Water Supply System Planning by Artificial Groundwater Recharge from Rooftop Rainwater Harvesting

Sirajul Islam and Bipul Talukdar

Abstract Rainwater harvesting could be an effective means of supplementing the growing demand of potable water in urban areas in India, especially in high rainfall areas such as the north-eastern part of the country. Although rainwater harvesting has been practiced in India since a long time, but it has never been seriously thought of as a component for integrated management of water resources for urban water supply. Among the various rainwater harvesting techniques, artificial recharge of groundwater from rooftop rainwater harvesting is the most effective means of collecting a sizeable quantity of the rainwater for future use. As per the estimation of the Central Ground Water Board (2000), about 0.008 m^3 of rainwater can be obtained from the rooftops of buildings per square metre area for 1 cm of rainfall. A mathematical model for simulation of the three-dimensional transient groundwater flow process is developed for a confined aquifer using finite difference method. The model results are validated by comparing with the results obtained from MODFLOW simulation. This model is then applied in a hypothetical urban water supply system to provide drinking water to a population of about 75,000 considering pumping from a set of discharge wells along with a set of recharge wells for rainwater harvesting. The recharge to the aquifer takes place from eight injection wells within the study area; each collecting rainwater from the roof tops of a network of buildings located within 200 m radius. The recharge to the aquifer is considered to be lumped for each well in quarterly time steps. The recharge wells are also used for pumping water to the supply system. The operation policy of the wells are designed in such a way that the recharge wells shall be used for pumping water to the supply system only for 6 months during the period from October to March. This will permit the recharge water from monsoon storm to settle down in the aquifer and will lead to substantial water quality improvement. However, the discharge wells shall be operated throughout the year. An optimization problem is

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formulated with the objective of minimizing the squared difference of supply and demand. This optimization model is externally linked to the groundwater flow simulation model to observe the tradeoff between withdrawal from aquifer and recharge from rainwater. The constraints to the optimization model are related to groundwater depletion and temporal and spatial operation of the two sets of wells. The optimization problem turns out to be nonlinear due to incorporation of the simulation constraints. A simple genetic algorithm is used to solve the problem and the results are obtained in the form of head distribution and drawdown after every successive time steps.

Keywords Groundwater flow · Rainwater harvesting · Artificial recharge · Genetic algorithm

1 Introduction

Migration of rural population to urban areas for economic and social reasons has led to rapid growth of urban population in India. The trend of urbanization has thrown up a serious challenge to the civic authorities to provide potable water to the urban mass both qualitatively and quantitatively. Nonavailability or distant location of perennial source of water, deterioration of aesthetic quality of water due to over treatment are some of the problems encountered while planning an urban water supply system with surface water sources. On the other hand, indiscriminate use of groundwater may lead to irreversible depletion of the aquifer system. Lack of a proper pricing policy has given freedom to the end users to misuse the precious water. Hence, if groundwater is the means of community water supply, it is necessary to plan the water supply system in an integrated manner considering recharge to the aquifer system. Natural recharge is a very slow process which is a function of the infiltration capacity of soil. Artificial recharge may therefore be considered as the best alternative to augment the depleting aquifer.

Rainwater harvesting could be a very effective way of augmenting the water table in an aquifer in areas with high rainfall such as the north-eastern Region of India. It has been shown that up to 40 % of onsite rainfall can be recharged back to the aquifer using appropriate techniques (Stout et al. 2015). Among the various rainwater harvesting techniques, collecting rainwater from the rooftops of buildings could be the best option in urban areas due to the presence of large structures such as commercial buildings, institutions and multi-family dwellings, etc. On an average, about 0.008 m^3 of rainwater is available for recharge from a rooftop area of 1 m^2 for 1 cm of rainfall (Central Groundwater Board 2000). Aquifer recharge from rainwater not only augments the water level in the aquifer but also improves the groundwater quality as rainwater is free of any impurity and hardness. Moreover, the problem of clogging of injection well is addressed to certain extent as they deal with a pure form of water (Central Groundwater Board 2007).

This study aims at developing a simulation-optimization model for integrated management of ground water with artificial recharge from rooftop rainwater harvesting for an urban water supply system. Model results are presented to show how rainwater harvesting could improve the sustainability of a medium scale water supply scheme.

2 Materials and Method

While making a long-term plan for extracting groundwater for any water supply scheme, it is very important to understand the aquifer characteristics, the hydraulic properties of the porous media, and the limits on drawdown and recharge characteristics.

A three-dimensional groundwater flow simulation model is developed for a confined aquifer using finite difference method. This model is applied to hypothetical urban water supply scheme to supply potable water to a population of about 75,000. A set of eight discharge wells are defined at specific locations. Another set of eight recharge wells are used to collect rainwater from rooftops of buildings located within a radius of 200 m. An optimization model is formulated with the objective of minimizing the squared difference of supply and demand. The groundwater flow model is externally linked to the optimization model to incorporate the simulation constraints viz., the limits on drawdown at each well. Discharge wells are operated throughout the year for supplying water to the distribution network, while the recharge wells are used only for pumping in water to the aquifer during the period from April to September assuming that most of the rainfall occurs during this period. These wells are also used for extracting water during the period from October to March. Such an operation policy of the wells permits the rainwater collected during rainy season to settle down in the aquifer and blend with ground water, resulting in water quality improvement. Further, the cost-effectiveness is improved by utilizing these wells for extracting water (which otherwise would remain idle) during the dry season. A simple binary-coded genetic algorithm is used to solve this nonlinear optimization problem in four quarterly time steps.

2.1 Ground Water Flow Simulation Model

The major components of a ground water flow model are the partial differential equation governing the flow process, the initial conditions and boundary conditions. The solution of such a model refers to the calculation of head values at each point of the system (Anderson and Woessner 1992).

2.1.1 Ground Water Flow Equation

The three-dimensional transient ground water flow through a porous medium can be represented by the following partial differential equation (McDonald and Harbaugh 1988).

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right)-W=S_{\mathrm{S}}\frac{\partial h}{\partial t} \tag{1}$$

where h is the hydraulic head (L); K_{xx}, K_{yy} and K_{zz} are values of hydraulic conductivity along the x, y, and z coordinate axes, which are assumed to be parallel to the major axes of hydraulic conductivity (L/T); W is a volumetric flow rate per unit volume and represents sources and/or sinks of water (T^{-1}); S_{S} is the specific storage of the porous medium (L^{-1}); and t is time (T).

2.1.2 Finite Difference Discretization

The finite difference discretization of the partial differential equation representing ground water flow converts the continuous variables into discrete variables that are defined at the grid points. Finite difference grids can be either block centred, where the nodes are located in the centre of each grid cell or mesh centred, where the nodes are located at the intersection of grid cells (Wang and Anderson 1982). In this study, block-centred grids as illustrated in Fig. 1 are used. ΔX, ΔY and ΔZ represent the size of grids in x, y and z directions, respectively.

The discretization of the ground water flow equation results in a set of simultaneous linear equations. The finite difference approximation of cell (i, j, k) can be represented by Eq. 2.

$$\begin{aligned}&C1h^t_{i-1,j,k}+C2h^t_{i,j-1,k}+C3h^t_{i,j,k-1}+C4h^t_{i+1,j,k}+C5h_{i,j+1,k}+C6h_{i,j,k+1}+C7h_{i,j,k}+Q_{i,j,k}\\&\quad=S_{i,j,k}(\Delta X\Delta Y\Delta Z)\frac{h^t_{i,j,k}-h^{t-1}_{i,j,k}}{\Delta t}\end{aligned} \tag{2}$$

The space derivatives are discretized using central difference scheme, while the time derivatives are discretized using backward difference scheme.

2.1.3 Initial and Boundary Conditions

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

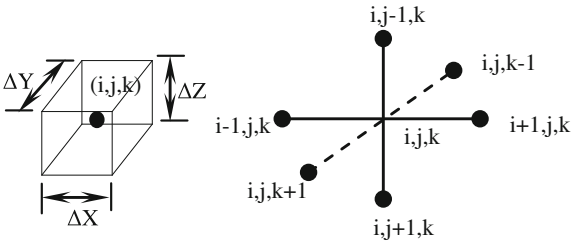

Fig. 1 *Block* centred grid system with associated indices

Mathematically,

$$h(x, y, z) = \Phi(x, y) \tag{3}$$

where Φ is a known function.

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

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

$$h(x, y, z) = \text{constant} \tag{4}$$

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

$$\frac{\mathrm{d}h(x, y, z, t)}{\mathrm{d}} n = \text{Constant} \tag{5}$$

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

$$\frac{\mathrm{d}}{h}\mathrm{d}n + \mathrm{ch} = \text{Constant} \tag{6}$$

The first two types of boundary conditions are applied in this model in terms of constant head and no flow boundaries. In finite difference approach, the aquifer domain is discretized into a system of rectangular grids. So, the aquifer boundaries are supposed to be along a straight line. But in most real life situations, the boundaries are of irregular nature, necessitating their proper representation in the mathematical model. This model represents such irregular boundaries as shown in Fig. 2.

2.1.4 Study Area and Aquifer Parameters

A hypothetical aquifer system is defined for simulating the ground water flow process which shall be used for optimal management of an urban water supply

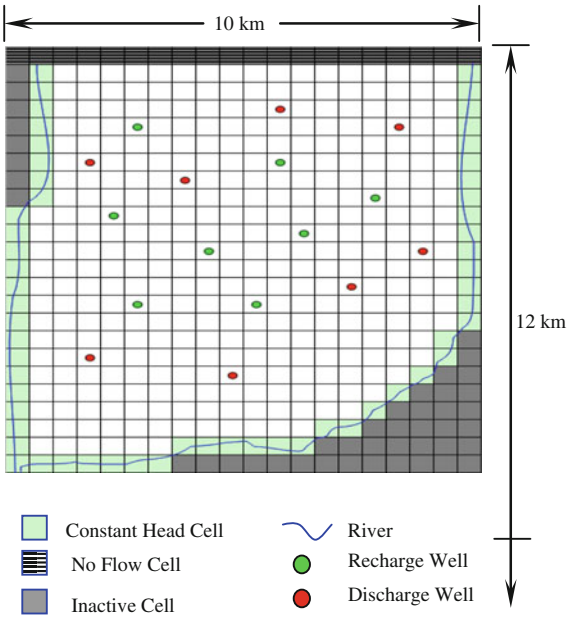

Fig. 2 Schematic diagram of hypothetical study area showing boundary conditions and well locations

system. Figure 2 shows the discretized study area, aquifer boundaries and pumping locations. The aquifer area of 10 km × 12 km is discretized into rectangular grids of 500 × 500 m size.

The eastern, western and southern boundaries of the aquifer are designated as constant head boundaries (river boundary) while the northern side is considered to be bounded by a hydraulic divide line representing no flow conditions across it. The hydraulic properties such as Horizontal hydraulic conductivity, storage coefficient are specified for each cell. However, vertical hydraulic conductivity is not taken into account as this model considers only a single confined layer of the aquifer. Also for simplicity, the aquifer is assumed to be homogeneous and isotropic in nature. Pumping locations are shown in Fig. 2. These locations are chosen in such a way that the radii of influence of any two wells do not intersect. Minimum distance of 500 m is maintained between two adjacent wells. The hydraulic gradient is considered to be in north–south and east–west directions. The head values at constant head boundaries are assumed to decrease linearly at a rate of 0.1 m. The constant head at the western boundary starts at 75 m and ends at 72.8 m. Similarly, a head value of 76.2 m is assumed at the starting point of eastern boundary, which extends through the southern part of the aquifer and merges with the end point of western boundary.

2.1.5 Method of Solution

The discretization of the flow equation results in a set of simultaneous linear equations of the form represented by Eq. 2. There are various methods for solving simultaneous linear equations. A computer code is written in MATLAB which solves this set of linear equations using matrix inversion method. This program returns the head values at different grid points as a function of discharge rates at different pumping locations and resulting draw down at these locations. MATLAB uses backlash operator to solve simultaneous equations.

$$[A]\{h\} = [B] \tag{7}$$

$$h = B/A$$

where A is the coefficient of flow equation, B is the constant on right-hand side and h is the peizometric head.

2.1.6 Model Validation

There are several numerical models for simulating the three-dimensional ground water flow process. In this study, MODFLOW, developed by the U.S. Geological Survey (McDonald and Harbaugh 1988), is used. MODFLOW is a complete ground water flow and transport model that uses block centred finite difference grid system. It consists of a main program and sub-routines called modules. The original MODFLOW program has a difficult interface, due to which different user friendly Graphical User Interfaces have been developed by software companies. In this study, PMWIN (Processing MODFLOW for WINDOWS), developed by Chiang and Kinzelbach (1995) is used for simulation. Identical flow parameters and discharge rates as presented in Table 1 and boundary conditions as discussed in Sect. 2.1.4 are used for simulation of both MODFLOW and this model. The results of simulation are shown in Fig. 3 as comparative plots of contours of head values (m) after each time step of simulation. The contours do not seem to deviate even after three continuous stress periods with high discharge rates. The numerical values differ in the order of 10^{-3} in decimal fractions.

2.2 Rainwater Harvesting Model

This study considers rooftop rainwater harvesting scheme for groundwater recharge in a confined aquifer through injection wells which would also be used for groundwater extraction. It consists of a hypothetical system of eight rainwater harvesting stations each collects rainwater from the rooftops of a network of buildings. These buildings include govenment establishments, institutions,

Table 1 Flow parameters and pumping rates for comparative simulation of flow model and MODFLOW

Flow parameter	Value	Discharge rates (m^3/day)				
		Well location	Steady state	$t = 1$	$t = 2$	$t = 3$
Horizontal hydraulic conductivity						
Saturated thickness of aquifer		(8,13)	–	6500	1500	4500
Storage coefficient	0.0001 m/s	(12,5)	–	2000	5000	3000
Time period		(17,11)	–	3000	4500	1000
	40 m					
	0.0001					
	30 days					

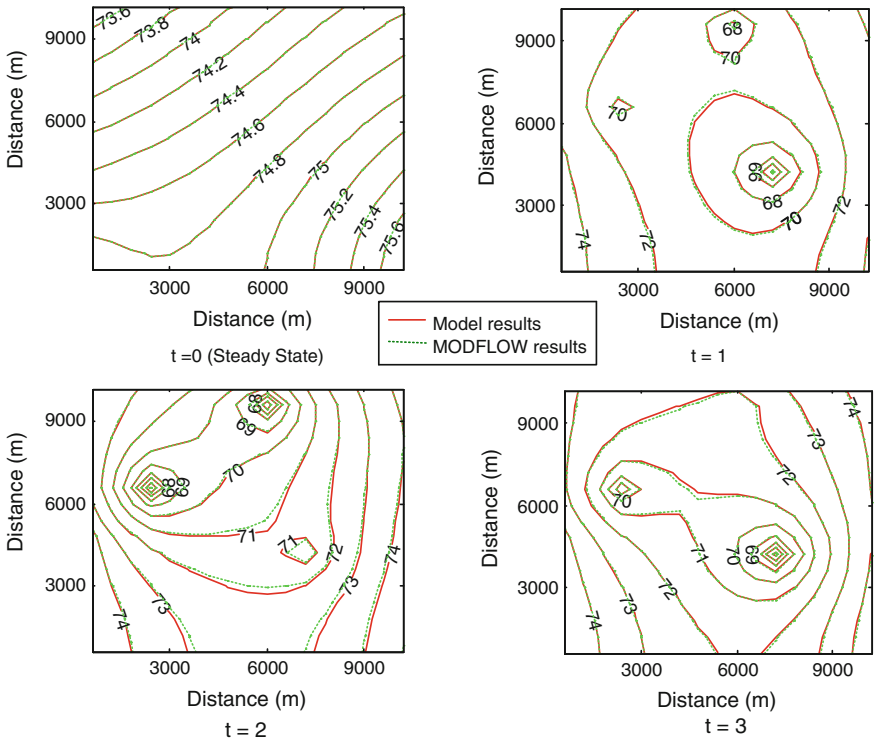

Fig. 3 Comparative contours of head values obtained from simulation of flow model and MODFLOW

Table 2 Rooftop area and quantity of rainwater collected at different recharge stations

Recharge station	no. of structures	Total roof area (m^2)	Quantity of rainwater collected (m^3)			
			Apr–June	July–Sept	Oct–Dec	Jan–Mar
RWS1	6	2119	1830.60	2237.40	305.10	339.00
RWS2	5	2073	1790.64	2188.56	298.44	331.60
RSW3	4	1543	1332.72	1628.88	222.12	246.80
RSW4	6	2713	2343.60	2864.40	390.60	434.00
RSW5	5	2102	1816.56	2220.24	302.76	336.40
RSW6	4	1079	932.04	1139.16	155.34	172.60
RSW7	5	1933	1669.68	2040.72	278.28	309.20
RSW8	6	2254	1947.24	2379.96	324.54	360.60

commercial buildings or multi-family dwellings. A set of data for each station showing rooftop areas and quantity of rainwater collected, is presented in Table 2 depicting a real life situation. The rainfall data is considered in four time steps of three months each viz. Apr–June (t = 1), July–Sept (t = 2), Oct–Dec (t = 3) and Jan–Mar (t = 4) with cumulative rainfall of 108, 132, 18 and 20 cm, respectively. For convenience in simulation, each of these time periods is taken as 90 days.

2.3 *Optimization Model*

The basic objective of this study is to provide drinking water facilities to a population of about 75,000 living in an urban area. The per capita water supply norm in India as specified by Central Public Health and Environmental Engineering Organization for towns with piped water supply without sewerage system is 70 L/day (Central Pollution Control Board 2010). As such a water demand of 5250 m^3 is the target to be fulfilled per day.

2.3.1 Objective Function

The objective function is formulated as the squared difference of supply and demand.

$$\text{Minimize:} f(x) = \left(Q_{\mathrm{d}}^{t} - \sum_{n=1}^{N} Q_{\mathrm{p}n}^{\ t} \right)^2 \tag{8}$$

where Q_{d} is the water demand (m^3), Q_{p} is the quantity of water to be pumped (m^3), n is the pumping location index, N the number of wells and t is the time index.

The decision variables in this optimization problem are the discharge rates of different pumping wells which may be expressed as $\boldsymbol{q}_{\mathbf{p}}$ (m^3/day)

$$F(x) = \frac{1}{1 + P(x)}.$$

2.3.2 Constraints

The groundwater flow model described in Sect. 2.1 is externally linked to the optimization model. An upper limit is specified for the drawdown at different pumping locations at any instance of pumping. This constraint function can be expressed as

$$g(x) = S_{\mathrm{s}} - S_n^t \geq 0 \tag{9}$$

where s_n^t is the drawdown at pumping location n at any time period t and s_{s} is specified value of drawdown (m). Apart from the drawdown constraint, upper and lower limits are fixed for the decision variables, i.e. the discharge rates of different pumping wells. This may be expressed as:

$$q^u \geq q_{\mathrm{p}} \geq q^l \tag{10}$$

where q^u is specified upper limit on discharge (m^3/day), q^l is the specified lower limit on discharge (m^3/day).

Due to incorporation of the simulation constraints, the optimization problem turns out be nonlinear in nature. A binary-coded genetic algorithm (GA) is used to solve this nonlinear minimization problem. The details of GA can be found in Goldberg (1989). Due to their evolutionary nature based on survival of fittest principle, GAs are naturally suitable for solving maximization problems. However, minimization problems can also be efficiently solved with appropriate conversions. The following conversions are made in this problem.

First, the constrained minimization problem is converted to an unconstrained problem using penalty parameter approach as given below:

$$p(x) = f(x) + \sum_{n=1}^{N} R_n \langle g_{n(x)} \rangle^2 \tag{11}$$

where $P(x)$ is the penalty function, $g(x)$ is the constraint function and R is the penalty coefficient.

This unconstrained minimization problem is then converted to an equivalent maximization problem by introduction of fitness function.

3 Results and Discussion

The optimization model is externally linked to the groundwater flow simulation model. The decision variables are the discharge rates of the wells as specified in Fig. 2. During first two time steps, i.e. during the periods April–June and July–September, only eight discharge wells are used for pumping, while the recharge wells are used only for pumping in rainwater to the aquifer. But during the other two time steps, the recharge wells are also used for pumping, thus constituting 16 decision variables altogether.

The optimization results (Scenario-I) are presented in Fig. 4 as the contours of drawdown after each time step. An upper limit of 10 m is fixed for drawdown after every time step. Table 3 shows the discharge rates of wells. The upper bound on discharge rate is set at 1000 m^3/day.

With same limits on drawdown and discharge rates, optimization is carried out in the same manner as above, but without considering recharge from rainwater harvesting (Scenario-II). Results show a different trend with drawdown level reaching the maximum limit and shortage in the supply level during the last time step (t = 4) (Fig. 5, Table 4).

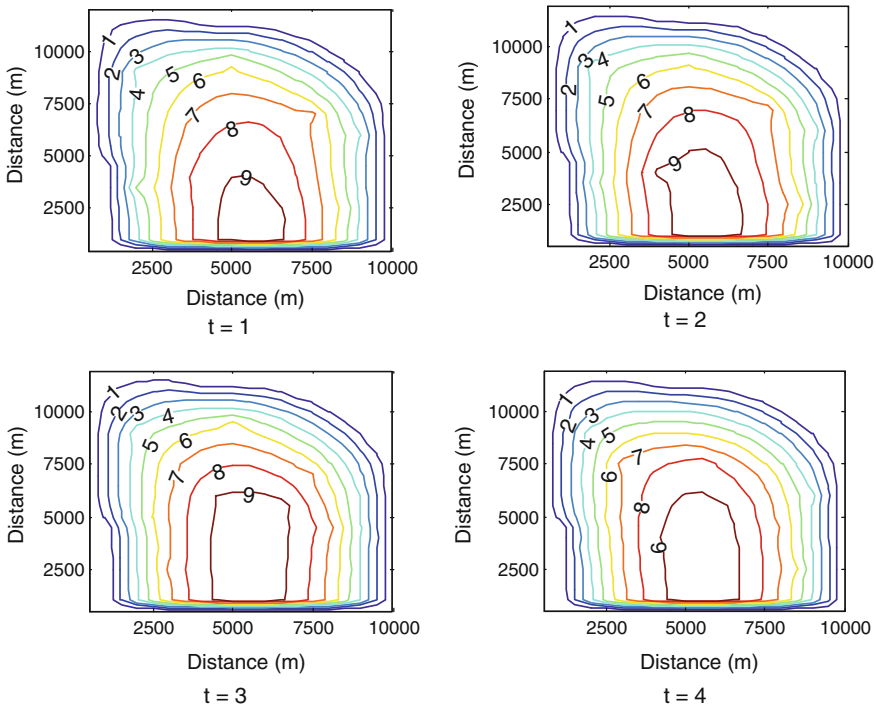

Fig. 4 Post-optimization contours of drawdown during different time steps (Scenario-I)

Table 3 Discharge rates of different pumping wells obtained from optimization using Genetic Algorithm during different time steps (Scenario-I)

Cell no.	Discharge rate (m^3/day)			
	$t = 1$	$t = 2$	$t = 3$	$t = 4$
(5,6)	0	0	357.25	153.43
(7,12)	0	0	54.64	135.08
(10,5)	0	0	728.32	245.23
(9,16)	0	0	252.75	310.53
(11,13)	0	0	538.11	210.65
12,9)	0	0	256.48	66.53
15,6)	0	0	202.83	758.29
15,11)	0	0	46.66	612.09
(4,12)	622.21	123.52	102.87	94.05
(5,17)	429.53	891.97	715.28	548.98
(7,4)	973.95	552.85	48.98	344.75
(8,8)	288.55	579.13	67.55	162.24
(12,18)	603.84	711.31	243.23	626.89
(14,15)	944.85	895.81	460.73	253.56
(18,4)	614.65	912.26	830.20	692.73
(19,10)	841.64	583.40	844.12	34.97

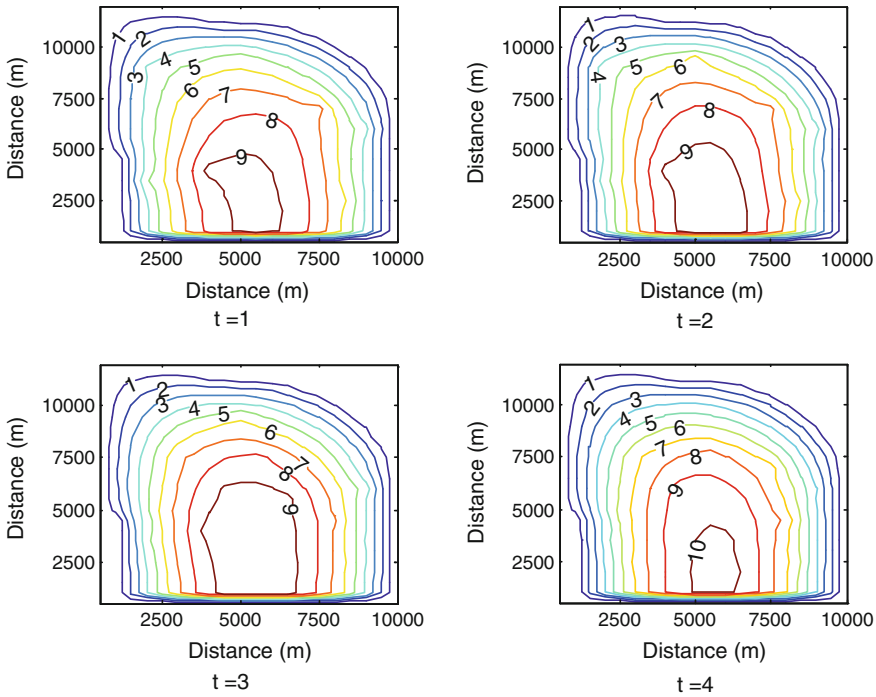

Fig. 5 Post-optimization contours of drawdown during different time steps (Scenario-II)

Table 4 Discharge rates of different pumping wells obtained from optimization using Genetic Algorithm during different time steps (Scenario-II)

Cell no.	Discharge rate (m^3/day)			
	$i = 1$	$t = 2$	$t = 3$	$t = 4$
(5,6)	0	0	27.70	168.93
(7,12)	0	0	55.12	749.57
(10,5)	0	0	369.33	99.90
(9,16)	0	0	279.26	367.20
(11,13)	0	0	388.50	596.78
12,9)	0	0	215.14	209.75
15,6)	0	0	403.32	327.82
15,11)	0	0	377.27	34.76
(4,12)	153.02	279.35	234.62	253.33
(5,17)	499.03	521.37	226.05	388.69
(7,4)	776.92	494.61	513.70	400.75
(8,8)	906.69	625.93	598.00	489.92
(12,18)	749.67	779.03	873.95	182.46
(14,15)	975.43	748.55	31.86	95.71
(18,4)	569.27	900.57	72.50	629.73
(19,10)	619.96	900.60	583.68	254.61

Table 5 Parameters of Genetic Algorithm used in optimization

Sl. no.	G.A. parameter	Specification
1	Population size	100
2	Selection operator	Binary tournament selection
3	Cross over probability	0.75
4	Mutation probability	0.005
5	No. of generations	40

Using the GA parameters presented in Table 5, the GA is run for 40 generations in each time step for both Scenario-I and II. Generation versus Fitness curves are drawn for both Scenario-I and II as shown in Fig. 6a, b.

These curves show different trends. In scenario-I, the maximum fitness is reached within 20 generation except for time period, $t = 4$, where the same is obtained after 35 generations. Contrary to this, scenario-II needs more number of generations to reach near the maximum fitness level. Due to imposition of high penalty for violating drawdown constraints ($R = 1000$), constraints are not violated in any case as evident from contours of drawdown (Figs. 4 and 5). During Time step, $t = 4$ of scenario-II, the drawdown level reaches the maximum level of 10 m and maximum fitness in this case is less than the desired value indicating shortage in water supply level.

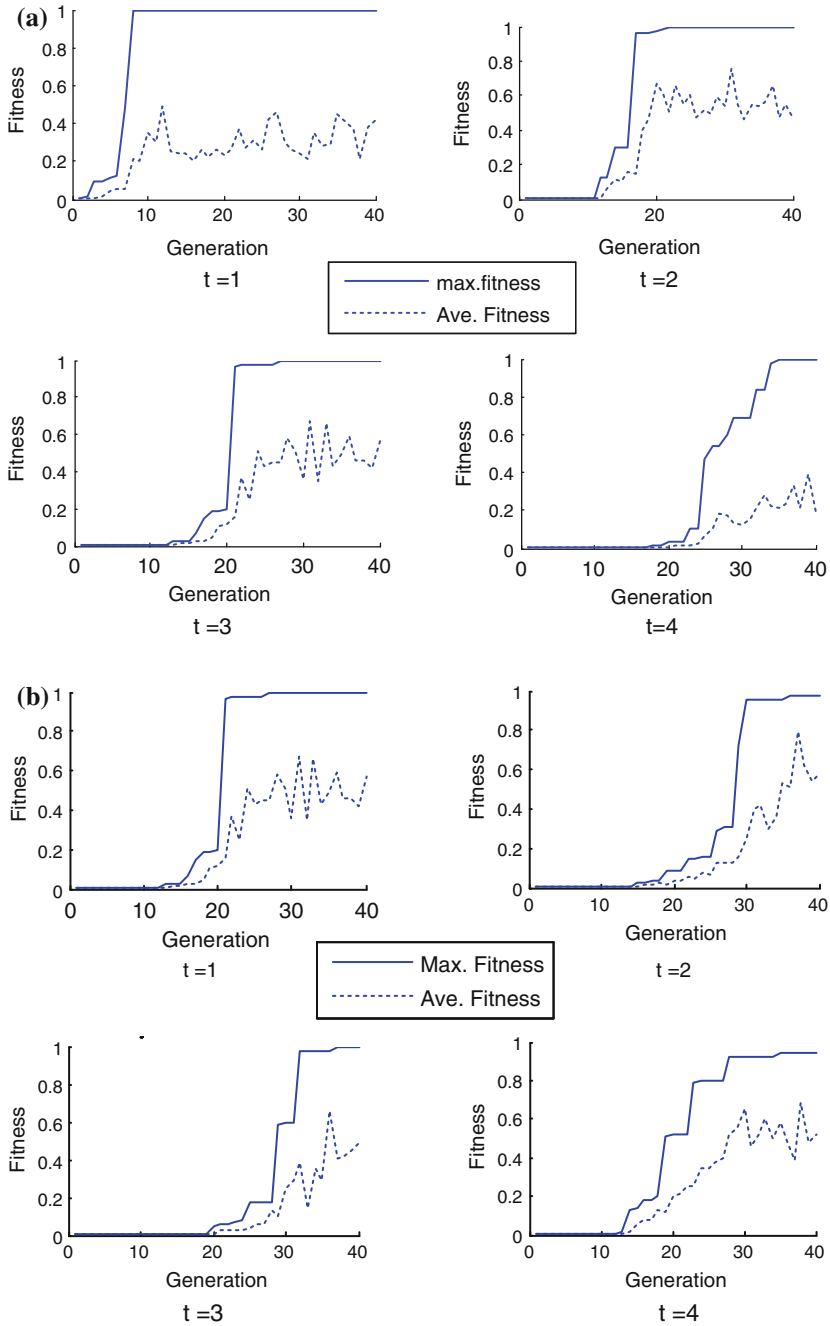

Fig. 6 **a** Fitness versus generation curves (Scenario-I). **b** Fitness versus generation curves (scenario-II)

4 Summary and Conclusion

A groundwater flow simulation model is developed for a confined aquifer using finite difference method considering artificial recharge from rainwater harvesting. This model is applied to a hypothetical urban water supply system to provide drinking water to a population of about 75,000. Optimization is done with the objective of minimizing the difference between supply and demand of water in quarterly time steps. Optimization is carried out under two different scenarios, first with aquifer recharge from rainwater and then without considering the recharge. Different trends in groundwater response are observed under the two scenarios of optimization. Although the water requirements are fulfilled under scenario-I (with rainwater recharge) during all the time steps, but scenario-II (without recharge) shows shortage in supply level towards the end with the same limits on drawdown.

This study considers only eight recharge stations with limited number of roof structures for collecting rainwater. But with more number of recharge points and a good number of buildings connected through piped network, rainwater harvesting could be a very effective component of urban water supply system. The water quality issues can also be addressed to a great extent by rainwater recharge, if the same is properly planned.

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