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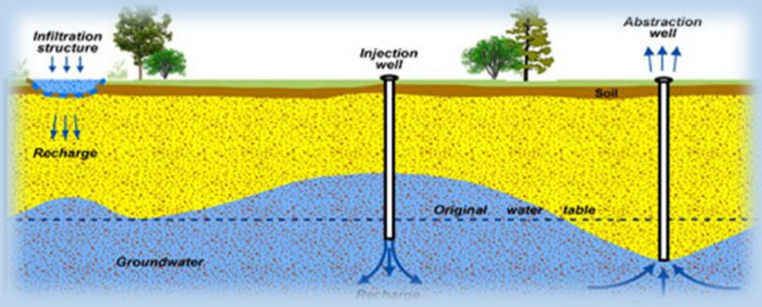
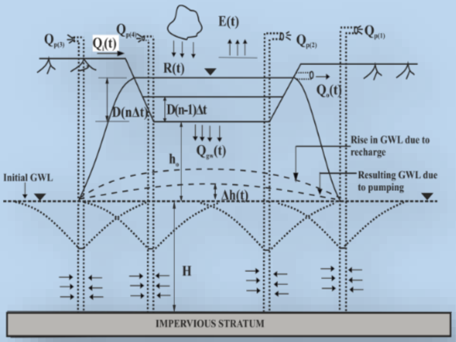
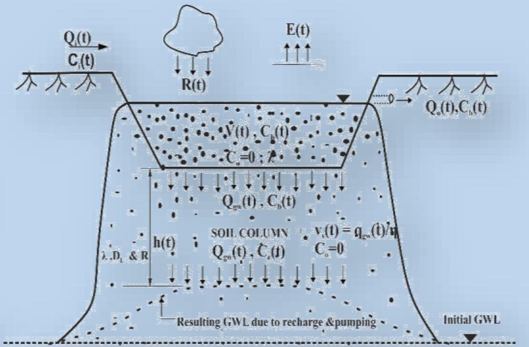
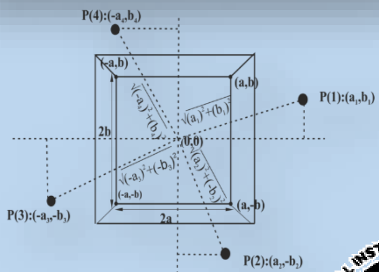
Web Enabled "Conjunctive Use Model for Management of Surface and Ground Water using concept of MAR and ASR"









NATIONAL INSTITUTE OF HYDROLOGY
JAL VIGYAN BHAWAN
ROORKEE - 247 667 (UTTARAKHAND)
2016-2018

NIH/GWD/NIH/16-16

Web Enabled "Conjunctive Use Model for Management of Surface and Ground Water using concept of MAR and ASR"

राष्ट्रीय जलविज्ञान संस्थान

आपो हि ष्ठा मयोभुवः

NATIONAL INSTITUTE OF HYDROLOGY
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PREFACE

Water resources management, in the backdrop of population growth, increasing pollution threat, rise of conflicts amongst stakeholders, climate change impacts, etc has become a challenging task in India. Surface water resources management has some constraints toward storages and retention, whereas groundwater being hidden resource its storages and management are somewhat difficult to comprehend than surface water resource. Replenishment of groundwater is reducing due to urbanization and moderation of surface land covers and depletion of groundwater levels is very common in many places. However, dependability on groundwater is increasing day by day. Natural aquifer recharge (NAR) that depends on soil and hydraulic properties of subsurface formation, etc is slow. Therefore, there is an urgent need for managed aquifer recharge (MAR) to augment groundwater resources in depleted aquifers. Water quality deterioration of both surface and ground water has posed a big gap in management of water between availability and demand. MAR together with natural treatment techniques appears to be a way forward to resolve the issue of water management, both quantity and quality.

The study titled as "Web-enabled Conjunctive Use Model for Management of Surface and Ground Water using concept of MAR and ASR" is a step towards seeking solution for integrated management of surface and ground water by building a web based computational platform, which will perform the required calculations satisfying the guiding principles of MAR and ASR (Aquifer Storage Recovery). This model is second in its series with the first one named as; WEGREM is already in the public domain. The present model is improved the WEGREM by additional capabilities to calculate recharge influenced by pumping and contaminant transport beneath the basin.

The study was carried out, and the web based computational platform was developed by a team of scientists from the Ground Water Hydrology Division with Ms. Suman Gurjar, Scientist 'C' as the lead and Dr. N. C. Ghosh, Scientist 'G' as the guide and other team members include Er Sumant Kumar, Scientist 'C', Dr Anupma Sharma, Scientist 'E' and Dr Surjeet Singh, Scientist 'E'. I put on record my appreciation for this good piece of work.

Place : Roorkee
Date : 6-02-2019

(Sharad Kumar Jain)
Director, NIH

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ABSTRACT

A comprehensive user friendly web-enabled "Conjunctive Use Model for Management of Surface and Ground Water using concept of MAR and ASR" has been developed using semi-analytical approaches for management of surface and ground water in a recharge basin, based on the concept of managed aquifer recharge (MAR) and aquifer storage treatment and recovery (ASTR). The model for simulation of aquifer responses due to recharge and extraction of recharged water has been developed by integrating the hydrologic components into the basic water balance equation; and the models for simulation of contaminants' fate in the recharge basin and through the soil column underneath have been developed by considering: (i) in-basin mass balance with decay of contaminant and, (ii) 1-dimensional advection-dispersion-decay equation coupled with linearized sorption isotherm kinetics, respectively. The estimate of hydrologic components included; inflow to the recharge basin from its catchment by SCS-CN model, water surface evaporation by combination of Priestley-Taylor and Penman method, recharge by Hantush's basic equation for water table rise due to recharge from a rectangular spreading basin in absence of pumping well, and drawdown due to pumping by Theis's well function equation. The contaminant's fate estimate included; time varying changes in concentration due to assimilation and detention of contaminant in the recharge basin and transport of assimilated materials through saturated soil column until they reached the groundwater table. Duhamel's convolution equation and method of superposition have been used to obtain the resulting water table position due to pumping and recharge, and also for computation of contaminants' fate. The performances of recharge-pumping and contaminants' transport models have been illustrated by examples. These models can successfully be used and upscale as potential tools for MAR and ASTR. The web-enabled GUI interface is an efficient tool to provide a platform to users and professionals for calculating time-varying depth of water in, and groundwater recharge from, a recharge basin consequent to the pumping in the vicinity of the basin for recovery of recharged water. This could also allow users to simulate the contaminant transport process in the basin and through the saturated soil column before mixing with the groundwater. Therefore, a web-enabled GUI interface has been developed for general access of the models. Using the web-enabled interface, users can use the models for computation and to visualize the outputs in graphical as well as tabular format.

Chapter 1

Introduction

India has diverse geological, geomorphological and hydro meteorological conditions, which outline the widely varying groundwater resource. Groundwater plays an important role in India by supporting about 85% of rural domestic needs and 50% of urban and industrial needs and about 65% of irrigation water requirements. India is the highest groundwater user in the world and thus faces the most groundwater challenges due to water scarcity because of its growing demands driven by population growth, economic development and the need for food..To meet these demands, groundwater is over pumped which have led or are leading to problem of groundwater scarcity in many aquifers.Increasing demand for water has resulted into over dependence on groundwater (GW), especially in regions where surface water resources are limited and temporal rainfall is uneven. Exploitation of groundwater for various purposes has resulted in the depletion of groundwater resources and rapid decline in water table in several parts of India. In order to balance the overdraft, it is necessary to increase the groundwater recharge to augment the storage of groundwater in depleted aquifers and also for improving the groundwater quality through dilution. Natural groundwater replenishment is slow because of lower hydraulic properties than the surface water storage and movement. The slower replishment of groundwater may not keep pace with excessive abstraction of groundwater resources.

To tackle the twin problems of de-saturation of aquifer zones and consequent deterioration of groundwater quality, augmentation of groundwater resources through suitable management interventions is essentially required. Managed aquifer recharge (MAR), popularly known by artificial recharge in India has now been accepted world-wide as a cost-effective method to augment groundwater resources in areas where continued overexploitation without due regard to recharge has resulted in various undesirable environmental consequences.

The conjunctive use of surface and ground water is one of the strategies for water management that has to be promoted for sustainable water resources development, management and conservation within a basin. Conjunctive use refers to the practice of sourcing water from both ground and surface water for demand-side management, and conservation of surface runoffs for aquifer recharge as groundwater development. Accordingly, conjunctive use can be characterised as being planned (where it is practiced as a direct result of management intention) compared with spontaneous use (where it occurs at a grass roots level). The planned conjunctive use of ground and surface water has the potential to offer benefits in terms of economic and social outcomes through significantly increased water use efficiency. At the resource level, groundwater pumping used in conjunction with surface water provides benefits that increase the groundwater supply or mitigate undesirable fluctuations in the supply and the issues like over-exploitation and water-logging.

Depleting level and deteriorating quality of groundwater from anthropogenic and geogenic sources of contaminants are common concerns in India (Shah *et al*, 2002; Shah, 2007; Mukherji and Shah, 2005; Mukherjee *et al*, 2015). The depletion of groundwater level (GWL) is mainly because of over exploitation resulting by growing demands, changing weather patterns, increasing pollution of surface water bodies, etc (IWMI, 2006, CGWB, 2007), on the other hand, the deteriorating GW quality is mainly due to the geochemical changes and trailing functions of the former one. Managed aquifer recharge (MAR) (Dillion, 2005; Gale, 2005, 2006; Dillion *et al*, 2009), also known as artificial recharge (AR), as a potential method for augmenting groundwater resources in depleted GWL area is encouraged in many countries, including India (CGWB, 1996; 2002; 2007; 2013). MAR by conserving excess surface runoff from monsoon rainfall in catchment scale can help provide an effective method for conjunctive management of surface, ground and recycled water to stretch water supplies, and can form a part of 'Integrated Water Resources Management (IWRM)' approach (Biswas, 2004; GWP, 2005; 2009). MAR together with aquifer storage, treatment and recovery (ASTR) (EPA, 2009), nearly covers the main 'quantity and quality' issues of IWRM, except its operational, management, economic and stakeholders parts (GWP, 2000; UNESCO, 2009).

India has a long tradition of practicing 'Rainwater Harvesting (RWH)' and AR by employing indigenously developed techniques and methods to fulfill requirements of agricultural and drinking water supply particularly, in rural areas. For the last one and a half decade, RWH and AR are promoted as the government supported national program (CGWB, 2007 and 2013; MoWR, 2012) for augmentation of groundwater resources in water stressed and groundwater problematic areas (SAPH PANI, 2012). India's MAR practices are mainly focused on how to plan, construct and operate MAR structures. There is large knowledge gap on social, economic and water quality aspects of MAR and on how best to organize the construction, maintenance and the most use of recharged water (Ghosh, 2013). MAR together with ASTR can address a number of scientifically challenged issues (Dillion *et al*, 2009), such as; identification of suitable location of pumping wells for aquifer storage recovery such that, pumping rate ≤ recharge rate, fate of contaminants' as they move beneath the recharge area till reaches the groundwater table, distance of the wells from the recharge area to satisfy the travel time for required treatment of the recharged water, etc.

There are different types of MAR structures practised all over the world depending upon the site conditions, hydrogeology, and water source. Amongst those, *'recharge basin or percolation pond'is* most commonly used MAR structure in India particularly in plain region. Keeping in view its widescale use in India, the study is focused to : (i) develop a generalized process based conjunctive use model for management of surface and ground water in/from a recharge basin considering both groundwater recharge and extraction, (ii) develop contaminant transport models for the in-basin assimilation and movement of contaminants through the soil column underneath the basin for variable recharge rates considering processes of advection-dispersion-decay-sorption, (iii) analyze performances of the developed models, and (iv) use of the models as the computational tool for MAR and ASTR. Finally to host this developed MAR and ASTR model as a web-enabled platform along with a graphical user interface to users and professionals for calculating time-varying depth of water in , and groundwater recharge from, a recharge basin consequent to the pumping in the vicinity of the basin for recovery of recharged water. This web-enabled platform will also allow users to simulate the contaminant transport process in the basin and through saturated soil column before mixing with the groundwater.

Chapter 2

Managed Aquifer Recharge (MAR) & Aquifer Storage and Recovery (ASR)

MAR describes intentional storage and treatment of water in aquifers for subsequent recovery or environmental benefits (Dillion et al, 2009; Sharma, et al., 2011). It also includes the techniques, Soil Aquifer Treatment (SAT), Aquifer Storage and Recovery (ASR), Aquifer Storage Transfer and Recovery (ASTR). The term “Artificial Recharge (AR)” commonly used in India also describes the similar activity as in MAR without consideration of quality of water resources, however, the term ‘MAR’ so far has not been as popular as the term **‘**AR**’**.

In MAR, aquifers, permeable geological strata that contain water, are replenished naturally by rain soaking through soil and rock to the aquifer below or by infiltration from streams. There are two categories of the aquifer recharge, unmanaged and managed aquifer recharge. Unmanaged recharge includes storm water drainage wells and sumps, and septic tank leach fields, usually for disposal of unwanted water without thought of reuse. Managed recharge are done through some mechanisms such as, injection wells, and infiltration basins and galleries for rainwater, storm water, reclaimed water, mains water and water from other aquifers that is subsequently recovered for all types of uses. With appropriate pre-treatment before recharge and sometimes post treatment on recovery of water, it may be used for drinking water supplies, industrial water, irrigation, toilet flushing, and sustaining ecosystems (Dillon et al, 2009).

The major reason for implementing and using MAR are to: secure and enhance water supplies, improve groundwater quality, prevent salt water intrusion into coastal aquifers, reduce evaporation of stored water, etc. MAR can be used to address a wide range of water management issues as depicted in Figure 2.1. Although, MAR has several advantages, a number of constraints and disadvantages, but it is promoted woprldover as the most promising technique for replenishing and re-pressurising depleted aquifers and also augmentation of groundwater resource by recharge of conserved excess monsoon surface runoffs and recovery of augmented water by the concept of aquifer storage and treatment.

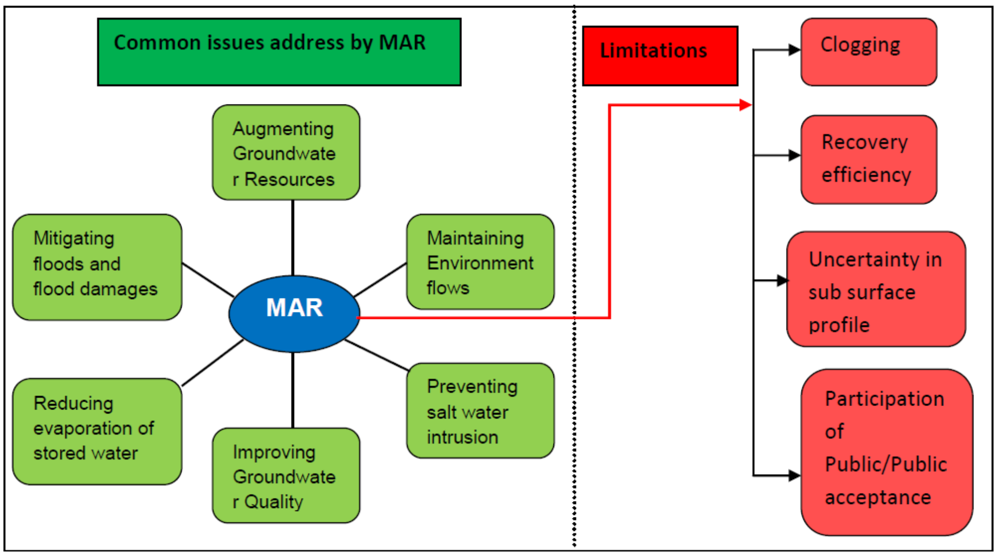

Fig 2.1: The common issues that can be addressed by MAR and its limitations

2.1 Structures of MAR

A wide range of methods are used for recharging aquifer to meet variety of local problems. These recharging structures are known by different names in different countries. However, they can broadly be divided into three main groups: surface-spreading, run-off conservation and sub-surface structures. Fig 2.2 covers a list of main types of structures mainly practiced in India (SAPH PANI, 2012).

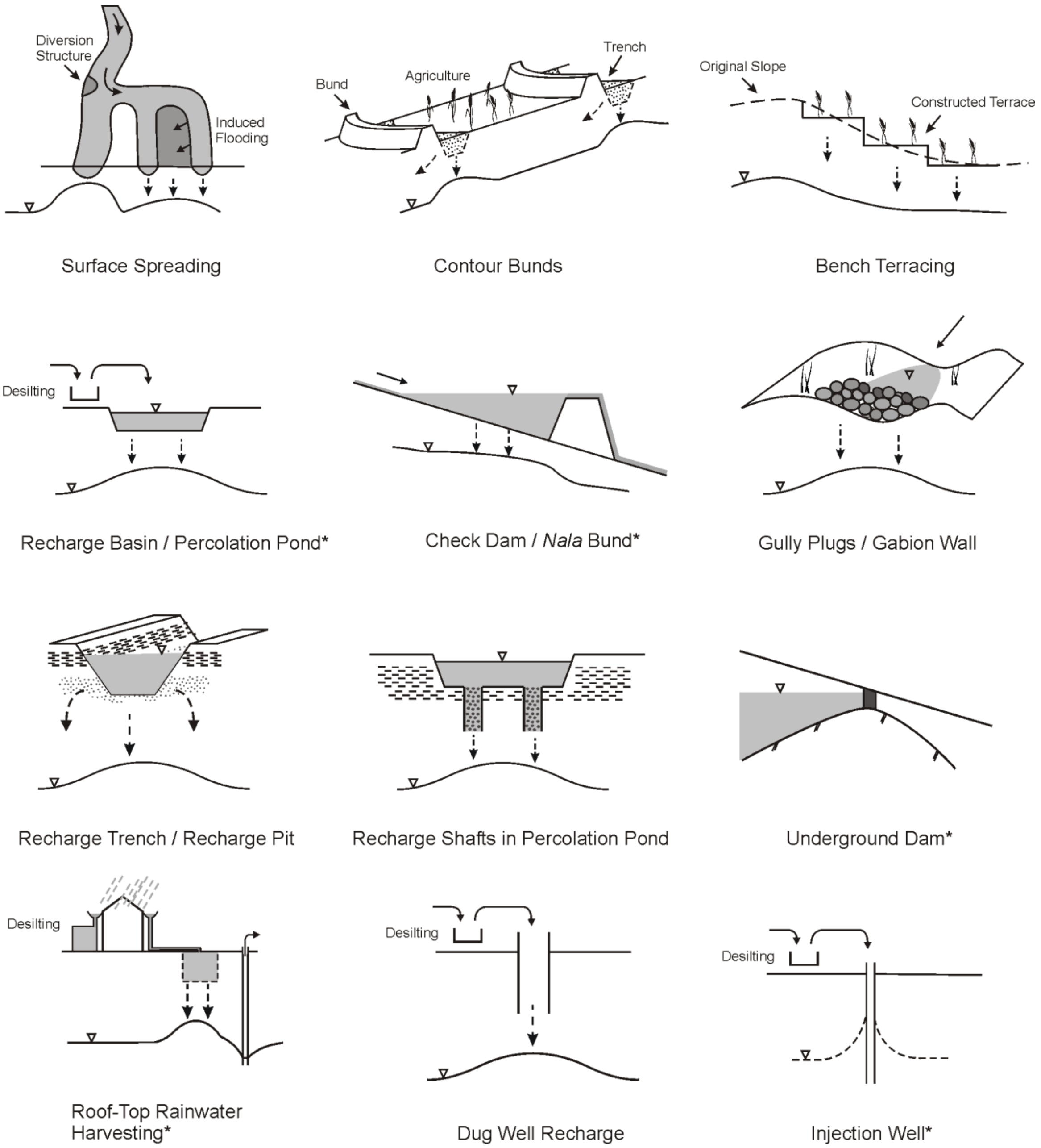

Fig 2.2: Different types of MAR structures commonly used in India (source: Saph Pani, 2012).

Selection of a suitable structure for MAR depends on the various factors like. total amount of surface runoff available, hydrogeology, topography, hydrology, land uses and socio economic conditions of the area. Amongst the wide range of methods used internationally and also in India for aquifer recharge as in Fig 2.2, the common methods employed for aquifer recharge are infiltration pond/recharge basin and different approaches of aquifer recharge and storage recovery. The description of few important methods are illustrated by Fig 2.3 below:

Aquifer storage and recovery (ASR): Injection of water into a well for storage and recovery from the same well. This is useful in brackish aquifers, where storage is the primary goal and water treatment is a smaller consideration.

Aquifer storage, transfer and recover (ASTR): Involves injecting water into a well for storage, and recovery from a different well This is used to achieve additional water treatment in the aquifer by extending residence time in the aquifer beyond that of a single well.

Infiltration ponds: Involve diverting surface water into off-stream basins and channels that allow water to infiltrate through unsaturated zone to the underlying unconfined aquifer.

Infiltration galleries: Underlying trenches in permeable soils to allow infiltration through the unsaturated zone to an unconfined aquifer.

Soil aquifer treatment (SAT): Treated sewage effluent is intermittently infiltrated through infiltration ponds to facilitate nutrient and pathogen removal in passage through the unsaturated zone for recover by wells after residence in the unconfined aquifer.

Percolation tanks or recharge weirs: Check dams built in ephermal streams detain water which infiltrate through bed to enhance storage in unconfined aquifers.

Rainwater harvesting for aquifer storage: Rooftop rainwater runoff is diverted into a well, sump or cassion filled with sand or gravel allows to percolate to water table where it is collected to pumping from a well.

Bank filtration (BF): Extraction of groundwater from a well or cassion near or under a river or lake to induce infiltration from the surface water body thereby improving the quality of water recovered.

Dry wells: Typically, shallow wells where water table is very deep, allowing infiltration of very high quality water to the unconfined aquifer.

Dune filtration: Infiltration of water from ponds constructed in dunes and extracted from wells or ponds at lower elevation for water quality improvement and to balance supply and demand.

Underground dams: In ephemeral streams where basement highs constrict flows, a trench is constructed across the streambed, keyed to the basement and backfill with low permeability material to help retain flood flows in saturated alluvium for stock and domestic use.

Surface Spreading

Surface spreading structures aim to increase the area which is in contact with surface water and also the time over which this contact takes place. In this way infiltration is improved and evaporation decreases. This can be achieved through managed flooding between constructed canals or streambeds or by constructing a system of ditches and furrows.

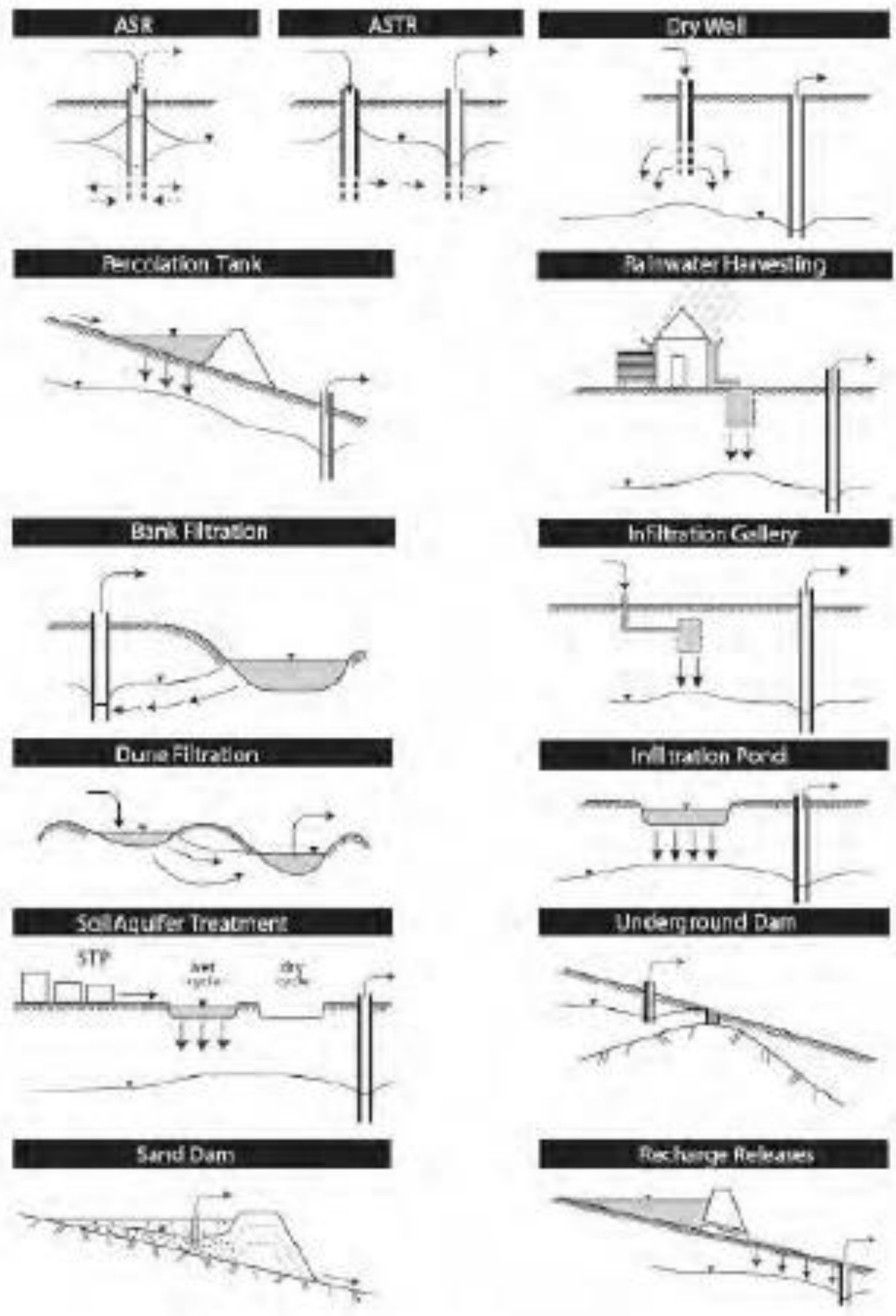

Fig 2.3: Schematic of types of Managed Aquifer Recharge (Source : Dillion, 2005)

Contour Bund and Contour Trench

A bund is an embankment of earth. Contour bunds and trenches break the flow of water and thus increase infiltration and limit erosion. They are constructed along contours of equal land elevation. Between two contours, agriculture can be practiced and tree plantation on the bund is possible. Bunds trees/ plants can help fix nitrogen in to the soil for the crop plants. During rainfall the contour bund acts as a barrier to the water flow, reducing the speed of run-off water thus also the washing out of nutrients.

Bench Terracing

Bench terracing is practiced in hilly areas where the original slope is levelled stepwise by cutting and filling. Under suitable conditions the structure helps to reduce surface run-off and enhances soil moisture conservation, crop production and aquifer recharge.

Percolation or Infiltration Pond or Tank and Recharge Basin

Percolation tank or pond is a term used in India to describe harvesting of water in storages built in ephemeral streams or off-stream where water is detained and infiltrates through the permeable base to enhance storage in unconfined aquifers. Recharge basins differ from percolation ponds in that they are designed to accommodate a flow through a series of basins not retaining the whole amount of water in a single basin like in a percolation pond. For both types of structures, the water is usually desilted to prevent clogging.

Check Dam, Nala Bund and Gully Plug

Check-dams are barriers built across the direction of water flow of rivers. These dams retain part of the water flow during monsoon rains in the area upstream of the structure. The increased pressure in the reservoir area increases the infiltration rate.

Gully Plug and Gabion Wall

Gullies are formed due to erosion of top soil by the flow of rain water. Gully plugs are built with local stones, sand, clay and plants. It is a simple technique for conservation of soil and moisture by reducing the speed of run-off water during floods. Gabions are wire mesh baskets filled with rocks and have a permeable, flexible structure. In connection with water management gabions walls are used often for erosion control, bank stabilization, channel linings and weirs. Gabion walls reduce the speed of run-off water. They are also constructed to protect the bank of lakes and rivers against the erosion due to water and waves. Sludge and small stones deposit in the interstices, leading to growth of vegetation and ultimately a natural reservoir is formed. It retains water for dry periods to serve agriculture and replenishes groundwater.

Recharge Pit

Recharge pits are dug out pits and trenches which have been dug through a layer of low permeability to improve infiltration to a shallow phreatic (unconfined) aquifer. They differ from percolation ponds and recharge basin in that they are deeper and frequently recharge takes place through the sides of the pit. Abandoned mine shafts and quarries are often converted to recharge pits if they are in contact with an underlying aquifer.

Recharge Shaft

Recharge shafts like recharge pits are recharge structures which penetrate an upper layer with low permeability into the underlying phreatic aquifer. They are constructed at the bottom of surface structures (ponds/tanks/channels) which do not connect to the permeable layer. In contrast to injection or recharge wells they are backfilled with coarse sand and stones thereby creating columns of porous, permeable soil which connect the recharge pit to the aquifer.

Injection Well or Recharge Well

Injection wells are tube wells constructed for the purpose of recharge. Injection wells are primarily used to recharge deep lying aquifers and the water is injected under pressure or using gravity alone. Many of them are constructed with slotted PVC pipe and surrounded with some kind of clogging protection.

Underground Dam

Underground dams are built in ephemeral streams where basement ridges constrict flows. A trench is dug across the streambed keyed to the basement and backfilled with low permeability material to help retain flood flows in saturated alluvium for stock and domestic use.

Rooftop Rainwater Harvesting Structure

Rooftop Rainwater harvesting collects and infiltrates the roof runoff from buildings. Most commonly injection will take place through dug or bore wells, but recharge through percolation ponds is also possible.

Dug Well Recharge

Dug wells which have run dry can be adapted for use as recharge structures. This is done by diverting surface water into the well. It is common to desilt the water before infiltration to avoid clogging.

2.2 Factors influencing feasibility and performance of MAR

2.2.1 Hydrogeology

The hydrogeology determines MAR feasibility and is the decisive factor for selecting the optimum location and suitable structure. The aim for MAR scheme is to identify aquifers that store large quantities of water and do not release them too quickly. Scientifically, the vertical hydraulic conductivity should be high, while the horizontal hydraulic conductivity should be moderate. However, coexistence of these two conditions is rare case in natural geologic settings. The main hydrogeological factors which influence MAR are geological boundaries, hydraulic boundaries, inflow and outflow of waters, storage capacity, porosity, hydraulic conductivity, transmissivity, natural discharge of springs, natural recharge, lithology, and depth of the aquifer etc.

2.2.2 Climate and Hydrology

Climatic conditions in the application site have an important role in determining the dimensions and type of structures that need to be implemented. The climatic factors are mean annual rainfall, number of rainy days, frequency of high intensity rainfall, variability in temperature etc. Hydrology is an important factor in locating appropriate areas for MAR and also in determining the amount of water available for recharge. The most important hydrological characteristics that influence MAR are terrain characteristics, land-uses, vegetation cover, flow availability and rate in streams, conveyance system for bringing the water to recharge structure.

2.2.3 Water Quality

Water quality concerns can come from naturally occurring or human induced contaminants; which can limit the use and availability of groundwater. Declining quality of the groundwater commonly cannot support all agricultural, industrial and urban demands and ecosystem functioning. Generally poorer quality source waters will need a higher level of treatment before recharge. The identified site for groundwater recharge structure, the groundwater itself shouldn't be contaminated.

2.3 Ground Water Contamination

Water in nature, on the surface or underground, is never free from impurities and typically contains many dissolved and suspended constituents (salts, other inorganic and organic chemicals, sediments, and microorganisms). Contamination of a water body or an aquifer occurs when the concentration of one or more substances increase to a level such that the resulting water quality undermines the use of resource and, in some instances, becomes a hazard to the environment and a risk to human, animal, or plant life (Morris et al. 2003). The principal causes of groundwater contamination due to human activity can be classified as agricultural, industrial, and urban (Foster et al. 2002). Human activity can add salts, chemicals, and microorganisms (pathogens) that affect quality of groundwater.

Ground and surface water are interconnected and can be fully understood and intelligently managed only when that fact is acknowledged. If there is a water supply well near a source of contamination, that well runs the risk of becoming contaminated. If there is a nearby river or stream, that water body may also become polluted by the ground water.

Depending on its physical, chemical, and biological properties, a contaminant that has been released into the environment may move within an aquifer in the same manner that ground water moves. It is possible to predict, to some degree, the transport within an aquifer of those substances that move along with ground water flow. For example, both water and certain contaminants flow in the direction of the topography from recharge areas to discharge areas. Soils that are porous and permeable tend to transmit water and certain types of contaminants with relative ease to an aquifer below. Just as ground water generally moves slowly, so do contaminants in ground water. Because of this slow movement, contaminants tend to remain concentrated in the form of a plume that flows along the same path as the ground water. The size and speed of the plume depend on the amount and type of contaminant, its solubility and density, and the velocity of the surrounding ground water.

In areas surrounding pumping wells, the potential for contamination increases because water from the zone of contribution, a land area larger than the original recharge area, is drawn into the well and the surrounding aquifer. Some drinking water wells actually draw water from nearby streams, lakes, or rivers. Contaminants present in these surface waters can contribute contamination to the ground water system. Some wells rely on artificial recharge to increase the amount of water infiltrating an aquifer, often using water from storm runoff, irrigation, industrial processes, or treated sewage. In several cases, this practice has resulted in increased concentrations of nitrates, metals, microbes, or synthetic chemicals in the water. Under certain conditions, pumping can also cause the ground water (and associated contaminants) from another aquifer to enter the one being pumped. This phenomenon is called inter-aquifer leakage. Thus, properly identifying and protecting the areas affected by well pumping is important to maintain ground water quality. Generally, the greater the distance between a source of contamination and a ground water source, the more likely that natural processes will reduce the impacts of contamination. Processes such as oxidation, biological degradation (which sometimes renders contaminants less toxic), and adsorption (binding of materials to soil particles) may take place in the soil layers of the unsaturated zone and reduce the concentration of a contaminant before it reaches ground water. Even contaminants that reach ground water directly, without passing through the unsaturated zone, can become less concentrated by dilution (mixing) with the ground water. However, because ground water usually moves slowly, contaminants generally undergo less dilution than when in surface water.

2.4 Aquifer Storage and Recovery

The term ASR is attributed to Pyne (1995): “Aquifer Storage and Recovery may be defined as the storage of water in a suitable aquifer through a well during times when water is available, and recovery of the water from the same well during times when it is needed”.

ASR can be used as a resource management tool where water from a source is treated and then stored underground (Fig 2.4). Large volumes of water may be stored underground thereby reducing the need to construct expensive surface reservoirs. ASR can also have added benefits in aquifers that have experienced long-term declines in water levels as a result of concentrated and heavy pumping. Groundwater levels can be restored if sufficient quantities of water are recharged.

There are a number of methods whereby the natural recharge processes can be accelerated and collectively be referred to as recharge enhancement or artificial recharge. ASR relates specifically to enhanced recharge using a well and is typically associated with deeper confined aquifer systems (Fig 2.5(a)). Individual injection and recovery wells (Fig 2.5(b)) can be used where groundwater quality is fit for intended use and the distance separating the wells provides opportunities for attenuation of contaminants. Infiltration basins are another method whereby water is collected in carefully constructed holding ponds and allowed to infiltrate through the base of the ponds to shallow water table aquifers (Fig 2.5(c)). Bank filtration is a third method whereby pumping wells adjacent to a watercourse are used to draw water from a stream into the aquifer (Fig 2.5(d)).

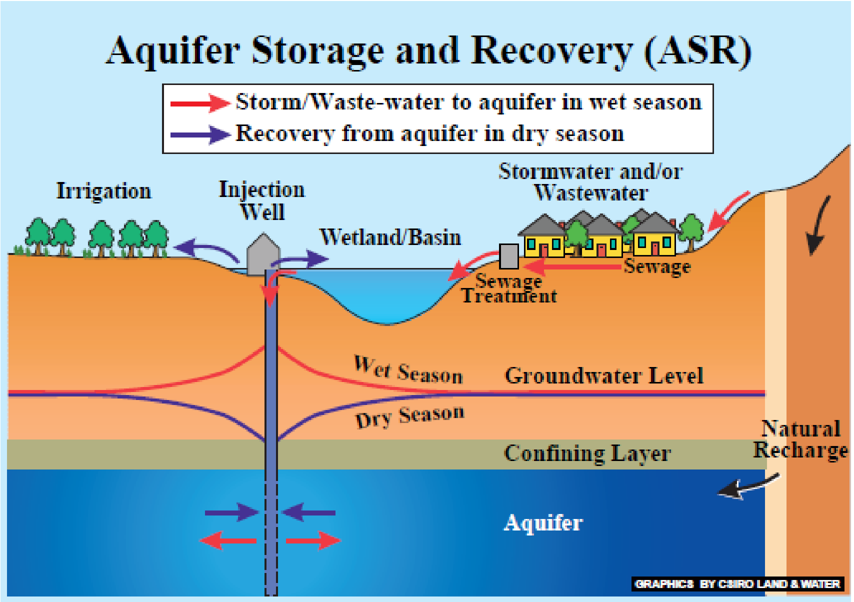

Fig 2.4. Schematic depiction of ASR. Stormwater or reclaimed water is recharged during the wet season and recovered in the dry season.(after Dillon *et al*, 2000)

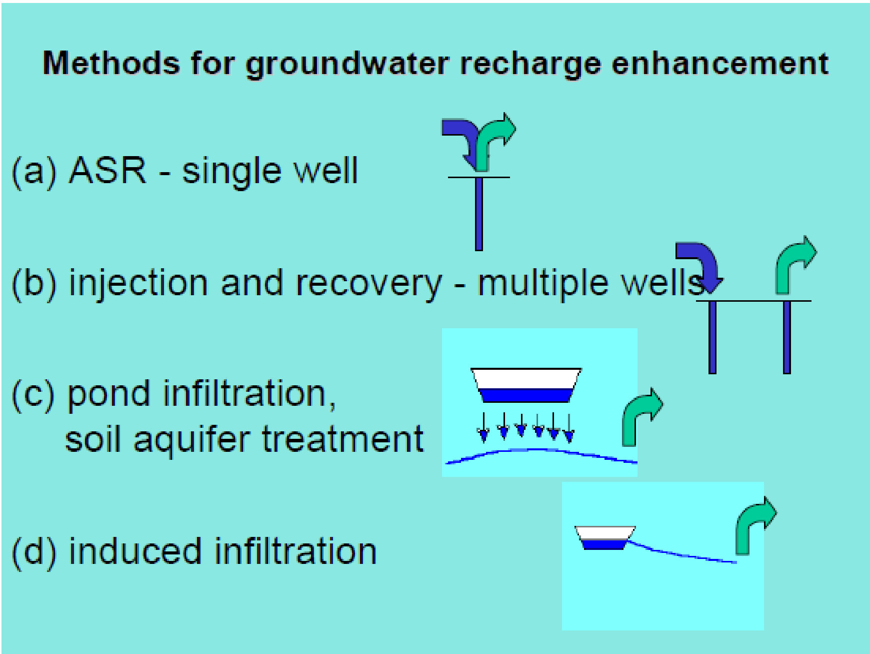

Fig 2.5: Types of groundwater recharge enhancement (a) ASR, (b) separate injection and recovery wells, (c) infiltration basins, and (d) bank filtration

2.4.1 Recharge wells

Recharge wells are typically the most difficult and expensive method of recharge enhancement. Recharge wells are used where the shallow lithology does not possess characteristics suitable for aquifer storage and recovery such as, low transmissivity or where land has such a high value that above ground storage ponds are not economically viable. On-going management and maintenance costs associated with the operation of recharge wells are also high. Appropriate well construction coupled with careful management is required to ensure its maximum life (up to 20 years). If any one of these aspects is ignored and the operator neglects to manage the system appropriately, failure of the ASR scheme will result or alternatively expensive remediation of the injection well will be necessary.

2.4.2 Infiltration basins

Infiltration basins are typically used where there is sufficient depth of sediment between the base of the infiltration pond and the underlying watertable and there are no low permeability layers. Infiltration occurs through the base of the pond with the infiltration driving force controlled by the depth of water in the pond. Infiltration basins are employed where the water source to be captured is stormwater runoff and there is sufficient room to establish the basins, often alongside the watercourse. Recovery of the stored water from these systems is generally via shallow collector wells.

2.5 Why is recharge basin selected for the study?

There are various structures (Figures 2.2 & 2.3) which can be used for recharging groundwater. Amongst these, *recharge basin* is the most prevalent structure in India as a measure to recharge the ground water. Recharge basin captures water from the upstream catchment and uses surplus surface runoffs for infiltration-percolation and recharge to the underneath aquifer as groundwater storage. When the source has the characteristics of poor water quality, the sub-surface formations act as natural filter to remove many physical, biological, and chemical pollutants from the water as it moves through the soil column. Water quality improvement, which is one of the main objectives of recharge, takes place naturally by the soil and aquifer treatment through additional treatment to the source water. Recharge basin can be of different sizes and shape viz., rectangular, trapezoidal, circular, etc. Depending upon the availability of land, hydrogeology of the area, scope of operation and maintenance, etc the size and shape are chosen.

Chapter 3

Theoretical Considerations

Groundwater recharge rate beneath a recharge basin depends on the potential head difference between the in-basin depth of water and the height of groundwater level at a given time, and the sub-surface soil permeability beneath the basin. The recharge rate can be computed by the Darcy's equation. Recharge from the basin evolves groundwater mound below the basin with its focal point at the centre. The evolution of mound depends on the recharge rate and the horizontal hydraulic conductivity. The changes in the potential water head difference make the recharge rates time variant. Pumping of groundwater around the basin creates drawdown that results into decrease in groundwater level. The recharge rate from the basin at a given time, in such case, depends on the head difference between the in-basin depth of water and the resulting groundwater level below it. While recharge from the basin increases the groundwater level, on the contrary, pumping depletes the groundwater level. The resulting rise/fall of groundwater level at any location below the recharge basin at a given point of time is the sum of groundwater levels describe by the recharge and pumping. Hantush (1967) gave an approximate analytical equation for the rise/fall of water table in a homogeneous and isotropic unconfined aquifer of infinite areal extent due to uniform percolation of water from a spreading basin in absence of a pumping well. Theis (1935) gave the analytical solution for drawdown due to pumping. Hantush's solution together with Theis's equation can be clubbed to obtain the resulting groundwater level position in a homogeneous and isotropic unconfined aquifer, which has horizontal water level and is in dynamic equilibrium prior to onset of the recharge or pumping.

The surface water source may contain contaminants of concern. In-basin accumulation of contaminants and recharge of accumulated contaminants may affect quality of groundwater. The fate of contaminants as they assimilate in the basin and move through the soil pores below the basin before mixing with the groundwater depends on the in-basin detention time, seepage velocity through soil pores, dispersivity, kinetics of contaminants, soil properties and heterogeneity, etc. The fate of influent contaminants will first be influenced by the in-basin assimilation and variable time of detention. Thereafter, the variable seepage velocity (= recharge rate/porosity) due to change of in-basin hydraulic conditions will make the contaminants' movement faster/slower and will change the dispersive characteristics of contaminants as well, which will result in variable travel time of contaminants. Travel time governs the fate of non-conservative contaminants. Further, rise/fall of the groundwater level below the basin changes the distance between the bed of recharge basin and groundwater level, which will result into longer/shorter flow path of contaminants. The soil particles beneath the recharge basin may have sorbing characteristics of contaminants. To simulate the in-basin transport processes and fate of contaminants through saturated soil column before mixing with the groundwater, the conservation equation with decay for the in-basin mass transport, and the analytical solution of 1-Dimensional advection-dispersion equation (ADE) given by Ogata and Banks (1961) together with the decay and Freundlich linear adsorption isotherm equation for contaminant transport through saturated soil column can be used.

3.1 *Statement of the Problem*

The schematic diagram (Fig.3.1(a)) shows different hydrological components associated with a trapezoidal recharge basin. Let $Q_i(t)$, $Q_o(t)$, $R(t)$, and $E(t)$ be the hydrological components representing respectively, runoff from the basin catchment, (L^3T^{-1}); outflow from the basin (L^3T^{-1}); rainfall over the basin, (LT^{-1}); and water surface evaporation from the basin, (LT^{-1}). The recharge basin has porous bed material and hydraulic passage between the basin and aquifer that allows infiltrated water to recharge to the underneath aquifer. Let K_v be the vertical hydraulic conductivity of the porous bed materials that have uniform permeability. Let K be the hydraulic conductivity, (LT^{-1}), of the underneath unconfined aquifer having homogeneous and isotropic properties and S be the storage coefficient (dimensionless). Let $Q_{gw}(t)$ be the groundwater recharge, (L^3T^{-1}), from the basin to aquifer due to water head difference between the basin and groundwater level underneath. Let $P(1)$, $P(2)$, $P(3)$ and $P(4)$ be the four production wells with coordinates respectively of : (a_1,b_1), (a_2,b_2), (a_3,b_3) *and* (a_4,b_4) being operated for recovery of aquifer storage (Fig.3.1(b)). Let H be the height of initial groundwater level measured upward from the impervious stratum, and h_0 be the height of sub-surface formation measured between the basin bed and initial groundwater level. Let the recharge basin be trapezoidal in shape having base dimension of 2a (length) and 2b (breadth) and side slope 1V:1H. The origin (0,0) of the axes is located at the centre of the basin and the coordinates of the four base corners are as shown in Fig.3.1(b). With reference to the centre of axes, the coordinates of the four pumping wells are, $P(1):(a_1,b_1)$; $P(2):(a_2,-b_2)$; $P(3):(-a_3,-b_3)$, and $P(4):(-a_4,b_4)$. Let the resulting height of the soil between the basin bed and the groundwater level be $h(t)= h_0 - \Delta h(t)$; $\Delta h(t)$ is the resulting rise/fall of groundwater level due to recharge and pumping. It is assumed that initially the groundwater level below the recharge basin is in dynamic equilibrium and horizontal. The time of recharge to the underneath groundwater level is reckoned since onset of time, t = 0, and all associated hydrological components are reckoned since t=0. Further, it is assumed that water to be pumped from the groundwater is in accordance with the some sectoral demands.

Fig.3.2 represents the parametric description of in-basin and through saturated soil column mass transport processes. Let contaminants enter into the basin along with the inflow be non-conservative in nature with influent concentration, $C_i(t)$, (ML^{-3}), and let the initial concentration of the respective contaminant in the basin and soil column be C_0, (ML^{-3}). Let the in-basin contaminants be governed by decay/growth, and the movement and transport of contaminants through the underneath saturated soil column from the basin be governed by dispersive characteristics, growth/decay, and sorption kinetics. Let $C_b(t)$, (ML-3), be the in-basin time-varying concentration of contaminant, which is also the influent concentration of contaminant through the saturated soil column. Let λ, (T^{-1}), be the decay/growth rate coefficient ; D_L, (L^2T^{-1}), be the longitudinal dispersion coefficient = $\alpha\, v_s^m$, in which α is the dispersivity, (L) and m is an exponent (dimensionless); R (dimensionless) be the distribution of sorbed material; and $v_s(t)$, (LT^{-1}) be the seepage velocity of the recharging water = $q_{gw}(t)/\eta$, in which $q_{gw}(t)$ is the Darcy's velocity, (LT^{-1}) and η is the porosity (dimensionless). Let $C_e(t)$, (ML^{-3}), be the effluent concentration of contaminant from the soil column before mixing with the groundwater. Let the initial concentration of contaminant, both in-basin and in the soil column, at time t =0, be zero.

It is reckoned that the groundwater recharge starts simultaneously from time, t = 0, and so is for the transport of contaminants. The soils beneath the recharge basin are considered to be homogeneous and isotropic with movement of water along the vertical direction only. It is intended to develop analytical models for determining time varying : (i) recharge rate, $Q_{gw}(t)$ owing to the interaction of different hydrological components in the recharge basin and enhanced recharge rate due to the pumping around the basin to recover recharge water, when all the hydrological components are time variant; (ii) variable depth of water in the basin, and (ii) concentration of contaminant in the basin as well as before mixing with the groundwater, when the contaminant transport is governed by decay in the basin and by advection-dispersion-decay-sorption characteristics through saturated soil column.

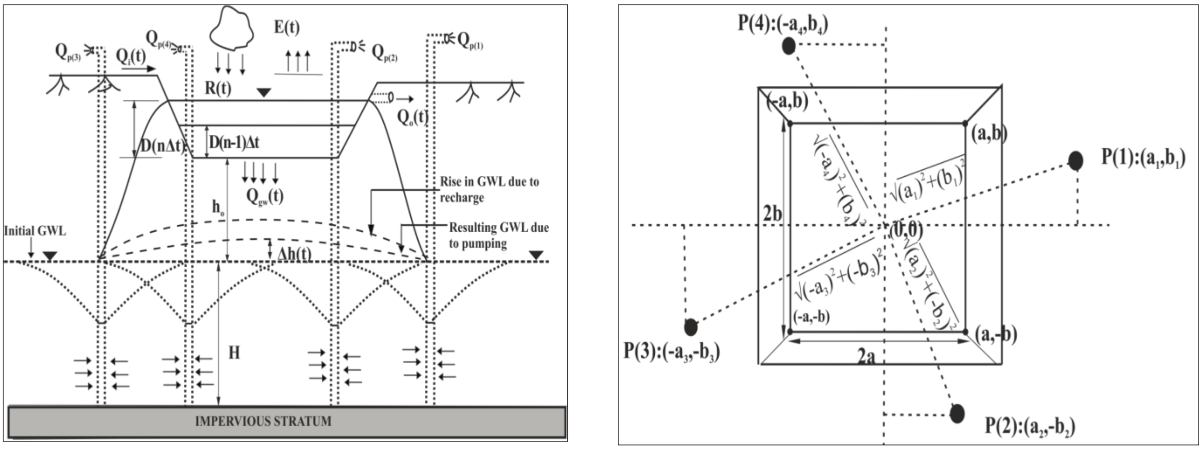

Fig 3.1 : Groundwater recharge basin showing hydrological components associated with it including arrangement of pumping for aquifer storage recovery, (a) sectional view, and (b) plan view along with coordinates of the pumping wells with reference to the basin origin.

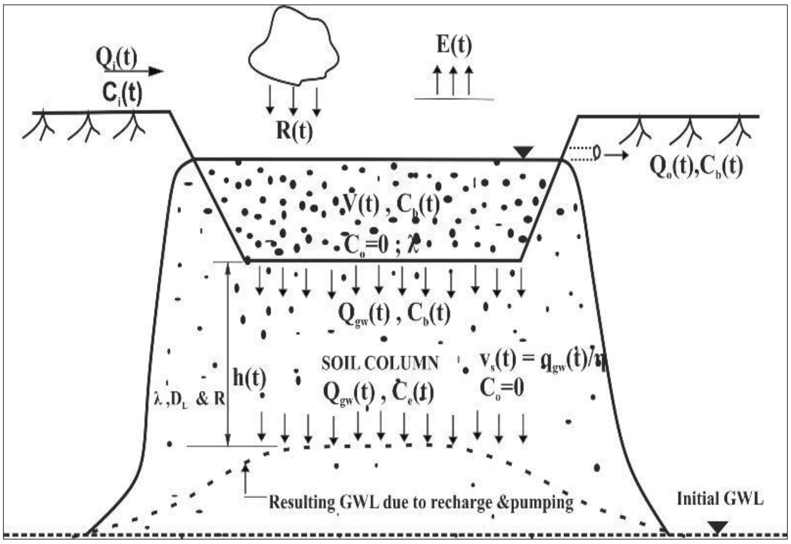

Fig 3.2: Parametric description of in-basin and through saturated soil pores underneath the basin mass transport processes.

3.2 Models Development

3.2.1 Recharge Model

The basic concept for estimate of the unsteady groundwater recharge consequent from variable inflows and outflows is the water balance of the basin (Figure 3.1a). In mathematical terms, the water balance equation, derived from the difference of all inflows and outflows to/from the basin at a given time is equal to the change in storage at that time, using notations of Fig. 3.1(a) can be written as (Ghosh et al, 2015):

$$[Q_i(t) + A\,R(t)\,]\varDelta t - \left[Q_o(t) + A_i\,(t)\,E(t) + Q_{gw}(t)\right]\varDelta t = \Delta V(t) \quad \text{..........} \ (1)$$

in which, $Q_i(t)$ is the inflow rate from its catchment to the recharge basin at time t; A is the gross surface area of the basin ; $R(t)$ is the rainfall rate at time t; $Q_0(t)$ is the outflow rate from the basin at time t; $A_i(t)$ is the water surface area of the basin at time t; $E(t)$ is the water surface evaporation rate at time t; $Q_{gw}(t)$ is the groundwater recharge rate from the basin at time t; $\varDelta V(t)$ is the change in storage in the basin between time t and t $+\Delta$t ; and $\varDelta t$ is the time step size.

In eq (1), all components are time varying and those, except $Q_{gw}(t)$, if known externally then $Q_{gw}(t)$ can be computed. The responses of $Q_{gw}(t)$ is governed by the hydraulic heads difference between the in-basin depth of water and water level height in the aquifer underneath the basin bed. The heads difference is influenced by the inflows and outflows to/from the basin and its geometry including the subsurface hydraulic parameters. The $Q_{gw}(t)$ in eq (1) is thus a time varying implicit function of hydraulic head.

For obtaining expression of $Q_{gw}(t)$ as a function of hydraulic head, Hantush's (1967) approximate analytical expression for the rise of water table due to uniform percolation of water from a spreading basin in absence of a pumping well together with well pumping equation given by Theis's (1935) is to be integrated into eq (1) to obtain resulting rise/fall of head beneath the basin. Hantush's (1967) analytical expression for rise/fall of water table due to uniform percolation of water from a rectangular spreading basin in absence of pumping well is given by:

$$h(x,y,t) = \left(\frac{w\,t}{4\,S}\right) f(x,y,t) \quad \text{....................................} \ (2)$$

in which, $h(x,y,t)$ is the rise in groundwater table below the rectangular spreading basin at location x, y, (L); w is the constant rate of percolation per unit area, (LT^{-1}); S is the storage coefficient of the aquifer (dimensionless); t is the time since the percolated water joins the water table, (T); and f(x,y,t) is the analytical expression derived by Hantush (1967), which is given by:

$$f(x,y,t) = \left\{ F\left[\frac{(a+x)}{2\sqrt{(Tt/S)}},\frac{(b+y)}{2\sqrt{(Tt/S)}}\right] + F\left[\frac{(a+x)}{2\sqrt{(Tt/S)}},\frac{(b-y)}{2\sqrt{(Tt/S)}}\right] \right. \quad \text{........................} (3)$$

$$\left. + F\left[\frac{(a-x)}{2\sqrt{(Tt/S)}},\frac{(b+y)}{2\sqrt{(Tt/S)}}\right] + F\left[\frac{(a-x)}{2\sqrt{(Tt/S)}},\frac{(b-y)}{2\sqrt{(Tt/S)}}\right] \right\}$$

where T is the transmissivity = $K\,H$; H is the weighted mean of the depth of saturation during the period of flow; K is the coefficient of permeability (i.e., hydraulic conductivity) of the aquifer material; a is the half of the length and b is the half of the width of the rectangular basin; x and y are the coordinates at which response is to be determined; $F(p,q)= \int_0^1 erf(p/\sqrt{z}).erf(q/\sqrt{z})\,dz$; and $erf(x)= \frac{2}{\sqrt{\pi}} \int_0^X e^{-u^2} du$.

Theis (1935) well response equation for drawdown in an observation well located at a distance r from the pumping well, with a uniform thickness and infinite in areal extent is given by :

$$s = \frac{Q_p}{4\,\pi\,T}\, W(u) \qquad \text{..............................}(4)$$

in which, s = drawdown (L); Q_p = constant pumping rate (L^3T^{-1}) ; T = transmissivity ($L^2\,T^{-1}$); $W(u)$ = the well function defined by an infinite series; and the dimensionless parameter u, which depends on time t, is defined as:

$$u = \frac{r^2 S}{4\,T\,t} \qquad \text{............. ....}\quad (5)$$

in which, r is the distance between the observation point and the well.

Let the time domain be discretized into uniform time steps, each of size, Δt, such that $t = n\Delta t$. Let the recharge from the wetted area of the basin be approximately equal to a train of pulses. Let $Q_{gw}(\gamma\Delta t)$ be the total recharge from the basin during the duration from $(\gamma\text{-}1)\Delta t$ to $(\gamma\Delta t)$. The resulting rise/fall in water table height, $\Delta h(t)$, consequent to a train of pulses of recharge and pumping can be derived using Duhamel's principle, as:

$$\Delta h(n\Delta t) = \sum_{\gamma=1}^{n} \frac{Q_{gw}(\gamma\Delta t)}{4\,a(\gamma)b(\gamma)}\, \delta_H(a(\gamma),b(\gamma),(n-\gamma+1)\Delta t) - \sum_{\xi=1}^{\xi_{max}} \sum_{\gamma=1}^{n} Q_p(\xi,\gamma)\ \delta_p(\xi,(n-\gamma+1)\Delta t) \text{....}\ (6a)$$

For rectangular basin, eq (6a) simplies to:

$$\Delta h(n\Delta t) = \sum_{\gamma=1}^{n} \frac{Q_{gw}(\gamma\Delta t)}{4\,ab}\, \delta_H((n-\gamma+1)\Delta t) - \sum_{\xi=1}^{\xi_{max}} \sum_{\gamma=1}^{n} Q_p(\xi,\gamma)\ \delta_p(\xi,(n-\gamma+1)\Delta t) \text{.......}\ (6b)$$

in which, $a(\gamma)$ is the half of the length; $b(\gamma)$ is the half of the width of the basin at a particular water depth, for rectangular basin, these are; a and b; $\delta_H(a(\gamma),b(\gamma),*)$ is the Hantush's discrete kernel coefficients corresponding to $a(\gamma)$ and $b(\gamma)$; ξ_{max} = number of pumping wells; $Q_p(\xi,\gamma)$ is the pumping rate of well ξ at time γ; and $\delta p(\xi,*)$ is the discrete kernel coefficients of the pumping well, ξ_p .

<u>Hantush's kernel coefficients:</u>

The unit step response function, $U_H(x,y,t)$ for rise/fall in water table height due to unit recharge per unit area per unit time, i.e., $w = 1\ (L^3\,L^{-2}\,T^{-1})$, from eq (2) is given by:

$$U_H(t) = \left(\frac{t}{4\,S}\right) f(x,y,t) \qquad \text{...}\ (7)$$

The unit pulse response function coefficients or the Hantush's discrete kernel coefficients, $\delta_H(x,y\ n\Delta t)$, in discrete time steps of size, Δt, $(t = n\ \Delta t)$, which takes place during the first time period, Δt, and no recharge afterwards, is given by:

$$\delta_H(x,y,n\Delta t) = \frac{U_H(x,y,n\Delta t) - \{U_H(x,y,(n-1)\Delta t)\}}{\Delta t} \qquad \text{....................................}\ (8)$$

Theis's kernel coefficients:

The unit step response function for drawdown, $U_p(t)$ due to unit pumping rate i.e., $Q_p = 1\ (L^3 T^{-1})$, from eq (4) is given by:

$$U_p(t) = \left(\frac{1}{4\pi T}\right) W\left(\frac{r^2 S}{4Tt}\right) \quad ,,,,,,,,,...................................... (9)$$

The unit pulse response function coefficients or the pumping discrete kernel coefficients, $\delta_p(n\Delta t)$, in discrete time steps of size, Δt, $(t = n\,\Delta t)$, is given by:

$$\delta_p(n\Delta t) = \frac{U_p(n\Delta t) - \{U_p(n-1)\Delta t\}}{\Delta t} \quad ....(10)$$

Assuming a linear relationship between the influent seepage and potential difference of heads, seepage during n^{th} time step can be obtained using Darcy's equation, as follows:

$$Q_{gw}(n\Delta t) = \frac{4\,a(n)\,b(n)\,K_v \left\lfloor H + h_0 + \overline{D}(n\Delta t) - (H + \Delta h(n\Delta t)) \right\rfloor}{h_0} \quad .... (11)$$

Putting $\Delta h(n\Delta t)$ from eq (6) into eq (11), we obtain:

$$Q_{gw}(n\Delta t) = \frac{4\,a(n)\,b(n)\,K_v \left[h_0 + \overline{D}(n\Delta t) - \left\{ \sum_{\gamma=1}^{n} \frac{Q_{gw}(\gamma)}{4\,a(\gamma)\,b(\gamma)} \delta_H(a(\gamma), b(\gamma), (n-\gamma+1`)\Delta t) - \sum_{\xi=1}^{\xi_{max}} \sum_{\gamma=1}^{n} Q_p(\xi,\lambda)\,\delta_p(\xi,(n-\gamma+1)\Delta t) \right\} \right]}{h_0} \quad .(12a)$$

For rectangular basin, eq (12a) simplies to :

$$Q_{gw}(n\Delta t) = \frac{4\,a\,b\,K_v \left[h_0 + \overline{D}(n\Delta t) - \left\{ \sum_{\gamma=1}^{n} \frac{Q_{gw}(\gamma)}{4\,a\,b} \delta_H((n-\gamma+1`)\Delta t) - \sum_{\xi=1}^{\xi_{max}} \sum_{\gamma=1}^{n} Q_p(\xi,\lambda)\,\delta_p(\xi,(n-\gamma+1)\Delta t) \right\} \right]}{h_0} \quad .... (12b)$$

$\overline{D}(n\Delta t)$ is the average water depth in the basin measured from its bed = $(D(n\Delta t) + D(n-1)\Delta t)/2$.

By separating the n^{th} term and rearranging, eq(12) is re-written, as:

$$Q_{gw}(n\Delta t) = \frac{4\,a(n)\,b(n)\,K_v \left[h_0 + \overline{D}(n\Delta t) - \left\{ \sum_{\gamma=1}^{n-1} \frac{Q_{gw}(\gamma\Delta t)}{4\,a(\gamma)\,b(\gamma)} \delta_H(a(\gamma), b(\gamma), (n-\gamma+1)\Delta t) - \sum_{\xi=1}^{\xi_{max}} \sum_{\gamma=1}^{n} Q_p(\xi,\gamma)\,\delta_p(\xi,(n-\gamma+1)\Delta t) \right\} \right]}{\left\{ h_0 + K_v\,\Delta t\,\delta_H(a(n), b(n), \Delta t) \right\}} \quad (13a)$$

For rectangular basin, eq(13a) reduces to:

$$Q_{gw}(n\Delta t) = \frac{4\,a\,b\,K_v \left[h_0 + \overline{D}(n\Delta t) - \left\{ \sum_{\gamma=1}^{n-1} \frac{Q_{gw}(\gamma\Delta t)}{4\,a\,b} \delta_H((n-\gamma+1)\Delta t) - \sum_{\xi=1}^{\xi_{max}} \sum_{\gamma=1}^{n} Q_p(\xi,\gamma)\,\delta_p(\xi,(n-\gamma+1)\Delta t) \right\} \right]}{\left\{ h_0 + K_v\,\Delta t\,\delta_H(\Delta t) \right\}} \quad .... (13b)$$

From eq (13 a& b), $Q_{gw}(n\Delta t)$ can be obtained by solving it in succession of time steps starting from time step 1.

The equation for computing the time varying in-basin depth of water, $D(n\Delta t)$ is obtained by substituting the recharge component from eq(13a) into eq (1). The equation for computing D(n Δt) is given by:

$$D(n\Delta t) = D(n\Delta - \Delta t)\frac{A_{ws}(n\Delta t - \Delta t)}{A_{ws}(n\Delta t)} + \frac{1}{A_{ws}(n\Delta t)}\left[Q_i(n\Delta t) + R(n\Delta t)\,A_s - E(n\Delta t)\overline{A_{ws}} - Q_0(n\Delta t)\right]\Delta - \frac{\overline{A_{rs}}}{A_{ws}(n\Delta t)}\left[K_v\frac{\left\{h_0 + \bar{D}(n\Delta t) - \left(\sum_{\gamma=1}^{n-1}\frac{Q_{gw}(\gamma\Delta t)}{4\,a(\gamma)\,b(\gamma)}\,\delta_H(a(\gamma), b(\gamma), (n-\gamma+1)\Delta t) - \sum_{\xi=1}^{\xi_{max}}\sum_{\gamma=1}^{n} Q_p(\xi,\gamma)\delta_p(\xi,(n-\gamma+1)\Delta t)\right)\right\}}{\{h_0 + K_v\,\delta_H(x,y,\Delta t\}}\right] \quad \text{............ (14a)}$$

in which, $\Delta V(n\Delta t) = [A_{ws}(n\Delta t)\,D(n\Delta t) - A_{ws}(n\Delta t - \Delta t)D(n\Delta t - \Delta t)]$; $D(n\Delta t)$ is the in-basin depth of water at time, t = n∆t; $D(n\Delta t-\Delta t)$ is the in-basin depth of water at the proceeding time step, i.e., $(n-1)\Delta t$; $A_{ws}(n\Delta t-\Delta t)$ and $A_{ws}(n\Delta t)$ are the water surface area of the basin at the proceeding and current time step, respectively ; A_s is the gross top surface area of the basin; $\overline{A_{ws}} = (A_{ws}(n\Delta t) + A_{ws}(n-1)\Delta t)/2$; and $\overline{A_{rs}} = 4\,a(n)\,b(n)$.

For rectangular basin, eq (14a) reduces to :

$$D(n\Delta t) = D(n\Delta - \Delta t) + \frac{1}{A_s}\left[Q_i(n\Delta t) + R(n\Delta t)\,A_s - E(n\Delta t)A_s - Q_0(n\Delta t)\right]\Delta t - \left[K_v\frac{\left\{h_0 + \bar{D}(n\Delta t) - \left(\sum_{\gamma=1}^{n-1}\frac{Q_{gw}(\gamma\Delta t)}{(4\,a\,b)}\,\delta_H((n-\gamma+1)\Delta t) - \sum_{\xi=1}^{\xi_{max}}\sum_{\gamma=1}^{n} Q_p(\xi,\gamma)\delta_p(\xi,(n-\gamma+1)\Delta t)\right)\right\}}{\{h_0 + K_v\,\delta_H(x,y,\Delta t\}}\right] \quad \text{.................... (14b)}$$

Eq (14a) has a single unknown $D(n\Delta t)$ and several other $D(n\Delta t)$ dependent variables. The unknown, $D(n\Delta t)$ and its dependent variables, $A_{ws}(n\Delta t)$ and $A_{rs}(n\Delta t)$ are in different mathematical forms at the RHS of eq (14a). $D(n\Delta t)$ in such case, can be solved by iteration procedure in succession of time steps when all other variables known externally. Substituting the estimated $D(n\Delta t)$ from eq (14 a & b) into eq (13 a & b), $Q_{gw}(n\Delta t)$ corresponding to the value of $D(n\Delta t)$ can be computed. The time varying $D(n\Delta t)$ and $Q_{gw}(n\Delta t)$ can thus be calculated in succession of time for n = 1, 2,3, ...

For computing $D(n\Delta t)$ and $Q_{gw}(n\Delta t)$ corresponding to unsteady inflows and outflows using eqs (14 a&b) and (13 a&b), respectively, the variables; $Q_i(n\Delta t)$, $R(n\Delta t)$, $Q_o(n\Delta t)$, $E(n\Delta t)$, $A_{ws}(n\Delta t)$, A_s, $D(n\Delta t-\Delta t)$, and the parameters H, h_0, T, S, and K_v are to be known *apriori*. In eq. (14b), $A_s = 4\,a\,b$.

3.2.2 Contaminant Transport Models

The transport models, for determining fate of contaminants as they accumulate and assimilate in-basin, and thereafter travel through the underneath porous formation below the recharge basin (Fig. 3.2), are developed based on the principle of conservation of mass that states: accumulation of mass in a control volume = (mass of contaminant in – mass of contaminant out ± sources/sinks of that contaminant), over a specific period of time. The models are developed separately for the in-basin mass accumulation and assimilation, and transport through soil column, considering outputs of the first as the inputs to the latter case.

3.2.2.1 In-basin Mass Transport

The transport equation, for the in-basin contaminants mass balance, Fig.3.2, can be written as:

$$[Q_i(t) + R(t)\, A_s - E(t)\, \overline{A_{ws}}\,]\, C_{ri}(t)\, \Delta t - Q_o(t)\, C_b(t)\, \Delta t - Q_{gw}(t)\, C_b(t)\Delta t - \lambda\, V(t)\, C_b(t)\, \Delta t = V(t)\, \Delta C_b(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in eq (15), $C_{ri}(t)$, is the resulting influent concentration of contaminants due to dilution by rainfall and intensification by evaporation = $(Q_i(t)\, C_i(t))/(Q_i(t) + R(t)\, A_s - E(t)\, \overline{A_{ws}}\,)$; $V(t)$ is the volume of water in the basin, at time, t; $\Delta C_b(t)$ is the change in effluent concentration at time, t = $C_b(t) - C_b(t-\Delta t)$; and all other terms are as explained earlier.

Discretizing the time, t into *'n'* number of equal time steps, each of size, Δt, such that, $t= n\Delta t$. By rearranging eq (15) , the expression for $C_b(n\Delta t)$ is obtained as:

$$C_b(n\Delta t) = \frac{\left[\{Q_i(n\Delta t) + R(n\Delta t)\, A_s - E(n\Delta t)\, \overline{A_{ws}}\}\, C_{ri}(n\Delta t)\Delta t + V(n\Delta t)\, C_b\big((n-1)\Delta t\big)\right]}{\left[V(n\Delta t) + \{Q_o(n\Delta t) + Q_{gw}(n\Delta t) + \lambda\, V(n\Delta t)\}\Delta t\right]} \quad .. \quad (16)$$

Having known, the influent concentration, $C_i(t)$; decay rate coefficient, λ; and all other time variant hydrological variables, viz. $Q_i(n\Delta t)$, $R(n\Delta)$, $E(n\,\Delta t)$, $Q_o(n\Delta t)$, $V(n\Delta t)$, *and* $Q_{gw}(n\Delta t)$, the in-basin concentration of contaminant,$C_b(n\Delta t)$ using eq (16) can be computed in succession of time step, n = 1, 2, 3 ... staring from n=1.

3.2.2.2 Contaminant Transport through Soil Column

For contaminant transport through porous media, one-dimensional advection-dispersion equation (ADE) together with decay and adsorption, as given below, is used. The one-dimensional contaminant transport equation describing advection, dispersion, decay, and sorption in soil and groundwater for steady and uniform flow (Taylor, 1954; Bajracharya and Barry, 1992; Wang and Chen, 1996; Sun, 1996) is given by:

$$\frac{\partial(C_e + C_s)}{\partial t} = D_L \frac{\partial^2 C_e}{\partial x^2} - v_s \frac{\partial C_e}{\partial x} - \lambda\, C_e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which, C_e is the effluent concentration in the water phase, C_s is the adsorbate concentration in the solid phase; x is the vertical distance = $h_0 - \Delta h(t)$.

For instantaneous linear adsorption sorption, the LHS of eq.(17) can be approximated as:

$$\frac{\partial(C_e + C_s)}{\partial t} = R\, \frac{\partial C_e}{\partial t} \qquad \ldots\ldots\ldots.\ldots\ldots\ldots\ldots\ldots.. \quad (18)$$

R, is a dimensionless constant termed as retardation factor= $\left(1+{}^{\rho_b K_d}/_{n}\right)$ in which $K_d = S/C_e$; S is the ratio of the mass of sorbed material to the mass of solid material, both contained in some volume of porous material; ρ_b is the bulk density of solid material , and η is the porosity. The unit of the distribution factor is (L^3M^{-1}) , and R > 1. The distribution factor is the quantity associated with retardation that is usually measured in the laboratory.

The analytical solution to eq *(17)* with replacement of LHS by eq*(18),* for step input, C_R and initial concentration, $C(x,0) = 0$, based on the solution given by Ogata and Banks (1963), is:

$$C_e(x,t) = \frac{C_R}{2}\exp\left(\frac{x\,v_s}{2\,D_L}\right)\left[exp\left(-\frac{x}{2\,D_L}\sqrt{v_s^2+4\,\lambda\,D_L}\right)\,erfc\left\{\frac{R\,x-\left(\sqrt{v_s^2+4\,\lambda\,D_L}\right)t}{2\sqrt{R\,t\,D_L}}\right\}+ exp\left(\frac{x}{2\,D_L}\sqrt{v_s^2+4\,\lambda\,D_L}\right)\,erfc\left\{\frac{R\,x+\left(\sqrt{v_s^2+4\,\lambda\,D_L}\right)t}{2\sqrt{R\,t\,D_L}}\right\}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.. \quad (19)$$

erfc(*) is the complementary error function.

For $C_R = 1$, *eq.(19)* turns to the unit step response function. Let $U_c(x,t)$ be the unit step response function of eq.*(40)* and the unit impulse response function, $u_c(x,t)$ is obtained as;

$$u_c(x,t) = \frac{dU_c(x,t)}{dt} = \frac{x\sqrt{R}}{2\,t\sqrt{\pi\,t\,D_L}}\exp\left[-\frac{(R\,x-\,v_s t)^2}{4\,R\,t\,D_L}-\frac{\lambda\,t}{R}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots. \quad (20)$$

in which, $x = h(t);\ v_s = {}^{q_{gw}(t)}/_{\eta}$; $q_{gw(t)=}\ {}^{Q_{gw(t)}}/_{A_{ws}}$;and both *h(t)* and $q_{gw}(t)$ are time variant.

In discrete time step size of Δt, the kernel coefficients of eq.(20) for $x = h(n\Delta t)$ and $v_s = v_s(n\Delta t)$, is given by:

$$\delta_c\left[h(n\Delta t),v_s(n\Delta t),n\Delta t\right]=\frac{h(n\Delta t)\sqrt{R}}{2\,(n\Delta t)\sqrt{\pi\,D_L(n\Delta t)}}\,exp\left[-\frac{\left(R\,h(n\Delta t)-v_s(n\Delta t)(n\Delta t)\right)^2}{4\,R D_L\,(n\Delta t)}-\frac{\lambda}{R}(n\Delta\,t)\right] \quad ....(21)$$

The concentration of contaminant through soil column before mixing with the groundwater at any time t, $C_e(t)$ is given by:

$$C_e(t) = \int_0^t C_b(\tau)\ u_c[h(t),v_s(t),t-\tau]\,d\tau \quad \ldots\ldots\ldots\ldots\ldots \quad (22)$$

where $C_b(\tau)$ is the in-basin concentration of contaminant at time, τ; $u_c[h(t),v_s(t),t-\tau]$ is the impulse response function for *h(t)* and $v_s(t)$, as given in eq (21) ; and τ is a dummy time variable. In discretized time domain of size, *Δt,* the convolution equation, eq (22) for variable seepage velocity, $v_s(t)$ and height, *h(t)* consequent to a train of pulses of the in-basin contaminant concentration using Duhamel's principle, can be written as:

$$C_e(n\,\Delta t) = \sum_{\gamma=1}^{n} C_b(\gamma\Delta t)\quad \delta_c[h(\gamma), v_s(\gamma), (n-\gamma+1)\Delta t] \quad \ldots\ldots\ldots\ldots \quad (23)$$

in which, $\delta_c[h(\gamma),v_s(\gamma),(n-\gamma+1)\Delta t]$ is the discrete kernel coefficient derived based on impulse response function (eq 21) for variable $h(t)$ and $v_s(t)$.

Knowing, $q_{gw}(n\Delta t)$ and $\Delta h(n\Delta t)$ from eq(13) and eq(6), $v_s(n\Delta t) = q_{gw}(n\Delta t)/\eta$ and $h(n\Delta t) = h_0 - \Delta h(n\Delta t)$ can be calculated. The kernel coefficients, $\delta_c[h(\gamma),v_s(\gamma),(n-\gamma+1)\Delta t]$ corresponding to each $v_s(n\Delta t)$ and $h(n\Delta t)$ for $n = 1,2.3...$ can be computed using eq. (21) . Making use of respective $\delta_c[h(\gamma),v_s(\gamma),(n-\gamma+1)\Delta t$ in *eq (23)*, $C_e(n\Delta t)$ corresponding to $C_b(n\Delta t)$ can be obtained by convolution.

3.3 An Illustrated Example

The performance of the derived mathematical models is demonstrated by a hypothetical example based on the real time data of rainfall and evaporation together with the databases in **Box 1**. Let us consider a trapezoidal and rectangular recharge basin on a depleted groundwater area. The recharge basin is meant for augmentation of groundwater resources. The basin receives inflows from its catchment area through runoffs from rainfall. To recover the created aquifer storage from recharge, groundwater withdrawal by four pumping wells located around the recharge basin (Fig.3.1(b)) is taken into consideration. It is assumed that the pumped water would be used for some beneficial purposes, viz. drinking or irrigation water supply. Let us consider that the inflows of water to the basin contain contaminants of non-conservative type, and the in-basin contaminants have the characteristics of decay and the sub-surface formation beneath the basin has the characteristics to decay and sorb the constituents' concentration. It is intended to: (i) determine the rate of groundwater recharge consequent to the hydrological interventions in the basin, (ii) the enhanced recharge due to pumping and also the rise/fall of groundwater level beneath the basin due to the resulting affect of recharge and pumping, and (iii) the improved constituents' concentration because of movement through soil pores and before mixing with the groundwater.

Box 1. Input Data : *Dimension of base of the trapezoidal and rectangular basin = 100 m x 100 m, i.e., 2a = 100 m and 2b = 100 m; side slope for trapezoidal basin: 1 V:1 H; rectangular basin has no side slope; maximum depth of the basin,* D_{max} *= 3.5 m; free board = 0.5 m; Outlet at 0.5 m below the basin top; Initial groundwater level above the impervious stratum, H = 40 m; height of porous material below the basin bed,* h_0 *= 5 m; coordinates of the four pumping wells in meter with reference to the origin at centre of the basin= P(1):(250, 50);P(2):(50, - 250); P(3):(-250, -50) ; P(4):(-50, 250); pumping wells operate for 8 hours in a day with pumping rate,* Q_p *= 40* m^3*/hr and pumping starts from* 5^{th} *day from start of inflow; initial depth of water in the basin, D(0) = 0 m; transmissivity of the aquifer, T = 800* m^2*/day; hydraulic conductivity of the aquifer, K = 20 m/day; storage coefficient, S = 0.1; the ratio of vertical hydraulic conductivity of soil material beneath the basin to the aquifer hydraulic conductivity,* K_v/K *=0.05, 0.075, and 0.1; porosity of soil materials beneath the basin,* η *= 0.39; and time step size,* Δt *= 1 day. The basin has a catchment area of 0.5 sq. km with varying soil-classes and land-use pattern; the inflow rate,* $Q_i(t)$ *is considered variable and dependent on the rainfall; the evaporation rate, E(t) is also considered variable and dependent on the meteorological data. The inflow to the basin starts at time, t = 0 and the simulation time period, t =*

120 days. It is assumed that soil below the basin is homogeneous and isotropic and groundwater level is horizontal.

The influent concentration of contaminants from the catchment, $C_i(t)$ = 50 mg/L; decay rate coefficient, λ = 0.025 day^{-1} ; distribution factor for the sorbed material, R = 1 and 1.2; longitudinal dispersion coefficient, $D_L = \alpha\, v_s^m$; dispersivity, α = 5m and exponent, m = 1.07; initial in-basin and soil column concentration of contaminant, C_0 at time, t=0 is zero.

For inflows to the basin, $Q_i(t)$, the real time rainfall data series, and for evaporation rate, E(t) a real-time meteorological data series of year- 2016 of Roorkee (India)(latitude: 29.8543° N and longitude: 77.888° E) are considered. The time-series data represent; daily rainfall, R(t) in mm, and daily value of wind speed at 2 m height, u in m/s; air temperature, T_a in ^{0}C; and relative humidity, RH in %.

3.3.1 Computation of Discrete Kernel Coefficients

For the trapezoidal basin, the discrete kernel coefficients will vary with varying in-basin depth of water and the corresponding change in dimension of wetted length and breadth. For computation of discrete kernel coefficients, in-basin depth of water has to be known *apriori* at every time step. For the rectangular basin, the length and breadth of the basin remain unchanged and hence, there will not be any change in the discrete kernel coefficients for changing depth of water in the basin.

Using T = 800 m^2/day *(K = 20 m/day)*, S= 0.1, a = 50 m, b = 50 m in eq (3) together with variable x and y i.e., $x = a + h(n\Delta t)$ and $y = b+h(n\Delta t)$, Hantush's unit step, $U_H(x, y, n\Delta t)$ and unit pulse, $\delta_H(x, y, n\Delta t)$ kernel coefficients are computed using eqs (7) and (8), respectively by taking average of 2 locations; the one at the centre, and the other one at the extreme wetted corner of the basin at the corresponding h(n∆t). The expression for average unit step kernel coefficients is :

$$U_H(x,y,t) = \frac{t}{8S}\left[\; f(0,0,t) + \; f(x,y,t)\right] \qquad \text{..................} \; (24)$$

Theis's pumping discrete kernel coefficients, $\delta_p(n\Delta t)$ are calculated using eqs (9) and (10). For calculating the well function, $W(u)$ by eq(5) corresponding to each well, the distance between the observation point and well, r is computed taking centre (origin) and the four base corners of the basin as observation points. The distances corresponding to these observation points in respect of four pumping wells are calculated using the expressions in Table 1.

Table 1: Expression of distances of pumping wells with respect to different observation points.

For origin (0,0)	For corner (a,b)	For corner (a,-b)	For corner (-a,-b)	For corner (-a,b)
$r_{O1} = \sqrt{(a_1)^2 + (b_1)^2}$	$r_{C1} = \sqrt{(a_1 - a)^2 + (b - b_1)^2}$	$r_{C1} = \sqrt{(a_1 - a)^2 + (b + b_1)^2}$	$r_{C1} = \sqrt{(a + a_1)^2 + (b + b_1)^2}$	$r_{C1} = \sqrt{(a + a_1)^2 + (b - b_1)^2}$
$r_{O2} = \sqrt{(a_2)^2 + (b_2)^2}$	$r_{C2} = \sqrt{(a - a_2)^2 + (b - b_2)^2}$	$r_{C2} = \sqrt{(a - a_2)^2 + (-b_2 - b)^2}$	$r_{C2} = \sqrt{(a + a_2)^2 + (-b_2 - b)^2}$	$r_{C2} = \sqrt{(a + a_2)^2 + (b - b_2)^2}$
$r_{O3} = \sqrt{(a_3)^2 + (b_3)^2}$	$r_{C3} = \sqrt{(-a_3 + a)^2 + (b - b_3)^2}$	$r_{C3} = \sqrt{(-a_3 + a)^2 + (b + b_3)^2}$	$r_{C3} = \sqrt{(-a_3 - a)^2 + (b + b_3)^2}$	$r_{C3} = \sqrt{(-a_3 - a)^2 + (b - b_3)^2}$
$r_{O4} = \sqrt{(a_4)^2 + (b_4)^2}$	$r_{C4} = \sqrt{(a - a_4)^2 + (b_4 - b)^2}$	$r_{C4} = \sqrt{(-a_4 + a)^2 + (b_4 + b)^2}$	$r_{C4} = \sqrt{(a + a_4)^2 + (b + b_4)^2}$	$r_{C4} = \sqrt{(a + a_4)^2 + (b_4 - b)^2}$

(r_{o1} represents with respect origin for pumping well, P(1); r_{c1} represents with respect to corner for pumping well, P(1); and so on.; coordinates of the pumping well should be as shown in Fig.3.1b) .

Having calculated the respective distances, using coordinates of the pumping wells (Box 1), *W(u)* for each observation point are determined for the given S and T. The discrete kernel coefficients, $\delta_p(n\Delta t)$ corresponding to *W(u)* of five observation points (centre, and four corners) are generated using eqs (9) and (10). The resulting kernel coefficients for unit step response function are obtained by taking average of kernel coefficients of five locations for all the wells, as follows:

$$U_p(\xi, n\Delta t) = \sum_{\gamma=1}^{n} \sum_{\xi=1}^{\xi_{max}} \frac{1}{N} \sum_{j=1}^{N} U_p(j, \xi, n\Delta t) \qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (25)$$

N = total number of observation location; and ξ_{max} = total number of well.

Because of rise/fall in GWL due to variable recharge rates, the travel length of contaminants, *h(t)* and seepage velocity, $v_s(t)$ through the soil column beneath the basin will also vary. The discrete kernel coefficients will change with varying *h(t)* and $v_s(t)$ and their related parameters. Therefore, for calculating the discrete kernel coefficients of contaminant transport, $\delta_c[h(n\Delta t), v_s(n\Delta t), n\Delta t]$; first $\Delta h(n\Delta t)$ from eq(6), and $v_s(n\Delta t) = \frac{Q_{gw}(n\Delta t)}{(\eta\, 4\, a(n) b(n))}$ from eq(13), for n = 1,2,3..., are calculated. For rectangular basin, $v_s(n\Delta t) = \frac{Q_{gw}(n\Delta t)}{(\eta\, 4a\, b)}$. [in which, η = soil porosity]. Using, $h(n\Delta t) = h_0 - \Delta h(n\Delta t)$; $D_L(n\Delta t) = \alpha\, v_s(n\Delta t)^m$, $\alpha = 5$; $m = 1.07$; $\lambda = 0.025$, and $R = 1.2$ in eq (21), $\delta_c[h(n\Delta t), v_s(n\Delta t), n\Delta t]$ are calculated for every time step with n=1,2,3 ....

3.3.2 Computation of Inflow/Runoff (Q) of the basin

The runoff yield by the SCS-CN model is given (SCS, 1993; Mishra & Singh, 1999) by:

$$Q = \frac{(R - \lambda S)^2}{R + (1 - \lambda) S} \quad \text{for } R > \lambda S \qquad \dots\dots\dots\dots\dots\dots\dots\dots (26)$$

$$Q = 0 \qquad\qquad for\ \ R \le \lambda S$$

$$S = \frac{25400}{CN} - 254 \qquad \text{...} \ (27)$$

where, Q is the runoff (mm/day); R is the precipitation (mm); S is the maximum potential retention (mm); λ is the initial abstraction weight as a fraction of S, normally $0 \le \lambda \le 0.3$, but conventionally 0.2; 25400 and 254 in eq. (27) are the arbitrary numerical constants in units of S; and CN is the Curve Number (dimensionless).

Theoretically, S varies between 0 to ∞ for the CN ranges from 100 to 0. The CN = 100 represents, $S = 0$, an impermeable watershed. Conversely, the CN = 0 represents, $S = \infty$, an infinitely abstracting watershed. Substituting S and λ =0.2, eq. (26), yields to:

$$Q = \frac{25.4\left[\frac{R}{25.4} - \frac{200}{CN} + 2\right]^2}{\left[\frac{R}{25.4} + \frac{800}{CN} - 8\right]} \quad ; valid\ for\ R \ge 0.2\ S \qquad \text{............................ ...} \ (28)$$

The watershed specific-CNs relating to the antecedent moisture condition (AMC) (SCS, 1985; Lewis *et al.*, 2000) are:

$$CN_I = \frac{4.2\ CN_{II}}{10 - 0.058\ CN_{II}} \qquad \text{....................................} \ (29)$$

$$CN_{III} = \frac{23\ \ CN_{II}}{10 - 0.13\ \ CN_{II}} \qquad \text{..} \ (30)$$

where, subscripts indicate the AMC, I being dry, II normal, and III wet.

3.3.3 Computation of Evaporation (E)

To overcome the requirement of R_n (net radiation on the water surface) and N (change in head stored in water) in Priestley-Taylor's equation(1972), by combining Penman and Priestley-Taylor method, de Bruin (1978) developed a simplified equation to estimate water surface evaporation as follows:

$$E = \frac{\alpha}{\alpha - 1}\left(\frac{\gamma}{\Delta + \gamma}\right) F(u)(e_s - e_a) \qquad \text{..............................} \ (31)$$

In which, E is the evaporation (watt/m^2); α is Priestley-Taylor coefficient, normally taken as 1.26; γ is Psychrometric coefficient; Δ is the slope of saturation vapour pressure-temperature curve; $F(u)$ is the wind function (W/m^2/mb); e_a and e_s are the actual and saturated vapour pressures respectively (millibars); E = $E_{megajoules}$*2.45(mm/day); and $E_{megajoules}$=E*0.0864 (Megajoules).

In eq *(31)*, γ is given by :

$$\gamma = \frac{C_{pa}\ P}{0.622 \times 1000 \times \lambda} \qquad \text{..................................} \ (32)$$

C_{pa} is the specific heat capacity, generally taken as $1.013 \times 10^{-3} MJ/kg/^0 C$; numerical value, *0.622 (=18.016/28.996)* represents the ratio of molecular weights of water to dry air; P is the atmospheric pressure, *(kPa)*; and λ is the latent heat of vaporization, ranges between 2.5 and *2.4 MJ/kg* for liquid water between 0°C and 40°C.

The atmospheric pressure, P at elevation, z, is given by (Allen, et al,1998):

$$P = 101.3\left(\frac{293 - 0.0065\, z}{293}\right)^{5.26} \qquad \text{.....................................} \; (33)$$

Z is the elevation (meter).

The slope of saturation vapour pressure-temperature curve, Δ is given by:

$$\Delta = \frac{4098\left[0.6108\, exp\left(\frac{17.27\, T_a}{T_a + 237.3}\right)\right]}{(T_a + 237.30)^2} \qquad \text{..} \; (34)$$

In which, T_a is the air temperature, (^0C)

The wind function, $F(u)$ is given by:

$$F(u) = 2.9 + 2.1\, u \qquad \text{....................................} (35)$$

In which, u is the wind speed measured at 2 m height, (m/s).

The actual vapour pressure (millibars), e_a is given by :

$$e_a = 33.8639\, [(0.00738 * T_a + 0.8072)^8 - 0.0000191|1.8 * T_a + 48| + 0.001316] \; ..(36)$$

The saturated vapour pressure (millibars), e_s is given by:

$$e_s = e_a / RH \qquad \text{...} (37)$$

RH is the relative humidity.

3.3.4 Computation of Contaminant Transport

The one-dimensional contaminant transport equation describing advection, dispersion, decay, and sorption in soil and groundwater for steady and uniform flow (Taylor, 1954; Bajracharya and Barry, 1992; Wang and Chen, 1996; Sun, 1996) is given by:

$$\frac{\partial (C_e + C_s)}{\partial t} = D_L \frac{\partial^2 C_e}{\partial x^2} - v_s \frac{\partial C_e}{\partial x} - \lambda\, C_e \quad \text{...} (38)$$

in which, C_e is the effluent concentration in the water phase, C_s is the adsorbate concentration in the solid phase; x is the vertical distance = $h_0 - \Delta h(t)$.

For instantaneous linear adsorption sorption, the LHS of eq(38) can be approximated as**:**

$$\frac{\partial (C_e + C_s)}{\partial t} = R\ \frac{\partial C_e}{\partial t} \qquad \text{………………..} \qquad (39)$$

R, is a dimensionless constant termed as retardation factor=$\left(1 + {}^{\rho_b\, K_d}/_{n}\right)$ in which $K_d = S/C_e$; S is the ratio of mass of sorbed material to the mass of solid material.

The analytical solution to eq *(38)* with replacement of LHS by eq*(39)*, for step input, C_R and initial concentration, $C(x,0) = 0$, based on the solution given by Ogata and Banks (1963), is:

$$C_e(x,t) = \frac{C_R}{2} \exp\left(\frac{x\, v_s}{2\, D_L}\right)\left[exp\left(-\ \frac{x}{2\, D_L}\sqrt{v_s^2 +\ 4\, \lambda\, D_L}\right)\ erfc\left\{\frac{R\, x - \left(\sqrt{v_s^2 + 4\, \lambda\, D_L}\right) t}{2\sqrt{R\, t\, D_L}}\right\} + \right.$$

$$\left. exp\left(\frac{x}{2\, D_L}\sqrt{v_s^2 +\ 4\, \lambda\, D_L}\right)\ erfc\left\{\frac{R\, x + \left(\sqrt{v_s^2 + 4\, \lambda\, D_L}\right) t}{2\sqrt{R\, t\, D_L}}\right\}\right] \qquad \text{………………………..} \ (40)$$

erfc(*) is the complementary error function. For $C_R = 1$, *eq.(40)* turns to the unit step response function. Let $U_c(x,t)$ be the unit step response function of eq.*(40)* and the unit impulse response function, $u_c(x,t)$ is obtained as;

$$u_c(x,t) = \frac{dU_c(x,t)}{dt} = \frac{x\sqrt{R}}{2\, t\sqrt{\pi\, t\, D_L}} \exp\left[-\frac{(R\, x -\ v_s t)^2}{4\, R\, t\, D_L} - \frac{\lambda\, t}{R}\right] \qquad \text{……………….} \ (41)$$

in which $x = h(t)$; $v_s = {}^{q_{gw}(t)}/_{\eta}$; $q_{gw(t)} = {}^{Q_{gw(t)}}/_{A_{ws}}$;and both $h(t)$ and $q_{gw}(t)$ are time variant.

In discrete time step size of Δt, the kernel coefficients of eq.(41 for $x = h(n\Delta t)$ and $v_s = v_s(n\Delta t)$, is given by:

$$\delta_c\left[h(n\Delta t), v_s(n\Delta t), n\Delta t\right] = \frac{h(n\Delta t)\sqrt{R}}{2\,(n\Delta t)\sqrt{\pi\ D_L\,(n\Delta t)}}\ exp\left[-\frac{\left(R\, h(n\Delta t) - v_s(n\Delta t)(n\Delta t)\right)^2}{4\, R D_L\,(n\Delta t)} - \frac{\lambda}{R}(n\Delta t)\right] \quad \text{…..}(42)$$

Chapter 4

Results and Discussion

4.1 Performance of Recharge Model

The recharge model has been applied for two basin types trapezoidal and rectangular shape basin. The results for both basin types are presented and discussed in the subsequent texts.

The simulation time period, t is, 120 days and the time step size, Δt is, 1 day that give a total number of time steps of 120. For computation of $Q_i(n\Delta t)$ by SCS-CN method corresponding to $R(n\Delta t)$ using eq (28), CN of the catchment has to be calculated first. The value of CN depends on the soil classes and land-use of the catchment. For demonstration purposes, the catchment's soil groups are considered as; 50% of group A; 25% group of B; and 25% group of C. Making use of the estimated CN from eqs (29 and 30) in eq (28), $Q_i(n\Delta t)$ (mm/day) corresponding to the time-series data of $R(n\Delta t)$ are calculated, and taken as input runoffs to the basin by multiplying with the catchment area. In the simulation period of 120 days, 25 events have rainfall ≥ 5 mm on different days. The variation of $Q_i(n\Delta t)$, corresponding to 25 $R(n\Delta t)$ events, which ranged between 5 mm/day and 100 mm/day, is estimated between 5 m^3/day and 100 m^3/day.

The discrete kernel coefficients for recharge, $\delta_H(x, y, n\Delta t)$ and pumping, $\delta_p(\xi, n\Delta t)$ for the given aquifer hydraulic properties and basin geometry are generated using eq (8) and eq (10), respectively and their unit pulse response functions given at eqs (7 and 9) are calculated by following the procedure explained in section 3.3.1. Using the respective, $\delta_H(x, y, n\Delta t)$, and $\delta_p(\xi, n\Delta t)$ in eq (6) together with $Q_p(\xi, n\Delta t)$ and $Q_{gw}((n-1)\Delta t)$, the recharge rate, $Q_{gw}(n\Delta t)$ for different, ${}^{K_v}/_{K} = 0.05, 0.075, and\ 0.1$ are calculated in succession of time step staring from $n = 1$. For calculating $Q_{gw}(n\Delta t)$ at every progressive time step, $D(n\Delta t)$ is to be computed first by using eq (14 a or b). Thus, the computation of $Q_{gw}(n\Delta t)$ and $D(n\Delta t)$ is a simultaneous process, replacing the former value at the current time step to the later one. Further, eq (12 a or b) is an implicit expression possessing $D(n\Delta t)$, and eq (12a) contains a number of $D(n\Delta t)$ dependent variables at the R.H.S. Therefore, the computation of $D(n\Delta t)$ at time step, $n\Delta t$, by eq (12 a or b) involves an iterative procedure. The cumulative variation of $Q_i(n\Delta t)$, $E(n\Delta t)$, $Q_0(n\Delta t)$, and $Q_{gw}(n\Delta t)$ *for* $K_v = 2$ *m/day* is given at Fig. 4. The outflows from the basin, $Q_0(n\Delta t)$ is the quantity in excess to the basin capacity beyond $D = 3$ *m*. The cumulative inflows of $Q_i(n\Delta t)$ showing horizontal line indicate no rainfall between two successive time intervals of rain. The variation of $E(n\Delta t)$ and $Q_{gw}(n\Delta t)$, in those no rainfall periods, also reduces because of reducing in-basin water depth.

(i) **For trapezoidal basin**

The time varying cumulative inflows for trapezoidal basin are shown in Fig 4.

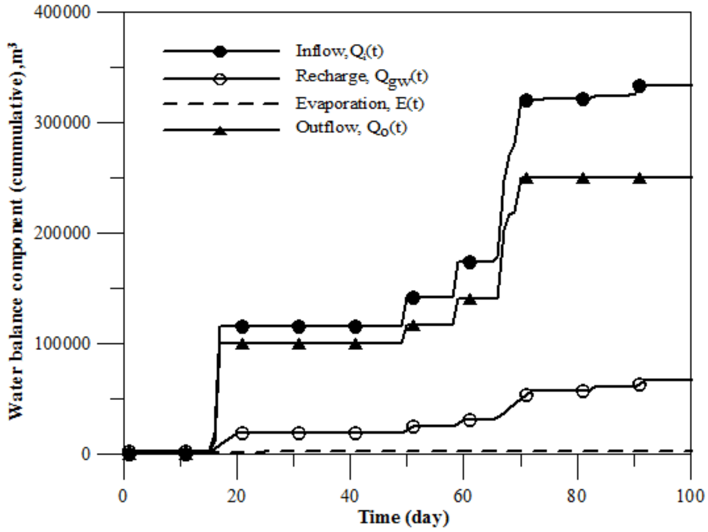

Fig. 4: Time varying cumulative water balance components of the basin.

Figs. 5 and 6 show the dimensionless plot of variation of groundwater recharge, $\left({Q_{gw}(n\Delta t)}/{(4\,a(n)b(n)\,K_v)}\right)$, and variation of in-basin depth of water, $\left({D(n\Delta t)}/{D_{max}}\right)$,versus time $\left({(n\Delta t)}/{t}\right)$ respectively for ${K_v}/{K}$ =0.1 with and without pumping for the trapezoidal basin. Fig. 5 clearly showed that aquifer pumping enhances, $Q_{gw}(n\Delta t)$ than without pumping because of increased head differences. Pumping underneath and around the basin has constraint to evolve the resulting groundwater level alike without pumping. The increased rate of recharge accelerates the time of empting the in-basin water (Fig.6).

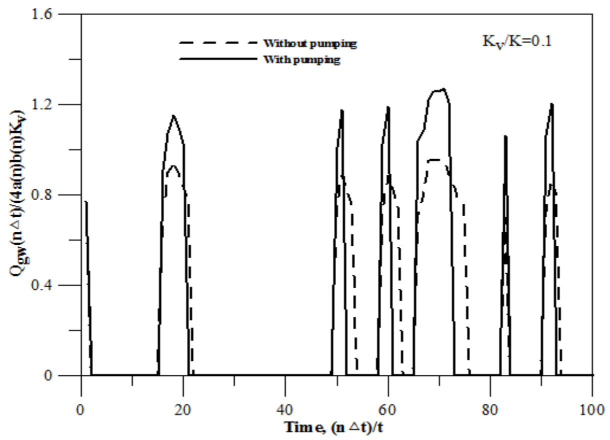

Fig. 5 : Dimensionless plot of time varying groundwater recharge, $\left({Q_{gw}(n\Delta t)}/{(4\,a(n)b(n)K_v)}\right)$, versus time $\left({(n\Delta t)}/{t}\right)$ for ${K_v}/{K} = 0.1$. with and without pumping beneath the trapezoidal basin.

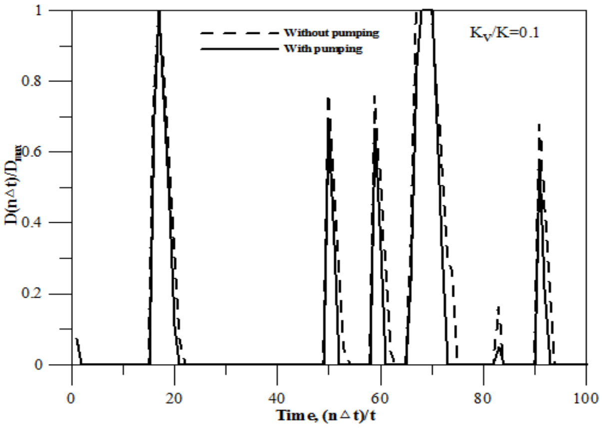

Fig. 6 : Dimensionless plot of time varying in-basin depth of water, $\left({D(n\Delta t)}/{D_{max}}\right)$ versus time $\left({(n\Delta t)}/{t}\right)$, for ${K_v}/{K} = 0.1$. with and without pumping beneath the trapezoidal basin.

The variation of $\left({Q_{gw}(n\Delta t)}/{(4\ a(n)b(n)\ K_v)}\right)$ and $\left({D(n\Delta t)}/{D_{max}}\right)$ versus $\left({(n\Delta t)}/{t}\right)$ for different ${K_v}/{K}$ with pumping underneath the basin are shown in Figs. (7) and (8). Higher the value of ${K_v}/{K}$, more is the recharge rate, $Q_{gw}(n\Delta t)$ (Fig. 7); and for more $Q_{gw}(n\Delta t)$, the basin will be emptied sooner than for the low value of ${K_v}/{K}$ (Fig. 7), if inflows are ceased for a long time. The variation of in-basin *D(nΔt)* (Fig. 8) also reaffirmed quick reduction of water depth due to pumping for all ${K_v}/{K}$ . . The smaller the ratio of ${K_v}/{K}$, longer is the duration of water stay in the basin (Figs. 7 and 8). These signify that by arranging recharge from, and pumping beneath a basin, more monsoon surface runoffs and aquifer storage beneath the basin can be conserved for subsequent recovery. The recovery of the stored water in the aquifer should be ≤ $Q_{gw}(t)$. $Q_{gw}(n\Delta t)$ also depends on the position of pumping wells; pumping near to the basin can help enhance $Q_{gw}(n\Delta t)$, but may associate risk of less treatment of recharged water because of inadequate travel time. The performances of the recharge model are thus advocated as the very promising and the model can be used as a tool for conjunctive management of surface and ground water.

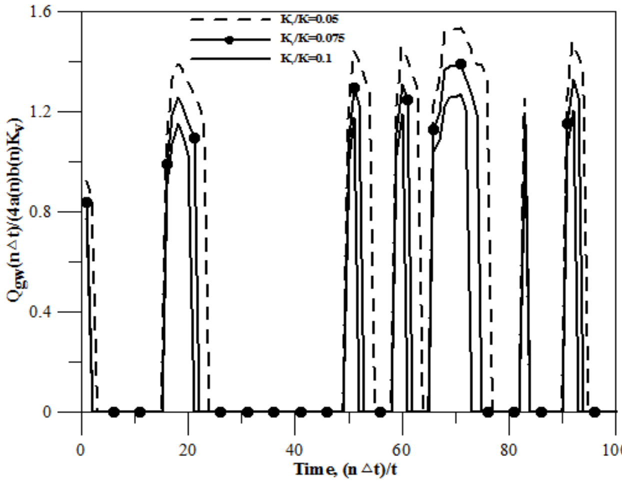

Fig. 7 : Dimensionless plot of time varying groundwater recharge, $\left({Q_{gw}(n\Delta t)}/{(4\,a(n)b(n)K_v)}\right)$, versus time $\left({(n\Delta t)}/{t}\right)$ for different values of ${K_v}/{K}$ with pumping beneath the trapezoidal basin.

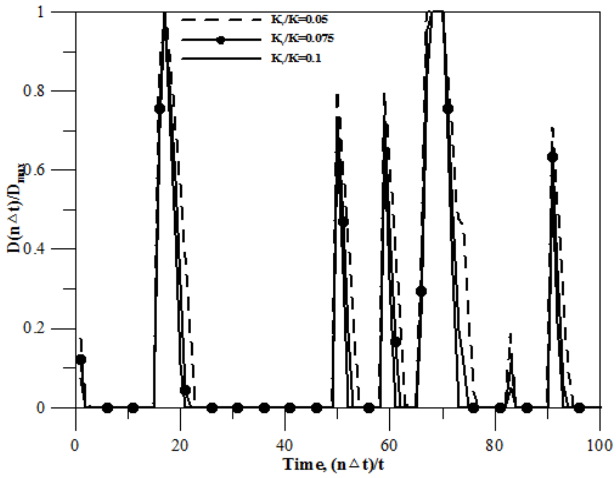

Fig. 8 : Dimensionless plot of time varying in-basin depth of water, $\left({D(n\Delta t)}/{D_{max}}\right)$ versus time $\left({(n\Delta t)}/{t}\right)$, for different values of ${K_v}/{K}$with pumping beneath the trapezoidal basin.

(ii) For rectangular basin

The time varying cumulative inflows for rectangular basin are shown in Fig 9. Figs. 10 and 11 showed the dimensionless plot of variation of groundwater recharge, $\left({Q_{gw}(n\Delta t)}/{(4ab\,K_v)}\right)$, and variation of in-basin depth of water, $\left({D(n\Delta t)}/{D_{max}}\right)$,versus time $\left({(n\Delta t)}/{t}\right)$ respectively for ${K_v}/{K}$ *=0.1* with and without pumping for rectangular basin. Fig. 10 also clearly demonstrated the similar picture as explained for the trapezoidal basin, except the differences in magnitude. Pumping underneath and around the basin has also showed the similar picyure as described for the trapeizodal basin level alike without pumping (Fig. 11)..

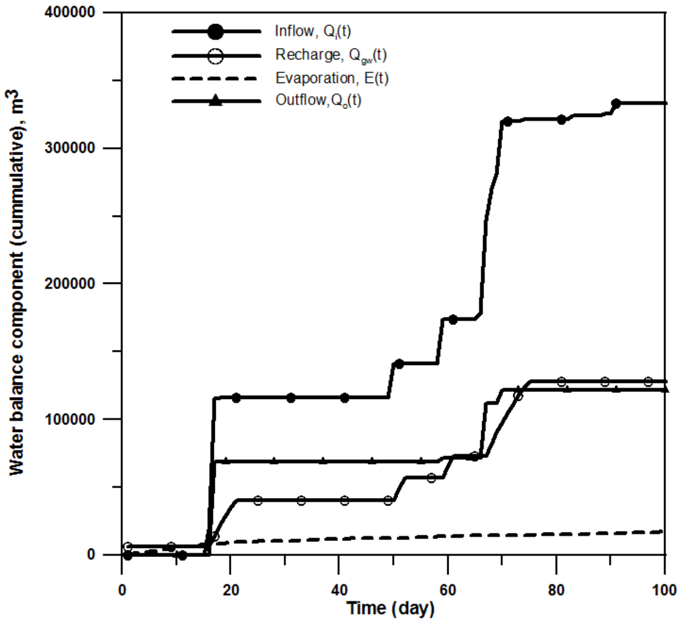

Fig. 9: Time varying cumulative water balance components of the rectangular basin.

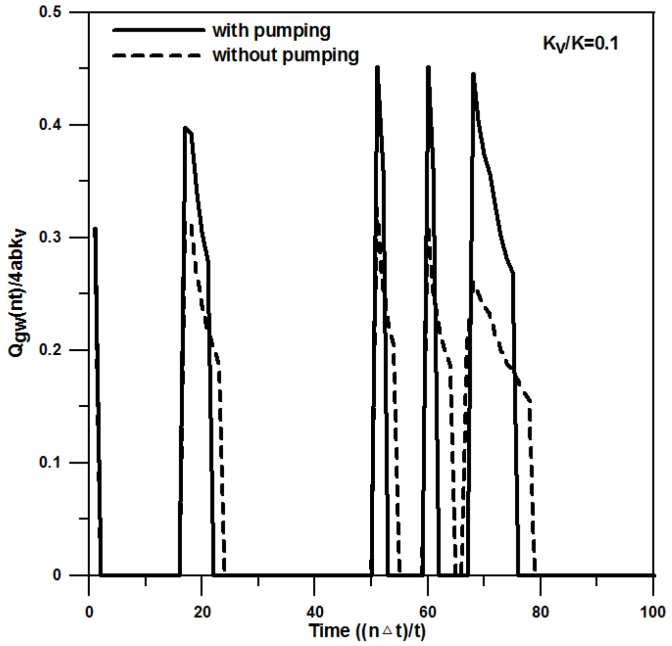

Fig. 10 : Dimensionless plot of time varying groundwater recharge, $\left({Q_{gw}(n\Delta t)}/{(4abK_v)}\right)$, versus time $\left({(n\Delta t)}/{t}\right)$ for ${K_v}/{K} = 0.1$. with and without pumping beneath the rectangular basin.

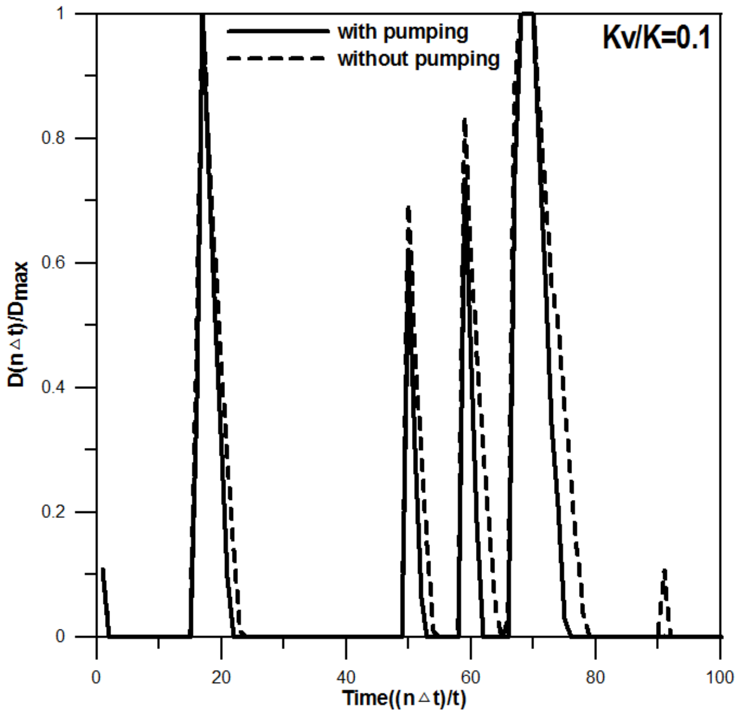

Fig. 11 : Dimensionless plot of time varying in-basin depth of water, $\left({}^{D(n\Delta t)}/_{D_{max}}\right)$ versus time $\left({}^{(n\Delta t)}/_{t}\right)$, for ${}^{K_v}/_{K} = 0.1$ with and without pumping beneath the rectangular basin.

The variation of $\left({}^{Q_{gw}(n\Delta t)}/_{(4abK_v)}\right)$ and $\left({}^{D(n\Delta t)}/_{D_{max}}\right)$ versus $\left({}^{(n\Delta t)}/_{t}\right)$ for different ${}^{K_v}/_{K}$ with pumping underneath the rectangular basin are in Figs. (12) and (13). Like the trapezoidal basin, the rectangular basin showed the similar results except magnitudinal differences, e.g,, higher the value of ${}^{K_v}/_{K}$, more is the recharge rate, $Q_{gw}(n\Delta t)$ (Fig. 12); and for more $Q_{gw}(n\Delta t)$, the basin will be emptied sooner than for low value of ${}^{K_v}/_{K}$ (Fig. 12), if inflows are ceased for a long time. The variation of in-basin *D(nΔt)* (Fig. 12) also reaffirmed quick reduction of water depth due to pumping for all ${}^{K_v}/_{K}$. The smaller the ratio of ${}^{K_v}/_{K}$, longer is the duration of water stay in the basin (Figs. 12 and 13). These signify that the performances of the derived recharge models are very promising.and the models can be used as a tool for conjunctive management of surface and ground water.

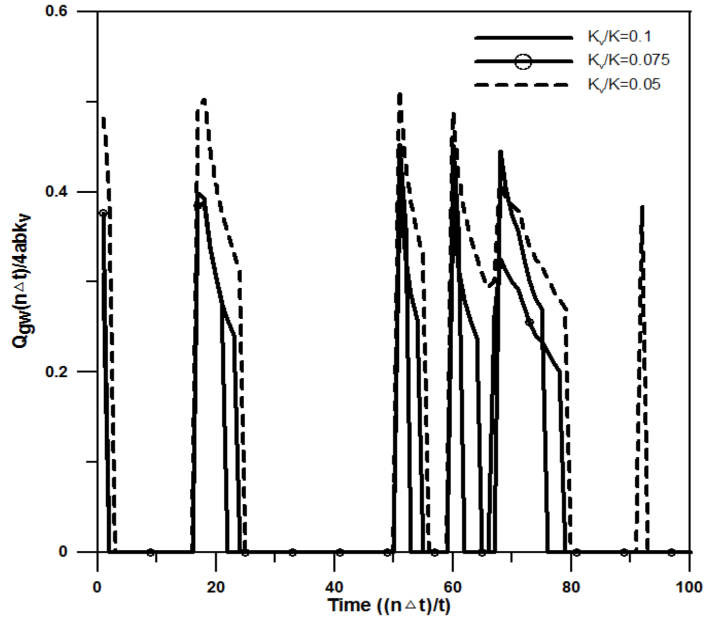

Fig. 12 : Dimensionless plot of time varying groundwater recharge, $\left({Q_{gw}(n\Delta t)}/{(4abK_v)}\right)$, versus time $\left({(n\Delta t)}/{t}\right)$ for different values of ${K_v}/{K}$ with pumping beneath the rectangular basin.

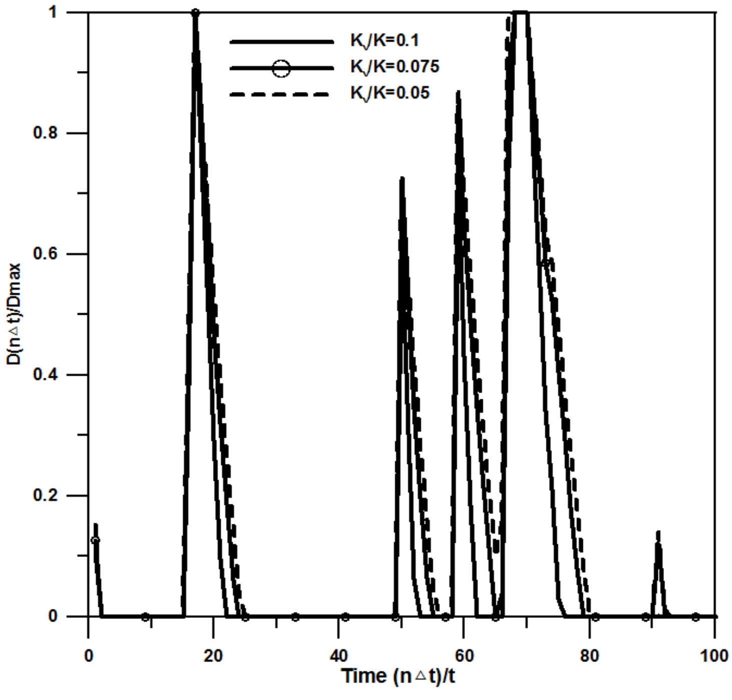

Fig. 13 : Dimensionless plot of time varying in-basin depth of water, $\left({D(n\Delta t)}/{D_{max}}\right)$ versus time $\left({(n\Delta t)}/{t}\right)$, for different values of ${K_v}/{K}$ with pumping beneath the rectangular basin.

4.2 Performance of Transport Models

The recharge rate, $Q_{gw}(n\Delta t)$ and in-basin depth of water, $D(n\Delta t)$ and the corresponding height of soil column beneath the basin, $h(n\Delta t)$ are time-varying because of variable $Q_i(n\Delta t)$, $E(n\Delta t)$, and $\Delta h(n\Delta t)$. Thus, the discrete kernel coefficients for contaminant transport through soil column, $\delta_c[h(n\Delta t),v_s(t),n\Delta t]$ *(eq 42)* will also vary. Using the estimated $h(n\Delta t) = h_0 - \Delta h$

(n∆t) and the corresponding $v_s(n\Delta t) = Q_{gw}(n\Delta t) / [\eta\, 4\, a((n)b(n)]$ and by following the procedure as explained in section 3.2.2.2, $\delta_c[h(n\Delta t), v_s(t), n\Delta t]$ for variable $h(n\Delta t)$, $v_s(n\Delta t)$, and $D_L(n\Delta t)$ together with the given value of λ and R, are generated using eq (42). By convoluting the in-basin concentration of contaminant, $C_b(n\Delta t)$ with the respective $\delta_c[h(n\Delta t), v_s(t), n\Delta t$, the effluent concentration, $C_e(n\Delta t)$ is computed using eq(19). The in-basin concentration of contaminant, $C_b(n\Delta t)$ for varying time in response to the influent concentration, $C_i(n\Delta t)$ is computed in succession of time step starting from n = 1 by using eq(16) together with the hydrological variables obtained from the recharge model. For computation of $C_b(n\Delta t)$, the in-basin concentration obtained in preceding time step, $C_b(n-1)\Delta t$ is taken as the initial in-basin concentration. Thus, the complete time-series of the in-basin concentrations of contaminant for the time varying hydrologic conditions of the basin are calculated. These time-series of computed $C_b(n\Delta t)$ are considered as influent concentration of contaminants for routing through soil column to compute $C_e(n\Delta t)$ using eq (19).

(i) For trapezoidal basin

The non-dimensional plot of $C_b(n\Delta t) / C_i(n\Delta t)$ versus $\left((n\,\Delta\,t) / t\right)$ for recharge rate in response to $K_v / K = 0.1$ and that for $C_e(n\Delta t) / C_i(n\Delta t)$ versus $\left((n\,\Delta\,t) / t\right)$ through soil column for R = 1 and 1.2 are respectively shown in Fig 14 and Fig. 15. The variation of $C_b(n\Delta t) / C_i(n\Delta t)$ (Fig. 14) is influenced by $E(n\Delta t)$, $Q_{gw}(n\Delta t)$, λ, in-basin accumulation and retention of influent concentration, $C_i(n\Delta t)$. Smaller the K_v / K, higher is the in-basin concentration of contaminants because of larger in-basin retention time. R = 1 means no sorption of contaminants. For R > 1, the contaminants are adsorbed in the beginning and desorbed at the later stages by the soils. These results in deviation of $C_e(n\Delta t)$ for R = 1.2 at any time than that of R=1 at that time (Fig. 15). For R > 1, $C_e(n\Delta t)$ continues for a longer time than *R = 1* because of desorption by the soils. From Fig.15, it can also be seen that constant $C_i(n\Delta t)$=50 mg/L has reduced markedly before mixing with the groundwater, because of the in-basin assimilation and transport through underneath saturated soil column by the process of dispersion, decay and sorption. The computational performances of the transport models are found to the expected lines.

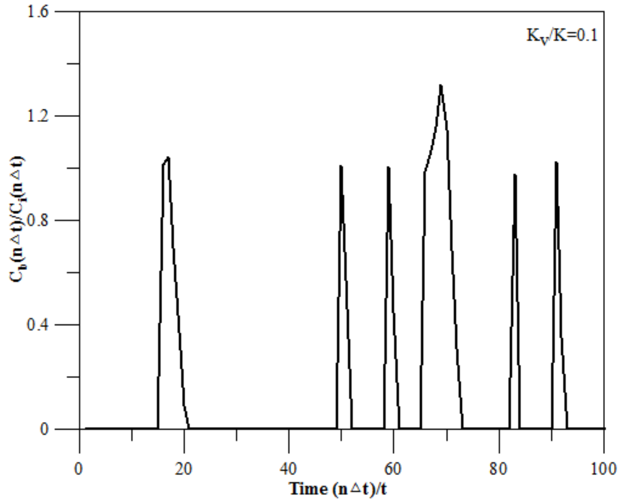

Fig. 14: Dimensionless plot of in-basin contaminant concentration, $C_b(n\Delta t)/C_i(n\Delta t)$ versus $((n\Delta t)/t)$ for recharge rate in response to $K_v/K = 0.1$ for trapezoidal basin.

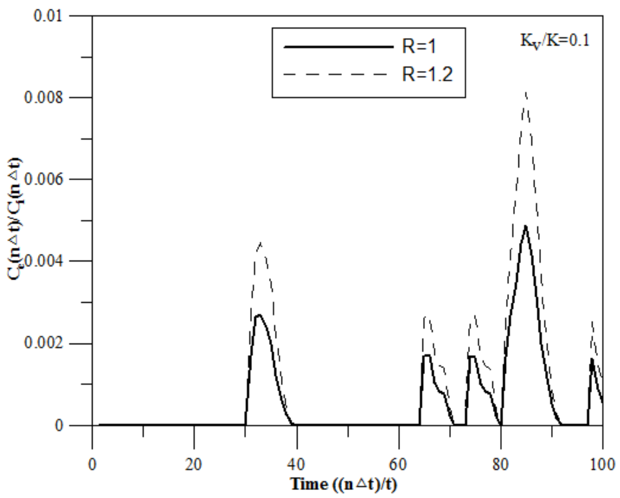

Fig. 15: Dimensionless plot of contaminant concentration through soil column, $C_e(n\Delta t)/C_i(n\Delta t)$ versus $((n\Delta t)/t)$ for R = 1 and 1.2 for recharge rate in response to $K_v/K = 0.1$ for trapezoidal basin.

(ii) For rectangular basin

In case of rectangular basin, the seepage velocity component will only be changed, viz., $v_s(n\Delta t) = Q_{gw}(n\Delta t)/[\eta\, 4ab]$ because of uniform cross-sectional area. The in-basin concentration of contaminants, $C_b(n\Delta t)$ will also be influenced by $E(n\Delta t)$, $Q_{gw}(n\Delta t)$, and λ. , The non-dimensional plot of $C_b(n\Delta t)/C_i(n\Delta t)$ versus $((n\Delta t)/t)$ for recharge rate in response to $K_v/K = 0.1$ and that for $C_e(n\Delta t)/C_i(n\Delta t)$ versus $((n\Delta t)/t)$ through soil column for R = 1 and 1.2 are respectively shown in Fig 16 and Fig. 17. The variation of $C_b(n\Delta t)/C_i(n\Delta t)$ (Fig. 16) showed similar characteristics as that of the trapezoidal basin except the differences in magnitude. Fig.(16) also showed that smaller the value of ${K_v}/{K}$, higher is the in-basin concentration of contaminants because of larger in-basin retention time. All other characteristics of the contaminants like, for R > 1 (Fig. 17), it showed the similar behaviour as that for the trapezoidal basin, except the differences in magnitude.

The computational performances of the transport models have been found to the expected lines. Hence, the effectiveness of the derived transport models demonstrates very potential tools for simulation of ASTR in MAR.

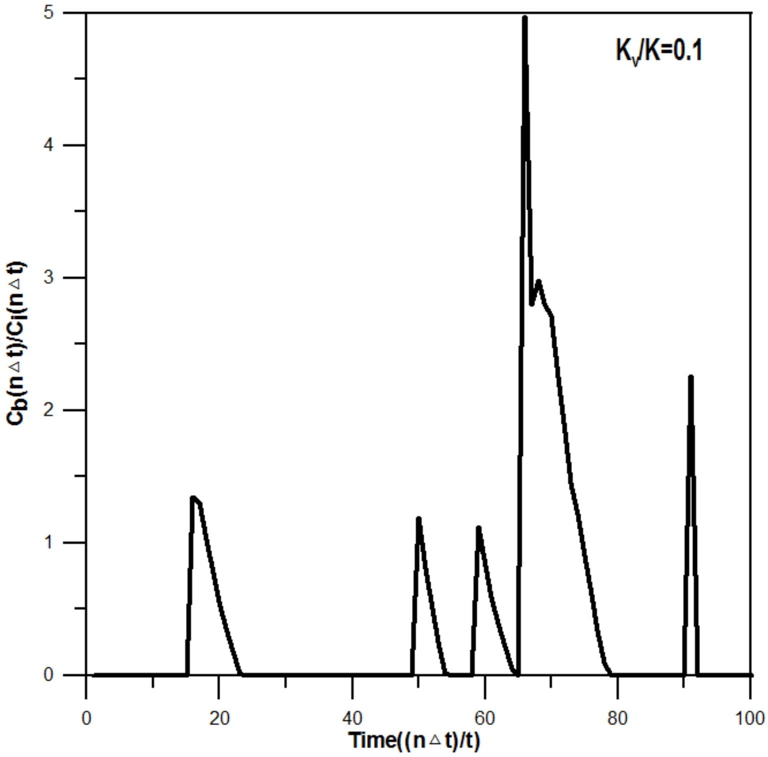

Fig. 16: Dimensionless plot of in-basin contaminant concentration, ${C_b(n\Delta t)}/{C_i(n\Delta t)}$ versus $\left({(n\Delta t)}/{t}\right)$ for recharge rate in response to ${K_v}/{K} = 0.1$ for rectangular basin.

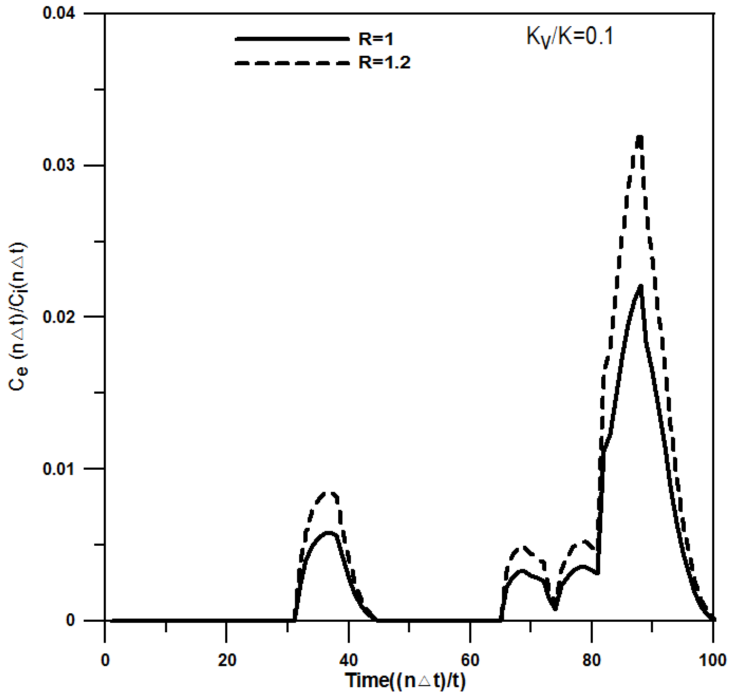

Fig. 17: Dimensionless plot of contaminant concentration through soil column, ${C_e(n\Delta t)}\big/{C_i(n\Delta t)}$ versus $\left({(n\Delta t)}\big/{t}\right)$ for R = 1 and 1.2 for recharge rate in response to ${K_v}\big/{K} = 0.1$ for rectangular basin.

Conclusions

Managed aquifer recharge (MAR), for augmentation of groundwater resource in depleted aquifers and also for subsequent recovery of recharged water, is promoted as an integral part of IWRM in many areas by conserving excess monsoon surface runoffs employing recharge basin. Groundwater recharge rates from a basin are functions of inflow rate, evaporation rate, and also accumulated depth of water in the basin, and hence, time varying. Pumping around the basin for recovery of recharged water enhances the recharge rates due to resulting affects of recharge and extraction of groundwater. These eventually complicate the computational process. Further, the fate of contaminants inflows to a basin changes due to in-basin accumulation and detention of contaminants, and transport through saturated soil column by the process of advection-dispersion-decay and sorption. Variable inflows, evaporations, detention times, and recharge rates make the contaminant transport computation further complicated.

As a scientific tool for conjunctive management of surface and ground water including contaminant transport simulation on similar line as that to the MAR and aquifer storage recovery (ASR), process based semi-analytical models for computation of recharge influenced pumping and contaminant transport from a recharge basin have been presented in this report. The process based models have been developed employing basic water balance equation for recharge estimation, and mass transport conservation equation for contaminant transport. For the recharge and extraction model, Hantush's analytical equation for rise/fall of groundwater level due to recharge from a rectangular basin and Theis pumping well equation have been integrated to the basic water balance equation.

The models for computation of concentration of contaminant have been developed by considering: (i) in-basin mass balance together with decay of contaminant and, (ii) one-dimensional advection-dispersion-decay equation coupled with non-equilibrium Freundlich isotherm equation, respectively. Duhamel's convolution equation and method of superposition have been used for determining the resulting water table position due to pumping and recharge, and also for computation of concentration of contaminants.

The performances of both recharge and contaminant transport models demonstrated by an illustrated example showed results to the expected lines. The developed models can promisingly be used as tools for estimation of time-varying recharge influenced by extraction for unsteady flows to/from a recharge basin together with computation of in-basin contaminant transport and through soil column underneath the basin. The web-enabled GUI interface of the derived models would provide a platform to users and professionals for calculating time-varying depth of water in, and groundwater recharge from, a recharge basin consequent to the pumping in the vicinity of the basin for recovery of recharged water. User can also visualize the output in graphical as well as tabular format. The web-enabled GUI is easy to handle and free to access and also would not require additional resource to work.

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