Assessment of artificial groundwater recharge using greenhouses runoff (North east Naivasha, Kenya)

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by

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Disclaimer

Dedication

To my parents and especially to my beloved wife,
Who had made a lot of affords looking after
the children while I was away for study at ITC.

Abstract

Feasibility study on artificial recharge was conducted in the horticultural area north of Lake Naivasha. The use of groundwater for irrigation purposes has tremendously increased since the last 8 years. This causes a decline of about 10m of the water level. This study is targeted to assess the feasibility of artificial ground water recharge using the runoff harvested from the greenhouses. The quality and quantity of the available water resource, identification of suitable sites for various alternatives and scenarios, and economics based on preliminary cost assessments are discussed.

During field study, Injection and pumping test was carried out to determine the intake capacity and hydraulic property of the aquifer. In addition soil and ground water samples were taken for grain size and water quality analysis respectively. A spreadsheet model developed to analyze the recharge efficiency and cost per cubic meter of recharge water for 31 years duration of daily rainfall.

The field experiment result shows the existence of vertical and horizontal variations of hydraulic parameters with the transmissivity value ranging from 800 to 1200 m^2day^{-1}. The porosity and specific yield of the aquifer are estimated 0.3 and 0.15 respectively. The injection rate with a pressure head of 30m with a hole of 40m deep is in the order of $160m^3hr^{-1}$. It is known that the 40-60m deep part of the aquifer has very high transmissivity. So the actual recharge potential may be double or more than the injection rate. Unfortunately this could not be tested due to collapsing of the boreholes starting around 38m depth.

The runoff volume calculated by rational empirical method results in a maximum of $35,000m^3day^{-1}$. Runoff coefficient of 0.89 is found for the present study using the field experiment of rainfall-runoff record. The scenario analysis of the water balance in the area shows that about 46% of the ground water abstraction can be saved by utilizing the runoff from greenhouses for aquifer recharge.

A cost per cubic meter surface (storage capacity vs recharge rate) was established from the spreadsheet model allowing finding the most cost effective scenario; a basin of $9600m^3$ combined with recharge wells with a total potential of $7000m^3day^{-1}$ is an optimum solution. In this case the the recharge efficiency is 89% and the cost per meter cube is 0.56 KES (Kenyan shilling) .The model set up implemented based on the work of Kibona (2000) results in 3.4m and 5m rise in water level around the productive wells and recharge wells respectively. The water budget simulated in two scenarios with and without artificial recharge indicates that 90% of the inflow is from constant heads boundaries simulating inflow from the lake and in the second scenario this value is lowered to 50% due to the artificial recharge.

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I would like to express my gratitude to the Government of Ethiopia, through Tigray Water Resources Development, for releasing me to pursue this course. My appreciation goes to the Canadian government through *WATER HARVESTING & INSTITUTIONAL STRENGTHENING TIGRAY (WHIST) AND A CANADIAN INTERNATIONAL DEVELOPMENT AGENCY (CIDA) PROJECT* who shouldered the financial burden of my study.

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AND above all, Praise be to the Merciful ALLAH who enabled me to accomplish this hectic course in sound health.
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1. Introduction

Ground water is the subsurface water that occurs beneath the water table in the soils and geologic formations that are fully saturated (Freeze, 1979).Safe ground water abstraction and proper groundwater management is important for sustainability of the resource. Safe yield is the amount naturally occurring groundwater that can be withdrawn from an aquifer on sustained basis, economically and legally, without impairing the native ground-water quality or creating an undesirable effect such as environmental damage (Fetter, 2001).

Water management can be defined as comprehensive planning for beneficial use, plus operation for optimum economic and social benefits, of total water resources (UN, 1975).The increase demand for water in many regions has led to the implementation of more intensive water management measures to achieve more efficient utilization of limited available water supplies. The natural replenishment of ground water occurs very slowly. If ground water is exploited at a rate greater than that of its natural replenishment it will cause declining ground water levels and lake, since the lake is hydraulically connected to the ground water system. In the long term, it causes destruction of the ground water resources. To increase natural replenishment of groundwater reserves, artificial recharge of ground water has become increasingly important.

1.1. Problem statement

Lake Naivasha is the only fresh water lake on the floor of the rift valley Kenya with good quality. Growth in industries, commercial irrigation agriculture, and municipal sectors around the lake has resulted in increasing utilization of water, which has caused considerable pressure on the development and sustenance of ground water reservoirs. Several wells have been drilled for the aforementioned purposes. However for the last few years study indicates that the groundwater and lake level have been declining. The groundwater level is fluctuating with the increasing well abstraction especially to the north east where intensive irrigation activities under development. Large cone of depression and lowering the groundwater level have been observed (Yihdego, 2005).The lake is hydro logically linked to the aquifers underlie it. In view of the increasing demands and limited resources, there is a need to obtain additional groundwater recharges for the sustainable use of groundwater resources and proper management.

1.2. Importance of the study

Lake Naivasha has social and economic importance to the surrounding area. The exploitation of the area started in the late 1960's and has accelerated since 1998. The abstraction rate has been constantly increasing over the last 15 years due to expanding area under irrigation (Becht and Harper, 2002). As a result of higher abstraction rate or lake level change cone of depression and inversion of natural hydraulic gradient has been observed around the north east. An artificial groundwater recharge can

significantly increase the sustainable yield of an aquifer. Increasing demand of water for irrigated agriculture has created awareness in the use of artificial recharges. Natural replenishment of groundwater recharge is very low around the lake sediments due to low hydraulic conductivity of the formation. The importance of this study is to assess artificial groundwater recharge for the sustainability and proper management of the groundwater resources

1.3. Research objective

- Major objective
- To improve the groundwater management using an artificial groundwater recharge from Greenhouses.
- Specific objectives
- To evaluate quantity and quality of the available runoff water from the greenhouse
- To evaluate artificial recharge scheme based on the geology, geo-physics and hydrogeology
- To analyse the preliminary cost of an artificial schemes.
- Minor objectives
- To evaluate the quantity of Artificial recharge schemes using groundwater flow modelling (MODFLOW)

1.4. Research questions

- How can the runoff generated from the surrounding area be used for artificial recharge?
- What are the possible alternatives location and method of artificial groundwater recharge?
- What is the effect of artificial recharge on the quantity and quality of the groundwater?
- What is the cost per cubic meter of recharged water under different alternatives?
- What type of artificial method is preferred and how to prevent clogging?

1.5. Literature review

1.5.1. Previous study

Water resources evaluation and management of the Naivasha basin was studied for many years to determine and understand the surface and groundwater resources occurrence, distribution and potential of the basin. The Naivasha catchment has an internal drainage system. There is no surface outlet. The water inputs to the lake include rainfall, inflow from rivers, and underground seepage. The outputs are evapotranspiration, underground seepage out of the lake and water abstraction. The groundwater flow system affects the lake level and causes the lake to be fresh.

The hydrogeology of the basin is complex due to extensive volcanism and cyclic deposition. The lacustrine sediments and volcanic deposits consist of clay, silt, fine sand, and course sand. The general groundwater flow direction is towards the lake from the eastern Mau escarpment and western side from Kinangop and Aberdare escarpments.

Groundwater modelling is essential tool to evaluate the groundwater flow and quantifying the potential. It also helps to understand and predicting the behaviour of the ground system in response to future stresses. A mathematical model simulates groundwater flow indirectly by means of governing equations though to represent the physical processes that occur in the system together, with equations that describe heads or flows along the boundaries of the model (Anderson and Woessner, 1992).

A number of groundwater models have been developed to estimate the long term water balance of the lake groundwater system of the Naivasha basin. (Owor, 2000) Developed conceptual model based on the hydro geological condition of the basin to analyse the long term interaction of groundwater system to determine the long term budget for the lake and estimate water abstraction from the surface-groundwater resources.

Kibona(2000) modelled the north Naivasha using two and three aquifer layer models. She sought to understand the variation of the groundwater levels due to the abstraction in space and time by setting up both transient and steady state. Groundwater numerical modelling developed by (Yihdego, 2005) was updating the conceptual model from existing geology, hydrogeology, geophysics, isotope analysis and additional field data. He used also model of (Owor, 2000) and enhanced the conceptual model of the basin for better understanding of the hydrological parameters and spatial–temporal variability of the hydrological stresses and boundary conditions.

1.5.2. Artificial groundwater recharge

Artificial groundwater recharge is the planned human activity of augmenting the amount of groundwater available through works designed to increase the natural replenishment or percolation of surface waters into the groundwater aquifers, resulting in a corresponding increase in the amount of groundwater available for abstraction. Artificial groundwater recharge is an important technology in water resources management (AMCE and EWRE, 2001).

1.5.2.1. Methods

Artificial recharge can be used for a number of reasons: Integrated water management, seasonal storage and recovery of water, long-term storage or water banking, emergency storage or strategic water reserve, short term storage, enhance well field production, restore groundwater level, replace over draft, raise water levels, reduce pumping cost, stop or reduce rate of land surface subsidence, improve groundwater quality to agriculture or municipal standards e.t.c (Raju et al., 1994). The recharge objectives are important to select and prioritize these that are applicable to the study area.

In general there are two methods of artificial recharge: Direct and Indirect

A. Direct method

Direct methods can be divided into surface recharge techniques and subsurface recharge techniques. In surface recharge, water moves from the land surface to the aquifer by means of infiltration through the soil. The surface is usually excavated and water is added to spreading basins, ditches, pits, and shafts and allowed to infiltrate. Surface infiltration

Surface infiltration consists of in-channel and off-channel facilities

A-On-channel system consists of Dams and weirs, canals, finger dikes or other structures in the stream bed or flood plain to impound and spread the water over as a large wetted area as possible, increasing infiltration volume.

B-Off-channel system consists of recharge basins, pits, ponds and ditches specially constructed by excavation, construction of berms (or both) or by use old gravel pits, borrow areas, or similar excavations. This method involves surface spreading of water in excavated basins. The amount of water entering the aquifer depends on three factors: the infiltration rate, the percolation rate, and the aquifer's capacity for horizontal water movement.

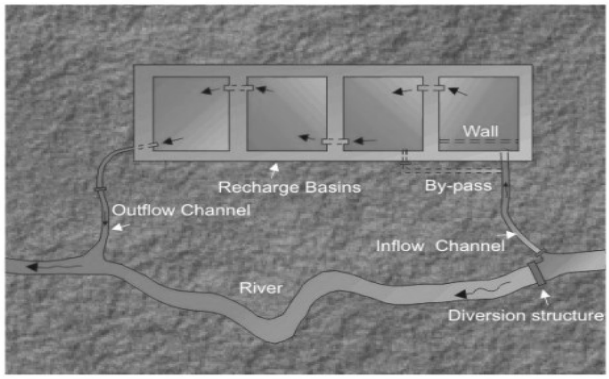


Figure 1.1Infiltration basins (adopted EOLSS)

Subsurface (Injection wells)

Injection techniques are used as an alternative to surface spreading operations when a zone with low permeability, within the unsaturated zone, impedes the recharge of the water to a designed aquifer.

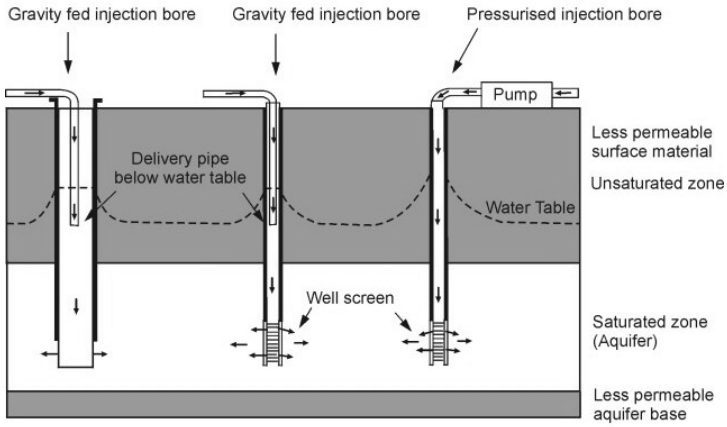


Figure 1.2 Gravity injection wells or bore (adopted EOLLS)

B. Indirect methods

Indirect method include installing groundwater pumping facilities near connected surface water bodies to lower groundwater levels and induce infiltration elsewhere in the drainage basin. Indirect methods include modifying aquifers to enhance groundwater reserves.

C. Combination system

The mixed recharge is the combination of infiltration and recharge wells. The advantage of this system is that the water has been pre filtered through the soil and the perched groundwater zone, so that its clogging potential is significantly reduced. In this way the risk of aquifer obstruction is reduced.

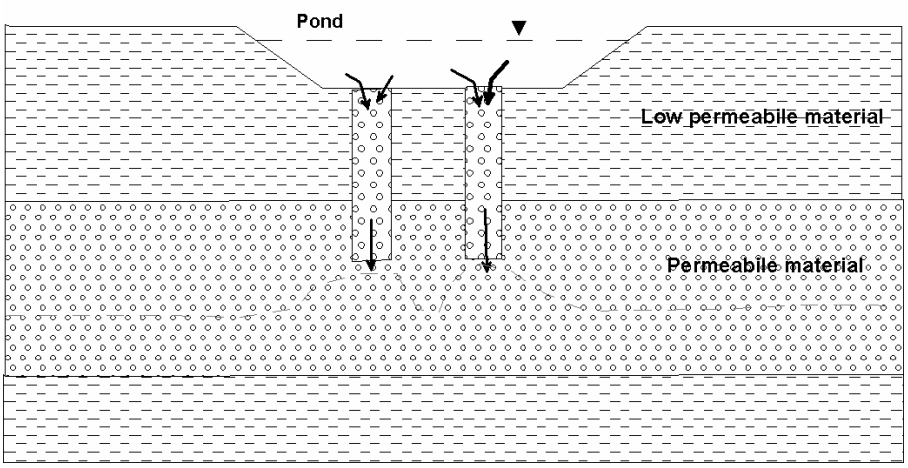


Figure 1.3 Combination of injection well and basin

1.5.2.2. Study of Artificial recharge

An Artificial recharge study should consider the water availability and quality, geology and hydrogeology and economical aspect of the recharge scheme.

The source of water needs careful consideration of the quantity and quality of alternative sources. The source should be evaluated as the average flow available; variability (daily to very long periods) in both flow rate and quality (Raju et al., 1994).

The geology and hydrogeology study determines the selection of suitable storage zones, recharge water sources as well as the location and type of recharge facilities. The general hydro geologic evaluation of the groundwater basin should consider the Surface soil and unsaturated zone characteristics, Aquifer characteristics: litho logy, areal extent and depth, Hydrologic boundaries, subsurface geologic structures (AMCE and EWRE, 2001).

The decision to construct a groundwater recharge project should be made after a comparison with alternative sources based on the unit cost of the water produced. The major costs to be included in the implementation of artificial recharge scheme are: Planning cost: cost incurred during each phase of

investigation, engineering cost, construction cost, operation and maintenance cost and contingency costs.

1.6. Methodology

The methodology followed in this study was based on the objectives of the study as stated in section 1.3. It consists of the activities, materials used and diagrammatic representation of the Methodology.

1.6.1. Pre_field work

- Literature review of the work done already in the area
- Data collection for and processing of already available data.
- Acquisition of equipment for field work
- Proposing test boreholes based on the geology and existing well logs.

1.6.2. Field work

A three week field work was carried out from September 13 to October7, 2004. The following activities were carried out in the field.

- Drilling test boreholes, test pits and auger holes
- Collection of recently drilled boreholes
- Conducting Injection and pumping test
- Collection of soil samples
- Collection of water samples from the wells
- Description of geological observation points

1.6.3. Post field work

Data organizing, processing and analysis of

- Injection and pumping test analysis
- Water sample analysis
- Soil sample analysis
- Geophysical analysis
- Conceptual model of the area in ILWIS

1.6.4. Frame work of the research

The research has two major tasks. These are interpretation and analysis of the geology hydrogeology, geophysics and water quality for the purpose of artificial recharge and secondly to apply modelling to analysis the artificial recharge. The schematic representation of the breakdown and sequence of the study process is below in flow chart Figure 1.4.

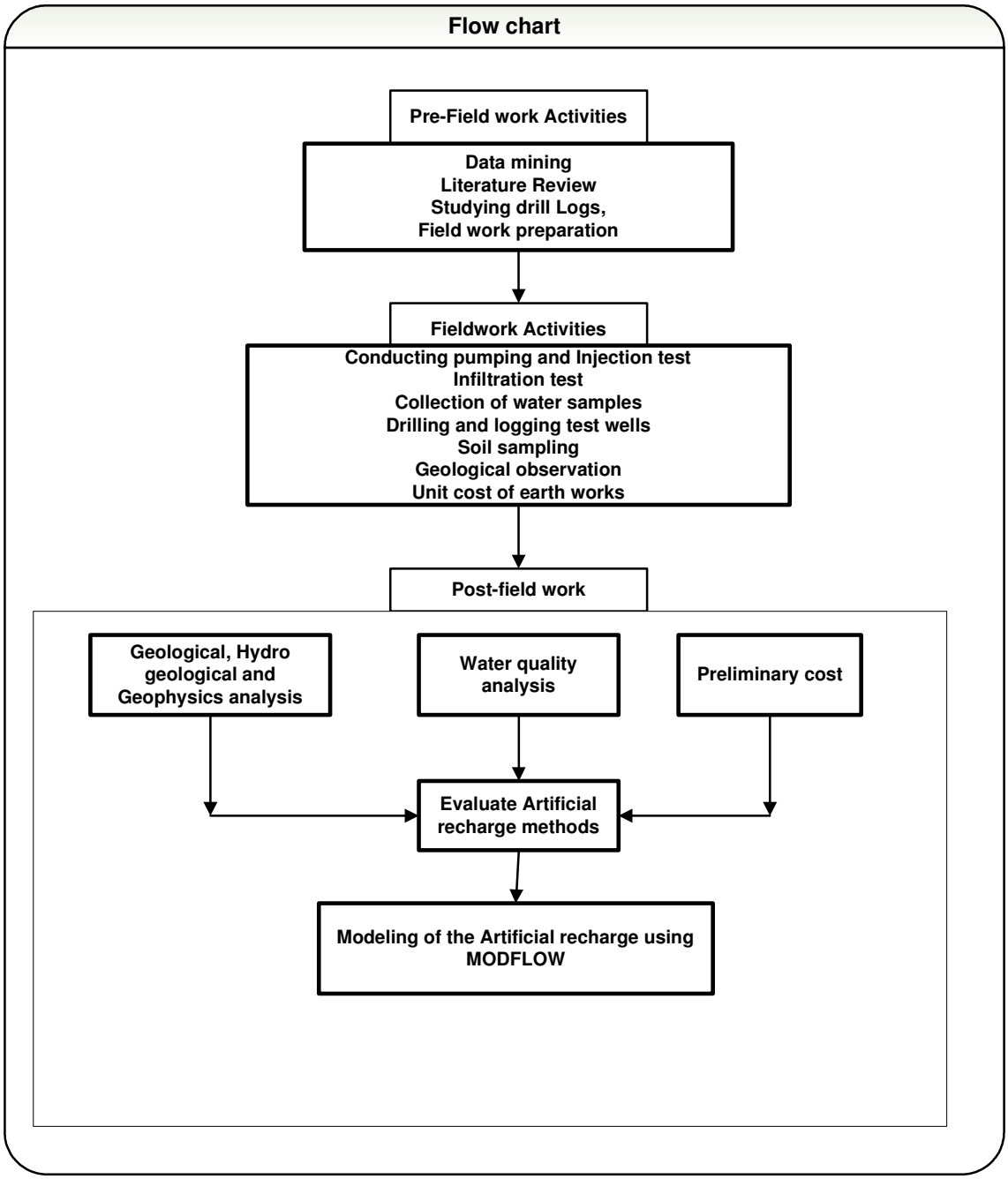


Figure 1.4 Flow chart of the study process

2. Description of the study area

2.1. Location and accessability

Naivasha basin is located in the Rift Valley Province of south-western Kenya, within the administrative district of Nakuru. It is located within the UTM zone 37 and its geographical coordinates are $0^0$00' to $1^0$00'S and $36^0$00' to $36^0$45'. The basin lies about 100 km to the Northwest of Nairobi. It is accessible by the mainline of the East African railways and a major road that services the western part of the country. There is an even distribution of all weather roads within the area. The study area is situated in North-eastern part of the Naivasha basin at a mean altitude of 1885m above mean sea level.

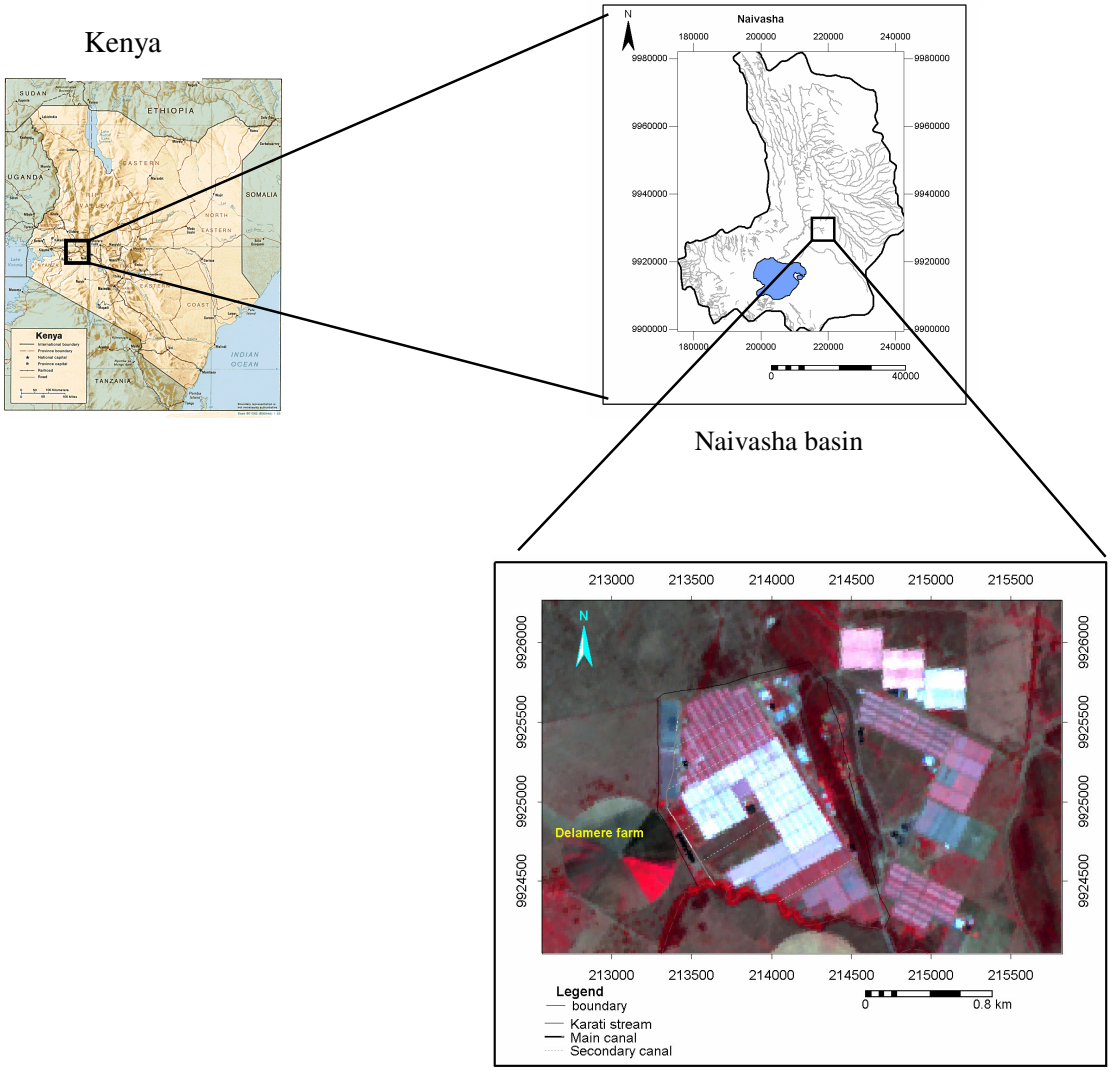


Figure 2.1 Location map of the study area

2.2. Physiography, Land use and climate

Three main geomorphologic units are found in the Naivasha basin. In the western part lies the Mau Escarpment, the Kinangop in the east and the rift valley plain that the lake basin forms a part. Lake Naivasha dominates the central part of the Navaisha basin. It has a mean surface area of 145 km^2 at an average altitude of 1887.3m.a.m.s.l (Gitonga, 1999).

The major land use units of the study area are agricultural (horticulture and flower growing), Settlements, Game sanctuaries, Rangeland (dairy) and natural vegetation and Forests Horticulture and flower growing is concentrated around the lake. The natural vegetation surrounding the lake is mainly papyrus swamp vegetation. Natural vegetation outside of the lake surroundings are shrub, acacia, cactus trees and savannah.

The climate of the basin lies within the semi arid belt of Kenya with mean annual precipitation of 600 to 800mm in the rifts and about 1300mm annual precipitation in the plateau (Abdulahi, 1999). The average monthly temperature ranges from 15.90 to 17.80c. The rain seasons are from April to May and October to November. There is annual potential evaporation estimated at about 1700mm (Ashfaque, 1999).

2.3. Drainage network

The main rivers that drain to the study area are the Gilgil (420km^2 watershed), Karati and Malewa (1750km^2 watershed). The rains on the Aberdare Mountains and Kinangop plateau maintain the perennial flow in the Malewa River. The Malewa and Gilgil drain from the northern part of the catchment while Karati River drains from the northeastern part. Flow in the Karati River and other streams are seasonal and often don't reach the lake as surface water. The Malewa River is one of the two main perennial rivers that drain the lake and flow in a graben at the foot of the kinanagop plateau. The Malewa and Turesha rivers have a combined drainage area of about 1,730 km^2.The Kinanagop rivers are captured by the main Malewa river in the north east of the basin. Further downstream the Malewa River is joined by the Turash a river and the two flow south wards. The Gilgil River flows in a narrow basin to the north of the basin and is the second major perennial river that drains

2.4. Regional Geology and structure

2.4.1. Geology

The Kenya rift valley was created by the tectonic and volcanic activities in the yearly to mid –miocen. The geology of the area is generally made of volcanic rocks and lacustrine deposits, which have been subjected to several tectonic processes leading to varying structural features. The geology of the Naivasha basin composed of the late tertiary and quaternary volcanics with lacustrine beds and alluvium of reworked debris. The volcanic rocks in the area consist of tephrites, basalts, trachyte, phonolites, ashes, tuffs, agglomerates and the acidic lavas rhyolites, commendite and obsidian. The lake beds are mainly composed of reworked volcanic materials or sub-aqueously deposited pyroclastics.

The Mau escarpment is largely composed of the ignimbrite succession dominated by tuffs with only rare outcrops of agglomerates and lavas. The rifting has produced blocks down-faulted to the east along the escarpment. The maximum exposed thickness is about 100m. The kinangop plateau appears in the north-eastern part of the area only, where it lies between the southern mountains of the Aberdare range and the rift floor.

The lake beds are mainly composed of pumiceous granules (pebble gravel, diatomites, coarse sand, silt and clay). The maximum thickness of exposed beds is about 15m.

Along the Malewa River valley are alluvial deposits that include silt, fine sand, some ferruginous coarse sand and boulder gravel. The stratigraphy of Naivasha basin was tentatively classified as follows according to their age by (Thomson and Dodoson, 1958) in Table 2.1.

Age	Archaeological Stages	Rock Types Approximate Thickness	Main Locality	Remarks
Holocene	Neolithic	Trachytes and ashes Obsidians	Longonot Southern slopes of Eburru and Cedar Hill	Climate as present day
		Basaltic ash cones Basaltic flows Silts 3.1 meter	Badlands Badlands Nderit River	Wetter than present
		Ashes	Longonot	Drier than present day
	Mesolithic	Gravels and silts 6.1 meter Trachytes	Nderit Longonot	Slightly wetter than present day Lake Naivasha 36.6 feet higher than present lake
Upper Pleistocene	Upper Paleolithic	Obsidians	Eburru	Drier that present day
		Lake beds 30.5 meter	Lake Naivasha	Lake Naivasha terraces
	Middle Paleolithic	Basaltic ash cones Rhyolites Phonolites Trachytes Basalts Comendites Phonolites Trachytes } with intercalated pyroclastic	Badlands Eburu & S.W. Naivasha Eburru Eburru Badlands Lower Eburru Lower Eburru East of Karterit	Drier than present day Faulting Much volcanic activity in the Rift floor
Middle Pleistocene	Lower Paleolithic	Swamp deposits Pyroclastics } minimum 15 meter	Kinangop and Mau Escarpments (diatomite) of Kariandus	Intense rifting and faulting Wetter than present day Erosion
		Welded tuff 90 cm Pyroclastics & sediments } 122 meter Trachytes 30.5 meter Pyroclastics & sediments w/ intercalated trachytes } 122 meter	Rift Walls	Wetter than present day
Lower Pleistocene		Kijabe-type basalt 45.7 meter		

Table 2.1 Summary of geological succession in the study area

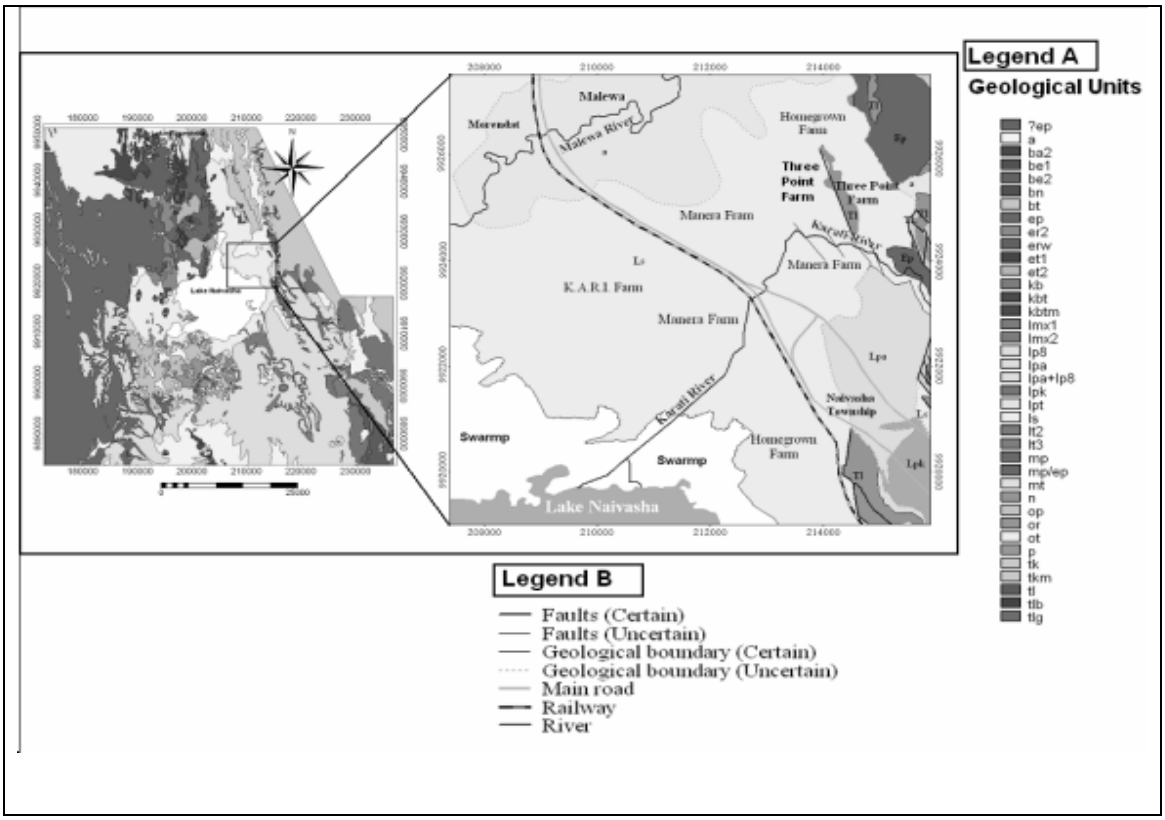


Figure 2.2 Geological map of the study area

Unit	Description
?ep	Eburru pumice, pantellerite, trachytic pumice, ash fall deposits
a	Alluvial deposit
ba2	Alkaria basalt, basalt and hawaiite lava flows, pyroclastic cones
be1	Older elementeita basalt, hawaiite lava flows, pyroclastic cones
be2	Younger elmenteita basalt, basalt, hawaiite and mugearite / benmoreite lava flows and pyroclastic cones
bn	Ndabibi basalt, hawaiite lava flows, pyroclastic cones
bt	Surtseyan / strombolian ash cones
ep	Eburru pumice, pantellerite, trachytic pumice, ash fall deposits
er2	Eastern eburru pantellerite and trachyte pumice, ash deposit
erw	Waterloo ridge pantellerite, welded and unwelded pyroclastics
et1	Older eburru trachyte, lava flows and pyroclastic
et2	Younger eburru trachyte, lava flows and pyroclastic cones
kb	Kijabe hill basalt
kbt	Surtseyan tuff cones
kbtm	Surtseyan tuff cones with laterally equivalent fall tuffs
lmx1	Lower longonot mixed basalt / trachyte lava flows and pyroclastic cones
lmx2	Upper longonot mixed basalt / trachyte lava flows and pyroclastic cones
lp8	Longonot ash
lpa	Longonot alkaria pumice
lpa+ip8	Longonot ash and alkaria pumice
lpk	Kedong valley tuff, trachyte ingimbrites and associated fall deposit
lpt	Longonot volcanic, pre-caldera welded pyroclastics and lava flows
ls	Lacustrine sediments
lt2	Lower longonot trachyte, lava flows and pyroclastic cones
lt3	Upper longonot trachyte, lava flows and pyroclastic cones
Mp	Maiella pumice, trachyte, pantellerite pumice and ash fall deposits
Mp/ep	Maiella pumice/trachyte pumice
Mt	Magaret trachyte, unwelded and welded pyroclastics
N	Ndabibi comedite lava flows, domes and pyroclastics
Op	Olkaria comendite, pyroclastics (include pre-lpk lacustrine sediments, reworked pyroclastics in ol Njorowa gorge
Or	Olkaria comendite, lava flows and domes (includeNjorowa pantellerite lava and welded pyroclastics
Ot	Olkaria trachyte, lava flows
P	Ndabibi pantellerite lava flows
Tk	Kinangop tuff (eastern rift margin)
Tkm	Mau tuff (western rift valley)
Tl	Limuru trachyte
Tlb	Karati and ol mogogo basalt
Tlg	Gilgil trachyte

Table 2.2 Description of legend of the geological map of the Naivasha basin

2.4.2. Structure

The Kenyan rift valley volcanics were erupted nearly continuously from Early Miocene to Holocene times. The volcanic centres are structurally controlled and most of the flows are erupted through fault zones.

The structure of the area comprises faulting on the flanks and in the floor of the rift valley and slight folding in the Njorwa gorge. Slight unconformities are present in the lake bed and can most clearly be seen along the Melewa river drainage. The West and south west of the kinangop plateau have been down faulted in a series of steps. The majority of the fault are short, and can be seen to die in one direction or another. Several of the fault scarps suggests slightly curved faults, with the downthrown blocks on the convex side.

2.5. Hydrogeological setting

The hydrogeology of Lake Naivasha is complex (Clarke, 1990). Hydrogeology is greatly influence by the geology, topography and climatic factors that pertain in the area. Topography in the vicinity creates two different hydro-geological environments, which affects significantly the hydrogeology. In the localized highland areas, there exist deep groundwater tables as well as steep groundwater gradient. This environment

is often associated with high rainfall values, which are sources of groundwater recharge if all conditions are fulfilled.

Groundwater occurrence is greatly determined by the geological conditions as well as the available water for storage. Fresh volcanic rocks tend to be compact with no primary porosity although secondary porosity may be well developed. These rocks underlying the rift valley therefore have low permeability though there are at times considerable variations where layers with poor hydraulic conductivity may be overlain with layers of good hydraulic properties.

The main aquifer is found in sediments covering parts of the rift floor. These, aquifers usually have relatively high permeability and are often unconfined with high specific yield (Stuttard, 1995). Clark et al (1990) estimated by inventory of boreholes and envisaged that the lake sediments have permeability of 12-148m/d.Besides, aquifers are found in fractured volcanic rocks and at times along weathered contacts between different lithological units and they are often confined or semi-confined with low storage coefficient.. Permeability is generally low in aquifers but there exist some variations locally as a result of some formations

Data from existing boreholes and wells reveal complex hydrogeological conditions and depth to water varies throughout the basin but it is generally ranging from 1.3m to about 240m. Estimated hydraulic conductivity average 10md-^{1} and well yield on average is 3 l/s/m (Nabide, 2002).

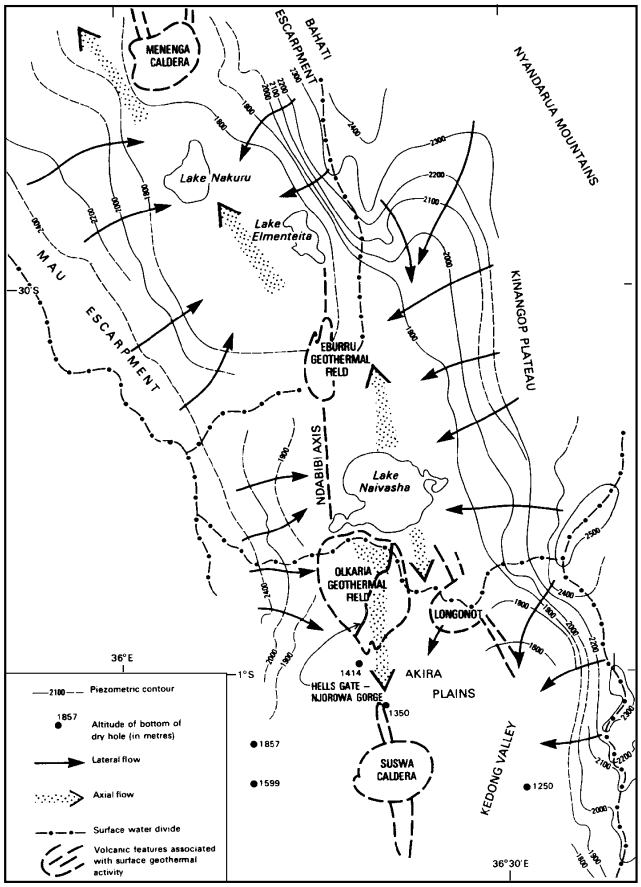


Figure 2.3 Piezometric map of Lake Naivasha & vicinities taken from (Clarke A.C.G., 1990)

3. Hydrology

The source of water availability is the basic requirement before undertaking any artificial recharge projects. Hydrological investigation was carried out to determine the source and quantity of the available water. The source of water for artificial recharge could be surplus surface water such as streams, groundwater from another aquifer and non potable waters such as waste water treatment plant effluent, contaminated surface water, storm runoff, irrigation return flow, surface (canal) supplies from large reservoir (Pyne, 1995).

3.1. Catchment characterstics

The catchments characteristics such as land use, soil type, slope, geology and geomorphology are the major factor that determines the runoff generated from the catchment. The main source of water for the artificial recharge is the runoff generated from the greenhouses. The main canal collects the runoff from the flat surrounding area occupied by greenhouses and drains to the Karati stream, which is ephemeral and part of the Naivasha basin. The total area of the catchment is 78.4ha, out of these area 53.6ha (68%) drains to the main canal and the remaining 24.8ha drains directly to the Karati stream.

The rain water draining from the greenhouses is collected by the nearby small earth canals, which finally drains to the main canal. Depending on the design of the greenhouse the rain water is collected from the roof into the canals in two ways. In the old designed greenhouses the water is collected from the roof through open plastics (Figure 3.1). In the lately designed greenhouses the water is collected through PVC pipes to the surface canal (figure 3.2). The recently implemented greenhouses are more efficient in collecting and delivering the water to the surface earth canals.

90% of the area is covered by the greenhouses and the remaining area is grass lands, narrow earth roads in between the green houses and buildings. The geomorphology of the area is very gentle to flat with small trachyte ridge on upper catchment. The canals are constructed from earth material of clayey silt and silty clay texture. The length of the main canal is about one km and its estimated bed slope is in the order of 1:1000. It is almost straight with dimension of 3m width and 5m depth. It has rectangular cross section. The left and right banks are vertical which are covered by silty clay, sandy silt and gravely sand.

Figure 3.1 Old design of Greenhouses

Figure 3.2 Recently implemented Greenhouses

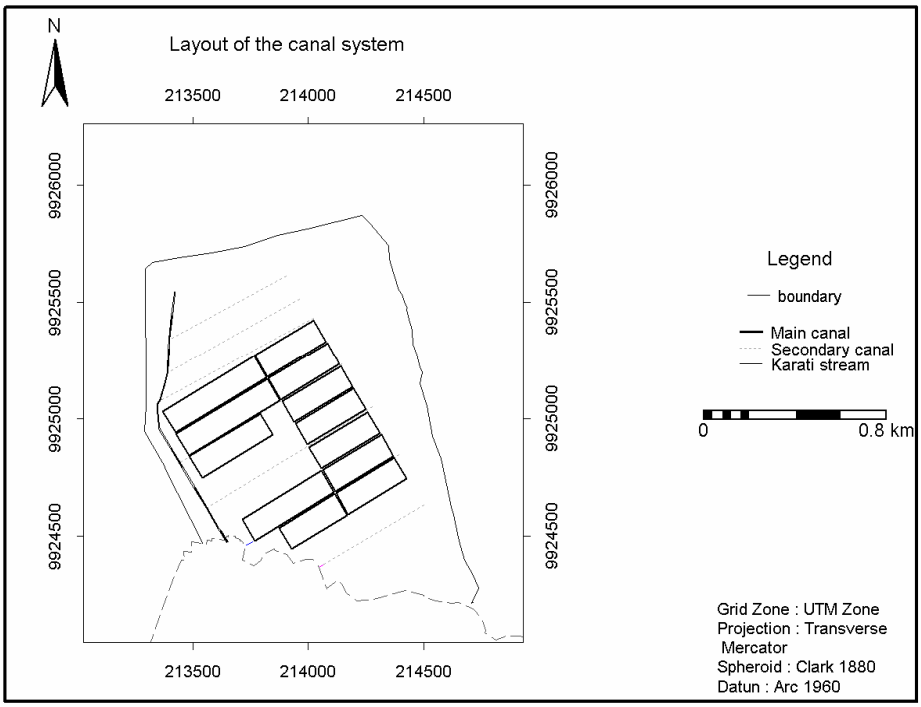


Figure 3.3 Lay out of the canal system

3.2. Rainfall analysis

Rainfall data is necessary to compute the runoff from the catchment area. For this purpose one rainfall station is selected from the Naivaha data base according to the distance, spatial distribution and data availability. Naivasha Division Office (D.O) station is situated on the Naivasha town to south direction at a distance of 4km and at an altitude of 1900.4m. The station has daily data for the duration 1957 to 2003 (Figure 3.4).

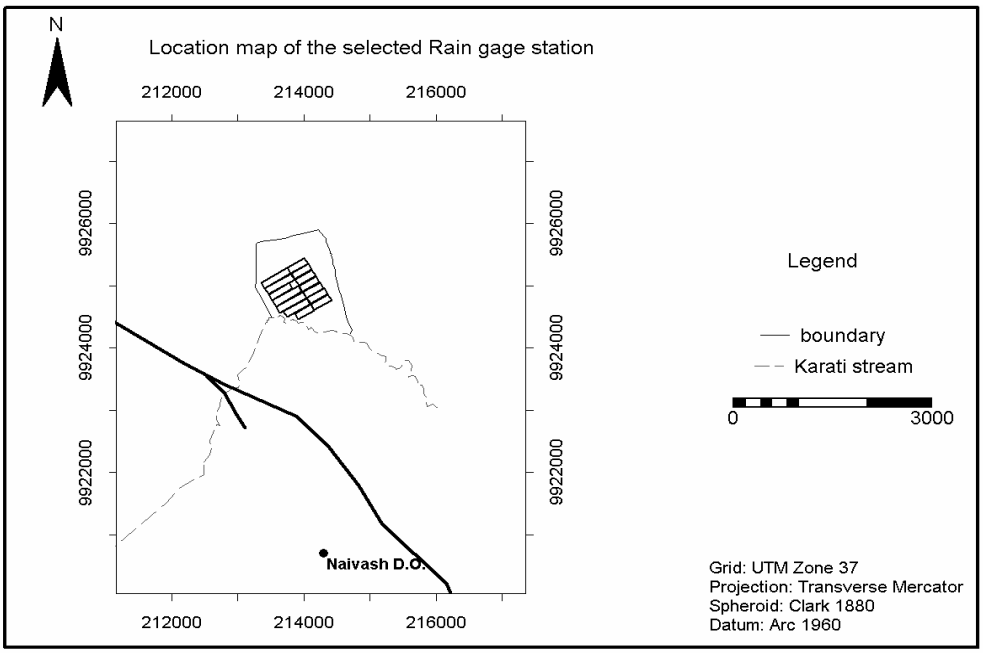


Figure 3.4 Location map of the selected rain gage station

The rainfall patterns was analysed from the Naivasha District Office (D.O) rain gage station. The area has two rainy seasons per year, the "long rains" occurring in April and May and part of June; sometimes these rains begin in March, the "short rains" in December and January, occasionally beginning in November. The general pattern of rainfall can be gauged from the graph of long-term average rainfall for the stations around the study area given in Figure 3.5.

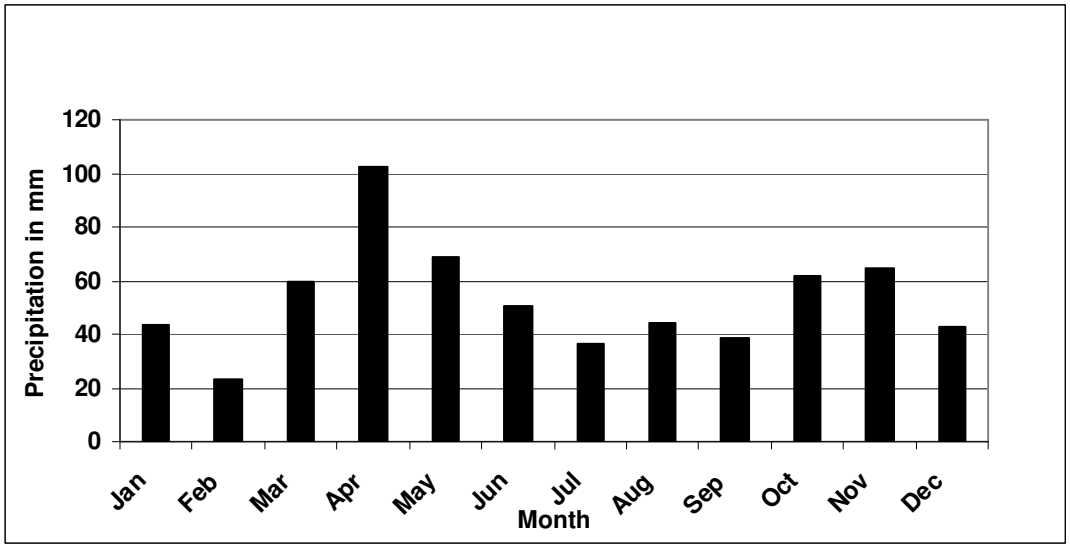


Figure 3.5 Mean monthly rainfall of Naivasha D.O (1981-2003)

Daily rainfall

The daily average and maximum precipitation was analysed as a main parameter regulating runoff generation. The values of the maximum daily rainfall of Naivasha show a cyclic pattern, varying within a defined umbra range between 20 mm and 60-70 mmday^{-1}. From the graph figure3.6 there is apparently no important change of regime, with a random variation around a mean value of 40-45 mmday^{-1}.

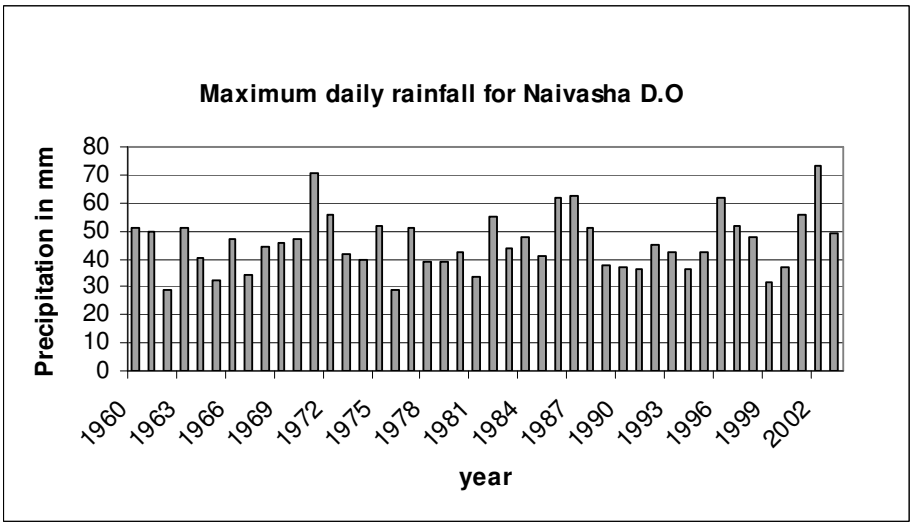


Figure 3.6 Maximum daily rainfall for Naivasha D.O

3.3. Runoff Calculation

Runoff is the portion of the precipitation that flow overland surfaces towards larger bodies of water. The runoff generated from the greenhouses is estimated using the rational empirical formula.

3.3.1. Runoff estimation using empirical formula

The rational formula represents a simple way of assessing the runoff of a watershed. It considers the entire basin area as a single unit (lumped model), estimates the flow at the most downstream point and makes the assumption that the rainfall is uniformly distributed over the drainage area and the runoff coefficient (C) is constant during the rain storm.

$$Q = C \times P \times A \qquad \textbf{3.1}$$

Where:

C = the Runoff Coefficient
P = daily rainfall ($mday^{-1}$)
A = the Area of the contributing catchment (m^2).
Q = Runoff rate (m^3day^{-1})

3.3.1.1. Runoff coefficient

Runoff coefficient *C* is defined as the ratio of the peak runoff rate per unit area to rainfall intensity and is dimensionless (Dingman, 2002). The runoff coefficients from the table (Viessman et al., 1989) suggested that for 5 to 10 years frequency design the minimum and maximum range for roofs and concrete streets land covers ranges from 0.75 to 0.95.The adopted values are similar to the findings of two days rainfall-runoff. The annual computed inflow volume distribution for the 31 years daily rainfall is shown in Figure 3.7.

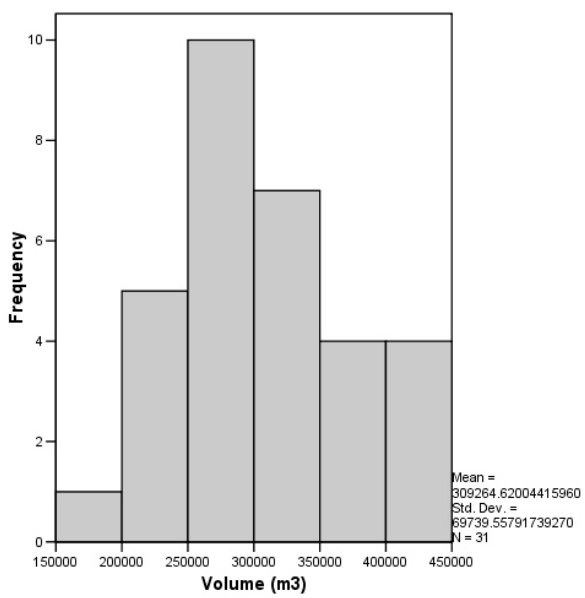


Figure 3.7 Inflow volume distribution for 31 year duration

3.3.2. Rainfall and Runoff field measurement

Rainfall and Runoff was measured during the field work in September 2005. The aim was to observe the flow during the event flow and to estimate the runoff coefficient.

●Rainfall

Rainfall was measured using tipping bucket gage. The rain gauges were installed in three places: Kenya Wildlife (KWS), Panda flower farm and Maridadi flower farms. Two days rainfall data was recorded in KWS, the other two stations were not record due technical problem. The KWS station is found around 5km to west direction, in Naivasha town. The daily rainfall record was 3 and 24mm for 4 and 5/10/2005 respectively.

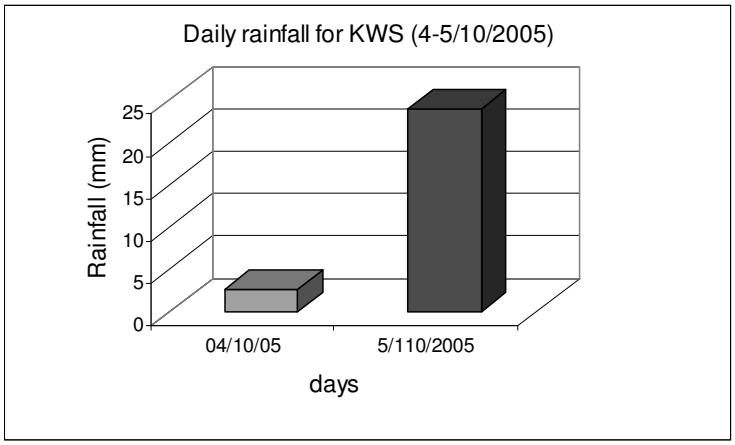


Figure 3.8 Daily rainfall for KWS

3.3.2.1. Runoff measurement

A weir was constructed on the outlet of main canal, before it joins to the karati stream to measure the flow head and calculate the discharge. The width of the main canal is about 3m with rectangular cross section. The weir is rectangular, 0.4m long and 0.48m high. It was constructed from masonry with 0.2m cut-off to minimize seepage beneath the foundation and to make it stable.

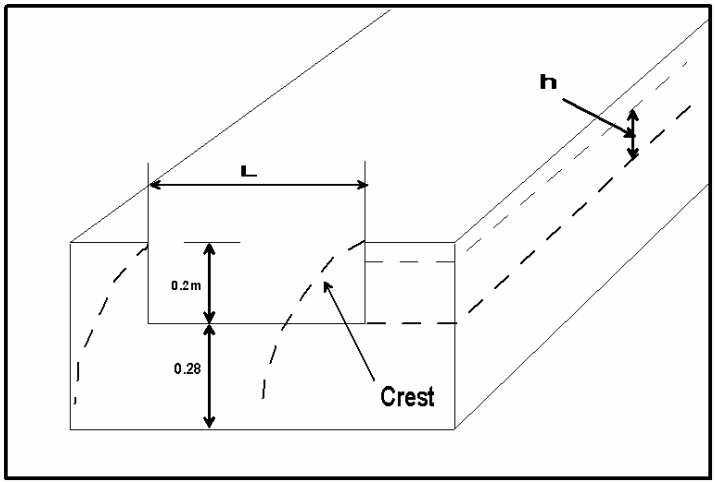


Figure 3.9 Sketch of the rectangular weir

The relation between the head and discharge of the weir varies according to the shape of the weir notch. Different equations are used with different weirs. One of the most common equations used with sharp-crested, rectangular weirs is known as the Francis equation (Driscoll, 1987). Francis equation for a weir with out end contraction is given as:

$$Q = 0.31 \times L \times h \times \sqrt{h} \qquad \textbf{3.2}$$

Where:

Q is the discharge rate in m^3sec^{-1}

L is the length of the crust in (m)

H is the difference in elevation between the crest of the weir and surface of the flowing stream (m)

Measurement

The measurement was carried by installing HOBO u20 water level logger. The logger is designed to measure water level in wells, river streams, lake and estuaries. It has an accuracy of ±2.1cm. The logger was calibrated to measure every 5minutes and it records absolute pressure which is later converted to water level reading by the software. The absolute pressure includes atmospheric pressure and water head.

Hydrographs

The inflow volume was calculated from the hydrograph. The calculation considered the overflow and the flow through the rectangular weir. The flow is measured every 5 minutes and hydrograph is drawn for the two days runoff. The total volume for the 04 and 05/05/2005 is about 1438 m^3 and 11349 m^3 respectively.

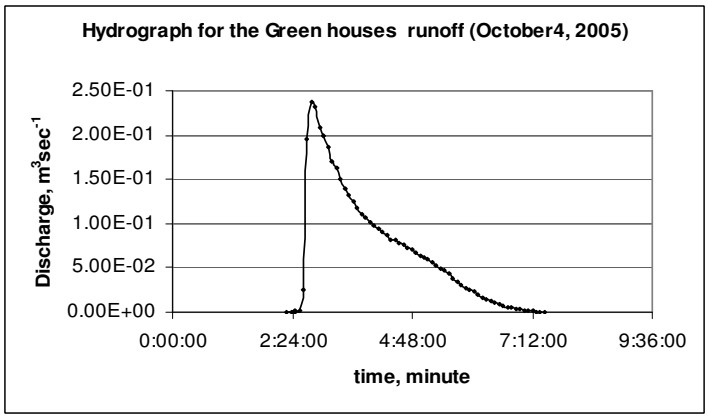


Figure 3.10 Hydrograph for the main canal (4/10/2005)

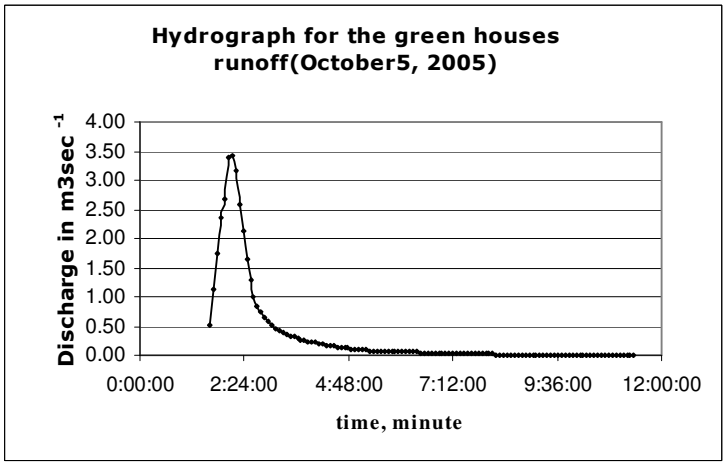


Figure 3.11 Hydrograph for the main canal (5/10/2005)

Runoff coefficient estimation from the field measurement

Even though the runoff coefficient estimated from the field can not be used directly in the calculation due to short period observation but it helps to support or strengthen the runoff coefficient adapted from literature.

Rainfall (mm)	Calculated, Q m^3day^{-1}	measured,Q m^3day^{-1}	Runoff coefficient,C_R
2.99	1603.45	1438	0.90
23.99	12865.17	11349	0.88

Table 3.1 Runoff coefficient estimated from field observation

Figure 3.12 the weir flooding during the field work

4. Data analysis

This chapter include collection and analysis of test pits and test borehole logs, pump tests, and hydraulic conductivity measurements. Test-holes were drilled and logged to collect core samples for grain size analysis and it aids in evaluating potential recharge rates through the unsaturated sediments. Infiltration tests and injection tests were conducted at the selected test borehole and test pits.

4.1. Subsurafce investigation

Characterization of the soils at and below the infiltrative surface, in the vadose zone, and below the water table is necessary to evaluate groundwater mounding and lateral flow (Alan, 2002). The study of subsurface layers and their extent is important in determining the path of recharged water, the possibility for perched water on the impermeable or semi-permeable layer and the lay out of recharge basins or recharge wells. The subsurface study was carried with the test pits, augur and drilling test boreholes. The test pits and test boreholes were located based on the practical and, hydro geological observation of the study area.

4.1.1. Test pits and Augers

The test pits are 2m by 2m with a depth of 5m. In some of the test pits the depth was extended using auger up to 11m depth.

The top layer is brownish clayey, which is compacted, dry and stiff. The thickness ranges from 1 to 2m. It is underlain by the whitish, lake sediment diatomaceous clayey silt. It is soft with thickness ranges from 0.6 to 1m. The next layer is dark brownish silty clay which is very stiff and compacted. The thickness increases from the upper reach of the dome shaped trachyte ridge towards the flat well field and karati stream. At the bottom there is a greenish decomposed volcanic ash and pumice fragments. It is dominantly medium to coarse sand composed of rounded to sub rounded pumice grains with silt. Rounded and well graded river channel deposits of sand and gravel has been observed as interbedded with thickness ranges from 0.10 to 0.40m. An auger hole was extended on the test pits to observe the continuity of the sediments. The samples obtained were disturbed as well as crushed into powder in which we are unable to describe the in situ property of the soil. The description of the test pits and fence diagram are shown in Appendix 2.

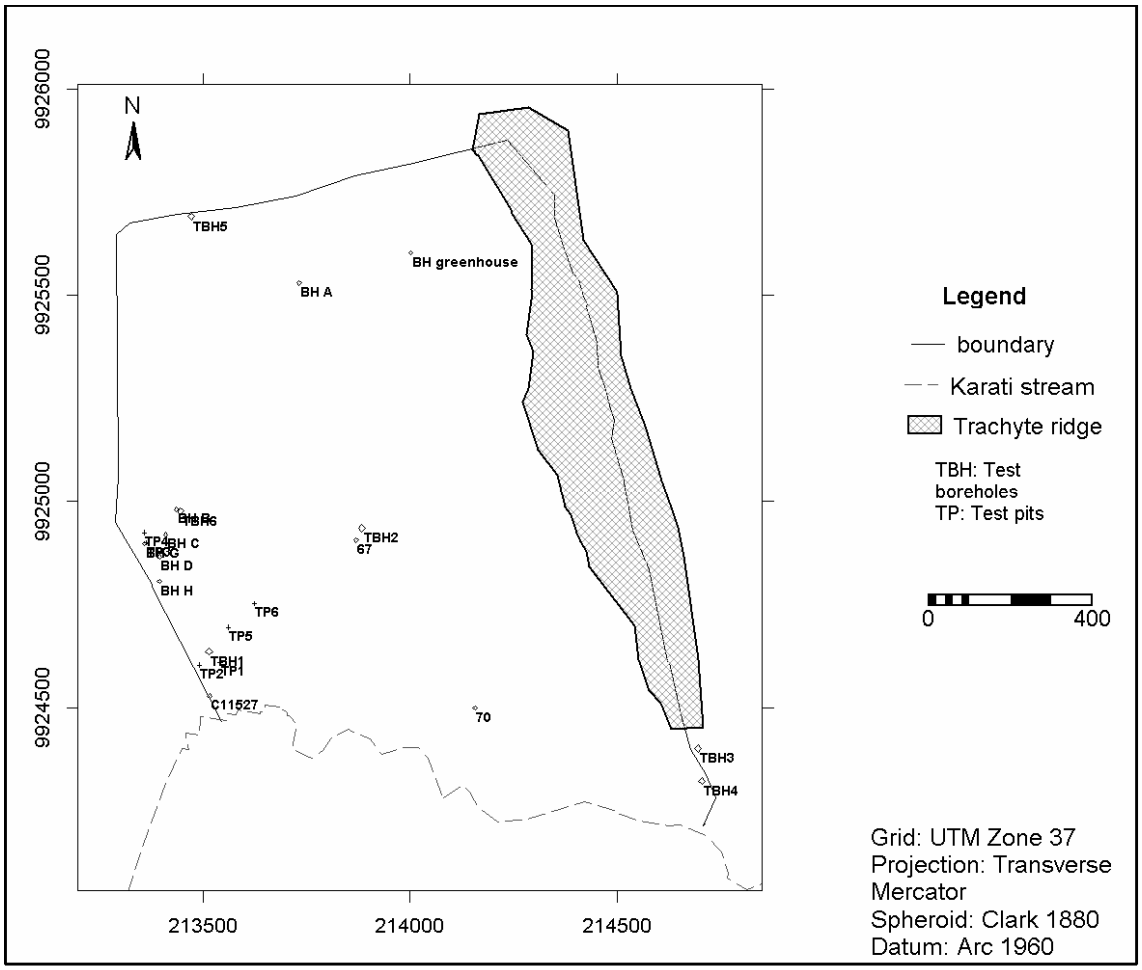


Figure 4.1 Location map of Test boreholes and Test pits

4.1.2. Drilling test boreholes

Before drilling a new well for the intended purpose it is common practice to put down a test hole (Todd, 1959). The purpose of the test hole is to determine depth to describe physical character and thickness of the unsaturated zone and aquifers materials.

The drilling method is cable tool with a standard well drilling rig, percussion tools, and a bailer. The drilling is accomplished by regular lifting and dropping of a string of tools. On the lower end, a bit with a relatively sharp chisel edge breaks the formation by impact. From top to the bottom, string of tool consists of a swivel socket, a set of jars, a drill stem and drilling bit. Drill cuttings are removed from the well by a bailer. The method is less effective when it encounters the unconsolidated fine sand because the loose material slumps and caves around the bit (Walton, 1970). The samples are provided every one meter but it is difficult to examine due to the fact it is mixed up when it bails. Accurate samples were obtained by pulling the drilling stem and using a core sampler at the bottom (http://www.eijkelkamp.nl/).

Figure 4.2 Core samples taken from TBH6 at every 5m

4.1.2.1. Geological logs

Six test boreholes have been drilled to study the subsurface geology of the unsaturated zone. A geologic log was constructed from the samples and examination of well cuttings collected at five meter interval during drilling the test boreholes. The sampling depth interval was determined depending on the geology of the area, time and machine capability. The logs furnish a description of the geologic character and thickness of each stratum encountered as a function of depth in meter. (Figure 4.3) shows description for the TBH1 from the disturbed samples where as TBH2 was logged from the core samples. The remaining logs are attached as appendix 2.

The top layer is covered with the lake sediments of clayey silt and silty clay. It is stiff and dry. It is underlain by the light brownish, poorly graded, fine-grained silty sand. The next layer is dark greyish sandy silt which is stiff and low plastic. The third layer is light brownish sandy silt which may be the result of weathered volcanic ashes and tuff. And it has few grains of volcanic glasses and pumice grains. The forth layer is decomposed silty sand with poorly graded, rounded to sub rounded and very fine sand. It contains few quartz and pumice grains. The bottom layer ranges from 25 to 30m depth with alluvial deposit of gravely sand silt. The sand is fine to medium grained, poorly graded and rounded. Volcanic glasses, pumice grains, very few quartz and elongated fibrous crystals (probably gypsum) have been observed during microscopic examination of the soil samples.

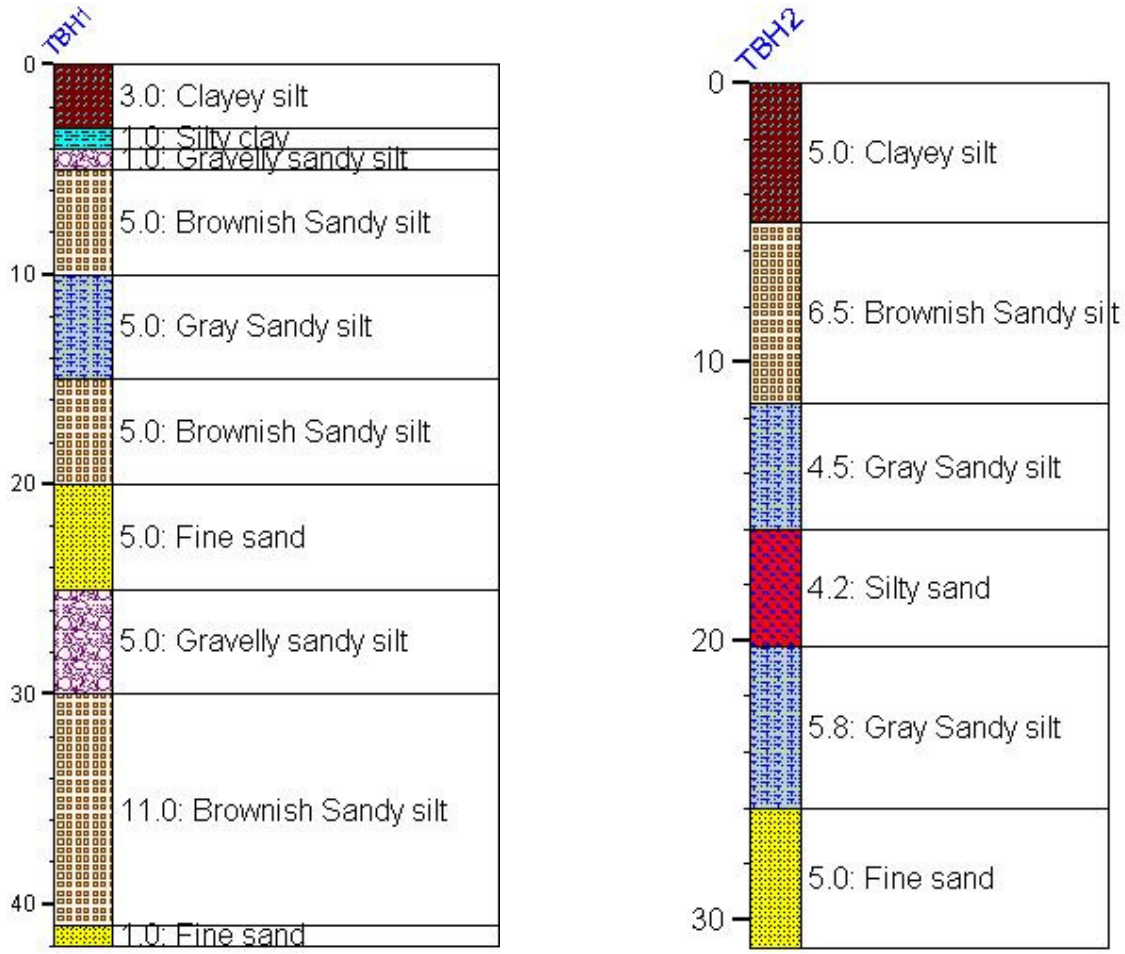


Figure 4.3 Geological logs of the Test boreholes from field observation

4.2. Soil sample analysis

Soil samples have been collected from the test boreholes and test pits. Six Samples were collected from the test borehole2 (TBH2) in every five meter. The purpose is to analyse the grain size and as an alternative method to estimate the hydraulic conductivity, specific yield and porosity.

4.2.1. Laboratory test

Particle size analysis is the separation of the mineral part of the soil in to various size fractions and determination of the proportion of these fractions. The analysis comprises all grain sizes from clay to gravel. The cementing materials usually of secondary origin such as organic matter and calcium carbonate were removed by complete dispersion. All pre-treatment were considered during the analysis.

A representative samples, about 50 to 100 grams, was taken in to the laboratory. After shaking the samples with a dispersing agent, sand was separated from clay and silt with 50μm sieve. The sand was fractionated in to very fine; fine, medium, coarse and very coarse sand by dry sieving. The clay and silt fractions were determined by the pipette method. The particle size analysis was done in according to FAO/ISRIC procedures (van Reeuwijk, 1993) and the particle size limit is attached in appendix 3.3

4.2.2. Soil classification

The relative proportion of soil particles in a particular soil determines its soil texture, which is based on the percentage of sand, silt and clay within the soil sample (Table 4.1). The soil samples were classified according to the USDA soil classification chart. The distribution of sediments at TBH2 shows that the clay proportion decreases and the sand proportion increases with depth. This might indicate that the top clay and silt are results from the lacustrine and reworked sediments and with depth the grain size increases as a result of the decomposed and weathered product of the volcanic ashes, tuff and pumices. The distribution of sediments in the test pit which indicates decreasing the percentage of silt and increasing of sand with depth. Similarly the top silty clay and clayey silt are lacustrine deposits and the silty sand is decomposed fragments of pumice grains and volcanic ashes. The figures are attached in appendix 3.4 to show the distribution of sediments.

Sample depth,m	%Clay,<0.002	%of fine Silt,0.02-0.02mm	%of medium to coarse Silt,0.02-0.05mm	%very fine sand,0.05-0.125mm	%of fine sand,0.125-0.25mm	%of medium sand,0.25-0.5mm	%of coarse sand,0.5-2mm	%of very coarse sand, >2mm	USDA soil classification
2	25.26	86.00	90.90	96.55	98.45	99.79	100.14	100.14	Sandy loam
3	45.67	92.85	101.80	104.75	106.82	107.52	107.67	107.67	Silty clay loam
4	13.17	24.86	28.68	39.48	55.68	67.13	76.13	97.73	Sandy loam
10	18.65	59.16	68.70	86.37	92.73	94.86	95.75	97.53	Loam
15	11.48	39.94	58.34	79.15	81.42	84.24	92.28	100.95	Sandy loam
20	11.46	40.52	51.51	5.6375	4.375	5.005	7.618	98.09	Sandy loam
25	10.73	66.15	84.49	95.40	96.40	97.30	98.39	99.49	Silty loam
30	2.08	8.45	8.45	57.14	92.81	98.53	99.49	100.28	sand
35	1.40	7.35	15.16	78.18	97.19	97.99	98.10	98.10	Loamy sand

Table 4.1 Grain size analysis and soil classification for the samples TBH2

4.2.2.1. Estimation of hydraulic properties

The hydraulic properties of an aquifer such as porosity, specific yield and hydraulic conductivity are estimated from the texture analysis of the soil samples. The hydraulic conductivity of unconsolidated sediments has a wide range(Fetter, 2001) .

Sample depth m	USDA Soil Classification	Field classification	Porosity %	Hydraulic conductivity $mday^{-1}$	Specific yield %	Remarks
2	Sandy loam	Clayey silt	33-60	$8.64x10^{-4}$-$8.64x10^{-2}$	12	
3	Silty clay loam	Silty clay	33-60	$8.64x10^{-7}$-$8.64x10^{-4}$	4	
4	Sandy loam	Silty sand	25-50	$8.64x10^{-3}$-$8.64x10^{-1}$	21	
10	Loam	Silty sand	25-50	$8.64x10^{-3}$-$8.64x10^{-1}$	21	
15	Sandy loam	Sandy silt	35-50	$8.64x10^{-4}$-$8.64x10^{-2}$	20	
20	Sandy loam	Silty sand	25-50	$8.64x10^{-3}$-$8.64x10^{-1}$	21	
25	Silty loam	Sandy silt	35-50	$8.64x10^{-4}$-$8.64x10^{-2}$	20	
30	sand	Silty sand	25-50	$8.64x10^{-3}$-$8.64x10^{-1}$	21	49% is very fine sand
35	Loamy sand	Silty sand	25-50	$8.64x10^{-3}$-$8.64x10^{-1}$	21	

Table 4.2 Estimation of hydraulic properties of the soil samples (Fetter, 2001)

4.2.3. Grading curves

A grain size distribution graph was prepared by plotting the cumulative percent retained by weight on the vertical scale against the logarithmic of the sieve on the horizontal scale. The effective grain size, d_{10}, is the size corresponding to the 10% line on the grain size curve. The slope is described by the uniformity coefficient. Uniformity coefficient, C_u is the ratio of the grain sizes that 60 percent finer by weight, d_{60}, to the grain size that is 10% finer by weight, d_{10} (Walton, 1970). It gives the idea of the grading or particle size distribution in the sample. If the ratio is less than 4 it is considered to be well sorted, and if it is greater than 6 it is considered to be poorly sorted (Fetter, 2001). The grading curves are attached as appendix 3.5.

$$C_u = \frac{D_{60}}{D_{10}} \qquad \textbf{4.1}$$

The uniformity coefficients (C_u) for the sample taken from depth 10 to 35m are all greater than 6, which imply that the grains are poorly sorted. On the contrary the samples from depth of 2 to 4m are less than 4 (Table 4.3).

Sample depth,m	D_{10}	D_{60}	C_u
2	0.002	0.0075	3.75
3	0.002	0.003	1.5
4	0.002	0.25	125
10	0.002	0.02	10
15	0.002	0.06	30
20	0.002	0.11	55
25	0.002	0.016	8
30	0.002	1.1	550
35	0.002	1.2	600

Table 4.3 Uniformity coefficients for the samples

4.3. Hydraulic conductivity measurements

Hydraulic conductivity of the soil and aquifer is the most significant factor to evaluate artificial recharge infiltration basin. The important factors controlling soil hydraulic conductivity are texture, structure, bulk-density (degree of compaction), percent coarse fragments, clay mineralogy, and organic content. The test was carried every five meter interval considering the heterogeneity and variability of the geological formations with depth.

4.3.1. Inverse auger hole method

A drill hole was drilled with average diameter of 40cm at a certain depth above water table and filled with water. Infiltration test has been carried every 5m depth. The hole was saturated before start measurement. The rate of fall of water was measured with electrical dipper. The following formula was used to compute the hydraulic conductivity (Herschy, 1995) and the results obtained from inverse auger-hole method were plotted on the semi-logarithmic paper and the hydraulic conductivity was calculated in $mday^{-1}$ (appendix 4).

$$K = 1.15r \frac{\log\left(h_o + \frac{1}{2}r\right) - \log\left(h_t + \frac{1}{2}r\right)}{t - t_o} \qquad \textbf{4.2}$$

Where:

K is the hydraulic conductivity in ($cmday^{-1}$)

T is time since the start of measuring (sec)

h_t the height of the water column in the hole at the time t (cm)

h_o the height of the water column at t =0

Hydraulic conductivity test was conducted in the test boreholes, TBH1, TBH2, TBH4, TBH5, and TBH6 with 5m head for all measurements. The depth graphs and their respective calculated hydraulic conductivity for all the inverse auger-hole are presented for each depth as follows: The field data measurements and basic principles of inverse auger hole are attached in appendix 4.

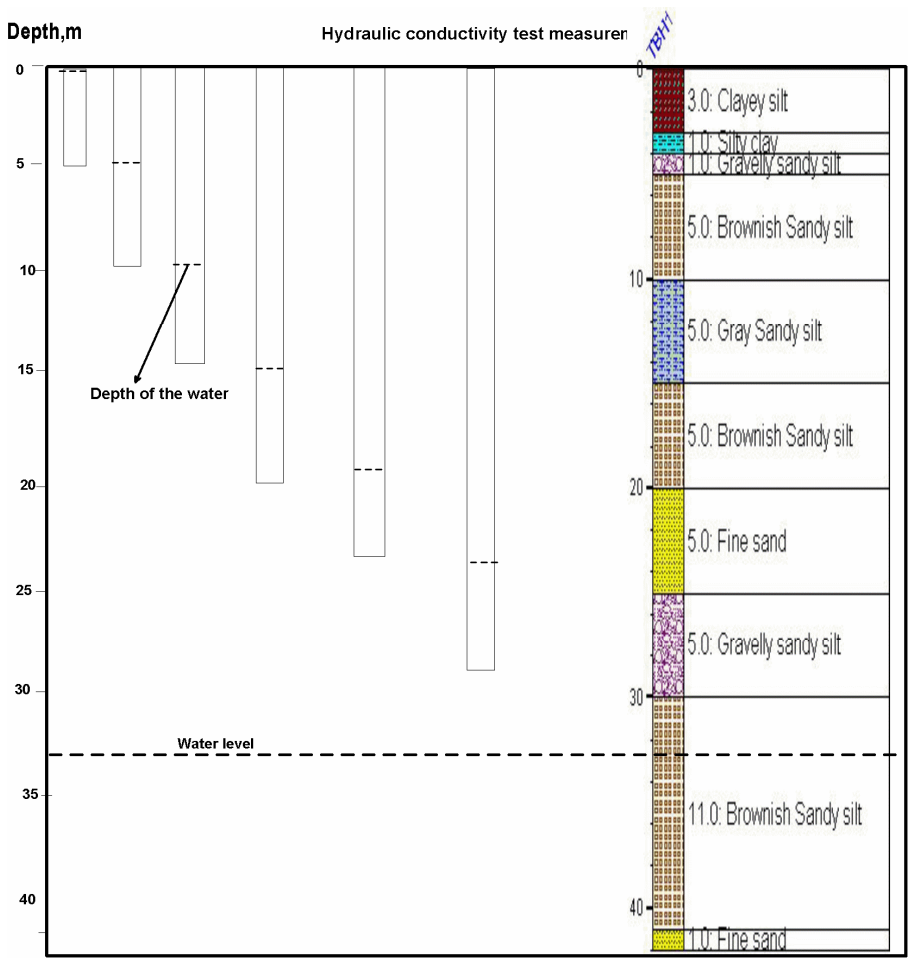


Figure 4.4 Infiltration test at 5m interval and the corresponding test boreholes log from disturbed samples at TBH1

TBH NO	Depth (m)	K in $mday^{-1}$	Lithology
TBH1	5	0.2	Silty Sand with decomposed pumice
	10	0.02	Sandy silt ,
	15	0.04	Silty sand
	20	0.02	Very fine sandy silt
	25	0.16	Sandy silt
	30	0.02	Silty sand
TBH2	10	0.12	Sandy silt with little rock
	15	0.04	Sandy silt
	20	0.12	Silty sand
	25	0.07	Silty sand with pumice
TBH4	11	0.1	Sandy silt with rock
TBH5	34.1	0.2	Sandy silt
TBH6	21.5	0.12	Sandy silt

Table 4.4 Summary of the hydraulic conductivity measurements at the test boreholes

The infiltration test results hydraulic conductivity in the range of 0.02 to 0.2$mday^{-1}$. The maximum infiltration rate was recorded in the decomposed greenish silty sand composed of pumice fragments which is comparable with field observation and texture analysis. The minimum is observed in very fine sandy silt sediments. Even though the lithology shows vertical heterogeneity there is no significant difference in hydraulic conductivity measurements. The possible explanation for the similarity of test result:

- Clogging of the formation due the mud cake resulted from the silt and very fine sediments
- Clogging of the test boreholes due to collapsing as the test borehole increasing depth

4.4. Pumping and Injection test analysis

4.4.1. Aquifer test

A lot of methods are there to obtain hydraulic information from aquifers, but perhaps the most common and best is the pumping test or aquifer test. The principle of a pumping test is that if we pump water well and measure the discharge of the well and the draw down in the well and in piezometers at known distance from the well, we can substitute these measurements in to an appropriate well-flow equations and calculate the hydraulic characteristics of the aquifer (Kruseman and de Ridder, 1983). The analysis of pumping data is closely connected with having an understanding of the geologic setting and a knowledge of the well-completion details (Weight and Sonderegger, 2001).

Aquifer test analysis methods

When performing a pumping test the presence of at least one observation well is desirable. In this case the tested borehole had no observation well. Therefore, analysis methods developed specifically to deal with this situation had to be used. In all cases one of the methods used for the pumping data analysis was Jacob's straight-line method, which can be applied to a single-well constant discharge test to estimate the aquifer transmissivity for confined aquifer. These tests are included in the package AQUITEST 3.5. The data was

interpreted using the confined equation due to the fact that for short duration test the observed drawdown in the first few minute is similar to the Theis type curve for confined aquifers since the pore water is released instantaneously from the elastic storage due to the compression of the aquifer matrix and expansion of entrapped air (Kruseman and de Ridder, 1983).

Jacob straight-line method using the equation:

$$KD = \frac{2.30Q}{4\pi\Delta s} \qquad 4.3$$

Where:

KD is the transmissivity (m^2day^{-1})
Q is discharge (m^3day^{-1})
ΔS is the drawdown per log cycle of distance (m).

Pumping test at Panda flower farm

Pumping test was conducted during the field work in BH_G. The well to be tested was stopped pumping for about a week and the static water level was measured 40.1m during the test. The borehole depth is 60m and the test duration was 72 minutes, which results a drawdown of about 6.2m. The discharge was measured on site by noting the time taken to fill a container of known volume, which is about $144m^3hr^{-1}$. The water was designed to flow in to the near by storage during the pumping test.

The drawdown was measured in the pumped well at frequent short intervals at the first since the water level drops fast then gradually decreased as pumping continuous. The discharge rates were constant. The time–drawdown data, analysed using Aquifer Test for windows with the Cooper Jacob's straight line method resulted in transmissivity of $816m^2day^{-1}$. The hydraulic conductivity was calculated about $51mday^{-1}$ assuming the thickness is about 16m.

Previous pumping test has been conducted in the study area at BH_A and BH_C (Hernandez, 1999) and (Kibona, 2000) respectively. Table 4.5 reveals the pumping test result of these wells.

BH Name	Method	Transmissivity (m^2day^{-1})	Remarks	Performed by
BH A	Pumping (Hantush)	1020	AQUITEST	Ramirez, 1998
BH A	Pumping (Jacob)	1150	AQUITEST	Ramirez, 1998

Table 4.5 Summary of the transmissivity

4.4.2. Injection test

A cone of recharge will be formed the water pass into the recharge well which is the reverse of cone of depression for a pumping well (Raghunath, 1982). Injection test was conducted in the open test boreholesl (uncased) and in a completed NBH7 (New Borehole7). The aim of this test was to determine the intake capacity and the hydraulic parameter of the test boreholes. The groundwater level in the injection well was raised and water levels in the well were measured for a short period of time using the electrical dipper. Cone

of recharge is the difference between the piezometric head at some time after test begins and the piezometric head at the existing at the start of the test.

Injection test at TBH1

The test was accomplished by conveying water from the existing well, BH_C located at a distance of 200 to 250m through 20cm P.V.C pipe. The rate of injection was measured on site by noting the time taken to fill a container of known volume. The test was carried at different depth interval from the top up to the bottom, top of the aquifer (Figure 4.5). The total depth of the test boreholes is 41m and water was strike at 31.4m. The formation is composed of mixtures of sand and silt. The test was performed by applying constant recharge of 80m^{3}hr-1 and looking the response of the aquifer system to the imposed recharge. The test duration was 15 minutes and it was kept constant in each depth.

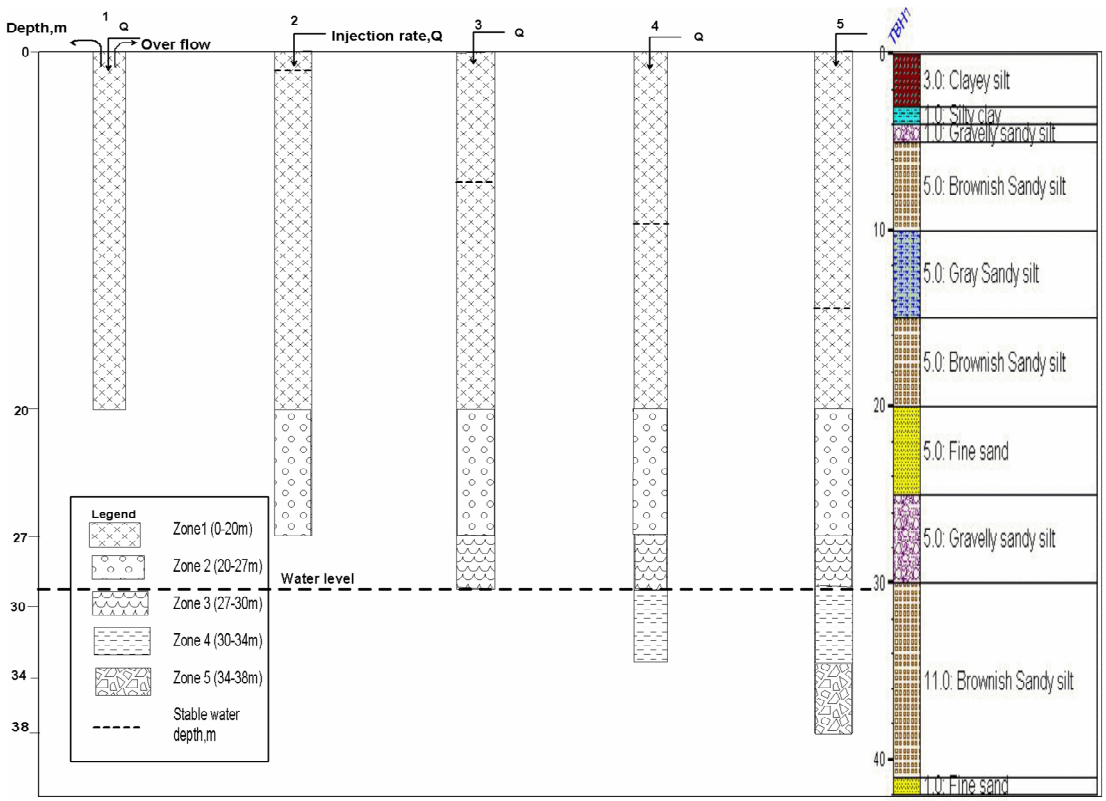


Figure 4.5 Injection test at different zones of TBH1

Interpretation of injection tests

The injection test results were analyzed in two approaches. The first approach is to plot the recharge head rise and fall with time and interpreting the curves. And the other is computing the injection rate of each depth interval assuming the linearity relationship of injection rate and average pressure. The test borehole is classified in 5 zones in according with the depth of the test accomplished as:

Zone	1	2	3	4	5
Depth (m)	0-20	20-27	27-30	30-34	34-38

Table 4.6 Injection test Zones

Injection plot curves

The first test was conducted at 20m depth, after injection of $80m^3hr^{-1}$, it over flows within 0.5 minutes. This implies that the lithology from the top to 20m depth is impervious or very low permeable. The next test was at 27m, the plot shows sharp change for the 3 minute response and become steady within very short time, which may be due to significant differences in hydraulic conductivity, such that the major part of the lithology is low permeable but there might be are thin layers which have higher permeability within this heterogeneous zone. The water level was raised to a maximum of 1.15m from the land surface at 27m depth test. The Figure 4.6 illustrates that the injection made at 30, 34 and 38m have similar response to the injected water, very high for the first 3 minutes then gradual changes up 10 minutes; at the end it became some what steady at injection head of 21.6m, 21.85m and 17.38m respectively.

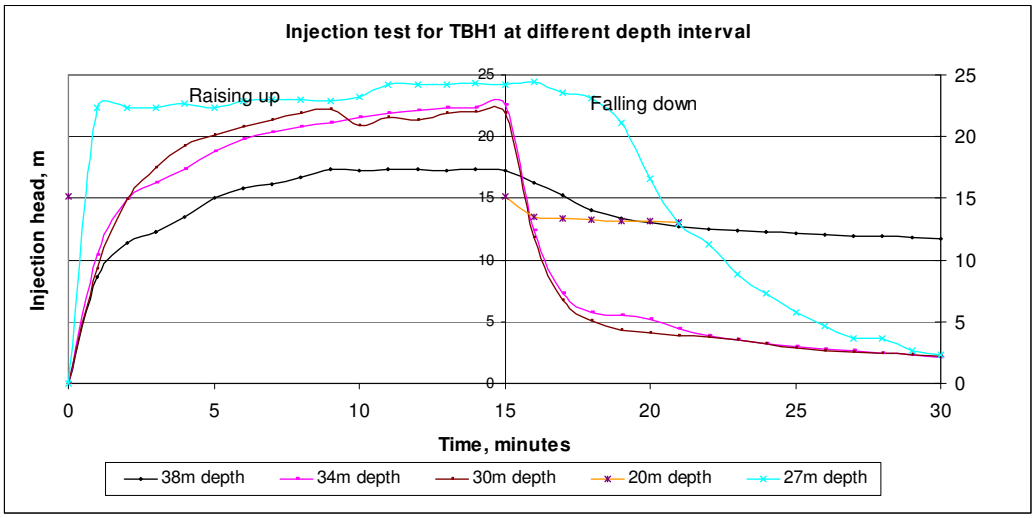


Figure 4.6 Water level changes in the injection test of TBH1

Injection rate and pressure

This method was applied to determine the infiltration potential of each zone. Assuming the linear relationship of injection rate and average pressure, the amount of water and the specific injection rate per the thickness of each zone was calculated. The injection rate was about $80m^3day^{-1}$ and it was kept constant in all tests. Based on this computation, the test result at depth 38m shows full picture of each zone. The first zone is impervious layer, from the first test. The computed amount of water of injected water in each zone is 29, 13.5, 3.3 and $34m^3hr^{-1.}$ The specific injection rate was computed by the ratio of the injection rate, average pressure head and the thickness of the formation which results in 0.52, 0.35, 0.05 and $0.42mhr^{-1}$for each zone respectively. Zone5 shows maximum intake rate of about $34m^3hr^{-1}$ and has higher infiltration rate. And the minimum is $3.2m^3hr^{-1}$ in the forth zone and it was interpreted as low permeable zone as compared to others. The summary of the result for each depth is shown in Table 4.7.

Zones	Parametrs	units	Bore hole depth,m				
			20	27	30	34	38
	Average pressure head	m					
	Sp.injection rate	mh^{-1}					
	Q	m^3hr^{-1}	Very low Impermeable zone				
Zone1	thickness	m	20	20	20	20	20
	Average pressure head	m		22	16	15	8
	Sp.injection rate	mh^{-1}		0.52	0.52	0.52	0.52
	Q	m^3hr^{-1}		80.00	58.18	54.55	29.09
Zone2	thickness	m		7	7.00	7.00	7.00
	Average pressure head	m			21.00	20.00	13.00
	Sp.injection rate	mh^{-1}			0.35	0.35	0.35
	Q	m^3hr^{-1}			21.82	20.78	13.51
Zone3	thickness	m			3	3.00	3.00
	Average pressure head	m				23.5	16.50
	Sp.injection rate	mh^{-1}				0.05	0.05
	Q	m^3hr^{-1}				4.68	3.28
Zone 4	Thickness	m				4	4.00
	Average pressure head	m					20.5
	Sp.injection rate	mh^{-1}					0.42
	Q	m^3hr^{-1}					34.12
Zone 5	Thickness	m					4

Table 4.7 Infiltration rate of each zone as computed

Injection test at NBH7

The second test was conducted at the new borehole which was drilled by the panda drilling crew in September 2005. It was not well developed and pumping test was not done. Groundwater was strike at 36m and the aquifer is well penetrated. The static water level is 34.5m with depth penetration of 72m and diameter of 0.2m.The depth from 30 to 60m is screened and the remaining part is plain casing. There is no completion record of the well but as to the observation of the materials drilled from the well the aquifer looks semi confined and consists of decomposed volcanic ashes, pumice sand and silt texture. The water level was 33.5m when the test started. The water was constantly discharged at $80m^3hr^{-1}$. After 15 minutes of recharge the water level stabilizes at 3.95m; to express this in terms of cone of recharge 29.95m attained as shown in Figure 4.7.

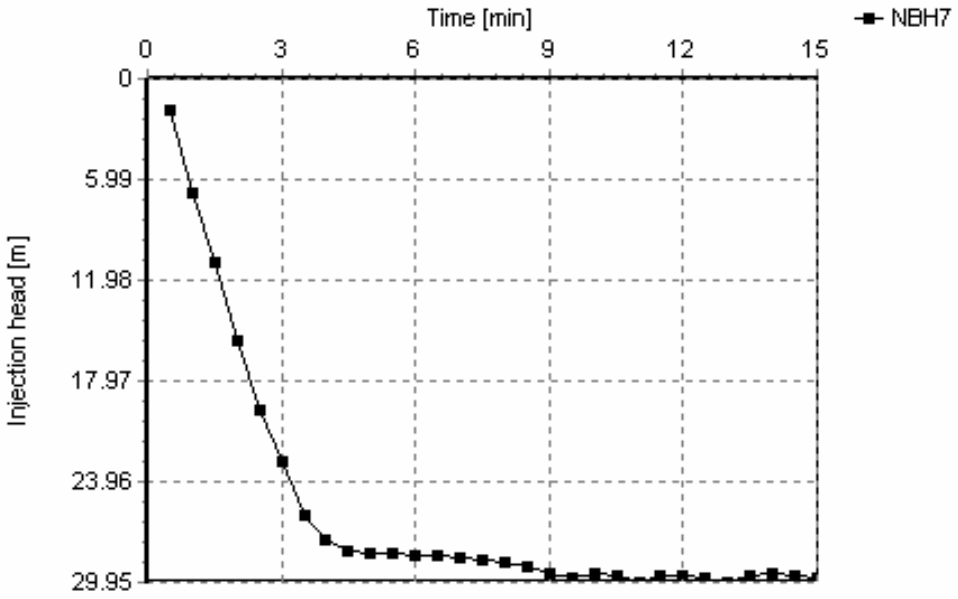


Figure 4.7 Injection head vs. time for NBH7

The output of pump test result using Jacob method found transmissivity and hydraulic conductivity is about $59m^2day^{-1}$ and $5.9mday^{-1}$ respectively. The field test data and plots are attached in appendix 6.

5. Hydrogeology

Geological and hydrogeological study plays a critical role in determining the suitability of a site for artificial recharge. A detailed geological and hydrological feature of the specific area was assessed for adequately selecting the site and the type of recharge structure. In particular, the features, parameters and data considered are: - geological boundaries; storage potential; porosity; hydraulic conductivity; transmissivity; lithology and depth of the aquifer. The sediments at and below the infiltrative surface has been characterized to evaluate the artificial recharge methods.

5.1. Aquifer characterstics

5.1.1. Hydrostratigraphy

The aquifer characteristics were studied based on the existing well records, pumping and injection tests and geophysical survey interpretation. The water level of the study area is encountered at depths of 34 to 36m below the surface. Geological cross section of the study area which is drawn based on the well logs of C11527, BH_A and geophysical data interpretation appeared in Figure 5.1. The aquifer is complex due to the sedimentation, which took place concurrently with tectonic history and associated volcanism. The hydrostratigraphic unit is composed of two aquifer system named shallow and deep aquifer. The shallow aquifer consists of unconsolidated lake sediment and reworked materials. They are composed of fine to medium grained sand, with lumps of silt, clay and gravel. The core samples of the geological log showed heterogeneous, built of alternating layers of sand, silt and gravel with thickness ranging in the order of 0.1 to 2m. This aquifer usually occurs as an unconfined and/or semi confined, but is locally confined under the clay silt. As explained by (Owor, 2000) the aquifer sediments have a thickness ranging from approximately 15m in areas of lower thickness (thins out to the scarps) to over 50m beneath the lake.

The deeper aquifer is confined and composed of reworked volcanic materials, gravely sand with silt. The material is more homogenous and course textured than the upper aquifer and contains no lumps of clay or silt. The thickness ranges from 40 to 50m.

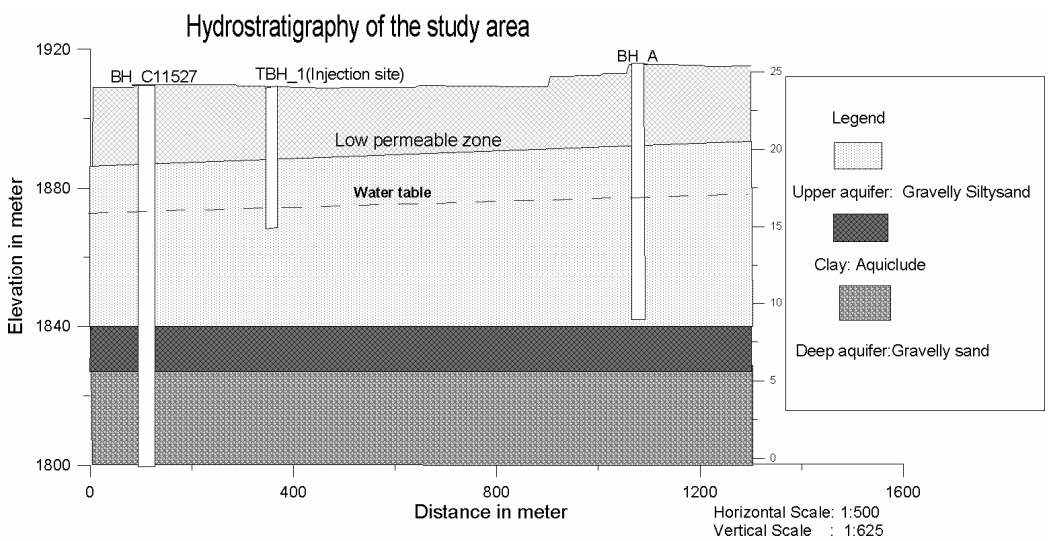


Figure 5.1 Hydrostratigraphy of from well logs and geophysical data interpretation

5.1.2. Geophysics

The purpose of applying geophysical methods for the selection of appropriate site for artificial recharge study is important in understanding the sub-surface hydro geological condition. The main objective of interpretation of geophysical data is to compliment the well logs, to correlate with hydro geological settings and to bring out a comparative picture of the subsurface manifestation of structures such as faults and fractures.

Geophysical survey was carried out by Tsiboah (2002) using Two-dimensional (2D) Resistivity Imaging and Transient Electro-Magnetic (TEM) data on three areas: Three Point (Panda flower farm), Manera, and Home-grown Figure 5.3. In all, thirteen Resistivity Imaging Survey profiles of different lengths and 137 Time-Domain EM (TEM) soundings were carried out to model the groundwater system in this area.

At that time two survey lines, which are Line 2 and 3, have been conducted in three point farms (Panda flower farm). The geological interpretation Figure 5.2 indicate the presence of a shallow material of medium to coarse sand of thickness between 15 to 20m spanning over some 300m. Towards the ends of the Karati stream it shows boulders and fractured lava of thickness of about 40m on top and sandy clay up to 80m beneath. Beyond this depth it is very conductive material which could be clay and /or saline water.

Survey Line3 runs almost perpendicular to Line2. It starts from the foot of the Trachyte ridge on the panda flower farm and runs approximately E-W across Pivot B. The results indicate the presence of a boulder trachyte layer at near surface depths (0 - 30m) from the foot of the trachyte ridge to some 300m away where a sub-vertical fault seems to have thrown the material further down. The materials beneath the boulder layer are silt and silty clay in origin. The depth of investigation by the TEM sounding in this area was much reduced due to the high conductive nature of the material below the silty layer. This could be clay or saline water. The rest of the section is made up to fine to medium sand with lenses of medium to coarse sand and pebbles as explained above on traverse Line2.

From the 2D geophysical interpretation the aquifer exists generally between depths of 20 to 80m in the study area. The top aquifer occurs between depths of 20-40m and the bottom between 50-80m.The main aquifer materials include fine sands, medium coarse sands, gravels, pebbles and fractured volcanics (Tsiboah, 2002). Laterally, the high quality and good yield portion of the aquifer occur within a radius of approximately 1km from where the Karati River turns from the NW direction to the SW (90^0 turn). The very low resistivity at depths greater than 80m has been identified as a mixture of clayey materials and saline water. A fault system has been inferred around the karati stream based on the geomorphology of the stream (Nabide, 2002). These fault systems were interpreted to be the source of recharge into the aquifer.

The correlation of drilled well logs, injection tests and geophysical survey clearly indicates that the top 20m depth is low permeable zone. The vertical and lateral extent of the unconsolidated sediments is observed from imaging profile. The upper aquifer sediments changes to coarser deposits of pebbles, cobbles and boulders towards the trachyte ridge and karati stream and at the middle it is fine to medium sand.

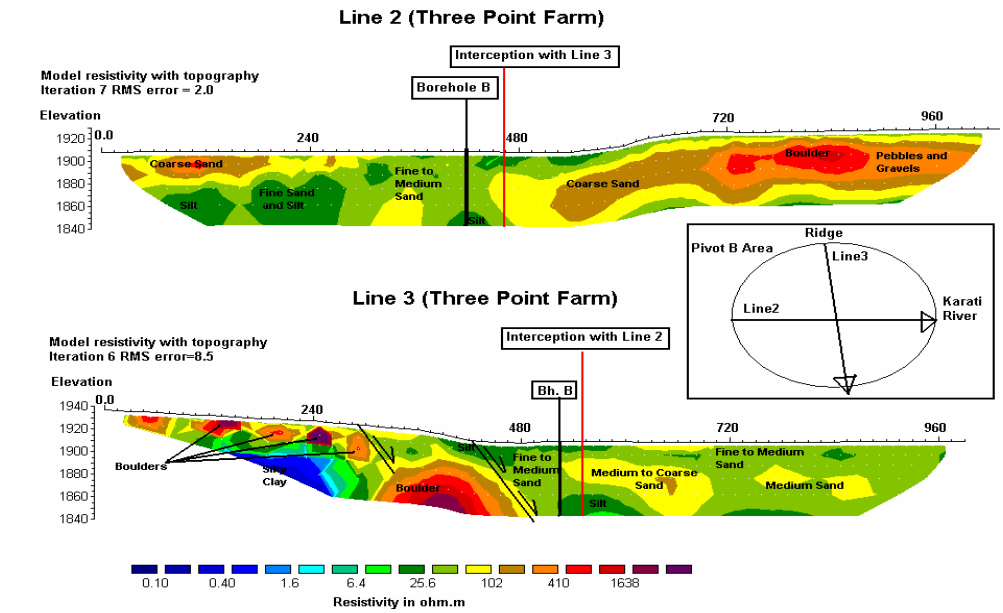


Figure 5.2 Geological interpretation of the 2_D resistivity imaging model section (Tsiboah, 2002)

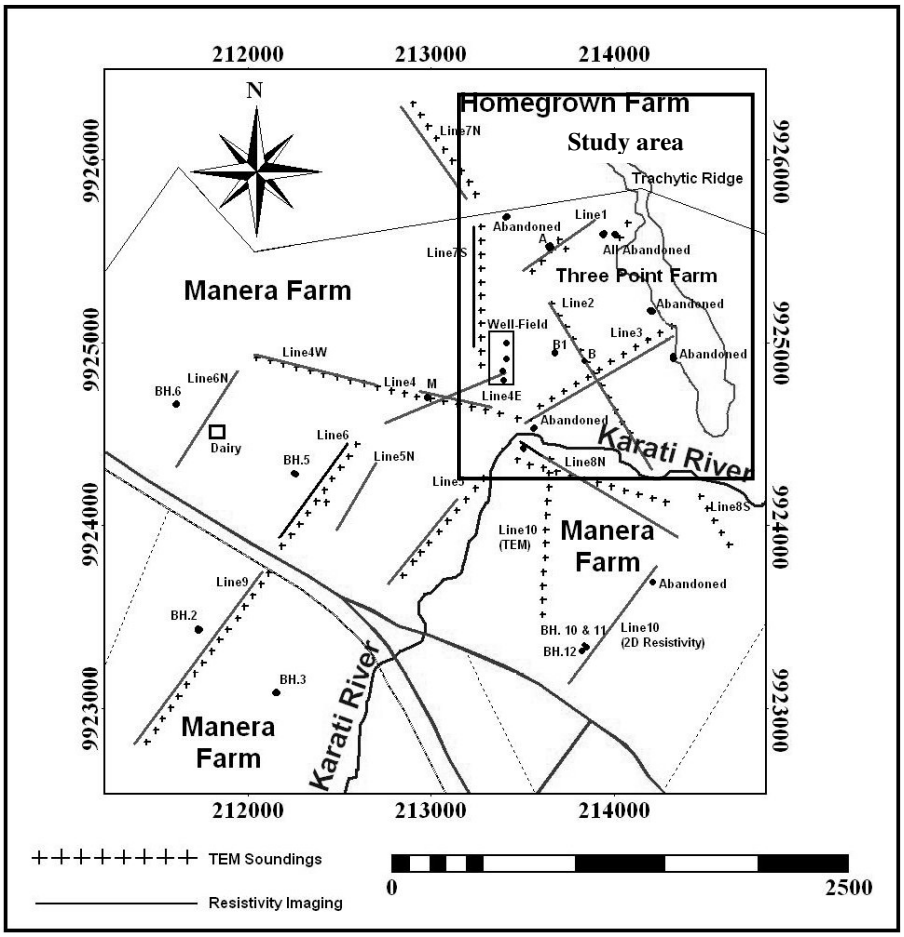


Figure 5.3 Time domain EM soundings positions and resistivity imaging locations (Tsiboah, 2002)

5.1.3. Aquifer storage

Aquifer storage potential and retention characteristics are determined largely by geological conditions at the site, such as the nature of the aquifer material, extent of the strata, storage of coefficient, and existence of fractures and faults. The storage potential of the groundwater must be adequate to accommodate the anticipated volume of recharge. Aquifers best suited for artificial recharge are those having a low value of the T/C coefficients where, T=Transmissivity and S=Storage coefficient, that is aquifers which absorb large quantities of water and do not release them quickly, these two conditions are not often encountered in nature (UN, 1975).

The hydraulic effect generated by the artificial recharge is a result of the increased head which is applied in the recharge area and the mass of the water which is introduced into the aquifer through the recharge area. These are: the piezometric effect and the volumetric effects.
The piezometric effect results in a rise of piezometric surface in the unconfined aquifer and/or a rise of the artesian pressure in the confined aquifer. The effect is related to factors which create a damping reaction. This damping effect is related to the shape the piezometric surface, to the geological and hydraulic boundaries of the aquifer and to the type and location of the recharge device. Secondly, it is related to the quotient T/C; thirdly, it is related to the artificial recharge yield and duration of the operation. The volumetric effect is related to the storage coefficient, transmissivity and boundary coefficients. Transmissivity is important for two reasons: (a) water must be transmitted from the area of recharge to points of extraction with a minimum gradient; (b) economic extraction requires large-capacity wells with small drawdown. Because specific capacity of wells is directly related to the transmissivity, a higher transmissivity is desirable.

The injection test conducted at different depth interval indicates that lateral spreading of the injected water was observed in depth range of 27 to 38m. Most of the water was infiltrated at the depth range of 38 to 40m. From the previous pumping test (Kibona, 2000) and current analysis results in a transmissivity in the range of 816 to 1050m^2day^{-1} was found. The porosity and specific yield of the formation is estimated from the laboratory core sample grain size analysis and are found to be ~ 0.3 and 0.15 respectively, for detail description refer chapter 4. From the short duration injection test result, the aquifer has good potential for the storage of the injected water. The water level of the area is 34 to 36m below the surface, which allows a sufficient storage space and hydraulic head build up of 10 to 15m above the static water with out over flow.

5.1.4. Groundwater level fluctuations

Fluctuations in the potentiometeric surface in the study area are indicative of changes in the amount of groundwater held in storage. The declined in groundwater level around the well field area creates favourable condition for the artificial recharge. Artificial recharge results in changes the groundwater level fluctuations depending on the aquifer characteristics.

Ground-water levels in the Naivasha basin, particularly in North east Naivasha are lower than they were prior to groundwater development in the basin since the early 1980.The decline in the water level of the study area probably may be related to increased withdrawals from wells for irrigation and/or lake level changes. Even though groundwater level measurement was not done during this study, the water level measurement of the existing wells from 1999 to 2004 by (Nabide, 2002) and (Yihdego, 2005) are summarized below in Table 5.1.

C_no	UTM_X	UTM_Y	Piezometric level	
			2001	2004
c11527	213518	9924527	1880.51	1874.4
BH A	213735	9925528	1884.61	1883.69
BH B	213713	9924977	1885.82	1871.13
BH C	213459	9924929	1882.31	1872.78

Table 5.1 Water level decline in the wells

Water levels decline have been observed in different part of the north east Naivasha such as in Panda flower farm, Manera and Milk oulet area. From the 1993 up to 2004 record of the well C11527, a decline of about 10m is apparent (Figure 5.4).

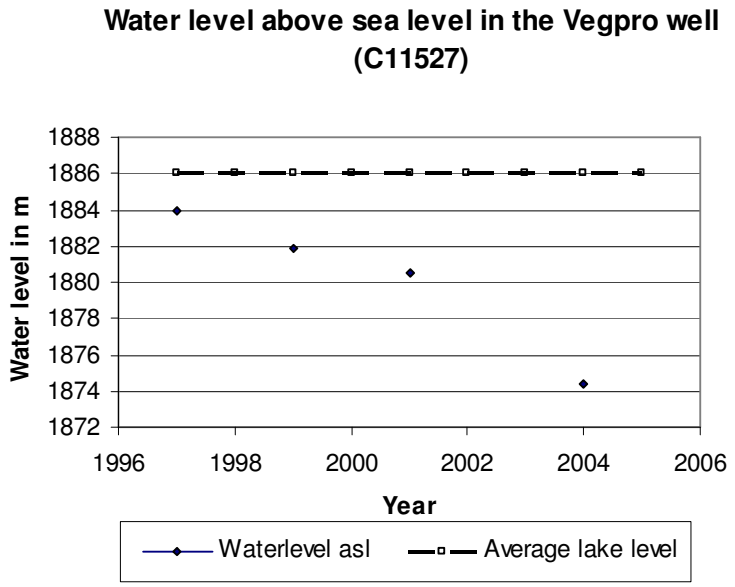


Figure 5.4 water level of C11527 well from 1997 to 2005

5.1.5. Groundwater flow

In the lake Naivasha basin, groundwater generally flows towards the lake from the Mau and Aberdare escarpments, although it is diverted locally by the presence of faults that form either barriers or conduits (McCnn, 1974). The historic contour map indicates that the regional groundwater flow direction is laterally from the eastern and western escarpments towards the Lake, and axially from Lake Naivasha northwards towards Lake Elmenteita and southwards from towards the Longonot area Figure 5.5.

Though the modified flow pattern by Owor has remained basically the same, there has been a change when details are observed. The piezometric contours indicate a development of sink on the North-eastern side of lake Naivasha around Panda flower farm and Manera Farms. There has not been a major change in the flow pattern. According to the 1980 piezometric contour map, there has been a fall in the piezometric heads in the North-western part of the study area. Consequently, the area around Three Point Farm (panda flower farm) and Manera Farm, could account for the change in the flow direction. The modified contour map is attached as appendix 8.The declined water level will result reversing the flow direction since the shallow aquifer is

hydraulically connected to the lake as it was explained by (Nabide, 2002) hypothesis. The artificial recharge can play important role in restoring the reversed natural groundwater flow.

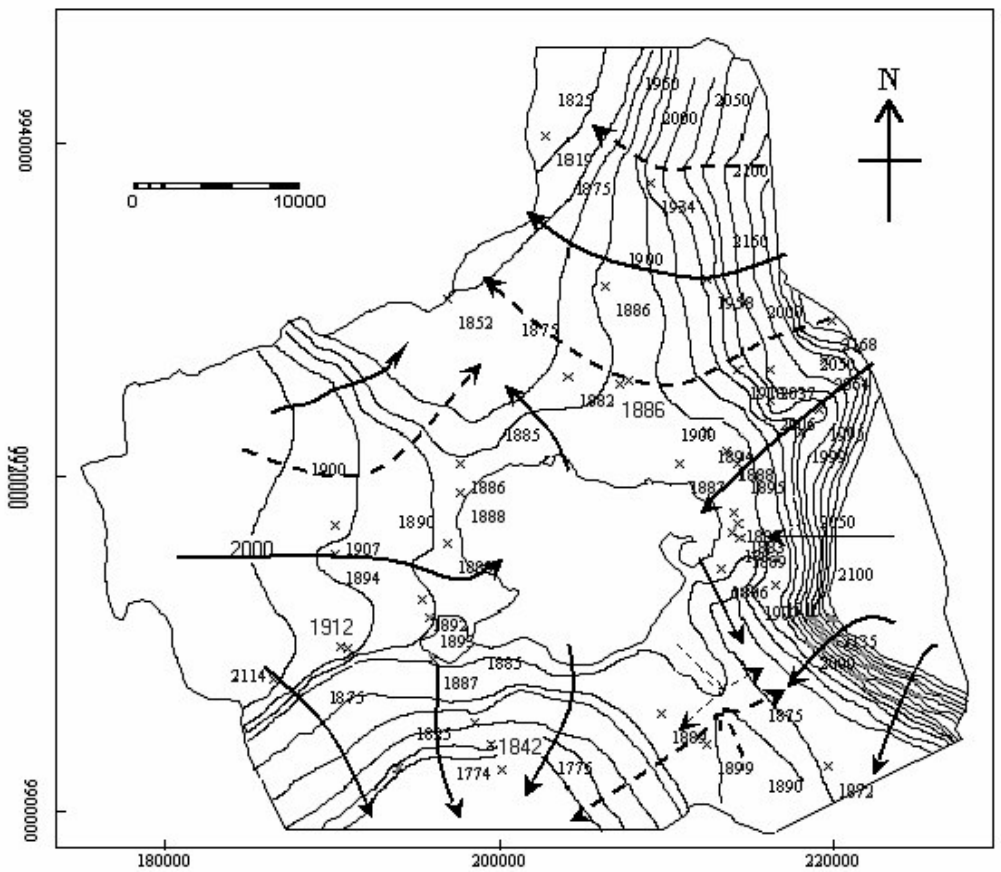


Figure 5.5 Flow direction as dictated by the historic heads of 1980. (Owor, 2000)

5.2. Water quality

The chemical compatibility of the recharge water and the groundwater is important in artificial recharge studies. If the native water in an aquifer reacts with the recharged water it can either degrade or improved. The objective of the geochemical analysis was to analyze the water quality of the recharge and groundwater. Proper sampling techniques and analysis in the laboratory have been done in order to validate the results.

Recharge water

The water to be used for the recharge is runoff generated from the green houses. The rain water collected from the green houses is free of mineral pollutants like florid and calcium salts that are found in the groundwater samples. The catchment is free from chemical contamination apart from the suspended sediments from the erosion of the canals.

Groundwater

5.2.1. Sampling

The sampling bottles were washed and rinsed thoroughly with distilled water before going to field. They were marked with a water proof marker and stocked with tape. The sampling quality was supported by

collecting 10 % of duplicated and one distilled water sampled with the same procedure. Five samples were collected from the well field.

The first sample bottle is 250ml acidified with HCl and were used for the analysis of anions except chloride. The second sample bottle is 100ml acidified with HNO_3 and it was used for the analysis of cations as well as chloride. The purpose of acidifying the samples is to avoid precipitation of carbonate and bicarbonate which affects the content of dissolved cations and also to prevent microbial activity that can alter the chemical composition of the water sample. During the field some parameters such as PH, EC, Temperature, Alkalinity, were measured.

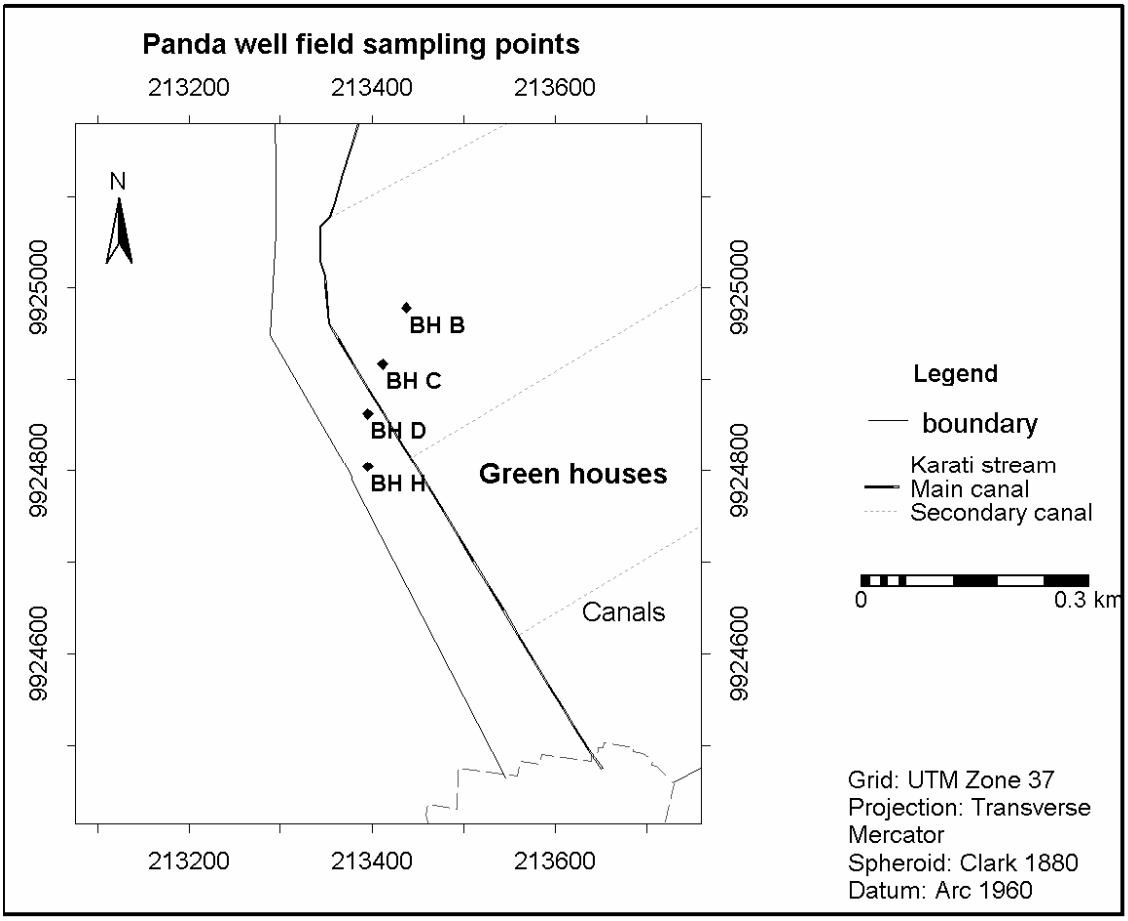


Figure 5.6 Location of water sample wells

5.2.2. Laboratory analysis

The water samples taken from the well field were analyzed for the major cations and anions. They were analyzed in ITC geochemical laboratory. The anions were analyzed with the Hatch spectrometer and the cations were analyzed using ICP method. EC, PH, Alkalinity and temperature were measured in the field using standard hand held-held calibrated field meter.

Reliability check

The accuracy of the water analysis was checked with the anion-cation balance because the solution must be electrically neutral. The sum of the cations in $meql^{-1}$ should be equal to the sum of anions in $meql^{-1}$.

$$Anion - cation = \frac{\sum cations - \sum Anions}{\sum Cations + \sum Anions} * 100 \quad \textbf{(5.1)}$$

Analysis of the groundwater with % difference<5% are regarded as acceptable but in very dilute and very saline waters, it may reach up to 10% is acceptable due to the greater errors incurred in measuring trace concentrations of common ions in dilute groundwater or in the multiple dilution required for analysis of concentrated groundwater (Gascoyne 2004). The sample analysis was found to be reliable as they area within +-10% difference range. The result is summarized below and the detail description is attached as appendix 8.

Code	Location		Sum of Cations	Sum of Anions	Ionic balance
	Easting(m)	Northing(m)			
BH B	213439	9924990	6.34	6.30	2.651
BH C	213404	9924908	6.42	6.11	0.612
BH D	213385	9924852	6.06	5.67	-0.046
BH H1	213396	9924806	5.66	5.38	1.681

Table 5.2 Ionic balance of the samples

5.2.3. Interpretation and analysis

Piper plots

All the water samples were taken from the production wells. Most of the waters in the study area are of alkali-bicarbonate or $Na^+K^+HCO_3$ type water as indicated by (Morgan, 1998). The result of the analysis suggests that all the samples are of the sodium bicarbonate type (Na-HCO3-) type which is in agreement with the study made by the aforementioned author. The results are plotted in piper diagram in Figure 5.7.

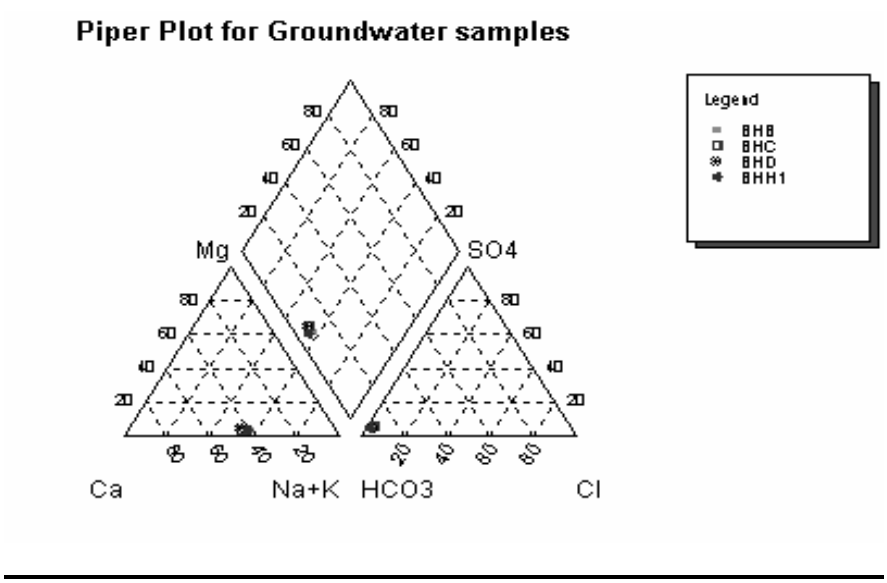


Figure 5.7 Piper plot for the groundwater samples

5.2.4. Comparison of recharge water and groundwater

During recharge geochemical reactions can occur that adversely affect aquifer permeability and cause changes in the quality of the recovered water. These chemical and physical changes are a function of recharge water quality, native groundwater quality, aquifer mineralogy and change in temperature and pressure that occur during recharge and recovery (Pyne, 1995). The most notable of the possible adverse geochemical reactions are precipitation of calcium carbonate (calcite), precipitation of iron and manganese oxide hydrate, and the formation, swelling or dispersion of clay particles. Rain water is of good quality with total dissolved solids (TDS) levels of about 1 to 50 mgl^{-1} with an average of about $10mgl^{-1}$ (Bouwer, 1978). The mean chemical composition of the ground water samples was compared with the standard rainwater of Northern Nigeria (Goni et al., 2001). A comparison was made by expressing the analysis of both in terms of $meql^{-1}$ of total cations and anions as shown in Table 5.3.

Constituent	**Groundwater**		**Standard rain water composition (Northern Nigeria)**	
	Mg/l	meq/l	Mg/l	meq/l
EC	621(µs/cm)			
PH	7.4			6.4
Temperature	20.5(^{0}c)			
Calcium	49.48	2.47	4	0.20
Magnesium	1.99	0.17	0.4	0.03
Sodium	60.61	2.64	0.75	0.03
Potassium	21.54	0.55	1.6	0.04
Chloride	7.8	0.22	1.28	0.04
Flourine	2.41	0.13		0.00
Sulphate	11	0.23	2.7	0.06
Bicarbonate	337.3	5.53		0.00

Table 5.3 Comparison of the recharge rain water and mean groundwater composition

6. Options and Scenarios of Artificial Recharge

The suitability of an artificial recharge site was determined by field and laboratory measurement of soil properties, field experiments and modelling. The technical feasibility of artificial recharge methods have been discussed based on the source water availability and quality, site specific hydrogeological studies, land availability and overall objectives of the artificial recharge. Since the water source of the recharge is a direct runoff from the rainfall and free of contamination, the most important factors determining the feasibility of artificial recharge method is the hydrogeological conditions of the underlying area of interest and recharge objective.

The artificial recharge methods considered under this study are surface infiltration basin and recharge wells. A spreadsheet model was developed to analyse and compare the two artificial methods based on their recharge efficiency, construction cost and space availability.

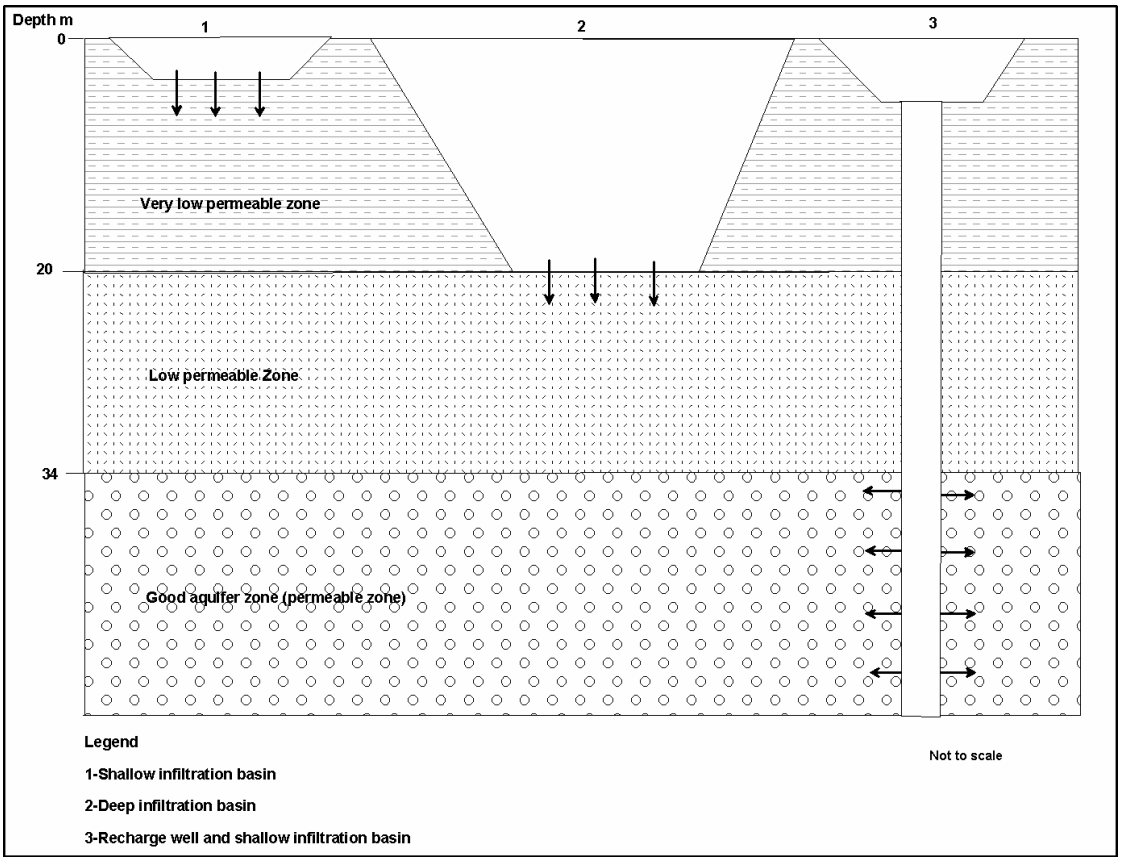


Figure 6.1 Artificial recharge methods

6.1. Shallow infiltration basin

Surface recharge techniques are feasible where the aquifer to be recharged is unconfined, permeable and sufficiently thick to provide storage space (Bouwer, 2002).

The shallow infiltration basin requires the presence of permeable soils at or near the land surface. The field investigation of the study area revealed that the top 20m depth is covered with heterogeneous, low permeable layers of silty clay, clayey silt, and sandy silt sediments. The infiltration rate test results a hydraulic conductivity in the range of 0.02 to 0.2mday^{-1}. The infiltration rate is limited due to the low permeable layers and therefore this option is not feasible to achieve the recharge objectives.

6.2. Deep infiltration basin

Deep infiltration basin can be designed on the main canal with a masonry wall and/or earth dyke. The major physical components of the structure are the earthen dyke and/or the masonry wall, the reservoir to harvest and infiltrate the water. The water in execs of the infiltration basin will be spilled over the side of the basin and disposed in to the Karati stream.

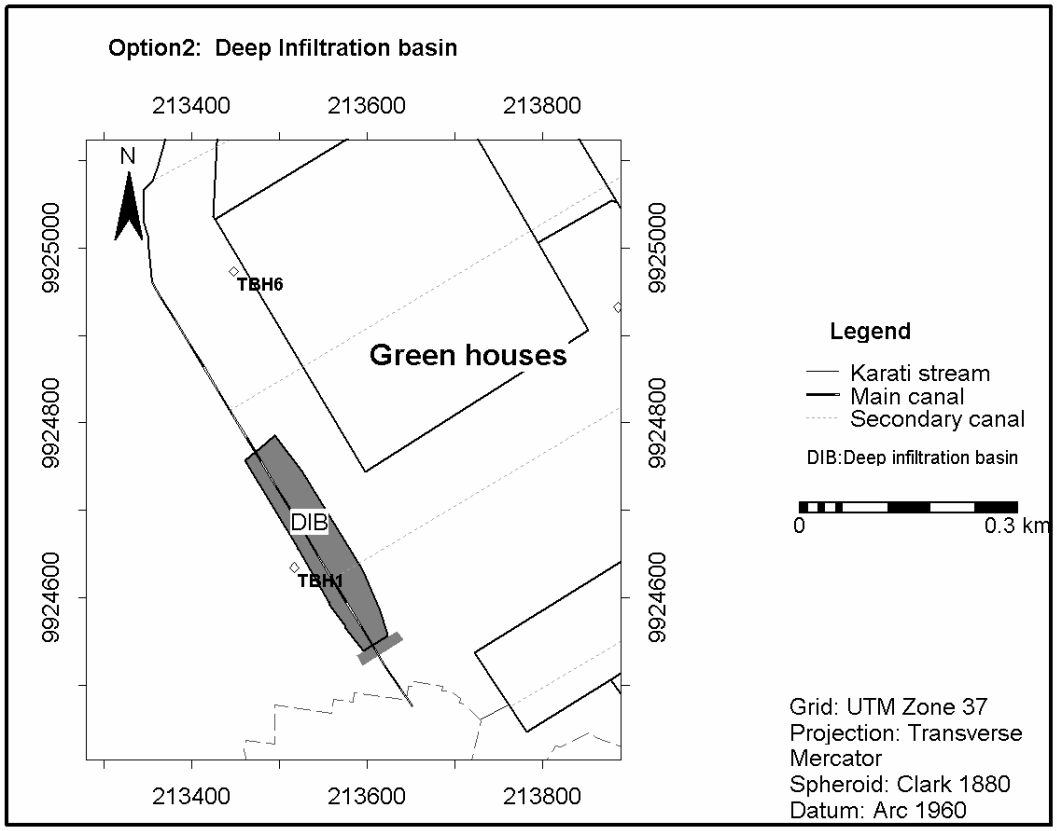


Figure 6.2 Location and layout of deep basin infiltration basin

Recharge efficiency and cost per cubic meter

The spreadsheet model developed compute the daily water balance of the storage, efficiency of the infiltration basin and cost of construction per cubic meter of recharged water for the duration of 31 years. The preliminary cost of construction includes the excavation and drilling costs. The model has components of daily rainfall, infiltration rate of the basin, inflow volume, spilled water and storage change. The daily runoff from the green houses was calculated from the long term daily rainfall (1973-2003), green houses area and runoff coefficient using the rational empirical formula here with. The daily infiltration rate of the basin was determined from the field test. The spread model is attached in appendix 9 with one year data sample to show its contents. The water balance is expressed as:

Inflow- Outflow =Change Storage

$$R-(I_r+S_w)=\Delta S \qquad 6.1$$

Where:

R: the daily runoff from the green houses (m^3day^{-1})

I_r: the daily infiltration rate of the recharge wells (m^3day^{-1})

S_w: the daily spilled water (m^3day^{-1})

ΔS: the change in of volume of the reservoir (m^3day^{-1})

The following assumptions were considered during the model development:

- The minimum depth of excavation should be 20m.
- The hydraulic conductivity of the basin is about from $0.5mday^{-1}$
- The unit cost of excavation is 500 KES (Kenyan shilling) per cubic meter.
- The model is computed for duration of 31 years life span.

Based on these assumptions the model results an infiltration volume of $1200m^3day^{-1}$. As shown in Figure 6.3 the response of the efficiency is insignificant with respect to the additional cost incurred to increase the storage capacity beyond $48000m^3$. The optimum output efficiency and cost per cubic meter are 91.6% and 1.64KES (Kenyan shilling) respectively. The results of the model are summarized below in Table 6.1.

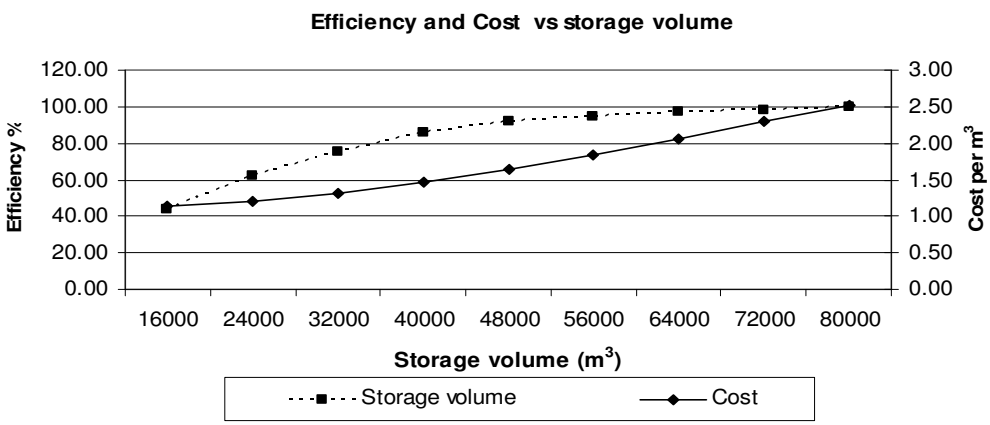


Figure 6.3Efficiency and cost versus storage capacity

Parametrs	Units	Amount	Remarks
Storage capacity	m^3	48000	
Hydraulic conductivity	mday-1	0.5	
Infiltration volume	m3day-1	1200	
Unit excavation cost: deep	$KESm^{-3}$	500	
Unit BH Cost	KES	60000	
Total Excavation cost	KES	14400000	
Cost per mean infiltration rate	$KESm^{-3}$	1.6400	
Number of years		31	KES:Kenyan shilling
Efficiency	%	91.64	

Table 6.1Summary of the output for deep infiltration basin

6.3. Recharge wells and shallow infiltration basin on the main canal

The recharge wells can be used to apply recharge water from the reservoir to the unconfined aquifer. The Shallow reservoir will be designed on the main canal and the recharge well can be drilled inside or outside of the reservoirs so that it can facilitate recharge to the specific aquifer and provides a direct hydraulic connection through overlying strata to the aquifer being recharged. The recharge head available is the elevation difference between the surface water level in reservoir and elevation of water table or piezometric head.

The recharge well potential was assessed from the field injection test, aquifer characteristics and modelling of the study area. The modelling enables to optimize the recharge well potential and spacing of well with their hydraulic head responses. The field test results in a recharge of about $80m^3hr^{-1}$ however since the recharge potential is decreasing with time due to the physical, chemical and biological processes, 75 % of the pumping rate is considered to estimate the potential (spender, 2006). Therefore the recharge well has a potential of $1400m^3day^{-1}$.Depending on the land availability the recharge wells are placed in linear arrangement along the reservoir. The lay out of the recharge well in the infiltration basin is shown in (Figure 6.4).

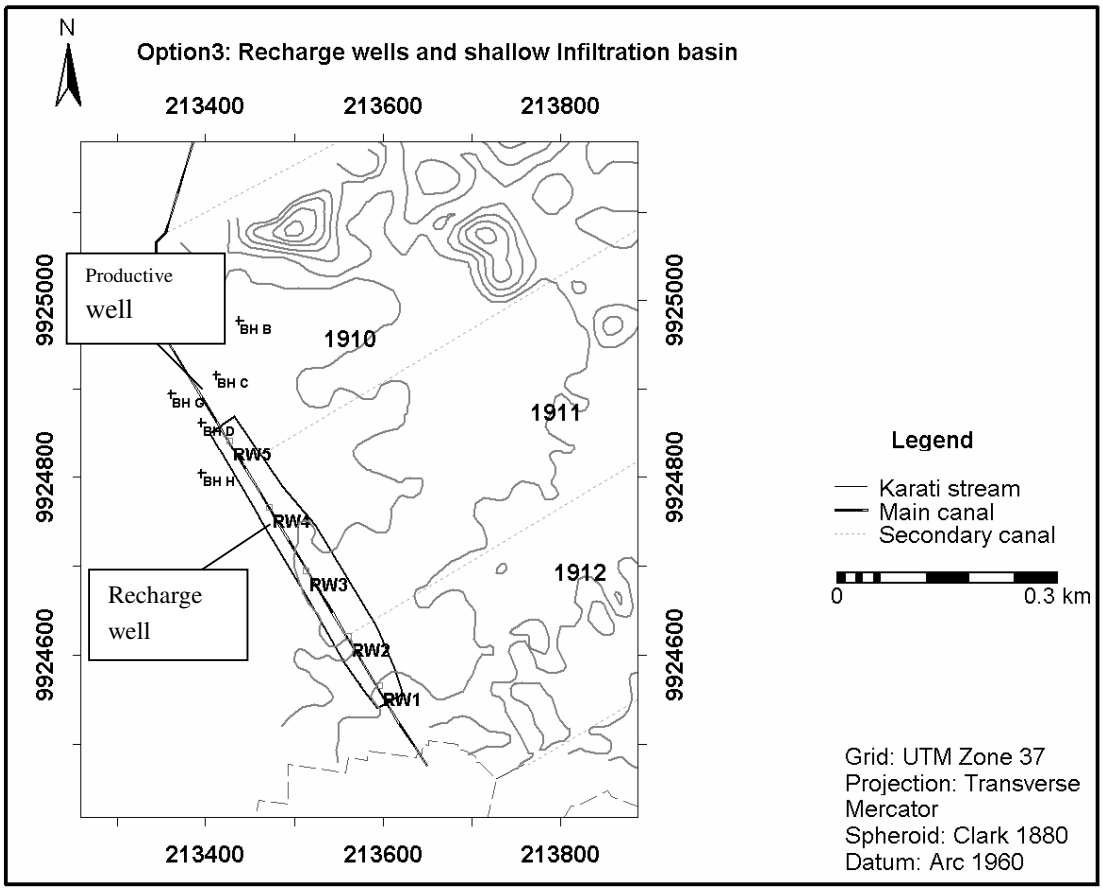


Figure 6.4 Layout of Recharge wells in the reservoir

The hydrogeology study indicates that the aquifers are mainly unconsolidated sediments consists of silty sand, poorly graded fine sand and gravely silty sand. These formations were observed collapsing during the test borehole drilling due to their loose and unconsolidated nature. Therefore, the recharge wells needs

casing to prevent the sides from collapsing. The casing should be cemented from the bottom of the casing to ground surface to ensure an adequate seal against flow movement outside the casing through possible channels opened during construction. A screen or perforation is necessary to be placed at the bottom of the casing to allow water to infiltrate from the injection well into the saturated zone. A filter can be placed below the basin to eliminate any suspended sediments which can block the recharge well. The filter pack consists of coarse sand, gravel and pebbles with thickness of 0.55m, 0.95 and 1.5m respectively from the top to the bottom (Ambast, 2005). The proposed recharge well and filter design is shown in Figure 6.5. Well cap is proposed on the well head so that to prevent the entry of suspended materials including sediment in to the well with the inflowing water

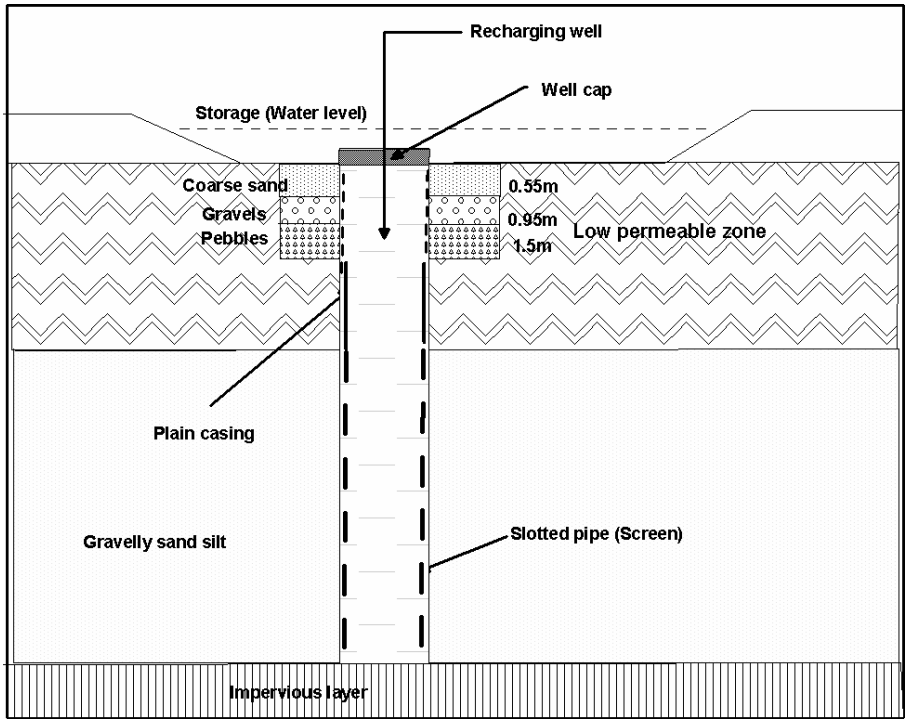


Figure 6.5 Design of recommended Recharge well (Not to scale)

Recharge efficiency and Cost per cubic meter

The same spreadsheet model as in section 6.2 was used to analyse the efficiency and cost per m^3, with additional inputs of Borehole recharge rate. The observations and assumptions are outlined below:

- The recharge wells have a potential of $1400m^3$.
- The spacing of the well should be 50 to 60m as per to the modelling result and space availability.
- Unit cost of excavation is 300 KES (Kenyan shilling) per cubic meter for shallow depth (up to 4m).
- Unit cost of drilling and complete construction for a well of 60m depth is 360000 KES (Kenyan shilling)

The graph plotted in Figure 6.6 is designed to show the effect of cost per cubic meter with respect to the change in BH infiltration rate and storage capacity of the shallow basin. From the graph it is clear that the cost per cubic meter increase with respect to the change in Borehole recharge rate and storage capacity increases in both directions but with large significance in the vertical direction.

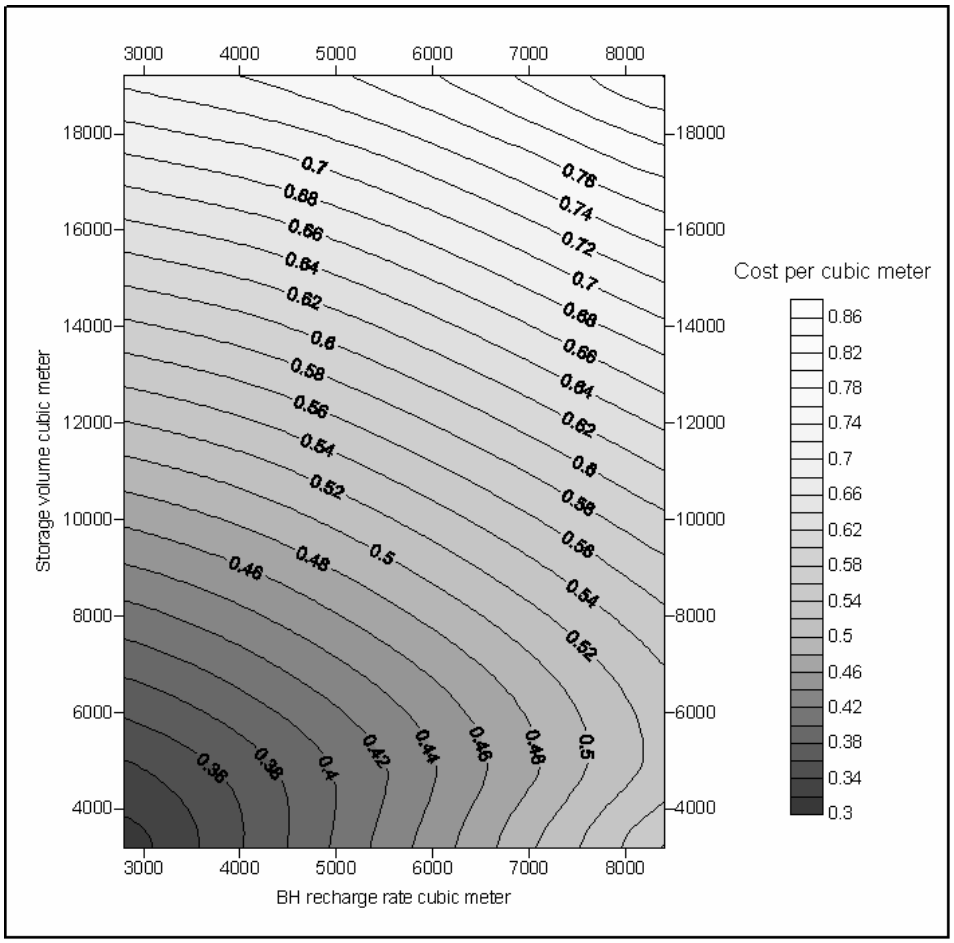


Figure 6.6 cost per cubic meter of recharge wells with respect to storage capacity and BH recharge rate

The model was displayed by changing the storage capacity and Borehole recharge rate. The optimum output is recharge rate of 7000m^{3}day^{-1}, with infiltration basin capacity of 9600m^3 and well spacing of 60m. The recharge efficiency and cost per cubic meter are 89% and 0.56KES (Kenyan shilling) respectively. The additional cost incurred to increase the storage capacity above 9600m^3 results insignificant change on the efficiency of the recharge wells, Figure 6.7. The output of the model is summarized in Table 6.2.

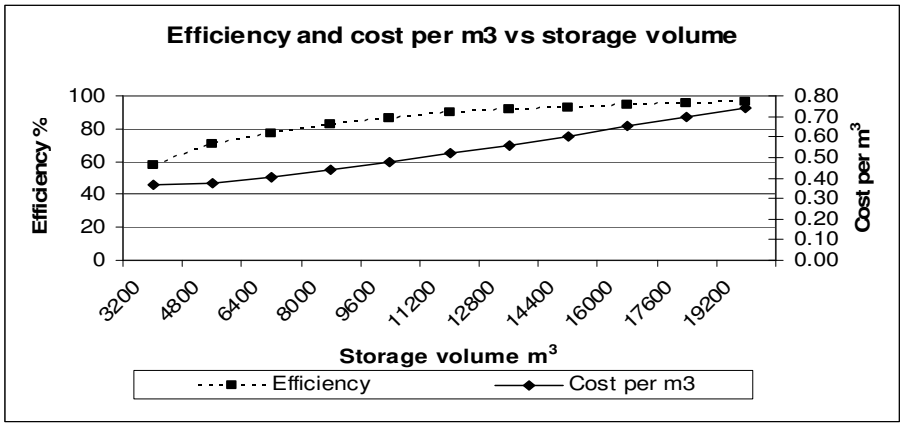


Figure 6.7 Efficiency and Cost vs storage capacity

Parametrs	Units	Amount	Remarks
Storage capacity	m^3	9600	
Total Recharge rate	m3day-1	7000	
Unit excavation cost: shallow	$KEShm^{-3}$	300	
Expected infiltration capacity of per BH	m^{-3}day-1	1400	
Number of BH required	-	5	
Unit BH Cost per 60m	KES	360000	
Total Excavation cost	KES	2880000	
Total BH cost	KES	1800000	
Total Cost	KES	4680000	
Cost per mean infiltration	$KESm^{-3}$	0.56	KES:Kenyan shilling
Number of years		31	
Efficiency	%	89	BH: Borehole

Table 6.2 Summary of the optimized output for Recharge wells with shallow basin

6.4. Recharge wells and shallow infiltration basin on the Karati stream

The second source of water for artificial recharge is the runoff from the karati stream. An infiltration basin can be designed with a masonry weir on the karati stream. Karati stream is one of the sub catchment which drains to Naivasha Lake and its catchments area is about 154.45km^2. It provides perennial flow in its upper reaches and cutting a deep gully as it descends the platforms east of Naivasha. The drainage is longitudinal linear channel with little drainage density.

The peak runoff from karati was estimated with different methods of empirical methods (Gorrotxategi Gonzalez, 2001). The applied methods were rational formula, SCS curve method and slope area methods. The results are summarized below in Table 6.3.

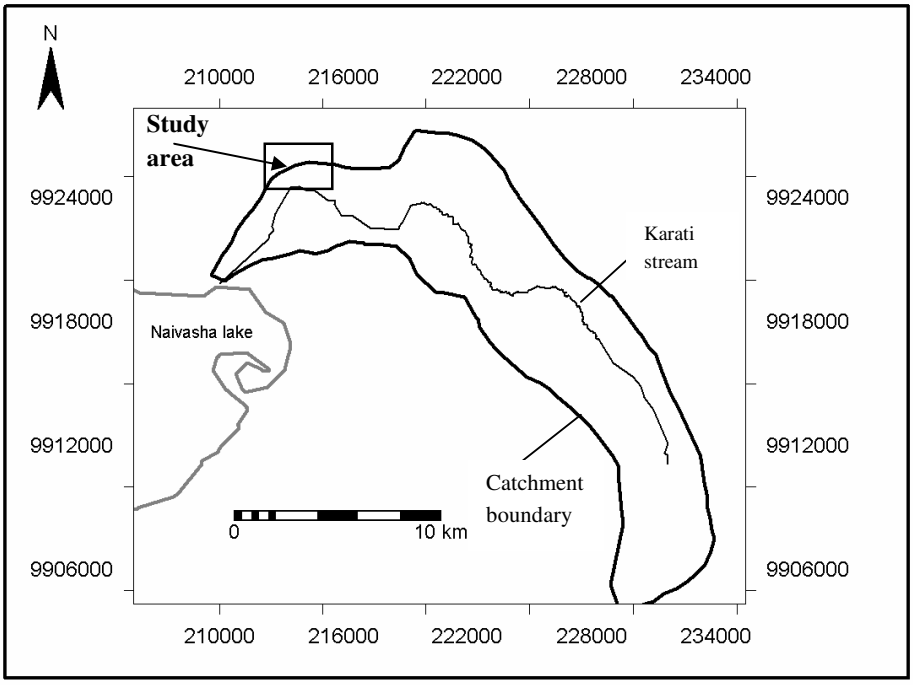


Figure 6.8 Karati drainage map

Method		Q in m^3sec^{-1}	Remark
Slope area		102	AMC: Antecedent moisture content
Rational		118.47	
Q=C*I*A			
SCS CN method	AMC I	27	
	AMC II	380	
	AMCIII	171	

Table 6.3 Summarized the estimated peak discharges of Karati (Gorrotxategi Gonzalez, 2001)

The volume of water harvested can be computed with the following assumptions and observations as:

- The masonry weir height is assumed to be 6m and the excess water is spilled on the top
- The length is estimated from the stream profile, 1000m
- The average depth of water is about 3m.

$Storage.volume = L \times W \times D$ Where

L= Length of the storage bank (m) estimated from the profile surveyed during the field using GPS (figure6.9)
W=Average width of the stream channel (m)
D=Average depth (m)

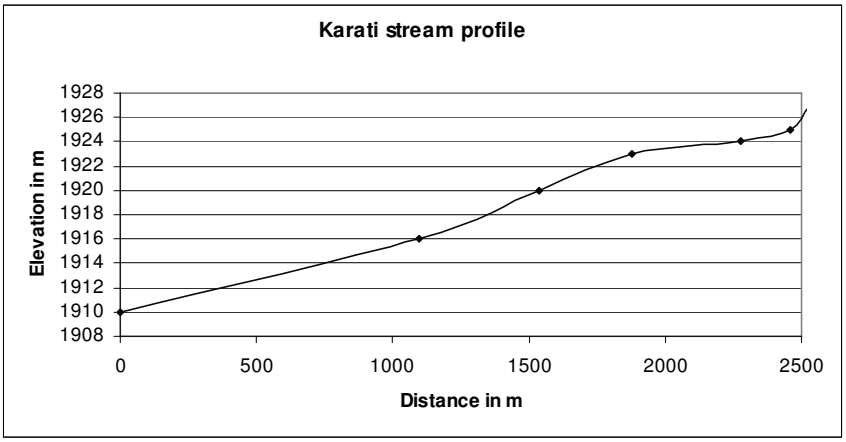


Figure 6.9 Karati stream profile from GPS survey

The recharge process can be facilitated by the recharge wells in side the storage. The design is similarly as mentioned in section 6.3. Based on the above mentioned assumptions the stream bank has a storage potential of about 37500m^3.
The main potential problem of this option is the sediments transport and accumulation from the catchment, which decreases the storage potential as wells the Bore hole recharge capacity.

6.5. Comparison of infiltration basin and recharge wells

6.5.1. Deep Infiltration basin

The possible advantages:

- It is easier for construction, operation and maintenance works
- As compared to recharge well, infiltration basins are less vulnerable to clogging because the infiltration rates into the aquifer around the borehole are much higher than surface infiltration.
- Remediation of clogging in surface infiltration systems is much easier than recharge well.

Disadvantage

- The recharge potential is only 3% of the storage volume
- The land requirement is relatively maximum as compared to the wells
- The evaporation loss is higher

6.5.2. Recharge wells and shallow basin

This method has the following advantages and disadvantage

Advantage:

- It has a recharge potential about 73% of the storage volume.
- As with all wells, land requirements are minimal.
- The recharge wells can be used as pumping wells the latter in turn makes maintenance by periodic well redevelopment is possible.
- Recharge wells can provide high rate in unsaturated zone since the layers of high and low hydraulic conductivity exists; it takes an advantage of the high horizontal hydraulic conductivity the aquifer and by pass the vertical restrictions.
- The losses of water in the form of soil moisture and evaporation, is much less than the infiltration basin
- The water has been profiteered through the filter, so that its clogging potential is significantly reduced.
- The recharge is fast and immediately delivers the benefit
- The cost of pumping decreases

Disadvantage:

- Monitoring the well performance is difficult to identify which well is performing better and worse.
- Maintenance of the recharge well will carried on during the dry period or when the reservoir is empty
- It is difficult to redevelopment if the recharge wells are not used for pumping
- Unlike a pumping well which is self developing an injection recharge well is self clogging which results in decline of the injection rate.

6.5.3. Conclusion of the comparison

According to the above mentioned comparison factors namely recharge efficiency, unit cost per cubic meter of recharged water and land availability the recharge wells combined with shallow infiltration basin is feasible method to meet the artificial recharge objectives.

6.6. Potential poblems and solutions of Recharge wells and shallow infiltration basin

6.6.1. Potential problems

The main problem for artificial recharge of groundwater is clogging of the pore spaces. Clogging is characterized by a reduction in the pore space of the aquifer, which can reduce the aquifer's capacity to store and transmit water (Perez-Paricio and J.Carrera, 1998). It is a result of physical, chemical and biological processes acting as the recharge water enters and infiltrates the recharge medium. The hydraulic characteristics of the well is changed due to clogging and results in a decreasing rate of recharge.

Physical clogging

The physical processes are accumulation of inorganic and organic suspended solids in the recharge water, such as clay and silt particles, algae and micro organism cells. Suspended material within the recharge water clogs pore spaces in the recharge medium. In natural water, suspended clay is the most frequent cause of aquifer clogging. The primary sites of plugging are the gravel pack, the borehole wall, and the formation immodestly surrounding the borehole wall (Pyne, 1995).

Chemical clogging

The interaction between the recharge water, groundwater or the recharge medium may form a chemical precipitate that deposits in pore spaces. Clogging of the recharge well due to chemical reaction may occur at the screen or casing perforations, the formation face, or in the aquifer itself. Chemical clogging can be caused by (AMCE and EWRE, 2001).

- Precipitated metabolic products of bacteria including iron oxide, ferrous bicarbonate, metallic sulfide (sulphur) or calcium carbonates
- Chemical interaction of the dissolved chemicals in the injected water and in the aquifer formation yielding precipitates, or solution and redeposit ion of soluble compounds such as gypsum.
- Reaction of high sodium water with soil particles causing deflocculation and swelling of the soil (clay) particles.

Biological clogging

The growth of algal or bacterial material in pore spaces, the accumulation of by product resulting from the decomposition of biological growth causes clogging. The bacterial contamination of the aquifer by the injection water and biological change in the injection water and the groundwater may result clogging.

6.6.2. Possible solutions

The operational life of artificial recharge facility, recharge well, will be shortened due to the problems incurred in artificial recharge There are many methods that can be implemented before and during recharge to reduce and even prevent many of the problems mentioned in section 1.3.1.

Physical clogging

- To prevent physical clogging of the recharge medium or recharge wells, the water coming from the green houses should be low in suspended sediments. There are two possible options to minimize the suspended sediments. These are to implement stilling basin to settle sediments before reaching the reservoir or to line the canals.
- Sand and gravel filters can be constructed next to recharge wells to reduce the level of turbidity in water before recharge.
- The recharge well can be surged or pumped in order to dislodge fine sediments from the well or gravel pack.
- Single injection wells are typically redeveloped by installing a vertical turbine pump, by air lift pumping or by swabbing and bailing with cable tool drilling equipment (Pyne, 1995).

Chemical clogging

- It is difficult to predict the geo chemical reactions that can adversely affect the aquifer permeability. But if it happens the chemical clogging is usually confined to the immediate vicinity of the well gravel pack or surrounding rock matrix.
- Pumping and surging can be trial tested to develop precipitates from around the well.
- If pumping is not successful, a chemical solution can be added to the recharge water to dissolve the chemical precipitate in pores spaces.

Biological clogging

- Surging or pumping the well may disturb organic matter and reduce clogging.

6.7. Artificial Recharge and water balance

The water balance of the lake Naivasha plays a great role in managing the water resources of the lake and artificial recharge has great contribution in this water resources management. Controlled artificial recharge of groundwater is an important tool for proper groundwater management (Attai et al., 1998). The lake and the surrounding catchment drained by ephemeral streams which disappear underground before reaching the lake have a catchment area of about 1000km^2. This makes the total basin area to be about 3376 km^2. The evaporation experienced by Naivasha average 1801mm annually for the period 1959 to 1990, as measured at Naivasha water Development office using a standard class A pan (Ashfaque, 1999).

Three scenarios were developed to overview the effect of artificial recharge on the water balance of the area under different hydrological conditions. As the water balance of Naivasha was explained by previous studies, the major components of the inflow in to the lake are the surface runoff from the surrounding catchment, Gilgil and karati, and groundwater inflow. The out flow components are evapotranspiration, groundwater out flow, abstraction for irrigation and other purposes.

6.7.1. Scenario 1: Natural catchment condition

The major components of the water balance are precipitation, runoff, natural recharge and evapotranspiration. The area receives an annual rainfall of 660mm, for the period 1960 to 2003, as measured at Naivasha D.O station, which is the characteristic of semi arid climate. The natural recharge is estimated 4.38mmyear^{-1}, which is less than 1% of the annual rainfall from the swap model (Nalugya, 2003). The runoff and evapotranspiration are 64 and 592mmyear^{-1} respectively from the simulated water balance model (Gitonga, 1999). Therefore 89% (592mm) of the water is lost through evapotranspiration. Therefore the contribution to the lake is 11% of the runoff, 7mm.

The relation for water balance is described as:-

$$P = I + R + E \qquad 6.2$$

Where:

P is the annual precipitation in mmyear^{-1}
I is the annual recharge in mmyear^{-1}
R is Runoff in mmyear^{-1}
E_t is Evapotranspiration in mmyear^{-1}

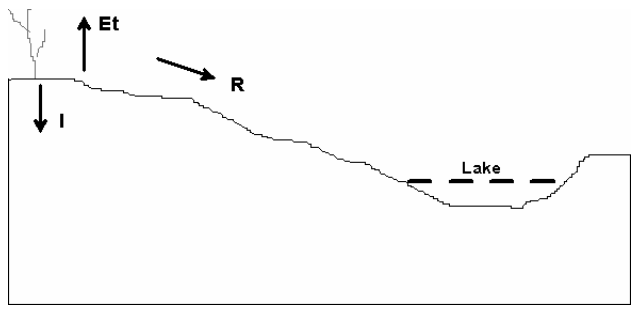


Figure 6.10 Sketch showing the water balance components scenario one

6.7.2. Scenario 2: Green houses catchments

The changes in land cover of the catchment in to green houses influence the water balance of the area. In this scenario the dominant process will be the direct runoff to the lake. Groundwater is abstracted for the purpose of irrigation development. The abstraction rate is calculated from the average consumed water for green house irrigation which is about 3mmday^{-1} (unpublished Tipis thesis). Therefore the net abstracted water is 1095mmyear^{-1}. Almost 90% (592mm) of the rain falling in the catchments will join the lake in the form of surface runoff. The water balance of lake indicates 83 percent of the water inflow from the lake is evaporated (Gitonga, 1999). For example 547mm can be evaporated from the annual rainfall of 660mm. Therefore the contribution to the lake is 17% the runoff, 101mm. The water balance can be described as:

P=I + R and for the lake balance I + R –Q-E=ΔS **6.3**

Where:

P is the annual rainfall in mmyear^{-1}
I is infiltration from irrigation return flow mmyear^{-1}
Q is groundwater abstraction
E is Evaporation in mmyear^{-1}

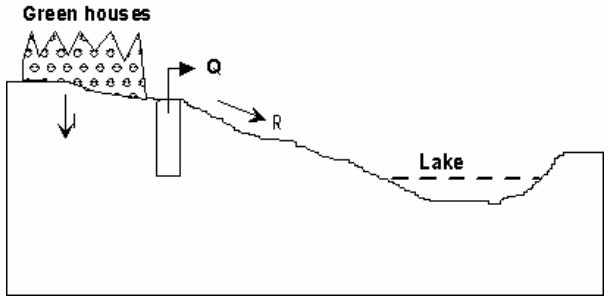


Figure 6.11 Sketch showing the water balance components under scenario two

6.7.3. Scenario 3: Green houses catchments and artificial recharge

In this scenario the runoff from the green houses will be harvested in reservoir and it will recharge the groundwater storage through the recharge wells. The significance of an artificial recharge is not only restoring the declined groundwater of the area but also to prevent from direct evaporation of the water. Therefore the net water demand for irrigation as it is calculated before is about 1095mm. The recharge water can be computed as:

$$R_w = P \times R_c \times R_{eff} \qquad \textbf{6.4}$$

Where:

R_w is Recharge water in mmyear^{-1}
P is the precipitation in mm
R_c is Runoff coefficient, 0.9
R_{eff} is Recharge efficiency, 0.89

The recharge water is 529mm and the contribution to lake will be 17% of the spilled and/or seep water, which is 16mm. Therefore the net water abstracted will be 550mm. The water balance can be described as:

$P = Q_{in} + L$

Where:

P Annual precipitation in
L Losses (seepage,)
Q_{in} Artificial recharge in mmyear^{-1}
Q_{out} Groundwater abstraction in mmyear^{-1}

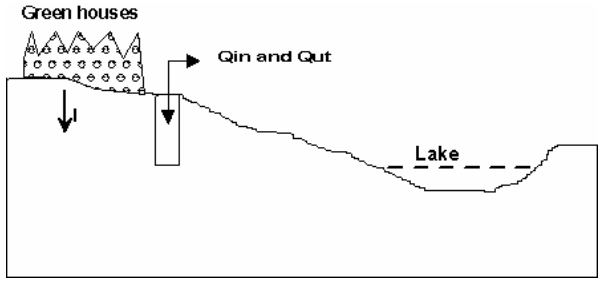


Figure 6.12 Sketch showing under scenario three

7. Modeling

A groundwater flow model can be a useful tool in developing an understanding of the hydrogeology of the groundwater basin, a portion of the basin or a site, by identifying which aquifer parameters have the greatest influence on the aquifer(s) being studied. Also it can be used to predict rise of groundwater mounds to determine if the aquifer can accommodate the lateral flow associated with the proposed recharge rates. This can also be used to estimate the lateral extent and movement of stored water (Brouwer.H, 1999).

A number of groundwater flow models were developed in Naivasha basin. For this purpose of study the regional model of North east Naivasha by (Kibona, 2000) was used to create the local area of interest for the analysis of the artificial recharges with modifications on the input parameters and grid refinements.

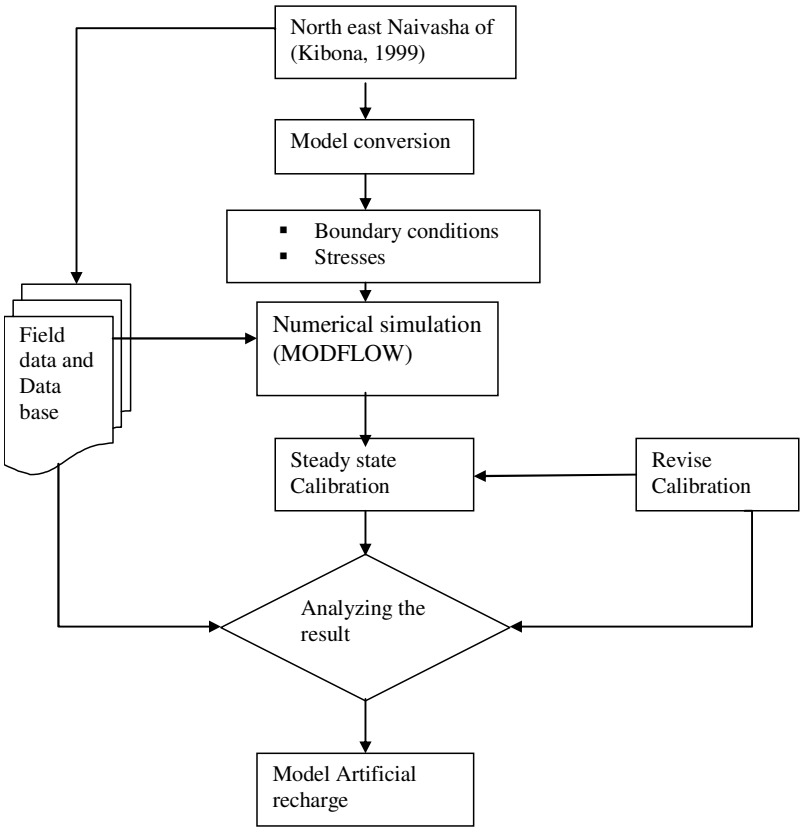


Figure 7.1 Flow chart of the modeling process

7.1. Model development

7.1.1. Methodology

Modeling of the local scale of the area was done in two phases. In the first phase a large scale model of the North east Naivasha constructed by (Kibona, 2000) was updated. In the second phase local scale model was constructed using the model converter of the PMwin functions that occupies a small area within the large model. The groundwater potential computed from the regional model is applied as specific head boundary

conditions to the local scale model. The layer data, including elevations and transmissivity, were interpolated from the regional to the local model.

7.1.2. Hydrostratigraphy

The aquifer system is bounded at the top by the lake sediments and at the bottom by an impermeable layer of undifferentiated basement volcanics. The layered groundwater aquifer is schematized in two layers. The first layer composed of the lake sediments consists of fine sand, silt, clay and diatomite with variable thickness 10 to 50m deep. The second layer, which is highly permeable (reworked volcanics or weathered contacts between different lithological units), was assigned an average thickness of 20m. The first and second layers were modelled using confined aquifers.

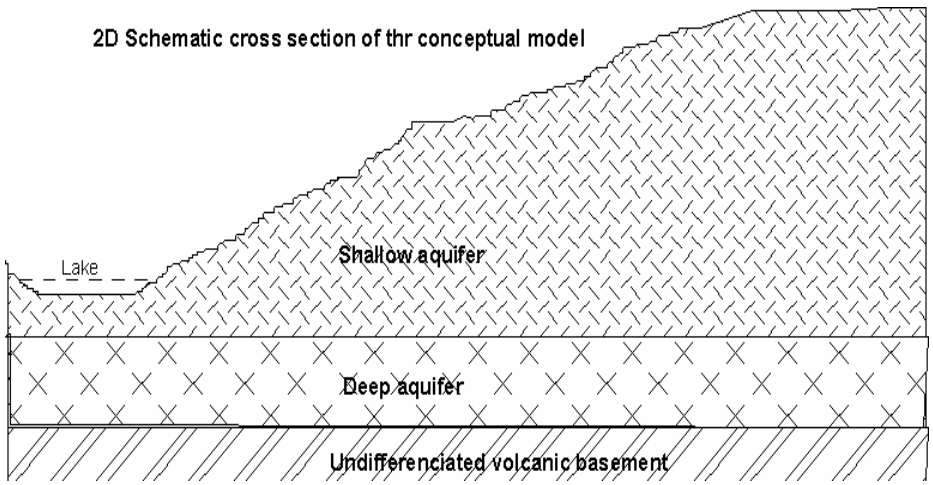
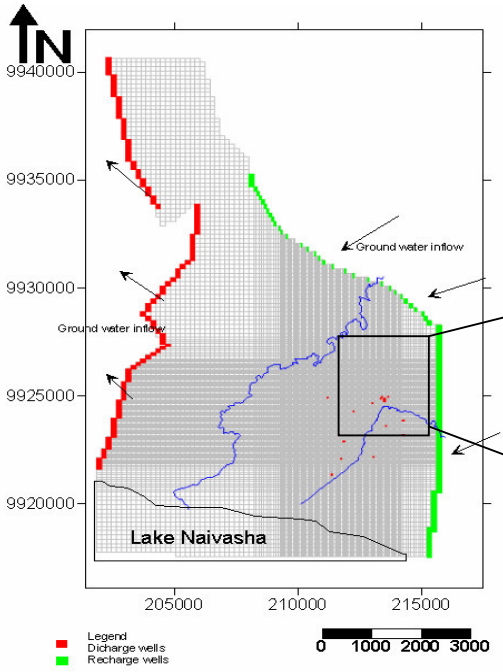


Figure 7.2A 2D schematic cross section of the conceptual model drawn from the Kinangop Fault (eastern) to the Lake Naivasha basin. (Not drawn to scale).

7.1.3. Grid geometry

The large scale model of Kibona (2000) has an area of 185km^2. The study domain was discretized into 157 columns and 211 rows with maximum grid size of 200X200m and minimum size of 50X50m. It was converted to local model area of 19.74km2 and refined the grids into 25X25 and 10X10m. The most distinct model refinement was done in the abstraction wells for better spatial representation of the hydrologic system.



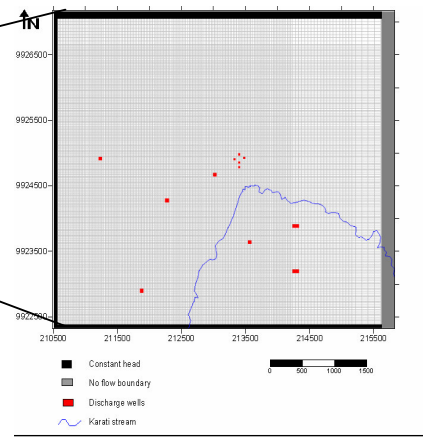


Figure 7.3Grid design and boundary condition of the large model adopted (Kibona, 2000)

Figure 7.4Grid design and boundary condition of the interest area

7.2. Model input parameters

7.2.1. Transmissivity

Hydraulic properties of the Shallow and deep aquifers in and near the study area have been estimated from the field tests and previous investigators. The transmissivity, of the aquifer used in the model for the first run were obtained from the analysis of the pumping test data collected during the field work. The transmissivity obtained from the pumping test result ranges from 800 to 1200m^2day^{-1}.

7.2.2. Recharge

Recharge to the (Kibona, 2000) model was applied to the top-most active cell in each vertical column. The recharge was assumed uniform over the entire area. There are two recharge mechanisms.

- The direct recharge is percolation through the vadose zone in excess of both soil moisture deficits and evaporation by. The natural recharge is estimated about 4.38mm/year (Nalugya, 2003), which is 1% of the total annual rainfall . The study area is flat, with no indication of valleys, so when it rains due to high rate of evaporation, much of it evaporates and that which remains infiltrates through the pumice sand to the deeper part of the unsaturated.
- The second recharge is assumed from groundwater inflow in the eastern boundary from through fault escarpment to the second aquifer (figure7.3).

7.2.3. Well abstraction

The modelled area has been characterized by an intensive agricultural development during the last 15 years. As it explained by Hernandez (1999) and field observation, the wells are found in clusters in north east area rather than being homogenously distributed this is because of those farms far away from the lake and the

rivers, the only source of fresh water available is groundwater. Well withdrawals in the study area occur from irrigation and domestic wells.

The average consumed amount for green houses and outdoor irrigation is estimated to be 3mmday^{-1} and 6mmday^{-1} respectively (unpublished Tipis thesis). The area under irrigation in the green houses and outdoor was determined from the early November, 2005 Aster imagery and field work. The area of green houses in the north eastern Naivasha is estimated about 173ha shown below in Figure 7.5 Pivot and Green houses irrigation with November 2006 ASTER Imagery as back ground and the area of pivot irrigation is about 255ha.Therefore amount of abstraction was calculated using the formula:

Abstraction rate (m^{3}day^{-1}) = area of irrigated land x depth of irrigation

The depth of irrigation is obtained from the unpublished Tipis thesis, 2006. Based on the above assumptions the abstraction rate for the greenhouses and outdoor irrigation is 5190m^{3}day^{-1} and 15300m^{3}day^{-1} respectively. The total abstraction rate in the model area is about 20490m^{3}day^{-1}.

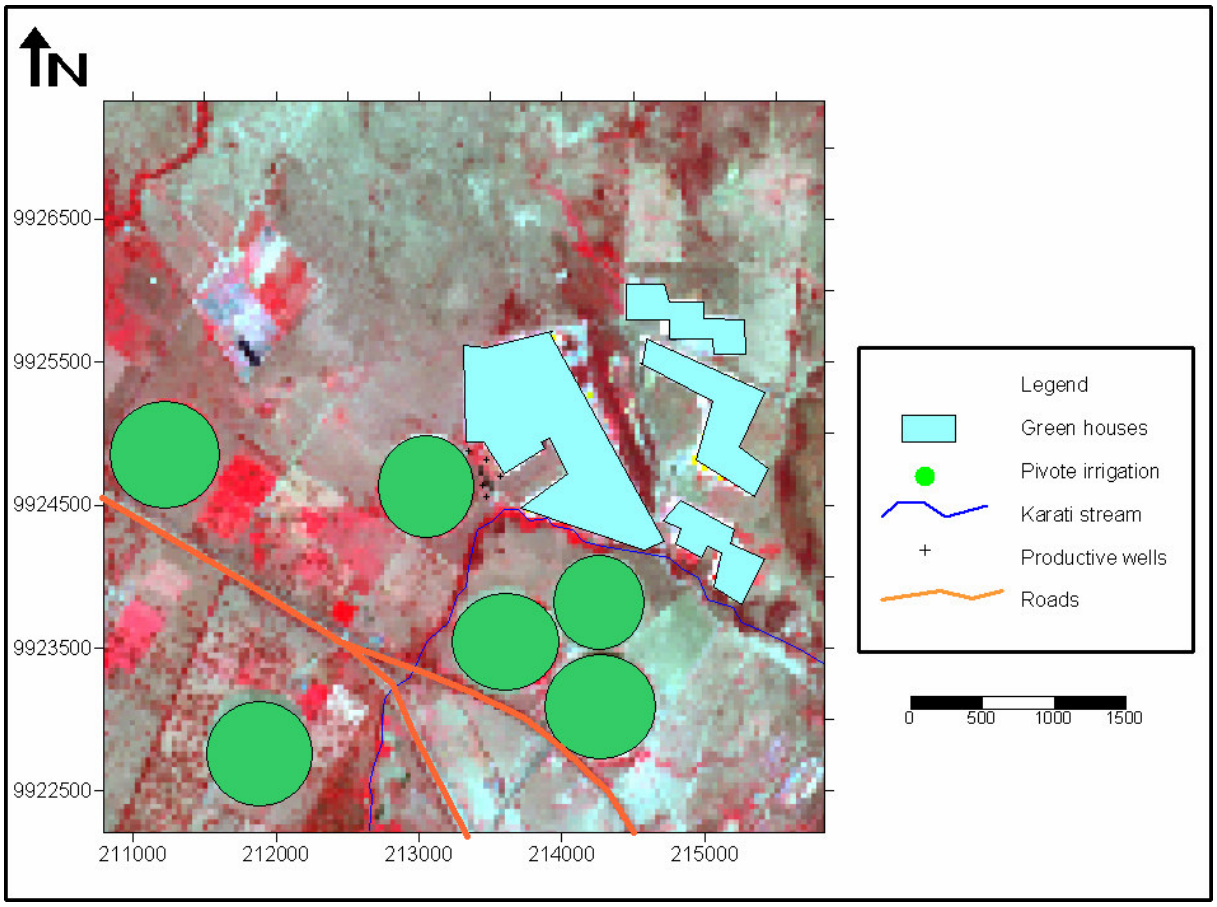


Figure 7.5 Pivot and Green houses irrigation with November 2006 ASTER Imagery as back ground

Name of Green house area	Area ($m^{2)}$
Bigot Flowers Ltd	504966.71
LINSEN ROSE Ltd	381681.52
MARIDADI FLOWERS Ltd	205453.55
PANDA FLOWERS Ltd	447525.78
STAR FLOWERS Ltd	195405.41
Total area	**1735032.97**

Table 7.1 Greenhouse areas in the Northeastern Naivasha

7.3. Steady state condition

7.3.1. Model calibration

Model calibration taking the initial estimates of the model parameters and solving the model to see how well it reproduces some known conditions of the aquifer initially calibrate a model (Fetter, 2001). The purpose of calibration is to establish that the model can reproduce field measured hydraulic head. During calibration a set of values for the aquifer parameters is found to approximate field measured heads. The groundwater flow model was calibrated by trial and error procedure of adjusting model-input data and model out put so that model results matched field observations within the acceptable level of accuracy. In calibration procedure there are three accepted steps (Anderson and Woessner, 1992):

- To first change the value in cells where the highest deviation occurs
- To change just one parameter in each run
- To determine if any change of that parameter has positive or negative effect in other cells.

7.3.2. Calibration results

The steady state model calibration involved minimizing the difference between the computed and the observed heads. A number of trials were made by changing the transmissivity and recharge parameters. The calibration process was focused on the well field area of the model area since the only available measurements are of the well around the well field are. The simulated versus measured heads for the selected observation points are shown in Figure 7.6. This figure indicates fairly agreement between the observed and calculated hydraulic heads. The average lake level was taken as 1886 m.a.s.l. The model was calibrated to reproduce observed heads by varying transmissivity and recharge.

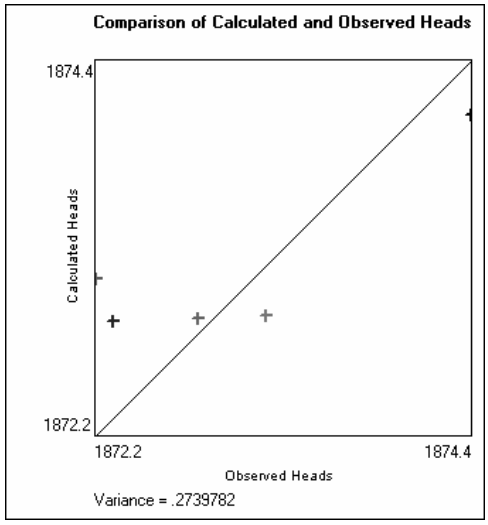


Figure 7.6 a scatter plot of observed and simulated heads.

The calibrated result was evaluated by calculating the difference between the measured and simulated heads using mean error (Me) mean absolute error (Mae) or mean squared error (Rmse). The formulas for these can be written as follows:

$$Me = \frac{1}{n}\sum_{i=1}^{n}\left(h_{cal} - h_{Obs}\right)_i \qquad \textbf{7.1}$$

$$Mae = \frac{1}{n}\sum_{i=1}^{n}\left|\left(h_{cal} - h_{Obs}\right)\right| \qquad \textbf{7.2}$$

$$Rmse = \left[\frac{1}{n}\sum_{i=1}^{n}\left(h_{cal} - h_{Obs}\right)^2{}_i\right]^{0.5} \qquad \textbf{7.3}$$

Where:

h_{cal}	Calculated head
h_{obs}	Observed head
Me	Mean error
Mae	Mean absolute error
Rmse	Root mean square error

The model was considered to be calibrated once the difference between the calculated heads minus the observed heads was in a range of ±2 meters. The observed and calculated heads for the observation well in the panda well field is shown in table7.2.

Code	X-coordinate	Y-coordinate	Observed head (m)	Simulated heads (m)	Difference (m)
C11527	213518	9924827	1874.4	1873.93	-0.47
BH_B	213713	9924977	1871.1	1872.76	1.66
BH_C	213459	9924959	1872.8	1872.49	-0.31
BH_G	213362	9924894	1872.3	1872.49	0.19
BH_H	213397	9924804	1874.5	1872.54	-1.96

Table 7.2 Observed and simulated heads for the observation points

The model accuracy was calculated using Equation 7.1, 7.2 and 7.3. The final model run gave the better results in this stage. At this level the model is considered to be calibrated as long as the simulated heads at the observed heads, the simulation error summary is given in Table 7.3.

Calculated parameter	Value(m)
Mean error	-0.24
Mean absolute error	1.28
Root mean square error	0.98

Table 7.3 Error summary of the calibrated model

7.4. Artificial Recharge and flow model

Based on the steady state calibrated model artificial recharge is simulated using recharge well to evaluate and analyse the effects on the existing groundwater condition of the area, and to estimate the radius of influence and optimum well spacing of the recharge wells.

7.4.1. Recharge wells and groundwater head responses

The groundwater head fluctuation in response to the proposed recharge wells around the panda well field has been evaluated based on the calibrated model. The positions of the productive wells and proposed recharge wells are shown in Figure 7.7.

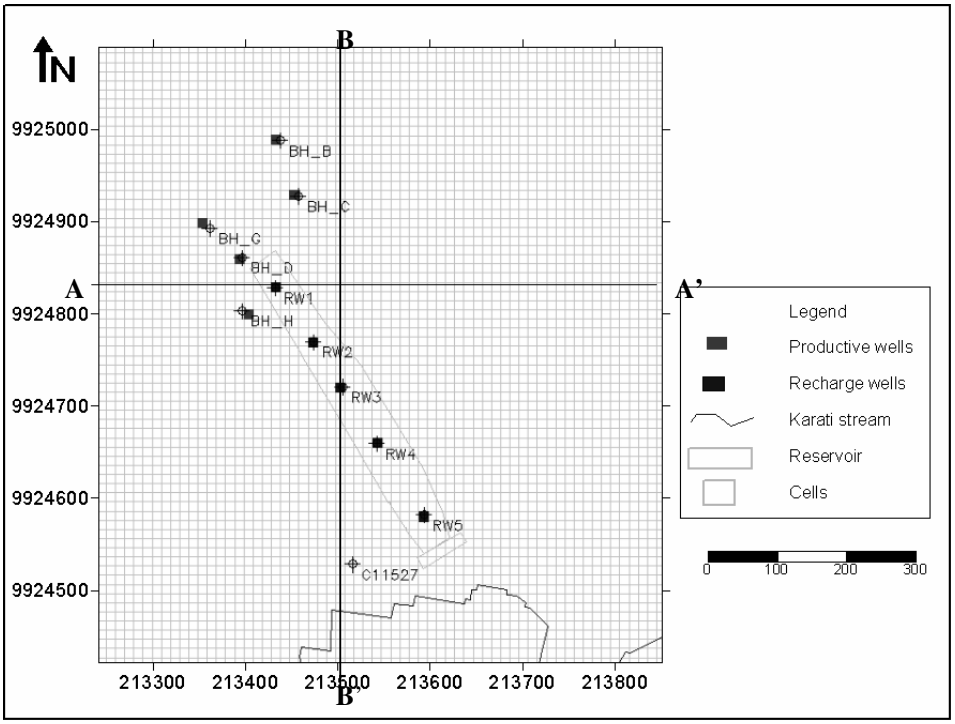


Figure 7.7 Location of the production and proposed recharge wells in the model area

The elevation of the ground surface around the well field area is 1910m a.m.s.l. The water level was analysed under the two pumping scenarios: One with the artificial recharge system fully operational and second with out the artificial recharge system in place. A number of trial and error have been made by changing the recharge volume and spacing of the recharge wells to obtain the optimum hydraulic head. By comparing water levels under the two conditions the water levels have been raised locally by the artificial recharge. Figure 7.8 and Figure 7.9 show water level along the cross section B-B' (North-South) and A-A' (West-East) under the two different scenarios. Water level was raised an average of about 5m at the centre of the recharge well fields as a result of injection of about 1500m^{3}day^{-1} of water from the reservoir. Water levels were raised more on the down gradient of the recharge wells than on the upper gradient side. The summary of the water level changes on the recharge well is shown in Table 7.5.

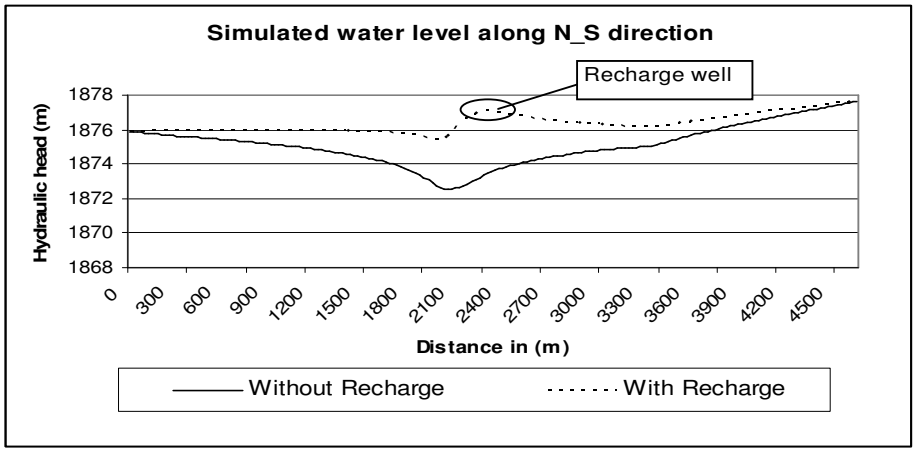


Figure 7.8 Simulated water level along the North-South direction

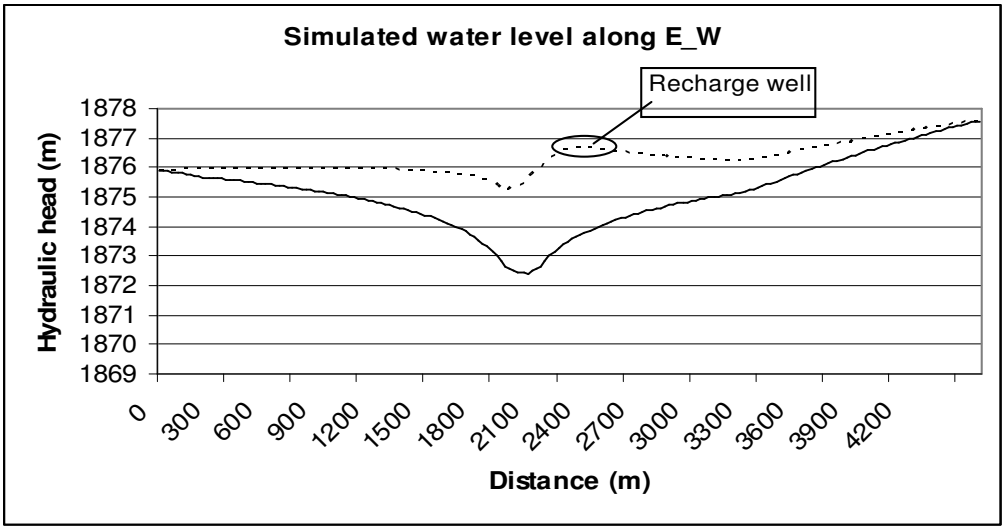


Figure 7.9 Simulated water level along the East-West direction

The water level in the productive wells raises an average of 3.4m. The maximum water level raise was observed on the BH_H and decreases with distance in the others. The results are summarized below showing the measured, simulated head with and without artificial recharge in Table 7.4.

Code	X-coordinate	Y-coordinate	measured head	First scenario simulated heads m	Second scenario Simulated recharge m	Head difference m
BH_B	213713	9924977	1871.1	1873.1	1876.03	2.93
BH_C	213459	9924959	1872.8	1872.9	1876.2	3.3
BH_D	213397	9924832	1876	1872.7	1876.3	3.6
BH_G	213362	9924894	1872.3	1872.8	1876	3.2
BH_H	213397	9924804	1874	1873	1877	4

Table 7.4 Summery the hydraulic heads of the productive wells

Code	X-coordinate	Y-coordinate	First scenario simulated head (m)	Second scenario Simulated head (m)	Head difference (m)
RW1	213423	9924843	1873.5	1878.1	4.6
RW2	213474	9924784	1873.9	1879	5.1
RW3	213521	9924713	1874.3	1879.4	5.1
RW4	213570	9924633	1874.6	1879.6	5
RW5	213604	9924559	1874.9	1879.4	4.5

Table 7.5 Summary of the hydraulic heads of the proposed recharge wells

The difference of the groundwater levels in the two scenarios is drawn in Figure 7.10. The minimum head increased around the boundaries of about 0.4m and maximum of 5.1m on the recharge wells.

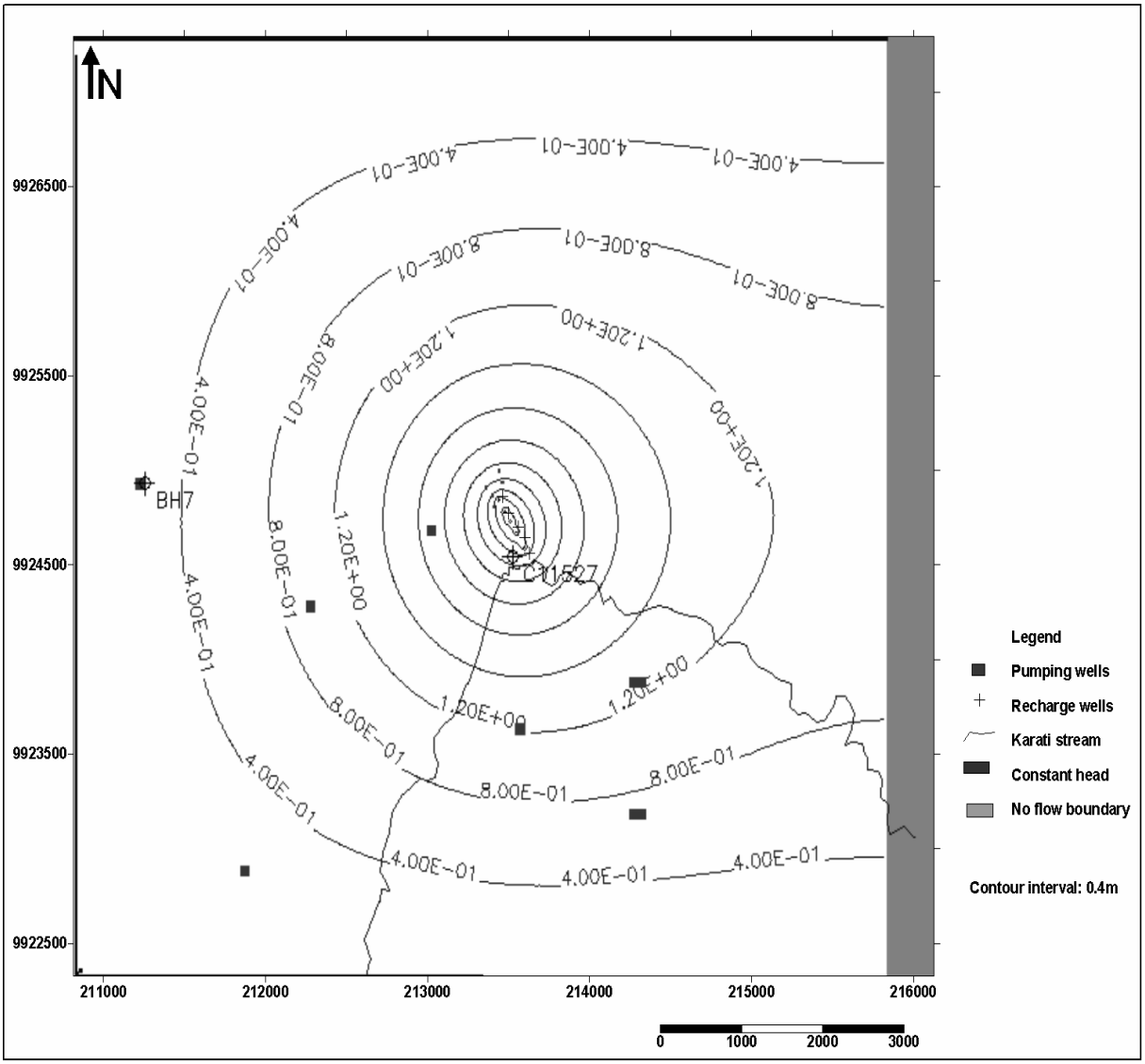


Figure 7.10 Simulated groundwater head difference map

7.4.2. Water balance of the modelled area

The model simulated water budget under two different scenarios is shown in Table 7.6. The first scenario is simulated water budget with out artificial recharge and the second scenario is with artificial recharge. The first scenario indicates that 90% of the inflow is coming to the aquifer through the constant boundary possibly from the lake and in the second scenario the constant head proportion decreases to 54.4 percent due the artificial recharge wells. In the outflow component pumping or abstraction from the aquifer is the major proportion in both scenarios.

Flow components	Without Artificial recharge	% of the total	With Artificial Recharge	% of the total	Remarks
Inflow					
Recharge	1502.67	7.67	1502.67	7.19	
Wells in	411.21	2.10	7911.21	37.87	
Constnt head in	17651.61	90.09	11445.74	54.79	from the lake
River leakage in	28.68	0.15	28.69	0.14	
Total inflow	**19594.1773**		**20888.31**		
Outflows					
Recharge	0	0	0.00	0	
Pumpage	17900.00	91.35	17900.00	85.69	
Constant head out	1694.18	8.65	2988.31	14.31	
River leakage out	0	0	0.00	0	
Total outflow	**19594.1846**		**20888.31**		
Inflow-Outflow	**-0.0073**		**0.00**		

Table 7.6 Water budget of model simulation under two different scenarios

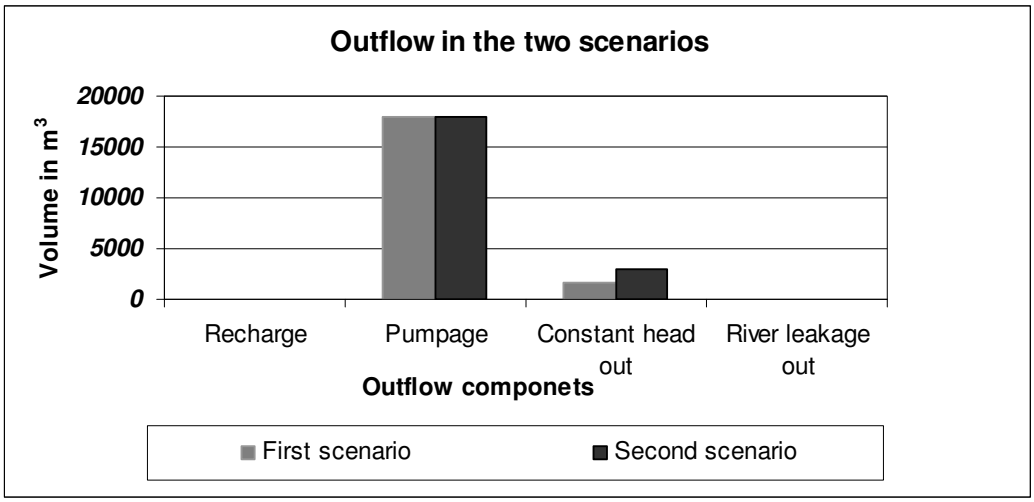


Figure 7.11 Inflow under two scenarios

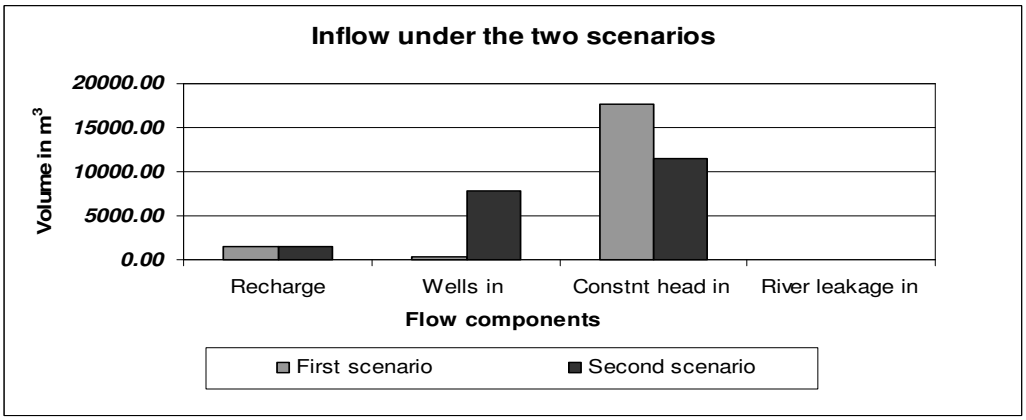


Figure 7.12 Outflow under two different scenarios

8. Conclusion and recommendation

8.1. Conclusion

- The water from the green houses is found to be the first priority source of artificial recharge. This is due to the fact that it is free of contaminates and very low suspended sediments, which in turn plays a great role in the water quality of the recharge water and efficiency of the recharge facilities.
- The two days rainfall and runoff measurement from field data analysis on the main canal results in runoff coefficient of about 0.89 which is comparable to the roofs catchment values in literatures.
- The maximum daily inflow volume estimated from the long term daily rainfall data using rational empirical formula is about 35000m^3 and the minimum is found to be zero.
- The Karati stream channel has a storage potential of about 37000m^3 which is promising to be used as second source of water to recharge the depleted groundwater by drilling recharge wells on the stream bed. However since the recharging site is within the river bed, the condition of sediment transport from the catchments should be studied before using this option.
- The field experiment briefed in chapter 4 reveals the texture variation within the top 30 to 32m unsaturated zone. Besides the injection test results in very low intake rate potential and the hydraulic conductivity is in the order of 0.01 to 0.2$mday^{-1}$. The above findings can lead to conclude infeasibility of shallow infiltration in the study area.
- The shallow aquifer is mainly composed of gravely and silty sand and the transmissivity and specific yield of the aquifer material are ranging between 800 and 1200m^2day^{-1} and 15 to 20%, respectively. Therefore the aquifer storage is adequate to accommodate the anticipated volume of recharge and the transmissivity of the water bearing formation is sufficient to allow extraction of the water from the aquifer.
- The chemical composition of the native groundwater is Na-Hco3 type and an average electrical conductivity of 600μSm^{-1}. Comparing to the standard rainwater composition the groundwater samples shows higher concentration of cations and anions which can be improved by the recharge water.
- A spreadsheet model was used to analyse the effective and cost effective method of artificial recharge. Recharge wells method combined with shallow infiltration basin is found to be feasible method with optimum solution of the model about 7000m^3day^{-1} of total recharge well potential, 89% recharge efficiency and estimated preliminary cost of 0.56KES(Kenyan shillings) per cubic meter of recharge water.
- The water balance of the area was assessed in three scenarios: In natural condition the net amount contributed to the lake was 7mm, in the Green house catchment the net abstraction from groundwater was 994mm and 83%(493mm) of the runoff was evaporated and the contribution to the lake was 100mm. In the third condition Green houses with artificial recharge the net abstraction 550mm and the contribution to the lake is 15mm. Therefore artificial recharge from the green houses runoff to the groundwater reservoir saves 50% the abstraction water.
- The main potential problem of artificial recharge through recharge wells is clogging of the reservoir and/or the recharge well due to the suspended sediments and recharge water quality. Of these two factors, for the present situation suspended sediments are the main source of clogging since the recharge water is direct runoff, free from contaminates.

- The model set up implemented based on the work of Kibona (2000) results in an average water level raise of about 5m underneath the recharge well and 3.5 meters around the existing wells with out over flow.
- The water budget of the steady state model reveals that the inflow through the constant head boundaries to the aquifer decreases from 90% to 40% after the artificial recharge was applied.
- Finally managing groundwater resources using artificial groundwater recharge through the recharge well structure plays a great role in restoring or maintaining the local water levels, increasing groundwater potential in the rapidly declining groundwater, further improving the water quality and it also prevents from direct evaporation.

8.2. Recommendations

- The possible solution to mitigate clogging of the recharge wells and shallow infiltration basin is to design filter below the recharge wells and/or to line the canals collecting the runoff from the green houses or to design stilling basin upstream of the reservoir.
- The main potential problem of karati stream as a second source option is the sediments transport and accumulation from the catchment, which decreases the storage potential as well as the BH recharge capacity. Therefore before realizing this option the watershed area should be studied.
- Further detail study and design of the artificial recharge structures such as weir, retaining walls, canals and related recharge works are essential
- There are many engineering options to place or to put the recharge wells such as to harvest the runoff from each green houses block in small ponds with networks of PVC pipes and then directly recharge to the well. This and other similar options need further consideration in the future.
- Pilot project is very important before full development of the artificial recharge project. This enables collecting of temporal data for the water quality, studying on the compatibility and mixing of the recharge and groundwater in terms of predicting the effects on the water quality improvement and clogging potential. Besides the quantity of the recharge water and the overall long term of performance of recharging facilities can be evaluated.
- To analyze the effects of artificial recharge on the regional groundwater it is important to have monitoring wells during the test program. The duration of the test should be long enough to observe and know the aquifer performance.

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Appendices

Appendix 1 Hydrology

Appendix1.1 Typical runoff coefficients for 5 to 10 years frequency design after (Viessman et al., 1989).

s.n	Description of the area	Runoff coefficient
1	Business	
	Downtown	0.70-0.95
	Neighborhood areas	0.50-0.70
2	Residential	
	Single-family area	0.30-0.50
	Multiunit, detached	0.40-0.60
	Multiunit, attached	0.60-0.75
3	Residential (suburban)	0.25-0.40
4	Apartment dwelling areas	0.50-0.70
5	Industrial	
	Light areas	0.50-0.80
	Heavy rains	0.60-0.90
6	Parks, cemeteries	0.10-0.25
	Playground	0.20-0.35
	Railroad yard areas	0.20-0.40
	Unimproved areas	0.10-0.30
7	Streets	
	Asphalt	0.70-0.95
	Concrete	0.80-0.95
	Brick	0.70-0.85
8	Drives and walks	0.75-0.85
9	Roofs	0.75-0.95
10	Lawns; Sandy soil	
	Flat,2%	0.05-0.10
	Average,2-7%	0.10-0.15
	Steep,7%	0.15-0.20
	Lawns; Heavy soil	
	Flat,2%	0.13-0.17
	Average,2-7%	0.18-0.22
	Steep,7%	0.25-0.35

Appendix1.2 Drainage area of Greenhouses

Blocks	Perimeter, m	Area m^2	Remarks
B5	1481	58011	Bigot flower farm
B6	1326	55945	Bigot flower farm
B7	1194	52545	Bigot flower farm
B8	1028	39619	Bigot flower farm
B9	1464	61526	Panda flower farm
P3	819	32685	Panda flower farm
P4	800	30061	Panda flower farm
P5	806	30698	Panda flower farm
P6	801	31668	Panda flower farm
P7	776	28973	Panda flower farm
P8	1016	42067	Panda flower farm
P9	1003	40758	Panda flower farm
P10	829	31716	Panda flower farm
	Total area	**536272.25**	

Area of the green houses draining in to the main canal

Blocks	Perimeter,m	Area, m^2	Drainge
B1	655	25095	Bigot flower farm
B2	675	26615	Bigot flower farm
B3	418	10923	Bigot flower farm
B4	1135	48031	Panda flower farm
P1	820	32806	Panda flower farm
P11	1027	43768	Panda flower farm
P12	791	30825	Panda flower farm
P2	791	29851	Panda flower farm
	Total area	**247913.6**	

Area of the green houses drain directly to the Karati stream

Appendix 2 Testpits and geological logs

Test pit No 01 Date 15/09/2005 Location 0213540E, 9924606, Elevation 1910m Land use grass land Slope flat

Depth(m)	Description	Remarks
0-1.80	Dark brownish clayey silt It id dry, stiff with shallow cracks and deep grass roots	Lake deposits
1.80-3.20	Diatomite Whitish, Clayey silt texture, dry and soft	Lake deposits
3.20-4.20	Fine to medium sand layer with little volcanic rock fragments which are sub rounded, max size of 10 cm in diameter The sand is loose, pervious	
4.20-4.80	Diatomite with clay texture, medium dense, dry	
4.80-5.00	Dark brownish clay soil It is wet, stiff and highly plastic **Auguring with diameter of 10 cm**	
5.00-6.00	Dark brownish stiff clay continues….	
6.00-6.40	Dark brownish silty clay With little % of fine sand	
6.40-8.00	Decomposed volcanic ashes, Light brownish sandy silt, fine to medium sand, low plastic in contact with water. It is dry and easily break with finger,	
8.00-11.0	Decomposed volcanic ashes, grayish sandy silt Fine to medium sand and poorly graded.	

Logged By: Abdulwab

Test pit No 02 Date 15/09/2005 Location 0213498 E, 9924592N, near the big reservoir
Land use grass land Slope flat

Depth(m)	Description	Remarks
0-2.00	Dark brownish silty clay soil It is dry and stiff with shallow cracks and grass roots Lake deposits of Diatomite	
2.00-2.30	Whitish, clayey silt texture, dry and moderate dense Fine to medium sand layer with little volcanic rock fragments which are sub rounded, max size of 10 cm in diameter The sand is loose, pervious	
2.30-2.80	Diatomite with clayey silt texture, soft, dry	
2.80-4.8	Dark brownish silty clay soil It is wet, stiff and highly plastic	
4.80-5.00	Dark brownish silty clay	

Logged By Abdulwab

Test pit No 03 Date 15/09/2005 Location 0213364 E, 9924896N Land use grass land
Slope flat

Depth(m)	Description	Remarks
0-2.20	Dark brownish silty clay soil It is dry and stiff with shallow cracks and grass roots	Lake deposits
2.20-3.00	Diatomite Whitish, silt texture, dry and moderate dense	
3.00-3.90	Dark brownish clay soil Very stiff and highly plastic in contact with water	
3.90-4.70	Fine to medium sand silt Decomposed volcanic ash With quartz grains	
4.70-5.00	Decomposed of volcanic rocks ashes and pumice Poorly graded sand Greenish color	

Logged By Abdulwab

Test pit No 04 Date 15/09/2005 Location 0213360 E, 9924924N, Land use grass land Slope flat

Depth(m)	Description	Remarks
0-1.10	Dark brownish silty clay soil It is dry, shallow cracks It is stiff With grass roots	
1.10-1.30	Diatomite Whitish, silt texture, dry and moderate dense	Lake deposits
1.30-2.30	Dark brownish stiff clay	
2.30-2.70	Diatomite, whitish and soft	
2.70-4.00	Dark brownish stiff clay	
4.00-5.00	Decomposed volcanic ash Sandy silt textures with little fragments	

Logged By Abdulwab

Test pit No 05 Date 27/09/2005 Location 0213564 E, 9924692N, Elevation 1926m
Land use grass land Slope flat

Depth(m)	Description	Remarks
0-2.7	Light brownish silty clay soil It is dry, fine to medium sand, With grass roots	
2.7-4.00	Decomposed volcanic ashes Grayish, sandy silt Fine sand, poorly graded	
4.00-5.00	Alluvial deposits of gravelly sand Unconsolidated, well graded and rounded	

Logged By_Abdulwab

Test pit No 06 Date 27/10/2005 Location 0213624 E, 9924768N, Elevation 1921m Land use grass land Slope flat

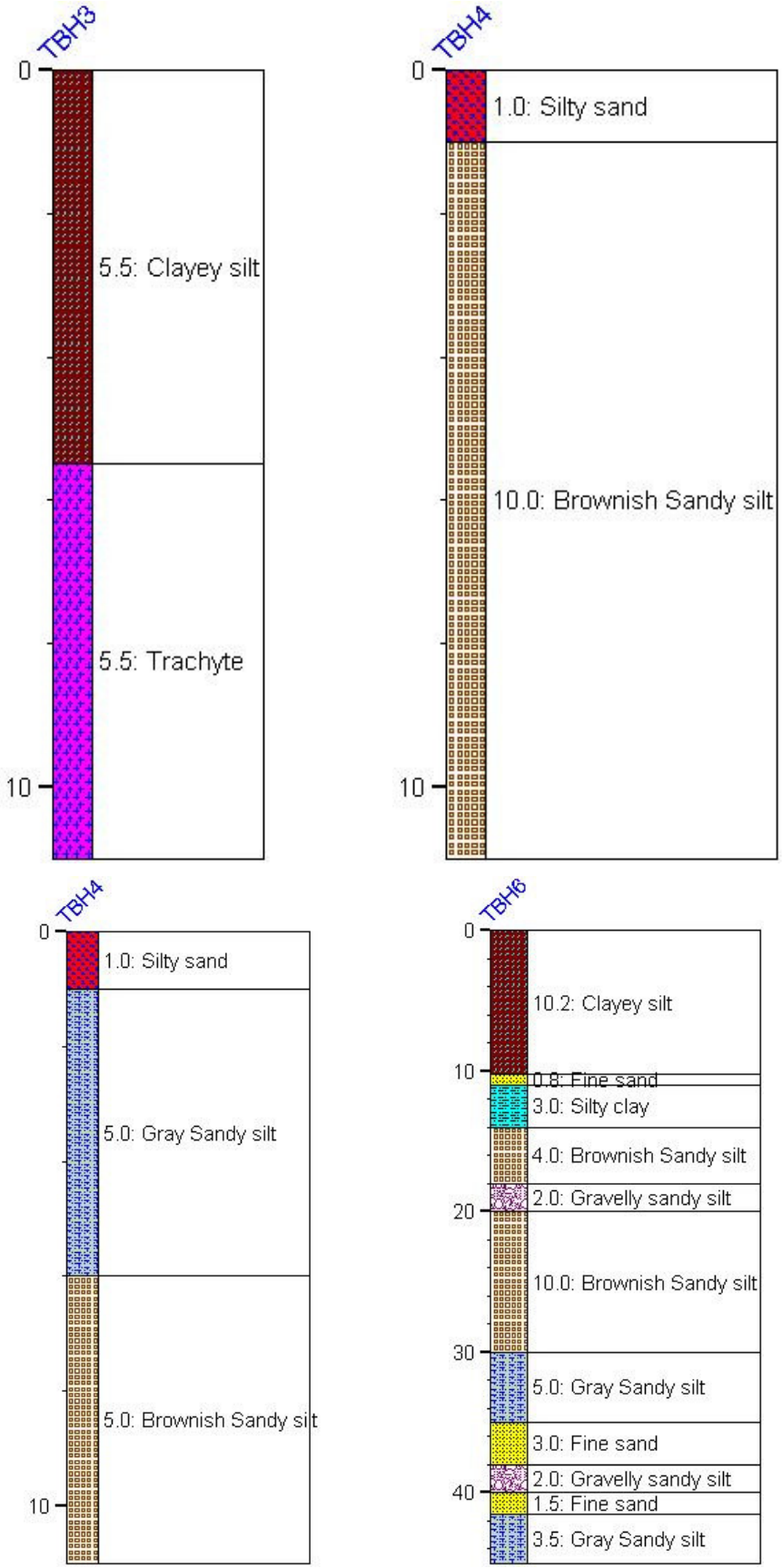
Depth(m)	Description	Remarks
0.00-2.6	Light brownish sandy silt With interbeds of 10cm sand layers	
2.6-3.3	Dark brownish clayey silt + fine sand	
3.3-4.1	Diatomite, whitish and soft Very soft, silt	
4.1-5.00	Dark brownish stiff clay Dry, highly compacted	

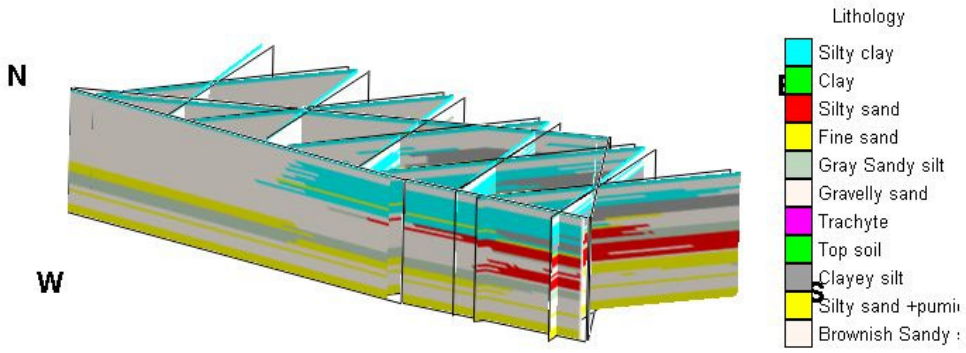
Logged By_Abdulwab

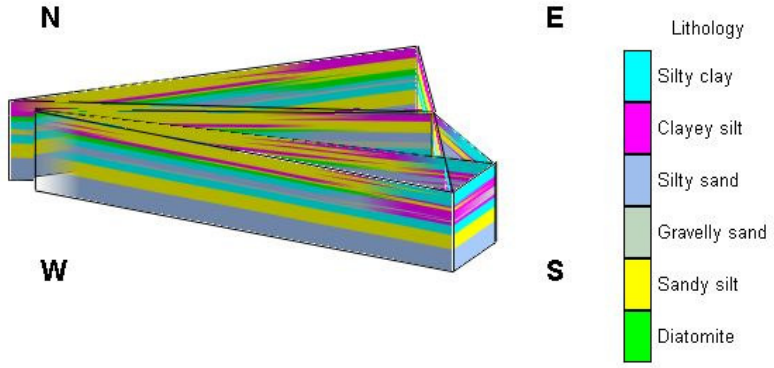
Test pit No 07 Date 27/10/2005 Location 0218853E, 9924914N, Elevation 1917m, Land use grass land Slope flat

Depth(m)	Description	Remarks
0-1.8	Light brownish sandy silt Fine sand	
1.8-2.2	Sandy clay with silt Medium plasticity	
2.2-3.0	Diatomite, whitish and soft	
3.0-3.5	Dark brownish stiff clay	
3.5-5.00	Decomposed volcanic ashes and pumice Medium sand, rounded to sub rounded, With quartz +dark volcanic minerals	

Logged By_Abdulwab


TBH3
0
5.5: Clayey silt
5.5: Trachyte
10
TBH4
0
1.0: Silty sand
10.0: Brownish Sandy silt
10
TBH4
0
1.0: Silty sand
5.0: Gray Sandy silt
5.0: Brownish Sandy silt
10
TBH6
0
10.2: Clayey silt
10
0.8: Fine sand
3.0: Silty clay
4.0: Brownish Sandy silt
2.0: Gravelly sandy silt
20
10.0: Brownish Sandy silt
30
5.0: Gray Sandy silt
3.0: Fine sand
2.0: Gravelly sandy silt
40
1.5: Fine sand
3.5: Gray Sandy silt


N
W
Lithology
Silty clay
Clay
Silty sand
Fine sand
Gray Sandy silt
Gravelly sand
Trachyte
Top soil
Clayey silt
Silty sand +pumi
Brownish Sandy


N
E
W
S
Lithology
Silty clay
Clayey silt
Silty sand
Gravelly sand
Sandy silt
Diatomite

Appendix 3 soil sample analysis, classification and distribution

Appendix 3.1 Grain size analysis results

Sample 1	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.002	3.73	19.13	19.13	100.00
0.02	8.10	41.53	60.66	80.87
0.05	1.91	9.79	70.45	39.34
0.125	0.355	1.82	72.27	29.55
0.25	0.178	0.91	73.18	27.73
0.5	0.427	2.19	75.37	26.82
1	1.272	6.52	81.89	24.63
2	3.533	18.11	100.00	18.11
Total	19.51	100.00		

Sample 2	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.002	2.30	11.38	11.38	100.00
0.02	5.69	28.19	39.57	88.62
0.05	3.68	18.22	57.80	60.43
0.125	1.734	8.59	66.38	42.21
0.25	1.607	7.96	74.34	33.62
0.5	0.565	2.80	77.14	25.66
1	0.454	2.25	79.39	22.86
2	4.162	20.61	100.00	20.61
Total	20.19	100.00		

Sample 3	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.002	2.29	11.60	11.6	100.00
0.02	5.78	29.26	40.86	88.40
0.05	2.37	11.98	52.84	59.14
0.125	1.52	7.71	60.55	47.16
0.25	1.00	5.07	65.62	39.44
0.5	0.88	4.43	70.05	34.38
1	1.13	5.71	75.76	29.95
2	4.79	24.24	100.00	24.24
Total	19.75	100.00		

Sample 4	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.002	2.15	10.78	10.78	100.00
0.02	11.08	55.71	66.49	89.22
0.05	3.67	18.43	84.92	33.51
0.125	0.22	1.11	86.02	15.08
0.25	0.22	1.10	87.12	13.97
0.5	0.18	0.90	88.03	12.88
1	0.20	1.00	89.03	11.97
2	2.18	10.97	100.00	10.97
Total	19.90	100.00		

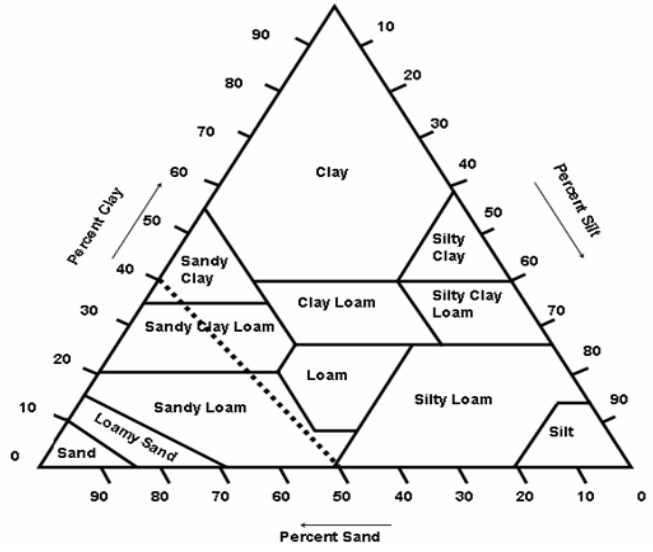
Sample 5	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.002	0.42	2.07	2.07	100.00
0.02	1.27	6.35	8.42	97.93
0.05	0.00	0.00	8.42	91.58
0.125	0.157	0.78	9.20	91.58
0.25	0.192	0.96	10.16	90.79
0.5	1.145	5.71	15.87	89.84
1	7.133	35.57	51.44	84.13
2	9.739	48.56	100.00	48.56
Total	20.06	100.00		

Sample 6	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.00	0.28	1.43	1.43	100.00
0.02	1.19	6.06	7.49	98.57
0.05	1.56	7.97	15.45	92.51
0.125	0.00	0.00	15.45	84.55
0.25	0.02	0.11	15.57	84.55
0.5	0.16	0.82	16.38	84.43
1	3.80	19.38	35.76	83.62
2	12.60	64.24	100.00	64.24
Total	19.62	100.00		

Sample 7	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.00	5.05	25.22	25.22	100.00
0.02	12.15	60.66	85.88	74.78
0.05	0.98	4.90	90.78	14.12
0.125	0.00	0.00	90.78	9.22
0.25	0.07	0.35	91.12	9.22
0.5	0.27	1.33	92.46	8.87
1	0.38	1.90	94.36	7.54
2	1.13	5.64	100.00	5.64
Total	20.03	100.00		

Sample 9	weight passed(g)	% passed	Cumulative % passed	Cumulative % retained
0.002	2.63	13.47	13.47	100.00
0.02	2.34	11.97	25.44	86.53
0.05	0.76	3.91	29.35	74.56
0.125	4.32	22.10	51.45	70.65
0.25	1.8	9.21	60.66	48.55
0.5	2.29	11.72	72.37	39.34
1	3.24	16.58	88.95	27.63
2	2.16	11.05	100.00	11.05
Total	19.55	100.00		

Appendix 3.2 USDA soil classification chart



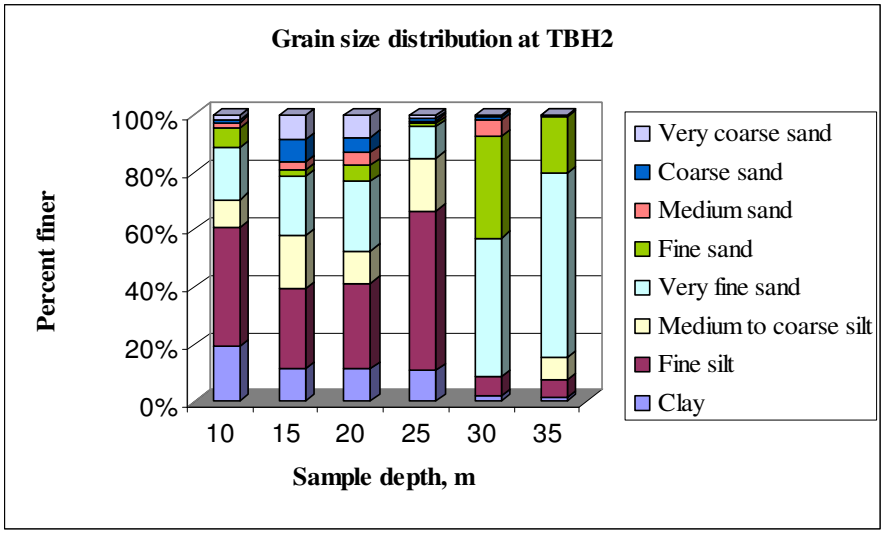
Material	Hydraulic conductivity cm/s
Clay	10^{-9}-10^{-6}
Silt, sandy silt, Clayay sands, till	10^{-6}-10^{-4}
Silty sands, fine sands	10^{-5}-10^{-3}
Well- sorted sands, glacial outwash	10^{-3}-10^{-1}
Well-sorted gravel	10^{-1}-1.00

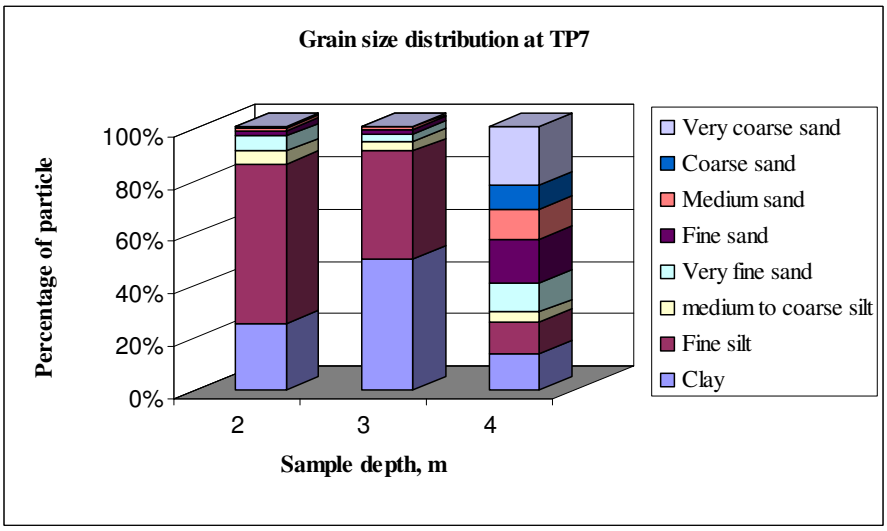
Range of Hydraulic conductivity test for unconsolidated sediments (Fetter, 2001)

Appendix 3.3 Grain size analysis limits

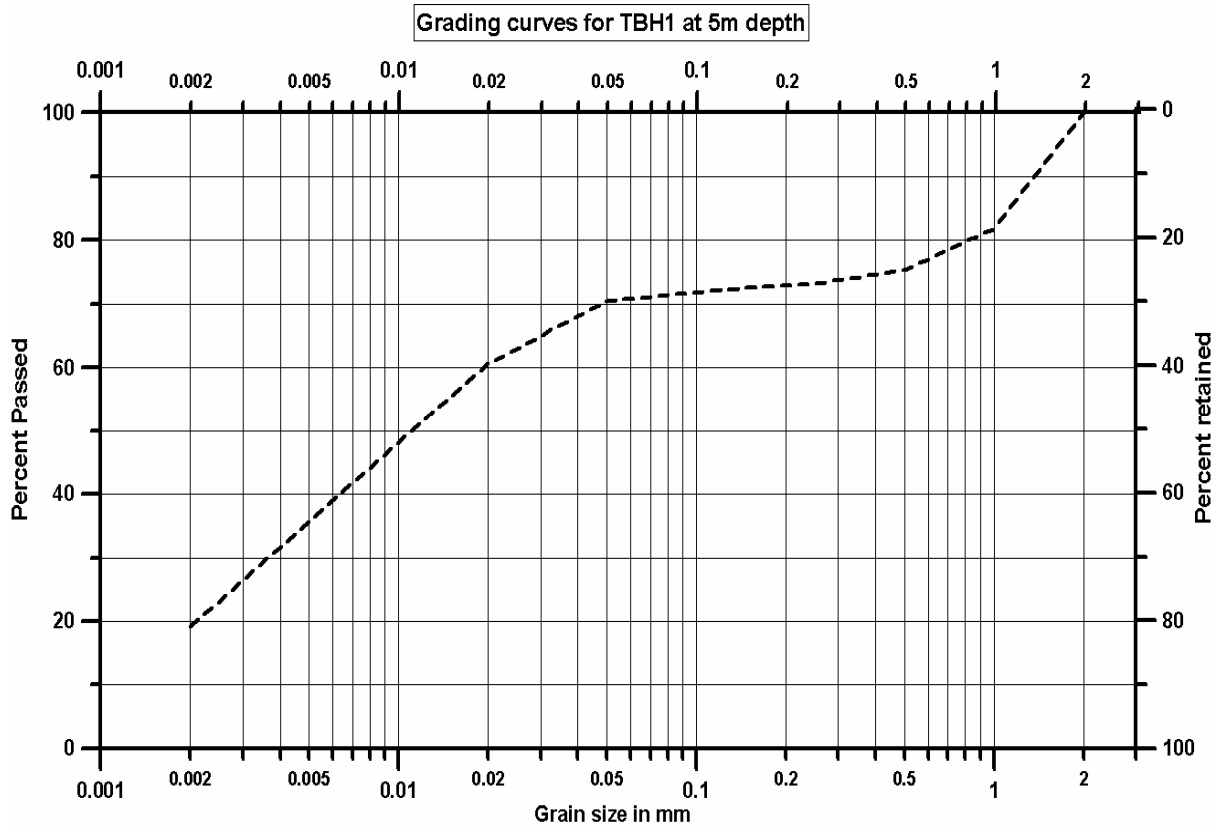
Size in mm	Texture
> 2.0mm	Very Coarse Sand
0.5 - 1.0mm	Coarse Sand
0.25 - 0.5mm	Medium Sand
0.125 - 0.25mm	Fine Sand
0.05 - 0.125mm	Very Fine Sand
0.02 - 0.05mm	Coarse Silt
0.002 - 0.02mm	Medium and Fine Silt
< 0.002mm	Clay

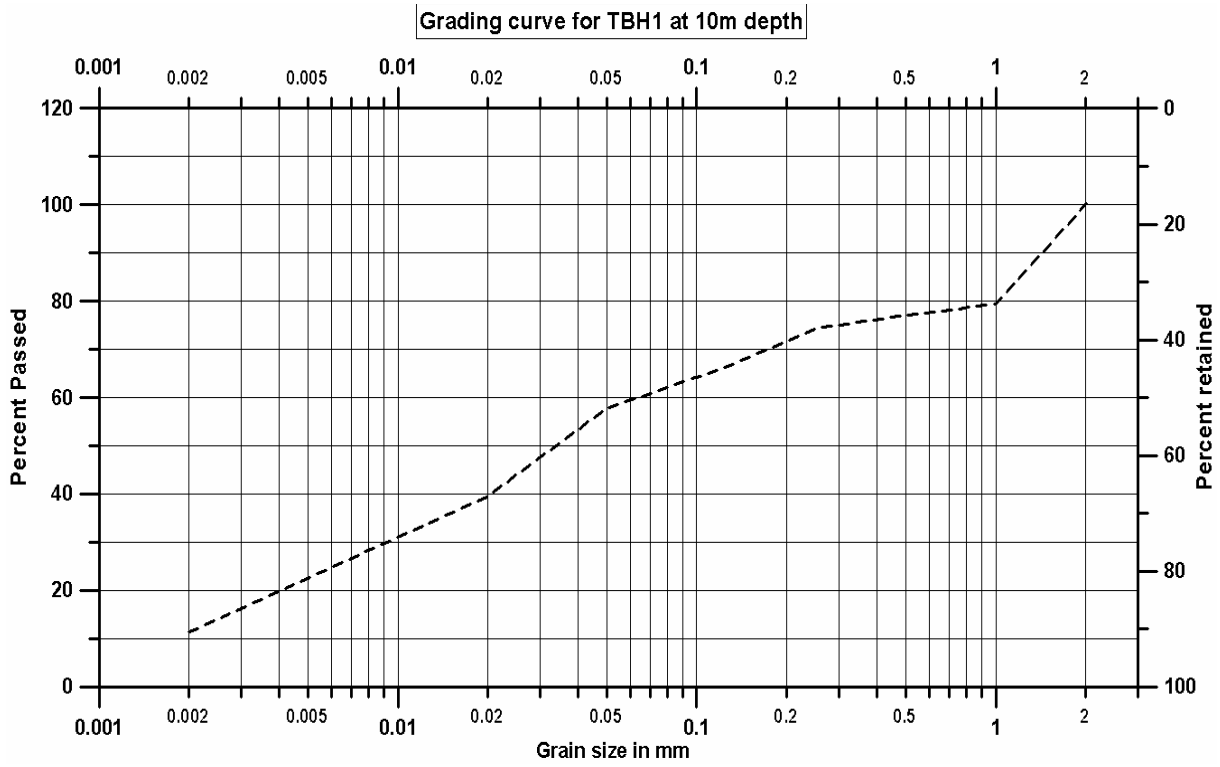
Appendix 3.4 Grains size distribution

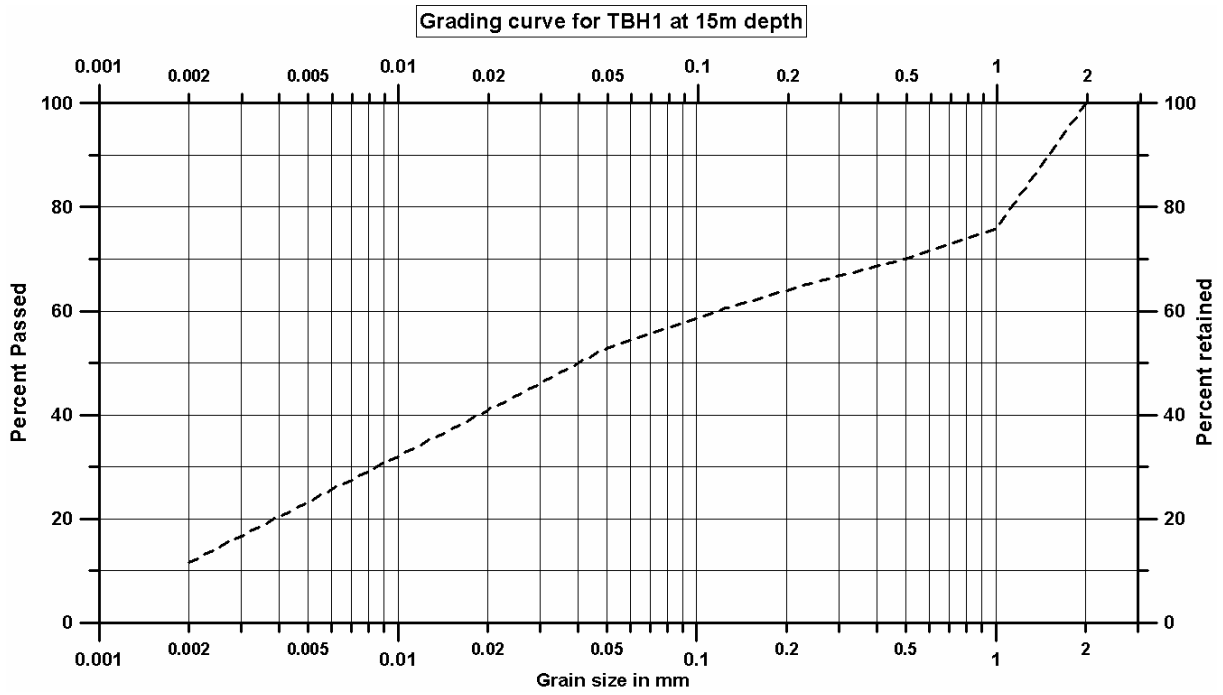


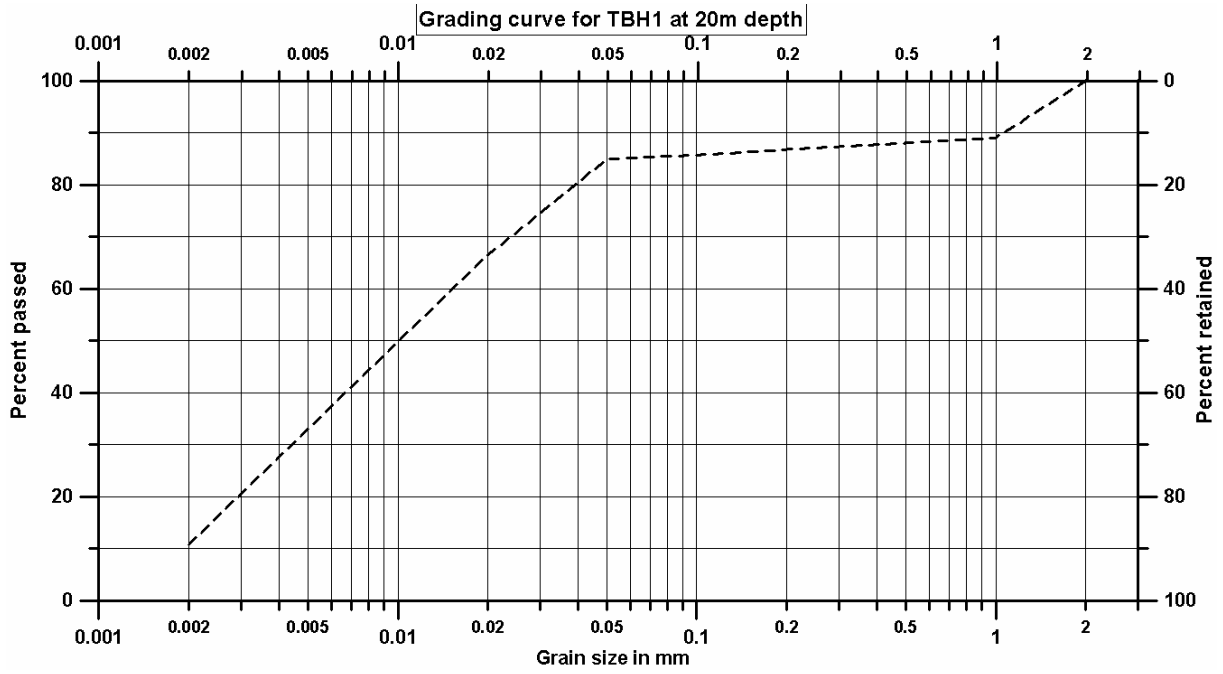


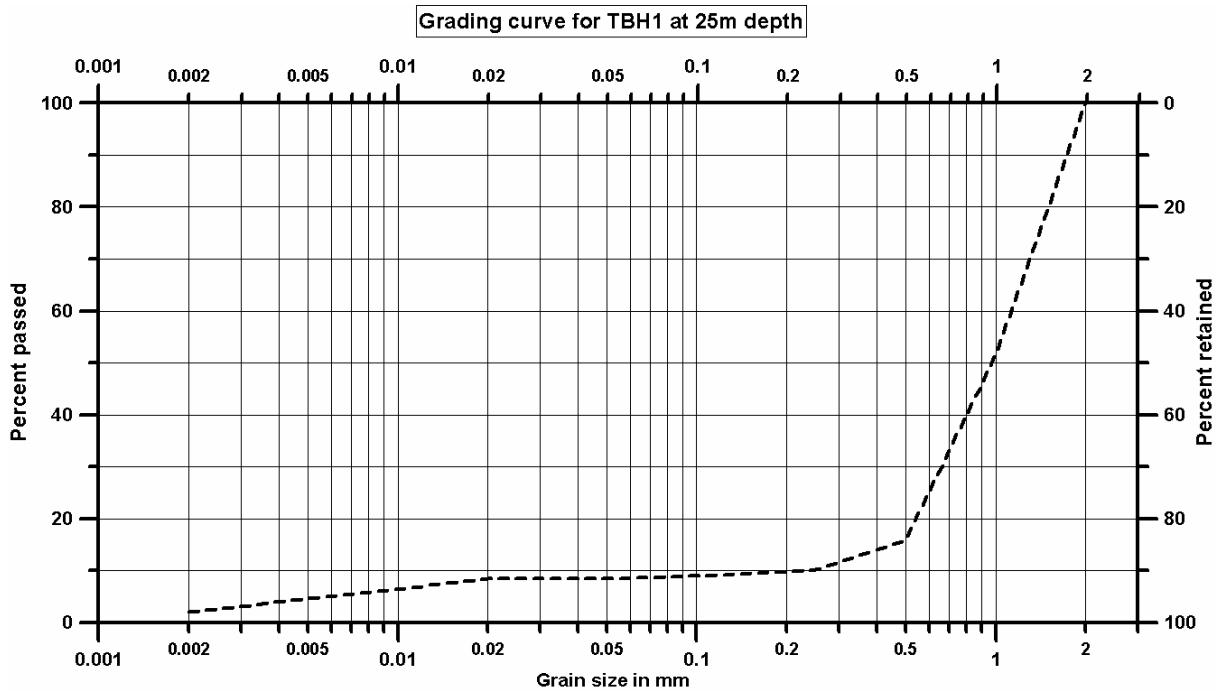
Appendix 3.5 Grading curves

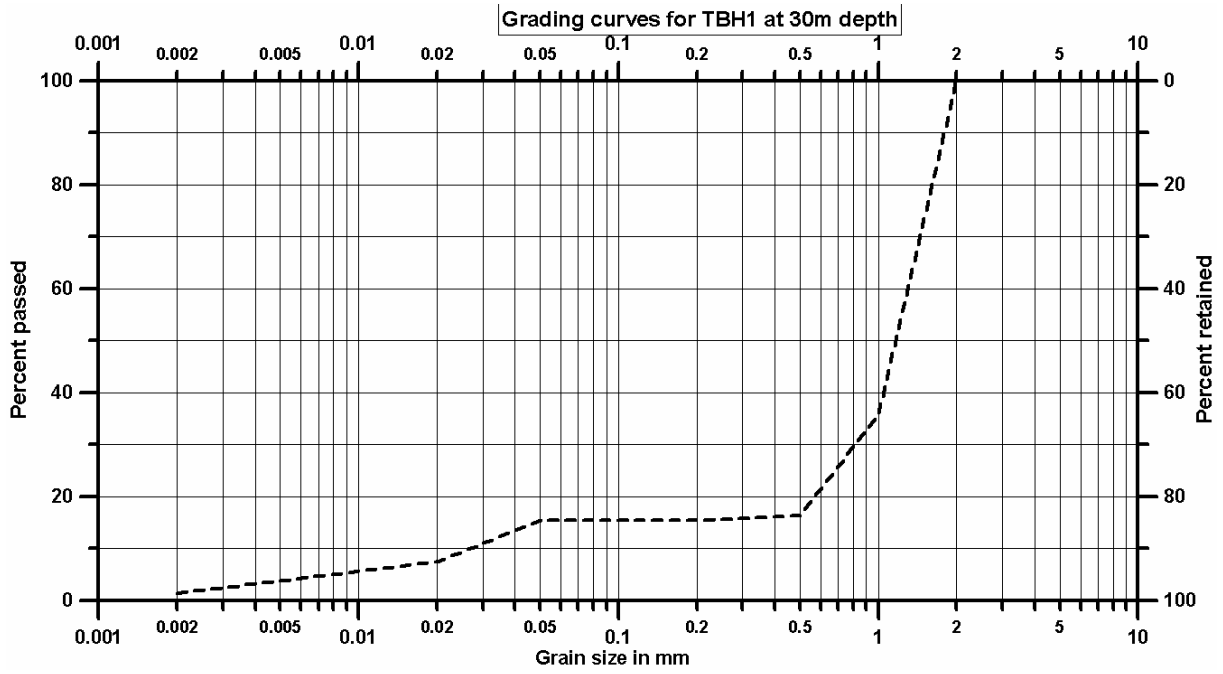


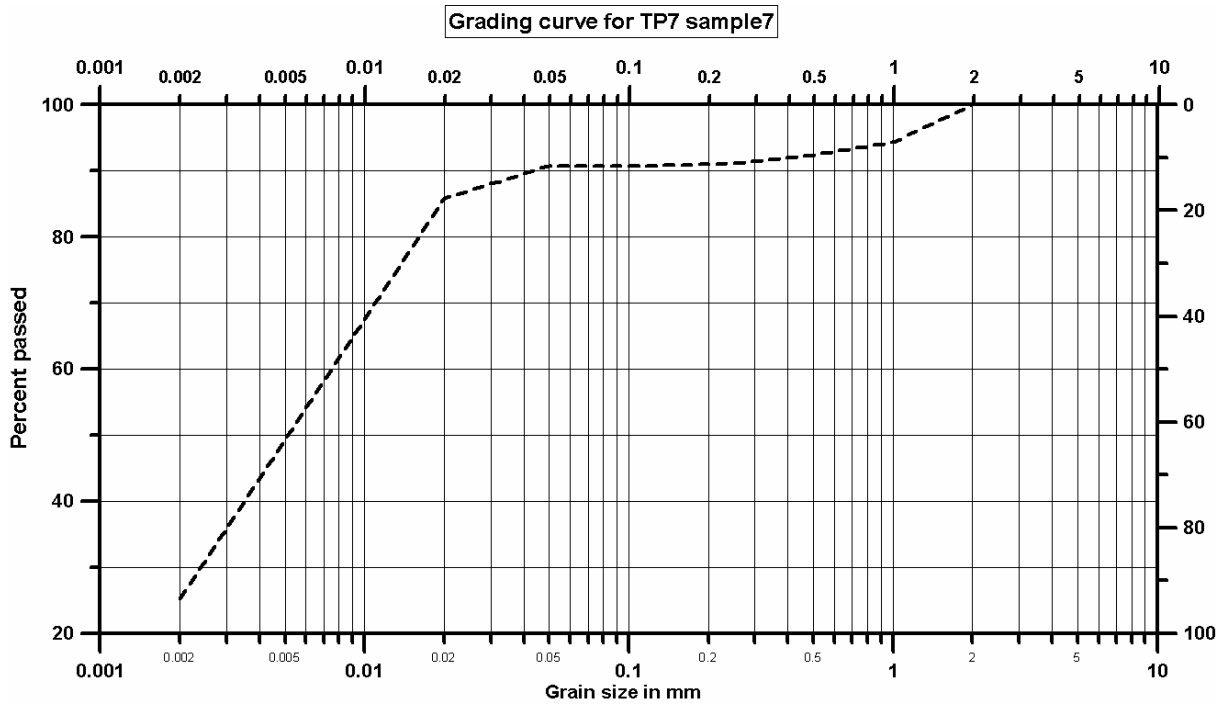


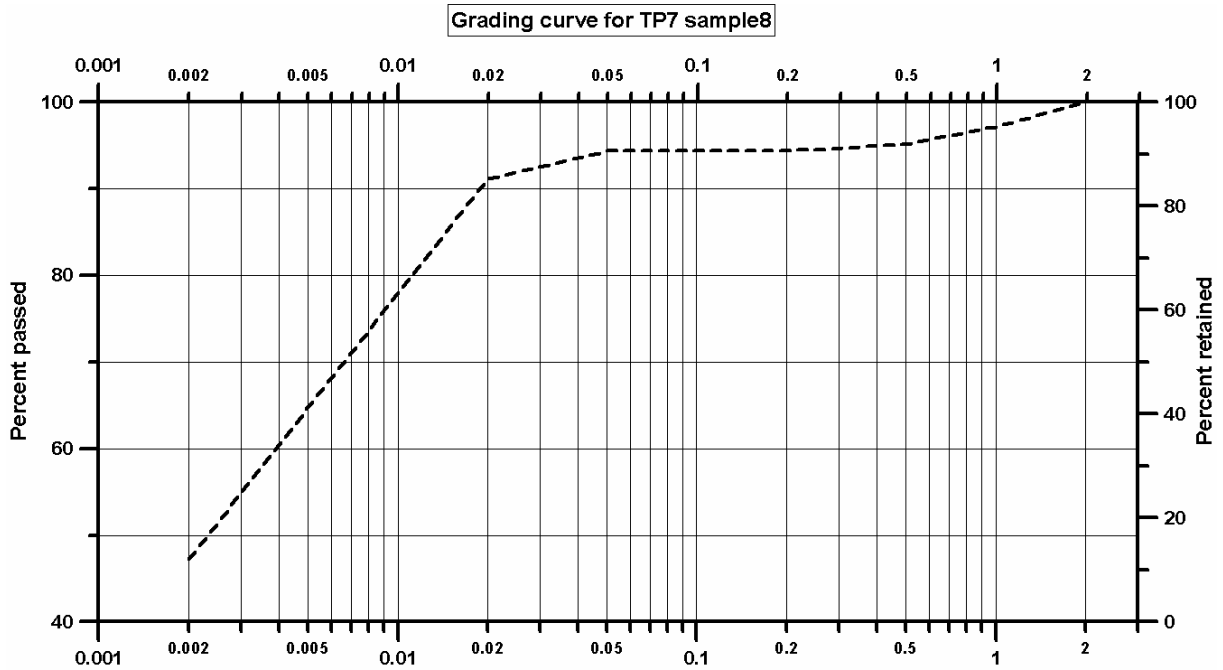

Grading curve for TBH1 at 15m depth
Percent Passed
Percent retained
Grain size in mm
0.001
0.002
0.005
0.01
0.02
0.05
0.1
0.2
0.5
1
2
0
20
40
60
80
100

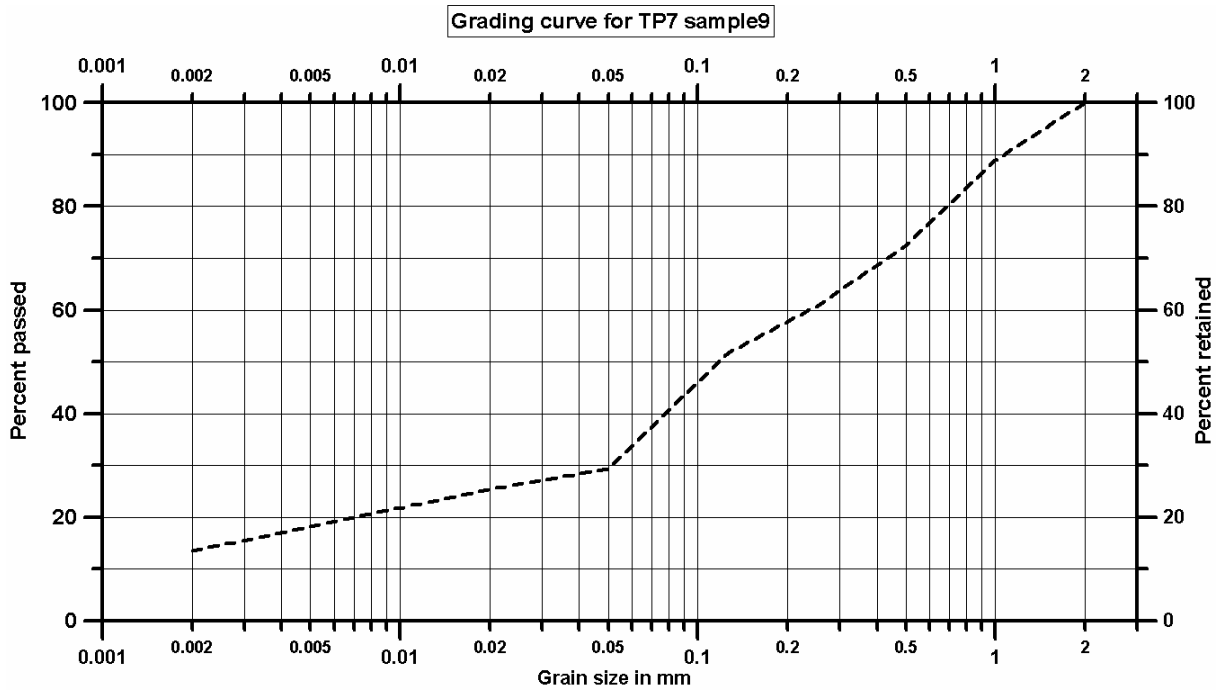

Grading curve for TBH1 at 20m depth
Percent passed
Percent retained
Grain size in mm
0.001
0.002
0.005
0.01
0.02
0.05
0.1
0.2
0.5
1
2
0
20
40
60
80
100


Grading curve for TBH1 at 25m depth
Percent passed
Percent retained
Grain size in mm
0.001
0.002
0.005
0.01
0.02
0.05
0.1
0.2
0.5
1
2
0
20
40
60
80
100

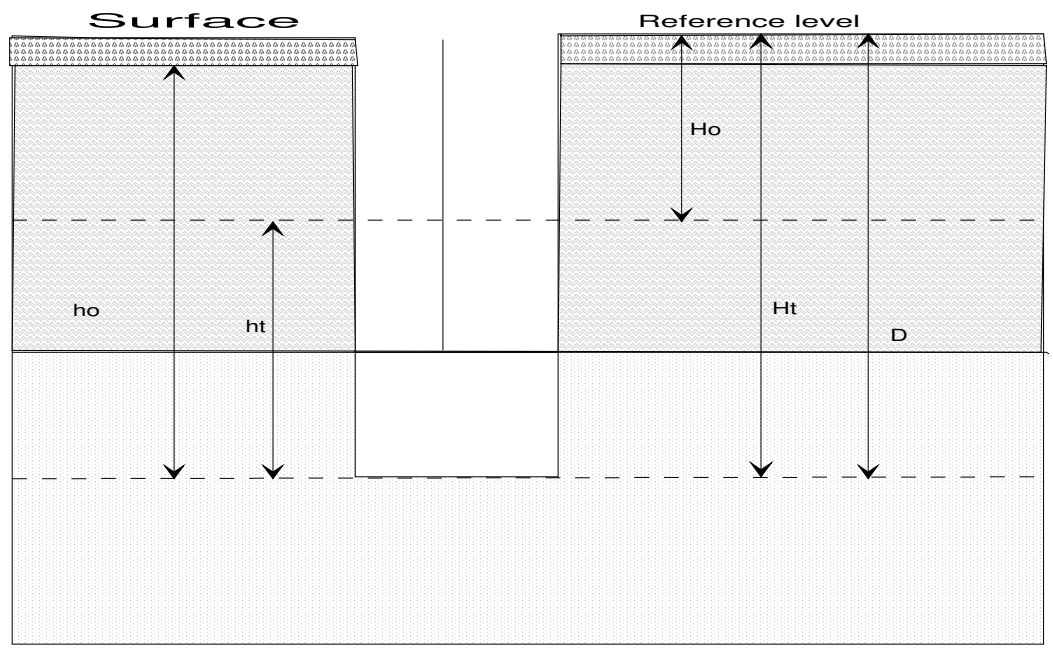

Grading curves for TBH1 at 30m depth
Percent passed
Percent retained
Grain size in mm
0.001
0.002
0.005
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
0
20
40
60
80
100


Grading curve for TP7 sample7
Percent passed
Grain size in mm
0.001
0.002
0.005
0.01
0.02
0.05
0.1
0.2
0.5
1
2
5
10
100
80
60
40
20
0


Grading curve for TP7 sample8
Percent passed
Percent retained
0.001
0.002
0.005
0.01
0.02
0.05
0.1
0.2
0.5
1
2
100
80
60
40
20
0



Appendix 4 Invers Auger hole field data and test results



Where
D: the depth of the hole below reference level (cm)
Ht: the depth of the water level in the hole below reference level (cm)
h_t: the height of the water column in the hole at time t(cm)
h_o: ht at time t=0

Basic principles

For the cylindrical auger hole and its flat base

$$A(ti) = 2\pi r h(ti) + r/2 \text{ ---------- } \mathbf{(0.1)}$$

Where

A(ti) = area through which water passes into the soil at time ti (cm^2)

r = radius of the auger hole (cm)

h(ti) = water-level in the hole at time ti (cm)

Supposing that the hydraulic gradient is approximately one, then according to Darcy's Law the volume rate of flow is given by:

$$Q(ti) = kA(ti) = 2k\pi(h(ti) + r/2) \text{ ---------- } \mathbf{(0.2)}$$

Where Q(ti) = volume rate of flow at time ti (cm^3s^{-1})

If during the time interval (dt) the water-level falls over a distance (dh), the volume rate of flow into the soil equals:

$$Q(ti) = -\pi r^2 \frac{dh}{dt}$$

Combining the last two equations gives: $2k\pi r(h(ti) + r/2) = -\pi r^2 \frac{dh}{dt}$

Integration between the limits
ti = t1, h(ti) = h(t1) and
ti = tn, h(ti) = h(tn)
Gives:
2k/r (tn-t1) = 1n (h(t1) + r/2) – 1n(h(tn) + r/2)
Changing to common logarithms and rearranging gives:

K = 1.15rlog (h(t1) – log(h(tn) + r/2)/ tn – t1

k = 1.15r tan α (cm/day)

By plotting ((hti) + r/2) against t(i) on semi-logarithmic paper, a straight line with a slope α is obtained.

k = 1.15* 864*r*Tan α (m/day)

Where:
r :is the radius of the auger-hole
α : is the slope of the line

INVERS AUGER HOLE METHOD

5m depth UTM X: 213517E

Site: Panda flower farm Ker UTM Y: 9924634N

Lithology Silty clay ,lake sediment

UTM Zone: 37

Hole diam. [cm]: 40

Hole depth [cm]: 500

point-surface [cm]: 0

Time	Level	(h+r/2)	log(h+r/2)
[sec]	[cm]	[cm]	log[cm]
600	32	473	2.675
1200	51	454	2.657
1800	62	443	2.646
2400	71	434	2.637
3000	81	424	2.627
3600	91	414	2.617

10m depth

Site: Panda flower farm

Lithology Sandy silt

Hole diam. [cm]: 40

Hole depth [cm]: 1000

point-surface [cm]: 487

Time	Level	(h+r/2)	log(h+r/2)
[sec]	[cm]	[cm]	log[cm]
600	488	1004	3.002
1200	489	1003	3.001
1800	490	1002	3.001
2400	491	1001	3.000
3000	492	1000	3.000
3600	493	999	3.000

15m depth

Site: Panda flower farm Kenya Naivasha

Lithology Silty sand

Hole diam. [cm]: 40

Hole depth [cm]: 1500

point-surface [cm]: 908

Time	Level	(h+r/2)	log(h+r/2)
[sec]	[cm]	[cm]	log[cm]
600	915	1498	3.176
1200	922	1491	3.173
1800	929	1484	3.171
2400	934	1479	3.170
3000	938	1475	3.169
3600	942	1471	3.168

20m depth

Site: Panda flower farm Kenya Naivasha

Lithology Sandy silt

Hole diam. [cm]: 40

Hole depth [cm]: 2000

point-surface [cm]: 1428

Time	Level	(h+r/2)	log(h+r/2)
[sec]	[cm]	[cm]	log[cm]
600	1433	2000	3.301
1200	1438	1995	3.300
1800	1443	1990	3.299
2400	1447	1986	3.298
3000	1451	1982	3.297
3600	1455	1978	3.296

25m depth

Site: Panda flower farm Kenya Naivasha

Lithology Silty sand

Hole diam. [cm]: 40

Hole depth [cm]: 2500

point-surface [cm]: 1998

Time	Level	(h+r/2)	log(h+r/2)
[sec]	[cm]	[cm]	log[cm]
600	1911	2592	3.414
1200	1865	2638	3.421
1800	1810	2693	3.430
2400	1762	2741	3.438
3000	1725	2778	3.444
3600	1697	2806	3.448

30m depth

Site: Panda flower farm Kenya Naivasha

Lithology Sandy silt

Hole diam. [cm]: 40

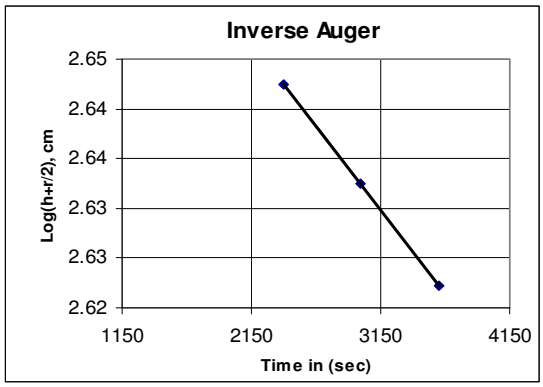
Hole depth [cm]: 3000

point-surface [cm]: 2428

Time	Level	(h+r/2)	log(h+r/2)
[sec]	[cm]	[cm]	log[cm]
600	2718	2715	3.434
1200	2721	2712	3.433
1800	2719	2714	3.434
2400	2716	2717	3.434
3000	2713	2720	3.435
3600	2710	2723	3.435

Inverse auger hole graph at 5m depth

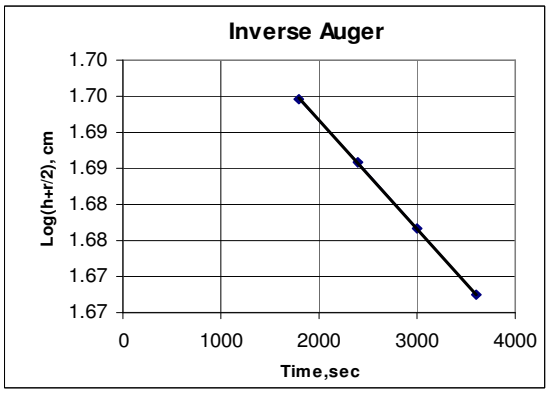
Location: (213517, 9924634)
Depth of the hole: 5m
Radius of the Hole: 0.20m



Equation:
Y = -0.00002x + 2.6785
k = 0.2mday^{-1}

Inverse auger hole graph at 10m depth

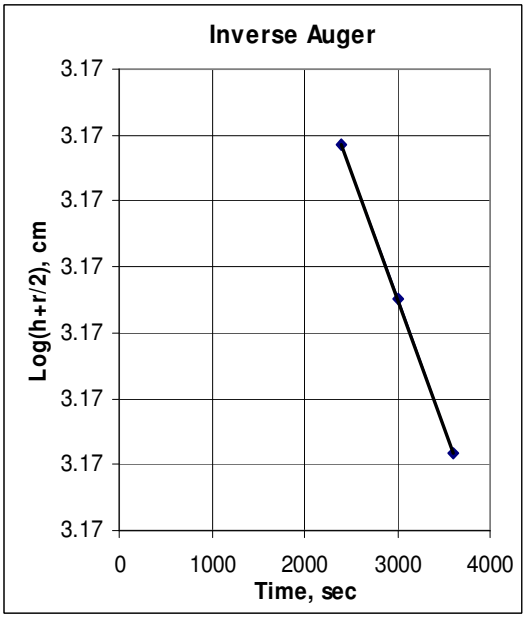
Depth of the hole: 10m
Radius of the Hole: 0.20m



Equation:
y = -0.000007x + 3.0022
k = 0.02mday^{-1}

Inverse auger hole graph at 15m depth

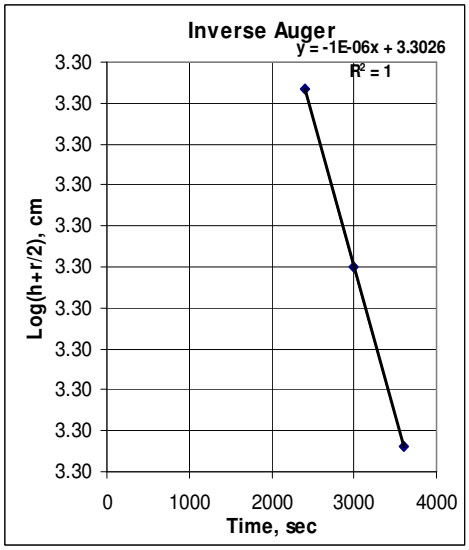
Depth of the hole: 15m
Radius of the Hole: 0.20m



Y=-0.00002x + 3.1751
K=0.04mday^{-1}

Inverse auger hole graph at 20m depth

Depth of the hole: 20m
Radius of the Hole: 0.20m

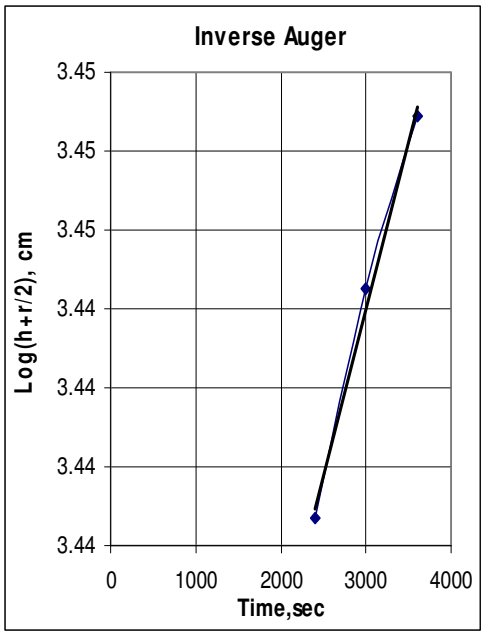


y = -0.00005x + 3.3015
K=0.02mday^{-1}

Inverse auger hole graph at 25m depth

Depth of the hole: 25m

Radius of the Hole: 0.20m



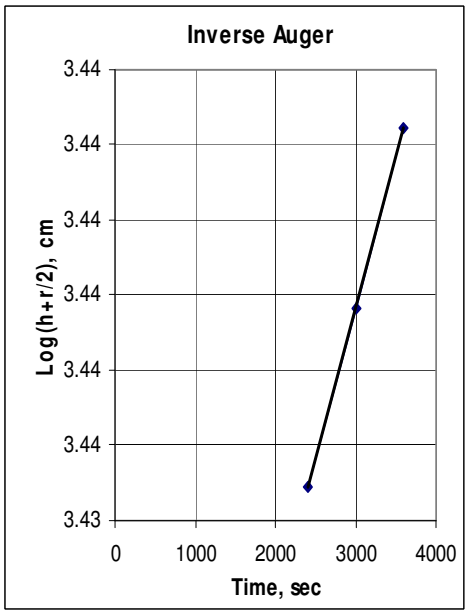
y = -0.00005x + 3.4178

K=0.16mday^{-1}

Inverse auger hole graph at 30m depth

Depth of the hole: 30m

Radius of the Hole: 0.20m



Y=-0.000008x + 3.4322

K=0.09mday^{-1}

Appendix 5 Pumping tests

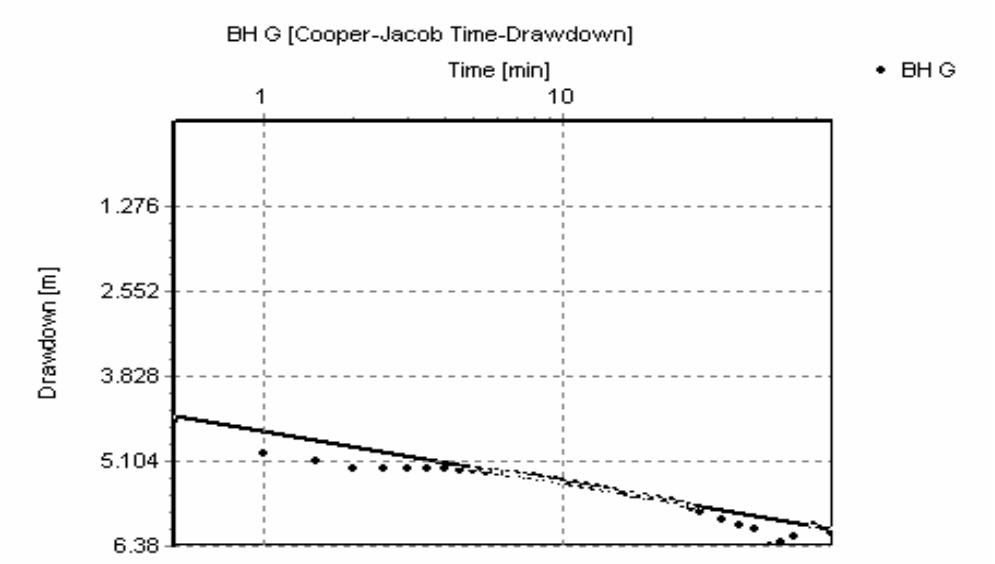
BH G

Three point farm p.o.box 884,Navaisha

Location	213359E
Coordinates	213359E,9924892N)
Radius of the Borehole	0.15m
Depth	60m
Pumping well	
Pump intake at	46.5m
Date	7/10/2005, Time start 4:10:00 p.m Time stop 5:29:00
Stataic water level	40.08
Final water level	46.55
Method of measuring Wate level	Electric Dipper
Disharge of the well	144 m^3/hr
Pumping duration	72 minutes

Draw down measument

Time hour	min	Depth to water level	DD
4:10:00	0	40.08	40.08
4:10:30	0.5	44.6	44.6
4:11:00	1	45.08	45.08
4:11:30	1.5	45.17	45.17
4:12:00	2	45.29	45.29
4:12:30	2.5	45.29	45.29
4:13:00	3	45.29	45.29
4:13:30	3.5	45.29	45.29
4:14:00	4	45.3	45.3
4:14:30	4.5	45.32	45.32
4:15:00	5	45.33	45.33
4:15:30	5.5	45.35	45.35
4:16:00	6	45.36	45.36
4:16:30	6.5	45.37	45.37
4:17:00	7	45.38	45.38
4:17:30	7.5	45.4	45.4
4:18:00	8	45.42	45.42
4:18:30	8.5	45.44	45.44
4:19:00	9	45.46	45.46
4:19:30	9.5	45.48	45.48
4:20:00	10	45.5	45.5
4:21:00	11	45.53	45.53
4:22:00	12	45.56	45.56
4:23:00	13	45.58	45.58
4:24:00	14	45.62	45.62
4:25:00	15	45.64	45.64
4:27:00	17	45.69	45.69
4:29:00	19	45.72	45.72
4:31:00	21	45.75	45.75
4:33:00	23	45.79	45.79
4:35:00	25	45.84	45.84
4:37:00	27	45.89	45.89
4:39:00	29	45.94	45.94
4:44:00	34	46.05	46.05
4:49:00	39	46.14	46.14
4:54:00	44	46.21	46.21
4:59:00	49	46.46	46.46
5:04:00	54	46.4	46.4
5:09:00	59	46.31	46.31
5:19:00	69	46.15	46.15
5:29:00	79	46.25	46.25



Pumping test result for BH_G

Appendix 6 Injection tests

TBH1

Date 27/09/2005

Location 213517E 9924634N

BH depth 27

Depth to water level 25.4

The borehole is filled up from 30 to 27m with silty clay soil

Pumping rate 80m3/hr

Formation Silty sand

Time in houre	Time in minute	Depth to water level	Injection head	Time in hou	Time in minute	Depth to water level	Recovery
16:25	0	25.4	0	16:37	0	1	0
16:26	1	3.1	22.3	16:38	1	1.85	-0.85
16:27	2	3.03	0.07	16:39	2	2.32	-0.47
16:28	3	3	0.03	16:40	3	4.25	-1.93
16:29	4	2.7	0.3	16:41	4	8.85	-4.6
16:30	5	3	-0.3	16:42	5	12.45	-3.6
16:31	6	2.5	0.5	16:43	6	14.13	-1.68
16:32	7	2.4	0.1	16:44	7	16.55	-2.42
16:33	8	2.35	0.05	16:45	8	18.15	-1.6
16:34	9	2.5	-0.15	16:46	9	19.66	-1.51
16:35	10	2.15	0.35	16:47	10	20.8	-1.14
16:36	11	1.15	1	16:48	11	21.8	-1
16:37	12	1.13	0.02	16:49	12	21.75	0.05
16:38	13	1.12	0.01	16:50	13	22.7	-0.95
16:39	14	1.11	0.01	16:51	14	23.1	-0.4
16:40	15	1.12	-0.01	16:52	15	23	0.1

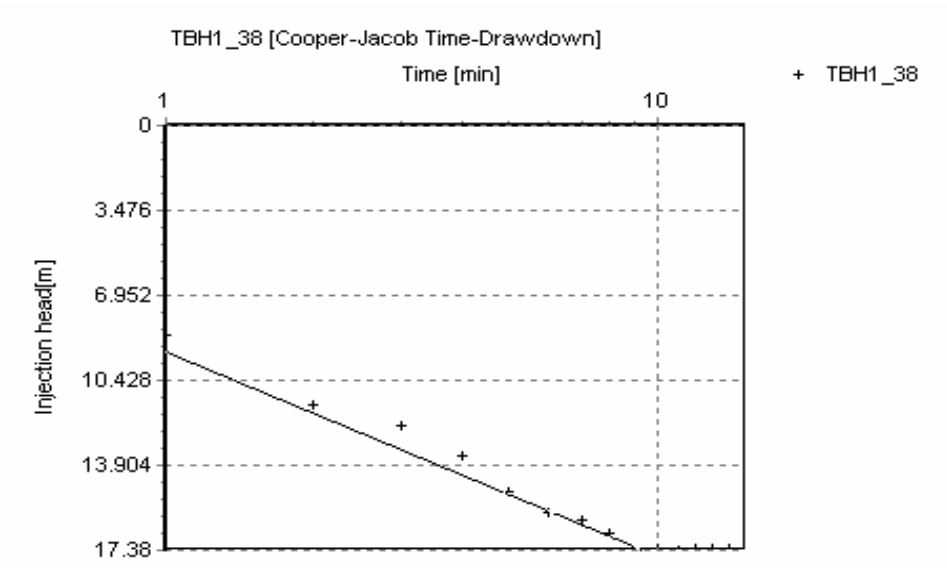
TBH1
Date 26/09/2005
Location 213517E 9924634N
BH depth 30
Depth to water level 29.55
The borehole is filled up from 35 to 30m with silty clay soil
Pumping rate 80m3/hr

Time in houre	Time in minute	Depth to water level	Injection head(m)	Time in houre	Time in minute	Depth to water level (m)	Recovery (m)
14:15	0	29.55	0	14:31	0	7.65	0
14:16	1	20.3	9.25	14:32	1	17.7	10.05
14:17	2	14.65	5.65	14:33	2	22.75	5.05
14:18	3	12.1	2.55	14:34	3	24.5	1.75
14:19	4	10.35	1.75	14:35	4	25.2	0.7
14:20	5	9.44	0.91	14:36	5	25.45	0.25
14:21	6	8.8	0.64	14:37	6	25.65	0.2
14:22	7	8.25	0.55	14:38	7	25.83	0.18
14:23	8	7.7	0.55	14:39	8	26.02	0.19
14:24	9	7.3	0.4	14:40	9	26.38	0.36
14:25	10	8.6	-1.3	14:41	10	26.68	0.3
14:26	11	7.95	0.65	14:42	11	26.86	0.18
14:27	12	8.25	-0.3	14:43	12	27.03	0.17
14:28	13	7.6	0.65	14:44	13	27.15	0.12
14:29	14	7.55	0.05	14:45	14	27.25	0.1
14:30	15	7.65	-0.1	14:46	15	27.35	0.1

TBH38 38m depth

Date	21/09/2005		SWL	32.2
Location	213517E	9924634N		
BH depth	38m	41m drilled but collapsed		
water depth	31.7m			
Recharge rate	80 m3/hr	for 15 minutes		
final water level	15.49	15.04		

Time in houre	Time interval in minute	Depth(m)	Injection head(m)	Time in houre	Time interval in minute	Depth of water, m	Recovery injection head,m
12:30:00	0	31.7	0	12:41:00	0	14.4	14.4
12:31:00	1	23.1	8.6	12:42:00	1	15.4	
12:32:00	2	20.3	11.4	12:43:00	2	16.43	
12:33:00	3	19.4	12.3	12:44:00	3	17.67	
12:34:00	4	18.21	13.49	12:45:00	4	18.3	
12:35:00	5	16.69	15.01	12:46:00	5	18.66	
12:36:00	6	15.87	15.83	12:47:00	6	18.94	
12:37:00	7	15.53	16.17	12:48:00	7	19.17	
12:38:00	8	15	16.7	12:49:00	8	19.35	
12:39:00	9	14.32	17.38	12:50:00	9	19.46	
12:40:00	10	14.4	17.3	12:51:00	10	19.55	
12:41:00	11	14.32	17.38	12:52:00	11	19.65	
12:42:00	12	14.36	17.34	12:53:00	12	19.72	
12:43:00	13	14.4	17.3	12:54:00	13	19.79	
12:44:00	14	14.38	17.32	12:55:00	14	19.91	
12:45:00	15	14.35	17.35	12:56:00	15	20	
				12:57:00	16	20.11	
				12:58:00	17	20.22	
				12:59:00	18	20.3	
				13:00:00	19	20.37	
				13:01:00	20	20.45	
				13:02:00	21	20.51	
				13:03:00	22	20.58	
				13:04:00	23	20.64	
				13:05:00	24	20.7	
				13:06:00	25	20.76	
				13:07:00	26	20.81	
				13:08:00	27	20.86	
				13:09:00	28	20.92	
				13:10:00	29	20.97	
				13:11:00	30	21	
				13:12:00	31	21.06	
				13:13:00	32	21.11	
				13:14:00	33	21.18	
				13:15:00	34	21.22	
				13:16:00	35	21.25	
				13:17:00	36	21.28	
				13:18:00	37	21.31	
				13:19:00	38	21.34	
				13:20:00	39	21.37	
				13:21:00	40	21.4	
				13:22:00	41	21.43	
				13:23:00	42	21.45	
				13:24:00	43	21.47	
				13:25:00	44	21.5	
				13:26:00	45	21.52	
				13:27:00	46	21.55	
				13:28:00	47	21.58	
				13:29:00	48	21.6	
				13:30:00	49	21.62	
				13:31:00	50	21.64	
				13:32:00	51	21.66	
				13:33:00	52	21.68	
				13:34:00	53	21.7	
				13:35:00	54	21.72	
				13:36:00	55	21.73	
				13:37:00	56	21.75	

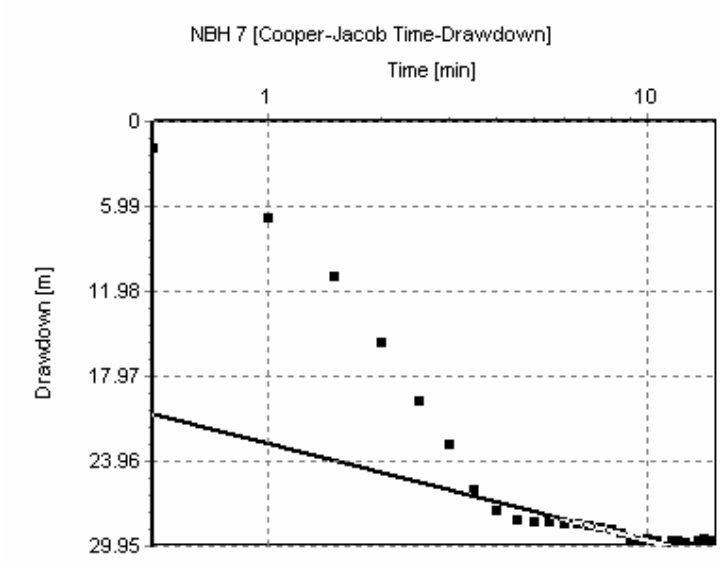


Pumping test result for TBH1 at 38m depth

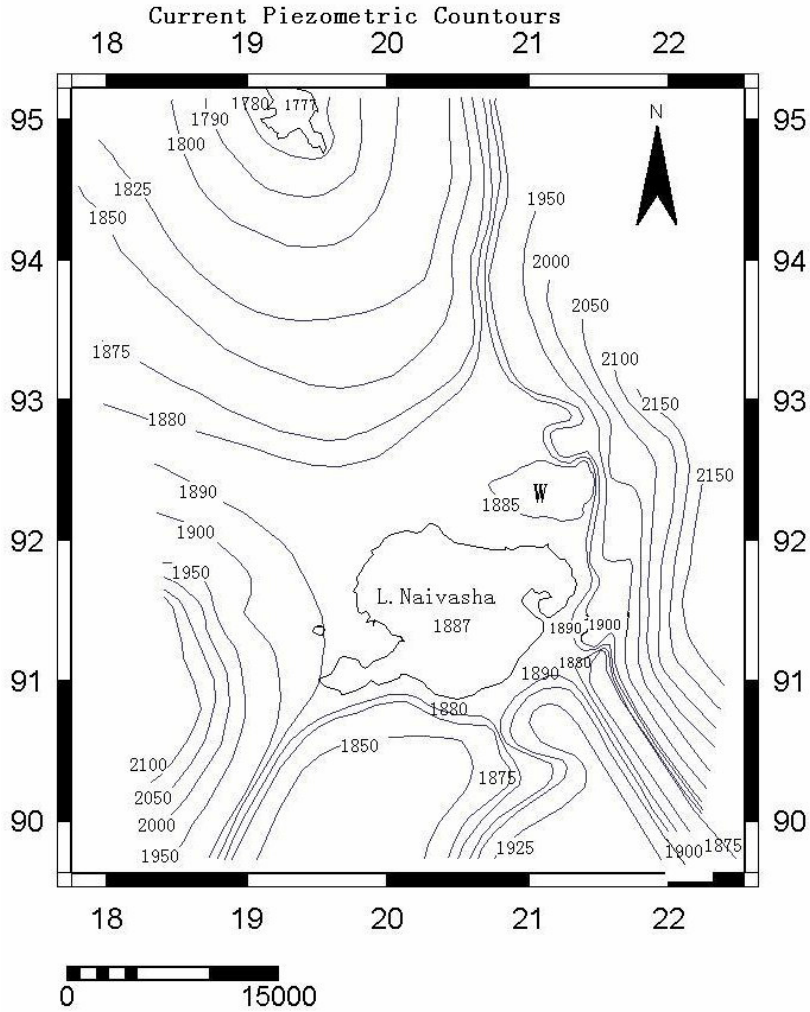
INJECTION TEST NEWBH7

Location	213413E	9924772N	
SWL	37.1	m	
Depth of water level during the test	33.7	m	
BH depth	70	m	
water depth	33.7	m	
Injection rate		80m3/hr	Recovery
Date		21/09/2005	

Time in houre	Time in minute	Depth to water level(m)	Injection head(m)	Time in houre	Time in minute	RecoveryDepth to water level(m)
10:15:00	0	33.7	33.7	10:31:00	0	3.95
10:16:30	0.5	31.89	1.81	10:31:30	1	10.5
10:17:00	1	26.85	6.85	10:32:00	1.5	12.1
10:17:30	1.5	22.8	10.9	10:32:30	2	13.25
10:18:00	2	18.15	15.55	10:33:00	2.5	14.5
10:18:30	2.5	13.95	19.75	10:33:30	3	15.6
10:19:00	3	10.95	22.75	10:34:00	3.5	16.6
10:19:30	3.5	7.75	25.95	10:34:30	4	17.55
10:20:00	4	6.33	27.37	10:35:00	4.5	18.35
10:20:30	4.5	5.6	28.1	10:36:00	5	18.95
10:21:00	5	5.5	28.2	11:37:00	6	20.35
10:21:30	5.5	5.48	28.22	12:38:00	7	21.85
10:22:00	6	5.4	28.3	13:39:00	8	23.25
10:22:30	6.5	5.35	28.35	14:40:00	9	23.6
10:23:00	7	5.2	28.5	15:41:00	10	24.15
10:23:30	7.5	5.12	28.58	16:42:00	11	24.8
10:24:00	8	4.95	28.75	17:43:00	12	25.35
10:24:30	8.5	4.64	29.06	18:44:00	13	25.8
10:25:00	9	4.3	29.4	19:45:00	14	26.3
10:25:30	9.5	4.05	29.65	20:46:00	15	26.75
10:26:00	10	4.25	29.45	21:47:00	16	27.25
10:26:30	10.5	4.1	29.6		17	27.65
10:27:00	11	3.75	29.95			
10:27:30	11.5	4.2	29.5			
10:28:00	12	4.1	29.6			
10:28:30	12.5	3.98	29.72			
10:29:00	13	3.75	29.95			
10:29:30	13.5	4.18	29.52			
10:30:00	14	4.25	29.45			
10:30:30	14.5	4.1	29.6			
10:31:00	15	3.95	29.75			



Appendix 7 Current piezometric map of Naivasha basin



Current Piezometric Head Contour W indicates the depression due to extraction from the well field (Owor, 2000)

Appendix 8 water quality

Appendix 8.1 Chemical analysis of water samples

Code	X	Y	Cl- [mg/l]	Fl- [mg/l]	SO42- [mg/l]	NO3- [mg/l]	PO43- [mg/l]	HCo3- [mg/l]	Ca [mg/l]	K [mg/l]	Mg [mg/l]	Na [mg/l]	Al[mg/l]	Fe[mg/l]	Mn[mg/l]
BH B	213439	9924990	8.7	2.07	13	0	0.43	347.81	49.84	25.68	1.91	68.07	0.19	0	0.4
BH C	213404	9924908	8.8	2.07	12	0	0.44	353.92	53.12	21.26	1.94	62.36	0.26	0.1	0.4
BH D	213385	9924852	7.8	3.4	11	0	0.88	329.5	51.36	19.26	2.33	54.43	0.3	0.02	0.4
BH H	213396	9924806	5.9	2.12	8	0	0.52	318	43.61	19.96	1.78	57.6	0.2	0.13	0.5

Appendix 8.2 Reliability check

Code	X	Y	Cl- [meq]	Fl- [meq]	SO42- [meq]	NO3- [meq]	PO43- [meq]	HCo3- [meq]	Sum Anions	Ca [meq]	K [meq]	Mg [meq]	Na [meq]	Al[meq]	Fe[meq]	Mn[meq]	Sum Cations	Reliability check
BH B	213439	9924990	0.25	0.11	0.27	0.00	0.01	5.70	6.34	2.49	0.66	0.16	2.96	0.02	0.00	0.01	6.30	-0.32
BH C	213404	9924908	0.25	0.11	0.25	0.00	0.01	5.80	6.42	2.65	0.54	0.16	2.71	0.03	0.00	0.01	6.11	-2.44
BH D	213385	9924852	0.22	0.18	0.23	0.00	0.03	5.40	6.06	2.56	0.49	0.19	2.37	0.03	0.00	0.01	5.67	-3.33
BH H	213396	9924806	0.17	0.11	0.17	0.00	0.02	5.20	5.66	2.18	0.51	0.15	2.51	0.02	0.00	0.02	5.38	-2.50

Appendix 9 Spreadsheet model to compute Recharge efficieny and cost per cubic meter of Recharge well and shallow infiltration basin

Daily water balance for the Recharge wells and shallow infiltration basin (Panda flower farm)					
Parametrs	**Amount**	**Units**	**Asumptions**	Remarks	
Storage capacity	9600	m^3		KES: Kenyan Shillings	
Top width	12	m			
Bottom width	4	m			
Length	300	m			
Depth	4	m			
Total infiltration rate	7000	m^3day-1			
Runoff coeffient	0.9				
Green house area	536272	m^2			
Unit excavation: cost shallow	300	KES/ m^3	<5		
Unit excavation: cost deep	500	KES/ m^3	>5		
Expected infiltration capacity per BH	1400	m^3day^{-1}			
No of bH required	5	-			
Unit BH Cost	360000	KES			
Total Excavation cost	2880000	KES			
Total BH cost	1800000.000	KES			
Total Cost	4680000.000	KES			
Cost per mean infiltration	cost/Infiltration	KES			
no years	30				
Efficiency	Infiltration volume/Infolw volume*100	%			
Spilled water	100-Efficiency	%			
Date	**Rain in (mm)**	**Inflow volume (m^3)**	**Storage balance (m^3)**	**Spilled volume (m^3)**	**Infiltraion ($m^{3)}$**
12/3/2003	0	0.00	0.00	0.00	0.00
12/4/2003	0	0.00	0.00	0.00	0.00
12/5/2003	18	8687.61	8687.61	0.00	7000.00
12/6/2003	0	0.00	1687.61	0.00	1687.61
12/7/2003	0	0.00	0.00	0.00	0.00
12/8/2003	0	0.00	0.00	0.00	0.00
12/9/2003	8	3861.16	3861.16	0.00	3861.16
12/10/2003	1.6	772.23	772.23	0.00	772.23
12/26/2003	0	0.00	0.00	0.00	0.00
12/27/2003	0	0.00	0.00	0.00	0.00
12/28/2003	0	0.00	0.00	0.00	0.00
12/29/2003	0	0.00	0.00	0.00	0.00
12/30/2003	0	0.00	0.00	0.00	0.00
12/31/2003	0	0.00	0.00	0.00	0.00
Sum	**Sum**	**Sum**	**Sum**	**Sum**	**Sum**

Appendix 10 Photo gallery

Preparation for injection test

Few to mention problem during the injection test pipe

Test pit and auguring