CHAPTER 3

METHODOLOGY

3.1 GENERAL

This study was carried out in the Arani-Korttalaiyar (A-K) river basin is located at a distance of around 45 km north of Chennai metropolitan city between the geographical co-ordinates of 79° 15’ 0” E - 80° 19’ 0” E longitude and 12° 50’ 0” - 13° 30’ 0” N latitude (Figure 3.1). Chennai is the 4th most popular metropolitan city located on the Coromandel coast of the Bay of Bengal and it has the 3rd largest expatriate population in India. This area was chosen for this study since it has been affected by seawater intrusion (UNDP 1987). This region experience a very dry (summer) period during April to June with a maximum temperature ranging from 32°C to 44°C and a colder (winter) period from December to January, when the temperature ranges from 23°C to 30°C. Relative humidity varies from 65% to 85% in the morning and varies from 40% to 70% during the afternoon (CGWB 2007).

The rainfall in this region depend on monsoons that is southwest (July to September) and northeast (October to December) monsoons. The average annual rainfall is around 1200 mm, 35 % of which fall during the southwest monsoon (July - September) and 60 % during the northeast monsoon (October – December). Figure 3.2 shows the annual variation in rainfall of nine raingauge station for a period of 16 years (1996 - 2012) collected from Tamil Nadu Public Works Department (TNPWD) (2012). Over the period of this study, maximum of an annual rainfall of 1,410 mm
was recorded in the Thiruvallur raingauge station and minimum of an annual rainfall of about 958 mm was recorded in Pernampet raingauge station.

The monthly average climatic parameters of temperature and relative humidity (TNPWD 2012) are shown in Figure 3.3. Arani and Korttalaiyar rivers flowing in this basin are non-perennial rivers and they flow only for about a few days during northeast monsoon season (October to December). Arani river originates at Sadasivakonda, Andhra Pradesh and it joins with the Bay of Bengal. Korttalaiyar river flowing in the southern part of the study area supplies water to Chozhavaram and Redhills reservoirs, thereafter it flows into the Bay of Bengal. The seawater enters into these rivers up to a distance of about 5 km from the coast whenever the river is not flowing. In southwestern side of the study area is bounded by the Palar river. Six well fields are located in the buried paleo channel of Palar river (Figure 3.1). The tube wells from these well fields supply water to the Chennai city. Buckingham canal, a manmade canal run parallel to the coast was constructed for navigation purpose during nineteenth century. The Buckingham canal always carries saline water as it is connected to the sea at a number of places. In order to achieve the objectives of the study, intensive field investigations were carried out several times and integrated model was developed. The methods adopted for carrying out this study and the various modeling tools used are described in the following sections.
Figure 3.1 Location of the Arani-Korttalaiyar (A-K) river basin

Figure 3.2 Temporal annual rainfall variation
3.2 MAPS AND DATA COLLECTED

The Survey of India (SOI) topographs number of 57O7, 57O8, 57O12, 57O15, 57O16, 57P5, 57P9, 66C3, 66C4, 66C7 and 66C8 (scale 1:25,000) covering the study area were used to prepare the base map. The Indian Remote Sensing satellite 1D (IRS 1D) –Linear Imaging Self Scanning sensor III (LISS-III) imagery (Row No.101, path No.62 dated 02nd Apr 2006, Row No.100, Path No. 63 dated 15th May 2006 and Row No.99 path No. 62 dated 10th May 2006) was procured from the National Remote Sensing Centre, Hyderabad and it was used for preparing the various thematic maps. Borehole logs of the study area were collected from CMWSSB. Groundwater head and groundwater abstraction rate from the well field wells for the periods from January 1996 to December 2012 were collected from CMWSSB. Further, the groundwater head from 27 monitoring wells of Tamil Nadu Public Works Department (TNPWD 2012) were also collected. The monthly rainfall from 9 raingauge stations from January 1996 to December 2012 was collected from TNPWD (2012). Monthly river discharge and water
level of the reservoirs were obtained from CMWSSB (2012). The maximum, average and minimum values of daily temperature, dew point, humidity, monthly evapotranspiration and wind speed are obtained from the Indian Metrological Department, Chennai. The topographical variation of the study area was derived from the image of Shuttle Radar Topography Mission (SRTM) for 90 m × 90 m resolution downloaded from the United States Geological Survey.

3.3 GEOSPATIAL ANALYTICAL TOOLS USED

In order to derive the sub catchments from the Arani and Korttalaiyar river basin, several hydrological tools namely reconditioning of raw digital elevation model, fill the sinks, flow direction, flow accumulation, stream order, pour point and watershed of ArcHydro tool box were used from ArcGIS 10.2 software developed by the Environmental Systems Research Institute (ESRI). ArcHydro is a set of data models and tools that operates within ArcGIS for the spatial and temporal analysis of data. Initially the raw digital elevation model was reconditioned by using AGREE method. This method adjusts the surface elevation of the digital elevation model to be consistent with a vector coverage of stream or ridge line. This system is written in ARC/INFO's Arc Macro Language.

After reconditioning, the image was generated free from the sinks. This was made by fill sink tool, it fills the sinks in a grid of reconditioned image. If the depressed cells is surrounded by higher elevation cells, the water is trapped in that depressed cell and it could not flow. The fill sinks tool modifies the elevation value of depressed cell to eliminate these problems. Once the image was corrected with free from sinks, the corrected image was used to generate the flow direction map by using a tool of flow direction. It computes the flow direction for the image. The values in the cells of the flow direction grid indicate the direction of the steepest decline from that cell. That
is the low value of flow direction indicates the flat decline of flow and higher value of the flow direction indicates the steepest decline of flow. The flow accumulation was derived from the flow direction map. It computes the flow accumulation of a image that contains the accumulated number of cells upstream of a cell, for each cell in the input image. The drainage pattern of the area was derived from the flow accumulation map. Further, the streams were ordered based on Strahler method for the purpose of delineating the watershed. Then the demarcation of catchments was made by the selection of pour points. The pour points were used to determine the sub-catchments based on the current and historic gauging station sites snapped to the adjacent cell of highest accumulation. The catchments of Arani and Korttalaiyar rivers were thus generated from the pour points and the flow accumulation.

Land use maps of the study area were prepared from IRS 1 D – LISS-III imagery by supervised image classification algorithm of maximum likelihood in ArcGIS. The characteristics of satellite imagery data, such as tone and texture patterns were used for level three classifications of crop patterns. Verification of these classifications was carried out by several field visits in Kharif and Rabi seasons.

3.4 FIELD INVESTIGATION

Initially, a well inventory survey was carried out during January 2011 to locate the monitoring wells. During this survey several dug and tube wells in this region were studied by measuring the groundwater head and electrical conductivity of the groundwater especially in the coastal part. Based on this survey, 27 dug wells and 22 tube wells were selected as representative wells for regular monitoring of groundwater head. Groundwater head was measured once in two months from January 2011 to December 2013 by using a water level indicator (Solinist 101) and groundwater samples collected in the field were analyzed the concentration of chloride in the laboratory using
Ion Chromotography (IC). The location of the dug and tube wells are shown in Figure 3.4. Figure 3.5 shows the photographs taken during the measurement of groundwater head in few wells. Details of the dug and tube wells selected for monitoring are given in Table 3.1. In order to convert the groundwater head measured below ground level with respect to the sea level, the elevation of the ground surface were measured using Differential Global Positioning System (DGPS) (Leica GS09 GNSS). A photograph taken during this survey is shown in Figure 3.6. Total station (Leica TS06) was used to survey the topographic elevation of the river profile at different locations. A photograph taken during this survey is shown in Figure 3.7.

![Figure 3.4 Location of the dug and tube wells monitored during this study](image-url)
Figure 3.5  Photograph taken during groundwater head measured in few wells
Table 3.1 Details of the dug wells monitored during this study

<table>
<thead>
<tr>
<th>Well No.</th>
<th>Location</th>
<th>Latitude (degree)</th>
<th>Longitude (degree)</th>
<th>Well depth (m)</th>
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Table 3.2 Details of the tube wells monitored during this study

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<th>Well depth (m)</th>
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Figure 3.6  Photograph taken during DGPS survey

Figure 3.7 Photograph taken during total station survey
3.5 INTEGRATED MODELLING APPROACH

The aim of this study is to develop an integrated model capable of simulating the groundwater head based on the rainfall-runoff and infiltration. Since, this approach is to be used in a seawater-intruded area the density variations in the groundwater also need to be incorporated in the model. This section describes the various modeling codes used for this study and the methods adopted for integrating them.

3.5.1 Rainfall-Runoff Model

Rainfall is the major source for surface runoff, river discharge and groundwater recharge. Hence the quantum of runoff that it will generate need to be assessed initially. Thus, rainfall-runoff modelling has to be performed as a first step. This was carried out by using a precipitation-runoff model namely NAM (abbreviation of the Danish word "Nedbør-Afstrømnings-Model"). NAM forms part of the rainfall-runoff (RR) module of the MIKE 11 river modelling system. The rainfall-runoff module can either be applied independently or used to represent one or more contributing catchments that generate lateral inflows to a river network. In this manner, it is possible to treat a single catchment or a large river basin containing numerous catchments and complex network of rivers and channels within the same modelling framework (DHI 2009a). This is a deterministic and lumped rainfall-runoff model which continuously accounts for the moisture content in the overland flow, inter flow, base flow and precipitation (DHI 2012a). This approach was applied independently to represent individual sub catchments of the study area to generate lateral inflows to the Arani and Korttalaiyar rivers. In this manner, the large river basins of the study area containing many catchments and complex network of rivers were considered within the modelling framework.
3.5.1.1 Model structure

This rainfall-runoff model is depends on physical structures and equations used together with semi-empirical ones. As this model is lumped, it treats each catchment as a single unit. The parameters and variables represent, therefore, average values for the entire catchment (DHI 2009a). As a result, some of the model parameters can be evaluated from physical catchment data, but the final parameter estimation must be performed by calibration against time series of hydrological observations. The model structure is shown in Figure 3.8. It simulates the rainfall-runoff process by continuously accounting for the water content in three different and mutually interrelated storages such as surface storage, lower or root zone storage and groundwater storage (DHI 2009a).

Figure 3.8 Conceptualization of rainfall-runoff model
3.5.1.2 Modelling components

The components used for rainfall-runoff modelling are explained in this section (DHI 2009a),

1. Surface storage

Moisture intercepted on the vegetation as well as water trapped in depressions and in the uppermost, cultivated part of the ground is represented as surface storage (DHI 2009a). $U_{\text{max}}$ denotes the upper limit of the amount of water in the surface storage. The amount of water ($U$) in the surface storage is continuously diminished by evaporative consumption as well as by horizontal leakage (interflow).

2. Lower zone or root zone storage

The soil moisture in the root zone, a soil layer below the surface from which the vegetation can draw water for transpiration, is represented as lower zone storage. $L_{\text{max}}$ denotes the upper limit of the amount of water in this storage. Moisture in the lower zone storage is subject to consumptive loss from transpiration. Moisture in the lower zone storage is subject to consumptive loss from transpiration.

3. Evapotranspiration

Evapotranspiration demands are first met at the potential rate from the surface storage. If the moisture content $U$ in the surface storage is less than these requirements ($U < E_p$), the remaining fraction is assumed to be withdrawn by root activity from the lower zone storage at an actual rate $E_a$. $E_a$ is proportional to the potential evapotranspiration and varies linearly with the relative soil moisture content, $L/L_{\text{max}}$, of the lower zone storage,

$$E_a = (E_p - U) \frac{L}{L_{\text{max}}}$$  \hspace{1cm} (3.1)
4. Overland flow

When the surface storage spills, i.e. when $U > U_{\text{max}}$, the excess water $PN$ gives rise to overland flow as well as to infiltration. $QOF$ denotes the part of $PN$ that contributes to overland flow. It is assumed to be proportional to $PN$ and to vary linearly with the relative soil moisture content, $L/L_{\text{max}}$, of the lower zone storage

$$QOF = \begin{cases} 
CQOF \frac{L}{L_{\text{max}} - TOF} P_N & \text{for } L/L_{\text{max}} > TOF \\
0 & \text{for } L/L_{\text{max}} \leq TOF 
\end{cases} \tag{3.2}$$

Where, $CQOF$ is the overland flow runoff coefficient ($0 \leq CQOF \leq 1$)

$TOF$ is the threshold value for overland flow ($0 \leq TOF \leq 1$)

The proportion of the excess water $PN$ that does not run off as overland flow infiltrates into the lower zone storage. A portion, $\Delta L$, of the water available for infiltration, $(PN - QOF)$, is assumed to increase the moisture content $L$ in the lower zone storage. The remaining amount of infiltrating moisture, $G$, is assumed to percolate deeper and recharge the groundwater storage.

5. Interflow

The interflow contribution, $QIF$, is assumed to be proportional to $U$ and to vary linearly with the relative moisture content of the lower zone storage.

$$QIF = \begin{cases} 
CKIF \left( \frac{L}{L_{\text{max}} - TIF} - U \right) & \text{for } L/L_{\text{max}} > TIF \\
0 & \text{for } L/L_{\text{max}} \leq TIF 
\end{cases} \tag{3.3}$$

Where, $CKIF$ is the time constant for interflow, and $TIF$ is the root zone threshold value for interflow ($0 \leq TIF \leq 1$).
6. Interflow and overland flow routing

The interflow is routed through two linear reservoirs in series with the same time constant $CK_{12}$. The overland flow routing is also based on the linear reservoir concept but with a variable time constant

$$CK = \begin{cases} CK_{12} & \text{for } OF < OF_{\text{min}} \\ CK_{12} \left( \frac{OF}{OF_{\text{min}}} \right)^{-\beta} & \text{for } OF \geq OF_{\text{min}} \end{cases} \quad (3.4)$$

Where, $OF$ is the overland flow (mm/hour), $OF_{\text{min}}$ is the upper limit for linear routing ($= 0.4$ mm/hour), and $\beta = 0.4$. The constant $\beta = 0.4$ corresponds to using the Manning formula for modelling the overland flow.

7. Groundwater recharge

The amount of infiltrating water $G$ recharging the groundwater storage depends on the soil moisture content in the root zone

$$G = \begin{cases} (P_N - QOF) \frac{L/L_{\text{max}} - TG}{1 - TG} & \text{for } L/L_{\text{max}} > TG \\ 0 & \text{for } L/L_{\text{max}} \leq TG \end{cases} \quad (3.5)$$

Where $TG$ is the root zone threshold value for groundwater recharge ($0 \leq TG \leq 1$).

8. Soil moisture content

The lower zone storage represents the water content within the root zone. After apportioning the net rainfall between overland flow and infiltration to groundwater, the remainder of the net rainfall increases the moisture content $L$ within the lower zone storage by the amount $\Delta L$

$$\Delta L = P_N - QOF - G \quad (3.6)$$
9. Base flow

The base flow BF from the groundwater storage is calculated as the outflow from a linear reservoir with time constant $CK_{BF}$

3.5.1.3 Basic parameters

The rainfall-runoff model needs 9 basic parameters to generate runoff from giving rainfall and evaporation. That are (1) maximum water content in surface storage, (2) maximum water content in the root zone storage, (3) overland flow runoff coefficient, (4) time constant for interflow, (5) time constant for routing interflow and overland flow, (6) root zone threshold value for overland flow, (7) root zone threshold value for interflow, (8) base flow time constant, and (9) root zone threshold value for groundwater recharge. These parameters are described below (DHI 2007a, DHI 2007b):

1. Maximum water content in surface storage (mm) ($U_{max}$)

The surface storage is moisture intercepted on the vegetation, surface depression storage and a few centimeters of the uppermost soil. The amount of water ($U$) in the surface storage is continuously diminishing by evaporation and interflow leakage. If the maximum storage is reached, some of the excess water (PN) will enter the streams as overland flow and the remainder is infiltrated into the lower zone groundwater storage. A better estimate of $U_{max}$ can be made by the amount of net rainfall occurred in dry periods prior to any observed overland flow. This is an important parameter and plays an important role in altering the values of the overland flow, recharge, amount of evapotranspiration and intermediate flow. By increasing the $U_{max}$, recharge and OF, BF decreases and IF and ActEP increase.
2. Maximum water content in the root zone storage (mm) ($L_{\text{max}}$)

This can be interpreted as the maximum soil moisture content in the root zone available for vegetation transpiration. Since the actual evapotranspiration is highly dependent on the water content of the surface and root zone storages, $U_{\text{max}}$ and $L_{\text{max}}$ are the primary parameters to be changed in order to adjust the water balance in the simulations. Ideally, $L_{\text{max}}$ can be estimated by the following equation:

$$L_{\text{max}} = (\text{field capacity} – \text{wilting point}) \times \text{Effective root depth}$$

In case of some special catchments an estimate of $L_{\text{max}} = 0.1 \times U_{\text{max}}$ can also be taken. It affects OF, Recharge, ActEP and IF, by increasing $L_{\text{max}}$ OF, BF, IF and recharge decreases and ActEP increases.

3. Overland flow runoff coefficient (CQOF)

CQOF is a very important parameter and it describes the fraction of excess rainfall that generates OF and magnitude of infiltration. It’s a dimensionless factor between 0-0.99. Physically in a lumped manner, it reflects the infiltration and also to some extent the recharge conditions. Smaller values of CQOF are expected for a flat catchment having coarse, sandy soils and a large unsaturated zone. Whereas, large CQOF values are expected for catchments having low permeable soils such as clay or bare rocks. By increasing CQOF the peak of OF increases and recharge, BF, IF decreases.

4. Time constant for interflow (hours) (CKIF)

This determines the rate at which surface water ($U$) drains into interflow storage. Physical interpretation of the interflow is difficult. Since interflow is seldom the dominant stream flow component, CKI: in general, is not a very important parameter. Usually, CKIF-values are in the range 500-
1000 hours. It affects the amount of drainage from the surface storage zone as intermediate flow.

5. **Time constant for routing interflow and overland flow (hours)**
   \((CK_{1,2})\)

   This time constant determines the shape of the hydrograph for the overland flow and interflow components. The value of \(CK_{1,2}\) depends on the size of the catchment and how fast it responds to rainfall. Typical values are in the range 3-48 hours. The time constant can be inferred from calibration on peak events. If the simulated peak discharges are too high or arriving too fast, increasing \(CK_{1,2}\) may correct this, and vice versa.

6. **Root zone threshold value for overland flow (TOF)**

   No overland flow occurs until the relative moisture content of the lower zone storage \((L)\) is above this threshold value. Amount of overland flow can be controlled by setting a high TOF value. Threshold value range between 0 and 70% of \(L_{\text{max}}\), and the maximum values allowed is 0.99.

7. **Root zone threshold value for interflow (TIF)**

   Similar to TOF, it determines the relative value of the moisture content in the root zone \((L/L_{\text{max}})\) above which interflow is generated.

8. **Base flow time constant (CK_{BF})**

   This determines the shape of the base flow hydrograph. If the resulting hydrograph is extended for a longer period than observed, decreasing the value may fix \(CK_{BF}\).

9. **Root zone threshold value for groundwater recharge (TG)**

   The root zone threshold value for groundwater recharge (TG) determines the relative value of the moisture content in the root zone \((L/L_{\text{max}})\)
above which groundwater recharge is generated. It affects the soil moisture content that must be satisfied for groundwater recharge to occur. High TG value results in lesser recharge.

Apart from these nine basic parameters, additional parameters related to groundwater were also considered (DHI 2009a)

1. Baseflow time constant $CK_{BF}$

The time constant for baseflow, $CK_{BF}$[hours], determines the shape of the simulated hydrograph in dry periods. According to the linear reservoir description the discharge in such periods is given by an exponential decay. $CK_{BF}$ can be estimated from hydrograph recession analysis. $CK_{BF}$-values in the range 500-5000 hours have been experienced. If the recession analysis indicates that the shape of the hydrograph changes to a slower recession after a certain time, an additional (lower) groundwater storage can be added to improve the description of the base flow.

2. Root zone threshold value for groundwater recharge $TG$

The root zone threshold value for recharge has the same effect on recharge as TOF has on the overland flow. It is an important parameter for simulating the rise of the groundwater table in the beginning of a wet season.

3. Recharge to lower groundwater storage $CQ_{LOW}$

In some cases the shape of the hydrograph recession changes to a slower recession after a certain period. To simulate this, a lower groundwater storage may be included. The parameter $CQ_{LOW}$ determines the proportion of the recharge that percolates to the lower groundwater storage. $CQ_{LOW}$ together with $CK_{low}$ can be estimated from hydrograph recession analysis.
4. Time constant for routing lower baseflow $C_{K_{LOW}}$

The baseflow from the lower groundwater storage is modelled using a linear reservoir with time constant $C_{K_{LOW}}$[hours]. The time constant can be estimated from hydrograph recession analysis. Usually, $C_{K_{LOW}}$ is larger than $C_{K_{BF}}$.

5. Ratio of groundwater catchment to topographical catchment area $C_{area}$

Drainage to or from neighbouring catchments can be modelled by specifying a value of $C_{area}$ different from 1. $C_{area}$ specifies the amount of recharge $G$ that is being drained. If $C_{area} < 1$, part of the recharge, $(1-C_{area})G$, is drained to another catchment, whereas for $C_{area} > 1$, the amount $(C_{area}-1)G$ is added to the catchment recharge.

6. Maximum groundwater depth causing baseflow $GWL_{BF0}$

The maximum depth to the groundwater table for which baseflow occurs, $GWL_{BF0}$ [m], represents the outflow level of the groundwater reservoir given as the distance between the average ground level of the catchment and the minimum level of the river to which it drains. In low, flat areas the annual variation of this distance may be of importance and the facility to allow $GWL_{BF0}$ to vary seasonally is provided in NAM. $GWL_{BF0}$ and the specific yield $S_Y$ can be calibrated by comparing the simulated groundwater level with observations.

7. Specific yield $S_Y$

Values of the specific yield for the groundwater storage may often be assessed from hydrological data e.g. pump tests. Alternatively, $S_Y$-values can be estimated from the literature for different soil types. Small values are found for clay (0.01-0.1) and high values for sand (0.1-0.3).
8. Groundwater depth for unit capillary flux \( GWL_{FL1} \)

\( GWL_{FL1} \) [m] is the depth to the groundwater table which yields an upward capillary flux of 1 mm/day when the moisture content of the upper soil layers is at wilting point, i.e. \( L = 0 \). This parameter will depend on the soil type.

3.5.2 River Flow Model

The discharge from the sub catchments entering into the Arani and Korttalaiyar rivers were estimated by considering the overland and inter flow using the NAM model. The outflows from all the sub catchments result in flow of rivers. This was simulated by using a river discharge and water level model. For this purpose MIKE11 HydroDynamic (HD) model developed by the Danish Hydraulic Institute (DHI) capable of simulating one dimensional (1D) river flow was used. The output obtained from the NAM model was used to assign the boundary conditions for the MIKE 11-HD model. Thus the NAM model described above was completely integrated with MIKE 11-HD. This model was used to simulate the unsteady flow in the Arani and Korttalaiyar rivers using an implicit finite difference scheme. Sub and super critical flow conditions were calculated by solving the full Saint-Venant equations (DHI 2003). The Saint-Venant equations is solved based on the following assumptions

- Water is incompressible and homogeneous,
- Bottom-slope is small, thus the cosine of the angle it makes with the horizontal may be taken as 1
- Wave lengths are large compared to the water depth. This ensures that the flow everywhere can be regarded as having a direction parallel to the bottom, i.e. vertical accelerations can
be neglected and a hydrostatic pressure variation along the vertical can be assumed

- Flow is subcritical

The derivation of the equations of continuity and momentum, as used by MIKE 11,

\[
\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \tag{3.7}
\]

\[
\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( Q \frac{Q^2}{A} \right) + gA \frac{\partial h}{\partial x} + \frac{gQ|q|}{c^2AR} = 0 \tag{3.8}
\]

Where, Q is the discharge, A is the flow area, q is the lateral flow, h is the stage above datum, C is chezy resistance coefficient, R is the hydraulic or resistance radius, \( \alpha \) is the momentum distribution coefficient.

MIKE 11-HD has the option to represent several types of structures in the model, some of which can be operated automatically according to the results of the running simulation. In the present study, a MIKE 11-HD model was set up and used to predict water levels, which was used as input to the groundwater flow model. Further, this model was also used to consider the effects of check dams located in both Arani and Korttalaiyar rivers.

3.5.3 Groundwater Flow and Solute Transport Model

Groundwater model is a representation of a real field aquifer system and it is used to understand the response of changes in hydrological input and output. Accuracy of the model results is dependent on the quality and quantity of the field data obtained to describe input parameters and boundary conditions. Groundwater modelling involves two major steps as conceptualization and translation of the conceptual model in the form of mathematical equations. The conceptual model is an assumption of how the
real aquifer system works with the help of available data. The mathematical model is the representation of processes as equations, physical properties as constants or the coefficients in the equation and measure of state as variables (Konikow 1996). The mathematical model for groundwater flow consists of a partial differential equation together with appropriate boundary and initial conditions that express conservation of mass and describe continuous variables of hydraulic head over the region of interest (Mercer & Faust 1980). According to Bear et al (1992) mathematical model consists of the following:

1. A definition of the geometry of the considered domain and its boundaries,
2. An equation that expresses the balance of the considered extensive quantities,
3. Flux equations that relate the fluxes of the considered extensive quantities to the relevant state variables of the problem,
4. Constitutive equations that define the behavior of the fluids and solids involved,
5. Sources and sinks referred to as forcing functions of the relevant extensive quantities,
6. An equation that expresses initial conditions that describe the known state of the considered system at some initial time, and
7. An equation that defines boundary conditions that describe the interaction of the considered domain with its environment

3.5.3.1 Modelling methods

Mathematical models can be solved by two methods such as analytical model and numerical models. An analytical model is the preferable
method to solve groundwater flow. It is obtained with a number of assumptions concerning the groundwater system and requires sound professional judgment and experience to field situations. However, as the complexity of realistic problems, the solution through this method is not achievable due to the irregularity of the domain’s shape, the heterogeneity of the domain with respect to various coefficients, irregular source functions and various nonlinearities (Bear et al 1992).

Numerical methods are generally used to solve the complex problems by mathematical model. Various numerical methods such as finite difference method, finite volume method, finite element method, boundary element methods, Eulerian - Lagrangian methods and matrix solution are used in order to transform the mathematical model into a numerical one, in which the partial differential equations are represented by their numerical counterparts. A computer program or a code is required in order to solve the numerical model (Bear & Cheng 2010). In recent years, finite difference method and finite element method are the two methods more commonly used for simulating the groundwater flow problems. Finite difference method is based on the application of a local Taylor expansion to approximate the differential equations. In finite difference method, the region of interest is discretized into square network and it uses the strong or the differential form of the governing equations for approximation of partial differential equation. This is a potential constriction of the method when handling complex geometries in multiple dimensions. This was the motivation to use of an integral form of the partial differential equation and subsequently the development of the finite element method.

Finite element method uses integral formulations to approximate partial differential equation. The use of integral formulations is advantageous as it provides a more natural treatment of Neumann boundary conditions as
well as that of discontinuous source terms due to their reduced requirements on the regularity or smoothness of the solution. Moreover, they are better suited than the finite difference method to deal with complex geometries in multi-dimensional problems, as the integral formulations do not rely on any special mesh structure (Peiro & Sherwin 2005).

3.5.3.2 Finite element modelling

In the present study, as the coastal alluvial aquifer has complex geometry, boundary and density-dependent flow, the finite element method of approximation was used. In the finite element method, the area of interest is divided into various irregular triangular shaped elements. It is possible to refine the size of the elements much smaller in regions where a better accuracy is required. The unknown value of groundwater head for different time period is computed at the triangular intersect nodes. Groundwater head of the interior of each cell is determined by interpolation between the nodal points.

Three dimensional variable density groundwater flow simulation in anisotropic and heterogeneous porous media was carried out by Finite Element subsurface FLOW (FEFLOW) version 6.2. FEFLOW is a professional software package for modeling fluid flow and transport of dissolved constituents and/or heat transport processes in the subsurface developed by DHI-WASY GmbH, the German branch of the DHI Group. It contains pre and post processing functionality and an efficient simulation engine, a user-friendly graphical interface provides easy access to the extensive modeling options. The program uses finite element analysis to solve the groundwater flow equation of both saturated and unsaturated conditions as well as mass and heat transport, including fluid density effects and chemical kinetics for multi-component reaction systems (Diersch 1996). Saturated groundwater flow is described by the equation of continuity with a Darcy flux
law. For unsaturated/variably saturated flow, FEFLOW solves the Richard’s equation that assumes a stagnant air phase that is at atmospheric pressure everywhere. FEFLOW provides two different options to define these relationships, such as spline models and empirical models. Spline models are used to derive the parametric relationships from tabular data using one of several different spline interpolation techniques. The following six empirical models are available: 1. Van Genuchten, 2. Modified Van Genuchten, 3. Brooks & Corey, 4. Haverkamp, 5. Exponential, and 6. Linear (DHI 2013). It solves the advection-dispersion balance equations for contaminant mass and heat in two and three dimensions.

3.5.3.3 Groundwater flow equation

Groundwater flow equation is derived by combining the equation of motion in the form of Darcy’s law with the continuity equation (also known as mass balance equation or mass conservation equations) for specific aquifer systems. Darcy’s law states that the rate of flow of water through a porous medium is related to properties of the water, porous medium and hydraulic gradient, which can be written as:

\[ q_i = - K_{ij} \frac{\partial h}{\partial x_j} \]  

(3.9)

Three dimensional groundwater flow equation in an unconfined aquifer given by Rushton (2003) is:

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

(3.10)

Where \( q_i \) is the specific discharge (LT\(^{-1}\)), \( K_{ij} \) is the hydraulic conductivity of the porous medium (LT\(^{-1}\)), \( K_x, K_y, \) and \( K_z \) are the hydraulic conductivities (LT\(^{-1}\)) along x, y and z-coordinate directions, \( h \) is the hydraulic head (L), \( x_i \) is
the Cartesian co-ordinates (L), \( S_s \) is the specific storage (L\(^{-1}\)), \( W \) is the volumetric flux per unit volume (LT\(^{-1}\)) and \( t \) is the time (T). Unsaturated flow involves the unknown variables of fluid pressure (\( \psi \)) and saturation (s). Basic Richards equation is written with these two unknown variables in one balance equation. Finite element model based on the Richards equation is written in the following form which has to be solved either for \( \psi \) or s (DHI 2009b)

\[
R(s, \psi) = S_0 \ s(\psi) \ \frac{\partial \psi}{\partial t} + \varepsilon \ \frac{\partial s(\psi)}{\partial t} - \nabla \cdot \{K_r(\psi)K[\nabla \psi+(1+\chi)e]\} - Q = 0 \quad (3.11)
\]

Where \( \psi \) is the pressure head, \( (\psi > 0 \text{ saturated medium, } \psi \leq 0 \text{ unsaturated medium}) \), \( s(\psi) \) is the saturation, \( (0<s\leq1, s = 1 \text{ if medium is saturated}) \), \( t \) is time, \( S_0 \) is the specific storage due to fluid and medium compressibility, \( \varepsilon \) is porosity, \( K_r(\psi) \) is relative hydraulic conductivity \( (0<K_r\leq1, K_r=1 \text{ if saturated at } s = 1) \), \( K \) is tensor of hydraulic conductivity for the saturated medium (anisotrophy), \( \chi \) is buoyancy coefficient including fluid density effects, e is gravitational unit vector, Q is specific mass supply, and R is residual. This equation was used to simulate the spatial and temporal variation in hydraulic head based on flow between the finite-element cells of the model.

### 3.5.3.4 Solute transport equation

A generalized form of the solute transport equation is presented by Grove (1976) as

\[
\frac{\partial (\varepsilon C)}{\partial t} = \frac{\partial}{\partial x_i} \left( \varepsilon D_{ij} \frac{\partial C}{\partial x_j} \right) - \frac{\partial}{\partial x_i} (\varepsilon CV_i) - C' W * +CHEM \quad (3.12)
\]

Where \( CHEM = -\rho_0 \partial C/\partial t \) for linear equilibrium controlled sorption or ion-exchange reactions. \( D_{ij} \) is the coefficient of hydrodynamic dispersion, (L\(^2\)T\(^{-1}\)), \( C \) is the concentration of the solute in the source or sink fluid, \( C' \) is the
concentration of the species adsorbed on the solid (mass of solute/mass of solid), \( \rho_b \) is the bulk density of the sediment, \((\text{ML}^{-3})\), \( V_i \) is the seepage velocity, \((\text{LT}^{-1})\), \( W^* \) is the volume flux per unit area, \((\text{LT}^{-1})\), and \( \varepsilon \) is the effective porosity of the porous medium. The first term on the right side of equation (3.12) represents the change in concentration due to hydrodynamic dispersion. The second term of equation (3.12) represents advective transport and it describes the movement of solutes at the average seepage velocity of the groundwater flow. The third term of equation (3.12) represents the effects of mixing with in solid phase into solution at the location of the recharge or injection. The fourth term of equation (3.12) lumps all of the chemical, geochemical, and biological reactions that cause transfer of mass between the liquid and solid phases or conversion of dissolved chemical species from one form to another (Konikow 1996).

After substituting the distribution coefficient \((K_d)\) and retardation factor \((R_f)\) in equation (3.12)

\[
R_f \frac{\partial c}{\partial t} = \frac{\partial}{\partial x_i} \left( D_{ij} \frac{\partial c}{\partial x_j} \right) - \frac{\partial}{\partial x_i} \left( CV_i \right) + \frac{C^i W^*}{\varepsilon} - R_f \lambda C \tag{3.13}
\]

Where \( K_d \) is the distribution coefficient, \((\text{L}^3\text{M}^{-1})\), \( R_f \) retardation factor (dimensionless), \( \lambda \) is the decay constant, \((\text{T}^{-1})\). The coefficient of hydrodynamic dispersion is defined as the sum of mechanical dispersion and molecular diffusion (Bear 1979). The mechanical dispersion is a function both of the intrinsic properties of the porous media (such as heterogeneities in hydraulic conductivity and porosity) and of the fluid flow. Molecular diffusion in a porous media will differ from that in free water because of the effects of porosity and tortuosity (Konikow 1996).

\[
D_{ij} = \alpha_{ijmn} \frac{\nu_m \nu_n}{|\nu|} + D_m \tag{3.14}
\]
Where \( \alpha_{ijmn} \) is the dispersivity of the porous medium, \( (L) \), \( V_m \) and \( V_n \) are the components of the flow velocity of the fluid in the \( m \) and \( n \) directions, \( (LT^{-1}) \), \( D_m \) is the effective coefficient of molecular diffusion, \( (L^2T^{-1}) \), and \( |V| \) is the magnitude of the velocity vector, \( (LT^{-1}) \). The dispersivity of anisotropic porous medium can be defined by longitudinal dispersivity \( (\alpha_L) \) of the medium and transverse dispersivity \( (\alpha_T) \) of the medium constants. These are related to the longitudinal and transverse dispersion coefficients by \( D_L = \alpha_L |V| \) and \( D_T = \alpha_T |V| \).

### 3.5.3.5 Density-dependent flow equation

Seawater intrusion occurs based on non-uniform density distributions in coastal aquifers. Variations in groundwater density, resulting from variations in salinity, can have a greater impact on groundwater. Flowing fresh and saline water will create sloping (density) interfaces from coast to inland. Three dimensional density-dependent mass transport is modeled in FEFLOW on the basis of the Darcy law and nonlinear (non-Fickian) dispersion law (DHI 2009b). In the linear Fickian law, the dispersive mass flux of a solute is proportional to the solute concentration gradient. However, if large concentration gradients exist, nonlinear effects become important and the linear Fickian type equation has to be replaced by an extended nonlinear (non-Fickian) dispersion law of

\[
J(\beta \Vert J \Vert + 1) = -D \cdot \nabla C
\]

Where \( \beta \) is a new high concentration (HC) dispersion coefficient, \( C \) is concentration, and \( D \) is dispersion tensor. Density coupled flow and transport processes is simulated by following equations (DHI 2009b)
\[
\frac{\partial (\varepsilon \rho)}{\partial t} + \nabla \cdot (\rho \mathbf{v}) = Q_\rho 
\] (3.16)

\[
\frac{\partial (\varepsilon \mathcal{C})}{\partial t} + \nabla \cdot (\mathbf{C} \mathbf{v} + J_C) = Q_C 
\] (3.17)

\[
\frac{\partial}{\partial t} [\varepsilon \mathcal{E}_f + (1 - \varepsilon) \rho^s \mathcal{E}_s] + \nabla \cdot (\rho \mathcal{E}_f \mathbf{v} + J_E) = Q_E
\] (3.18)

Where the fluid velocity \( \mathbf{v} \) is explicitly given by

\[
\mathbf{v} = -\frac{k}{\mu} \cdot (\nabla p - \rho g) 
\] (3.19)

\[
\mathbf{v} = -Kf_\mu \cdot (\nabla h + \tilde{\rho} \mathbf{e}) 
\] (3.20)

\[
\tilde{\rho} = \frac{\rho - \rho_0}{\rho_0} 
\] (3.21)

Where \( \mathcal{C} \) is the concentration, \( \mathcal{E}_f \) is internal energy density for fluid, \( \mathcal{E}_s \) is internal energy density for solid, \( \varepsilon \) is gravitational unit vector with respect to global coordinates, \( f_\mu \) is fluid viscosity relation function, \( g \) is gravitational acceleration, \( J_C \) is Fickian mass flux vector, \( J_E \) is Fourier thermal energy flux vector, \( K \) is tensor of hydraulic conductivity, \( k \) is tensor of permeability for the porous medium, \( \mathbf{p} \) is fluid pressure, \( Q_\rho \) is bulk fluid flow sink/source, \( Q_C \) is bulk mass sink/source, \( Q_E \) is bulk thermal sink/source, \( \mathbf{v} \) is Darcy velocity vector, \( \alpha \) is solute expansion coefficient, \( \varepsilon \) is porosity, \( \rho \) is fluid density, \( \rho_0 \) is reference fluid density, \( \tilde{\rho} \) is relative fluid density, \( \mu \) is dynamic viscosity of fluid, \( \mathbf{U} \) is fluid density, \( \mathbf{U}_0 \) is reference fluid density, \( \mathcal{E}_s \) is solid density.

Divergence (3.22) and convergence (3.23) form of transport equation using in FEFLOW (DHI 2009b) is given below
\[ \frac{\partial}{\partial t} (RC) + \frac{\partial}{\partial x_i} \left( q_i^f C - D_{ij} \frac{\partial c}{\partial x_j} \right) + R \partial C = Q_c \]  
(3.22)

\[ R_d \frac{\partial c}{\partial t} + q_i^f \frac{\partial c}{\partial x_i} - \frac{\partial}{\partial x_i} \left( D_{ij} \frac{\partial c}{\partial x_j} \right) + (R \partial + Q_p)C = Q_c \]  
(3.23)

With the constitutive equations

\[ \rho_f = \rho_0^f \left[ 1 + \frac{\bar{a}}{(C_s - C_0)} (C - C_0) - \bar{\beta} (T - T_0) \right] \]  
(3.24)

\[ h = \frac{p_f}{\rho_0^f \rho} + x_t \]  
(3.25)

\[ K_{ij} = \frac{k_{ij} \rho_0^f \rho}{\mu_0^f} \]  
(3.26)

\[ \bar{a} = \left[ \frac{\rho_f(c_0) - \rho_0^f}{\rho_0^f} \right] \]  
(3.27)

\[ f_\mu = \frac{\mu_0^f}{\mu^f(C,T)} \]  
(3.28)

\[ D_{ij} = (\epsilon D_d + \beta_T V_q^f) \delta_{ij} + (\beta_L - \beta_T) \frac{q_i^f q_j^f}{V_q^f} \]  
(3.29)

\[ R = \epsilon + (1 - \epsilon) \chi (C) \]  
(3.30)

\[ R_d = \epsilon + (1 - \epsilon) \frac{\partial \chi(C,C)}{\partial C} \]  
(3.31)

\[ \lambda_{ij} = \lambda_{ij}^{cond} + \lambda_{ij}^{disp} \]  
(3.32)

\[ Q_T = \epsilon \rho^f Q_T^f + (1 - \epsilon) \rho^s Q_T^s \]  
(3.33)

\[ \lambda_{ij}^{cond} = [\epsilon \lambda_f + (1 - \epsilon) \lambda_s] \delta_{ij} \]  
(3.34)
\[ \lambda_{ij}^{disp} = \rho_f c_f \left[ \alpha_T V_q^f \delta_{ij} + (\alpha_L - \alpha_T) \frac{q_i^f q_j^f}{V_q^f} \right] \] (3.35)

Where \( h \) is hydraulic head, \( q_i^f \) is Darcy velocity vector of fluid, \( C \) is concentration of chemical component, \( T \) is temperature, \( \rho_f \) is density of fluid, \( \rho_o^f \) is density of reference fluid, \( S_o \) is specific storage coefficient, \( K_{ij} \) is tensor of hydraulic conductivity, \( e_j \) is gravitational unit vector, \( f_{\mu} \) is constitutive viscosity relation function, \( R \) is retardation factor, \( R_d \) is derivative term of retardation, \( D_{ij} \) is tensor of hydrodynamic dispersion, \( \varepsilon \) is porosity, \( c^f, c^s \) is specific heat capacity of fluid and solid respectively, \( \lambda_{ij} \) is tensor of hydrodynamic thermo dispersion, \( Q_x \) is source/sink function of fluid (\( x = \rho \)), of contaminant mass (\( x = C \)) and heat (\( x = T \)), \( \alpha \) is fluid density difference ratio, \( \beta \) is fluid expansion coefficient, \( C_o, T_o \) are reference concentration and temperature respectively, \( C_s \) is maximum concentration, \( p^f \) is fluid pressure, \( g \) is gravitational acceleration, \( k_{ij} \) tensor of permeability, \( \mu^b, \mu_o^f \) are dynamic viscosity and its reference value respectively, of fluids, \( \zeta \) is normalized temperature, \( \omega \) is mass fraction, \( D_d \) is the molecular diffusion coefficient of fluid, \( V_q^f \) is absolute Darcy fluid flux, \( \beta_L, \beta_T \) is longitudinal and transverse dispersivity, \( \chi(C) \) is concentration-dependent adsorption function, \( \lambda_{ij}^{cond} \) is conductive part of thermo dispersion tensor, \( \lambda_{ij}^{disp} \) is dispersive part of thermo dispersion tensor, \( \lambda^f, \lambda^s \) is the thermal conductivity of fluid and solid respectively, \( \alpha_L, \alpha_T \) is longitudinal and transverse thermo dispersivity of fluid respectively. The equations 3.15 to 3.35 are given by DHI (2009b).

### 3.6 COUPLING OF RIVER FLOW AND GROUNDWATER MODEL

The groundwater model needs to consider the recharge from the two major rivers flowing in the study area. For this purpose, the nodes in the
rivers were considered as fluid transfer boundary. The river water level estimated by MIKE 11-HD based on the river discharge generated by considering the input from the various sub catchments, was assigned at the fluid transfer boundary. That is the river water level simulated by MIKE11-HD at the end of the previous time step was used to define the actual boundary values at the nodes in the rivers of the FEFLOW model. In FEFLOW, a river described by fluid transfer boundary type is the only type supported by the coupling module IfmMIKE11. At the end of each FEFLOW time step the exchange rates to these FEFLOW boundary nodes are calculated by the module within FEFLOW. The time step of the groundwater model is controlled by FEFLOW. The spatial overlay of both meshes is automatically integrated within IfmMIKE11 (Monninkhoff & Hartnack 2011). The nodal exchange of discharge (Q) between the ground and surface water is calculated within FEFLOW separately for each single fluid transfer boundary node. The main parameter to control this discharge is an elemental parameter called transfer coefficient $\phi_h$ (Monninkhoff 2014)

$$Q = \phi_h A (h_{ref} - h_{gw})$$

(3.36)

Where $Q$ is the discharge of the fluid (m\(^3\)/day), $\phi_h$ is the transfer rate or leakage factor (d\(^{-1}\)), $A$ is the nodal representative exchange area of the boundary node (m\(^2\)) and $h_{ref}$ and $h_{gw}$ are heads in the river and groundwater (m) respectively.

### 3.7 INTEGRATION OF MODELS

As explained earlier the output obtained from the rainfall-runoff (NAM) model was given as input for assigning the boundary conditions for river flow (MIKE 11-HD) model. The river flow simulated by MIKE 11 predicts the river water level, which was given as the boundary conditions for the groundwater model. The groundwater model (FEFLOW) estimates the
recharge from the river and this volume was transferred again to the MIKE 11-HD for calculating the head during the next time step (Figure 3.9). Further, the MIKE 11-HD will also consider the inflow if any generated by the NAM model. The groundwater head simulated by FEFLOW is again given as an input for NAM model to generate the overland and base flow going out of the sub catchment. This approach is pictorially represented in the Figure 3.10.

Figure 3.9  Integration of rainfall-runoff, river flow and groundwater model
Figure 3.10 Conceptualization of rainfall-runoff, river flow and groundwater model