SEEPAGE LOSSES FROM A CANAL CONVEYING A VARYING DISCHARGE

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
WATER RESOURCES DEVELOPMENT
(CIVIL)

By

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CANDIDATE'S DECLARATIONS

I hereby certify that the work which is being presented in the Dissertation entitled **SEEPAGE LOSSES FROM A CANAL CONVEYING A VARYING DISCHARGE** is being submitted in partial fulfillment of the requirements for the award of the Degree of Master of Technology in Water Resources Development, Indian Institute of Technology, Roorkee, is an authentic record of my own work carried out from July, 24th 2002 to June, 2004 under the supervision of Dr. G.C. Mishra, Professor, WRDTC, and Dr. M.L. Kansal, Associate Professor, WRDTC, Indian Institute of Technology, Roorkee

The matter embodied in this Dissertation has not submitted by me for the award of any other degree.

Dated: June 24th, 2004

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This is to certify that the above statements made by the candidate are correct to the best of my knowledge and belief.

24/6/04

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SYNOPSIS

Due to increasing demands of water day by day either for agriculture or for an industry it is imperative to conserve water by reducing loss of water during transmission from source to place of utility. It has been observed that about 1/3 of the water drawn from an irrigation source is lost as transmission losses before it reaches the agricultural land. The transmission losses are comprised of leakage, waste, evaporation, transpiration, and seepage.

The losses due to seepage are by far the most significant. There are various factors like soil texture, water turbidity, depth of water in the canal, velocity, temperature, position of water table and drainage condition, etc. which affect the seepage rate. The effects of many of these factors are difficult to determine and their relative importance has not yet been definitely known.

Before any measure for controlling seepage from canals are taken up it is necessary to locate where such losses do occur and estimate how much quantity is being lost as seepage. For estimating this seepage loss there are several analytical and practical methods in vogue.

For infinite depth of permeable medium with very deep water table, Vedernikov has given solution to the steady state seepage from trapezoidal and triangular canals. Vedernikov has used conformal mapping and hodograph method and has determined seepage from a canal. According to him, seepage per unit length of the canal is given by the formula:

$$q = k(B + A h)$$

Thus for computation of seepage from a canal, the width of the canal,B, at the water surface and the maximum depth of water, h, in the canal are to be known a priori. In practice, through hydraulic control structure, a certain known discharge is conveyed in the canal. Only the bed width of the canal is known. Because of seepage losses, the discharge varies along the canal, the depth of water at any section is unknown unless it is measured..

While applying conformal mapping to compute seepage from the canal, Vedernikov has considered the entire inverse hodograph plane and neglected the symmetry in the inverse hodograph plane. This has resulted in complex formulation. With more number of vertices, the integrations appearing Schwarz-Christoffel transformation become intractable. In the present thesis, the equations for computing seepage have been rederived. The symmetry in the inverse hodograph plane has been considered and simplified solution has been derived. Applying Manning's equation, and using mass balance, the depth of water along the canal have been computed. The variation of seepage along the canal is presented.

CONTENTS

NOTATIONS

A	Area (m^2) ; A factor which is function of the geometry of the canal; A is function of B/H and m
m	Cot α
B	Width of the canal at the water surface (m)
B( )	Beta function
b	bed width below the canal (m)
C	Constant
g	Gravitational constant (m/s^2)
h	Depth of water of the canal, (m)
i	$\sqrt{-1}$; hydraulic gradient
k	Coefficient of permeability, (m/sec)
L	Length, (m)
M	Constant ; moment
n	porosity ; parameter ; Manning Roughness
P	pressure force
q	Quantity of seepage (per unit normal to direction flow)
Q	Canal discharge, (m^3/sec)
R	hydraulic radius (m)
r	real axis of t - plane
So	Bed slope of the canal
S	Slope of energy grade line
s	Imaginary axis of t – plane
t	Tangential direction
V	Tangential velocity
w	complex potential
$\alpha\pi$	Angle of inclination of the canal over horizontal line; parameter
γ_w	unit weight of water
θ	Zhukovsky function
ϕ	potential function
ψ	stream function

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CHAPTER I

INTRODUCTION

1.1 GENERAL

The fundamental fact about water is that its supply is constant. The earth has exactly as much water today as it ever had – no less, no more. The ultimate need of water is much more than the useable water we have. The need of water, therefore, advocates the need to utilize the water properly in a well-planned and scientific manner so that the future generations do not stand in danger of running short of water.

Much of water that is drawn from an irrigation sources is lost during transit from the source to its field of utilization i.e. the agricultural land or any industry. These losses are called transmission losses. These transmission losses consist of mainly (i) evaporation losses (ii) seepage through canals. Therefore, it is of utmost importance to decide about the seepage losses before deciding whether a canal or distributary's system is to be lined or not or if decided not to line, to what capacity the unlined canal has to be designed.

It has been observed that nearly 40% of water is lost in unlined canals before reaching the fields mainly due to seepage in canals. Any endeavor to minimize seepage losses would be very useful for the rapid development of new areas with irrigation facilities.

The term seepage may be defined as the process of water movement into and through the soil from a body of surface water such as canal, ditch, stream or a reservoir.

The hydrodynamics of the seepage flow are different from the usual hydrodynamics of fluid flow since the former is characterized by channels of irregular cross section, non uniformity of soil in horizontal as were as vertical extent, changing elevations of the water surfaces in the channel and of the water table in the soil and other complications.

Canal is the main link between the source and the point of application in an irrigation system. A large quantity of total utilizable water is lost from these irrigation canals to the adjacent area through seepage.

The seepage from the canals, apart from the fact that it causes loss of valuable water has got another detrimental effect on the adjoining land by causing water logging.

A land is said to be water logged when the subsoil water level rises nearest to the surface such that aeration of the roots of the plants stops, alkaline salts rise to the root zone with capillary water rendering the soil useless for irrigation. If proper drainage arrangements are not made this may also result in unsanitary and unhealthy conditions to the inhabitants dwelling around it and in the long run convert once a bountiful land into a huge waste and marsh.

Estimation of seepage losses from canal is important for canal design. To compensate for these seepage losses, the design must provide for the supply of extra water. Computation of seepage is also important for evaluation of the likely benefit resulting from canal lining and for the evaluation of damages due to water logging and salinity.

Seepage loss from an irrigation canal is mainly a function of soil properties, flow depth and the shape of the cross section. The flow depth in a canal varies with distance even for the case of long prismatic channel with uniform sub-surface conditions because of spatially varying discharge. Thus, there is strong mutual dependence of water surface profile and seepage rate on each other. In this dissertation, using analytical method the mutual interaction between seepage rate and water surface profile is quantified. Depth of water along a canal of known cross section (bottom width and side slope) and consequent seepage losses are determined. Solution to the seepage problem is obtained through conformal mapping of the inverse hodograph plane and complex potential plane to an intermediate auxiliary planes. The flow depth in the canal is obtained by Manning's equation.

The total seepage loss is given by the difference in discharge at the upstream end of the reach (prescribed) and the discharge at the downstream end (computed). Expression for the quantity of seepage is based upon the potential theory of ground water percolation.

1.2 TWO-DIMENSIONAL FLOW

Physically, all flow systems extend in three dimensions. However, in many cases of the ground water motion are essentially planar, with the motion being substantially the same in parallel planes. Such a flow is a two dimensional flow and the corresponding seepage problem can be solved as a two-dimensional one. In the solution, the length of the flow region in the direction normal to the plane of flow is taken to be equal to unity. The total flow for the entire flow region is then obtained by multiplying of the plane problem by the actual length of the region.

The fundamental equations for two-dimensional seepage flow through a homogeneous and isotropic medium, the Darcy's law, could be written :

$$v_x = \frac{\partial\phi}{\partial x} = \frac{\partial\psi}{\partial y} = -k\frac{\partial h}{\partial x} \qquad \text{(A1.1)}$$

and

$$v_y = \frac{\partial\phi}{\partial y} = \frac{\partial\psi}{\partial x} = -k\frac{\partial h}{\partial y} \qquad \text{(A1.2)}$$

where v_x and v_y are the component of discharge velocity in the direction of the coordinate axes, ψ (x, y) is the stream function, and ψ (x, y) = C, a constant, depicts locus of stream line, ϕ; (x, y) is the velocity potential function defined as

$$\phi = -kh + C_1 \qquad \text{(A1.3)}$$

where C_1 is a constant; h(x, y) is the hydraulic head at the point (x, y). For the direction of the coordinate axes positive down ward,

$$h = \frac{p}{\gamma_w} - y \qquad \text{(A1.4)}$$

where p(x, y) is the water pressure at the point (x, y), the constant C_1 is dependent on the choice of the reference plane used in defining the potential function ϕ.

1.3 HYDRAULIC CHANNEL CROSS SECTIONS

Whenever the depth and the other features of flow, such as the cross-sectional area, the velocity, and the hydraulic slope, vary from section to section, flow becomes *varied*.

As an example, take flow in the canal (Fig. 1.1), in the event that the depths y_2 and y_1 are no longer equal, in which $y_2 < y_1$, where the depth will be decreasing down stream with a falling curved surface.

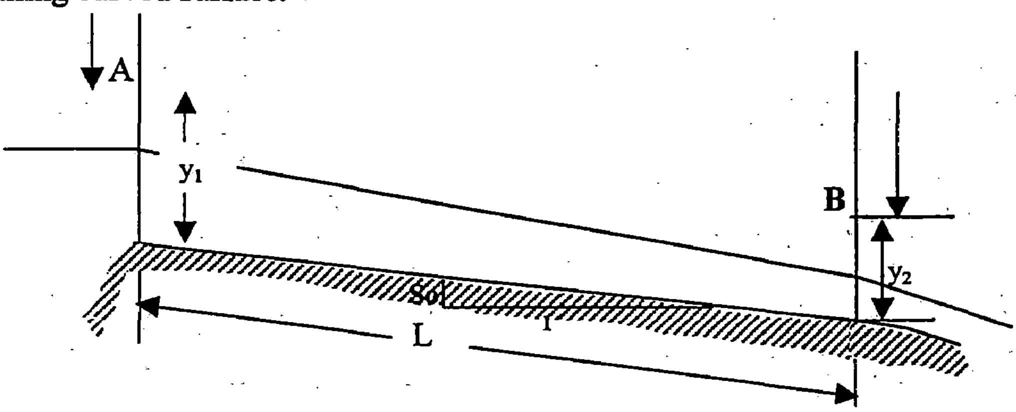

Fig. 1.1. **Varied flow in a canal connecting two sections**

In Fig. 1.1, in case levels at A and B do not remain permanent and the depths y_1 and y_2 change in time, the permanent character of the movement would no longer prevail and the flow would become variable.

A canal of given cross sectional form is laid for a length L between section A and B with a bottom slope So. The levels, and accordingly the depths y_1 and y_2, in different sections vary owing to seepage loss. The problem is to determine the discharge and depth of water at section B for a given value of discharge and width of the canal at section A. The gradually varied flow can be solved using Manning's equation.

1.4 OBJECTIVES OF THE STUDY

In the light of the status of the studies on the seepage losses from a canal conveying a varying discharge, the objectives of the present study are :

1. Computation rate of seepage from a canal for various discharge along the canal using conformal mapping.
2. Computation of discharge and depth of flow along the canal using Manning's equation.

The following assumptions have been made in study:

1. The flow is two-dimensional.
2. The soil is homogeneous and isotropic.
3. There is no loss due to evaporation or infiltration from surface.
4. Component of seepage velocity in the downstream direction is negligible
5. The water table occurs at infinite depth below the canal.

1.5 ORGANIZATION OF THE DISSERTATION

The presentation of the studies has been organized as follows :

In Chapter 1. A general introduction to the seepage from a canal conveying a varying discharge has been presented. It includes the subject matters on two dimensional flow and the hydraulic channel cross sections. The objectives of the studies have been identified here.

Chapter 2 deals with the pertinent review of literature. It includes the subject matters on estimation of seepage loss.

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In chapter3, analytical solution for seepage rate from a trapezoidal canal has been obtained. The results obtained using the simplified solution are compound with Vedernikov's result.

In chapter 4, using analytical solution and Manning's equation, effective dimensions of the canal are solved. Results for various of discharge, and seepage loss along the canal are presented.

In chapter 5, the important conclusions of the study have been summarized.

CHAPTER II

REVIEW OF LITERATURE

A literature review on estimation of seepage loss and Manning's equation has been made in this dissertation

II.1 ESTIMATION OF SEEPAGE LOSS

Average values of canal seepage can be derived readily in established irrigation areas. Inflow and outflow measurements are made on a typical reach of an existing canal to reveal total loss. Alternatively, water is ponded and the rate of fall measured. Estimated evaporation is subtracted from total loss to give seepage loss. This, expressed as a loss per unit length, can be applied to canals of similar cross-section. Seepage loss expressed as a loss per unit area of wetted surface, is likely to give false results when applied to canal of different cross section.

Owing to the variable nature of groundwater conditions and channel bed and bank materials, the problem of estimating seepage in the absence of any field data from comparable canals is formidable. To quote the USBR Design Standard No. 3, 'An accurate prediction of seepage loss is extremely difficult to make and the result are at best uncertain'. It is notable that, in practice, seepage frequently can be seen to occur in small zones where a lens of highly permeable material has been included in the bank and compaction has been inadequate.

A number of studies, experimental as well as theoretical, have been conducted in the past to estimate seepage losses from irrigation canals [Kozeny (1931), Vedernikov (1934), Garg and Chawla (1970), Subramanya et al. (1973), Sharma and Chawla (1979), Bouwer (1969), Pontin et al. (1979), Deacon (1983)]. The formulation given by Kozeny and Vedernikov are given in Appendix 2.

Before any measures for controlling seepage from canals are taken up it is necessary to locate where such losses occur and estimate how much quantity is being lost as seepage.

Estimation of seepage losses from canals can be categorized as follows:

1. Methods for pre construction period
2. Methods for post construction period

Methods pre construction period are useful for finding the seepage loss from canals before finalizing the designs of a scheme or project. Quantitative analysis helps in finding the capacity of a canal, or branch or distributary to be designed or for aligning the canal so that there may be minimum conveyance loss. This methods further classified into *Empirical methods* and *analytical methods*. These methods can also be used for determination of seepage from existing canal.

The *empirical methods* have been in vogue in India and in other countries from a very old time. These formulae are simple to use with a minimum of field investigation.

The *analytical method for computation of canal seepage* depends on the canal dimension, the permeability of the subsoil, distance of governing drainages and the difference in the water levels in the canal and at the drainage boundary.

Mortoz, Offengeden and Molesworth (Vide Yates, 1975) have also given empirical equations for estimating seepage loss rate as a function of soil characteristic, flow depth and discharge.

Analytical solutions for steady state seepage loss for various cross sections can be obtained by solving the Laplace equation for flow in a porous medium satisfying simple boundary conditions (Garg and Chawla 1970, Subramanya et al. 1973, Sharma and Chawla 1979). Analytical solutions are available for evaluating seepage losses from canals under steady conditions for the following cases of channels located in homogeneous and isotropic mediums extending up to (Bouwer 1969):

1. Infinite depth with swallow water table
2. Finite depth with swallow water table
3. Infinite depth with very deep water table.

Subramanya et al. (1973) have found analytical solutions for partially lined trapezoidal channels. Analytical solutions assume uniform flow conditions in the canal and give the seepage loss per unit length of the channel.

For condition at (3) the velocity distribution tends to become uniform at deep layers and the percolation velocity, V, equals to the coefficient of permeability, k, since the hydraulic gradient approaches unity with infinite depth of the water table.

As $\frac{d+H}{d} \Rightarrow 1$, $v \approx k$

where

H = Depth of water in the canal

d = depth to the water table below the canal bed.

Methods for post construction period are methods adopted to find the losses due to seepage in an existing canal. This information would be very useful to decide upon where a canal reach has to be lined or not and also useful in water logging studies at places of high water table.

The usual methods adopted in India and in other countries are:

1. Ponding method
2. Seepage meters
3. Inflow – outflow methods
4. Tracer Technique
5. Radioisotope methods.

II.2 HYDRAULIC CHANNEL CROSS SECTIONS

II.2.1 Manning Formula

Of the empirical formulae available for channel design, when channel conveying clear water is to be lined or the earth used for its construction is non-erodible in the normal range of canal velocities, the Manning equation is the one most commonly used. It has simple form but gives satisfactory results.

Some channel cross sections are more efficient than others in that they provide more area for a given wetted perimeter. In general, when a channel is constructed, the excavation, and possibly the lining, must be paid for. From the Manning formula it is shown that when the area of cross section is a minimum, the wetted perimeter is also a minimum, and so both lining and excavation approach their minimum value for the same dimension of channel. The Manning formula is

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad (2.1)$$

where R is the hydraulic radius (area divided by perimeter P), S is the slope of energy grade line, and n is the Manning roughness factor (table 2.1).

The value of the roughness coefficient, n is affected by numerous factors :

1. Surface roughness
2. Irregularity of section
3. Obstructions to flow
4. Channel geometry
5. Height and density of vegetation
6. Flattening of vegetation at high flows
7. Sediment load.

There is, as yet, no satisfactory method for evaluating n from these factors. For a first approximation in design a value can be chosen from a table such as that given in table 2.1.

Table 2.1. Values of the roughness coefficient *n*

Type of surface	Range of roughness coefficient
Neat cement	0.10 - 0.013
Cement mortar	0.11 - 0.015
Planed planks	0.10 - 0.014
Concrete	0.11 - 0.018
Dry rubble	0.25 - 0.035
Cement rubble	0.17 - 0.030
Earth. Straight and uniform	0.17 - 0.025
Rock cuts. Smooth and uniform	0.25 - 0.035
Rock cuts. Jagged and irregular	0.35 - 0.045
Dredged earth channels	0.25 - 0.033
Canal with rough stony beds, weeds on earth banks	0.25 - 0.040
Canal with earth bottom, rubble sides	0.028 - 0.035

Even for a particular channel, roughness varies during use from a minimum shortly after maintenance or construction to a maximum tolerated level when maintenance is due. It is usually possible to find existing canals similar to those proposed for the new system. If no canal exist, a conservative value of n must be used in design and checked in the field at the earliest opportunity.

II.2.2. Computation of Canal Cross Sections

A straight reach of canal of uniform section is selected. Discharge and slope of the water surface are measured. The cross section is found by sounding at several sample positions, and cross section areas and hydraulic depth are calculated.

The discharge of flow in a channel may be expressed as the product of the velocity, and the water area, or

$$Q = V \cdot A \tag{2.2}$$

Where Q is the discharge (m^3/s), V is the velocity (m/s) and A is the cross sectional flow area. When the Manning formula is used, the equivalent formula for discharge is

$$Q = \frac{1}{n} R^{2/3} S_0^{1/2} A \tag{2.3}$$

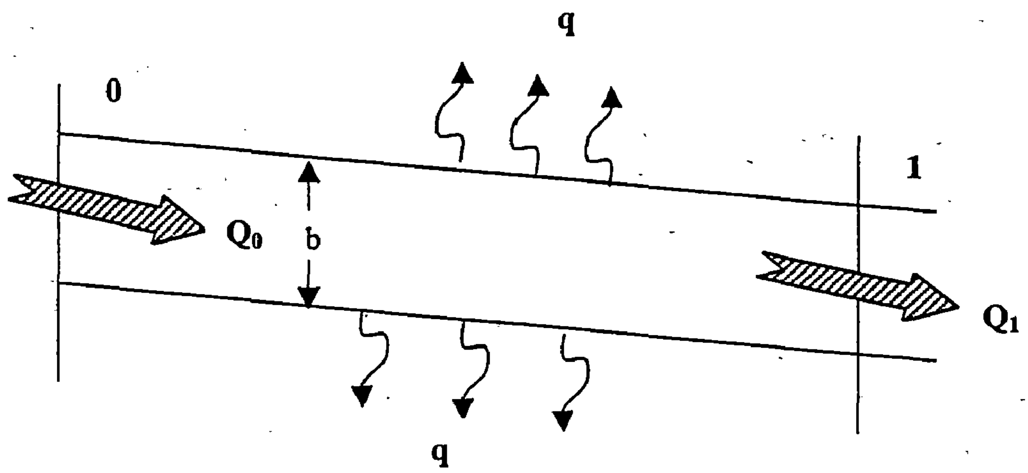


Fig. 2.2 **Illustration of varied flow**

When the discharge, roughness, slope and width of the canal are known at section 0 (Fig. 2.2), equation (2.3) can be used to compute the depth of water in section 0.

The computation for the cross section at another section may be performed by the use of two equations. Equation (2.2) and equation (2.3) is very useful tool for the computation cross sections of the canal at each section.

$$Q = A_1 \, . \, V_1 + (q_1 \, L) = A_2 \, . \, V_2 + (q_2 \, L) = \; .... \qquad (2.4)$$

In which

$$A = b\,h + h^2 \cot \alpha \qquad (2.5)$$

$$R = \frac{A}{P} = \frac{b\,h + h^2 \cot \alpha}{b + 2\,h \operatorname{cosec} \alpha} \qquad (2.6)$$

For the first reach

$$Q_1 = A_1 \; V_1 = Q_0 - (q_{av})*L \qquad (2.7)$$

$$q_{av} = k\,(B + A\,(0.5\,(h_0 + h_1))) \qquad (2.8)$$

where

B = width of canal at water surface

A = seepage parameter given by Vedernikov

$\frac{1}{2}(ho + h_1)$ = average depth of water in a canal reach of length L

For a given values of discharge, bed slope, roughness and width of the canal at section 0, we find value of ho, and apply equations (2.4), (2.5), (2.6), (2.7) and (2.8) to find depth of water at section 1.

CHAPTER III

SEEPAGE FROM A TRAPEZOIDAL CANAL

III.1 GENERAL

The section of a canal can be conveniently assumed as triangular, rectangular or trapezoidal for computation of seepage by analytical method. Trapezoidal section is adopted for canal conveying large discharge. For small distributory , canal section can be assumed to be triangular. The mathematical complexity for computation of seepage is least for triangular section.

A number of studies, experimental as well as theoretical, have been conducted in the past to estimate seepage losses from irrigation canal [Garg and Chawla (1970), Subramanya et al. (1973), Sharma and Chawla (1979), Bouwer (1969), Pontin et al. (1979), Deacon (1983)]. Usually inflow-outflow method is used for directly determining the seepage loss in the field. In this method, the actual canal discharge entering a reach (inflow) and going out of the reach (outflow) are measured carefully over a time interval. The difference between the two per unit time is taken as the *seepage loss from the canal.* The degree of error present in the above method is quite high and large number of measurements are required to reduce the level of uncertainty (Deacon 1983). Moreover, theoretical or analytical estimation of seepage is required at the time of designing a new irrigation system.

The governing Laplace equation with the associated complex boundary conditions can also be solved numerically using finite-difference or finite element methods. Analytical solutions assume uniform flow conditions in the canal and give the seepage loss per unit length of the channel.

In this chapter the analysis of steady seepage from a canal has been derived using conformal mapping. A solution much simpler than Vedernikov's solution has been derived.

III. STATEMENT OF THE PROBLEM

Figure (3.1) shows a schematic cross section of canal in z-plane. The depth of water in the canal is *h* and *b* is bottom width of the canal.

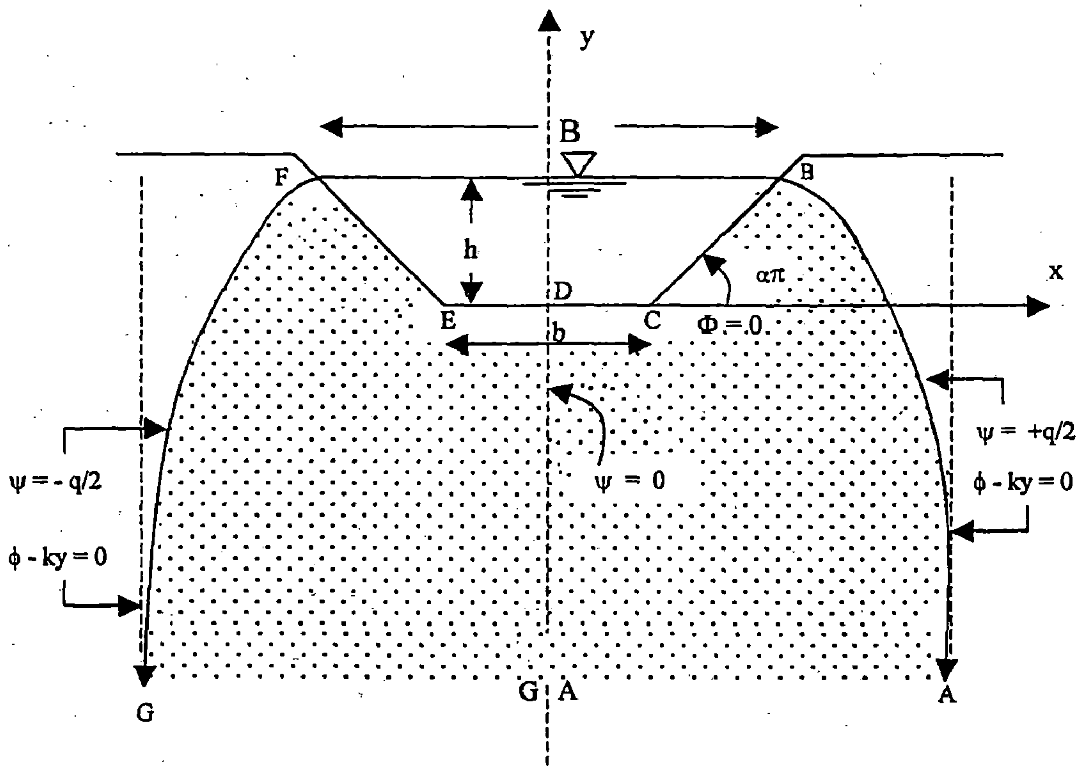


Fig. 3.1 **Physical flow domain in z-plane, z = x + i y**

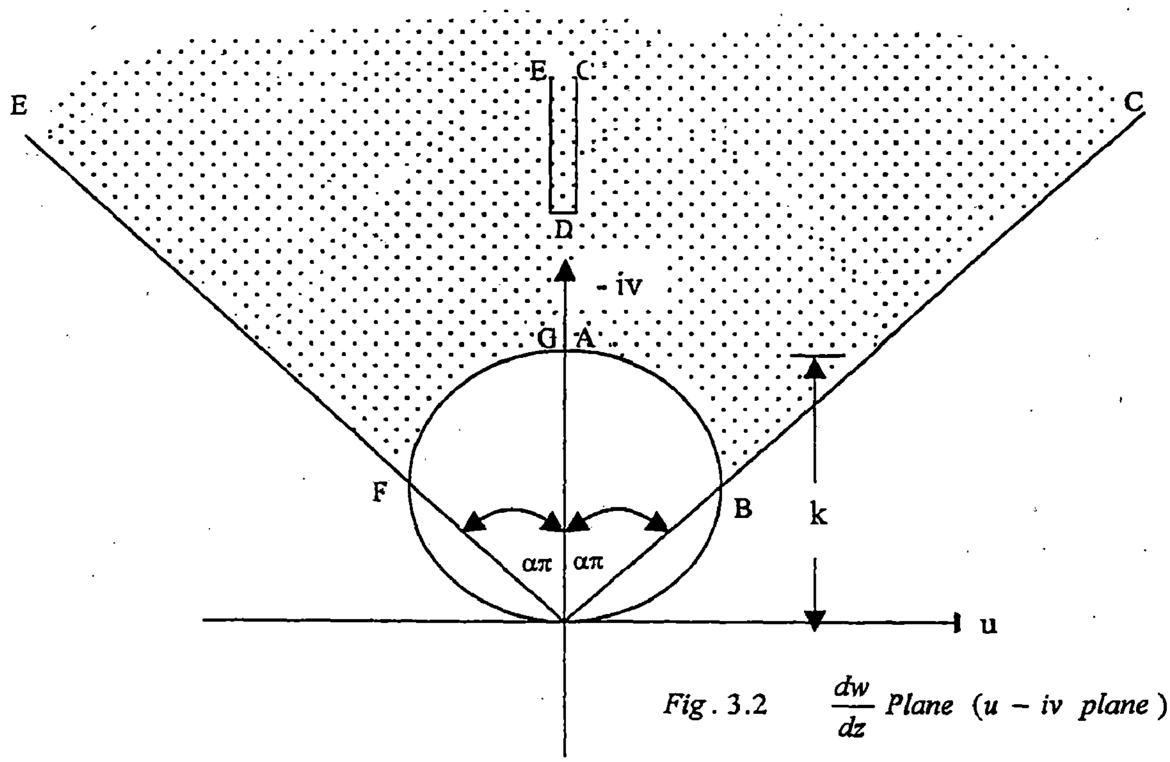


Fig. 3.2 $\frac{dw}{dz}$ *Plane (u − iv plane)*

The hodograph and the inverse hodograph planes are shown in Fig. (3.2) and (3.3) respectively.

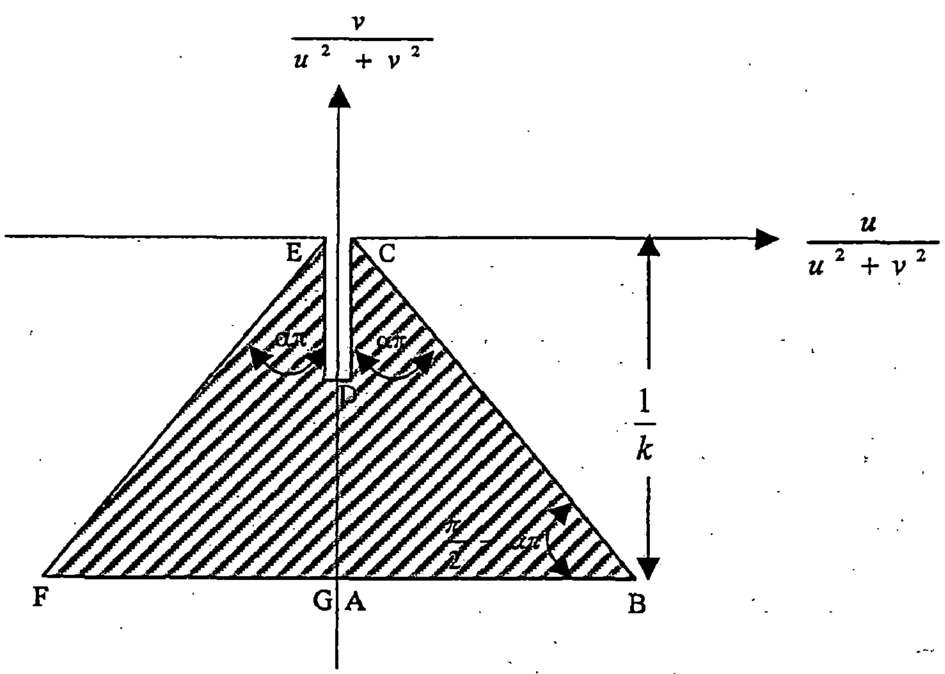


Fig. 3.3. Inversion of the hodograph ($\frac{dz}{dw}$ - plane)

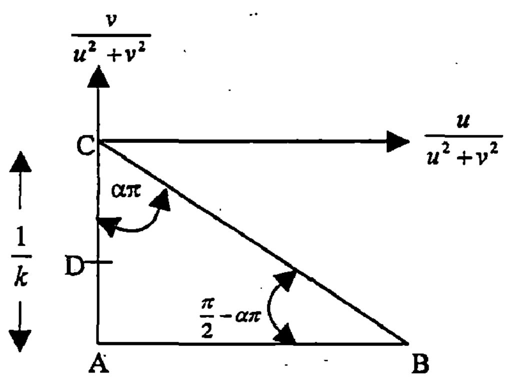


Fig. 3.4 Half of the inverse Hodograph

There are five vertices (B, C D, E and F) in the hodograph plane. Vedernikov has conformally mapped the entire inverse hodograph plane onto the auxiliary t plane.

The inverse hodograph is symmetrical about $\frac{v}{u^2 + v^2}$ axis. Therefore, half of the inverse hodograph plane can be considered in the mapping procedure. In such formulation the number of vertices is three.
The right half of the inverse hodograph is shown in Fig. (3.4).

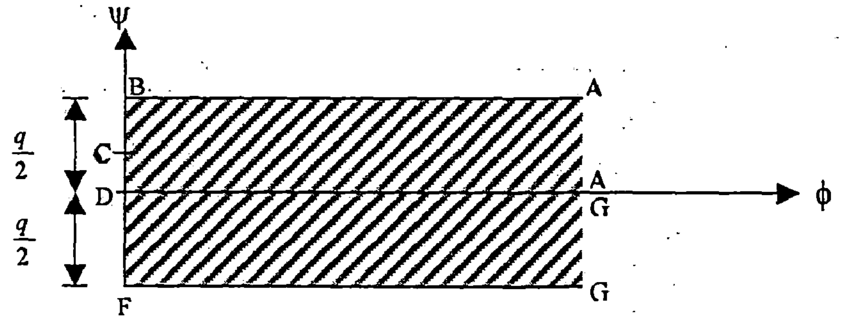


Fig 3.5 **w-plane (w = ϕ + i Ψ)**

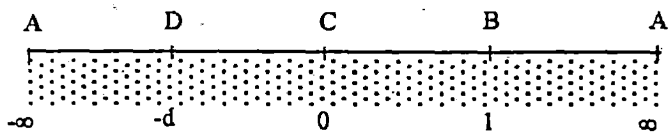


Fig. 3.6 **t-plane (t = r + is)**

The complex potential plane corresponding to the flow domain is shown in Fig. (3.5)

III.3 ANALYSIS

III.3.1 Mapping of the One Half of the Inverse hodograph $\left(\frac{dz}{dw}\right)$ Plane to An Auxiliary t-Plane

According to the Schwarz-Christoffel transformation, the conformal mapping of the triangle ACB in $\frac{dz}{dw}$ plane onto lower half of the auxiliary t plane (the vertices C and B being mapped onto 0 and 1 in t plane respectively) is given by

$$\frac{dz}{dw} = M \int_0^t \frac{dt}{t^{1-\alpha}\ (1-t)^{1-(\frac{1}{2}-\alpha)}} + N \qquad (3.1)$$

$$= M \int_0^t t^{\alpha-1}\ (1-t)^{(\frac{1}{2}-\alpha)-1}\ dt + N$$

in which M and N are constant.

At the lower limit (t = 0), $\frac{dz}{dw} = 0$; hence, constant N = 0

For $0 \le t \le 1$

$$\frac{dz}{dw} = M\ B_t(\alpha\ ,\ \tfrac{1}{2}-\alpha) \qquad (3.2)$$

where $B_t(\alpha\ ,\ \frac{1}{2}-\alpha)$ is incomplete beta function.

For point B, t = 1 and $\frac{dz}{dw} = \frac{1}{k}\sec\alpha\pi\ e^{i\left(\frac{3\pi}{2}+\alpha\pi\right)}$

Applying this condition in Eq. (3.2)

$$M\ B(\alpha\ ,\ \tfrac{1}{2}-\alpha) = \frac{1}{k}\sec\alpha\pi\ e^{i\left(\frac{3\pi}{2}+\alpha\pi\right)} \qquad (3.3a)$$

where $B(\alpha\ ,\ \frac{1}{2}-\alpha)$ is complete beta function $= \frac{\Gamma(\alpha)\ \Gamma(\frac{1}{2}-\alpha)}{\Gamma(\frac{1}{2})}$

and $\Gamma(.)$ is complete Gamma function.

From Eq. (3.3a)

$$M = \frac{\frac{1}{k}\ \sec\alpha\pi\ e^{i\left(\frac{3\pi}{2}+\alpha\pi\right)}}{B\left(\alpha\ ,\ \frac{1}{2}-\alpha\right)} = \frac{\frac{1}{k}\left(\tan\alpha\pi - i\right)}{B\left(\alpha\ ,\ \frac{1}{2}-\alpha\right)} \qquad (3.3b)$$

Substituting M in equation (3.2), we find:

$$\frac{dz}{dw} = \frac{\frac{1}{k}\ (\tan\alpha\pi\ - i)}{B(\alpha\ ,\ \frac{1}{2}-\alpha)}\ B_t\ (\alpha\ ,\ \tfrac{1}{2}-\alpha) \quad ,\ 0 \le t \le 1 \qquad (3.4)$$

For $1 \le t \le \infty$ $\frac{dz}{dw}$ *is given by:*

$$\frac{dz}{dw} = M \int_1^{t} \frac{dt}{t^{1-\alpha}\ (1-t)^{1-(\frac{1}{2}-\alpha)}} + \frac{1}{k}\sec\alpha\pi\ e^{i(3\pi/2+\alpha\pi)} \qquad (3.5a)$$

or

$$\frac{dz}{dw} = M\,(-1)^{-\frac{1}{2}-\alpha} \int_{1}^{t} t^{\alpha-1}\,(t-1)^{(\frac{1}{2}-\alpha)-1}\,dt + \frac{1}{k}\sec\alpha\pi\, e^{i(\frac{3\pi}{2}+\alpha\pi)} \qquad (3.5b)$$

and incorporating constant M in equation (3.5b)

Substituting $t = \frac{1}{p}$; $dt = -\frac{1}{p^2}\,dp$

$$\frac{dz}{dw} = -\frac{1}{k}\,\frac{\sec\alpha\pi}{B(\alpha\,,\frac{1}{2}-\alpha)}\left[B\left(\tfrac{1}{2}\,,\,\tfrac{1}{2}-\alpha\right) - B_{\frac{1}{t}}\left(\tfrac{1}{2}\,,\,\tfrac{1}{2}-\alpha\right)\right]$$

$$+\frac{1}{k}\,\sec\alpha\pi\, e^{i(\frac{3\pi}{2}+\alpha\pi)} \qquad (3.6)$$

For $-\infty \le t \le 0$, $\frac{dz}{dw}$ is given by

$$\frac{dz}{dw} = M\int_{0}^{-t^1} t^{\alpha-1}\,(1-t)^{(\frac{1}{2}-\alpha)-1}\,dt \qquad (3.7)$$

With substitution $t = -\xi$, equation (3.7) reduces to

$$\frac{dz}{dw} = \frac{1}{k}\,\frac{\sec\alpha\pi}{B\left(\alpha\,,\frac{1}{2}-\alpha\right)}\,e^{i(\frac{5\pi}{2})}\int_{0}^{t^1}\xi^{\alpha-1}\,(1+\xi)^{\frac{1}{2}-\alpha-1}\,d\xi \qquad (3.8)$$

Substituting $\xi = \frac{p}{1-p}$; $d\xi = \frac{1}{1-p} + \frac{p}{(1-p)^2}$, at the lower limit $\xi = 0$, $p = 0$ and at the upper limit $\xi = t^1$, $p = \frac{t^1}{1+t^1}$, where p is dummy variable, the integral above is converted to the following integral:

$$\frac{dz}{dw} = -\frac{1}{k}\,\frac{\sec\alpha\pi}{B\left(\alpha\,,\frac{1}{2}-\alpha\right)}\,e^{i\left(\frac{5\pi}{2}\right)}\int_{0}^{\frac{t^1}{1+t}} p^{\alpha-1}\,(1-p)^{-\frac{1}{2}}\,dp$$

or

$$\frac{dz}{dw} = -\frac{1}{k}\,\frac{\sec\alpha\pi}{B\left(\alpha\,,\frac{1}{2}-\alpha\right)}\; i\; B_{\frac{t^1}{1+t^1}}\left(\alpha\,,\tfrac{1}{2}\right) \qquad (3.9)$$

III.3.2 Mapping of The Complex Potential w-plane to the Auxiliary t-Plane

There is symmetry in w plane about ϕ axis. Considering upper half of the w-plane the conformal mapping is given by :

$$\frac{dw}{dt} = \frac{M_1}{(t+d)^{1/2}\ (1-t)^{1/2}} \tag{3.10}$$

$$w = M_1 \int \frac{dt}{(t+d)^{1/2}\ (1-t)^{1/2}} + N \tag{3.11}$$

$$= M_1 \sin^{-1} \frac{2t+d-1}{1+d}$$

For point D ; t = - d and w = 0 ; hence,

$$0 = M_1 \sin^{-1}(-1) + N$$

Hence,

$$N = M_1 \left(\pi/2\right) \tag{3.12}$$

For point B, t = 1 and w = $\frac{iq}{2}$. Applying this condition we find :

$$M_1 = \frac{iq}{2\pi}, \text{ and } N = \frac{iq}{4} \tag{3.13}$$

So, w is given by:

$$w = \frac{iq}{2\pi} \sin^{-1} \frac{2t+d-1}{1+d} + i\frac{q}{4} \tag{3.14}$$

III.3.3 Relation between The Physical Flow Domain z-plane and the Auxiliary t-Plane

$$\frac{dz}{dt} = \frac{dz}{dw} \cdot \frac{dw}{dt}$$

For $0 \le t \le 1$,

$$\frac{dz}{dt} = M\ B_t(\alpha\ ,\ 1/2-\alpha)\ \left(\frac{iq}{2\pi}\right) \frac{1}{(t+d)^{1/2}\ (1-t)^{1/2}} \tag{3.15}$$

and

$$z = \int_0^{t^1} M\ B_t(\alpha\ ,\ 1/2-\alpha)\ \left(\frac{iq}{2\pi}\right) \frac{dt}{(t+d)^{1/2}\ (1-t)^{1/2}} + z_c \tag{3.16}$$

At point B, $t^1 = 1$, and $z = z_b$. Hence

$$z_b = \frac{-i\,M\cdot q}{2\pi} \int_0^1 \frac{B_t\left(\alpha\,,\tfrac{1}{2}-\alpha\right)\,dt}{(t+d)^{1/2}\,(1-t)^{1/2}} + z_c \tag{3.17a}$$

or

$$z_b - z_c = h\cot\alpha\pi + ih = \frac{q}{2k\pi}\,\frac{(1+i\tan\alpha\pi)}{B\left(\alpha\,,\tfrac{1}{2}-\alpha\right)} \int_0^1 \frac{B_t\left(\alpha\,,\tfrac{1}{2}-\alpha\right)dt}{\sqrt{(t+d)(1-t)}} \tag{3.17b}$$

Equating real parts (or imaginary part) on either sides in Eq. (3.17b)

$$\frac{q}{kh} = \frac{2\pi\,B\left(\alpha\,,\tfrac{1}{2}-\alpha\right)}{\tan(\alpha\pi)\displaystyle\int_0^1 \frac{B_t\left(\alpha\,,\tfrac{1}{2}-\alpha\right)dt}{(t+d)^{1/2}\,(1-t)^{1/2}}} \tag{3.18}$$

For $-\infty \le t \le 0$, $\dfrac{dz}{dt}$ is given by:

$$\frac{dz}{dt} = \frac{q}{2k\pi}\,\frac{\sec(\alpha\pi)}{B\left(\alpha\,,\tfrac{1}{2}-\alpha\right)}\,B_{\frac{-t}{1-t}}\left(\alpha\,,\tfrac{1}{2}\right)\,\frac{1}{(t+d)^{1/2}\,(1-t)^{1/2}} \tag{3.19a}$$

and

$$z = \frac{q}{2k\pi}\,\frac{\sec(\alpha\pi)}{B\left(\alpha\,,\tfrac{1}{2}-\alpha\right)} \int_0^{t^1} B_{\frac{-t}{1-t}}\left(\alpha\,,\tfrac{1}{2}\right)\frac{dt}{(t+d)^{1/2}\,(1-t)^{1/2}} \tag{3.19b}$$

At $t = -d$, $z = z_d$. Hence

$$z_d = \frac{q}{2k\pi}\,\frac{\sec(\alpha\pi)}{B\left(\alpha\,,\tfrac{1}{2}-\alpha\right)} \int_0^{-d} B_{\frac{-t}{1-t}}\left(\alpha\,,\tfrac{1}{2}\right)\frac{dt}{(t+d)^{1/2}\,(1-t)^{1/2}} + z_c \tag{3.20}$$

Substituting $-t = p$ and $dt = -dp$

Where p is dummy variable Eq. (3.20) reduces to

$$z_c - z_d = \frac{q}{2k\pi}\,\frac{\sec(\alpha\pi)}{B\left(\alpha\,,\tfrac{1}{2}-\alpha\right)} \int_0^{d} B_{\frac{p}{1+p}}\left(\alpha\,,\tfrac{1}{2}\right)\frac{dp}{(d-p)^{1/2}\,(1+p)^{1/2}} \tag{3.21}$$

or

$$b = \frac{q}{k\pi}\,\frac{\sec(\alpha\pi)}{B\left(\alpha\,,\tfrac{1}{2}-\alpha\right)} \int_0^{d} B_{\frac{p}{1+p}}\left(\alpha\,,\tfrac{1}{2}\right)\frac{dp}{(d-p)^{1/2}\,(1+p)^{1/2}} \tag{3.22}$$

Substituting q in above

$$\frac{b}{h} = \frac{4}{\sin \alpha\pi} \frac{\int_0^d B_{\frac{p}{1+p}} (\alpha , ½) \frac{dp}{(d - p)^{½} (1 + p)^{½}}}{\int_0^1 B_t (\alpha , ½ - \alpha) \frac{dt}{(t + d)^{½} (1 - t)^{½}}} \qquad (3.23)$$

III.4 RESULT AND DISCUSSIONS

The computation of seepage from a trapezoidal canal (known bed width, bank slope and depth of water) is straight forward. The integration appearing in Eq. (3.18) is performed numerically. The integral $\int_0^1 \frac{B_t (\alpha , ½ - \alpha)dt}{(t + d)^{½} (1 - t)^{½}}$ is an improper integral, since the integral is infinite at the upper limit of integration i.e. at t = 1.

With substitution 1 - t = v^2, dt = 2v dv the improper integral reduces to the proper integral as given below

$$\int_0^1 \frac{B_t (\alpha , ½ - \alpha)dt}{(t + d)^{½} (1 - t)^{½}} = 2 \int_0^1 \frac{B_{1-v^2} (\alpha , ½ - \alpha)dv}{(1 - v^2 + d)^{½}}$$

The incomplete Beta function $B_{1-v^2} (\alpha , ½ - \alpha)$ is computed using Hypogeometric series i.e.

$$B_x (p , q) = \frac{x^p}{p} \; {}_2F_1 (p , 1 - q , 1 + p , x)$$

$${}_2F_1 (a,b,c,x) = 1 + \frac{ab}{c} x + \frac{a(a+1)b(b+1)}{c(c+1)} \frac{x^2}{2!} + \frac{a(a+1)(a+2)b(b+1)(b+2)}{c(c+1)(c+2)} \frac{x^3}{3!} + ...$$

Assuming the parameter $\frac{q}{kh}$ is obtained from (3.18) after converting the improper integral to proper one as explained above. The corresponding b/h is obtained from Eq. (3.23). The improper integral $\int_0^d B_{\frac{p}{1+p}} (\alpha , ½ - \alpha) \frac{dp}{(d-p)^{½} (1 + p)^{½}}$ is inverted to a proper integral with substitution $d - p = v^2$, dp = - 2 v dv

$$\int_0^d B_{\frac{p}{1+p}} (\alpha , ½ - \alpha) \frac{dp}{(d-p)^{½} (1 + p)^{½}} = 2 \int_0^{\sqrt{d}} B_{\frac{d-v^2}{1+d-v^2}} (\alpha , ½ - \alpha) \frac{dv}{(1+d-v^2)^{½}}$$

The variation of $\frac{q}{kh}$ with $\frac{b}{h}$ is shown in Fig. (3.1) for a trapezoidal canal and Fig. (3.2) for a triangular canal.

The variation of $\frac{q}{kh}$ with $\frac{b}{h}$ is quarry linear making it easy for extrapolation of $\frac{q}{kh}$ for higher $\frac{b}{h}$. However seepage $\frac{q}{kh}$ can be obtained easily for any higher value ($\frac{b}{h} > 5$) of $\frac{b}{h}$. The seepage quantity at $\frac{b}{h} = 0$ corresponds to a triangular canal. The seepage quantity computed compares exactly with Vedernikov's result. However the mathematical complexity does not exist in the resent analysis. In the present method the dimensionless seepage quantity has been expressed as a function of a single variable $\frac{b}{h}$ unlike in Vedernikov's method in which the dimensionless seepage is a function of $\frac{b}{h}$ and A*.

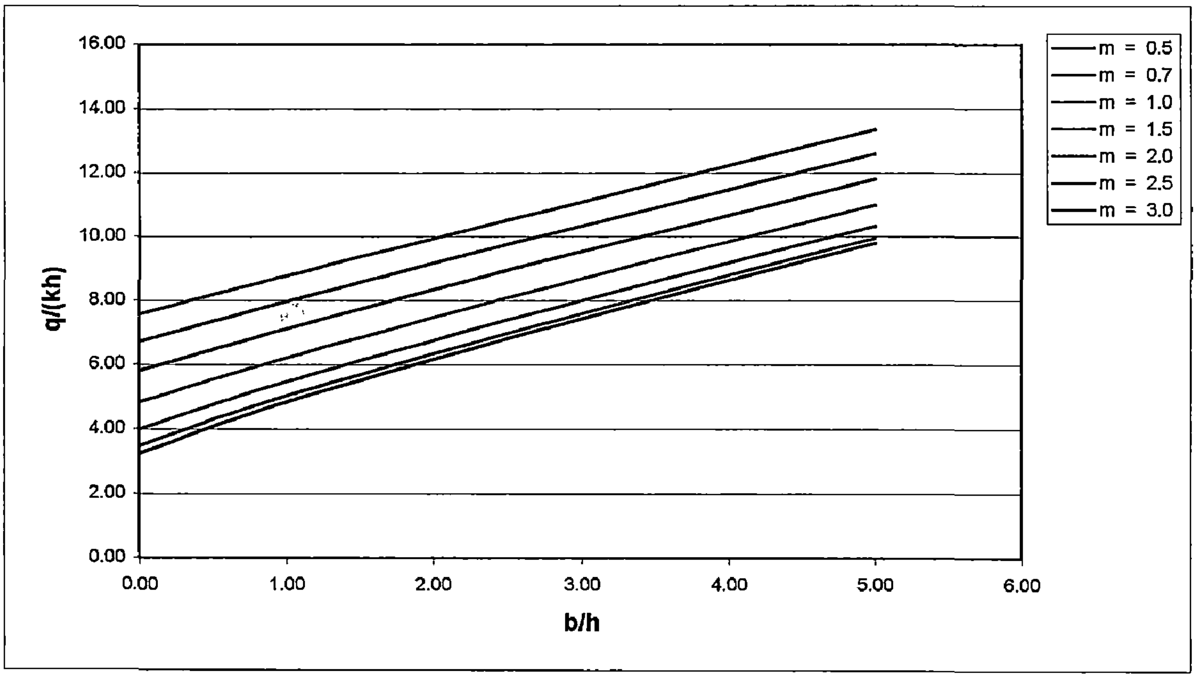
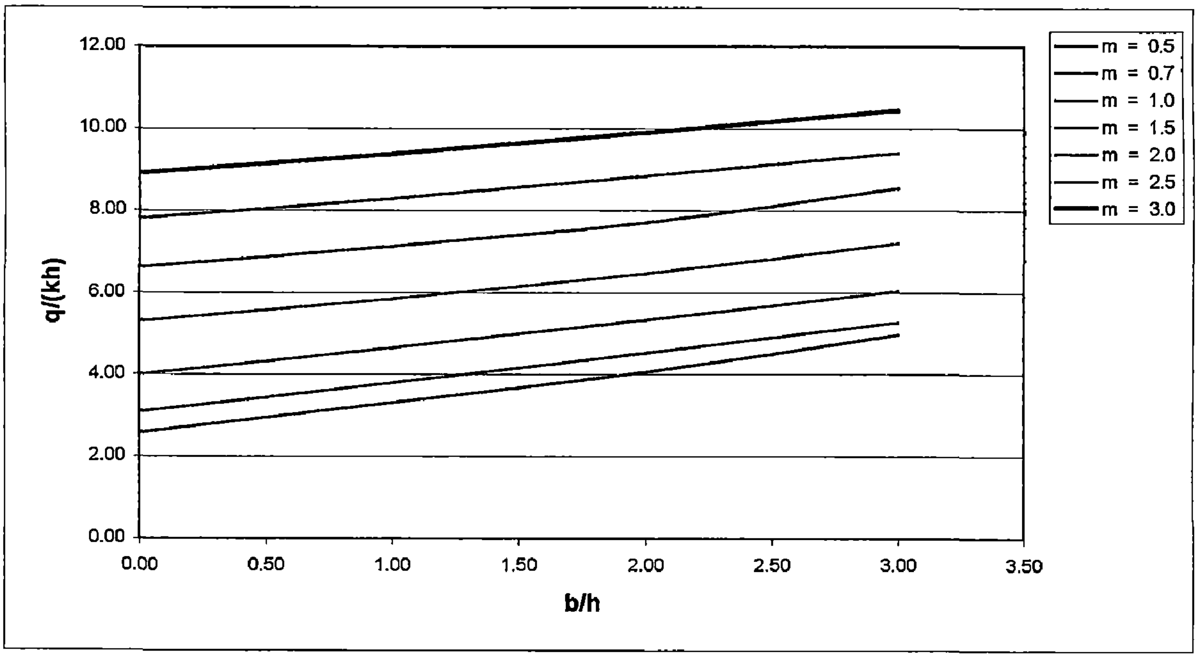


Fig. 3.1. Variation of q/(kh) with b/h for a Trapezoidal Canal



CHAPTER IV

SEEPAGE FROM A CANAL FOR VARIED DISCHARGE

IV.1 GENERAL

Artificial channels, including irrigation and navigation canals, spillway channel, sewers, and drainage ditches are all man-made. They are normally of regular cross sectional shape and bed slope, and as such are termed prismatic channels. Their construction material are varied; commonly used materials include concrete, steel and earth. The surface roughness characteristics of these materials are normally well defined within engineering tolerances. In designing an artificial channel, cost is prime consideration, so the engineer must use a channel whose geometry minimizes the cost of excavation and lining.

Natural channels are normally very irregular in shape, and their materials are diverse. The surface roughness of natural channels changes with time, distance and water surface elevation.

IV.2 UNIFORM FLOW COMPUTATIONS

Manning formula may be used to determine steady uniform flow. There are two types of commonly occurring problems to solve. The first is to determine the discharge given the depth, and the second is to determine the depth given the discharge. The depth is referred to as the normal depth, which is synonymous with steady uniform flow. As uniform flow can only occur in a channel of constant cross section, natural channels should be excluded. However, in solving the equations of varied flow applicable to natural channels, it is still necessary to solve Manning's equation (Chow, 1959).

The value of the roughness coefficient n determines the frictional resistance of a given channel. It can be evaluated directly by discharge and stage measurements for a known cross-section and slope.

The uniform flow equation can be used to give some indication of the optimum section shape from hydraulic viewpoint.

For the case of artificial lined channels, n may be estimated with reasonable accuracy.

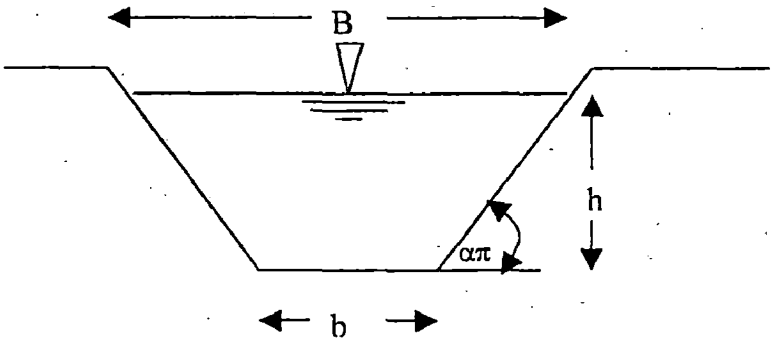


Fig. 4.1. **Trapezoidal Canal**

To find the hydraulic trapezoidal section (Fig. 4.1) we apply

$$A = b\,h + h^2 \cot \alpha\pi \tag{4.1}$$

and

$$R = \frac{b\,h + h^2 \cot \alpha\pi}{2\,h\,\operatorname{cosec}\,\alpha\pi + b} \tag{4.2}$$

Applying Manning's equation $V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$ (4.3)

For a given value of discharge, Q_0 , b, α, and bed slope of the canal, we can compute depth of water by applying equation (4.3).

IV.3 COMPUTATION CROSS SECTIONS ALONG A CANAL

At entrances to channels and at changes in cross section and bottom slope, the structure that conducts the liquid from the upstream section to the new section is known as control structure. Its purpose is to change the shape of flow and surface profile in such a manner that minimum losses result. A transition for a flow is illustrated in Fig 4.2.

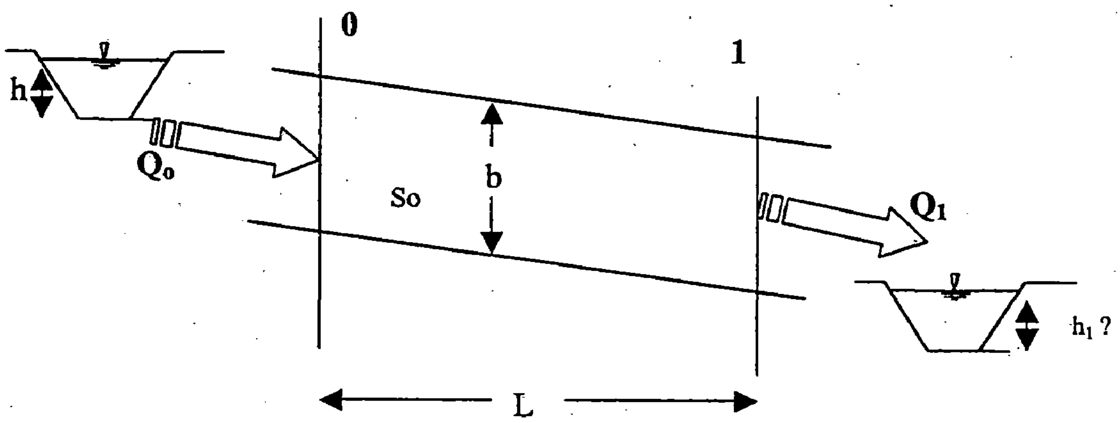


Fig. 4.2. **Illustration transition of flow**

Assuming stedy flow condition and applying mass balance for the reach of length L

$$Q_1 = Q_o - q_{av} \, . \, L \tag{4.5}$$

where

q_{av} = average seepage loss per unit length of the reach.

At section 1

$$Q_1 = A_1 \, . \, V_1 = Q_0 - q_{av} \, L \tag{4.6}$$

or

$$Q_o - q_{av} \, . \, L - A_1 \, . \, V_1 = 0 \tag{4.7}$$

According Vedernikov formula

$$q = k(B + A^* h) \tag{4.8}$$

the parameter A^* is

$$A^* = \frac{q}{kh} - \frac{B}{h} = C^* - \frac{B}{h} \tag{4.9}$$

C* is given by Eq. (3.18)

The width of the canal at water surface, B, is

$$B = b + 2 \, h \cot \alpha\pi \tag{4.10}$$

The average seepage loss

$$q_{av} = k \, (b + 2 \, \bar{h} \cot \alpha\pi + A^* \bar{h})$$

$$= k \left[b + (h_0 + h_1) \cot \alpha\pi + A^* \left(\frac{h_0 + h_1}{2} \right) \right] \tag{4.11}$$

Incorporating Eq. (4.11) in Eq. (4.7)

$$Q_0 - k\left[b + (h_0 + h_1)\cot\alpha\pi + A^*\left(\frac{h_0 + h_1}{2}\right)L - \frac{1}{n}\frac{(b\,h_1 + h_1^2\cot\alpha\pi)^{5/3}}{(b + 2h\operatorname{cosec}\alpha\pi)^{2/3}}\right] = 0 \quad (4.12)$$

From Eq. (4.12) the only unknown h_1 is obtained by iteration.

Incorporating Eq. (4.9) in Eq. (4.13) for replacing A*

$$F = Q_0 - k\,\{(b + 2\,(h_0 + h_1)\cot\alpha\pi) + \left[\left(C^* - \frac{b + (h_0 + h_1)\cot\,\alpha\pi}{\frac{1}{2}(h_0 + h_1)}\right)\frac{1}{2}(h_0 + h_1)\right] - \frac{1}{n}\frac{(b\,h_1 + h_1^2\,\cot\,\alpha)^{5/3}}{(b + 2\,h_1\,\operatorname{cosec}\,\alpha)^{2/3}} \quad (4.13)$$

$$Q_0 = \frac{(b\,h_0 + h_0\,\cot\,\alpha)^{5/3}}{(b + 2\,h_0\,\operatorname{cosec}\alpha)^{2/3}} \quad (4.14)$$

h_o is computed for known Q_0

Thus for a given value of bed width, length, roughness, hydraulic conductivity ,angle of the canal and bank slope, the unknown h_1 is known and seepage loss, a function of $\left(\frac{h_0 + h_1}{2}\right)$ is computed.

IV.4 RESULT AND DISCUSION

For computing dimensions of the canal, whose section conform to a trapezoidal one, the parameter and data required are:

1. Discharge at entrance
2. The hydraulic conductivity, k
3. Length along the canal
4. Bed slope of the canal
5. Bed width of the canal
6. Manning's roughness

Typical hydraulic conductivity values are given in table 4.1. The roughness coefficient (n) are given in table 2.1. The fraction of canal discharge getting lost as seepage loss is presented in Fig. (4.1.1a) through Fig. (4.5.2a). The fraction of discharge the canal losses is also decreases exponentially with distance. The losses decreases exponentially. The variations of dimensionless distance $\frac{q}{kb}$ with dimensionless distance $\frac{L}{b}$ for various inlet discharges Q_0 are presented in Fig. (4.1.1b) through Fig. (4.5.2b). The variation of q with L is shown in Fig. (4.1.1c) through (4.5.2c) for various discharges. The rate of seepage loss decreases with length as the depth of water decreases. Variations of depth of water along the length of the canal due to seepage losses for different discharges at the canal inlet, are presented in Fig.(4.1.1d) through (4.5.2d) The depth of water decrease in the depth of water due to seepage is quite significant. For example, for $Q_0 = 40\ m^3/sec$ the head decreases from 4.18 m at inlet to 2.09 m at a distance of 2000 m at m = 0.5, and k = 0.001. This reduction in depth of water due to seepage loss would hamper irrigation water supply under gravity flow.

Table 4.1 Some typical Values of Coefficient of Permeability

Soil Type	Coefficient of permeability k, cm/sec.
Clean gravel	1.0 and grater
Clean sand (coarse)	1.0 - 0.01
Sand (mixture)	0.01 - 0.005
Fine sand	0.05 - 0.001
Silty sand	0.002 - 0.0001
Silt	0.0005 - 0.00001
Clay	0.000001 and smaller

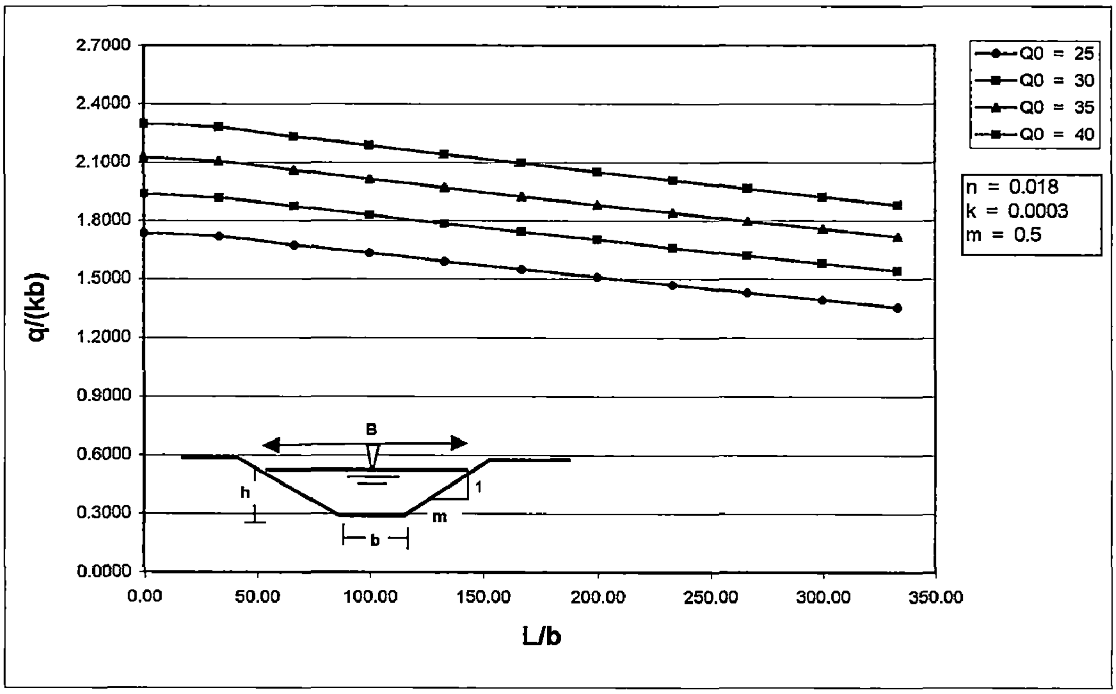


Fig. 4.1.2a. Variation of seepage losses with length of the canal for different discharges

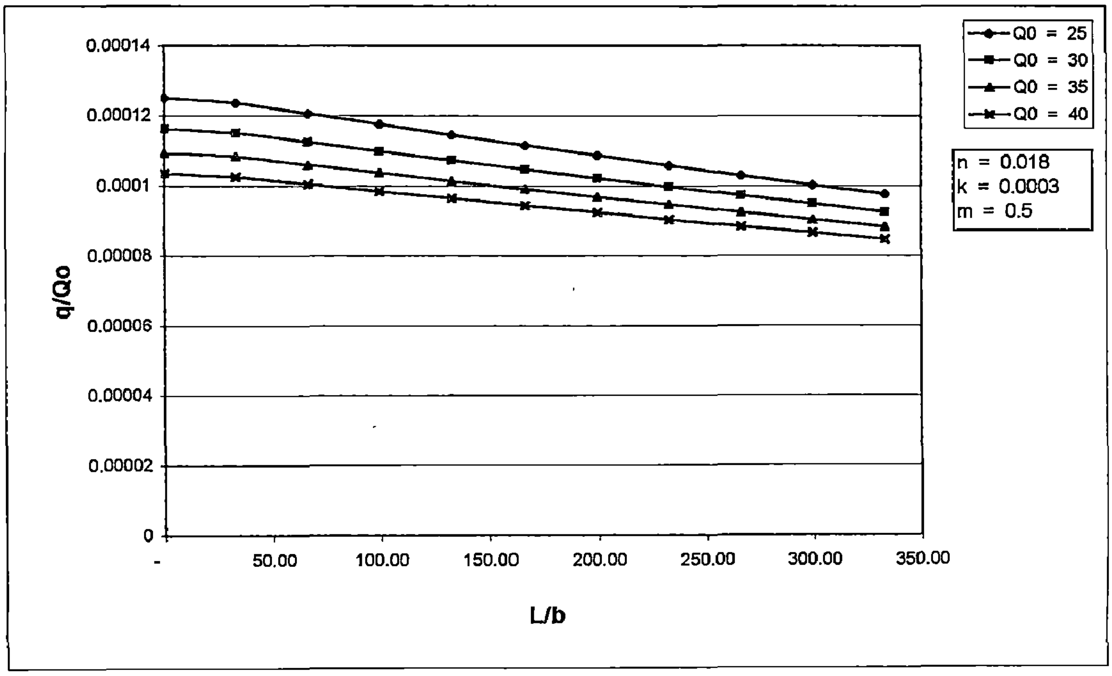


Fig. 4.1.2b. Relation between q/Qo and L/b for different discharges

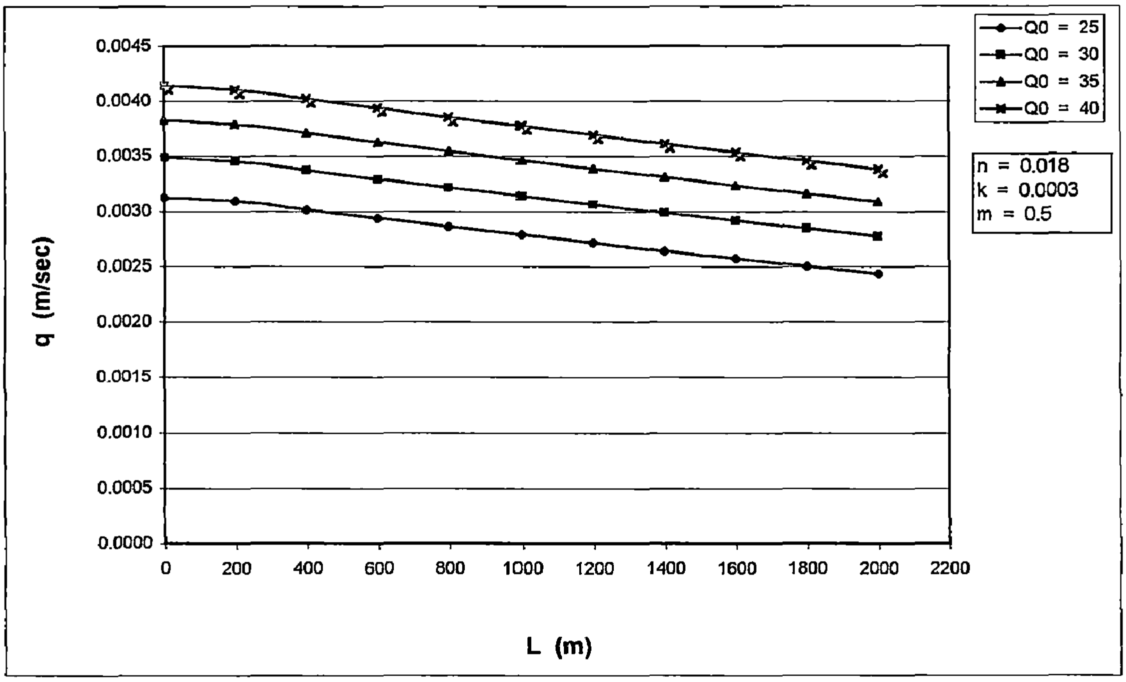


Fig. 4.1.2c. **Relation between seepage losses with length of the canal for different discharges**

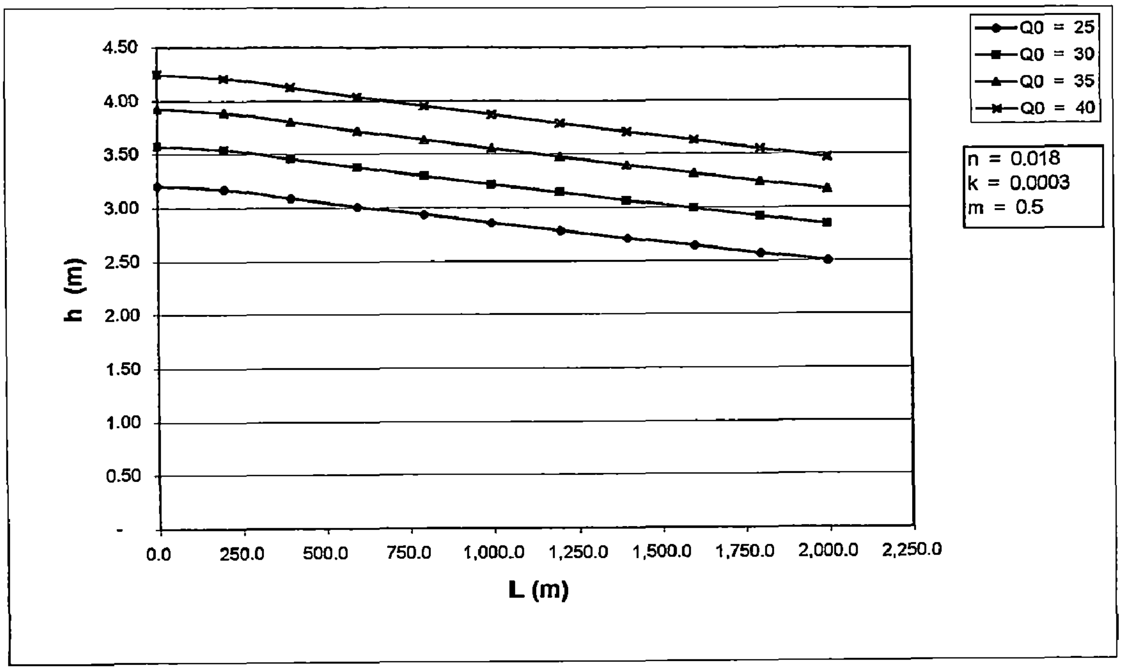


Fig. 4.1.2d. **Relation between depth of water with length of the canal for different discharges**

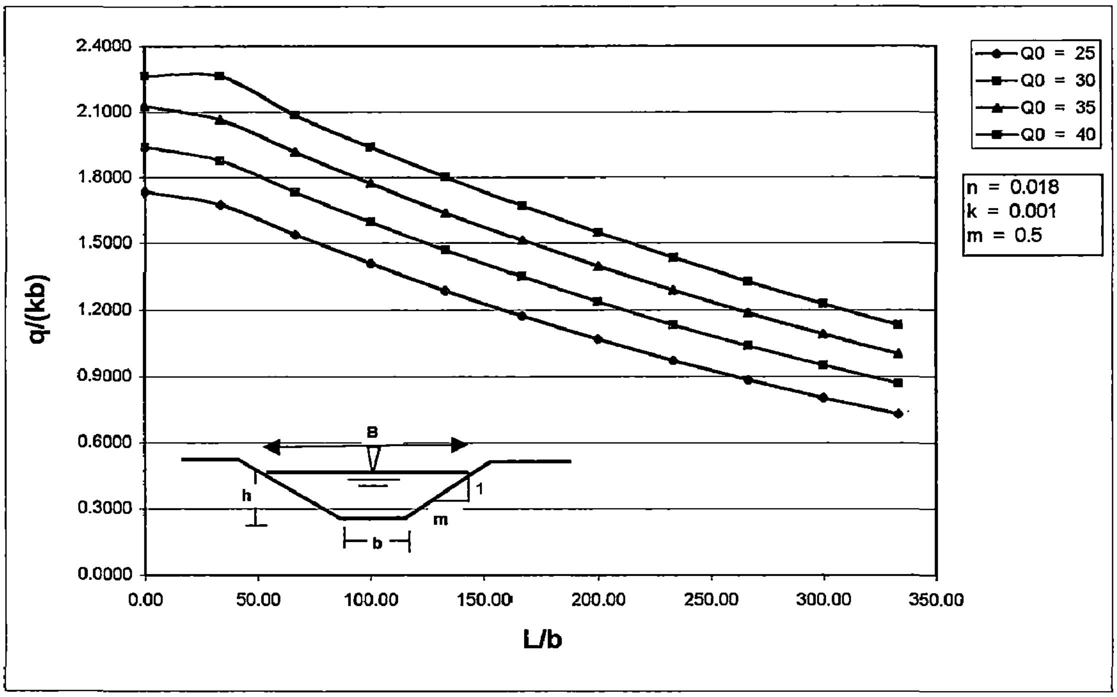


Fig. 4.1.1a. Variation of seepage losses with length of the canal for different discharges

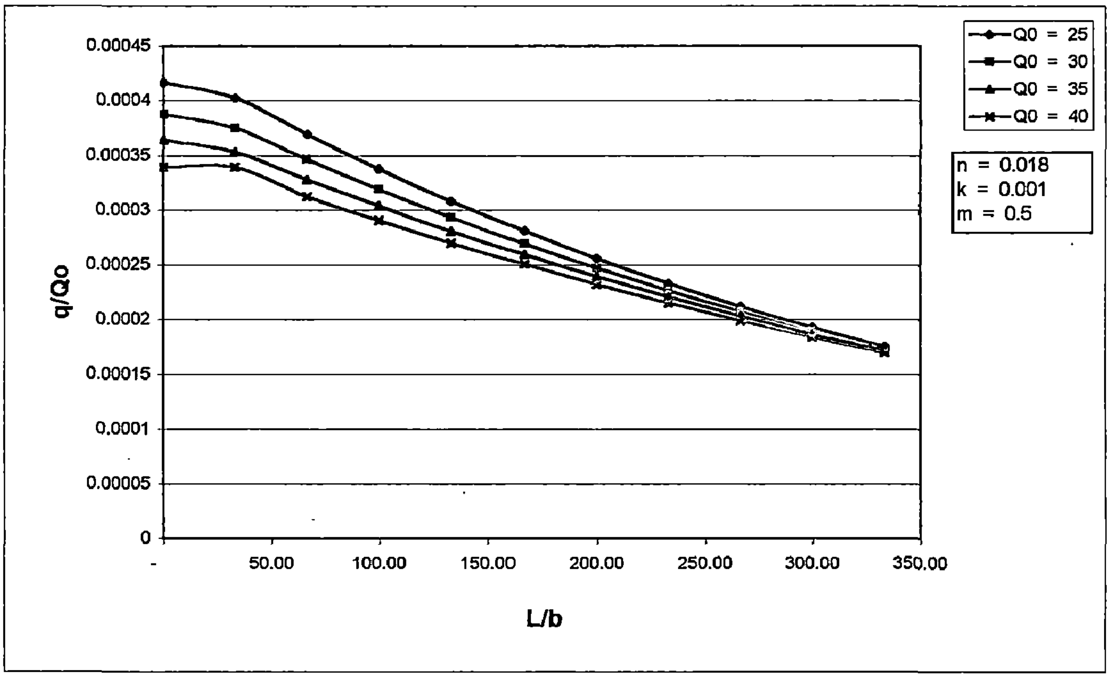


Fig. 4.1.1b. Relation between q/Qo and L/b for different discharges

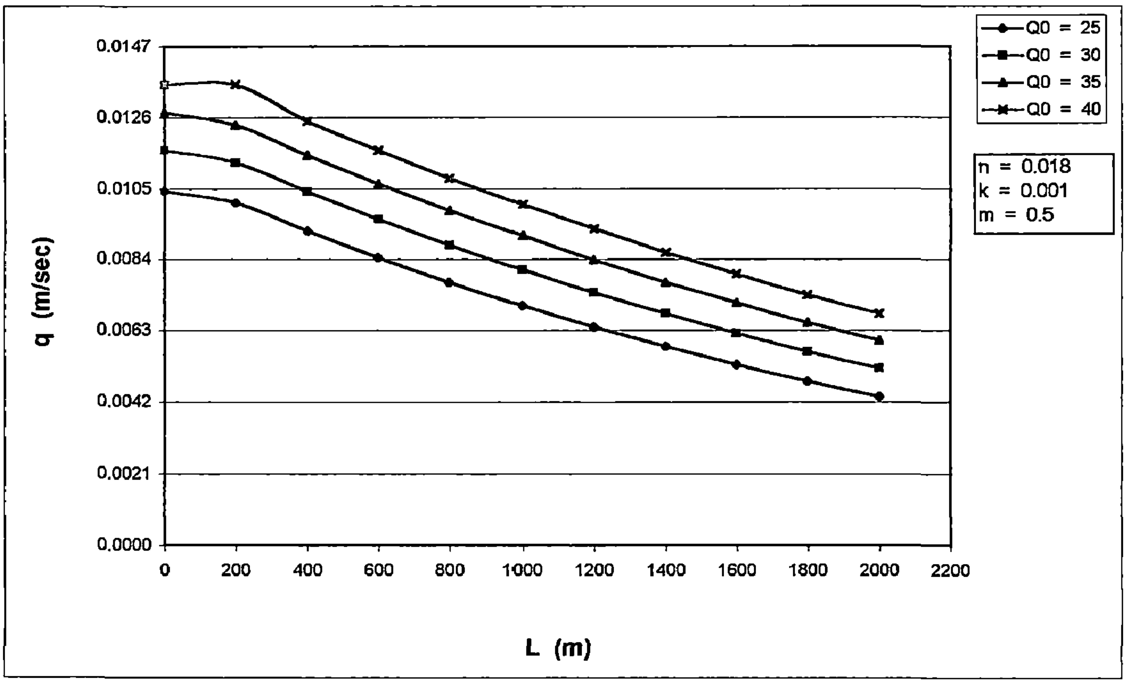


Fig. 4.1.1c. **Relation between seepage losses with length of the canal for different discharges**

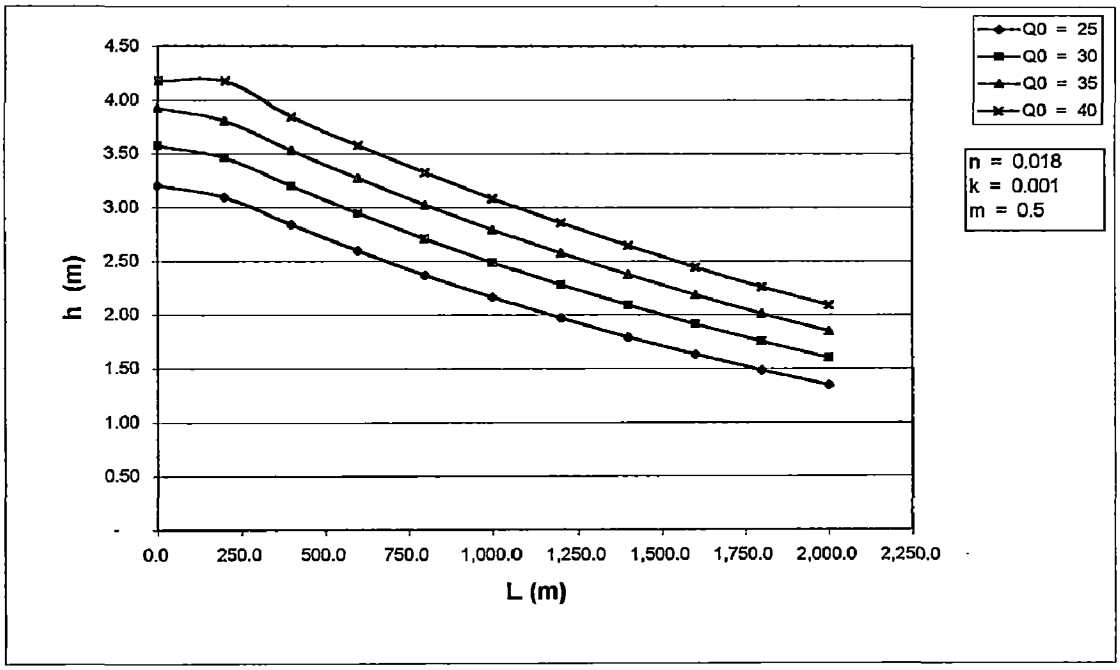


Fig. 4.1.1d. **Relation between depth of water with length of the canal for different discharges**

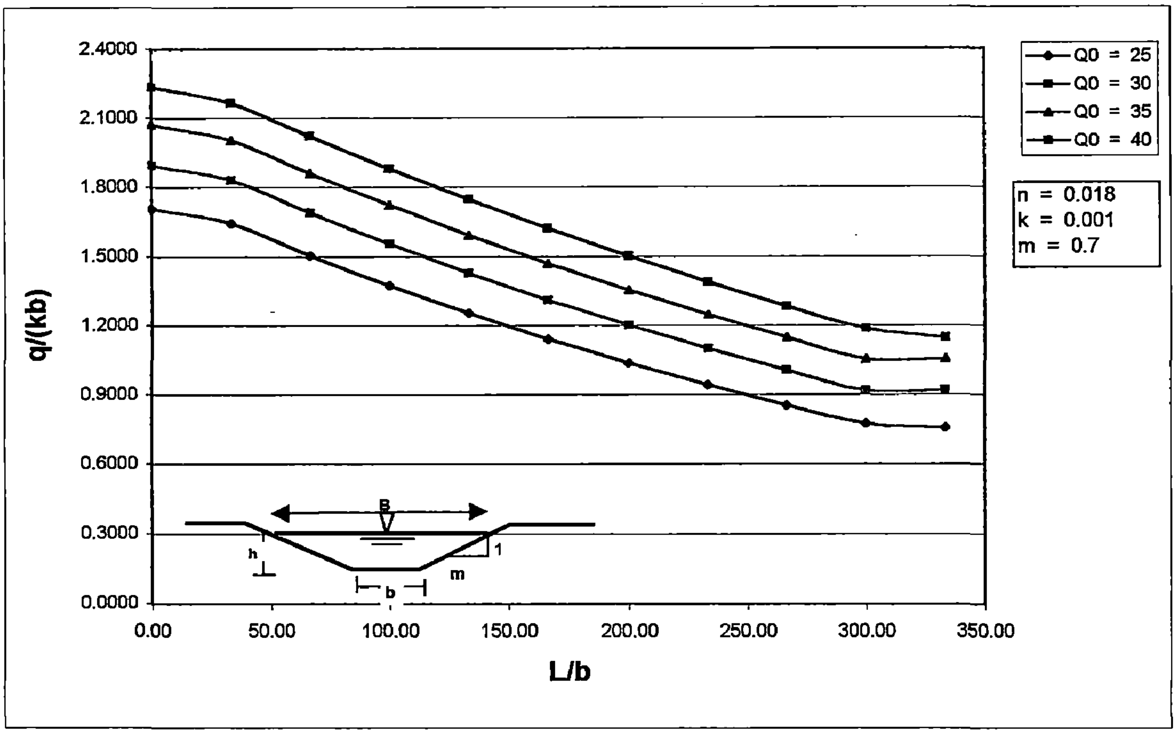


Fig. 4.2.1a. Variation of seepage losses with length of the canal for different discharges

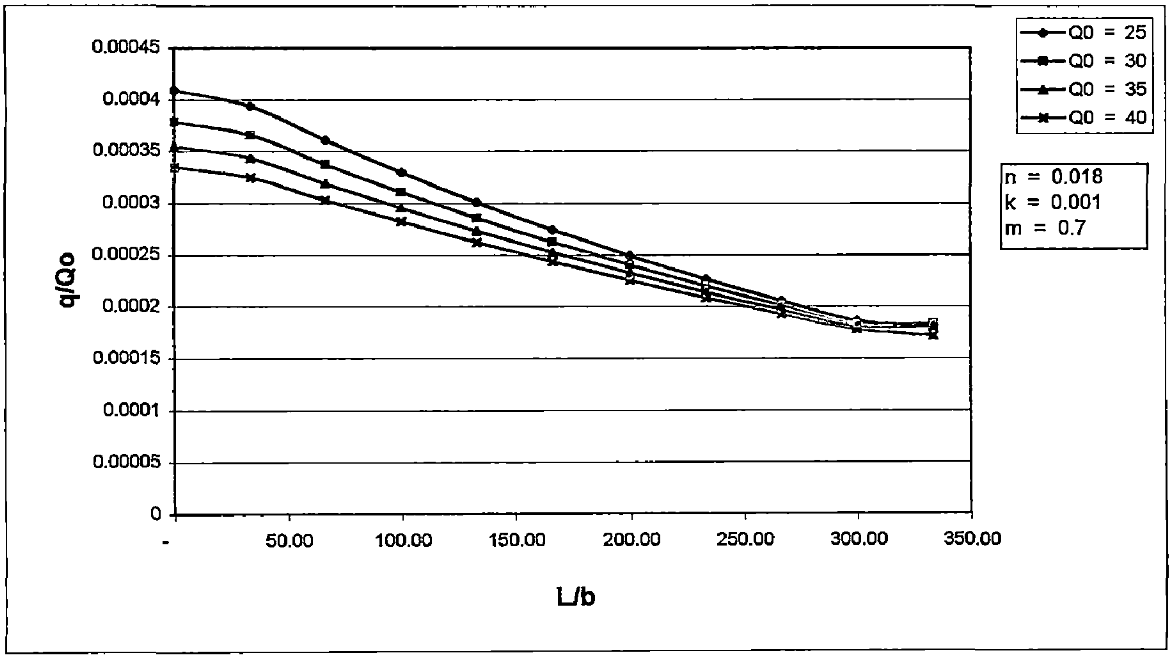


Fig. 4.2.1b. Relation between q/Qo and L/b for different discharges

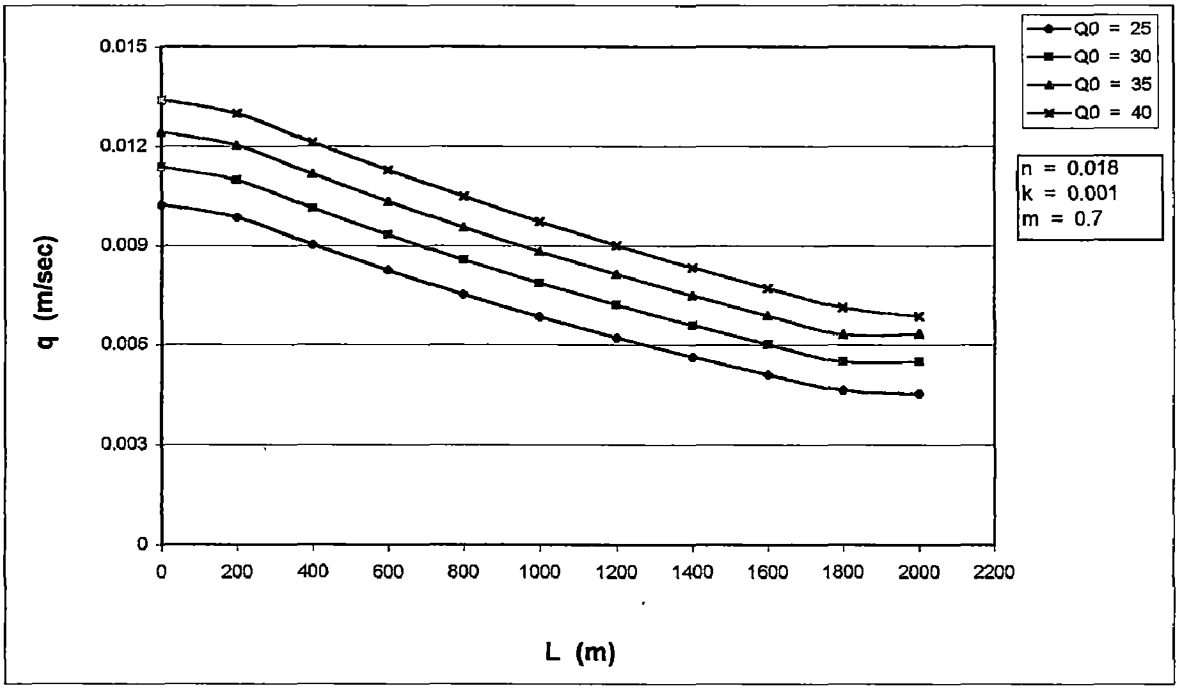


Fig. 4.2.1c. **Relation between seepage losses with length of the canal for different discharges**

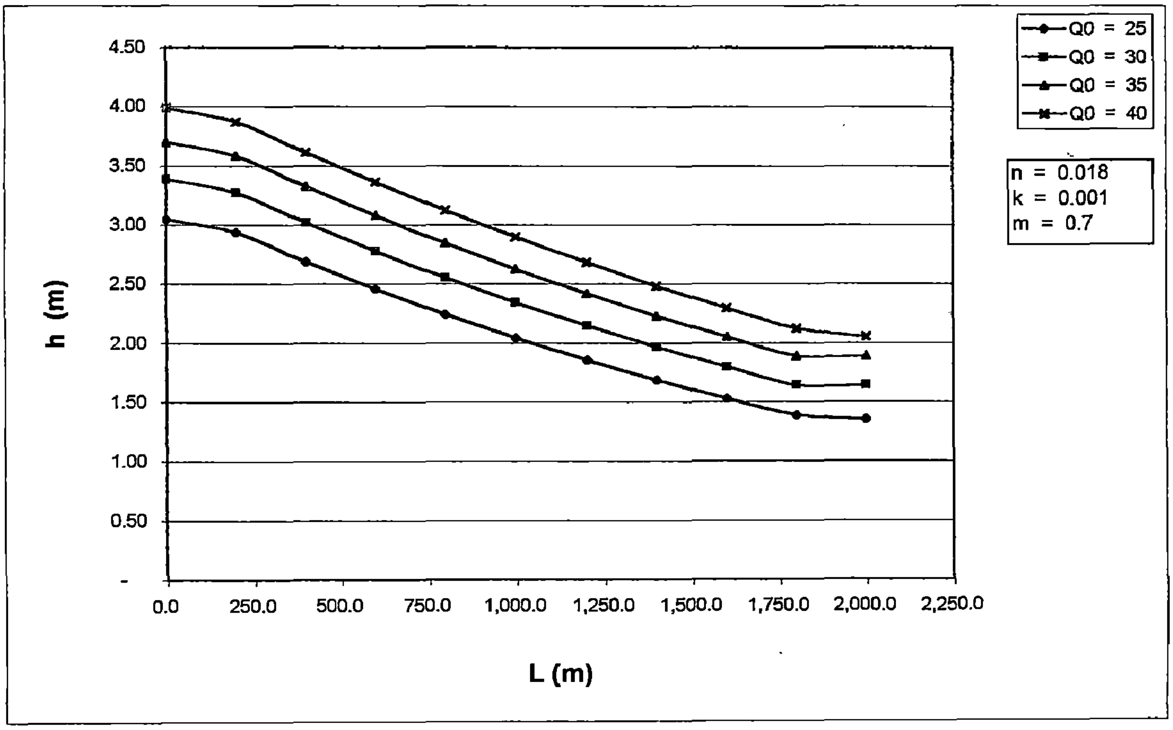


Fig. 4.2.1d. **Relation between depth of water with length of the canal for different discharges**

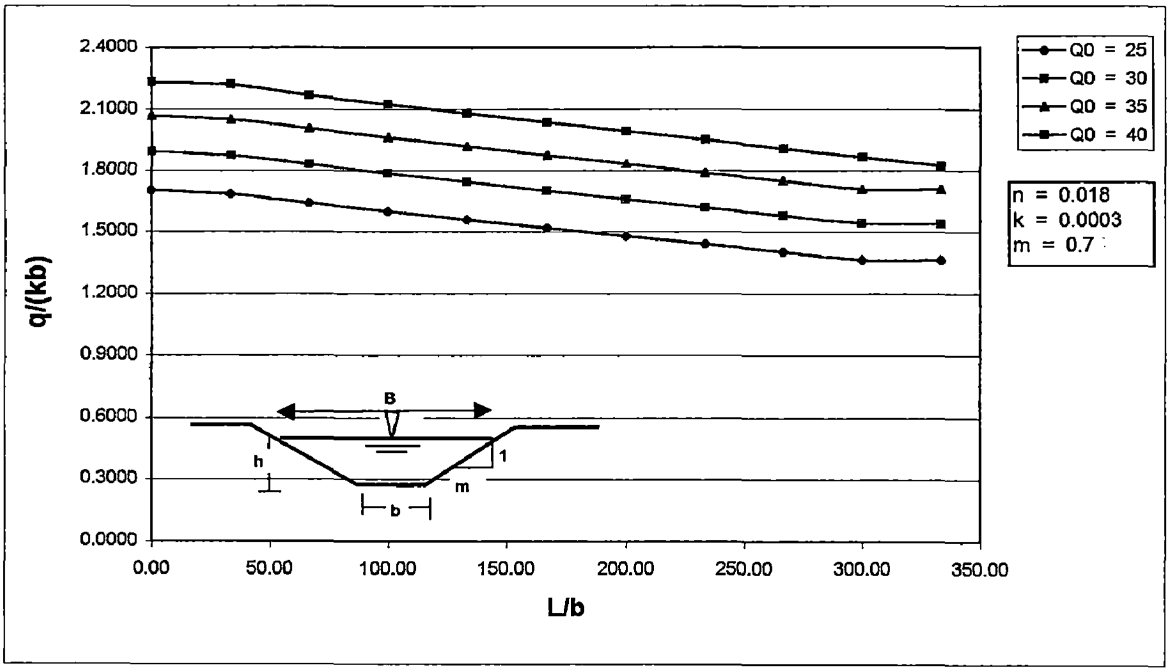


Fig. 4.2.2a. Variation of seepage losses with length of the canal for different discharges

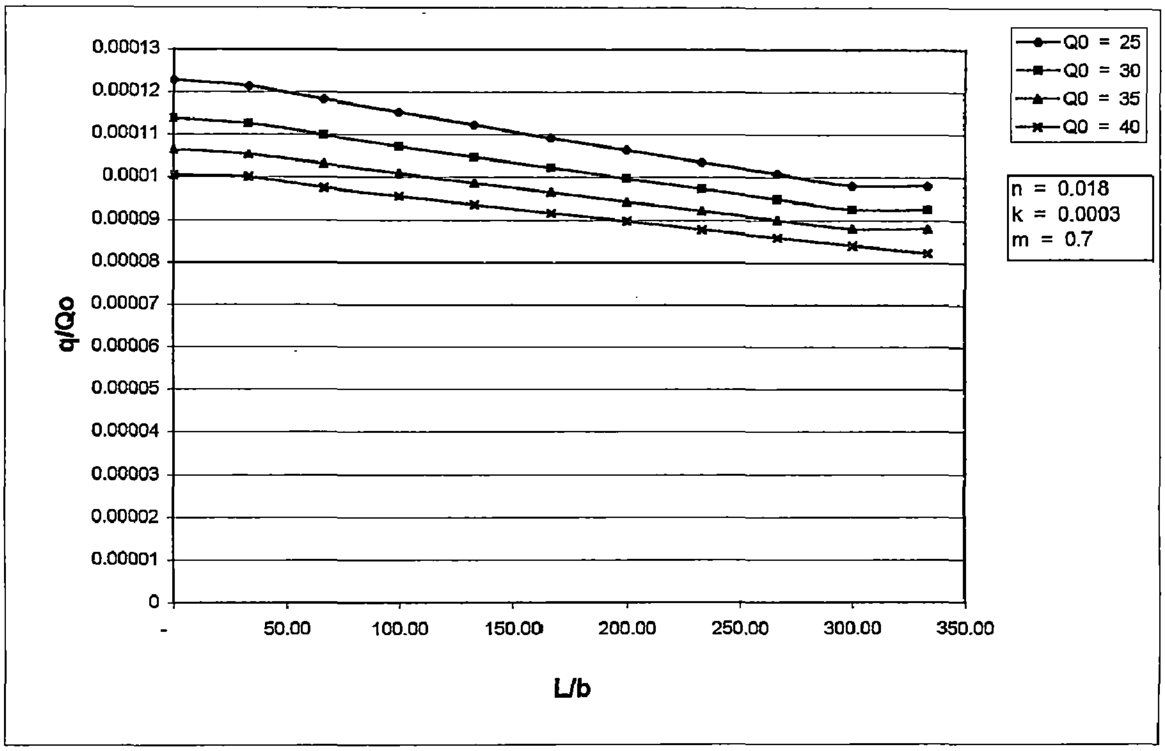


Fig. 4.2.2b. Relation between q/Qo and L/b for different discharges

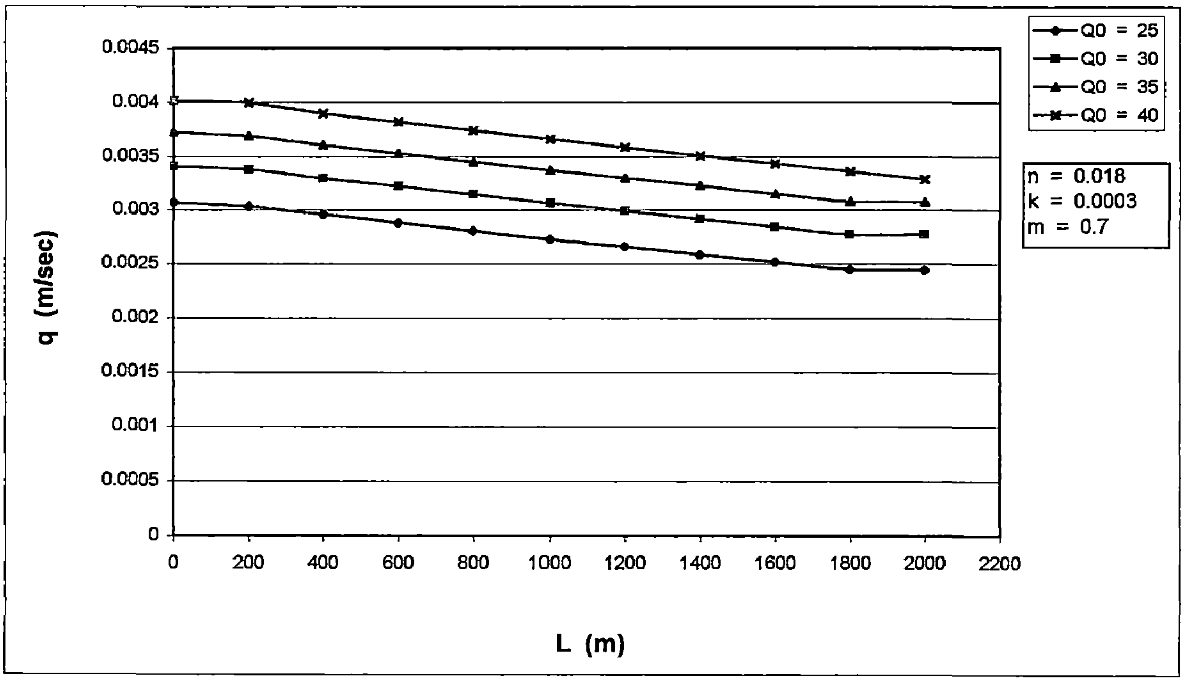


Fig. 4.2.2c. **Relation between seepage losses with length of the canal for different discharges**

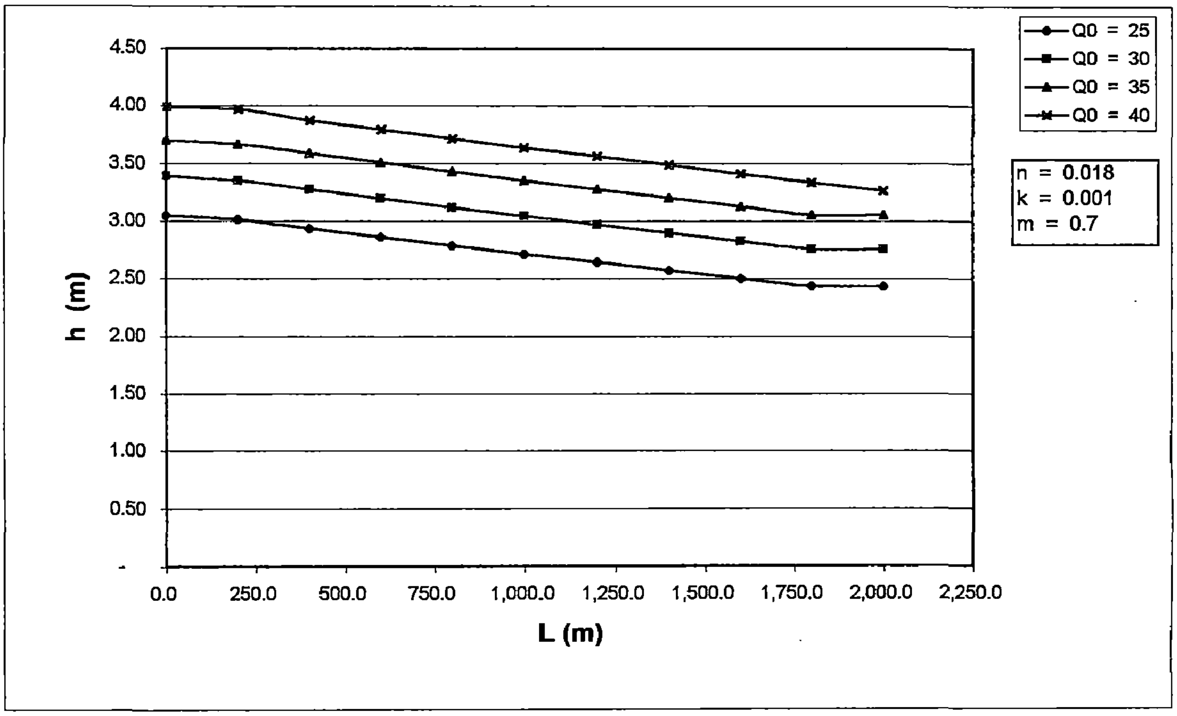


Fig. 4.2.2d. **Relation between depth of water with length of the canal for different discharges**

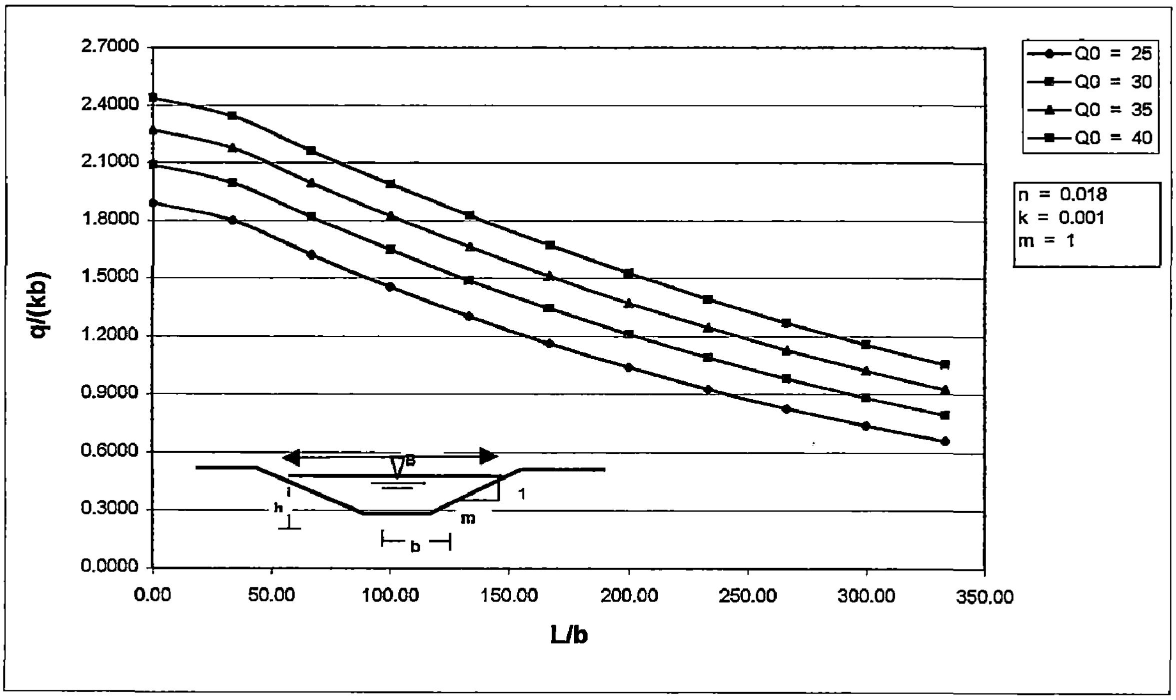


Fig. 4.3.1a. Variation of seepage losses with length of the canal for different discharges

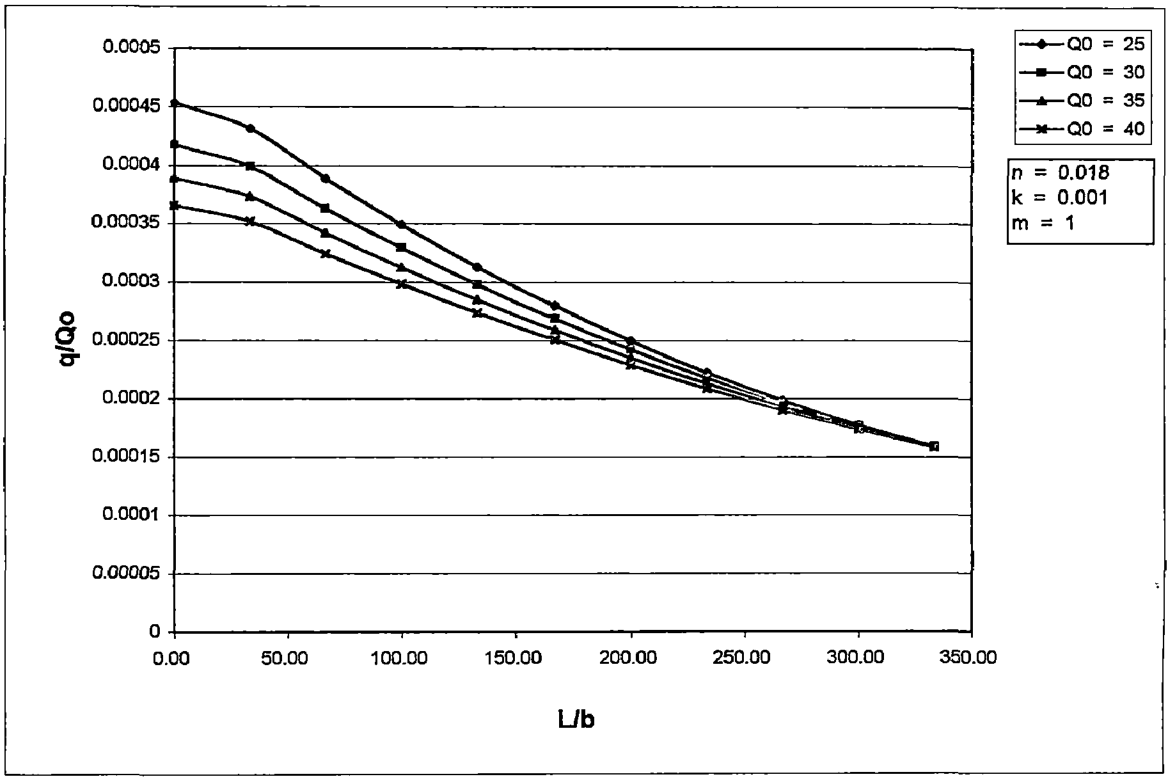


Fig. 4.3.1b. Relation between q/Qo and L/b for different discharges

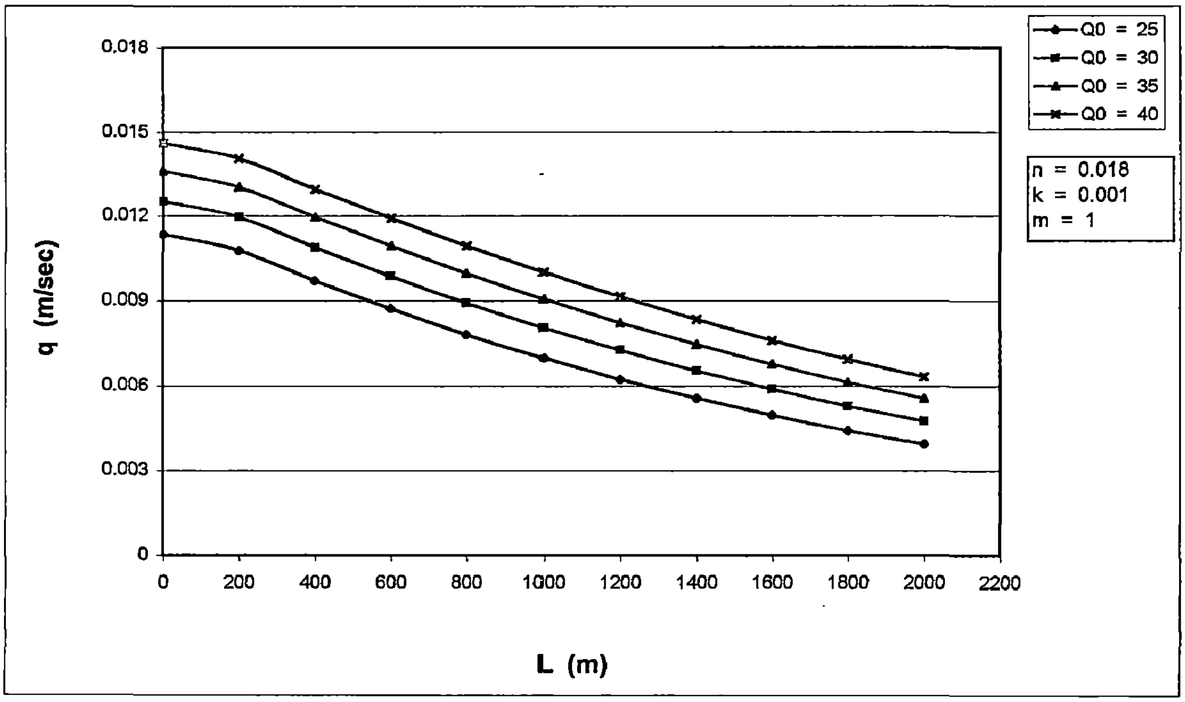


Fig. 4.3.1c. **Relation between seepage losses with length of the canal for different discharges**

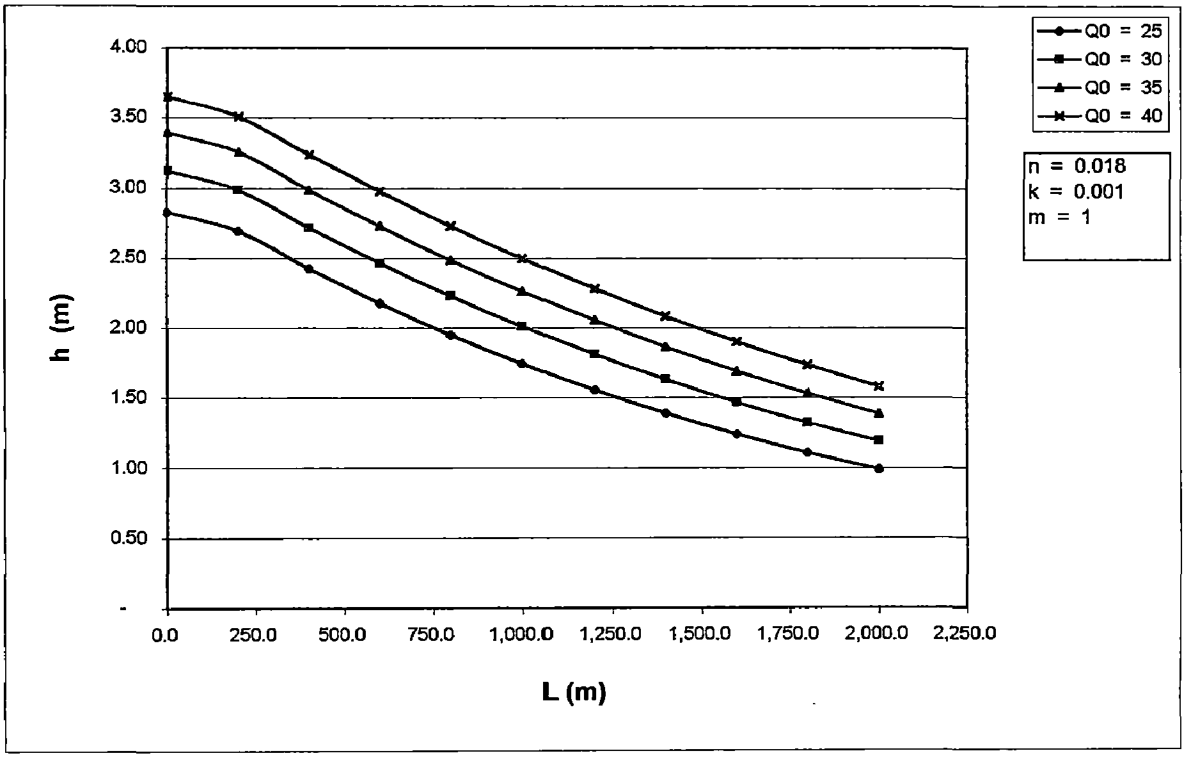


Fig. 4.3.1d. **Relation between depth of water with length of the canal for different discharges**

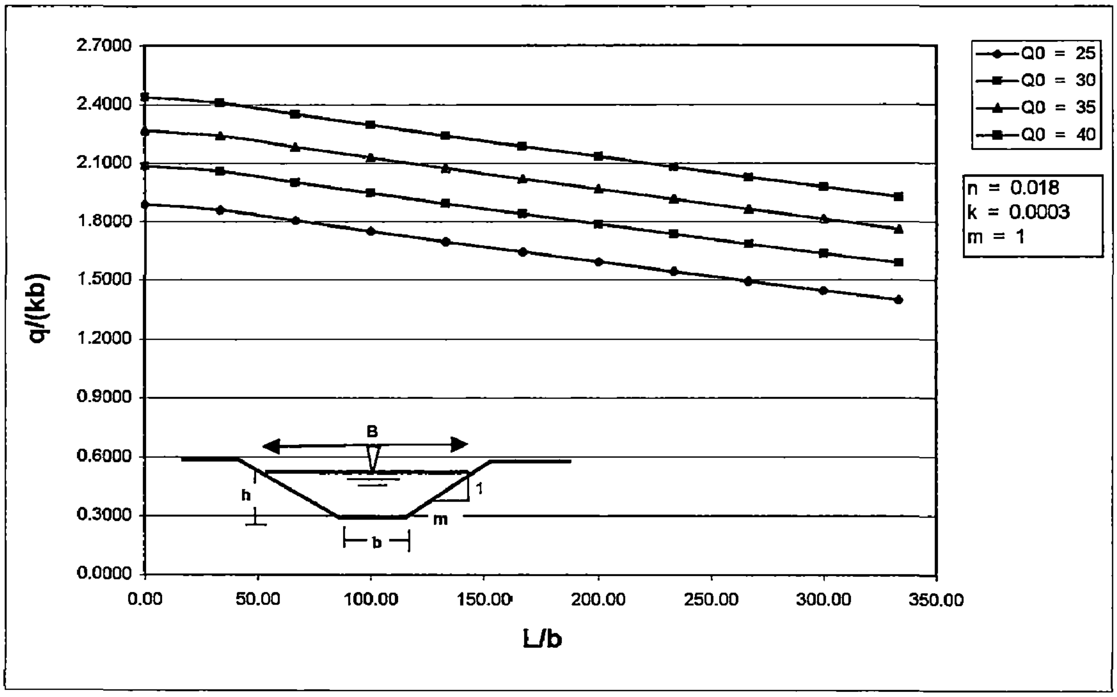


Fig. 4.3.2a. Variation of seepage losses with length of the canal for different discharges

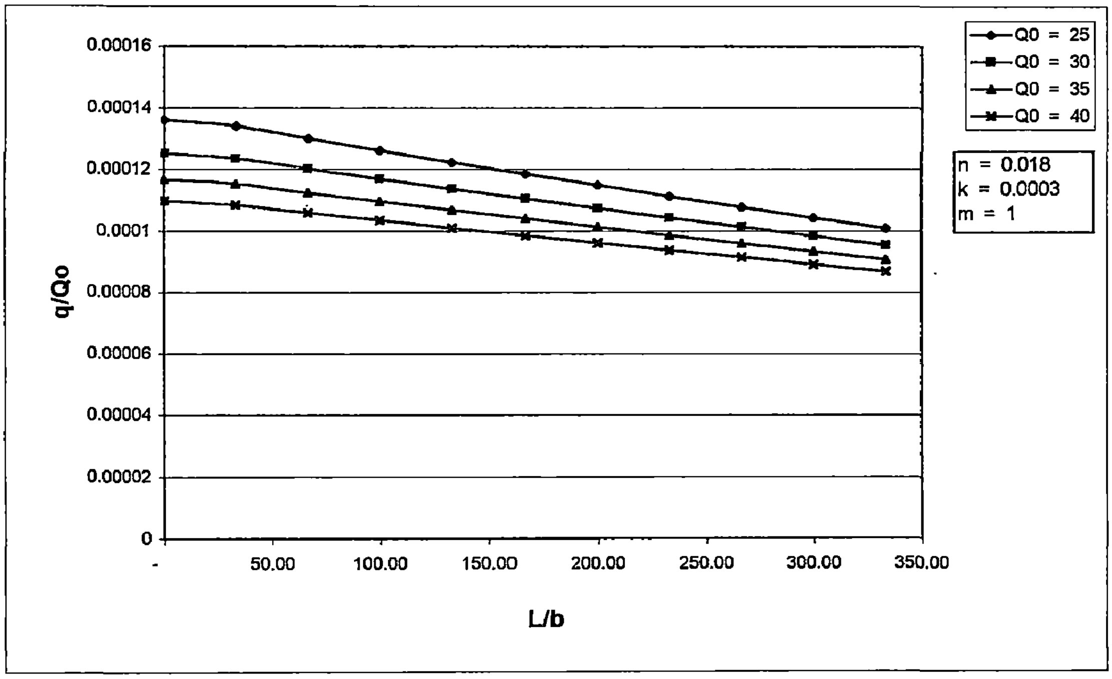


Fig. 4.3.2b. Relation between q/Qo and L/b for different discharges

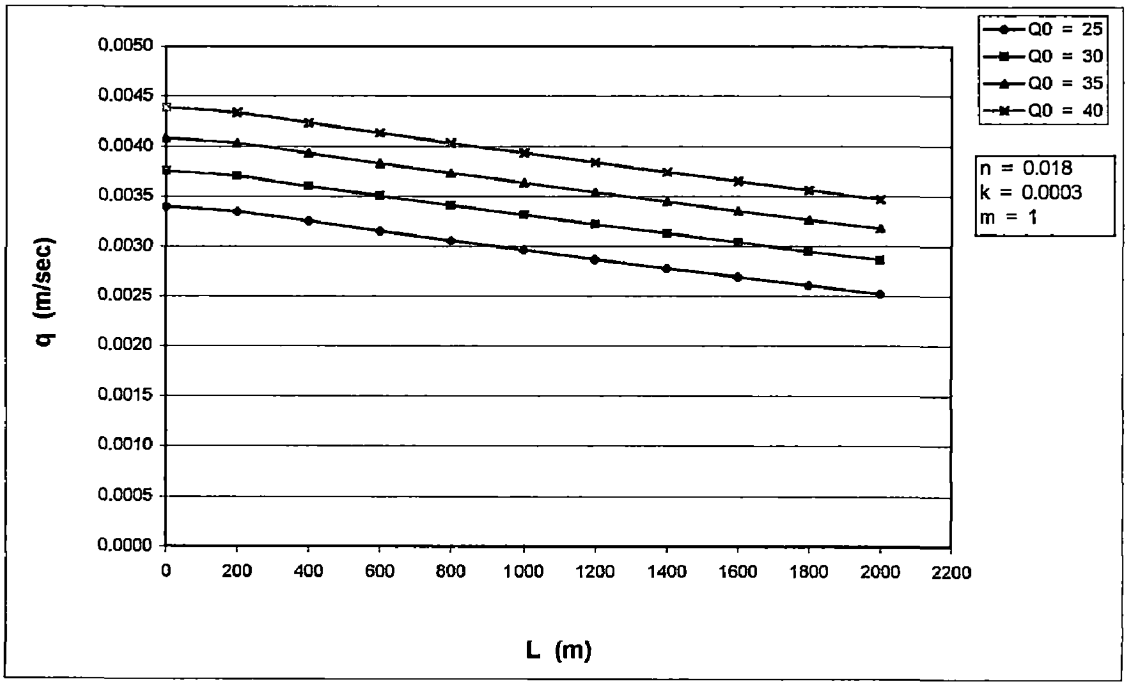


Fig. 4.3.2c. **Relation between seepage losses with length of the canal for different discharges**

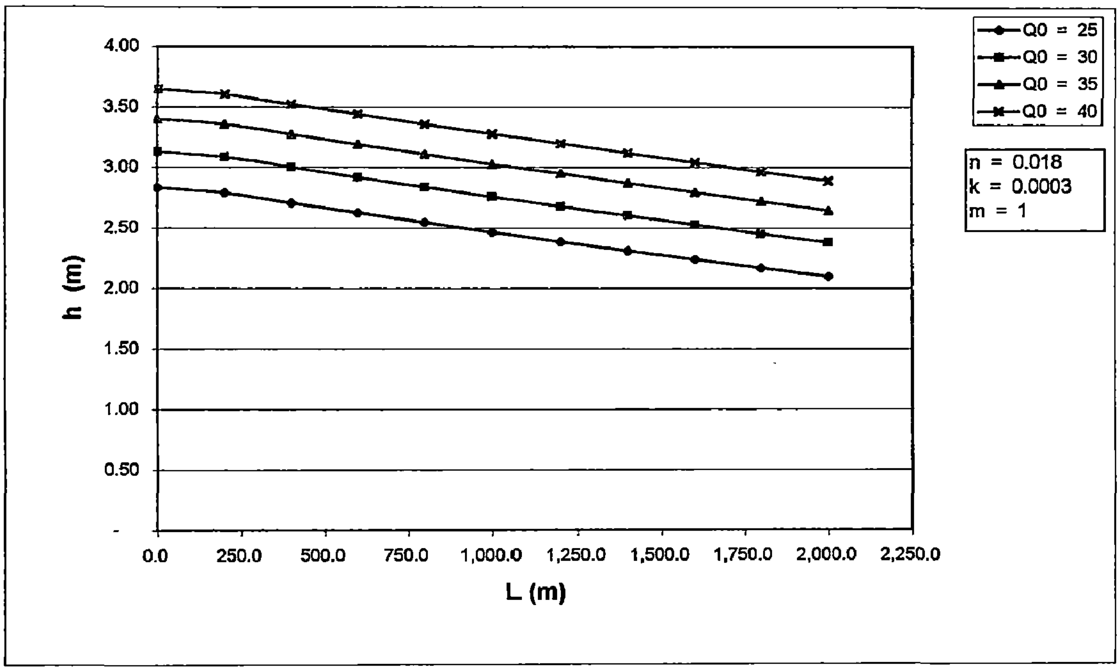


Fig. 4.3.2d. **Relation between depth of water with length of the canal for different discharges**

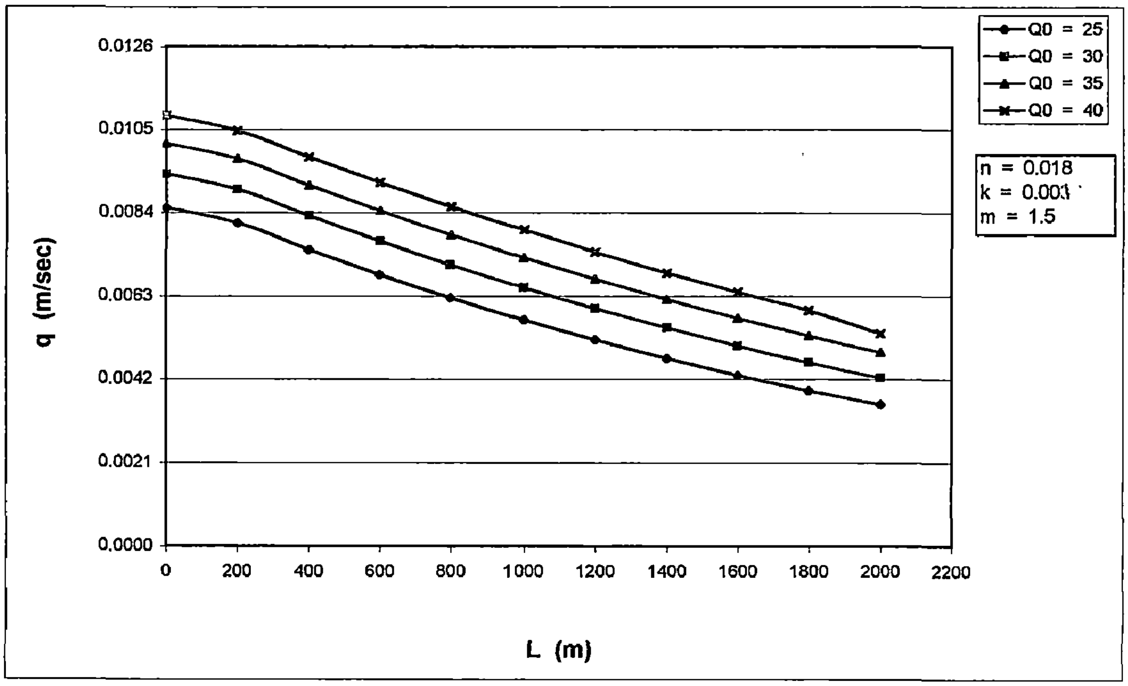


Fig. 4.4.1c. **Relation between seepage losses with length of the canal for different discharges**

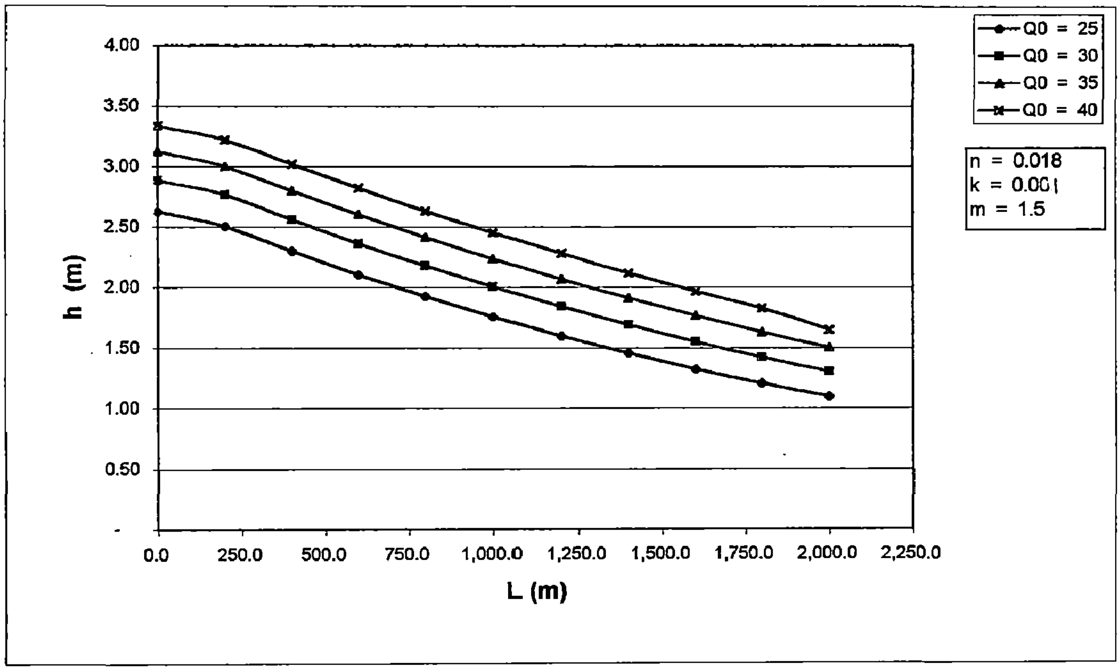


Fig. 4.4.1d. **Relation between depth of water with length of the canal for different discharges**

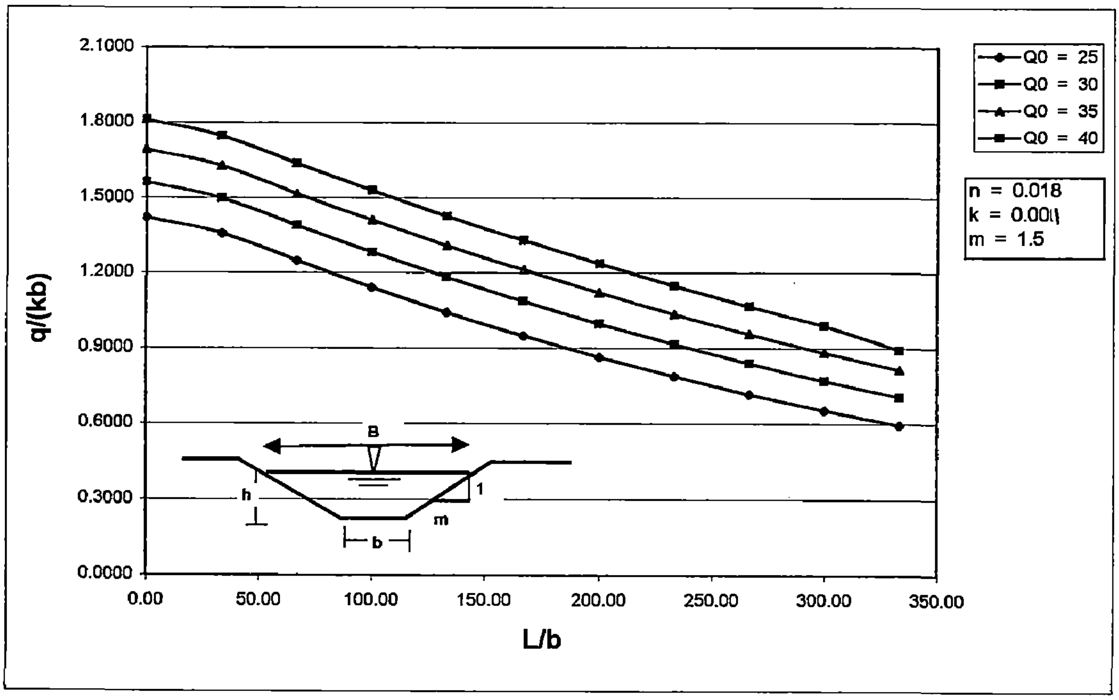


Fig. 4.4.1a. Variation of seepage losses with length of the canal for different discharges

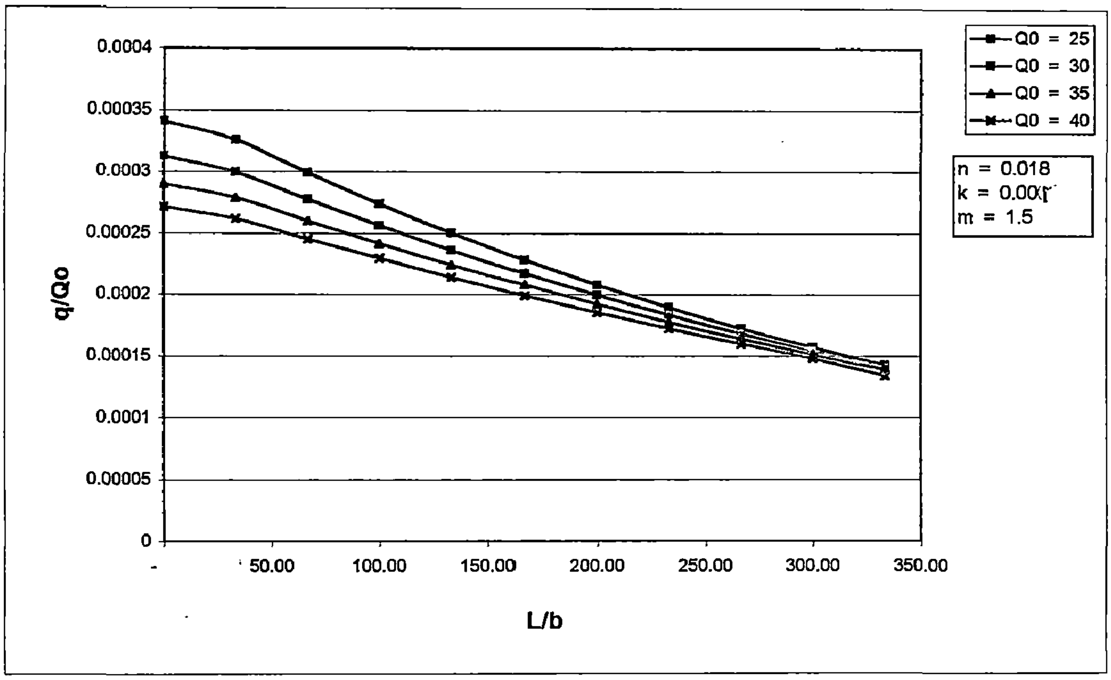


Fig. 4.4.1b. Relation between q/Qo and L/b for different discharges

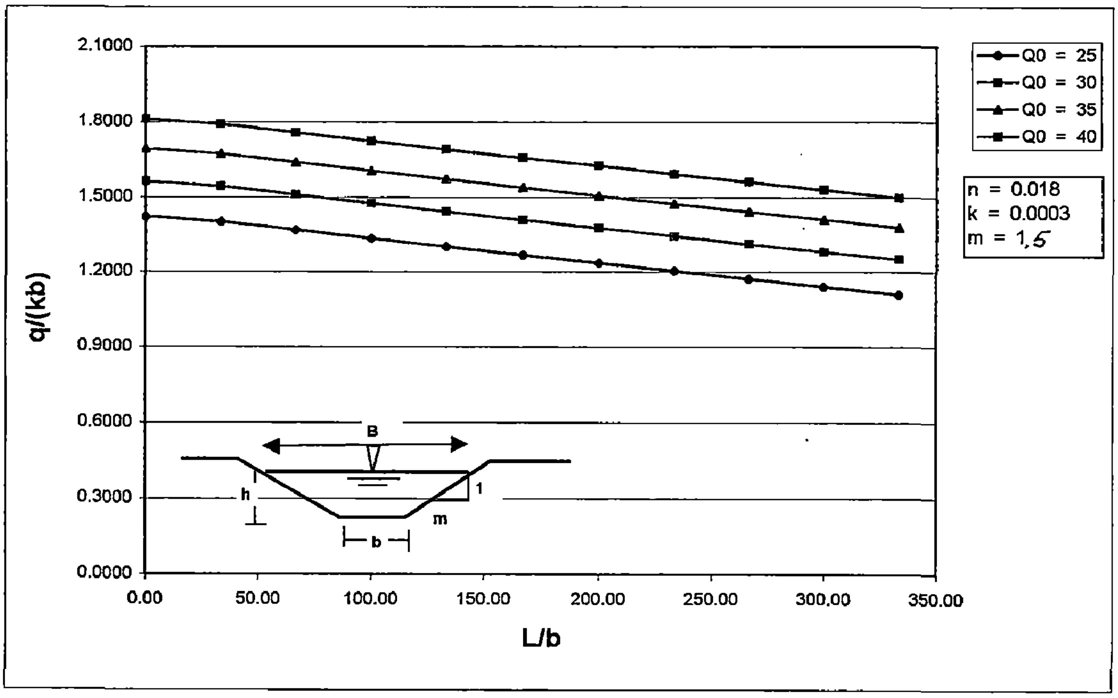


Fig. 4.4.2a. Variation of seepage losses with length of the canal for different discharges

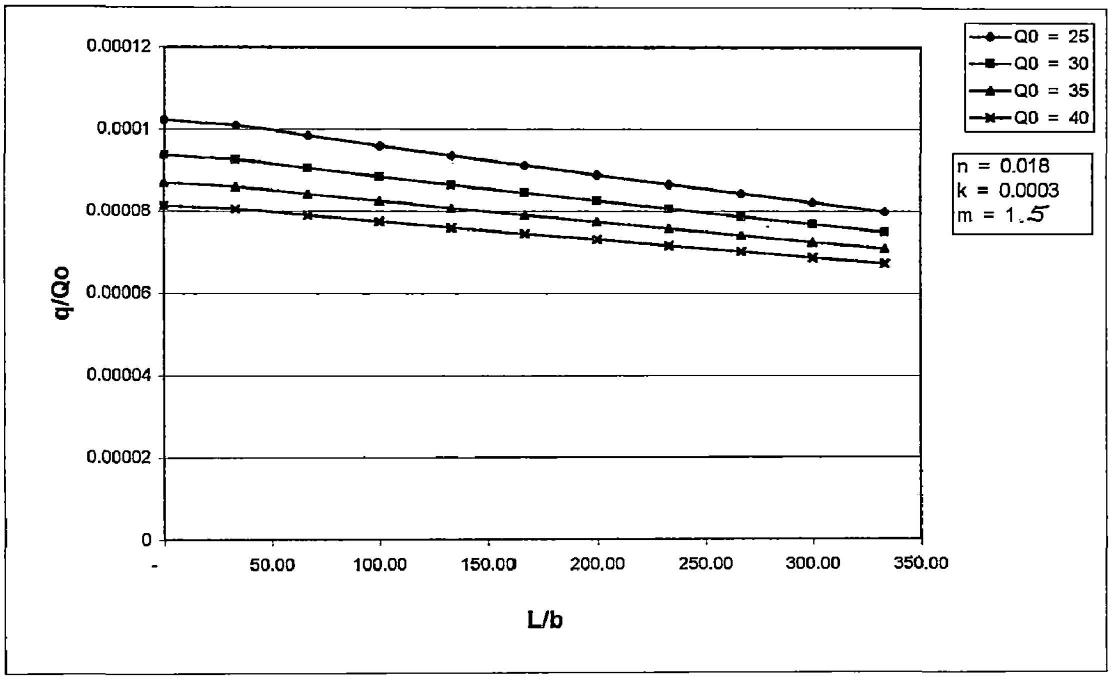


Fig. 4.4.2b. Relation between q/Qo and L/b for different discharges

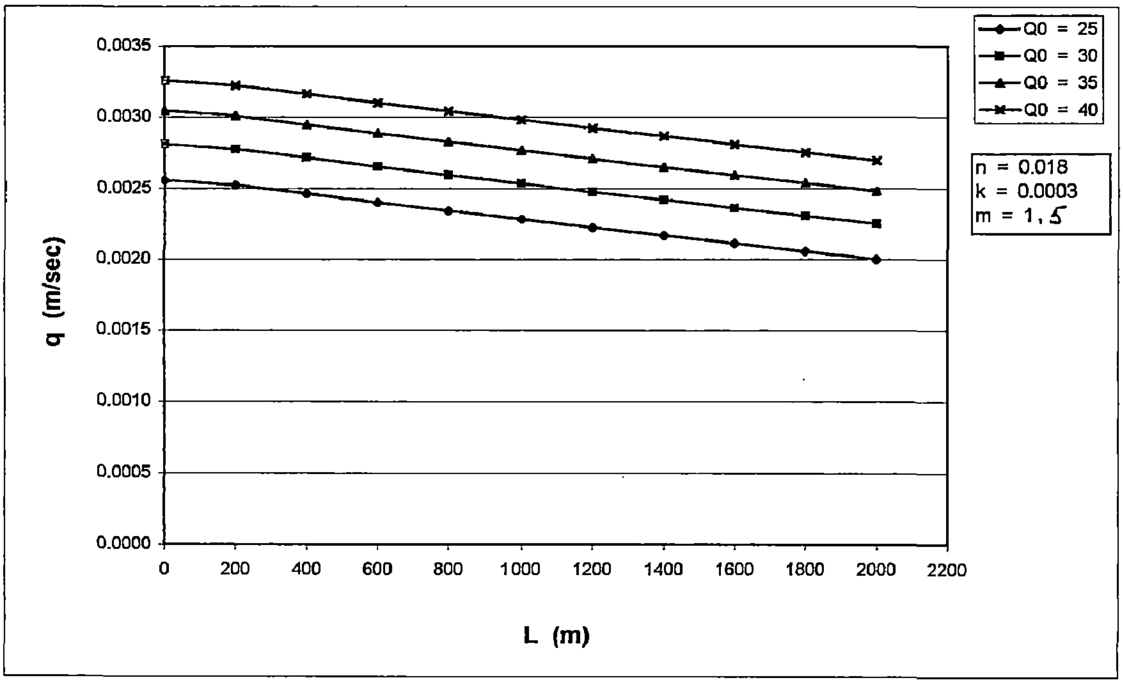


Fig. 4.4.2c. **Relation between seepage losses with length of the canal for different discharges**

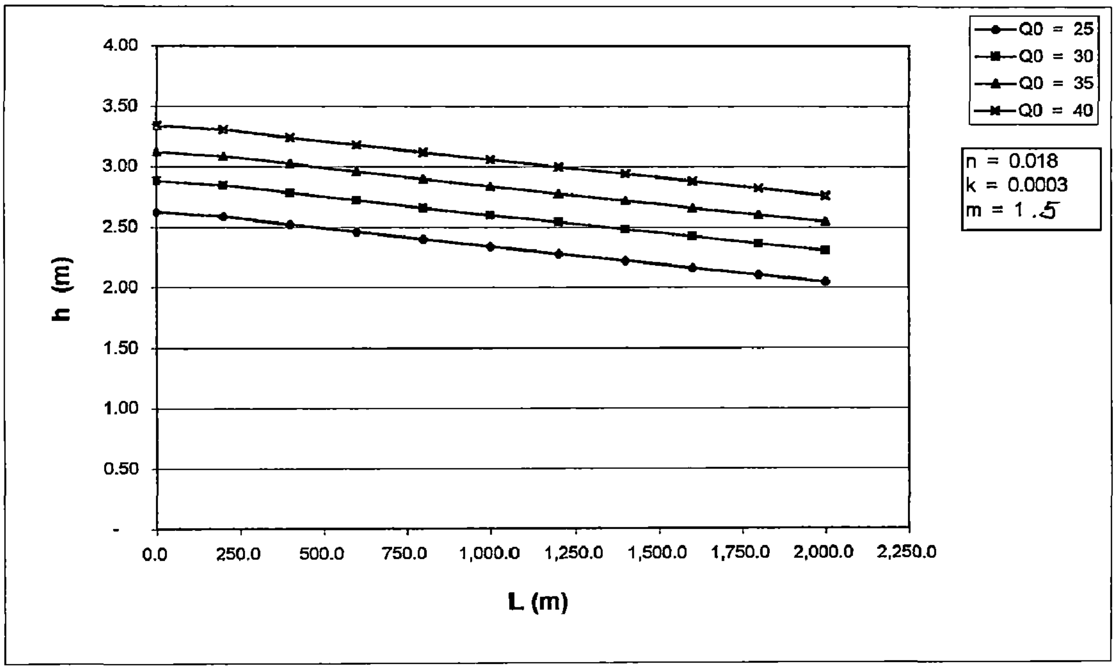


Fig. 4.4.2d. **Relation between depth of water with length of the canal for different discharges**

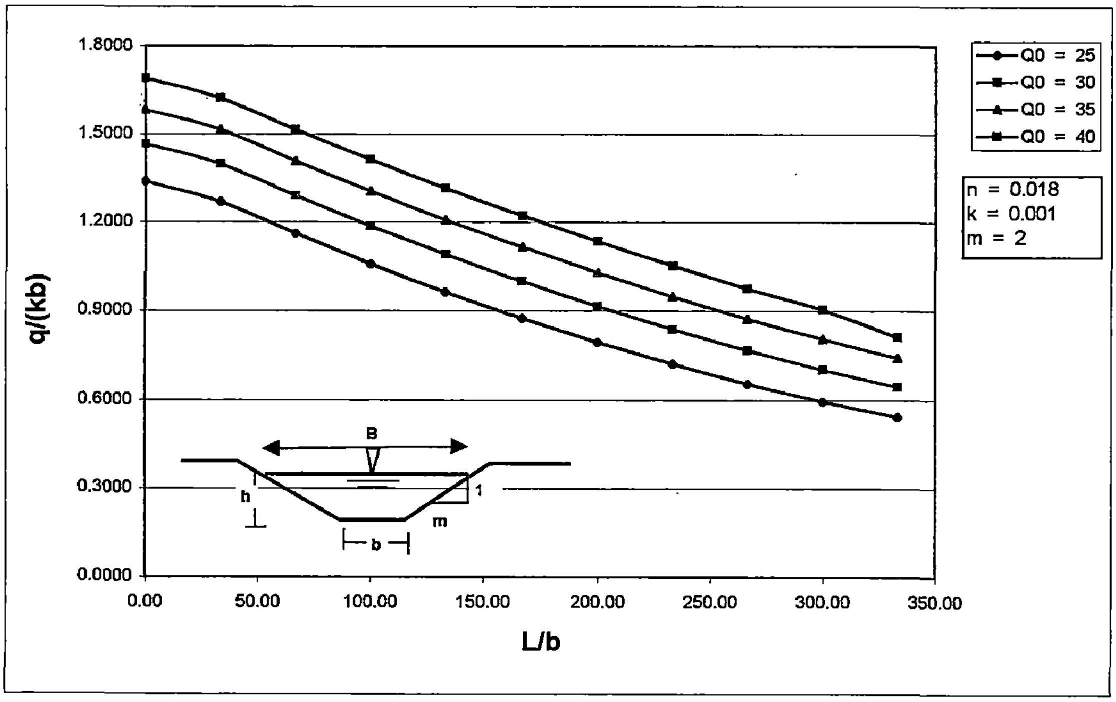


Fig. 4.5.1a. Variation of seepage losses with length of the canal for different discharges

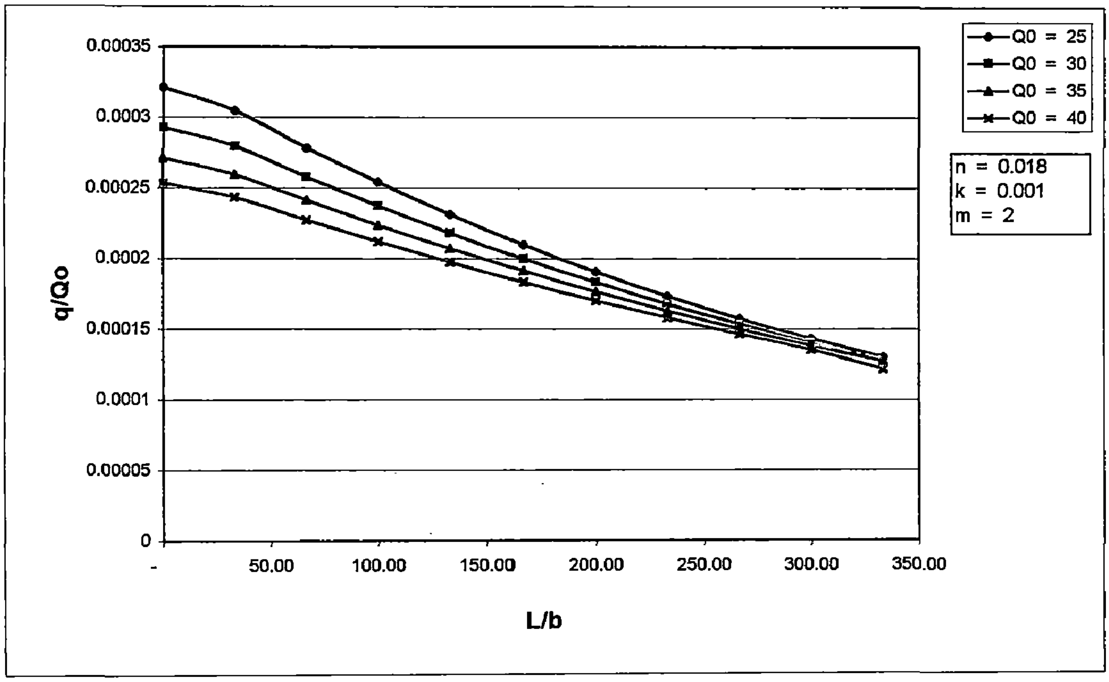


Fig. 4.5.1b. Relation between q/Qo and L/b for different discharges

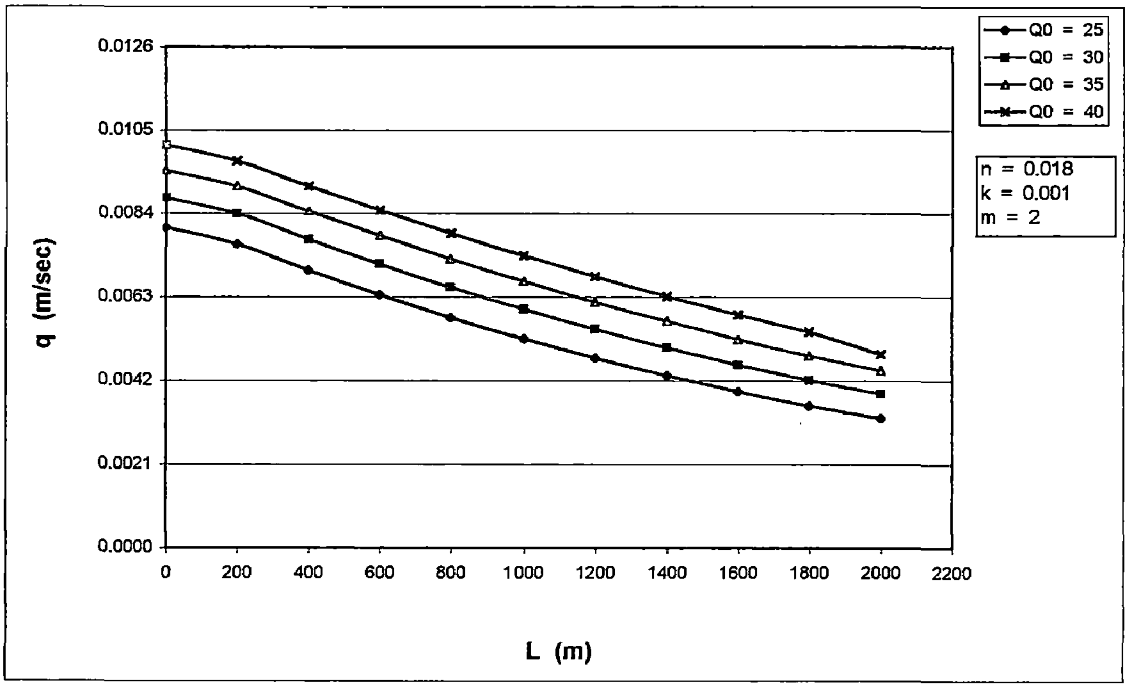


Fig. 4.5.1c. **Relation between seepage losses with length of the canal for different discharges**

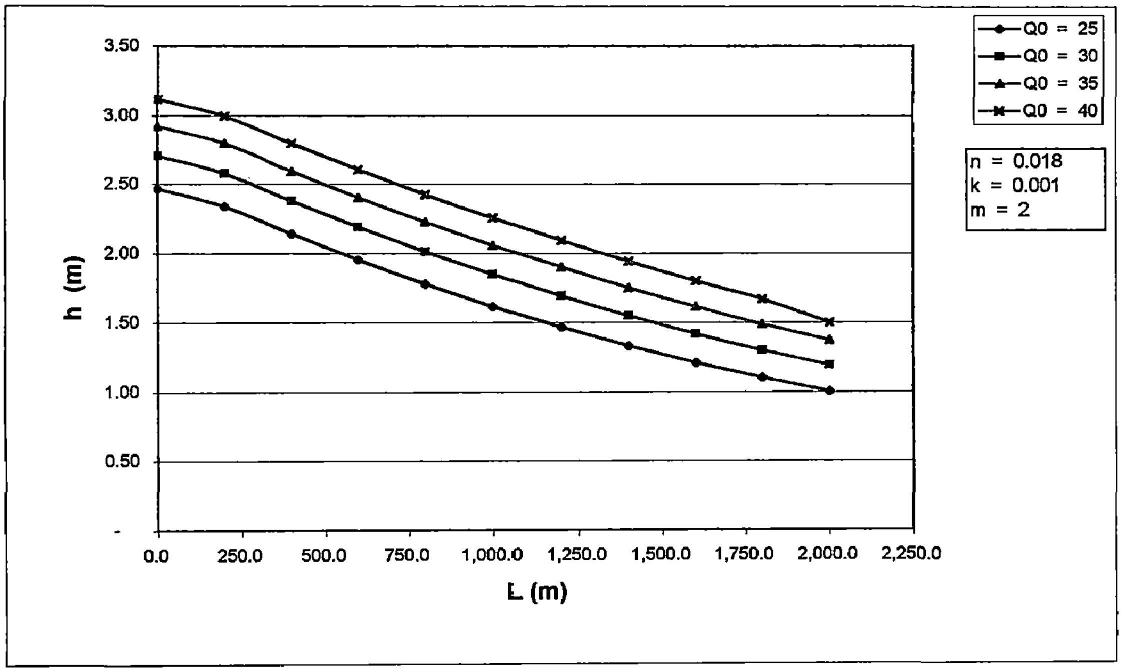


Fig. 4.5.1d. **Relation between depth of water with length of the canal for different discharges**

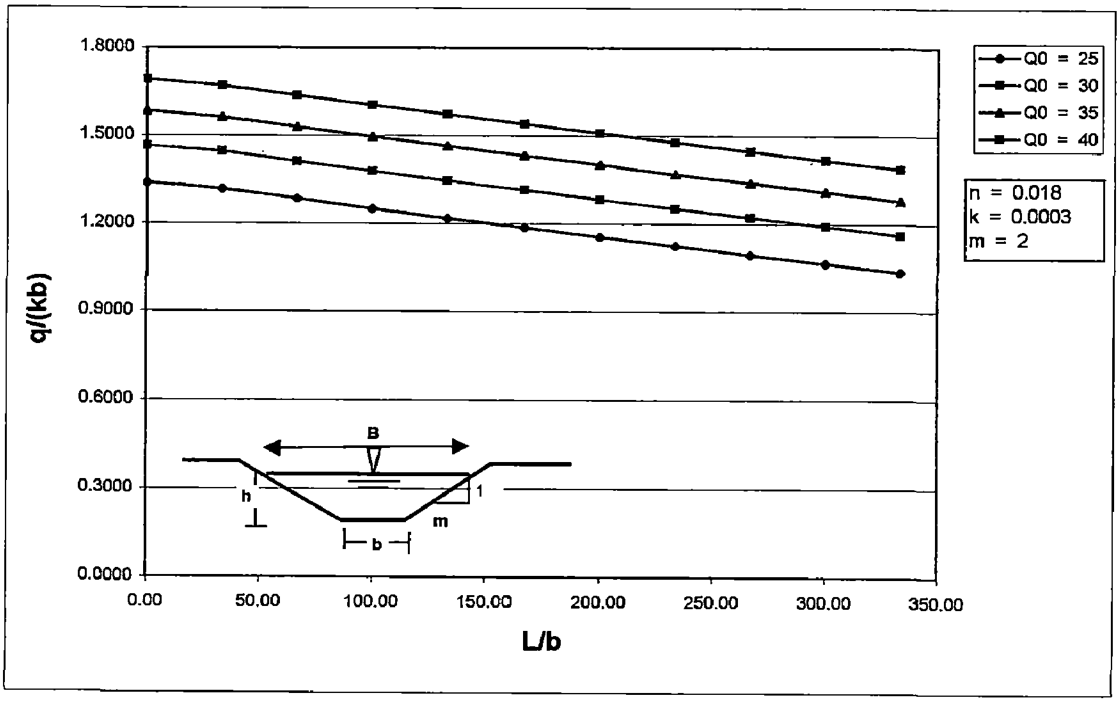


Fig. 4.5.2a. Variation of seepage losses with length of the canal for different discharges

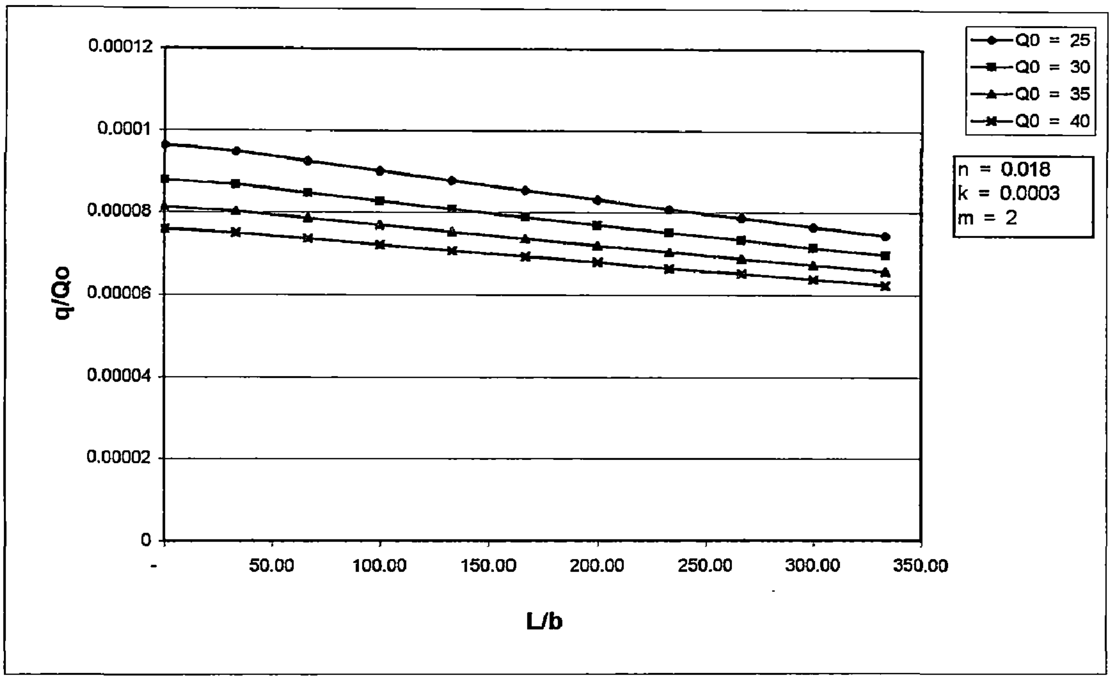


Fig. 4.5.2b. Relation between q/Qo and L/b for different discharges

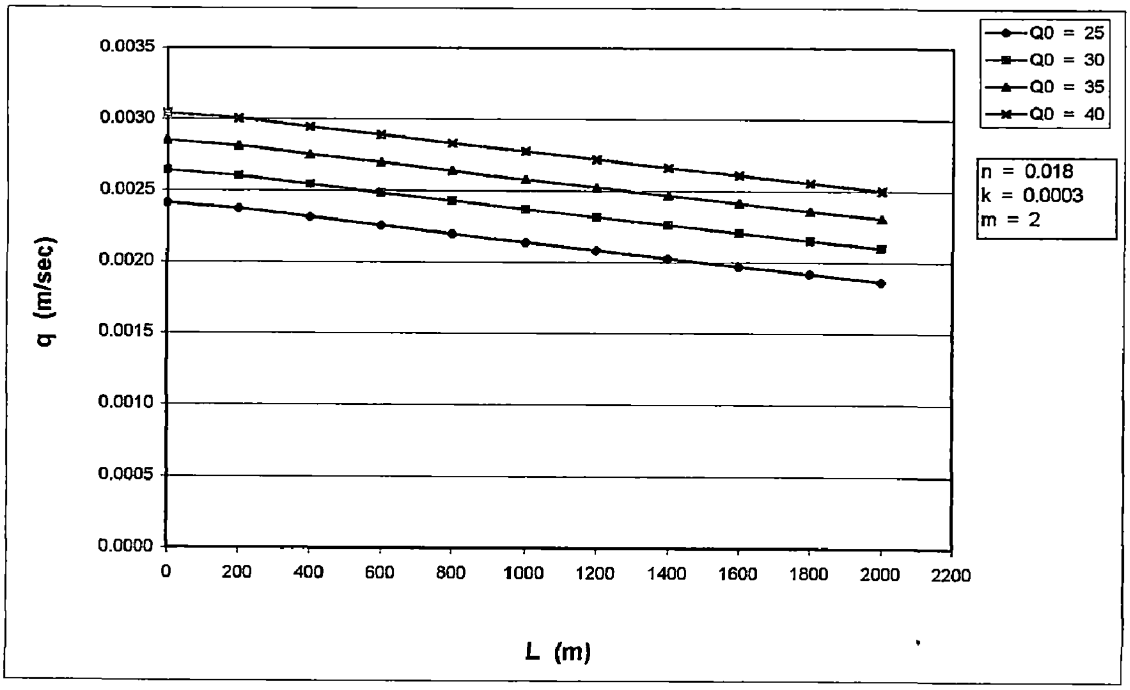


Fig. 4.5.2c. **Relation between seepage losses with length of the canal for different discharges**

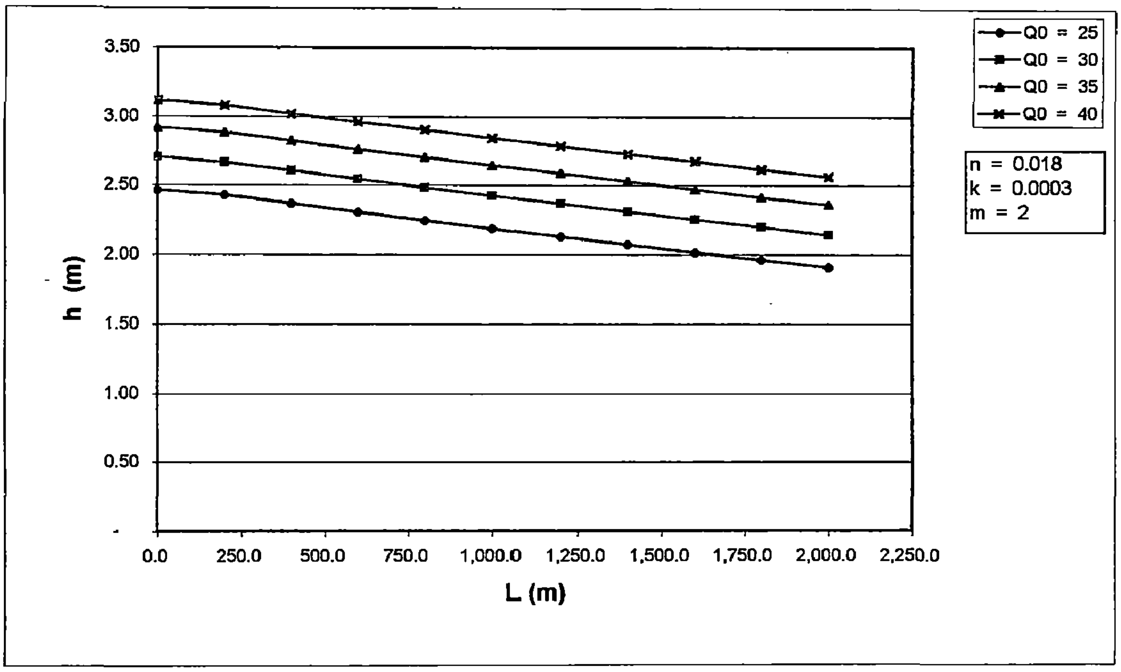


Fig. 4.5.2d. **Relation between depth of water with length of the canal for different discharges**

CHAPTER V

CONCLUSIONS

Various authors have given several empirical formulae for evaluation of seepage losses from the canals and most of them have a limited application only as they can be applied to a particular region only.

Several analytical formulae also have been developed for solving of seepage problems from open channels for various positions of ground water table and positions of impermeable or highly permeable layer relative to the channel. All these derivations have some application in the field at one time or other.

Vedernikov has given analytical solution to compute seepage from a trapezoidal canal for the situation where water table lies at large depth below the canal bed. The entire hodograph plane has been mapped conformally because of which the mapping functions are complex. In the present thesis, taking advantage of the symmetry in hodograph, simple and straightforward analytical solution for computation of seepage has been derived. Unlike in the computation of seepage in Vedernikov's method, in which both B, the width of the canal at water surface (which is a function of depth of water), and h, the depth of water in the canal, are involved, in the present analysis only depth of water is used for computation is straight forward and easy.

In a canal certain known volume of water per unit time is conveyed. The depth of water would change due to seepage from the canal.

The canal seepage at any section can not be computed unless the depth of water is known. In the present thesis the depth of water in the canal which varies along the length of canal has been found using mass balance principle and Manning equation for flow velocity.

It is found that the depth of water decreases exponentially. Thus would affect the irrigation water supply of water has to flow under gravity to the field.

For computing dimensions of the canal, whose section conform to a trapezoidal one, the parameter and data required are:

1. Discharge at entrance
2. The hydraulic conductivity, k
3. Length along the canal

4. Bed slope of the canal
5. Bed width of the canal
6. Manning's roughness

For varied discharge along the canal, applying Manning's equations and mass balance, including seepage losses per unit length of the canal, the depth of water has been computed. The result are presented in table V.1a through V.5b.

Table V.1b Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 0.5 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.0003

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	3.21	1.87	0.00313	0.000125	1.73726
33.33	200.00	25.00	3.17	1.89	0.00309	0.000124	1.71924
66.67	400.00	24.38	3.09	1.94	0.00302	0.000121	1.67557
100.00	600.00	23.78	3.01	1.99	0.00294	0.000118	1.63269
133.33	800.00	23.19	2.94	2.04	0.00286	0.000115	1.59059
166.67	1,000.00	22.62	2.86	2.10	0.00279	0.000112	1.54928
200.00	1,200.00	22.06	2.78	2.16	0.00272	0.000109	1.50877
233.33	1,400.00	21.52	2.71	2.21	0.00264	0.000106	1.46905
266.67	1,600.00	20.99	2.64	2.27	0.00257	0.000103	1.43013
300.00	1,800.00	20.47	2.57	2.34	0.00251	0.000100	1.39206
333.33	2,000.00	19.97	2.50	2.40	0.00244	0.000098	1.35477
		30.00	3.58	1.68	0.00349	0.000116	1.93872
		30.00	3.54	1.69	0.00346	0.000115	1.92015
		29.31	3.46	1.73	0.00338	0.000113	1.87515
		28.63	3.38	1.78	0.00330	0.000110	1.83081
		27.97	3.30	1.82	0.00322	0.000107	1.78720
		27.33	3.22	1.86	0.00314	0.000105	1.74433
		26.70	3.14	1.91	0.00306	0.000102	1.70216
		26.09	3.06	1.96	0.00299	0.000100	1.66079
		25.49	2.99	2.01	0.00292	0.000097	1.62017
		24.91	2.92	2.06	0.00284	0.000095	1.58029
		24.34	2.84	2.11	0.00277	0.000092	1.54116
		35.00	3.92	1.53	0.00383	0.000109	2.12564
		35.00	3.89	1.54	0.00379	0.000108	2.10664
		34.24	3.80	1.58	0.00371	0.000106	2.06055
		33.50	3.72	1.61	0.00363	0.000104	2.01507
		32.77	3.64	1.65	0.00355	0.000101	1.97027
		32.07	3.55	1.69	0.00347	0.000099	1.92615
		31.37	3.47	1.73	0.00339	0.000097	1.88272
		30.69	3.40	1.77	0.00331	0.000095	1.83999
		30.03	3.32	1.81	0.00324	0.000092	1.79796
		29.38	3.24	1.85	0.00316	0.000090	1.75663
		28.75	3.17	1.89	0.00309	0.000088	1.71601
		40.00	4.25	1.41	0.00414	0.000104	2.30066
		40.00	4.21	1.43	0.00411	0.000103	2.28135
		39.18	4.12	1.46	0.00402	0.000101	2.23438
		38.37	4.04	1.49	0.00394	0.000098	2.18798
		37.59	3.95	1.52	0.00386	0.000096	2.14222
		36.82	3.87	1.55	0.00377	0.000094	2.09709
		36.06	3.79	1.58	0.00369	0.000092	2.05260
		35.32	3.71	1.62	0.00362	0.000090	2.00877
		34.60	3.63	1.65	0.00354	0.000088	1.96559
		33.89	3.55	1.69	0.00346	0.000087	1.92307
		33.20	3.47	1.73	0.00339	0.000085	1.88121

Table V.1a Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 0.5 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.001

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	3.21	1.87	0.01042	0.000417	5.79086
33.33	200.00	25.00	3.10	1.94	0.01007	0.000403	5.59467
66.67	400.00	22.99	2.84	2.11	0.00924	0.000369	5.13097
100.00	600.00	21.14	2.60	2.31	0.00845	0.000338	4.69424
133.33	800.00	19.45	2.37	2.53	0.00772	0.000309	4.28704
166.67	1,000.00	17.91	2.16	2.77	0.00704	0.000281	3.90919
200.00	1,200.00	16.50	1.97	3.04	0.00641	0.000256	3.56017
233.33	1,400.00	15.22	1.79	3.35	0.00583	0.000233	3.23903
266.67	1,600.00	14.05	1.63	3.68	0.00530	0.000212	2.94492
300.00	1,800.00	12.99	1.48	4.05	0.00482	0.000193	2.67628
333.33	2,000.00	12.03	1.35	4.46	0.00438	0.000175	2.43169
		30.00	3.58	1.68	0.01163	0.000388	6.46241
		30.00	3.47	1.73	0.01127	0.000376	6.25999
		27.75	3.20	1.88	0.01040	0.000347	5.77936
		25.67	2.95	2.04	0.00958	0.000319	5.32376
		23.75	2.71	2.21	0.00881	0.000294	4.89581
		21.99	2.49	2.41	0.00809	0.000270	4.49566
		20.37	2.28	2.63	0.00742	0.000247	4.12301
		18.88	2.09	2.87	0.00680	0.000227	3.77738
		17.52	1.91	3.13	0.00622	0.000207	3.45799
		16.28	1.75	3.43	0.00569	0.000190	3.16380
		15.14	1.60	3.75	0.00521	0.000174	2.89375
		35.00	3.92	1.53	0.01275	0.000364395	7.08546
		35.00	3.81	1.58	0.01238	0.000353727	6.87803
		32.52	3.53	1.70	0.01149	0.00032832	6.38399
		30.23	3.27	1.83	0.01064	0.000304109	5.91323
		28.10	3.03	1.98	0.00984	0.00028123	5.46835
		26.13	2.80	2.15	0.00909	0.000259703	5.04977
		24.31	2.58	2.33	0.00838	0.000239524	4.65741
		22.63	2.38	2.53	0.00772	0.000220682	4.29105
		21.09	2.19	2.74	0.00711	0.000203151	3.95016
		19.67	2.01	2.98	0.00654	0.000186893	3.63403
		18.36	1.85	3.24	0.00601	0.000171856	3.34165
		40.00	4.18	1.44	0.01359	0.000339792	7.55094
		40.00	4.18	1.44	0.01358	0.000339493	7.54429
		37.28	3.85	1.56	0.01251	0.000312755	6.95012
		34.78	3.58	1.68	0.01164	0.000290962	6.46582
		32.45	3.33	1.80	0.01081	0.000270316	6.00702
		30.29	3.09	1.94	0.01003	0.000250785	5.57300
		28.29	2.86	2.10	0.00930	0.000232384	5.16410
		26.43	2.65	2.27	0.00860	0.000215103	4.78006
		24.71	2.45	2.45	0.00796	0.000198935	4.42077
		23.11	2.26	2.65	0.00735	0.000183852	4.08560
		21.64	2.09	2.87	0.00679	0.000169825	3.77389

Table V.2a Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 0.7 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.001

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	3.05	1.97	0.01023	0.000409	1.70552
33.33	200.00	25.00	2.94	2.04	0.00985	0.000394	1.64229
66.67	400.00	23.03	2.69	2.23	0.00903	0.000361	1.50454
100.00	600.00	21.22	2.46	2.44	0.00825	0.000330	1.37479
133.33	800.00	19.57	2.24	2.68	0.00752	0.000301	1.25371
166.67	1,000.00	18.07	2.04	2.94	0.00685	0.000274	1.14133
200.00	1,200.00	16.70	1.85	3.23	0.00623	0.000249	1.03762
233.33	1,400.00	15.45	1.68	3.56	0.00565	0.000226	0.94239
266.67	1,600.00	14.32	1.53	3.92	0.00513	0.000205	0.85536
300.00	1,800.00	13.30	1.39	4.32	0.00466	0.000186	0.77614
333.33	2,000.00	12.37	1.36	4.42	0.00456	0.000182	0.75922
		30.00	3.39	1.77	0.01137	0.000379	1.89506
		30.00	3.27	1.83	0.01098	0.000366	1.83048
		27.80	3.02	1.99	0.01013	0.000338	1.68915
		25.78	2.78	2.16	0.00933	0.000311	1.55508
		23.91	2.55	2.35	0.00857	0.000286	1.42887
		22.20	2.34	2.56	0.00786	0.000262	1.31079
		20.62	2.15	2.80	0.00720	0.000240	1.20078
		19.18	1.96	3.05	0.00659	0.000220	1.09881
		17.86	1.80	3.34	0.00603	0.000201	1.00472
		16.66	1.64	3.66	0.00551	0.000184	0.91825
		15.56	1.64	3.65	0.00552	0.000184	0.91977
		35.00	3.70	1.62	0.01242	0.000355	2.06995
		35.00	3.58	1.67	0.01203	0.000344	2.00434
		32.59	3.33	1.80	0.01116	0.000319	1.86034
		30.36	3.08	1.95	0.01034	0.000295	1.72292
		28.29	2.85	2.11	0.00956	0.000273	1.59277
		26.38	2.63	2.28	0.00882	0.000252	1.47009
		24.62	2.42	2.48	0.00813	0.000232	1.35498
		22.99	2.23	2.69	0.00748	0.000214	1.24749
		21.50	2.05	2.92	0.00689	0.000197	1.14752
		20.12	1.89	3.18	0.00633	0.000181	1.05491
		18.85	1.89	3.18	0.00634	0.000181	1.05652
		40.00	3.99	1.50	0.01340	0.000335	2.23298
		40.00	3.87	1.55	0.01300	0.000325	2.16662
		37.40	3.61	1.66	0.01212	0.000303	2.02052
		34.98	3.36	1.78	0.01128	0.000282	1.88051
		32.72	3.12	1.92	0.01048	0.000262	1.74717
		30.62	2.90	2.07	0.00972	0.000243	1.62079
		28.68	2.68	2.24	0.00901	0.000225	1.50151
		26.88	2.48	2.42	0.00834	0.000208	1.38939
		25.21	2.30	2.61	0.00771	0.000193	1.28448
		23.67	2.12	2.83	0.00712	0.000178	1.18663
		22.24	2.05	2.93	0.00688	0.000172	1.14670

Table V.2b Variation L/b ; q/Qo and q/(kb) for different discharges along the canal
for m = 0.7 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.0003

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	3.05	1.97	0.003070	0.000123	1.70551
33.33	200.00	25.00	3.01	1.99	0.003035	0.000121	1.68615
66.67	400.00	24.39	2.94	2.04	0.002958	0.000118	1.64309
100.00	600.00	23.80	2.86	2.10	0.002881	0.000115	1.60071
133.33	800.00	23.23	2.79	2.15	0.002806	0.000112	1.55908
166.67	1,000.00	22.66	2.71	2.21	0.002733	0.000109	1.51817
200.00	1,200.00	22.12	2.64	2.27	0.002660	0.000106	1.47803
233.33	1,400.00	21.59	2.57	2.33	0.002590	0.000104	1.43870
266.67	1,600.00	21.07	2.50	2.40	0.002520	0.000101	1.40012
300.00	1,800.00	20.56	2.44	2.46	0.002452	0.000098	1.36231
333.33	2,000.00	20.07	2.43	2.47	0.002451	0.000098	1.36150
		30.00	3.39	1.77	0.003411	0.000114	1.89506
		30.00	3.35	1.79	0.003376	0.000113	1.87534
		29.32	3.27	1.83	0.003296	0.000110	1.83136
		28.67	3.20	1.88	0.003218	0.000107	1.78800
		28.02	3.12	1.92	0.003142	0.000105	1.74531
		27.39	3.04	1.97	0.003066	0.000102	1.70327
		26.78	2.97	2.02	0.002992	0.000100	1.66196
		26.18	2.90	2.07	0.002918	0.000097	1.62134
		25.60	2.83	2.12	0.002847	0.000095	1.58142
		25.03	2.76	2.18	0.002776	0.000093	1.54221
		24.47	2.76	2.18	0.002775	0.000092	1.54161
		35.00	3.70	1.62	0.003726	0.000106	2.06996
		35.00	3.66	1.64	0.003690	0.000105	2.04996
		34.26	3.58	1.67	0.003610	0.000103	2.00530
		33.54	3.51	1.71	0.003530	0.000101	1.96122
		32.83	3.43	1.75	0.003452	0.000099	1.91774
		32.14	3.35	1.79	0.003375	0.000096	1.87487
		31.47	3.28	1.83	0.003299	0.000094	1.83264
		30.81	3.20	1.87	0.003224	0.000092	1.79104
		30.16	3.13	1.92	0.003150	0.000090	1.75008
		29.53	3.06	1.96	0.003078	0.000088	1.70975
		28.92	3.06	1.96	0.003077	0.000088	1.70942
		40.00	3.99	1.50	0.004019	0.000100	2.23299
		40.00	3.97	1.51	0.004001	0.000100	2.22268
		39.20	3.88	1.55	0.003902	0.000098	2.16755
		38.42	3.80	1.58	0.003821	0.000096	2.12288
		37.66	3.72	1.61	0.003742	0.000094	2.07880
		36.91	3.64	1.65	0.003663	0.000092	2.03528
		36.17	3.56	1.68	0.003586	0.000090	1.99233
		35.46	3.49	1.72	0.003510	0.000088	1.94997
		34.75	3.41	1.76	0.003435	0.000086	1.90821
		34.07	3.34	1.80	0.003361	0.000084	1.86704
		33.40	3.27	1.84	0.003288	0.000082	1.82649

Table V.3a Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 1 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.001

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	2.83	2.12	0.011336	0.000453	1.88937
33.33	200.00	25.00	2.69	2.23	0.010786	0.000431	1.79771
66.67	400.00	22.84	2.43	2.47	0.009724	0.000389	1.62066
100.00	600.00	20.90	2.18	2.75	0.008735	0.000349	1.45586
133.33	800.00	19.15	1.95	3.07	0.007824	0.000313	1.30404
166.67	1,000.00	17.59	1.75	3.44	0.006993	0.000280	1.16552
200.00	1,200.00	16.19	1.56	3.85	0.006242	0.000250	1.04028
233.33	1,400.00	14.94	1.39	4.32	0.005568	0.000223	0.92794
266.67	1,600.00	13.83	1.24	4.84	0.004968	0.000199	0.82793
300.00	1,800.00	12.83	1.11	5.42	0.004436	0.000177	0.73927
333.33	2,000.00	11.94	0.99	6.06	0.003967	0.000159	0.66110
		30.00	3.13	1.92	0.012524	0.000501	2.08731
		30.00	2.99	2.01	0.011969	0.000479	1.99489
		27.61	2.72	2.21	0.010892	0.000436	1.81531
		25.43	2.47	2.43	0.009880	0.000395	1.64659
		23.45	2.23	2.69	0.008937	0.000357	1.48942
		21.66	2.01	2.98	0.008066	0.000323	1.34426
		20.05	1.81	3.31	0.007268	0.000291	1.21134
		18.60	1.63	3.67	0.006543	0.000262	1.09052
		17.29	1.47	4.08	0.005889	0.000236	0.98148
		16.11	1.32	4.53	0.005302	0.000212	0.88358
		15.05	1.19	5.03	0.004777	0.000191	0.79612
		35.00	3.40	1.77	0.013612	0.000544	2.26873
		35.00	3.26	1.84	0.013055	0.000522	2.17582
		32.39	2.99	2.01	0.011968	0.000479	1.99468
		30.00	2.73	2.20	0.010939	0.000438	1.82317
		27.81	2.49	2.41	0.009972	0.000399	1.66204
		25.81	2.26	2.65	0.009071	0.000363	1.51186
		24.00	2.06	2.92	0.008237	0.000329	1.37291
		22.35	1.86	3.22	0.007472	0.000299	1.24530
		20.86	1.69	3.55	0.006773	0.000271	1.12883
		19.50	1.53	3.92	0.006139	0.000246	1.02318
		18.27	1.39	4.32	0.005566	0.000223	0.92774
		40.00	3.65	1.64	0.014622	0.000585	2.43693
		40.00	3.51	1.71	0.014063	0.000563	2.34378
		37.19	3.24	1.85	0.012970	0.000519	2.16163
		34.59	2.98	2.02	0.011929	0.000477	1.98816
		32.21	2.73	2.20	0.010944	0.000438	1.82406
		30.02	2.50	2.40	0.010020	0.000401	1.66994
		28.01	2.29	2.63	0.009157	0.000366	1.52621
		26.18	2.09	2.88	0.008358	0.000334	1.39307
		24.51	1.90	3.15	0.007623	0.000305	1.27047
		22.99	1.73	3.46	0.006949	0.000278	1.15824
		21.60	1.58	3.79	0.006336	0.000253	1.05598

Table V.3b Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 1 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.0003

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	2.83	2.12	0.00340	0.000136	1.88936
33.33	200.00	25.00	2.79	2.15	0.00335	0.000134	1.86134
66.67	400.00	24.33	2.70	2.22	0.00325	0.000130	1.80582
100.00	600.00	23.68	2.62	2.29	0.00315	0.000126	1.75132
133.33	800.00	23.05	2.54	2.36	0.00306	0.000122	1.69788
166.67	1,000.00	22.44	2.46	2.44	0.00296	0.000118	1.64548
200.00	1,200.00	21.85	2.39	2.51	0.00287	0.000115	1.59417
233.33	1,400.00	21.27	2.31	2.60	0.00278	0.000111	1.54399
266.67	1,600.00	20.72	2.24	2.68	0.00269	0.000108	1.49494
300.00	1,800.00	20.18	2.17	2.77	0.00260	0.000104	1.44706
333.33	2,000.00	19.66	2.10	2.86	0.00252	0.000101	1.40035
		30.00	3.13	1.92	0.00376	0.000125	2.08731
		30.00	3.08	1.95	0.00371	0.000124	2.05917
		29.26	3.00	2.00	0.00361	0.000120	2.00317
		28.54	2.92	2.06	0.00351	0.000117	1.94805
		27.84	2.84	2.12	0.00341	0.000114	1.89389
		27.15	2.76	2.18	0.00331	0.000110	1.84066
		26.49	2.68	2.24	0.00322	0.000107	1.78839
		25.85	2.60	2.31	0.00313	0.000104	1.73714
		25.22	2.53	2.38	0.00304	0.000101	1.68685
		24.62	2.45	2.45	0.00295	0.000098	1.63760
		24.03	2.38	2.52	0.00286	0.000095	1.58940
		35.00	3.40	1.77	0.00408	0.000117	2.26873
		35.00	3.35	1.79	0.00403	0.000115	2.24043
		34.19	3.27	1.83	0.00393	0.000112	2.18422
		33.41	3.19	1.88	0.00383	0.000109	2.12880
		32.64	3.11	1.93	0.00373	0.000107	2.07419
		31.89	3.03	1.98	0.00364	0.000104	2.02042
		31.17	2.95	2.04	0.00354	0.000101	1.96747
		30.46	2.87	2.09	0.00345	0.000099	1.91542
		29.77	2.79	2.15	0.00336	0.000096	1.86428
		29.10	2.72	2.21	0.00327	0.000093	1.81405
		28.44	2.64	2.27	0.00318	0.000091	1.76479
		40.00	3.65	1.64	0.00439	0.000109662	2.43693
		40.00	3.61	1.66	0.00434	0.000108388	2.40862
		39.13	3.52	1.70	0.00423	0.000105852	2.35228
		38.29	3.44	1.74	0.00413	0.000103349	2.29665
		37.46	3.36	1.79	0.00404	0.000100879	2.24176
		36.65	3.28	1.83	0.00394	9.84425E-05	2.18761
		35.86	3.20	1.88	0.00384	9.60405E-05	2.13423
		35.10	3.12	1.92	0.00375	9.36751E-05	2.08167
		34.35	3.04	1.97	0.00365	9.13449E-05	2.02989
		33.62	2.96	2.02	0.00356	8.90523E-05	1.97894
		32.90	2.89	2.08	0.00347	8.67982E-05	1.92885

Table V.4a Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 1.5 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.001

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	2.62	2.29	0.00853	0.000341	4.73791
33.33	200.00	25.00	2.50	2.40	0.00814	0.000326	4.52280
66.67	400.00	23.37	2.30	2.61	0.00747	0.000299	4.15215
100.00	600.00	21.88	2.10	2.85	0.00684	0.000274	3.80236
133.33	800.00	20.51	1.92	3.12	0.00625	0.000250	3.47408
166.67	1,000.00	19.26	1.75	3.42	0.00570	0.000228	3.16877
200.00	1,200.00	18.12	1.60	3.75	0.00520	0.000208	2.88680
233.33	1,400.00	17.08	1.45	4.12	0.00473	0.000189	2.62833
266.67	1,600.00	16.13	1.32	4.53	0.00431	0.000172	2.39289
300.00	1,800.00	15.27	1.21	4.97	0.00392	0.000157	2.17961
333.33	2,000.00	14.49	1.10	5.45	0.00358	0.000143	1.98717
		30.00	2.88	2.08	0.00937	0.000312	5.20809
		30.00	2.76	2.17	0.00899	0.000300	4.99360
		28.20	2.56	2.34	0.00832	0.000277	4.62234
		26.54	2.36	2.54	0.00768	0.000256	4.26908
		25.00	2.18	2.75	0.00708	0.000236	3.93442
		23.59	2.00	2.99	0.00652	0.000217	3.61971
		22.28	1.84	3.26	0.00599	0.000200	3.32576
		21.08	1.69	3.55	0.00550	0.000183	3.05304
		19.99	1.55	3.87	0.00504	0.000168	2.80153
		18.98	1.42	4.22	0.00463	0.000154	2.57078
		18.05	1.31	4.59	0.00425	0.000142	2.36004
		35.00	3.12	1.92	0.01015	0.000289894	5.63683
		35.00	3.00	2.00	0.00976	0.000278914	5.42332
		33.05	2.80	2.15	0.00909	0.000259844	5.05252
		31.23	2.60	2.31	0.00846	0.000241582	4.69743
		29.54	2.41	2.49	0.00785	0.000224164	4.35875
		27.97	2.24	2.68	0.00727	0.000207649	4.03762
		26.51	2.07	2.90	0.00672	0.00019209	3.73507
		25.17	1.91	3.14	0.00621	0.000177515	3.45167
		23.93	1.76	3.40	0.00574	0.000163944	3.18781
		22.78	1.63	3.68	0.00530	0.000151368	2.94327
		21.72	1.50	3.99	0.00489	0.000139768	2.71770
		40.00	3.34	1.80	0.01086	0.000271469	6.03264
		40.00	3.22	1.86	0.01048	0.000261915	5.82034
		37.90	3.02	1.99	0.00981	0.000245278	5.45061
		35.94	2.82	2.13	0.00917	0.000229267	5.09482
		34.11	2.63	2.28	0.00856	0.00021391	4.75356
		32.40	2.45	2.45	0.00797	0.000199263	4.42806
		30.80	2.28	2.63	0.00741	0.000185364	4.11920
		29.32	2.12	2.83	0.00689	0.00017225	3.82778
		27.94	1.97	3.05	0.00640	0.000159937	3.55415
		26.66	1.83	3.29	0.00594	0.00014844	3.29867
		25.48	1.65	3.64	0.00537	0.000134129	2.98063

Table V.4b Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 1.5 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.0003

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	2.62	2.29	0.00256	0.000102	1.42137
33.33	200.00	25.00	2.59	2.32	0.00252	0.000101	1.40178
66.67	400.00	24.50	2.52	2.38	0.00246	0.000098	1.36733
100.00	600.00	24.00	2.46	2.44	0.00240	0.000096	1.33339
133.33	800.00	23.52	2.40	2.50	0.00234	0.000094	1.29995
166.67	1,000.00	23.06	2.34	2.57	0.00228	0.000091	1.26697
200.00	1,200.00	22.60	2.28	2.63	0.00222	0.000089	1.23456
233.33	1,400.00	22.15	2.22	2.70	0.00216	0.000087	1.20268
266.67	1,600.00	21.72	2.16	2.78	0.00211	0.000084	1.17136
300.00	1,800.00	21.30	2.10	2.85	0.00205	0.000082	1.14063
333.33	2,000.00	20.89	2.05	2.93	0.00200	0.000080	1.11044
		30.00	2.88	2.08	0.00281	0.000094	1.56243
		30.00	2.85	2.11	0.00278	0.000093	1.54291
		29.44	2.78	2.16	0.00272	0.000091	1.50858
		28.90	2.72	2.20	0.00265	0.000088	1.47467
		28.37	2.66	2.26	0.00259	0.000086	1.44116
		27.85	2.60	2.31	0.00253	0.000084	1.40813
		27.34	2.54	2.36	0.00248	0.000083	1.37553
		26.85	2.48	2.42	0.00242	0.000081	1.34340
		26.37	2.42	2.48	0.00236	0.000079	1.31176
		25.89	2.36	2.54	0.00231	0.000077	1.28057
		25.43	2.31	2.60	0.00225	0.000075	1.24991
		35.00	3.12	1.92	0.00304	8.69683E-05	1.69105
		35.00	3.08	1.95	0.00301	8.59707E-05	1.67165
		34.40	3.02	1.99	0.00295	8.42132E-05	1.63748
		33.81	2.96	2.03	0.00289	8.24749E-05	1.60368
		33.23	2.90	2.07	0.00283	8.07545E-05	1.57023
		32.67	2.84	2.12	0.00277	7.90556E-05	1.53719
		32.11	2.78	2.16	0.00271	7.73764E-05	1.50454
		31.57	2.72	2.21	0.00265	7.57179E-05	1.47229
		31.04	2.66	2.26	0.00259	7.40798E-05	1.44044
		30.52	2.60	2.31	0.00254	7.24659E-05	1.40906
		30.02	2.54	2.36	0.00248	7.08736E-05	1.37810
		40.00	3.34	1.80	0.00326	8.14413E-05	1.80981
		40.00	3.30	1.82	0.00322	8.05739E-05	1.79053
		39.36	3.24	1.85	0.00316	7.90448E-05	1.75655
		38.72	3.18	1.89	0.00310	7.75307E-05	1.72290
		38.10	3.12	1.92	0.00304	7.60312E-05	1.68958
		37.49	3.06	1.96	0.00298	7.45469E-05	1.65660
		36.90	3.00	2.00	0.00292	7.30782E-05	1.62396
		36.31	2.94	2.04	0.00287	7.16257E-05	1.59168
		35.74	2.88	2.08	0.00281	7.01893E-05	1.55976
		35.18	2.82	2.13	0.00275	6.87711E-05	1.52825
		34.63	2.76	2.17	0.00269	6.73701E-05	1.49711

Table V.5a Variation L/b ; q/Qo and q/(kb) for different discharges along the canal for m = 2 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.001

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	2.47	2.43	0.00803	0.000321	4.46243
33.33	200.00	25.00	2.34	2.56	0.00762	0.000305	4.23255
66.67	400.00	23.48	2.14	2.80	0.00697	0.000279	3.87102
100.00	600.00	22.08	1.95	3.07	0.00636	0.000254	3.53126
133.33	800.00	20.81	1.78	3.37	0.00578	0.000231	3.21376
166.67	1,000.00	19.65	1.62	3.71	0.00526	0.000210	2.92004
200.00	1,200.00	18.60	1.47	4.09	0.00477	0.000191	2.65085
233.33	1,400.00	17.65	1.33	4.50	0.00433	0.000173	2.40612
266.67	1,600.00	16.78	1.21	4.96	0.00393	0.000157	2.18526
300.00	1,800.00	16.00	1.10	5.45	0.00358	0.000143	1.98696
333.33	2,000.00	15.28	1.00	5.99	0.00326	0.000130	1.80978
		30.00	2.71	2.22	0.00880	0.000293	4.88852
		30.00	2.58	2.33	0.00839	0.000280	4.66094
		28.32	2.38	2.52	0.00774	0.000258	4.30124
		26.77	2.19	2.74	0.00713	0.000238	3.96005
		25.35	2.01	2.98	0.00655	0.000218	3.63781
		24.04	1.85	3.25	0.00600	0.000200	3.33606
		22.84	1.69	3.55	0.00550	0.000183	3.05587
		21.74	1.55	3.87	0.00504	0.000168	2.79765
		20.73	1.42	4.23	0.00461	0.000154	2.56126
		19.81	1.30	4.62	0.00422	0.000141	2.34610
		18.96	1.19	5.04	0.00387	0.000129	2.15117
		35.00	2.92	2.05	0.00950	0.000317	5.27586
		35.00	2.80	2.15	0.00909	0.000259746	5.05062
		33.18	2.60	2.31	0.00845	0.000241373	4.69336
		31.49	2.41	2.49	0.00783	0.000223823	4.35211
		29.93	2.23	2.69	0.00725	0.00020712	4.02733
		28.48	2.06	2.91	0.00670	0.000191338	3.72045
		27.14	1.90	3.16	0.00618	0.000176532	3.43257
		25.90	1.75	3.43	0.00570	0.000162747	3.16453
		24.76	1.61	3.72	0.00525	0.000149985	2.91638
		23.71	1.49	4.03	0.00484	0.00013824	2.68801
		22.74	1.37	4.37	0.00446	0.000127481	2.47880
		40.00	3.12	1.92	0.01014	0.000253467	5.63259
		40.00	2.99	2.00	0.00974	0.000243435	5.40967
		38.05	2.80	2.14	0.00910	0.00022748	5.05512
		36.23	2.61	2.30	0.00849	0.000212159	4.71465
		34.54	2.43	2.47	0.00790	0.000197492	4.38871
		32.96	2.26	2.66	0.00734	0.000183532	4.07850
		31.49	2.10	2.86	0.00681	0.00017034	3.78533
		30.12	1.94	3.09	0.00632	0.000157948	3.50995
		28.86	1.80	3.33	0.00586	0.000146376	3.25280
		27.69	1.67	3.60	0.00543	0.000135629	3.01398
		26.60	1.50	4.00	0.00488	0.000121935	2.70967

Table V.5b Variation L/b ; q/Qo and q/(kb) for different discharges along the canal
for m = 2 ; s = 0.00015 ; b = 6 m ; n = 0.018 and k = 0.0003

L/b	L (m)	Q (m^3/sec)	h (m)	b/h	q (m^3/sec)	q/Qo	q/(kb)
-	-	25.00	2.47	2.43	0.00241	0.000096	1.33873
33.33	200.00	25.00	2.43	2.47	0.00237	0.000095	1.31784
66.67	400.00	24.53	2.37	2.53	0.00231	0.000092	1.28428
100.00	600.00	24.06	2.31	2.60	0.00225	0.000090	1.25122
133.33	800.00	23.61	2.25	2.67	0.00219	0.000088	1.21864
166.67	1,000.00	23.17	2.19	2.74	0.00214	0.000085	1.18658
200.00	1,200.00	22.75	2.13	2.82	0.00208	0.000083	1.15504
233.33	1,400.00	22.33	2.07	2.89	0.00202	0.000081	1.12404
266.67	1,600.00	21.93	2.02	2.97	0.00197	0.000079	1.09360
300.00	1,800.00	21.53	1.96	3.06	0.00191	0.000077	1.06376
333.33	2,000.00	21.15	1.91	3.14	0.00186	0.000074	1.03449
		30.00	2.71	2.22	0.00264	0.000088	1.46656
		30.00	2.67	2.25	0.00260	0.000087	1.44589
		29.48	2.61	2.30	0.00254	0.000085	1.41268
		28.97	2.55	2.36	0.00248	0.000083	1.37989
		28.47	2.49	2.41	0.00243	0.000081	1.34751
		27.99	2.43	2.47	0.00237	0.000079	1.31553
		27.52	2.37	2.53	0.00231	0.000077	1.28402
		27.05	2.31	2.60	0.00226	0.000075	1.25297
		26.60	2.26	2.66	0.00220	0.000073	1.22239
		26.16	2.20	2.73	0.00215	0.000072	1.19230
		25.73	2.15	2.80	0.00209	0.000070	1.16273
		35.00	2.92	2.05	0.00285	0.000081	1.58276
		35.00	2.88	2.08	0.00281	0.000080	1.56234
		34.44	2.82	2.13	0.00275	0.000079	1.52945
		33.89	2.76	2.17	0.00269	0.000077	1.49695
		33.35	2.70	2.22	0.00264	0.000075	1.46478
		32.82	2.64	2.27	0.00258	0.000074	1.43301
		32.30	2.59	2.32	0.00252	0.000072	1.40161
		31.80	2.53	2.37	0.00247	0.000070	1.37061
		31.31	2.47	2.43	0.00241	0.000069	1.34003
		30.82	2.42	2.48	0.00236	0.000067	1.30985
		30.35	2.36	2.54	0.00230	0.000066	1.28013
		40.00	3.12	1.92	0.00304	0.000076	1.68978
		40.00	3.08	1.95	0.00301	0.000075	1.66959
		39.40	3.02	1.99	0.00295	0.000074	1.63705
		38.81	2.96	2.03	0.00289	0.000072	1.60484
		38.23	2.90	2.07	0.00283	0.000071	1.57293
		37.67	2.84	2.11	0.00277	0.000069	1.54134
		37.11	2.79	2.15	0.00272	0.000068	1.51011
		36.57	2.73	2.20	0.00266	0.000067	1.47922
		36.03	2.67	2.24	0.00261	0.000065	1.44869
		35.51	2.62	2.29	0.00255	0.000064	1.41854
		35.00	2.56	2.34	0.00250	0.000062	1.38877

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APPENDIX 1

CONFORMAL MAPPING

1.1 VELOCITY HODOGRAPH

The transformation of the region of flow from the z plane into the w plane is called the velocity hodograph. The complex potential $w = \phi + i\psi$ is an analytic function of the complex variable z, i.e. $w = f(z)$. Differentiating w with respect to z, one finds

$$\frac{dw}{dz} = \frac{\partial \phi}{\partial x} + i\,\frac{\partial \psi}{\partial x}$$

Substituting the velocity components, $W = \frac{dw}{dz} = u - iv$ (A1.1)

The utility of the hodograph stems from the fact that, although the shape of the free surface and the limit of the surface of seepage are not known initially in the z plane, in the W plane their hodographs are completely defined.

1.3. THE SCHWARZ-CHRISTOFFEL TRANSFORMATION

Theoretically, the transformation exists which will map any pair of simply connected regions conformally onto each other. Generally these regions will be polygons having a finite number vertices (one more of which may be infinity). The method of mapping a polygon from one or more planes onto the upper half of another plane is of particular importance.

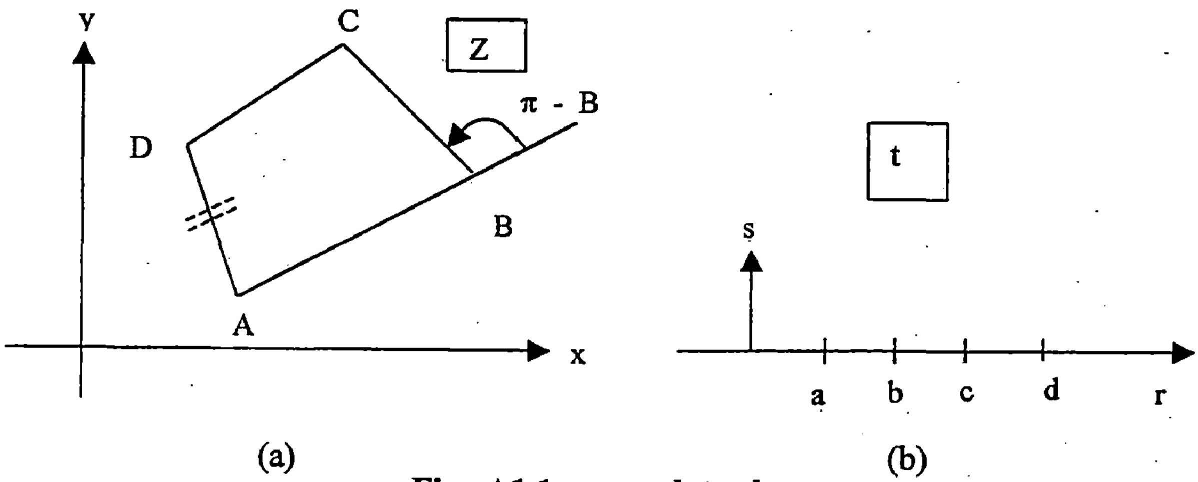

Fig. A1.1 z and t plane

If a polygon is located in the z plane, then the transformation that maps (Fig. A1.1) the polygon conformally onto the upper half of the t plane (t = r + is) is :

$$z = M \int \frac{dt}{(t-a)^{1-\alpha/\pi}(t-b)^{1-\beta/\pi}(t-c)^{1-\gamma/\pi}\ldots\ldots\ldots} + N \qquad \text{(A1.2)}$$

where M and N are complex constants, A, B, C,……., are the interior angles (in radians) of the polygon in the z plane (Fig. A1.1a), and a, b, c,….(a < b < c < ….) are points on the real axis of the t plane corresponding to the respective vertices A, B, C, …(Fig A1.1b).We note, in particular, that the complex constant N corresponds to the point on the perimeter of the polygon that has its image at t = 0.

The name the Schwarz-Christoffel transformation is given in honor of the two mathematicians, the German H. A. Schwarz (1843 – 1921) and the Swiss E. B. Christoffel (1829 – 1900), who discovered it independently.

The transformation can be considered as the mapping of a polygon from the z plane onto a similar polygon in the t plane in such a manner that the sides of the polygon in the z plane extend through the real axis of the t plane. This is accomplished by opening the polygon at some convenient point, say between A and E of Fig. A1.1a, and extending operation side to t = - ∞ and the other to t = + ∞ (Fig. A1.1b). In this operation the sides of the polygon are bent into straight line extending from t = - ∞ to t = + ∞ and are placed along the real axis of the t plane. The interior angle at the point of opening may be regarded as π (in the z plane) and as noted in Eq. (A1.6), takes no part in the transformation. The point of opening in the z plane is represented in the upper half of the t plane by semicircle with a radius of infinity. Thus the Schwarz-Christoffel transformation, in effect, maps conformally the region interior to the polygon A, B, C, …. Of the z plane into the interior of the polygon bounded by the sides ab, bc, … and a semicircle with a radius of infinity in the upper half of the t plane, or more simply, into the entire upper half of the t plane.

1.4 ZHUKOVSKY FUNCTIONS

Special mapping techniques, of particular value when dialing with unconfined flow problems, make use of an auxiliary transformation called Zhukovsky's function.

Noting that the relationship between the velocity potential and the pressure $[\phi = -k(p/\gamma_w + y)]$ can be written as $-kp/\gamma_w = \phi + ky$, if we define $\theta_1 = -kp/\gamma_w$, then

$$\theta_1 = \phi + ky \tag{A1.3}$$

θ_1 is seen to be an harmonic function of x and y as $\nabla^2\theta_1 = \nabla^2\phi \equiv 0$.

Hence, its conjugate is the function

$$\theta_2 = \psi - kx \tag{A1.4}$$

Defining $\theta_1 + i\theta_2 = \theta$, we observe that

$$\theta = \theta_1 + i\theta_2 = w - ikz$$

where $w = \phi + i\psi$, and $z = x + iy$ (A1.5)

Definition (A1.5) and any function with its real or imaginary part differing from it by a constant multiplier is called Zhukovsky function.

APPENDIX 2

SEEPAGE LOSSES FROM A CANAL

C.1 SEEPAGE FROM A CANAL WITH A CURVED PERIMETER INTO AHORIZONTAL DRAINAGE LAYER

Let we introduce a new function, namely

$$\theta = iz + \frac{w}{k} = Ae^{w/\alpha} \tag{A2.1}$$

with reference to the section shown in Fig. A2.1a. Here θ is Zhukovsky's function, w = ϕ(x, y) + iΨ(x, y) , α is a parameter, and A is a real constant. Separating this expression into real and imaginary parts, we obtain

$$-y + \frac{\phi}{k} = A\,e^{\phi/\alpha}\cos\frac{\psi}{\alpha}$$

$$x + \frac{\psi}{k} = A\,e^{\phi/\alpha}\sin\frac{\psi}{\alpha} \tag{A2.2}$$

Substituting - Ψ for Ψ and - x for x in Eq. (A2.2), we see that the system of streamlines defined by Ψ in these equations is symmetrical about the y axis. Hence, the y axis can be taken as the streamline Ψ = 0. Now a free surface must satisfy the conditions – y + ϕ/k = 0 and, say Ψ = -q/2 ; from the first of Eq. (A2.2) we find

$$\cos\left(-\frac{q}{2\alpha}\right) = 0$$

or

$$q = -(2n + 1)\,\alpha\pi \tag{A2.3}$$

where n is an integer.

In particular, taking n = 0 and substituting Eq. (A2.3) with Ψ = - q/2, and ϕ = k y into the second of Eq. (A2.2), we obtain for the free surface

$$x - \frac{q}{2k} = Ae^{-kyx/q} \tag{A2.4}$$

which has the asymptote (at y = ∞)

$$x_{y=x} = \frac{q}{2k} \tag{A2.5}$$

Letting y = 0 in Eq. (A2.4), we obtain for the half width of the ditch (Fig. A2.1a)

$$x_{y=0} = \frac{B}{2} = \frac{q}{2k} + A \tag{A2.6}$$

Now, taking ϕ = 0 in Eq. (A2.2), we find parametric equations for the perimeter of the canal :

$$-y = A \cos\frac{\psi}{\alpha}$$

$$x + \frac{\psi}{k} = A \sin\frac{\psi}{\alpha} \tag{A2.7}$$

As Ψ = 0 at the bottom of the canal, we find from the first of these equations that y = - A = h is the maximum depth of water in the canal. Hence the quantity of seepage from the canal section is found from Eq. (A2. 6) to be

$$q = k(B + 2h) \tag{A2.8}$$

C.2 TRAPEZOIDAL SHAPE

A much more direct method of solution for the seepage from ditches, canals, etc., was given by Vedernikov, using the method of inversion. The section to be investigated is shown in Fig. A2.1a. The hodograph is given in Fig. A2.1b, and the inversion of the hodograph (dz/dw = 1/(u - iv)) in Fig. A2.1c.

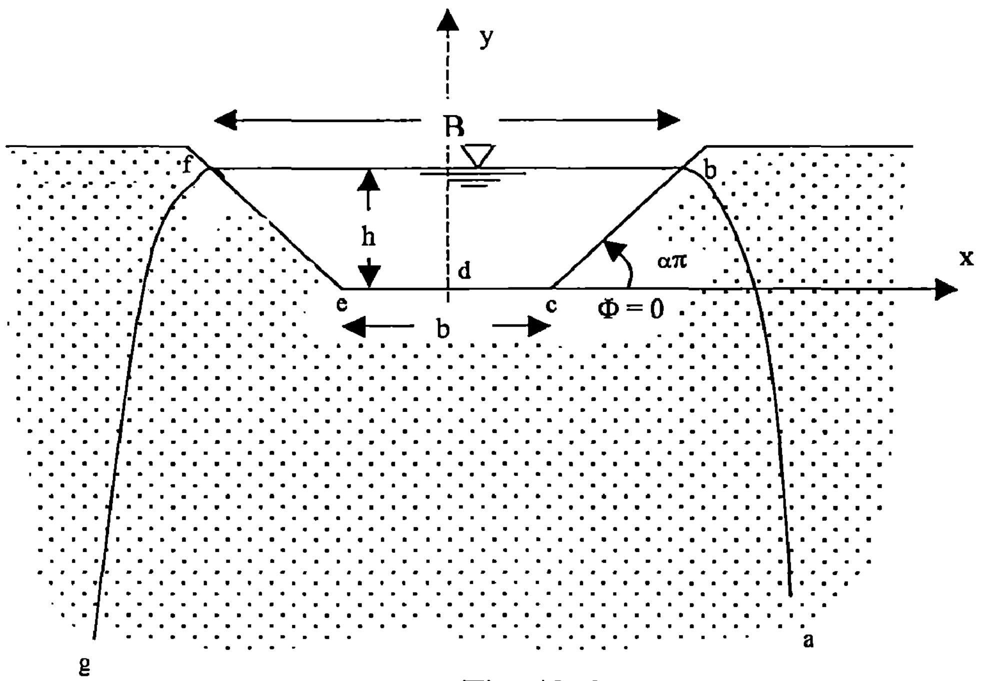

Fig. A2 .1a

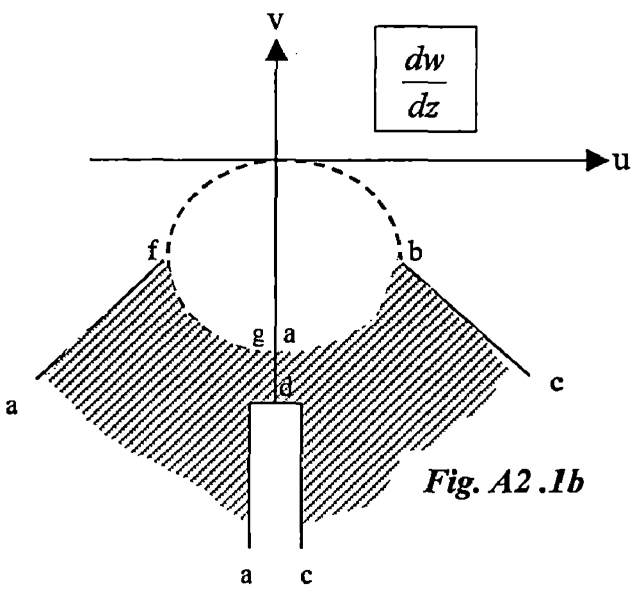

Fig. A2 .1b

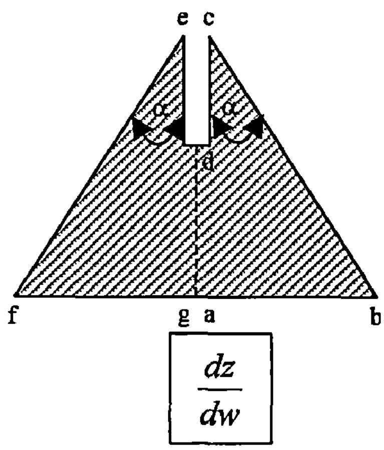

Fig. A2. 1c

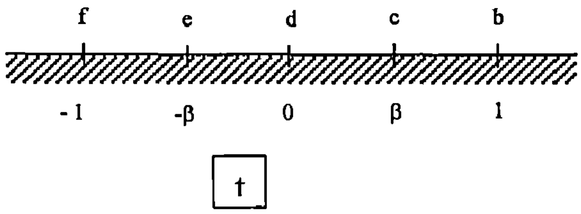

Fig. A2.1d

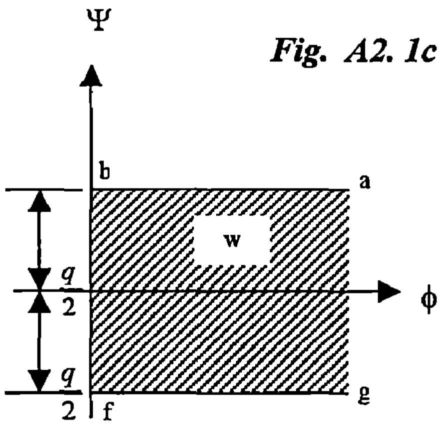

Fig. A2. 1e

Taking an auxiliary t plane as shown in Fig. A2.1d, we obtain for the mapping of the $\frac{dz}{dw}$ plane onto the lower half plane of t

$$\frac{dz}{dw} = M \int_0^t \frac{t\ dt}{(1 - t^2)^{½ + \sigma}\ (\beta^2 - t^2)^{1-\sigma}} + N = M\,\Phi + N \qquad \text{(A2.9)}$$

where $\sigma = \frac{\alpha}{\pi}$, and Φ (t) is indicated integral. In particular, we shall define

$$\Im_1 = \Phi(\beta) = \int_0^\beta \frac{t\ dt}{(1 - t^2)^{½ + \sigma}\ (\beta^2 - t^2)^{1-\sigma}}$$

$$\Im_2 = \int_\beta^1 \frac{t\ dt}{(1 - t^2)^{½ + \sigma}\ (t^2 - \beta^2)^{1-\sigma}} \qquad \text{(A2.10)}$$

Substituting ($\frac{(t^2 - \beta^2)}{(1 - \beta^2)} = x$ into second of equations (A2. 10), we find that

$$\Im_2 = \frac{1}{2\beta^1} \int_0^1 x^{\sigma - 1}\ (1 - x)^{-½ - \sigma}\ dx = \frac{1}{2\beta^1} B(\sigma\,,½ - \sigma) = \frac{\Gamma(\sigma)\Gamma(½ - \sigma)}{2\,\beta^1\,\sqrt{\pi}} \qquad \text{(A2.11)}$$

where $\beta^1 = (1 - \beta^2)^{½}$.

To eliminate the constant N in Eq. (A2.9), we note that at points c, t = β, and the velocity is infinite $\left(\frac{dz}{dw} = 0\right)$; hence

$$M\ \Im_1 - N = 0$$

and

$$\frac{dz}{dw} = M\,[\Phi(t) - \Im_1] \qquad \text{(A2.12)}$$

To evaluate the constant M in Eq. (A2.12) we note that at point b, where u = k sin πσ, v = - k $\cos^2$ πσ, $\frac{dw}{dz} = u - iv = k\,i e^{-\pi i \sigma} \cos \pi\sigma$, and t = 1,

$$\frac{1}{ki} e^{\pi i \sigma} = M\,[\Phi(1) - \Im_1] \cos \pi\sigma$$

Now, noting in the second of Eq. (A2.10) that

$$(\beta^2 - t^2)^{1-\sigma} = -e^{-i\pi\sigma}\,(t^2 - \beta^2)^{1-\sigma}$$

We have

$$\Phi(t) = \Im_1 - e^{i\pi\sigma} \int_{\beta}^{1} \frac{t\ dt}{\left(1 - t^2\right)^{1/2+\sigma} \left(t^2 - \beta^2\right)^{1-\sigma}} \tag{A2.13}$$

hence

$$M = \frac{i}{k\ \Im_2\ \cos\pi\sigma} \tag{A2.14}$$

and

$$\frac{dz}{dw} = \frac{i}{k\ \Im_2\ \cos\pi\sigma} \left[\Phi(t) - \Im_1\right] \tag{A2.15}$$

The mapping of the w plane (Fig. A2.1e) onto the lower half of the t plane is given by

$$w = \frac{i\,q}{\pi} \sin^{-1} t \tag{A2.16}$$

Now multiplying Eq. (A2.15) by derivative of Eq. (A2.16) with respect to t, we find that

$$\frac{dz}{dt} = \frac{dz}{dw}\,\frac{dw}{dt} = -\frac{q\left[\Phi(t) - \Im_1\right]}{k\pi\ \Im_2\ \cos\pi\sigma\ \sqrt{1-t^2}}$$

which, after integration with respect to t, yields

$$z = -\frac{q}{k\pi\ \Im_2 \cos\pi\sigma} \left[\int_0^1 \frac{\Phi(t)\, dt}{\sqrt{1-t^2}} - \Im_1\ \sin^{-1} t\right] \tag{A2.17a}$$

For the integral in Eq. (A2.17a) we integrate by parts.

$$\int_0^t \frac{\Phi(t)\ dt}{\sqrt{1-t^2}} = \Phi(t)\sin^{-1} t - \int_0^t \frac{t\ \sin^{-1} t\ dt}{\left(1-t^2\right)^{1/2+\sigma}\left(\beta^2 - t^2\right)^{1-\sigma}} \tag{A2.17b}$$

We shall now consider Eqs. (A2. 17) for the various parts of the flow region.

Along the bottom of the canal, where $0 < t < \beta$,

$$z = -\frac{q}{k\pi\Im_2\cos\pi\sigma}\left\{\sin^{-1} t\left[\int_0^t \frac{t\ dt}{\left(1-t^2\right)^{1/2+\sigma}\left(\beta^2 - t^2\right)^{1-\sigma}} - \Im_1\right] - \int_0^t \frac{t\ \sin^{-1}\ t\ dt}{\left(1-t^2\right)^{1/2+\sigma}\left(\beta^2 - t^2\right)^{1-\sigma}}\right\} \tag{A2.18a}$$

At point c, where t = β and z = $\frac{b}{2}$, we find

$$\frac{b}{2} = \frac{q}{k\pi \Im_2 \cos\pi\sigma} \left. \int_0^\beta \frac{t \sin^{-1} t \, dt}{(1-t^2)^{1/2+\sigma} (\beta^2 - t^2)^{1-\sigma}} \right\} \tag{A2.18b}$$

Along the side of the canal bc, where $\beta < t < 1$,

$$z = \frac{b}{2} + \frac{q}{k\pi \Im_2 \cos\pi\sigma} e^{\pi\sigma i} \left\{ \sin^{-1} t \left[\int_\beta^t \frac{t \, dt}{(1-t^2)^{1/2+\sigma} (\beta^2 - t^2)^{1-\sigma}} \right] - \int_\beta^t \frac{t \sin^{-1} t \, dt}{(1-t^2)^{1/2+\sigma} (\beta^2 - t^2)^{1-\sigma}} \right\} \tag{A2.19a}$$

At point b, where t = 1 and z = $\frac{B}{2} + ih$, we obtain

$$\frac{B-b}{2} = \frac{q}{k\pi \Im_2} \left[\frac{\pi}{2} \Im_2 - \int_1^\beta \frac{t \sin^{-1} t \, dt}{(1-t^2)^{1/2+\sigma} (t^2 - \beta^2)^{1-\sigma}} \right] \tag{A2.19b}$$

$$H = \frac{q}{k\pi \Im_2} \tan \pi\sigma \left[\frac{\pi}{2} \Im_2 - \int_1^\beta \frac{t \sin^{-1} t \, dt}{(1-t^2)^{1/2+\sigma} (t^2 - \beta^2)^{1-\sigma}} \right] \tag{A2.19c}$$

Along the free surface ba, where $1 < t < \infty$, from Eq. (A2.19a) we find

$$z = \frac{B}{2} + ih + \frac{q}{k\pi \Im_2 \cos\pi\sigma} \left\{ -\cosh^{-1} t \left[\int_1^t \frac{t \, dt}{(t^2-1)^{1/2+\sigma} (t^2 - \beta^2)^{1-\sigma}} \right] - i\Im_2 e^{\pi i\sigma} \cosh^{-1} t + \int_1^t \frac{t \cosh^{-1} t \, dt}{(t^2-1)^{1/2+\sigma} (t^2 - \beta^2)^{1-\sigma}} \right\} \tag{A2.20a}$$

Separating this equation into real and imaginary parts, we obtain for the equation of the free surface ba,

$$x - \frac{B}{2} = \frac{q}{k\pi \Im_2 \cos\pi\sigma} \left[\int_1^t \frac{t \cosh^{-1} t \, dt}{(t^2-1)^{1/2+\sigma} (t^2 - \beta^2)^{1-\sigma}} + \Im_2 \sin \pi\sigma \cosh^{-1} t - \cosh^{-1} t \int_1^t \frac{t \, dt}{(t^2-1)^{1/2+\sigma} (t^2 - \beta^2)^{1-\sigma}} \right. \tag{2.20b}$$

We shall now derive the expression for the discharge from the canal. Defining

$$\int_0^\beta \frac{t \sin^{-1} t \, dt}{\left(1-t^2\right)^{\frac{1}{2}+\sigma}\left(t^2 - \beta^2\right)^{1-\sigma}} = \int_1 (\sigma , \beta)$$

$$\int_\beta^1 \frac{t \sin^{-1} t \, dt}{\left(1-t^2\right)^{\frac{1}{2}+\sigma}\left(t^2 - \beta^2\right)^{1-\sigma}} = \int_2 (\sigma , \beta) \qquad \text{(A2.21)}$$

We have, in place of Eq. (A2.18b),

$$b = \frac{2 q}{k\pi \, \Im_2 \cos\pi\sigma} \int_1 (\sigma , \beta) \qquad \text{(A2.22a)}$$

and in place of Eqs. (A2.19b) and (A2.19c)

$$\frac{B-b}{2} = \frac{q}{k\pi \Im_2}\left[\frac{\pi}{2}\Im_2 - \int_2 (\sigma , \beta)\right]$$

$$H = \frac{q}{k\pi \Im_2} \tan \pi\sigma \left[\frac{\pi}{2}\Im_2 - \int_2 (\sigma , \beta)\right] \qquad \text{(A2.22b)}$$

Whence

$$B = b + \frac{q}{k}\left[1 - \frac{2 \int_2 (\sigma , \beta)}{\pi \Im_2}\right]$$

$$H = \frac{q}{2 k} \tan \pi\sigma \left[1 - \frac{2 \int_2 (\sigma , \beta)}{\pi \Im_2}\right] \qquad \text{(A2.22c)}$$

We note in Eqs. (A2.22c) that the quantity of seepage is dependent upon the the parameters σ and β and one of the dimensions B, b, or h, which are related by B - b = 2 h cot σπ. As was done in the previous sections, Vedernikov takes the quantity of seepage in the form

$$q = k (B + Ah) \qquad \text{(A2.23a)}$$

where \, from Eqs. (A2.22c), A is given by

$$A = \frac{2}{\tan \sigma\pi} \, \frac{\int_2 (\sigma , \beta) - \int_2 (\sigma , \beta)/\cos \sigma\pi}{\int_2 \pi/2 - \int_2 (\sigma , \beta)} \qquad \text{(A2.23b)}$$

Taking a series of values for α and β, Vedernikov obtained the correspondence between A and B/h as given in Fig. A2.2. In this figure m = cot α is the side slope of the canal.

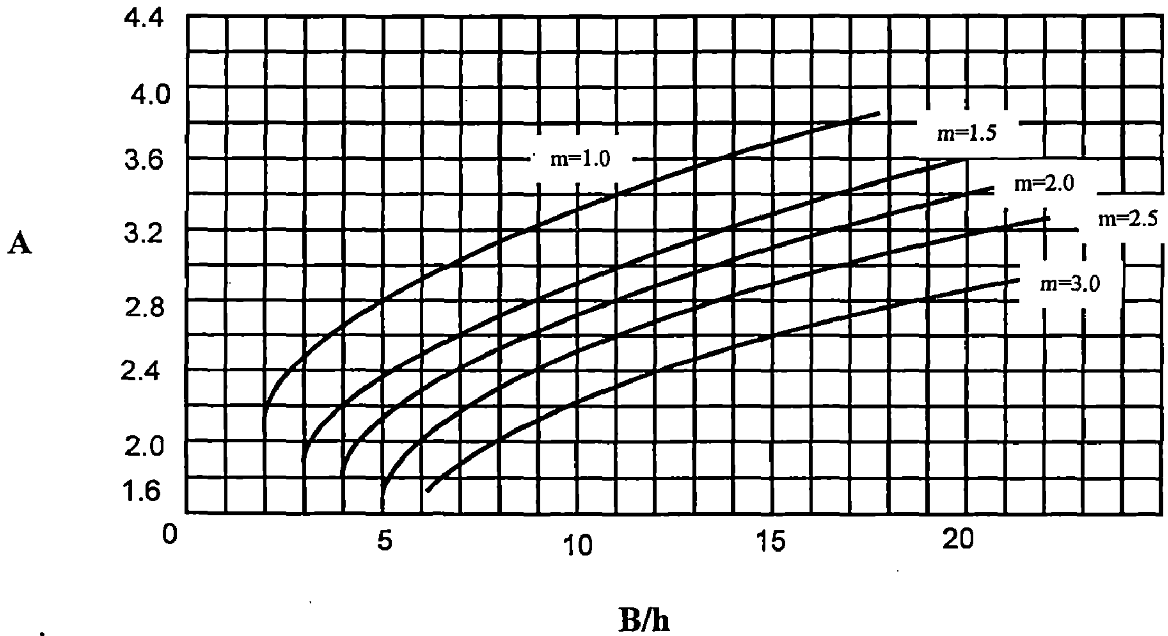

Fig. A2. 2. **(After Vedernikov)**

Comparing Eq. (A2.15a) with the expression for the quantity of seepage obtained in Eq. (A2.8) for canal with a curved perimeter ($q = k(B + 2h)$), we see that the latter solution implied $A = 2$, whereas in Fig. A2.3, for the trapezoidal section, A is seen to vary from 2 to 4 for typical values of B/h. Noting that the velocity at infinity equals the coefficient of permeability, we find that the width of the flow at infinity (Fig. A2.3) is

$$L = B + Ah \tag{A2.24}$$

Thus we see that for a trapezoidal section, as was also shown to be the case for the curved perimeter, the equipotential lines rapidly approach the horizontal. Hence the solution of this section may also be considered to provide a sufficiently valid approximation for seepage into deep horizontal filters, as shown in Fig. A2.3.

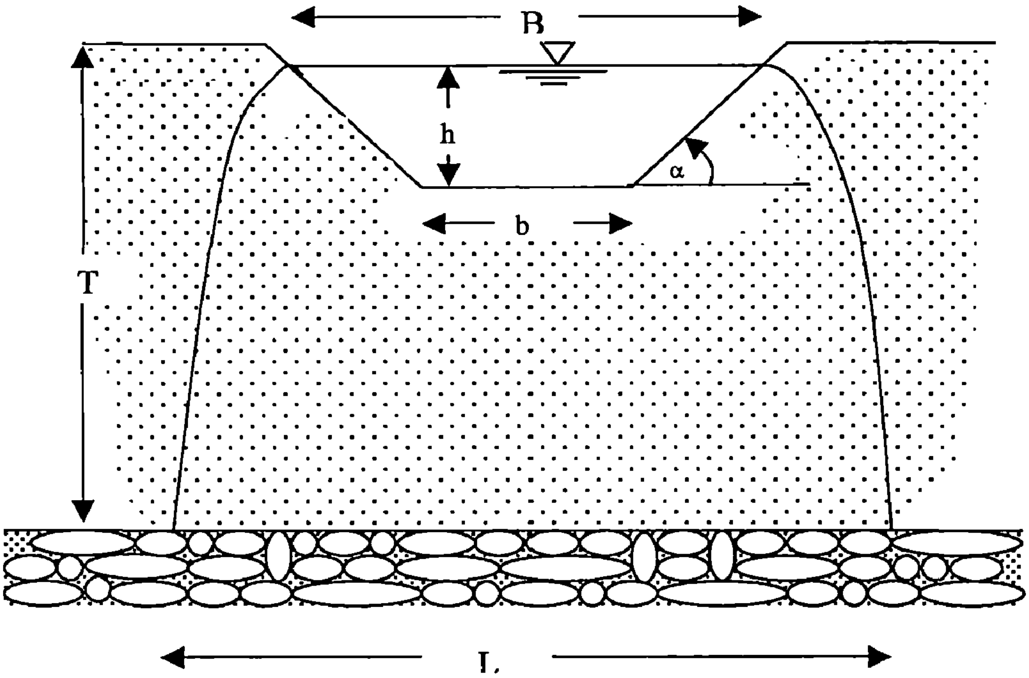

Fig. A2 .3

C.2. TRIANGULAR SHAPE

In this case of triangular canal (Fig. A2.4a), the solution of the previous section is modified by taking β = 0. When then have

$$q = k(B + A\,h) \qquad \text{(A2.25)}$$

Where (Eq. (A2. 23b))

$$A = \frac{2}{\tan \sigma\pi} \; \frac{\int_2 (\sigma)}{\Im_2 \; \pi/2 - \int_2 (\sigma)} \qquad \text{(A2.26)}$$

$$\sigma = \frac{\alpha}{\pi}$$

$$\Im_2 = \frac{\Gamma(\sigma)\,\Gamma(\tfrac{1}{2} - \sigma)}{2\sqrt{\pi}}$$

A plot of the relationship between A and α was obtained by Vedernikov and is given in Fig. (A2.4b).

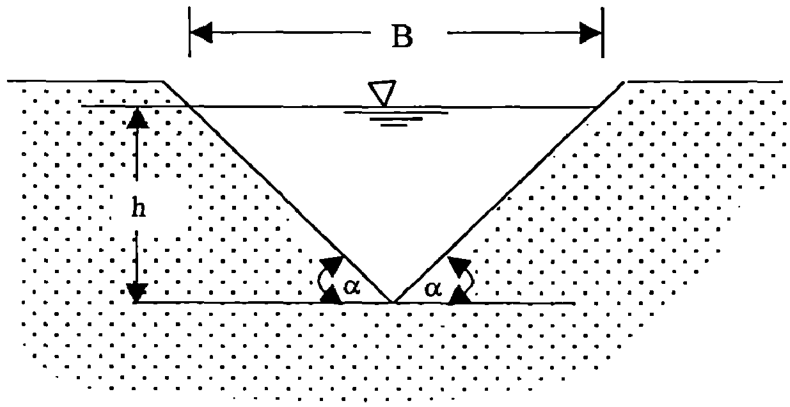

Fig. A2. 4a

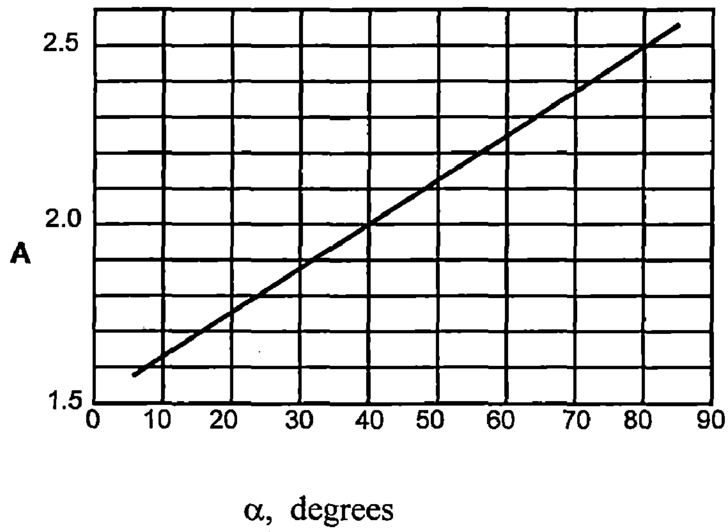

Fig. A2. 4b

APPENDIX 3

THE MANNING'S FORMULA

In 1889 the Irish Engineer Robert Manning presented a formula, which was later modified to its present well-known form

$$V = \frac{1}{2} R^{2/3} S^{1/2} \qquad \text{(A3.1)}$$

where V is the mean velocity, R is the hydraulics radius, S is the slope energy line, and n is the coefficient of roughness, specifically known as Manning's n. This formula was developed from seven different formulas, based on Bazins's experimental data, and further verified by 170 observations. Owing to its simplicity of form and to the satisfactory results it lends to practical applications, the Manning formula has become the most widely used of all uniform flow formulas for open channel flow computations.

Within the normal ranges of slope and hydraulic radius, the values of Manning's and Kutter's n are generally found to be numerically very close. For practical purposes, the two values may be considered identical when the slope is equal to or greater than 0.0001 and the hydraulic radius is between 1.0 and 30 ft.

Comparing the Chezy formula with the Manning's formula, it can be seen that

$$C = \frac{1}{n} R^{1/6} \qquad \text{(A3.2)}$$

This equation provides an important relationship between Chezy's C and Manning's n.

The exponent of the hydraulic radius in the Manning formula is actually not a constant but varies in a range depending mainly on the channel shape and roughness. For this reason, some hydraulicians prefer to use the formula with a variable exponent.

Factor affecting Manning's Roughness Coefficient

1. Surface Roughness

The surface roughness is represented by the size and shape of the grains of the material forming the wetted perimeter and producing a retarding effect on the flow. This is often considered the only factor in selecting a roughness coefficient, but it is actually just one of several majors factors.

2. *Vegetation*

   Vegetation may be regarded as a kind of surface roughness, but also markedly reduces the capacity of the channel and retards the flow. This effect depends mainly on height, density, distribution and type of vegetation, and it is very important in designing small drainage channels.

3. *Channel Irregularity*

   Channel irregularity comprises irregularities in wetted perimeter and variations in cross section, size and shape along the channel length.

4. *Channel Alignment*

   Smooth curvature with large radius will give a relatively low value of n, whereas sharp with severe meandering will increase n.

5. *Silting and Scouring*

   Silting may change a very irregular channel into a comparatively uniform one and decrease n, whereas scouring may do the reverse and increase n. However, the dominant effect of silting will depend on the nature of the material deposited.

6. *Obstruction*

   The presence of log jams, bridge piers, and the like tends to increase n. The amount of increase depends on the nature of the obstructions, their size, shape, number and distribution.

7. *Size and shape of Channel*

   There is no definite evidence about the size and shape of a channel as an important factor affecting the value of n. An increase in hydraulic radius may either increase or decrease n, depending on the condition of the channel.

8. *Stage and Discharge*

   The n value in most streams decreases with increase in stage and in discharge.

9. *Seasonal Change*

   Owing the seasonal growth of aquatic plants, grass, weeds, willow and trees in the channel or on the banks, the value of n may increase in the growing season and diminish in the dominant season.

10. *Suspended Material and Bed Load*

    The suspended material and the bed load, whether moving or not moving, would consume energy and cause head loss or increase the apparent channel roughness.

APPENDIX 4

COMPUTER PROGRAMING FOR SEEPAGE LOSSES FROM TRAPEZOIDAL CANAL

```
$DEBUG
C       IMPLICIT REAL*8 (A-H,O-Z)
C       OPEN(UNIT=1,STATUS='OLD',FILE='HARIYANA.DAT')
        OPEN(UNIT=2,STATUS='UNKNOWN',FILE='TRAPFIN.OUT')
        OPEN(UNIT=3,STATUS='OLD',FILE='GAUSS.DAT')
C       ALPHA*PAI=ANGLE SUBTENDED BY THE CANAL BANK WITH HORIZON
C       B=BOTTOM WIDTH OF CANAL
C       H=MAXIMUM DEPTH OF WATER IN THE CANAL
        ALPHA=0.25
        BBYH=0.001
        CALL SEEPAGE(ALPHA,BBYH,BBYHC,QBYKH)
        WRITE(2,10)
10      FORMAT(2X,'ALPHA',10X,'BBYHC',10X,'QBYKH')
        WRITE(2,11)ALPHA,BBYHC,QBYKH
11      FORMAT(2X,1F5.4,4X,1F10.4,6X,1F10.4)
        STOP
        END
C
        SUBROUTINE SEEPAGE(ALPHA,BBYH,BBYHC,QBYKH)
        PAI=3.14159265
        XX=ALPHA
        CALL GAMAP(XX,RES)
        TERM1=RES
        XX=0.5-ALPHA
        CALL GAMAP(XX,RES)
        TERM2=RES
        XX=0.5
        CALL GAMAP(XX,RES)
        TERM3=RES
        CBETAF=TERM1*TERM2/TERM3
        M=2000
        D=0.00001
```

```
      DELD=0.1
      ACC=0.000001
      TERMS =SIN(PAI*ALPHA)
1     CONTINUE
      D=D+DELD
      CALL  FIRSTIM(M,D,ALPHA,FINTGM)
      CALL SECONDIM(M,D,ALPHA,SINTGM)
      BBYHC=2.*SINTGM/(FINTGM*TERMS)
      RESIDUE=BBYH-BBYHC
      IF(RESIDUE.GT.0.00) GO TO 1
      IF (ABS(RESIDUE).LT.ACC)GO TO 2
      DR=D
      DL=DR-DELD
3     D=0.5*(DL+DR)
      CALL  FIRSTIM(M,D,ALPHA,FINTGM)
      CALL SECONDIM(M,D,ALPHA,SINTGM)
      TERMs =SIN(PAI*ALPHA)
      BBYHC=2.*SINTGM/(FINTGM*TERMS)
      RESIDUE=BBYH-BBYHC
      IF (ABS(RESIDUE).LT.ACC)GO TO 2
      IF(RESIDUE.GT.0.00) GO TO 4
      IF(RESIDUE.LT.0.00) GO TO 5
4     DL=D
      GO TO 3
5     DR=D
      GO TO 3
2     CONTINUE
      CALL FIRSTIM(M,D,ALPHA,FINTGM)
      CALL SECONDIM(M,D,ALPHA,SINTGM)
      BBYHC=2.*SINTGM/(FINTGM*TERMS)
      QBYKH=2.*PAI*CBETAF/(TAN(ALPHA*PAI)*FINTGM)
      RETURN
      END

      SUBROUTINE BETAINH(P,Q,X,HBETAI)
      TERM1=P
      TERM2=Q
      IF(X.GT.0.6) GO TO 10
```

```
        C4=P
        C5=1.0-Q
        C6=1.0+P
        C7=1.0
        C9=1.0
        C10=1.0
7       C9=C9*X*C4/C6*C5/C7
        C10=C10+C9
        C4=C4+1.0
        C5=C5+1.0
        C6=C6+1.0
        C7=C7+1.0
        A=ABS(C9)
        IF(A.GT.0.0000001) GO TO 7
        HBETAI=X**P*C10/P
        RETURN
10      CONTINUE
        XX=P
        CALL GAMAG(XX,RES)
        GAMAP=RES
        XX=Q
        CALL GAMAG(XX,RES)
        GAMAQ=RES
        PPLUSQ=P+Q
        XX=PPLUSQ
        CALL GAMAG(XX,RES)
        GPLUSQ=RES
        BETAPQ=(GAMAP*GAMAQ)/GPLUSQ
        P=TERM2
        Q=TERM1
        X=1.-X
    C4=P
        C5=1.0-Q
        C6=1.0+P
        C7=1.0
        C9=1.0
        C10=1.0
77      C9=C9*X*C4/C6*C5/C7
```

```
      C10=C10+C9
      C4=C4+1.0
      C5=C5+1.0
      C6=C6+1.0
      C7=C7+1.0
      A=ABS(C9)
      IF(A.GT.0.000001) GO TO 77
      HBETAI=BETAPQ-X**P*C10/P
      RETURN
      END

      SUBROUTINE GAMAP(XX,RES)
C     THIS SUBROUTINE IS ONLY VALID FOR XX LESS THAN 1.
C     WE ARE COMPUTING GAMA(XX)
      DIMENSION B(8)
      B(1)=-0.577191652
      B(2)=0.988205891
      B(3)=-0.897056937
      B(4)=0.918206857
      B(5)=-0.756704078
      B(6)=0.482199394
      B(7)=-0.193527818
      B(8)=0.035868343
      SUM=1.0
      DO  M=1,8
      SUM=SUM+B(M)*XX**M
      END DO
      RES=SUM/XX
      RETURN
      END

      SUBROUTINE GAMAG(XX,RES)
      IF(XX.LT.1.000) GO TO 100
      XX1=XX
      N=XX
      XX=XX-N
      PRODUCT=1.
      DO I=1,N
```

```
      PRODUCT=PRODUCT*(XX+I-1)
      END DO
      CALL GAMAP(XX,RES)
      RES=PRODUCT*RES
      XX=XX1
      RETURN
100   CONTINUE
      CALL GAMAP(XX,RES)
      RETURN
      END

      SUBROUTINE BETAPQ(P,Q;BPQ)
      XX=P
      CALL GAMAG(XX,RES)
      TERM1=RES
      XX=Q
      CALL GAMAG(XX,RES)
      TERM2=RES
      XX=P+Q
      CALL GAMAG(XX,RES)
      TERM3=RES
      BPQ=TERM1*TERM2/TERM3
      RETURN
      END

      SUBROUTINE FIRSTIM(M,D,ALPHA,FINTGM)
      DIMENSION FV(2001)
      P=ALPHA
      Q=0.5-ALPHA
      CALL BETAPQ(P,Q,BPQ)
      FV(1)=BPQ/(1.+D)**0.5
      FV(M+1)=0.
      V=0.
      DELV=1./M
      DO I=2,M
      V=V+DELV
      P=ALPHA
      Q=0.5-ALPHA
```

```
X=1.-V**2
CALL BETAINH(P,Q,X,HBETAI)
FV(I)=HBETAI/(D+1.-V**2)**0.5
END DO
FINTGM=0.
DO I=1,M
FINTGM=FINTGM+0.5*(FV(I)+FV(I+1))*DELV
END DO
FINTGM=2.*FINTGM
RETURN
END

SUBROUTINE SECONDIM(M,D,ALPHA,SINTGM)
DIMENSION FV(2001)
P=ALPHA
Q=0.5
X=D/(1.+D)
CALL BETAINH(P,Q,X,HBETAI)
FV(1)=HBETAI/(D+1.)**0.5
FV(M+1)=0.
V=0.
DELV=D**0.5/M
DO I=2,M
V=V+DELV
P=ALPHA
Q=0.5
X=(D-V**2)/(1.+D-V**2)
CALL BETAINH(P,Q,X,HBETAI)
FV(I)=HBETAI/(D+1.-V**2)**0.5
END DO
SINTGM=0.
DO I=1,M
SINTGM=SINTGM+0.5*(FV(I)+FV(I+1))*DELV
END DO
SINTGM=2.*SINTGM
RETURN
END
```

APPENDIX 5

COMPUTER PROGRAMMING FOR CANAL DIMENSION

```
PROGRAM CANAL
CHARACTER*16 FILENAME

REAL H1,AA,BB1,BB2,CC1
REAL Q0,H0,B,ALFA1,ALFA2,N,L,I
WRITE(*,*)' ENTER INPUT DATA FILENAME'
READ(*,'(A)')FILENAME
OPEN (UNIT=2,FILE=FILENAME,STATUS='OLD')
WRITE(*,*)' ENTER OUTPUT DATA FILENAME'
READ(*,'(A)')FILENAME
OPEN (UNIT=3,FILE=FILENAME,STATUS='UNKNOWN')
READ (2,*)Q0,H0,B,ALFA1,ALFA2,N,L,I
!        I IS SEEPAGE LOSSES
G=9.81
BB1=B+(2*(0.5*(H0+H1))*ALFA1)
BB2=I/(K*(0.5*(H0+H1)))
CC1=(1/N)*(((B*H1+H1**2*ALFA1)**(5/3))/((B+2*H1*ALFA2)**(2/3))

READ(2,*)H1
READ (2,*) DELH1
!          WRITE(3,*)'DATA H1'
!          WRITE(3,*)H1
1          H1=H1+DELH1
CALL F(Q0,H1,BB1,BB2,CC1,B,ALFA1,L,H0,AA)
!          WRITE(3,*)'CALL 1'
           WRITE(3,*)H1,AA
IF (AA.LT.0) GOTO 1
H1R=H1
H1L=H1R-DELH1
2   H1=(H1L+H1R)/2
```

```
        CALL F(Q0,H1,BB1,BB2,CC1,B,ALFA1,L,H0,AA)
!       WRITE(3,*)'CALL2'
        WRITE(3,*)H1,AA
        IF (ABS(AA).LT.0.00001) GOTO 5
        IF (AA.LT.0) GOTO 3
        IF (AA.GT.0) GOTO 4
3    H1L=H1
     GOTO 2
4    H1R=H1
     GOTO 2
5    CONTINUE
     WRITE(3,*)'OUTPUT F=0'
     WRITE (3,*)H1
STOP
END PROGRAM COBA

SUBROUTINE F(Q0,H1,BB1,BB2,CC1,L,G,H0,AA)
REAL Q0,H1,AA,BB1,BB2,CC1
REAL L
AA=Q0-K*(BB1+((BB2-((BB1/(0.5*(H0+H1)))*(0.5*(H0+H1))*L)-CC1
RETURN
END SUBROUTINE
```