GROUND WATER MODELING OF VARUNA BASIN

A DISSERTATION
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the requirements for the award of the degree
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in
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By

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CERTIFICATE

Certified that the dissertation entitled " GROUND WATER MODELLING OF VARUNA BASIN", which is being submitted by Er. Gopal Chandra Verma in partial fulfilment of the requirement for award of the degree of Master of Engineering in Hydrology by the University of Roorkee is a record of his own work, carried out under our supervision and guidance. The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for more than nine months from October 1974 to January 20, 1976 for preparation of this dissertation.

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SYNOPSIS

The existing water resources though plentiful can not be exploited in-discriminately without an adverse effect on the total water supply. Hence it is necessary to find means of better utilisation of available surface and ground water resources for increasing living standard of people. For ground water exploitation one has to investigate and evaluate ground water potential of the basin and understand the mechanism of the system. Ground water problems can be studied either by carrying out field studies or with the help of models.

Field study is always time consuming and expensive, so model study is preferable. Models are useful tools for prediction of influence of management practices on water table or piezometric level within an aquifer. There are two types of models, physical models and mathematical models.

The physical model needs laboratory facility as well as more time for construction of model and is built for specific problem. The mathematical model which removes those difficulties is based upon mathematical expression or group of expressions that describe the aquifer functioning. Under this study a mathematical model was employed for case study of Varuna Ground Water Basin.

The river Varuna, a tributary of river Ganga has its confluence at Varanasi. The basin which lies in four districts, Pratapgarh, Allahabad, Jaunpur and Varanasi of eastern part of Uttar Pradesh, covers total area of 2,57,870 Hect.

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Contd.

In the present study both lumped and distributed models were used to evaluate the ground water potential and study the ground water response. Lumped model was employed to estimate the ground water potential of the basin as a whole using hydrological balance equation which incorporates all the components of hydrological cycle related to ground water movement.

Ground water potential so arrived, comes to 16,868 hectare meters. The annual water balance on the basis gave the recharge to the aquifer as 34.3% of total rainfall over the basin.

In the distributed model the basin was divided into ten small areas and the water balance equation was applied to these small areas, through a nodal points. This model used the hydrological parameter related to nodal area. In this model the solution of the partial differential equation of unsteady flow through numerical technique was possible. This model was employed for prediction of water level at nodes located at the distributed points of the basin, on imposed conditions. The predicted water levels show reasonable agreement with observed water levels at those nodal points.

This study has led to a better understanding of the method of analysis of ground water problem.

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CHAPTER-I

INTRODUCTION

Water is essential for existence of mankind. The demand of water is increasing more or more due to rapid industrialisation steady, rise in population and needs for increasing agriculture production. The existing water resources though plentiful can not be exploited indiscriminately without an adverse effect on the total water supply of the world. Hence it is necessary to find means of better utilisation of available water resources for increasing living standard of people.

There is plentiful supply of water over the earth, but only 1% of water in the hydrosphere is in a form that can be readily and economically exploited by present day technology. This 1% includes both surface and ground water. Thus water, the natural resource, which is most vital resource, is limited. In our country, total water which can eventually be utilised is only 93.5 million h.m., out of which ground water contains 26.5 million h.m. These figures are still tentative and approx. and need further estimation with the aid of advanced technology.

However, on the basis of present figure it can be well realised that existing surface water resource can not cope alone with the increasing demand in future. Under such situation due importance is to be given for utilisation of ground water resource. Ground water basin may be considered as reservoir, in which water enters, moves through it and leaves it. The natural course which the water itself follows can be altered by man in the bulk of basin by artificial recharge, pumping from and into wells. This may cause undesirable effect, if proper treatment to the basin is not given. As for example, excessive extraction of ground water may cause lowering of water table beyond economic pumpage limit, and also ground subsidence in other hand excessive contribution to ground water by irrigation and other artificial recharge may create problem of water logging and consequent damage to crop. Hence it is necessary to have most efficient use of available water. With the objective of full utilisation of natural resource, nature of the resource and nature of use must be considered. For this, it is first essential to investigate and evaluate ground water potential of the basin and understand mechanism of the System. After such assessment it is necessary to develop a plan for optimum utilisation of ground water resource. Planning includes besides evaluation of water resource, prediction of future use, and various ways to meet predicted demand.

Ground water problems can be studied with the help of following :-

1. By Field Studies
2. Use of Models

Field study is always time consuming and expensive and so generally models are preferred to study ground water problem.

Models are useful tools for prediction of influence of management practices on water table or Piezometer water levels within an aquifer. There are two types of model ((i) Physical model (ii) Mathematical model. Physical models are experimental studies. Various types of physical models such as sand model, electrical models, viscous models elastic membrane are in use to study the aquifer.

However physical model to represent water system requires equipment and considerable material, some require precise machine shop work, and many require more time for construction. In addition a physical model built for specific problem, which is suited to that problem, needs considerable modification to study another aquifer. Hence these models can not be treated as convenient.

Recently, due to availability of electronic computer, it has now become possible to use mathematical model based on theoretical analysis for ground water movement studies. These mathematical model is based on mathematical expression or group of expressions that describes the aquifer functioning.

Mathematical models have been employed in studies of several basins. H.N. Tyson[13] & E.M. Weber describe the developmental verification of a mathemetical model to study hydrologic characteristic of ground water basin of the coastal plane. In this problem, similar model has been developed to study the ground water problem of various basin.

Mathematical models can be further classified as (a) lump model (b) distributed model. In the lump model the entire basin is taken as composite unit, using parameters, representing hydrological characteristic of whole area. In the distributed model, the basin is divided into small areas using hydrological parameter related to small area.

In this thesis work, both types of mathematical models have been developed to study the ground water problem of Varuna Basin lying in eastern part of U.P.

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CHAPTER-II

MODELS FOR GROUND WATER STUDIES

Introduction:

Hydrologic models[1] are physical or symobolic representation of known or assumed functions expressing the relationship of segments of hydrologic cycle, or expressing the influence of management practice within an aquifer. Ground water model can be represented by any thing which copies or describes the function of aquifer. Models are valuable tools to study natural process. Models for ground water are used to simulate some real ground water system so that it can be predicted what would happen if the system properties are changed. Models are used for convenience and to avoid costly experiment with the real system.

2.1. Necessity of Review:

There are several types of models, but no model can really reproduce all the varieties in a real ground water system. Hence it is necessary to review the available techniques and select the appropriate one for solution ofproblem at hand. This section reviews several such methods with back ground informations for understanding the application, and selection of a suitable model for case study.

2.2. Approach to Model:[2]

Hydrologic process may be complex or simple. Depending on circumstances, models can be developed using either statistical or rational method based on deterministic approach.

2.2.1. Statistical Approach:

In case of most complex natural process such as next week's weather which involves so many input and variable datas that they can not be enumerated or measured. Under such situation the statistical methods can be employed to determine the output, which may be based either on law of probability or on the basis of sequence of occurrence. Thus the methods using statistical approach, develops stochastic or probabilistic models as discussed below:

2.2.1.1. Stochastic Model[3] :

Use of stochastic model becomes necessary for large no of variables, having good range of variance in observation and data point. Stochastic model is true representative model of hydrology process as hydrologic process, a time dependent process, involving sequence of occurrence, behaves as stochastic process, to great extent. In these models, statistical regression by co-relating all conceptually measured variables with the variable of interest is done, and model is further developed using random or non random number depending upon sequence of occurrence of variate.

2.2.1.2. Probabilistic Model[3]

This model is used for sake of simplicity, in which neither sequence of variates involved in the process, nor definite law of certainity is considered. However, the model is developed using chance of occurrence of variables and concept of probabilistic nature. These statistical method needs large no. of input variables for statistical fitting of model, which in turn would need expensive and tremendous work for data collection. Hence this technique is not useful for ground water studies.

2.2. Deterministic Approach[4]

Rational Method:- Rational methods have been used to develop model on deterministic approach. In this chance of occurrence of variables involved in process is ignored, and model is considered to follow physical law, the law of certainity. The model is specified both functionally and numerically without initial recourse to observational data, by assignment of numerical values to coefficient and exponent of the model based upon prior information or physical laws. The model may be applied to data as test, by computing value of dependent variable for selected value of independent variable. Necessary adjustment in co-efficient or exponents to have good agreement with observed and computed data are done. The rational methods would be more useful when the number of unknown and number of observations are small. Models based on rational method can be physical or mathematical model, based on theoretical analysis.

2.2.2.1. Physical Model:- Physical model of ground water is actually material representation of the aquifer. The physical model is used when the natural processeses which are complex can not be easily understood. Physical model may be either true model or Analogous of ground water flow. True model of aquifer is only represented by sand model, other models are model analogous to aquifer.

Physical models may further be grouped under two types of Model, Mechanical models and Electrical Models.

Mechanical Models:

2.2.2.1.1. Sand Model It is simplest model and most clearly pertinent to hydrologic problem in ground water studies, sand model is a single type of model which employs the same material as of aquifer, and has its process similar its prototype. The flow in both aquifer and model is governed by Darcys law.

Sand models are constructed in water tight boxes, which may be of either rectangular, column or sector form. In the model sand is used as test medium representing the aquifer. However use of fluid which may be either water or but more viscous liquid, depends upon the extent of modification necessary to compensate for permeability of sand as being high compared to size of model.

Sand model thus resembles closely except magnitude and parameter.

Models can be made to represent either confined or unconfined aquifer by allowing water table to act as upper boundary and providing and impervious cover to apply pressure respectively. In order to have test medium of uniform permeability, coarse sand is placed in small quantities and compacted consistently to remove air, where as for anistropic permeability different types of sands in layer are placed.

There are dynamic and kinematic similarities between model and prototype as flows in model as well as in aquifer occurs in porous media. So (Todd[6]) formulas have been developed for interpretation of test data from sand model.

Geometric similarity is defined by the model prototype length rated.

Eqn (1) $$Lr = \frac{Lm}{Lp}$$ where subscript r,m and p refer to ratio model, and prototype respectively.

As flow in medium of model, as well as in aquifer is governed by Darcy's law provided fluid movement in the model lies in laminar range, the velocity ratio for isotropic condition can be expressed as:

Eqn.(2) $$\frac{V_m}{V_p} = \frac{K_m \, i_m}{K_p \, i_p}$$ where K is co-efficient of permeability and i is hydraulic gradient.

Having same slope, prototype velocity and flow rate can be evaluated using following equation.

Eqn. (3) $$V_p = \frac{V_m}{K_r} \quad \text{and} \quad Q_p = \frac{Q_m}{K_r \, L_r^2}$$ Eqn.(4)

<u>Advantage:</u>

In laboratory various ground water problem can be studied with the aid of said model, out of which unsaturated, and multiple fluid flow sudh as sea water intrusion can be considered as useful. By adding dye such as potasimdichromate at various points in sand, flow lines can be made visible, for study of flow and dispersion.

However several problems affect the use of sand model.

1. The reduction in size becomes a problem when the model is too small. Further problems a sealing, obtaining effective ratio between the viscosity of the fluid and characteristic of the sand add further difficulties.

2. Though in case of confined aquifer, capilary rise does not affect much but in case of unconfined aquire magnitude of capillary rise being high compared to size of model and disproporsionate with actual rise in aquifer, causes much difficulty in use.

3. Extension of piezometer into the model as necessary for measurement, presence of air, and organic growth cause interference with flow.

4. In case of optimisation, various trial, by applying experimental data is to be done, hence optimatise of very simple case can be made with the aid of model.

2.2.2.1.2. Elastic Sheet Model(7)

Zwarokin proves that when a thin elastic sheet located in the horizontal x, and y plane is placed under uniform tension and attached to a flat frame, the sheet adjusts itself to any vertical deformation in frame by taking a shape, is governed by following equation.

$$\frac{\partial^2 z}{\partial x^2}\left[1+\left(\frac{\partial z}{\partial x}\right)^2\right] + \frac{\partial^2 z}{\partial y^2}\left[1+\left(\frac{\partial z}{\partial y}\right)^2\right] - 2\frac{\partial^2 z}{\partial x\,\partial y}\cdot\frac{\partial z}{\partial x}\cdot\frac{\partial z}{\partial y}$$

Where Z is vertical displacement.

If the vertical displacements of sheet are kept constant so that sheet is nearly horizontal every where, then

$$\frac{\delta z}{\delta x} \approx 0 \qquad \text{and} \qquad \frac{\delta z}{\delta y} \approx 0$$

then in that case equation 5 can be simplified to $\frac{\delta^2 z}{\delta x^2} + \frac{\delta^2 z}{\delta y^2}$

This is nothing both laplace equation which corresponds to differential equation of gr. water flow $\left(\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} \right)$

This analogy was developed by Hansen(4) for ground water studies. To study shape of free water surface around a well a membrane of surgical rubber is streched over a wooden frame, and clamped under uniform and high tension (not exceeding elastic limit). The frame is placed over glass plate, so that membrane can be deflected vertically to represent draw down, by aluminium strips. Lines of equal vertical displacement corresponding to equipotential line can be mapped by means of micrometer, duly mounted over the frame in proper manner in order to control its vertical position and have free horizontal movement. In order to study path of flow steel ball bearing of 3/16" dic is released at point of high potential, and path of the ball, followed during rolling down ward is photographed with the help of moving camera mounted on the model.

2.2.2.1.3. Viscous Model(8,9)

Hele Shaw, was first man to develop model with certain assumption to demonstrate flow pattern based on analogy between movement of viscous fluid between two closely spaced parallel plates, and two dimensional ground water movement in an aquifer.

After presentation of mathematical analysis by Sri G.C.Stroke and proving the assumption of Hele Shaw by him. This model, commonly known as Hele Chaw model, has come in extensive use.

Formula of Hagen[6]-Poisemille for the flow of a viscous fluid through the narrow inter space between two parallel plate corresponds to Darcy's law which may be seen from following comparison:

Poisemille Law $V_m = \frac{b^2}{12} \frac{\rho_m g}{\mu_m} \frac{dh}{dx}$ V_m = mean velocity of flow in model

Darcys law $V_p = K \cdot \frac{dh}{dx}$ V_p = Velocity in prototype

in which Q = Quantity of water, b = width of space between two parallel plates, ρ_m, μ_m are the model fluid density and viscosity respectively, g is acceleration due to gravity dh/dx is the surface slope where as in darcys law K is permeability of aquifer, and dh is the head difference between the two points of flow path at distance dx.

From above analogy between two laws, the velocity ratio can be written as $V_r = \frac{b^2}{12} \frac{\rho_r}{k} \mu_r$ and for given length ratio Lr, of the model, time ratio can be found from Tr = Lr/Vr (10)

By the use of two sheets other of glass or plastic keeping them parallely at a fixed spacing, models are constructed. Reservoirs to control fluid flow are attached either at end or sides to the model. The versatility of the model as a tool for investigation of both steady and unsteady ground water flow, depends upon simulation of different values of hydraulic conductivity of the soil through variation of spacing and of the fluid properties density and viscosity. As for law viscosity of fluid such as light oil or water very narrow spacings, a fraction of millimeter or so, is used to keep the flow movement in laminar range, where as in case of glycerline or heavy oils, spacings of a few millimeter can be provided. Regional variation of K can be provided by the insertion of thin strips in the desired region between the plates.

These models can be either vertical or horizontal model. Recharge, either due to rainfall or under flow or infiltration from surface water body can be simulated in the model. Rainfall is initiated by sprinkling water over the length of channel in vertical model where as ground water withdrawl or artificial replishment is reproduced by withdrawing or adding liquid at respective plates in the channel. The piezometric level or water table can be observed and flow lines can be seen as well as photographed if dye is used. The simple boundary condition of constant and variable head may be materialised by adjusting and the vessel connecting it to interspace of model. Two liquid problem such as salt water, fresh water relation can also be represented with the aid of two fluids of different density whereas fluids of different colour, having equal density and viscocity may be used to study multiple aquifers.

Though the model has proved its usefulness but difficulty arises in adoption of same in present type of problem on account of following drawbacks:

1. Viscocity and density should be kept constant while using model, and this condition can be rarely satisfied until and unless the model is set up in temperature controlled room.

2. The errors in slot width have very marked influence upon the co-efficient of permeability. As thickness of plate are order of milimetre, great care is required to keep the distance constant for which insertion of specially prepared metallic or plastic rings between the plates at regular interval and tightening the model with the aid of screws is required.

3. Sufficient experience of study on such model is must to resolve particular problem in better way.

2.2.2.1.4. <u>Blotting Paper Model</u>:

Blotting Paper Model: This model has been used by R.J. Seven Huysen[10] for simulating ground water flow. Simulation of infiltration, saturated flow and leaching with the aid of above model, has been described by him. This material can be easily cut into any shape, lines can be drawn on it and samples can be taken out. The shed of blotting paper forming the model, are hung vertically, the flow is introduced by saturated sheets placed vertically or horizontally as required. A brief description of method of use of above model is described here.

In vertical sheet two furrows are cut away, and a net work on rest area of paper is work is drawn by dots using water soluble ink. This paper is vertically hung and water is brought into the paper by means of strips of same material drawing water from the tank. After few minutes (10 minutes taken by Seven Huysten in his experiments) stream lines and distance of displacement is traced. For saturated flow a net work of ink dots is applied on the model after saturated flow has become well established. Impermeable boundary can be represented by means of application of point on the profile with a laqueous solution. Where layer of low permeability is desired, lacquer on humid blotting paper resulting some of the pores remained open, is to be painted. With aid of this model salt concentration change can also be studied using a paper duly soaked into solution of copper sulphate and dried thereafter.

This is very simple and easy model to demonstrate ground water phenomena, but quantitive evaluatic is possible for very simple and isolated cases. Hence for most quantitive work, this can not be used.

2.2.2.1.5. Electrical Model:

The electrical models have been developed to study ground water problem, on account of correspondence of Ohm's law with Darcys law, which may be apparent from following

I = 1/R A V/L (Ohm law) where I is the amount of current 1/R electrical conductivity (reciprocal of resistance), A is cross-sectional areas through which currents flows, V is the electrical potential or voltage, and L is length. It is thus clear that electric current is analogous to discharge, electric conductivity to permeability, voltage to head.

On above approach model is constructed using conductive material which can be (i) liquid conductor (ii) solid conductor (iii) Gelation.

2.2.2.1.5.1. Conductive Liquid Model[6,7,9]

It is electrolytic tank of shallow depth made of a water tight container of non-conductive material, filled with weak solution of an electrolyte such as copper sulphate.

The conductive liquid must be one which has neither reactance nor chemical reaction on electrode and possessess uniform conductivity in entire model with low evaporation rate so that there is no appreciable alteration in conductivity.

In the model, alternating current of low voltage is used for safety as well as to avoid heating of electrolytie.

A recharge boundary is simulated by an electrode, through which current enters the model, where as, discharge point line is simulated by another electrode. In case of constant head in prototype, electrode is kept at constant potential. An impervious boundary is simulated by an insulator. The distribution of voltage corresponds to head distribution.

In this potential devider is used for accurate division of voltage into large no. of sub-division whereas use of oscillograph helps in the identification of the potential. When the potential at desired part is equal to potential adjusted in potentiometer, no current flows through oscillograph, and on this approach lines of constant voltage can be traced by a probe attached to voltameter circuit as described below. A mark is left over the drawing paper placed on drawing board by depressing the pencil attached to panotograph, whose other limb lines over the model. This limb contains the detecting probe of electrical circuit fixed diagnolly opposte to the pencil, by which constant potential drop is traced. Hence due to interdependence, equipotential lines can be plotted on the principle described earlier.

The potential is measured at points on surface of liquid in case of the dimensional flow, and if same is measured within the liquid, three dimensional flow can be studied. Currents in each electrodes are adjusted by suitable resistance to correspond to well flow rate, and similarly study on effects of recharge wells on ground water, can also be studied. In case of aquifer of variable thickness, the bottom of tank is filled with parafin and the configuration carved in the parafin after solidifcation. The tank when fully filled with electrolyte the aquifer thickness is modelled. Thus transmissivity distribution can be studied.

2.2.2.1.5.3. Solid Conductive Model (6,7,9)

In this shed of electrically conducting material of sufficiently uniform, isotropic and high resistance is fastered to broad or other flat surface. Most commonly Teledelitos paper which, contains dispersion carbon, a conductor, and coated with relatively non conducting layer of lacquer on one side and aluminium on the other is used. Recharge boundary can be made of either copper strips pressed on paper, or of siliver paint, or made of a copper wire soldered to paint. A copper strip can be used for straight boundary, but for arbitrary boundary use of paid is easy. This paper of isotropic nature can be cut in slot to represent impermeable layer or other in homogenates. Potential distribution is measured along flow lines and equipotential lines on the surface of the paper with the help of galvanometer as per method described earlier. Flow lines and equipotential lines can be drawn directly on the model during the experiment.

2.2.2.1.5.3. Gelation Conductor:

This model is constructed by pouring hot gelation with the addition of sodium chloride or copper sulphate into mould, having boundary similar to aquifer.The gelation represents aquifer when it solidifies. With the help of appropriate salt concentration, different permeability can be simulated in the model after insertion of gelation at desired location. The shape, size and variable thickness can be easily made in the model to resemble the aquifer condition. Besides other studies, free surface around a well in unconfined aquifer can be determined by cutting the gelation by a hot wire.

2.2.2.1.6. Resistence Capacity Analog[11,12,13]

This model can be constructed to study two dimensional or three dimensional movement of water in aquifer. This model employs resistence net work which consists of junction of resisters and capacitors, forming finite difference grids to represent finite difference form of partial equation of ground water flow. In the model current in various branches of net works are analogous to flow of water, where as voltage at nodes correspond to water level. Resistors, inverse of which corresponds to conductivity of aquifer, dissipates energy in same manner as porous medium of aquifer consumes ground water energy in the movement of water through its void.

In order to simulate pumping rate and variation of drawdown in reservation well, excitation response apparatus is used. The input and output devices used in conjunction with

R.C. net work are strip chart recorder. Cathode ray oscilliscope square wave or step function generators and timing circuit. There are two approaches, one consideration of long time duration (2 to 15 minutes) also known as one shot and other short time duration known as repetiative, for solution of the ground water problem. In short time repetitive network, voltage across each capacitor are brought back to initial condition using de-energising circuit which removes accumulated charge in last part of the cycle period. However, for variable parameters, involving trial and error method for adjustment of system, this type of equipment needs specialised and intricate electronic equipment.

In case of long time computer, the steady state and transient excitation are applied at time t = 0, and resulting voltage transient can be recorded, based on following principle(9). In this wave form generate, which produces saw-tooth pulses, sends positive pulse to the oscilloscope to start horizontal sweep and also it sends a single negative pulse of saw tooth form to pulse generate. The trigger of pulse generator on receipt of initiating pulse, fires and large negative pulse of rectangular shape is applied over a resistor connected to the node representing the pumped well. This pulse is also brought to the grid of a tube having cathode resistor which is jointly used by another tube which in turn is connected to any node of system(observation well). Prior to receipt of above pulse there was large voltage drop across the cathode resistor due to plate current of tube connected to trigger. Hence voltage of grid of tube connected to node representing observation well was not transferred to ossciloscope. The application of negative pulse from triffer cut off the current and allows the other tube to conduct. On account of this high voltage difference between plate and cathode develops which ignates the tube and voltage of node is thereby transferred through the D.C. amplifier to the osscilloscope. Thusthe electron beam is swept across the cathode tube providing a time voltage graph which is analoger to water level fluctuation due to pumpage of known quantity and time. The screen of ossciloscope is accurately corrected to read voltage and time as vertical and horizontal scale. On due consideration of analogy between duration of the pulse from generator as pumping period and amplitude of pulse to pumping rate, the pumping rate can be computed using formula based on Ohm law.

(Chow) $Q = \frac{V}{Rt} a_3$ where V is voltage drop across small resistor R, between pulse generator and the analog model, and a_3 = factor depending upon relationship between Q and I. Based on superposition theory, the total water level change due to pumpage at several nodes, can be determined.

Electric-analog computer is versatile model to study the complex problem of ground water. However in case of ground water flow which involves two or more independent variable, the exclusive use of analog model is not suitable as in this model. Only one independent variable is used.

Secondly this equipment can be handled by electrical man as largest errors are introduced by the input and output equipment. The accurate application of current and voltage excitation of arbitrary shape involves practical difficulties. The cost of model construction can be treated as high as compared operational cost in use of digital computer.

2.2.2.1.7. Analog Model (Electronic Differential Analyser)

Tysen and Weber[14] employed electronic differential analyser to study ground water proble, as an additional equipment in electrical network. Differential analyser is actually equation solver rather than physical analog system. The differential analyser can solve directly finite difference equation at each node of field, governed by partial differential equation. Tysen used a pair of resistor between two junction node of net work for compatibility and connected voltage divider potentiometer between the two resistor, terming non-standard, voltage divider circuit. Another circuit using operational amplifier, parallel-by connected to integrating capacitor, joined with another potentiometer for obtaining variable capacitance, was formed.

The replenishment and extraction data in form of time dependent source flow rate were inserted at the appropriate node of analog circuit. The resulting water level response (node voltage vrs. time) was recorded graphically using x,y ploter.

On due comparison of computed water level with observed water level by superposing on these plots, the portion of basin having sufficient variation was marked. The transmissibility or storage co-efficient or both are then changed to adjust water level and new response was obtained. The cut and try process of adjusting physical constant was continued in this manner until satisfactory agreement was reached between computed and observed water level of the basin.

This technique can be considered as useful determination of local transmissibility and storage co-efficient of aquifer. However use of this type of model, may be limited for study of dynamic behaviour of the aquifer under certain specific conditions of replishment or extraction. In this model, response of entire network, due to application of unit flow rate, at a node (all other source flow rate set to zero) could be plotted. Similar response would be obtained for successive application of unit flow rate at other nodes. Any desired solution could be constructed from these influence function by superposition. This procedure would require large nos. of computer run and construction of model is not economical and therefore not very much suitable for general study of aquifer.

2.2.2.2. Mathematical Model:

Mathematical model is mathematical analogue of the hydrologic basins and so ground water mathematical model simulates the behaviour of aquifer. Mathematical model of dynamic process can be obtained by two different ways theoretical analysis and/or experimental analysis. In this chapter, discussion on the two methods are made in order to select the best way to develop mathematical model.

Actually theoretical analysis of dynamic process is called theoretical model, which can be defined by[15].

"Process modelling is the theoretical analysis of the time behaviour of process. Physical laws are used to obtain mathematical model". There are four different types of basic equation for general consideration:-

1. Balance equation for stored mass energy or momentum
2. Physical chemical state equation
3. Phenomological process
4. Entropy balance

In case of those ground water basin in which change of salt concentration is not processed(change in salt concentration occurs incoastal area) only first and third equations are relevant:-

(1) For the first equation, the common equation of ground water flow known as equation of continuity can be described. In case of given volume of ground water basin, with due consideration of negligible elasticity in water as well as in

aquifer the form of equation is

Inflow- Outflow = Change in storage

(2) For the phenomologic process, the equation involving state variable known as Darcy's law is used. Based on this equation, amount of flow from given area can be determined using the equation.

Q = P.I.A. where Q = amount of flow,
P is permeability,
I is hydraulic gradient
A is the cross sectional area

These equations are applied to the whole process or to a part of the hydrologic process. In this type of model structures and parameters are analytic function of physical process. However, though it appears very simple, but actually model is complicated, due to the fact that model involves partial differential equation of linear or non-linear form whose analytical solution is not very easy except in very simple boundary condition. Internal function of aquifer in the form of mathematical relationship is not known. Process parameter are not completely known.

Experimental Analysis:

In the experimental analysis model is determined by measured input and output data of process. With known structure or idea of linearisation, the model is determined by the identification method. In this method, another knowledge of internal function is not required. The complexibility of process does not necessarily influence the identification process. The model so developed may be more accurate than theoretical model.

However the model may be valid for a special process and special operating point. Analytic relationship between the model parameter and physical process parameter can not be obtained.

It is desirable to develop suitable model, so that adequacy of model is proved by available historical data and also can be used for computation in situation other than those expressed by observational data. This can be achieved by appropriate combination of theoretical and experimental analysis.

The mathematical model, which can be developed to meet above requirements, necessiates following steps to be done:-

1. Development of theoretical model, by combing basic equations.
2. Obtain numerical solution to these equation
3. Investigate the applicability of the model by identification.

Mathematical model can be further divided into 2 types

(1) Lumped mathematical model, and

(2) distributed mathematical model.

2.2.2.2.1. <u>Lumped Model</u>:-

Lump model is developed for the entire basin, and this model is applicable for any point within the basin.

Under such conditions all the parameters are lumped together, & mathematically this model can be expressed by ordinary differential equation. This type of model may be based on theoretical analysis, experimental analysis or combination of both as described earlier.

2.2.2.2.1.1 Model based on theoretical analysis:

Lump model which can be developed purely on theoretical analysis is based upon concept of consideration of relationship between hydrologic envrionment and ground water regime. Hydrologic envrionment is dependent upon topography, geology and climate where as ground water regime, comprises two sets of parameters physical and chemical state of ground water. The envrionment and regime are related quantitatively hence ground water regime may be defined(33) as sum of six envrionmental parameters:

$$R = \overline{W}(p) + f(\text{grade } \phi) + q_i + Cw + \Theta + \sum_1^5 \frac{\partial p}{\partial t}$$

Where $\overline{W}(p)$ = is amount of water presented in theoretical zone

$f(\text{grade } \phi)$ = Pattern of ground water flow

q_i = Volume discharge

Cw = Chemical composition of water

Θ = temperature of water

$$\sum_1^5 \frac{\partial p_i}{\partial t} = \frac{\partial \bar{\omega}(p)}{\partial t} + \frac{\partial f}{\partial t}(\text{grade } \phi) + \frac{\partial q_i}{\partial t} + \frac{\partial c_w}{\partial t} + \frac{\partial \phi}{\partial t}$$

The last parameter is indicative of type and intensity of time dependent changes of the regime properties at any point within the flow region characterizing the variability of ground water conditions.

2.2.2.2.1.2. Model based on process analysis: .

In previous section it was advocated to combine theoretical analysis with experimental analysis. In such situation based on physical law model can be developed for particular basins and its applicability may be extended to other similar basins. In such case, if particular basin is considered where chemical state of ground water does not undergo appreciable changes can be described by physical parameters only and for such general equation (34) can be described as

$$\Delta d = f(\ dn,\ L\ )$$

Where Δd = change in water table elevation for the time period,

dn = water table elevation at the end of time interval

and L = a lumps factor combining all inflow and outflow to ground water basin.

The lumps factor can be obtained using water budget equation. On the basis of this model can be developed for other basin as well.

2.2.2.2.1.3. This model gives overall picture of the basin. With the aid of this model, at a time unknown value of one component can be determined easily, using water budget equation. However, in lumps model, total inflow and outflows are balanced, but their locations are not considered. Also many local inflow and outflow relationship may not appear as transfer across the area boundries and thus never appear as budget equation. For such detailed information distributed model is used.

The size of the nodal area depends on the variation in replishment, extraction, transmission, storage and water level data. Where there are large spatial rates of change in water level elevation smaller subdivision is necessary.

The shape of nodal area may be rectangular, square or polygonal shape. Construction of nodal area of polygonal shape is preferable for irregular location of observation wells and boundary condition. Such condition exists in particular basin, which is under study, the polygonal shape is desired.

2.2.2.2.2. Preparation of Nodal net work:

The preparation of nodal net work depends on the choice of number and distribution of nodes, with due consideration of size and boundary. On basis of selection of above the actual construction of nodal net work can be done. Hence discussion on this issue is made prior to actual construction.

2.2.2.2.2.3. Selection of size and Boundary[17]

Size of nodal area besides hydrologic conditions as discussed earlier depends upon boundary condition. The boundary conditions have very real and important consideration in deciding the size of nodal area. The boundary condition may be either geologic boundary or hydrologic boundary. The boundary condition can be further classified as internal boundary or external boundary. The external boundary needs special attention in selection of nodal area.

2.2.2.2.2. Distributed Model[16]

This model is employed for distributed answers, such as locations of discharging wells, their radius of influence, discharge and recharge from stream, major recharge and discharge areas. This type of model divides a particular basin area into small sub-surface that may be rectangular square or some other polygonal shape. The size of zone is dependent on the variation in replenishment, extraction, transmission, storage and water level data. A single well point, known as nodal point is selected to represent the smaller area. The dynamic response of nodal area is represented by water level elevation at this node. In this model, differential equation describing ground water movement is used, whose solution is not simple. For distributed model, vast data is necessary which is a very difficult problem at present stage of development in field of hydrology in our country. In spite of above this model, which is a versatile tool for ground water study is preferable, for carrying detailed study of ground water problem.

2.2.2.2.2.1. Nodal Area :

The development of distributed mathematical model is based upon sub-division of basin in smaller area. These areas are called as nodal area, due to the fact that a single node point represents the water level of small area. The node representing the sub-area is connected hydrologically with its neighbour. It has been assumed that all the extraction and replishments into the nodal area occurs at the nodal point for simplification of model.

The size of the nodal area depends on the variation in replishment, extraction, transmission, storage and water level data. Where there are large spatial rates of change in water level elevation smaller subdivision is necessary.

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1. Geologic boundary:-

Impervious boundary is formed due to presence of clay layer (which generally occurs) or other relatively impervious rock under lying, over lying, or adjacent to an aquifer. External boundary of this type of geologic feature, can be considered as boundary having no flow for modelling purpose.

2. River, Lake or Occen boundary:-

If the large water body is in free or partial contact with the ground water body will not be influenced by the events of ground water basin. Hence for the purpose of ground water studies level can be assumed constant, or same level based on river level fluctuation with time.

3. Arbitrary boundary:

Across this type of boundary ground water flow takes place. It is better to assume impervious boundary where there is low flow. In case of sufficient flow, the sub-surface flow is known and accounted for assigned input to concerned node, and boundary treated as closed. In place of first assumption, another consideration is to treat boundary as impervious and apply image theory. Another possibility is similar as applied to river, lake or open boundary depending upon availability of water level data.

For such case of arbitrary boundary condition, it is desirable to verify and modify with the help of model test.

2.2.2.2.2.4. Number and distribution of Nodes:

Identification of region:- In order to locate the area of large spatial rates of change of water level, for consideration of smaller nodal area in the region following basic map informations are desired.

1. Water level contour
2. Surface topography, geology and limit of aquifer.
3. Lines of equal transmissibility or permeability and storage co-efficient if known.
4. Locations of well with water table measurement.
5. Location of major area of recharge or heavy pumping.

Other considerations, the approximate no. of nodes depend upon complexities in data preparation as well as storage capacity of the available digital computer. With the aid of high speed digital computer IBM 7094, it is possible to solve the mathematical model, even of 7500 nodes but in case of digital model of very low capacity IBM 1620 which is at present available for study. The solution of mathematical model containing 20 nodes is not even assured. R.G. Thomes[12] has made following views for number of nodes." It is recommended that first time this method is used, the number of nodes be only 10 to 15 for entire basin. Experience has shown that this number is easy to handle is good for learning at the same time shows that measure problems to be expected in more detailed net work ".

In view of limitation of storage capacity of the digital computer available for the studies and recommendation made by Thomas regarding the number for study initial attempt, ten nodes can be considered as suitable in present situation. On this decision, nodal net work can be constructed as discussed below.

2.2.2.2.5. Construction of Nodal Net work :

For construction of nodal net work, map showing location of observation well is to obtained and boundary drawn Node point is located at point of well observation having complete data of water level of the period in the area of pumping with above mentioned guide lines. The nodes are connected by lines to form triangles. Selection of node should be done in such a manner no any interior angle of triangle is greater than 90^o. If any triangle so formed is having any of its interior angle greater than 90, another well point is to be selected and procedure repeated till following coditions are satisfied.

(1) No. of node so selected is only 10 and all the nodes have complete data of water level.

(2) No any interior angle is greater than 90^o.

(3) The triangles are more or less symetrical

(4) No any area is too big nor too small.

All the lines connecting the nodes as discussed earlier is perpendicuarly bisected, and bisected line extended to meet other relative bisectors to join at one point to form a corner of polygoner.

product of hydraulic conductivity K and average saturated thickness of grid, as the water level varies with the time so does h and thus T. But in the present study variation is not much, and compared to thickness of the aquifer, the variation may be treated very less. Hence transmissibility T has been assumed constant. It has also been assumed that the properties of the unconfined aquifer do not very in vertical direction.

Based with these conditions, equation can be now expressed.

2.2.2.2.3.1. Mathematical Expression:(17)

1. Equation of continuity : -

$$\nabla \bar{V} = S \frac{\partial h}{\partial t} + Q' \quad (1) \text{ in which } h = m+Z$$

$$m = m_r + m_o$$

2. Equation of motion based on Darcy's law

$$\bar{V} = - P g K/\mu \nabla h$$

The symbols used in these two equations are defined as follows:-

h	=	head	L
Z	=	reference elevation	L
m	=	local thickness of saturated portion of aquifer	L
$\bar{V}$	=	velocity	
s	=	Storage co-efficient	
Q'	=	Volumetric flow rate per unit area	LT^{-1}
P	=	density	ML^{-3}

The polygon thus formed around each node are boundaries of the effective area, controlled by representive node, and is popularly known as Theissen Polygon.

In fact this very model, the distributed model was employed for case study. Hence a nodal net work for the basin was geometrically constructed based on above considerations described under the preceding para. The procedure for construction of nodal net work, which has been actually carried out can easily be followed from map showing net work for the basin under study, vide Fig.2. In this context, it is to submit that for easiness in further discussion as well as requirement for computational work employing digital computer. Consecutive Numbers from 1 to 10 were assigned to each node. Consequtive numbers 11 to 27 were also given to lines joining the nodes. By same number perpendical bisector, now acting as a boundary of polygonal can also be easily denoted.

Length of line joining the two nodes denoted by DL(I) I refersto number assigned to theline) and its perpendicular bisector DJ (I) as well as area A(I) (I refers to number assigned to node) were measured. The measured values of area and length of those lines have been produced in table-5, and table 5.4 respectively. Development of distributed model has to be made for this network of ground water basin.

2.2.2.2.3. <u>Development of Theoretical Model:</u>

The model can be constructed by combining the two equation, equation of continuity and equation of flow based on following assumptions. Transmissibility of aquifer T involved in flow equations needs discussion. Transmissibility is the product of hydraulic conductivity K and average saturated thickness of grid, as the water level varies with the time so does h and thus T. But in the present study variation is not much, and compared to thickness of the aquifer, the variation may be treated very less. Hence transmissibility T has been assumed constant. It has also been assumed that the properties of the unconfined aquifer do not very in vertical direction.

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$\bar{V}$	=	velocity	
s	=	Storage co-efficient	
Q'	=	Volumetric flow rate per unit area	LT^{-1}
P	=	density	ML^{-3}

g	=	acceleration due to gravity	LT^{-2}
K	=	Permeability	L^2
u	=	absolute viscocity	$ML^{-1}T^{-1}$
t	=	time	T

2.2.2.2.3.2. <u>Simplification of Equations</u>(17)

The thickness of aquifer may be assumed very small compared to its. lateral dimension, and so flow can be considered two dimensional. On this approach above two equations can be combined and linearised to yield a single equation that, subject to boundary condition, describes the dynamic of flow in the aquifer. This equation is

$$\nabla T \nabla h - S\frac{\Delta h}{\Delta t} - Q' = 0$$

in which $T = \frac{m \, e \, g \, k}{\mu}$

The quantities T and S are respectively local transmissibility and storage co-efficient of the aquifer. T is product of field permeability (K) and aquifer thickness m.

The source flow rate is time dependent. The flow rate is algebric sum of several components of extraction and replishment. The replishment flow consists of percipitation, stream percolation, artificial recharge due to canal and tube well irrigation, subsurface inflow across boundary. The extraction flow consist. Water pumpage from the aquifer for consumptive use, subsurface outflow across the boundary, effluent flow, loss due to evapotranspiration from forest and water logged area.

Now the evaluation of equation is needed for which necessary discussion is made hereafter.

2.2.2.2.3.3. Solution of Equation:

There are several methods of solving boundary value problems. These methods can be divided into two broad field (1) Analytical solution involving the classical approach of formal mathematics (2) Numerical solution using finite difference approach.

2.2.2.2.3.3.1. Analytical Solution:[18]

Analytical solution of equation is based upon Founer Seres theory and also employs the technique of separation of variable. This method has been generally used in case of relatively uniform aquifer. With simple geometry and have been used primarily for problems of small portion of aquifer or aquifer of small area.

The analytical method is applied to two dimensional flow in the plane, parallel to direction of slope of water. (2) As analytical solution of Richard's equation is not known hence in this method, Laplace equation can be only used. This restriction means, consideration of homogeneous case of permeability of the aquifer which is not possible in actual system. However aquifer may be considered, as geological formation in two or more layers of different permeability. Each geologic unit must be homogenous and isotropic with respect to permeability.

(3) The available methods of solving laplace equation are limited to region of regular shape. Hence the conditions of aquifer is to be suitably idealised. There are many ways to do so, without sacrifying the accuracy of analysis.

This solution of simple and ideal case of aquifer is possible. In case of definite and simple boundary conditions solutions have been made available. Verijugt, Dewest, Todd Bear are among many to describe the solution.

2.2.2.3.3.2. Numerical Solution:

In this method, the three restriction of analytical methods are removed. The true shape of the field may be represented to a very close approximation. By this method general non homogeneous anistropic case can be solved. It is also possible to construct three dimensional models representing ground water basin. This method was adopted which has two components, same are discussed below.

2.2.2.2.3.3.3 Approach of Derivation-Finite Difference:

In numerical solution derivative at any point making up the field and its boundaries are replaced by the ratio of changes in appropriate variables over a small but finite time interval. The objective implied in process of discretization known as finite difference method, is to solve differential equation concerned with sub-surface hydrology. It would appear that finite difference make use of approximation.

$$\frac{d\theta}{dx} = \lim_{\Delta x \to 0} \frac{\Delta \phi}{\Delta x} = \frac{\Delta \phi}{\Delta x}$$

How small x must be for above derivative, to an acceptable approximation depends on the particular problem. This type of approximation is made at finite number of points and reduces a continuous boundary value problem to set of algebric equations. Finite difference method can be conveniently applied subject to limitation ~~at~~ that it should satisfy following condition.

(1) They should be convergent and unconditionally stable i.e. the error due to such approximation, and also the errors are not amplified as computation work marches on.

(2) Solution should be easy to compute.

2.2.2.2.3.4 Derivation of Equation:

Considering the said limitations and compromise with boundary condition the differential equation described earlier is replaced by algebric equation using finite difference method. For this, said equation at first is rewritten in difference equation form.

$$\Sigma (h_I - h_N)\, Y_I = A_I \quad S_I \quad \frac{dh_I}{dt} \quad + A_I\, Q'_I$$

$$\text{in which } Y_{IN} = \frac{DJ_{IN}}{DL_{IN}} \quad x \quad T\, R_I$$

Where

A_I	=	Area associated with node I	- L^2
Y_{IN}	=	Conductence of path between node I & N	- $M^{-1}L^{-1}T^{-1}$
S_I	=	Storage co-efficient of polygon zone associated with node I.	

Q'_I = Volumetric flow rate per unit area at node I $M^{-3}L^{-2}T^{-1}$

TR_I = Transmissibility at mid point between I & N

DL_{IV} = Distance between nodes I and N L

DJ_{IN} = Length of perpendicular bisector associated with node I and N L

The above equation on application of finite difference method comes to

$$\Sigma (h_i^K - h_N^K) Y I,_N = \frac{A_I \; S_I}{t} (h_I^K - h_I^J) + A_I Q_I^K$$

$$Y_{IN} = \frac{DJ_{IN}}{DL_{IN}} x TR_I \text{ and } K = J + \Delta t$$

Where ubscript J denotes point at time co-ordinate at initial and K is time co-ordinate after interal Δt.

In this method time step does not depend on stability criterion. Thus the system of equations need appropriate method of solution.

2.2.2.2.3.5. Method of Solution :

Solution of above system of equations which are now in form of algebric equations can be achieved by two general categories of methods. (1) Direct Method (2) Iteration Method.

2.2.2.2.3.5.1. Direct Method(19,20)

In the direct method, method of ellumination is employed. This method is exemplified by the well known method of Gaussian elementation. In this method, co-efficient matrix related to set of equation is decomposed into a lower and triangular matrix. There after, it is easy to obtain solution dealing both lower triangular and upper triangular matrix separately. The lower triangular matrix is solved by forward substitution. This is accomplished by solving the first equation for unknown vector and value of this is used in second equation to solve next vector where as in the upper triangular matrix it is solved by starting with last equation. This is known as backward substitution.

Except for round off error, they yield exact solution of a system of difference equation using a finite numbers of operations. In practice, the number of equation is usually very large in case of ground water problem. Also matrices for those equation are sparce, i.e. it contain more no. of zero element, solution of which be obtained easily using iterative method. Therefore, normally second method, as described below is adopted.

2.2.2.2.3.5.2. Iteration method (19,20)

This method is ideal forsolution of sparce matrices. This method involves sequence of approximation computed by an iterative method with the sequence hopefully converging to the solution of system of equations. This converging matrices requires that largest value of the iteration matrix be less than unity in absolute value. The smaller the dominant value, the more rapid the convergence. The iteration methods may be applied directly to any linear algebric equations and can be extended to more than two spaces. There are several iteration method. Out of these method, the simplest and useful method is successive, Over relaxation method. This method is considered to be first good iterative method. Relaxation method has been applied in the present study, and so discussion on same method is limited here.

2.2.2.2.3.5.2.1. Relaxation Method:

It is essentially a reiteration or trial and adjustment method. Significant advantage of the method lies on the fact that this method is simple and the relaxation procedure can easily determine answers to above difference equations. It will be better to explain the procedure by way of description of process involved in the specific problem of Varune Basin which is under this study. Prior to this some characteristic of model related to numerical solution is discussed first. The model under this study has been divided into assymetric net works and it is a case of transient flow. In this model, each node is junction for one or more branches and the grid is of polygonal shape. Hence general procedure as adopted in case of steady flow, with rectangular grid represent the basin[18,19] can not be directly applied.

2.2.2.2.4. Operation[17]

For this model, first of all to start with water level at initial value is taken and arbitrary value, of water level at time increment Δt is assigned to all nodes of the basin. On the basis of known value of Y_{IN} and given data of source flow rate with assigned value of water level at initial time solution can be obtained by executing the following computation in serial order.

1. Subsurface flow to each node from all connected nodes are to be determined.,using the first term of difference equation, as described herewith.

$$Q'_I = \Sigma (h_N - h_I) Y_{IN} \cdot \Delta t$$

2. Change in storage due to difference in head during time step Δt is calculated using first term of right hand of the equation.

$$S_I = A_I \times (H_I^K - H_I^I)$$

3. Total source flow is determined, using last term of said equation.

$$DIB_I = A_I \times Q'_I \times \Delta t$$

In case of balanced stage $Q_I + DIB_I - S_I = 0$, but it will not happen till there is difference in assumed level and required level. Naturally, this would give error,called as residue. To minimise this error, water level at the node is to be adjusted by the magnitude of residual value. As stated earlier, the grid is not rectangular, hence proper distribution co-efficient is to be used to determine change in water level at respective node due to the residue. This co-efficient has been called as Relaxation Co-efficient. In order to calculate change of water level, on account of residual which is a flow, the co-efficient is to be multiplied to residual. Hence the co-efficient must be an equivalent impedance of branches joining the node to its neighbours. This may be represented by equation. Relaxation Co-efficient= $\dfrac{1}{Y_{IN} + \dfrac{A_I \ S_I}{\Delta t}}$

4. Hence water level is adjusted by the magnitude of residual, duly multiplied by above relaxation co-efficient By this process, water level of all nodes are adjusted in first operation and the same repeated till sum of residual on all nodes are within presented tolerance limit.

2.2.2.2.5. Computation Device :

In case of entire area of the basin, it is difficult to do iteration work by hand. Hence use of some other device is must. In solving such problems following three types of computation device are in use.

1. Analog computer
2. Hybrid computer
3. Digital computer

2.2.2.2.5.1. Analog computer :

Analog computer consists of an analog model with an excitation response apparatus. Walton has used this type of computer in his extensive work. Tyson and Weber has also employed to determine aquifer character of ground water basin. Regarding this model, discussion has been done in previous chapter. In all the analog computer, the space variables are descretized by replacing the space derivatives by suitable finite difference expression and leaving the time as continuous. The analog computer technique yields very quickly any reasonable solution. However major difficulty arises when it is necessary to simulate non-linear and time varying ground water problem.

Hence the application of analog computer technique is not very practical in case of transient field problems.

2.2.2.2.5.2. Hybrid Computer[21,22]

Hybrid computer system which combines both a digital and analog computer, linked by a matching interface and incorporating the best feature of both, have been developed. This computer combines the accuracy, memory and capacity of the digital computer with the speed of analog. In the solution of complex inverse problem, ninety percent of digital computer running time is utilised in matrix inversion, and considerable amount of storage is occupied for this. Hence under such condition, expensive high speed digital computer with large memories is required for numerical solution. With the development of Hybrid computer this work has become now easier. Discrete space or discrete time hybrid computer represents hybridization of analog digital method.

Hybrid computer uses a passive resistance net work composed of fixed carbon film resistors as a simultaneous equation solver and acts as a matrix inversion, Subroutine in digital computer. The time taken in the operation of matrix inversion is very small due to the fact that net work of passive resistor relaxes almost instantaneously to its steady state.

In this flow chart is first programmed for conventional digital computer. The flow chart is then examined for any loops of resistence network involving matrix inversion subroutine.

Data from the digital computer can be converted through a digital to analog computer and applied to input nodes of analog resistence net work through a distributer. Thus when boundary and input voltage are applied to net work, the out put voltage yield the solution. The output voltage are sampled at the node of analog resistence net work by multiplied and converted and transmitted to the digital computer. Subsequently digital computer reads out these data or modified form and test the convergence and store the solution namely potential at time ($t_o + \nabla t$) at each node in order that these data may be employed as initial condition at the subsequent time. This process repeated till test of convergence is satisfied.

Hence development of hybrid computer has become helpful for quick solution of problem and is more economical and accurate than analog method.

However such type of facility is not available for use in present stage of study. On the other hand, digital computer as described below is readily available so same has been employed forsolution ofproblem at hand.

2.2.2.2.5.3. Digital Computer(23,24)

It is needless to say the importance and utility of digital computer. Rapid advances in numerical methods,increased ability in the programming and faster computing speeds have made the Digital Computer a very useful computational aid in the analysis of various aspects of ground water basin.

The digital computer is versatile and requires no major equipment changes to analyse drastically, different situation Fortunately this machine is also available for use, for problem hand hence the same was used in solution of equation. Initially the digital computers were mechanical computers. With advancement of technology electronic computers are now available which is composed of three major elements, the processer central processing unit (CPU), the memory and the input/output (I/O) device.

(i) The processor performs the control&computational function is generally characterized by its size, computation time,&special capability. The precessor operates in step wise single instruction mode, on vectors containing both data and instruction.
(ii) Memory:- It is second major element of digital computer. Memory is the storage facility for all data instruction and characterized by speed capacity and method of address of the three most common type of memory core, disc and magnitudes tape. Core store proves the fastest cycle time which is fortunately in terms of words which are made up of bits. This has been used here.
(iii) Input/Output:- This element is composed mainly of card readers and line printers. Graphic display technique is also developing which would bring new life in field of hydrology.

For execution of solution, datas and instructions are to be given to computer in the computer language, which is known as FORTRAN (Formula translatic) for which programme is to be written in coding system using FORTRAN SYSTEM.

2.2.5.5.4. Data for Computer:

Prior to discussion, it is necessary to reproduce the difference equation to find out the form and requirement of data for computer operation for its solution.

$$(H_I^K - H_N^K)\, Y_{IN} = \frac{A_I\, S_T}{t}\,(H_I^K - H_I^I) + A_I\, Q_I^K$$

$$Y_{IN} = DJ_I/DL_I \times T_{RI} \text{ and } K = J + \nabla t$$

Hence on reviewing the equation, necessity of following data was observed which are discussed.

1. H. Water level for initial stage for computation of water level for subsequent stage, and also for other stages for identification.

   These datas are observed water level from Jan. 1973 to Dec. 1973 for all the ten nodes, which have been presented in table 5.1.

2. Area A, distance from node to connecting node DJ, and its perpendicular bisector DL.

   These are measured and prepared data after construction of net work. The same is also presented in table 5.6.

3. Storage co-efficient 'S' and Transmissibility 'T' of the aquifer.

This has been determined, on the basis of pump test data, actually carried in field. The value so obtained are also presented. At present value, of S and T considered as same to all node (Chapter IV).

4. Q_I Source flow rate to concerned node.

This is algebric sum of all the flow rates. Against for this, values of all the hydrological components of hydrological cycle were computed, net flow determined. Result is produced in table 5.3.

On basis of all these oonsideration, mathematical model was preferred for present stuudy of problem associated with a particular case of Varuna Basin. For this study both types of model, Lump Mathematical Model as well as distributed mathemetical models were employed, which are discussed in succeeding chapter.

CHAPTER - 3

CHARACTERISTIC OF THE BASIN UNDER STUDY

THE BASIN

The Varuna river, a tributary to the River Ganges flows from west to east in direction and meets the River Ganges near Varanasi. The basin covering an area of 2,57,870 Hectare lies in four districts, Pratapgarh, Allahabad, Jaunpur and Varanasi of eastern part of Uttar Pradesh.

The river Ganges is nearly flowing parallel to Varuna basin keeping close distance to it in entire length exceed near the point of confluence. There is another stream also running at parallel distances, on the northern side of the basin area. The boundary of basin as demaracted is an arbitary boundary, due to the fact that a tributary to Varuna basin has its upper reach out side the area, and possibility of impervious boundary in vast gaustic plane is remote. Secondly there is no any natural boundary as per discussion made earli

There is a branch canal Maahun Branch which just follows along the arbitary boundary on north side upto middle of length of northern boundary and goes straight leaving away this very boundary of the basin. There are systems of canals, taking off from right bank of said branch canal to irrigate narrow belt of northern part of the basin. There are six rainfall stations, names and location of which are available in Fig.1.

3.1 AVAILABLE DATA

Related to above informations following data were available in project report of Varuna GR Water basin[25].

1. Monthly rainfall data of all the six stations complete/incomplete for 10 years upto 1973.

2. Data for average evapotranspiration rate as pan evaporation rate (month-wise).

3. Water Level Data for 67 observation wells for ~~only~~ one particular year of 1973 is only available for entire period.

4. Pump test data for only two observation wells (Well No.81 and Well No.25).

5. The canal system in the basin comprises of 18 channels (distributaries, minors and a branch) excluding the Branch Canal as described earlier but data for canal water supply and running days during month are available for 7 channels including branch canal for the period 1968-1971. Average figures of monthly discharge has been shown in Table 3.1. However design particulars design discharge, bed width and depth of each channels are available in the report.

6. Year-wise data from 1968-1971 regarding State Tube-wells showing no.of wells average discharge and total running hour during Kharif season and Rabi season are available.

DEPTH OF WATER TABLE FROM GROUND IN METRE UNDER DIFFERENT WELLS LYING IN VARUNA BASIN

OBSERVATION LINE 'O' | OBSERVATION LINE 'OP'

Well No.	5	7	8A	8	9A	9	2A	3A	3	4	5	6
Datum / Month	92.75	95.94	97.32	97.48	96.90	96.52	87.48	88.7	89.61	92.08	95.30	92.94
January	1.83	6.28	7.46	6.60	4.35	11.12	13.26	4.40	2.53	5.60	5.58	8.20
February	2.00	6.14	7.60	6.71	4.40	11.20	13.40	4.52	2.64	5.73	5.72	8.32
March	3.10	6.70	7.93	6.91	4.43	11.68	13.90	5.55	3.40	7.15	6.80	8.46
April	2.63	7.01	8.13	6.92	4.56	11.74	19.05	5.65	3.56	7.30	6.91	8.67
May	-	7.20	8.33	7.14	4.82	11.92	14.15	5.78	3.72	7.47	7.08	8.78
June	3.37	8.50	9.17	7.25	5.27	12.42	14.55	7.00	4.50	7.70	8.80	10.65
July	3.35	8.55	9.25	7.35	5.32	12.40	14.64	7.10	4.60	7.73	8.84	10.67
August	2.00	6.80	8.70	6.45	3.20	11.15	13.00	3.50	6.75	4.65	4.35	8.00
September	3.29	7.65	8.17	6.60	5.10	13.10	10.80	1.85	6.55	4.30	4.29	7.90
October	2.10	3.20	4.80	3.10	3.80	7.20	10.84	1.90	0.52	4.35	4.35	7.65
November	2.30	2.30	5.80	3.30	3.81	7.50	11.10	2.20	0.75	4.43	4.46	7.36
December	2.00	2.40	5.90	3.50	3.95	7.70	11.21	2.69	1.15	4.30	4.35	7.97

CONTD. A

TABLE - 3.1

OBSERVED DEPTH OF WATER TABLE FROM GROUND IN METRE UNDER DIFFERENT WELLS LYING IN VARUNA BASIN

	OBSERVATION LINE 'VA'						OBSERVATION LINE 'P'					
	2	3	4	5	3A	3	4A	4	5A	5	6B	6A
Datum / Month	85.21	83.71	82.85	82.19	85.81	84.37	85.09	84.43	86.46	86.31	86.15	84.94
January	9.67	8.25	9.42	10.33	7.10	5.76	6.24	6.59	5.74	4.55	3.83	6.69
February	9.81	8.41	9.55	10.45	7.25	5.91	6.36	6.12	5.91	5.70	5.80	6.80
March	11.05	8.75	10.65	10.95	7.40	6.65	6.95	7.25	6.65	6.15	6.50	7.00
April	11.26	8.95	10.83	11.95	7.48	6.82	7.06	7.42	6.80	6.27	6.71	7.15
May	11.41	9.22	11.02	12.32	7.70	7.08	7.20	7.62	7.06	6.41	7.01	7.34
June	11.62	9.60	11.22	12.40	7.89	7.23	7.33	7.78	7.22	5.56	7.15	7.50
July	11.72	9.67	11.00	12.48	8.30	7.58	7.40	8.00	7.80	7.85	7.75	8.70
August	9.00	7.90	8.80	10.35	5.10	4.40	5.55	6.50	5.10	4.15	4.70	4.30
Sept.	9.74	7.55	7.75	8.32	3.30	3.30	4.90	5.70	4.15	3.70	3.35	4.26
October	9.71	7.50	7.82	8.29	3.27	3.26	4.28	5.64	4.10	3.65	3.31	4.68
November	9.84	7.60	7.99	8.26	3.48	3.43	5.03	5.75	4.26	3.88	3.50	4.86
December	9.96	7.82	8.11	8.38	4.35	7.00	5.90	5.90	5.00	3.85	5.30	5.05

CONTD. B

TABLE - 3.1

OBSERVED DEPTH OF WATER TABLE FROM GROUND IN METRE UNDER DIFFERENT WELLS LYING IN VARUNA BASIN

	OBSERVATION LINE 'P'			OBSERVATION LINE 'VB'				OBSERVATION LINE 'VC'				
Well No.	7B	7A	7	2	3	4	5	3	4	5	6	7
Datum / Month	85.23	80.95	86.42	79.31	79.50	80.36	81.41	83.63	82.42	81.95	81.75	81.70
January	8.36	3.31	8.73	12.84	10.14	2.88	11.92	13.99	13.63	2.78	10.22	10.30
February	8.85	3.95	8.85	12.96	10.34	3.05	12.08	14.07	13.80	2.87	10.35	10.42
March	9.10	4.25	9.35	14.20	10.85	3.35	13.90	16.50	13.15	3.70	12.70	10.85
April	9.33	4.37	9.52	14.27	10.94	3.36	14.01	17.20	14.05	4.20	13.50	11.80
May	9.55	4.56	9.73	14.67	11.30	4.17	14.27	18.70	15.20	5.20	11.80	10.90
June	9.60	9.77	9.91	14.70	11.35	4.23	14.39	19.10	15.35	5.15	11.83	10.85
July	9.65	7.00	9.50	14.68	11.20	4.00	14.21	18.80	15.20	5.08	11.00	10.90
August	7.60	3.60	8.00	11.30	9.28	3.48	11.38	14.20	11.72	3.15	8.90	7.98
Sept.	8.10	3.24	8.35	11.60	9.00	2.50	10.90	13.50	11.50	2.85	9.05	7.70
Oct.	8.05	3.21	8.50	11.89	8.95	2.42	10.87	13.48	11.43	2.76	9.20	7.80
November	8.20	3.37	5.67	12.30	8.15	2.83	11.23	14.10	11.89	3.15	10.30	8.18
December	7.90	3.70	9.10	13.30	10.58	2.85	11.45	15.20	11.70	2.45	10.70	7.80

CONTD. C

TABLE - 3.1

OBSERVED DEPTH OF WATER TABLE FROM GROUND IN METRE UNDER DIFFERENT WELLS LYING IN VARUNA BASIN

	OBSERVATION LINE VC				OBSERVATION LINE 'VD'					OBSERVATION LINE 'BQ'		
Well No.	8	9	4	5	6	7	8	9	3	4	5B	5A
Datum / Month	87.13	84.43	83.33	81.64	80.51	79.82	84.18	78.89	80.40	79.34	77.39	77.49
January	8.92	7.18	7.45	13.45	7.25	12.25	10.25	12.05	5.33	12.57	3.65	5.45
February	9.05	1.26	7.67	13.58	7.36	12.39	10.57	12.17	5.45	12.69	3.60	5.57
March	9.15	7.50	8.10	13.95	7.85	12.90	11.02	12.85	5.45	13.00	4.15	5.25
April	9.90	8.10	8.40	14.75	8.70	11.50	11.45	12.60	5.90	13.50	4.90	5.75
May	11.00	8.90	9.22	15.05	9.15	12.23	11.95	13.27	6.43	13.92	5.35	6.00
June	11.05	9.05	9.35	15.00	9.23	12.45	12.12	13.40	7.10	14.12	5.38	6.23
July	11.10	8.80	9.25	14.90	9.10	12.40	12.00	13.28	7.05	14.05	5.30	6.26
August	8.20	6.31	6.55	13.05	6.32	9.35	10.31	10.98	4.89	10.42	3.40	5.23
Sept.	8.02	6.32	6.35	11.60	6.03	9.48	10.35	10.78	3.30	9.80	1.80	4.67
Oct.	8.03	6.31	6.32	11.52	6.03	9.52	10.40	10.73	3.22	9.73	1.75	4.63
November	9.22	7.86	7.49	12.77	7.42	10.52	11.73	11.80	4.73	10.55	3.18	5.30
December	8.96	7.80	6.65	13.00	7.10	10.85	8.00	11.73	3.10	9.30	2.15	4.90

END

TABLE - 3.2

TABLE SHOWING AVERAGE MONTHLY* RELEASE OF WATER IN CUSEC-MONTH AND RUNNING DAYS OF GIVEN CHANNEL

…IAHUN BRANCH		PHULPUR DISTRY		NARAINPUR MINOR		PALI MINOR		KUWARPUR DISTRY		JHANJHARPUR MINOR		TILORA MINOR	
…lease	Running Days	Release	Running days	Release	Running days	Release	Running days	Release	Running days	Release	Running days	Release	Running days
4	15	153	5	31	5	180	5	116	5	7.5	2	NA	NA
5	12	258	8	13.5	3	5.5	3	103	6	5	5	1	1
3	17	115	8	19	5	7.2	4	28	2	1.5	1	3	5
2	5	93	3	1	1	1	1	38	4	1.0	1.0	-	-
1	16	256	9	44	8	20	7	65.4	13	5	2	NA	2
7	23	372	13	64	12	18	7	6	4	3	4	1.3	1
5	20	275	9	58	6	20	7	140	14	5	5	1	1
9	22	408	13	14	5	7	4	54	8	3	2	2	1
2	9	282	9	2.7	1	8.6	2	74	8	3	1	-	-
[illegible]	22	393	11	16	2	11	6	5	3	-	-	1.8	1
[illegible]	21	412	10	61.3	9	23.3	8	80	6	1	1	4	7
[illegible]	26	49.3	16	89	14	66	16	55	2	-	-	2.0	3.0
0.0		43.0		6.0		5.8		21.0		8.0		1.8	

*This is based upon available data of release and running days for the years 1958-1971

TABLE - 3.3

AVAILABLE DATA OF PUMP TEST OF WELL NO!81 AND 25 LYING IN BASIN

Well No.81 Spring Level 29 FT 5.5 inch Well No.25 Spring Level 12.22

Distance of conservation well = 150 Ft. Distance of Observation well = 147 ft.

Discharge of well in Gallons per day = 45,000 45,000

Time in Minutes	Water level in Feet	- inches	Time in minutes	Water level in Feet	- Inches	Time in Minutes	Water level in Metres
1.	29	5.6	40	31	6.50	1.0	12.230
2.	30	1.0	50	31	7.00	2.5	12.240
3.	30	4.0	60	31	8.00	8.0	12.245
4.	30	5.5	70	31	8.50	21.0	12.255
5.	30	6.75	80	31	9.00	32.0	12.270
6.	30	8.00	100	31	9.50	40.0	12.285
7.	30	9.00	120	31	10.5	50.0	12.295
8.	30	9.75	140	31	10.5	60.0	12.310
9.	30	10.25	180	31	11.75	80.0	12.345
10.	30	11.00	220	32	00	100.0	12.385
12.	31	0.00	260	32	0.25	120.0	12.425
14	31	1.25	320	32	0.25	150.0	12.475
16	31	2.00	420	32	0.25	180.0	12.530
18	31	2.50	-	-		210.0	12.580
20	31	3.25	-	-		245.0	12.610
25	31	4.25	-	-		300.0	12.675
30	31	5.25	-	-		320.0	12.690
35	31	6.50				346.0	12.710

*Time in minute since pumping started.

7. Regarding private tube wells, pumping sets and dug wells only no.of wells under use in different year 1968-1971 are available.

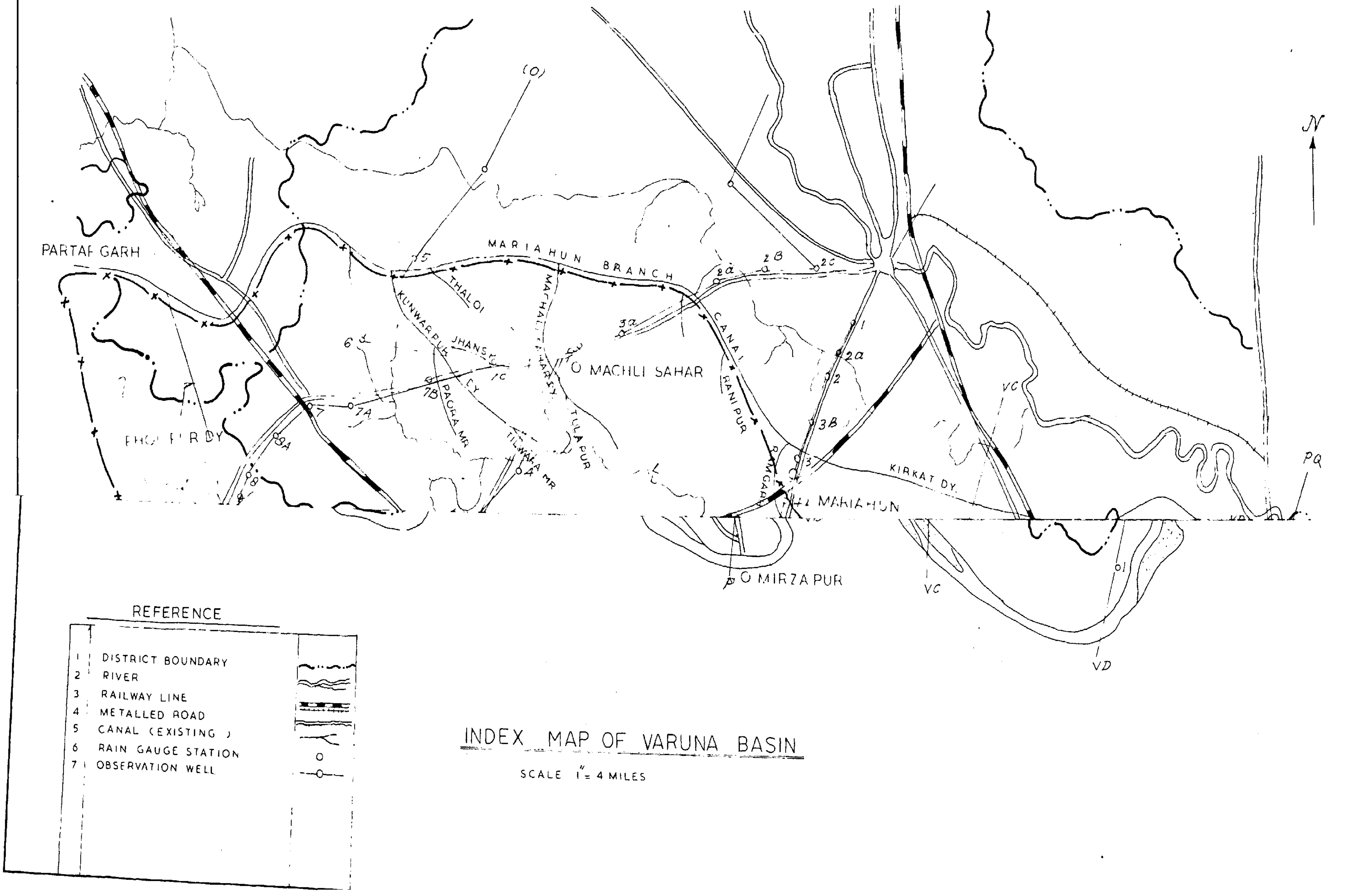
8. Following information are available in the map which can also be seen in Fig.1.

   (i) In the other map locations of 54, private tube wells and 8 pumping sets have been shown, with their discharge.

   (ii) Streams joining the Varuna River have also been shown.

   (iii) Location of Rain-gauge stations has been shown.

3.2 The success of development of model depends on the accuracy of value of components of aquifer characteristic. It can be seen from the information described earlier that water level data, details of drawl through different types of well, pump test data are in sufficient, however an attempt has been made to prepare required data for the model on this stage. The water level data is the key point for the development of model. Unfortunately data of all observation wells are available for the year 1973. Hence there was no alternative left but to select same year for present study. Accordingly data of other components required for the model were prepared, as per following discussions.



INDEX MAP OF VARUNA BASIN

SCALE 1" = 4 MILES

3.3 AQUIFER CHARACTERISTIC

In this Chapter determination of aquifer characteristics which play most vital role in hydrologic cycle of ground water movement has been dealt in-dependently where as in next Chapters all other parameters have been described, alongwith application of models. Reliability of model represented by finite differences equation depends largely on the accuracy of numerical values of ~~determine-value-of~~ the formation factors of ground water basin. There are several methods to determine value of formation factors, which can be classified under three methods. Laboratory methods (Direct and Indirect), Field methods (velocity and Potential method, Discharging well (Pumping test) method. Out of these methods, pumping test method may be considered as most useful method. For this very purpose, data of only two pumping tests are available. With the available data of pump test, aquifer characteristics were to be determined. It would appear from the equation of mathematical model that only two formation factors of aquifer, transmissibility and storage co-efficient are involved in the equation. Prior to discussion of methods employed for determination of value of these two factors, these factors are first defined.

3.3.1. Storage Co-Efficient: The storage co-efficient may be defined as volume of water released or stored per unit surface area of the aquifer per unit change in the component of head, normal to that surface. This formation factor is generally designated by S and are dimensionless. The volume of water stored in pore spaces of soil, if expressed as a percentage of total volume of soil, is equivalent to porosity of rock. Not all the water stored in pore spaces can be released by gravity due to pumping but a part of it is retained by inter-molecular and surface tension force. In case of unconfined aquifer this term is named as specific yield which is actually effective porosity. In books both name are used for unconfined aquifer.

3.3.2. Transmissibility:

Transmissibility indicates capacity of an aquifer to transmit water through its entire thickness and is product of the average hydraulic conductivity (or permeability) and thickness of aquifer. Transmissibility is defined as rate of flow under a hydraulic gradient equal to unity through a cross section of unit width over the whole thickness of aquifer. It is generally designated by symbol T. It has dimension of $Length^3/ L \times T$ or simply $Length^2/Time$ which may be expressed as m^2/T. Transmissibility has been defined by Theis as number of gallons of water which will move in one day through vertical strip of aquifer one feet wide and having the height of aquifer when the hydraulic gradient is unity. Based on this definition T is also expressed as gal/day/ft. As in pumping test unit of length has been taken in feet as well as metre and pumping rate in gallon per hour. Hence both the dimensions have been freely used, where-ever necessary in this study.

3.4 METHODS OF ESTIMATION

The field values of these two formation factors, can be determined based on Non-equilibrium theory developed by these or other investigators. However in this Chapter discussion is limited to methods described by Theiss, Jacob, Chow and Boultan, which are applicable to unconfined aquifer. Analog method based on water table data may be treated best method, but need sufficient time and money and so could not be adopted in present study. The discussion is here limited to pump test method only.

3.4.1. Theis Method[26]

Theis actually developed the method for determination of formation factors of confined aquifer. However Mienzer[23]m abd Jacob[26] have recommended the application for determination of these constants of unconfined aquifer. Prior to Theis theory, the mathematical theory of gr.water hydraulics was based upon a postulate that equilibrium has been attained and therefore the water levels are no longer falling such condition rarely arises. As the previous theories were not strictly applicable, in unequilibrium state of water level. Theis made an investigation and developed equation as discussed below.

3.4.1.1. Development of Formation:

Theis developed an equation for non-equilibrium condition of ground water flow on basis of equation of conduction of heat in solid for the temperature at any point in an infinite plane with initial temperature zero at any time, after observing analogy between these two flows. The equation of conductive can be described as

$$v = \frac{F}{4\pi\ tk}\ e^{-\frac{x+y}{4\ kt}}$$

Where v = change in temperature at the point x,y at time t,

V = the strength of source or sink, ϕ is amount of heat per unit volume;

K = Kelvin constant of diffisuvity which is equal to co-efficient of conductivity devided by the specific heat per unit volume, and t the time. The effect of continuous source a sink of constant strength is derived from equation (1).

$$F = \phi\ (I')\ dt'$$

then
$$v_{x,y.t} = \int_0^t \frac{\phi(t')}{4\pi k(t-t')}\ e^{\frac{-(x^2+y^2)}{4K(t-t')}}\ dt'$$

Let $\phi(t') = \lambda$ a constant

$$\underset{(t)}{v} = \frac{\lambda}{4\ \pi\ k} \int_0^t \frac{e^{\frac{-(x^2+y^2)}{4K\phi t-t')}}}{(\ t\ -\ t')}\ dt'$$

Let $u = \frac{x^2\ +\ y^2}{4\ K(t-t')}$

$$v_t = \frac{\lambda}{4\ \pi\ K} \int_{\frac{x^2+y^2}{4Kt}}^{\infty} \frac{e^{-u}}{t-t'} \cdot \frac{x^2+y^2}{4\ K} \cdot \frac{du}{u^2}$$

$$= \frac{\lambda}{4\pi K} \int_{\frac{x^2+y^2}{4Kt}}^{\infty} e^{-u}\ du$$

$$= \text{simply } \frac{\lambda}{4\pi k} \int_{u}^{\infty} e^{-u}$$

The definite integral which is uniform of exponential integral can be expanded in following equation:

$$W(u) = \int_{u}^{\infty} e^{-u}/u \, du = -0.577216 - \log_e u + u + \frac{u^2}{2.2!} + \frac{u^3}{3.3!}$$

Where e is base of natural logarithm.

This equation can be applied to ground water hydraulics due to analogy between following components of gr.water and heat problems.

Diffisuivity K is analogous to transmissivity of aquifer divided by specific yield. Where as F continuous strength of sink is analogous to pumping rate divided by the specific yield, and change in temperature is analogous to drawdowns in ft. at any point in the vicinity of the well pumped at uniform rate. Taking these substitution, equation for gr.water flow can be written, using the term Q; for pumping rates in gall/min.

$$s = \frac{Q}{4\pi T} \int_{\frac{r^2 s}{4Tt}}^{\infty} e^{-u} \, du$$

in which argument u can be expressed as u =

In the system of conventional unit as discussed earlier, these two equations can be written as :

$$s = 114.6\, Q \int_{\frac{1.87 r^2 s}{Tt}}^{\infty} e^{-u}/u \cdot du = 114.6\, Q\, W(u)$$

$$u = \frac{1.87\, r^2 s}{Tt}$$

Where r is distance of observation of well from the pumped well in ft. s = co-efficient of storage as a ratio of fraction, and t = time since pumping began in days. W(u) is, well function of u, whose value can be determined by evaluation of expansion given in equation (2).

3.4.1.2. Basis of Determination of Formation Constant;

The co-efficient of transmissibility and storage can not be determined directly because T occurs both in the argument of function and as a divider of the integral. However Theis developed graphical method that makes possible to obtain a solution, on following approaches. Taking log on both side for equation (3) and (4) we get

$$\text{Log}s = \text{Log}\left[\frac{114.6}{T} Q\right] + \text{Log}\, w(u)$$ and rearranging to equation (4) $\text{Log}\,\frac{r^2}{t} = \text{Log}\left[\frac{T}{1.87\, s}\right] + \text{Log}\, u$ The value of members under bracket is constant which shows that Logs differs with Log w(u) only by constant, in the same manner as Log $r^2/1$ with Log u have only difference in the value of constant members. Hence the similarity between equations gave scope to detérmine the value of unknown by way of graphical methods of superposition. Details of further work such as plotting of curves, and determination of value of co-efficient can be well described by actual procedure adopted for determination of above factors in case of ground water basin under study.

3.4.1.3. Procedure Followed:

Mienzer[27] has simply recommended, the use of Theis method for unconfined aquifer, but Jacob[28] has suggested to use the method after replacing the term of drawdown ~~and down~~ by $s' = s - s^2/2D$ where D is thickness of aquifer. Weznel gave tabulation of exponential integral written symbolically w(u) for values of u from 10^{-15} to 9.9, by evaluating the series of expansion ~~and~~ as described in eqn.(2). A condensed version of his table is available in standard books of Ground Water Hydrology. These values are useful in plotting the desired curves.

For the present study, pump test data for only 2 wells Well No.81 and Well No.25 of basin area was made available to determine aquifer characteristic, which is not sufficient for entire basin. However, due to lack of data the formation factors were assumed as uniform through out the basin, and values determined by carrying out following works in steps.

3.4.1.3.1. Graphical Work:

First of all, values of w(u) vrs u on logarithmic paper having w(u) on y and u on x - axis were plotted, with the help of values of well function w(u) for u given in table 4.1 of book by Todd. The Logarithm was used for convenience in plotting of variables having wide range of values. Next value of drawdown with certain modification ($s-s^2/2D$ where d depth of well) on y axis against the value of r^2/t on the x axis of similar logarthim paper of equal size,was plotted.

The observed data curve was superimposed on the type curve (plot for w(u) vrs u), with the co-ordinate axis of both graphs held parallel, the field data was moved either parallel to vertical axis or horizontal axis. Until a position was found where most of the plotted points of well observation overlie on a segment of type curve as shown in Figure 3. An arbitory match point was selected in the region where two points overlapped. The pair of co-ordinate values for this point was recorded from each of two plots. These values were substituted on equations and values of transmissibility and storage co-efficient was computed as described below.

3.4.1.3.2. Computation Work

For Well No.81: Discharge per minute Q = 750 gall/minute

Distance of observation well from well r=150 f

Co-ordinates of matching point = $r^2/t = 4.63 \times 10^{+6}$ ft^2/day

on observed data curve $s = 1.274$ ft.

Coordinate of matching point on type curve = $u = 5 \times 10^{-2}$ and

$w(u) = 2.47$

For the transmissibility equation (5) was used

$$s = \frac{114.6}{T} Q \cdot w(u) \text{ or } T = 114.6\, Q\, w(u).$$

On substitution of the values in Eqn.

$$\text{Transmissibility } T = \frac{114.6 \text{x} 750 \text{x} 247}{1.274} = 167000 \text{ gall/day/}$$

For storage co-efficient equation (6) was used

$$u = \frac{1.87\, r^2 S}{T\, t} \quad \text{or} \quad S = \frac{T \times t \times u}{1.87 \times r^2}$$

Putting the values $S = \dfrac{5 \times 10^{-2} \times 167000}{1.87 \times 4.63 \times 10^{6}} = 9.68 \times 10^{-4}$

For Well No. 25

Discharge rate of pump Q = 750 gall/minute

Distance of observation well from well r = 147

Coordinates of matching point r^2/t = 2.16×10^5 ft^2/day

On observation data curve s = 0.83 ft.

On type curve u = 4×10^{-1} and w(u) = 0.70

Hence Transmissibility $T = \dfrac{114.6 \times 750 \times 0.70}{0.85} = 72500$

Storage co-efficient = 7.20×10^{-2}

3.4.2. Jacob Method[29]

The Theis method of superposition offers a very ingeneous solution to the mathematical difficulties described previously. However when the field data curve is flat which generally occurs, fitting of curve with type curve by trial would be not exact and this will cause errors due to personal judgement. Jacob, in order to obviate the curve fitting work, developed an approximate method, straight line method. Jacob has stated that this method, where as interpretation. The method is applicable in particular condition, hence this method can be treated as supplimentary method to Theis method.

3.4.2.1. Theory

Jacob has given following basis for adopting the method of approximation,in order to avoid curve fitting work.

'For small values of r^2/t compared to the value of $(4T/s)$, u will be so small that the series following two terms may be neglected'. On this approach Theis equation can be simplified as follows.

Theis equation $s = \frac{Q}{4 \pi T} W(u) = \frac{Q}{4 \pi T} \; -.5772 - \log_e u +$

$$u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!}$$

reduces to $s = \frac{Q}{4 \pi T} \; \log_e \frac{1}{u} - .5772$ on putting $u = \frac{4 Tk}{r^2 s}$

We get $s = \frac{Q}{4 \pi T} \log_e \; 4e^{-.5772} \, . \frac{T t}{r^2 s} =$

$$\frac{Q}{4 \pi T} \log_e \frac{2.25 \; T \; t}{r^2 \; s}$$

Jacob's tolerance for the application is based on limiting value of u, which should be less than 0.02. The above equation after conversion on normal logarthim can be written in three different forms:

$$s = \frac{-2.303 \; Q}{2 \pi T} \left[\mathrm{Log}_{10} \, r - 1/2 \log_{10} \left(\frac{2.25 \; Tt}{s} \right) \right] \quad .....(7)$$

$$s = + \frac{2.303 \; Q}{4 \pi T} \left[\mathrm{Log}_{10} \, t - \log_{10} \frac{r^2 \; s}{2.25 \; T} \right] \quad .....(8)$$

$$s = - \frac{2.303 \; Q}{4 \pi t} \left[\mathrm{Log}_{10} \, (r^2/t) - \mathrm{Log}_{10} \left(2.25 \, \frac{T}{s} \right) \right] \quad .....(9)$$

The only variable in these equations are drawdown s, the distance r, and time t. In case of constant time t, plot of S vrs $\log_{10}$ r will be straight line as per eqn.(7). When distance of observation well is kept constant, on the basis of Equation 8, plot of S vs $\log_{10}$ t will represent straight line. If r and t is combined with single variable, Equation 9 will be equation of straight line, with the plot of s against $\log_{10}$ r/t.

3.4.2.2. Approach for Solution

As all the three equations are in form of linear equation, with last term as constant, the slope of the corresponding line is represented by the quantity on out side of bracket, and the intercept of the straight line on the zero drawdown line is represented by the second term within the bracket. Hence with the help of value of slope of plot, the value of T can be determined and subsequently value of storage co-efficient (rather specific yield) can be known as per procedure described below.

3.4.2.3. Simplification of Equation

THE Equation is re-written for further simplification needed for computation work, prior to computation work.

$S = \frac{2.30\ Q}{4\ T} \log_{10} \frac{2.25\ T}{r^2\ s} + \frac{2.30\ Q}{2\ T} \log t$ (8A). In light of this equation value of t can be selected one logarthmic cycle, so that $\log_{10}\ t_2/t_1 = 1$. For this value of s, differ in drawdown between time t_2 and t_1, can be noted under his condition the equation could be written as :

$$s = \frac{2.30\ Q}{4\ \pi\ T} \cdot \log \frac{t_2}{t_1} = \frac{2.30\ Q}{4\ \pi\ T} \text{ or } T = \frac{2.30\ Q}{4\ \pi\ T} \quad .....(10)$$

Storage co-efficient (s) can be determined from intercept (Equation - 8A) to of straight line on the log t axis. Drawdown s is zero only when

$$\frac{2.25\ T\ t_o}{r^2 s} = 1 \text{ or } s = \frac{2.25\ T\ t_o}{r^2} \quad .....11(a),11(b$$

After converting above two equations in conventional unit the equation can be described as $T = \frac{2.64\ Q}{s}$ and

$$s = 0.3\ T\ t_o/r^2 \quad ...12(a),12(b)$$

3.4.2.4. Method of Solution:

The determination of formation factors based on above two equations 12(a), 12(b) needs following steps to be carried on.

3.4.2.4.1. Graphical Work

Using data of observed drawdown, with respect to variation of time, Field data curve was plotted on semi log paper utilising Y axis, on arithmatic scale, for drawdown and x axis on logarthmic scale for time t in days. It is to mention here that plot of data of observation well no.81 is not following straight line, where as in case of well no.25 the condition is just reversed. Such type of situation may occur due to gradual changing in formation constant due to pumping, as per view of Chow.

3.4.2.4.2. Computational Work

For Well No.81

From Graph:- s difference in drawdown per cycle = 1.12 and

$t_o = 3.5 \times 10^{-4}$ day.

$$\text{Hence } T = \frac{2.64 \times 750}{1.12} = 176800 \text{ gall/day/ft.}$$

$$S = \frac{0.3 \times T \times t_o}{\cancel{216000}\, r^2} = \frac{0.3 \text{x} 176800 \text{x} 3.5 \text{x} 10^{-4}}{2.2500} = 8.35 \text{x} 10^{4}$$

For Well No.25

From graph: s difference in drawdown per cycle = 1.90 and

$t_o = 3.9 \times 10^{-2}$

$$\text{Hence } T = \frac{2.64 \times 750}{1.99} = 99000 \text{ gall/day/ft.}$$

$$S = \frac{0.3 \times 99{,}000 \times 3.9 \times 10^{-2}}{216000} = 5.42 \times 10^{-2}$$

3.4.3. Chow Method[31]

Chow has basically modified the method of Jacob and extended the limit of application of striaght line ~~fe~~ method for determination of value of storage co-efficient and Transmissibility Use of Jacob method, the method of approximation, gives an error of hardly one percent in the value of Transmissibility T, and four percent error in the value of storage co-efficient, in case of 4 less than 0.01. Procedure described by Chow, based on tolerable error of five percent, for which condition laid down by him that u should be less than 0.05. Chow has also viewed that matching of observed curve to any portion of type curve is not possible due to following facts. The view of Chow in this context is noted.

'Experience has shown that formation constants are gradually changing during the time of pumping. This phenomenon becomes evident particularly in case of severe pumping or at the early stage of the pumping test, under which condition the sudden release in pore water pressure would cause compression in formation that affects the realizable value of constant.

3.4.3.1. Theory

Chow has described in his paper two approaches for determination of formation factors. Development of an independent equation using the variable of said equation on basis of which these values can be known on the basis of plot of data using the variable of said equation (11). Determination of error involved in use of method of approximation for any problem, in order to modify the value accordingly. These two approaches avoid curve fitting work, and remove the restriction imposed by Jacob for r and t. Chow has developed the equation as discussed below, to find out apparent value of formation constant at any instant as pumping goes on.

3.4.3.2. Development of Equation

From equation (6), taking log on either side

$$\log_e u = \log_e 1.87 \frac{r^2 s}{T t^2} - \log_e t$$

Differenting u, log u, w(u) and s of equation of (6), (10) (5) with respect to $\log_e t$, the resulting equations are :-

$$\frac{u}{\log et} = \frac{1.87\ r^2\ s}{T\ t^2} \cdot \frac{dt}{d\ \log\ et} = \frac{-1.87\ r^2 s}{T\ t} \times \left(\frac{1}{t}\ dt\right) \times$$

$$\frac{1}{d\ \log_e t} = -u \quad ..(1$$

$$\frac{\log_e u}{\log_e t} = 1 \qquad .....(12)$$

$$\frac{W(u)}{\log_e t} = \frac{-\ \log_e u}{\log_e t} + \frac{u}{\log_e t}\ t \left(1 - \frac{u}{21} + \frac{u^2}{31}\right)$$ on putting the value from (10) & (11)

$$= 1 - u \left(1 - \frac{u}{21} + \frac{u^2}{31}\right)$$

$$= 1 - u + \frac{u^2}{21} - \frac{u^3}{31} = e^{-4} \qquad .....(12)$$

$$\frac{s}{\log_e t} = \frac{114.6\ Q}{T}\ \frac{W(u)}{\log_e t} = \frac{114.6\ Q}{T} \times e^{-4} \qquad .....(13)$$

Converting the natural logarthim to common logarthim the equation (13) becomes

$$\frac{s}{\log_{10} t} = \frac{264\ Q}{T}\ e^{-4} \qquad .....(14)$$

dividing (5) by (14)

$$s\ /\ (\ s/\ \log_{10} t) = \frac{W\ (u)\ e^4}{2.3} \qquad .....(15)$$

Let

$$F(u) = s/\ (\ s/\ \log t\ ) \qquad .....(16)$$

Then

$$F(u) = W\ (u)\ e^4\ /\ 2.3 \qquad .....(17)$$

Therefore, for a given value of u, the value of W(u)

Therefore, for a given value of u, the value of w(u) and w(u) can be computed by equation (2) and (17) respectively. Hence on this approach, the relation between F(u), w(u) and u was plotted by Chow as shown in Fig.(1) and (2) of his paper. Thus when F(u) is known from equation (17) the value of w(u) and u can be found from those figures, which are also available in Todd book. Procedure as described by Chow will be better explained by actual work done for the same, which is described after discussion on the approach for determination of error involved in approximation method.

3.4.3.3. Simplification of Equation

Determination of error on approximate method is based upon following :

Theory:

When u is very small, that is less than 0.01 e^u approaches to unity then equation (17) becomes

$$F(u) = \frac{w(u)}{2.3} \qquad .....(18)$$

and thus $$T = \frac{264\ Q\ F(u)}{s} = \frac{264\ Q}{\partial s/\partial \log_{10} t} \qquad .....(19)$$

In case of consideration of one log cycle $\partial s/\partial \log_{10} t$, the equation turn to Jacob equation (17) and further replacing w(u) by equation (17) in equation (2) ignoring all terms beyond second we may get

$$u = e^{-0.5772 - 2.3\ F(u)}$$

Putting the value of u in this equation (6)

$$s = T\ t\ e^{-.5772 - 2.3\ F(u)/1.87\ r^2} \qquad .....(20)$$

When drawdown is zero, w(u) = 0, and so F(u) = 0, and intercept with zero drawdown axis, the equation turns to Jacob equation for s, it can be further proved that the relation between t and t_o

$$\frac{t_o}{t} = e^{-2.3\ F(u)}$$

and thus the relation between approx. T and T, as well as relation between approx. s by approximation method, and s by Chow method can be found out.

$$\frac{T_{approx}}{T} = \frac{264\ Q\ F(u)/s}{114.6\ Q\ W(u)/s} = \frac{264}{114.6 x 2.3} \times \frac{W(u)e^{u}}{W(u)} = e^{u}$$

Similarly

$$\frac{S_{approx.}}{S} = \frac{T_{approx}}{T} \times \frac{t}{t} \times e_u \frac{-.5772-2.3\ F(u)/1.87\ r^2}{u\ /1.87\ r^2}$$

$$= \frac{e^4\ .\ e^{-.5772\ -\ 2.3\ F(u)}}{4}$$

On the basis of above two equations error can be known as described below. The over estimated error in T = $e^4 - 1$

The under estimated error in S = $1 - 0.561\ e^{4-2.3\ F(u)}$

3.4.3.4 Method of Solution

For convenience Chow has given a graph, from which it is easier to determine percentage of error, when values F(u) is known. The same graph was used in this study for computation of error in order to rectify the value of transmissibility and storage co-efficient, as determined earlier by Jacob method. Similar to other two methods, this work also needs graphical and computation work to obtaine desired value.

3.4.3.4.1. Graphical Work

For doing graphical work, same procedure was described by Chow. Hence in the present study same graph was used. Only an arbitary point was selected, and coordinate values of t and s noted.

3.4.3.4.2. Computation Work

(Well No.81) The coordinates of point from graph as noted are $t = 1 \times 10^{-2}$ days and $s = 1.62$ ft

$\partial s/\partial$ loget per cycle = $s = 1.12$ ft.

Hence $F(u) = 1.62/1.12 = 1.445$

From graph value of $W(u) = 3.25$ and $u = .023$

$$\text{Then } T = \frac{114.6 \times 750 \times 3.25}{1.62} = 172500 \text{ gall/day/ft.}$$

$$s = \frac{0.023 \times 172500 \times 1 \times 10^{-2}}{1.87 \times 22500} = 9.22 \times 10^{-4}$$

Method (B): The value of approx. T and s will be same as computed in Jacob method, which may be used here.

$T_{approx.} = 176800$ gall/day/ft, and storage co-efficient

$s = 8.35 \times 10^{-4}$

From graph (4) of Paper Chow

Over estimated error in T = 2.5 percent

Under estimated in error in s = 8 percent

Hence corrected value of $T = \frac{176800}{1.025} = 172500$

and corrected value of $s = 8.35 \times 1.08 \times 10^{-4} = 9.05 \times 10^{-4}$

For Well No.25

The co-ordinates of point from observed data graph noted $t = 1.7 \times 10^{-1}$ days, $s = 1.26$ ft

$\partial s/\partial \log_e t$ per cycle $= \quad s = 1.99$ and

Hence $F(u) = s/ \quad s = \frac{1.264}{1.99} = 0.636$

From graph value of $W(u) = 1.15$ and $u = 0.218$

$$\text{Then } T = \frac{114.6 \times 750 \times 1.15}{1.264} = 780000 \text{ g.}$$

$$s = \frac{78000 \quad \times 0.218 \times 1.7 \times 10^{-1}}{1.87 \times 147 \times 147} = 7.25 \times 10^{-2}$$

(Method B) From prepage $T_{approx.} = 99000$ gall per day

and $s = 5.42 \times 10^{-2}$

From graph of Chow:

Over estimate error in T = 24.0 percent

Under estimated error in s = 23.5 percent

Hence corrected value of $T = 99000 \times \frac{1}{1.24} = 79800$

Corrected $S \quad = 5.42 \times 1.235 \times 10^{-2} = 6.7 \times 10^{-2}$

3.4.4. Boulton[36] Method

The drawdown vrs time plot curve for well no.25 is not exactly in conformity with the type curve of unconfine aquifer as described by Krusenan. The field curve of said well observation point has three segments in which middle segment is showing decrease in slope which can be seen in Fig.3. Hence there was doubt about the possibility of delayed yield and so same was investigated on the following basis.

3.4.4.1. Theory

Boulton has stated in his paper that Theis formula may not give good approximation for the drawdown of water table unless following dimensionless quantities satisfy given condition $\tau > 5$, $\rho > 0.2$ in which $\tau = Ku\ t/(\ 5\ he)$ and $P = r/h_e$.

Where K_u = co-efficient of permeability in vertical direction, t = time from start of pumping effective short term co-efficient of storage, h_e height of the undisturbed water table above impermeable stratums. He has further advocated the analysis of distance drawdown for determination of storage co-efficient for which data was not available. However he has also described an analysis for time drawdown curve and developed an equation described below:

$$W = 2\int_0^\infty \frac{o\,(U.P)}{u} \quad 1 - \frac{\cos h_u (1-\zeta)\ F(t)}{\cos h_u} \quad du \text{ also}$$

$$W = \frac{S}{\frac{\theta}{4\pi t}}$$

Where $F(t) = e^{-f_1} (\cos h\ f_3 + \frac{f_2}{f_3} \sinh f_3)$

and $2 f_1 = \alpha t\ (\eta + r\ u \tan h_u)$

$2 f_2 = \alpha t\ (\eta - r\ u \tan h_u)$

$2 f_3 = \alpha t \sqrt{(\gamma\ u \tan h_u + \ )^2 - 4\ r\ u \tan h_u}$

In equation $\zeta = Z/h_e$, Z being depth of the observation well below the initial water level, $\gamma = k_u/(\alpha\beta\ h_e)$, n = effective long term co-efficient (S + S') divided by short term co-efficient (S). The equation described by him as stated above cannot be computed at the present stage. However on the basis of another method also described by Boulton having use of type curve was employed. The approval of same is described in brief here.

3.4.4.2. Graphical Solution[30]

Boulton has developed a family of type curves occuring in different situation of unconfined aquifer. All these curves form three segments, out of which central part of curves are flatter than other two segments. The break in slope in type curves are probably due to replenishment by gravity drainage from inter-slices above the depression cone. During this time there is marked discrepancy between the observed data curve and Theis type curve.

The development of type curves are based upon the function $W = F(U_A, r/B)$ for earlier time condition

$= F(U_y, r/B)$ for latter time condition.

in which r is horizontal distance from well axis to any point and $B = \sqrt{T/\alpha S'}$ in which T is transmissibility, α ~~represent~~ reciprocal of delay under and S' storage co-efficient of latter time condition.

These two Functions W and U_A, (U_y) are also related to field parameters as described below:

$$W = \frac{S}{Q/(4\pi\ T)}\ ;\quad U_A = \frac{r^2\ S}{T\ t} \quad \text{and } U_y = \frac{r^2\ S'}{T\ t}$$

Where S = drawdown of water table at distance , Q constant rate of discharge from pumped well.

3.4.4.3. Graph Plot

On basis of values of well function W, related U_A and U_B available in book by Kruseman,[30] two families of curves for different values of r/B were drawn on logarithmic paper. A family of curve for earlier time condition was drawn starting from the left end where as for latter time condition, it was done from opposite end of paper, which subsequently formed the relatives curves of other family. Field curve was then drawn on the basis of observed data, on logarthim paper, utilising Y axis for drawdown S and X axis for time t.

3.4.4.4. Superposition of Curves

The field curve was superimposed first on the curves of earlier time, for matching. It actually matched with the type curve for r/B= 1 but when transallted on right side the field curve did not agree with these curves, but corresponded to These curves. Hence it was realised that the problem is not exactly same as discussed in this section, and so the Boulton's theory was considered as in applicable.

3.4.5. In recent, a few other investigators have developed the method, which are a little or more based on Theis Theory.

For information sake it is to describe that Hurr[32] has developed a method with use of type curve for U W(u) and u {W(u)} / S_c and plot of field curve of specific capacity vrs time, to determine the transmissibility only. Where as Kriz[33] has transformed differential equation in non-dimensional form to develop type curves. However these methods could not be employed in this study due to certain difficulties.

3.5 VALUE OF FORMATION FACTORS

3.5.1. The value of storage co-efficient and transmissibility, as obtained by application of different methods are reproduced here.

Well No.		Theis Method	Jacob Method	Chow Method	Chow Method
81	Transmissibility	1,67,000 gpa/ft.	176,800 gpd/ft	1,72500 gpd/ft	172,500 gpd/ft
	Storage Co-efficient	9.65×10^{-4}	8.35×10^{-4}	9.22×10^{-4}	9.05×10^{-4}
25	Transmissibility	72,500 gpd/ft	99,000 gpd/ft	78.00 gpd ft	79,800 gpd/ft.
	Storage Co-efficient	7.20×10^{-2}	5.12×10^{-2}	7.25×10^{-2}	6.7×10^{-2}

3.5.2. This this context it is to mention that Jacob's method is very approximate. In this light, when other three methods are compared, the values so obtained are close to each other. It is also to add that values of storage co-efficient obtained through observed data of well No.81 is too low, This shows that it is either data of confined aquifer or presence of clay lense has caused so low result.

3.5.3. As present model study is related to isotropic unconfined aquifer, hence the values of formation factors obtained for well no.81 was ignored, and values of other well no.25 was adooted, after converting the same in metric unit. Theis final values of formation factors used in the model are as given below.

Transmissibility of aquifer 'T' = 1.227 m^2 per day
Storage co-efficient = 0.072

CHAPTER-IV

LUMP MODEL

4. Through lump model, the whole basin is represented as a unit. In the lump model all the hydrological parameters are lumped parameters representing the process of entire basin. The general[34] function representing the lump model which has been described earlier, is reproduced for further discussion.

General form of model is $\Delta d = f\ (dn, L)$

Where as equation of continuity is $\dfrac{\Delta S}{\Delta t} = I - 0$

4.1. Simplification of Model:

In case, if the whole area is taken as single unit and all the quantities are determined over some fixed time space in terms of an equivalent depth[35] over the area, the left hand terms of above two equations can be equalised. Hence other two terms will correspond to each other. It is to add further that water level (dn) of basin has also direct relation with water level of streams lying in the basin and also water level of adjacent basin. (in case of pervious boundary) as inflow or outflow to/from the basin which is governed by Darcy's law is dependent upon water level difference of these water bodies. For sake of simplification, if it is assumed for some time, that inflow or outflow of above two cases are either known or to be known by other means, then lump model which employs ordinary differential equation as described above can be represented by simple algebrical expression which is based upon equation of continuity.

4.2. Water Balance Equation:

This equation which incorporates all the components of hydrological cycle is popularly known as water balance equation. The water balance equation[39] is thus simplest form of lump model, which can be expressed as follows :-

$$R_r + R_c + R_{IC} + R_{IT} + P_L + I + S_I = E_{TW} \quad E_{TP} \quad + O + S_E + T_P \pm \Delta S$$

Where

R_r = recharge due to rainfall

R_c = recharge due to canal seepage

R_{IC} = recharge due to irrigation through canal system.

R_{it} = recharge due to minor irrigation (wells)

P_L = recharge due to percolation from lakes and ponds.

I = inflow from other basins

S_I = influent seepage from streams

E_{TW} = evapotranspiration from water logged area

E_{TP} = Evapotranspiration from forest area.

O = outflowto other basin

S_E = Effluent seepage to stream

T_p = Extraction of water through wells and pumps

ΔS = change in storage

This mathematical equation is the simplest form of axpression and lump model in which differential equation though actually involved is not apparent.

4.3. Use of Model :

The lumpedmodel which describes above mathematical expression can be employed for development of ground water resources, determination of specific yield and also same can beutilised for computation of unknown value of any one component,provided all other components of hydrological cycles are known. This model is thus useful tool in planning of ground water project. However this needs determination or estimation of hydrological components which requires further computation. Under present study the model was employed to study over all water balance of Varuna Ground Water Basin as well as supplement data to distributed model, for which further work was carried on. For over all ground water balance, estimation of hydrological components of model was made, as needed for the same. The procedures adopted for above work is described below with very brief theory. However prior to determination of value of components of lump model, it appears necessary to give theoretical discussion of rainfall, specially regarding its movement towards ground water storage, actual contribution and computation of weighted rainfall, as these basis were needed for discussion as well as computation of several components.

4.4. Rainfall :

4.4.1. Movement to Ground Water Basin:

The rainfall that falls over the catchment area gives rise to certain number of different phenomena,evaporation, infiltration, transpiration, surface run-off depending upon such factors as the initial status of basin(dry or wet) rain intensity, geology, vegetation.Thus only a part of rainfall infiltrates through the surface of ground. There is further loss even after infiltration in the intermediate zone, as part of water, is retained in this zone to meet the soil moisture deficiency. The loss may be further aggrevated, if deep roots penetrate through the zone of aeration to a capillary fringe. In this particular condition water will be lost from the saturated zone, to make good the deficiency. Thus the unsaturated zone above the water table affects both the quantity and timing of direct recharge from precipitation. Important parameters are the rates, and duration of rainfall the subsequent condition of the land surface the steady or unsteady rainfall, back ground of wet condition recharge or discharge rate, the antecedent soil moisture condition, the water level depth, the allowable depth of surface water ponding and the soil type. The interaction of these parameters result in a lag and attenuation of the recharge as a function of depth.

4.4.2. Contribution to Ground Storage :

The ultimate source of all the ground water of good quality is precipitation. Contribution of rainfall as recharge to ground water body is very high compared to any other source of recharge such as recharge through influent seepage from stream, artifical recharge through surface irrigation etc. Hence estimate of recharge due to rainfall is very important in the field of hydrology, but there is no any direct method for estimation of same. However, there are a few indirect methods such as method based on measurement of chlorine[36] content, Isotop[37] technique etc. Isotop technique which is of recent development is very precised but at the same time experiment is costly. Karmanskhif[38] has described method of estimation of recharge, which is based upon water level fluctuation. He actually employed the basis for field experiment, for small area. However using same principle, recharge can be estimated based on water fluctuation data using the mathematical model The estimation of recharge can be treated as in verse problem of ground water balance, as this value of recharge can only be computed until all other components of hydrological cycle, involved in the model is known. Hence actual estimation of recharge has been dealt at the end, under overall balance study.

4.4.3. Weighted Rainfal:

Since the depth of rainfall occurring over the basin is not uniform over the whole area, the average depth of rainfall either for entire basin or for significant area is needed for solving hydrological problems. For this model, average rainfall for entirebasin as well as its sub-area was necessary. The observed rainfall data recorded at different stations can be used for computation of average rainfall for area in question. The computation of average depth of precipitation can be done by following methods :-

1. Arithmatic average Method
2. Thiessen Method
3. The Isohyetal Method

Thiessen method was preferred for computation of weighted rain-fall for the basin under study. The procedure as followed is described here in very brief.

There are six rain gauge stations in the command area of the basin, location of which have been shown in Fig.1. An area in form of Thiessen polygon was described around each rainfall station, the procedure for the same has already been described in nodal construction work. The areas of all the six polygons were measured with the help of plani-meter, and converted as fraction of the total area in question. The rainfall of respective stations was multiplied by the partial areas. Finally all the products were added together for obtaining the average rainfall for the whole basin. (Table 5.5.) Similar procedure was followed for computation of districtwise weighted rainfall, as needed in course of study.

4.5. Estimation of Data:

For overall balance of ground water basin, it was necessary to evaluate the hydrological components of lump model. In this respect it is to inform that for preparation of required data, as needed for the model, input data was not to available in complete scenee. However from available information estimation of following components were done, based on certain assumptions. The procedures followed is hereby described with its background.

4.5.1. Recharge due to Canal Seepage:

Computation of recharge due to canal seepage is based upon following considerations. It is to note that only a part of basin is under canal irrigation, which will be apparent if table 4.1.(c) is looked into.

4.5.1.1. Canal Losses[40]

While carrying water through channels, a good amount of water is lost. These losses are due to evaporation and percolation. These losses, when combined is termed as conveyance loss. Evaporation depends on several factors such as surface area, relative humidity, wind velocities and temperature. Maximum evaporation takes place during hot and dry season, particularly in month of May and June. However evaporation loss is very less compared to seepage loss.

4.5.1.1.1. Seepage Loss :

Seepage loss, which is a source of recharge to ground water body depends upon nature and porosity of the soil; the depth, turbidity, and temperature of water; age and shape as well section of canal, and the position of ground water table. All the water which seeps through embankment do not reach ground water body, but a part of it is lost due absorption of water by the soil in intermediate zone lying between soil saturated by the canal water at the top and capillary fringe of water table.

4.5.1.1.1.1. Absorption Loss :

Absorption loss actually controls the percolation loss. This loss is dependent on the deficiency of moisture content of the intermediate zone. The water which seeps through canal is initially utilised in filling up the pores of the soil around it and after that it frickles down. Hence at initial supply of water in canal, good amount of water is lost in absorption. Only after the intermediate zone becomes saturated, water percolates downward. Still, even the intermediate zone becomes saturated, all the water do not reach ground water storage as some of water is lost by evaporation and transpiration of seepage water re-emerging at the surface by capillary fringe.

4.5.1.1.1.2. Percolation :

Hence, after meeting all the losses as discussed above, water percolates to ground water table. The percolation may be considered as high compared to other losses. Determination of percolation is important for ground water balance study. The application of physical laws for same is not an easy approach. Hence some emperical formulas or common practice are in use. In Uttar Pradesh, where the present study is being carried out following approaches are in practice.

4.5.1.2. Approach of Estimation of Seepage Loss :

For design of project works, conveyance loss which include percolation loss as well as evaporation loss, is calculated based on following percentage of discharge of canal system.

i)	Loss in main canal or branch canal	- 18 percent
ii)	Loss in distributary and minors	- 7 percent
iii)	Loss in field channels	- 20 percent

2. It is also in common practice to consider loss in terms of discharge in cusecs per million sq.ft. of wetted perimeter or cumecs per million sq.ft. The usual figures as used in U.P. are from about 2.5 cumecs (8 cusecs) for ordinary day loam to about 5 cumecs (16 cusecs) for sandy loam with average of nearly 3 cumecs (10 cusecs).

3. Emperical formula also gives comparable results. It expresses the losses in cusecs per mile length of the channel. The formula is

$$\text{Loss} \quad q_{if} = \frac{C}{8} \left(B + D \right)^{2/3} \quad \text{Where C is constant}$$

Value of c is 1 for intermittent canal, and 0.75 for canal running constantly. B and D are respectively bed width of depth of water in canal. In metric unit, above equation can be written as

$$\text{Loss} = \frac{C}{200} \left(B + D \right)^{2/3}$$

In present work last equation was employed for determination of seepage loss. Value of C was taken as 1, as supply through canal is intermittent. This equation is in common use.

4.5.1.3. <u>Estimation of Seepage loss</u> :

It may appear that use of any one approach as discussed above needs value of discharge of channels. It has already been described that required value of flow is wanting. Hence for computation of seepage, at first, water supply through various channels were estimated as described below.

4.51.3.1. Estimation of flow in canal:

In project report(41) of Varuna Basin, discharge data of seven channels of command area were available, against 19 nos. of channels lying in the basin. The monthly discharge figures are for available period 1968-71 and so not related to study year. However estimation was done as per following procedures.

On basis of above data, of 4 years average discharge and thereafter ratio between this average discharge and design discharge was determined. Similarly average running days for all those seven channels were computed. Result is shown in table 3.2. Discharge ratio of Machlisahar distributory was computed by taking average of discharge ratios (as computed) of other two distributaries and parent channel, Maridahun Br. canal. Similarly running days of above distributary was also computed by taking average of running days of above three canals.

In case of minors similar method was adopted, considering discharge ratio of its parent channel (distributary or branch canal) and sister channels only. However, some adjustment was made, for keeping the sum of discharge through different channels equal to total discharge of parent channel at its head. For running days approach remained the same, but in some cases release of water in the distributary was treated in a part of its length only when a few minors at the end is drawing water from its parent channel, and days of run is lesser than running days of the distributary.

4.5.1.3.2. Estimation of recharge due to canal seepage:

As computation of seepage using the emperical equation as described below for 12 months through 19 channels was a difficult task, hence following procedure was adopted for computation of same by equation

$$q_i = \frac{1}{200} \quad (b + d)^{2/3}$$

4.5.1.3.2.1. Simplification of Method of Estimation:

It is needless to say that discharge and seepage losses, both are junction of bed width and depth of water. Hence establishment of interrelationship in dimensionless form was made, on following procedures. For design discharge bed width and depth of all the channels are available in said report. Hence for various depth of water running discharge was computed. Discharge ratio was obtained for those selected depth by simply dividing running discharge by design discharge. For those various depth, used in above computation, seepage losses~~for~~ of respective channels were computed, together with seepage loss for design depth. By simple division of former value to latter, seepage loss ratio was also known.

Thus seepage loss ratio for respective discharge ratio, as obtained was tabulated. For any intermediate value of discharge ratio seepage loss ratio was obtained by interpolation.

4.5.1.3.2.2. Computation of Seepage Loss :

Running discharge of all the channels, as described earlier was arrived in terms of discharge ratio (running discharge/design discharge). Hence seepage loss was computed by taking the product of length of channel lying under the nodal area in question, days of run, seepage loss for design depth multiplied by seepage loss factor for running discharge. Thus for all the channels lying under different nodes, seepage loss monthwise for effected nodal areas were computed. A statement showing computation for representative month is available vide table 4.1.(a).

4.5.1.3.2.3. Computation of Recharge due to seepage loss:-

Losses from seepage due to evaportranspiration and absorptions during transit was taken as 20 percent of above loss. Based on this factor, recharge due to seepage was taken as 80 percent of sctual seepage. On this approach recharge under effected nodes forall the 12 months were computed. The value obtained for various nodes were added to obtain desired value of recharge for entire basin to use the same in lump model. The result is in available in table 4.1.(d).

4.5.2. Recharge due to Canal Irrigation:

This includes recharge due to seepage through water course field losses.

4.5.2.1. Seepage through water course:

Seepage through water courses of canal system was included under irrigation over the field, as computation work was not possible earlier due to following reason:

i) Design particulars of water courses was not available, hence this was to be determined on other alternative method, which was as applicable was the basis percentage of water available at head of water course.

ii) Availability in water course could be determined only when seepage loss through minors, distributaries are pre-determined.

4.5.2.1.1. Estimation of Flow in Water Course:

For this total flow in quantity (H.M.) through minors, distributaries were determined for entire month by taking product of design discharge, discharge ratio and no. of running days. Net volume available in minors for supply to water course directly taking branch canals or taking from distributary was known by simply substracting seepage loss from total flow during month. However in case of distributary feeding the water course net quantity was obtained first by deducting seepage loss through it, and then balanced net quantity after feeding minors were found out, which gave the value available to its water courses.

4.5.2.1.2. Recharge through Water Course :

As computation of recharge due to seepage loss is in term of percentage, with consideration of deduction in percentage dueto evaporation and evaportranspiration, computation work of this recharge was combined together with field loss. Usually 15 to 20% of discharge available at head of water courses is considered as loss due to evaportranspiration and other losses.

4.5.2.2. Field loss:

The amount of water which is applied over the field is not fully utilised by the crop, but good amount of water is lost in evaporation, deep percolation and surface runoff.

There are various factors influencing extent of loss. Hence to meet the needs for consumptive use, greater amount of water is supplied and amounts so needed depends upon field application efficiency.

4.5.2.2.2.1. Field Application Efficiency[42]

Field application efficiency also depends upon several factors, as described below :-

i) Skill and attention paid by the irrigators. If the water is not properly distributed over the field and left uncared for this can cause either over irrigation or surface runoff resulting considerable loss. Some times wastage occurs due to lack of skill, as irrigator does not know how to divert water to another field, after meeting the requirement.

ii) Intake characteristic of soil :- In case of soils having very high infiltration rates, deep percolation will take place which is generally expected in coarse textured soil, where as in case of fine textured soil whose infiltration rate is low, loss through surface runoff will be more. On account of growing different crops by rotation, over a field variation in filtration rates takes place, which causes further reduction in efficiency.

iii) Topography :- In case of plane land irrigation efficiency is more compared to sloping land where major quantity is lost in surface run-off causing lower efficiency.

iv) Irrigation method:- Water intake varies with the method of irrigation. In case of sprinklers method infiltration rate is influenced by those factors as in case of rain fall. In case of partially flooding in which water is applied by running small streams in furrows, water moves vertically and laterally depending upon capacity of soil as well as wetted perimeter of furrows. In case of complete flooding water moves in sheds.

v) Depth of Irrigation:- There is also some influence in irrigation efficiency due to depth of irrigation, in case of a few irrigation methods. Lighter application of irrigation causes lower efficiency where as heavier application subject to water holding capacity of soil increases the efficiency.

Hence field irrigation efficiency is to be decided based on all above factors. At present stage of development of irrigation system for existing methods in the Uttar Pradesh. 60% irrigation efficiency is considered.

4.5.2.3. Estimate of Recharge due to canal irrigation :-

Recharge due to above two losses, based on percentage discussed in relative section, was computed by taking overall percentage of water available at the head of water course, based on following derivation.

4.5.2.3.1. Derivation of percentage of recharge :-

Net recharge as percentage of flow available at the head is based on following calculation:-

Seepage plus evaporation loss in water course = 20 percent

Field loss 1.00 - 60 (Field efficiency) of discharge reaching to field = 32 percent

Deduct 20% loss due to evaporation and evaportranspiration - 10.4 percent

Hence net recharge to ground water body 41.6 percent

OF WATER TO FIELD FOR JANUARY

Sl. No. Name	GE LOSS UNDER EACH NODE 2 HM	4 HM	5 HM	Net Qty. to Water Course vide(e)	DISTRIBUTION OF WATER IN THE FIELD OF EACH NODE 1 HM	2 HM	4 HM	5 HM
1. PHULPU								
a. Sahpur	2.85			7.05		7.05		
b. Narain	1.14			6.23		6.23		
c. Pali	1.58			2.84		2.84		
d. Nai ko				1.32	1.32			
e. Water					5.86			
2. Kuwarp	4.33	3.74		vide 2(d)				
a. Jhanjh	0.51	0.71		6.01		2.50	3.51	
b. Talwar		0.632		1.59			1.59	
c. Paora	1.00			3.25		3.25		
d. Water				1.28		0.68	0.60	
3. Machli		8.68		Vide 3(c)				
a. Mirzap		0.98		2.38			2.38	
b. Tulapu		0.67		1.61			1.60	
c. Water							12.72	
4.a. Tikar M		0.54		1.53			1.53	
Ranipu		1.20		2.83			2.83	
Pawapu	3.29			4.57		4.57		
Thalio	0.77	1.01		4.02		1.75	2.27	
Ramgar			1.77	3.93				3.93
Rampur	1.55			2.48		24		
5. Mariah	61.68	45.50						
	77.70	63.66	1.77		7.18	31.35	29.03	3.93

TABLE -4.1(a)

TABLE SHOWING SEEPAGE LOSS THROUGH CANAL SYSTEM UNDER DIFFERENT NODES & SUPPLY OF WATER TO FIELD FOR JANUARY

Sl. No.	Name of Channel	Design Discharge in Cumecs	RELEASE OF WATER Days	Release factor	Total supply during month H.M.	Rate of Seepage loss in reach on full discharge in entire length Cumec/sec.	Seepage loss factor for actual discharge cumec/sec	Seepage loss in full length per day	Total seepage during month	Balance Qty. H.M.	SEEPAGE LOSS UNDER EACH NODE 1 HM	2 HM	4 HM	5 HM	Net Qty. to Water Course vide(e)	DISTRIBUTION OF WATER IN THE FIELD OF EACH NODE 1 HM	2 HM	4 HM	5 HM
1.	PHULPUR DISTY.	1.222	5	0.70	36.95	0.1766	0.965	1.47	7.38	29.57	7.38								
a.	Sahpur Minor	0.3695	5	0.578	9.90	0.0704	0.938	0.57	2.85	7.05		2.85			7.05		7.05		
b.	Narainpur "	0.1705	5	1.00	7.37	0.02758	0.948	0.228	1.14	6.23		1.14			6.23		6.23		
c.	Pali minor	0.1648	5	0.62	4.42	0.03787	0.959	0.316	1.58	2.84		1.58			2.84		2.84		
d.	Nai kot minor	0.0767	5	0.578	1.92	0.015036	0.935	0.121	0.605	1.32	0.605				1.32	1.32			
e.	Water course ex Disty				5.86		INCLUDED IN FIELD LOSS									5.86			
2.	Kuwarpur Disty.	0.5968	5	1.0	25.78	0.10026 / 0.08647	1.00	0.866 / 0.747	8.01	18.77		4.33	3.74		vide 2(d)				
a.	Jhanjharpur Minor	0.2274	5	0.746	7.23	0.01234 / 0.01728	0.960	0.1024 / 0.1435	1.22	6.01		0.51	0.71		6.01		2.50	3.51	
b.	Talwara minor	0.05116	5	1.00	2.21	0.01462	1.00	0.1263	0.632	1.59			0.632		1.59			1.59	
c.	Paora	0.086	5	1.00	4.26	0.02318	1.00	0.2003	1.00	3.25		1.00			3.25		3.25		
d.	Water course ex Disty.	-				INCLUDED IN FIELD LOSS									1.28		0.68	0.60	
3.	Machli Sahar Disty.	0.8924	5	0.80	26.84	0.20456	0.983	1.735	8.08	18.76			8.68		Vide 3(c)				
a.	Mirzapur Minor	0.0986	5	0.80	3.36	0.02318	0.972	0.195	0.98	2.38			0.98		2.38			2.38	
b.	Tulapur minor	0.07162	5	0.80	24.8	0.017018	0.972	0.134	0.67	1.61			0.67		1.61			1.60	
c.	Water course ex Disty.				12.72		INCLUDED IN FIELD LOSS											12.72	
4.a.	Tikari Direct Minor	0.0597	5	0.80	2.07	0.01277	0.972	0.107	0.54	1.53			0.54		1.53			1.53	
	Ranipur Minor	0.1165	5	0.80	4.03	0.028673	0.966	0.240	1.20	2.83			1.20		2.83			2.83	
	Pawapur minor	0.2274	5	0.80	7.86	0.07808	0.975	0.657	3.29	4.57		3.29			4.57		4.57		
	Thalio Minor	0.1648	5	0.80	5.80	0.018348 / 0.02399	0.966	0.153 / 0.262	1.78	4.02		0.77	1.01		4.02		1.75	2.27	
	Ramgarh Minor	0.7055	5	0.80	5.70	0.04233	0.966	0.354	1.77	3.93				1.77	3.93				3.93
	Rampur Minor	0.1165	5	0.80	4.03	0.03699	0.966	0.309	1.55	2.48		1.55			2.48		24		
5.	Mariahun Branch	9.55/2.8415		0.70		4.192/3.092	0.987	4.112/ 3.033	107.18	-		61.68	45.50						
	Total under the Node										7.99	77.70	63.66	1.77		7.18	31.35	29.03	3.93

ISE) FIGURES IN HECT.METRES

onth	WA Su in Br Ca at	B. WATER SUPPLY TO FIELD CHANNEL AND RECHARGE TO GR.W.BRANCH Node 1 Supply to Field Channel	Rech. to GW B	2 Supply to Field Channel	Rech. to GW B	3 Supply to Field Channel	Rech. to GW B	4 Supply to Field Channel	Recharge to G.W. Body
anuary	8	7.18	2.98	31.35	13.05	29.03	12.10	3.93	1.62
ebruary	6	7.24	7.19	44.52	18.57	56.25	23.43	7.26	3.02
arch		.67	1.53	17.65	7.35	14.10	5.87	1.73	0.72
pril		.54	3.14	16.35	6.81	17.89	7.45	2.46	1.03
ay	1	3.44	5.60	38.46	15.60	40.24	16.73	1.85	0.77
une	1	9.53	12.30	65.32	27.20	55.83	24.20	8.20	3.42
uly	1	7.14	7.14	54.17	22.56	57.39	23.83	6.97	2.80
ugust	1	1.74	13.22	58.78	24.48	56.34	23.42	5.29	2.20
eptember		0.95	8.72	44.51	18.50	36.63	15.22	4.89	2.09
ctober	14	7.91	11.58	47.43	19.30	37.28	15.49	5.20	2.16
ovember	1	4.76	10.48	79.39	33.02	80.93	33.68	9.81	4.08
ecember	1	7.53	11.47	82.21	34.22	41.49	17.28	12.89	5.37
otal in ear			95.35		240.66		218.71		29.38
une to ctober			52.96		112.04		102.17		12.77

TABLE - 4.1(b)

TABLE SHOWING COMPUTATION OF RECHARGE DUE TO CANAL SEEPAGE AND CANAL IRRIGATION (MONTH-WISE)

FIGURES IN HECT.METRES

Month	WATER SUPPLY IN CANAL			A. SEEPAGE AND EVAPORATION(CONVEYANCE)LOSS AND RECHARGE TO G.W.B.								B. WATER SUPPLY TO FIELD CHANNEL AND RECHARGE TO GR.W.BRANCH							
				NODE - 1		2		4		5		Node 1		2		3		4	
	Supply in Branch Canal at Head	Canal System at Basin end	Total canal supply to Distr. System in this Basin	Conveyance loss	Recharge to G.W.	Conveyance loss	Recharge to G.W.	Conv.Loss	Recharge G.W.	Conv. Loss	Recharge G.W.	Supply to Field Channel	Rech. to GW B	Supply to Field Channel	Rech. to GW B	Supply to Field Channel	Rech. to GW B	Supply to Field Channel	Recharge to G.W. Body
January	889.0	268.1	119.06	7.99	6.41	77.70	62.16	63.66	50.93	1.77	1.41	7.18	2.98	31.35	13.05	29.03	12.10	3.93	1.62
February	652.4	180.8	188.50	12.32	9.85	72.06	57.65	64.78	51.82	3.13	2.50	17.24	7.19	44.52	18.57	56.25	23.43	7.26	3.02
March	584.8	140.4	71.28	9.95	7.95	82.15	57.72	63.95	51.16	1.62	1.29	3.67	1.53	17.65	7.35	14.10	5.87	1.73	0.72
April	918.9	239.0	51.30	4.79	3.83	30.65	24.52	26.05	20.84	1.30	1.04	7.54	3.14	16.35	6.81	17.89	7.45	2.46	1.03
May	1090.7	248.1	171.20	14.06	11.23	98.46	78.77	79.35	63.48	4.34	3.47	13.44	5.60	38.46	15.60	40.24	16.73	1.85	0.77
June	1562.2	391.1	220.20	20.44	16.35	122.51	98.01	81.25	65.00	3.96	3.17	29.53	12.30	65.32	27.20	55.83	24.20	8.20	3.42
July	1324.4	331.5	238.80	14.26	11.44	141.11	112.89	84.27	67.41	4.77	3.81	17.14	7.14	54.17	22.56	57.39	23.83	6.97	2.80
August	1825.6	454.7	245.70	21.17	16.94	123.91	99.13	104.71	83.76	4.51	3.60	31.74	13.22	58.78	24.49	56.34	23.42	5.29	2.20
September	607.6	183.0	178.0	14.28	11.42	61.64	49.31	44.44	35.55	3.31	2.66	20.95	8.72	44.51	18.50	36.63	15.22	4.89	2.09
October	1464.4	350.5	174.20	17.80	14.24	113.84	91.07	89.84	71.08	2.08	2.20	27.91	11.58	47.43	19.30	37.28	15.49	5.20	2.16
November	1444.8	361.7	265.00	16.49	13.19	119.21	95.37	103.55	82.84	4.59	3.67	24.76	10.48	79.39	33.02	80.93	33.68	9.81	4.08
December	1763.4	442.0	318.90	34.43	24.54	144.34	115.47	119.62	95.68	3.60	2.88	27.53	11.47	82.21	34.22	41.49	17.28	12.89	5.37
Total in Year			2242.14		1470.36		942.07		740.55		31.70		95.35		240.66		218.71		29.38
June to October					70.36		450.41		323.70		15.44		52.96		112.04		102.17		12.77

TABLE SHOWING NODAL AREAS UNDER PRESENT SYSTEM OF IRRIGATION(DISTRICTWISE) IN VARUNA BASIN

NODE	PRATAP GARH		ALLAHABAD		JAUNPUR		VARANASI	
	Canal Irrigation	Minor Irrigation	Canal Irrigation	Minor Irrigation	Minor Irrigation	Canal Irrigation	Canal Irrigation	Minor Irrigation
1.	840		5510	14,890**				
2.	6420		960	3,720**	6190(11,430)*	13,474		
3.	-	12,009	-	16,803	-	-	-	9,601
4.	-	-	-	-	14,511(22,740)	3,005	-	-
5.	-	-	-	-	825(6,000)	23,884**	-	1,443
6.	-	-	-	-	-	600	-	25,505
7.	-	-	-	-	-	8,164	-	14,763
8.	-	-	-	-	-	1,740	-	22,321
9.	-	-	-	-	-	-	-	16,803
10.	-	-	-	-	-	-	-	15,243

*Figures in Bracket shows command area of canal

** Area of Irrigation through state tube wells
For other system some area (in %) of canal command was also been included.

TABLE SHOWING RECHARGE DUE TO CANAL SEEPAGE AND DUE TO LOSSES THROUGH FIELD CHANNEL AND IRRIGATION

Month	RECHARGE DUE TO CANAL SEEPAGE Node 1	Node 2	Node 4	Node 5	Total	RECHARGE DUE TO LOSSES THROUGH FIELD CHANNELS&IRRIGAT Node 1	Node 2	Node 4	Node 5	Total
January	6.41	62.16	50.93	1.41	120.91	2.98	13.05	12.10	1.62	28.75
February	9.85	57.65	51.82	2.50	121.82	7.19	18.57	23.43	3.02	52.21
March	7.95	57.72	51.16	1.29	118.12	1.53	7.35	5.87	0.72	15.47
April	3.83	24.52	20.84	1.04	50.23	3.14	6.81	7.45	1.03	18.43
May	11.23	78.77	63.48	3.47	156.95	5.60	15.60	16.73	0.77	38.70
June	16.35	98.01	65.00	3.17	182.53	12.30	27.20	24.20	3.42	67.12
July	11.41	112.89	67.41	3.81	195.52	7.14	22.56	23.84	2.90	56.44
August	16.94	99.13	83.76	3.60	203.43	13.22	24.48	23.42	2.20	63.32
September	11.42	49.31	35.55	2.66	98.94	8.72	18.50	15.22	2.09	44.53
October	14.24	91.07	2.20	179.49	11.58	19.30	15.49	2.16	49.53	49.53
November	13.19	95.37	82.84	3.67	195.07	10.48	33.02	33.68	4.08	81.26
December	24.54	115.47	95.68	2.88	238.57	11.47	34.22	17.28	5.37	68.35
	147.36	942.07	740.45	31.70	1861.58	95.35	240.6	218.71	29.38	584.10

4.5.2.3.2. Computation of recharge :

Using above percentage of recharge monthly recharge under different nodes were computed by taking 41.6 percentage of total volume available at the head of water course lying in the nodal areas. Finally these values were added to obtain desired value of monthly recharge for entire basin to use the same in lump model.

4.5.3. Ground Water Drawl Through Wells:

Major area of the basin is under minor irrigation system for which ground water extraction through wells and pumps is being done. It may be stated here that on basis of available informations regarding no. of wells during different years from 1968-72, it appears that there is rapid development of ground water exploitation in the basin. Hence it was necessary to estimate the ground water drawl based on appropriate information for use in the model. Unfortunately the informations which could be obtained were quite in complete. However an effort was made to estimate drawl on basis of available informations, as described below :-

There are following types of well under use :-

i) Dug wells

ii) Pumping sets

iii) Private tube wells

iv) State tube wells.

Hence it was necessary to have following informations for computation of monthly drawl through the area in question.

(1) Discharge of wells (ii) No. of wells (iii) Running hour during different months. It will appear from coming discussion that informations for all above factors are lacking. Hence it was necessary first to estimate above factors for computation of drawl, for which following approaches were folloed :-

4.5.3.1. Estimation of discharge of wells(6,40)

4.5.3.1.1. Dug wells ;

Since ancient times, dug wells have been used for domestic purposes. They still exist in villages for utilisation of damesti needs. However, where there is no any permanent water distribution system, and water table lies at reasonable depth these wells are also used for irrigation purposes, and so, such is the case of Varuna Basin.

The depth of such wells vary from about 10 to 40 ft. depending upon the position of water table.

The wells are generally extended a few feet preferably 15 ft. to 25 ft. below the water table. It is also in common practice to sink the wells below the lowest spring level only to the extent necessary for filling the vessel and to sink a pipe 30 to 40' deeper in the centre of well, the bottom of well being sealed by concrete plug. The diameter of such wells may be of various sizes but generally does not exceed 5 metres. The wells may be resting on mota layer and drawing water through a hole bored in that layer. Such wells are termed as deep wells. When the wells rest on pervious strata and draw water from surrounding materials known as shallow wells. The wells are constructed with masonry linings or dry brick lining or left without any artificial lining. Drawl of water for irrigation may be either persian wheel, pulley, mot or dhekuli.

Hence keeping into consideration of such variations in depth of sinking, diameter, construction and means of drawl of water it is very difficult to assess correct figure of discharge. However, Todd(6) has stated that properly constructed wells penetrating a permeable aquifer can yield 30,000 to 9000 gallon/hr. although most domestic dug wells yield less than 6000 gallon/hr. Bharat Singh(40) has ascertained that with a lift of 7.5 metre and using the churus as lifting device a well can irrigate 1.5 hectare of area in a year Based on this figure, and consideration of required depth of water for rice as 90 cm. and for wheat 37.5 cm. total quantity of drawl during Kharif and rabi comes to 1.35 hectare metre, and 0.53 hectm respectively. In light of above informations discharge of dug wells was assumed as 2500 galls/hr. during Kharif when water table is high and eighty percent of above discharge was taken during rabi, when water level in well goes down.

4.5.3.1.2. <u>Pumping Sets</u> :-

The pumping set described in project report is as understood as pumps of low discharge employed for extraction of water through shallow wells. Hence based on this conception discussion follows.

Types of Well :

Shallow wells may be open wells which is usually dug wells as described earlier or tube wells of smaller depth in an unconsolidated aquifer. Such shallow tube wells are either bored wells or driven wells or jetted wells. A brief idea of all these wells are given in order to assess average discharge for all types of wells under use, for pumping sets.

4.5.3.1.2.1. Bored Wells :

These wells are constructed with hand operated auger or power driven earth auger. Hand bored wells are generally preferred for 6" to 8" dia. having depth not more than 50 ft. where as power driven wells can be of larger bore holes even upto 36 inch diameter and depth more than 100 ft. in case of favourable condition. This type of wells can supply small quantitities of water, but at minimum cost (Todd) .

4.5.3.1.2.2. Driven well

These wells have smaller diameter following the range of 1¼ to 4" and depth generally less than 50 ft., but in certain condition may exceed 100 ft. These wells are constructed bythe impact of column ofpipes having driven point at its lowest end. Water is extracted through this driven point. The pump is generally used by individuals for irrigation where water table is nearer, 10 to 15 ft. from the ground water. Yield is small.

4.5.3.1.2.3. Jetted well :

These wells more or less have generally same range of size and length as driven well, but it can be extended to 12" dia. and depth more than 50 ft. For construction of this type of well water is pumped inside the casing pipe through a smaller diameter of pipe kept within the casing pipe. Water coming with high velocity causes cutting action, at the same time washes the earth away and returns back through the spacing between the casing pipe and nozzle pipe. The wells have smaller discharge and are best adopted forunconsolidated formation.

Generally water is drawn through the pumps of centrifugal type and disel engine or electromotor. However there may be other type of pumps depending upon size of intake and discharg pipes. Discharge may vary from a few to 6000 gph (Todd). Hence such large variation of discharge needs further support for arriving at reasonable value of discharge. In the map available in project report locations of few pumping sets with their discharge is given. The discharge of these pumping sets are withi the range as described above. Hence average figure of discharge for each districts were computed on above figures for further work as needed for the computation of ground water drawl.

4.5.3.1.3. Private tubewells :

Fortunately in available map locations of 94 nos. of private tube wells with their discharge through unevently distributed in three districts have been shown. On basis of this, average discharge for both the seasons were computed on following basis.

1. It was assumed that discharge as shown on map of wells is maximum discharge which can hold good during Kharif season, when water table is high.

2. During dry season, discharge was reduced to eighty percent of maximum discharge when water table goes down.

3. Average discharge for each districts are good enough, if average discharge of all the wells located in the disctrict area as shown in map is computed.

Thus on above assumptions for all the three districts where well irrigation system exists, district wise pumpage rate of private tube wells was arrived at :-

4.5.3.1.4. Statetubewell :-

These are deep tube wells through which extraction of water from more than one strata is possible. Discharge of these type of wells have large range. For this type of well, districtwise figures of pumpage rate during Kharif as well as rabi season of previous years, prior to study years are available From the figures available in said respect it appears that there is large variation of discharge from year to year, probably based on variation in water requirement of crop. On further investigation it was found that the year which had higher rainfall, drawl was low, and vice verse. Under such situation an effort was made to select the year in which the rain fall in season is alike corresponding season of 1973, with a condition that complete data for the period are available, and value of discharge which was obtained was considered probable discharge of pump during the year in question.

4.5.3.2. Estimation of Number of Wells :

District wise figures of number of wells of all these 4 types of well for previous years (1968-1972) are available in the report. As laid down that every year the strength of wells showed increment, hence it was necessary to estimate probable no. of wells during the period of Kharif and rabi of the year 1973. This was achieved on following method.

Increment of wells in successive years was obtained Gradient of increment of successive years, and then average gradient of increment was found. Thus based on incremental method, no. of wells on Ist Jan. 1973, and June 1973 was separately estimated for all types of wells.

4.5.3.3. Estimation of Running Hour of Wells :-

Running hour of state tube well for previous years was available, but rest types of wells was wanting. Similar to other giures, this also necessary for computation of drawl. So this was estimated on following consideration.

4.5.3.3.1. State tube wells :

As discussed earlier, season wise figures of running hour of state tube well forprevious years are available. Running hour of the selected year as discussed was adopted for further computational work.

4.5.3.3.2. Other types of wells :

Saxena[43] in his work has computed depth of water as applied over the known area of Ram Ganga Doab through private irrigation system employing dug wells, Pumping sets andprivate tube wells. Figures of number of wells of all types of well and their discharge, as well as figures of state tube wells with respect to its total number average discharge, and running hour are available in his work. Based on these informations volume applied by private irrigation was obtained by taking product of area irrigated by said irrigation system and depth of water. Total volume of extraction of water through those wells per hour was computed by taking sum of products of total no. of wells of different types in question and their respective average discharge. Average running hour of private wells was obtained by simple division to total volume of irrigation by total drawl per hour. The relation between running hour ofprivate wells and state tube well was established in term of ratio. It was assumed that this ratio will hold good for private tube wells of the Varuna basin during Kharif when water table of the basin is high. For rabi season, above ratio was reduced to 80 percent of said ratio. For rabi season 80% running hour of private tube well was considered in case of pumping sets and dug well where as in rabi this percentage was further reduced due to much lowering of water table in latter types of well (dug well and pumping set) compared to their shallow depth.

With these estimated values, total drawl during Kharif as well as rabi computed as discussed below :-

4.5.3.4. Seasonwise Total drawl:

Based on the values of required component, as described in succeeding section, total district wise drawl of water through different types of well for both the seasons were separately computed by taking product of no. of respective types of well lying in the district, their running hour during the season, together with their discharge. Total drawl per unit area was further computed by dividing above value by the area of respective district.

However, the interest was in monthly rate of drawl through various types of wells for which further work was carried out as discussed below. Result and calculation for total drawl per unit area is available in table 4.2.(a).

4.5.3.5. Monthly Distribution of Drawl:

For computation of above it was assumed that probable monthly distribution of drawl will be corresponding to their theoretical requirement of water in different months by the crop which in turn is based up on consumptive use. On this approach monthwise theoretical requirement of water for the crop under the three districts were determined separately as follows :-

4.5.3.5.1. Theoretical water requirement for the crop :

The estimation of water requirement is based upon a following components :-

4.5.3.5.1.1. Consumptive Use :

Irrigation water which is applied over the soil, a part of it is removed from the soil by a plant, moved through the plant to the leaves and lost to the atmosphere from its surface. It is to note that plant hardly retains water more than 2% This process of loss is known as transpiration. Rest part of water is mostly lost through the soil by evaporation. Hence these combined operations, known as evaportranspiration consumes most of water during building of plant tissues. Thus consumptive use, which is actually a requirement of water needed for plant growth, is practically evaportranspiration loss.

<u>Evaportranspiration</u> :-

It is combination of both evaporation and ~~evapo~~transpiration, hence it is dependent on the factors associated with those two terms. Rate of transpiration is dependent upon moisture availability in the soil, kind and density of plant growth, amount of sunshine, temperature as well as wind movement, where as evaporation from the soil is dependentupon wind movement temperature and humidity. Hence on the approach of interrelationship between transpiration and evaporation from the soil, a co-relation between consumptive use and class A pan evaporation was determined which showed high degree of co-relation. On this theory, Hargreaves[42] proposed a function.

$$E = K. E_p$$

Where K is the consumptive use Co-efficient, E_t is evapotranspiration and E_p is the evaporation. Value of K proposed for 8 groups of crops at different stages of growth is available in table 8 of Book[44] " A Guide for Estimating Irrigation Water Requirement". With help of this table, crop wise requirement was computed using the equation described earlier. Computation of gross water requirement for each of representative crop of both season, Rice and wheat (Kharif/Rabi respectively) is shown in table 4.2.(b).

4.5.3.5.1.2. <u>Net Water Requirement</u>[42,44]

Rainfall which falls over the land meets the necessity of water requirement for the crop. However utilisation by crop and storage within the root zone of crop is dependent upon the consumptive use. Higher the rate of consumptive use, the greater will be rainfall effectiveness due to the fact for higher value of consumptive use available moisture gets depleted rapidly, and so provides storage capacity at relatively rapid rate for subsequent rainfall. Hence contribution of rainfall is to be evaluated based on following factor.

4.5.3.5.1.2.1. Effective Rainfall :

This is basically a fraction of rainfall, which is governed by consumptive use as well as net depth of application of water. Based on these factors, table of effective rainfall for different consumptive uses for 8 groups of crops against various depth of rainfall is available in the book[44] referred above. As the values in the table is based upon 75 mm. of net depth of application so far other depth of application values given in table are to be modified by the factor relating to other depth of application.

Hence based on above approach, effective rainfall for different crops of the basin, which are under irrigation system was computed. An example of same can be seen in table

4.5.3.5.1.2.2. Estimation:

Net irrigation requirement for each crop was obtained by substracting effective rainfall from consumptive use. Based on these values, monthly distribution work was carried out as follows :

4.5.3.5.2. Total District-wise monthly water requirement:

The district wise figures of total area covered by different crops both in Kharif and rabi season are available in the report. Based on these values, district wise monthly requirements of all the crops of aboth seasons were obtained by taking sum of products of crop areas and net water requirement for respective crop, during the month. When these monthly values were divided by the sum of the monthly values of respective season monthly distribution ratio was obtained.

4.5.3.5.3. Rate of Monthly drawl:

District wise monthly rate of drawl for hectare was obtained by multiplying monthly distribution ratio to total drawl of water per unit area of respective district for related season. However these values so obtained are based upon no. of wells on Ist Jan. Hence proportionate increase in rate of drawl beyond June was made corresponding to increase in no. of wells in June compared to January figure.

4.5.3.5.4. Monthly Drawl of Water :

Total monthly drawl through partial area of node lying in particular district was obtained by taking sum of products of area under different type of well irrigation and their rate of drawl. By taking sum of drawl through partial areas lying in different districts, required values of drawl through complete areas of different nodes were obtained.

These node wise monthly values so obtained were further added to obtain desired value for the lump model.

ON	RABI SEASON(JAN.73 TO MAY 73 & 16TH OCT.TO DEC.73)						KHARIF SEASON (JUNE 73 TO 15TH OCT. 1973		
of	No.of well in Jan.73	No.of well in Oct.73	Increment factor	Drawl in Ham.during season	Area Covered by well	Drawl per Hact.in HMx10^{-2}	Drawl during season	Area covered by well Hact.	Drawl per Hect. in H.M. x 10^{-2}
RICT-ALLAHABAD									
te Tube l	78	82	0.0514	2652	35,412	7.490	783	35,412	2.811
.TubeWell	460	521	0.118	1107	37,027	2.990	627	37,027	1.693
; Well	3503	3524	0.006	2260	37,997	5.948	1480	37,997	3.895
ping Set	263	284	0.058	332	36,382	0.993	229	36,382	0.630
CT-JAUNPUR									
ate Tube ll	132	138	0.044	3789	62,870	6.027	1816	62,870	2.889
;.TubeWell	1025	1141	0.104	1937	67,825	2.856	1410	67,825	2.079
g Well	10603	10764	0.016	7467	86,395	8.643	5869	86,395	6.793
nping Set	1000	1101	0.101	1549	71690	2.161	1320	71,690	1.841
ICT-VARANASI									
ate Tube ll	207	224	0.082	8860	105,850	8.370	4770	105,850	4.506
t.TubeWell	1130	1255	0.110	2425	105,850	2.291	1710	105,850	1.616
g Well	6589	6714	0.019	4192	105,850	3.960	3890	105,850	3.675
mping Set	1364	1367	0.002	1786	105,850	1.687	1630	105,850	1.540

Further Distribution of rate of drawl through various types of wells and pump were made as per consumptive use of water by the Crop under different months.

DISTRICTS LYING IN THE BASIN (RICE CROP)

AD DISTRICT		JAUNPUR DISTRICT			VARANASI DISTRICT		
ffec- ive rainfall mm	Net water reqt. mm	Rain-fall mm	Effective rainfall mm	Net water require-ment mm	Rain-fall mm	Effective rainfall mm	Net water requiremen mm
20.1	151.9	28.6	28.6	143.4	15.6	15.6	156.4
20.1	168.8	28.6	28.6	161.3	15.6	15.6	173.4
181.0	93.0	318.6	181.0	93.0	312.2	181.0	93.0
157.1	93.0	299.1	157.1	93.0	243.3	150.2	93.0
89.8	133.2	209.9	131.2	91.8	274.6	133.0	90.0
-	41.1	-	-	41.1	-	-	41.0

WHEAT CROP

-	3.69	-	-	3.69	-	-	3.69
-	19.53	-	-	19.53	-	-	19.53
4.4	39.5	12.3	7.8	34.1	4.6	3.5	38.4
0.4	82.3	8.8	6.3	76.4	4.8	3.45	79.25
-	76.4	-	-	76.4	-	-	76.4
-	55.8	-	-	55.8	-	-	55.8

half of month (16-31st) 40% area under plantation and 5% nursery.

TABLE - 4.2(b)

TABLE SHOWING COMPUTATION OF WATER REQUIREMENT FOR RICE CROP UNDER DIFFERENT DISTRICTS LYING IN THE BASIN (RICE CROP)

PERIOD	Mid Point	Percent of growing season	Value of K	Pan Evaporation Rate mm	Consumptive use mm	Special requirement	Percolation mm	Total requirement	ALLAHABAD DISTRICT			JAUNPUR DISTRICT			VARANASI DISTRICT		
									Rain-fall mm	Effective rainfall mm	Net water reqt. mm	Rain-fall mm	Effective rainfall mm	Net water requirement mm	Rain-fall mm	Effective rainfall mm	Net water requiremen mm
June 1 to 15	7 days	5.38 days	0.9	141.0	127.0	-	45.0	172 mm	20.1	20.1	151.9	28.6	28.6	143.4	15.6	15.6	156.4
June 16-30	22 "	16.9 "	1.02	141.0	143.9	-	45.0	188.9	20.1	20.1	168.8	28.6	28.6	161.3	15.6	15.6	173.4
July	45 "	34.6 "	1.17	155.0	189.0	-	93.0	274.0	282.2	181.0	93.0	318.6	181.0	93.0	312.2	181.0	93.0
August	77 "	59.1 "	1.30	120.7	157.1	-	93.0	230.1	350.2	157.1	93.0	299.1	157.1	93.0	243.3	150.2	93.0
September	107 "	82.1 "	1.08	123.0	133.0	-	90.0	223	122.2	89.8	133.2	209.9	131.2	91.8	274.6	133.0	90.0
October	126 "	97.0 "	0.56	304	17.1	-	24.0	41.1	-	-	41.1	-	-	41.1	-	-	41.0
Total 130 days																	

WHEAT CROP

60 mm for land preparation

PERIOD	Mid Point	Percent of growing season	Value of K	Pan Evaporation Rate mm	Consumptive use mm	Special requirement	Percolation mm	Total requirement	Allahabad Rain-fall mm	Allahabad Effective rainfall mm	Allahabad Net water reqt. mm	Jaunpur Rain-fall mm	Jaunpur Effective rainfall mm	Jaunpur Net water requirement mm	Varanasi Rain-fall mm	Varanasi Effective rainfall mm	Varanasi Net water requiremen mm
November 16-30	7 days	5.15%	0.082	45.0	3.69	-	-	3.69	-	-	3.69	-	-	3.69	-	-	3.69
Dec.	31 "	22.60	0.30	65.1	19.53	-	-	19.53	-	-	19.53	-	-	19.53	-	-	19.53
January	62	45.60	0.588	71.3	41.9	-	-	41.9	6.2	4.4	39.5	12.3	7.8	34.1	4.6	3.5	38.4
February	91	67.00	0.844	98.0	82.7	-	-	82.7	0.6	0.4	82.3	8.8	6.3	76.4	4.8	3.45	79.25
March 1-15	112	82.5	0.85	89.9	76.4	-	-	76.4	-	-	76.4	-	-	76.4	-	-	76.4
March 16-31	128	94.0	0.62	89.9	55.8	-	-	55.8	-	-	55.8	-	-	55.8	-	-	55.8

NOTE (A) In June, 50% area is under nursery for early half 1 to 15, where as in latter half of month (16-31st) 40% area under plantation and 5% nursery.

(B) For land preparation - one third area considered.

TABLE SHOWING MONTHLY DRAWL THROUGH DIFFERENT TYPES OF WELL (COMBINED VALUE) IN HECTARE-METRE DURING MONTH

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10	Total ing mo
January	305.16	487.89	800.83	239.92	658.52	826,33	490.79	489.63	337.34	306.02	4642.1
February	509.3	671.2	1136.2	318.1	867.3	601.6	595.1	564.5	383.9	348.2	5996.0
March	1165.6	1526.3	2669.4	721.2	1979.3	1583.0	1475.8	1472.3	1014.5	920.4	14527.8
April	123.6	319.1	507.3	167.8	467.0	479.4	407.8	439.9	309.3	280.5	3501.7
May	111.0	286.4	454.9	150.7	419.2	429.1	365.3	393.8	276.8	251.1	3138.3
June	695.8	2019.2	2516.0	1139.6	2858.6	1512.6	1686.4	1450.0	955.4	866.7	15700.3
July	173.5	258.7	468.0	133.3	346.1	383.8	317.0	350.6	248.0	225.0	2904.0
August	173.5	258.7	468.0	133.3	346.1	383.8	317.0	350.6	248.0	225.0	2904.0
September	245.0	272.7	534.0	131.2	340.0	367.5	306.1	336.1	237.4	215.3	2985.3
October	120.1	210.9	366.5	108.8	292.2	316.7	264.0	289.8	204.6	185.4	2359.0
November	255.5	381.8	619.3	184.6	506.6	350.4	347.0	329.1	233.6	202.8	3410.[illegible]
December	246.8	336.1	560.7	159.4	437.1	298.1	297.1	280.2	190.0	172.4	2977.[illegible]
	4124.9	7028.7	11101.1	3587.9	9518.6	7232.3	6869.4	6746.5	4638.8	4198.8	65047.[illegible]

4.5.4. Recharge due to well Irrigation:

Actually theoretical basis of computation of recharge due to irrigation through wells could not be known. However there is a common practice to adopt certain percentage of water drawl through well for estimation of recharge. Recharge is actually return flow of water to the reservoir from where it was extracted. However it is to submit that all the water which infiltrates through the soil does not reach water table. These are several factors, which influence the recharge A few of which is discussed here as needed for consideration of recharge as percentage of drawl.

4.5.4.1. Seepage Loss:

Water is applied over the field, through water courses, for the irrigation. There is naturally seepage loss through these channels alike the water courses under canal irrigation, system. Hence same percentage of loss in case of water course for well irrigation was also considered.

4.5.4.2. Field Loss:

It has already been described that field loss depend upon so many factors. The field loss under well irrigation system is not same as canal irrigation due to following reasons.

1. All the wells except state tube wells are mostly owned by cultivators. So they use economically as they are aware of the labour and cost of watering.

2. Location of well in central place, minimises the length of water course.

3. Use of water at proper time, and only when actually needed minimises further loss during irrigation.

On these considerations, it is expected that efficiency in irrigation may be higher than irrigation under canal system. So, 5% higher efficiency compared to canal irrigation efficiency was assumed, on the other hand, due to above conditions, evaporation loss will be on lower side, and for which 15% of evaporation loss was considered against 20% loss under canal system.

4.5.4.3. <u>Computation of Recharge</u> :

On basis of above considerations percentage recharge has been computed as described below:

1. Conveyence loss (seepage loss + evaporation loss)

2. Field loss (1-0.65) where 0.65 is field irrigation efficiency) of discharge reaching to the field (100- conveyance loss) = 80%) 28%

3. Deduct 15% losses due to evaporation and evapotranspiration = - ... = -7.2

40.8

The gain to ground water storage due to recharge through well irrigation under different nodal areas were computed by taking 40.8 percent of total ground water drawl in respective nodal areas. All the values of recharge under different nodes were added for obtaining required value of above component for the use in lumped model. Result of same is available in table 4.3.

TABLE SHOWING MONTHLY RECHARGE FROM MINOR IRRIGATION(WELLS&PUMPS) UNDER DIFFERENT NODES IN HECT.METR

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10	T
January	125.1	199.9	328.3	98.4	270.0	215.8	201.2	200.8	138.3	125.4	19
February	208.8	275.2	465.8	130.4	355.8	246.7	244.0	231.4	157.4	142.8	24
March	477.9	625.8	1094.5	295.7	811.5	649.0	605.1	603.6	415.9	377.4	59
April	50.7	130.8	208.0	68.8	191.5	196.6	167.2	180.4	126.8	112.2	14
May	45.5	117.4	186.5	61.8	171.9	175.9	149.8	161.4	113.5	102.9	12
June	285.3	827.9	1031.6	467.2	1172.0	620.2	691.4	594.5	391.7	355.3	64
July	71.1	106.1	191.9	54.6	141.9	157.3	130.0	143.8	101.7	92.3	11
August	71.1	106.1	191.8	54.6	141.9	157.3	130.0	143.8	101.7	92.3	11
September	100.5	111.8	218.9	53.8	139.4	150.6	125.5	137.8	97.3	88.3	12
October	49.2	86.5	150.3	44.6	119.8	129.8	108.2	118.8	83.9	76.0	9
November	104.8	156.6	253.9	75.7	207.7	143.7	142.4	134.9	91.6	83.1	13
December	101.2	137.8	229.9	65.4	179.2	122.2	121.8	114.9	77.9	70.7	12
	1691.2	2881.9	4551.4	1471.0	3902.6	2965.1	2816.6	2766.1	1897.7	1718.7	266

4.5.5. Percolation from lakes and ponds:

4.5.5.1. Area & Depth:

It was assumed that area covered by lakes and ponds are 2½% of total area, and average depth of water in lake on Ist November is 20 metre. Further it was assumed that this is evenly distributed in entire area, and to lake areas under different nodes were computed using above percentage.

4.5.5.2. Permeability of Media:

For consideration of permeability, the following views as extracted from book is reproduced. " Alluvial plain further from high land may yield lesser water than the case of alluvial fans. However these planes have developable ground water reservoir unless the plain were formed by slowly moving streams depositing a high percentage of clay and sand". On the said process it is concluded that in the lake or tank which is still water body, silt and clay have been laid down.

By examination of specific yield of aquifer obtained by pump test (0.072) and comparing the same with the figure given in table under reference (47) it can be said that upper layer of aquifer contains either silt or sand. These two variations gave rise for consideration of media between lake surface and water table, as composite of three layers. Based on above consideration following assumptions were made.

Layer	Depth below lake bottom	Type of soil	Permeability
Ist layer	upto 1 metre	silly clay	4.0×10^{-6}
2nd layer	1-2	clay silt	1.0×10^{-5}
3rd layer	2 to water table	clay-sand or silt	5×10^{-3}

On basis of permeabilities of above three layers, mean permeability was computed in the process of computation using following equation[45].

$$\text{Mean permeability } K = \frac{T}{\frac{T_1}{K_1} + \frac{T_2}{K_2} + \frac{T_3}{K_3}}$$

Where T is the total mean depth of media, between water surface and water table at any stage, T_1, T_2, T_3 are respectively thickness of Ist, 2nd, 3rd layers, as defined earlier and K_1, K_2, K_3 are permeabilities of relative layers.

4.5.5.3. Computation of Percolation :

As evaporation takes place simultaneously from water surface direct computation of percolation was not possible. Hence under such situation iterative method was adopted as described below.

4.5.5.3.1. Theory:

i) It was assumed that as the lake area is large, so width of lake compared to depth, the flow lines are vertical for practical consideration. Hence percolation rate can be computed using following equation, based on Darcy's law.

$$Q = A \times \quad = A \times K \times i = A \times K \times Z/T$$

Where K is the mean permeability, i is the hydraulic gradient, Z is the head above water table i.e. distance between lake surface and water table, T is the total mean depth of media.

ii) Evaporation rate from water surface, taken as 0.7 times pan evaporation rate, as given in the project respect.

Evaporation loss during month = A x E x 0.7 where A is the average lake area during the month, and E is the pan evaporation rate.

5.5.5.3.2. Procedure :

Based on these two equations, loss during month was obtained by cut and by method ~~by~~ in computing reduced volume, depth of water at the end for reduced volume, lake area, percolation loss, evaporation loss, total loss, and volume of lake water at the end. Error of 1 hect. metre was considered to end iteration and proceed further.

Thus percolation loss under all the ten nodes during November to May was calculated.

Recharge due to Percolation:-

All the water percolating towards water table do not actually reach ground water storage, but a part of it is lost during transit. Hence based on present practice recharge due to percolation under different nodes were obtained by taking eighty per cent of percolation loss. Values of monthly recharge so obtained for different nodes were further added to achieve desired value of recharge for entire basin to use the same in lump model. Result as discussed above has been produced in table 4.4.

4.5.6. Evapotranspiration from Water Logged Area :

In certain areas either due to excessive irrigation or canal seepage or excessive rainfall on one hand or lower porosity and/or poor permeability on the otherhand, water table rises in root zone, even upto top of surface. Under such

TABLE - 4.4

TABLE SHOWING MONTHLY PERCOLATION FROM LAKES AND PONDS UNDER DIFFERENT NODES IN HECT.METRES

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10	Tota duri mont
January	175.1	262.1	353.7	79.6	201.2	288.6	229.7	241.4	218.9	115.5	2165.
February	192.5	289.6	397.4	98.2	235.4	335.9	254.5	277.8	252.6	131.5	2465.
March	91.1	133.5		51.5	110.5					59.9	446.
April											
May											
June											
July											
August	FROM JUNE TO OCTOBER DURING MONSOON -										
September	CONSIDERED UNDER RECHARGE THROUGH RAINFALL DUE TO COMPLICATE PHENOMENON										
October											
November	100.3	149.2	252.6	54.6	145.4	217.6	179.2	187.4	172.0	78.0	1536.
December	136.1	201.0	300.0	66.0	174.3	250.1	201.3	212.5	191.4	90.4	1823.
	695.1	1035.4	1303.7	349.9	866.8	1092.2	864.7	919.1	834.9	475.3	8437.

situation when water table is near the surface, the area is said to be water logged. In such areas, plants known as phreatophytes commonly extract water through the root system. It has been observed by the hydrologists that evaportranspiration quantity is affected by the depth to ground water, vegetations types and quantities, in addition to other factors already discussed in previous chapter. David C. Lewis(35) has stated that total transpiration decreases as the depth to water table increases in the range of 2 to 8 ft.

4.5.6.1. Effective depth of water table :

Based on above discussion, it was considered that evapotranspiration in water logged area is more prominent in root zone, and so depth of water table upto 2 metre will be reasonabl depth for the said process. Beyond above, the loss can be ignore

4.5.6.2. Estimation of Evapotranspiration Loss :

On above criterion of effective depth, water logged areas were located on the basis of observed water level data, during different months.

It will appear from that table 4.5 that major areas of water logged lies in nodes 2 and 4, where extensive canal irrigation system exists. It is also to note that water logging condition exists during September to December only. The water logged areas so demarcated were planimitered.

Based on evapotranspiration rate as per actual observation, available in project report, evapotranspiration loss was computed by taking the product of water logged area under effected node, and evaportranspiration rate of relative month.

TABLE 4.5

TABLE SHOWING COMPUTATION OF POTENTIAL EVAPOTRANSPIRATION LOSS FROM WATER LOGGED AREA IN BASIN

	SEPTEMBER		OCTOBER		NOVEMBER		DECEMBER		
Water logged	Area	P.Evap. Tr.Loss	Area in Hect.	Pot.Evap.Tr. Loss	Area in Hect.	Evap.Trans. Loss	Area	Evap.trans. Loss	Total
NODES	(Evaporation Rate-12m) Total During Month		(12.18 Cm.in month) H.M.		8.026 Cm.in month H.M.		5.793 Cm in month H.M.		Ev.Tr.Los Hect,M.
2	-	-	600	73.0	14970 Hect.	1201.5	540	31.3	1305.8
4	4800	576.0	9438	1149.5	24160	1939.1	3570	206.8	3871.4
5	-	-	240	29.2	746	59.8	-	-	99.0
10	5	0.6	5	0.6	5	0.4	5	0.3	1.9
		576.6		1252.3		3200.8		238.4	5268.1

Finally these monthwise losses under different nodal areas were added together to obtain total loss during respective month for use the same in lump model.

4.5.7. Evapotranspiration loss from forest area:

In the forest, greater density of trees and shurbs exists. In case of occurrence of these forest near the banks of streams and valleys or other areas where the water table is high, the roots of vegetation penetrate into capillary fringe. Thus these trees and shurbs draw water from ground water body, and water is lost through transpiration. Evaporation loss from soil also becomes effective in such zones. Thus evaportranspiration loss, depending upon factor as discussed earlier takes place. Hence for estimation of same, following works were carried on.

4.5.7.1. Forest area :

For want of actual figures of forest areas within the basin, it was assumed that 3% area of the basin is covered by the forest, and distribution of same is uniform throughout the basin. Based on this assumption forest area under each node was considered as three percent of its nodal area.

4.5.7.2. Estimation of Evapotranspiration loss :-

Based on evapotranspiration rate of different months, nodewise monthly evaportranspiration loss was computed in the beginning by taking product of forest area and loss rate. The monthwise loss under different nodes was added together to obtain required monthly loss for the entire basin to employ the same in the lump model.

TABLE - 4.6 FIGURES IN HECT.METRE

COMPUTATION OF EVAPOTRANSPIRATION FROM ORCHARD,FOREST AND BIG TREES AREA UNDER THE BASIN

Month	Evapo.Range Rate in a month AREA	Node 1 630 HECT	Node 2 1080 H	Node 3 1130HECT	Node 4 A777	Node 5 939	Node 6 788	Node 7 690	Node 8 720	Node 9 504	Node 10 486	Tot 257
January	62.06 mm	39.0 HM	66.6 HM	70.8 HM	58.0 HM	58.5 HM	48.6 HM	42.9HM	44.7HM	31.2 HM	28.5	478.8
February	90.06	56.7	97.2	102.6	72.6	84.6	70.5	62.1	64.8	45.3	39.6	696.
March	151.43	95.4	163.5	172.8	117.0	142.5	118.5	104.4	109.2	76.2	69.0	1168.
April	195.43	123.0	210.9	222.9	150.6	184.2	153.0	129.0	141.0	92.4	88.5	1495.
May	273.2	172.5	295.2	311.1	210.3	257.1	213.6	188.7	196.5	137.5	124.5	210
June	198.56	124.8	214.2	225.3	153.0	186.0	155.1	136.5	142.5	99.9	91.5	152
July	128.9	81.3	139.2	147.0	99.6	121.5	100.8	89.1	93.0	64.8	58.8	995.
August	113.6	73.1	123.0	129.6	87.6	106.8	89.1	78.6	81.9	57.3	51.9	876.
September	120.0	75.6	129.6	136.8	92.4	112.8	93.9	82.8	86.4	60.6	54.6	925.
October	121.76	76.8	131.7	138.9	93.3	114.6	95.4	84.0	87.6	61.5	55.5	939.
November	80.26	50.4	86.7	91.5	60.9	75.3	62.7	55.5	57.9	40.5	36.6	618
December	57.93	36.6	62.4	66.0	44.7	54.6	45.3	39.9	41.7	29.1	26.4	446.
TOTAL		1003.2	1720.2	1815.3	1230.0	1498.5	1246.5	1093.5	1147.2	796.5	7254	1227

NOTE: Area covered by Forest, Orchard etc.assumed as 3% of Nodal Area

4.5.8. Inflow and Outflow to/from the basin:

The lump model, represents the entire basin as a unit. Hence sub-surface flow across any boundary within basin area is not accounted. The sub-surface flow which is important for consideration is flow across outer boundary of the basin, due to hydrological connection with adjacent basin.

For such consideration, detailed information regarding hydrological and geological condition of other basin was necessary, but the same was not available for present study. Hence following procedure was adopted.

4.5.8.1. Flow Lines drawl:

From available water level data, contour maps of water table for different months were prepared, for the entire basin. For the study contour lines along the boundary of the basin were selected. Flow lines were drawn orthogonally at selected points on the contour lines and inflow or outflow to basin was decided on basis of gradient of the contours. Length of contour along the basin boundary which indicated flow were measured nodewise. Average gradient, on the basis of distance between a pair of contours at several places and their difference in head, within the nodal areas were computed. When these were known discharge difference across the boundary lines were computed as described below:

4.5.8.2. Estimation of Net inflow or outflow:

For computation of inflow and outflow across the basin boundary following equation was employed.

TABLE - 4.7

FIGURES IN HECT.METRES

TABLE SHOWING MONTHLY INFLOW AND OUTFLOW FROM/TO OTHER BASIN

	INFLOW TO VARUNA GR.WATER BASIN				OUT FLOW TO OTHER BASIN				
Month	Node 1	Node 2	Node 10	Total Inflow	Node 4	Node 5	Node 8	Node 9	Total
January	51.50	88.60	66.13	206.23	58.70	9.80	53.96	223.60	346.06
February	52.10	79.49	73.22	204.81	72.32	11.70	63.51	241.07	388.60
March	40.80	79.16	49.12	169.08	57.40	12.77	58.76	218.57	347.50
April	28.50	78.82	25.02	132.34	42.48	13.76	54.00	196.07	306.31
May	15.11	78.46	0.91	94.48	27.55	14.76	49.23	173.57	265.11
June	16.04	90.80	16.11	122.95	26.55	13.06	45.22	214.80	299.63
July	16.97	103.14	31.29	151.40	25.55	11.36	41.22	214.80	~~299.63~~ 334.26
August	17.94	115.49	46.49	179.92	24.54	9.87	37.20	297.66	369.27
September	28.70	111.32	49.97	189.99	26.78	8.41	36.42	261.19	332.80
October	38.45	107.15	51.45	197.05	29.02	7.25	35.68	214.82	286.77
November	50.19	102.99	53.93	207.11	31.26	5.88	34.86	188.66	260.66
December	50.19	95.10	60.03	205.32	44.98	7.84	34.41	206.13	303.36
Total in Year	406.59	1130.52	523.67	2060.68	467.13	126.53	554.46	2692.27	3840.43

$$Q = A \times V = l \times d \times v = l \times (d \times k) \times i = L \times T \times i$$

Final equation comes to $Q = l \times T \times \Delta h / \Delta S$. Where Q is flow where l is length of contour within respective nodal area.

T is transmissibility i.e. product of permeability K and depth d

$\Delta h / \Delta s = i =$ gradient

Using above equation monthwise inflows or outflows to/from the basin were computed for effected nodal areas. Monthwise values for different nodes were then summed up to obtain desired monthly values of net inflow as well as out flow for lump model.

4.5.9. Influent seepage:

There are numbers of stream associated with the Varuna River. However neither their discharge nor their sections were given in project report. Hence following steps were followed to estimate seepage loss.

1. Drawl of catchment area on the map.
2. Estimation of average runoff in stream
3. Determination of wetted perimeter
4. Computation of seepage loss.

4.5.9.1. Catchment Area :

On the consideration of point of confluence of tributaries the Varuna River was divided into three reaches, and catchment of tributaries were first marked, keeping in view of boundary lines. The area left after such demarcation on either side of parent river was considered as a portion of catchment directly contributing runoff to the main stream.

Then catchment areas of branches, and reaches of main so demarcated were planimetered.

4.5.9.2. Estimation of Runoff:

For Indian condition strange[40] has established relationship of total volume of runoff over relatively long period, with rainfall by certain percentage. For this purpose he has also divided the catchment into "good"," average" or "bad" depending upon runoff yielding capacity. On the basis of values given by him vide table 14.6 of Booch by Bharat Singh[40], runoff was computed as discussed below.

Considering catchment of basin as good, for annual rainfall of 855 mm over the basin, runoff factor as interpolated from above mentioned table, comes to 0.30. On basis of above two components, rainfall contribution was computed by taking products of catchment area, rainfall and runoff factor. Runoff in the main stream was computed by incorporating the runoff of its branches in successive order. The average discharge for entire length/reach of individual streams were determined as per table 4.8.(a)

4.5.9.3. Computation of Pemmeter:

All time of high flood a river is temporarily in regime. For such case Iarcy has described relationship between regime. Mean velocity Q and hydraulic mean depth R $V = CR^{\frac{1}{2}}$ where C is constant when the silt factor is constant, Lindly[40] then pointed out about possibility of existence of relationship between regime velocity and width as well. Iarcy[40] on this approach carried out finite study and developed an equation as described below. With this equation wetted perimeter of channel can be determined by considering discharge alone.

$$P = 47.5\ Q^{\frac{1}{2}}$$

Where P is wetted perimeter, Q is discharge in cumecs. Using this equation for wetted perimeter of all the streams main and branch as were calculated and for small value of wetted perimeter was assumed and then converted in f.p.s. unit.

4.5.9.4. Computation of Seepage loss:-

Etevery and Harding[40] suggested values for seepage loss for different characters of materials. From the table available in book by Bharat Singh at P.57 value of conveyance loss. at the rate of 4 cusecs for million sq.ft. of wetted perimeter was arrived at by consideration of river having clay loam. It was assumed that influent seepage is effective during rainy season only and for rest period effluent seepage takes place. The length of channel were determined by measurement of stream length as shown in map lying in different nodal areas 10% additional length also considered as the stream are flowing in Zig Zag way.

Hence based on these consideration seepage loss per mile per ft. width of per mile was calculated first as described below. Conveyance loss per million square ft. of wetted perimeter = 4.0 cusecs.

Deduct 20% for evaporation & evapotranspiration loss = 0.8

Net recharge due to seepage from stream = 3.2

Recharge due to seepage per mile length for unit width of perimeter $= \frac{5280 \times 3.2}{10^6} = 1.46 \times 10^{-2}$ cusecs

Recharge for seepage loss from stream during a month per mile length with 1 ft. wetted perimeter

$$= 1.46 \times 10^{-2} \times 30 \times 3600 \times 24 \times 0.2892 \times 10^{-4}$$

$$= 0.1265 \text{ HM}$$

Add 10% extra for Zig Zag length of stream $= \underline{0.0126}$

Total Recharge rate $= 0.139$ HM

Subsequently recharge due to seepage was computed by taking product of length in mile and wetted perimeter in ft. for the portion of stream lying in the nodal area in question. These values of recharge from all the streams lying in the node was added to obtain total recharge under the node. Finally values so obtained for different nodes were combined to obtain desired value of recharge for the lump model.

4.5.10. Change in Ground Water Storage[35]

Though the change in moisture content in the zone of aeration takes place, but determination of water content is not possible. Secondly the amount of water in unsaturated zone at the beginning and end of period is same. In irrigated area period of limits are beginning and end of irrigation season. Hence based on above considered for practical purposes change in storage is limited for saturated zone only.

TABLE – 4.8(a)

TABLE SHOWING COMPUTATION OF RECHARGE DUE TO INFLUENT SEEPAGE FROM STREAMS LYING IN VARUNA BASIN FOR MONTH – SEPTEMBER

A. STREAM PARTICULARS	MAIN STREAM – VARUNA RIVER: 1st Reach 0-26 Mile	MAIN STREAM – VARUNA RIVER: 2nd Reach 26-80 mile	MAIN STREAM – VARUNA RIVER: 3rd Reach 80-106 mile	Right Tributary R_1	Right ~~Left~~ Tributary R_2	Left Tributary L	Minors	REMARKS
Catchment area of Stream	38400 Hect.	34,260 HECT.	95,400 Hect.	28200 Hect.	55,500 Hect.	97,000 Hect.		Rainfall =219.5 mm
Total runoff volume of rainfall	2519×10^4 cu.mt.	2248×10^4 cu.mt.	6260×10^4 cu.mt.	1850×10^4 cu.m.	3641×10^4 cu.m.	6364 cu.m.	Perimeter	
Runoff contribution in Reach/stream	9.77 cumecs	8.71 cumecs	24.25 cumecs	7.18 cu.m.	14.1	24.66	Assumed	Runoff Factor = 0.30
Discharge at Upper End	1.95 cumecs	16.96 cumecs	64.43 cumecs	1.43 cu.m.	nil	nil		
Mean discharge in cumecs	5.86 "	21.31 "	76.54 "	5.02 "	7.06	12.33		
Perimeter of stream/ reach in ft.	37.7 feet	71.85 feet	136.2 feet	34.98 feet	41.40 feet	71.85 feet		

B. RECHARGE COMPUTATION UNDER

Node	Nodal Area	Length in mile	Recharge H.M.	Length mile	Recharge H.M.	Length mile	Recharge H.M.	Length mile	Recharge H.M.	Length mile	Recharge H.M.	Length mile	Recharge H.M.	Length mile	Recharge H.M.	Total Recharge
1		12.5	63.7	–	–	–	–	–	–	–	–	–	–	1.25	3.3	69.0
2		9.0	47.4	–	–	–	–	–	–	–	–	14.0	106.4	11.25	29.3	182.9
3		4.5	23.4	22	220.0	–	–	10	49.0	–	–	–	–	1.0	2.6	295.0
4		–	–	–	–	–	–	–	–	–	–	12.0	91.0	6.0	18.8	106.8
5		–	–	10.5	105.0	–	–	–	–	~~14~~	~~80.9~~	16.1	121.2	7.6	19.8	246.0
6		–	–	6.5	65.0	–	–	–	–	14	80.9	–	–	–	–	145.9
7		–	–	–	–	10	188.6	–	–	–	–	8.2	62.0	–	–	250.6
8				15.0	150.0					6.2	35.2					185.2
9						10	188.6									188.6
10						6	113.3									113.3

TABLE – 4.8(b)

FIGURES IN HECT METRES

TABLE SHOWING MONTHLY INFLUENT SEEPAGE FROM STREAMS AND RIVER DURING MONSOON TO DIFFERENT NODES

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10	Total
June	36.2	95.4	149.0	54.3	125.1	74.7	126.4	93.1	96.0	57.6	907.8
July	81.5	217.6	344.8	127.1	290.8	171.4	296.4	216.4	224.0	134.4	2104.4
August	76.5	204.6	29.2	120.6	275.5	165.0	282.0	209.3	214.2	128.4	2005.3
September	69.0	182.8	295.0	106.8	246.0	145.9	250.6	185.2	189.6	113.3	1783.2
TOTAL	255.1	677.5	1081.6	394.3	906.6	537.4	922.1	678.7	697.0	418.4	6568.7

Change in storage as used here means net addition of water or net withdrawl of water from an aquifer. Water table fluctuations are result of change intotal ground water storage. The fluctuation of water table may be gradual in case of natural recharge or discharge such as rainfall recharge or discharge by spring. Where as in case of sudden rise in surface water level of river or canal, the increase in storage will be abrupt similar is the case for discharge through pumps. On this very approach computation of change in storage was made as described below.

4.5.10.1. <u>Construction of Contour Maps of Water Table Fluctuation</u>:

Difference of water level at each observation wells, between successive months were computed. This showed water table fluctuation at the point of observation well, during the month. Contour maps for water table fluctuation for all the 12 months were prepared for the entire basin.

4.5.10.2. <u>Computation of Change in Storage</u> :

For different months areas lying between two contours as shown on maps were planimitered and mean depth of water table fluctuation (rise/fall) were noted by taking average of both the contours. Change in storage was obtained by taking product of areas between contours and mean height of change. Subsequently these were added algebrically to obtain net value of change in storage for entire basin.

As water is actually stored in the pore spaces of soil only, hence this quantity was multipliyed by storage co-efficient, to obtain net volume of change in water content in ground water storage.

Thus for all the 12 months change in storage was determined, result of which has been shown in table 4.9.

4.6. Overall Ground Water Balance :

It is to note here that values of two components are still undermined. These are following components:

1. Recharge due to rainfall
2. Effluent seepage to stream

Hence prior to balance study these two components were to be determined for which following studies were carried out.

4.6.1. Computation of Monthly Net flow :

Monthly values of all the components as described earlier were tabulated (vide table 4.10) and net flow was determined. It was further compared with change in quantity of ground water storage, and difference obtained for the respective month. These difference in quantity in various months was supposed to be due to non-inclusion of these two components otherwise it would have been equalised. In order to evaluate these components, seperation of base flow was considered as desirable. This was achieved on following concept.

4.6.2. Estimation of Effluent Seepage to Stream:

4.6.2.1. Basic Theory :- Regarding ground water recession following views are reproduced (RICHARD)[35]

" Following a seasonal high ground water elevation related to recharge, a natural ground water recession may be seen in the well hydrograph. The rate of decline in the water level is a function of the local values of hydraulic conductivity, storage co-efficient, aquifer thickness and hydraulic gradient.

TABLE – 4.9

TABLE SHOWING CHANGE IN GROUND WATER STORAGE DURING DIFFERENT MONTHS

Month	Storage effected	INCREASE IN STORAGE Volumetric Change Hect.Mets	INCREASE IN STORAGE Change in Water content Hect.Mets	DECREASE IN STORAGE Volumetric change Hect.Mets	DECREASE IN STORAGE Change in Water content Hect.Mets.
January	0.072			36,694.9	2498.8
February	"			42,891.7	3088.2
March	"			1,53,211.5	11031.2
April	"			73,259.1	5274.4
May	"			75,638.9	5446.0
June	"			1,51,035.5	10874.6
July	"			1,465.6	105.6
August	"	6,30,608	45,403.3		
September	"	66,579	4,793.7		
October	"	1,85,232	13,336.7		
November	"			84,952.8	6110.6
December	"			32,613.0	2348.1
Total			63,533.7		46,666.2

Net change in ground water storage = Increase in Qty. = 16,868

UNA BASIN (MONTHWISE) FIGURES IN HECTOMETRES

Month	THE BASIN Drawl of water by wells and pumps	Total outflow	Inflow minus outflow	Residual flow	Net flow
Janua	4642.1	8467.0	-1041.1	-1457.8	-2498.9
Febru	5996.0	7080.6	-1778.1	-1310.0	-3088.2
March	14527.8	16043.8	-9338.2	-1693.0	-11031.2
April	3501.7	5303.5	-3669.9	-1604.2	-5274.1
May	3138.3	5510.6	-3933.8	-1512.2	-5446.0
June	15700.3	17528.7	-9811.2	-1063.4	-10874.6
July	2904.0	4233.4	-535.0	+429.4	-105.6
Augus	2900.0	4150.2	-507.6	+45910.9	+45403.3
Sept.	2985.3	4820.2	-1479.7	+6273.4	+4793.7
Oct.	2359.0	4837.4	-3444.1	+16880.8	+1336.7
Nov.	3410.7	7490.2	-4076.0	-2035.8	-6116.6
Dec.	2977.9	3966.4	-410.0	-1938.1	-2348.1
Total Year	86432.0		-40024.7		+16862

culated in Table 5.17

TABLE – 4.10

TABLE SHOWING INFLOW AND OUTFLOW OF VARIOUS COMPONENTS OF HYDROLOGICAL CYCLE OF VARUNA BASIN (MONTHWISE)

FIGURES IN HECTOMETRES

Month	INFLOW TO GROUND WATER BASIN							OUTFLOW FROM THE BASIN							
	Recharge from canal seepage	Recharge from canal irrigation	Recharge from minor irrigation	Inflow from other basin	Influent seepage from stream	Percolation from lakes, Ponds	Total Inflow	Outflow to other basin	Evap. transpirn. from water logged area	Evap. transpirn. from forest orchard	Drawl of water by wells and pumps	Total outflow	Inflow minus outflow	Residual flow	Net flow
January	120.9	28.8	1903.2	206.2	–	2163.8	4425.9	346.1	–	478.8	4642.1	8467.0	-1041.1	-1457.8	-2498.9
February	121.8	52.2	2458.3	20 4.8	–	2465.4	3382.5	388.6	–	696.0	5996.0	7080.6	-1778.1	-1310.0	-3088.2
March	118.1	15.5	5956.4	169.1	–	446.5	6705.6	347.5	–	1168.[illegible]	14527.8	16043.8	-9338.2	-1693.0	-11031.2
April	50.2	18.4	1433.0	132.3	–	–	1633.9	366.3	–	1495.5	3501.7	5303.5	-3669.9	-1604.2	-5274.1
May	157.0	38.7	1286.6	94.5	–	–	1576.8	265.1	–	2107.2	3138.3	5510.6	-3933.8	-1512.2	-5446.0
June	182.5	67.1	6437.1	123.0	907.8	–	7717.5	299.6	–	1528.8	15700.3	17528.7	-9811.2	-1063.4	-10874.6
July	195.5	56.4	1190.7	151.4	210.44	–	3698.4	334.3	–	995.1	2904.0	4233.4	-535.0	+429.4	-105.6
August	203.4	63.3	1190.7	179.8	2005.3	–	3642.6	369.3	–	876.9	2900.0	4150.2	-507.6	+45910.9	+45403.3
Sept.	98.9	44.5	1223.9	190.0	1783.2	–	3340.5	332.8	576.6	925.8	2985.3	4820.2	-1479.7	+6273.4	+4793.7
Oct.	179.5	49.5	967.1	197.1	–	–	1393.2	286.8	1252.3	939.3	2359.0	4837.4	-3444.1	+16880.8	+1336.7
Nov.	195.1	81.3	1394.4	207.1	–	1536.3	3414.2	260.7	3200.8	618.0	3410.7	7490.2	-4076.0	-2035.8	-6116.6
Dec.	238.6	68.4	1221.0	205.3	–	1823.1	3556.4	303.4	238.4	446.7	2977.9	3966.4	-410.0	-1938.1	-2348.1
Total in Year	1861.5	584.1	26662.4	2060.7	6800.7	46407.5	3840.5	5268.1	12276.3	65047.1	86432.0		-40024.7		+16862

Residual flow is balance quantity calculated in Table 5.17

Rate of decline is measure of excess of subsurface outflow at a given point and decreases expotentially with time to the decrease in saturated thickness and hydraulic gradient. In the absence of new recharge the natural ground water recession can be plotted on semilogarithmic graph paper to obtain a straight line.

Regarding this study, following views are also relevant (COTTON)[48]

" It is hard to reconsile the concept of interflow, the ground water discharge occurs in the recession of storm, with release of the water of identifiable zone. Normally there is considerable time lag between infiltration from precipitation and rise in water level in a deep well. The seasonal rise in water table due to percolation usually therefore begins upto 2 months after the last months and can be early as September or as late as January ".

On the other hand, computation of effluent is not easy as hydrograph is largely is indeterminate, any so computation is based on arbitrary rules. Two extremes of shapes have been defined by Linslay. It may be assumed that with the beginning of surface runoff ground water flow becomes zero, and remains so until the net head begins to decrease as a result of withdrawl from ground water storage". At the other extreme it might be assumed that ground water hydrograph begins to rise at or shortly after the beginning of rain reaches a peak and decles with a hydrograph shape resembling that of the other components of flow. True conditions lies some where between these two extremes".

4.6.2.2. <u>Approach:</u> Hence considering above views following approaches for computation were established.

(1) At the beginning of surface runoff, in the month of June, base flow is minimum, and hydrograph begins to rise shortly after the beginning of rain and reaches the peak in month of September and then recession takes place.

(2) It follows expontential law for both rising as well as recession condition.

(3) Rainfall during non-monsoon period though influences the hydrograph, but do not change the pattern.

(4) As rain fall during non-monsoon period is very low, hence rainfall contribution during subsequent months can be ignored without causing appreciable error.

Based on these assumptions, base flow during different months were estimated, as per following procedures.

4.6.2.3. Plot of Hydrograph:

From the table 4.11, it will appear that there is no rainfall during non monsoon period except January and Feb. Hence the balance quantity as difference between net flow and change in storage during rest months are only related to effluent seepage from streams. The values of flow for November and December were plotted at the end with a purpose that this can be extended in back for earlier periods, October and September. The values of March, April and May were accordingly plotted on semilogarthim paper and obtained straightline plot on either case. Increase of plot figures of March, April and May the line was extended upto June, when the flow is minimum and also extended in back upto beginning of January. A third line, was drawn by the joining the lower end of second line, representing flow during June to upper end of former line, showing peak flow.

4.6.2.4. Estimation of Flow:

With The help of above plots, values flows of intermediate months were obtained. Value so obtained for the rest months have been produced in table 4.11.

4.6.3. Estimation of Recharge due to rainfall:

In this context it is to state that total quantity of base flow was met by depletion of storage only, when the rainfall during that month is nil. Where as on occurrence of rainfall during the month in question, base flow is either contributed fully or supplemented by rainfall to follow natural phenomenon, governed by exponential law. On this basis difference between base flow and net flow contribution of ground water basin to the stream causing depletion was accounted as rainfall contribution.

By combining the values rainfall contribution to base flow, and rainfall contribution to ground water storage total rainfall recharge was obtained for entire basin during different months.

4.6.4. Estimation of Overall Ground Water balance:

With an object to have prospective view of ground water basin, over all balance for annual period was necessary. The period under this study was taken from January 1973 to December 197 Hence annual values of all the hydrological components of lump model was incorporated in the balance sheet as shown in table 4.12.

It would appear that during the year of study there is net increase of storage of Ground water basin is order of 16,86.8HM Based on one year balance definite conclusion can not be arrived. However it appears that there is further scope for development and utilisation of ground water resource.

HECT. METRE)

Month	Inf Out as p form tabl	RAINFALL onth	RECHARGE Cumulative figure	Percentage of Recharge As per monthly rainfall	On Basis of cumulative Figure
1	2		12	13	14
	Hect.				
January	-10		-	20.4%	20.4
February	-17		898	18.6%	19.4
March	-93		898	-	19.4
April	-36		898	-	19.4
May	-39		898	-	19.4
June	-986		1274.6	2.62	6.75
July	-53.4		3404.0	2.68	3.45
August	-500.9		51314.9	65.4	29.8
Sept.	-14.4		69778.3	15.2%	26.4
Oct.	-340.8		78769.1	No Rain	34.3
Nov.	-40		78769.1	-	34.3
Dec.	-41		78769.1	-	34.3
Total during the year	-469.1		78769.1	-	34.6%

*V

TABLE - 4.11

COMPUTATION OF BASE FLOW AND RAINFALL RECHARGE FOR ENTIRE BASIN (UNIT PER VOLUME - HECT. METRE)

Month	Inflow-Outflow as per former* table	Changem storage (as per W.T.fluctuation	Difference in Qty. Col.(3) - Col.(2)	Area = 2,57,870 Hect Rainfall Particulars: Depth of rain	Total volume: During month	Total volume: Cumulative	Base Flow: When no rain	Base Flow: When There is rain	Rainfall contribution to Base flow (9)-(4)	RAINFALL RECHARGE: During month	RAINFALL RECHARGE: Cumulative figure	Percentage of Recharge: As per monthly rainfall	Percentage of Recharge: On Basis of cumulative Figure
1	2	3	4	5	6	7	8	9	10	11	12	13	14
	Hect.Met.	Hect.Mt.		mm			as per balance qty.in col.(4)	as per graph	Max.limit to qty.,Base flow				
January	-1041.1	-2498.8	-1457.9	7.91	2049.8	2049.8	-	1877	420	418	-	20.4%	20.4
February	-1778.1	-3088.2	-1310.0	10.02	2581.3	4631.1	-	1790	480	480	898	18.6%	19.4
March	-9338.2	-11031.2	-1693.0	-	-	-	1693	-	-	Nil	898	-	19.4
April	-3669.9	-5274.1	-1604.2	-	-	-	1604.2	-	-	Nil	898	-	19.4
May	-3933.8	-5446.0	-1512.2	-	-	-	1512.2	-	-	Nil	898	-	19.4
June	-9811.2	-10874.6	-1063.4	55.52	1431.9	18948.0	-	1440	376.6	376.6	1274.6	2.62	6.75
July	-535.0	-105.6	+429.4	307.58	79315.6	98263.6	-	1700.0	1700	2129.4	3404.0	2.68	3.45
August	-507.6	+45403.3	+45910.9	285.73	73681.2	171944.8	-	2000.0	2000.0	47910.9	51314.9	65.4	29.8
Sept.	-1479.7	+4793.7	6273.4	219.48	56597.6	228542.4	-	2190	2190	8463.4	69778.3	15.2%	26.4
Oct.	-3444.1	+13336.7	16880.8	-	-	228542.4	-	2110	2110	18990.8	78769.1	No Rain	34.3
Nov.	-4076.1	-6116.6	-2035.5	-	-	228542.4	2035.5	-	-	-	78769.1	-	34.3
Dec.	-410.0	-2348.1	-1938.1	-	-	228542.4	1938.1	-	-	-	78769.1	-	34.3
Total during the year	-40256.7	+16862 (Net)	-	-	228542.4	228542.4	8730.0	+13105.0 =21888 HECT.	9274.6	78769.1	78769.1	-	34.6%

*Value tabulated as per value computed vide Table 4.10 Col.14.

TABLE - 4.12

GROUND WATER BALANCE OF VARUNA BASIN

S. No.	Assets	Qty. in Hect. Metrs	Liabilities	Qty. in Hect. Met
1.	Recharge to basin due to Seepage from Canal	1861.5	Outflow from the basin to other basin	3840.5
2.	Recharge to basin due to Irrigation by Canal System	584.1	Evapotranspiration from forest area of the basin	5268.1
3.	Recharge to basin due to Minor Irrigation (Weiss & Pumps)	26662.4	Evapotranspiration from Water logged area	12,275.0
4.	Inflow to basin from other basin	2060.7	Drawl of water from aquifer(Tubewell dugwell, pump)for Irrigation including domestic & industrial supply	65047.1
5.	Recharge through rainfall during year	79011.1	Effluent seepage from the basin to Stream	21888
6.	Percolation due to lakes & Ponds(Nonmonsoon periods)	8437.1	Increase in Storage in monsoon period	63533.7
7.	Inflow to the basin due to Seepage from rivers	6568.7		
8.	Decrease in storage	46666.2		
		1,71,852.		1,71,852

CHAPTER-V

DISTRIBUTED MODEL

General:

For use of this model, the basin was divided into 10 modal areas. Each nodal areas have individual balancing equation. The boundary conditions of distributed model, is different compared to lumped model. On the consideration of the boundary condition associated with the distributed model, two additional terms are involved in the model.

The general equation which is applicable for each of the nodel areas representing the distributed model can be written as:

5.1. Balancing Equation:

$$\sum_{i=K}^{n} I + S_i' + R_c + R_{IC} + R_{II} + P_L + S_I + R_r = E_{TW} + E_{TF} + T_P + \sum_{L=1}^{n} O + S_E \pm \Delta S + S_o'$$

Where

I is inflow from other connected nodes to the node.

O is outflow to other connected nodes from the node

R_C = recharge due to canal seepage in nodal area.

R_{IC} = recharge due to canal irrigation in nodal area.

R_{II} = recharge due to well irrigation in nodal area

P_L = recharge due to Lake Percolation in the area

S_I = Influent seepage from streams in the area

R_r = recharge due to rainfall over the nodal area

S_i' = Inflow from other basin in the nodal area

T_p = Extraction of water through wells & pumps

S_I = inflow from other basin

S_o = outflow to other basin

ΔS_T = change in storage in nodal area

5.2. Mathematical Model:

The mathematical distributed model which has been described earlier is reproduced here for further discussion

$$\sum_{L=K}^{n} (\overset{K}{H_I} - \overset{K}{H_N}) \; Y_{IN} = A_i \; S_i \; (\overset{K}{H_I} - \overset{J}{H_I}) + A_I \, Q_I$$

in which $K = J + \Delta t$ and $Y_{IN} = DI_{IN}/DL_{IN} \times T_R$

where K and J are periods at end and beginning respectively. Other notations have been already defined. The model employs only three terms as discussed below.

5.2.1. Sub-surface flow :- Left hand term describes sum of inflow and out flow from/to other connected nodes. This is dependent upon hydraulic gradient between two connected nodes I and N; and conductence of path between above nodes. The change in hydraulic gradient is primarily dependent on water level fluctuating of either of two nodes. Water table fluctuation is also dependent upon all the hydrological components involved in the model. The term can be compared with term of sum of inflow and outflow from/to other nodes in hydrologic balance equation, and so this can be expressed as follows :

$$(\overset{K}{H_I} - \overset{K}{H_N}) \quad Y_{IN} = I - O$$

5.2.2. Change in Storage :

It has been pointed out earlier that in the distributed model response of aquifer in the nodal area is represented by single water level elevation at the node. Hence the first term on the right hand side represents change in storage during time interval t. This term is similar to s which represents the same. Hence these two terms can be equated as follows :

$$A_I \; S_I \; (H_I^K - H_I^J) = \pm \Delta S$$

5.2.3. Source Flow :

Last term is net source flow during the period. This is algebric sum of all other inflow and ou flow to or from node. Under such state, sum of all the terms can be equated, to the rest of terms involved in the hydrological balance equation, as follows.

$$A_I . Q_I = R_r + R_c + R_{IC} + P_L + S_I - E_{TW} - E_{TF} - T_p - S_E$$

5.3. Objective for the Model:

The model was employed for prediction of water level at nodal points on the imposed condition. For which based on the considerations of only three terms, data was accordingly needed for solution of the mathemetical model. The datas were obtained in the same forms as described below, Considering the period as one month.

5.4. Preparation of data :

For use of distributed model following data were needed:

5.4.1. Water level data : For prediction of water level at nodal points, data of water levels of the wells representing the nodes for initial month was only necessary, but for identification of model data for subsequent months were required. Hence monthly water level of nodes (represented by wells) for all the twelve months were computed on the basis of available data of depth of water table from ground surface as per project report. The value of same is available vide table 5.1.

5.4.2. Rainfall data :-

For the use of model, weighted rainfall for all the nodal areas were separately required. The data of rainfall at all the six rain gauge stations were used for computation of weighted rainfall for respective nodes. It is to submit that thesian polygon as constructed earlier representing the controlling areas of different rainfall stations were utilised for this work. The nodal areas are ~~of~~ mostly covered by partial areas of different rainfall stations. Hence on this basis, the areas of rainfall station lying in each nodal areas were demarcated by method of superposition. Areas were planimetered and ratios of partial areas controlled by respective rainfall stations to total nodal areas were computed for each node separately. The result of same is available in table 5.6. Then products of values of depth of rainfall controlling the respective partial areas of the node in question, and partial areas in fraction as discussed above were calculated.

These products were further added to obtain average rainfall for the node in question. On above consideration weighted rainfall of all the 10 nodes were calculated, and value so obtained produced in table 5.7.

5.4.3. Source flow rate:

This is algebric sum of all the hydrological components involved in right hand of the equation described under section 5.23. So it was necessary to evaluate the same. These were obtained an basis of available informations and certain assumptions which have been described in earlier chapter. Only a brief information for all these components are given here.

5.4.3.1. Recharge from canal seepage :-

Recharge from stream lying in the respective nodal areas were computed, and values obtained. Result of same is available in table 4.1.

5.4.3.2. Recharge due to canal irrigation :-

Node wise values of recharge for the effected nodes which were also computed as described in Chapter IV. The value of same is available vide table 4.1.

5.4.3.3. Extraction of water through wells:

Similar steps were made for computation of extraction of water through tube wells, as discussed in chapter IV and values of which have been produced in table 4.2.

5.4.3.4. Recharge through well irrigation:-

This is based on consideration of 40.8% of quantity extraction through wells. On this basis recharge for each node was calculated, and value so obtained has been produced in table 4.3.

5.4.3.5. Recharge due to percolation from lakes and ponds:-

This is based upon assumption of lake area, depth permeability and thickness of first two layers and consideration of water table depth. The procedure has been discussed in Chapter IV. The values of above for respective nodes are available in table 4.4.

5.4.3.6. Evapotranspiration from water logged area :-

This is based upon assumption of effective depth of water table for above process, and construction of water logged area on map. On this approach values were obtained as per earlier discussion and produced the same in table 4.5.

5.4.3.7. Evapotranspiration from forest area :-

This is based upon assumption of forest area in terms of percentage of basin area. Evapotranspiration rate calculated for each node by taking product of respective area and evapotranspiration rate. Values so obtained have been produced in table 4.6.

5.4.3.8. Inflow and outflow from/to other boundaries:-

Nodewise inflow and outflow from/to other boundaries were computed based on water table gradient and Darcy's law for incorporating the same in source flow rate. On the other

MONTHLY OBSERVED WATER LEVELS OF WELLS REPRESENTING NODES OF MODEL OF THE VARUNA BASIN

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10
January	85.40	89.66	84.79	84.35	78.85	75.46	78.21	69.36	68.19	66.77
February	85.32	89.80	84.67	84.23	78.73	75.30	78.08	69.16	60.06	66.65
March	84.84	89.24	84.53	83.20	78.14	74.96	77.98	68.65	67.69	66.34
April	84.78	88.87	84.32	83.12	78.03	74.76	77.23	68.56	66.89	65.84
May	84.60	88.74	84.21	82.97	77.89	74.49	76.13	68.20	66.59	65.42
June	84.10	87.44	82.34	81.75	77.76	74.11	76.08	68.15	66.64	65.22
July	84.12	87.39	82.32	81.65	77.69	74.04	76.03	68.30	66.74	65.29
August	85.37	89.14	84.99	85.25	79.54	75.81	78.93	70.22	68.59	68.92
September	83.42	88.29	85.09	86.90	80.19	76.19	79.11	70.50	70.04	69.54
October	89.32	92.74	85.34	86.85	80.81	76.21	79.10	70.55	70.12	69.61
November	89.02	93.64	85.62	86.55	80.06	76.05	77.91	71.32	68.87	68.79
December	88.82	93.54	85.02	86.02	79.14	75.89	78.17	68.95	68.64	70.04

hand, the boundary of the basin was assumed as impermeable boundary with no flow, for computational purpose, reason of such assumption has been described in chapter-2.

5.4.3.9. Effluent Seepage :

The lumped model, involving water budget equation was employed for determination of monthly effluent seepage to streams for entire basin. This monthly flows of effluent seepage to streams were distributed under each nodal areas, proportionate to total length of stream lying in nodal area in question. Values so obtained were considered for use in the distributed model, and same have been produced in table....

Hence finally considering monthly values of all the components, net flow for all the 10 nodes were separately obtained by taking algebric sum of all those. Components related to respective nodes. Table 5.2. shows computation of above for representative month January. Whereas table 5.3.(b) gives value of net flow from January-December.

5.4.4. Other Data:

5.4.4.1. Nodal Area :-

After geometrical construction of nodal area (as discussed in Chapter 2) area of each node was measured using planimeter. Figures of same are available in table.

5.4.4.2. Distance between nodes and sides of Polygons:-

The distance between connected nodes (DL) and perpendicular bisector (DT) representing the sides of polygon was measured on scale and values so obtained have been produced in table 5.4.

MONTH OF JANUARY FIGURE IN HECT.METRE

Node	rawl from round ater hrough ells	Total Outflow	Difference	Residue flow	Net Flow
1.	305.2	344.2	+16.2	-91.8	-75.6
2.	87.6	544.2	+71.6	-228.9	-157.3
3.	800.8	871.6	-189.6	-250.7	-440.3
4.	239.9	346.6	-105.6	-119.6	-225.2
5.	658.5	726.8	-252.6	-227.4	-480.0
6.	526.3	574.9	-70.5	-172.8	243.3
7.	490.8	533.7	-102.8	-119.6	-222.4
8.	489.6	588.3	-146.1	-139.9	-286.0
9.	337.3	592.1	-234.9	-67.0	-301.9
10.	306.0	334.5	27.4	-40.8	-68.2
		TOTAL DURING MONTH	1040.3	1488.5	2498.8

NOTE n connected nodes and also contribution by Rainfall.

TABLE – 5.2

COMPUTATION OF NET FLOW UNDER DIFFERENT NODAL AREAS OF VARUNA BASIN DURING MONTH OF JANUARY

FIGURE IN HECT.METRE

Node	Recharge from canal seepage	Recharge from canal Irrigation	Recharge from minor Irrigation	Inflow from other basin	Influent seepage from stream	Percolation from lakes and Ponds	Total Inflow	Outflow to other basin	Evap. transpiration from water logged area	Evap. transpiration from forests etc.	Drawl from Ground water through wells	Total Outflow	Difference	Residue flow	Net Flow
1.	6.4	3.0	125.1	51.5	–	175.1	361.1	–	–	39.0	305.2	344.2	+16.2	–91.8	–75.6
2.	62.2	13.0	199.9	88.6	–	262.1	625.8	–	–	66.6	87.6	544.2	+71.6	–228.9	–157.3
3.	–	–	328.3	–	–	353.7	682.0	–	–	70.8	800.8	871.6	–189.6	–250.7	–440.3
4.	50.9	12.1	98.4	–	–	79.6	241.0	58.7	–	48.0	239.9	346.6	–105.6	–119.6	–225.2
5.	1.4	1.6	270.0	–	–	201.2	474.2	9.8	–	58.5	658.5	726.8	–252.6	–227.4	–480.0
6.	–	–	215.8	–	–	288.6	504.4	–	–	48.6	526.3	574.9	–70.5	–172.8	243.3
7.	–	–	201.2	–	–	228.7	430.9	–	–	42.9	490.8	533.7	–102.8	–119.6	–222.4
8.	–	–	200.8	–	–	241.4	442.2	–	54.0	54.0	489.6	588.3	–146.1	–139.9	–286.0
9.	–	–	138.3	–	–	218.9	357.2	223.6	–	31.2	337.3	592.1	–234.9	–67.0	–301.9
10.	–	–	125.5	66.1	–	115.5	307.1	28.5	–	28.5	306.0	334.5	27.4	–40.8	–68.2
												TOTAL DURING MONTH	1040.3	1488.5	2498.8

NOTE Net flow under different nodes given in this table does not include sub-surface flow from connected nodes and also contribution by Rainfall.

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10
January	101.2	222.9	512.1	285.6	545.0	291.8	256.7	326.2	321.3	81.0
February	208.2	329.0	683.5	306.0	656.7	300.8	308.5	356.5	342.6	90.4
March	784.0	1052.0	2038.9	630.1	1474.7	1252.3	1113.9	1199.2	971.3	550.4
April	261.6	541.0	798.1	395.3	721.8	625.1	501.1	608.5	544.8	276.8
May	301.4	528.7	839.6	370.5	750.8	645.2	528.2	623.3	544.2	314.1
June	544.3	1320.7	1808.5	826.7	1978.8	1142.6	1123.2	1088.3	848.8	569.3
July	173.9	102.6	371.3	125.0	304.6	356.9	119.2	287.8	321.4	73.4
August	174.9	145.9	420.5	127.0	351.4	386.6	147.6	308.6	379.0	65.7
September	240.3	272.6	532.9	795.0	413.0	422.9	192.8	335.9	374.3	79.9
October	217.4	42.6	717.0	1422.0	648.0	531.7	413.3	497.3	494.0	173.0
November	155.1	1452.4	554.4	2136.0	604.3	292.0	247.8	295.2	292.8	81.8
December	82.0	150.5	430.2	370.4	440.1	199.8	172.8	225.0	245.1	32.2

This does not include rainfall contribution influencing change in storage but include rainfall contribution to effluent seepage.

TABLE - 5.3(b) FIGURES IN HECT METS

TABLE SHOWING NET MONTHLY FLOW FROM/TO DIFFERENTS NODAL AREAS OF VARUNA BASIN

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10
January	74.9	157.3	440.3	225.2	480.0	242.5	222.4	286.0	202.0	68.2
February	178.0	253.6	600.9	266.6	572.0	244.1	266.1	309.4	320.6	76.9
March	784.4	1052.0	2038.9	630.1	1474.7	1252.3	1113.9	1199.2	971.3	550.4
April	261.6	541.0	798.1	395.3	721.8	625.1	501.1	608.5	544.8	276.8
May	301.4	528.7	839.6	370.5	750.8	645.2	528.2	623.3	544.2	314.1
June	520.6	1261.4	1744.0	795.8	1920.2	1098.1	1092.4	1052.1	831.4	558.8
July	66.7	164.4	78.3	144.0	39.6	155.9	20.3	124.6	243.2	25.8
August	49.0	168.1	76.5	37.0	39.4	170.2	16.4	116.6	287.0	9.7
September	102.3	71.4	156.9	615.0	71.0	164.9	12.8	135.9	273.3	18.9
October	84.4	111.6	355.1	1248.5	319.0	282.3	239.8	294.3	397.L	114.0
November	155.1	1452.4	554.4	2136.0	604.3	292.0	247.8	295.2	292.8	81.3
December	82.0	150.5	430.2	370.4	440.1	199.8	172.8	225.0	245.1	32.2

This does not include rainfall contribution.

NOTE: Sign given as plus (+) shows inflow to the basin, otherwise outflow for the basin.

TABLE – 5.4

TABLE SHOWING GEOMETRICAL DETAILS OF DIFFERENT NODAL AREAS OF VARUNA BASIN

NODE	DISTANCE JOINING TWO CONNECTED NODES REPRESENTED BY					PERPENDICULAR BISECTOR DISTANCE REPRESENTED BY				
	DL(I)in cm	DL(I) in Cm	DL(I)in Cm.	DL)I)in Cm	DL(I)in cm.	DJ(J)in cm	DJ(I)in cm	DJ(I)in cm	DJ(I)in cm	DJ(I)in cm
1.	(11)=5.5	(12)=9.4	–	–	–	DJ(12)=118	DJ(12)= 3.9	DJ(I)	DJ)I)	DJ(I)
2.	(11)=5.5	(13)=8.5	(14)=10.2	–	–	(11)=11.8	(13)=5.8	(14)=5.0	–	–
3.	(13)=8.5	(15)=10.3	(17)=10.0	(13)=6.75	(12)=9.4	(13)=5.8	(15)=4.3	(17)=3.75	(18)=7.80	(12)=3.
4.	(14)=10.2	(15)=10.3	(16)= 7.8	–	–	(14)=5.0	(15)=4.3	(16)=5.85	–	–
5.	(17)=10.0	(16)=7.8	(19)=9.65	(22)=10.4	(21)=10.1	(17)=3.75	(16)=5.85	(19)=4.55	(22)=5.0	(21)=2.
6.	(18)=6.75	(19)=9.65	(20)=8.05	–	–	(18)=7.80	(19)=4.55	(20)=5.00	–	–
7.	(22)=8.0	(23)=7.15	(24)=8.2	(26)=9.9	–	(22)=5.00	(23)=6.75	(24)=8.2	(26)=1.25	–
8.	(21)=10.1	(20)=8.05	(23)=7.15	(25)=9.1	–	(20)=5.00	(21)=2.6	(23)=6.75	(25)=2.3	–
9.	(24)=8.2	(25)=9.10	(27)=5.8	–	–	(24)=5.2	(25)=2.3	(27)=6.35	–	–
10.	(26)=9.9	(27)=5.8	–	–	–	(26)=1.25	(27)=6.35	–	–	–

Distance on the map scale 1 cm = 2.989 km.

TABLE - 5.5

TABLE SHOWING COMPUTATION OF WEIGHTED RAINFALL OVER THE BASIN

Rainfall Station	Rainfall station Area In Hectare	Rainfall station Area In Factor	JANUARY Rainfall MM	JANUARY Area x Rainfall in Factor	FEBRUARY Rain-fall MM	FEBRUARY Area Factor x Rainfall
Phulpur	45,669	0.1771	4.6	0.80	-	-
Mirzapur	14,673	0.0569	5.0	0.28	25.8	1.47
Handia	29,268	0.1135	9.0	1.02	-	-
Gangapur	51,807	0.2009	1.0	0.20	1.0	0.20
Marianhun	58,896	0.2284	7.0	1.56	1.0	0.23
Machu Saint	57,557	0.2232	18.2	4.05	36.4	8.12
Total	2,57,870	1.000	-	7.99	-	10.02
WEIGHTED RAINFALL			7.91 mm		10.02 mm	

DURING KHARIF SEASON

Rain-fall Station	JUNE Rain-fall in mm	JUNE Area in factor x Rain-fall	JULY Rain-fall in MM	JULY Area Factor x Rain-fall	AUGUST Rain-fall in MM	AUGUST Area Factor x Rain-fall	SEPTEMBER Rainfall in MM	SEPTEMBER Area in Factor x Rainfall
Phulpur	31.3	55.4	222.7	39.44	347.7	615.8	116.7	20.67
Handia	59.4	6.74	401.4	45.56	360.8	409.5	128.7	14.61
Mirzapur	24.40	13.88	318.5	18.12	213.8	12.16	143.1	8.14
Gangapur	-	-	291.1	58.48	197.4	39.66	378.0	75.94
Mariahun	99.2	22.66	278.8	63.62	271.5	62.01	228.7	52.24
Machu Sahar	30.0	6.70	369.0	82.36	310.4	69.37	214.4	47.85
Weighted Rainfall		55.52		307.58		285.73		219.45

Rainfall Area in Factor = Rainfall Station Area; Total Basin Area

TABLE - 5.6

TABLE SHOWING PARTIAL AREAS OF NODES UNDER DIFFERENT RAINFALL STATION ZONES IN PERCENT

Node	Node Area in Hect.	Total Basin Area in Hect.	Percent of Total Area	Phulpur	Handia	Machli Sahair	Mariahun	Mirzapur	Gangapur
1.	21,243	2,57,870	8.22	98.02	1.98				
2.	26,006	2,57,870	13.97	58.50	-	41.50			
3.	38,406	2,57,870	14.90	10.30	46.30	41.80	1.60		
4.	25,745	2,57,870	9.99	-	-	94.50	5.50		
5.	31,331	2,57,870	12.17	-	-	5.83	94.17		
6.	26,105	2,57,870	10.16	-	43.10	0.16	35.82	20.92	
7.	22,924	2,57,870	8.87	-	-	-	47.12	-	52.88
8.	24,064	2,57,870	9.30	-	-	-	29.60	38.80	31.60
9.	16,803	2,57,870	6.52	-	-	-	-	-	100.00
10.	15,243	2,57,870	5.90	-	-	-	-	-	100.00
Total	2,57,870	2,57,870	100.00						

TABLE - 5.7

TABLE SHOWING WEIGHTED MONTHLY RAINFALL UNDER DIFFERENT NODAL ZONES IN millimetre

Month	Node 1	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7	Node 8	Node 9	Node 10
January	4.7	10.2	12.4	17.6	6.7	7.5	5.9	4.3	1.0	1.0
February	-	15.1	15.2	34.5	3.1	6.5	1.0	10.6	1.0	1.0
March	-	-	-	-	-	-	-	-	-	-
April	-	-	-	-	-	-	-	-	-	-
May	-	-	-	-	-	-	-	-	-	-
June	31.9	30.8	44.9	33.8	95.2	66.3	59.6	36.8	-	-
July	226.2	283.4	367.5	364.0	284.1	346.1	299.8	298.1	291.1	291.1
August	348.0	332.2	659.3	308.3	273.8	298.0	341.0	225.7	197.4	197.4
September	116.9	157.4	164.9	328.4	227.9	167.7	183.4	232.7	378.0	378.0
October	-	-	-	-	-	-	-	-	-	-
November	-	-	-	-	-	-	-	-	-	-
December	-	-	-	-	-	-	-	-	-	-

5.4.4.3. Storage coefficient and transmissibility:-

On the basis of pump test data, storage co-efficient and transmissibility of aquifer was computed for entire basin. It was assumed for the present study the values are applicable for all the 10 nodes and constant for the period. Values so obtained are given below:

Storage coefficient = 0.072

Transmissibility = 1223 m^2/day

= 3.369 Hm/month

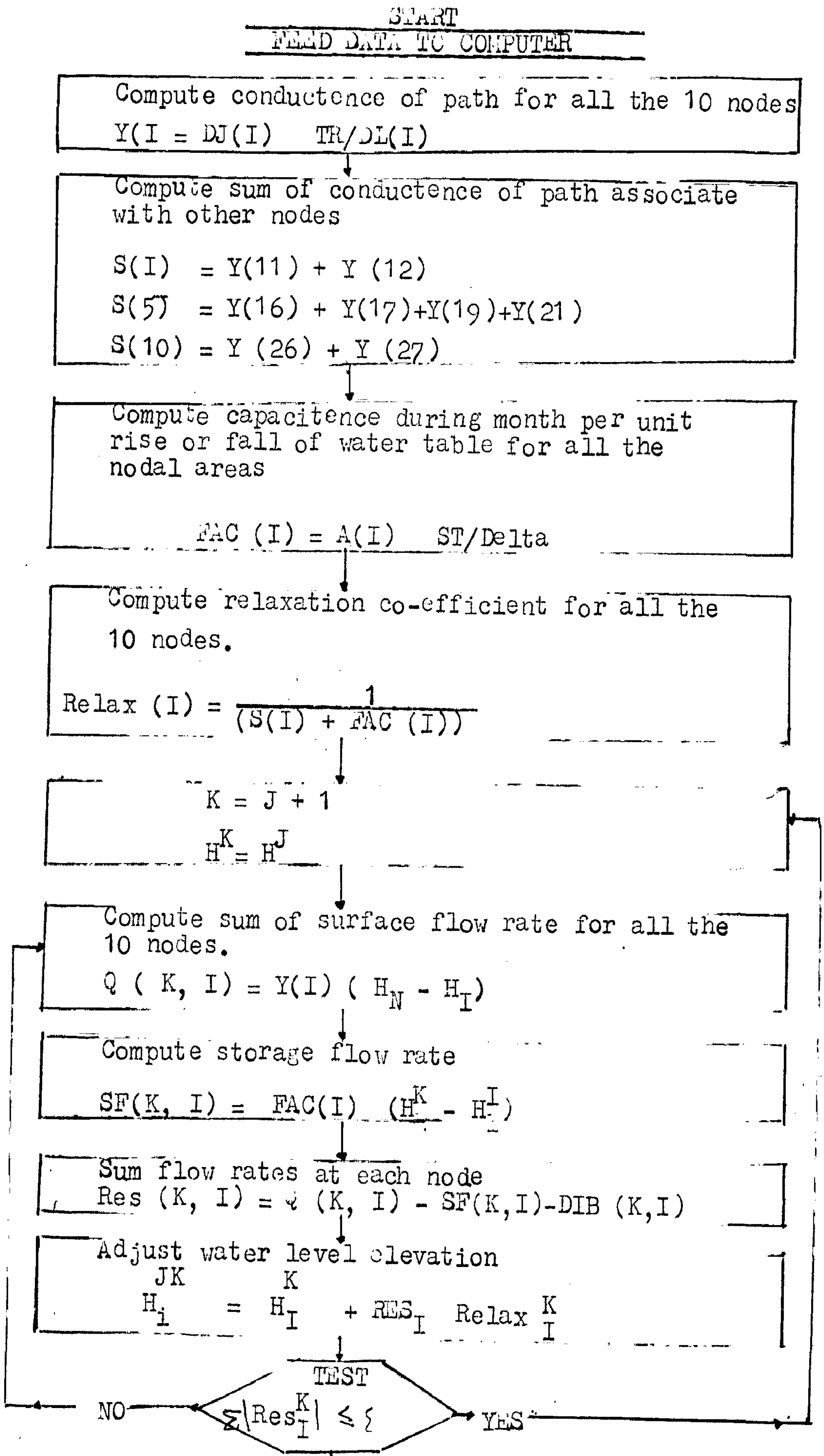
5.5. Solution of equation:

By using the mathematical expression involved in the model, water level at nodal points can be determined for above values of data. However it would need computation device.

5.6. Flow diagram:

The solution of above equation was only possible with the aid of digital computer. Hence prior to preparation of programme for the operation of computer, flow diagram was drawn, which is reproduced here.

The computer programme data and results as listed are placed, below above mentioned diagram.



SIMPLIFIED FLOW CHART FOR DIGITAL COMPUTER SOLUTION

DEFINITION

DELTA	=	Time step in 9 months
	=	Error = maxm. sum of nodel flow residuals at any time.
DIB	=	Source flow rate at any time
Y	=	Conductence of node to node branch
S(I)	=	Sum of conductence of associated branch for node in question
Relax	=	Relaxation co-efficient at node (I)
RES(K,T)	=	Node flow residual node (I) during month K
ST	=	Storage coefficient
TR	=	Transmissibility
FAC(I)	=	Capacitence of nodal area
Q(K,T)	=	Subsurface flow at node I, during month K
H_I^K	=	Water level at node I for month K
J	=	Previous month

```
C   C   MATHEMATICAL MODEL OF VARUNA GROUND WATER BASIN (U. C. VERMA)
C       PREDICTION OF WATER TABLE FLUCTUATION         1 TO 4
        DIMENSION A(10), DL(20), DU(20), Y(20), S(10), FAC(10)
        DIMENSION RELAX(12), H(12,12), DIB(12,12), Q(12,12), REC(12,12)
        DIMENSION ST(12,12)
101     FORMAT(8 I5)
102     FORMAT (8F 10.2 )
103     FORMAT(4F10.5)
104     FORMAT (5F 10.2)
400     READ 101,II,N,NN,MN,JK,KK
        PUNCH101,II,N,NN,MN,JK,KK
401     READ  102,(A(I),I =II,N)
        PUNCH 102,(A(I),I =II,N)
402     READ102,((DIB(J,I),I=II,N),J = JK,KK)
        PUNCH 102,((DIB(J,I),I = II,N),J =JK,KK)
403     READ 103,TR,ST,DELTA,CERR
        PUNCH 103,TR,ST,DELTA,CERR
404     READ  102, (DU(I), I = JN,NN)
        PUNCH 102, (DU(I), I = JN,NN)
405     READ  102,(DL(I),I =JN,NN)
        PUNCH 102,(DL(I),I =JN,NN)
406     READ  102,(H(1,I),I = II,N)
        PUNCH 102,(H(1,I),I = II,N)
407     READ 104,((H(J,I),J=KK,MM),I=5,6)
        PUNCH 104,((H(J,I),J=KK,MM),I=5,6)
        DO 11 I =JN, NN
    11  Y(I) =DU(I)* TR/DL(I)
408     S(1) = Y(11) + Y(12)
        S(2) = Y(11) + Y(13) + Y(14)
        S(3)  =Y(12) +Y(13) +Y(15) + Y(17) + Y(18)
        S(4) = Y(14) + Y(15) + Y(16)
        DO12 I = II, N
        FAC(I) =A(I) *ST/DELTA
    12  RELAX(I) =  1.0/(S(I) + FAC(I))
        L = MM-1
        DO 29  J =1,L
        K = J + 1
        DO 28 I= II,N
    28  H(K,I) =H(J,I)
    29  FL = Y(11)*(H(K,2) - H(K,1))
415     Q(K,1) =Y(12)*(H(K,3)-H(K,1)) + FL
416     FL =Y(12)*(H(K,1) - H(K,2)) + Y(12)*(H(K,3) - H(K,2))
417     Q(K,2) =Y(14)* (H(K,4) -H(K,2)) + FL
418     FL =Y(12)*(H(K,1) - H(K,3)) + Y(13)*(H(K,2 ) - H(K,3))
419     CL = Y(15)*(H(K,4) - H(K,3)) + Y(17)*(H(K,5)- H(K,3))
420     Q(K,3) =Y(18)*(H(K,6) - H(K,3)) + FL + CL
421     FL = Y(14 )*(H(K,2) -H(K,4)) + Y(15) *(H(K,3) -H(K,4))
422     Q(K,4) = Y(16)*(H(K,5) - H(K,4)) + FL
```

DATA AS READ AND PUNCHED BY COMPUTER

```
      DO 55 I = 1,9,
425   ST(K,I) = FACT(I) ([illegible](K,I) - H(J,I))
424   RES(K,I) = Q(K,I) - ST(K,I) - SIS(K,I)
55      H(K,I)=H(K,I)+RES(K,I) [illegible]
422   SQ = 0.0
        TQ=0.
        DO 57 I=1,9,
426   TQ = TQ + Q(K,I) - SIS(K,I)
427   TQ = ABSF(TQ)
57      SQ=SQ+ABSF(RES(K,I))
        ERR=TQ*CERR
        STQ=SQ-ERR
        IF(STQ) 56,56,27
56      PUNCH 105,SQ,TQ,ERR,STQ
        DO 40 I=1,9,
        PUNCH 106,I,K
106      FORMAT(2H I=,I5,2H K=,I5)
40       PUNCH 107,H(K,I),Q(K,I),ST(K,I)
107        FORMAT(3F20.2)
  20    CONTINUE
      PUNCH 107,((H(J,I),J=1,9),I=[illegible])
        STOP
        END
```

1 4	[illegible] 10	11 1		
[illegible]	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	440.??	[illegible]	
170.00	[illegible]	[illegible]	[illegible]	
7??.??	10??.??	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	
501.??	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	
11.??	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	
1?.??	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

RESULT

PROGRAM No 2


```
[illegible]
      DO 11 I = [illegible]
   11 [illegible]
      [illegible]
      DO 12 I = 1,9
      [illegible]
   12 DELTAX(I) = [illegible]
      L = NN-1
      DO 20 J = [illegible],L
      K = J + 1
      DO 20 I = 1,9
   20 H(K,I) = H(J,I)
   25 FL = Y(22)*(H(K,[illegible]) [illegible]) + Y([illegible])*(H(K,[illegible]) - H(K,7))
  416 Q(K,7) = Y(2[illegible])*(H(K,[illegible]) - H(K,7)) + Y(2[illegible])*(H(K,10) - H(K,7)) + FL
  417 FL = Y(25)*(H(K,6) - H(K,8)) + Y(21)*(H(K,[illegible]) - H(K,8))
  418 Q(K,8) = Y(23)*(H(K,7) - H(K,8)) + Y(2[illegible])*(H(K,9) - H(K,8)) + FL
  419 FL = Y(25)*(H(K,6) - H(K,9)) + Y(24)*(H(K,7) - H(K,9))
  420 Q(K,9) = Y(2[illegible])*(H(K,10) - H(K,9)) + FL
  421 FL = Y(2[illegible])*(H(K,9) - H(K,10))
  42[illegible] Q(K,10) = Y(26)*(H(K,7) - H(K,10)) + FL
```

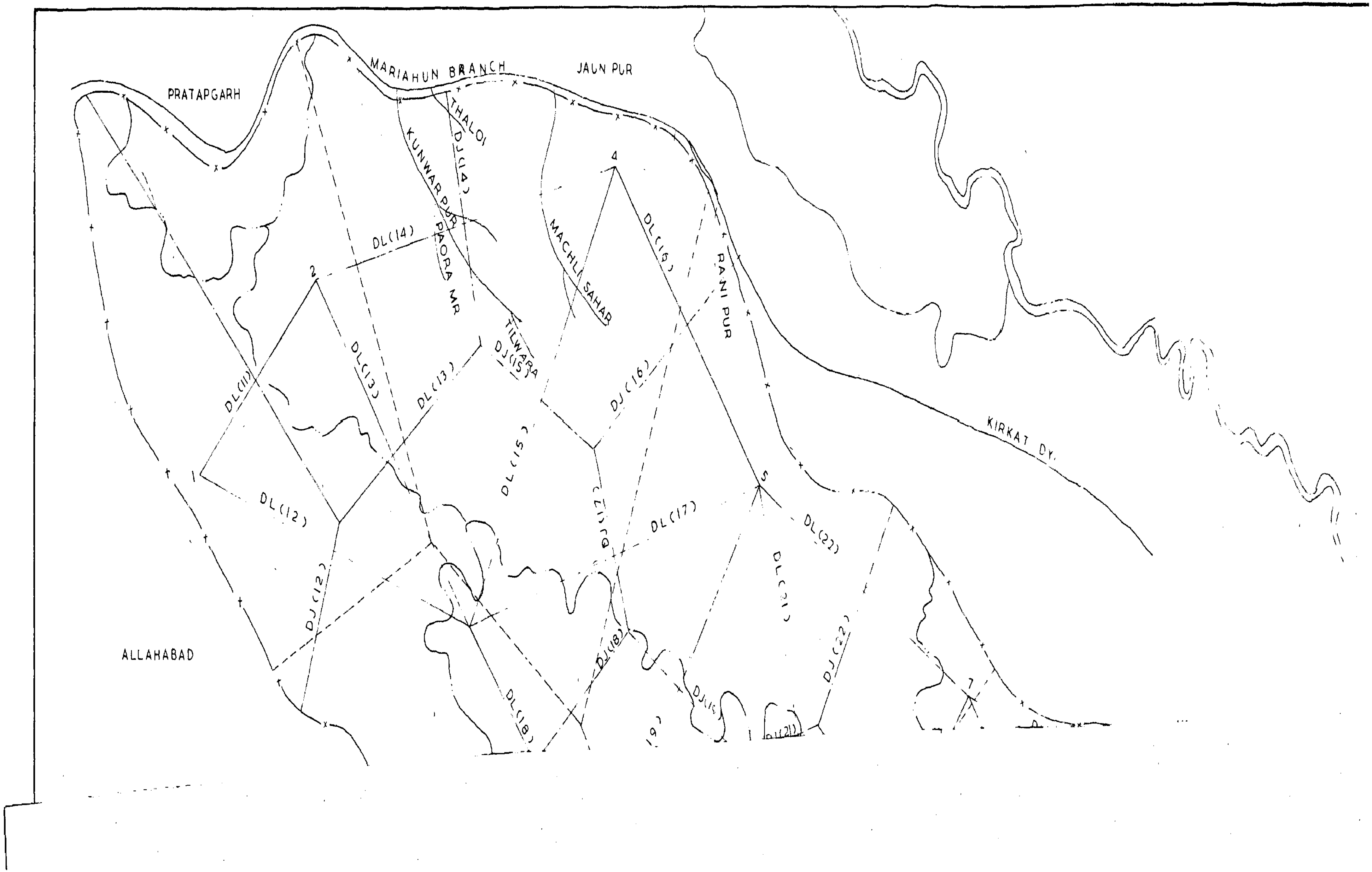
DATA AS READ AND PUNCHED COMPUTED


```
      DO 55 I = 1,9
425   D(K,I) = (ACT( )*(H(K,I) - H(J,I))
427   RES(K,I) = Q(K,I) - QP(K,I) - DIS(K,I)
55     H(K,I)=H(K,I)+RES(K,I)/AREA(I)
428   SQ = 0.0
       TQ=0.
        DO 57 I=1,9
426   TQ = TQ + Q(K,I) - DIS(K,I)
427   TQ = ABSF(TQ)
57     SQ=SQ+ABSF(RES(K,I))
       ERR=TQ*CLAR
       STQ=SQ-ERR
       IF (STQ) 30,30,25
30     PUNCH 105,SQ,TQ,ERR,STQ
105   FORMAT(2HSQ, F10.2, 2HTQ,E10.6, 3HERR, F10.2, 3HSTQ, F10.2)
       DO 40 I=1,N
       PUNCH 106,I,K
106     FORMAT(2HI=,I5,2HK=,I5)
40      PUNCH 107,H(K,I),Q(K,I),QP(K,I)
107       FORMAT(3F20.2)
  20   CONTINUE
       STOP
       END
```

7 10	5 21	20 1		
22924.00	24004.00	16005.00	15245.00	
222.40	226.00	302.00	66.20	
200.10	305.40	320.00	75.90	
1113.90	1135.20	971.00	350.70	
501.10	600.50	347.05	275.00	
520.20	625.50	341.20	314.10	
5.005	.072	1.000	.005	
5.00	2.00	5.00	6.75	
5.20	2.50	1.25	6.55	
6.55	10.10	6.00	7.15	
6.20	5.10	5.50	5.00	
76.21	67.[illegible]	66.[illegible]	66.77	
76.05	76.75	76.1[illegible]	76.0[illegible]	77.09
75.40	75.50	74.50	74.70	77.45

RESULT

```
SQ      5.941Q   .16'8215552+05ERR     15.4251Q    -[illegible].47
I=      7K=      2
                 76.51                 -57.52               -522.50
I=      8K=      2
                 69.21                 52.17                -250.51
I=      9K=      2
                 67.94                 19.00                -500.10
I=     10K=      2
                 60.70                 10.16                -65.02
SQ     13.201Q   .5041511102+05ERR     56.755TQ    -15.52
I=      7K=      3
                 77.50                 -57.10               -1160.73
I=      8K=      3
                 66.54                 52.05                -1145.35
I=      9K=      3
                 67.15                 21.12                -945.94
I=     10K=      3
                 66.21                 6.91                 -556.01
SQ      7.75TQ   .2521550642+05ERR     20.125TQ    -12.59
I=      7K=      4
                 76.96                 -56.75               -555.65
I=      8K=      4
                 66.22                 55.50                -555.92
I=      9K=      4
                 66.72                 22.20                -520.54
I=     10K=      4
                 65.97                 6.11                 -200.75
SQ      6.011Q   .2601577062+05ERR     10.0151Q    -7.55
I=      7K=      5
                 76.01                 -56.51               -562.25
I=      8K=      5
                 67.90                 55.15                -560.54
I=      9K=      5
                 66.25                 25.00                -516.07
I=     10K=      5
                 65.65                 7.47                 -504.75
```


PRATAPGARH
MARIAHUN BRANCH
JAUN PUR
THALOI
DJ(14)
KUNWARPUR
PAORA MR
DL(14)
MACHLI SAHAR
DL(16)
RANI PUR
TILWARA
DJ(15)
DL(11)
DL(13)
DL(13)
DJ(16)
DL(15)
KIRKAT DY.
DL(12)
DJ(17)
DL(17)
DL(22)
DL(21)
DJ(12)
DJ(18)
DL(18)
DJ(22)
ALLAHABAD
1
2
4
5
7

CHAPTER - VI

DISCUSSION OF RESULT AND CONCLUSION

6.1 DISCUSSION OF RESULT

The application of the verification principle involves induction rather than rigid proof, that is, a model can reproduce what has happened. It seems reasonable to believe what will happen. Verification with respect to any particular phenomenon requires a similarity of resultant components of the phenomenon. Any individual component may or may not be quantitively simulated. Model verification does not obviate the fact that natural hydrograph and other phenomenon can not be exactly predicted.

In light of above principles the applicability of the model by comparision with computed values and observed values is to be examined.

Under present study the mathematical lump model as well as distributed model were employed for solution of different problems of Varuna Ground Water Basin. Result of above are discussed as below:-

6.1.1. Result of Lumped Model

The lumped model was employed primarily to determine the values of effluent seepage, recharge due to rainfall and overall water balance of the basin. However for solution of above components, following components were first evaluated, for incorporating them in water balance equation.

1. Recharge due to canal seepage and field irrigation.
2. Extraction of ground water through wells.
3. Recharge due to tube well irrigations.
4. Recharge due to percolation from lakes & ponds
5. Inflow and outflow from or to other basin
6. Influent seepage from streams
7. Evapotranspiration from water logged area
8. Evapotranspiration from forest area
9. Change in ground water storage.

Monthly values of above components were computed based on available data, as well as other considerations including certain assumptions the result of same is available in tables 5.1 to 5.9, in same serial order, as components have been enumerated. Based on these values above three components as described earlier were evaluated, as discussed below.

6.1.1.1. <u>Effluent Seepage</u>

Effluent Seepage for the month of March to May and November to December, was determined originally using lump model. The values are available vide table 4.11. These values when plotted on semi-logarithim paper plot came as straight line, which shows agreement with theory[50]. Effluent seepage for rest months were determined by extension of above plot and arbitary rule, as shown in above table.

6.1.1.2. Recharge due to Rainfall

Values of recharge during different months as obtained by use of model is available in Table 4.11. The percentage of total recharge during the year comes to 34.3 This can be compared with following emperical formulas.

6.1.1.2.1. I.R.I.Formula

This formula as developed by I.R.I. Roorkee is as follows:

$$R = 1.35 (R_p - 14)^{1/2}$$

Where R = annual recharge, and R_p = annual rainfall in inches.

Based on above equation, the percentage of recharge comes to 17.8 percent. Compared to this percentage, percentage of recharge as per model is very high.

Amritsar Formula:

This formula developed by Irrigation and Power Institute, Punjab, Amritsar and is as follows:

$$R = 2.5 (R_p - 16)^{1/2}$$

Based on above equation, the percentage of recharge comes to 31.4 percent against 34.3% as computed by model. This shows reasonable agreement in the case.

6.1.1.3. Overall Balance

Overall ground water balance was computed for the year 1973, result of same can be seen vide table 4.12.

It may appear from the table that there is net increase of storage during the year and value of which comes 16,868 HM [illegible].). Based on one year balance study definite decision cannot be made. However it appears from the study that there is scope for further development of ground water in the Varuna Basin.

6.1.2. Result of Distributed Model

The model was employed for predictions of water level at nodal points with the aid of digital computer. The computed water level, as well as observed water level at those nodes are available in table 3.1, which is based upon result given by digital compute. On perusal of Table 6.1 it is noted that at node No.1 and 2 predicted water levels compare very well with observed ones. The same trend has been observed for the water level at node 8, 9, 10. However the computed water level at node 3, 4 & 7 are not in close agreement to the observed water level for the months of March, April and May. This variation could be attributed to either the abstraction by tube well being different from those assumed or due to different value of the specific yield and transmissibility. The difference is maximum at node 3 where the water level differs by about 0.7 to 1 metre in different months. If instead of node, the well no.5 (OP line) lying in the same nodal area, would have been exhibited in the month of March in observed water level 83.59 metre as against 84.53 metre which would have more close to computed value.

TABLE - 6.1

TABLE SHOWING COMPUTED VALUES OF MONTHLY WATER LEVEL OF DIFFERENT NODES AGAINST OBSERVED LEVEL IN METRE

NODE	Base Month January	FEBRUARY		MARCH		APRIL		MAY	
		Observed	Computed	Observed	Computed	Observed	Computed	Observed	Computed
1	85.40	85.32	85.30	84.84	84.81	84.78	84.66	84.60	84.48
2.	89.66	89.80	89.55	89.24	89.13	88.87	88.92	88.74	88.70
3.	84.74	84.62	84.56	84.48	83.81	84.27	83.51	84.16	83.19
4.	84.35	84.23	84.20	83.20	83.85	83.12	83.64	82.97	83.43
7.	78.21	78.08	78.01	77.98	77.30	77.23	76.96	76.13	76.61
8.	69.36	69.16	69.21	68.65	68.54	68.56	68.22	68.20	67.90
9.	68.19	68.06	67.94	67.69	67.15	66.89	66.72	66.59	66.29
10.	66.17	66.65	66.70	66.34	66.21	65.84	65.97	65.42	65.69

In order to estimate the variation of water level a data of 2 to 3 years would be better against one year available.

In this context it is to point out that in present study, the data were inadequate and incomplete, hence several assumptions were made for carrying out studies. In view of above facts, the working of the models can be considered to be satisfactory.

6.3 CONCLUSION

As a result of ground water modelling for river Varuna following conclusions are drawn:-

(1) Test pump data indicate that the aquifers are unconfined and value of specific yield and transmity which were found out as 0.072 and 1223 m^2/day.

(2) The area is partly covered by canal and partly by tube wells and the water used for irrigation by canal is 2242 Hect.Metres and by minor irrigation (wells) is 65047 Hect.Metres.

(3) On the basis of water table maps it ~~is~~ was found that Varuna basin is receiving water from other basin through western, and north-eastern boundary of the order 2061 Ha m whereas it releases water through the middle region of northern boundary as well as south-eastern boundary of the order of 3841 Ha.m.

(4) On the northern side of the basin a part of area remains water logged during September to December, During month of November the maximum area which remains water logged comes to 38,876 Ha, and total loss due to evapotranspiration from water logged area comes to 5268 Ha.m.

(5) There is also loss due to evapotranspiration from forest area, which comes to 12275 ha.metre during the year.

(6) Based on water fluctuation map and table showing change in storage, it appears that basin is loosing water causing depletion of storage during November to June where as recharge takes place during July to October.

(7) On the basis of water balance it is found that during non-monsoon period, base flow follows exponential law, with its minimum value in the months of June, as 1440 Ham.

(8) The annual water balance on the basis of lumped model gives the recharge to the aquifer as 34.3% total rainfall over the basin.

(9) On the basis of annual water balance, ground water potential comes to 16,868 Hectare metre.

(10) The prediction of water level by the distributed model was made. It was found that there is reasonable agreement in major nodes, but in case of some nodes there is large variance which is

attributed to either the abstraction by tube well being different from those assumed or due to different value of specific yield and the transmissibility or due to variation in average level, compared to level at the nodal point. The working of model on a whole can be considered as satisfactory.

The accuracy of distributed model can be further increased on following considerations:

1. Considerations of increase in numbers of nodal points subject to capacity of digital computer. This would need large no. of data for various components, hence compromise between accuracy and feasibility is necessary.

2. Determination of storage co-efficient and transmissibility at more points so that the different values for the nodal areas can be used to work out recharge and water level fluctuation.

3. Combined use of analog computer and digital computer will make the solution of ground water problem easier, as well as accurate. However this needs appreciable expenditure for construction of analog model. Use of Hybid computer is further advanced approach for above work.

4. Collection of data of various parameters and for longer period will make the analysis more accurate and reliable.

R E F E R E N C E S

1. Trelease, Frank J, "Mechanics of Mathematical Model of Ground Water Flow", Proc. of A.S.C.E. Irrigation & Drainage Division, Vol.89.

2. Rogere Smith & David A Woolhiso, "Mathematical Simulation of Infiltrating Water Shed", Hydrology Paper 47, Colarado State University,Colarodo.

3. Chow, Ven Te, "Handbook of Applied Hydrology"

4. Willard M Synder & John B.Stale, "Men, Models, Methods & Machines in Hydrological Cycle", Proc.of A.S.C.E., Hydraulic Division, Vol.91.

5. Heath, R.C., "An Introduction to Ground Water Hydrology".

6. Todd David, "Ground Water Hydrology".

7. Karpulas, "Analog Simulation"

8. Verujit, "Ground Water Hydrology"

9. Dewist, David De, "Geohydrology"

10. Huysen, R.J. Seveny "Journal of Hydrology", Vol.X (1970) No.3.

11. Walton, C.William, "Ground Water Resource Evaluation"

12. Walton, C.William, "Hydrogeologic Electric Analog Computer", Proc. A.S.C.E. Hyd.Dn.Vol.89(1963).

13. ~~Brede~~ Tysen & Weber, "Ground Water Management for the Nations Future - Computer Simulation of Ground Water Basin", Proc. ASCE Vol.90(1964).

14. Isermann R., Paper " Modelling and Identificationof Dynamic Process" book computer simulation of water resource system ed. G.C. Vanstee Nkiste.

15. Stuttgart F.R. " Modelling the conjuctive availability of surface and ground water of a garben in the arid South West of the United State ".

16. Richard L. Cooley, " Principles of Ground Water Hydrology Volume 10 Chapter IV, 1970.

17. R.G. Thomas, " Ground Water Model "

18. Freeze R.A. Allen " Theoretical Analysis of Regional Ground Water flow" Water Res. Vol.2 No.4.

19. George S. Taylor and James N. Luthin," Computer Methods for Transient free surface flow Problem", Water Resources Research Volume 5 No.1.

20. Mauris Todsen," On the Solution of Transient Free Surface Flow Problems in porous media by Finite Difference Method) Journal of Hydrology Vol.12, 1974.

21. Robert G. Thomas, " Comments on Analysis of non-linearties in ground water Hydrology" A Hybrid computer approach.

22. Vemuri & Karplus - " Identification of non-linear parameters of ground water basin by Hybrid Computation.

23. Garg S.P., Fortran IV and Engineering Applications.

24. Keshave Rao, M.N. " Introduction to Fortran Program ".

25. Project Report, " Project report of Varuna Ground Water Basin International Hydrology Course, University of Roorkee 1974.

26. Theis, C.V., " The relation between the lowering of the piezometric surface and rate and duration of discharge using ground water storage " - Trans. Amer. Geophysical Union Vol.16.

27. Jacob, CE & H.H. Cooper, " A generalized graphical method for evaluating formation constant and summarising well field history ". TRANS. Am. Geo. Union Volume 27

28. Mienzer, " Chapter Ground Water" Page 471, Book Hydrology by Mienzer.

29. Jacob C.E., " Flow of Ground " Book Engineering Hydraulics ed by Hunter Rouse.

30. Krueman, G.P. and De-Ridder N.A. " An analysis and Evaluation of Pumping test data" International Inst. of Land Recl. and Imp. Bul No.11.

32. Hurr, R. Theodore, " A new approach for estimating transmissibility ". W.R. Research

33. Hoth J. " A conceptual model of ground water regime and hydrogeo logical envrionment" Journal of Hydrology Volume X, (1970) No.2.

34. J.C. Wilkinsen & D.K. Smith, " Designing Simulate Models for optimisation study" - Computer simulation of Water Resource System ed. by G.C. Vansteakiste.

35. David C. Lewis, " Ground Water Levels and Fluctuations" Book Primciples of Ground water Hydrology.X (1970)

36. Boulton Norman S, "Analysis of data from pumping test in unconfined anisotropic aquifers" Jour. of Hydrology.10 (1970)

37. International Atomic Energy Agency, Viena- Istotope Techniques in Hydrology ".

38. Wister C.O. & Brater E.F. " Hydrology "

40. Bharat Singh, " Fundamentals of Irrigation Engineering "

41. " Other Papers of Project reports of Varuna Basin ".

42. Water Management Division, Deptt. of Agriculture, Govt. of India, " Hand book on Irrigation Management ".

43. Saxena, R.S., " Ground Water balance study of Ram Ganga Doab", M.E (Hydrology) DISSERTATION, Univ. of Roorkee.

44. Water Managment Division, Dept. of Agriculture, Govt. of India., " A Guide for Estimating Irrigation Water Requirement ".

45. Tarzaghi. K and Peck, R.B., " Soil Mechanics in Engg. Practice".

48. Cotton J.N., " Development of Ground Water Resources " River Engg. Works & Water Conservation ed. R.B. Thoru.

50. G. Wbber N.B. " The base flow recession curve, its derviative and application ", Jour. of Institution of Water Engineers, Volume 15, 1960.