5.0 SYSTEM DESIGN PROCEDURE

5.1 Surface Drainage

5.1.1 Unit hydrograph approach for estimation of design flood for agricultural drainage system

The main source of excess water on the land surface is the rain falling over the catchment area. A part of the rainfall reaching on the land surface infiltrate into the ground which percolates through the layers of the soil and contributes to its water supply or feeds the lower aquifer. Simultaneously, depending upon the intensity of the rainfall and antecedent soil moisture conditions, a portion of the rainwater appears as the overland flow which runs on the surface of the soil down the slope to ditches and rivers. Climatic demands and plant growth cause the losses of water in the form of evapotranspiration. These process of water movement represent the components of hydrologic cycle. Due to uneven distribution of rainfall in time and space, some parts of the catchment receive excessive rainfall which is harmful for the plants in the agricultural areas. In addition to this excess flood water also causes distress and hardship to the population and damage to the environment. Land drainage is the process through which the excess flood water can be removed from the agricultural fields or low lying area. In general the principle objectives of land drainage works are as follows:

a) Drainage, to reduce the water content of agricultural land to prevent water logging

b) Flood protection, to minimise the overflowing of rivers on to agricultural crops and on to built up areas

c) Conservation, to contain the rivers in their channels for the benefit of riparian users and navigation

d) Disposal of surface water, to provide outfalls and proper drainage channels for surface water flowing from urban areas

e) Sea defences, to prevent sea water penetrating on to land areas.

In order to design efficient land drainage works, the Civil Engineer, Agricultural Engineer, and water resources engineer need design flood estimates. The methods which have been more often used for design flood estimation include rational method, empirical method, flood frequency method, unit hydrograph technique and watershed models.

The application of the rational method is limited to small size (<50 Km2) catchments. The empirical formulae are essentially the regional formulae based on statistical correlation of the observed peak and important catchment properties. These formulae are only applicable in the region from which they were developed within the range of flood peaks used. If such formulae are applied to other areas they can at best give approximate values. The frequency analysis approach comprises statistical methods to predict the flood peaks of a specified return period. The unit hydrograph method is basically a rainfall-runoff relationship normally applicable to moderate size catchments with area more than 25 sq.km. but less than 5000 sq.km. (IS:5477-1971).

In the last two decades, with the advent of high speed digital computers, multiparameter watershed models which include U.S.G.S. model, Betson model, USDAHL Model, CREAMS Model, NWSRFS have been developed which as their authors claim give satisfactory results. Still the unit hydrograph approach is the most widely used technique for the estimation of design flood although it is nearing its 60th anniversary. The reason for this is that the uncertainty involved in the estimation of parameters of the complex watershed models may lead to erroneous design flood estimates. Neverthless, many researchers presently involved in rainfall-runoff modelling, are still trying to improve upon the parameter estimation techniques in order to provide better estimates for the parameters specially for the design conditions. In this chapter the procedure applicable to small catchments with gauged/ungauged flood flows for the design of agricultural drains based on unit hydrograph approach has been discussed.

5.1.1.1 UNIT HYDROGRAPH THEORY AND ASSUMPTIONS

A unit hydrograph is a hydrograph of direct surface runoff that would result at a given point in a stream from unit rainfall excess occurring in unit time uniformly over the catchment area above that given point. The basic assumptions of the unit hydrograph theory are:

i) Effective rainfall occurs at a uniform time rate during the selected unit hydrograph interval.

ii) Effective rainfall is uniformly distributed over the whole catchment for which the unit hydrograph is developed and applied.

iii) Ordinates of the direct runoff hydrographs are proportional to those of the unit hydrograph or the total direct runoff of each hydrograph is proportional to the volume of unit hydrograph; and

iv) Unit hydrograph reflects the combined effects of all the physical characteristics of a catchment.

These assumptions are only approximately satisfied for any catchment. It is often claimed, and verified in practice, that in case of flood events carefully selected for small catchments these assumptions are not significantly violated, with approximations acceptable for practical purposes.

5.1.1.2 PROCEDURE FOR COMPUTATION OF DESIGN FLOOD HYDROGRAPH BASED ON UNIT HYDROGRAPH APPROACH

The following computational steps are involved in the determination of design flood hydrograph for the design of agricultural drainage.

i) Determining a Design Rainfall

The amount of rain that falls on the ground in a certain period is expressed as a depth (mm, inches etc.) to which it would cover a horizontal plane on the ground. The rainfall depth may be considered a statistical variate, its value depending on:

i) the season of the year
ii) the duration selected
iii) the area under study.

In a design, the frequency, season, and duration chosen depend on the type of problem under consideration. The detailed description of the methodology for determining the design rainfall of the selected duration under different data situations is given by ILRI (1974).

As per Indian standard guidelines for planning and Design of Surface drains (IS:8835-1978), the drains should be designed for three days rainfall of 5 year frequency for maintaining optimum cost ratio. However,in specific cases requiring a higher degree of protection, the frequency of 10 or 15 years can also be adopted after proper justification in terms of the economics. The recommended periods of disposals depending upon the tolerance of individual crops are given below :

i)	Paddy	7 to 10 days
ii)	Maize, bajra and other similar crops	3 days
iii)	Sugarcane and bananas	7 days
iv)	Cotton	3 days
v)	Vegetables	1 day (in the case of vegetables, 24 hour rainfall will have to be drained in 24 hours)

Cross drainage works are always designed for a higher discharge than the cut sections of the drains on account of the fact that the damage caused to the structures in the event of flows resulting from rainfall higher than the design rainfall, can be much more than to the drain. A 3 day rainfall of 50 years frequency is recommended (IS:8835-1978). However, the time of disposal remains the same depending on the type of crop.

(ii) Time Distribution

For the distribution of Design Flood using unit hydrograph approach, short interval increments of design storm would be required. In India, in view of the sparse network of SRRG stations, it is not an easy job to have the correct design rainfall increments at shorter interval. As such a recourse is taken to determine the short interval design rainfall increments by applying a time distribution based on the hourly rainfall data observed at a group of self recording raingauge stations during a severe storm in the meteorologically homogeneous region under study. The steps involved in deriving the time distribution are indicated below:

i) Select all major storms in the region

ii) Compute the maximum hourly rainfall totals for 1,2,3,6,9,12,15......72 hours using only consecutive hourly rainfall data.

iii) Express the maximum rainfall totals computed in step (ii) as percentage to the total rainfall amount of the 72 hour duration (or other durations as required).

iv) Repeat the procedure in step (ii) and (iii) for all selected storm spells

v) Plot the percentages of the different durations from each spell on a graph paper and draw smooth curves.

vi) Drawn an envelope curve passing through the maximum percentages for each duration from out of different storm spells selected.

Fig. 5.1 shows the average time distribution curves of storms of various duration.

NOTE: Though it is the general practice by IMD to draw envelope curve of percentages obtained at individual SRRG stations, it would be desirable if a weighted average of hourly rainfall at SRRG stations is obtained and then the percentage ratios worked out.

vii) Use the above percentage time distribution to distribute the design rainfall (3 day 5-year rainfall for drainage design) in time.

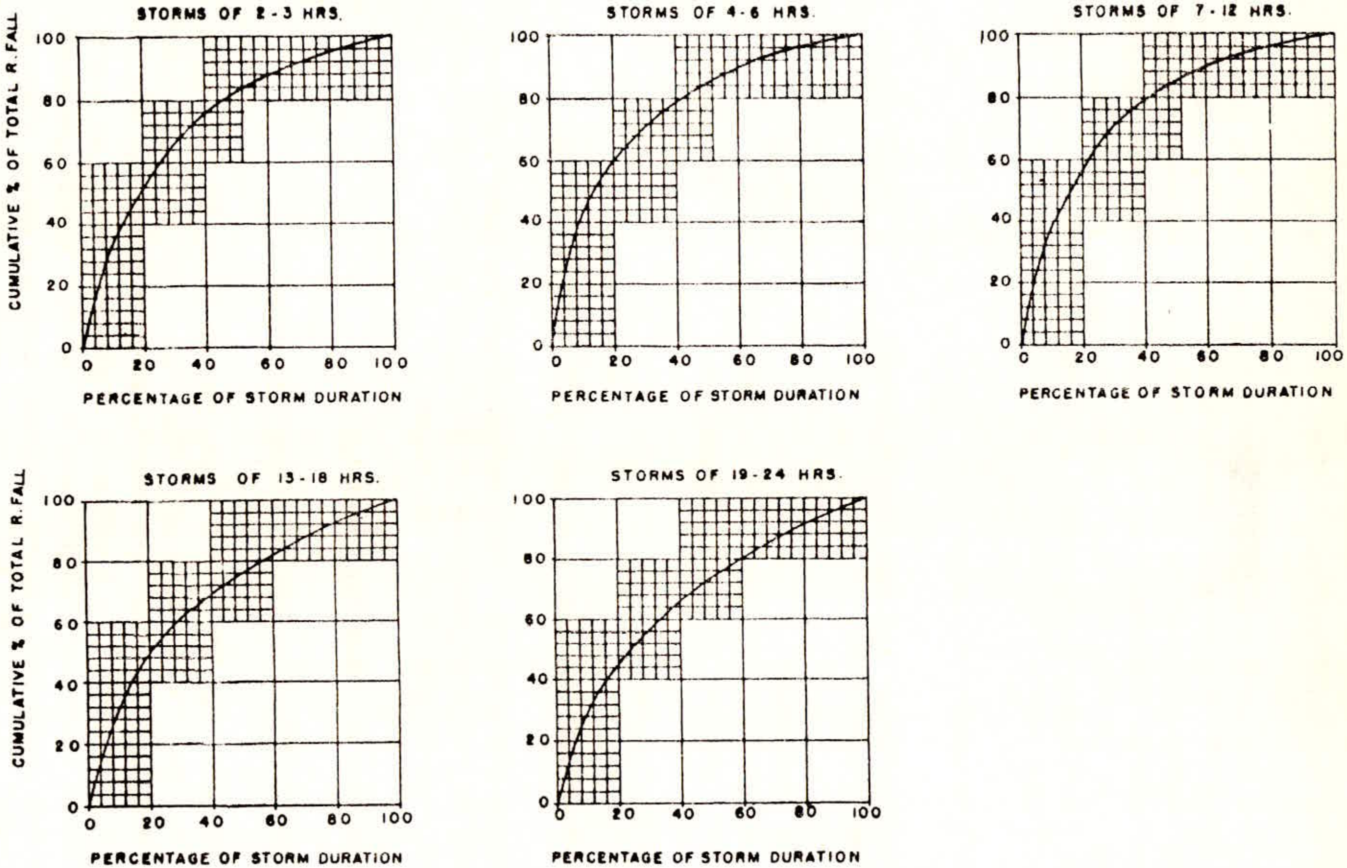


Figure 5.1 A mass curve representation of the SCS rainfall runoff relationship

(iii) Computation of effective design rainfall

Generally the infiltration and other losses are derived from the available rainfall-runoff records for the several storms in the basin using a suitable infiltration model. Assuming the basin would be saturated at the time of design rainfall, the minimum loss rate values may be considered. In case the available data for the basin are limited, the loss rate for the design situation may be estimated using the following runoff coefficients for different soils recommended for plain areas: (IS:8835-1978)

i)	Loam, lightly cultivated or covered	0.40
ii)	Loam, largely cultivated and suburbs with gardens, lawns, macade mized roads	0.30
iii)	Sandy soils, light growth	0.20
iv)	Parks, lawns, meadows, gardens, cultivated area	0.05-0.20
v)	Plateaus lightly covered	0..70
vi)	Clayey soils stiff and bare and clayey soils lightly covered	0.55

Depending upon the soil type and cover conditions the runoff coefficients may be multiplied by the design rainfall to get the design excess rainfall which may be distributed in time using the time distribution.

5.1.1.3 Derivation of Design Unit Hydrograph

a) Derivation of unit hydrograph for gauged catchments

Unit hydrographs can be derived from analysis of rainfall and runoff records in those catchments where such data are available. The procedures used to derive a unit hydrograph are dependent upon whether the storm from which a unit hydrograph is to be derived is a simple or single period storm or a multi period storm. Thunder storms are usually intense and of short duration, and more likely to be treated as single period storms. Frontal storms are usually of longer duration and therefore, are generally not suitable for single period analysis. A multiperiod approach should be followed for these.

The methods available in literature for the derivation of unit hydrograph from the multiperiod storms are mainly based on two approaches:

i) Non Parametric System Analysis Approach
ii) Parametric System Synthesis Approach

i) Non Parametric System Analysis Approach

In this approach the input (excess rainfall) and output (direct surface runoff) are related in the form of a mathematical expression through the system response function.

The methods based on non parametric system analysis approach may be classified in five different groups:

* Conventional Methods
* Matrix Methods
* Transform Methods
* Time Series Methods
* Linear Programming Methods

ii) Parametric System Synthesis Approach

This requires the use of conceptual linear models. These models evaluate the system response in terms of certain number of parameters which can be estimated from the given input and outputt data. Some of the well known models which have been developed, based on this approach, and widely used include.

* Clark Model	(Clark, 1945)
* Nash Model	(Nash, 1957)
* Dooge Model	(Dooge, 1959)
* O'Kelly's Model	(O'Kelly, 1955)
* Diskin's Model	(Diskin, 1972)
* Singh's Model	(Singh, 1964)
* Kulandaiswamy Model	(Kulandaiswamy, 1965)
* Laurenson Model	(Laurenson, 1964)
* Prasad Model	(Prasad, 1967)

It is to be noted that the system response is a continuous mathematical function, while using parametric system synthesis approach. This continuous mathematical function is termed as instantaneous unit hydrograph (IUH) or impulse response. In response the relationship between IUH and T-hour unit hydrograph is utilised. In the case of non parametric system analysis approach the discritization interval t defines the response function which becomes the unit pulse response rather than inpulse response. The methods based on the parametric system synthesis approach have some distinct advantages over the methods based on non-parametric approach. Those include:

i) They always estimates ordinates of unit hydrograph as positive ordinates.

ii) They preserve the shape of the unit hydrograph.

iii) The errors in input data are not able to distort the shape of the unit hydrograph.

iv) The optimisation procedure based methods are capable of admitting a number of events simultaneously for the reprresentative unit hydrograph derivation.

v) Since only few parameters define the complete shape of the unit hydrograph, therefore, the parameters obtained from the gauged basins of the region can be easily correlated with catchment characteristics to develop the regional relationships for use in the derivation of the unit hydrographs for the ungauged catchments of the region.

The methods based on the above two approaches have been briefly described by Singh (1984-85).

b) Derivation of unit hydrograph for ungauged catchments

Whenever sufficient and reliable records on stream flow and rainfall are available the unit hydrograph for those catchments can be derived from the rainfall-runoff data of storm events using one of the methods described. However, most of the small agricultural catchments are generally not gauged and many land drainage works are being planned in those catchments. Therefore, it becomes necessary to have the estimates of floods at the proposed sites in small agricultural catchments. The unit hydrographs for such catchments are estimated by using data on climatological, physiographic and other factors of these catchments. This necessitates the development of suitable regional relationships for unit hydrograph derivation. The procedure used for this purpose involves the derivation of the parameters that describe the unit hydrograph for gauged catchments and then the development of the regional relationships between the unit hydrograph parameters with pertinent physiographic and storm characteristics of the catchments. The catchments considered for such regional study have to be similar in hydrological and meteorological characteristics. The methodology for carrying out regional unit hydrograph analysis has been described elsewhere (NIH, 1988-89).

c) Averaging of unit hydrographs

Generally the unit hydrographs derived from various events are not the same for the gauged catchment. The estimation of design flood requires a representative unit hydrograph for the catchment. There are two possible approaches for the derivation of the representative unit hydrograph for the catchment, in the first approach, the unit hydrographs derived from various events are averaged by the conventional averaging methods. However, the second approach considers the joint event, obtained from clubbing the various events together for the analysis and provides a single representative unit hydrograph for the catchment. Based on the above approaches, the methods for averaging of unit hydrographs of the same duration are:

i) Text book method of averaging

ii) Ordinate by ordinate averaging

- Mean method
- Median method
- Mean peaks aligned method
- Median peaks aligned method

iii) Shape factor averaging

iv) Joint analysis technique

- Event concatenation
- Event super position.

The above methods of averaging unit hydrographs have been described elsewhere (NIH, 1990).

d) Change of unit period of a unit hydrograph

The unit hydrograph is a conversion factor which converts the excess rainfall to direct surface runoff. For this conversion, the duration of unit hydrograph should be the same as the duration of the unit hydrograph and excess rainfall blocks differ, then the duration of unit hydrograph should be changed to the duration of excess rainfall, and the unit hydrogaph with changed duration should be used for converting the excess rainfall to the direct surface runoff. Generally short duration and intense rainfall events are most suited for the derivation of unit hydrograph. It provides an estimate for unit hydrograph of shorter duration. For the recommended duration of design rainfall, a 3 day duration unit hydrograph is required. Thus the shorter duration of the unit hydrograph should be changed to 3 day duration unit hydrograph which may be used to convert the 3 day excess design rainfall to the design direct surface runoff hydrograph. Two approaches based on S-curve and superimposition method are generally available (NIH, 1990) for changing the duration of unit hydrograph. The former approach of S-curve is more general than the later one which is used for changing the duration of unit hydrograph only when the new duration is the integer multiple of the original duration.

5.1.1.4 Computation of Design Flood Hydrograph

The design frequency rainfall for the selected duration are applied to the design unit hydrograph to obtain the total design direct surface runoff hydrograph also known as the design flood hydrograph considering the negligible baseflow. For the estimation of design direct surface runoff or design flood hydrograph (negligible base flow) the following convolution summation equation may be used:

$$Q_j = \sum_{i=0}^{j-1} R_i \, U_{j-i}$$

where,

Q_j is the direct surface runoff at time jT

R_i is the excess rainfall block at time iT for duration of time T and

U_j is the ordinate of T-hour unit hydrograph at time jT.

In large areas, there are often different types of crops grown. In such a situation, the runoff for composite crops would be needed to design the surface drains. It has been recommended (IS:8835-1978) that the field and link drains can be designed on outfall drain, either a composite discharge can be worked out or the total discharge can be worked out by taking into account the discharges from individual link drains. As the area grows longer, the chances of synchronization of discharge from the entire area become less. As such, working out a composite discharge may also serve the purpose.

5.1.1.5 REMARKS

Unit hydrograph approach is one of the most useful and simple techniques available in literature for converting the excess rainfall into direct surface runoff hydrograph. For gauged catchments that short duration rainfall runoff records for some past events can be analysed to derive the unit hydrograph. The unit hydrographs, thus derived from various events, are not the same. A representative unit hydrograph for the catchment can be derived by averaging the different unit hydrographs. The above procedure can not be applied for the derivation of unit hydrograph for ungauged catchments. Unit hydrograph for such catchments can only be estimated by using data on climatological, physiographic and other factors affecting the runoff in the regional unit hydrograph relationships which can be developed by relating the unit hydrograph parameters of the gauged catchments in the region with their pertinent physiographic and storm characteristics. While converting the excess rainfall to direct surface runoff, the unit hydrograph and the excess rainfall block should be of the same duration. If both are not the same, the unit hydrograph of the changed duration should be derived.

The recommended design rainfall for the design of agricultural drainage system is 3 day 5 year rainfall. A critical time distribution for this design rainfall would be needed in order to compute the short interval increments of design rainfall. Subsequently the effective design rainfall can be computed after substracting the design loss rate from the rainfall increments. The design excess rainfall can be converted to the design flood hydrograph using the representative unit hydrograph.

5.1.2 ESTIMATION OF DIRECT RUNOFF FROM STORM RAINFALL BY SCS METHOD

The principal application of the method is in estimating quantities of direct runoff from storm rainfall. The SCS rainfall-runoff relation uses the total of one or more storms occurring in a calendar day and nothing is used about the time distribution. The relation therefore excludes time as an explicit variable, which means that rainfall intensity is ignored. If every thing but storm duration or intensity is the same for two storms, the estimate of runoff is the same for both storms. Soil properties influence the process of generation of runoff from rainfall and they must be considered, even if only indirectly, in methods of runoff estimation. When runoff from individual storms is the major concern, as in flood prevention work, the properties can be represented by a hydrologic parameter: the minimum rate of infiltration obtained for a bare soil after prolonged wetting. The influences of both the surface and the horizons of a soil are thereby included.

The infiltration rate is the rate at which water enters the soil at the surface and which is controlled by surface conditions, and the transmission rate is the rate at which the water moves in the soil and which is controlled by the horizons. The hydrologic soils classification takes into consideration the minimum rate of infiltration for bare soil after prolonged wetting. The hydrologic soil groups, as defined by SCS soil scientists, are:

A. (Low runoff potential) Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission of the order 7.5-11.5 cm/hr.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission of the order 4-7.5 cm/hr.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures. These soils have a slow rate of water transmission of the order 0.13-4 cm/hr.

D. (High runoff potential). Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission of the order 0-0.13 cm/hr.

Besides, infiltration characteristics the antecedent moisture condition in a catchment governs significantly the fraction of storm precipitation which is converted to direct runoff. SCS method recognises three stages of antecedent moisture conditions which are:

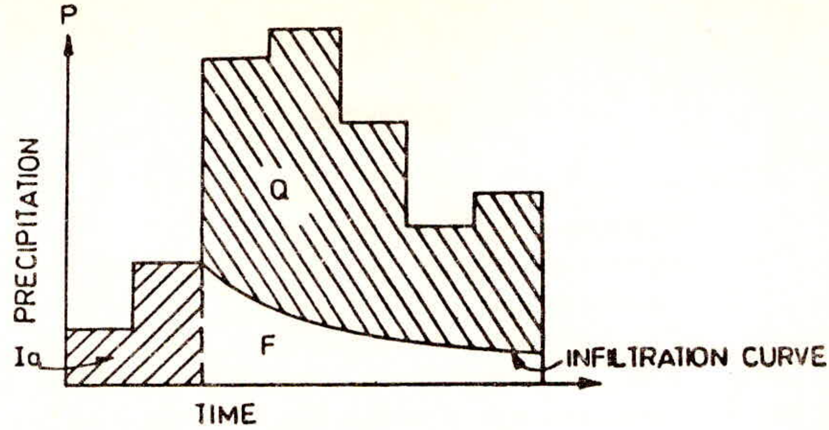
AMC-I (Lowest runoff potential).
If the watershed soils are dry enough for satisfactory plowing or cultivation to take place, the moisture condition is classified as AMC-I

AMC-II For the average condition, the soil moisture condition is classified as AMC-II

AMC-III (Highest runoff potential).
If the watershed is practically saturated from antecedent rains the moisture condition is considered as AMC-III. The AMC can be estimated from 5-day antecedent rainfall by the following table which gives the rainfall limits by season categories.

AMC Group	Dormant season (cms)	Growing season (cms)
I	< 1.3	< 3.6
II	1.3 to 2.8	3.6 to 5.4
III	> 2.8	> 5.4

SCS method assumes the following rainfall-runoff relation which is shown schematically in Fig. 5.1:



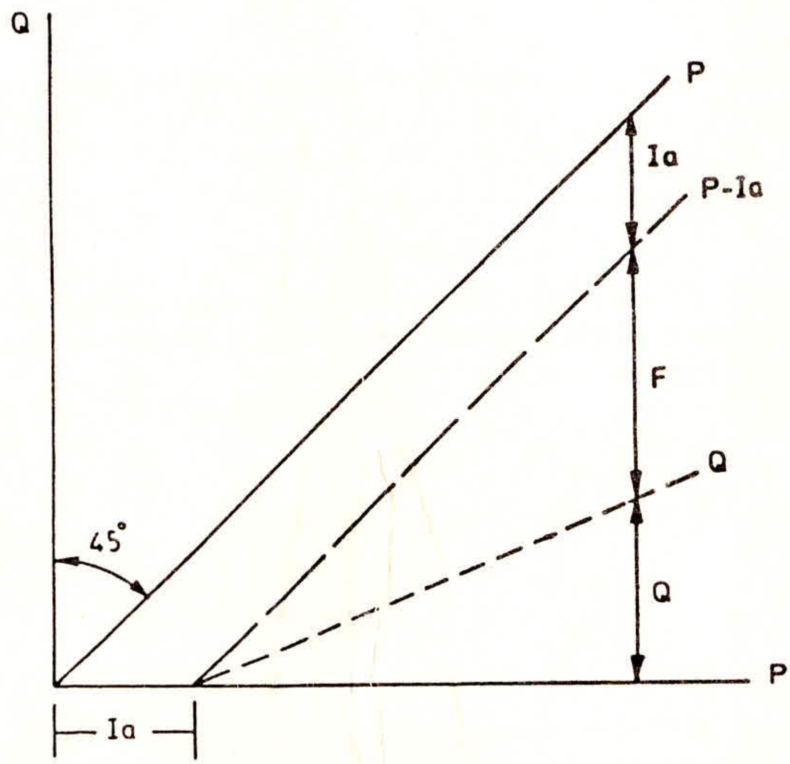


Fig.5.1 A Mass Curve Representation of the SCS Rainfall-Runoff Relationship

$$\frac{F}{S} = \frac{Q}{P\text{-}I_a} \qquad (5.1)$$

in which,

S	=	Potential maximum retention,
P	=	Volume of precipitation,
Q	=	Volume of runoff,
F	=	Actual retention equal to $P\text{-}I_a\text{-}Q$,
I_a	=	Initial abstraction which consists of interception, infiltration and surface storage all of which occur before runoff begins. The initial abstraction is a function of land use, and antecedent soil moisture condition.

The actual retention has been expressed as:

$$F = (P - I_a) - Q \qquad (5.2)$$

Equations (5.1) & (5.2) have been combined and the following relation has been derived:

$$Q = \frac{(P - I_a) - Q}{P - I_a + S} \qquad (5.3)$$

An empirical relationship between Ia and S has been suggested which is based on analysis of rainfall and runoff data from several experimental watershed in USA and the relationship is given by:

$$I_a = 0.2S \qquad (5.4)$$

Substituion of Ia value in equation (5.3) leads to

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \; ; \; p \geq 0.2S \qquad (5.5)$$

Equation (5.5) is the hydrologic relation between storm rainfall, soil site storage, and storm runoff used in the SCS method of estimating direct runoff from storm rainfall. The direct runoff consists of channel runoff, surface runoff and subsurface flow (interflow). It does not include the baseflow component. In equation (5.5) the storage parameter, S, is an indicator of land condition. It may vary from zero for an impervious watershed to infinite for a completely absorbent watershed. The value of S is still unknown. Based on analysis of rainfall runoff data in several watersheds having the same cover complex the storage in the form of curve number, CN, have been defined by SCS for various hydrologic cover complex. A hydrologic complex is a combination of a hydrologic soil group and a land use and treatment class.

The relation between curve number (CN) and storage index (S) is given by

$$CN = \frac{1000}{S+10} \tag{5.6}$$

where CN is a dimensionless number, having no intrinsic meaning. It gives a convenient transformation of S to establish a 0 to 100 scale. The CN incorporates the effects of infiltration characteristics of the soil, land use and agricultural practices.

Curve Numbers for different vegetative types and hydrologic soil groups are presented in Table 5.1 for AMC-II condition. The conversion factor to find curve number for other AMC conditions are given in Table 5.2. A graphical presentation of the variation of direct runoff, Q, with rainfall, P, for I_a = 0.2S has been made in Fig. 5.2

5.1.2.1 Determination of Direct Runoff Consequent To Several Days Consequtive Rainfall

Direct runoff volume when rainfall continues for several days can be predicted using SCS method that has been outlined by Hawkins (1978). The procedure is described below which is based on updating the curve number by continuous accounting of storage.

Using the basic SCS rainfall runoff equation (5.5), expanding the numerator and applying polynomial division

$$Q = P - S\,[1.2 - S/(P + 0.8\,S)]\,,\ P \geq 0.2S \tag{5.7}$$

It can be seen from the equation (5.7) that the possible difference (as $P \to \infty$) between rainfall (P) and direct runoff (Q) is not S but 1.2 S.

Table 5.1 **Runoff curve numbers for hydrologic soil-cover complexes**
(Antecedent moisture condition, II, and $I_a = 0.2S$)

Cover			Hydrologic soil group			
Land use	Treatment or practice	Hyrologic condition	A	B	C	D
Fallow	Straight row	---------	77	86	91	94
Row crops	"	Poor	72	81	88	91
	"	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	"	Good	65	75	82	86
	" and terraced	Poor	66	74	80	82
	" " "	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	" and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes 1/ or rotation meadow	Straight row	Poor	66	77	85	89
	" "	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	"	Good	55	69	78	83
	" and terraced	Poor	63	73	80	83
	" and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	"	Fair	25	59	75	83
	"	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	73	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		------	59	74	82	86
Roads (dirt)2/		------	72	82	87	89
(hard surface) 2/		------	74	84	90	92

1/ Close-drilled for broadcast
2/ Including right-of-way

Table 5.2 Curve numbers (CN) and constants for the case $I_a = 0.2\ S$

CN for condition II	CN for conditions I	CN for conditions III	CN for condition II	CN for conditions I	CN for conditions III
100	100	100	60	40	78
99	97	100	59	39	77
98	94	99	58	38	76
97	91	99	57	37	75
96	89	99	56	36	75
95	87	98	55	35	74
94	85	98	54	34	73
93	83	98	53	33	72
92	81	97	52	32	71
91	80	97	51	31	70
90	78	96	50	31	70
89	76	96	49	30	69
88	75	95	48	29	68
87	73	95	47	28	67
86	72	94	46	27	66
85	70	94	45	26	65
84	68	93	44	25	64
83	67	93	43	25	63
82	66	92	42	24	62
81	64	92	41	23	61
80	63	91	40	22	60
79	62	91	39	21	59
78	60	90	38	21	58
77	59	89	37	20	57
76	58	89	36	19	56
75	57	88	35	18	55
74	55	88	34	18	54
73	54	87	33	17	53
72	53	86	32	16	52
71	52	86	31	16	51
70	51	85	30	15	50
69	50				
68	48	84	25	12	43
67	47	83	20	09	37
66	46	82	15	06	30
65	45	82	10	04	22
64	44	81	05	02	13
63	43	80	00	00	00
62	42				
61	41				

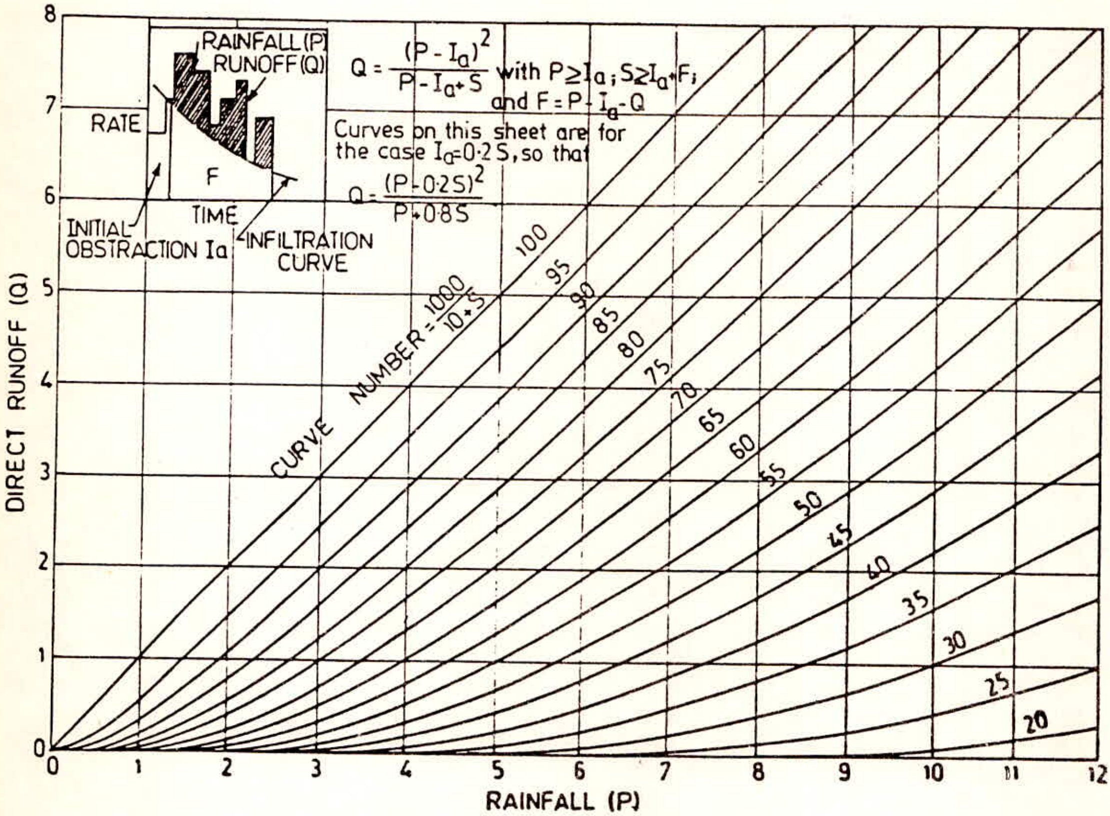


Fig.5.2 Variation of Q vs. P for $I_a = 0.2S$

Let $V = 1.2\ S$

This can be envisaged as the total storage available on the site for a given condition of soil vegetation and moisture status.

At time t , the storage available is

$$V_1 = 1.2\ S_1 = 1.2\ (1000/CN_1 - 10) \qquad (5.8)$$

Any change in V_1, will be due to the actual evapotranspiration losses (ETA), rainfall (P) and runoff (Q) occurring at that time.

So at time t_2

$$V_2 = V_1 + ETA_1 - (P_1 - Q_1) \qquad (5.9)$$

Therefore

$$V_2 = 1.2\ (1000/CN_1 - 10) + ETA1 - (P_1 - Q_1) = 1.2\ S_2 \qquad (5.10)$$

By defintiion

$$CN_2 = \frac{1000}{10 + V_2/1.2}$$

Substituting value of V_2

$$CN_2 = \frac{1200}{1200/CN_1 + [ET_1 - (P_1 - Q_1)}$$

at time t_n

$$V_n = V_{n-1} + ETA_{n-1} (P_{n-1} - Q_{n-1}) = 1.2\, S_n \qquad (5.11)$$

Soil moisture storage by SCS method can be estimated by using equation (5.11) for any real value of P, Q and ETA.

Actual evapotranspiration can be estimated by the relationship:

$$ETA = \frac{ETP \times (\theta - WP)}{(FC - WP)} \qquad (5.12)$$

where

θ	=	Soil moisture (Volume ratio)
WP	=	Average wilting point (Volume ratio)
FC	=	Average field capacity (Volume ratio)
ETP	=	potential evapotranspiration

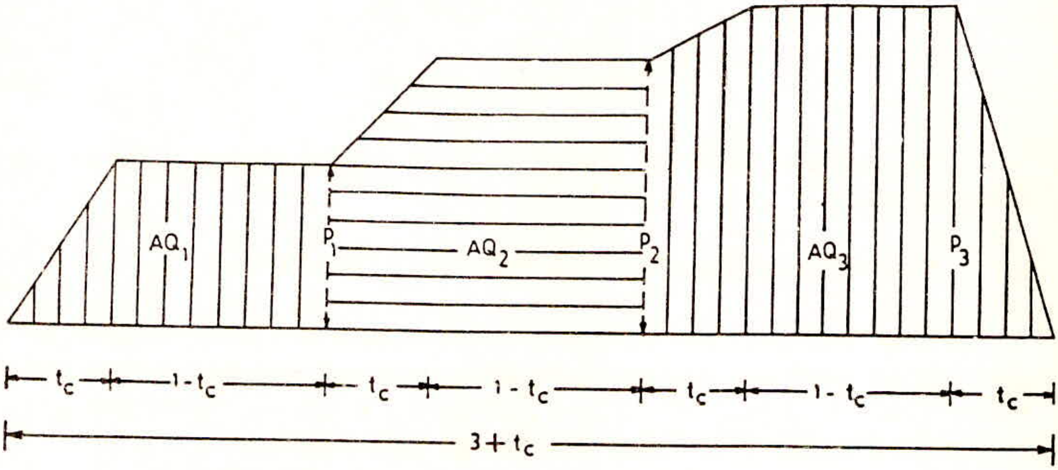
Here WP and FC are the properties of the soil and ETP depends on meteorological factors. However daily soil moisture shall vary depending upon the part of the precipitation retained by the catchment and the actual evapotranspiration occured during the day.

Once the curve numbers are updated, the daily surface runoff volumes can be known using the respective curve number making use of equation (5.6) and (5.5).

5.1.2.2 Estimation of Design Discharge

Let Q_1, Q_2, and Q_3 be the direct surface runoff volume on the first, second and the third day due to a storm that continues for three days. It may be noted that, according to Indian Standard, the surface drainage system is to be designed for a storm of three day duration. Let A be the area of the catchment and t_c is the time of concentration in fraction of a day. Let the rising limbs and falling limb of the hydrograph be straight lines. The peak discharge rates on different days can be estimated as given below. Refering to Fig. 5.3.

$$A\, Q_1 = 0.5_{p1}\, tc + (1\text{-}tc)_{p1}$$



Q_1 = Direct runoff on the first day

Q_2 = Direct runoff on the second day

Q_3 = Direct runoff on the third day

Fig.5.3 Runoff Hydrograph consequent to a 3-day storm

or	$p1$	=	$AQ1/(1-0.5tc)$	(5.13)
	AQ_2	=	$0.5tc(p1+p2) + (1-tc)p2$	
or	$p2$	=	$AQ_2/(1-0.5tc) - 0.5tcAQ1/(1-0.5tc)^2$	(5.14)
	AQ_3	=	$0.5tc(p2-p3) + (1-tc)p3$	
or	$p3$	=	$AQ3/(1-0.5tc) - 0.5tcAQ2/(1-0.5tc)^2$	
			$+ 0.25\ tc2AQ1/(1-0.5tc)^3$	(5.15)

The drainage channel should be designed for peak discharge rate p3 given in equation (5.15)

5.1.3 TIME-AREA METHOD OF DISCHARGE ESTIMATION

Time Area Method of estimation of discharge from agricultural lands for a given design storm can be considered as an extension and improvement of the rational method. In this method the variations in the intensity of effective rainfall can be considered and the complete discharge hydrograph can be obtained.

5.1.3.1 *Concept of Time-Area Method*

In time-area method, the catchment contribution to the runoff at outlet are sub-divided in time. The varying intensities within a storm are average over discrete periods according to the isocrone time internal selected.

An isocrone is a contour joining those points in the watershed that are havig the same travel time. The travel time of a point in the catchment is defined as the time difference between the occurrence of an element of effective rainfall at that point and the realisation of its effect at the outlet.

Determination of runoff discharge by time-area method basically involves following steps-

i) Determination of rainfall excess
ii) Construction of time area diagram
iii) Calculation of outflow hydrograph

i) Determination of rainfall excess

In deriving the peak discharge for design purposes a design storm with a critical sequence of intensities is selected. An effective rainfall rate is assessed by assuming a suitable runoff coefficient and applying this to rainfall element is each time unit or a constant loss rate is assumed and substracted from rainfall element in every time unit. Fig.(5.4) illustrates the determination of rainfall excess by the two methods.

ii) Relative travel time method

The fundamental assumption made here is that the travel time for any element of area is approximately proportional to $(L/\sqrt{S})$, where L and S are the length and slope of any reach of the flow path, respectively, and the summation is perfomred along the flow path from the point in question to the watershed outlet.

Furthermore, in this method, the abscissa is taken as relative travel time and hence is dimensionless. The relative travel time is the proportion of the maximum travel time, i.e. t/tmax. Therefore, the TA diagram can be first proposed without knowing the travel time for any point in the watershed. When the travel time is determined for ahy point, then the abscissa can be made dimensional. The mechanics of constructing TA diagram can be explained in the following steps:

Step 1 Draw the elevation contours on the topographic map of the watershed under study

Step 2 Mark a large number of points on the contours. Distribute these points uniformly over the watershed.

Step 3 Tabulate for each point the distances between adjacent contours along the flow path to the watershed outlet.

Step 4 Raise these individual distances to the power 3/2, as the travel time is assumed to be proportional to $L/\sqrt{S}$ or $L^{3/2}/E^{1/2}$, where E is the contour interval. Since E is constant, the travel time is proportional to $L^{3/2}$.

The watershed outlet may not be on a contour, and a correction would have to be made for the lowest channel segment. This correction amounts to multiplying its length by $(E/E_1)^{1/2}$ where E_1 is the elevation drop of this length.

Step 5 Sum the lengths raised to the 3/2 power for each point.

Step 6 Reduce the sums obtained in Step 5 proportionally to give a delay time of unity for the extreme upstream portion of the watershed.

Step 7 Mark the relative delay time obtained in Step 6 on the points in the map. Draw the isochrones of relative travel times.

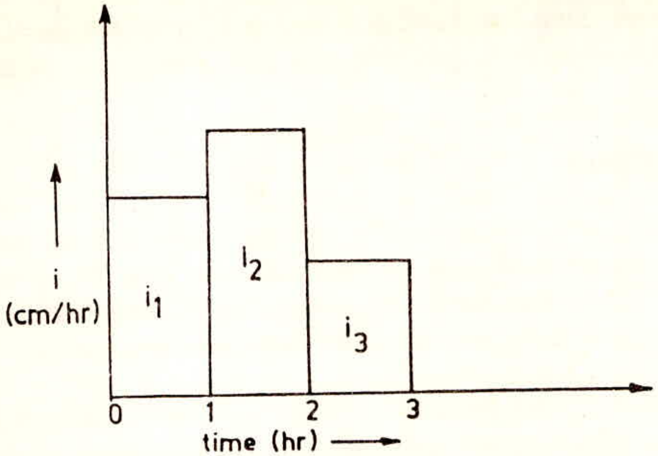


Fig. 5.4 (a) – TOTAL RAINFALL

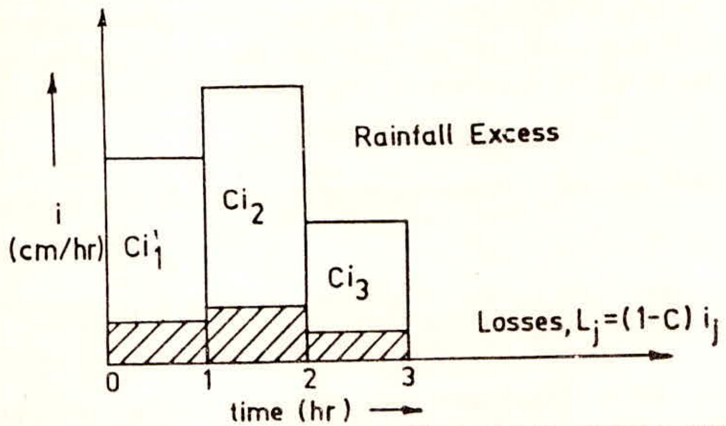


Fig. 5.4 (b) - DETERMINATION OF RAINFALL EXCESS BY ASSUMING A CONSTANT RUNOFF COEFFICIENT, C

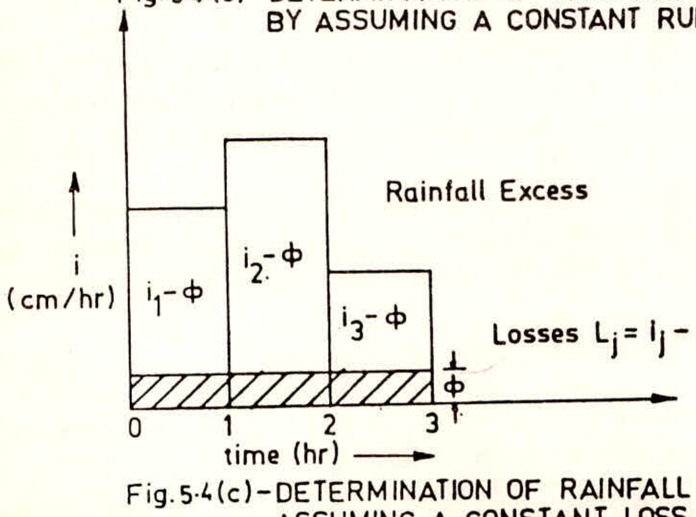


Fig. 5.4 (c) - DETERMINATION OF RAINFALL EXCESS BY ASSUMING A CONSTANT LOSS RATES ϕ

Step 8 Measure the area between the isocrones. This can be easily done by planimetering. These areas are denoted by a , i=1,2,..............,10 in Fig. 5.5.

Step 9 Find out the time of concentration for the watershed by using the formulae given in section 3. It will be nothing but tmax, the travel time of the most remote point on the watershed.

Step10Convert the relative travel time values of isocrones drawn in step 7 into absolute travel times by multiplying them with time of concentration obtained above.

Step11Plot the cumulative areas upto an isocrone against the corresponding values of absolute travel times as shown in Fig. 5.6. It will give on time-area concentration diagram.

Step12Choose a suitable time interval and find out with the help of the TAC diagram the increase in area in successive time intervals.

Step13 Plot the incremental area against the corresponding time interval to given a time area histogram as shown in Fig. 5.7.

iii) Average Velocity Method

The average velocity method was developed by Soil Conservation Service (1972) and is based on computing the length of flow and the average velocity of flow. The procedure of constructing TA or diagrams using this method is explained in the following steps:

Step 1 Mark a large number of points, preferably uniformly distributed along elevation contours, on the topographic map of the watershed.

Step 2 Measure the distance of flow from these points to the contributing channels. These are overland flow distances.

Step 3 Compute the average velocity of flow corresponding to each of these distances with the help of Fig. 5.8 given by S.C.S.

Step 4 Divide the distance of flow by the corresponding velocity and obtain the overland flow time.

Step 5 The channel flow time upto the outlet from a point channel can be computed by using an open channel flow formula. If Manning's formula is used the values of roughness coefficient (n) given in Table 5.3 can be used.

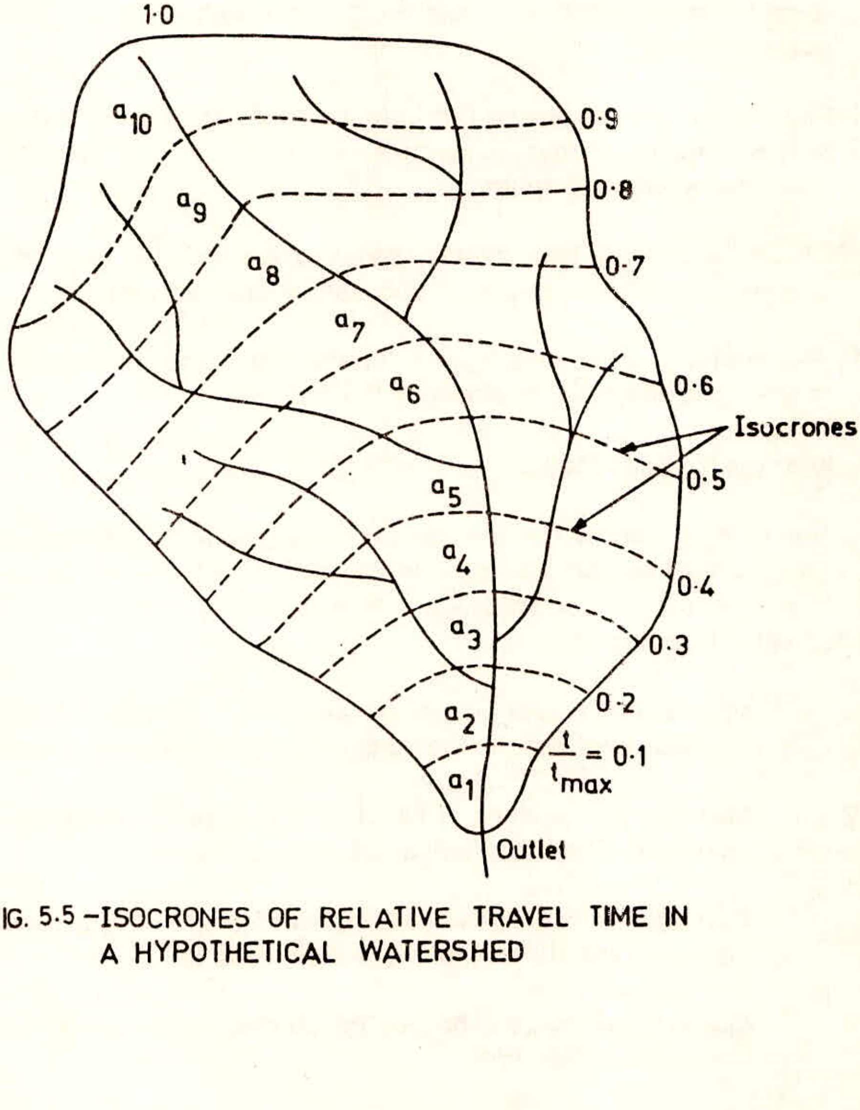


FIG. 5.5 —ISOCRONES OF RELATIVE TRAVEL TIME IN A HYPOTHETICAL WATERSHED

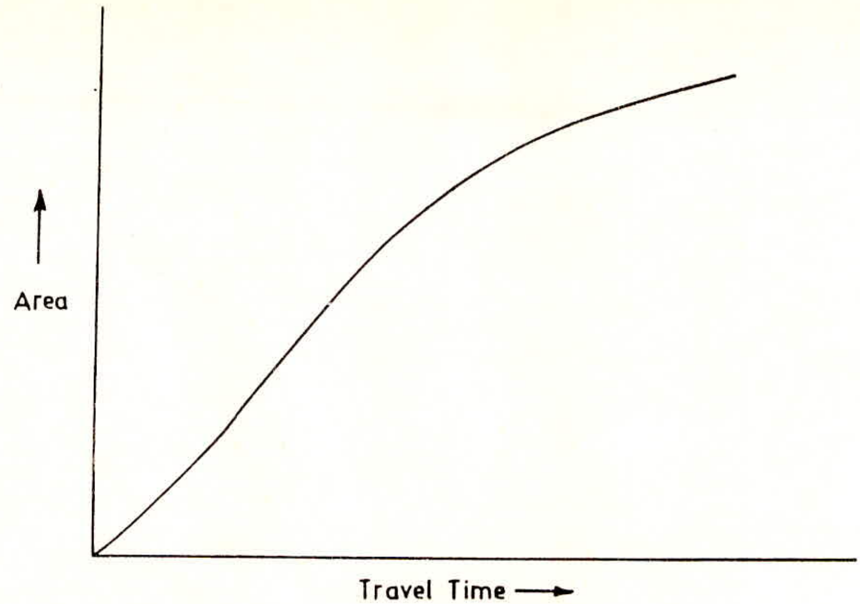


Fig. 5.6 – TIME AREA CONCENTRATION DIAGRAM

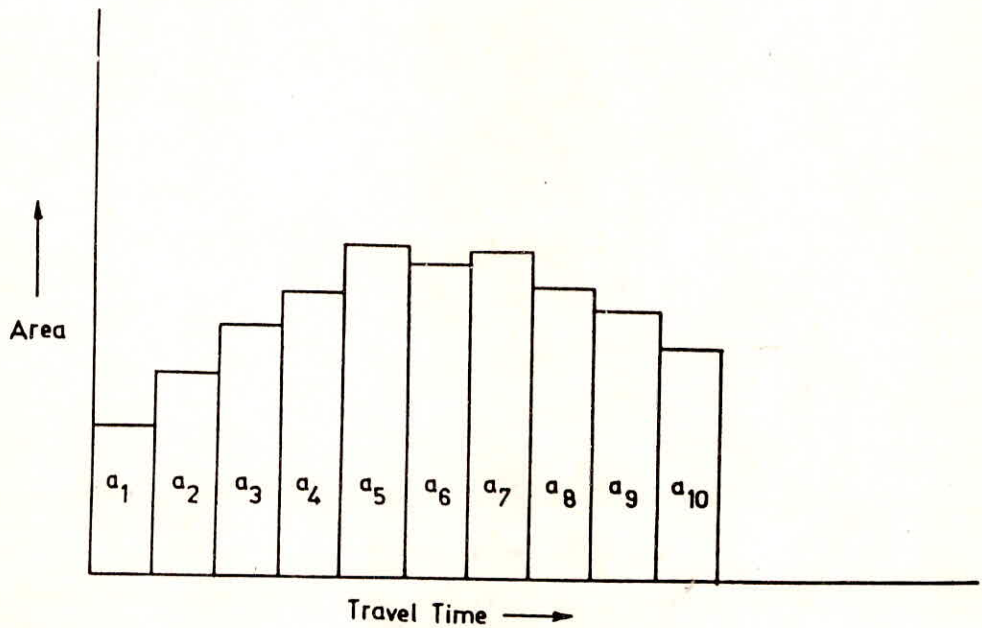


Fig. 5.7 – TIME AREA HISTOGRAM

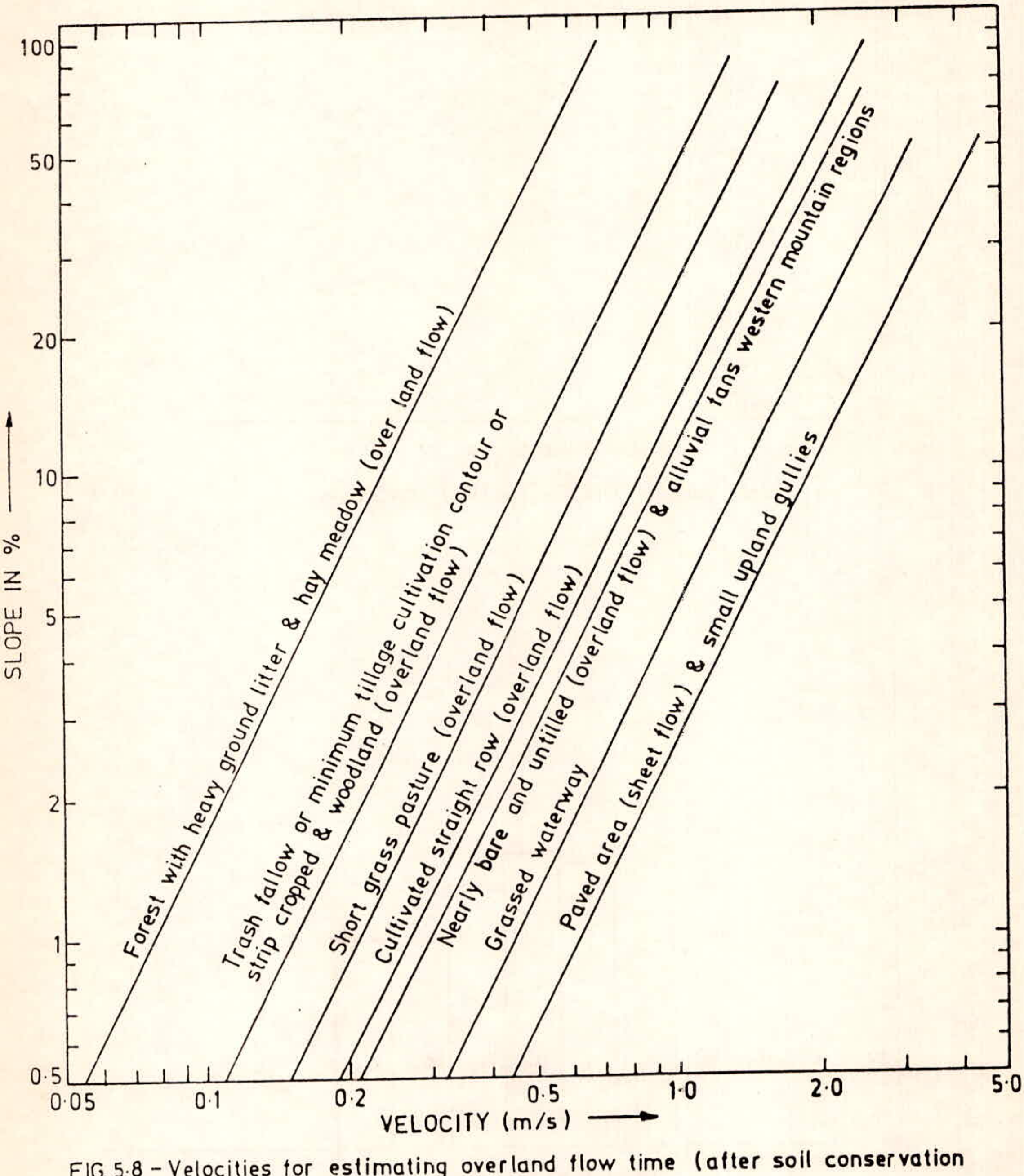


FIG. 5·8 – Velocities for estimating overland flow time (after soil conservation service 1972); converted to metric system.

Table 5.3 : Mannings roughness Coefficients for various Boundaries

S.N.	Boundary	Mannings Coeef. n
1.	Very smooth surface such as glass,plastic or brass	0.010
2.	Very smooth concrete and planed timber	0.011
3.	Smooth concrete	0.012
4.	Ordinary concrete lining	0.013
5.	Earth channels in best conditions	0.017
6.	Straight unlined eatrh canals in good condition	0.020
7.	Rivers and earth canals in fair conditions - some moss growth	0.025
8.	Winding natural streams and canals in poor condition - considerable moss growth	0.035
9.	Mountain streams with rocky beds and rivers with variable sections and some vegetation along banks	0.040 to 0.050

Step 6 The overland flow time and the channel flow time are added to give the total travel time for a point on the watershed.

Step 7 After knowing the travel times of various points on the catchment the isocrones may be interpolated.

The remaining steps to draw a time-area histogram and a time area diagram are the same as described earlier.

The overland flow time is normally a significant portion of the total travel time in smaller watersheds but a much smaller portion in larger watersheds. The computation of overland flow time should be restricted to watersheds smaller than 2000 acres and to subwatershed portions of larger areas above and beyond the point where hydraulic measurements are impractical to make.

5.1.3.2 Calculation of outflow Hydrograph

Once the excess rainfall hyetograph of the design storm and the time-area diagram of the catchment are obtained, the outflow hydrograph ordinates can be calculated from the principle that

the runoff at any time 't' is equal to the area enclosed by the 1 hr. isocrone multiplied by the mean intensity of effective rainfall of the hour previous to time, t plus the additional area enclosed by the 2 hour isocrone multiplied by the mean intensity of effective rainfall during the period from 1 hour to 2 hour before time 't' and so on.

Mathematically the runoff hydrograph can be expressed as

$$Q_t = \Sigma a_k I_{t-k+1} \qquad (5.16)$$

where a_i is the area enclosed by the i^{th} and the $(i-1)^{th}$ isocrone and I_i is the intensity of effective rainfall in the i^{th} interval. Equation (5.16) is nothing but a discrete convolution of I and a.

Care has to be taken in the method to keep the time interval of excess rainfall hyetograph the same as taken in the time-area histogram.

5.1.3.3 Assumption and Limitations

The fundamental assumption in the TA methods is one of translation. They allow for the delay experienced by water in reaching the watershed outlet. They also allow for spatially nonuniform effective rainfall. However, they do not allow for the storage effects, which are primarily responsible for producing attenuation in peak flow and for increasing the time base of the hydrograph. It is therefore not surprising that the TA methods tend to overestimate the peak discharge. These methods may yield acceptable results for small watersheds, where storage effects are minimal.

5.2 Sub Surface Drainage

Sub surface drainage problems can be solved either by horizontal sub surface drains or by pumping ground water by Tubewells. The design of Subsurface Drainage is mainly depends on the Geohydrological condition of the area, crops cultivated in the command area and the drainage coefficient. Tube well drainage, under some conditions is an effective method of lowering a high watertable and reducing salinity hazard in an irrigated area. Before a tube well drainage scheme is installed, careful engineering investigations should be carried out to evaluate the feasibility of drainage. Of particular importance is to ascertain the interconnection of the underground water in the upper layers to that of the pumped aquifer.

The following conditions contribute to the feasibility of a tube well drainage scheme:

i) Large areas of flat lands with extensive high water table with or without salinity problem.

ii) Well defined continuous, thick phreatic aquifers with a good hydraulic conductivity.

iii) Ground water under artesian pressure.

iv) Ground water quality good enough to be used for irrigation with or without mixing with the fresh surface waters.

v) When pipe drainage is feasible but difficult or costly because of inadequate gravity outlets or unstable soils which do not permit digging of deep trenches.

vi) When ground water lowering beyond 2.5 m is desired and a shallow impervious layer does not exit.

vii) Power available at reasonable cost.

5.2.1. Description of the Study Area

The procedure of designing a sub-surface drainage system is illustrated below with respect to a plot of land on Indira Gandhi Nahar Pariyoja. The Indira Gandhi Nahar Pariyojna with a command area of 1.543 million hectare is the largest irrigation and drinking water project in the north western Rajasthan. The main canal gets water from the river Sutlej in Punjab through a feeder canal which takes off from Hari Ka Barrage. The project has been taken up in two stages (Fig.5.9), the first stage has already been completed and the second stage is under execution.

The Stage II area of IGNP starts from Pugal and comprises main canal from 620 RD to 1458 RD. It has been envisaged to irrigate seven flow command and five lift commands.

The study area selected for the present subsurface drainage study lies near RD 838 along the main canal, at a distance of about 60 km from Bikaner. The study area is located near Bajju town and has an area of about 60 hectares.

The climate of the region is arid with an average annual rainfall of about 200-250 mm. The temperature ranges from freezing point in winters to above 50^{o} C in summers.

The area covered by IGNP is comprised of sandy undulating plains with various types of low to medium sand dunes. The top aeolian soils have permeability but underlying sediments, comprising of sand silty clay and kankar, have low permeability.

Prior to the introduction of the canal irrigation, only rainfed agriculture was being practised with Moth and Bajra as the major crops. But the introduction of the canal irrigation has changed the agricultural practices. Now Ground nut, Gram, Mustard and Guar are the main crops grown in the area.

5.2.2 DESIGN OF FIELD DRAINS

The design of subsurface drains is governed by the hydrogeological condition of the area, drainage coefficient and the types of crops grown in the command area. A case study has been taken in the command area of Indira Gandhi Nahar Pariyojna. The various investigations needed for design of subsurface drainage system are given below:

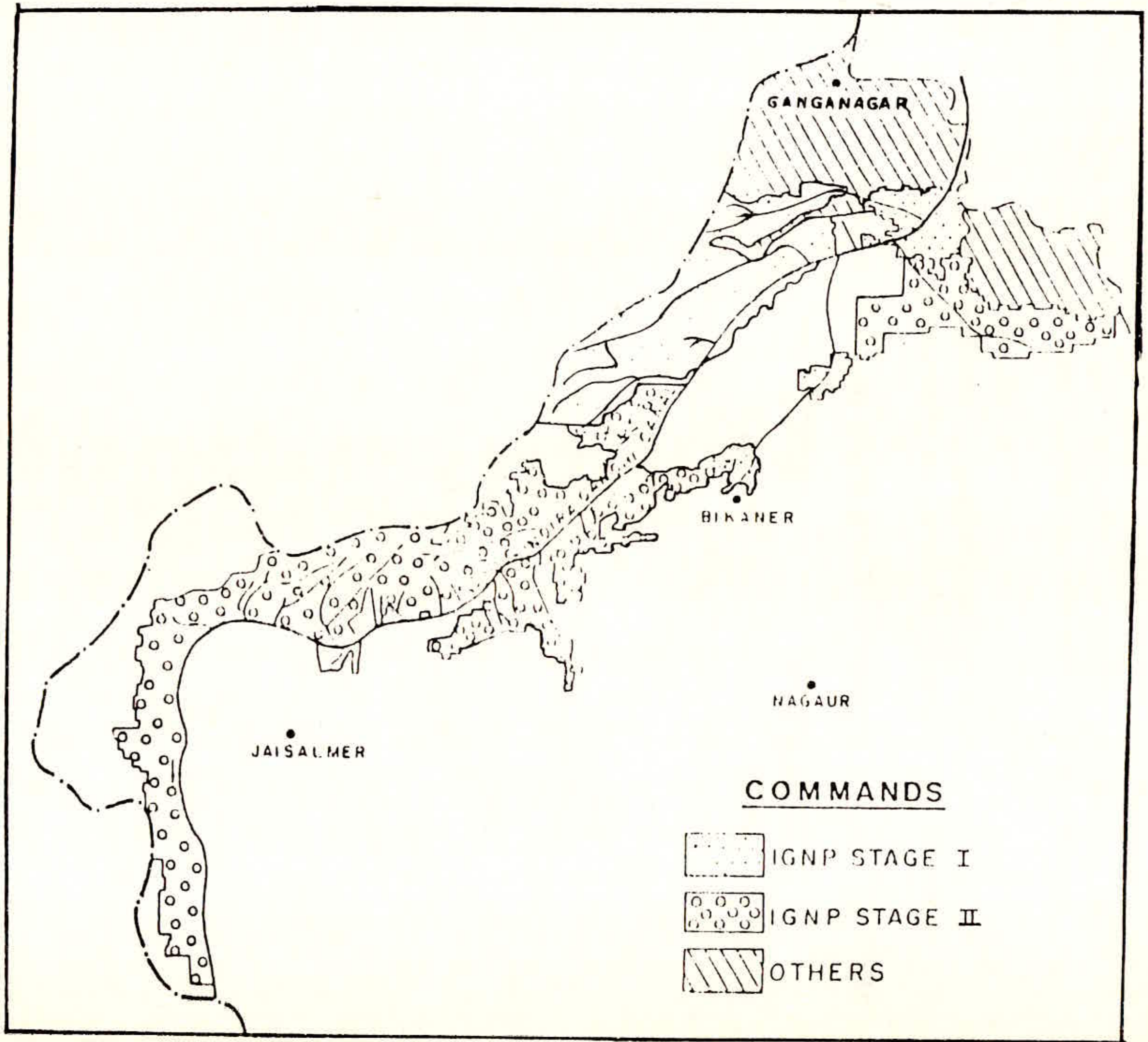


FIG. 5.9 COMMAND AREA OF INGP AND ADJOINING SYSTEMS

5.2.2.1 Hydrogeological Investigation

The lithologs of the study area which were supplied by Ground Water Division, CAD were used to identify the first and second layer in the study area.

The hydraulic conductivities of the top layer have been determined at several locations using Guelph permeameter. These values are given in Table 5.4.

The conductivity of the second layer is determined at two locations by piezometric method. The details of the test are given below:

TABLE 5.4 : Values of Hydraulic Conductivity (K_S) of First Layer

ORIGINAL SITE CODE	GUELPH'S VALUES
A_1(1)	0.9912
A_2(2)	0.2541
G(3)	0.81
B_2(4)	0.3034
B_3(5)	0.1169
B_6(6)	2.909
B_7(7)	0.697
G(8)	0.35
B_5(9)	0.615
B_4(10)	0.1745
G(11)	0.11
A_4(12)	0.00766
A_6(14)	0.0959
B_8(15)	1.5039
A_7(16)	0.4231
G(17)	0.025
A_8(18)	1.0223
B_9(19)	1.12299
B_{12A}(20)	1.6975
B_{12B}(20)	-
G(21)	1.71
A_{10}(22)	0.0416
A_9(23)	0.2703

G(24)	0.098
B_{10}(25)	0.362
B_{11}(26)	0.7053
G(27)	2.79
A_{11}(28)	0.234
A_3	-
A_5	-

Hydraulic Conductivity of Second Layer by Piezometric Method

The piezometric method is used to measure the insitu hydraulic conductivity of the second layer. This test measures the hydraulic conductivity of individual soil layers below the water table. In this method, a hole is augured into the soil to the depth below the water table at which one wishes to measure the hydraulic conductivity. A pipe is inserted into the hole, having a small unprotected cavity at the bottom. After the water level in the pipe has reached equilibrium with the groundwater, the water in the pipe is removed. Due to pressure difference, water will flow from the surrounding soil into the uncased cavity, causing the water level in the pipe to rise. The rate of rise is measured and the K value is calculated by a formula which describes the relation between the rate of rise, the flow condition and the K value of the soil. The formula used is:

$$K = \frac{3600\,\pi\,(D/2)^2 \log_e(Y_1/Y_2)}{A\,(t_2 - t_1)} \qquad (5.17)$$

where,

K	=	hydraulic conductivity (inches per hour)
Y_1 and Y_2	=	distance from static water level to water level at time t_1 and t_2 (inches)
D	=	diameter of casing (inches)
$t_2 - t_1$	=	time for water level to change from Y_1 to Y_2 (Sec)
A	=	a constant for a given flow geometry (inches)

The constant A is taken from the curve drawn by Luthin & Kirkham, in which A is a function of D and length of cavity.

The hydraulic conductivity of the second layer was measured at two locations. For this study, the following values are used.

D = 4 inches

Length of Cavity = 4 inch

For above values, the value of constant A from Luthin & Kirkham comes as 27.

SITE 1

Y_1 = 71.26 inch
Y_2 = 70.86 inch

$t_1 - t_2$ = 390 sec

The computed value of K is 0.0145 m/day

SITE 2

Y_1 = 70.86 inch

Y_2 = 70.47 inch

$t_1 - t_2$ = 195 sec

The computed value of K is 0.0288 m/day

Specific yield of the first layer has been determined using Johnson graph. As the textural class of the first layer is sand, the specific yield has been taken as 0.3.

5.2.2.2 Determination of Drainage Coefficient

The drainage coefficient for which the field drains are designed is obtained from a water balance of the study area. The study area is bound by Indira Gandhi main canal on one side. Since an intercepting drain has been proposed parallel to the canal, the seepage from the canal will not enter into the study area. Therefore, inflow to the study area will be recharge from rainfall and irrigation return flow. The outflow is comprised of evaporation loses from high water table and subsurface outflow from three sides of the study area. The monthly irrigation return flow is assumed to be proportional to the monthly irrigation water depth or delta which is given in Table 5.5.

Table 5.5: Monthly Delta (mm) assumed for the Study Area (Yearly Delta = 0.41233 m)

June	July	Aug.	Sept.	Oct.	Nov.
36.21	27.0	46.71	54.42	53.57	32.57

Dec.	Jan.	Feb.	March	April	May
42.85	43.71	28.28	24.42	8.14	14.45

a) Recharge from Rainfall

The average values of the rainfall and evaporation recorded at Bikaner and Jaisalmer have been assumed to represent the rainfall and evaporation of the study area. The meteorological data recorded at Bikaner and Jaisalmer and data adopted for the study area are presented in Table 5.6.

Table 5.6: Rainfall and Potential Evaporation Data for Bikaner and Jaisalmer Area

	RAINFALL (mm)			POTENTIAL EVAPORATION(mm)		
MONTH	BIKANER	JAISALMER	STUDY AREA	BIKANER	JAISALMER	STUDY AREA
JAN.	3.57	2.35	2.96	53.30	70.60	61.95
FEB.	8.96	4.65	6.81	75.30	91.60	83.45
MAR.	8.18	3.06	5.62	131.40	153.20	142.30
APR.	2.46	2.73	2.60	172.40	203.50	187.95
MAY	15.01	7.04	11.02	236.80	281.00	258.90
JUNE	27.39	15.32	21.35	258.30	317.40	287.85
JULY	70.38	56.72	63.55	228.20	247.60	237.90
AUG.	63.72	67.71	65.71	196.60	210.70	203.65
SEPT	54.61	21.93	38.27	177.70	192.00	184.85
OCT.	2.99	1.26	2.13	124.70	147.00	135.85
NOV.	2.80	1.82	2.31	68.30	83.70	76.00
DEC.	3.92	1.88	2.90	48.80	64.10	56.45

The percentage of rainfall recharge in Sikar basin comprised of quaternary aeolian sand is reported to be 8 percent (UNDP, 1976). The recharge from rainfall in area west of Barmer-Jhunjunu axis is of the order of 5% (Gupta and Prakash, 1980). In the present study, the recharge from rainfall has been assumed to be 5 percent of the rainfall which occurs during monsoon months (June to September).

b) Evaporation Losses from Water Table

Kumar (1992) studied steady state evaporation rate from bare soils under high water table conditions by using a finite difference numerical scheme for solution of the one dimensional Richards equation. Functional relation, as reported by Haverkamp et.al. (1977), were used for characterizing the hydraulic properties of a sandy soil. For different values of depth to water table (45 cm to 210 cm), Kumar has estimated the evaporation rates.

The actual steady evaporation rate is determined by the external evaporativity and by the water transmitting properties of the soil. Where the water table is near the surface the evaporation rate is determined by the external conditions. When the water table is deeper, the evaporation rate approaches a limiting value regardless of how high external evaporativity may be. With the water table at large depth, the evaporative flux decreases markedly because the low hydraulic conductivity of the unsaturated soil becomes the limiting factor.

Arid climate is generally favourable to high evaporation rates as solar radiation values are very high and humidity of the air is low. A notable exception is the desert of Rajasthan, where the high dust content of the atmosphere reduces the intensity of solar radiation and the relative humidity is comparatively higher. The light textured soils (like loss of western Thar Desert) have the property of conserving moisture in face of strong atmospheric moisture deficit. The phenomenon of reduced soil moisture evaporation and moisture conservation has been widely observed in the wind blown sands of western Thar desert region in IGNP, Stage-II area. The dune and depression sands have been found moist after removing the top crusted or dry sand mulched layer of about 15 to 30 cms (Bhanwar Dan Bithu, 1993).

Considering the mulching action of soils of Thar desert, for the present study it has been assumed that potential evaporation occurs when water table is at a depth equal to or less than 30 cm below ground surface. Evaporation loss was assumed to be zero if water table lies at a depth more than or equal to 1m below ground surface.

Evaporation losses from water table are assumed to occur at average potential rates for depth to water table range 0-30cm. Kumar (1992) observed that evaporation losses from water table become negligible for depth to water table greater than 100cm in case of a sandy soil. The actual variation of evaporation losses from water table with depth to water table as considered in the lumped ground water balance is given in Table 5.7.

Table 5.7: Evaporation Loss from Water Table

Depth to Water Table (m)	Evaporation losses from water table as percentage of potential evaporation
0.30	100.00
0.45	24.84
0.65	6.46
0.85	2.26
1.00	0.00

c) Lumped Groundwater Balance

A lumped monthly ground water balance has been carried out to find the evolution of water table in the irrigated areas. The water balance equation for a balance period Δt is

$$\sum I_i = \sum O_i + \Delta S \qquad (5.18)$$

in which I_i is the i^{th} inflow to the flow domain during Δt, O_i is the i^{th} outflow from the flow domain during Δt and ΔS is the change in storage that occurs in time interval Δt. The inflows include recharge due to rainfall, and irrigation return flow. The outflows include flow to the three sides of the study area, and evaporation losses.

Assuming that evaporation losses from water table get activated when the water table rises upto a level of 1.0m below the ground surface, the water balance has been carried out. The average hydraulic conductivities and thickness of the first, second, and third layer have been considered for estimating the subsurface outflow. The average storativity of the first layer has been considered while computing rise of water level. Based on the field observation the initial water level found to be at a depth of 2m below ground surface. The gradient of the water table has been assumed to be 0.005 for computing subsurface outflow. The recharge due to rainfall is taken as 5% of the rainfall which occurs during the monsoon months i.e. from June to September. The recharge from irrigation fields has been assumed to be 30% of the water applied at field. The water balance has been made for irrigation application of 3.0 cusecs for 1000 acres. For the purpose of water balance, June has been taken as the starting month.

The following data have been used in the water balance:

(i) Average thickness of the first layer = 2.6m

(ii) Average hydraulic conductivity of the first layer = 0.6 m/day,

(iii) Average thickness of the second layer = 3.4m

(iv) Average hydraulic conductivity of the second layer = .02 m/day,

(v) Average thickness of the third layer = 12.0 m

(vi) Average hydraulic conductivity of the third layer = 0.4m/day,

(vii) Average storativity of the first layer = 0.3

(viii) Irrigated areas = 90 hectares,

(ix) Area for the water balance = 130 hectares,

(x) Width of outflow = 3.48 km,

(xi) Hydraulic gradient for outflow computations = 0.005,

(xii) Delta = 0.41233m per year,

(xiii) Irrigation return flow = 30% of the water applied

(xiv) Time step size = 1 month

The evalution of water table is given in Fig. 5.10. Without drainage water table will stabilise at a depth of 0.7 m. Therefore, provision of subsurface drainage is necessary.

The spacing of the field drains has been determined so that the minimum depth to water table ground surface is equal to 1m.

d) *Spacing of Field Drains*

The conductivity of the second layer is less than one tenth of the conductivity of the first layer. Therefore the second layer can be considered as

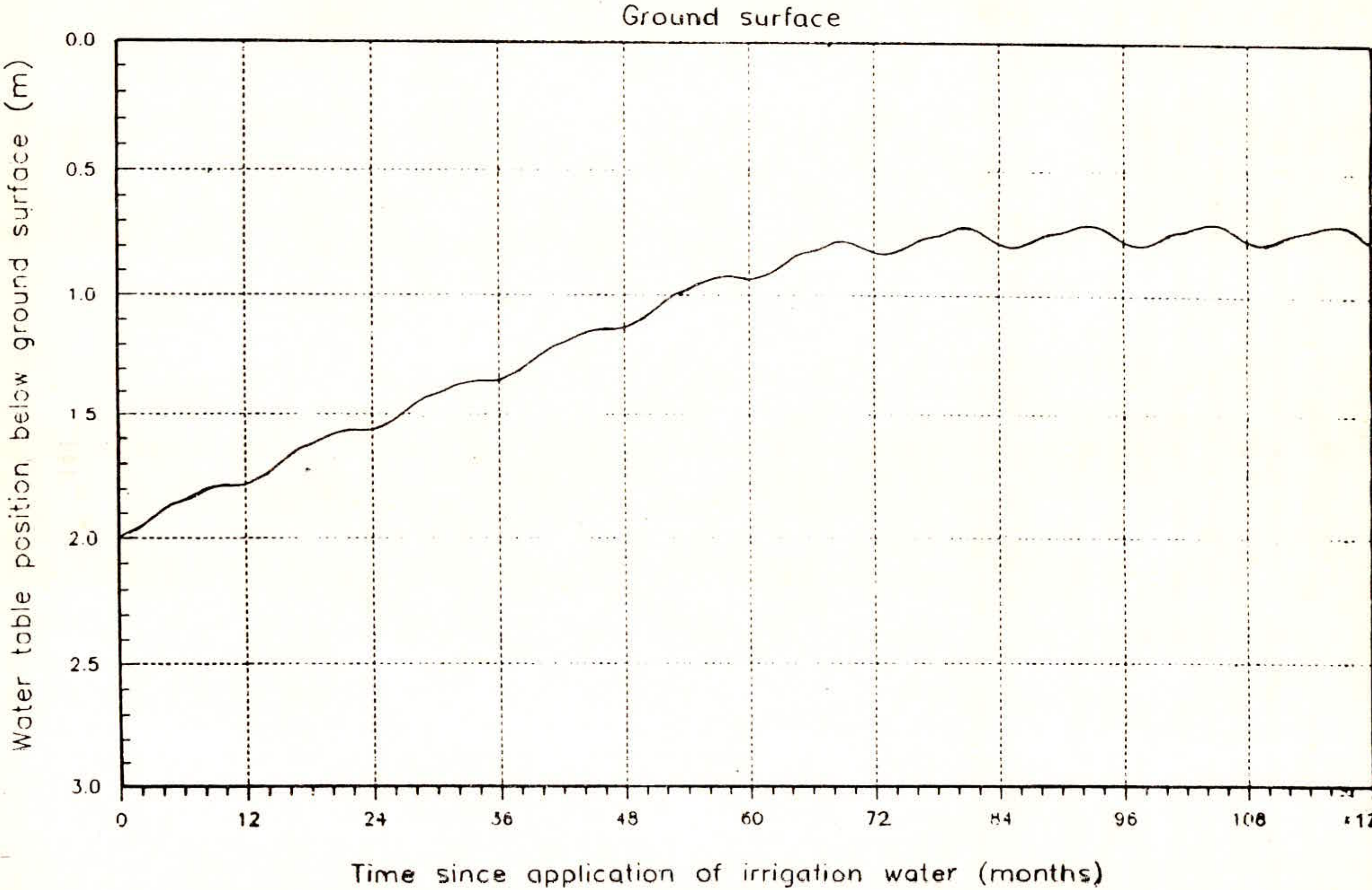


Fig. 5.10 : Water Table Evalution at RD838 (with provision of intercepting drain)

impervious for the purpose of finding the drain spacing. Hooghoudt equation has been used for finding the spacing.

In the present study, the steady state drainage criteria is used for determination of drain spacing. The drainage coefficient was found from the trend of water table rise given in Table 5.8 to 5.13. If water table is maintained at a depth of one metre from ground surface the maximum possible water table rise in a month will be 0.035m. Therefore, the drainage coefficient for which the drain should be designed is (0.035/30.) multiplied by porosity of the first layer. The drainage coefficient works out to be 0.00035m. The drains have been designed for a drainage coefficient of 0.00035m per day.

Field drains are to be provided in an area of 42 ha only. The remaining area of 48 ha is covered by intercepting drains which are spaced at a distance of 100 m apart.

TABLE 5.8: MONTHLY GROUND WATER BALANCE FOR THE FIRST AND SECOND YEAR IN THE CANAL COMMAND NEAR RD838 ALL VOLUMETRIC UNITS ARE IN CUBIC METRE

MONTH AFTER IRRIGATION	RAINFALL RECHARGE	IRRIGATION RETURN FLOW	EVAPORATION LOSSES FROM WATER TABLE	OUTFLOW	CHANGE IN STORAGE	RISE (M)	DEPTH TO WATER TABLE BELOW GROUND SURFACE (m)
1.	1387.8	9483.4	.0	2985.4	7885.7	.020	1.980
2.	4130.8	7071.3	.0	2989.2	8212.8	.021	1.959
3.	4271.1	12233.4	.0	2989.4	13515.1	.035	1.924
4.	2487.6	14252.6	.0	2991.9	13748.0	.028	1.889
5.	.0	14030.0	.0	2992.0	11038.0	.028	1.861
6.	.0	8530.1	.0	2990.7	5539.4	.014	1.846
7.	.0	11222.4	.0	2988.1	8234.3	.021	1.825
8.	.0	11447.6	.0	2989.4	8458.3	.022	1.804
9.	.0	7406.5	.0	2989.5	4417.0	.011	1.792
10.	.0	6395.6	.0	2987.6	3408.0	.009	1.783
11.	.0	2131.9	.0	2987.1	-855.2	-.002	1.786
12.	.0	3784.5	.0	2985.0	799.4	.002	1.784
13.	1387.8	9483.4	.0	2985.8	7885.3	.020	1.763
14.	4130.8	7071.3	.0	2989.2	8212.8	.021	1.742
15.	4271.1	12233.4	.0	2989.4	13515.1	.035	1.708
16.	2487.6	14252.6	.0	2991.9	13748.2	.035	1.762
17.	.0	14030.0	.0	2992.0	11038.0	.028	1.644
18.	.0	8530.1	.0	2990.7	5539.4	.014	1.630
19.	.0	11222.4	.0	2988.1	8234.3	.021	1.609
20.	.0	11447.6	.0	2989.4	8458.3	.022	1.587
21.	.0	7406.5	.0	2989.5	4417.0	.011	1.576
22.	.0	6395.6	.0	2987.6	3408.0	.009	1.567
23.	.0	2131.9	.0	2987.1	-855.2	-.002	1.569
24.	.0	3784.5	.0	2985.0	799.4	.002	1.567

TABLE 5.9: MONTHLY GROUND WATER BALANCE FOR THE THIRD AND FOURTH YEAR IN THE CANAL COMMAND NEAR RD838 ALL VOLUMETRIC UNITS ARE IN CUBIC METRE

MONTH AFTER IRRIGA-TION	RAINFALL RECHARGE	IRRIGATION RETURN FLOW	EVAPORA-TION LOSSES FROM WATER TABLE	OUTFLOW	CHANGE IN STO-RAGE	RISE (M)	DEPTH TO WATER TABLE BELOW GROUND SURFACE (m)
25.	1387.8	9483.4	.0	2985.8	7885.3	.020	1.547
26.	4130.8	7071.3	.0	2989.2	8212.8	.021	1.526
27.	4271.1	12233.4	.0	2989.4	13515.1	.035	1.491
28.	2487.6	14252.6	.0	2991.9	13748.2	.035	1.456
29.	.0	14030.0	.0	2992.0	11038.0	.028	1.428
30.	.0	8530.1	.0	2990.7	5539.4	.014	1.413
31.	.0	11222.4	.0	2988.1	8234.3	.022	1.392
32.	.0	11447.6	.0	2989.4	8458.3	.022	1.371
33.	.0	7406.5	.0	2989.5	4417.0	.011	1.359
34.	.0	6395.6	.0	2987.6	3408.0	.009	1.351
35.	.0	2131.9	.0	2987.1	-855.2	.002	1.353
36.	.0	3784.5	.0	2985.0	799.4	.002	1.351
37.	1387.8	9483.4	.0	2985.8	7885.3	.020	1.331
38.	4130.8	7071.3	.0	2989.2	8212.8	.021	1.309
39.	4271.1	12233.4	.0	2989.4	13515.1	.035	1.275
40.	2487.6	14252.6	.0	2991.9	13748.2	.035	1.240
41.	.0	14030.0	.0	2992.0	11038.0	.028	1.211
42.	.0	8530.1	.0	2990.7	5539.4	.014	1.197
43.	.0	11222.4	.0	2988.1	8234.3	.021	1.176
44.	.0	11447.6	.0	2989.4	8458.3	.022	1.154
45.	.0	7406.5	.0	2989.5	4417.0	.011	1.143
46.	.0	6395.6	.0	2987.6	3408.0	.009	1.134
47.	.0	2131.9	.0	2987.1	-855.2	-.002	1.136
48.	.0	3784.5	.0	2985.0	799.4	.002	1.134

TABLE 5.10: MONTHLY GROUND WATER BALANCE FOR THE FIFTH AND SIXTH YEAR IN THE CANAL COMMAND NEAR RD838 ALL VOLUMETRIC UNITS ARE IN CUBIC METRE

MONTH AFTER IRRIGATION	RAINFALL RECHARGE	IRRIGATION RETURN FLOW	EVAPORATION LOSSES FROM WATER TABLE	OUTFLOW	CHANGE IN STORAGE	RISE (M)	DEPTH TO WATER TABLE BELOW GROUND SURFACE (m)
49.	1387.8	9483.4	.0	2985.8	7885.3	.020	1.134
50.	4130.8	7071.3	.0	2989.2	8212.8	.021	1.093
51.	4271.1	12233.4	.0	2989.4	13515.1	.035	1.058
52.	2487.6	14252.6	.0	2991.9	13748.2	.035	1.023
53.	0	14030.0	.0	2992.0	11038.0	.035	0.995
54.	.0	8530.1	.0	2990.7	5539.4	.014	0.981
55.	0	11222.4	43.4	2988.1	8190.9	.021	0.960
56.	.0	11447.6	178.9	2989.4	8279.4	.021	0.938
57.	.0	7406.5	502.4	2989.4	3914.7	.010	0.928
58.	.0	6395.6	1307.4	2987.3	2100.9	.005	0.923
59.	.0	2131.9	2008.3	2986.5	-2862.9	-.007	0.930
60.	.0	3784.5	2974.5	2984.1	-2174.1	-.006	0.936
61.	1387.8	9483.4	2991.8	2984.4	4894.9	.013	0.923
62.	4130.8	7071.3	2274.8	2987.8	5939.5	.015	0.908
63.	4271.1	12233.4	2328.6	2988.3	11187.6	.029	0.879
64.	2487.6	14252.6	2533.6	2990.8	11215.7	.029	0.851
65.	.0	14030.0	2443.4	2990.8	8595.8	.022	0.829
66.	.0	8530.1	1693.0	2989.6	3847.5	.010	0.819
67.	.0	11222.4	1510.6	2987.3	6724.5	.017	0.802
68.	.0	11447.6	1784.6	2988.7	6674.4	.017	0.784
69.	.0	7406.5	2702.5	2988.6	1715.4	.004	0.780
70.	.0	6395.6	5113.7	2986.3	-1704.3	-.004	0.784
71.	.0	2131.9	6925.7	2984.6	-7778.4	-.020	0.804
72.	.0	3784.5	9305.3	2981.7	-8502.6	-.022	0.826

TABLE 5.11: MONTHLY GROUND WATER BALANCE FOR THE SEVENTH AND EIGHTH YEAR IN THE CANAL COMMAND NEAR RD838 ALL VOLUMETRIC UNITS ARE IN CUBIC METRE

MONTH AFTER IRRIGATION	RAINFALL RECHARGE	IRRIGATION RETURN FLOW	EVAPORATION LOSSES FROM WATER TABLE	OUTFLOW	CHANGE IN STORAGE	RISE (M)	DEPTH TO WATER TABLE BELOW GROUND SURFACE (m)
73.	1387.8	9483.4	9154.6	2981.4	-1264.9	-.003	0.829
74.	4130.8	7071.3	6490.0	2984.8	1727.2	.004	0.825
75.	4271.1	12233.4	5418.6	2986.3	8099.6	.004	0.804
76.	2487.6	14252.6	5088.3	2989.3	8662.6	.022	0.782
77.	.0	14030.0	4324.9	2989.6	6715.5	.017	0.765
78.	.0	8530.1	2769.7	2988.7	2771.7	:007	0.765
79.	.0	11222.4	2258.9	2986.8	5976.7	.015	0.742
80.	.0	11447.6	2570.4	2988.3	5889.0	.015	0.727
81.	.0	7406.5	3727.8	2988.3	690.5	.002	0.725
82.	.0	6395.6	6802.5	2985.8	-3392.7	-.009	0.734
83.	.0	2131.9	9053.8	2983.8	-9905.7	-.025	0.760
84.	.0	3784.5	12004.2	2980.7	-11200.5	-.029	0.788
85.	1387.8	9483.4	11829.5	2980.1	-3938.5	-.010	0.798
86.	4130.8	7071.3	8359.2	2983.6	-140.7	.000	0.799
87.	4271.1	12233.4	6729.0	2985.4	6790.1	.017	0.781
88.	2487.6	14252.6	6094.0	2988.7	7657.4	.020	0.762
89.	.0	14030.0	4969.3	2989.1	6071.5	.016	0.746
90.	.0	8530.1	3089.7	2988.4	2452.1	.006	0.740
91.	.0	11222.4	2477.2	2986.6	5758.6	.015	0.725
92.	.0	11447.6	2799.4	2988.2	5660.1	.015	0.711
93.	.0	7406.5	4026.6	2988.2	391.8	.001	0.710
94.	.0	6395.6	7294.6	2985.6	-3884.7	-.010	0.720
95.	.0	2131.9	9673.9	2983.6	-10525.7	-.027	0.746
96.	.0	3784.5	12790.8	2980.4	-11986.7	-.031	0.777

TABLE 5.12: MONTHLY GROUND WATER BALANCE FOR THE NINTH AND TENTH YEAR IN THE CANAL COMMAND NEAR RD838 ALL VOLUMETRIC UNITS ARE IN CUBIC METRE

MONTH AFTER IRRIGATION	RAINFALL RECHARGE	IRRIGATION RETURN FLOW	EVAPORATION LOSSES FROM WATER TABLE	OUTFLOW	CHANGE IN STORAGE	RISE (M)	DEPTH TO WATER TABLE BELOW GROUND SURFACE (m)
97.	1387.8	9483.4	12609.2	2979.7	-4717.7	-.012	0.789
98.	4130.8	7071.3	8904.1	2983.2	-685.2	-.002	0.791
99.	4271.1	12233.4	7111.0	2985.1	6408.3	.016	0.775
100.	2487.6	14252.6	6387.2	2988.5	7364.4	.019	0.756
101.	.0	14030.0	5157.2	2989.0	5883.8	.015	0.741
102.	.0	8530.1	3182.9	2988.3	2358.9	.005	0.735
103.	.0	11222.4	2540.9	2986.6	5695.0	.015	0.720
104.	.0	11447.6	2866.2	2988.2	5593.3	.014	0.706
105.	.0	7406.5	4113.7	2988.1	304.7	.001	0.705
106.	.0	6395.6	7438.2	2985.6	-4028.2	-.010	0.715
107.	.0	2131.9	9854.8	2983.5	-10706.5	-.027	0.743
108.	.0	3784.5	13020.2	2980.3	-12216.0	-.031	0.774
109.	1387.8	9483.4	12836.5	2979.6	-4945.0	-.013	0.787
110.	4130.8	7071.3	9063.0	2983.1	-844.0	-.002	0.789
111.	4271.1	12233.4	7222.4	2985.0	6297.0	.016	0.723
112.	2487.6	14252.6	6472.7	2988.5	7279.0	.019	0.754
113.	.0	14030.0	5212.0	2988.9	5829.1	.015	0.739
114.	.0	8530.1	3210.1	2988.2	2331.8	.006	0.733
115.	.0	11222.4	2559.4	2986.6	5676.5	.015	0.719
116.	.0	11447.6	2885.6	2988.2	5573.9	.014	0.704
117.	.0	7406.5	4139.1	2988.1	279.3	.001	0.704
118.	.0	6395.6	7480.0	2985.6	-4069.9	-.010	0.714
119.	.0	2131.9	9907.5	2983.5	-10759.1	-.028	0.742
120.	.0	2784.5	13087.0	2980.3	-12282.8	-.031	0.773

TABLE 5.13: MONTHLY GROUND WATER BALANCE FOR THE THIRTY NINTH AND FORTIETH YEAR IN THE CANAL COMMAND NEAR RD838 ALL VOLUMETRIC UNITS ARE IN CUBIC METRE

MONTH AFTER IRRIGATION	RAINFALL RECHARGE	IRRIGATION RETURN FLOW	EVAPORATION LOSSES FROM WATER TABLE	OUTFLOW	CHANGE IN STORAGE	RISE (M)	DEPTH TO WATER TABLE BELOW GROUND SURFACE (m)
457	1387.8	9483.4	12929.9	2979.6	-5038.4	-.013	0.786
458	4130.8	7071.3	9128.2	2983.0	-909.2	-.002	0.788
459	4271.1	12233.4	7268.2	2985.0	6251.3	.016	0.772
460	2487.6	14252.6	6507.8	2988.4	7243.9	.019	0.753
461.	.0	14030.0	5234.6	2988.9	5806.5	.015	0.738
462.	.0	8530.1	3221.3	2988.2	2320.6	.006	0.734
463.	.0	11222.4	2567.0	2986.6	5668.8	.015	0.718
464	.0	11447.6	2893.6	2988.2	5565.8	.014	0.704
465	.0	7406.5	4149.6	2988.1	268.8	.001	0.703
466	.0	6395.6	7497.2	2985.6	-4087.2	-.010	0.713
467	.0	2131.9	9929.2	2983.5	-10780.8	-.028	0.741
468	.0	3784.5	13114.4	2980.3	-12310.3	-.032	0.773
469	1387.8	9483.4	12929.9	2979.6	-5038.4	-.013	0.786
470	4130.8	7071.3	9128.2	2983.0	-909.2	-.002	0.788
471	4271.1	12233.4	7268.2	2985.0	6251.3	.016	0.782
472	2487.6	14252.6	6507.8	2988.4	7243.9	.019	0.753
473	.0	14030.0	5234.6	2988.9	5806.5	.015	0.738
474	.0	8530.1	3221.3	2988.2	2320.6	.006	0.732
475	.0	11222.4	2567.0	2986.6	5668.8	.015	0.718
476	.0	11447.6	2893.6	2988.2	5565.8	.014	0.704
477	.0	7406.5	4149.6	2988.1	268.8	.001	0.703
478	.0	6395.6	7497.2	2985.6	-4087.2	-.010	0.713
479	.0	2131.9	9929.2	2983.5	-10780.8	-0.28	0.741
480	.0	3784.5	13114.4	2980.3	-12310.3	-.032	0.773

Drain spacing for different depth of drain has been found and are given in Table 5.14. The cost of excavation has been assumed to be Rs. 17.00 per cubic meter. The cost of 0.2 m dia drain pipe has been assumed to be Rs. 75.00. From Table 5.14 it can be seen that the optimal spacing of the drain is 93 meters. At this spacing, total length of field drain is 4513 meters and the cost of excavation and field drains is Rs. 3,95,083.00.

TABLE 5.14: DRAIN SPACING AND COST AT VARIOUS DEPTHS

DEPTH OF DRAIN (m)	DRAIN SPACING (m)	TOTAL LENGTH OF PIPE (m)	COST OF PIPE (Rs)	COST OF EX-CAVA-TION (Rs)	TOTAL COST OF EXCAVA-TION AND PIPE (Rs)
1.10	43.0	9771	732894	74813	807707
1.20	59.9	7015	526180	58595	584775
1.30	71.3	5890	441792	53297	495089
1.40	79.6	5278	395916	51437	447353
1.50	85.5	4910	368299	51267	419566
1.60	89.6	4685	351442	52182	403624
1.70	92.1	4560	342023	53957	395980
1.80	93.0	4513	338534	56548	395082
1.90	92.5	4539	348249	64635	412884
2.00	90.5	4643	348249	64635	412884
2.10	86.7	4841	363112	70763	433875
2.20	81.2	5174	388079	79230	467309
2.30	73.3	5732	429917	91761	521678
2.40	62.2	6756	506740	112861	619601
2.50	45.5	9222	691678	160469	852147

5.2.2.3 Design of Subsurface Drainage

Considering the seepage losses from the canal and percolation loses from irrigation application in the area, intercepting drains and field drains will be required to contain the water table position at the desired depth.

a) *Design of intercepting drains*

A schematic section of two intercepting drains running parallel to the canal is shown in Fig.5.11. It is assumed that the intercepting drains are perforated in the lower half portion only and run half filled. Let D_1 be the depth to the impervious layer below the first intercepting drain. Let the flow is radial within a zone $D_1/\sqrt{2}$ from the drain. Let the first intercepting drain to be at a distance of L_1 from the canal bank. Let the height of water in the second intercepting drain be h_3 above the impervious layer. Let the second drain be at a distance of L_2 from the

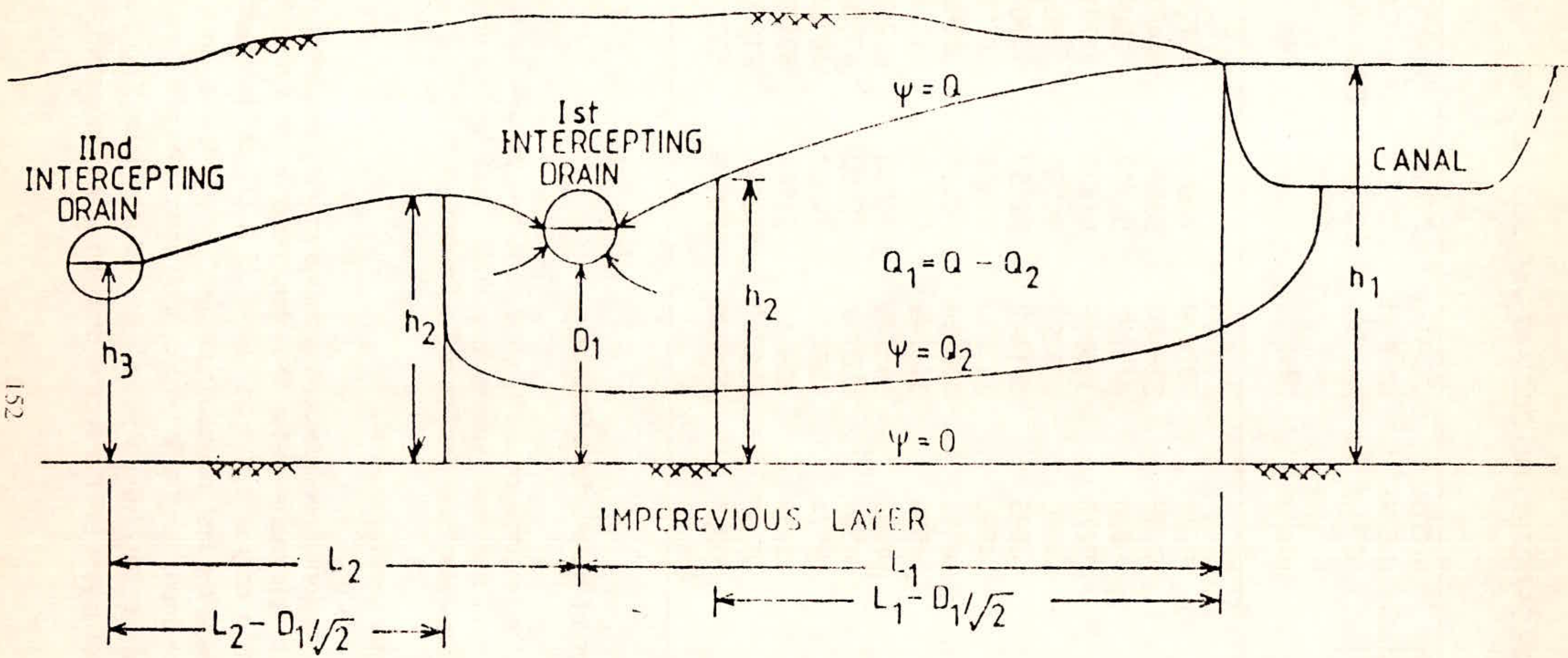


Figure 5.11 Configuration of flow to two parallel intercepting drains

first drain. Assuming that Dupuit-Forchheimer's assumptions are valid in the zone L_1-$D_1/\sqrt{2}$ the flow from the canal is

$$Q = k \frac{h^2{}_1 - h^2{}_2}{2(L_1 - D_1/\sqrt{2}} \tag{5.19}$$

Considering the flow to the first intercepting drain to be radial, the quantity of flow to the drain can be derived as

$$Q_1 = \frac{\pi k(h_2 - D_1 - d/2)}{\log_e(\sqrt{2}D_1/d)} \tag{5.20}$$

Flow entering into the second intercepting drain is given by

$$Q_2 = k \frac{h^2{}_2 - h^2{}_3}{2(L_2 - D_1/\sqrt{2})} \tag{5.21}$$

Equating $Q_1 + Q_2 = Q$, unknown h_2 is found to be

$$h_2 = \frac{-b + \sqrt{b^2 - 4ac}}{2a} \tag{5.22}$$

where,

$$a = \frac{1}{2(L_1 - D_1/\sqrt{2}} + \frac{1}{2(L_2 - D_1/\sqrt{2})}$$

$$b = \frac{\pi}{\log_e(\sqrt{2}D_1/d)}$$

and,

$$c = \frac{\pi(D_1 + d/2)}{\log_e(\sqrt{2}D_1/d)} + \frac{h^2{}_1}{2(L_1 - D_1/\sqrt{2})} + \frac{h^2{}_3}{2(L_2 - D_1/\sqrt{2})}$$

Hence, Q, Q_1 and Q_2 are solved for known value of h_1, h_3, L_1, L_2, d, D_1 and hydraulic conductivity, k. Once Q_1 is known, drain diameter can be obtained by using the following equation (Wesseling and Homma, 1967):

$$d = \left[\frac{0.1153\, Q_1 L}{\pi S^{0.572}}\right]^{0.3686} \tag{5.23}$$

In which d is the diameter of the intercepting drain in meter, L is the length of the intercepting drain in meter, Q_1 is discharge in comec per unit length of the *drain*.

From the field permeability tests it has been found that the hydraulic conductivity of the top layer with 50% probability is 0.62 m/day. The average *hydraulic conductivity estimated at 15 locations is 0.67 m/day.*

From the study of the topographic map (Fig.5.12) it is proposed to construct the first and second intercepting drains at a distance of 100 m and 200 m from the canal respectively and accordingly the design diameters have been computed. Diameter of the first and second intercepting drains are given in Table 5.15.

Table 5.15: Diameter of interceptor drains

Length	Internal Diameter of intercepting drain	
	Drain I (cm)	Drain II (cm)
Up to 500 m	7.0	3.5
500 - 1,000 m	9.0	4.0
1,000 - 1,500 m	10.0	5.0

However, this proposed distance has to be further confirmed from detailed field investigation.

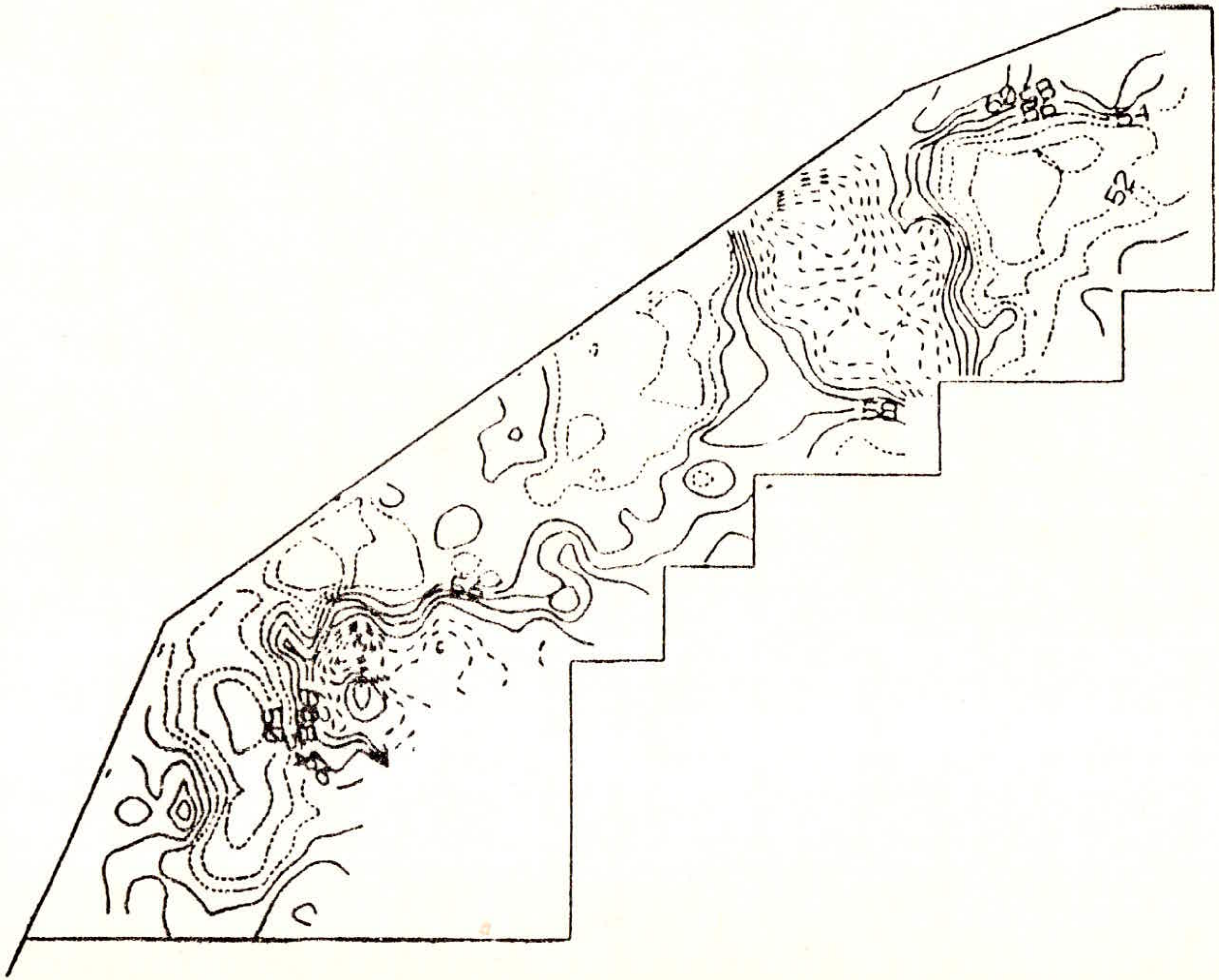


Figure 5.12 Surface allegation map of the study area

b) Filter design

The principal requirements for a satisfactory filter are that it should be more pervious than the protected soil and at the same time fine enough to prevent particles of protected soil from washing into its voids. Terzaghi's work is perhaps the first rational approach to filter design. Bertrom conducted several laboratory tests on uniform sands and developed procedure for evaluating the efficacy of filters. USBR and Army Corps of Engineers later confirmed and extended the work of Bertrom. Following are the rules widely used for filter design.

a. The 15% size of filter (D_{15}) should be at least five times as large as D_{15} size of protected soil.

b. The D_{15} size of filter should not be larger than the five times the D_{85} size of protected soil.

c. The gradation curve of the filter .should have same shape as that of protected soil.

d. Filters should not contain more than about 5% fines passing through 200 no sieve and should be cohesion less.

The Soil Conservation Service (1971) has reported the following criteria for filter design

a. The D_{50} size limits for filter are obtained as 12 and 58 times the D_{50} size of protected material.

b. The D_{15} size limits for filter are 12 and 40 times the D_{15} size of protected material.

c. All filter material should pass 40 mm sieve, 90% should pass through 20 mm sieve and not more than 10% should pass through 250 micron sieve.

d. Where filter and bare material are uniformly graded, the D_{15} size of filter should not be larger than the five times the D_{85} size of protected soil.

Based upon these procedure envelope curves are developed for the project area as given in Fig 5.13. The dotted line on the map indicate the limits using Bertrom and USBR approach and the thick lines indicates the limiting values using SCS approach. The hatched area in Fig.5.13 gives the average size of filter material ranging from 0.2 to 10 mm.

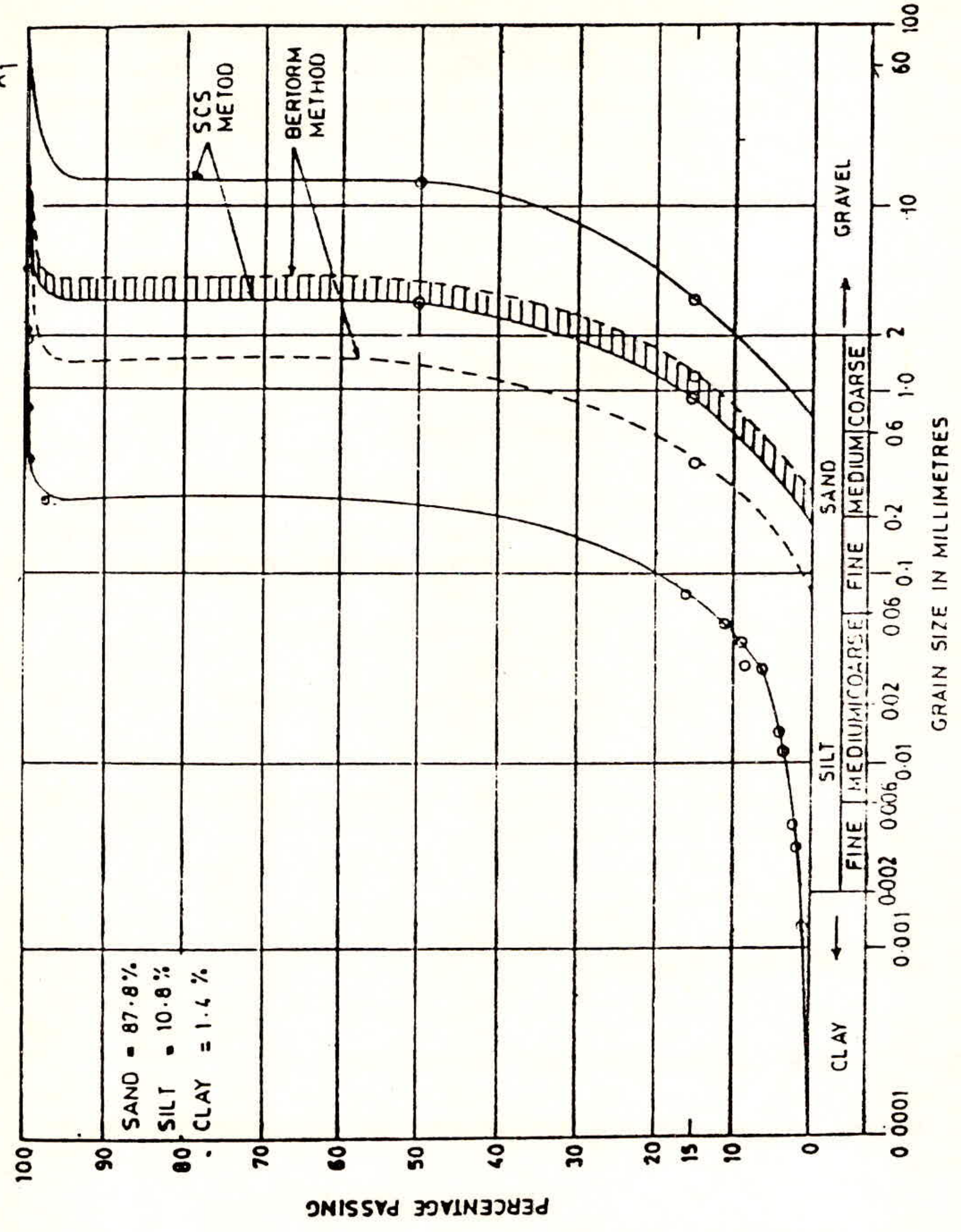


Figure 5.13 Particle size distribution curve and filter design

5.2.3 Design of a Tube Well Drainage System

The tube well with pumping equipment is the element of a sub surface vertical drainage system. The required number of tube wells for controlling the water table height, their layout and pumping schedule, the gravel pack design around each tube well are the important aspects of a vertical drainage design.

5.2.3.1 Data Requirement:

The data required for the design of a vertical drainage system are as follows:

i) Area in which the water table is to be lowered for controlling water logging;

ii) Types of crops grown and method of irrigation;

iii) Required drawdown to contain the water table below the critical depth in the problematic area, the required drawdown can be ascertained from the hydrograph records at a few observation points in the problematic area;

iv) The duration of high water table in the water logged area;

v) Location of hydrologic boundary i.e. presence of river and no flow boundary;

vi) Aquifer parameters i.e. transmissivity, and storage coefficient and grain size distribution of the aquifer material;

vii) Capacity of the pump; and

viii) Utility of the pumped water.

5.2.3.2 An Example for Vertical Drainage Design

A vertical drainage system is required to be designed for the following field condition.

i) The transmissivity of the aquifer, T=300 m^3 per day,

ii) storage coefficient = 0.05,

iii) Area vulnerable

iv) Capacity of each pump = 600 m^3 /day,

v) Water table rise = 0.8m above the critical level and it is at 0.2 m below ground surface,

vi) The crop grown in the area is wheat,

vii) A pump can operate only 8 hours a day because of the non availability of power supply in the vulnerable area,

viii) As per the irrigation water requirement during the first two months irrigation is required once in every 15 days and 6cm of water is applied in each irrigation,

ix) The groundwater is suitable for irrigation.

For controlling water logging through vertical drainage in an area where the ground water is not saline, it is appropriate to make use of ground water for irrigation. If each well operates for 8 hours a day it can irrigate an area equal to 600/(3x0.06)=3333m^2 in each day. Since one irrigation is given once in every 15 days each well can irrigate an area equal to 3333x15=5 hectare. Total number of well required for irrigation in an area of 50 square km is 1000. The spacing between two wells is 224m. In case power is available continuously 334 number of wells will be required for irrigation and spacing between the wells will be 388m.

The adequacy of the wells for controlling water logging is verified as follows:

The maximum volume of water that remains in the water logged area during the period of water logging = area x (depth from ground surface to water table position that initiates water logging-depth to water table position during cropping period) x storage coefficient. For the present case the maximum volume of water is found to be $50\times10^6 \times (1.0-0.2) \times 0.05 = 2\times10^6$ m^3 . This volume of water is required to be pumped within a specified time which is found to be 5 days from Fig.5.14 (ILRI, 1973). If this volume of water is pumped from the water logged area some quantity of water also enters from the adjoining area. The volume of water that enters to the pumped zone can be found using the following relation:

$$V = Q(t)\,[\,t \exp\{-R^2 S / (4 T t)\} - R^2 S/(4 T)\, E_1 \{R^2 S/ (4 T t)\} \qquad (5.24)$$

in which

5.37

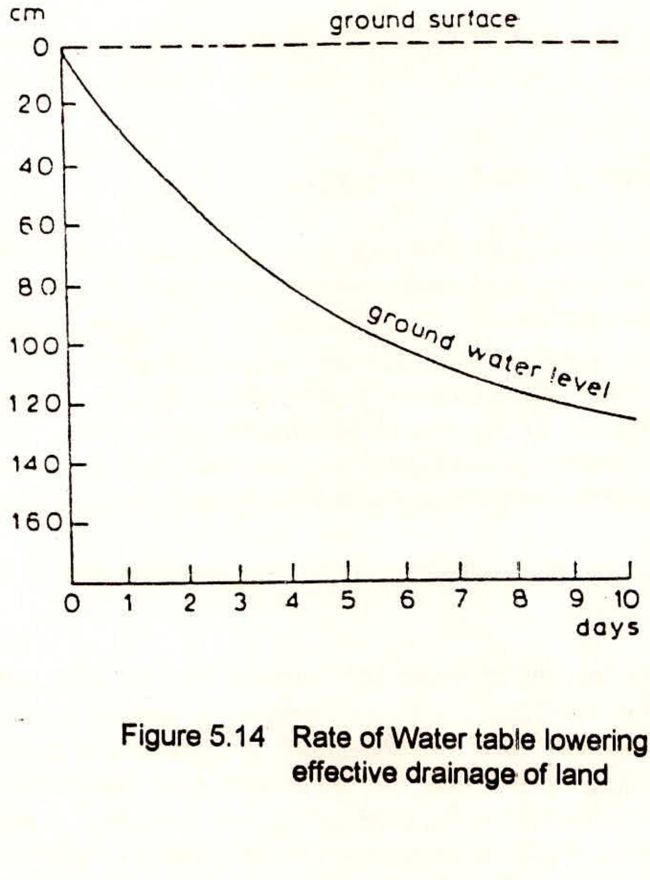


Figure 5.14 Rate of Water table lowering required for effective drainage of land

$Q(t)$	=	uniform pumping rate,
R	=	radius of the equivalent area of the water logged region,
$E_1(X)$	=	$\int_X^{\infty} \frac{e^{-u}}{u}$, an exponential integral equal to the This well function for a confined non leaky aquifer.

This formula has been derived assuming that pumping is continuous and is lumped at the centre of the water logged area.

For the present case R=4000 m; $Q(t)=2\times10^6/5=4.0\times10^5\ m^3$ per day. The inflow to the pumped area computed from the above equation comes out to be very small. Therefore, the quantity of water to be pumped in each day is $4.0\times10^5\ m^3$ to control water logging. The total number of wells required is $4.0\times10^5/200=2000$. Because of non availability of power the pumps are run only for 8 hours a day. If these can be run continuously only 667 wells are required. Hence, for controlling water logging 667 wells out of the 1000 wells need to be operated for 5 days. The minimum wells required for controlling water logging are = 667.

5.2.3.4 Sub Surface Horizontal Drainage System

Most of the equations used for findind the spacing of drains to contain the water table below root zone depth are based on the Dupuit-Forchheimer assumptions. Consequently they have to be considered as approximate solutions only. Such approximate solutions, however, are generally accepted as having such a high degree of accuracy that their application in practice is completely jutified. Though the flow to the drains remains in an unsteady state condition the drain spacings are generally computed using solution of steady state flow. The drain spacing can also be computed considering the unsteady state flow condition.

The spacing of fully penetrating vertically walled parallel ditches to contain the water level away from root zone when prolonged recharge is taking place at uniform rate can be computed using the Donnan's equation. The spacing of tile drains can be worked out using Hooghoudt's equations which considers the radial flow to the drains. In the following paragraphs the details of the derivations of Donnan's equation and the Hooghoudt's equation have been given.

5.2.3.5 The Donnan's Equation:

Under condition of steady state flow, the inflow to the column ABCD is equal to the outflow from the column as shown in Fig. 5.15:

$$q_x + R\,\delta x = q_{x+\delta x}$$

$$\text{or} \quad \frac{dq_x}{dx} = R \qquad (5.25)$$

in which R is the uniform recharge rate per unit area. According to Darcy's law and Dupuit Forchheimer's assumptions

$$q_x = -K h \frac{dh}{dx} \qquad (5.26)$$

in which h is the saturated thickness of the aquifer at x.

Incorporating equation (5.26) in equation (5.25)

$$\frac{d}{dx}\left[-Kh \frac{dh}{dx}\right] = R$$

or

$$\frac{d^2}{dx^2}[h^2] = -\frac{2R}{K} \qquad (5.27)$$

Integrating twice

$$h^2 = -\frac{R}{K}x^2 + Ax + B \qquad (5.28)$$

Since water level in both the ditches are equal, therefore at x=0, and x=L, h=D. Applying these conditions the constant are found to be A = RL/K and $B=D^2$. The *final expression of h is given by*

$$h^2 = \frac{R}{K} x\,[L-x] + D^2 \qquad (5.29)$$

Let H be the maximum height of water level above the impervious floor at mid way between the ditches, i.e. h=H at x = L/2. Hence,

$$H^2 = RL^2/4K + D^2 \qquad (5.30)$$

or

$$R = (H^2 - D^2)\, 4K/L^2 = 4K(H+D)\,(H-D)\,/L^2$$

$$= 4K\,(H-D+2D)\,(H-D)\,/\,L^2 \qquad (5.31)$$

Equation (5.31) is known as Donnan's equation.

Let at $x = L/2$, h_{max} be the height of water table above the water level in the ditches. Hence,

$$R = \frac{4\,K\,h_{max}\,(2D+h_{max})}{L^2}$$

$$= \frac{4Kh^2_{max}}{L^2} + \frac{8KDh_{max}}{L^2} \qquad (5.32)$$

The term $4Kh^2_{max}/L^2$ apparently represents part of the recharge rate R coming to the ditch above the water level in the ditch. With this assumption the equation for flow in a layered soil system is

$$R = \frac{4K_a h^2_{max}}{L^2} + \frac{8K_b D\, h_{max}}{L^2} \qquad (5.33)$$

in which K_a is the hydraulic conductivity of the soil above the water level in the ditch, and K_b is the hydraulic conductivity of the soil mass below the water level in the ditch.

For a several soil layers the equation can be extended as

$$R = \frac{4K_1 h^2_{max}}{L^2} + \frac{8K_1 D_2 h_{max}}{L^2} + \frac{8K_2 D_2 h_{max}}{L^2} + \frac{8K_3 D_3 h_{max}}{L^2} \qquad (5.34)$$

D being equal to $D_1+D_2+D_3$. Knowing the recharge rate R (m/unit time), the thickness of each stratum D (m) and the saturated hydraulic conductivity K_i (m/unit time), the allowable water table height h max (m) above the water level in the ditches, the required ditch spacing L(m) can be found.

5.2.3.6 *Hooghoudt's Equation*

If the ditches do not reach the impervious floor, the flow lines will not be parallel and horizontal but will converge towards the drain. In this region the flow system cannot be simplified to a flow field with parallel and horizontal streamlines without introducing large errors. The radial flow causes a lengthening of the flow lines. This lengthening causes a more than proportional loss of hydraulic head since the flow velocity in the vicinity of the drains is larger than else where in the flow region. Consequently, the elevation of the water table will be higher when the vertically walled ditches are replaced by pipe drains, the drain level remaining the same.

Hooghoudt derived a flow equation for the flow as presented in Fig. 5.16, in which the flow region is divided into a part with horizontal flow and a part with radial flow. Up to a radial distance of $D/\sqrt{2}$ from each drain the flow is assumed to be radial and in the central portion over a length of $L-D\sqrt{2}$ the flow lines are assumed to be horizontal. Neglecting the variation in radial flow quantity the equation for the radial flow can be expressed as

$$-q_r = (2\pi p/4)(-)K \frac{dh}{dr} \tag{5.35}$$

or

$$\frac{dr}{r} = -\frac{\pi}{2q_r} K \, dh \tag{5.36}$$

in which q_r is the quantity of flow entering to the drain. Integrating and applying the boundary condition $h=0$ at $r=r_0$

$$h = 2q_r/(\pi K) \log_e \frac{r}{r_0} \tag{5.37}$$

The quantity q_r is equal to $RL/2$. Substituting the value of q_r in equation (5.37)

$$h = RL/(K) \log_e \frac{r}{r_0} \tag{5.38}$$

Let h_2 be the hydraulic head dissipated in the zone of radial flow. Hence,

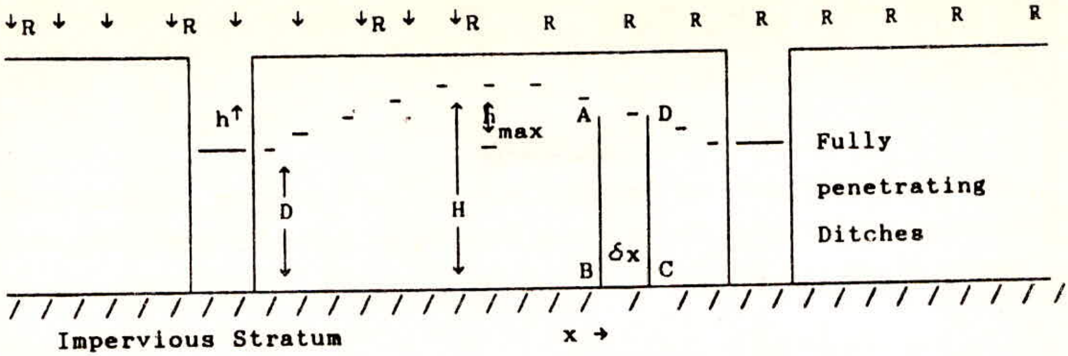


Figure 5.15 .- Steady flow to two fully penetrating parallel ditches for which Donnan's equation is applicable

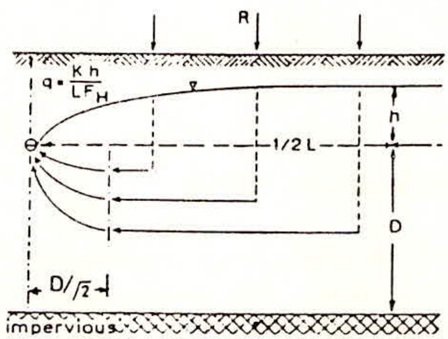


Figure 5.16 - Decomposition of the flow domain made for derivation of HOOGHOUDT'S equation

$$h_2 = RL/(\pi K) \log_e \frac{D/\sqrt{2}}{r_0} \qquad (5.39)$$

Let h_1 be the hydraulic head dissipated in zone of horizontal flow. Neglecting the component of R leaving the horizontal flow region above the level $D+h_2$ and applying equation (5.33)

$$R = 8K(D+h_2)h_1/(L-D\sqrt{2})^2 \qquad (5.40)$$

Assuming $D+h_2 \cong D$, the component h_1 can be approximated as

$$h_1 = R(L-D\sqrt{2})^2/(8KD)$$

Since, $h_{max} = h_1+h_2$,

$$h_{max} = R(L-D\sqrt{2})^2/(8KD) + RL/(\pi K) \log_e \{D/(\sqrt{2}r_0\} \qquad (5.41)$$

$$= (RL/K)[L-D\sqrt{2})^2/(8DL) + (1/\pi) \log_e \{D/(\sqrt{2}r_0)\}] \qquad (5.42)$$

or

$$h_{max} = RL/K) F_H \qquad (5.43)$$

in which

$$F_H = (L-D\sqrt{2})^2/(8DL) + (1/\pi) \log_e \{D/\sqrt{2}r_o\} + f(D,L) \qquad (5.44)$$

f(D,L) is a function of D and L, generally small compared with other terms in equation (5.44) and the term is neglected. For known values R, K, D, r_0 the desired drain spacing L can be computed to contain the water level at h_{max} by solving the quadratic equation given in equation (5.42). Out of the two roots the positive root gives the drain spacing.

Hooghoudt further developed the concept of equivalent depth. Instead of working with equations (5.43) and (5.44), he considered it more practical to have a formula similar to the equation describing flow to fully penetrating ditches. To account for the extra resistance caused by the radial flow, Hooghoudt introduced a reduction of the depth D to a smaller equivalent depth d. By so doing the flow pattern is replaced by a model with horizontal flow only.

By considering flow below the drain level

$$R = 8\,K\,d\,h_{max} / L^2 \tag{5.45}$$

From equation (5.43)

$$R = K\,h_{max} / (L\,F_H) \tag{5.46}$$

Equating equations (5.45) and (5.46) the equivalent depth d is found to be

$$d=L/(8F_H) \tag{5.47}$$

The factor d is like F_H a function of L D and r_0 . Hooghoudt tabulated the value of equivalent depth for different values of D,L and $r_0 = 0.1$ which are presented in Table 5.16. In order to take radial flow into account the following equation using equivalent depth can be written:

$$R = (\,8\,K_b\,d\,h_{max} + 4\,K_a\,h^2_{max}) / L^2 \tag{5.48}$$

Equation (5.48) is Hooghoudt equation and can be used to find the required drain spacing L (ILRI, 1973).

The Hooghoudt equation is commonly used to calculate the drain spacing L if the factors q h K, D and r_0 are known (Fig. 5.17). Since the drain spacing L depends on the equivalent depth d which in turn is a function of L, its use involves a trial and error procedure. The following example is given for highlighting the use of Hooghoudt's formula in finding the drain spacing.

Example:

For the drainage of an irrigated area drain pipes with a radius of 0.1m will be used. They will be placed at a depth of 2.0m below the soil surface. A relatively impermeable soil layer was found at a depth of 6.0m below the soil surface. From auger hole tests the hydraulic conductivity of the upper soil layer was estimated at 1.0 m per day. An irrigation is to be applied once in 20 days. The average irrigation losses in 20 days amount to 40 mm. Find the drain spacing so that the water table is maintained at a depth of 1.2m below ground surface.

From the above information r_o=0.1m; K =1m/day; D = 4m; h=0.8m; R=0.002m/day. From the nomograph for K/R=500; D/h=5; $h/(\pi r_o)$ =2.55 the trial value of L/h=130. Hence L=104m.

For L=104m F = 3.97 and h=0.825m. Since the computed value of h is more than 0.8m the spacing is to be decreased. Let the drains are placed at a

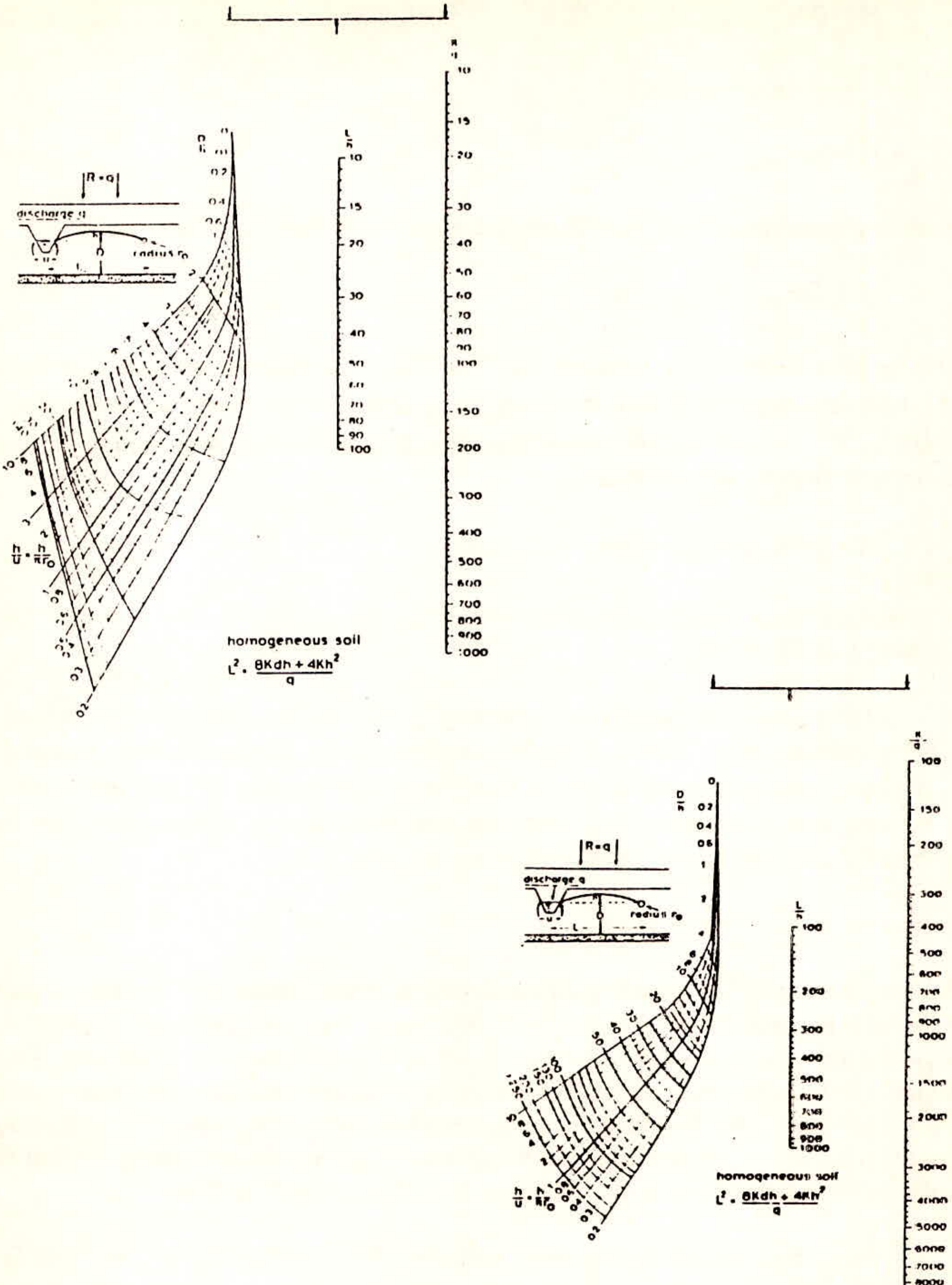


Figure 5.17 - Nomograph for evaluating approximate drain spacing

Table 5.16 Values of the equivalent depth d of Hooghoudt evaluated for r_0= 0.1m, and for different values of D and L in m.

L	5m	7.5	10	15	20	25	30	35	40	45	50
D											
0.5m	0.47	0.48	0.49	0.49	0.49	0.50	0.50	-------	-------	-------	-------
0.75	0.60	0.65	0.69	0.71	0.73	0.74	0.75	0.75	0.75	0.76	0.76
1.0	0.67	0.75	0.80	0.86	0.89	0.91	0.93	0.94	0.96	0.96	0.96
1.25	0.70	0.82	0.89	1.00	1.05	1.09	1.12	1.13	1.14	1.14	1.15
1.50		0.88	0.97	1.11	1.19	1.25	1.28	1.31	1.34	1.35	1.36
1.75		0.91	1.02	1.20	1.30	1.39	1.45	1.49	1.52	1.55	1.57
2.00			1.08	1.28	1.41	1.50	1.57	1.62	1.66	1.70	1.72
2.25			1.13	1.34	1.59	1.69	1.69	1.76	1.81	1.84	1.86
2.50				1.38	1.57	1.69	1.79	1.87	1.94	1.99	2.02
2.75				1.42	1.63	1.76	1.88	1.98	2.05	2.12	2.18
3.00				1.45	1.67	1.83	1.97	2.08	2.16	2.23	2.29
3.25				1.48	1.71	1.88	2.04	2.16	2.26	2.35	2.42
3.50				1.50	1.75	1.93	2.11	2.24	2.35	2.45	2.54
3.75				1.52	1.78	1.92	2.17	2.31	2.44	2.54	2.64
4.00					1.81	2.02	2.22	2.37	2.51	2.62	2.71
4.50					1.85	2.08	2.31	2.50	2.63	2.76	2.87
5.00					1.88	2.15	2.38	2.58	2.75	2.89	3.02
5.50						2.20	2.43	2.65	2.84	3.00	3.15
6.00							2.48	2.70	2.92	3.09	3.26
7.00							2.54	2.81	3.03	3.24	3.43
8.00							2.57	2.85	3.13	3.35	3.56
9.00								2.89	3.18	3.43	3.661
10.00									3.23	3.48	3.74
∞	0.71	0.93	1.14	1.53	1.89	2.24	2.58	2.91	3.24	3.56	3.88

Table 5.31 contd.

L	50	75	80	85	90	100	150	200	250
D									
0.5	0.50								
1	0.96	0.97	0.97	0.97	0.98	0.98	0.99	0.99	0.99
2	1.72	1.80	1.82	1.82	1.83	1.85	1.00	1.92	1.94
3	2.29	2.49	2.52	2.54	2.56	2.60	2.72	1.92	1.94
4	2.71	3.04	3.08	3.12	3.16	3.24	3.46	3.58	3.66
5	3.02	3.49	3.55	3.61	3.67	3.78	4.12	4.31	4.43
6	3.23	3.85	3.93	4.00	4.08	4.23	4.70	4.97	5.15
7	3.43	4.14	4.23	4.33	4.42	4.62	5.22	5.57	5.81
8	3.56	4.38	4.49	4.61	4.72	4.95	5.68	6.13	6.43
9	3.66	4.57	4.70	4.82	4.95	5.23	6.09	6.63	7.00
10	3.74	4.74	4.89	5.04	5.18	5.47	6.45	7.09	7.53
12.5		5.02	5.20	5.38	5.56	5.92	7.20	8.06	8.68
15		5.20	5.40	5.60	5.80	6.25	7.77	8.84	9.64
17.5		5.30	5.53	5.76	5.99	6.44	8.20	9.47	10.4
20			5.62	5.87	6.12	6.60	8.54	9.97	11.1
25			5.74	5.96	6.20	6.79	8.99	10.7	12.1
30							9.27	11.3	12.9
35							9.44	11.6	13.4
40								11.8	13.8
45								12.0	13.8
50								12.1	14.3
60									14.6
	3.88	5.38	5.76	6.00	6.26	6.82	9.55	12.2	14.7

distance of 103m. For L=103m F_H = 3.9388 and h = 0.81. The drain spacing is to be further decreased. Let L be 102m. For L=102m F_H =3.9077 and h=0.797m. Hence the required spacing of the drains is 102m.