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Conjunctive Use of Surface Water and Groundwater for Sustainable Water Management

H. Ramesh and A. Mahesha
Dept. of Applied Mechanics & Hydraulics, National Institute of Technology Karnataka, Surathkal, Mangalore India

1. Introduction

A critical problem that mankind had to face and cope with is how to manage the intensifying competition for water among the expanding urban centers, agricultural sectors and in-stream water uses. Water planner can achieve a better management through basin-wide strategies that include integrated utilization of surface and groundwater which may be defined as conjunctive use (Todd, 1956). Conjunctive use is the simultaneous use of surface water and groundwater. Investment in conjunctive use raises the overall productivity of irrigation systems, extends the area effectively commanded, helps in preventing water logging and can reduce drainage needs. Lettenmaier and Burges (1982) distinguished conjunctive use which deals with the short term use from the long term discharging and recharging processes known as cycle storage. Until late 1950s, development and management of surface water and groundwater were dealt separately, as if they were unrelated systems. Although the adverse effects have been evident, it is only in recent years that conjunctive use is being considered as an important water management practice.

Conjunctive use of surface and groundwater is not a new concept but it has been in practice since last three decades. The term 'conjunctive' used here is to integrate surface and groundwater resources. It includes interaction between surface water and groundwater through groundwater recharge, hydrological cycle, water balance components etc. These parameters will be used for modeling the groundwater flow and its interaction with surface water. Buras (1963) used dynamic programming to determine design criteria and operating policy for a conjunctively managed system supplying water to agricultural fields. Chun et. al., (1964) used a simulation model to examine alternative plans for conjunctive operation of surface water and groundwater in California, USA. Dracup (1965); Longenbaugh (1970) and Milligan (1969) developed a parametric linear programming model for a conjunctive surface water and groundwater system in southern California, USA.

A GIS linked conjunctive use groundwater – surface water flow model (MODFLOW) was done by Ruud et al, (2001); Sarwar (1999). An overview paper on conjunctive use of surface water and groundwater was presented by Wranchien et al, (2002) giving more emphasis to holistic approach of management. The interaction between surface and groundwater was also studied by various authors elsewhere (Sophocleous, 2002; Ozt et al, 2003; LaBolle et al, (2003). A simple groundwater balance model was developed (Peranginangin et al, 2004) based on 15–20 years (1980–1999) of hydro-meteorological, land use, soil and other relevant data to generate the hydro-geologic information needed for the water-accounting procedure in conjunctive use. A regional conjunctive use model was developed by Rao et al, 2004; Schoups et al, 2005 for a near-real deltaic aquifer system irrigated from a diversion system with some reference to hydro-geoclimatic conditions prevalent in the east coastal deltas of India. A numerical model for conjunctive use surface and groundwater flow was developed and alternating direction implicit method was applied for model solution (Chuenchooklin et al, 2006).

2. Conjunctive use optimization

Optimization techniques were introduced by Castle and Linderborg (1961), who formulated a linear programming to allocate water from two sources (surface water and groundwater) to agricultural areas. Due to the development of advanced digital computer and optimization technique, later, a dynamic programming model (Aron 1969) developed to determine the optimum allocation of surface water and groundwater. Yu and Haimes (1974) discussed hierarchical multi-level approach to conjunctive use of surface water and groundwater systems, emphasizing hierarchical decision making in a general sense. Integrated Groundwater and Surface water Model (IGSM) was first developed by Yan and Smith (1994) at the University of California, USA. The major constraint in IGSM is the semi-explicit time descretization and its incapability that fails to properly couple and simultaneously solve groundwater and surface water models with appropriate mass balance head convergence under practical conditions. An extensive examination of the literature covering conjunctive use of groundwater-surface water summarized chronologically (Maknoon and Burges, 1978); Miles and Rushton (1983); (McKee et al, 2004), reveals in nearly all cases that the analysis of conjunctive use was dominated by one or several parameters which were extensively modeled.

The optimization models were developed by Menenti et al, (1992) and Deshan, (1995) and Karamouz et al, (2004) to allocate optimum water for agricultural benefits in the river basins. Effective use of groundwater simulation codes as management decision tools requires the establishment of their functionality, performance characteristics and applicability to the problems at hand (Paul et al, 1997). This is accomplished through systematic code-testing protocol and code selection strategy. The protocol contains two main elements: functionality analysis and performance evaluation. Functionality analysis is the description and measurement of the capabilities of a simulation code; performance evaluation concerns the appraisal of the code's operational characteristics (e.g., computational accuracy and efficiency, sensitivity for problem design and parameter selection and reproducibility). Testing of groundwater simulation codes may take the form of (1) benchmarking with known independently derived analytical solutions; (2) intra-comparison using different code functions inciting the same system responses; (3) inter-comparison with comparable simulation codes; or (4) comparison with field or laboratory experiments. The results of the various tests are analyzed using standardized statistical and graphical techniques to identify performance strengths and weaknesses of code and testing procedures. The solution of optimization model was done by dynamic programming. A multi-stage decision model was developed by Azaiez (2002) for the conjunctive use of groundwater and surface water with an artificial recharge. He assumed certain supply and a random demand and an integrated opportunity cost explicitly for the unsatisfied demand. He also incorporated the importance of weight attributed by the decision-makers to the final groundwater level at the end of the planning horizon. An integrated hydrologic-economic modeling framework for optimizing conjunctive use of surface and groundwater at the river basin scale (Velázquez et al, 2006).

3. Conjunctive use modeling options

Conjunctive use modeling of surface water and ground water has wide applications in water resources management, ecology, eco-hydrology and agricultural water management. Conjunctive use model are developed based on the purpose and objective. Conjunctive use model are developed based on the technique used and may be classified as :

- Simulation and prediction models,
- Dynamic programming models,
- Linear programming models,
- Hierarchical optimization models,
- Nonlinear programming models and others.

Simulation approaches provide a framework for conceptualizing, analyzing and evaluating stream–aquifer systems. Since the governing partial differential equations for complex heterogeneous ground water and stream–aquifer systems are not amenable to closed form analytical solution, various numerical models using finite difference or finite element methods have been used for solution) simulation and optimization models and decision-support tools that have proven to be valuable in the planning and management of regional water supplies (Chun et al., 1964; Bredehoeft and Young, 1983, Latif and James, 1991; Chaves-Morales et al., 1992; Marino, 2001).

The system dynamics, initially developed by Jay W. Forrester (Forrester 1961), uses a perspective based on information feedback and mutual or recursive causality to understand the dynamics of complex physical, biological, social, and other systems. In system dynamics, the relation between structure and behavior is based on the concept of stock-flow diagrams. The process of model development, combining program flowchart with spatial system configuration, provokes modeler can build model easily. System dynamics is a computer-aided approach to evaluate the interrelationships of components and activities within complex systems. The most important feature of this approach is to elucidate the endogenous structure of the system under study, to see how the different elements of the system actually relate to one another, and to experiment with changing relations within the system when different decisions are included. Dynamic programming (DP) has been used because of its advantages in modeling sequential decision making processes, and applicability to nonlinear systems, ability to incorporate stochasticity of hydrologic processes and obtain global optimality even for complex policies (Buras, 1963; Aron, 1969; Provencher and Burt, 1994). However, the ''curse of dimensionality'' seems to be the major reason for limited use of DP in conjunctive use studies as it considers physical system as lumped.

Linear Programming (LP) has been the most widely used technique in conjunctive use optimization models.. However, nonlinearities may arise due to the physical representation of the system or the cost structure for surface and groundwater use. For example, Stream-aquifer interaction can be represented by a linear function of stream stage and groundwater elevation where groundwater level is at or above the streambed. However, the stream stage is a nonlinear function of discharge or reservoir release.

Hierarchical optimization was first defined by Bracken and McGill (1974) as a generalization of mathematical programming. In this context the constraint region is implicitly determined by a series of optimization problems which must be solved in a predetermined sequence. Hierarchical optimization models were developed and applied in conjunctive use by Maddock (1972, 1973); Yu and Haimes (1974) and Paudyal and Gupta (1990).

Non linear programming models: The solution of a conjunctive use problem with nonlinear constraints because of very complex and some parameters are non linear. Hence such a model is called nonlinear conjunctive use optimization model. E.g,. In order to solve the conjunctive use problem, the ground water flow and mass transport models will need to be run numerous times that the problem may not be solvable (Taghavi et al. 1994). E.g., groundwater quality problems and groundwater head constraint.

Despite the many different optimization models and techniques that have been applied, most conjunctive use optimization work reported in the literature deal with hypothetical problems, simple cases or steady state problems. The lack of large-scale complex real world conjunctive use optimization studies is probably due to the great size of the problem resulting when many nodes-cells and long time periods are under consideration for modeling groundwater flow and the interaction between surface and groundwater. Most conjunctive use models reported are created "ad hoc" for a particular problem. Water resources engineers and scientists around the world are trying to develop the different kind of conjunctive use models based on purposes and objectives.

Following are some of the conjunctive use models.

- A simple groundwater balance model
- A GIS linked conjunctive use groundwater – surface water flow model (MODFLOW)
- Interaction of surface water and ground water modeling,
- Integrated Groundwater and Surface water Model (IGSM)
- Conjunctive use optimization model
- Linear optimization model
- Non-linear optimization models
- Multi objective conjunctive use models

Apart from the methods of development of conjunctive use models, there is lot of scope in conjunctive use modeling options. Here is some of the conjunctive use modeling options.

- Surface water and groundwater interaction model.
- Managing soil salinity through conjunctive use model
- Groundwater pumping through conjunctive use model.
- Irrigation water management in command area through conjunctive use model.
- Optimal crop planning and conjunctive use of surface water and groundwater.
- Crop scheduling, nutrients and agricultural water management through conjunctive use model.
- Surface water modeling and management
- Groundwater recharge estimation,

- Optimal allocation of surface water and groundwater in a basin.
- Climate change on surface water and groundwater through conjunctive use model., etc.

4. Conceptual conjunctive use model

The conceptual model of the surface water and groundwater was developed at catchment scale (after Sarwar, 1999) and shown in figure 1. The surface water model was developed based on simple water balance which accounts for input and outputs in the system causing change in storage. The water balance is based on law of conservation of mass. The objective of this model was to find the net groundwater recharge in the basin and this net recharge will be the input to the groundwater model. Hence, mathematically one can represent water balance in a basin as

$$I - O = \pm \Delta S_t \tag{1}$$

where I = total inflow, O = total outflow, ΔS_t = change in groundwater storage.

The conjunctive use surface water and groundwater model was developed based on the concept of hydrologic cycle. It consists of three sub-models viz. surface water model, groundwater model and optimization model. An attempt has been made to bring all the three models under one theme. The conceptual model of the present research is presented in figure 2. The surface hydrological processes follow the law of conservation of mass and are modeled using the water balance. This is identified as surface water model. Out of the infiltrated (net recharge) water into the soil, some percentage contributes to the base flow/subsurface flow and rest of it contributes to the aquifer recharge. The quantum of recharge depends mainly on geo-morphological, soil and hydro-geological parameters. The process of flow of water through the porous media is conceptualized as groundwater model.

Due to increased pressure on water resources (domestic, industrial and agricultural), the equilibrium of these two resources gets affected. So the use of surface water in conjunction with the groundwater may play a significant role in marinating the equilibrium and sustainability of the related system. The detailed descriptions of all the three models are given in the subsequent sections.

Over-exploitation of groundwater causes many problems like groundwater table depletion, water quality degradation and sea water intrusion in coastal areas. This is mainly because of shortage of surface water storage resources and the high investment required for storage. The solution for these challenging tasks may be sought through an optimization model. Usually a conjunctive use optimization model has socio-economic and hydraulic constraints. But in the present study, only hydraulic constraints like maximum allowable groundwater level and maximum stream flow utilization were taken into account to satisfy the demand (domestic and agricultural) leading to the optimal utilization of both surface water and groundwater. The three models represented in the conceptual model leads to a Decision Support System (DSS) where a suitable decision would be taken considering optimal utilization of water resources.

The model will help the decision makers, policy makers, practicing engineers and agricultural scientists to prepare the action plans for the overall development in the basin. The plausible policies and action plans should be sustainable water supply schemes for both

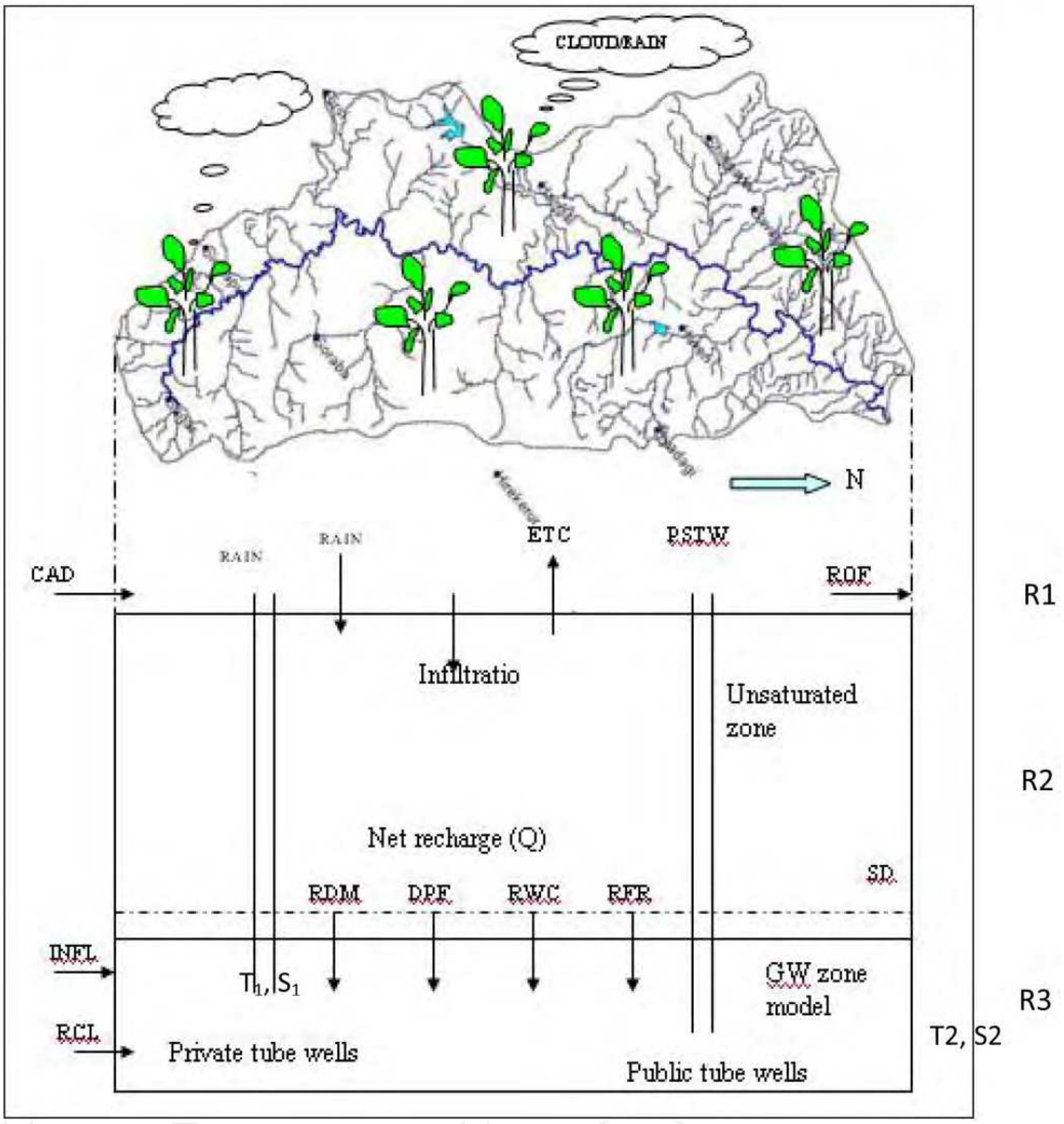

Fig. 1. Conceptual model of surface water & groundwater (modified after Sarwar, 1999).

domestic and agricultural sector in terms of groundwater pumping / surface water utilization to avoid over-exploitation and wastage of energy. It may also involve water resources development plans like construction of recharge structures to compensate for the groundwater level depletion, adoption of suitable cropping pattern and crop schedule to achieve better yield and economy with available water resources in the basin.

The net recharge to the groundwater will be computed by integrating the water balance elements considering R1 and R2 together. The flow in the saturated zone i.e. (groundwater reservoir R3) will be simulated using the groundwater model. The net recharge of a catchment area is then given by

$$\begin{aligned} Q = RFR + DPF + RDM + RWC + RCL + INFL - ROF - ET_a - EFL \\ - PSTW - PPTW - SD \end{aligned} \tag{2}$$

where

Q = Net recharge to the aquifer
RFR = Recharge from rainfall
DPF = Deep percolation from field
RDM=Recharge from distributory & minors
RWC = Recharge from water courses
RCL = Recharge from link canals
INFL = Inflow from adjacent area
ROF = Surface runoff
ET_a = Crop evapo-transpiration
EFL = Evaporation from fallow/ bare soil
PSTW = Pumpage by public tube wells
PPTW = Pumpage by private tube wells
CAD = Canal deliveries
SD = Seepage from water table to surface drains

It is assumed that, there is no interflow from adjacent areas into the catchment. Also, the basin is assumed to be geologically and hydrologically single system. The above equation is not applicable every where, suitable modification can be done to suit the interested area by considering all the above components or deleting some of the components.

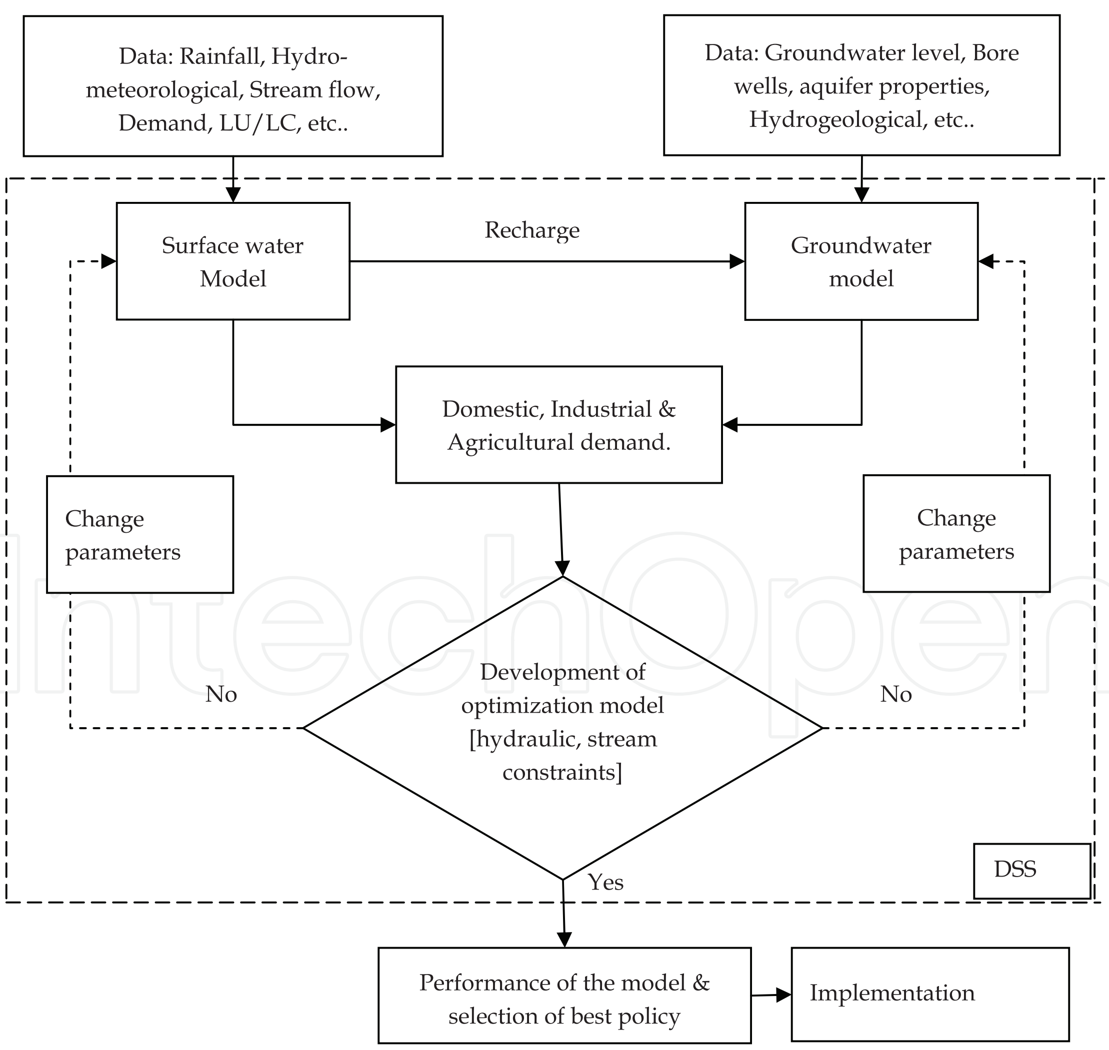

Fig. 2. Conceptual model of conjunctive use policy (Ramesh, 2007)

5. Conjunctive use model

5.1 Computations of water balance components

Water balance components will be compute using available standard models. The modeler can look in to suitable model and the inputs for such models should be supplied through field experiments. Some of models of water balance equation are given below.

5.1.1 Recharge from rainfall (RFR)

Many rainfall recharge models are available to estimate recharge from rainfall. One of the methods is regression based model to estimate recharge from rainfall.

5.1.2 Deep Percolation from field (DPF)

This component is to be estimated based on gross draft plus additional recharge of 5% (GEC, 1997). To estimate groundwater draft, an inventory of wells and a sample survey of groundwater draft from various types of wells (state tube wells, private tube wells and open wells) are required. For state tube wells, information about their number, running hours per day, discharge, and number of days of operation in a season is available in the concerned departments. To compute the draft from private tube wells, pumping sets and rates etc., sample surveys have to be conducted regarding their number, discharge and withdrawals over the season.

5.1.3 Recharge from distributaries (RDM)

It can be estimated separately for lined and unlined canals. Suitable loses can be used or estimated values form past studies can be used. As reported by the Indian Standards (IS 9452, 1980), the loss of water by seepage from unlined canals in India varies from 0.3 to 7.0 m^3/sec / million square meter of wetted area. It is calculated by the following relation:

$$\text{Losses in m}^3\text{ / sec / km} = \left\{\frac{C}{200}\right\} * (B+D)^{2/3} \tag{3}$$

where B=bed width, D=depth of water in meters, C=constant varies from 1 for intermittent to 0.75 for continuous.

As per GEC (1997) recommendations:

i. for unlined canals in normal soils
   - 1.8 to 2.5 m^3/sec / million square meters of wetted area

ii. unlined canals in sandy soils with some silt content
   - 3 to 3.5 m^3/sec / million square meters of wetted area.

5.1.4 Recharge from water courses (RWC)

Recommendations made by GEC, India (1997) are based on average water spread area. Recharge from storage tanks and ponds may be taken as 1.4 mm per day for the period in which tank has water. If the data on average water spread area is not available, then 60% of the maximum water spread area may be used. Recharge due to check dams and nala bunds may be taken as 50% of gross storage.

5.1.5 Surface runoff (ROF)

Direct runoff in a catchment depends on soil type, land cover and rainfall. Of the many methods available for estimating the runoff from rainfall, the curve number method (USDA-SCS, 1964) is the most popular. The curve number method makes use of soil categorization based on infiltration rates and land use i.e., the manner in which the soil surface is covered and its hydrologic conditions are important parameters influencing the runoff. The advantage of this method compared to other methods lies in the fact that the parameters used here are relatively easy to estimate. The final empirical equation given by USDA-SCS (1964) is as follows:

$$Q = \frac{(P - I_a)^2}{[S + (P - I_a)]} \tag{4}$$

where Q - actual runoff, P – rainfall, I_a- initial abstraction and S - Potential maximum retention after runoff begins which is expressed in terms of Curve Number (CN) given by the relation.

$$S = \frac{25400}{CN} - 254 \tag{5}$$

The parameter CN depends on a combinations of hydrologic soil, vegetation and land use complex (SVL) and antecedent moisture condition of a watershed. But this method has been modified by the Ministry of Agriculture, India (1972) to suite Indian conditions. The initial abstraction (I_a) is usually taken as equal to 0.2S for Indian conditions. Hence, equation (4) becomes

$$Q = \frac{(P - 0.2S)^2}{P - 0.8S} \tag{6}$$

The Curve Numbers for different SVL and AMC condition can be taken from Handbook of Hydrology (Ministry of Agriculture, India, 1972).

5.1.6 Crop Evapo-transpiration (ET_a)

This is the major loss in the water balance studies. It is the combined loss of water in the form of evaporation from soil surface / water and the transpiration from plant or vegetation. It can be calculated by the following equation as suggested by FAO (1956)

$$ET_a = K_C * ET_0 \tag{7}$$

Where ET_a = evapo-transpiration of specific crop (L/T)
ET_0 = potential / reference crop evapo-transpiration (L/T)
K_c= crop coefficient (dimensionless)

The reference crop evapo-transpiration is estimated according to Penman-Monteith (1980) equation.

$$ET_0 = \frac{0.408\Delta(Rn - G) + \gamma \frac{900}{Ta + 273} Uz(e_a - e_d)}{\Delta + \gamma(1 + 0.34U_2)} \tag{8}$$

Where ET_0 = reference evapo-transpiration [mm day^{-1}],
Rn = net radiation at crop surface [MJ m^{-2} day^{-1}],
Rnl = net outgoing long wave radiation [MJ m^{-2} day^{-1}],
Ra = net incoming shortwave radiation [MJ m^{-2} day^{-1}],
Ra = extra terrestrial radiation [s m^{-1}],
G = soil heat flux [MJ m^{-2} day^{-1}],
Ta = average air temperature in deg C,
U_2 = wind speed at 2 meter height [m s^{-1}],
e_a = saturation vapour pressure [kPa],
e_d = actual vapor pressure [kPa], e_s - e_a saturation vapour pressure deficit [kPa],
Δ = slope of the saturation vapor pressure [kPa °C^{-1}],
γ = psychrometric constant [kPa °C^{-1}].

5.1.6 Evaporation from fallow and barren soil (EFL)

It is estimated by making use of the following equation:

$$EFL = EPF * FSE(1 - XR) * CCA \tag{9}$$

where EFL = evaporation from fallow land (L^3/T),
EPF = equivalent evaporation factor,
FSE = free surface evaporation (pan evaporation), XR = the ratio of cropped to cultivable area, CCA = cultivable command area (L^2), EPF is calculated as

$$EPF = \left[\left(\frac{0.55}{(0.66 + WTD)}\right) + 0.009\right] \tag{10}$$

where WTD = depth to water table below soil surface.

5.1.7 Pumpage from tube wells (PTW)

Groundwater Pumpage from private and public tube wells is calculated by the following relation to account for the groundwater abstraction.

$$PTW = 0.083 * NPTW * UTF * AD * TOH \tag{11}$$

where NPTW = no. of private tube wells,
UTF = the utilization factor for each month,
AD = the actual discharge of private tube wells (m^3/sec),
TOH = total operational hours in a year (hrs), 0.083 = conversion factor

5.2 Development of numerical groundwater model

5.2.1 Model selection

Understanding the physics of groundwater flow and its interaction with surface water is a complex task. This is mainly because of the heterogeneity of the geo-hydrological formation, the complexity in the recharge and the boundary conditions of the aquifer system. Thus the role of numerical models has got utmost importance in the field of aquifer simulation. There are many numerical models available to simulate groundwater system. The numerical models are mainly based on finite difference (FD), finite element (FE), finite volume (FV) and finite boundary (FB) approaches. For many groundwater problems, the finite element method is superior to classical finite difference models (Willis and Yeh, 1987). Heterogeneities and irregular boundary conditions can be handled easily by the finite element method. This is in contrast to difference approximations that require complicated interpolation schemes to approximate the complex boundary conditions. Moreover, the size of element can be easily modified to reflect rapidly changing state variables or parameter values in the finite element method.

5.2.2 Governing equations

The groundwater flow modeling methodology given by American Society for Testing Materials (ASTM) presented in figure 3 was used in the present study.

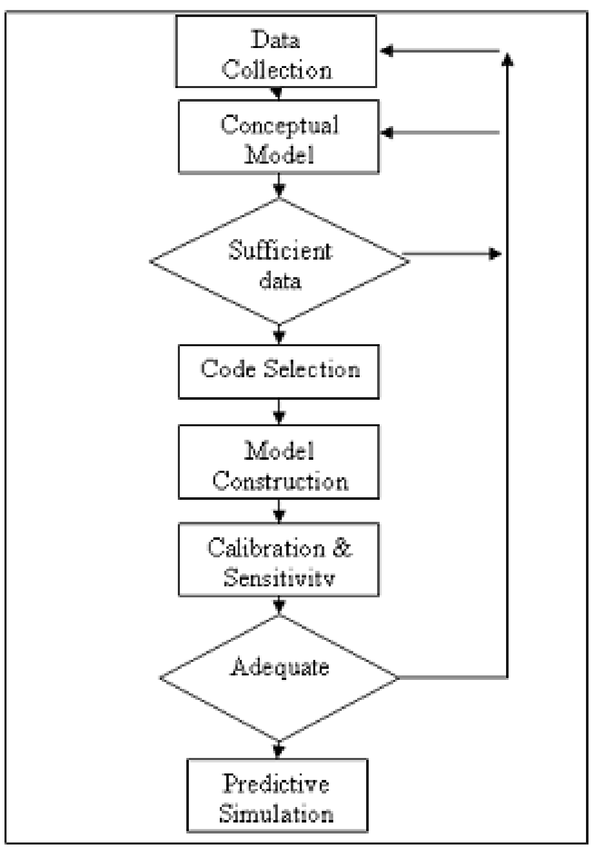

Fig. 3. Groundwater flow methodology (ASTM, D5447-2004)

The groundwater flow in an aquifer is represented by the following differential equations (Jacob, 1963),

For steady state condition:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right) \pm G(x, y) = 0 \tag{12}$$

For transient condition:

$$\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right)=S\frac{\partial h}{\partial t}\pm G(x,y,t) \tag{13}$$

where T_x and T_y are the x and y – direction transmissivities respectively (m^2/day);
h- Piezometric head (m); S – Storage coefficient (dimensionless);
G(x,y,t) – Pumping/Recharge (m^3/day); t – Time (days);
x & y – Coordinate axes.

5.2.3 Initial and boundary conditions

The initial groundwater level is provided as initial condition.

$$h(x_i,y_i,0)=h\ (x_i,y_i) \tag{14}$$

where, $h(x_i, y_i)$ is initial piezometric head.

The boundary condition is the combination of Dirichlet and Neumann conditions

$$h=\overline{h}(x,y,t) \quad \text{on } \Gamma_1 \tag{15}$$

and

$$T_x\frac{\partial h}{\partial x}l_x+T_y\frac{\partial h}{\partial y}l_y=q(x,y,t) \quad \text{on } \Gamma_2 \tag{16}$$

where $\overline{h}$ – specified piezometric head and l_x, l_y, l_z are the direction cosines between the normal to the boundary surface and the coordinate axes; Γ_1 represents those parts of the boundary where h is known and is therefore specified. q is prescribed for the remaining part of the boundary (Γ_2), which is the flow rate per unit area across of the boundary. For the general case of transient flow with phreatic surface moving with a velocity V_n normal to its instantaneous configuration, the quantity of flow entering its unit area is given by

$$q=V_nS+I*l_x \tag{17}$$

where S is the specific yield coefficient relating the total volume of material to the quantity of fluid which can be drained. I is the infiltration or evaporation.

The pumping or recharging well at a particular point in the domain is represented as:

$$Q_h^w(x_i,t)=\sum_m Q_m^w\prod\left\{\delta\left(x_i-x_i^m\right)\right\} \quad \text{for} \quad \forall(x_i-x_i^m)\in\Omega \tag{18}$$

where Q_h^w = a well function, Q_m^w = pumping or recharge rate of a single well (m^3/sec)

X_i^m = coordinate of well (m)

5.2.4 Finite element formulation

The finite element solution of equations (12 & 13) with initial and boundary conditions (14 - 16) is derived using Gelarkin's weighted residuals method. The Galerkin finite element method is a widely used technique for sub-surface flow simulations due to its efficiency and suitability (Pinder and Grey, 1977). The variable *h* is approximated as

$$h \approx \hat{h} = \sum_{i=1}^{n} N_i h_i \tag{19}$$

Over the domain; where N_i are the interpolation functions; h_i are the nodal values of *h*; n is the number of nodes.

The application of Galerkin method to the steady state equation yields following integral equation:

$$\int_{\Omega} R N_i d\Omega = 0\,; \qquad i = 1, 2, \ldots n \tag{20}$$

in which

$$R = \left[\frac{\partial}{\partial x}\left(T_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(T_y \frac{\partial h}{\partial y}\right)\right] \pm G(x,y) \tag{21}$$

where Ω refers to the area of flow domain

By applying Green's theorem, equation (20) can be modified to

$$\begin{aligned}\int_{\Omega}\left(\frac{\partial N_i}{\partial x}\sum_{1}^{n} T_x \frac{\partial N_j}{\partial x} + \frac{\partial N_i}{\partial y}\sum_{1}^{n} T_y \frac{\partial N_j}{\partial y}\right) h_i d\Omega - \\ -\int_{\Gamma} N_i \left(\sum_{1}^{n} T_x \frac{\partial h}{\partial x} l_x + \sum_{1}^{n} T_y \frac{\partial h}{\partial y} l_y\right) d\Gamma \pm \int_{\Omega} N_i G_i d\Omega = 0\end{aligned} \tag{22}$$

where Γ refers to external boundary. Equation (22) leads to a system of simultaneous equations which can be expressed as

$$[P]\{h\} = \{F\} \tag{23}$$

where [P] – conductivity matrix; {h}- vector of nodal values; {F} – load vector

$$P_{ij} = \sum \int_{E}\left(\frac{\partial N_i}{\partial x} T_x \frac{\partial N_j}{\partial x} + \frac{\partial N_i}{\partial y} T_y \frac{\partial N_j}{\partial y}\right) d\Omega \tag{24}$$

and

$$F_i = \sum \int_{\Gamma E} N_i q d\Gamma \pm \int_{E} N_i G_i d\Omega \tag{25}$$

where E denotes an element; ΓE refers to elements with an external boundary. The element equations are assembled into global system of equation. The prescribed boundary conditions are inserted at this stage and the solution is obtained using Gauss elimination routine.

5.2.5 Development of transient model

Rewriting the equation (13) describing linearized unsteady groundwater flow

$$T\left\{\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}\right\}-S\frac{\partial h}{\partial t}=\pm G(x,y,t) \tag{26}$$

To solve this equation, homogeneous and isotropic domain with boundary Γ in the time interval (0, t_n) is assumed. Both an essential and natural boundary conditions are imposed on the boundary.

$$h(x,y,t)=h_0(x,y,t) \text{ on } \Gamma_1 \tag{27}$$

$$T\left(\frac{\partial h}{\partial x}l_x+\frac{\partial h}{\partial y}l_y\right)+q_0=0 \text{ on } \Gamma_2 \tag{28}$$

where l_x and l_y are directional cosines of the outward normal to Γ. h_o- specified piezometric head; q_o- specified flux

The following initial condition is imposed on the domain Ω

$$h(x,y,0)=H(x,y) \text{ in } \Omega \tag{29}$$

where H – Initial piezometric head. Applying the Galerkin method to equation (26),

$$\int_\Omega N_i\left\{T\left[\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}\right]-S\frac{\partial h}{\partial t}\pm G(x,y,t)\right\}d\Omega=0 \qquad i=1,2,3\ldots.n \tag{30}$$

where n is number of nodes in the finite element mesh and N_i are the shape functions. Now applying Green's theorem yields

$$\int_\Omega T\left\{\frac{\partial N_i}{\partial x}\frac{\partial h}{\partial x}+\frac{\partial N_i}{\partial y}\frac{\partial h}{\partial y}\right\}d\Omega-\int_{\Gamma_2}N_i q_0 d\Gamma+\int_\Omega N_i S\frac{\partial h}{\partial t}d\Omega\pm\int_\Omega N_i G_i d\Omega=0 \tag{31}$$

The resulting system can be conveniently written in matrix form:

$$[P]\{h\}+[L]\left\{\frac{\partial h}{\partial t}\right\}=-\{F\} \tag{32}$$

where [P] - conductivity matrix; [L] - storativity matrix.

The elements of the matrices are given as

$$p_{ij}=\int_{\Omega^e}T\left\{\frac{\partial N_i}{\partial x}\frac{\partial N_j}{\partial x}+\frac{\partial N_i}{\partial y}\frac{\partial N_j}{\partial y}\right\}d\Omega \tag{33}$$

$$l_{ij} = \int_{\Omega^e} S N_i N_j d\Omega \tag{34}$$

$$f_i = \int N_i q d\Gamma \pm \int N_i G dA \tag{35}$$

For linear triangular element shown in figure 4, the interpolation function is given as

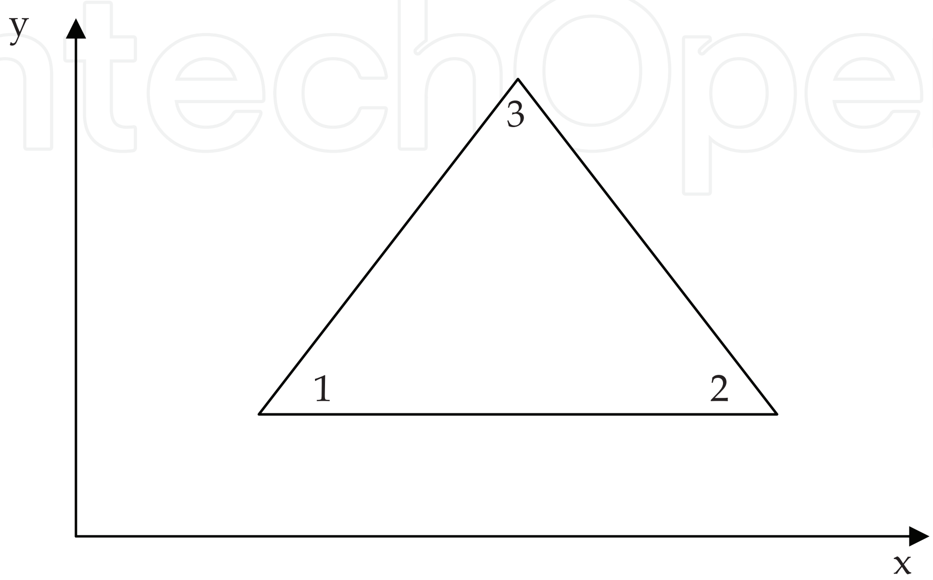

Fig. 4. Linear triangular finite element

$$N_i = \frac{\left(a_i + b_i x + c_i y\right)}{2A} \tag{36}$$

the element matrix is given below

$$\left[K^e\right]\left\{h^e\right\} =$$

$$= T \int_{\Omega^e} \left(\begin{Bmatrix} \dfrac{\partial N_1}{\partial x} \\ \dfrac{\partial N_2}{\partial x} \\ \dfrac{\partial N_3}{\partial x} \end{Bmatrix} \begin{Bmatrix} \dfrac{\partial N_1}{\partial x} & \dfrac{\partial N_2}{\partial x} & \dfrac{\partial N_3}{\partial x} \end{Bmatrix} + \begin{Bmatrix} \dfrac{\partial N_1}{\partial y} \\ \dfrac{\partial N_2}{\partial y} \\ \dfrac{\partial N_3}{\partial y} \end{Bmatrix} \begin{Bmatrix} \dfrac{\partial N_1}{\partial y} & \dfrac{\partial N_2}{\partial y} & \dfrac{\partial N_3}{\partial y} \end{Bmatrix} \right) d\Omega \left\{h^e\right\} \tag{37}$$

where Ω^e is the element domain

performing integration after substituting the shape functions we get

$$\left[K^e\right] = T \begin{bmatrix} k_{11} & k_{12} & k_{13} \\ k_{21} & k_{22} & k_{23} \\ k_{31} & k_{32} & k_{33} \end{bmatrix} \tag{38}$$

in which

$$k_{11} = \frac{1}{4A}\left[(x_3 - x_2)^2 + (y_2 - y_3)^2\right] \tag{39}$$

$$k_{12} = \frac{1}{4A}\left[(x_3 - x_2)(x_1 - x_3) + (y_2 - y_3)(y_3 - y_1)\right] \tag{40}$$

$$k_{13} = \frac{1}{4A}\left[(x_3 - x_2)(x_2 - x_1) + (y_2 - y_3)(y_1 - y_2)\right] \tag{41}$$

$$k_{21} = k_{12} \tag{42}$$

$$k_{22} = \frac{1}{4A}\left[(x_1 - x_3)^2 + (y_3 - y_1)^2\right] \tag{43}$$

$$k_{23} = \frac{1}{4A}\left[(x_1 - x_3)(x_2 - x_1) + (y_3 - y_1)(y_1 - y_2)\right] \tag{44}$$

$$k_{31} = k_{13} \tag{45}$$

$$k_{32} = k_{23} \tag{46}$$

$$k_{33} = \frac{1}{4A}\left[(x_2 - x_1)^2 + (y_1 - y_2)^2\right] \tag{47}$$

where A is the area of triangle.

Considering forward difference scheme for the time derivative term in the equation (32)

$$\frac{\partial h}{\partial t} = \frac{h_{t+1} - h_t}{\Delta t} \tag{48}$$

Also, considering the system of equations marching with time, a time stepping scheme is introduced with a factor θ. The solution accuracy and the numerical stability depends the choice of values of θ, is of decisive significance. Most frequently 1, ½, or 0 are substituted for θ. Equation (32) thus obtains the form:

Case (i): θ =1, Backward scheme;

$$(L + P_{t+1}\Delta t)h_{t+1} = Lh_t - F_{t+1}\Delta t \tag{49}$$

Case (ii): θ =1/2, central (Crank-Nicolson) scheme

$$\left(L + \frac{1}{2}P_{t+1}\Delta t\right)h_{t+1} = \left(L - \frac{1}{2}P_t\Delta t\right)h_t - \frac{1}{2}(F_t + F_{t+1})\Delta t \tag{50}$$

Case (iii): θ =0, forward scheme;

$$Lh_{t+1} = (L - P_t\Delta t)h_t - F_t\Delta t \quad (51)$$

In the present study, an implicit scheme with θ = 1/2, (Crank-Nicholson scheme) was adopted. The model was operated on a monthly basis to suit the availability of data. A computer code was developed in Visual C++ for the entire process and programme is given in appendix II. The results are presented in GIS platform (ESRI, 2004) for better visualization.

5.2.6 Model calibration

In the present study, trial and error calibration (figure 5) procedure is adopted. Initially, the aquifer parameters such as transmissivity (T) and storativity (S) are assigned based on the field test results. The simulated and measured values of piezometric heads were compared by adjusting the model parameters to improve the fit. The recharge components were varied within the range presented in table.

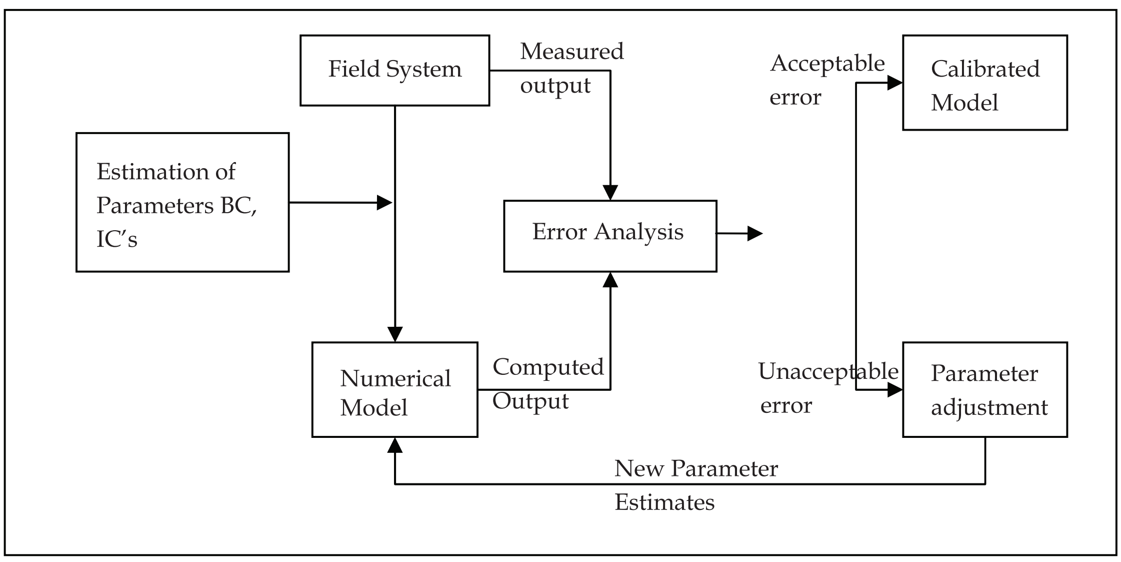

Fig. 5. Trial and error calibration procedures (Anderson and Woessner, 1992)

The following input parameters have received particular attention during the calibration.

- Specific yield/storage coefficient, transmissivity of aquifer.
- Factor for recharge from distributaries and minors
- Factor for recharge from watercourse
- Factor for discharge to surface drains
- Factor for recharge from rainfall
- Factor for evaporation from watercourse surfaces and bank vegetation

For the second and third parameters, an empirical equation has been used to compute recharge to groundwater depending on the depth of rainfall as discussed earlier. For the rest of parameters, following generalities of transient calibration (Boonstra and Ridder, 1990) were followed.

- First, change the input parameters for those areas where the largest deviation occurs

- Change one type of input parameter in each run
- Determine whether any change of input parameter in one area will have positive or negative effect in other areas.

5.2.7 Convergence criteria

A modeler must decide what levels of accuracy are appropriate for comparative assessment of alternatives. A generalized model with limited accuracy doesn't provide the required level of confidence in the selection of a water management strategy, while beyond certain limits that is required to provide a rational basis for comparing alternatives is wasteful. This can be achieved by imposing convergence criterion and tolerance limits in the model to stop the number of iterations. The following convergence criterion is used in the present study.

$$\frac{\sqrt{\sum_{j=n}^{n} h_i^2} - \sqrt{\sum_{j=1}^{n} h_{i-1}^2}}{\sqrt{\sum_{j=1}^{n} h_i^2}} \le \varepsilon \tag{52}$$

where i = iteration index, j = no. of nodes, ε = tolerance limit (0.001).

The attainment of steady sate is also monitored using the above relationship with 'i' representing the time level.

5.2.8 Model validation

The objective of model performance analysis is to quantify how well the model simulates the physical system and to identify the problem if any, in the model. The method typically used to quantify model error is to compute the difference between predicted and observed values of piezometric heads (Residual) at the measuring location. The scatter diagrams, together with computed coefficient of determination indicate where the greatest discrepancies occur and whether there are few major discrepancies or general disagreement between predictions and observations (Karlheinz and Moreno, 1996).

The performance of the calibrated model could be quantified by a number of statistics comparing the observed and simulated hydraulic heads (ASTM, 1993). Following measures of the goodness of fit between measured and simulated water levels (Sarwar, 1999) were calculated in this study.

Mean Error (ME)

$$ME = \frac{1}{N}\sum_{i=1}^{N}(P_i - O_i) \tag{53}$$

Root Mean Square Error (RMSE)

$$RMSE = \left[\sum_{i=1}^{N}\frac{(P_i - O_i)^2}{N}\right]^{\frac{1}{2}}\left[\frac{100}{\overline{O}}\right] \tag{54}$$

where P is simulated value, O is the observed value, $\bar{O}$ is the mean observed value and N is the number of observations.

The error parameters, generally used for evaluating the calibration quality (Frey Berg, 1988; Anderson & Woessner, 1992, Madan et al., 1996) are to be tabulated.

5.3 Development of conjunctive use optimization model

Management optimization is a powerful technique for computing optimal solutions for challenging management problems, such as maximizing quantity of water or minimizing operating costs. The management problem is mathematically formulated to represent the desired objectives of the decision maker (e.g., minimize costs), as well as the associated constraints (e.g., required water supply rate). Algorithms compute the optimal solution (e.g., pumping rates of individual wells) and quantify its sensitivity to various problem components (e.g., cost coefficients, constraint limits, etc.). Surface water and groundwater systems are often intimately connected. Industrial, commercial, and agricultural land uses affect aquifer recharge and discharge, which in turn impact spring discharge to, and seepage from, surface water bodies. Irrigated agriculture is a significant component of river and aquifer water budgets in many areas of the world. Surface water applied in excess of crop consumptive requirements enters the groundwater system increasing aquifer water levels and spring discharge. Groundwater pumping for irrigation or other consumptive uses creates the opposite effect. The Snake river in southern Idaho is a prime example of a surface water system that is greatly affected by groundwater conditions which changes in response to irrigation practices (Miller et al, 2003). Integrated river basin modeling with distributed groundwater simulation and dynamic stream-aquifer interaction allows a more realistic representation of conjunctive use and the associated economic results (Velázquez et al, 2006).

In the present study, optimization problem was formulated as a linear programming problem with the objective of maximizing water production from wells and from streams given by John et al, (2003) with a little modification. The objective function has the following constraints:

1. Maintaining groundwater level at or above specified level.
2. Utilization of stream flow at or below maximum specified rates.
3. Limiting the maximum increase in groundwater withdrawals.

5.3.1 Water demand

The total water demand in the basin is considered to be of domestic, agricultural and industrial sectors. The water demand will be projected over next two decades based on past decadal census data.

Domestic Water Demand

Domestic water demand is the total quantity of water that is being used for drinking, cooking, washing, cleaning etc. therefore it is mainly depending on the number of population. The domestic water demand will be calculated by population forecast based on arithmetic progression, geometric progression, incremental increase and national average.

However, specific assessment of growth potential shall be taken into consideration while arriving at the final population forecast.

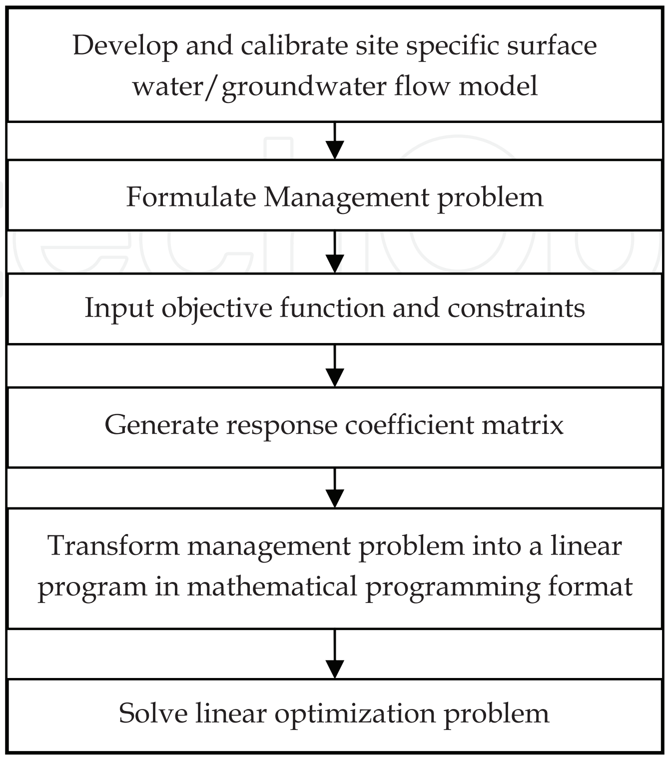

Fig. 6. Flow chart of optimization modeling process (Czarnecki, 2003)

Arithmetic progression: This method is based on the assumption that population increases at constant rate. A constant increment growth is added periodically based on the past records. This method generally gives a low rate of population growth and can be used where growths are not conspicuous.

Population forecast for $P_n = P_{2001} + (X) * n$ (55)

where X= Average population increase / decade
n = No. of decades

Geometrical progression method: In this method, percentage increase or percentage growth rate per decade is assumed to be constant, and the percentage increase is compounded over existing population every decade. This method normally predicts greater values of population and is used for the areas with scope for huge expansion plans.

Population forecast for $P_n = P_{2001} + (1 + M / 100)^n$ (56)

where M = Average percentage increase in population

Incremental increase method: In this method, the average incremental increase is calculated from the available data. To the present population, the average incremental increase per decade is added and the population of next decade is obtained. Like this, the process is repeated till the population in the desired decade is reached.

Population forecast for $P_n = P_{2001} + (X + Z) * n$ (57)

where X = Average population increase / decade = Total increase/ No. of decades
Y = Net incremental increase; Z = Average incremental increase; n = No. of decades.

Final decision on the estimation of domestic water demand shall be based on the realistic projection for the project period using the methods described above. However, the decade growth rate must be limited to 20 percent (National average growth, India), when the projected population growth is more than 20 percent per decade. However, under exceptional circumstances where the growth rate beyond 20 percent, it should be substantiated by data.

5.3.2 Objective function

The objective of the present optimization model is to maximize the water production from both groundwater and the surface water resources. The objective function has the following form:

$$Maximize \cdots Z = \sum q_{well} + \sum q_{river} \quad (58)$$

where Z – is the total managed water withdrawal in Mm^3/day
Σq_{well} – is the sum of groundwater withdrawal rates in Mm^3/day
Σq_{river} – is the sum of surface water withdrawal rates from all managed river reaches in Mm^3/d.

The following constraints are formulated to solve the objective function.

i. Hydraulic head constraints

This is the constraint imposed based on the groundwater level fluctuation in an aquifer. For achieving sustainability, a critical groundwater level is to be worked out by analyzing. The following hydraulic constraint is to be satisfied for sustainable groundwater management.

$$h_c \leq h_{\max imum} \quad (59)$$

where h_c is the hydraulic head (water level) at the given location c, in meter.

$h_{maximum}$ is the groundwater level altitude at half the thickness of the aquifer in meter. There is a flexibility of fixing h_c in the model based on hydrogeology and groundwater levels fluctuation. The above equation allows an aquifer to drain up to critical hydraulic head (h_c).

ii. Stream flow constraints

stream flow constraint as the maximum utilization of the stream flow within the basin. The stream flow constraint was derived based on simple mass balance equation as follows:

$$q_{head} + \sum q_{overland} \pm \sum q_{groundwater} - \sum q_{diversion} - \sum q_{river} \leq q_{\max imum} \quad (60)$$

where q_{head} is the flow rate into the head of stream in m^3/d
$\Sigma q_{overland}$ is the sum of all overland and tributary flow into stream reach in m^3/d
$\Sigma q_{groundwater}$ is the net sum of all groundwater flow to or from stream reach *R*, in m^3/d

$\Sigma q_{diversion}$ is the sum of all surface water diversions from stream reach in m^3/d
Σq_{river} is the sum of all potential withdrawal excluding diversions from stream in m^3/d
$q_{maximum}$ is the minimum permissible surface water flow rate for stream in m^3/d

But the data on river head and groundwater are not available in the study area. The selection of maximum stream flow rate ($q_{maximum}$) depends on the downstream requirement. Therefore the above constraint reduces to the following form:

$$\sum q_{overland} - \sum q_{diverion} - \sum q_{river} \leq q_{\max imum} \tag{61}$$

iii. Groundwater pumping limits

If no limits are imposed on the potential amount of water that can be pumped at each managed well, then those wells nearest to the sources of water, such as rivers or general head boundaries will be the first to be supplied water, thus capturing flow that would otherwise reach wells farther from the sources.

$$0 \leq \sum \ q_{well} \ \leq mq_{well(year)} \tag{62}$$

where, $\sum q_{\ wells}$ is the optimal groundwater withdrawal, Mm^3/d
m is a multiplier to account for annual increase in pumping rate
$q_{\ well\ (year)}$ is the total amount withdrawn in the particular year from the wells in Mm^3/d.

iv. Surface water withdrawal limits

No limits are imposed on optimized withdrawal from river such that the range in optimal withdrawal was between zero and maximum amount of water available at a given point in a river. This specification permitted the analysis where water could be withdrawal and the maximum quantity available. Withdrawals will be allowed only at one point where river constraint is specified i.e. at measuring point.

5.4 Case Study

A humid, tropical river basin is chosen for the application of conjunctive use model. The Varada river basin of southern India lies between latitude 14° to 15° 15′ N and longitude 74° 45′ to 75° 45′ E (Fig. 7). The river originates at an altitude of 610 m above the mean sea level (MSL) in the western ghats (mountainous forest range parallel to west coast) and drains an area of about 5020 Km^2. The river flows towards north-east for about 220 Km and joins the river Tungabhadra. Physiographically, Varada basin consists of western ghats on the west and a plateau region in the east. Sirsi, Siddapur, Soraba, Sagar, and part of Hanagal taluks are covered by the western ghat region and form a dense tropical forest zone. The remaining area falls under the plateau region. The average annual rainfall in the western ghat and the plateau regions are 2070mm and 775mm respectively. The rainfall is mainly confined to June to November and the rest of the year is usually a dry season.

5.4.1 Model calibration

Steady state calibration

The basin is discretized into 329 linear triangular elements with 196 nodes (fig.8). The aquifer condition of January 1993 was used as initial condition for the steady state model calibration. A number of trail runs were made by varying both transmissivity and storativity values of the aquifers so that root mean square (RMS) error was kept below 0.5m. The simulated (computed) versus observed heads for selected observation points (wells), are shown in figure 9. The figure indicates a good agreement between the simulated and observed water levels. This was also found to true for other observation wells.

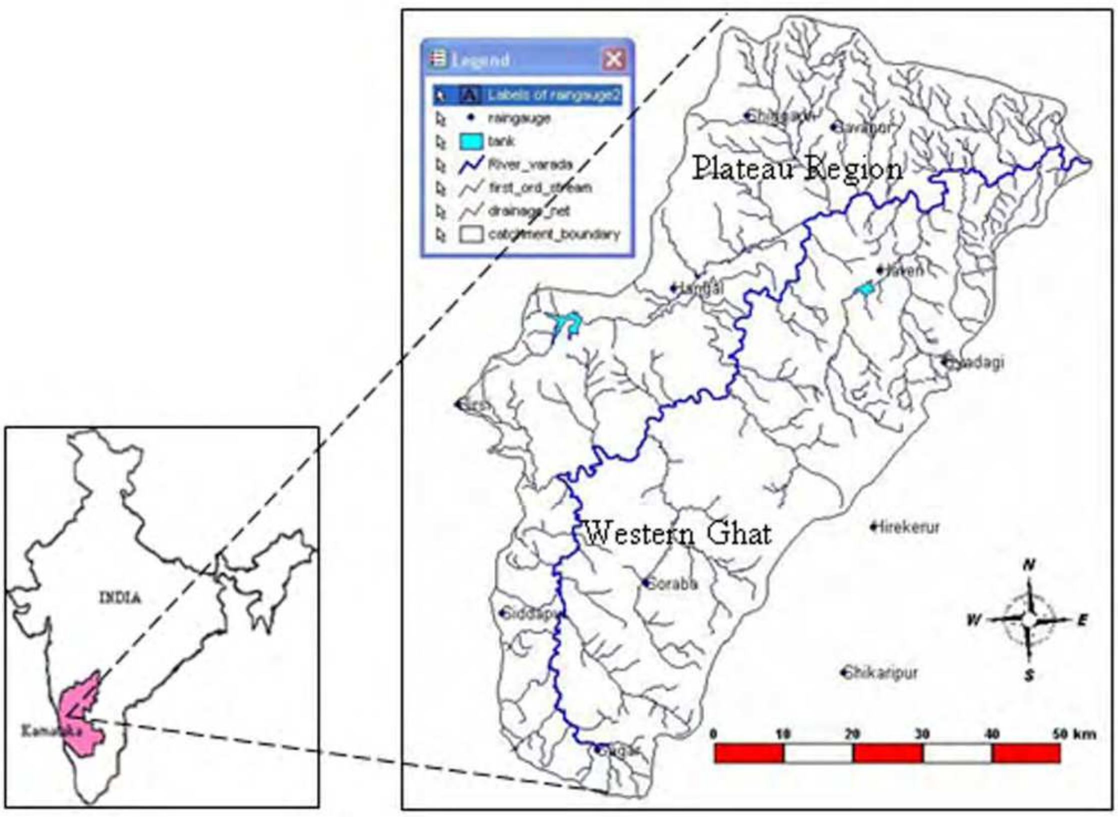

Fig. 7. Study area- Varada Catchment (Ramesh and Mahesha, 2008)

Error Measures	Well Location (nodes)						
	6	14	43	105	139	175	185
ME	-0.08	-0.31	0.47	0.55	-0.21	-0.43	-0.08
RMSE	0.65	0.46	0.73	0.78	0.56	0.76	0.69
R^2	0.83	0.89	0.89	0.89	0.91	0.78	0.86

Table 1. Goodness of fit statistics for comparison between observed and simulated heads (Ramesh and Mahesha, 2008).

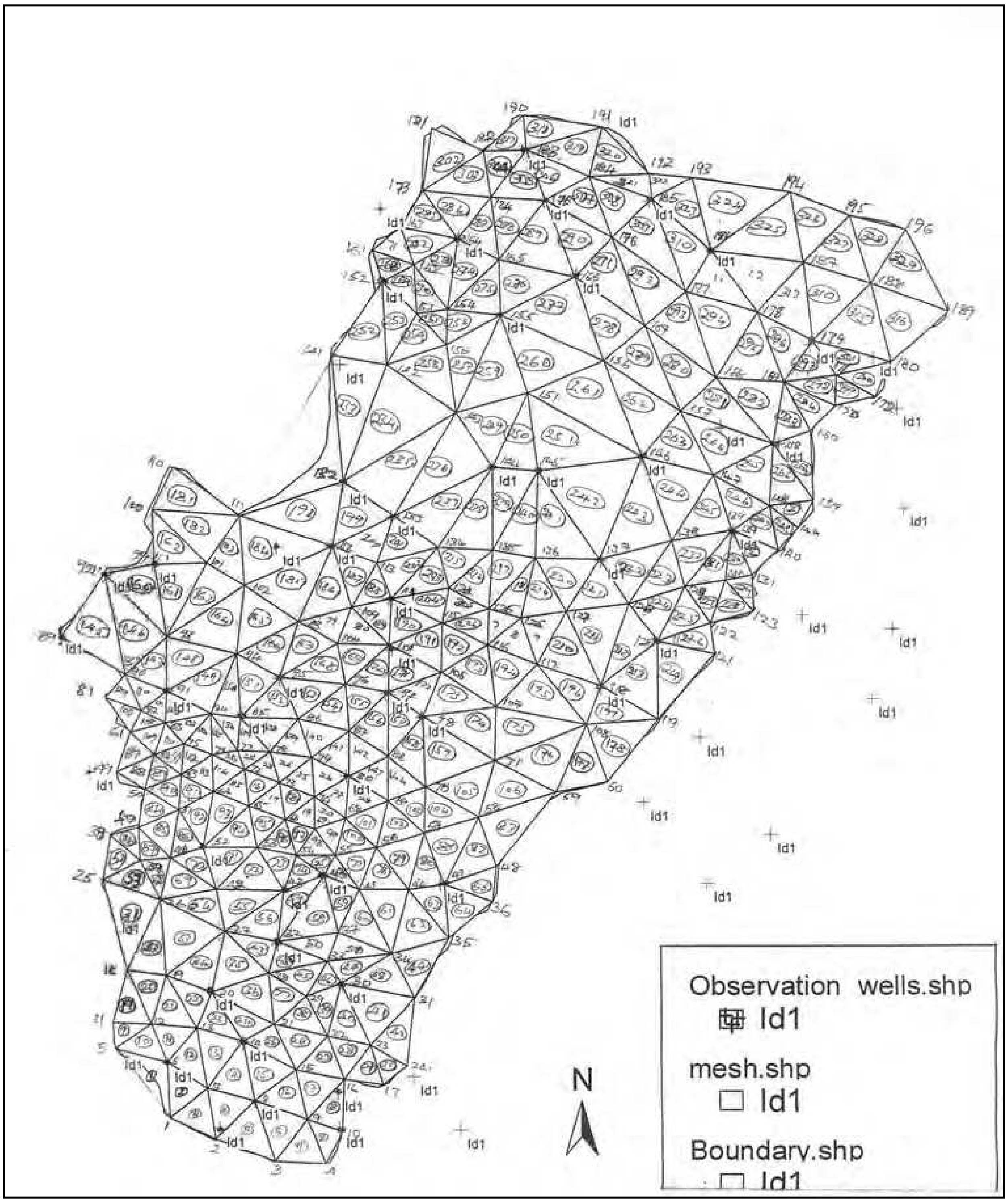

Fig. 8. Finite element mesh for Varada river basin (mod. after Ramesh & Mahesha, 2008)

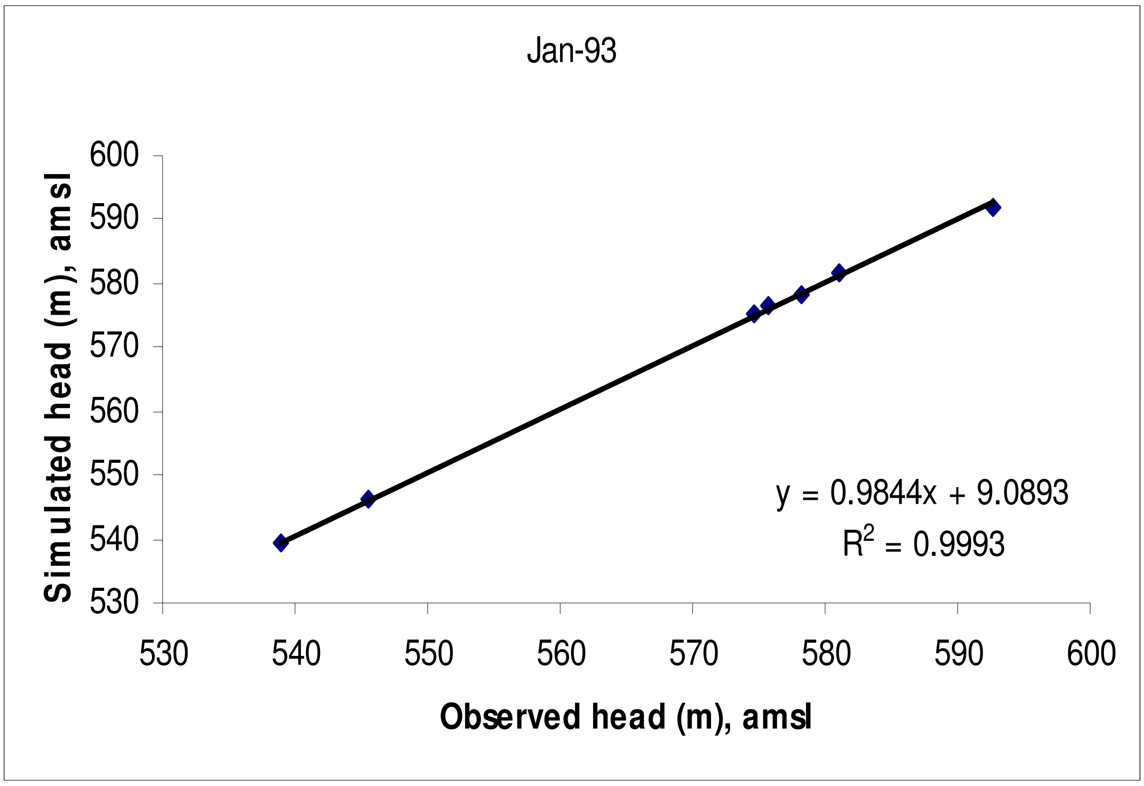

Fig. 9. Observed and simulated groundwater levels during calibration

Transient calibration

The transient calibration was carried out for the period January 1993 to December 1998. The hydraulic conductivity values, boundary conditions and the water levels, arrived through the steady state model calibration were then used as the initial condition in the transient model calibration. These were used along with the storage coefficient distribution and time variable recharge and pumping distribution. A numbers of trial runs were made by varying the storage coefficient (S) values within the observed range so that a reasonably good match was obtained between computed and observed water levels. The transmissivity values are already arrived at during the steady state calibration. Forty seven observation wells were selected as the fitting wells after consideration of their data availability and distribution in the region. The calibrated storage coefficient values for the western ghat zone and plain area zone were found to be 0.0025 and 0.0063 respectively.

The computed well hydrographs for these boreholes show a fairly good agreement with the field values. The disagreement observed in some observation wells (OW-3) is generally attributed to differences in the initial head conditions arrived through steady state calibration, variation in pumping pattern and insufficient bore well data. Nearly 10% of the total bore wells are unauthorized and hence pumping rate and pattern differs from the official data.

5.5 Simulation of predicted scenarios (Groundwater levels)

The calibrated model is applied to predict the basin response over the short term i.e. 2004 – 2010 under various aquifer stress scenarios. The total water demand in the basin was

Year	RFR	RWC	RLC	INFL	ROF+BF	ETc	PTW	Net Recharge (R)	Recharge /Rainfall	% increase PTW	Classification
1993	1200.59	61.28	30.64	2.45	367.54	596.70	98.49	232.23	0.19	0.00	-----
1994	1443.55	75.73	37.86	3.03	364.22	596.70	123.51	475.74	0.33	25.40	Wet year
1995	968.67	48.47	24.24	1.94	263.91	596.70	134.61	48.10	0.05	36.67	-----
1996	1030.93	53.79	26.89	2.15	210.25	596.70	161.74	145.07	0.14	64.22	-----
1997	1487.62	74.68	37.34	2.99	242.84	596.70	183.60	579.49	0.39	86.41	-----
1998	1293.58	64.73	32.37	2.59	186.67	596.70	198.87	411.03	0.32	101.92	-----
1999	1270.48	66.60	33.30	2.66	241.17	596.70	188.25	346.92	0.27	91.14	-----
2000	1232.28	62.04	31.02	2.48	219.11	596.70	181.08	330.92	0.27	83.86	-----
2001	713.05	36.32	18.16	1.45	80.61	596.70	164.54	-72.87	-0.10	67.06	Dry Year
2002	936.65	46.83	23.42	1.87	96.71	596.70	203.13	112.23	0.12	106.24	-----
2003	902.14	45.41	22.70	1.82	85.76	596.70	207.05	82.55	0.09	110.22	-----
Average	1134.50						167.72	244.26	0.19	70.29	Average

Table 2. Water balance components for the period 1993-2003 (in mm)

		Jan	Mar	Feb	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1993	Q	0.000	0.546	0.000	0.682	1.692	5.985	9.821	6.867	1.358	9.771	1.463	1.887
	P	1.615	2.936	2.479	3.302	3.734	5.798	7.266	5.078	2.683	9.626	3.467	2.112
1994	Q	0.000	0.081	0.000	2.248	1.081	10.247	19.046	6.491	3.333	6.619	0.302	0.000
	P	1.990	1.719	1.734	1.749	2.698	8.023	15.430	4.992	2.628	6.431	0.776	0.980
1995	Q	0.066	0.000	0.000	1.181	1.015	2.319	12.049	5.335	4.340	4.160	1.210	0.000
	P	2.035	2.035	1.779	2.065	2.902	2.050	12.170	4.358	2.326	5.996	0.962	1.133
1996	Q	0.000	0.000	0.022	1.386	1.109	7.439	9.811	5.669	3.773	4.982	0.297	0.859
	P	2.020	2.065	2.045	2.115	2.574	5.770	12.383	4.279	2.325	5.975	1.031	1.233
1997	Q	0.083	1.768	0.000	1.170	1.039	9.822	13.038	12.507	0.765	4.708	2.689	1.215
	P	2.389	2.136	2.494	2.414	2.079	7.511	11.686	10.470	2.263	5.980	1.165	1.318
1998	Q	0.000	0.004	0.000	0.067	1.733	9.177	10.723	6.821	7.565	4.907	1.413	0.094
	P	2.269	2.404	2.429	2.014	2.113	7.166	10.786	7.809	5.319	5.760	1.182	1.207
1999	Q	0.000	0.018	0.024	0.615	3.657	8.144	17.128	4.089	2.173	7.585	0.054	0.000
	P	0.140	0.601	0.632	0.803	3.165	7.551	17.151	4.375	2.251	7.197	0.380	0.179
2000	Q	0.295	0.000	0.021	0.788	1.283	7.145	11.565	8.680	6.455	4.315	0.069	0.035
	P	0.123	0.573	0.602	0.813	0.961	6.664	10.967	8.755	5.565	3.489	0.410	0.212
2001	Q	3.420	0.000	0.004	1.676	0.718	5.541	5.382	5.264	3.253	1.645	0.229	0.152
	P	0.116	0.582	0.636	1.303	0.778	5.385	5.301	5.029	2.782	1.337	0.395	0.186
2002	Q	0.000	0.651	0.020	1.467	1.891	7.288	4.570	8.234	1.295	5.236	0.054	0.028
	P	0.137	0.711	0.649	1.187	0.952	6.611	4.779	8.097	1.179	4.975	0.405	0.199

Table 3. Recharge (Q) & extraction (P) rates in study area (in Mm^3/d)

Node No.	Village	Jan-02	Scenario-2				Scenario-3			
			5 % increase in pumping				10% increase in pumping			
			Jan-07	Jan-08	Jan-09	Jan-10	Jan-07	Jan-08	Jan-09	Jan-10
2	Yadagigalemane	608.61	606.80	605.08	603.44	601.88	604.99	601.72	598.53	596.66
5	Talaguppa	578.73	578.64	578.54	578.44	578.33	578.55	578.34	576.55	571.25
6	Alahalli	572.79	572.52	572.23	571.94	571.62	572.25	571.65	569.69	566.04
10	Ullur	628.20	625.13	622.22	619.46	616.83	622.07	616.56	612.05	609.25
14	Keladi	577.70	577.58	577.47	577.36	577.25	577.46	577.24	576.32	574.49
16	Bommatti	609.13	606.85	604.69	602.63	600.66	604.58	600.46	598.26	596.53
32	Hosabale	592.09	591.68	591.29	590.92	590.57	591.27	590.54	589.49	586.25
43	Yalsi	574.99	574.72	574.45	574.18	573.91	574.44	573.91	571.91	565.72
49	Tyagali	546.10	542.66	539.05	535.26	531.28	539.23	531.66	499.33	436.29
80	Kuppagadde	565.75	564.84	563.98	563.16	562.36	563.94	562.29	562.72	559.22
92	Isloor	626.63	626.70	626.76	626.84	626.91	626.76	626.90	624.86	613.35
95	Jade	553.35	552.33	551.25	550.12	548.94	551.30	549.05	537.38	511.32
97	Anavatti	542.34	542.26	542.17	542.08	541.98	542.17	541.99	538.42	525.95
105	Agasanahalli	540.05	539.92	539.78	539.64	539.48	539.79	539.50	538.00	529.98
114	Makaravalli	549.35	548.97	548.56	548.14	547.69	548.58	547.73	541.81	525.14
139	Motebennur	576.45	575.90	575.38	574.89	574.43	575.35	574.37	574.09	572.63
144	Adur	545.56	545.19	544.81	544.41	543.99	544.83	544.03	538.72	523.45
179	Haleritti	517.91	517.72	517.51	517.30	517.07	517.52	517.09	512.97	502.62
180	Negalur	513.60	513.47	513.34	513.21	513.07	513.35	513.08	513.03	512.30
185	Yalavigi	596.41	596.36	596.32	596.27	596.22	596.32	596.22	594.51	588.74
189	Outlet	501.08	500.79	500.49	500.17	499.84	500.50	499.87	495.02	481.41

Table 4. Predicted groundwater levels (in m with respect to mean sea level) for the scenarios –2 & 3 (January)

Node No.	Village	May-02	Scenario-2				Scenario-3			
			5 % increase in pumping				10% increase in pumping			
			May-07	May-08	May-09	May-10	May-07	May-08	May-09	May-10
2	Yadagigalemane	607.54	606.41	605.32	604.28	603.28	605.27	603.20	601.29	599.52
5	Talaguppa	578.86	577.96	577.01	576.02	574.97	577.06	575.07	572.89	570.48
6	Alahalli	570.34	569.82	569.28	568.70	568.10	569.30	568.16	566.90	565.52
10	Ullur	626.53	625.26	624.06	622.93	621.86	624.00	621.74	619.72	617.93
14	Keladi	576.55	576.28	575.99	575.69	575.37	576.01	575.40	574.73	574.00
16	Bommatti	569.25	568.59	567.91	567.20	566.46	567.94	566.53	564.98	563.29
32	Hosabale	589.05	588.77	588.50	588.24	587.99	588.48	587.97	587.50	587.08
43	Yalsi	573.48	573.38	573.27	573.19	573.10	573.28	573.10	572.92	572.76
49	Tyagali	604.61	601.82	598.88	595.80	592.56	599.02	592.87	586.10	578.65
80	Kuppagadde	565.23	564.21	563.23	562.31	561.42	563.18	561.34	559.66	558.14
92	Isloor	626.05	625.90	625.74	625.57	625.40	625.75	625.41	625.04	624.63
95	Jade	552.35	551.12	549.83	548.48	547.06	549.90	547.20	544.23	540.96
97	Anavatti	542.25	542.21	542.17	542.12	542.07	542.17	542.08	541.97	541.85
105	Agasanahalli	540.31	539.97	539.60	539.22	538.82	539.62	538.86	538.02	537.10
114	Makaravalli	547.56	546.76	545.91	545.03	544.10	545.95	544.19	542.24	540.10
139	Motebennur	569.74	569.74	569.74	569.74	569.74	569.74	569.74	569.73	569.73
144	Adur	557.96	557.89	557.82	557.75	557.67	557.83	557.68	557.51	557.32
179	Haleritti	522.46	521.87	521.26	520.61	519.93	521.29	520.00	518.58	517.02
180	Negalur	517.20	516.43	515.62	514.77	513.88	515.66	513.96	512.10	510.04
185	Yalavigi	590.10	589.98	589.85	589.72	589.57	589.86	589.59	589.29	588.96
189	Out let	509.01	508.18	507.32	506.41	505.46	507.36	505.55	503.57	501.38

Table 5. Predicted groundwater levels (in m with respect to mean sea level) for the scenarios – 2 & 3 (May)

Node No.	Village	Sep-02	Scenario-2				Scenario-3			
			5% increase in pumping				10% increase in pumping			
			Sep-07	Sep-08	Sep-09	Sep-10	Sep-07	Sep-08	Sep-09	Sep-10
2	Yadagigalemane	611.12	610.26	609.43	608.63	607.87	609.39	607.80	606.35	605.00
5	Talaguppa	585.43	584.26	583.03	581.74	580.38	583.09	580.51	577.68	574.56
6	Alahalli	578.13	577.52	576.89	576.21	575.51	576.92	575.58	574.11	572.49
10	Ullur	629.67	628.24	626.89	625.60	624.35	626.84	624.26	621.86	619.62
14	Keladi	578.67	578.43	578.08	577.71	577.32	578.09	577.36	576.55	575.66
16	Bommatti	580.15	579.96	579.44	578.90	578.34	579.46	578.39	577.22	575.94
32	Hosabale	594.20	596.49	596.21	595.94	595.68	596.20	595.66	595.15	594.66
43	Yalsi	577.32	577.30	576.93	576.55	576.15	576.95	576.18	575.34	574.42
49	Tyagali	644.03	640.89	637.59	634.12	630.49	637.74	630.83	623.23	614.87
80	Kuppagadde	565.36	567.45	567.15	566.86	566.57	567.15	566.56	565.99	565.44
92	Isloor	633.22	632.65	632.06	631.43	630.78	632.09	630.84	629.48	627.97
95	Jade	570.13	571.16	570.31	569.43	568.50	570.35	568.59	566.65	564.52
97	Anavatti	543.50	545.05	544.55	544.02	543.46	544.57	543.51	542.34	541.06
105	Agasanahalli	543.72	544.39	544.07	543.74	543.39	544.09	543.43	542.70	541.90
114	Makaravalli	549.83	550.13	549.31	548.45	547.55	549.35	547.64	545.76	543.69
139	Motebennur	570.77	571.59	571.49	571.39	571.29	571.48	571.28	571.10	570.94
144	Adur	561.02	561.42	561.37	561.32	561.26	561.37	561.27	561.15	561.03
179	Haleritti	522.39	522.07	521.72	521.35	520.97	521.74	521.01	520.20	519.32
180	Negalur	514.65	514.18	513.68	513.16	512.62	513.70	512.67	511.53	510.27
185	Yalavigi	598.44	598.38	598.32	598.26	598.20	598.32	598.20	598.90	599.00
189	Outlet	510.80	510.56	510.31	510.04	509.76	510.32	509.79	509.21	508.57

Table 6. Predicted groundwater levels (in m with respect to mean sea level) of the scenarios-2 & 3 (September)

predicted based on the historical data. Various levels of increase in water demand are considered to have different options of water management. The aquifer response under different stress scenarios was studied in order to evolve optimal groundwater extraction along with surface water utilization for the sustainable development of water resources. In all, six different scenarios were considered to evolve the optimal management schemes.

The rainfall and pumping data were analyzed for the last 11 years (table 2). It indicates that there is deficient rainfall of about 6% per year with respect to the normal rainfall (1200 mm) and pumping increases by about 7% every year. The years 1994 and 1997 may be considered as wet years with surplus rainfall of more than 10%. The rainfall deficiency of 40% was observed during 2001 which may be considered as dry year. The recharge and extraction of groundwater was estimated for the last 11 years and the results are shown in table 3. Based on these statistics, six scenarios are predicted for the estimation as follows:

1. 2 % increase in the pumping rate of 2003 every year up to 2010.
2. 5 % increase in the pumping rate of 2003 every year up to 2010.
3. 10 % increase in the pumping rate of 2003 every year up to 2010.
4. 5 % increase in pumping with 2 % increase in recharge rate of 2003 every year up to 2010.
5. 20 % increase in the pumping rate of 2003 every year up to 2010.
6. Increase in recharge rate due to proposed inter-linking of Bedti-Varada river.

The simulated groundwater levels for some of the above scenarios are given in table 4 to 6.

5.6 Optimization model

The ultimate objective of the optimization model is to provide estimates of sustainable yield from both groundwater and surface water. Sustainable yield is defined here as a withdrawal rate from the aquifer or from a stream that can be maintained indefinitely without causing violation of either hydraulic-head or stream flow constraints. The optimization problem was solved by graphical method shown in figure 10 for the year 2003. The optimum withdrawals of surface water and groundwater limits were given in table 8. The amount of surface water and groundwater withdrawals in the feasible region (points 1-5) are indicated here. Table 8 clearly indicates that the total sustainable yield of 11.8 Mm^3/d is possible with conjunctive use of surface water (1.6 Mm^3/d) and groundwater (10.2 Mm^3/d) in the Varada basin. The sustainable yield from groundwater is a function of the withdrawal limit specified which accounts for annual average increase of 5-20% of extraction rates from 2003. The distribution of optimal withdrawal rates with upper limits being specified as 5%, 10 % and 20 percent multiples of 2003 groundwater withdrawal rates is continued for a short period up to 2010. The results are listed in table 9.

Considering the minimum possible growth rate of 5%, the sustainable yield of groundwater and surface water are 4.22 Mm^3/d and 0.422 Mm^3/d respectively (figure 11). Specifying an upper withdrawal limit of 10 percent of the 2003 withdrawal rate and continuing every year (scenario 2), the sustainable yield of groundwater from the basin is 5.81 Mm^3/d (table 9), which was about 3.3 Mm^3/d in 2003. If the upper withdrawal limit is increased to 20 percent annually, the sustainable yield of groundwater from the basin is about 12.88 Mm^3/d (table 9). But this rate violates the hydraulic constraint of h_c = 50m and most part of the basin would be subjected to groundwater mining. Hence this projected increase is not feasible with increase in pumping rate of 20% every year. The only option available is to increase surface water withdrawal i.e. stream flow to cater this growth rate. Now, the upper withdrawal limit of river flow increased to 10 percent of the 2003 withdrawal rate (scenario 4) with 10% increase in groundwater withdrawals. The sustainable yield from groundwater for the basin is 5.81 Mm^3/d (table 9) and the surface water withdrawal is about 0.585 Mm^3/d during the year 2010. However, there is further scope in increasing the river flow withdrawal up to 20% per year. If that would be the case, the sustainable yield from the surface water and groundwater is about 1.289 Mm^3/d and 5.81 Mm^3/d respectively.

The optimal conjunctive use surface water groundwater thus leads to sustainable development of the region within the given constraints. Policy decisions need to be centered around these results while planning the overall water resources development of the region. In this study, the numerical model gave an useful insight into the developmental scenarios for the conjunctive use of surface water and groundwater resources in the Varada river basin.

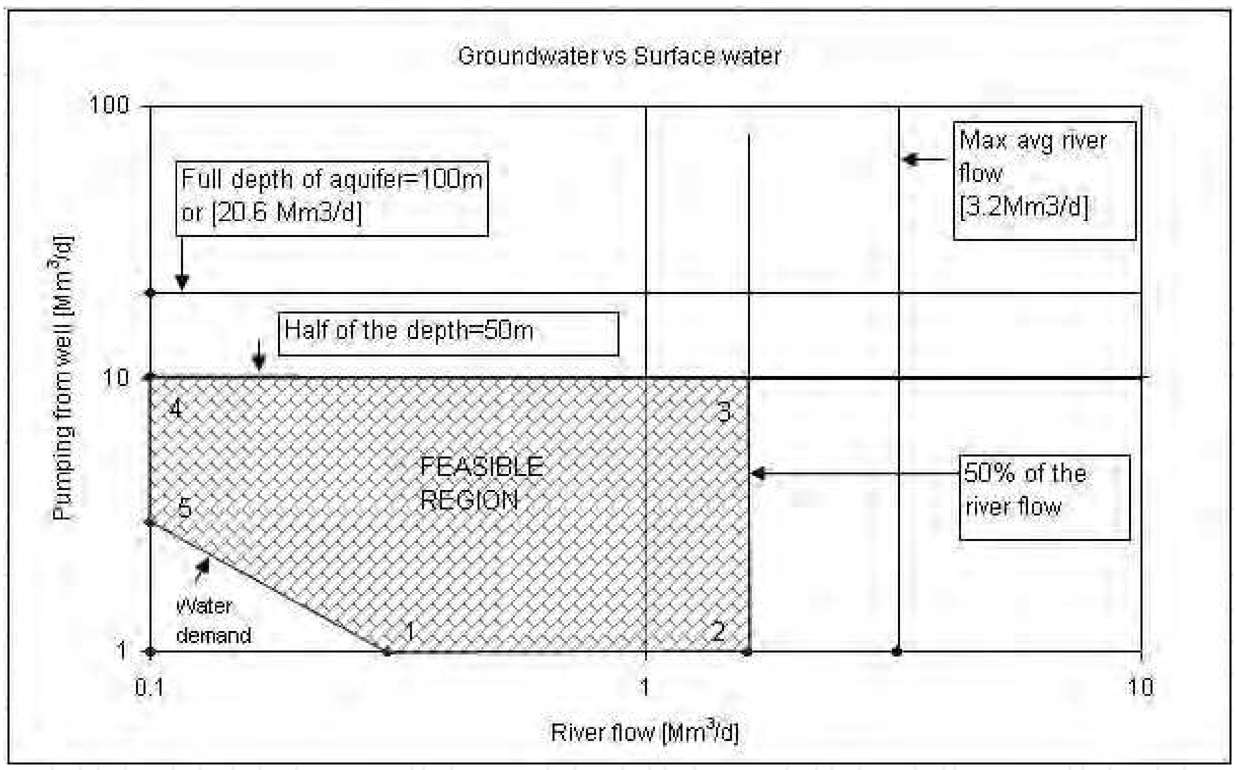

Fig. 10. Results of optimization model for the year 2003 (Ramesh and Mahesha, 2009)

Points	q_{well} [Mm^3/day]	q_{river} [Mm^3/day]	$Z = \Sigma q_{well} + \Sigma q_{river}$ [Mm^3/day]	Remarks
1	1	0.3	1.3	
2	1	1.6	2.6	
3	**10.2**	**1.6**	**11.8**	**Optimum**
4	10.2	0.1	10.3	
5	3	0.1	3.1	

Table 8. Optimum withdrawal rates of surface water and groundwater for the year 2003 (Ramesh and Mahesha, 2009)

Sources	Increase in extraction / year	2003	'04	'05	'06	'07	'08	'09	'10	Upper limits
Ground water (wells)	5% increase from 2003 every year	3.00	3.15	3.30	3.48	3.65	3.83	4.02	4.22	
	10% increase from 2003 every year	3.00	3.30	3.63	3.99	4.38	4.81	5.29	5.81	10.20
	20% increase from 2003 every year	3.00	3.60	4.32	5.18	6.22	7.46	8.95	10.74	
Surface water (river)	5% increase from 2003 every year	0.30	0.32	0.33	0.35	0.37	0.38	0.40	0.42	
	10% increase from 2003 every year	0.30	0.33	0.37	0.40	0.44	0.48	0.53	0.59	1.60
	20% increase from 2003 every year	0.30	0.36	0.43	0.52	0.62	0.75	0.90	1.08	

Table 9. Sustainable yield for different upper limits on withdrawals and demand rates [in Mm^3/d]

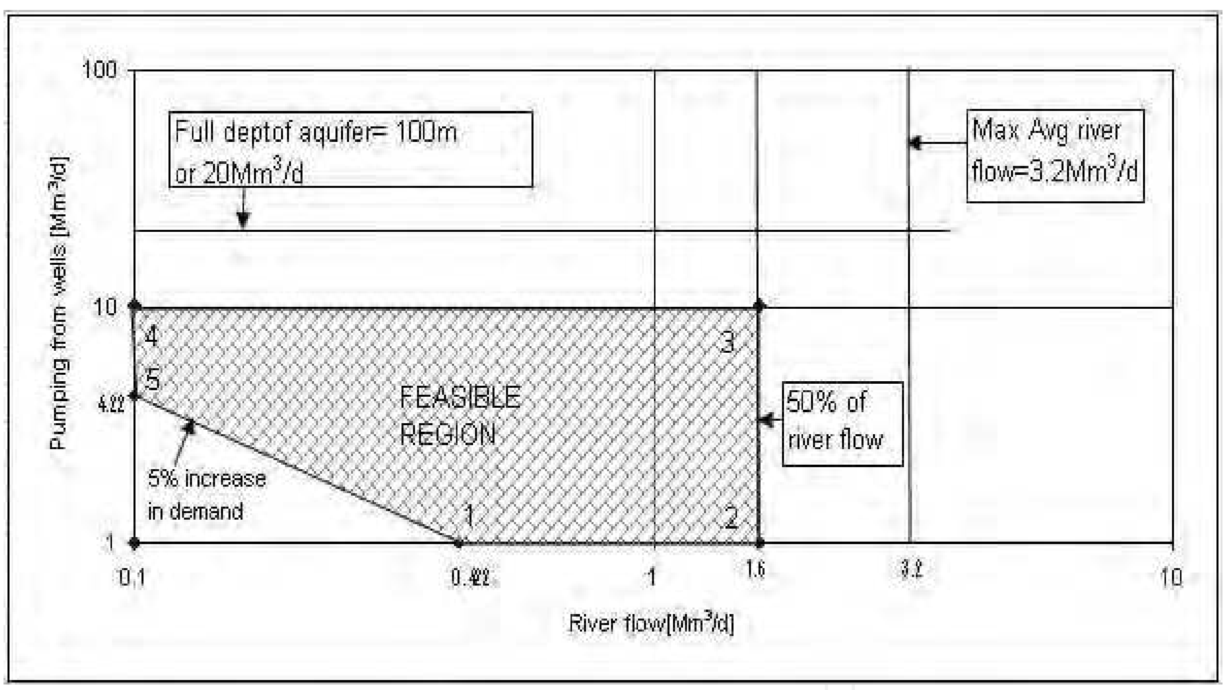

Fig. 11. Results of Scenario-1 for the year 2010 [5% increase in pumping from both surface water & groundwater / year]

5.7 Conclusions

The use of groundwater in conjunction with surface water is gaining prominence in the recent years as a part of water conservation measures in the water stressed regions worldwide. Considering the case study of India, increasing demand for fresh water has put enormous pressure on agriculture and domestic sectors due to population explosion, urbanization, industrialization of expansion of agricultural activities. The agricultural activities in the basin are predominantly controlled by the monsoon rains which are limited to four months in a year and there is an immense need for efficient utilization of available water resources during the rest of the year. To address the increasing demand for fresh water, the government of Karnataka, India is implementing World Bank assisted 'Jal Nirmal' project on the sustainable watershed development programme to ensure supply of safe drinking water to north Karnataka districts. The Varada basin is one of the beneficiaries of the project and is taken up for the present investigation. The results from the study would be useful feedback on the success of the project and the options available for the sustainable development of the region.

An attempt was made in the present study to simulate and allocate the available water resources of the basin for various demands with sustainability approach. The following conclusions may be drawn from the present study:

- The surface water model is based on the water balance approach and the groundwater recharge estimated by it compares well with the other methods. The study evaluated the effect of recharge due to rainfall and other surface water bodies on groundwater through field observations and methods proposed by Groundwater Estimation Committee. The annual average recharge in the basin is estimated to be about 1200 Mm^3
- The numerical solution was effective and accurate enough to simulate the aquifer system with mean error ranging between -0.43 to 0.55 and the correlation coefficient between from 0.78 to 0.91.
- Based on the past records on the increase in freshwater demand, an average increase of 7 % in the groundwater/surface water extraction is estimated. The simulation was carried out to predict the decline in groundwater level for various levels of development. It was predicted that up to 2% increase in extraction rate every year, the system is sustainable. The growth rates more than this may produce undesirable results with groundwater mining.
- The option of surface water supply through run-of-river supply and storage structures may be considered seriously to meet this situation. The present level of river water utilization is 0.2 Mm^3/day which can be increased up to 1.6 Mm^3 /day through adequate canal network and storage structures.
- Operation of additional conjunctive use facilities and storage capacity under flexible water allocation (water transfers) can generate substantial economic benefits to the region. Conjunctive use adds operational flexibility required for water transfers which in tern ensures water allocation flexibility needed to take economical advantage of conjunctive use.
- The optimization model provides a sustainable solution considering different water demands (domestic and agriculture) and available groundwater/surface water resources. Considering a maximum growth rate of 10% every year in the water demand, the optimal conjunctive utilization could be 5.81 Mm^3/day from groundwater resources and 0.585 Mm^3/day from surface water resources. The effective implementation of the developed policies ensures sustainable groundwater development in the study area.
- The proposed Bedti-Varada link system could augment the groundwater/surface water system of the surrounding region significantly even if a minimum utilization of 25% of total transferable amount of 242 Mm^3 is considered.

Scope for Further Investigations

To ensure sustainability, water resources systems need to be planned, designed and managed in such a way as to fully meet the social and economical objectives of both present and future generations and maintaining their ecological, environmental and hydrological integrity. This imposes constraints on every stage of development from project planning to final operation and maintenance. Water Managers and decision makers have to consider a large number of often conflicting demands on the available water and operate water resources systems under numerous social, economic and legal, as well as physical constraints. Economic constraints are equally important in water resources development in a market oriented economy and the concerned agencies may not support it without economic feasibility. In view of this, the present work can be attempted as a nonlinear optimization subjected to the social and economic constraints along with the hydraulic and stream flow constraints. The parameters which will be considered in socio-economic constraints are the gross domestic product (GDP), equity, etc. With the above issues being included, the problem may be viewed as an Integrated Water Resources Management which is the ultimate objective of sustainable development of any region.

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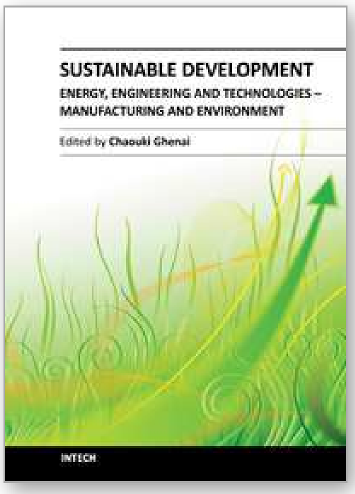

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The technological advancement of our civilization has created a consumer society expanding faster than the planet's resources allow, with our resource and energy needs rising exponentially in the past century. Securing the future of the human race will require an improved understanding of the environment as well as of technological solutions, mindsets and behaviors in line with modes of development that the ecosphere of our planet can support. Some experts see the only solution in a global deflation of the currently unsustainable exploitation of resources. However, sustainable development offers an approach that would be practical to fuse with the managerial strategies and assessment tools for policy and decision makers at the regional planning level. Environmentalists, architects, engineers, policy makers and economists will have to work together in order to ensure that planning and development can meet our society's present needs without compromising the security of future generations.