# Saph Pani

Enhancement of natural water systems and treatment methods for safe and sustainable water supply in India

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Deliverable D 5.4

Synopsis of modelling and monitoring approaches in the Indian context

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| **Authors in alphabetic order** | Ahmed S., Alazard M., Amerasinghe P., Bhola P., Boisson A., Elango L., Fahimuddin M., Feistel U., Fischer S., Ghosh N. C., Grischek T., Groeschke M., Grützmacher G., Hamann E., Indu S.N., Kloppmann W., Mahesh J., Mondal N.C., Monninkhoff B., Pettenati M., Picot-Colbeau G., Rajaveni S. P., Rao S., Sandhu C., Sarah S., Schneider M., Sklorz S., Sprenger C., Thiéry D., Tomsu C., Zabel A. K. |
| **Contact for queries** | Wolfram Kloppmann<br>BRGM, LAB/ISO, 3 Av. Claude Guillemin, F-45060 Orléans, France<br>Phone: +33 2 38643915<br>Email: w.kloppmann@brgm.fr |
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# Content

# Introduction

This deliverable was initially planned as a “draft paper for stakeholders”, synthesizing the experiences of modelling water quantity and quality of the SAPH-PANI case studies. The consortium decided to edit a Handbook as main interface with stakeholders, both those with technical interest in the project findings (water managers, engineers) and those who might use the project results as basis for regulation and political decision making. It was therefore agreed upon an integration of the synthesis on modelling techniques and applications of deliverable D5.4 into two book chapters instead of one draft paper. This deliverable provides an overview of modelling techniques that can be applied to natural treatment systems as MAR-SAT and river bank filtration (chapter 15 of the handbook) to show the range of technically more or less demanding tools available for initial planning, implementation and optimisation of NT systems. The experience and feedback from the case studies where modelling has been applied is provided in chapter 16 with a common and very straightforward structure for all sites answering to a shortlist of concrete questions water managers might ask about the usefulness of modelling in a given context:

- Where? Site description
- Why? Problems to be solved
- How? Tools and modelling strategy
- So what? Outcome, added value and perspectives

A general conclusion resumes the potential of modelling for operational planning and the scientific challenges still to be met for certain types of more complex problems to be solved (e.g. modelling of water quality impacts through reactive transport modelling).

# Chapter 15

# Models for natural water treatment systems in the Indian context

*Sprenger, C., Monninkhoff, B., Tomsu, C., Kloppmann, W.*

## 15.1 MODELS AS TOOLS TO ASSESS FEASIBILITY OF NWTS TO CHECK OPTIONS FOR IMPLEMENTATION TO OPERATE/OPTIMISE AND TO ASSESS IMPACTS ON WATER QUALITY

### 15.1.1 Why modelling Indian Natural Treatment Systems?

Modelling either by numerical or analytical approaches is a widespread tool in hydrogeology, but requires careful considerations of the type and level of effort to be spent which in turn depends on the purpose of the study. Before starting to model, Anderson and Woessner (1992) proposed to answer the following generic questions:

- Is the model to be constructed for prediction, system interpretation, or a generic modelling exercise?
- What do you want to learn from the model? What questions do you want the model to answer?
- Is a modelling exercise the best way to answer the question(s)?
- Can an analytical model provide the answer or must a numerical model be constructed?

In this chapter we will outline the use of groundwater models for NT-Systems focusing on the strategic phases of NTS (feasibility in a given watershed, NTS design and implementation, NTS operation) as well as on the model selection (types of models). Generic numerical models are then developed for two NTS types, bank filtration and Management of Aquifer Recharge (MAR) via infiltration ponds. The purpose of this modelling exercise is to characterize important NTS performance parameters such as recovery rates, infiltration rates or travel times of the infiltrate in the subsurface. Investigation of modelling scenarios for each of the NTS types and comparison of numerical and selected analytical solutions is another aim.

Applications of different types of models within the case studies of the Saph Pani project will illustrate this synthesis (chapter 16).

The specificity of natural treatment systems is that they rely on natural processes depending on complex interactions of surface water, wastewater and groundwater and the contaminants they may contain with the aquifer matrix, with microorganisms and plants. Contrarily to completely engineered systems, the functioning of NTS needs to be understood first to be able to predict their performance. Once a conceptual model of the relevant processes has been established on the basis of a variety of measurements, analytical or numerical models can be set up, that simulate the behaviour of the NTS as closely as possible through an iterative process of model setup and model calibration.

Both a completely natural and even a partly engineered NTS can be, at the very beginning of their implementation, considered as a black box contrarily to engineered treatment systems. Their potential impact or performance can be estimated on the basis of general knowledge, as acquired in the SAPH PANI project, but will largely depend on the local climatic, hydrological, geological, and biological (including macrophytes, microphytes, microbia) conditions, on land use and, last but not least, on the quality and quantity of water to be treated. Measurements and monitoring are indispensable to get an insight into this black box system and are a prerequisite to any establishment of a model, even a conceptual one. The data situation will largely determine the complexity and the reliability of NTS models. Setting up a highly sophisticated model on a weak data basis will only create artificially precise simulations and predictions. In sum, modelling is inseparably linked to monitoring providing

A series of key questions need to be addressed before, during and after implementing an NTS facility

- What will be the impact of a NTS system at local scale in terms of water availability and water quality improvement (or deterioration)?
- What will be the radius of influence on water quality and quantity of an individual NTS?
- Will its performance be durable over time?
- How will it behave in cases of changing boundary conditions (climatic, hydrologic, landuse, …) or in the case of extreme events (droughts, floods)
- How can it be improved through adapted configuration (e.g. position of wells with respect to a river, pumping...), or by adding engineered components to the system like including in-situ or post treatment measures for water quality improvement?
- What will be the impact of NTS at basin scale when a large number of individual systems are implemented and if different systems are combined within a watershed?

These questions will be asked from the very beginning of the planning phase and over the whole lifetime of the NTS project and coupled surface-groundwater models, potentially with contaminant transport, provide the unique possibility to preview the feasibility of an NTS system in the regional context, to optimize the choice of the site, the configuration, to optimize operating conditions in a way to meet fixed quantity and quality targets. Those targets are most frequently quantified through key parameters, like water quality acceptable for given uses, groundwater level evolution, salinity.

Especially in India groundwater is an important resource, accounting for approximately 60 % of irrigation water and 85% of drinking water and it is estimated that 60 % of groundwater sources will be in a critical state of quantitative degradation within the next twenty years (Worldbank, 2010 and referenced therein). Managed aquifer recharge (MAR) is identified as a strategy to cope with dwindling water resources and “The National Groundwater Recharge Master Plan” is developed to assess the nationwide feasibility of MAR (CGWB, 2005). Modelling can accompany the implementation of NTS over different generic phases, common to all NTS types and regional contexts (table 15.1).

Table 15.1 Planning phases of NTSs and case studies in India (Saph Pani project), see chapter 16

| Phase | Examples from Saph Pani |
|---|---|
| Phase 1: initial feasibility study in the regional context and choice of the NT-System(s) | • Choice of NT-Systems (MAR) for saline intrusion management in the coastal Arani and Koratalaiyar watershed, Chennai, Tamil Nadu |
| Phase 2: estimation of the radius of influence and positive/negative impact of an individual NTS | • Simulation of the behaviour of individual percolation tanks, Maheshwaram, Telangana |
| Phase 3: planning of NTS implementation at watershed scale | • Implementation of check dams in the coastal Arani and Koratalaiyar watershed, Chennai, Tamil Nadu |
| Phase 4: estimation of the impacts on water quality and quantity at aquifer and watershed scale | • Scenarios of wetland impacts on water balance in the Musi watershed, Hyderabad, Telangana<br>• Simulation of contaminant transport/attenuation in an alluvial aquifer: RBF at Yamuna River, New Delhi |
| Phase 5: optimisation of individual and watershed scale solutions | • Optimisation of well technology and exploitation schemes assisted by flow modelling in Haridwar, Uttarakhand |

### 15.1.2 What models for Indian Natural Treatment Systems?

The variety and degree of complexity of models is large and, as stated above, has to be adapted to the problem to be solved and to the available data situation. Geometry of groundwater models range from 1D to full 3D and the chosen spatial resolution will determine the calculation times. Processes used for natural water treatment mainly take place at the interfaces of different compartments of the local or regional water cycle (surface flow, unsaturated flow, groundwater flow, seawater intrusion) so that there is need for integration of different types of model (river models, unsaturated-saturated groundwater models, density driven flow models) which revealed a major challenge for the simulation of the behaviour e.g. constructed wetlands at basin scale (Musi river study site, chapter 16).

A complete response to the questions listed above, also addressing contaminant transport and water quality in general, may need the use of reactive transport models or even state of the art bio-geochemical reaction modelling. Even simple models (analytical models) can provide sufficient information at least for preliminary design or evaluation of NTS systems but, most frequently, numerical models will be used. Standard numerical models will nowadays be able to simulate up to full 3D advective and dispersive flow and transport of water and solutes. Supplementary features may be needed as, in the order of increasing complexity:

- Density driven flow (in the context coastal aquifers salinisation), e.g. the Chennai case study (chapter 16)
- Sorption and (bio-)degradation of solutes (e.g. through sorption isotherms, degradation factors) e.g. the New-Delhi RBF case study investigating ammonium transport (chapter 16)
- Variable saturation flow (in the case of a significant thickness of the unsaturated zone, in particular if the latter plays an important role for water quality improvements in SAT systems) e.g. the Maheshwaram case study looking upon infiltration processes when using infiltration ponds/tanks for MAR-SAT[1] (chapter 16)
- Geochemical reactions through the combined use of flow-transport models and thermodynamic equilibrium models or thermo-kinetic models taking into account the reaction kinetics e.g. the Maheshwaram case study dealing with Fluoride mobilisation upon MAR (chapter 16)
- Biologically mediated geochemical reactions (specific models available)

In this chapter we will outline, through simulation of generic benchmark tests, the use of some types of groundwater models available for NTSs, in particular MAR and river bank filtration (BF) allowing for model selection in function of the problem to be treated and available capacity and means. Applications of different types of models within the case studies of Saph Pani will illustrate this synthesis in chapter 16.

## 15.2 SOME ANALYTICAL SOLUTIONS FOR NTS SYSTEMS

Analytical solutions are simplifications and generally assume hydraulic properties to be homogenous and isotropic. Boundary conditions are often simplified and assumed to be constant. Nevertheless, analytical solutions for NT-systems often provide a straight-forward approximation of important performance parameters such as recovery rates, infiltration rates or travel times of the infiltrate in the subsurface.

### 15.2.1 Bank Filtration

Bank filtration (BF) systems in India are often utilized as the sole purification treatment along with limited post-treatment such as chlorination (Sandhu et al., 2010). The purification capacity of the BF systems depends to a large extend on hydraulic parameters such as mixing ratio between native groundwater with induced surface water (bank filtrate) and the travel time of the bank filtrate to the abstraction well.

Simple analytical solutions for BF systems were developed by (Rhebergen and Dillon, 1999) and (Dillon et al., 2002) to approximate travel time of bank filtrate from the surface water to the abstraction well. The authors assume an initially horizontal water table and do not consider riverbed clogging. Both river and well are fully penetrating the aquifer (Figure 15.1). All the water which is pumped is assumed to come from the surface water body in the final steady state condition.

[1] Managed aquifer recharge (MAR) combined with soil-aquifer treatment (SAT)

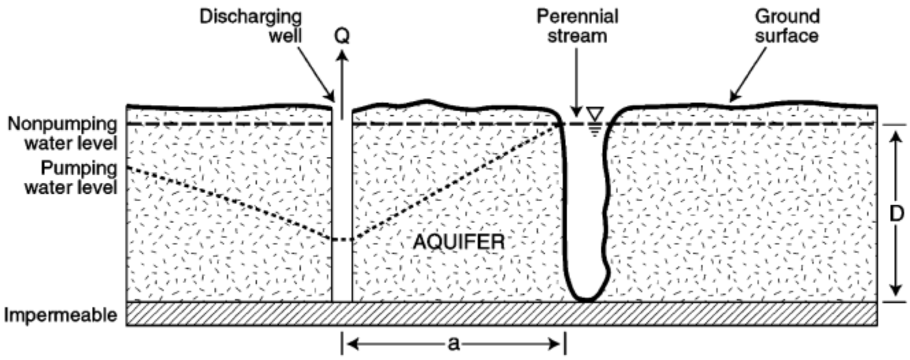


Figure 15.1 Cross sectional view of setting for analytical solution (Dillon et al. 2002)

The minimum travel time ($t_{min}$) of bank filtrate to the abstraction well is calculated according to:

$$t_{\min} = \frac{2\pi D n_e a^2}{3Q} \quad \text{(Dillon et al., 2002)} \qquad \text{eq. 15.1}$$

where:
$t_{min}$ = minimum travel time (d)
D = average saturated thickness (m)
$n_e$ = effective porosity of the aquifer (-)
a = distance of the well from the bank (m)
Q = the abstraction rate ($m^3$/d)

The calculated $t_{min}$ is calculated under steady-state conditions and underestimates travel time for transient conditions (Dillon et al., 2002). It overestimates the proportion of bankfiltrate in the abstraction well because rivers in nature are usually only partially penetrating the aquifer and analytical solution which assume fully penetration will overestimate the infiltration from the river (Chen, 2001).

The share of bankfiltrate in the abstraction well ( $q/Q$ ) of the above example changes in time and can be calculated according to a generalized solution developed by Glover and Balmer (1954) based on the equations developed by Theis (1941):

$$\frac{q}{Q} = erfc\left(\frac{a}{\sqrt{4\alpha t}}\right) \quad \text{(Glover and Balmer, 1954)} \qquad \text{eq. 15.2}$$

where:
q = rate of induced infiltration from the river (bankfiltrate) ($m^3$/d)
α = aquifer diffusivity = transmissivity/storage coefficient, for unconfined conditions it can be calculated according to $KD/n_e$
t = time of pumping (d)
erfc = the complementary error function

As t increases to values when steady-state conditions can be assumed (approx.one year in the test cases) the solution approaches close to 1 (equal to 100 %).

Eq.15.1 and eq.15.2 were used for a first assessment and the critical parameters travel times are underestimated and share of bankfiltrate overestimated which it makes it a conservative approach, since both parameters are assumed to be better in reality in terms of purification capacity: Higher travel times will lead to a longer contact time with aquifer material and biofilms and lower bankfiltrate share means stronger dilution so that the overall system performance will be higher than estimated by the analytical model.

In addition to the above example, Hunt (1999) derived a solution which takes into account the situation in which a river only penetrates partially into the aquifer system, the river has a semi pervious sediment layer and the river is not necessary located at the boundary of the model. The system Hunt describes gives a non-stationary solution for a phreatic aquifer system in which a well extracts groundwater and this extraction causes inflow from the river into the groundwater (Figure 15.2). Hunt presents a solution for the drawdown of the groundwater, both in space and in time, as well as a solution for the discharges from the river into the groundwater. The equations he provides for these two solutions are given in eq.15.3 (drawdown ω (x, y, t) [m]) and eq.15.4 (ratio between the infiltration and the extraction rate $\Delta Q/Q_w$):

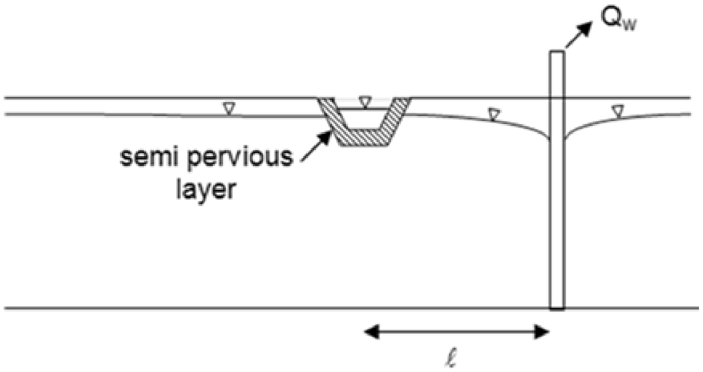


Figure 15.2 The problem considered by Hunt (1999)

$$\omega(x,y,t)=\frac{Q_w}{4\pi T}\left\{W\left[\frac{(\ell-x)^2+y^2}{4Tt/S}\right]-\int_0^\infty e^{-\theta}W\left[\frac{(\ell+|x|+2T\theta/\lambda)^2+y^2}{4Tt/S}\right]d\theta\right\} \qquad \text{eq. 15.3}$$

where λ [m/d] is a constant of proportionality between the seepage flow rate per unit distance (in the y direction) through the streambed and the difference between river and groundwater levels at x = 0 (location of the river). W is the Theis well function (for example in Barry, 2000) and S [-] stands for the porosity and T [m²/d] for the transmissivity of the aquifer.

$$\frac{\Delta Q}{Q_w}=erfc\left(\sqrt{\frac{S\ell^2}{4Tt}}\right)-\exp\left(\frac{\lambda^2 t}{4ST}+\frac{\lambda\ell}{2T}\right)erfc\left(\sqrt{\frac{\lambda^2 t}{4ST}}+\sqrt{\frac{S\ell^2}{4Tt}}\right) \qquad \text{eq. 15.4}$$

Unfortunately, also Hunt assumes that the water level in the river does not change in time and as a result of the infiltration into the groundwater.

### 15.2.2 Surface spreading methods

Surface spreading methods consist of NT-Systems such as infiltration ponds, soil-aquifer treatment or surface flooding. During surface spreading the source water such as river water or surface run-off is collected and diverted to the area of recharge. Recharge takes place by percolation through the unsaturated zone to the groundwater table. In India, surface spreading is often operated without managed abstraction and the artificially recharged groundwater is consumed by the local community mostly for agricultural purposes (Gale et al., 2006). Important hydraulic parameters during surface spreading are the infiltration rate of the system or the development of the groundwater mound beneath the recharge area. Infiltration rates during surface spreading are subject to large temporal and spatial variations. This is caused by geological heterogeneities but also by operational needs such as dry/wet cycles. The hydraulic capacity of an infiltration system is therefore best expressed in long-term infiltration rates or hydraulic loading rates (Bouwer, 2002).

The Green-Ampt equation was developed to calculate the infiltration rate ($V_i$) from a ponded surface (e.g. infiltration basin) into a deep homogeneous porous media with uniform initial water content. The Green-Ampt model has been found to apply best to infiltration into initially dry, coarse textured media which exhibit a sharp wetting front as shown in Figure 15.3.

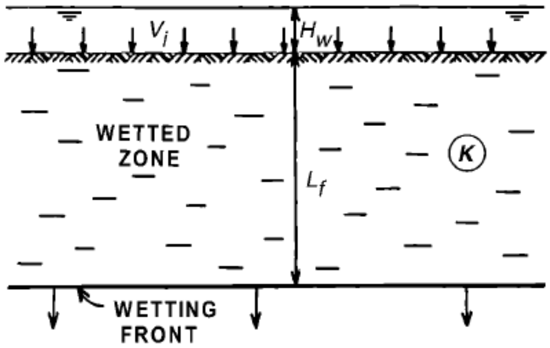


Figure 15.3 Cross sectional view showing geometry and explanation for Green Ampt model (Bouwer, 2002).

The Green-Ampt solution was developed in 1911 and is based on Darcy´s law:

$$V_i = K\left(\frac{H_W + L_f - h_{we}}{L_f}\right) \quad \text{(Green and Ampt, 1911)} \qquad \text{eq. 15.5}$$

where:
$V_i$ = the infiltration rate or hydraulic loading rate (m/s),
K = hydraulic conductivity (m/s), K is less than K saturated
$H_w$ = depth of water in the pond or infiltration facility (m)
$L_f$ = depth of the wetting front below the bottom of the pond (m)
$h_{we}$ = capillary suction or negative pressure head at the wetting front (m). Approximately equal to the air entry pressure or bubbling pressure

Unsaturated K values are lower than saturated K ($K_{sat}$) values, because of the entrapped air. Bouwer (1978) refers to factors 0.5 × $K_{sat}$ for sandy soils and 0.25 × $K_{sat}$ for clays. Values of $h_{we}$ describe the suction at the wetting front (negative pressure head). Typical values of $h_{we}$ along with other important hydraulic properties for various soils can be found in Table 15.2.

Table 15.2 Hydraulic properties for various soils (Rawls *et al.*, 1983)

| Texture | Porosity n | Residual Porosity $\Theta_r$ | Effective Porosity $\Theta_e$ | Suction Head ψ (cm) | Conductivity K (cm/hr) |
|---|---|---|---|---|---|
| Sand | 0.437 | 0.020 | 0.417 | -4.95 | 11.78 |
| Loamy Sand | 0.437 | 0.036 | 0.401 | -6.13 | 2.99 |
| Sandy Loam | 0.453 | 0.041 | 0.412 | -11.01 | 1.09 |
| Loam | 0.463 | 0.029 | 0.434 | -8.89 | 0.34 |
| Silt Loam | 0.501 | 0.015 | 0.486 | -16.68 | 0.65 |
| Sandy Clay Loam | 0.398 | 0.068 | 0.330 | -21.85 | 0.15 |
| Clay Loam | 0.464 | 0.155 | 0.309 | -20.88 | 0.10 |
| Silty Clay Loam | 0.471 | 0.039 | 0.432 | -27.30 | 0.10 |
| Sandy Clay | 0.430 | 0.109 | 0.321 | -23.90 | 0.06 |
| Silty Clay | 0.470 | 0.047 | 0.423 | -29.22 | 0.05 |
| Clay | 0.475 | 0.090 | 0.385 | -31.63 | 0.03 |

Operators of infiltration ponds may also be interested in the height of groundwater mound which is created by a MAR facility. This is important to e.g. control water quality where it might be necessary to ensure a minimum thickness of unsaturated zone (Figure 15.4).

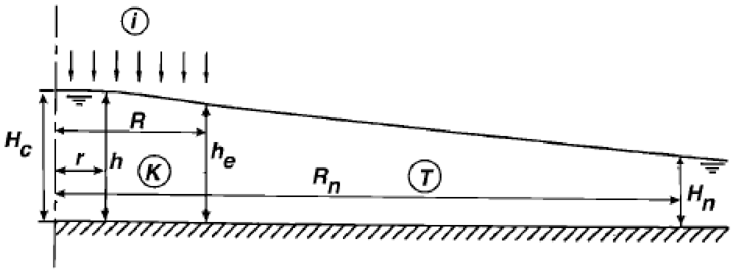


Figure 15.4 Cross sectional view illustrating geometry and parameters used for calculation of groundwater mound (Bouwer, 2002).

(Bouwer et al., 1999) developed an analytical solution for round or square area type of recharge ponds, where the groundwater flow is radially away from the point of recharge. The steady-state height of the groundwater mound right below of the centre of the recharge pond is calculated according to:

$$H_c - H_n = \frac{V_i R^2}{4T}\left(1 + 2\ln\frac{R_n}{R}\right)$$ (Bouwer et al., 1999) eq. 15.6

where:
R = radius or equivalent radius of the recharge area (m)
$R_n$ = distance from the centre of the infiltration pond to the control area (m)
$H_c$ = height of groundwater mound in the centre of recharge area (m)
$H_n$ = height of water table in control area (m)
$V_i$ = average infiltration rate (total recharge divided by total area) (m/s)
T = transmissivity of the aquifer ($m^2$/s)

Control area is here defined as the area where the groundwater table is stable. The value of transmissivity in eq.15.5 must reflect the average transmissivity of the aquifer at the steady-state stage of the mound.

In numerical groundwater models, infiltration rates or exchange fluxes (q) between the ground- and surface water are usually calculated by introducing a transfer or leakage coefficient $\phi_h$ [$d^{-1}$]:

$$q = \phi_h (h_{ref} - h_{gw})$$ eq. 15.7

In which:
q = Darcy flux [$md^{-1}$] of fluid (positive from river to groundwater) and
$h_{ref}$, $h_{gw}$ = heads [m] in the river and groundwater respectively.

Assuming a simple surface spreading infiltration system represented by an initially fully rectangular canal with no other in- or outflow than the fluxes to or from the connected groundwater, the conservation of mass equation for such a unit can be written as follows:

$$\frac{\delta h_{ref}}{\delta t} = -\frac{Q_o}{A_r}$$ eq. 15.8

In which t [d] represents time and $A_r$ represents the cross section area of the canal [m²].

It is assumed that the groundwater is initially way below the bottom of the canal and that even after the canal has been drained completely, the groundwater still has no direct contact to the surface water. Substituting eq.15.7 in eq.15.8, taking into account that the width of the canal ($B_r$ [m]) is water level independent, infiltration takes place along the complete wetted perimeter of the canal (bottom and lateral infiltration) and $h_{gw}$ is limited to the bottom of the canal (constraining the infiltration rate in contrast to the above equation of Green and Amt), the time $T_e$ [d] to empty the canal from a water depth $wd_r1$ to a depth $wd_r2$ can be calculated by:

$$T_e = -\left[ \frac{B_r(\ln(wd_r) - \ln(B_r\phi_h + 2\phi_h wd_r))}{B_r\phi_h} \right]_{wd_{r1}}^{wd_{r2}} \qquad \text{eq. 15.9}$$

In case that the canal is triangular with a constant slope of the banks 1/η [-], the solution of $T_e$ can be expressed in a slightly more convenient way:

$$T_e = -\left[ \frac{\eta}{\phi_h \sqrt{(1+\eta^2)}} \ln(wd_r) \right]_{wd_{r1}}^{wd_{r2}} \qquad \text{eq. 15.10}$$

Describing also non stationary infiltration processes with varying water levels in the infiltration unit, these equations can be used to verify the behaviour of numerical groundwater models in describing the infiltration processes related to surface spreading MAR structures.

## 15.3 USE OF NUMERICAL MODELS FOR MAR SYSTEMS

In contrast to analytical solutions, numerical models can be adapted to a wide range of site specific conditions and problem statements. A large number of numerical models have been used to analyse various MAR systems covering basic hydraulic problems (Neumann et al., 2004) to complex temperature-dependant redox zonation and associated contaminant removal (Henzler et al., 2014;  Greskowiak et al., 2006). Most numerical models, no matter if based on finite elements or finite differences, are generally capable to simulate three-dimensional advective, diffusive and dispersive flow and solute transport.

In the following we provide a brief description of commonly used codes:

**MODFLOW** (Harbaugh, 2005) is a modular finite-difference flow model developed since the 1980´s. The code is public domain free software, but there are several commercial and non-commercial graphical user interfaces available. MODFLOW can be combined with several packages to simulate fate and transport (MT3DMS, Zheng and Wang (1999)), density driven flow (SEAWAT, Langevin et al. (2007)), and reactive multicomponent transport including PHREEQC (PHT3D, Prommer (2006)). It also provides packages for parameter estimation and uncertainty analysis (e.g. PEST, Welter et al. (2012)). With this set of packages the MODFLOW is a very powerful, robust and flexible modelling tool.

**FEFLOW** (Diersch, 2014) provides an advanced 3D graphically based modelling environment for performing complex groundwater flow, contaminant transport, and heat transport modelling. Regarding contaminant transport, both multiple species as well as kinetic reactions between the species can be modelled. Both saturated and unsaturated flow regimes can be described. It uses a Galerkin-based finite element numerical analysis approach with a selection of different numerical solvers and tools for controlling and optimizing the solution process. FEFLOW is a completely integrated system from simulation engine to graphical user interface including a public programming interface for user code. By this interface also integrated surface water – groundwater interactions can be modelled, for example using the integrated coupling to the surface water modelling engine MIKE11 (DHI, 2014a). Its scope of application ranges from simple local-scale to complex large-scale simulations. Special features like biodegradation, density dependent flow and random walk analyses enable the use of FEFLOW also in very specific cases. With FEPEST FEFLOW offers a powerful tool for auto calibration and parameter uncertainty analyses.

**MARTH**E v7.0 is a complete numerical hydrosystem code designed for hydrodynamic and hydrodispersive modelling of groundwater flow and mass and energy transfer in porous media (Thiéry, 1990; 1993; 1995; 2010a). This code allows the three-dimensional  simulation of flow and transport under saturated conditions and in the vadose zone using a finite volume method for hydraulic calculations (Thiéry, 2010b) and integrates a hydroclimatic balance (precipitation, evapotranspiration, runoff, infiltration, recharge) using the GARDENIA scheme (Thiéry, 2010c) as well as density driven flow. Interaction between surface, subsurface and groundwater is implemented in MARTHE v7.0 and has been applied to river basins to predict the influence of climatological changes on river flows and to anticipate floods (Thiéry and Amraoui, 2001; Habets et al., 2010; Thiéry, 2010d). Furthermore, MARTHE was applied in the context of MAR systems (Gaus et al., 2007; Klopmann et al., 2012).

However in the release of MARTHE used for earlier studies, surface water is connected to the river network and is allowed to flow out of the hydrosystem, but it cannot be stored in topographic depressions and re-infiltrate to the aquifer. For this reason, a specific module (LAC) has been implemented and tested on the Saph Pani project study site of Maheshwaram watershed (Picot *et al.*, 2013)

**MIKE SHE** (DHI, 2014b) is a fully distributed, process-based hydrological model and includes process models for evapotranspiration, overland flow, unsaturated flow, groundwater flow, and channel flow and their interactions including solute transport. Each of this process is described by its governing equation or by a simpler conceptual representation and a user can tailor the model structure by choosing processes to be included and the solving methods. MIKE SHE is a comprehensive catchment modelling framework with applications ranging from aquifer management and remediation to wetland management, flooding and flood forecasting. MIKE SHE is dynamically coupled to MIKE11, which is a one-dimensional surface water model that simulates fully dynamic channel flows and is therefore able to represent river processes and river management. While the process-based approach allows different model structures to be applied within the same modelling framework, in the original concept the different flow processes are described by the governing partial differential equations and these are then solved by discrete numerical approximations in space and time using finite differences.

### 15.3.1 Calculating mixing proportions by water balance modelling

Water balance modelling or water budget calculation is the easiest procedure to analyse e.g. shares between the different water sources. Typical problem statements in MAR systems comprise characterisation of sources of water which is abstracted in a well. By specifying sub-regions in the model domain such as boundary conditions the flow between each of the adjacent zones is calculated. This method may be applied in two- or three-dimensions using any groundwater flow model that includes water balance calculations (for example MODFLOW and FEFLOW). In case of more complex flow patterns, e.g. multiple wells and transient boundary conditions, it may be necessary to assess the water budget by more demanding particle tracking or solute transport approaches.

### 15.3.2 Calculating mixing proportions and travel times by particle tracking

Particle tracking is used to trace flow lines by simulating the movement of imaginary particles in a given flow field. When advective transport is the dominant process controlling solute mobility, particle tracking in groundwater flow models can be an alternative to more demanding solute transport models. Particle-tracking using model packages such as MODPATH (Pollock, 1994) or FEFLOW (DHI-WASY, 2013) provide a tool to calculate e.g. travel time of water between two points. In MAR and RBF systems, particle tracking was used e.g. by Abdel-Fattah et al. (2008) to investigate travel time of bank filtrate, riverbed infiltration zone length and well capture zone. During particle tracking, it is assumed that solute movement is controlled entirely by advection and that density-dependent flow, dispersion and diffusion are negligible. Random-Walk Particle Tracks, however, incorporate diffusion and dispersion, bringing field-line analysis a large step closer to the capabilities of a full advection-dispersion solution. As this option does not require the setup of a complete transport problem, pre-processing effort and computational cost remain comparably low. Random-Walk particle tracking is available in FEFLOW (DHI-WASY, 2013) and MARTHE (Thiéry, 1990; 1993; 1995; 2010a). Flow models can be transient or steady state and particle tracking can be calculated forward or backward in time.

Important performance parameters of MAR systems such as the share of bankfiltrate in the abstraction well can be approximated by backward modelling of particles released around a well screen. The angle between the uppermost and lowermost streamline gives an approximation of the share of bankfiltrate in the abstraction well (Chen, 2001). River water between these two lines flows to the abstraction well, while the water outside of these two lines does not flow to the well. The angle is then measured by visual inspection close to the well screen and compared to a complete full circle. This method will give only a rough approximation for simple models (e.g. single layer models). It has to be taken into account that in heterogeneous, multi-layer models particles for each layer must be weighted according to the layer specific flow. This leads to laborious calculation and other methods may apply better. Travel times of bank filtrate to the abstraction well can be approximated by e.g. end point calculations in MODPATH. Travel time of particles from the abstraction well to their point of termination is calculated backward in time. Termination points are model boundaries e.g. the river.

### 15.3.3 Calculating mixing proportions and travel times by solute transport

In transient models, conservative solute transport can be used to approximate travel times and mixing proportions in MAR systems. Among other transport options, conservative transport can be simulated with MT3DMS (Zheng and Wang, 1999). In FEFLOW a new feature called Groundwater Age offers a new method to calculate groundwater ages, mean lifetime expectancies and mean exit probabilities within a model domain also for non-conservative transient transport processes involving 1st order decay and linear retardation processes (DHI-WASY, 2013), providing value information to estimate risk vulnerabilities or evaluate outlet capture zones and the origin of water, also under density dependent dominated conditions.

In MAR systems the point of recharge (e.g. the river or lake during bank filtration, infiltration pond or injection well) is assigned to species concentration of 1, while the rest of the model domain is assigned to species concentration of 0. The resulting breakthrough curve is shown schematically in Figure 15.5.

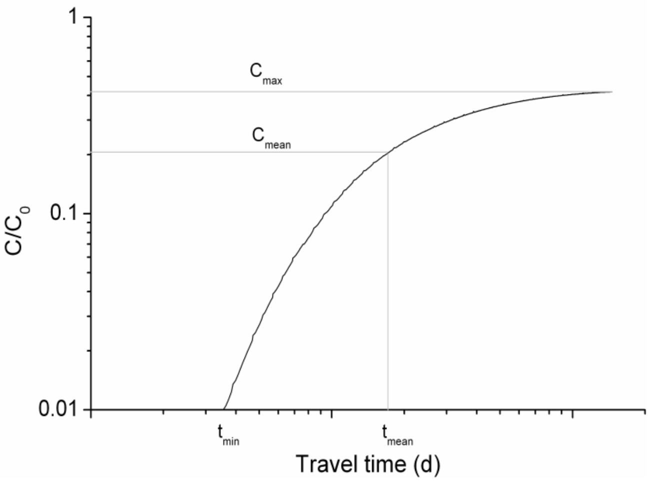


Figure 15.5 Log-log scale of an exemplary breakthrough curve (BTC) of an ideal tracer and calculation of mean, minimum travel time and bankfiltrate share in the abstraction well.

The proportion or share of bankfiltrate in the abstraction well water is defined as the maximum concentration ($C_{max}$) during late, quasi steady state conditions. The mean travel time (or dominant travel time) of e.g. bankfiltrate to the abstraction well can be calculated by differentiating the cumulative breakthrough curve and retrieving the time at which its mean value is reached. Minimum travel time can for example be defined when one percent ($C/C_0 = 0.01$) of the maximum concentration is reached.

## 15.4 COMPARISON OF ANALYTICAL AND NUMERICAL SOLUTIONS

### 15.4.1 Bank filtration

#### *15.4.1.1 Model descriptions*

The BF model domain and associated boundary conditions are illustrated in Figure 15.6.

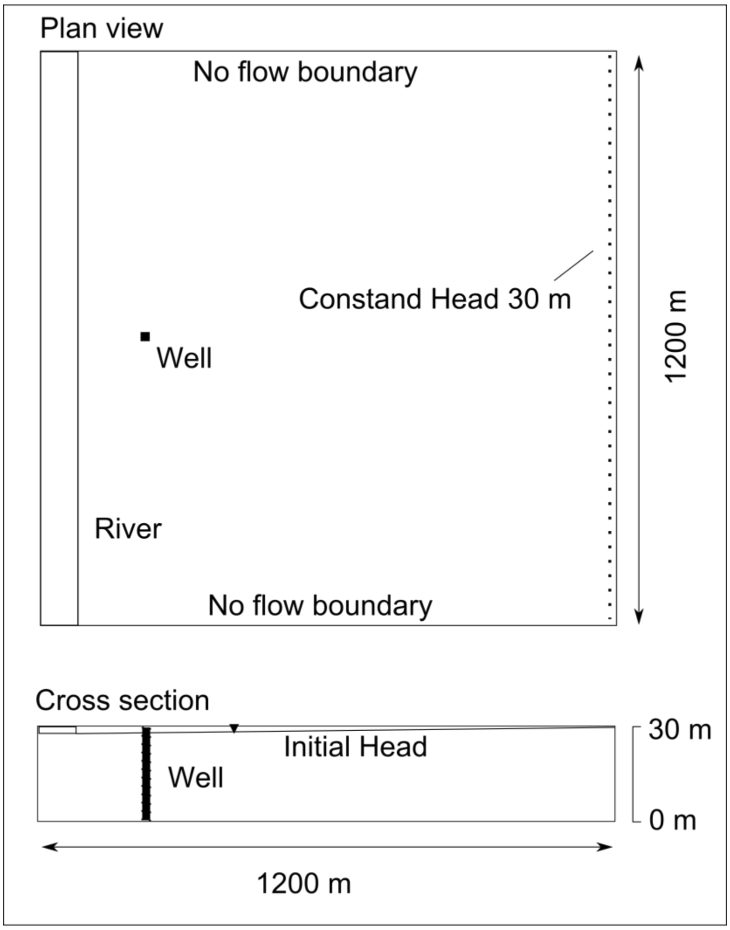


Figure 15.6 Model domain and boundary conditions for the Bank filtration scenarios.

The cell size in the model domain is 20 × 20 m without any grid refinement. The abstraction well is fully penetrating. River stage is kept constant 1 m below the constant head boundary and vertical hydraulic conductivity is 1/10 of horizontal hydraulic conductivity. Two exemplary simple model scenarios have been used to compare analytical and numerical solutions for BF systems. Differences in the two BF scenarios are shown in Table 15.3 These scenarios represent two arbitrary, basic and simple capture zone characteristics, which can be approximated by using the presented analytical solutions. It was not intended to analyse the differences between the two scenarios in this study.

Table 15.3 Differences in model parameter between scenarios.

| Parameter | Scenario 1 | Scenario 2 |
|---|---|---|
| Hydraulic conductivity (m/s) | $1\times10^{-4}$ | $1\times10^{-3}$ |
| Effective porosity (-) | 0.15 | 0.25 |
| Pumping rate ($m^3$/d) | 1000 | 2000 |
| Riverbed conductance ($m^2$/s) | 0.04 | 0.4 |
| Distance of pumping well from river bank (m) | 60, 80, 100 | 60, 80, 100 |

The riverbed conductance is in FEFLOW applied as transfer rates $T_{in}$ and $T_{out}$ , which are calculated as follows:

$T_{in} = T_{out}$ = Riverbed conductance (m²/s) / Element area (m²) = 0.04 / 20 = $2\times10^{-4}$ (1/s)

In each of the scenarios the riverbed conductance was adjusted to the hydraulic conductivity of the aquifer in order to ignore any riverbed clogging effects. The river is simulated in MODFLOW as well as FEFLOW a head dependent flux boundary. The head can be a temporal variable boundary, but is kept constant during all model scenarios. Water is flowing from the river to the groundwater when the head in the nearby cell is lower than the river stage. The flux between river and aquifer ($q_{riv}$) is calculated with riverbed conductance ($C_{riv}$) and the head difference between the river stage and the adjacent groundwater head (Δh):

$$q_{riv} = C_{riv} \times \Delta h \quad \text{eq. 15.11}$$

Clogging of the riverbed is expressed by riverbed conductance ($C_{riv}$) according to:

$$C_{riv} = \frac{K \times L \times W}{M} \quad \text{eq. 15.12}$$

where:
$C_{riv}$ = riverbed conductance ($m^2$/s)
K = hydraulic conductivity of riverbed (m/s)
L = river length (m) in cell
W = river width (m) in cell
M = thickness of clogging layer (m)

Both equations are solved individually for each model time step at each model grid cell, which is identified as a river cell. This approach enables consideration of temporally and spatially variable extent of the interactions between the groundwater and the surface water.

The BF model scenarios were first run in steady-state mode to calculate the water budget and particle tracking. In a last step the flow model was coupled to a MT3DMS solute transport model using a third-order total-variation-diminishing (TVD) scheme for solving the advection term in transient mode. The TVD scheme is mass conservative and does not produce excessive numerical dispersion or artificial oscillation (Zheng and Wang, 1999). In FEFLOW, the particle tracking analyses taking into account dispersion as well as diffusion was performed using classic mass transport simulations as well as the calculation of lifetime expectancies with the Groundwater Age feature.

In a second example, the shown benchmark of Hunt (1999) was simulated using a coupled FEFLOW and MIKE11 setup using the plug-in IfmMIKE11 (Monninkhoff et al., 2009). The model setup is shown in the next Figure 15.7.

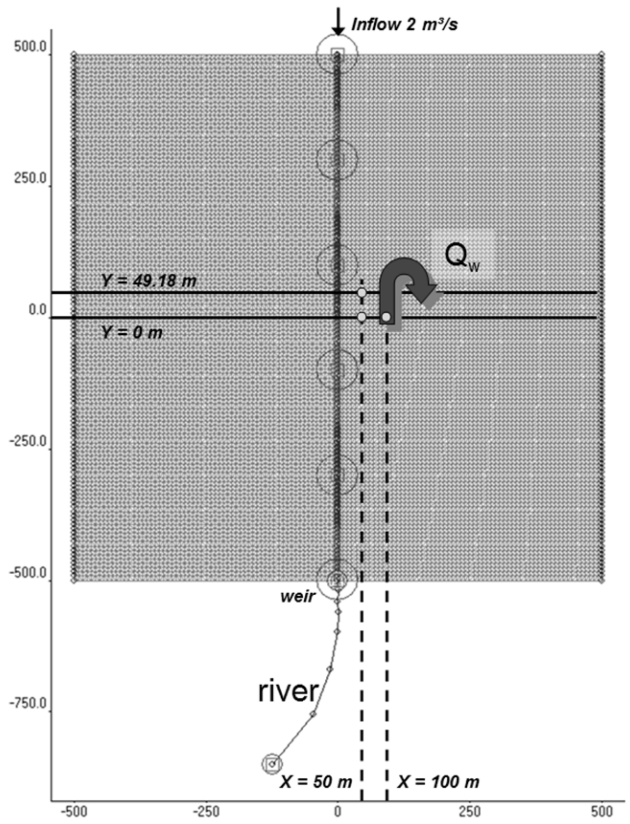


Figure 15.7 The problem considered by Hunt (1999) described by coupled groundwater and surface water system.

For this example, the MIKE11 model had to be built in a way that the river water level would not or hardly change as a result of the infiltration. For that purpose, a rather wide (200 m) and very smooth river bed (Kst = 80 $m^{1/3}/s$) has been defined. Furthermore, a constant upstream inflow rate of 2 m³/s and a weir level of approximately 10 m at the downstream end of the coupled region ensured that the water level in the river along the FEFLOW model was infiltration rate independent as well as constant along the river itself. MIKE SHE has been verified using the Hunt benchmark as well (Illangasekare, 2001). For that verification $\lambda$ in eq.15.4 was set equal to $1*10^{-5}$ m/s. With a 200 m wide rectangular channel with a constant water depth of 2 m (the river bed was set at 8 m) an identical value for $\lambda$ can be achieved using a global transfer coefficient equal $\phi_h$ of $42.4 * 10^{-4}$ $d^{-1}$. With furthermore a porosity of 0.2, a transmissivity of 0.001 m²/s, a distance $\ell$ between river and well of 100 m and an extraction rate $Q_w$ equal to 10 000 m³/a exactly the same conditions could be tested with IfmMIKE11.

### 15.4.1.2 Results

Differences of calculated minimum travel time by analytical and numerical approaches were found for the two BF scenarios (Table 15.4). These can be partly explained by the differences in the numerical approximation and discretization (MODFLOW and FEFLOW) and by the differences in conceptualization (in the numerical models the river was not fully penetrating the aquifer and a certain clogging effect has been taken into account). Furthermore, it has to be noted that the calculation methods for deriving minimum travel times using particle tracking and solute transport are fundamentally different. In case of solute transport an (arbitrary) minimum concentration at the well breakthrough curve is determining the minimum travel time, taking into account dispersion, diffusion and mixing processes within the well capture zone. Using particle tracking, the minimum travel time among different starting points within the capture zone is derived. With the simple model setup at hand, the minimum travel time path always coincides with the shortest distance between the well and the river.

Table 15.4 Minimum travel time (d) calculated by different analytical and numerical approaches (longitudinal dispersivity = 5 m, transversal dispersivity = 0.5 m for solute transport).

| | Well distance from riverbank (m) | Dillon et al. (2002) | Particle tracking (advection only) | | solute transport (advection + dispersion) | |
|---|---|---|---|---|---|---|
| | | | MODFLOW | FEFLOW | MODFLOW (TDV) | FEFLOW[1] |
| Scenario 1 | 100 | 94 | 104 | 91 | 75 | 60 |
| | 80 | 60 | 67 | 57 | 39 | 43 |
| | 60 | 34 | 39 | 30 | 25 | 18 |
| Scenario 2 | 100 | 79 | 132 | 109 | 101 | 91 |
| | 80 | 50 | 79 | 62 | 64 | 50 |
| | 60 | 28 | 43 | 31 | 29 | 24 |

[1]: classic FEFLOW mass transport simulation

Despite these differences in model setup and calculation methods, the results clearly show that, compared to particle tracking results, on average the analytical solution from Dillon et al. (2002) produces lower travel times and confirms the conservative approach of the analytical solution, especially for scenario 2. If compared to the solute transport solutions the analytical solution yields substantial higher travel times and underestimates dispersive effects during subsurface passage.

In the next Table 15.5 a comparison between FEFLOW simulations using particle tracking, the classic mass transport simulation and using the Age Problem Class (Life Time Expectancy) is shown. The minimum travel time in the classic mass transport simulation is based on the explanation in Figure 15.5 and is therefore derived from the concentrations in the well. The evaluation of travel times using life time expectancies is based on mean travel times including dispersion and diffusion. From the resulting travel times, the minimum travel time between the river and the well is selected. Dispersion and diffusion in reality causes both longer and shorter travel times compared to mere advection based simulations. In this, the proportion of the longer travel times has the tendency to easily shift the mean towards older values, causing on average longer single travel times compared to pure advection based analyses (particle tracking). This results also into larger minimum travel times using the Age Problem Class. Like in the particle tracking, the minimum travel time is mostly located along the minimum distance between the river bank and the well. The results show that according to the choice of definition of minimum travel time, significant different results can be obtained. It is therefore important to determine which definition is most appropriate for the problem statement under consideration.

Table 15.5 Minimum travel time (d) calculated by different approaches in FEFLOW (longitudinal dispersivity = 5 m, transversal dispersivity = 0.5 m for solute transport simulations).

| Scenario | Well distance from riverbank (m) | minimum travel times (d) | | |
|---|---|---|---|---|
| | | Particle tracking (advection only) | Life Time Expectancy | classic mass transport |
| Scenario 1 | 100 | 91 | 103 | 60 |
| | 80 | 57 | 68 | 43 |
| | 60 | 30 | 41 | 18 |
| Scenario 2 | 100 | 109 | 120 | 91 |
| | 80 | 62 | 73 | 50 |
| | 60 | 31 | 41 | 24 |

In the next Figure 15.8 an exemplary result of Scenario 1 with a well distance of 100 m from the riverbank, calculated with FEFLOW, taking into account advection and dispersion using the Age Problem Class (Life Time Expectancy), is shown.

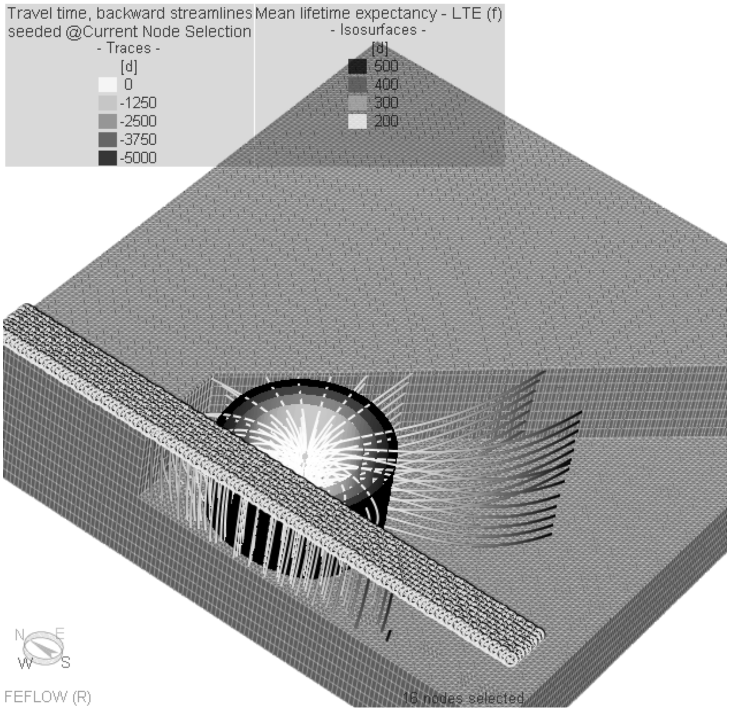


Figure 15.8 Exemplary result of a FEFLOW bank filtration simulation showing Lifetime Expectancies (advection and dispersion), Backward Streamlines (purely advection) and the river boundary nodes in slice 1. In FEFLOW, the Lifetime Expectancy (days) is defined as the time required for the water molecules to reach an outlet boundary of the aquifer.

Calculation of the share of bank filtrate in the abstraction well based on zone budget, particle tracking and solute transport come to almost identical results, whereas analytical solution by Glover and Balmer (1954) largely overestimates the share of bankfiltrate (%) compared to numerical solutions (Table 15.6).

Table 15.6 Percentage share of bankfiltrate calculated by analytical and numerical (advection only) approaches.

| | Well distance from river bank (m) | Glover and Balmer (1954)* | Zone Budget (water balance) | MODFLOW | FEFLOW |
|---|---|---|---|---|---|
| Scenario 1 | 100 | 86 | 61 | 61 | 62 |
| | 80 | 86 | 63 | 63 | 64 |
| | 60 | 86 | 66 | 66 | 67 |
| Scenario 2 | 100 | 94 | 17 | 17 | 17 |
| | 80 | 94 | 22 | 22 | 22 |
| | 60 | 94 | 27 | 27 | 28 |

*calculated with $t_{min}$ derived from eq.15.1

Minimum travel time and share of bank filtrate in the abstraction well can also be calculated based on breakthrough curves (BTC´s) of conservative solute transport calculated for the abstraction well for different well distances (like shown in Figure 15.9 for the MODFLOW simulations).

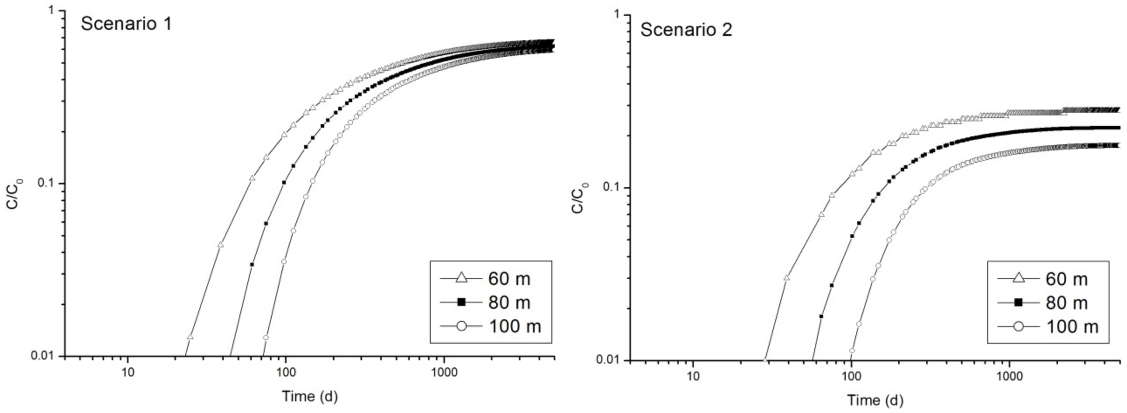


Figure 15.9 Log-log scale breakthrough curves of tracer indicating travel time (d) and share of bankfiltrate ($C/C_0$) calculated by solute transport for 60 m, 80 m and 100 m distance of the abstraction well from the river bank, MODFLOW simulations.

During these additional solute transport simulations longitudinal dispersivity was kept constant with 10 m, which was approximated by one tenth of the maximum flow distance from the river bank to the abstraction well according to (Adams and Gelhar, 1992) and transversal dispersivity was neglected.

In the next two figures the results for the benchmark of Hunt (1999) using IfmMIKE11 are shown. These figure show both the simulation results for the infiltration rate along the coupled river (Figure 15.10) as well as the drawdown along the line $y = 0$ and $y = 49.18$ m at day 23 of the simulation (Figure 15.11) are shown. Within the figures also the analytical solutions presented by Hunt are included. The analytical solutions and the IfmMIKE11 results are nearly identical and show that these kind of MAR applications can be simulated using a coupled setup of MIKE11 and FEFLOW. The same setup has been successfully benchmarked by an OPENMI based coupling between FEFLOW and MIKE SHE (Yamagata et al., 2012).

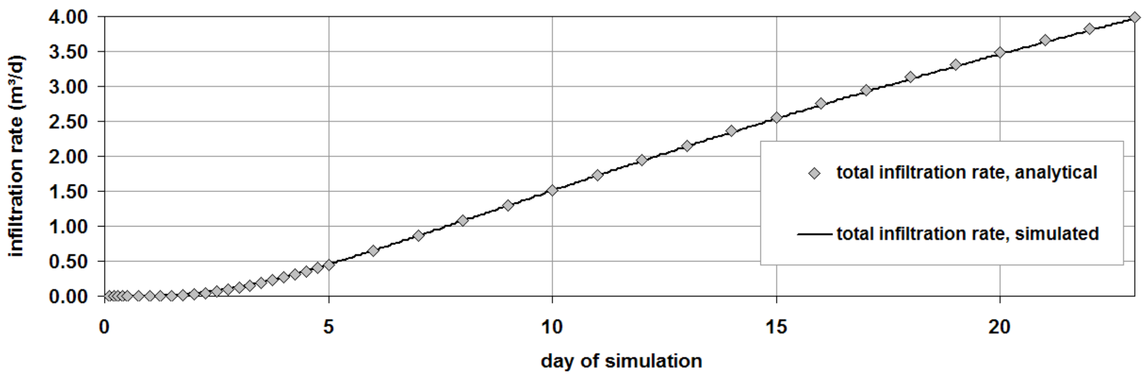


Figure 15.10 Comparison between the analytically solved and simulated total infiltration rates in time along the coupled river branch using FEFLOW and MIKE11.

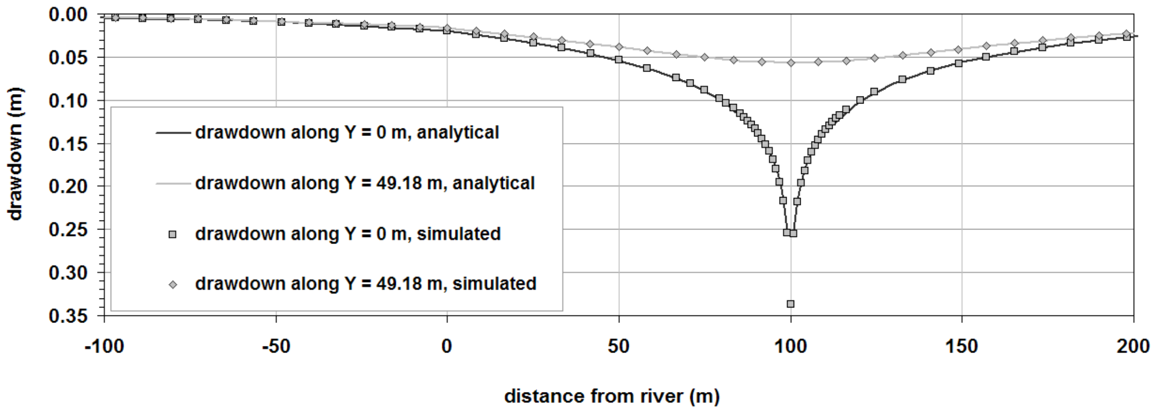


Figure 15.11 Comparison between the analytically solved and simulated drawdown along Y = 0 and Y = 49.18 m at day 23 of the simulation using FEFLOW and MIKE11.

## 15.3.2 Infiltration Pond

### *15.3.2.1 Model description*

A simple model has been constructed to compare analytical and numerical solutions for infiltration ponds (IP). The IP model domain and associated boundary conditions are illustrated in Figure 15.12.

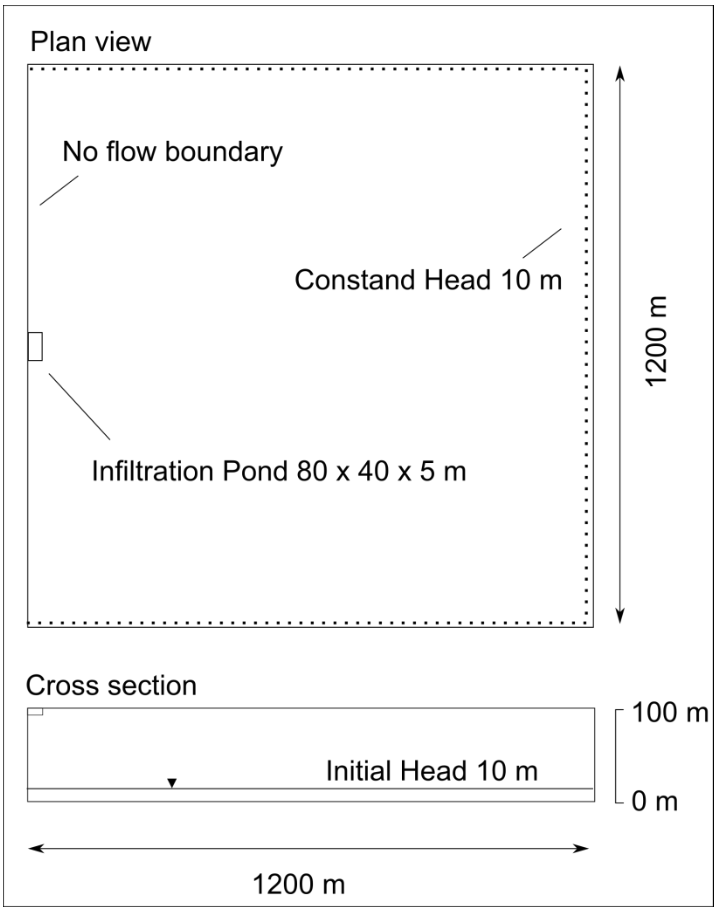


Figure 15.12 Model domain and boundary conditions for infiltration pond scenarios.

The infiltration pond is square type with 80 × 80 m area, but, in order to save computational time, only half of the infiltration pond is represented by the model. The development of the groundwater mound beneath the infiltration pond was simulated using the unsaturated-zone flow package coupled to MODFLOW 2005 under steady-state conditions. Unsaturated flow is calculated based on a simplified Richard´s equation (Niswonger et al., 2006). In FEFLOW, the fully integrated 3D Richard’s equation is used, applying a simplified Van Genuchten scheme (Table 15.7).

Table 15.7 Differences in model parameter for infiltration pond scenarios.

| Hydraulic loading rate (m/d) | Hydraulic conductivity (m/s) | Effective porosity (-) |
|---|---|---|
| 1 | $1\times10^{-4}$ | 0.15 |
| 2 | $1\times10^{-4}$ | 0.15 |
| 3 | $1\times10^{-4}$ | 0.15 |
| 1 | $1\times10^{-3}$ | 0.25 |
| 2 | $1\times10^{-3}$ | 0.25 |
| 3 | $1\times10^{-3}$ | 0.25 |

### *15.3.2.2 Results*

Analytical solution from Bouwer et al. (1999) and numerical solution for the development of groundwater mound beneath a recharge pond for different hydraulic loading rates and hydraulic conductivities of the aquifer are shown in Figure 15.13.

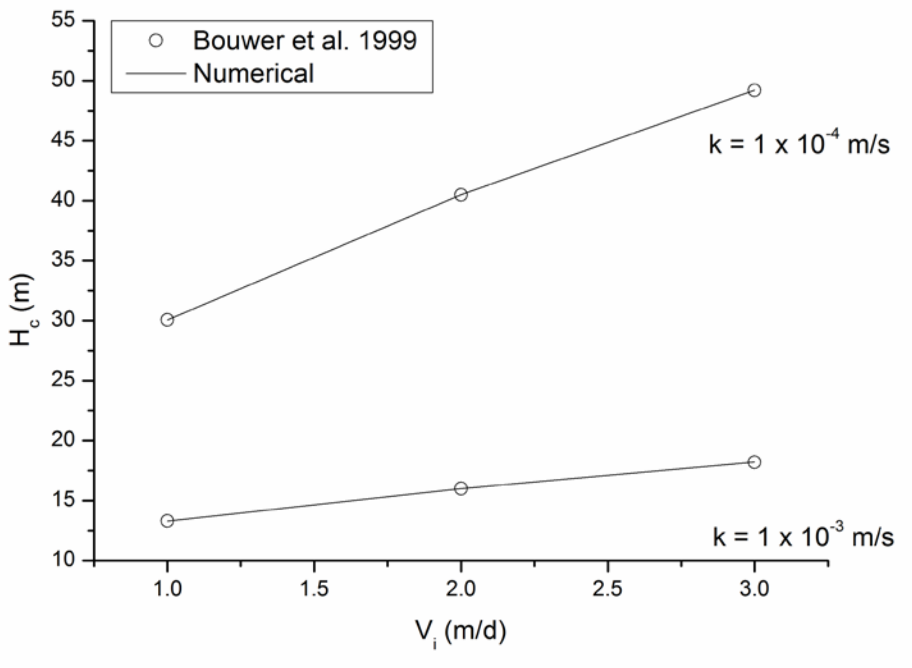


Figure 15.13 Comparison of numerical and analytical solution of mounding height ($H_c$) below infiltration ponds using MODFLOW.

Calculations with MODFLOW and FEFLOW show that a sandy aquifer with $k = 1\times10^{-4}$ m/s and a 10 m groundwater thickness at the control area creates a mounding height of approx. 50 m ($V_i$ = 3 m/d). Please note that these values are steady-state solutions and it may take years (up to 10 years) to develop equilibrium between percolation from the recharge structure and radial flow away from the recharge area. If the groundwater mounding exceeds the thickness of the unsaturated zone, the groundwater mound must be controlled e.g. by pumping or by reducing the long-term infiltration rate. In the next Figure 15.14 an example of the FEFLOW setup and a resulting groundwater mounding beneath the infiltration pond for this example is shown.

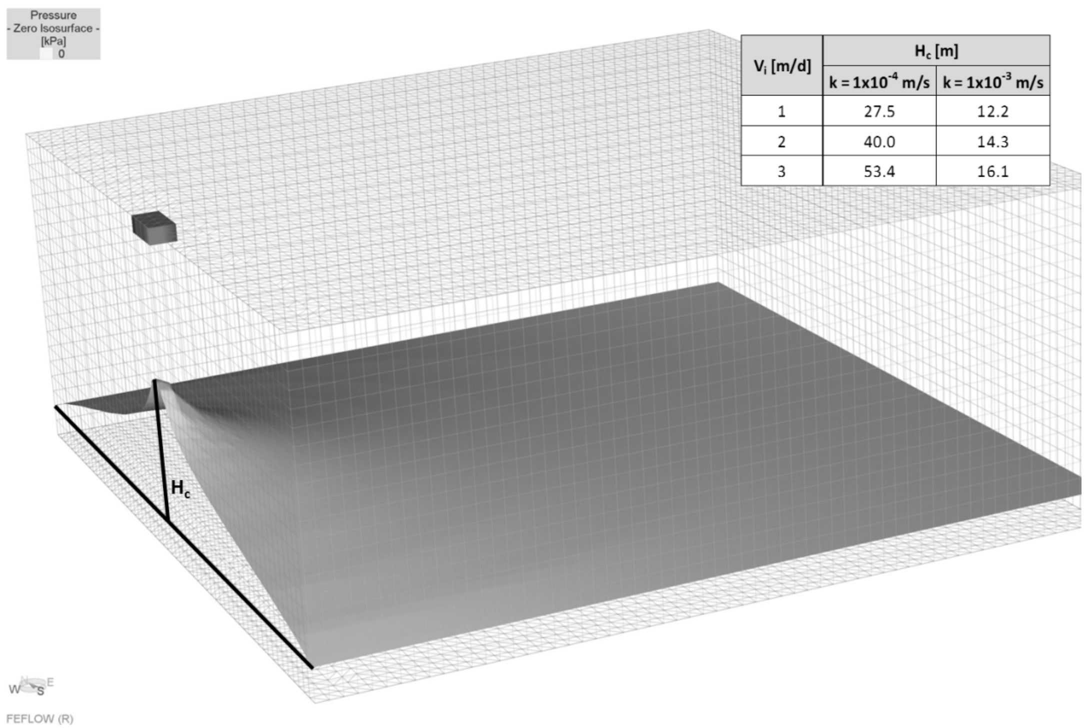


Figure 15.14 Results of FEFLOW simulations for different infiltration pond scenarios

The value of transmissivity (T) in eq.15.6 must reflect the average transmissivity of the aquifer during steady-state mound height. If T of the entire aquifer is used, eq.15.6 underestimates the mounding height and if T only for the initially saturated thickness is used, eq.15.6 overestimates the mound rise. Hence, the challenging part is to find a representative aquifer transmissivity. Bouwer et al. (1999) proposed to run pilot infiltration areas and to calculate T from that mound rise. The authors also highlight the importance of calibrated numerical models.

Eq.15.9 and eq.15.10, describing a transient infiltration process out of a rectangular and triangular canalized infiltration pond, have also been verified using a coupled numerical model using FEFLOW and MIKE11. The initial water depth of the river in both models is 10 m, the river width of the rectangular cross section is 25 m and the slope of the bank of the triangular cross section amounts 1:2. The transfer coefficient is equal to 0.1 $d^{-1}$ and the length of the river is 500 m. Using a maximum allowed time step of FEFLOW of 0.05 d, both the exchange discharges and the water depths of the latest version of IfmMIKE11 fit perfectly to the analytical solutions (Figure 15.15). As IfmMIKE11 is using an explicit coupling approach, also the influence of the maximum allowed time step of FEFLOW was tested. It was found that the lack of an iterative coupling causes discrepancies between the analytical and numerical solution if the FEFLOW time step exceeds a length of approximately 0.25 d. Nevertheless, these simulations show that also surface spreading MAR structures can be simulated using numerical coupled ground and surface water models. With the coupling of MIKE11 and FEFLOW even integrated multi-species non-conservative mass transport processes can be described (Monninkhoff et al., 2011).

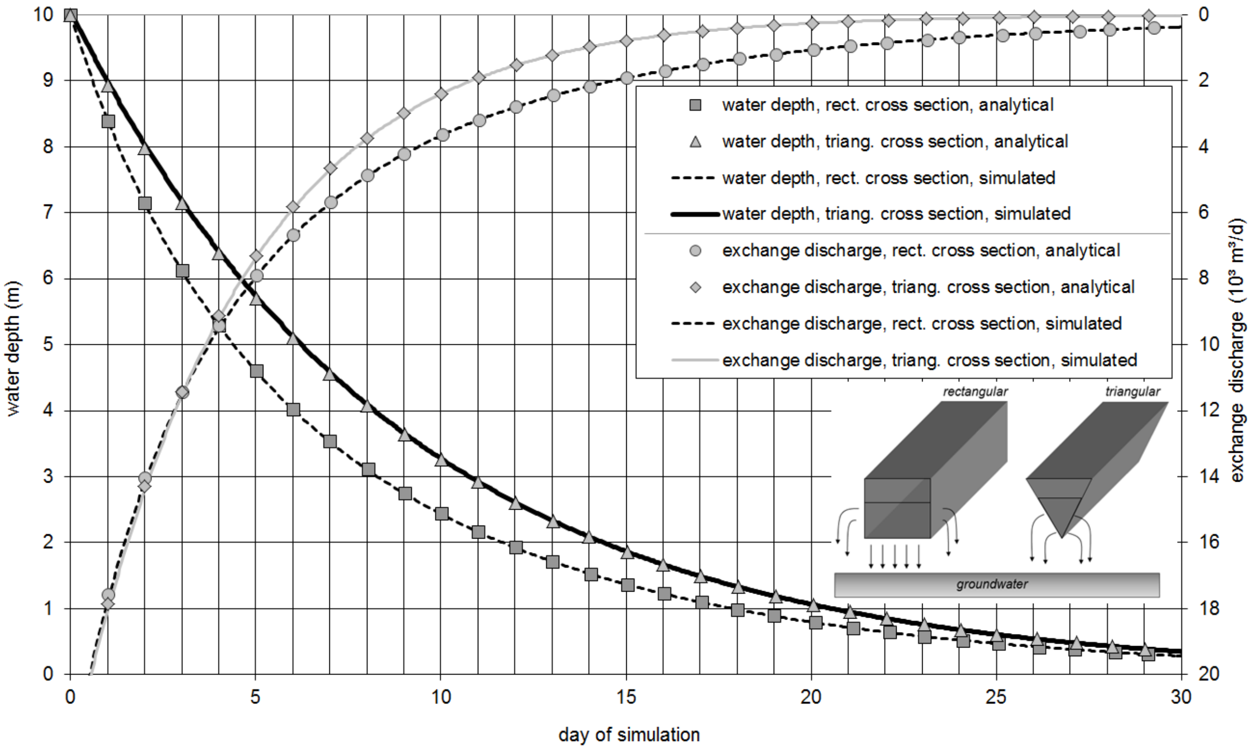


Figure 15.15 Comparison between the analytical solutions and simulated results for a rectangular as well as a triangular river cross section using a maximum FEFLOW time step of 0.05 d.

## 15.5 CONCLUSIONS

The examples shown in this chapter have shown that by using numerical modelling a variety of different NTS techniques (MAR, RBF) can be described. However, it was also shown that particle based, pure advective numerical calculations may underestimate minimum travel times of infiltrated source water to the abstraction well substantially. It is therefore recommended to use classical mass transport simulations to evaluate the minimum travel times in case of casualties in the surroundings of a well, taking into account both dispersion and diffusion driven processes. The analyses also showed, that, compared to mere advection driven processes, mean travel times can be longer in case dispersion and diffusion is taken into account. It is therefore important to determine the purpose of the travel time analyses, before detailed calculations are being started.

Besides the numerical tools presented in this chapter, the included analytical solutions may provide a first approximation of important performance parameters of NT-systems, but limitation and assumption have to be taken into account. Integrated analyses of surface water and groundwater interactions, especially at complex geometrical MAR structures, in diverse geological environments or at a regional catchment level can only be performed using numerical modelling tools.

In the following chapter, applications of different types of models on typical problems of NTS performance assessment and prediction are be shown.

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# Chapter 16

# Modelling of natural water treatment systems in India: learning from the Saph Pani case studies

*Ahmed S., Alazard M., Amerasinghe P., Bhola P., Boisson A., Elango L., Fahimuddin M., Feistel U., Fischer S., Ghosh N. C., Grischek T., Groeschke M., Grützmacher G., Hamann E., Indu S.N., Kloppmann W., Mahesh J., Mondal N.C., Monninkhoff B., Pettenati M., Picot-Colbeau G., Rajaveni S. P., Rao S., Sandhu C., Sarah S., Schneider M., Sklorz S., Thiéry D., Zabel A. K.*

## 16.1 MODELLING RIVER BANK FILTRATION (RBF)

Direct surface water abstraction from the river network for drinking water and irrigation bears important sanitary risks, microbial as well as chemical (pathogens, organic contaminants including emerging substance classes, inorganic major, minor or trace compounds). Those risks can be considerably reduced through indirect river water abstraction from wells and boreholes within the accompanying alluvial aquifer. Water pumped from such wells will contain a variable fraction of river water (up to 100%) and recharge coming from direct infiltration of rainwater or groundwater flow from the piedmont area. The river water fraction, called river bank filtrate, will be naturally purified through the passage through (1) the river bed, often rich in clay minerals and organic matter and (2) the alluvial aquifer. Main processes are (1) physical retention of suspended matter and microbes depending on the porosity of the filtering media, (2) chemical interaction of the migrating water with the aquifer material, notably sorption-desorption processes, ion exchange on clay minerals, dissolution-precipitation reactions and (3) microbiologically mediated processes mainly taking place in contact with biofilms on the aquifer material, in particular biodegradation of organic substances, transformation of organic matter, nitrification-denitrification processes. Those processes all have their proper kinetics, are therefore time-dependent, the purifying action of RBF will largely vary in function of the contact time of the migrating water with minerals and biofilms.

In the following sub-chapter we will outline some crucial aspects of RBF, including the determination of key parameters for purification capacity as the mixing proportion of river bank filtrated in the pumped alluvial groundwater and the travel time from the river to the pumping wells (Haridwar case study) as well as the simulation of transport of nitrogen species within the aquifer material, both on lab scale and field scale (Delhi case study).

### 16.1.1 RBF at River Ganga, Haridwar, Uttarakhand: groundwater flow modelling

*Where? Site description*

The importance of riverbank filtration (RBF) as a sustainable year-round natural treatment technology for the provision of drinking water to the permanent residents of Haridwar (>225,000) and the highly variable number of pilgrims (at least 50,000 daily, with up to 8.2 million on specific days such as *Kumbh Mela*; Gangwar and Joshi, 2004) in terms of being able to remove bacteriological indicators (total coliforms and E. coli) and to meet the dynamic drinking water demand has been highlighted in Chapters 1 and 2 (Wintgens et al., 2014; Sandhu et al., 2014).

As of 2013 at least two-thirds, or 59,000 to 67,000 $m^3$/day (Bartak et al., 2014), of the total raw water for drinking is abstracted from a total of 22 RBF wells with the remainder supplied by deep groundwater abstraction wells. The RBF wells are located on the west-bank of the Ganga River in the North, on Pant Dweep Island and on a narrow stretch of land between the Upper Ganga Canal (UGC) and the Ganga River in the southern part of the city (Figure 2.3). Thus, by virtue of their proximity to the Ganga River and UGC that form natural recharge boundaries, the RBF wells abstract around 40 to 90 % bank filtrate (Bartak et al., 2014). The portion of bank filtrate abstracted by the wells located on Pant Dweep Island and further South is greater than those to the North. This is due to their location in an area influenced by the naturally occurring flow of bank filtrate between the UGC and Ganga River due to the difference in hydraulic gradient. The naturally pre-treated RBF water is abstracted from the upper unconfined alluvial aquifer, which is in hydraulic contact with the Ganga River and UGC. The aquifer comprises fluvial deposits of poorly graded sand (0.0075-4.75 mm) beneath which lies a lower layer of silty sand (Dash et al., 2010). After abstraction, the water is disinfected with sodium hypochlorite at the well prior to being distributed to the consumer. Although these wells are relatively shallow (7-10 m deep), they have a large storage capacity on account of their large diameter (~10 m). The abstraction from these wells is highly variable (790 to 7530 $m^3$/day) and dependent on the season, with higher daily abstractions in monsoon as a result of longer operating hours due to a greater water demand, but also due to an increase in groundwater levels due to greater recharge from the surface water bodies, compared to the non-monsoon period.

The discharge of partially treated sewage and untreated storm water run-off in to the Ganga River (and UGC in Haridwar) and its upstream tributaries, as well as large-scale ritualistic bathing, are a source of thermotolerant coliforms (TTC; E. coli) present in the surface water. In this context, mean TTC numbers measured in the 22 RBF wells, calculated from long term water quality data (2005 - 2013), were 18 TTC/100 ml during monsoon and 1 TTC/100 ml during non-monsoon compared with $10^4$ - $10^5$ TTC/100 mL in the Ganga, including UGC (Bartak et al., 2014). This highlights the significant removal efficiency of 3.5 to 4.4 $\log_{10}$ of TTC by RBF (Dash et al., 2010; Bartak et al., 2014).

### *Why? Problems to be solved*

Despite the observed high TTC removal efficiency, TTC counts up to 93 MPN/100 ml were still observed in some RBF wells (Bartak et al., 2014). In this context it has been observed that some RBF wells which are very close to the surface water body show the presence of coliforms, such as wells 40, 42, 17, 21 and 24 that are located at a distance of 6 - 36 m from the UGC and its escape channel.

However, it is also evident that RBF wells which are comparatively further away from the Ganga or UGC (48 - 190 m) and are located in the area where the Ganga enters Haridwar in the northern part of the city (thus low impact of upstream pollution), such as wells 3, 4, 26, 1 and 31, also have comparably high coliform counts of <2 - 93 MPN/100 ml (Bartak et al., 2014). Normally one would not expect wells to be contaminated at such a relatively far distance. But considering that the lack of well head protection zones, social land use practices such as public bathing/washing at the well heads, well head housing, cattle in and around the RBF wells and unsanitary defecation practices near/around the wells were identified as high risks for the Haridwar RBF system (Bartak et al., 2014), it is conceivable that the origin of coliforms in some RBF wells is not the bank filtrate from the Ganga River but rather ambient landside groundwater.

Thus, the objective of the numerical groundwater flow modelling study for Haridwar was to identify the flow paths of the bank filtrate to the RBF wells and the travel time in order to analyse the source of contamination of the wells. Additionally, an overall understanding of the hydrogeological system in response to dynamic hydrological regime of the Ganga River (high monsoon and low non-monsoon water levels) was to be achieved. The degree of confidence in the numerically simulated portion of bank filtrate abstracted by the RBF wells was to be ascertained through comparisons with analytical calculations from mean conductivity (EC) and Oxygen-18 isotope values.

### *How? Tools and modelling strategy*

A three dimensional finite element two layered numerical groundwater flow model was set up in Visual MODFLOW (version 2011.1). The spatial extent of the entire model area is 5000 m (East-West) × 6000 m (North-South). However, the active model domain is assigned only to the area of the Ganga floodplain, with the remaining cells inactivated. The model domain was discretised to obtain a cell area of 12.5×12.5 to 100×100 m. The upper model layer coincides in general with the upper sandy layer of the aquifer (thickness from 0 to around 12 m below ground level (BGL)) and with the bottom of the partially penetrating RBF wells. The lower layer represents the silty sand (thickness 12 to around 21 m BGL). The Ganga River, UGC and its escape channel are represented by the river boundary condition. The hydraulic conductivity of the riverbed was determined from sieve analyses at various points and accordingly assigned to calculate the riverbed conductance. Reference day water level measurements were conducted on three specific days, each in August 2012, October 2012 and January 2013 to represent monsoon, post- and pre-monsoon conditions respectively and a relatively good calibration was achieved for each of these conditions. The actual hourly abstraction rates of the RBF wells for the monsoon and non-monsoon operating hours were normalised for a continuous operation (24 hour period) and assigned using the well boundary condition for the 22 RBF wells in the model at their respective locations. Subsequently the particle tracking tool was used in MODPATH to visualize the flow paths and travel times of water to the RBF wells (Figure 16.1). The zone budget method in MODFLOW was used to determine the portion of bank filtrate abstracted by the RBF wells.

*So what? Outcome, added value and perspectives*

The simulated flow paths of the water to the RBF wells (Figure 16.1, left) corroborate to the portion of bank filtrate abstracted by them that have been calculated from long-term mean electric conductivity values (Bartak et al., 2014) and Oxygen-18 isotope values (Saph Pani D1.3, 2013). The flow of water to the wells in Figure 16.1 indicate that the RBF wells located in the northern part of Haridwar also receive a considerable portion of groundwater in addition to some bank filtrate. For the wells 3, 4, 26 and 1, the portion of ambient landside groundwater is between 40 - 60 % and consequently the remainder being bank filtrate, with a greater portion of bank filtrate abstracted in monsoon due to an increase in the Ganga River levels and thereby the water line of the river moving closer to the bank and the wells. But as the area that lies in the groundwater catchment of the RBF wells is densely populated and substantially large, there is a greater risk of contamination from decentralised sewage disposal (septic tanks) and leaky wastewater drains. This would also explain the high TTC counts in in the RBF wells in relation to a relatively low portion of bank filtrate.

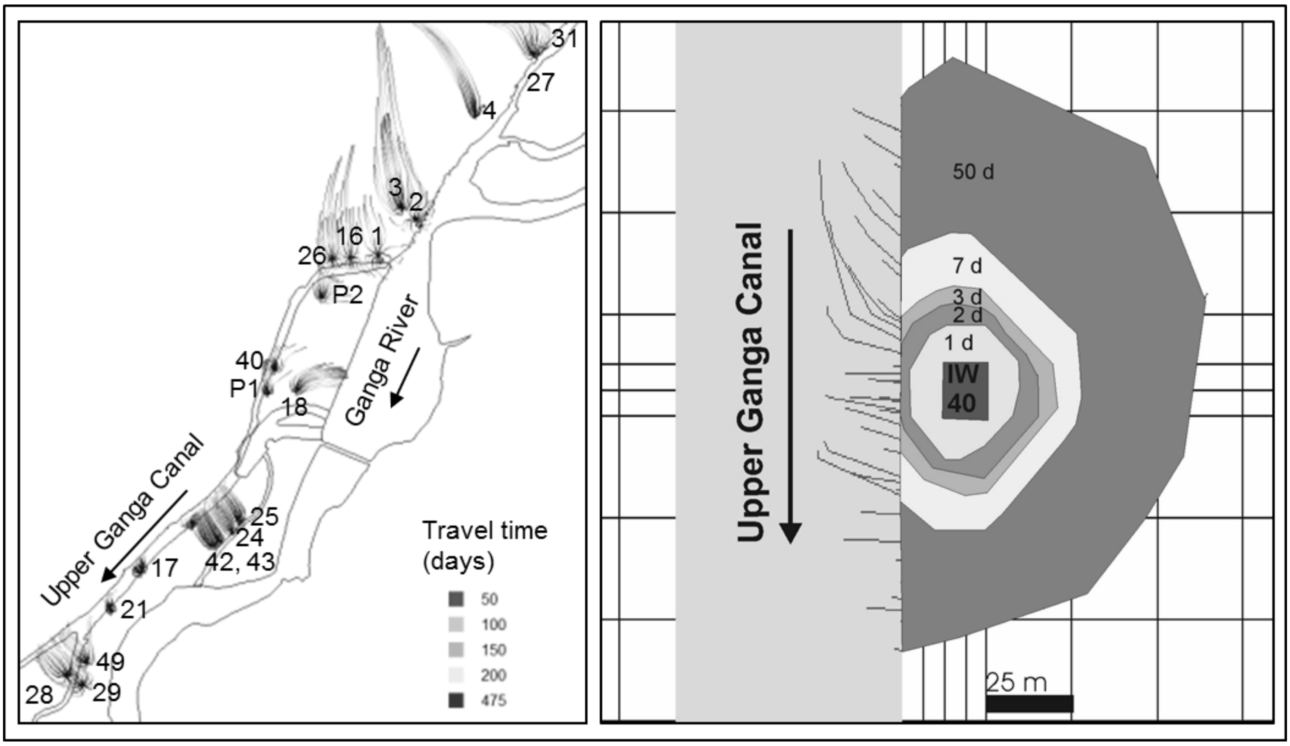


***Figure 16.1 Travel time and flow paths of bank filtrate for RBF system in Haridwar (left) and travel time of bank filtrate during monsoon for RBF well 40***

On Pant Dweep Island the shortest travel time of the bank filtrate to the wells 40 and P1, located only 15 m from the UGC, is around 3 days during the non-monsoon period that decreases to 2 days during monsoon (Figure 16.1, right). The mean portion of bank filtrate abstracted is 60 - 70 % and while the TTC counts in well 40 are <2 - 93 MPN/100 ml, they are <3 MPN/100 ml or below the detectable limit in well P1 (Bartak et al., 2014). Although both wells exhibit short travel times of bank filtrate, bathing and washing activities take place immediately next to well 40 by means of a tap attached to the main distribution pipe at the well. Thus the higher TTC count in well 40 and other wells located close to the UGC bank with similar short travel times, can be explained due to preferential flow of water in to the RBF wells from above ground and around the wells (not river / canal water) due to flooding, an intensive rainfall event or regular seepage / drainage of wastewater from bathing and washing activities (Saph Pani D1.2, 2013), which result in very short travel times (45 minutes to 4.5

hours) as demonstrated by a NaCl tracer experiment on well 40 in Chapter 2 (Sandhu et al., 2014). For RBF wells 18 and P2, located between 110 and 320 m from the UGC and Ganga River, the travel time of the bank filtrate is substantially longer (up to a year, Figure 16.1, left). Compared to well 40, the TTC count is lower with a maximum of only 15 MPN/100 ml. As the bank filtrate to these wells has considerably long travel times, the likely reason is above ground contamination from wide spread defecation on the vast open spaces of the Pant Dweep Island that has an extremely large influx of pilgrims and tourists daily, especially during festivals like the *Kanwar Mela*. During longer festivals like the *Kumbh* and *Ardh Kumbh Melas*, pilgrims reside on the island for up to 4 months. Unlined pit-latrines are dug for such events that have been assessed as a risk to the wells (Bartak et al., 2014).

On the other hand, the remaining wells that are located at a distance of 15 m and more from the UGC in the southern part of Haridwar abstract the highest portion of bank filtrate of all RBF wells in Haridwar (80 - 90 %). The simulated portion of bank filtrate (using the zone budget tool in MODPATH) abstracted by these wells lies within a ±10% confidence limit of the analytically calculated portions using EC and Oxygen-18 isotope data. The maximum TTC count observed in some wells was up to 15 MPN/100 ml while in the others it was below the detectable limit of <3 MPN/100 ml, with the exception of one well having a maximum TTC count of 93 MPN/100 ml (Bartak et al., 2014). As the area between these wells and the UGC, its escape channels and Ganga River is not residential, the impact from domestic sewage (septic tanks, pit-latrines) is low. However, occasional high TTC counts can be attributed to washing and bathing activities and inappropriate drainage of water (wastewater, rainfall and/or storm water runoff) accumulated near/around the wells.

Most importantly, the comparatively overall low TTC counts highlights the high removal efficiency of the RBF system, because most of the public bathing takes place daily in this stretch of the UGC from which the bank filtrate to these wells originates. Furthermore, the annual monsoon and the location of the wells in an area result in a natural recharge to the RBF wells thereby ensuring sustainable operation during periods of peak water demand.

### *Conclusions*

The Haridwar RBF system is operating sustainably since 1965. The groundwater flow modelling study of the RBF system in Haridwar has identified the flow paths of bank filtrate and the groundwater catchment areas of the RBF wells. In conjunction with investigations on the risk of floods and health risk assessment to RBF wells using Haridwar as an example (Chapter 2; Sandhu et al., 2014; Bartak et al., 2014), the groundwater flow modelling investigation has helped to identify potential sources of contamination to the wells. Consequently the study has shown that the wells which abstract the highest portion of bank filtrate, have overall lower or at the most an equal magnitude (only in some cases) of thermotolerant coliform counts compared to RBF wells that abstract an equal or greater portion of ambient groundwater. On one hand the flow modelling study has helped to signify the effectiveness of the natural RBF system to remove pathogens, and on the other hand it illustrates the risk of contamination to unconfined aquifers from inhabited areas without appropriate collection, treatment and discharge of domestic sewage and wastewater. This highlights the need for the implementation of well-head and catchment protection zone measures. These measures have to be prioritised in lieu of the growing pressure on land use and conflicting interests. The flow modelling study has also shown the benefit of locating RBF wells on islands and in areas where a natural flow between surface water bodies occurs to ensure sustainable abstraction. The groundwater flow model of the Haridwar RBF system is a useful tool to compliment the water quality and isotope investigations and can be integrated into a regional hydrogeological assessment of the Haridwar urban area.

### 16.1.2 RBF at Yamuna River, New Delhi: Ammonium reactive transport modelling

*Where? Site description*

A further RBF field site investigated within Saph Pani is located in New Delhi. The capital city is located in North India in the Indo Gangetic Plain along the banks of the Yamuna River. Within the city the river is dammed by two barrages, one in the north of the city and one in the south. In between both barrages, treated, partially treated, and untreated sewage water feed the river through 16 drains (Government of Delhi 2006). Numerous production wells draw water from the floodplain aquifer, a shallow sand and kankar aquifer made up of river deposits. Due to high groundwater abstraction in the city, losing stream conditions are dominating (Lorenzen et al. 2010) and therefore some of the wells draw a high share of bank filtrate. Through the infiltration of river water, sewage-borne contaminants can enter the aquifer and, depending on their retention and degradation rates in the sediments, can eventually reach the production wells. In this context one parameter of concern is ammonium (drinking water limit: 0.5 mg/L, BIS 10500: 2012).

The study site in New Delhi is a transect of observation points across the flood plain on the East bank of the Yamuna River in central Delhi. The transect comprises several hand pumps and observation wells as well as Ranney wells, large horizontal collector wells (Fig. 16.2). Because the well field was not specifically designed for RBF, the production wells are not arranged parallel to the river bank but were constructed across the complete width of the undeveloped flood plain. The main focus of the study lies on a Ranney well (P3) located at a distance of 500 m from the river bank. A detailed description of the field site is given in chapter 4.

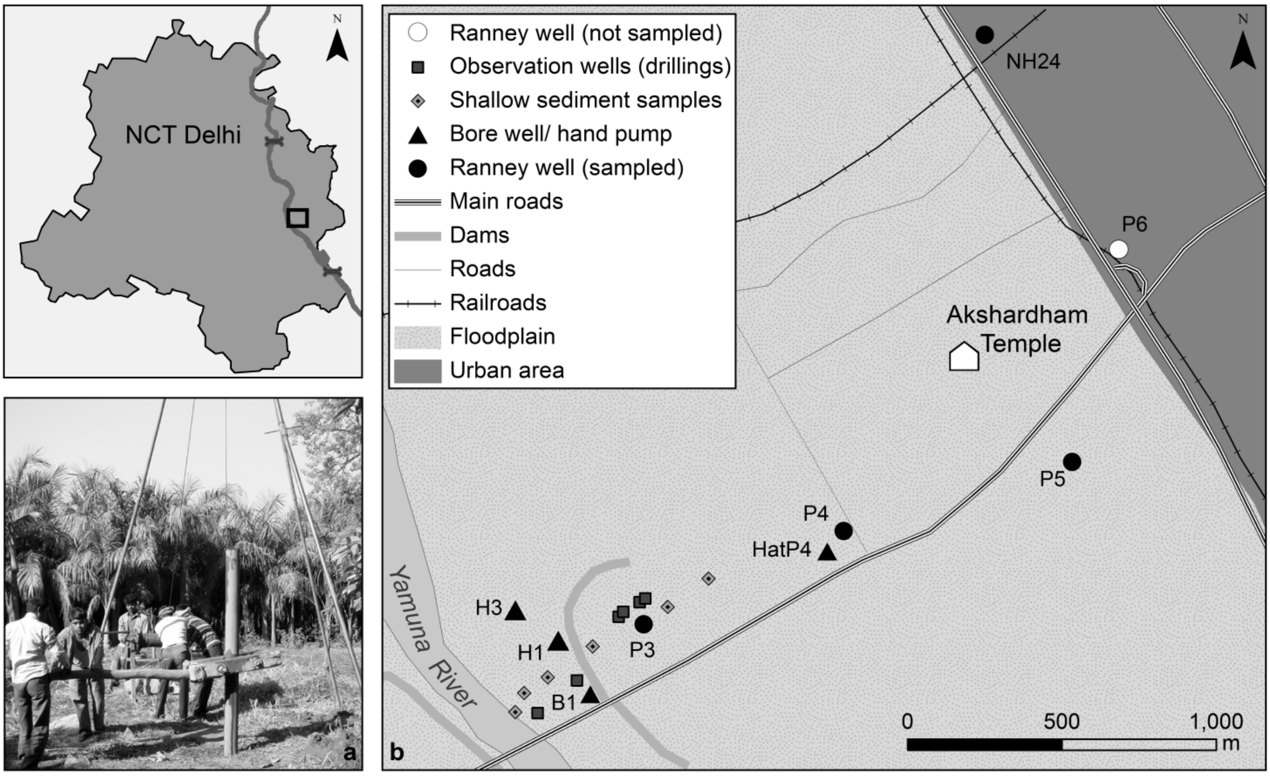


***Figure 16.2 Location of the field site***

*Why? Problems to be solved*

The floodplain aquifer is the aquifer with the highest fresh water potential in Delhi (Kumar et al. 2006). However, strongly elevated ammonium concentrations were found in the river water and in the aquifer close to the river. In the river water ammonium concentrations up to 20 mg/L were measured during the Saph Pani sampling campaigns and concentrations up to 33.3 mg/L were reported by the Central Pollution Control Board (2006). In the groundwater, the trend in ammonium concentrations in 2012 at the three sampling points B1, H1, and H3 was similar with values between 4.5 mg/L in June and 26 mg/L in December (Groeschke 2013). In 2013, ammonium concentrations still fluctuated (between 6.4 and 35 mg/L), but no trend could be discerned. In the Ranney well P3 at a distance of 500 m from the river, an increase of ammonium concentrations has been observed for the past years. In 2012 and 2013, the concentrations varied between 5.5 and 8 mg/L and the well was not used for the production of drinking water (personal communication DJB 2012). In wells farther away from the riverbank, ammonium concentrations remained below 1.7 mg/L in both years. Ammonium concentrations in December 2013 are shown in Figure 16.3. Due to the high ammonium concentrations, the aquifer might be only of limited use for the production of safe drinking water in the future if no appropriate post treatment or remediation concept is installed. For this, the prediction of future ammonium concentrations is of utmost importance.

Various processes influence the transport and fate of ammonium in an aquifer. Due to interactions with the sediment particle surfaces (cation exchange), ammonium does not move with linear groundwater flow velocity but is retarded. The retardation of ammonium strongly depends on site-specific sediment characteristics and therefore transport cannot be predicted using the retardation factors already published, which vary in magnitudes between $10^0$ and $10^2$ for sands and gravel, depending on the clay content and the feed concentrations of ammonium (Buss et al. 2003). Furthermore, the presence of the reduced nitrogen species ammonium is strongly dependent on the redox conditions in the aquifer. Under oxic conditions it can be biologically oxidised to nitrate in the process of nitrification. Under anoxic conditions it can be also oxidised by the anammox process if nitrate or nitrite are present as electron acceptors (van de Graaf et al. 1995, Clark et al. 2008). In addition, the irreversible fixation of ammonium in clay minerals could also occur as well as the mineralization of organic N as an additional source of ammonium. A detailed description of the redox states, occurrence and effects of nitrogen is given in chapter 4.

Laboratory column experiments show that fixation or degradation of ammonium takes place to some extent in the sediments of the unsaturated zone and that no degradation takes place in the saturated zone under suboxic and anoxic conditions (see chapter 4). These results give indications as to the processes occurring at the field site. However, to completely understand the developments of the ammonium concentrations, especially the strong variations, it is necessary to set up a 2D or 3D reactive transport model of the field site. In order to be able to set up such a model, several small scale modelling approaches were applied to determine the necessary input data and to test different hypotheses.

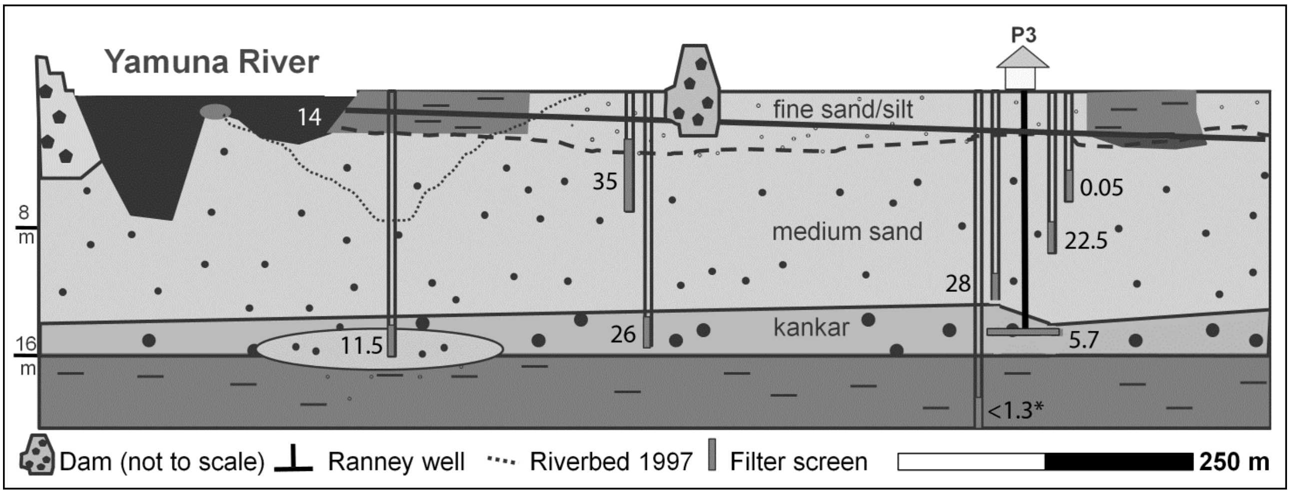


***Figure 16.3 Ammonium concentrations in mg/L in December 2013. *Observation well not sampled in December 2013 but all previous concentrations were below 1.3 mg/L.***

*How? Tools and modelling strategy*

The following modelling techniques were applied to gain a better understanding of the processes occurring in the columns and at the field site:

- Inverse geochemical modelling to determine precipitation and dissolution processes occurring during infiltration
- 2D flow and nonreactive transport modelling of column experiments to determine transport parameters of the different lithological units
- 2D and 1D reactive transport modelling of column experiments implementing cation exchange by adapting selectivity coefficients

- 2D and 1D reactive transport modelling of column experiments adding a nonreactive tracer to determine retardation factors
- 1D modelling of two 500 m flow paths

*Inverse modelling* was conducted with PHREEQC v3 (Parkhurst and Appelo 2013) to identify reactions which can explain the evolution of the water composition from infiltration to the wells. A sample from the Yamuna River taken in October 2012 was used as the initial water composition and a sample from bore well B1 taken in June 2013 was used as the final solution. Calcite, clay minerals, iron bearing minerals (iron hydroxides, iron sulphides), organic matter, and the exchange species $NH_4X$, NaX, KX, $MgX_2$, $CaX_2$ were included as potential reacting phases (X is a cation exchanger like clay minerals). A travel time of approximately eight months for the distance of 250 m is in accordance with the average linear groundwater velocity of 0.9 m/d published for this field site (Sprenger 2011).

A *2D flow and nonreactive transport model* was developed to determine the effective porosities and the dispersivities of the different sediments. The flow simulations were carried out with MODFLOW and the advective-dispersive transport of the NaCl tracer was simulated with the transport simulator MT3DMS and additionally with PHT3D. Tracer breakthrough curves were fitted by adjusting dispersivities and effective porosities, taking into account measured total porosities and literature values (e.g. Johnson 1967). To ensure that no numerical dispersion or oscillations occurred, the simulations were run with TVD and MMOC solver and selected models were furthermore rerun with smaller grid spacing.

*Reactive transport modelling with adapted selectivity* coefficients was carried out in 2D and 1D. Using the transport parameters determined with the non-reactive tracer modelling, flow and reactive transport models were developed with MODFLOW and PHT3D to simulate the adsorption and desorption experiments. Many investigations show that at contaminant sites, where the infiltrating water is strongly influenced by one contaminant, simple sorption isotherm models are insufficient to describe the ammonium behaviour at field scale (Buss et al. 2003). Ion exchange models, which consider all species that compete for the exchange sites give better results (Hamann 2009). Therefore, reactive transport models were developed which consider ion exchange of all main cations present (Haerens 2002). The reactive transport was computed with PHT3D using the Amm.dat database provided with the software PHREEQC v2. The Amm.dat database decouples ammonium from the nitrogen system, which means that no oxidation of ammonium can occur in the model, which is in accordance with the experimental results showing no oxidation of ammonium to nitrite and nitrite at significant levels (Groeschke et al. submitted). The cation exchange selectivity coefficient is the relative preference of an exchanger to adsorb different cations. It is not a thermodynamic constant, but varies with the exchanger composition (e.g. Tournassat et al. 2007 after Jensen 1973). For the exchanger phases of the three sediment types (sand, sand with kankar, kankar), equilibrium equations for Na/K, Na/Mg, Na/Ca, Na/$NH_4$ were set up using the Gaines Thomas convention (Gaines and Thomas 1953) and measurements of the cation compositions on the exchanger as well as the corresponding activities in groundwater samples.

*Retardation factors* for ammonium were determined simulating a conservative tracer test by adding a non-reactive conservative tracer to the reactive transport model (Groeschke et al. submitted). The retardation factors of NH4 were then calculated from the modelled conservative tracer and NH4 breakthrough curves from the time required for the ammonium to reach a relative concentration ($C/C_0$) of 0.5 at the outlet of the column compared to the time required for the tracer to reach $C/C_0$=0.5 (Steefel et al. 2003), whereby the conservative tracer represents the velocity of the water (Appelo and Postma 2007 after Sillén 1951).

*1D modelling of flow paths* was applied to determine how long it would take to flush out the ammonium from the 500 m wide strip near the river. Detailed description of the model set-up is given in chapter 4.

### *So what? Outcome, added value and perspectives*

With the help of 2D models, the transport parameters of the two main lithological units of the aquifer (sand and kankar) as well as a transitional unit (sand with kankar) were determined. Reactive transport models were set-up using adapted selectivity coefficients for the different lithological units. This modelling approach gives good results with respect to the development of ammonium concentrations as well as the development of the concentrations of the main cations. In the sand it takes about 10-12 flushes until the 100% ammonium breakthrough is reached and in the kankar it takes about 30-35 flushed pore volumes (Groeschke et al. submitted). The measured and modelled results of one sand and one kankar column experiment are shown in Figure 16.4. Retardation factors were determined by adding a non-reactive tracer to the models; resulting factors are higher than retardation factors published previously for sand and gravel aquifers - between 6.7 and 19.8 (Groeschke et al. submitted) as opposed to between 2.8 and 6.4 (Böhlke et al. 2006). Using the information from all modelling steps, two simplified 1D flow paths from the river were modelled, one in the gravel and one in the kankar layer. Under conditions where only cation exchange occurs and no oxidation of ammonium takes place, it would take 19 years to flush the ammonium from the sand layer and 61 years to flush the ammonium from the kankar layer, provided that the assumed linear flow velocity is accurate.

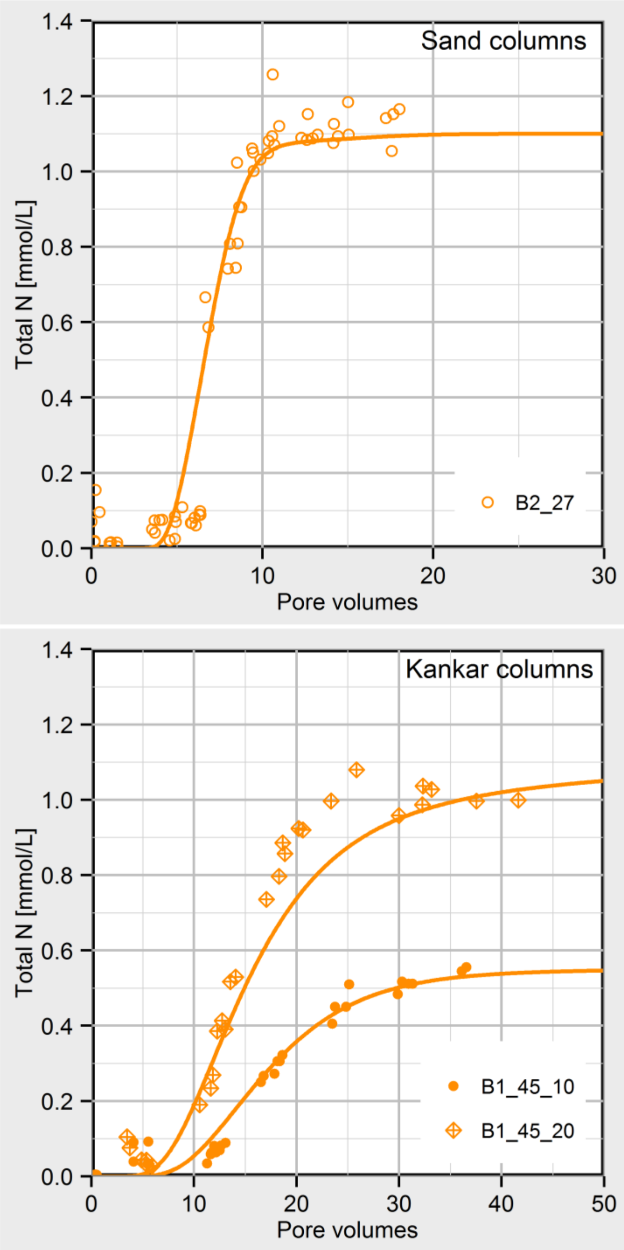


***Figure 16.4 Adsorption experiment modelled and measured***

The enhancement of the river water quality by effective sewage treatment is essential for long term improvement of the groundwater quality in the floodplain aquifer. However, even if this would occur on short term, it would still be a long-term measure due to the strong retardation of ammonium in the aquifer. Therefore, remediation measures or an adapted post treatment have to be installed as short and medium term measures, especially as the ammonium concentrations in well P3 will further increase. For this it is essential to know how the ammonium concentrations will develop in the future and if the ammonium plume will reach the drinking water production wells farther away from the river. To obtain more detailed predictions, the information obtained with the modelling techniques described above, have to be used to set up a 2D or 3D hydrogeological flow and (reactive) transport model for the floodplain and different source water qualities and pumping scenarios have to be considered.

## 16.2 MODELLING MAR (MANAGED AQUIFER RECHARGE)

Managed Aquifer Recharge (MAR) is a method to enhance groundwater quantity and, particularly when combined with Soil-Aquifer-Treatment (SAT), groundwater quality, through the implementation of different types of structures. MAR structures include aquifer storage and recovery (ASR), aquifer storage, transfer and recovery (ASTR), infiltration ponds, infiltration galleries, soil aquifer treatment (SAT), percolation tanks and check dams. One important aspect of water quality improvement is also the remediation of saline intrusion into coastal aquifers through increased groundwater recharge upstream in the watershed or through MAR systems (injection well galleries) at the very limit of the salt water wedge.

A thorough understanding of hydrodynamics, at local and watershed scale is crucial for the selection of the type, dimension and location of MAR structures within a watershed. Flow and transport models, established for those different scales, are important tools to estimate the benefits of MAR-SAT systems for water quality and quantity before implementation and to optimise existing structures. This sub-chapter will look in more details on modelling of a coastal watershed in Tamil Nadu, impacted by over-exploitation and saline intrusion (Chennai case study) and on MAR implementation in a watershed typical for Central India with crystalline fractured bedrock overlain by a more or less porous weathering zone (saprolite).

### 16.2.1 MAR in a coastal aquifer affected by seawater intrusion: Chennai, Tamil Nadu

*Where? Site description*

The Arani and Koratalaiyar (A-K) watershed is located around 40 km north of Chennai. Surface water from reservoirs and groundwater mostly from well fields of this watershed are one of the major sources for Chennai city water supply. Excessive and heavy pumping of groundwater from the A-K basin, tidal water ingress, relatively low recharge, poor land and water management, are the most obvious causes for seawater intrusion. Artificial recharge methods include rainwater harvesting, construction of infiltration wells, percolation tanks, recharge pits and shafts, managing runoff water and facilitating utmost recharge (Asano, 1985). Several check dams were constructed across the Arani and Koratalaiyar rivers flowing north of Chennai to mitigate the problem of seawater intrusion by increasing the groundwater recharge.

The study area comprises two non-perennial river basins Arani-Koratalaiyar (A-K) which are flowing through north of Chennai. The rivers generally flow only for few days during the north east monsoon (November - January). A very dry period occurs in this region during April to May when the temperature rises above 45°C. A colder (winter) period occurs during November to January, experiencing an average temperature of 25°C. The average annual rainfall is around 1200 mm, 35% falling in the south west monsoon (June - September) and 60% during the north east monsoon (October – December). Modelling work has been carried out for an area of 1455 $km^2$ in a part of A-K river basin. The Eastern model boundary is delimited by the Bay of Bengal and the south western side is bounded by the Palar river. The elevation in the model area ranges from sea level in the eastern side to 130 m above mean sea level in south west side as observed in the survey of India toposheet. Groundwater has being exploited for the purpose of agricultural and Chennai city water supply. Five well fields were constructed to withdraw groundwater to supply the city with water (Figure 16.5).

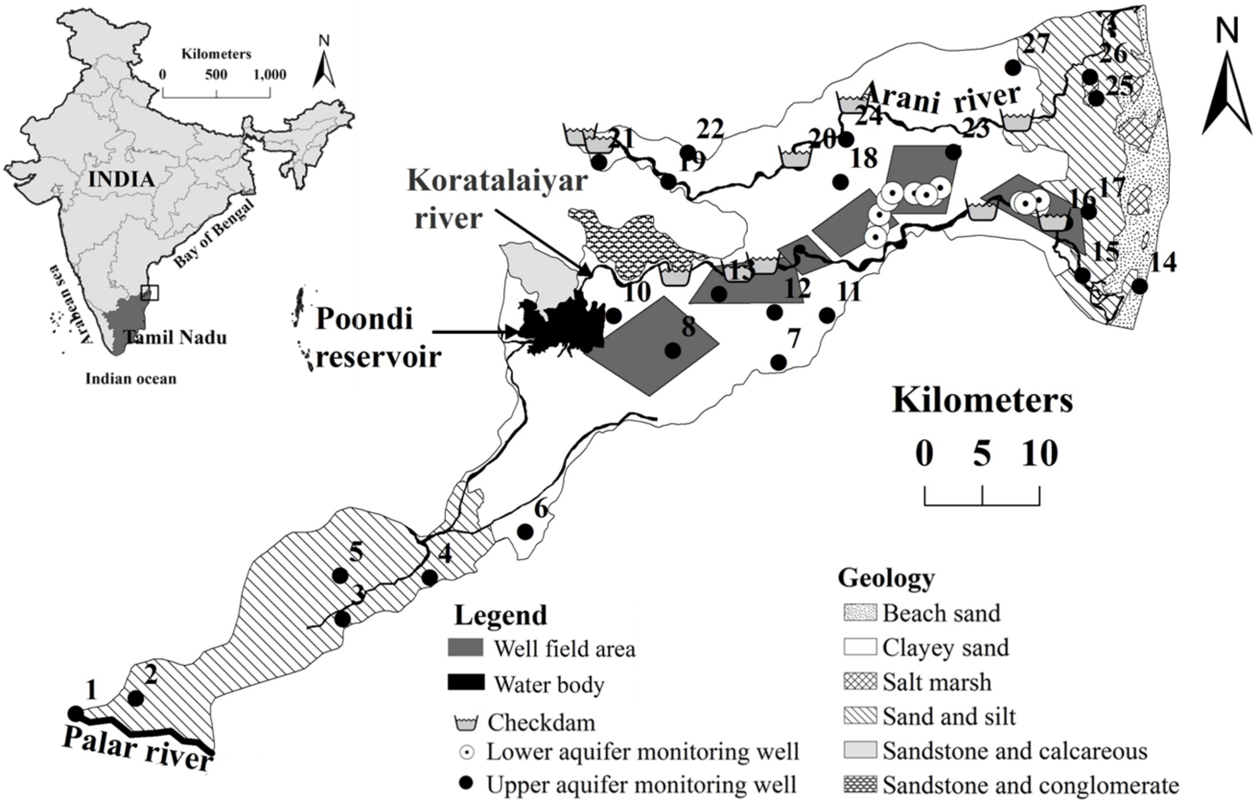


*Figure 16.5 Geology of the study area (adapted from Rajaveni et al., 2014a)*

***Geology and Hydrogeology :*** In this area, the basement archaean rocks are overlained by boulders, clay, shale and sandstone of Mesozoic age, the stratigraphic succession of the geologic formations is given in Table 16.2.1 (UNDP, 1987). The geological outcrop of the A-K basin is shown in Figure 16.5.

Table 16.2.1. Stratigraphic succession of the geological formation (after UNDP, 1987)

| Stratigraphic age and Thickness | Geological description |
|---|---|
| Quaternary (up to 40m) | Fine to coarse sand, gravel, laterite |
| Tertiary (45-50m) | Shale, clay and sand stone |
| Mesozoic | Gondwana shale and clay |
| Archaean | Crystalline rocks |

The main aquifer in the area is the quaternary alluvium and predominantly consists of fine grained material, reflecting a buried channel system. The subsurface lithology has been characterized by boreholes with depths of 50 m thickness penetrating the coastal alluvium with thicknesses up to 35 m. Groundwater in the area occurs in shallow alluvial zone near the coast and the depth to groundwater level increases with the elevation of the area. The thick clay lenses form a semi confined aquifer system. The groundwater levels in the unconfined aquifer ranges from 2 to 6 m bgl (below ground level) and in semi confined aquifer it ranges from 14 to 20 m bgl. A west to east geological cross section is given in Figure 16.6 (A-A').

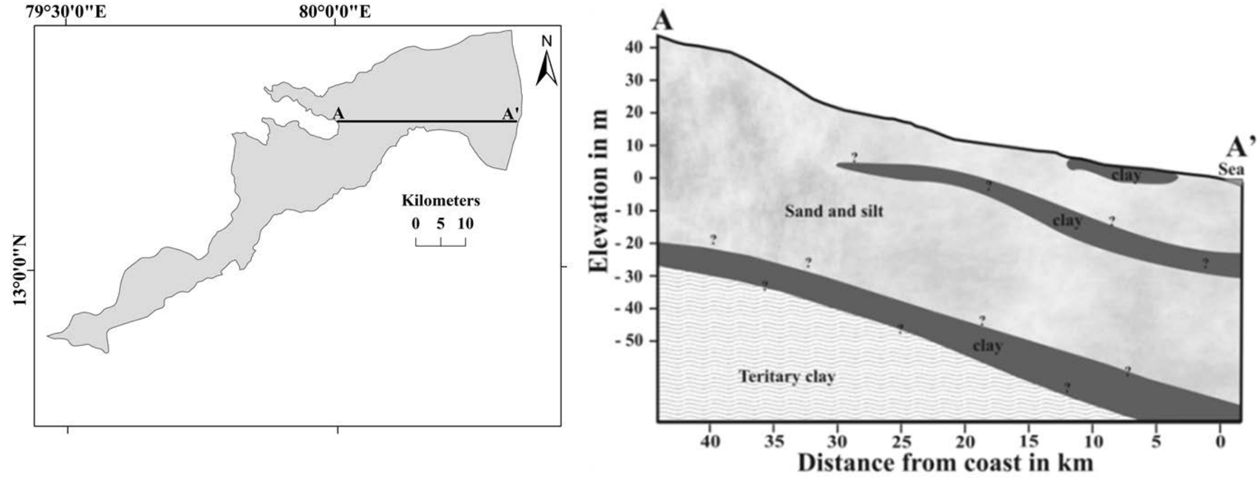


***Figure 16.6 Geological cross section along A-A'***

### *Why? Problems to be solved*

The A-K basin is characterized by severe over-extraction of groundwater for agricultural activities and water supply to the Chennai metropolitan area, which has been identified to cause significant seawater intrusion. Numerical modelling can help to analyse seawater intrusion by using models to simulate different pumping conditions and quantifying the effect of possible mitigation measures. The general Objectives are:

- Simulate the current seawater intrusion
- Representation of check dams and potential other artificial recharge structures in the model, to predict future seawater intrusion and to analyse measures to push back the saltwater front.

### *How? Tools and modelling strategy*

The methods and tools used to generate the coupled model are as follows (Bhola et al., 2014):

(1) A rainfall-runoff model (NAM) to produce surface water inflow at the sub-catchment scale for the boundary conditions into the surface water system as well as the infiltration into the subsoil, integrated in the 1D surface water model.

(2) A 1D surface water model (MIKE 11) for the two rivers Arani and Koratalaiyar.

(3) A 3D groundwater model (FEFLOW) for the alluvial aquifers of A-K basin which is coupled to the MIKE 11 model using the coupling interface IfmMIKE11 (Monninkhoff 2011), to describe the interaction between the ground- and surface water in detail.

### *So what? Outcome, added value and perspectives*

The NAM model parameters were calibrated and extended homogeneously over the entire A-K basin. Since the model was calibrated for an eight year time period, it covers a wide range of hydrologic and climate conditions, which builds confidence in the model's ability to predict stream flow conditions under a variety of scenarios. The model gives a satisfactory comparison with observed flow records with an $R^2$ value of 0.6. Main focus was given to achieve least volume and peak errors (Figure 16.7). The model over-predicts the total volume in eight years by 10.5%, and a peak error of almost 7% for a discharge greater than 300 $m^3$/day. The NAM model does not predict low flow accurately due to high surface and root zone storage coefficients. These coefficients define the water holding capacity of the soil, i.e. overland flow will occur once the rainfall is greater than the thresholds of these coefficients. In the observed discharge records, it was found that the response of a rainfall that results in runoff is relatively high and therefore in the model it was implemented accordingly.

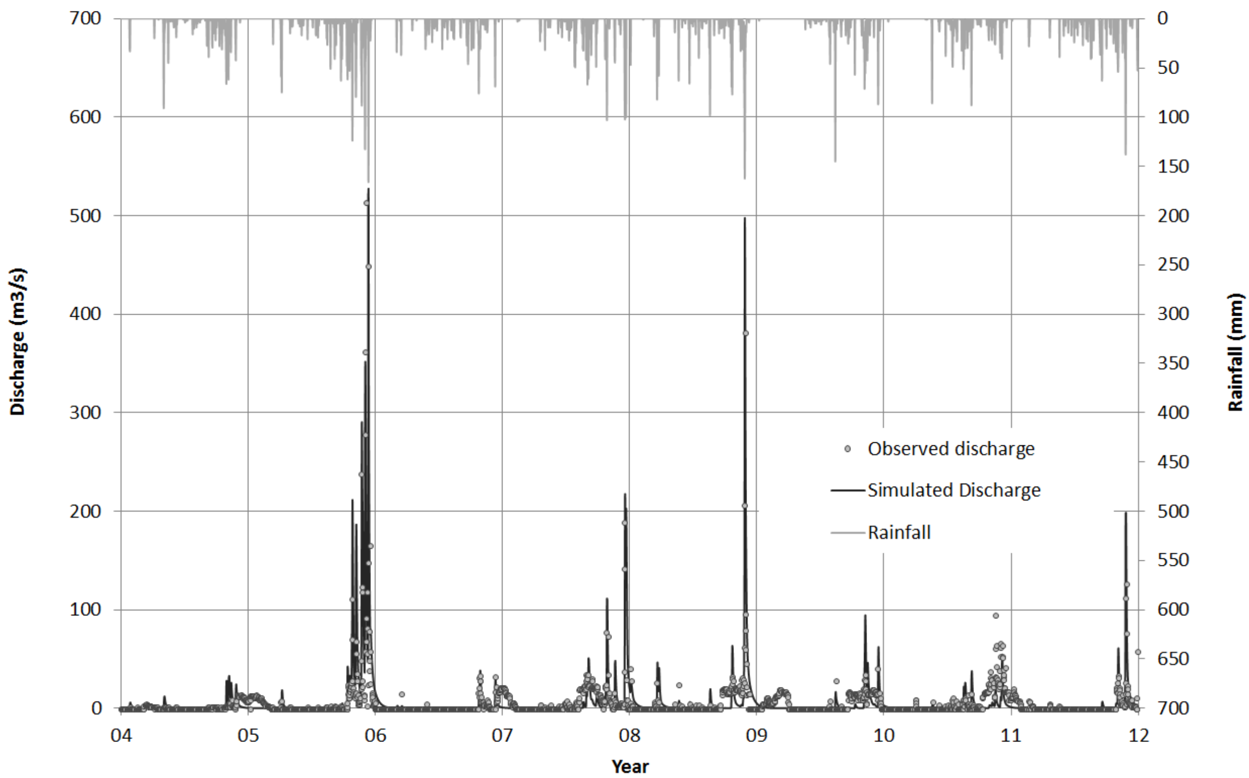


*Figure 16.7 Comparison of observed and simulated discharge from 2004 to 2012 at the inlet of Poondi reservoir (Bhola et al., 2014)*

***Groundwater model:*** The model was calibrated in two stages, a steady state and a transient condition. The steady state calibration was carried out to achieve an average match between the available observed and simulated groundwater heads and to define a suitable distribution of conductivities. The transient state was carried out for a period of 13 years from January 1996 to March 2009 (Rajaveni et al., 2014b). Transient state calibration was conducted by basically adapting local conductivities and porosities until the best fit curve was obtained for observed and simulated groundwater heads. A root mean square of 0.901 was obtained during steady state calibration. In the transient state calibration, the simulated groundwater heads were accurately describing the groundwater dynamics of the observed groundwater head in most of the wells. The observed and simulated groundwater head variations in the transient state calibration are exemplarily shown in Figure 16.8 for one of the observation wells.

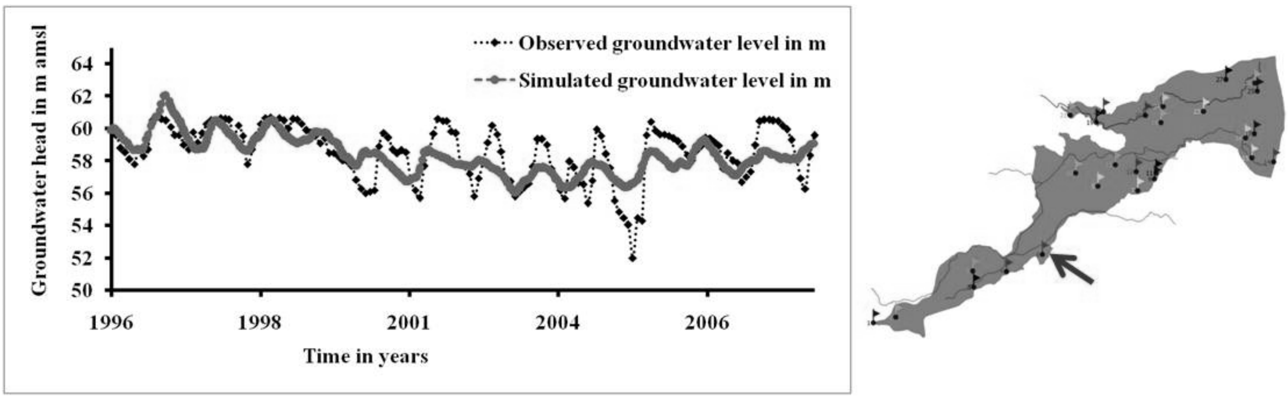


*Figure 16.8 Observed and simulated groundwater head variations during transient state calibration in the single aquifer system*

***Density dependent model:*** Density dependent parameters were applied to the uncoupled 3D-groundwater model, to report fresh water-seawater interactions. The hydraulic head boundary condition (BC) at the Bay of Bengal was assigned as saltwater head BC. Mass concentration BC was assigned as 500 mg/l in the existing hydraulic head location and 35000 mg/l in the eastern boundary (coast). An initial mass-concentration distribution was defined, according to the range used in the boundary conditions. To avoid numerical instabilities, the mesh was refined in the coastal region (high density gradient area), which increased the total number of elements from 1 to 1.5 million (Rajaveni et al., 2014b). An uncoupled density dependent seawater intrusion was simulated and the result shows seawater has intruded from 3.5 km in the year May 1997 to 7 km during May 2003 (Figure 16.9).

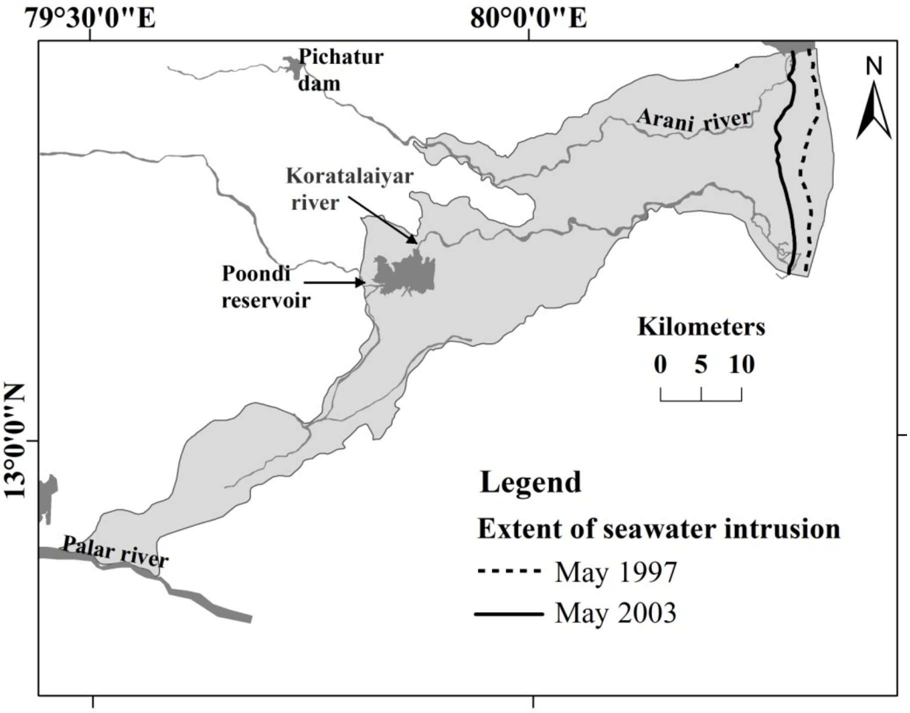


***Figure 16.9 Extent of seawater intrusion for two time periods such as May 1997 and May 2003(Rajaveni et al., 2014b)***

***Principle simulation of the effect of MAR:*** The general aim of this study is to improve groundwater quantity and quality through MAR structures. As a first step the calibrated 3D groundwater model was used to evaluate the effect of recharge from MAR structures on groundwater heads in the basin. A total of 9 check dams, 4 in the Arani river and 5 in the Koratalaiyar river, are existing during 1996 in the study area and were implemented in the uncoupled model. The effect of check dams was computed and predicted by representing the check dams as a 3rd Kind BC with different realistic time series (Rajaveni et al., 2014b). Groundwater head variations were simulated under 2 scenarios (i) with and (ii) without check dams. Observation point 8 has been chosen to explain this study since this observation well is located at the centre of the modelled area. Figure 16.10 shows a maximum of 3 m increase in groundwater heads with the implementation of check dams in the model at this location. Highest differences can be observed during the monsoon seasons. During non-monsoon seasons the groundwater head at this location will eventually reach the level representing the situation without check dams.

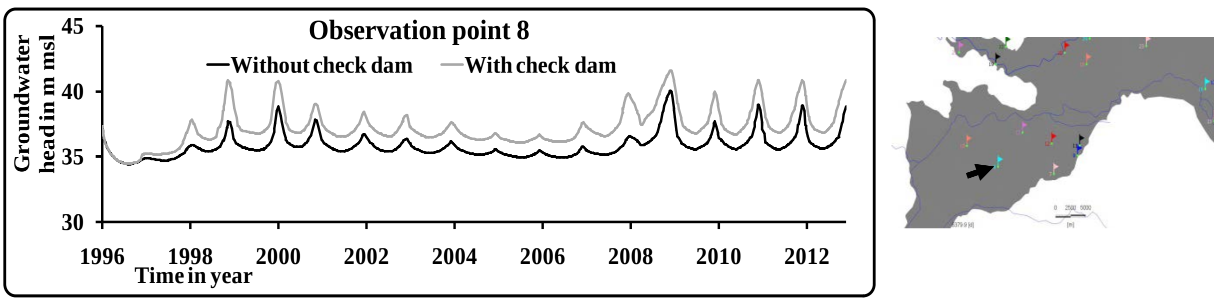


***Figure 16.10 Groundwater head variations with and without check dam***

***Coupled surface and groundwater model***: An uncoupled groundwater flow model was coupled with the surface water model MIKE11 and simulations were performed for the period 2004 to 2009. Three scenarios were simulated: one scenario without check dams, scenario 1 considering most of the existing check dams until date (9 in total) and scenario 2 with three additional check dams (12 in total) as well as an increased dam crest of about1 m at the already existing check dams (Rajaveni et al., 2014b)

Figure 16.11 compares the simulated groundwater head along the Arani River, at locations close to the implemented check dams in scenario 2. The last figure (bottom) also includes the simulated water level in the check dam, calculated by the coupled surface water model MIKE11.The following table gives an overview of the situation at 4 selected locations in Figure 16.7. The scenario without check dams represents a situation with approximated natural river courses.

Table 16.2.2Overview of the scenario definitions at 4 selected locations at the Arani River

| Location | Scenario without check dams | Scenario 1 | Scenario 2 |
|---|---|---|---|
| A1 | No check dam | Check dam | Check dam raised |
| A2 | No check dam | No check dam | Check dam implemented |
| A3 | No check dam | Check dam | Check dam raised |
| A4 | No check dam | Check dam | Check dam raised |

At location A1 there is an existing check dam in scenario 1 and in scenario 2 the same check dam has been raised by 1 m (Rajaveni et al., 2014b). As expected, the results show an increase in groundwater levels by the implementation of the check dam in scenario 1 and a further, though less significant increase in groundwater heads by raising the dam wall in scenario 2.

Scenario 1 has no check dam at location A2, leading to no difference compared to the scenario without check dams. Higher groundwater heads were obtained through the additional check dam in scenario 2.

For location A3 both scenario 1 and 2 have a check dam implemented, while the check dam in scenario 2 has been raised (similar to location A1). The simulation results show that the check dam in scenario 1 increased the groundwater heads and an additional increase in levels can be obtained by constructing a higher check dam wall. At the beginning of the simulation scenario 1 has slightly higher groundwater heads at this location, which is related to the fact that the water is not retained upstream in this scenario, since there is no check dam between location A1 and A3 in this simulation, while in scenario 2 there are two check dams which can retain the water along this river stretch, preventing that the check dam at location A3 is continuously being refilled (Rajaveni et al., 2014b). After 2006, the results show that during wet periods also the check dam at A3 can be filled with the release water from the newly implemented check dams upstream. The higher crest level in scenario 2 causes higher groundwater heads in scenario 2 at this location during that period.

At location A4, the situation is identical to A3, a check dam has been implemented in scenario 1 and the dam level has been raised in scenario 2, leading to the highest groundwater heads through scenario 2. A lag was identified between scenario 1 and 2, due to the retention effect of the upstream check dams in scenario 2. This can also be seen in the additionally displayed water levels directly at the check dam (Rajaveni et al., 2014b).

In sum, the results indicate that additional check dams have a positive (local) effect on the groundwater heads, just as the raising of the dams, though the effect is considerably smaller. The results also show that the implementation of additional check dams can retain water further upstream, possibly leading to a delay or even a lack of groundwater recharge in the downstream part of the catchment.

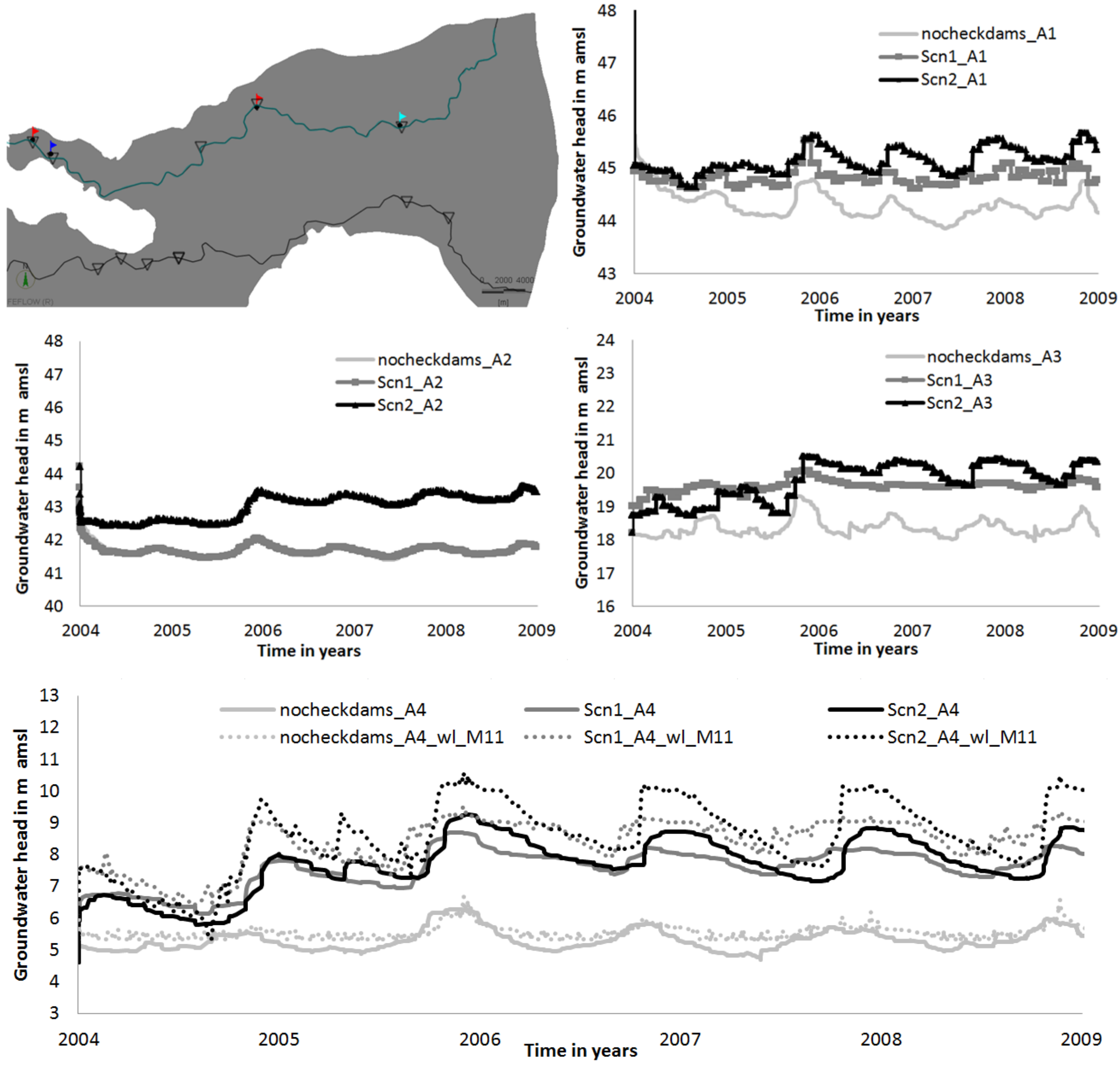


***Figure 16.11 Comparison of groundwater heads in the vicinity of existing and plannedcheck damsalongthe Araniyar River for different scenarios (Rajaveni et al., 2014b)***

***Conclusions and outlook***: An integrated surface and groundwater model using MIKE11 and FEFLOW has been setup and was successfully calibrated. With the model it was possible to display salt water intrusion processes. Using the scenarios presented in this chapter, which show a significant local effect of the MAR structures on groundwater levels, the model is now ready to use to analyse the benefits of MAR structures also on the saltwater intrusion process. For this, long-term analyses will be necessary. These simulations could be set up using yearly returning seasonal cycles for climatological conditions as well as natural groundwater recharge conditions. The simulations should cover a period of approximately at least 50 years to analyse the effect of MAR structures on the long-term perspective. Furthermore, the model could also be used to predict the effect of climatological changes on the long-term.

### 16.2.2 MAR in a weathered crystalline hardrock aquifer: Maheshwaram, Telangana

*Where? Site description*

One of the main experimental watersheds relevant for MAR studies in the Saph Pani project is located around the town of Maheshwaram (Figure 1.12) near Hyderabad, Telangana. With a total area of 53 km$^2$, it is situated, under semi-arid climate, on a weathered crystalline rock substratum, a geological and climatic context typical for the entire region where the saprolite weathering layer (10-20 m thick) is usually unsaturated. It is a watershed with a high density of groundwater production wells (>700) mostly for paddy irrigation; changes in land use have occurred since 2006, the new Hyderabad international airport being located less than 10 km away. It is expected to become a peri-urban area within the coming years as significant housing projects are planned. MAR systems exist throughout the watershed in the form of percolation tanks, check dams, defunct dugwells, etc.

Intensive groundwater exploitation for irrigation has resulted in aquifer over-exploitation and deterioration of groundwater quality (fluoride above maximum permissible limit of 1.2 mg/L (BIS, Bureau of Indian Standards), salinisation and agricultural inputs). MAR is an attractive concept for groundwater augmentation and enhanced groundwater quality nearby wells exploited for domestic uses.

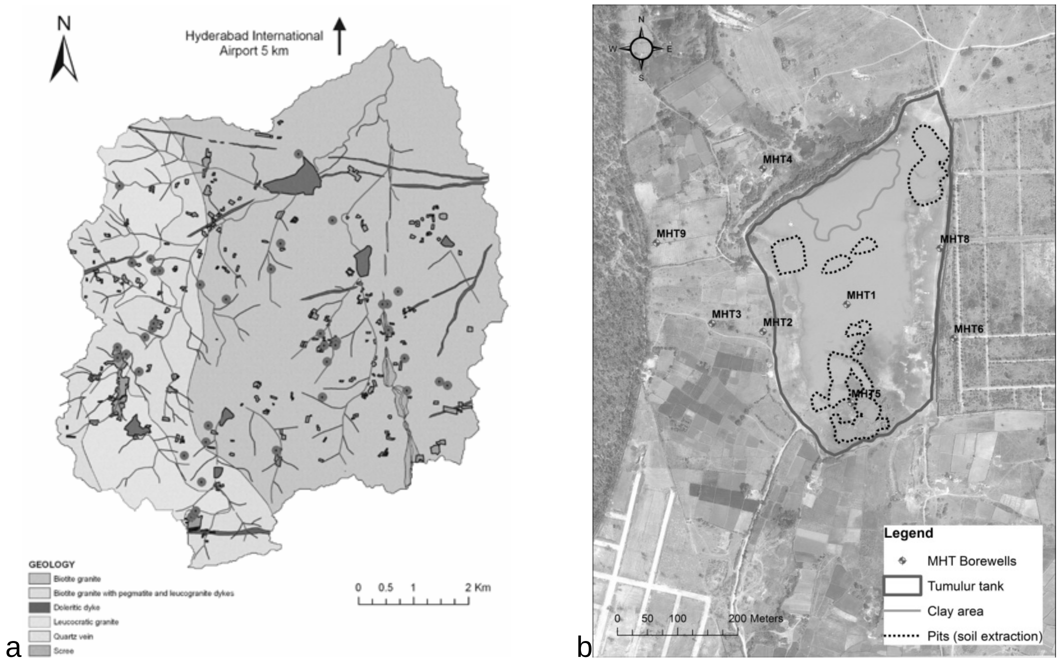


***Figure 1.12 a Geological map of Maheshwaram watershed. Main MAR (percolation tanks and defunct dugwells) structures are indicated in dark grey. Middle grey areas are irrigated paddy fields. b Tumulur tank structure with pumping and observation wells (open circles).***

*Why? Problems to be solved*

The modelling objective for the case study site in Maheshwaram is triple:

(1) Develop tools to take into account the highly variable geometry of percolation tanks on weathered crystalline basement rocks under the specific Indian climate (dry season vs. monsoon) through a specifically developed module for the 3D finite difference transient groundwater flow model MARTHE (Thiéry, 2010).

(2) to assess the influence on water quantity of percolation tanks at local and regional scale

(3) to assess the influence of percolation tanks on crystalline basement rocks on water quality, in particular on fluoride concentrations, triggered by water-rock interactions with F-containing minerals and evaporation together with agricultural backflow on paddy fields.

*How? Tools and modelling strategy*

***Modelling infiltration from percolation tanks of variable geometry via a partially saturated weathering zone:*** To assess the performance of percolation tanks, the three-dimensional finite difference transient state numerical groundwater code MARTHE was optimized by implementing three-dimensional non-perennial surface water bodies in continuity with groundwater via an unsaturated zone. Implementation included the spatiotemporal evolution of the natural percolation tanks (i.e., volume and geometry) linked to topography, taking into account heavy rainfalls during monsoon, evapotranspiration, infiltration, runoff, and groundwater dynamics. Part of the rain water stored in such a tanks during the monsoon season infiltrates into the soil (variably-saturated media) and reaches the aquifer whereas part is evaporated. Theoretical simulations show that the new developed module "LAC" is able to simulate the relation between surface water and groundwater while respecting the water balance (Picot-Colbeaux *et al.*, submitted) and to assess the highly variable geometry of infiltration tanks over the dry and wet season (figure 16.13).

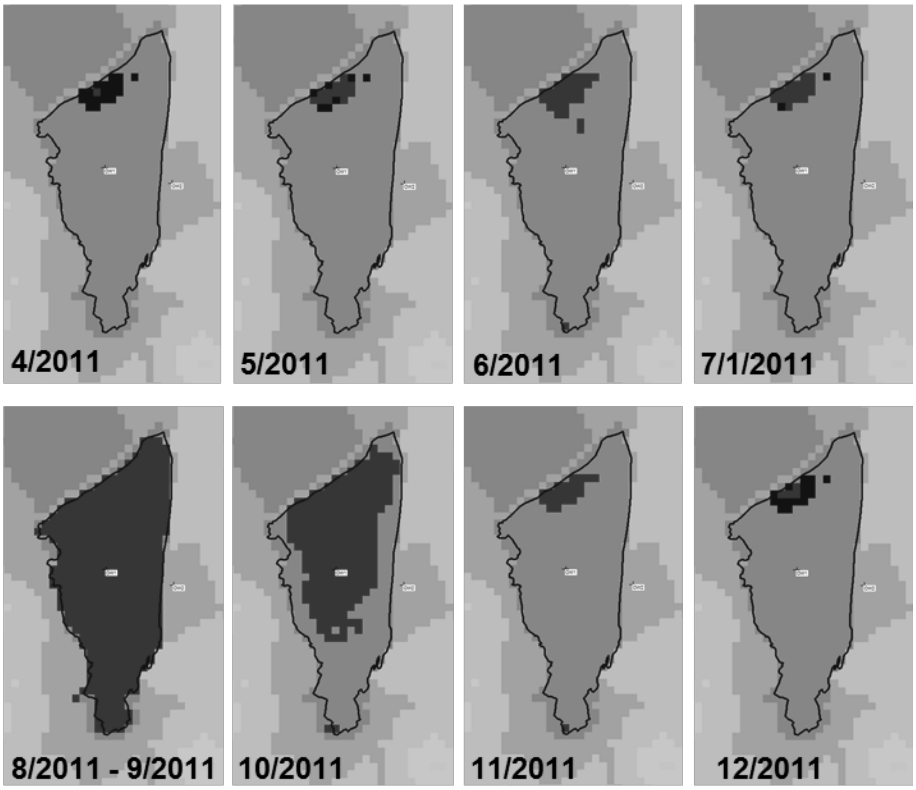


***Figure 16.13 Simulated variable extension of the Tumulur tank filling (dark grey) over a monsoon season in 2011, Maheshwaram study site near Hyderabad, Telangana, India***

Modelling influence of percolation tank systems on fluoride concentrations: The geochemical model of solute recycling developed for paddy field irrigation (Pettenati *et al.*, 2013) using a 1D PHREEQC reactive-transport column (Parkhurst and Appello, 1999) was further developed and adapted to the percolation tank problem, on the basis of new monitoring data in order to test the conceptual geochemical model of MAR (figure 16.14).

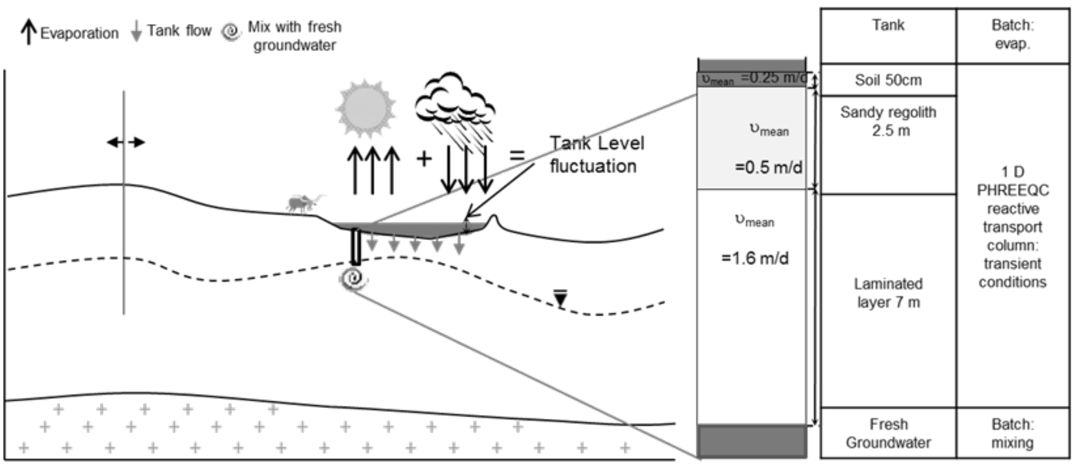


***Figure 16.14 Conceptual model of hard-rock aquifer in southern India with Managed Aquifer Recharge (MAR) through an infiltration tank used for the development of a1D Phreeqc reactive column model.*** $\upsilon_{mean}$ ***is the mean pore flow velocity (Pettenati*** **et al.,*****2014)***

Reactive transport column modeling was performed over a period of 110 days with calculated pore flow velocity taking into account the mineral composition of the 3 distinct layers of altered biotite granite determined by XRD analysis, the Cation Exchange Capacity (CEC) of the weathering profile determined by cobalt hexamine chloride solution and the measured initial groundwater composition.

*So what? Outcome, added value and perspectives*

The 3D MARTHE software, first developed in 1990 already integrating surface-groundwater flow under varying saturation states, including density driven flow (Thiéry, 2010), is now ready, with the implementation of a specific module for percolation tanks, to be applied to MAR systems on weathered crystalline basement rocks in India and elsewhere. Such massively integrated models are still the exception and will be increasingly used as decision-making tool for assessing the quantitative effects of MAR on groundwater resources at the watershed scale.

The geochemical 1D reactive transport model, using PHREEQC, investigated the role of managed aquifer recharge under variable climatic conditions and its impact on groundwater chemistry. Based on a previous model which reproduced satisfactorily the solute behaviour in Maheshwaram groundwater under the influence of paddy fields (Pettenati *et al.*, 2013), the reactive transport model of the Tumulur tank infiltration through the critical zone helps to understand the evolution of fluoride enrichment or depletion in groundwater when MAR is implemented in a watershed. Results of the first scenarios simulation show that the beneficial effect of MAR may be variable over the year being strongest during monsoon where significant dilution of whereas during the dry period, F accumulation occurs. In sum, the beneficial effects observed during monsoon are countered by adverse effects during the dry period so that no overall water quality improvement related to the MAR system can be expected at local, and, most likely, also at regional scale. Extrapolating at regional scale would require integration of 3D groundwater flow approaches with the developed geochemical model.

## 16.3 Modelling Wetlands

Natural wetlands play an important role in regulating surface water and groundwater flow within a watershed and also have a purifying action through intensive and divers biological processes, ranging from macrophyte uptake of nutrients and contaminants to microbiological processes. Those processes are voluntarily used and optimised in constructed wetlands. In an intermediate position between natural and engineered systems are situated man-made wetlands used for agriculture, notably paddy fields, with important effects through (1) supplementary water abstraction from the watershed, both from surface water and groundwater, (2) enhanced evaporation, (3) nutrient and trace element uptake by crops and (4) agricultural return flow towards the aquifer. Effects on groundwater quality may be beneficial (through filtration, water-rock interaction, biological processes in soil, the underlying variably saturated zone and the aquifer, in an analogous way to SAT systems) or, on the contrary, adverse (mainly through evaporation and return flow with enhanced salinity, trace elements and wastewater-related contaminants and pathogens). In this sub-chapter, we investigate the impact of indirect wastewater recycling for irrigation in a peri-urban watershed in Telangana through an integrated modelling approach.

### 16.3.1 Integrated modeling of the Musi river wetlands: Hyderabad, Telangana

*Where? Site description*

The Musi River is a major tributary of Krishna River, originates in the North West of Hyderabad in Rangareddy district and flows down towards South east direction passing through the Hyderabad city and joins Krishna River at Wazirabad in Nalgonda District. The Musi River has been intercepted by two major reservoirs, Himayat sagar and Osman sagar, upstream of the City. Below these two reservoirs, the Musi River receives only city's wastewater and storm water. It receives water from its upper catchment only when excess flood is released from these reservoirs. The study area lies between coordinates 17° 15' N, 17° 30.'N and 78° 30'E, 78° 45.0'E and includes villages Peerzadaguda, Kachiwani singram and Mutialguda situated in peri-urban Hyderabad, and on the northern side of the Musi River, Figure 16.15.

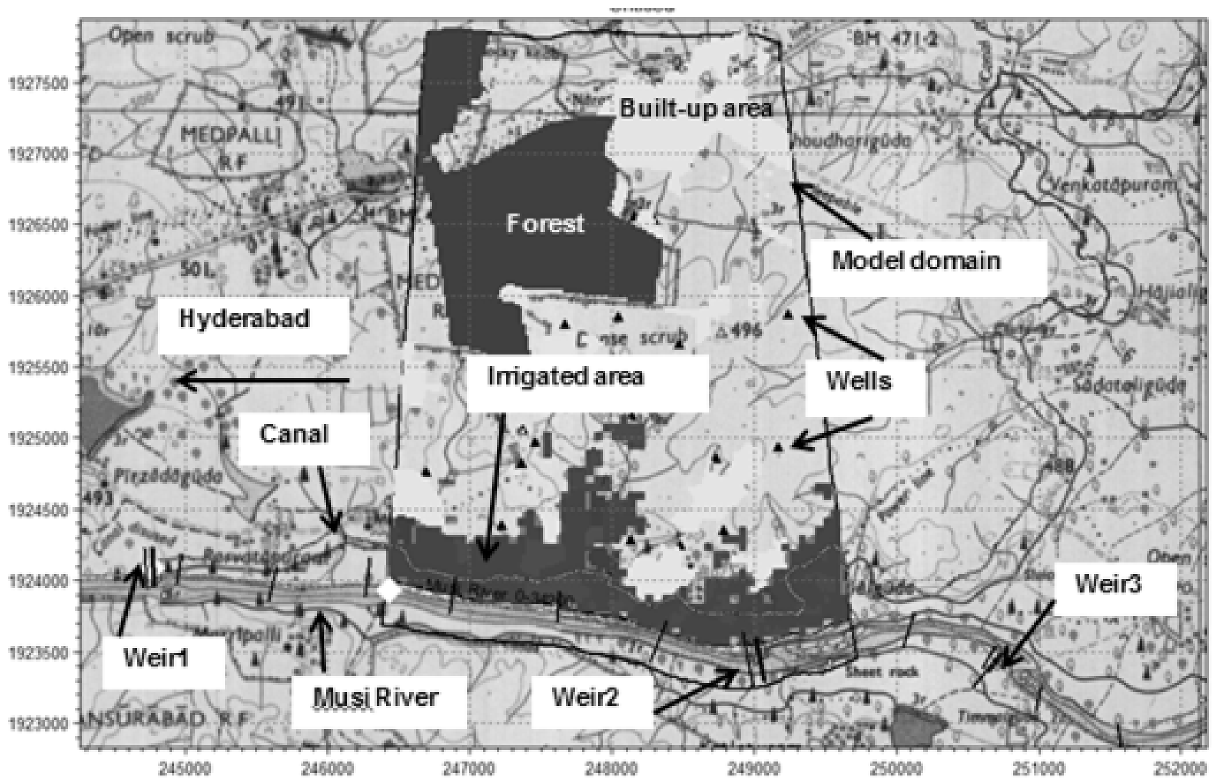


***Figure 16.15 The study area east of Hyderabad,, showing the Musi river, and the irrigation canal***

The Musi River downstream of Hyderabad has a cascade of overflowing weirs/pond at which water is diverted into irrigation channels on both sides of the river. The wastewater flow in the river has made it a perennial river which is a significant resource in this semi-arid peri-urban environment where the cultivation of fodder grass, paddy and vegetables has provided economic benefits to many peri-urban inhabitants. Year round cultivation, water in the irrigation canal, overflowing diversion structures (weirs), storage ponds etc has resulted in the rise in the water table and converted the riparian zone along the river into a wetland.

*Why? Problems to be solved*

***Wastewater irrigation vs natural treatment systems:*** Wastewater reuse has become a major area of interest to engineers, biologists, chemists, agronomists, water supply authorities, industries, water resources authorities, etc. Different agencies and stakeholders have different concerns like preventing surface water pollution, conserve and recycle soil nutrients, development of additional water sources for agriculture, industries or non-potable supplies. Irrigation practice with wastewater is one of the reuse options for wastewater. The livelihood and economic activities of the peri-urban farmers are the key drivers of wastewater reuse, especially irrigation for agriculture production and its secondary advantages like by-products and indirect benefits, for example, cheap water, perennial supply, reduction in surface water pollution, increase in soil nutrient and groundwater recharge (increase in specific yield of underlying aquifer increases). And, some obvious downside aspects are soil degradation, degradation of ambient groundwater, cropping pattern change, aesthetics, and health risk for consumers and farmers.

***Wetlands:*** "Wetlands are areas where water plays an important role creating a suitable environment for the associated plant and animal life. They occur where the water table is at or near the surface of the land, or where the land is covered by shallow water" (Ramsar, 2013). Wetlands, whether human made, (i.e., constructed) or purely natural, are also considered to be a cheaper and low-cost alternative technology for wastewater natural treatment. Distribution and differences in type of natural wetlands are caused by topography, soil, drainage, vegetation, geology, climate, land use, as well as infrastructures like canal, controlled and impounded natural drainage, or human-induced disturbance. Water table depth and its temporal variability, movement of water from one level to another through the wetland are the key parameters in characterizing the types and behaviour of a wetland.

***Groundwater surface water interactions:*** Surface water and groundwater have been managed separately often by completely different branches of the government. It is now recognized that water resources problems cannot be treated in isolation. Problems like wetland protection or the conjunctive use of surface water and groundwater resources require the integrated management of surface water and groundwater together with the water chemistry and ecology. Increasingly, water resources are being managed on a watershed basis, while addressing problems at the local scale. Watershed-based water management system requires new and more sophisticated tools. Traditional groundwater and surface water models were not designed to answer questions related to conjunctive use of groundwater and surface water, water quality impacts of surface water on groundwater, impact of land-use changes and urban development on water resources, and floodplain and wetland management. Instead, fully integrated hydrologic models of the watershed behaviour are required.

***Objectives:*** The main objective of the study was to help understand the hydrodynamic behaviour of the groundwater-surface water systems under the influence of anthropogenic activities like irrigation, canal construction (seepage), weirs/ponding in the natural drainage of a riverine wastewater impacted (agriculture) wetland. The better understanding of the surface and sub-surface hydrologic processes in an integrated manner will help in assessing the positive and negatives impacts of wastewater irrigation practice on the groundwater and surface water systems. Considering the overall objective of the study dealing with models, it was required to understand the movement and exchange of water among various zones of the system like overland surface, unsaturated zone ( sub surface), aquifer, vegetation, exchange with surface water body ( river/canal), therefore, an distributed hydrologic tool, MIKE SHE was selected for carrying out the study.

### *How? Tools and modelling strategy*

***Integrated catchment modelling: Application of MIKE SHE:*** MIKE SHE has been widely used for integrated hydrologic modeling. MIKE SHE's process based framework allows each hydrologic process to be represented according to the problem needs at different spatial and temporal scales. The Water Movement module of the software has a modular structure which includes six process-oriented components of the hydrological cycle that are interception/evapotranspiration, overland/channel flow, unsaturated zone, saturated zone, snow melt and the exchange between aquifers and rivers (Figure 16.16) ( DHI , 2014) . MIKE SHE uses MIKE 11 to simulate channel flow and interact with surface water. MIKE SHE 's strength lies in its features to provide a simulation of coupled unsaturated-saturated zone, interaction between evaporation and shallow water table , better evapotranspiration module with root zone exchange apart from efficient coupling with open channel.

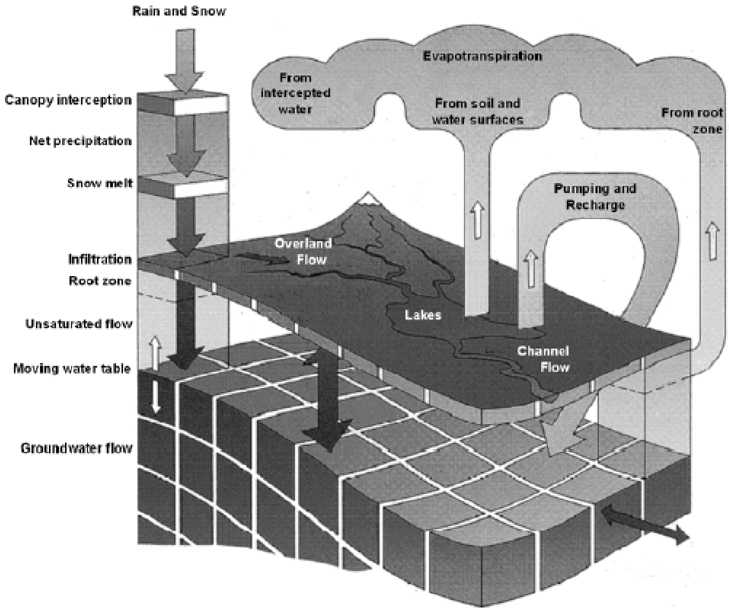


*Figure 16.16 Hydrological Processes in MIKE SHE ( DHI,2014)*

***Modelling strategy:*** In the present case, the area of interest was wastewater irrigated area along the Musi River which includes the river, weirs, cultivation practices, pumping etc. However, in this instance also, it appeared that the model domain needed to be up-scaled suitably to have a realistic groundwater boundary. For this reason, the model catchment was upscaled towards upland on the northern side up to near village Narapalli Figure 16.15 above. The main input parameters for the model setup include data such as: topography, soils, land use and land cover, natural and canal drainage network, locations of weirs and their hydraulic parameters, well numbers and locations, agriculture and irrigation data, rainfall, potential evapotranspiration, aquifer parameters etc which were collected from field visits, primary survey and monitoring and also taken from secondary sources and research reports conducted in the area. The model domain (12.68 Km$^2$) was divided into 60 m x 60 m cells. The irrigated area inside the model domain is about 1.73 Km$^2$. In the present model, the study area is very small and highly vegetative and in the catchment no stream or ditch of significant size is present which carries significant surface runoff during dry or even rainfall period. However, there were good number of observed groundwater table data across the model domain, hence, the model was calibrated with groundwater depth only. All the processes like overland flow, unsaturated zone flow, saturated zone, evapotranspiration and exchange with surface water were included in the model setup and simulations considering their roles in the wastewater irrigation practice as a natural treatment system, i.e., soil-aquifer-treatment. MIKE 11 was setup and simulated as stand-alone including the Musi River, canal and Weir 2 and later on was integrated/coupled with MIKE SHE. The coupled length of the Musi and the canal with MIKE SHE are 2.28 Km and 4.05 Km respectively

### *So what? Outcome, added value and perspectives*

Two basic scenarios were simulated. The pristine scenario assuming that there were no canals and weirs/ponds across the Musi River and no additional irrigation except rainfall, and the second was the baseline scenario, i.e., the existing condition. Irrigation, weirs and canal seepage have changed the hydrodynamic characteristic of the area and the area is functioning like a wetland where there is direct exchange groundwater with overland flow and the water table is close to the surface. The results shows that the area with groundwater table within 6 m from surface, i.e. Wetland, (Ramsar, 1971) has been increased from 0.74 Km$^2$ (Pristine condition) to 1.47 Km$^2$ (Baseline) over the years, Figure 16.17 below.

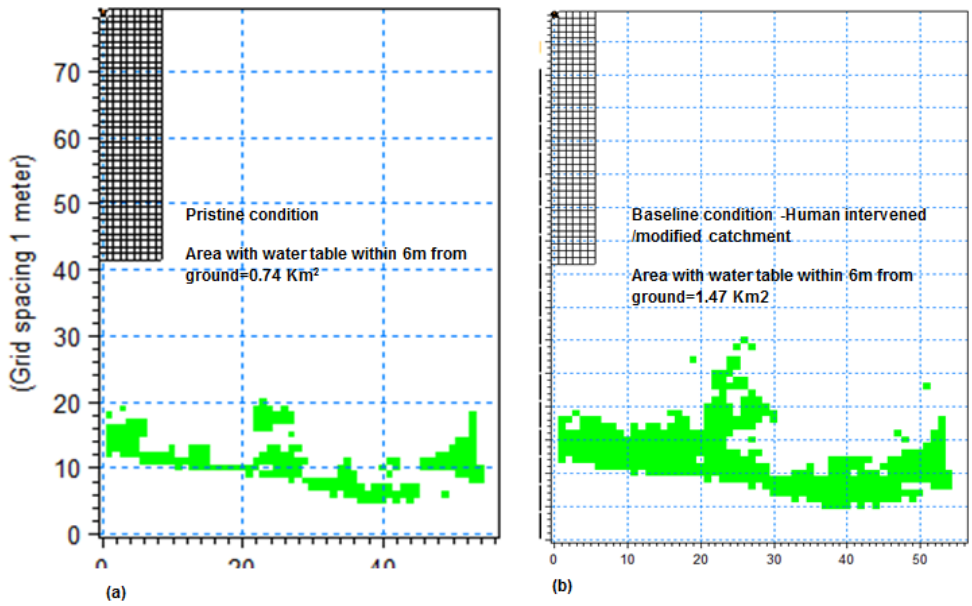


***Figure 16.17 : Impact of irrigation , canal and weirs on water table: (a) pristine condition and (b) baseline condition***

In order to evaluate wastewater irrigation practice, through the purifying action of agricultural return flow through soil, the variably saturated zone above the groundwater level and the aquifer itself, as a natural treatment system, the first requirement is to know the movement of water through various zones and quantify the exchange of water among them through water balance analysis. The MIKE SHE water balance tool provides a detailed account of the water balance. The water balance in terms of mean annual flow (Million cubic meters, $Mm^3$) is presented in Table 16.1 which is self-explanatory and presents the losses and the return flows from different components of the system.

Overall groundwater flow gradient is towards the Musi River, gradient inverses locally due to pumping. Overland zone and saturated zone are interacting and exchanging water directly which is a typical feature of a wetland. Soil and saturated zone evaporation also add salinity apart from the reason due to wastewater application. Farmers apply water when the deficit reaches in the range 50 % (Maximum allowable deficit=0.5). Even though wastewater supply is continuous and free, and farmers are conscious of the benefits related to the free nutrients wastewater contains, water application in the area is limited mainly by two factors, (1) the energy need to lift water from canal to upland area and interruption of power supply and (2) the farmers' fear of unnecessary contact with poor quality water. Even though, over-irrigation and -pumping occur, especially in the paragrass and vegetable growing areas. In the irrigated area, the consumptive loss is about 25% of total inflow and the return flow is therefore, 75%. The modelling confirms the infiltration from Musi to aquifer in the upstream of the weir where ponding level is above the groundwater level. Seepage from the canal contributes to a rising water table and return flows. The stretch just downstream of the weir receives water from the aquifer (base flow) and is in gaining reach.

Table 16.1 Detailed mean annual inflows, losses and return flows in $Mm^3$

| Components | Pristine ( model domain-12.68 $Km^2$) | Baseline ( Model domain-12.68 $Km^2$) | Baseline ( Irrigated area-1.73$Km^2$) |
|---|---|---|---|
| **Inflow** | | | |
| Precipitation | 10.22 | 10.22 | 2.14 |
| Irrigation | 0.00 | 10.29 | 10.17 |
| Infiltration from canal & Musi river | 0.00 | 2.57 | 2.56 |
| **Total** | 10.22 | 23.08 | 14.87 |
| **Loss** | | | |

| | | | |
|---|---|---|---|
| Canopy evaporation | 0.55 | 0.62 | 0.15 |
| Overland evaporation | 2.44 | 5.26 | 2.95 |
| Soil evaporation | 0.71 | 0.64 | 0.13 |
| Saturated zone evaporation | 0.02 | 0.03 | 0.03 |
| Plant transpiration | 2.40 | 2.11 | 0.48 |
| **Total** | 6.12 | 8.67 | 3.73 |
| **Return flow** | | | |
| Direct surface return flow | 1.56 | 7.35 | 5.73 |
| Sub surface( interflow) | 0.16 | 5.11 | 4.27 |
| Base flow-through aquifer | 1.40 | 0.92 | 0.95 |
| Groundwater recovery | 0.97 | 0.97 | 0.17 |
| Storage | 0.00 | 0.04 | -0.01 |
| **Total** | 4.09 | 14.39 | 11.10 |
| Error | -0.01 | -0.02 | -0.03 |

The key system parameters for wastewater irrigation and the natural treatment related to agricultural return flow through the different milieus, soil, variably saturated zone and the groundwater body, are suitable topography and boundary conditions apart from other parameters like soil, geology, vegetation, irrigation practice etc. Human interference in terms of irrigation infrastructures has increased the area of the riverine wetland. Canal seepage and ponding at weirs have made a significant contribution to the rise in water table. Wastewater application on land has increased salinity but high water table might have also contributed to soil salinity. However, the positive outcome is that specific capacity of wells in and around the irrigated area has increased. The movement of wastewater in the wastewater irrigated area could be managed to protect native groundwater resources outside the wastewater irrigated system (soil aquifer treatment system) with well-planned recovery wells – appropriate locations , capacity, types and depths pumping schedules, etc , and artificial or natural collector drains – depth , size, locations etc. Therefore the share of groundwater and (wastewater containing) surface water use for irrigation should be optimised through watershed wide integrated modelling to maximize the benefit and minimize the negative impact of wastewater irrigation practices. Several agencies need to play an important role in encouraging and regulating wastewater irrigation practice in this area. For example the departments of irrigation and agriculture and state pollution control board

The distributed hydrologic modelling to the Musi wetland using MIKE SHE has demonstrated its ability to represent complex hydrological systems found within many wetland environments where groundwater, surface water interactions are common hydrological processes. The detailed water balance analysis helps us understand the movement and quantity of water from one level to other.

## 16.4 CONCLUSIONS

Different types of NT systems (river bank filtration, constructed wetlands and MAR) have been modelled in a large variety of geological and hydro-climatic settings, representative of the Indian subcontinent, thus demonstrating the utility of state-of-the art integrated surface-groundwater flow and transport models as planning and management tools.

Analytical or numerical models can be used at all stages of NTS implementation, from initial planning of individual systems, over upscaling at watershed scale to system optimisation to reach defined water quantity and quality targets. Such models enable water managers to test diverse scenarios so that they can be used to optimise implementation of NTSs within a watershed (which type? where? how big?). At local scale, models also may be useful for improving any individual NTS by fine-tuning technical options. Overall, they are management tools that help avoiding costly real-size trial and error testing of NTSs and also may avoid surprises with respect to the expected impact of NTSs on water quantity and quality.

The biggest challenge for modelling NTS systems is model integration. When looking on NTSs like constructed wetlands or percolation tanks (soil-aquifer treatment) we need to take into account surface runoff, the unsaturated soil zone, complex but crucial for water purification, the saturated groundwater flow and even the density driven saltwater flow in coastal aquifers. Water flow is a continuum but most currently available models are not yet able to treat it as such. One of the major advances in the Saph Pani was to establish integrated models that take into account the whole water cycle at watershed scale from surface over unsaturated to saturated and density driven flow. The project studies also integrated scales: Modelling NTSs needs both a close look on their behaviour at a very local scale but also upscaling to a watershed to simulate effects if a large number of them were implemented. A typical example is percolation tanks. Our observations at the Maheshwaram site showed that their extension in all three dimensions varies widely with rainfall from close-to-nil during the dry season to maximum extension during monsoon. Treating their geometry as constant over time is an oversimplification that can lead to erroneous results if we want to estimate their real impact on groundwater recharge. For this reason, a specific module was developed for the MARTHE software, already massively integrated with respect to all flow types (surface flow, unsaturated, saturated and density driven flow), able to simulate realistically the behaviour of infiltration tanks from rainfall and evaporation data and surface topography, simulating infiltration.

Another type of integration that revealed crucial was that of water flow with water quality changes. Here the most instructive example from the Saph Pani project is the simulation of ammonium transport from the heavily polluted Yamuna River, across the alluvial aquifer before reaching the wells that pump river bank filtrate. Ammonium breakthrough was first measured and modelled at laboratory scale, through percolation experiments in sediment columns, then upscaled to aquifer scale through reactive transport modelling. An important result is the considerable residence time of several decades of ammonium in the aquifer due to sorption onto the aquifer material.

Models have been developed for all three types of NTSs studied in Saph Pani, managed aquifer recharge combined with soil-aquifer treatment, constructed wetlands and river bank filtration. This has demonstrated how these approaches can be used for understanding, planning and optimising NTSs. The modelling tools used are widespread and accessible (e.g. MODFLOW, MARTHE, MIKE-SHE,…). Even though, integrated modelling of complex systems like NTS on different scales up to basin scale needs specialists trained in the application of those tools on the specific problems of NTS implementation in the Indian context and the knowledge and knowhow created in the project needs to be transmitted widely to young scientists and engineers through training programmes organised by the Indian institutions who were involved in the development of those methods within Saph Pani.

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