CHAPTER 7

MODEL CALIBRATION AND APPLICATION

7.1 INTRODUCTION

The problem of seawater intrusion can be formulated in terms of a continuity equation for fluid, a continuity equation for the salt concentration and a constitutive equation relating fluid density to concentration. While studying the problem involving mass transport, it is necessary to solve both the equations as the solution to one depends on the solution of the other. The advection-dispersion equation cannot be solved correctly before the flow velocity distributions are known. These are provided by the flow equation. However, if the flow is density dependent, then the concentrations are to be known to determine the effect of the changed density on the flow pattern. In practical groundwater modelling the flow equation is solved first followed by the advection - dispersion equation.

7.2 SALT CONTAMINATION

As mentioned earlier, the Minjur wellfield of North Chennai aquifer has been intruded by seawater since 1969. This is due to overtapping of groundwater which breaks the delicate balance between the fresh and saltwater flow. The heavy pumping in the Minjur wellfield created pronounced reversal of hydraulic gradient ie from sea to land. As the seawater intruded into the freshwater zone, the chloride concentration of the groundwater increased. Recovering back the contaminated aquifer to the original condition seems to be a formidable task. The main objective is to
prevent further intrusion of seawater and to reclaim the affected aquifer through a proper management strategy.

7.3 SOLUTE TRANSPORT MODEL

Advection, mechanical dispersion, and molecular diffusion are the three distinct processes (Bear, 1979) that contribute to the transport of solute in aquifers. At the seaward boundary, there will be an influx of seawater migrating to the bottom of the aquifer displacing the freshwater upward because of its greater density. Wherever freshwater and seawater come into contact, mechanical dispersion causes mixing and a transition zone will be formed. As the mixed water is lighter than the seawater, it will be displaced upward along with the freshwater and the mixed water will discharge through the window into the sea. This discharge causes a loss of salt from the system replenished by new seawater moving in. This dispersion seems to establish a salt convection cell keeping the system in continuous motion.

If the boundary conditions remain constant, a state of dynamic equilibrium will eventually be attained by the system. At equilibrium, the total fluid mass entering both ends of the aquifer plus the leakage influx at landward side will be balanced by the outflow to the sea. Likewise, the salt mass entering the seawater boundary will be balanced by the salt mass swept out by outward discharge. The role of freshwater at equilibrium is to continuously sweep out the intruding salt and thus prevent further advance of seawater. In all cases, a body of seawater, often in the form of a wedge, exists underneath the freshwater. Freshwater and seawater are actually miscible and therefore the zone of contact between them takes the form of a transition zone caused by hydrodynamic dispersion.

Hydrodynamic dispersion is an unsteady irreversible process in which the salt mass mixes with the liquid. It causes dilution of the solute.
It occurs because of mechanical mixing during fluid advection and molecular diffusion due to the thermal-kinetic energy of the solute particles. Diffusion is a dispersion process that takes place at low velocities. It is time dependent and its effect on the overall dispersion will be more significant at low velocities. The dispersion caused by the motion of the liquid is known as mechanical dispersion.

Mechanical dispersion is viewed as a microscopic process. On the microscopic scale, dispersion is caused by three mechanisms. The first occurs in individual pore channels because molecules travel with different velocities at different points across the channel due to the drag exerted on the fluid by the roughness of the pore surface. The second process is caused by the difference in pore sizes along the flow paths followed by water molecules. Because of differences in surface area and roughness relative to the volume of water in individual pore channels, they have different bulk fluid velocities. The third dispersive process is related to tortuosity, branching, and inter fingering of pore channels. The spreading of the solute in the direction of bulk flow is known as longitudinal dispersion. Spreading in directions perpendicular to the flow is called transverse dispersion. Longitudinal dispersion is normally more stronger than lateral dispersion.

Dispersion is a mixing process. Qualitatively, it has a similar effect in turbulence in surface-water regimes. For porous media, the concepts of average linear velocity and longitudinal dispersion are closely related. Longitudinal dispersion is the process whereby some of the water molecules and solute molecules travel either more rapidly or more slowly than the average linear velocity. The solute therefore spreads out in the direction of flow and has a reduction in concentration. The effects of diffusion are usually negligible compared to those of mechanical dispersion, provided the transport is not in unfractured clays, clayey silts or shales where groundwater velocities are normally low and diffusion is significant.
In the present modelling procedure, miscible fluid method was adopted to simulate the salt transport in the MMAS. The governing equation for flow (MODFLOW) and solute transport (MT3D) models are discussed below.

### 7.3.1 Groundwater Flow Equation

Saline movement is dominated by advective transport. An essential prerequisite for an accurate simulation of transport is an accurate description of the flow obtained by applying Darcy’s law and law of conservation of mass to a control volume. A unit volume of porous media is called an elemental control volume. The law of conservation of mass for steady state flow through porous media requires that the rate of fluid mass flow into any elemental control volume be equal to the rate of fluid mass flow out of any elemental control volume. The equation of continuity that translates this into mathematical form can be written as,

\[
\frac{\partial}{\partial x} \left( K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_z \frac{\partial h}{\partial z} \right) = \frac{\partial}{\partial t} \left( S_s \frac{\partial h}{\partial t} \right) + w(x,y,z,t)
\]

or

\[
\frac{\partial}{\partial x} \left( T_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( T_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( T_z \frac{\partial h}{\partial z} \right) = \frac{\partial}{\partial t} \left( S \frac{\partial h}{\partial t} \right) + w(x,y,z,t)
\]

Where,

\[
S_s = \text{specific storage [L}^{-1}] ,
\]

\[
S = \text{storage coefficient [dimensionless]},
\]

\[
h = \text{hydraulic head [L]},
\]

\[
t = \text{time [T]},
\]
\( K_x, K_y \), hydraulic conductivity in the principal horizontal directions \([LT^{-1}]\),
\( K_z \), hydraulic conductivity in the vertical direction \([LT^{-1}]\),
\( T_x, T_y \), transmissivity in the principal horizontal direction \([L^2T^{-1}]\),
\( w(x,y,z,t) \), the rate of groundwater discharge/recharge per unit area \([LT^{-1}]\) and
\( x, y, z \), Cartesian coordinates directions.

The differential equation (7.1) is replaced by a set of difference equations, one for each point or block. This results in \( n \times m \) simultaneous equations to be solved, where \( n \) is the number of rows and \( m \) is the number of columns of the grid. Surrounding each node is a model block with dimensions \( x, y \) and \( z \), in which the hydraulic properties are assumed to be uniform. Thus to simulate the field conditions using the computer program, the aquifer was divided into blocks and values of aquifer properties were estimated for each block. To solve for the unknown head, spatially and temporally through the model, any one of the solvers, such as, strongly implicit procedure (SIP), Slice-Successive Over Relaxation approach (SSOR) and Preconditioned Conjugate Gradient 2 approach (PCG2) can be used. The unknown head is estimated by solving the finite difference equations for each node until the head change between iterations is less than a specified value. Once this convergence criterion is met, the model advances to a new time interval.

### 7.3.2 Solute Transport Equations

The development of differential equations to describe the transport of solutes in porous materials is to consider the flux of solute into and out of a fixed elemental volume within the flow domain. A conservation of mass statement for this elemental volume is
The physical processes that control the flux into and out of the elemental volume are advection and hydrodynamic dispersion. Loss or gain of solute mass in the elemental volume can occur as a result of chemical or biochemical reactions or radioactive decay. Advection is a component of solute movement attributed to transport by the flowing groundwater. The rate of transport is equal to the average linear groundwater velocity, \( v = V/n \), \( V \) being the specific discharge and \( n \) the porosity. The advection process is sometimes called convection. In addition to advection, mechanical dispersion and molecular diffusion, several other phenomena may affect the concentration distribution of a salt as it moves through a porous medium. The salt may interact with the solid surface, deposition, solution of the solid matrix, ion exchange, etc. All these phenomena cause changes in the concentration of a salt in the flowing liquid. Radioactive decay and chemical reactions within the liquid also cause salt concentration changes. In this research work, chloride ion concentration in the solute is considered as the nonreactive constituent that undergoes only the physical process.

The general form of the advective dispersive equation in cartesian coordinates for solute transport in three dimensional flow through the aquifer can be described by (Freeze and Cherry, 1979).
\[
\frac{\partial}{\partial x}\left[D_x \frac{\partial c}{\partial x}\right] + \frac{\partial}{\partial y}\left[D_y \frac{\partial c}{\partial y}\right] + \frac{\partial}{\partial z}\left[D_z \frac{\partial c}{\partial z}\right] - \left[\frac{\partial}{\partial x}(V_x c)\right] - \left[\frac{\partial}{\partial y}(V_y c)\right] - \left[\frac{\partial}{\partial z}(V_z c)\right] - \frac{\partial c}{\partial t} = 0
\]  
(7.2)

Where,

\(x, y\) and \(z\) are cartesian coordinate directions,

\(V_x, V_y\) and \(V_z\) are the seepage velocities in the respective direction \([LT^{-1}]\),

\(D_x, D_y\) and \(D_z\) are the dispersion coefficients in the respective direction \([L^2T^{-1}]\),

\(C\) stands for solute concentration \(ML^{-3}\) (mass of solute per unit volume of solution) and

\(T\) is the time \([T]\)

The dispersion coefficients in equation (7.2) are related to the dispersivity of the medium, in respective directions and the resultant seepage velocity as

\[D_x = \alpha_l \frac{V_x^2}{|V|} + \alpha_{th} \frac{V_y^2}{|V|} + \alpha_{tv} \frac{V_z^2}{|V|} + D^*\]  
(7.3)

\[D_y = \alpha_l \frac{V_y^2}{|V|} + \alpha_{th} \frac{V_x^2}{|V|} + \alpha_{tv} \frac{V_z^2}{|V|} + D^*\]  
(7.4)

\[D_z = \alpha_l \frac{V_z^2}{|V|} + \alpha_{th} \frac{V_x^2}{|V|} + \alpha_{tv} \frac{V_y^2}{|V|} + D^*\]  
(7.5)

Where, \(\alpha_l\) is the longitudinal dispersivity \([L]\),

\(\alpha_{th}\) is the horizontal transverse dispersivity \([L]\),

\(\alpha_{tv}\) is the vertical transverse dispersivity \([L]\),

\(D^*\) is the effective molecular diffusion coefficient \([L^2T^{-1}]\) and:
The finite difference method is the more frequently used technique
to solve the flow equation. However, it is not often used to solve the
advective-dispersion equation because of a phenomenon known as numerical
dispersion. The numerical solution usually appears to advance the solute at
a rate greater than the physically possible one. Numerical dispersion is
carried by neglecting the second order and higher order terms in the
approximation of $\partial c/\partial x$. Finite difference schemes can be developed to
minimise dispersion. However they are liable to cause either overshooting
or undershooting that appears in the solution as oscillations. There are some
rules helpful in minimising the effects of numerical dispersion. They use a
form of Peclet number, $P$ and Courant number, $C$. The grid should be
designed in such a way that $P (= V \Delta x/D_x) < 2$, where $\Delta x$ is some
characteristic grid size and $D_x$ is the characteristic dispersivity. The
discretisation of time should be such that $C (= V \Delta t/\Delta x) < 1$, where $V$ is the
velocity of flow and $\Delta t$ is the time step. This is same as saying that the time
step should be less than the time it takes for the solute to be advected by
one grid distance.

The method of characteristics (MOC) was developed to solve
hyperbolic partial differential equations and was first used for flow through
porous media by Gardner et al. (1964). For large flow velocities the
dispersion equation is hyperbolic one. The MOC has been extensively
applied for solving the advection - dispersion equation. The basic concept
underlying the application of the method is to decouple the advective and
the dispersive components of the equation and then solve them sequentially.
The solutions are $x = x(t)$, $y = y(t)$ and $C = C(t)$, where $x$ and $y$ are the
coordinates in a cartesian system, $C$ is the pollutant concentration and $t$ is
time. These are called as characteristic curves of the advection - dispersion
equation. Once these solutions are available then a solution of the advection
- dispersion equation can be obtained by following the characteristic curves.

$$|V| = (|V_x|^2 + |V_y|^2 + |V_z|^2)^{1/2}$$ (7.6)
Numerically this is accomplished by inserting a number of reference particles into the finite difference grid and tracing their movement. A simpler way of viewing this approach is to imagine that the solution is being produced by tracking a series of particles such that, at any time, a snapshot can be obtained on each particle's position and pollutant concentration.

A set of moving particles is distributed in the flow field at the beginning of simulation. A concentration and a position in the cartesian coordinate system are associated with each of these particles. Particles are tracked through the flow field using a small time increment. At the end of each time increment, the average concentration at a cell due to advection alone is evaluated from the concentration of particles, which happen to be located within the cell. One of the most desirable features of the MOC technique is that it is virtually free from numerical dispersion, that creates serious difficulty in many numerical schemes. In the solute transport model, instead of placing a uniform number of particles in every finite-difference cell, a dynamic approach is used to control the distribution of moving particles.

The physical phenomena occurring in the seawater intrusion process are represented as partial differential equations. These equations require certain simplifying assumptions to arrive at analytical solution. But in real life problems, it does not satisfy these assumptions. These difficulties have paved way for the development of numerical models. Many assumptions must be made in order to apply the numerical model to field problems as discussed in following sections.

7.4 ASSUMPTIONS

For the flow in aquifers of large horizontal extent, compared to their thickness, it may be assumed that the variations of the groundwater head in a vertical direction are so small that it can be neglected, which leads
to a vertically averaged equation and a two dimensional model in the horizontal plane. In the present analysis, the confined aquifer alone is considered. The recharge to the aquifer is mainly leakage through the aquitard which caused due to difference in the water table and confined aquifer. The movement of the solute is assumed to be horizontal and groundwater flow is assumed not to be affected by the presence of salt in solution. The density and viscosity of the groundwater are assumed to be constant. Considering these assumptions, the solute transport model involves two parts. The predicted hydraulic head distribution arrived from the first part i.e., flow model is utilised in the second part, the solute transport model to estimate the chloride distribution. Solute Transport model is based on the assumption that changes in the concentration field will not affect the flow field significantly. After the flow simulation is complete, transport model retrieves the stored hydraulic heads for further computations. Figure 7.1 shows the working algorithm of the flow and transport model.

7.5 BOUNDARY CONDITIONS

Model boundaries were selected and located to approximate the natural hydrological boundaries of the groundwater flow system. The western most boundaries (landside) of the model is flow boundary calculated from Darcy's equation as shown in Figure 7.2. At the landward side boundary the concentration is constant and is equal to that of freshwater concentration $C_f$ as shown in figure 7.3. On the seaward side constant head boundary is assumed subjected to hydrostatic pressure distribution. Similarly on the seaward side exposed to sea is equal to that of seawater concentration $C_g$. No flow of water and salt are assumed laterally because the aquifer ends at the northern and southern side of aquifer system. The bottom of the aquifer is impermeable i.e., the normal flux through the bed for both fluid and salt is equal to zero. The top of the aquifer is assumed as a recharge boundary through which water leaks into the aquifer. Figure 7.4a
Data Input

Grid generation

Boundary conditions

Time Loop

Flow Loop

solve flow

Flow converged?

Yes

Last time step?

Head Output

Velocities

Time Loop

Transport loop

Solve transport

Transport converged?

Yes

Last time step?

Concentration output

Using MODFLOW

Using MT3D

Fig. 7.1 Flow chart for the flow and transport model
Fig. 7.2  Initial and boundary conditions for flow model

SCALE  1 : 170000
Fig. 7.3 Initial and boundary conditions for solute transport model

SCALE 1 : 170000
Fig. 7.4a Location of pumping and observation wells

Fig. 7.4b Discretisation of the domain

Fig. 7.4c Calibrated hydraulic conductivity

Fig. 7.4d Calibrated storage coefficient

SCALE 1 : 340000
indicates the location of seven piezometers (observation wells) and pumping wells in the Minjur well field. The observation wells considered for the study are 11038, 11548, 11526, 11553, PW32, 11529 and PW 17 which are represented as 1, 10, 6, 12, 19, 13 and 17 respectively in the model domain. First said five piezometers lie in the critical zone and other two in the northern and southern side of critical zone. The critical zone is a high permeable zone resulting into high pumping that causes reduction in piezometric pressure and increase in chloride ion concentration.

7.6 DISCRETISATION OF THE SPACE AND TIME DOMAIN

To simulate the groundwater flow and solute transport with the numerical model, the aquifer was divided into nodal blocks with longitudinal dimension of 250 m and 1000 m as a longitudinal dimension and 300m and 900 m as a lateral dimension. A finite difference grid of 67 rows and 45 columns for a total of 3015 blocks was super imposed over a map of the study area. It covers approximately an areal extent of 320 square kilometres. Certain cells are taken as inactive cells at which there is no flow of water and salt. The discretisation should be done on the basis of stability criteria as explained below.

Both finite difference (FDM) and Finite element (FEM) methods suffer from numerical damping and dispersion errors. Most numerical schemes generate computational errors near source points and sharp gradients (fronts). It is the growth of the numerical dispersion errors which give rise to the generation of wave packets of concentration (negative and positive regions resembling waves). Complex flow regions are often regions with high gradients and large numerical errors. The basic idea behind grid adaptations is to increase the number of grid points in regions of high gradients and reduce the number of grid points (Figure 7.4 b) where the flow is smooth thereby increasing both the solution accuracy and speed of convergence.
The accuracy of the numerical solution is controlled through constraints on the grid discretisation by means of the Peclet and Courant number criteria. The grid peclet criteria, which constrain the spatial discretisation, are

\[ P = \frac{(V \cdot \Delta x)}{D_x} \leq 2 \]

The Courant criterion, which constraints the time step is,

\[ C = \frac{(V \cdot \Delta t)}{\Delta x} \leq \frac{P}{2} \]

The stability requirements for numerical schemes are to be met out. In single time step the fluid moves a distance equal to \((V \cdot dt/n)\). The physical interpretation of the stability requirement is that the distance moved by the fluid in any time step will be less than \(dx\), \(dy\) or \(dz\) or in other words, that the outflow from the cell in any time step will be less than the volume of water in the cell at the beginning of the time step. If the stability requirement of the time step is not met, the calculated solute concentration tend to oscillate with each time step. In some cases, the concentration may be negative.

The domain is subdivided into six subdomains and each subdomain is divided into a number of rectangular elements. The rectangular elements are smaller in size in the regions where the variation in the concentration gradient is relatively high. An intensive grid is provided near the sea boundary. Figure 7.4b shows the discretisation of the domain. The time is discretised at every 30 days for the flow model. But, for the solute transport model, the time is discretised at every 10 days. The same space discretisation is adopted for both the flow and transport model.
7.7 SIMULATION OF SALTWATER MOVEMENT

Three models were used to describe the groundwater mass transport system in MMAS. They are conceptual, numerical flow and mass transport models. The conceptual model can be developed with the help of geology, hydrogeology, boundary and physical parameters that describes the groundwater flow system in the MMAS. The information on the lithology, geometry of the geohydrologic units, the piezometric map, the recharge and discharge rates of the aquifer were required to develop a conceptual model of the MMAS. This conceptual model was translated with the boundary condition and model parameters that were used to formulate numerical flow and transport model. Flow and transport models are computer models that simulate certain aspects of the groundwater flow system. The computer models were used to quantify plume and flow velocities within the aquifer and to aid the visualisation of the groundwater flow system. The primary objective of the transport model was to quantify the migration of a contaminant plume located in the confined aquifer towards a pumping well. The model was used to locate the solute concentration in the aquifer and to predict the movement of the fronts.

Numerical models were used to test the validity of the conceptual model, combines a mathematical model with a conceptual model of the aquifer and computes response variables. These response variables then are compared with field measurements of the same properties. When the simulated response variables from the numerical model approximate the measured response variables, the numerical model is considered to be a reasonable approximation of the modelled aspects of the flow and transport system and by extension the conceptual model of the aquifer is reasonable. Good agreement can be achieved between the response variable and field measurements by specifying realistically, proper aquifer properties and boundary conditions.
The MMAS was modelled under transient state system with all stresses. The aquifer was overstressed resulting in a disturbed situation. Boundary conditions were established to represent as closely as possible the conceptual model of the flow system. The eastern boundary was established as a constant head boundary, Western boundary as variable head boundary and the northern and southern boundaries as a no flow boundaries. Similarly the boundary condition for the solute transport model is constant concentration at sea side, concentration equal to that of freshwater is assumed at western boundary and no solute transport across the northern and southern boundaries. The physical parameters, irrigation pumping, rainfall recharge and areal salinity distribution were assigned to every cell of the geologic unit.

The numerical flow model was used to compute heads at every cell and volumetric flux between active cells and through boundaries. Similarly the solute transport model was used to compute concentration at every cell and salt balance between active cells and through boundaries. The response variables simulated heads and concentration, should be similar to the measured heads. This may rarely occur on the basis of the initial estimates of the aquifer properties. After the initial simulation, the parameters in the model that represent aquifer properties are adjusted to produce better agreement between response variables and measured variables. This process is known as calibration and is discussed in the section "Calibration". There are three steps involved when the theoretical models are applied to the physical system.

They are

i. Establishment of Initial conditions and Model Calibration
ii. Testing the Calibration and
iii. Future projection
7.7.1 Establishment of Initial Conditions and Model Calibration

Calibration is the process by which parameters in the model that represent aquifer properties are adjusted to produce agreement between model response variables and measured properties. Calibration is needed because of uncertainties in formulating the conceptual model of the aquifer and because of measurement uncertainties associated with the determination of the aquifer properties. Calibration reconciles these uncertainties, providing the model parameters are adjusted within reasonable ranges. Some aquifer properties are known with more precision than other properties and during calibration those aquifer properties with small uncertainties are either adjusted or not. For example, the altitudes of the geohydrologic units in the aquifer have small uncertainties and were not adjusted during calibration, whereas recharge and discharge rate that have a larger uncertainty was adjusted as part of the calibration process. For the MMAS, the calibration process consisted of adjusting the recharge and discharge rates for the flow model and dispersivity for the solute transport model to match simulated heads and concentration with measured values.

The goodness of fit was measured by the Mean Difference (MD) and Mean Percentage Error (MPE) between measured and simulated heads or concentrations. The MD was computed by summing the difference between measured and simulated head or concentrations at each wells dividing by the total number of wells. Ideally, the MD should be reduced to nearly zero during the process of calibrating the model. A near zero MD indicates that the deviation between measured and simulated heads or concentrations is nearly zero on the average and the positive differences are balanced by the negative differences. The MPE was computed by averaging the percentage error between measured and simulated heads or concentrations at each time step and at each well. During the calibration process the MD and MPE are reduced by adjusting the aquifer parameters and the leakage. In this process, first the MD and MPE was reduced for the head using flow model.
After achieving a reasonable accuracy the MD and MPE for concentration was reduced by adjusting the dispersivity using the solute transport model.

The numerical model was calibrated in two steps. First, the groundwater flow portion of the model was calibrated and then the solute transport model was calibrated until measured head and concentration matched with simulated water head and concentration. Before calibration, initial distribution of head and concentration must be arrived at to get a good match.

7.7.1.1 Flow analysis

The principal technique used to analyse groundwater flow and yield of the aquifer was a digital computer model. The model was first calibrated for a period of average conditions with little groundwater pumpage (1976-82) and then recalibrated for selected periods of varying climatic conditions and changes in groundwater pumpage (1983-96). The calibrated model was used to simulate long term trends that describe probable future response to selected groundwater development. The steps involved in arriving at initial condition and calibration of the model are discussed below.

For the simulation of the aquifer system for the period of 21 years from 1976 to 1996, the spatial distributions of the water table elevation in the upper formation and the groundwater head of the lower aquifer at the beginning of the year 1976 (initial conditions) are required. Rushton and Wedderburg (1973) have discussed the importance of starting an aquifer simulation from correct initial conditions. Rushton and Redshaw (1979) have suggested the following three methods to represent the initial conditions:

(a) Performing a calculation which attempts to represent the previous history until a dynamic balance is reached.
(b) Using field data as the starting condition.

(c) Applying typical inflows and outflows with very small storage coefficient everywhere. Using the proposed method of solution, the calculation is continued until a steady state is reached.

Method (a) is the most reliable one. But with a digital computer solution it is an expensive in computing effort to represent a large number of historical years. Method (b) requires adequate field data to represent the initial conditions. Otherwise it will lead to serious error. Probably the most efficient approach is to obtain a steady state by the method (c) and then run about five annual cycles with the typical inflows and outflows. As the field data available on the groundwater level are sparse and inadequate, method (b) cannot be applied and hence method (c) is adopted to arrive at the initial conditions. The computation of the initial conditions involves the following steps.

**Step 1.** As a first step to arrive at the initial condition, the model was run with seven years (1976-82) average inflows and outflows and the resultant contour of piezometric heads was compared with the average of seven years observed piezometric heads.

**Step 2.** The spatially distributed seven years average inflows and outflows data were taken and the model was run under steady state for which initial condition and storage coefficient values are not required.

**Step 3.** With the same data used in step 2, the model was run under the transient condition. The steady state head contour obtained from step 2, was taken as initial condition. The actual storage coefficient was multiplied by $10^{-7}$. 
Step 4. The model was run with the actual storage coefficient under transient condition. The head contour obtained from step 3, was taken as initial condition.

Step 5. The parameters such as leakage and irrigation abstraction were adjusted till the computed head contour matched closely with the observed seven years average head contour.

Step 6. To get the steady state condition, the model was run many times by substituting the final head contour of the previous trial as the initial condition of the current trial until the change in the head contour was negligible. The purpose of this step is to arrive at the predevelopment condition of the system.

Simulation of past is an integral part of model development. During this history matching process (model calibration), the model boundaries and hydraulic characteristics are calculated and adjusted until the computed responses match the measured responses.

In this study, the model was calibrated using the data pertaining to the period 1976-82. The initial estimates of aquifer parameters and boundary conditions were adjusted until the model was capable of simulating the historical hydrologic condition of the period 1976-82. The end condition was used as the initial condition for the period 1983 through 1996.

Step 1. Seven years (1976-82) corresponding monthly average inflows, outflows and piezometric head contours were considered for calibration of the model.

Step 2. The head contour arrived at the section 7.7.1.1 in step 5 was used as an initial condition. With this initial condition, the model was tested first for the January month. The ensemble average head
contour for the month of January was compared with the model result.

**Step 3.** Through many trials the physical parameters are adjusted and improved to get a better match with the ensemble average head contour of January. The simulated piezometric head contour of January is the initial condition for the month of February. Again through trials the physical parameters are adjusted to get a match for both January and February. This process was continued for remaining ten months.

**Step 4.** After finalising the aquifer parameters, to get the steady state condition, the model was run for many times by substituting the head contour at the end of December of the previous trial as the initial condition (January) of the current trial till the change in the head contour was negligible.

**Step 5.** The head contours developed at the end of December in Step 4 were used as a starting head distribution (Fig. 7.2) for the calibration period 1976-82.

The model is allowed to simulate the calibration period with step 4 head distribution. The model is considered to be calibrated only when the difference (error) between the observed and simulated head is 0.3 to 1.0 m depending upon the position of the well. If the observation well is near pumping well the allowed error is 1.0 m otherwise 0.3 m. Till a good match is arrived the other parameters like vertical leakage and irrigation pumping were adjusted.

Either recharge or discharge can be fixed and the other quantity has to be adjusted to produce a simulated head distribution that matches the observed head distribution. Early in the process of modelling the MMAS, a decision was made to hold the irrigation pumping constant and to vary
vertical leakage to produce a good match between measured and simulated response variables. After getting the better match, the leakage rate is kept constant and the irrigation pumping was calibrated to produce agreement between measured and simulated heads better than the previous one.

To get good starting head distribution which represents the previous history, it was to undergo 106 trials in a PC-AT 486 with a speed of 66 MHz which took 12 minutes CPU time to run each trial.

After the parameters were adjusted to produce a reasonable match between simulated and measured heads, the solute transport model was used to compute chloride concentration and their movement.

7.7.1.2 Salt transport analysis

Once a satisfactory hydrologic model was obtained, the mass average flux of fluid estimated from the hydrologic model was used in the mass transport equation to simulate the chloride distribution. The physical domain considered for regional simulation was the same as that used for the flow analysis. In addition to the flow pattern, to simulate saltwater movement it is required to have an appropriate boundary and initial conditions and the variation of porosity and dispersivity of the medium on a regional scale. High chloride concentration area, particularly in areas already contaminated, is inadequate to assign appropriate initial condition. This aspect is being considered as another adjustable parameter in the simulation of salt movement. Usually the first approximation of the initial chloride concentration distribution in the contaminated area was done through the consideration of one-dimensional transport along selected streamlines. Depending upon the flow system, the values of dispersivity and porosity can be critical to the modelling purpose. Measurement of dispersivity on a regional scale is difficult. In aquifers consisting of sand and gravel, porosity can be considered more or less uniform on a regional basis.
and is expected to be in the range of 0.3 to 0.45. The simulated solute transport is not sensitive to porosity values in this range.

Steady state simulations were performed using various boundary conditions to evaluate the sensitivity of the system to these changes. The resulting chloride concentration is somewhat similar to the required initial condition, because of the uncertainty in the predevelopment chloride concentration. Although the model used initial conditions that represented a steady state flow and transport, the actual field conditions were probably not at steady state. The chloride concentration assessed by the steady state flow and transport was kept as the initial concentration and it was tried to bring the 1976 initial condition by varying the dispersivity alone without changing other parameters. The dispersivity was tried in the range of 100 m to 1000 m.

After establishing the initial concentration distribution for the year 1976 as shown in Fig. 7.3, the solute transport model was used to simulate the calibration period 1976-82. If it was not able to produce a better match between the observed and simulated concentration, then the parameter dispersivity was adjusted. Only upto 1000 m dispersivity the present system was sensitive, beyond which there was no significant change. Hence it was tried for various initial conditions. It was found that the present aquifer system was very sensitive to initial concentration distribution. By trial and error, attempts were made to minimise the error. In this process, 52 trials were attempted to arrive at a minimum error. After calibrating the solute transport model for the period 1976-82, it was used for testing the calibration for the known period 1983-96.

7.7.2 Testing the Calibration

Once the aquifer parameters are calibrated, their validity should be confirmed by testing the system with data which are not used for
calibration. For this purpose, normally the known time series is divided into two parts as calibration part and test part. In this study the calibration part is from 1976 to 1982 and the test part is from 1983 to 1996. The calibrated parameters using the calibration part (1976-82) are tested by simulating the system for the test part (1983-96) for which the head and concentration contour records are available. The computed head and concentration contours for December 1982 were taken as the initial conditions for January 1983 and the system was tested using the model for the years 1983-96.

7.8 RESULTS OF NUMERICAL SIMULATION

In this section results of calibration part and test part are presented. The results of the calibrated model show that it actually simulates the system and justifies future analysis. Here the goodness of fit between the simulated and observed heads and concentrations, water and salt balances, frontal movement of the interface and trend of temporal changes in head and chloride concentration are discussed.

1. The hydraulic conductivity distribution was estimated in the range of 80 to 160 m/day near Vallur anicut and tend to increase near the Minjur well field as shown in Fig. 7.4c. Specific storage value of $10^{-7}$ to $10^{-5}$ per m (Fig. 7.4d) and porosity of 0.4 was estimated for the modeled area.

2. The calibrated leakage to the aquifer and the quantity of irrigation pumping are shown in Fig. 7.5. It was calibrated that the average leakage and the average irrigation pumping were 23 mcm/year and 30 mcm/year respectively.

4. After the adjustment of recharge and pumping were completed, aquifer dispersivity was calibrated in the range of 100 to 1000 m to get a reasonable match.
Fig. 7.5 Comparison of calibrated leakage with irrigation pumping
5. Figures 7.6 to 7.11 indicate the match between the simulated head with observed head and the simulated concentration with observed concentration. The model is considered to be calibrated if the average percentage error between the observed and simulated head were within 5% at the critical zone. Similarly the goodness of fit was assumed for the observed concentration and simulated concentration only when the average percentage error was within 12% at the critical zone. Figures 7.6a through 7.11b indicate that the simulated head follows a similar trend of observed head resulting in a representation of the physical system. The percentage error for the piezometric heads in the critical zone at the well 11038, 11526 and 11553 were 7%, 4.5% and 1.7% respectively. The MPE for the observation wells PW 32 and PW 17 near the production wells were 9.6% and 13.8% respectively. This higher error is due to draw down and radius of influence of the production wells.

6. The MPE for the concentration was 6.2%, 11.4%, 10.2% and 11.4% for the observation wells 11526, 11553, PW 32 and PW17 respectively.

7. The observed concentration was higher than the simulated concentration at the well 11526 during the period 1986 to 1990 as shown in Fig. 7.7b. It was tried to match with various initial, boundary conditions and dispersivity. This extreme points may be due to some other local contaminations which cannot be simulated in the regional modelling.

8. A reasonable match at the wells 11553 and 11529 was possible with variations at few points (Fig. 7.8b and 7.9b).
Fig. 7.6a  Comparison of observed and simulated heads at the well 11038

Fig. 7.6b  Comparison of observed and simulated piezometric heads at the well 11548
Fig. 7.7a Comparison of observed and simulated piezometric heads at the well 11526

Fig. 7.7b Comparison of observed and simulated chloride ion concentration at the well 11526
Fig. 7.8a  Comparison of observed and simulated piezometric heads at the well 11553

Fig. 7.8b  Comparison of observed and simulated chloride ion concentration at the well 11553
Fig. 7.9a  Comparison of observed and simulated piezometric heads at the well 11529

Fig. 7.9b  Comparison of observed and simulated chloride ion concentration at the well 11529
Fig. 7.10a Comparison of observed and simulated piezometric heads at the well PW 32

Fig. 7.10b Comparison of observed and simulated chloride ion concentration at the well PW 32
Fig. 7.11a Comparison of observed and simulated piezometric heads at the well PW 17

Fig. 7.11b Comparison of observed and simulated chloride ion concentration at the well PW 17
9. The data on observed concentration at the well PW32 was very limited for which the simulated concentration was matching well. The simulated concentrations were neither showing fluctuation nor increasing or decreasing trend. They lie between 40 and 60 mg/l. (Fig. 7.10b)

10. A good match between observed and simulated concentration was observed at the well PW17 (7.11b).

11. The simulated contour show the direction of flow of water. Fig. 7.12 and 7.13 indicate the change in piezometric head contours over the years 1976 to 1996. It is quite evident that the piezometric pressure reduced from -12 m (1976) to -27 m (1988) and it increase to -12 m in 1991 and further improved to -9 m in 1996. The change in minimum piezometric pressure was shown in Fig. 7.14. The piezometric pressure head ranges between -12 m and -17 m during 1976-82, -11 m and -27 m during 1983-90 and -8 m and -15 m during 1991-96. This indicated that the pumping was maximum during the period 1983-90.

12. The shape of the contours were entirely changed after monsoon, ie. the critical zone is separated from the southern zone of the aquifer. The river recharges the southern zone and the water flows to the critical zone due to a head gradient and geological setup. There was rapid improvement in the piezometric pressure after monsoon (Figures 7.15 and 7.16). Seawater flows into the aquifer at the central zone and the freshwater outflows to the sea at the southern zone after the monsoon. The contours before monsoon indicated that the seawater flow to the aquifer was through the entire sea boundary.
Fig. 7.12 Simulated piezometric head contours before monsoon for the years 1976, 1977, 1981 and 1983
Fig. 7.13 Simulated piezometric head contours before monsoon for the years 1986, 1988, 1991 and 1996
Simulated minimum piezometric heads

Fig. 7.14 Simulated minimum piezometric heads
Fig. 7.15 Simulated piezometric head contours after monsoon for the years 1976, 1977, 1981 and 1983

SCALE 1 : 320000
13. The 1000 mg/l isochlor occupied 6.84 km from the coast from 1976 and it moved to 7.30 km in 1981. This may be due to concentrated pumping and low rainfall. It suddenly moved to 11.2 km in 1983 (Fig. 7.17 and 7.18). Actually there was a heavy rainfall of 2000 mm above normal rainfall at the end of the year 1983. Till then the city demand was met by groundwater pumping at Minjur during the period 1982 to October 1983. As the Minjur was overstressed the front suddenly moved to 3.7 km inland. Even though there was a heavy rainfall in the year 1983, it was able to move back the front. After that the front was under a slight regression. This may be due to good rainfall in subsequent years (except 1993) and reduction in the irrigation pumping and industrial pumping. Finally the 1000 mg/l isochlor front occupied 9.95 km in 1996.

14. The area of seawater contamination increased year after year as was quite evident from the three dimension figures 7.19 and 7.20. Figure 7.19d indicate the sudden movement of the front in the year 1983. Figures 7.20 a,b,c and d indicate the areal intensity of chloride ion concentration during subsequent years. It is clearly seen from the figure 7.20d that the isochlor moved back with a reduction in the concentration level.

15. Average inflow of salt was 550 tonnes per year during the period 1976-82; 450 tonnes per year in 1983-90 and 140 tonnes per year in 1991-96. The salt inflow was very high (Figure 7.21) during the first period (1976-82) because of shortfall of rainfall. At the same time it was more during the higher pumping period called second stress period (1983-90). There was heavy rainfall in the period 1983-90 which may be the reason for the reduction in the seawater intrusion and the salt inflow. The salt inflow was reduced by one third during the period 1991-96. This may be due to 5% reduction
Fig. 7.17 Simulated 1000 mg/l isochlor contours for the period 1976-96
Fig. 7.18  Position of 1000 mg/l isochlor
Fig. 7.19  Comparison of the simulated three dimensional frontal movement for the years 1976, 1977, 1981 and 1983
Fig. 7.20  Comparison of the simulated three dimensional frontal movement for the years 1986, 1988, 1991 and 1996
Fig. 7.21 Comparison of rainfall and salt inflow

Fig. 7.22 Comparison of the major inflows and outflows
in the irrigation pumping every year, drastic reduction in the industrial pumping and heavy rainfall.

16. The change in storage was not that much significant as the aquifer is supported continuously by seawater. The reduction in the piezometric pressure is supported by seawater intrusion. For the period 1976-82, rainfall was less, pumping was more as compared to leakage resulting in more seawater intrusion compared to other two periods. Even though there was higher pumping in the second period, the seawater intrusion quantity was lesser than the first period which may be due to heavy rainfall in the second period. Seawater intrusion quantity was reduced drastically during the third period due to reduction in pumping and heavy rainfall. All these information could be seen in the figure 7.22.

17. Table 7.1 summarises the annual rainfall, water and salt balance, minimum head, chloride concentration at 6.5 km, 1000 mg/l isochlor position and their movement for the entire simulation period of 1976-96. Table 7.2 shows the average leakage, irrigation pumping, industrial pumping, total pumping, seawater intrusion quantity, salt inflow, minimum piezometric head, chloride concentration and the position of 1000 mg/l contour for the period 1976-82, 1983-90 and 1991-96.
<table>
<thead>
<tr>
<th>Year</th>
<th>Rainfall Position</th>
<th>Leakages at 6.5 km</th>
<th>Industrial Supply</th>
<th>Total Contour</th>
<th>Irrigation Contour</th>
<th>Min. Head</th>
<th>Net Flow Contour</th>
<th>Position from sea</th>
<th>Rate of Intrusion m/year</th>
<th>qty. m³/day/year</th>
<th>1000 mg/l head</th>
</tr>
</thead>
<tbody>
<tr>
<td>1977</td>
<td>1986</td>
<td>2.79</td>
<td>3.44</td>
<td>3.13</td>
<td>3.44</td>
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<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
</tr>
<tr>
<td>1984</td>
<td>1993</td>
<td>2.79</td>
<td>3.44</td>
<td>3.13</td>
<td>3.44</td>
<td>3.44</td>
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<td>1995</td>
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<td>3.44</td>
<td>3.13</td>
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<td>3.44</td>
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<td>3.44</td>
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<tr>
<td>1987</td>
<td>1996</td>
<td>2.79</td>
<td>3.44</td>
<td>3.13</td>
<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
<td>3.44</td>
</tr>
</tbody>
</table>

Note: The table provides data on rainfall, leakages, industrial supply, total contour, irrigation contour, minimum head, net flow contour, position from sea, rate of intrusion, and quantity of flow per day for the period 1976-1996.
Table 7.2 Summary of simulation results

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Leakage mcm/year</td>
<td>13.36</td>
<td>27.94</td>
<td>27.34</td>
</tr>
<tr>
<td>Irrigation abstraction mcm/year</td>
<td>20.68</td>
<td>34.50</td>
<td>31.42</td>
</tr>
<tr>
<td>Industrial supply mcm/year</td>
<td>11.65</td>
<td>9.56</td>
<td>5.02</td>
</tr>
<tr>
<td>Total pumping mcm/year</td>
<td>32.33</td>
<td>44.06</td>
<td>36.44</td>
</tr>
<tr>
<td>Seawater intrusion mcm/year</td>
<td>18.91</td>
<td>16.74</td>
<td>9.22</td>
</tr>
<tr>
<td>Salt inflow Tonnes/year</td>
<td>550.00</td>
<td>380.00</td>
<td>140.00</td>
</tr>
<tr>
<td>Piezometric head w.r.t. MSL (m)</td>
<td>-14.71</td>
<td>-20.50</td>
<td>-12.00</td>
</tr>
<tr>
<td>Chloride concentration mg/l</td>
<td>1366.00</td>
<td>3490.00</td>
<td>8133.00</td>
</tr>
<tr>
<td>Position of 1000 mg/l contour km</td>
<td>7.16</td>
<td>10.90</td>
<td>10.28</td>
</tr>
</tbody>
</table>

7.9 DISCUSSION

Above figures and table indicate that the MMAS behaved separately in the three periods. The aquifer was overtapped during the period 1983-90 compared to the other two periods 1976-82 and 1991-96. Even though the aquifer was overpumped during the second stress period (1983-90) the rate of seawater intrusion was more in the first period (1976-82) because of low rainfall and sudden increase in the pumping. This sudden pumping created high depression in the piezometric surface which invited the front suddenly to 11 km from the coast. Even though the rainfall was more in the subsequent years and reduction in the total pumping, it was not able to move back the front to the position occupied in the year 1980.

7.10 SIMULATION OF THE SYSTEM FOR FUTURE

As it was discussed earlier, the observation wells considered for the simulation study are 11038, 11548, 11526 and PW 32 which lie in the
critical zone. The critical zone is the zone of high permeability and high pumping that results in seawater intrusion in this zone. The other two observation wells 11529 and PW 1 are situated in the north and south to the critical zone. Observation wells 11529 and PW 1 have not been affected significantly compared with the critical well 11526. The observation well PW 32 has not been affected. In brief, the area upto 6.5 km was affected heavily during the crucial years (1983-90). The deterioration of quality of water at 6.5 km, forced to construct 32 pumping wells between 9 km to 13.5 km. In addition to irrigation pumping, nine to eleven mcm per year was pumped for drinking water from this zone. This created deeper trough between 9 to 13.5 km than the trough at 6.5 km. For the future projections, the observation well 11526 (at 6.5 km from coast) is considered as critical well and the discussions are particular about it.

After simulating the system, the simulated results are used to find the system behaviour for the future. The system was projected and analysed for the period from 1997 to 2020 as future period. The head and concentration contours at the end of December 1996 was taken as initial condition at the start of 1997 and the system was projected for the future. Demand and rainfall recharge are the data to be given for future years. The rainfall data is assumed cyclic and the past records were used for the future. The Government of Tamil Nadu has already been started to purchase the groundwater rights from the farmers in this area. This will considerably reduce the groundwater pumping. To augment the Chennai city water supply an inter-basin transfer scheme called Krishna Water Supply Project is being implemented. According to this project about 340 million cubic metre of water will be released at Poondi reservoir. This water will reduce the requirement from already available sources and that can be utilised for industrial uses. Considering the above points and the water use and demand in the year 1996, different scenarios are formulated. The scenarios considered for analysis are listed below.
Continuing the demand pattern as of 1996 in future.

In this scenario, the demand pattern as of 1996 was continued for the period 1997 - 2020 in order to see the effect of 1996 demand.

i. The Figure 7.23 a,b,c and d indicates that the minimum heads range between -13 m to -10 m in 24 years from 1996 - 2020.

ii. These minimum heads are near the pumping wells located at a distance of 13.5 km from the coast.

iii. Already the industrial pumping was reduced to 3 mcm per year from 9 mcm per year over the period 1983-1996 and the irrigation pumping reduced five percent per year during the period 1991-96, resulting in an improvement in the piezometric surface.

iv. The improvement in the piezometric pressure does not reduce the concentration level. It is having an increasing trend.

v. The 1000 mg/l isochlor reaches to 9.8 km from the coast during 2020 which was originally at 9.95 km (Table 7.3) from the coast in 1996. This isochlor moves back by 195 m seaward.

vi. During the first ten years, the 1000 mg/l isochlor retreats back and then moves inland (Fig 7.24)

vii. About 131 mcm of seawater intruded into the aquifer over 24 years which brings 4.5 million kilogram of salt into the aquifer.
Fig. 7.23  Projected piezometric head contours for Scenario 1
Fig. 7.24  Projected isochlors at an interval of 1000 mg/l for Scenario 1
Table 7.3  Summary of results for scenario 1

<table>
<thead>
<tr>
<th>Year</th>
<th>Minimum head (m)</th>
<th>Concentration at 6.5 km (mg/l)</th>
<th>Position of 1000 mg/l isochlor from the sea (km)</th>
<th>Distance moved by isochlor (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1996</td>
<td>-8</td>
<td>8329</td>
<td>9.95</td>
<td>-</td>
</tr>
<tr>
<td>2005</td>
<td>-13</td>
<td>12756</td>
<td>9.46</td>
<td>-400</td>
</tr>
<tr>
<td>2010</td>
<td>-10</td>
<td>13451</td>
<td>9.46</td>
<td>0</td>
</tr>
<tr>
<td>2015</td>
<td>-11</td>
<td>13884</td>
<td>9.54</td>
<td>+80</td>
</tr>
<tr>
<td>2020</td>
<td>-10</td>
<td>13810</td>
<td>9.79</td>
<td>+250</td>
</tr>
</tbody>
</table>

Negative sign indicates that the 1000 mg/l isochlor moved back or retreated back towards the sea.

7.10.2  Elimination of all Groundwater Withdrawals

In this scenario, all the pumping viz. industrial and irrigation pumping were assumed to be zero. This analysis was made to study the consequences of stopping the pumping completely. This gives an idea on the rate of recovery of the intruded aquifer system. The results obtained for this scenario are not much different from that of the scenario 4.

i. The head at control zone ranges between -8m below MSL (in 1996) and +32 m in 2020 (Fig. 7.25)

ii. In the 24 years, the 1000 mg/l isochlor moves back to 6.2 km from 9.95 km (Fig. 7.26)

iii. The isochlor moves back by a distance of 3750 m in 24 years.

iv. In this scenario, 756 mcm of water flushes 5.7 million kilograms of salt into the sea.

v. The concentration at the critical well reduces from 8300 to 600 mg/l in 24 years.
Fig. 7.25 Projected piezometric head contours for Scenario 5
Fig. 7.26 Projected isochlors at an interval of 1000 mg/l for Scenario 5
Table 7.4 Summary of results for scenario 5

<table>
<thead>
<tr>
<th>Year</th>
<th>Minimum head (m)</th>
<th>Concentration at 6.5 km (mg/l)</th>
<th>Position of 1000 mg/l isochlor from the sea (km)</th>
<th>Distance moved by isochlor (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1996</td>
<td>-8</td>
<td>8329</td>
<td>9.95</td>
<td>-</td>
</tr>
<tr>
<td>2005</td>
<td>+40</td>
<td>2522</td>
<td>9.70</td>
<td>- 250</td>
</tr>
<tr>
<td>2010</td>
<td>+44</td>
<td>1508</td>
<td>7.94</td>
<td>- 1760</td>
</tr>
<tr>
<td>2015</td>
<td>+44</td>
<td>944</td>
<td>7.14</td>
<td>- 800</td>
</tr>
<tr>
<td>2020</td>
<td>+44</td>
<td>606</td>
<td>6.26</td>
<td>- 880</td>
</tr>
</tbody>
</table>

Negative sign indicates that the 1000 mg/l isochlor moved back or retreated back towards the sea.

7.10.3 Comparison of Scenarios 1,2,3,4 and 5

i. If the same pattern of pumping is continued as of 1996 for the future years the fluctuations in the piezometric head reaches dynamic equilibrium (Figure 7.27a) and the frontal movement is also somewhat controlled. However the concentration increases exponentially at 6.5 km (Fig. 7.27b) from the coast due to deep trough created during the period 1976-82 which was continued during the period 1983-90. This greater reduction in piezometric pressure invites seawater rapidly towards the trough at 6.5 km. It may also certain percent due to dispersion effect.

ii. It was observed that even if the pumping rate is increased by ten percent as of the 1996 pumping, the 1000 mg/l front moves towards the land by about 1.36 km and the chloride concentration increases exponentially to about 15500 mg/l at 6.5 km (first trough) from the coast. The piezometric pressure head reduces to 15 m below MSL.
Fig. 7.27a Comparison of fluctuations of projected head between scenarios 1 and 5 at the well 11526

Fig. 7.27b Comparison of variations of projected chloride ion concentration between scenarios 1 and 5 at the well 11526
at the end of 2020 at a distance of 13.5 km (second trough) from the coast. The main reason of increase in chloride concentration at first trough is due to concentrated pumping at the second trough.

iii. To understand the effect of cutting down the industrial pumping alone, in the scenario three, the irrigation pumping alone is considered. It is quite interesting that the 1000 mg/l front moved back approximately by one kilometre and the head improved from -8 m to -4 m with respect to MSL at the end of 2020. This improvement is just due to elimination of three million cubic metre of industrial pumping at second trough.

iv. The scenario four represents the impact of eliminating irrigation pumping. It is quite explicit that the head would definitely increase. The 1000 mg/l front moves back by about 3.5 km and the concentration reduced to about 670 mg/l at the first trough at the end of 2020.

v. Similar improvements are seen in the scenario five, where all the groundwater withdrawals are eliminated. The front retreated back by about 3.7 km and the head improves to 32 m above MSL and the concentration reduces to nearly 600 mg/l at the end of year 2020.
Table 7.5  Summary of results of all the five scenarios at the end of 2020

Initial Piezometric head at the critical zone is -8.0 m
Initial Chloride concentration at the critical well is 8329 mg/l
Initial position of the 1000 mg/l Isochlor is 9.95 km from the coast

<table>
<thead>
<tr>
<th>Scen</th>
<th>Piezometric Head (m)</th>
<th>Chloride Concentration (mg/l)</th>
<th>Position of the front (km)</th>
<th>Net distance moved (km)</th>
<th>Seawater intrusion Quantity (mcm)</th>
<th>Net Salt Flow (mkg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seen 1</td>
<td>-10</td>
<td>13810</td>
<td>9.79</td>
<td>-0.16</td>
<td>131</td>
<td>4.5</td>
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<tr>
<td>Seen 2</td>
<td>-15</td>
<td>15503</td>
<td>11.31</td>
<td>1.36</td>
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<td>Seen 3</td>
<td>-4</td>
<td>12458</td>
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<td>-0.97</td>
<td>97</td>
<td>3.6</td>
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<tr>
<td>Seen 4</td>
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<td>670</td>
<td>6.42</td>
<td>-3.53</td>
<td>-236</td>
<td>-5.8</td>
</tr>
<tr>
<td>Seen 5</td>
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<td>-3.77</td>
<td>-756</td>
<td>-5.7</td>
</tr>
</tbody>
</table>

Negative sign indicates that the flow of water and salt out of the aquifer to the sea.

Negative sign for the net distance moved indicates the frontal movement to the sea.

7.10.4  Inferences

i. Any more increase in pumping is not suggestable after noting the scenario two (110% of 1996 pumping).

ii. At the same time, just eliminating the industrial pumping, it is possible to control the frontal movement (scenario 3).

iii. Both scenarios 4 and 5 behave in the similar manner and there is no significant difference. However it is practically not possible as far as the MMAS is concerned. Even it may not be possible for any aquifer system.
iv. The variations of head reaches dynamic equilibrium but the concentration does not (Fig. 7.27 to 7.30). It is also seen that the head fluctuates below MSL for the scenario one and fluctuates above MSL for the scenario five.

v. The concentration exponentially increases for the scenario one and exponentially reduces for the scenario five. The concentration either increases or decreases rapidly in the first ten years and the change is slower in the next ten years. This indicates that the system started reaching dynamic equilibrium for concentration also.

vi. The trend shows an exponential increasing pattern in concentration at the well 11529 (Fig. 7.28b) representing a northern side of the critical zone if the same pattern of pumping is continued as of 1996.

vii. The well PW 17 shows increasing trend in concentration (Fig. 7.29b). However it increases steadily.

viii. The concentration in the observation well PW 32 increases initially and reaches dynamic steady state latter (Fig. 7.30). The piezometric pressure reduces at the end of 2020 as it compared with the year 1996. It can be expected that the reduction in pressure may cause increase in chloride concentration, which is not so in this case. Because in nature, the bottom of the aquifer is at higher elevation in between 6.5 km and 13.5 km from the coast. If this bottom surface undulation is not existed, by this time the entire length of the critical zone of Minjur wellfield might have intruded by seawater for the concentrated pumping at the second trough.
Fig. 7.28a Comparison of fluctuations of projected head between Scenarios 1 and 5 at the well 11529

Fig. 7.28b Comparison of variations of projected chloride ion concentration between Scenarios 1 and 5 at the well 11529
Fig. 7.29a Comparison of fluctuations of projected head between Scenarios 1 and 5 at the well PW 17

Fig. 7.29b Comparison of variations of projected chloride ion concentration between Scenarios 1 and 5 at the well PW 17
Fig. 7.30a Comparison of fluctuations of projected head between Scenarios 1 and 5 at the well PW 32

Fig. 7.30b Comparison of variations of projected chloride ion concentration between Scenarios 1 and 5 at the well PW 32
After getting exposed to these five scenarios, it was found that, if the pumping is increased the chloride concentration and isochlor movement increases. If the pumping is stopped completely or pumping very little quantity reduces the concentration. But the piezometric pressure head increases very high. It is essential to reduce the pumping in such a way to maintain the piezometric head after meeting the minimum demand. Then the question on the quantity of pumping is to be decided. It is necessary to identify proper pumping pattern to reduce the concentration without increasing the piezometric pressure enormously.

To estimate optimal pumping or recharge simulation alone is not sufficient. It is essential to adopt combined simulation-optimisation technique. Optimal pumping strategy and recharge strategy are evaluated using nonlinear programming and they are discussed in detail in the next chapter.