CHAPTER 4
DEVELOPMENT OF A GROUNDWATER FLOW SIMULATION MODEL FOR THE COMMAND AREA OF PAGLADIYA DAM PROJECT

4.1 INTRODUCTION

The groundwater flow process is an indispensable part of any study involving conjunctive management of the irrigation system. The dynamic behaviour of the aquifer system must be incorporated into the management model, so as to arrive at any optimal conjunctive use policy decision. The best way to represent the response of an aquifer to external stresses is through mathematical modeling of the complex groundwater flow process. Numerous mathematical models have been developed for simulation of the groundwater flow and transport processes. Some of these models have been described in Chapter 2. Depending on their application domain, these models have robust capabilities to describe different physical phenomena involving groundwater. But despite the availability of various tested mathematical models of groundwater flow and transport process, it is sometimes necessary to develop problem specific, simplified models depending on the desired goal to be achieved. Moreover, the computational complexities associated with the LSO models can be addressed to a great extent by linking a simple code to an optimization model developed under the same computing platform, rather than using the complex codes developed under different programming environments.

4.2 THE MODELING PROCESS

A model is a tool designed to represent simplified version of reality (Anderson and Woessner, 1992). Groundwater flow models describe the rate and direction of flow of
groundwater using mathematical equations that are based on certain simplifying assumptions.

The various steps involved in groundwater flow modeling process are - construction of the conceptual model, translating the conceptual model in to a mathematical model, solution of the mathematical model, model calibration and sensitivity analysis.

4.2.1 Model Conceptualization

The first step in groundwater flow modeling is the construction of the conceptual model. A conceptual model comprises a set of assumptions that verbally describe the system’s composition, the mechanisms that govern the flow process and the properties of the porous medium (Bear et al., 1992). The conceptual model should be as simple as possible, but at the same time it should be complex enough to represent the system behavior adequately (Anderson and Woessner, 1992).

Although the contents of the conceptual model may vary with the modeling objectives, following are some of the basic ingredients, as suggested by Bear et al. (1992):

1) The geometry of the boundaries of the investigated aquifer domain
2) The kind of solid matrix comprising the aquifer (with reference to homogeneity, isotropy etc.)
3) Mode of flow in the aquifer (e.g. one dimensional, two dimensional or three dimensional)
4) Relevant state variables in the area (e.g. hydraulic conductivity, storativity etc.)
5) Sources and sinks of water within the domain and its boundary
6) Initial and boundary conditions.

4.2.2 The Mathematical Model

The major steps in mathematical modeling of the ground water flow process consist of
transforming the partial differential equation governing the flow process into discrete equations using some numerical technique and defining the initial conditions of flow and the boundary conditions of the aquifer domain. The solution of such a model refers to the calculation of head values at each point of the system (Anderson and Woessner, 1992). The solution obtained from such a mathematical model should be validated by some established modeling procedures.

4.2.3 Model Calibration and Sensitivity Analysis

Calibration is the process of adjusting the model inputs to achieve a desired degree of correspondence between the model simulations and the natural groundwater flow system. A flow model is considered calibrated when it can reproduce, to an acceptable degree, the hydraulic heads and the groundwater fluxes of the natural system being modeled (Anderson and Woessner, 1992).

A sensitivity analysis is a quantitative evaluation of the influence on the model outputs from variation of model inputs. Sensitivity analysis can be used to aid in model construction by identifying inputs requiring more definition.

4.3 DEVELOPMENT OF A GROUNDWATER FLOW MODEL FOR THE COMMAND AREA OF PAGLADIYA DAM PROJECT

4.3.1 The Conceptual Model

The on-going Pagladiya Dam Project is designed to irrigate a Gross Command Area (GCA) of about 54160 ha on the right bank of river Pagladiya, covering parts of Nalbari district and parts of Baksa district of Assam. Central Groundwater Board (CGWB) under the Ministry of Water Resources, Govt. of India has carried out detailed hydrogeological survey in the undivided Nalbari district (Central Groundwater Board, 1997, 2008, 2011). CGWB has not only made a comprehensive groundwater
resource evaluation in the area through exploratory drilling, but also has come up with suitable groundwater management strategies. The conceptual model in this study is based on the above reports of CGWB.

4.3.1.1 Study area and boundaries

One of the most important steps in groundwater flow modeling is the identification of the aquifer domain and its boundaries. The selection of boundaries of an aquifer domain is determined, based on the presence of some physical features around it. The boundaries should not be too close to the study area, which may be affected by the hydrological activities within the aquifer domain. On the other hand, boundaries far away from the actual area of interest may increase the computational cost in the mathematical model (Anderson and Woessner, 1992). However, the actual location of boundary is determined by the presence of the representative boundary features.

The eastern boundary of the study area is covered by the Pagladiya river. The structural hill of Bhutan, located at about 18 km north of the command area has been considered to be the northern boundary of the study area. On the other hand, the Brahmaputra river has been designated as the southern boundary of the aquifer. The only prominent hydrological feature with available data on the western side of the study area is the river Manas, which is located at about 35 km away from the command area. This river has, therefore, been considered as the western boundary of the aquifer domain despite its distant location.

4.3.1.2 Topography

The general topography of the area is divided into three units- the northern hilly region near Bhutan, the central alluvial region and the southern flood plains of the river Brahmaputra. The average elevation of the hilly region is about 160 m above msl. The central alluvial parts have the elevations of about 60-90 m above msl. This area has a
gentle slope towards the southern direction ending at an elevation of about 45 m above
msl in the flood plains of the Brahmaputra river. The area also has a local gradient in
the east-west direction.

4.3.1.3 Water table

The river Brahmaputra flowing from east to west forms the regional drainage and its
tributaries like Pagladiya, Buradiya, Baralia and Manas rivers flowing from north to
south form the local drainage of the area. Due to the perennial nature of these rivers,
the water table is shallow. Fig. 4.1 shows the water table contours of the study area

Fig. 4.1 Water level contours in parts of the study area during April, 1993
(Source: Central Groundwater Board, Govt. of India)
during April, 1993 (CGWB, 1997). The depths of water level in most of the central and the southern parts of the study area are between 2 m to 4 m bgl. However, in the northern Bhabar and Terai (piedmont) zones, the depth of water table is between 10 m to 20 m bgl. The long term water level fluctuation in the area is quite small with the average pre-monsoon rise/fall in water level of about 0.005/0.038 m/year and the average post monsoon rise of about 0.25 m/year.

4.3.1.4 Aquifer characteristics

CGWB has categorized the aquifer system in the undivided Nalbari district (now divided into Nalbari and Baksa districts) into two categories- the shallow aquifers in the depth range of 0-50 m bgl and the deeper aquifers in the depth range of 50-300 m bgl. No hard rock formation is encountered in the entire area. Groundwater occurs mostly under table conditions with intermittent confining beds of varying thicknesses. The average discharge rate of shallow aquifer is about 30 m$^3$/hr and that of deeper aquifer varies from 120 to 200 m$^3$/hr. The specific yield is in the range of 8-12 %. The hydraulic conductivity values range from 25-30 m/day in the alluvial central and south region to about 45-50 m/day in the northern Terai and Bhabar zones. The western part of the study area extends to Barpeta district. Although groundwater data are not available for Barpeta district, the reports of CGWB suggest more or less uniform aquifer properties of the vast alluvial formations in the entire area (CGWB, 1997, 2008, 2011). Therefore, the aquifer properties of the western parts have been assumed to be similar to the rest of the study area.

4.3.2 The Mathematical Model

A mathematical model involving three dimensional groundwater flow in confined conditions, was earlier developed by using the finite difference technique (Islam and
Talukdar, 2012a). This model was demonstrated through its application in a hypothetical urban water supply system. Similar modeling procedure has been adopted in this study too. However, on the basis of the aquifer characteristics and flow conditions in the study area as described in the conceptual model, a two dimensional model for unconfined flow conditions is proposed in this study.

4.3.2.1 Groundwater flow equation

The theoretical background of groundwater flow is complex. The governing equation is based on the combined physical theories of Darcy’s law and the continuity equation. The two dimensional transient ground water flow through a heterogeneous, porous medium in an unconfined aquifer can be represented by the following partial-differential equation (Trescott et al., 1980):

\[
\frac{\partial}{\partial x} \left( T_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( T_{yy} \frac{\partial h}{\partial y} \right) - W = S_y \frac{\partial h}{\partial t} 
\]

(4.1)

where,

\( h \) = the hydraulic head (m)

\( T_{xx} \) = values of transmissivites along the x - co-ordinate axis (m²/day)

\( T_{yy} \) = values of transmissivites along the y- co-ordinate axis (m²/day)

\( W \) = volumetric flow rate (m³/day)

\( S_y \) = the specific yield of the porous medium (dimensionless)

\( t \) = time (day)

The flow equation for steady state condition can be written as:

\[
\frac{\partial}{\partial x} \left( T_{xx} \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( T_{yy} \frac{\partial h}{\partial y} \right) - W = 0
\]

(4.2)
4.3.2.2 Finite difference discretization of the flow equation

The discretization of the partial differential equation representing ground water flow, converts the continuous variables into discrete variables that are defined at the block centres or nodes. The finite difference and the finite element are the two methods used for numerical modeling of the ground water flow process (Wang and Anderson, 1982). The finite difference technique is conceptually simple and requires less input data than the finite element method (Anderson & Woessner, 1992). Due to its stated advantages, the finite difference technique has been applied in this study. Fig.4.2 shows the block centred finite difference grid system used in the present study. ΔX and ΔY represent the size of grids in x and y directions respectively. The indices i and j increase in positive x and y directions respectively.

![Block centred finite difference grids and associated indices](image)

Fig. 4.2 Block centred finite difference grids and associated indices

The step by step discretization procedure of the flow equation is described below. The space derivatives have been discretized by using the central difference scheme, while
The time derivatives have been discretized by using the backward difference scheme.

The first term on the left hand side of Eq.4.1 has been discretized for cell \((i,j)\) as follows:

\[
\frac{\partial}{\partial x} \left( T_{xx} \frac{\partial h}{\partial x} \right) = \frac{1}{\Delta X_i} \left[ (T_{xx})_{i+1/2,j} \left( \frac{h_{i+1,j} - h_{i,j}}{\Delta X_{i+1/2}} \right) - (T_{xx})_{i-1/2,j} \left( \frac{h_{i,j} - h_{i-1,j}}{\Delta X_{i-1/2}} \right) \right]
\]

\[
= \frac{(T_{xx})_{i+1/2,j}}{\Delta X_i \Delta X_{i+1/2}} (h_{i+1,j} - h_{i,j}) + \frac{(T_{xx})_{i-1/2,j}}{\Delta X_i \Delta X_{i-1/2}} (h_{i,j} - h_{i-1,j}) \quad (4.3)
\]

Similarly, the discretization of the second term of Eq.4.1 yields

\[
\frac{\partial}{\partial y} \left( T_{yy} \frac{\partial h}{\partial y} \right) = \frac{(T_{yy})_{i,j+1/2}}{\Delta Y_j \Delta Y_{j+1/2}} (h_{i,j+1} - h_{i,j}) + \frac{(T_{yy})_{i,j-1/2}}{\Delta Y_j \Delta Y_{j-1/2}} (h_{i,j-1} - h_{i,j}) \quad (4.4)
\]

The source/sink term, \(W\) consists of two parts, i.e.

\[
W_{i,j} = \frac{(q_p)_{i,j} - (q_r)_{i,j}}{\Delta X_i \Delta Y_j} \quad (4.5)
\]

where,

\(q_p\) = rate of pumping (m\(^3\)/day)

\(q_r\) = rate of recharge (m\(^3\)/day)

The time derivative of head on the right hand side of Eq.4.1 has been discretized using the backward difference scheme as:

\[
S_y \frac{\partial h}{\partial t} = (S_y)_{i,j} \frac{h_{i,j}^t - h_{i,j}^{t-1}}{\Delta t} \quad (4.6)
\]

Combining Eq.4.3 through Eq.4.6 and multiplying both sides by \(\Delta X_i \Delta Y_j\), the finite
difference approximation of Eq.4.1 for cell (i,j), at time period t (in implicit form) has been obtained as:

\[
\begin{align*}
CF_1 & = h^{i-1,j} + CF_2 h^{i+1,j} + CF_3 h^{i,j-1} + CF_4 h^{i,j+1} + CF_5 h^{i,j} + CF_6 h^{i,j} \\
& + CF_7 h^{i-1,j} = 0
\end{align*}
\]  

(4.7)

where,

\[
CF_1 = \left( \frac{2(T_{xx})^{i+1,j} (T_{xx})^{i,j}}{(T_{xx})^{i+1,j} \Delta X_{i-1} + (T_{xx})^{i,j} \Delta X_i} \right) \Delta Y_j
\]  

(4.8)

The term in bracket of Eq. 4.8 is the harmonic mean of \(\frac{(T_{xx})^{i-1,j}}{\Delta X_{i-1}}\) and \(\frac{(T_{xx})^{i,j}}{\Delta X_i}\), which represents the ratio of \(\frac{(T_{xx})^{i-1/2}}{\Delta X_{i-1/2}}\).

The use of harmonic mean serves two purposes. Firstly, it ensures continuity across the cell boundaries at steady state, if variable grid size is used. It also makes the appropriate coefficients zero at the no flow boundaries (Trescott et al., 1980).

Similarly,

\[
\begin{align*}
CF_2 & = \left( \frac{2(T_{xx})^{i+1,j} (T_{xx})^{i,j}}{(T_{xx})^{i+1,j} \Delta X_{i-1} + (T_{xx})^{i,j} \Delta X_i} \right) \Delta Y_j \\
CF_3 & = \left( \frac{2(T_{yy})^{i,j+1} (T_{yy})^{i,j}}{(T_{yy})^{i,j+1} \Delta Y_{j-1} + (T_{yy})^{i,j} \Delta Y_j} \right) \Delta X_i \\
CF_4 & = \left( \frac{2(T_{yy})^{i,j+1} (T_{yy})^{i,j}}{(T_{yy})^{i,j+1} \Delta Y_{j-1} + (T_{yy})^{i,j} \Delta Y_j} \right) \Delta X_i \\
CF_5 & = -(CF_1 + CF_2 + CF_3 + CF_4) \frac{(S_y)_{ij}}{\Delta t} \\
CF_6 & = (q_r)^{i,j} - (q_p)^{i,j} \\
CF_7 & = \frac{(S_y)_{ij}}{\Delta t} \Delta X_i \Delta Y_j
\end{align*}
\]  

(4.9) - (4.14)

The transmissivity in an unconfined aquifer is a function of head. The change in head should, therefore, be reflected in the transmissivity values during any transient.
simulation. For example,

\[(T_{xx})_{ij}^t = (K_{xx})_{ij} (b_{i,j}^{t-1})_{ij}\]  \hspace{1cm} (4.15)

where,

\[(T_{xx})_{ij}^t\] = Transmissivity of cells \((i,j)\) along the x-direction at time \(t\)

\[(K_{xx})_{ij}\] = hydraulic conductivity of cell \((i,j)\) along the x direction

\[(b_{i,j}^{t-1})_{ij}\] = saturated thickness of cell \((i,j)\) at time \(t-1\).

For steady state condition, the finite difference equation may be written as:

\[CF1 \ h_{i-1,j} + CF2 \ h_{i+1,j} + CF3 \ h_{i,j-1} + CF4 \ h_{i,j+1} + CF5 \ h_{ij} + CF6_{ij} = 0\]  \hspace{1cm} (4.16)

The coefficients \(CF1, CF2, CF3, CF4\) and \(CF6\) in Eq.4.16 can be obtained in the same manner as those in Eq.4.8, Eq. 4.9, Eq.4.10, Eq.4.11 and Eq.4.13 respectively (without time index), while the coefficient \(CF5\) is given by:

\[CF5 = -(CF1 + CF2 + CF3 + CF4)\]  \hspace{1cm} (4.17)

### 4.3.2.3 Initial and boundary conditions

While simulating the ground water flow process, the initial and boundary conditions representing the aquifer system must be specified. The initial conditions are specified in the form of hydraulic head distribution for the aquifer domain at the initial time. Mathematically,

\[h(x, y) = \Omega(x, y) \ \text{where,} \ \Omega \ \text{is a known function.} \]  \hspace{1cm} (4.18)

Boundary conditions refer to the physical features that act as hydrologic boundaries in an actual ground water system. Generally, three types of boundary conditions are encountered in an aquifer system (Reilly, 2001).

(a) Specified head (Dirichlet) boundary conditions mathematically represented as:

\[h(x, y, t) = \text{constant} \]  \hspace{1cm} (4.19)
A constant head boundary condition is a special category of boundary under specified head boundary conditions, where the head remains constant throughout simulation of the flow model irrespective of time period i.e.

\[ h(x, y) = \text{constant} \quad (4.20) \]

(b) Specified flow (Neumann) conditions mathematically represented as:

\[ \frac{\partial h(x, y, t)}{\partial \perp} = \text{constant} \quad (4.21) \]

where, \( \perp \) is the direction normal to the flow boundary. A no flow boundary is a special category of specified flow boundary where the flow lines can not cross the aquifer boundary, i.e.

\[ \frac{\partial h(x, y, t)}{\partial \perp} = 0 \quad (4.22) \]

(c) Head-dependent flow (Cauchy) conditions mathematically represented as:

\[ \frac{\partial h}{\partial \perp} + C_n h = \text{constant} \quad (4.23) \]

where, \( C_n \) is a constant.

4.3.2.4 Discretization of the study area and the input parameters

A modeling area larger than the irrigation command area has been chosen to include some natural hydrological boundaries of the aquifer, as explained in the conceptual model. The modeled study area lies between the latitudes of 26° 48' N and 26° 15' N and the longitudes of 90° 95'E and of 91° 32', measuring a total area of about 65 km x 60 km. The entire area has been divided into a system of rectangular grids. The discretization of the study area is shown in Fig. 4.3.

The selection of the grid dimension is an important factor in numerical modeling of groundwater flow. Grid size depends on factors like modeling objectives, size of the study area and variation in the hydraulic properties. If the cell size is too large, some
important features of the framework may be left out or poorly represented. On the other hand, too small grid size will lead to increase in the number of model grids and hence will result in enormous computational burden (Reilly and Harbaugh, 2004).

Depending on the modeling objectives and the dimension of the study area, a grid size of 750 m on both directions has been chosen for the actual area of interest (i.e. the command area), while the same for the rest of the study area has been taken as 1000 m. Thus, a total of 5250 (75×70) finite difference grids have been obtained for the entire study area.

![Fig.4.3 Discretization of the study area showing boundary conditions and proposed locations of deep tube wells](image)
As explained in the conceptual model, the eastern, southern and western boundaries of the study area are bounded by perennial rivers (Pagladiya, Brahmaputra and Manas rivers respectively) and hence these boundaries have been designated as specified head boundaries. The river stage data for the three rivers have been collected from the Water Resources Department under the Govt. of Assam. Table 4.1 shows the average monthly river stages at specific locations of these rivers. The heads along the entire boundary reaches have been obtained by linearly manipulating the observed data, following the natural gradients along the flow boundaries. The northern side of the study area is bounded by the structural hills of Bhutan. So, this boundary has been designated as a no flow boundary (hydraulic divide).

Table 4.1 River stage data for aquifer boundaries (Source: Water Resources Deptt., Govt. of Assam)

<table>
<thead>
<tr>
<th>Month</th>
<th>Pagladiya (NH Crossing)</th>
<th>Manas (NH Crossing)</th>
<th>Brahmaputra (Guwahati)</th>
<th>Brahmaputra (Goalpara)</th>
</tr>
</thead>
<tbody>
<tr>
<td>May</td>
<td>50.95</td>
<td>45.95</td>
<td>45.01</td>
<td>32.77</td>
</tr>
<tr>
<td>June</td>
<td>51.20</td>
<td>46.29</td>
<td>46.65</td>
<td>34.09</td>
</tr>
<tr>
<td>July</td>
<td>51.40</td>
<td>46.51</td>
<td>48.01</td>
<td>35.26</td>
</tr>
<tr>
<td>August</td>
<td>51.37</td>
<td>46.62</td>
<td>48.44</td>
<td>35.34</td>
</tr>
<tr>
<td>September</td>
<td>51.34</td>
<td>46.54</td>
<td>47.60</td>
<td>34.31</td>
</tr>
<tr>
<td>October</td>
<td>51.24</td>
<td>46.27</td>
<td>45.67</td>
<td>33.38</td>
</tr>
</tbody>
</table>

The initial hydraulic head distribution in the discretized area has been obtained from the available water level contours for the month of April, 1993, as shown in Fig.4.1. The head values at different grid points have been obtained by linear manipulation of the observed data, depending on the water level contours. The representation of the above data as the initial hydraulic heads for the aquifer domain is justified by the fact that the month of April marks the beginning of monsoon season in
Assam. Further, the water level data in the area shows neither an increasing trend nor a decreasing trend over a long period of time (CGWB, 1997).

The locations of tube wells generally depend on the spacing requirements of different types of well. On the basis of the aquifer test data of the network hydrographs stations located within the study area, the radial distances of 500 m for the shallow tube wells and 3000 m for the deep wells, have been recommended by the CGWB (1997), for the study area. On the basis of these recommendations, 50 deep tube wells and 1050 shallow tube wells have been found to be feasible in the command area of PDP. The proposed locations of the deep tube wells are shown in Fig. 4.3.

4.3.3 Method of Solution

The finite difference discretization of the partial differential equation of groundwater flow results in a set of simultaneous equations. These equations can be written in a generalized form in matrix notation as:

\[
[\text{COEF}][h] = \{\text{CONS}\}
\]

where, \([\text{COEF}]\) is a square matrix of the coefficients of the head vector \([h]\), \([\text{CONS}]\) is a vector of constants whose size is equal to the number of grids. A computer code has been written in MATLAB platform, for solving these simultaneous equations. This program returns the head value at each grid point. MATLAB uses backlash (\(\backslash\)) operator to solve a set of simultaneous equations, i.e.

\[
{h} = [\text{COEF}] \backslash \{\text{CONS}\}
\]

Two sets of codes have been written respectively for the steady state and the transient simulations. Different aquifer parameters such as the grid dimensions, the initial hydraulic head distribution, the hydraulic conductivity, the boundary parameters and the storage coefficient etc. were initially stored in separate worksheets in an EXCEL Spreadsheet. From the EXCEL sheets, these data have been called into a .mat
file of MATLAB and stored in matrix form. The reason for storing the variables in a .mat file is that the MATLAB program file requires much less time to invoke these variables from a native file format than from an external application.

With the initial and the boundary conditions as described in the preceding section, the flow model has been simulated first for the steady state condition and the hydraulic head values obtained from steady state simulation have then been used as the initial conditions for the transient simulations. After each simulation, the values of hydraulic head at the specified head boundaries have been set to their respective specified head values. Further, the change in head at each transient period has been taken into account, while calculating the transmissivity values in the next transient simulation.

4.3.4 Model Validation

Before a mathematical groundwater flow model is calibrated in the field, the numerical results obtained from the model should be validated. In this study, MODFLOW has been used as the model for comparison. MODFLOW is one of the most widely used three dimensional groundwater flow and transport models which was developed by McDonald and Harbaugh (1988). This model simulates steady and non-steady flow in an irregularly shaped flow system in which aquifer layers can be confined or unconfined or both. Flow from external stresses, such as flow to wells, areal recharge, evapotranspiration, flow to drains, and flow through river beds, can be simulated. All types of boundary conditioned can be incorporated into the model. Although MODFLOW has been developed as a 3D model, it eventually turns into a 2D model when a single unconfined layer is used for simulation.

The original program of MODFLOW has a difficult interface, due to which a number of user friendly graphical preprocessors and processors have been developed using MODFLOW and other related programs. In this study, Processing MODFLOW
for Windows (PMWIN), a simulation system developed by Chiang and Kinzelbach (1992), has been used. PMWIN is a complete model provided with a professional Graphical User Interface (GUI). All the important features of MODFLOW are included in this model.

For the purpose of model verification, a uniform saturated thickness of 150 m has been considered for the entire aquifer at steady state condition. The hydraulic conductivity and specific yield have been taken as 25 m/day and 0.01 respectively. Using these flow parameters and boundary conditions as described in the preceding section, simulations have been carried out for both the developed model and PMWIN. Three stress periods of 30 days each, have been considered for comparison under stress conditions with a uniform discharge rate of 2000 m³/day in each of the 50 proposed deep tube wells.

The goodness of fit criterion chosen for comparison is that the Normalized Root Mean Squared Error (NRMSE) between the head values obtained from simulation of the PMWIN model and the developed model at the pumping locations should be less than 10%. This type of criterion is generally used in model validation and calibration process (Waterloo Hydrogeologic Inc., 2000). NRMSE is mathematically expressed as:

\[
NRMSE = \sqrt{\frac{1}{50} \sum_{i=1}^{50} \left( \frac{h_{PM} - h_{GWFM}}{(h_{PM})_{\max} - (h_{GWFM})_{\min}} \right)^2}
\]  

(4.26)

where,

\( h_{PM} \) = the hydraulic head obtained from PMWIN

\( h_{GWFM} \) = the hydraulic head obtained from the developed groundwater flow model

\( i \) = the pumping location index

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The NMRSE values are tabulated in Table 4.2. These values are good enough to make a conclusion that the numerical results obtained from the developed groundwater

Table 4.2 Goodness of fit measures for comparison of results obtained from Processing MODFLOW (PM) and the developed model (GWFM).

<table>
<thead>
<tr>
<th>Goodness of fit criterion</th>
<th>t = 1</th>
<th>t = 2</th>
<th>t = 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>NRMSE</td>
<td>0.023</td>
<td>0.024</td>
<td>0.029</td>
</tr>
</tbody>
</table>

flow model are comparable with those obtained from an established model namely, MODFLOW. In addition to the goodness of fit measures, the hydraulic head values obtained from simulation of the developed flow model and PM are also plotted of comparative contour levels, so as to make a physical comparison of results. The contours, as shown in Fig. 4.4, are in good agreements at all the stress periods, thus ascertaining the validity of the modeling procedures applied in this study.

Fig. 4.4 (a) Comparative plots of contour levels of head values after the first stress period (t = 1)
Fig. 4.4 (b) Comparative plots of contour levels of head values after the second stress period (t = 2)

Fig. 4.4 (c) Comparative plots of contour levels of head values after the third stress period (t = 3)
4.3.5 Calibration and Sensitivity Analysis

Groundwater model calibration is generally performed in two ways—the manual trial and error procedure and automated calibration. For automated calibration, various inverse modeling softwares are available. PEST is one such model, which is used with MODFLOW.

In this study, manual calibration procedure has been adopted. Both steady state calibration and calibrations under transient conditions have been carried out. For this purpose, four Network Hydrograph Stations located within the study area have been chosen as the target locations. The hydraulic head data for these locations, for the period from April, 1993 to January, 1994, are shown in Table 4.3. For steady state calibration, hydraulic head data for the month of April has been chosen as the representative steady state head data, considering the fact that this month marks the beginning of monsoon season in Assam. The model has been run with the input parameters already described in section 4.3.4. Then the computed and observed heads at the specified well locations have been matched. The parameters chosen for steady state calibration are the hydraulic conductivity and the boundary heads.

Table 4.3 Water level data of four observation wells in the study area during the period 1993-1994 (Source: Central Groundwater Board)

<table>
<thead>
<tr>
<th>Location</th>
<th>Elevation (m)</th>
<th>April, 93 Water level (m bgl)</th>
<th>Head (m)</th>
<th>August, 93 Water level (m bgl)</th>
<th>Head (m)</th>
<th>November, 93 Water level (m bgl)</th>
<th>Head (m)</th>
<th>January, 94 Water level (m bgl)</th>
<th>Head (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hazaregoan</td>
<td>56.00</td>
<td>1.22</td>
<td>54.78</td>
<td>1.03</td>
<td>54.97</td>
<td>1.42</td>
<td>54.58</td>
<td>1.68</td>
<td>54.32</td>
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<tr>
<td>Dhamdhama</td>
<td>61.00</td>
<td>3.63</td>
<td>57.37</td>
<td>2.87</td>
<td>58.13</td>
<td>3.23</td>
<td>57.77</td>
<td>4.01</td>
<td>56.99</td>
</tr>
<tr>
<td>Barama</td>
<td>54.00</td>
<td>2.18</td>
<td>51.82</td>
<td>1.51</td>
<td>52.49</td>
<td>2.98</td>
<td>51.02</td>
<td>3.50</td>
<td>50.50</td>
</tr>
<tr>
<td>Mukalmua</td>
<td>45.00</td>
<td>2.75</td>
<td>42.25</td>
<td>0.46</td>
<td>44.54</td>
<td>2.59</td>
<td>42.41</td>
<td>3.17</td>
<td>41.83</td>
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</tbody>
</table>
The convergence criterion considered in this case is the minimization of the Root Mean Squared Error (RMSE). Additionally, R-Squared ($R^2$) measures have also been considered for observing the goodness of fit. These two criteria are explained by the following equations:

\[
RMSE = \sqrt{\frac{\sum_{i=1}^{M} (h_{cal}^i - h_{obs}^i)^2}{M}}
\]  

(4.27)

where, $h_{cal}$ is the calculated head; $h_{obs}$ is the observed head and $M$ is the number of target locations.

\[
R^2 = \frac{\left( \frac{\sum_{i=1}^{M} (h_{cal}^i - \overline{h_{cal}})(h_{obs}^i - \overline{h_{obs}})}{\sum_{i=1}^{M} (h_{cal}^i - \overline{h_{cal}})^2} \right)^2}{\sum_{i=1}^{M} (h_{obs}^i - \overline{h_{obs}})^2}
\]  

(4.28)

where, $\overline{h_{cal}}$ and $\overline{h_{obs}}$ are the means of the calculated and observed heads respectively.

The chosen parameters have been varied gradually. The model has been repeatedly run until the RMSE value has come down to 10%.

With the optimized values of the input parameters, calibration has been carried out again under transient conditions. On the basis of available water table data, three different transient periods have been chosen. The parameters considered for transient calibration are the specific yield and boundary heads at the specified head boundaries during different time periods. In addition, a uniform rate of recharge (rainfall recharge) has also been inserted into the model. Finally, an optimum set of parameters has been obtained, which includes a hydraulic conductivity value of 30 m/day and specific yield of 0.01, apart from the calibrated hydraulic heads at the specified head boundaries. Fig.4.5 shows the plot of observed heads versus simulated heads. The goodness of fit measures obtained from calibration are shown in Table 4.4.
Fig. 4.5 Comparison of observed and computed hydraulic heads at calibration

![Graphs showing simulated and observed heads for different months](image)

(a) Steady state (April, 1993)  
(b) August, 1993  
(c) November, 1993  
(d) January, 1994

Table 4.4 Goodness of fit measures during groundwater model calibration

<table>
<thead>
<tr>
<th>Goodness of fit measure</th>
<th>Calibration period</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>April (steady)</td>
</tr>
<tr>
<td>RMSE</td>
<td>0.323</td>
</tr>
<tr>
<td>$R^2$</td>
<td>0.9978</td>
</tr>
</tbody>
</table>
After calibration of the model for different input parameters, the sensitivity of the model to different parameters has also been analyzed. Parameters selected for sensitivity analysis are the hydraulic conductivity and the specific yield of the aquifer. Sensitivity of the model to one parameter has been tested at a time, keeping the others fixed. In this way, sensitivity analysis has been carried out for each of the input parameters chosen. It has been found that the model is most sensitive to change in the value of specific yield.

4.4 CONCLUSION

A two dimensional groundwater flow simulation model has been developed for the command area of Pagladiya River Project in Assam, using the finite difference technique in MATLAB platform. The validity of this model has been tested by way of comparison with an established groundwater flow and transport model namely, MODFLOW. The results from this model have compared well with those from MODFLOW. The developed flow model has also been calibrated in the study area using the observed field data for steady state as well as transient flow conditions. Finally, a sensitivity analysis has been carried out for selected input parameters of the model.