4.0 Introduction

A variety of groundwater problems being encountered in the planning and development of water resources. Many of these problems are related to the nature of the field techniques employed and the quality and reliability of the interpretation models. It also involves the analysis of depletions caused by pumping, estimates of seepages, computation of return flows, analysis of draw down and estimation of permeabilities for selection of pump capacities. In analysing these and other groundwater problems, theoretical charts, tables and solution equations are of immense help in solving the specific problems for the hydrogeologists in quantifying the groundwater potential of an area. The behaviour of groundwater and the diagnostic characteristics of the aquifer in the study area of Ramanathapuram have been discussed in this chapter.

4.1 Behaviour of Groundwater

Information on the status of water table is of vital importance in hydrogeological evaluation of an area. Apart from indicating the extent of the zone of saturation, the fluctuation of water level is an index of recharge and discharge of groundwater. Commonly used thematic maps to study the behaviour of
groundwater are the water level contour map, fluctuation maps, depth to water level maps, water level profiles and well hydrographs.

If details of well construction and aquifer geometry are known, water level contour maps can be classified more precisely as piezometric maps, watertable maps or potentiometric maps (David et al, 1966). Representation of watertable by conventional maps, showing the depth of water level with respect to MSL, is rather generalized in nature, being not capable of bringing out the pertinent features of the groundwater flow and is often liable to subjective errors. (Biswas, et al., 1967).

4.1.1 The data

To study the present day condition of the water level 60 monitoring wells relating to December 1991 distributed randomly in the study area were studied (Fig.4.1). Among the 60 wells, 20 years water level data from 25 wells were studied for preparing water level fluctuation and grid deviation map.

4.1.2 Status of Water Table

The water table below groundlevel of the 60 monitoring wells were reduced with respect to mean sea level with the help of spot levels given in the relevant topographic sheets and a water table contour map (Fig.4.2) has been prepared. It is found that the water table contour generally follow the topography of the area.
4.1.3 Water level fluctuation

David, et. al., (1960) classified the water level fluctuation into four basic types, namely,

(a) fluctuation owing to change in groundwater storage,
(b) fluctuation brought about by atmospheric pressure in contact with the water surface in wells,
(c) fluctuations resulting due to deformation of aquifers and
(d) fluctuation owing to disturbances within the well. Minor fluctuation are attributed to chemical or thermal change in and near the wells.

All the above said factors also affect the seasonal variation in the water table, though the fluctuations of water level in wells are due to several causes, the long term fluctuation in water levels in the wells is due to changes in groundwater storage and are important for exploitation of groundwater. The average annual water level fluctuations of 25 wells have been taken and the areal distribution of water level fluctuation is presented (Fig.4.3). The fluctuation varies from a minimum of 0.78 m to a maximum of 4.32 m in the study area.

4.1.4 Well hydrograph

The stress and strain in water table due to groundwater recharge, discharge and vagaries of monsoon are reflected in the waterlevel fluctuations with time. The hydrographs of selected
network stations show corresponding peaks and lows with monthly rainfall distribution. The best fit line for the hydrographs will bring out the secular trend of the water table with time. The hydrograph of representative well Tiruvadanai monitored in the study area along with rainfall distribution curves is shown in Fig.4.4.

4.1.5 Grid deviation watertable

Grid deviation method of representing the geological data (Seha, et. al., 1963) is more convenient, objective and informative and brings out more sharply the regional trend by eliminating the local interference (Biswas, Op. Cit). Hence, this method has been adopted in the present study and analysis. The water level data collected from 25 wells monitored by Public Works Department for 20 years in the study area have been used and the following steps have been adopted in preparing the grid deviation water table map.

1. Monthly water levels measured below the measuring points have been recalculated to water level depths above M.S.L.

2. Average elevation of water table \( A_1 \) for each observation well has been computed after calculating the monthly average of water levels.

3. An average value \( A_2 \) of all average elevations of water table computed in step 2, has been determined for the study area.

4. The deviation \( D = (A_1 - A_2) \) from the average waterlevel depths and the average depths of water levels of individual observation wells have been determined.
5. The desired deviation map (Fig. 4.5) results from an objective contouring of the deviations (result of step 4) of each location of the study area (Table 4.1) has been made.

Based on the grid deviation concept of water table analysis (Biswas and Chatterjee, 1967) the entire study area is divided into positive as well as negative horizons representing zones of discharge and recharge.

The negative areas bordering entire eastern coastal region indicate steep gradient of water table and low permeability of the formation materials, whereas the central and the NW portion of the study area falls under positive area are suggestive of flat gentle gradient of water table and high permeability of the formation materials.

Artificial recharge methods can be suggested in the recharge areas and if any such activities in the discharge zones will develop sea water incursion.

4.2 Types of aquifers

The groundwater potential of an aquifer depends largely upon its ability to store, transmit and yield water. Most of the aquifer are of large areal extent and may be visualised as under groundwater storage reservoirs. Water enters into the systems from natural or artificial recharge, it flows under the action of gravity or is extracted by the abstraction structures like wells.
Fig. 4.4: Rainfall distribution and well hydrographs

TIRUVADANAI

Water level

Rainfall
Table-4.1 : Ramanathapuram District Grid Deviation of Average Water Levels

<table>
<thead>
<tr>
<th>Sl.No.</th>
<th>Location</th>
<th>Water level in M.AMSL</th>
<th>Grid deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Periyapattinam</td>
<td>4.88</td>
<td>-0.25</td>
</tr>
<tr>
<td>2</td>
<td>Valantharuvai</td>
<td>7.55</td>
<td>2.52</td>
</tr>
<tr>
<td>3</td>
<td>Sembadayarkulam</td>
<td>5.57</td>
<td>0.54</td>
</tr>
<tr>
<td>4</td>
<td>Uchupuli</td>
<td>2.44</td>
<td>-2.59</td>
</tr>
<tr>
<td>5</td>
<td>Idyarvalasai</td>
<td>3.23</td>
<td>-1.80</td>
</tr>
<tr>
<td>6</td>
<td>Uttarkosamangai</td>
<td>3.29</td>
<td>-1.74</td>
</tr>
<tr>
<td>7</td>
<td>Vaddakinarripayar</td>
<td>4.77</td>
<td>-0.26</td>
</tr>
<tr>
<td>8</td>
<td>Thirupullani</td>
<td>5.90</td>
<td>0.87</td>
</tr>
<tr>
<td>9</td>
<td>Rajasingamangalam</td>
<td>3.12</td>
<td>-1.91</td>
</tr>
<tr>
<td>10</td>
<td>Sevalpatti</td>
<td>5.98</td>
<td>0.95</td>
</tr>
<tr>
<td>11</td>
<td>Tirumalagankottai</td>
<td>5.16</td>
<td>0.13</td>
</tr>
<tr>
<td>12</td>
<td>Sikkal</td>
<td>5.05</td>
<td>-0.04</td>
</tr>
<tr>
<td>13</td>
<td>Thondi</td>
<td>4.16</td>
<td>-0.87</td>
</tr>
<tr>
<td>14</td>
<td>Ervadi</td>
<td>4.85</td>
<td>-0.18</td>
</tr>
<tr>
<td>15</td>
<td>Edambadal</td>
<td>4.32</td>
<td>-0.71</td>
</tr>
<tr>
<td>16</td>
<td>Chattirakudi</td>
<td>4.07</td>
<td>-0.96</td>
</tr>
<tr>
<td>17</td>
<td>Kilakarai</td>
<td>8.89</td>
<td>3.86</td>
</tr>
<tr>
<td>18</td>
<td>Terkumukkaiyur</td>
<td>3.95</td>
<td>-0.96</td>
</tr>
<tr>
<td>19</td>
<td>Perunali</td>
<td>4.40</td>
<td>-0.63</td>
</tr>
<tr>
<td>20</td>
<td>Devipatttnam</td>
<td>3.30</td>
<td>-1.73</td>
</tr>
<tr>
<td>21</td>
<td>Mandapam</td>
<td>3.85</td>
<td>-1.18</td>
</tr>
<tr>
<td>22</td>
<td>Ramanathapuram</td>
<td>7.69</td>
<td>2.66</td>
</tr>
<tr>
<td>23</td>
<td>Govindamangalm</td>
<td>4.86</td>
<td>-0.17</td>
</tr>
<tr>
<td>24</td>
<td>Kamudi</td>
<td>5.35</td>
<td>0.32</td>
</tr>
<tr>
<td>25</td>
<td>Paramakudi</td>
<td>6.32</td>
<td>1.29</td>
</tr>
<tr>
<td></td>
<td><strong>Average</strong></td>
<td><strong>5.09</strong></td>
<td></td>
</tr>
</tbody>
</table>
In the study area, the aquifers are of two types sand aquifers and sandstone aquifers. Towards the coastal area the sand aquifer are more dominant running for about 40 to 100 mts. thick and towards the inland it is sandstone as inferred from the bore hole lithology. A fence diagram is drawn to bring out the regional sub surface lithology (Fig.4.6). Water is saturated in the sandy and silt formation and groundwater being extracted more from the Sandstone formations which form deeper aquifers. Lime stone formation are invariably present at some places. The analysis of hydraulic properties of these aquifers has given greater stress in the present study.

4.3 Types of groundwater flow

Groundwater movement and storage are governed by the established hydraulic principles. All general flow equations of groundwater are derived from Darcy's Law and the equation of continuity. There are two major types of flow:

(i) Steady state; steady state flow occurs when at any point in a flow field the magnitude and direction of the flow velocity are constant with time. (ii) Unsteady state flow; transient (non-steady or unsteady) flow occurs when at any point in a flow field the magnitude and direction of the flow velocity change with time (Freeze, et. al, 1979). Steady flow condition exists in the aquifers of the study area.
4.4 Pumping tests

The exploitation of groundwater in any region leads to water level decline that serve to limit yields. One of the primary goals of groundwater resource evaluation must hence be prediction of hydraulic head (drawdowns) in aquifers under proposed pumping schemes. Pumping test is one of the most useful means for not only determining the aquifer hydraulic characteristics but also in the determination of yield, drawdown and specific capacity.

The hydraulic properties in sedimentary formations depend on the nature of infiltration, porosity and permeability, which are more pronounced near the surface and decreases with depth. Hence, a major part of the work in this chapter is concentrated in the analysis of the open wells. Once a sufficient depth of water column is achieved, the farmers stop further deepening of their wells and hence the dug wells were selected for pumptest. About twenty wells were selected for pumping test distributed in the entire district (Fig.4.7).

4.4.1 Choice of Wells

No particular systematic method has been used to choose the wells for the pumping test since they are not uniformly distributed. In short, the availability of energised dug wells with a satisfactory hydraulic response was the most important criteria for well selection.
4.4.2 Field Measurements

The measurements to be taken during an actual pumping test fall into two types: 1. Measurement of the water level and 2. Measurement of the discharge/recharge rate. Ideally, a pumping test should not be started before the already existing water level changes in the aquifer are known, including both long term regional trends and short term variations. Hence, for some days prior to the actual pumping test, the water levels in the wells should be measured twice a day.

If the trend is not expected to change during the study period, pumping test can be conducted (Kruseman, et al, 1970). After the completion of the test, water level readings should continue for one or two days.

Since the agriculturists and the well owners rely on one or two wells for their water needs, a long duration study could not be conducted. At the most, on any particular day when a pumping test has to be conducted, a request is made to the well owners a day in advance, not to start the pump and take the water until the observer informs them to do so on the day of the test.

A. Water Level Measurement

The most important part of a pumping test is the measurement of the depth to water in the well both during pumping and recovery phases. The difference between the static water level and the pumping
water level can give the drawdown (s) and the difference between the maximum drawdown and the recovery water level can give the recovered drawdown (s'). The difference between maximum drawdown and recovered drawdown will be the residual drawdown (s'').

B. Time Intervals of Water Level Measurements

Immediately after the starting of pumping or its stoppage, the water levels fall or rise very rapidly and hence the water level measurements should also be very rapid during this period. Kruseman et al. (1970) proposed a range of time intervals for measurements in the pumping wells of small diameter. A time interval ranging between 5 and 30 minutes during both pumping and recovery phases has been followed, depending upon the aquifer response. It may also be added here that a satisfactorily long duration pumping could not always be achieved either due to the limited saturated thickness of the aquifer or the reluctance of the farmers to oblige our request.

C. Recharge / Discharge Measurements

Amongst the arrangements to be made for an aquifer test is the control of the discharge rate. In order to avoid complications in computation, the discharge rate should preferably be kept constant throughout the test. But, in the existing field conditions, this is not possible. Repeated measurements of discharge using the container method were done during the entire phase of pumping and average
taken for computational purposes. Recovery rate has been computed by multiplying the drawdown ($s'$) with the cross-sectional area of the well.

4.5 **Aquifer parameter evaluation (Methodology)**

Numerous techniques are available for analysing the pumping test data of wells. The classical methods Cooper, et. al., 1946; (Papadopulous, et. al, 1967; Raju et. al., 1967; Narasimhan, 1968; Adyalkar and Wani 1972; Sammel, 1974; Streltsova, 1974; Zhdankus, 1974; Neuman, 1975; Boulton and Streltsova, 1976; Black and Kipp 1977; Fenske, 1977a, 1977b; Walton, 1979; Rushton and Singh 1983) are most graphical in nature and there is room for error in the individual judgement. Many computer methods have been proposed and successfully utilised during the decades, for analysing the pump test data (Saleem, 1970; Walton, 1970; McElwee, 1980; Rayner, 1980; Dumble and Eullen 1983; Bradbury and Rothschild 1985; Butt and McElwee 1985; Mukhopadhyay, 1985 and Balasubramanian, 1986).

Digital numerical procedures have been adopted in this study for the determination of aquifer transmissivity ($T$) and storage coefficient ($S$). A composite computer program - APE - in BASIC language (Appendix A) has been used to analyse the pumping and recovery test. Following segments have been included in the program and the pump-test standard conversion factors have been incorporated in the program in order to get all the results in metric units.
4.5.1 Simple Least Squares Analysis

A segment of the program performs (a) a summation of the cumulative pumping test time - drawdown entries and (b) a least square analysis of the logarithmic curve of the form

\[ d = a + b \ln t \]  \hspace{1cm} \text{...(4.1)}

where,

\[ d = \text{drawdown (in ft) in the pumping well} \]
\[ t = \text{cumulative time since pumping began (in hrs)} \]

\[ b = \frac{\Sigma d \ln t - \frac{1}{n} \Sigma \ln t \Sigma d}{\Sigma (\ln t)^2 - \frac{1}{n} (\Sigma \ln t)^2} \]  \hspace{1cm} \text{...(4.2)}

\[ a = \frac{1}{n} (\Sigma d - b \Sigma \ln t) \]  \hspace{1cm} \text{...(4.3)}

\[ n = \text{number of time-drawn pairs} \]

The shape of the curve described by equation 4.1 would resemble the curve obtained by plotting the well function of \( u(W_u) \) as the ordinate and \( l/u \) as the abscissa-the 'type curve' (Rayner Op. Cit). The program then computes the aquifer parameters using the following equations.
\[ T = \frac{114.59\, Q}{b} \] ........(4.4)

\[ S = \frac{0.0125\, T}{r_w^2\, e^{(a/b)}} \] ........(4.5)

where,

- \( Q \) = discharge rate in gallons per minute
- \( T \) = transmissivity in gpd/ft
- \( S \) = storage coefficient (dimensionless)
- \( r_w \) = radius of the well in ft
- \( e \) = natural antilogarithm

### 4.5.2 Simple Iterative Technique

Theis (1963) described a method for estimating the transmissivity (\( T \)) of an aquifer from the specific capacity of wells. This is based on the Jacob's equation.

\[ T = \frac{Q}{4\pi S} \ln \left( \frac{2.25\, T\, t}{r_w^2\, s}\right) \] ........(4.6)

- \( T \) = transmissivity in m\(^2\)/day
- \( S \) = storage coefficient (dimensionless)
- \( Q \) = discharge rate m\(^3\)/day
- \( s \) = drawdown in metres.
- \( t \) = period of pumping in days, and
- \( r_w \) = radius of the well in m
As $T$ appears on both sides of the equation, this cannot be solved directly. Approximations in the determination of $T$ values have been done using simple iterative procedures as shown in the flowchart (Fig.4.8). Appropriate $S$ values have to be given for this method.

**ITERATIVE METHOD OF SOLVING EQUATION 4.6**

1. Set $T_{\text{GUESS}} = 1000$
2. $F_1 = Q/(4 \, I_2 s)$
3. $F_2 = (2.25 \times T_{\text{GUESS}} \times t) / r^2 S$
4. $T_{\text{CALC}} = F_1 \times \text{ALOG} (F_2)$
5. If $T_{\text{CALC}} = T_{\text{GUESS}}$, then print results.
6. Else, set $T_{\text{GUESS}} = T_{\text{CALC}}$ and repeat from step 2.

**FIG.4.8 : FLOW CHART OF SPECIFIC CAPACITY METHOD**
4.5.3 Parameter Estimation by Sensitivity Analysis

Numerous automated fitting procedures have been suggested for pumping test analysis (Saleem, 1970; Labadie, 1975; Holzschuh, 1976), but no attempt was made to use the sensitivity analysis to find out the certainty of the results in output. McElwee, (1980b), Cobb, et. al., (1982) and Mukhopadhyay, (1985) proposed algorithms for computer automated least squares fit to the experimental data, yielding approximations for transmissivity (T), storage coefficient (S), given the average drawdown errors by sensitivity analysis. These procedures use the Theis equation

\[ s = \frac{Q}{4 \pi T} \int_0^\infty \frac{e^{-u}}{r_w^2 s/4Tt} du \]

where,
- \( T \) = transmissivity in gpd/ft,
- \( S \) = storage coefficient (dimensionless),
- \( t \) = time duration of pumping in days,
- \( s \) = drawdown in ft,
- \( Q \) = discharge rate in gallons per day and
- \( r_w \) = radius of the well in ft.

A. Numerical Approximation

The integral in this equation is symbolically represented by \( W(u) \). The drawdown can then be written as

\[ s = \frac{Q}{4 \pi T} W(u) \]
where,
\[ u = \frac{r_w^{2S}}{4Tt} \]

In order to evaluate this equation, one needs an explicit expression for \( W(u) \) involving only simple arithmetic operations. (Mc Elwee, Op.cit). For \( 0 \leq u \leq 1 \)

\[ W(u) = -\ln u + a_0 + a_1 u + a_2 u^2 + a_3 v^3 + a_4 v^4 + a_5 u^5 + E(u) \ldots (4.9) \]

\[ | E(U) | < 2 \times 10^{-2} \]

\[ a_0 = -0.57721566 \quad a_3 = 0.05519968 \]

\[ a_1 = 0.99999193 \quad a_4 = -0.00976004 \]

\[ a_2 = -0.24991055 \quad a_5 = 0.00107857 \]

\( E(u) \) is the error in the approximation.

For \( u > 1.0 \)

\[ W(u) = e^{u/u} [u^2 + a_1 u + a_2] / (u^2 + b_1 + b_1 u + b_2) + E(u) \] \( (4.10) \)

\[ | E(u) | < 5 \times 10^{-5} \text{ for } 1 \leq u \leq \sigma \]

\[ a_1 = 2.334733 \quad b_1 = 3.330657 \]

\[ a_2 = 0.250621 \quad b_2 = 1.681534 \]
B. Sensitivity Analysis

Sensitivity analysis is the study of a system's response to various disturbances. In this study, disturbances of aquifer parameters are considered. The solution to an aquifer model may be written in the form \( h = h(x, y, t; T, S, Q) \), where \( h \) is the hydraulic head. Varying a parameter say \( T \), by a small amount \( \Delta T \), the solution becomes

\[
h^* = h(x, y, t; T + \Delta T, S, Q).
\]

It is assumed that the solution of the aquifer model depends analytically upon the parameters \( T \) and \( S \) and that \( T \), \( S \) and \( Q \) are independent of each other. The function \( h^* \) can be expanded into Taylor series. If \( \Delta T \) is small, the second or higher order terms may be neglected.

\[
h^*(x, y, t; T + \Delta T, S, Q) = h(x, y, t; T, S, Q) + U_T \Delta T \ldots (4.11)
\]

\[
U_T (x, y, t; T, S, Q) = \frac{\delta h (x, y, t; T, S, Q)}{\delta T} \ldots (4.12)
\]

Thus, the new head produced by a perturbation or change in transmissivity \( (\Delta T) \) may be calculated from equation \( 4.11 \) if the sensitivity coefficient \( (U_T) \) and the unperturbed head are known. Similarly, for a change in storage coefficient \( (\Delta S) \), the changed head is given by (McElwee, op.cit).
\[ h^* (x,y,t; T, S + \Delta S, Q) = h(x,y,t; T,S,Q) + U_s \Delta S \quad \ldots \ldots(4.13) \]

\[ U_s = \frac{\delta h(x,y,t; T,S,Q)}{\delta S} \quad \ldots \ldots(4.14) \]

to first order in \( \Delta S \).

Equations 4.11 and 4.13 show that it is possible to calculate the sensitivity coefficients of \( T \) and \( S \) that is \( U_t \) and \( U_s \) for a given model. The response of the model to various perturbations can be calculated from equation 4.11 or 4.13 without actually evaluating the model equations again (McElwee. Op. cit)

The sensitivity coefficients for \( \Delta T \) and \( \Delta S \) can be obtained from Theis equation by applying the definitions given in equations 4.12 and 4.14. By the application of Leibnitz's rule for differentiating an integral (Hildebrand, 1962, McElwee, Op. Cit) to the Thies equation, we get

\[ U_t = \frac{\delta s}{\delta T} = -\frac{S}{T} + \frac{Q}{4\pi T^2} \exp \left( -\frac{r^2}{4Tt} \right) \quad \ldots \ldots4.15 \]

\[ U_s = \frac{\delta s}{\delta S} = -\frac{S}{T} + \frac{Q}{4\pi T^s} \exp \left( -\frac{r^2}{4Tt} \right) \quad \ldots \ldots4.16 \]

The coefficients calculated from equation 4.15 and 4.16 can be used in equation 4.11 and 4.13 to calculate what the drawdown would be if \( T \) and \( S \) were changed by \( \Delta T \) and \( \Delta S \) respectively.
C. Least Squares Fit

The main objective of using the sensitivity analysis is to obtain a least squares fit of the experimental pumping test data to the Theis equation and thus to obtain the "best" fit transmissivity and storage coefficient. The new drawdown for the $\Delta T$ and $\Delta S$ is given by

$$S^* = S + U_T \Delta T + U_S \Delta S$$

.........(4.17)

This is obtained from equation 4.11 and 4.13 by observing that $s = h - h_0$, where $h_0$ is the original head before pumping starts and is a constant independent of $T$ and $S$. Best fit $T$ and $S$ for the experimental data, can be obtained by minimizing the error function (McElwee, Op.Cit).

$$E(\Delta T, \Delta S) = \sum_i \left[ S_{ob} (t_i) - S^* (t_i) \right]^2$$

$$= \sum_i \left[ S_{ob} (t_i) - S_c (t_i) \right]^2$$

$$- 2 \Delta T \sum_i U_T (t_i) \left[ S_{ob} (t_i) - S_c (t_i) \right]$$

$$+ 2 \Delta S \sum_i U_T (t_i) \left[ S_{ob} (t_i) - S_c (t_i) \right] + \sum_i U_s^2 (t_i) \Delta S^2$$

$$+ 2 U_T (t_i) U_s (t_i) \Delta S \Delta T + U_T^2 (t_i) \Delta T^2$$

.........(4.18)

t_i = the time since pumping started (days)

$S_{ob} = observed$ drawdown (in ft) for time (t )

$S_c = calculated$ drawdown (in ft) for time (t )
Figure 4.7 shows the computational steps involved (as per the notations of the program) in minimizing the sum of squared errors and solving the equations for $T$ and $S$.

The values for $T$ and $S$ can be used to update the first estimate of $T$ and $S$. This improved estimate for $T$ and $S$ is then used again in the least squares routine to calculate new values for $T$ and $S$. This process is repeated until the coefficients become so small to be insignificant.

D. Initial Estimation of $T$ and $S$

In order to give an initial estimate of $T$ and $S$ for sensitivity analysis a linear-fit segment has been included as per the flow chart shown in Figure 4.8.

4.5.4 Theis Recovery Method

According to Theis (1935), the residual drawdowns ($s''$) during the recovery period is

$$s'' = \frac{Q}{4nT} \left( \ln \frac{4Tt}{r_w^2S} - \ln \frac{4Tt''}{r_w^2S''} \right) \quad \ldots \ldots (4.19)$$

Where

$s'' = \text{residual drawdown in m}$

$r_w = \text{effective radius of the pumping well in m}$
\( S^* \) = Coefficient of storage during recovery (dimensionless)

\( S \) = Coefficient of storage during pumping (dimensionless)

t = time since pumping started in days

t" = time since pumping stopped in days

T = transmissivity in \( m^2 / \text{day} \) and

Q = rate of recharge in \( m^3 / \text{day} \)

When \( S \) and \( S^* \) are constant and equal and \( u = r^2 S / 4 T t \) is sufficiently small, equation 4.19 can be written as

\[
S^* = \frac{2.30 Q}{4 \pi T} \log \frac{t}{t''} \quad \ldots \ldots (4.20)
\]

The plots of \( s'' \) v/s \( t/t'' \) on semi-logarithmic paper fall on a straightline whose slope is equal to \( 2.30 Q / 4 \pi T \) and hence the residual drawdown per log cycle of \( t/t'' \) can be calculated numerically and substituted for evaluating \( T \) in the equation

\[
T = \frac{2.30 Q}{4 \pi a} \quad \ldots \ldots (4.21)
\]

where,

\[
a = \frac{M \sum xy - \Sigma x \Sigma y}{M \sum x^2 - (\Sigma x)^2} \quad \ldots \ldots (4.22)
\]

where,

\( M \) = no. of time-residual drawdown pairs

\( x = t/t'' \)

\( Y \) = residual drawdown (in m)
EVALUATION OF SENSITIVITY COEFFICIENTS

\[ S_5 = \sum_{i=1}^{m} U_s^2 (T_i) \]

\[ S_6 = \sum_{i=1}^{m} U_T^2 (T_i) \]

\[ S_7 = \sum_{i=1}^{m} U_s (T_i) U_T (T_i) \]

\[ S_8 = \sum_{i=1}^{m} U_s (T_i) \left[ S_{obs} (t_i) - S_{cal} (t_i) \right] \]

\[ S_9 = \sum_{i=1}^{m} U_T (T_i) \left[ S_{obs} (t_i) - S_{cal} (t_i) \right] \]

\[ \Delta S = \frac{(S_8 \times S_8 - S_7 \times S_9)}{(S_5 \times S_6) - (S_7)^2} \]

\[ \Delta T = \frac{(S_9 - \Delta S \times S_7)}{S_6} \]

FOR CONVERGENCE CHECK

FIGURE 4.9. FLOW CHART OF SENSITIVITY ANALYSIS
ANALYSIS OF PUMPING DRAW DOWN PAIRS

\[ \Sigma X = \sum_{i=1}^{m} \log(t_i) \]

\[ \Sigma X^2 = \sum_{i=1}^{m} \log(t_i)^2 \]

\[ \Sigma Y = \sum_{i=1}^{m} S_i \]

\[ \Sigma XY = \sum_{i=1}^{m} (\log t_i * s_i) \]

SLOPE = \( \frac{(M * XY - Y * X)}{(M * X^2 - \{(X)^2\})} \)

CON = \( \frac{(-SLOPE * X - Y)}{M} \)

TRANS = \( \frac{Q}{(SLOPE * 4 * 3.1416)} \)

ALNTO = \(-CON / SLOPE\)

SCALC = \( \frac{(4 * TRANS)}{\{R * 2 * EXP (0.5772 - ALNTO)\}/7.48} \)

ASSIGN TT = TRANS ; SC = SCALC

FOR SENSITIVITY ANALYSIS

FIG.4.10 FLOW CHART OF INITIAL T & S COMPUTATION
4.5.5 Specific Capacity Analysis

The specific capacity of a well is the ratio of discharge to the drawdown (Summers, 1972). It is not constant because the drawdown varies with a number of factors, including length of time since pumping began, rate of pumping, well construction, boundary conditions within the aquifer and the influence of nearby pumping wells. It is a measure of effectiveness of the well (Todd, 1959). Slichter, (1906) gave the formula computing the specific capacity as

\[ C = 2303 \times \frac{A}{t} \times \log \left( \frac{s}{s''} \right) \]  \hspace{1cm} (4.23)

where,

- \( C \) = specific capacity of the well in lpm/mdd
- \( A \) = area of cross-section of the well in sq.mts.
- \( t \) = time since pumping stopped in minutes
- \( s \) = drawdown in metres just before pumping stopped and
- \( s'' \) = residual drawdown in metres at any time \( t \) after pumping stopped

This formula is valid for the wells penetrating poorly permeable formations where the recovery is slow and may not hold good for such aquifers where the inflow is equal to or more than outflow. This does not take into account the duration of pumping prior to the shut down of the pump.
Sammel, (1974) mentions that although it is not a means of determining aquifer constants, Slichter's formula has been used to compare the recovery performance of dug wells. It is expressed as a linear function of time and a logarithmic function of drawdown. Thus it cannot be used to calculate transmissivity (T) as done by other methods (Theis, 1964; Hurr, 1966). He added that it provides a useful basis for comparison of wells of similar types in similar geological environments if one adds the requirement that

\[ \frac{Q_1}{A_1} \times t_1 = \frac{Q_2}{A_2} \times t_2 \]  

where, \( Q_1 \) and \( Q_2 \) are the pumping rates in two wells, \( t_1 \) and \( t_2 \) are respective duration of pumping and \( A_1 \) and \( A_2 \) are the cross-sectional areas, respectively. In the present study the rates of recovery and recovery times have been used in place of pumping rates and pumping time.

Since Slichter's specific capacity values cannot be used directly for purposes of comparison, Walton, (1962) introduced the concept of specific capacity index, which is obtained by dividing the Slichter's specific capacity (average) by the saturated thickness of the aquifer (lpm/m drawdown/m thickness of aquifer tapped). Zeozel, et. al., (1962); Marilingaiah, (1978) Nautiyal, et. al., (1980); Sharma, (1982); Sastri, et. al., (1983) and Prasad, (1984) have used this index for determining the relative productivity of different units in predicting aquifer yields.
Narasimhan, (1965) proposed, on the other hand, the unit area specific capacity (lpm/m drawdown/sq.m) by dividing Slichter's specific capacity with the cross-sectional area of the well and this has been successfully utilised by Summers, (1972), Krupanidhi et. al., (1973); Subramanyan, (1975), Viswanathiah, (1978 a); Marilingaiah (Op.cit) and Prasad (Op. cit.).

Singhal, (1993) suggested that since both diameter and the saturated thickness of the aquifer tapped vary, it would be better if the Slichter's specific capacity is divided by the well (lpm/m drawdown/ sq.metres of total surface area tapped). However, Limaye's suggested during the International Seminar on Groundwater Development at Madras in 1973, that instead of the use of total contributing surface area of the aquifer tapped, it should be a sum of the total surface area of the well under study. This modification of Limaye's (lpm/mdd/sq.mt) has been taken into consideration for discussing the aquifer properties of the study area.

4.5.6 Time for Full Recovery Analysis

Rajagopalan, et. al., (1983) have (a) carried out tests on open wells under constant well discharge conditions (b)recorded the drawdowns during pumping and recovery phases and (c) analysed the data using two approximate equations derived under conditions of two alternative assumptions made on the distribution of the hydraulic gradient in the aquifer (at \( r_w = \) radius of the well itself) and under condition of an additional assumption that the hydraulic gradient at
the well face is linearly related to the drawdown within the well diameter.

\[
\log s = \frac{1}{1.1515} \frac{D}{r_w} \log s (4.25) + \log s \quad \ldots \ldots (4.26)
\]

\[
\log \left( \frac{2D - s}{s} \right) = \frac{1}{1.1515} \frac{D}{r_w} \log k + \log \left( \frac{2D - s_0}{s_0} \right) \quad \ldots \ldots (4.26)
\]

where,

\[
\begin{align*}
\text{r}_w & = \text{radius of the well}, \\
D & = \text{the depth of water column in the well prior to pumping}, \\
k & = \text{permeability of the aquifer}, \\
s & = \text{drawdown with the well diameter}, \\
t & = \text{time since recovery started}, \\
s_0 & = \text{drawdown at } t = 0 \text{ and}
\end{align*}
\]

B and E are constants (all units in Metric system).

They have also pointed out that the equation 4.25 is incidentally of the same form as that given by Slichter (1902) and Basak, (1982). The equation 4.26 is incidentally of the same form as that given by, Kumaraswamy, (1973).
If the equation 4.25 is to be used, the recovery data should be plotted on semilogarithmic graphs taking 's' on logarithmic scale. The slope of the resultant straight line is equal to

\[
\frac{1}{1.1515} \frac{D}{r_w} \text{ KB and}
\]

substituting the values D and r_w, the numerical value of KB can be found out.

To use the equation 4.26, the recovery data should be so plotted that \[(2D - s)/s\] is taken on the logarithmic scale and 't' on linear scale. The slope of the resultant straight line is equal to

\[
\frac{1}{1.1515} \frac{D}{r_w} \text{ KE and}
\]

substituting the values for D and r_w, the numerical value of KE can be determined.

In both these determinations, it may be noted that the numerical values of KB or KE are not unique for a particular dug well, but are actually a function of the well discharge conditions during the abstraction phase (Rajagopalan, 1982). The wells could be considered to have fully recovered if s = 0.01 D. If the equation 4.25 defines recovery within the well diameter then the total time required for full recovery can be computed using
1.1515 \, r \, 100 \, s^* 

\[ t_{fr} (\text{max}) = \frac{1.1515 \, r_w}{D \, KB} \log \frac{100 \, s_o}{D} \] 

If the well is dewatered completely and if \( s_o = D \), then

\[ t_{fr} (\text{max}) = \frac{2.303 \, r_w}{D \, KB} \] 

If the equation 4.26 defines the time required for full recovery within the well diameter, then

\[ t_{fr} (\text{max}) = \frac{1.1515 \, r_w}{D \, KE} \log \frac{100 \, (2D - 0.01 \, s_o)}{2D - s_o} \] 

If the well is dewatered completely then

\[ t_{fr} (\text{max}) = \frac{1.1515 \, r_w}{D \, KE} \log 199.00 \] 

It has been concluded that (a) for the same conditions of \( D \), \( Q \) and \( t_{fr} (\text{max}) \) are directly proportional to \( r_w \) and (b) for the same conditions of \( r_w \), \( Q \) is directly proportional and \( t_{fr} (\text{max}) \) is inversely proportional to \( D \).

**4.5.7 Optimum Yield Determination**

One of the important characteristics of aquifers is the "Optimum Yield" which is the basis for any type of planning and implementation. Karanjac, (1975) proposed the following formula for computing the optimum yield:
\[ Q_o = Q_p \frac{t_p}{t + t_{fr}(\text{max})} \] 

where,

- \( Q_o \) = Optimum yield in m\(^3\)/day
- \( Q_p \) = discharge rate of pumping in m\(^3\)/day
- \( t_p \) = time duration of pumping in days and
- \( t_{fr} \) = total time required for full recovery in days.

It has been found that a higher optimum yield is suggestive of higher permeability. The regions of high optimum yield can be chosen for well field development.

4.6 Conclusion

The water level observations and the results of pump test analysis of the study area has been summarised below.

The analysis of the water table contours generally follow the topography of the area. The groundwater flow is directed towards Vaigai and Gundar river. The configuration of water table contours clearly bring out the effluent nature of these two rivers.

The average annual water level fluctuation varies from a minimum of 0.78 m to a maximum of 4.32 m.
The well hydrographs show not much fluctuation between water level and rainfall, both have the same trend.

A grid deviation map is attempted to delineate positive and negative areas for recharge and discharge.

The salient features of the analysis of the pumping test data are presented below:

1. Table (4.2) shows that the aquifer parameters of the study area, evaluated using the computer program referred to under section 4.5 of this chapter. The range of transmissivity and storage co-efficient values obtained by Jacob's Straight Line Sensitivity Analysis and Theis recovery methods are calculated for the aquifers of Ramanathapuram region. The results of the sensitivity analysis and automated fit of T and S to the pumping test data have been checked up for their usefulness. Fig.4.11 shows the computed and observed drawdowns of the sensitivity analysis for the best fit T and S of some of the wells.

It could be seen that, in Figures, they almost match perfectly and the average drawdown values have been shown in Table (4.3). Hence, T values of this sensitivity analysis have been used for further studies and interpretation. Sandy formations constitute the major sedimentary aquifers of the study area and they show a wide range of T values from 96.5 m²/day to 235 m²/day. The aquifer transmissivity contours of the study area are shown in Fig.4.12. It could be seen that the zones of higher transmissivity are associated with high permeable zones. The median value of T has
therefore been obtained for sandy formations in order to classify a well tapping such aquifers as fundamentally a success or failure. The probability plots have given median T value of 15.75 m²/day (Fig.4.13).

2. Slichter's specific capacity value with increasing recovery time intervals are shown in Table.(4.4). A general observation is made to find out the increase of recovery time with respect to specific capacity.

(a) Increase of specific capacity with Recovery time is observed in wells, 2,3,4,16 and 20.

(b) There is a general decrease of specific capacity with increasing recovery time in wells 1, 5, 6, 9, 11, 13, 15, 17 and 18.

(c). An initial increase and then decrease of specific capacity is observed in some of the wells like 7,8,10,12.

d) There is an initial decrease and then increase in specific capacity in well no. 19 only.

From the first two facts that (i) the sides of the wells are contributing substantially for recovery and (ii) there are horizons with higher aquifer permeability at shallower depths. From the last two factors indicate that the recouperation is accomplished by contribution from the bottom of wells.

The computed specific capacity indices of the dug wells of the study area following the various authors are presented in Table(4.5). In
Fig. 4.14 the percentage probability plotted against Limaye’s Specific Capacity Index with reference to Varyins ranges in the saturated thickness of the wells tapping the sandy aquifers. This figure clearly brings out the fact that the Limaye’s Index is optimum (0.52 lpm/mdd/m²) for wells having STA < 2.50m and increase of STA beyond this does not improve the well performance.

3. Following the method of Rajagobalan (1983) the total time required for full recovery of dugwells of the area have been computed and presented in Table-4.6. It could be seen that (i) the slopes of the straight lines (M₁ and M₂) fitted for the plots of both the methods and are very nearly equal to one another in most of the wells and (ii) tfr (max) calculated from KB and KE are not much different and fall within tolerable limits.

4. Fig. 4.15 has been drawn to demarcate zones based on time required for full recovery. Eventhough 89.10 median value of tfr is coming about 4.5 days (96 hrs), areas characterized by a tfr of less than one day (24 hours) can be considered quite favourable from the point of view of the agriculturists since the wells in these areas permit daily withdrawal of waters and optimum usage. A consideration of this in conjoinction with the optimum yield map Fig. (4.16). will help in picking out locations where a good optimum yield as well as lesser recouperation time co-exist for groundwater extraction.
**TABLE-4.2 : EVALUATED AQUIFER PARAMETERS OF RAMANATHAPURAM DISTRICT**

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FIG-4.11 SENSITIVE ANALYSIS OF PUMP TEST DATA OF RAMANATHAPURAM DISTRICT
TABLE-4.3 : AVERAGE DIFFERENCE BETWEEN OBSERVED AND COMPUTED DRAWDOWNS BY SENSIVITY ANALYSIS

|   | 0.04157547 | 0.06612117 | 0.04586078 | 0.02148496 | 0.03319939 | 0.04548169 | 0.07776989 | 0.02054024 | 0.03555229 | 0.05183223 | 0.01743043 | 0.05741028 | 0.08907078 | 0.02937248 | 0.12084900 | 0.08870447 | 0.08440869 | 0.18147760 | 0.01575049 | 0.01325807 |
TRANSMISSIVITY MAP

RAMANATHAPURAM DISTRICT

SCALE

FIG. 4.12

96
**TABLE-4.4 : SLITCHER’S SPECIFIC CAPACITY (LPM/MDD/M) WITH RECOVERY TIME VALUES**

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NORMAL PROBABILITY PLOT FOR LIMAYE'S INDEX

(1pm/mdd/m)

CUMULATIVE PERCENT

0.1  1  5  20  50  80  95  99  99.9

0  4  8  12  16  20  24

FIG. 4.14
### TABLE-4.6 : TIME FOR FULL RECOVERY OF DUG WELLS

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OPTIMUM YIELD

RAMANATHAPURAM DISTRICT

OPT YIELD (M^3/D)

- < 100
- 100 - 200
- > 200

FIG. 4.16