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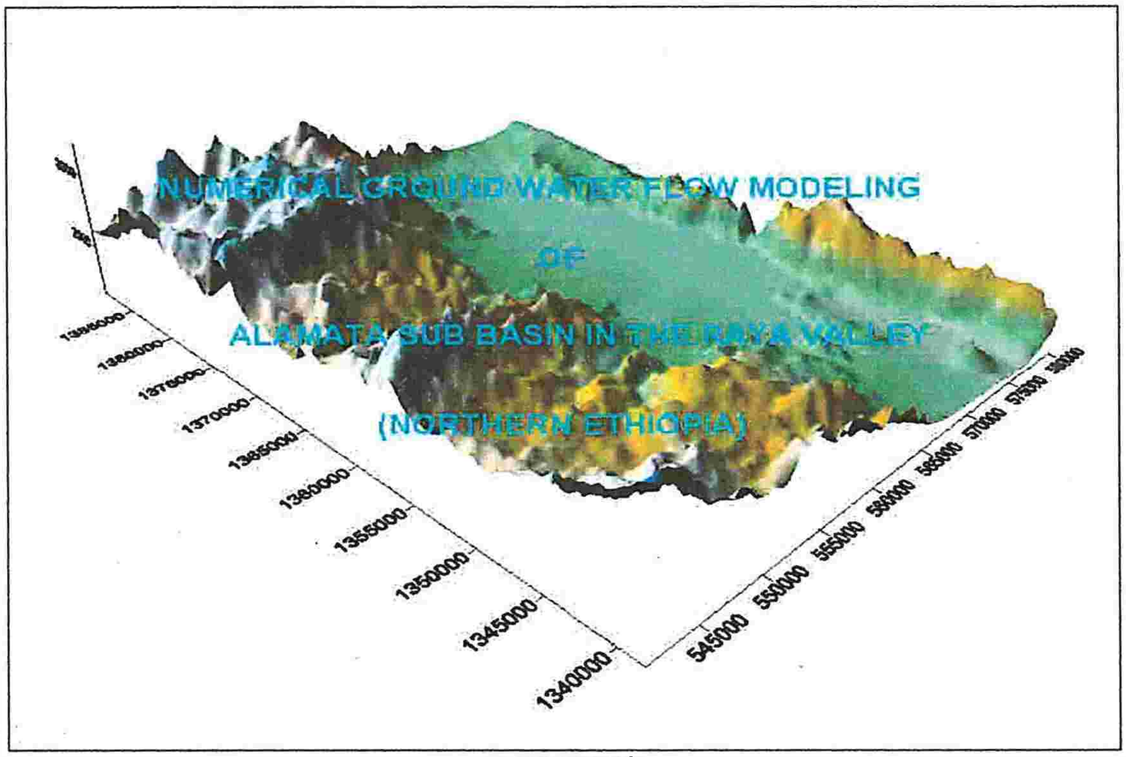
DEPARTEMENT OF IRRIGATION ENGINEERING



A THESIS

SUBMITTED TO SCHOOL OF GRADUATE STUDIES OF

ARBA MINCH UNIVERSITY

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE IN IRRIGATION ENGINEERING

BY GETACHEW WELAMO

JULY, 2009

ARBA MINCH UNIVERSITY
SCHOOL OF GRADUATE STUDIES

NUMERICAL GROUND WATER FLOW MODELING OF ALAMATA SUB-BASIN IN THE RAYA VALLEY (NORTHERN ETHIOPIA)

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CERTIFICATION

The undersigned certify that he has read the thesis entitled "Numerical Groundwater Flow Modeling of Alamata sub basin in the Raya valley" and here by recommend for acceptance by Arba Minch University in partial fulfillment of the requirement for the degree of Master of Science in Irrigation Engineering

Dessie Nedaw, (Ph.D)
Supervisor

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(Getachew Welamo)

Dedicated to

Welamo and Azmera with Love and Gratitude

"Who encourage me to knowledge"

Approval by Board of Examine

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3. Mr Samuel Dagalo (MSc.) (Internal Examiner)	Sign	20/08/09 Date
Dean of SGS	Sign	Date

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ABREVATIONS

Acc.pwl	Accumulated potential water loss
AET	Actual Evapotranspiration
AMC	Antecedent Moisture Condition
BH	Bore Hall
BOARD	Bureau of Agriculture and Rural Development
CN	Curve Number
DD	Draw Down
EC	Electrical Conductivity
ET	Evapotranspiration
FAO	Food and Agriculture Organization
FC	Field Capacity
GIS	Global Information System
GPS	Global Positioning System
Gr	Ground water recharge
GW	Groundwater
IFAD	International Fund for Agricultural Development
KAADP	Kobo Alamata Agricultural Development Project
M	meter
M .a. s. l	Meter Above See Level
M.M.RF	Mean Monthly Rain Fall
MSE	Mean square Error
MODFLOW	Modular Three Dimensional Finite Differences Groundwater Flow Model
MoWR	Ministry of Water Resource
OB	Observation well
PET	Potential Evapotranspiration
PMWIN	Processing MODFLOW for windows
REST	Relief Society of Tigray
RF	Rain Fall
RH	Relative Humidity
RMSE	Root Mean Square Error
Ro	Runoff

RVADP	Raya Valley Agricultural Development Project
RVPIP	Raya Valley Pressurized Irrigation project
ΔS	Change in soil moisture storage
SCS	Soil Conservation Services
UNDP	United Nations Development Program
USGS	United State Geological Survey
UTM	Universal Transverse Mercator
WHO	World health organization
WP	Wilting Point
WS	Wind Speed

ABSTRACT

Alamata sub-basin which is found in the Raya Valley is located in the southern part of Tigray Regional State of the Northern Ethiopia. The area covers 51% of the Raya valley with area coverage of 1292.57km^{2}. The sub-basin water resource is used for irrigation and domestic water supply and is covered by tertiary volcanic in western and eastern highlands and alluvial deposit in the valley fill.

The main purpose of this project work is to make a quantitative estimation for sustainable use of groundwater resource for domestic, live stock and agricultural supply. To achieve this objective careful understanding of hydrology, hydrogeology and dynamics of groundwater flow in and around the study area is mandatory.

As numerical groundwater flow models represent the simplification of complex natural systems, different parameters were assembled into conceptual model to represent the complex natural system in a simplified form. The conceptual model was put into the numeric model to examine system response.

Numerical groundwater flow was simulated in the model by the finite-difference method using MODFLOW, 1996 (McDonald and Harabaugh, 1988). The finite difference grid consisted of 1 layer, 113 rows and 97 columns. Two dimensional profile model was developed considering the system to be under steady state condition and assuming flow system view point.

Model calibration was carried out by trial and error calibration method using groundwater contours constructed from heads collected in 14 wells. The calibration showed that about 100% of simulated heads were within the calibration target and the overall root mean square error for simulated hydraulic heads is about 4.24m. The poor fit at some points was due to numerous limitations associated with the model.

Model sensitivity analysis was conducted by considering the horizontal hydraulic conductivity and recharge because they are sensitive. A change in hydraulic conductivity by -55%,-45%,-25% and 55%, 45% 25% resulted in root mean square

(RMS) 76.31, 53.7, 25.43 24.5, 45.6 and 51 respectively. A change in recharge by the same amount as the horizontal hydraulic conductivity RMS head changes by 53.3, 40.52, 23, 22.17, 39.9, and 47.8.

As the model is intended to study the response of the hydrologic system the following four scenarios were assigned: In the first and second scenarios which apply for the sub-basin and, withdrawals were increased by 25 and 50% of the average existing withdrawals. In the third and forth scenarios, 25% and 50% reduction in recharge is applied. The effects of the scenarios are evaluated with respect to change induced groundwater heads to the steady state simulated heads. Accordingly, an increase of withdrawals by 25% over the whole area resulted in an average decline of steady state water level with minimum decline of 3.747m at well OB4 and a maximum decline of 7.74 at well PZ3. Again an increase of withdrawals by 50% results with a minimum drawdown of 7.985m at well OB4 and a maximum of 31.472m at well PZ3.

On the other way, reduction of recharge by 25% results with a minimum decline of 20.333m at well BH21 and a maximum decline of 32.485m at well PZ3. A reduction of 50% recharge resulted with a complete drying of well PZ3.

1. INTRODUCTION

WATER is the basic need for any life to exist in this world. Prehistoric man was leading a nomadic life on the banks of rivers. With natural calamities such as floods, earthquakes etc. he was disturbed and up rooted from his dwelling place. With the advent of civilization the use of water increased by leaps and bounds, first for his demostic needs, then for supplementing agriculture, irrigation municipal requirements and later for industrial growth. Naturally, when surface water is in short supply one has to depend partly or wholly on ground water.

Groundwater is more desirable than surface water at least for the following six reasons:-

- It is commonly free of pathogenic organisms and no need of elaborating the purification for domestic and industrial uses.
- Temperature is nearly constant which has a great advantage if the water is used for heat exchange.
- Turbidity and color are generally negligible.
- Chemical composition is generally constant.
- Ground water storage is always greater than surface water storage, so that ground water supplies are not seriously affected by short duration droughts.
- Biological contamination in ground water is seldom noticed.

Groundwater is the main source for rural water supplies in many semi-arid developing countries. Over recent years, increasing abstraction to meet rising demand for domestic supplies and irrigation has raised concerns for the sustainability of the resource and the livelihood it supports.

Additionally, changing land use as well as hydrological interventions and climate change will have impacts on natural recharge and groundwater storage. Consequences of overexploitation include declining water levels and increasing competition for scarce water resources between domestic and agricultural users and rural and urban communities. This creates a new situation that request proper planning and sound management of groundwater.
Therefore this thesis is dealing with ground water flow modeling and its sustainable utilization for domestic, livestock and agriculture purpose.

1.1 Back ground and justification

Ethiopia has 12 river basins. The total mean annual flow from all the 12 river basins is estimated to be 122 BMC (MoWR 1999). Figure 1.1 below shows the map of Ethiopian River Basins. It also has 11 fresh and 9 saline lakes, 4 crater lakes and over 12 major swamps or wetlands. Majority of the lakes are found in the Rift Valley Basin; see also Figure 1.1. The total surface area of these natural and artificial lakes in Ethiopia is about 7,500 km^2. The majority of Ethiopian lakes are rich in fish. Most of the lakes except Ziway, Tana, Langano, Abbaya and Chamo have no surface water outlets, i.e., they are endhoric. Lakes Shala and Abiyata have high concentrations of chemicals and Abiyata is currently exploited for production of soda ash.

As compared to surface water resources, Ethiopia has lower ground water potential. However, by many countries' standard the total exploitable groundwater potential is high. Based on the scanty knowledge available on groundwater resources, the potential is estimated to be about 2.6 BMC (Billion Metric Cube) annually rechargeable resources.

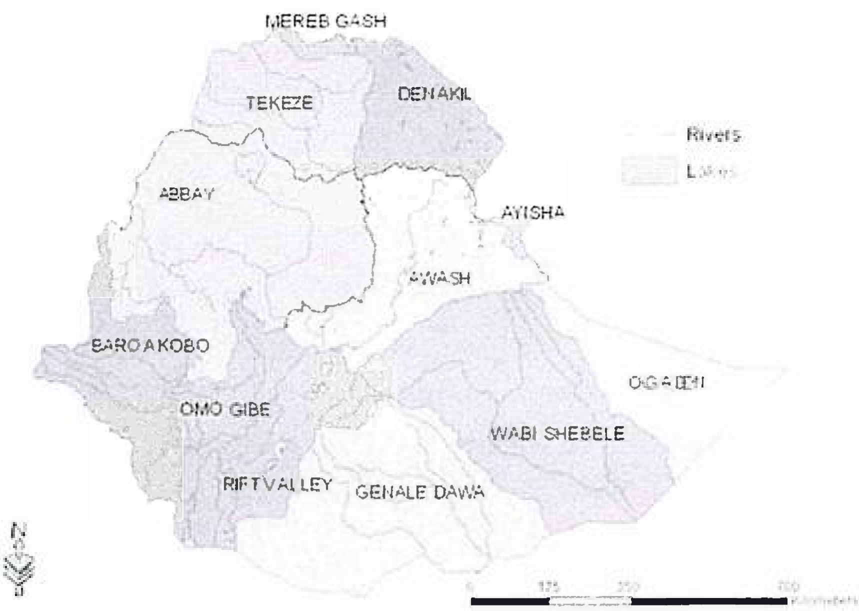


Figure 1. 1 Ethiopian river basins map (Source, International Water Management Institute. 78p.Working Paper 123)

Though the country possesses a substantial amount of water resources little has been developed for drinking water supply, hydropower, agriculture and other purposes.

The water supply coverage was estimated to be 30.9 percent, thus the rural water supply coverage being 23.1 percent and that of urban being 74.4 percent (UNESCO 2004). The great majority of the rural Ethiopian population community water supply relies on groundwater. The safe supply of water in rural areas is usually derived from shallow wells, spring development and deep wells. People who have no access to improved supply usually obtain water from rivers, unprotected springs, hand-dug wells and rainwater harvesting. Despite its immense relevance and importance, the groundwater sector has been given less attention until recently.

Based on the present indicative information sources, the potential irrigable land is about 3.7million hectares. This figure is believed to be on a lower side, and could change as more and reliable data emerge particularly on small-scale irrigation potential. Estimates of the irrigated area presently vary, but range between 150,000 and 250,000 hectares less than five percent of potentially irrigable land (Werfring 2004; Awulachew et al. 2005). Estimates of irrigated area according to MoWR, is 107,265.65 hectares, which is less than 5 percent of the potential. This data does not contain schemes which are under construction, or in operational/suspended for some reasons.

Nevertheless Ethiopia has been stricken repeatedly by severe droughts and it is one of the poorest countries in the world which is ranked 170th out of 177 countries (UNDP, 2005). Though poverty is highly spread through out the country it is the worst in Tigray. In 34 rural woreda 621,000 household or 75% total populations of four million are food insecure and seriously threatened by drought, which hit the region every 3-4 years (Hugo rams 2003). To reveres this situation Ethiopian government introduce a national poverty reduction strategy of modern agricultural development approach using all available water resource we have.
Depending on the national poverty reduction strategy, the Tigray national Regional Government has set an ambitious goal to eradicate 80% of the food deficit with in the last 3 years (2005-2007 Regional rural development strategy plans). Water

harvesting with ponds and groundwater extraction by shallow wells were the main component to increase the agricultural production through irrigation.

In the last 5 years as part of the effort to attain the regional objective water works such as 31,374 hand dug wells, 71,406 house hold ponds, 53 small earth dams and 61 diversion weirs have been constructed at house hold and community level. Many of the people living in the project area using hand dug wells to achieve their house hold food security through home garden micro irrigation. These water works currently irrigating 12876 hectares which is 24 fold what was irrigated before 10 years (Report of Tigray BOARD, 2006).

Due to the expansion of irrigated agriculture there is excessive withdrawal of groundwater. The most evident problem resulted from this is lowering of the water table beyond the reach of the existing well depth. In some places (Shire and Mekele) the water table has dropped a lot of meters due to excessive well pumping. This lowered water table cause severe shortage for domestic and agricultural water supply.

In order to utilize the ground water resource properly, understanding of the groundwater occurrence and distribution in space and time, proper management and efficient exploitation is necessary. The available studies on the groundwater resources of the region (Tigray) are very limited, in that, the delineation of aquifer systems, the water balance and determination of the aquifer characteristics has not been conducted. Any sustainable utilization of groundwater resources demands systematic study and raising the technical and man power capability. In this regard the region has a long way to go, yet.

In response to the above problem, groundwater modeling for its sustainable utilization should be conducted in Tigray region at watershed level.

1.2 General overview of the Study Area

1.2.1 Location

The catchment area is found in Raya valley Alamata wereda, southern zone of Tigray National Regional State, around 600km from Addis Ababa and 180km from the regional capitalm, Mekele. Geographically the study area is located between

UTME from 0550799 to 0580416 and UTMN from1343384 to 1381274 at Alamata site. Hydrologically it is found in western Denakil river basin and estimated to have area coverage of 1292.57 sqkm.

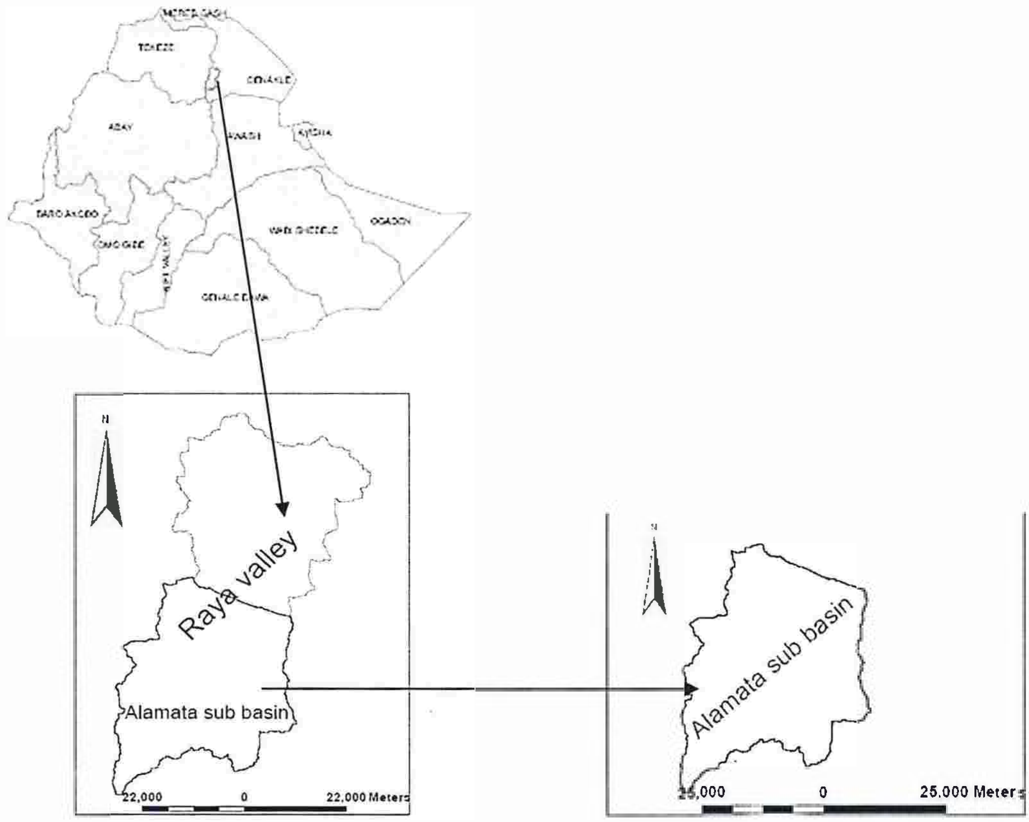


Figure 1. 2 Location of study area

1.2.2 Rainfall

The area receives bi-modal rainfall with erratic distribution and therefore the bi-modal pattern of the rainfall is not consistent. The mean annual rainfall varies from 400-800mm. The main rainy season (meher) lasts from end June to early September and the highest amount of rain is recorded in July & August where as the short rainy season (Belg) starts in February and ends in end March.

The rainfall data of the study area was taken from Waja and Alemata metrological stations which are with in the target area respectively. The mean is tabulated and presented in the Table 1.1

Accordingly, the mean annual rainfall of the study area is 728.5 mm. The mean monthly rainfall averaged over the period of records for the Alamata basin is shown in Figure 1.3.

Table 1. 1 Mean monthly rainfall of Alamata basin

Months	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Sum
M.M. RF ofWaja	29	43	53	78	76	29	164	191	58	28	20	26	727
M.M. RF of Alamata	42	48	78	97	59	8	142	168	53	27	26	41	730
Avg. M.M. RF(mm)	35.5	45.5	65.5	87.5	67.5	18.5	153	179.5	55.5	27.5	23	33.5	728.5

Where M.M. RF is mean monthly rain fall
Avg. M.M.RF is average mean monthly rain fall

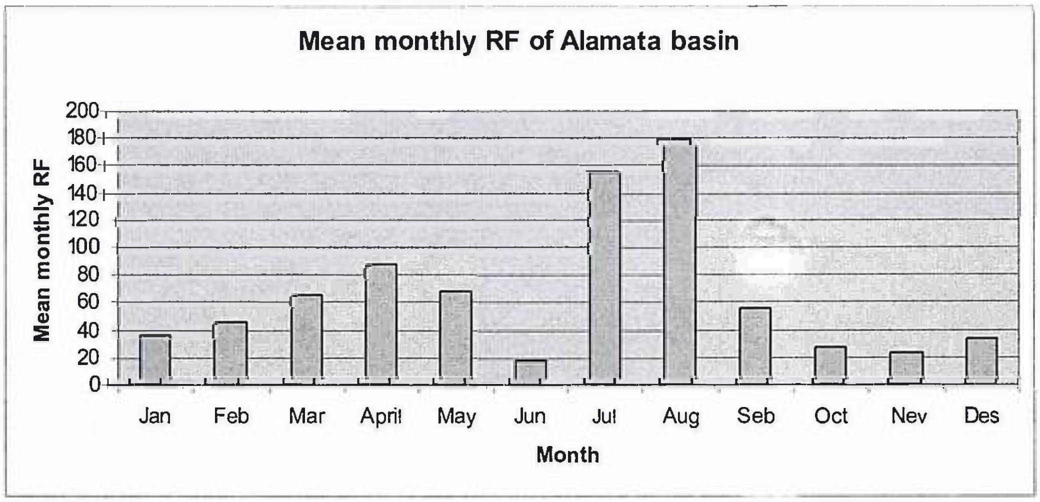


Figure 1. 3 Mean monthly RF of Alamata basin

1.2.3 Temperature

The temperature data for the study area is taken as the mean from Waja and Alamata stations which are inside the study area. From this mean temperature data analysis, the minimum air temperature is 19.5 °C in December and the maximum air temperature is 25.78 °C in June. The annual range of temperature is 6.28 °C.

Table 1. 2 Mean monthly temperature of Alamata station

Months	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
M.M.Max.Temp	27	27.5	29.5	39	32.5	34.22	31.62	30.5	30.53	30.07	28.51	27.5
M.M.Mini.Temp	12	13	14	15.62	16.5	17.53	17	15.73	15	13.62	12.88	12.31
M.M.Temp	19.5	20.25	21.75	27.31	24.5	25.87	24.31	23.11	22.8	21.84	20.7	19.9

Table 1. 3 Mean monthly temperature of Waja station

Months	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
M.M.Max.Temp	27.24	28.51	29	29.78	31.73	34.17	32.17	30.5	30.4	29.4	28.82	27.84
M.M.Mini.Temp	12.04	13.6	15.11	15.42	15.73	17.18	18.1	18.62	15.8	13.58	12	10.36
M.M.Temp	19.64	21.055	22.055	22.1	23.73	25.68	25.14	24.56	23.1	21.5	20.41	19.1

Table 1. 4 Average Mean monthly temperature of Alamata and Waja stations

Month	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
M.M.Temp (Alamata)	19.5	20.25	21.75	27.31	24.5	25.87	24.31	23.11	22.8	21.84	20.7	19.9
M.M.Temp (Waja)	19.64	21.055	22.055	22.1	23.73	25.68	25.14	24.56	23.1	21.5	20.41	19.1
M.M.Temp	19.57	20.65	21.90	24.71	24.12	25.78	23.74	23.84	22.95	21.67	20.56	19.5

Where

M.M. Max .Temp is mean monthly maximum temperature;

M.M. Mini. Temp is mean monthly minimum temperature; and,

M.M .Temp is mean monthly air temperature

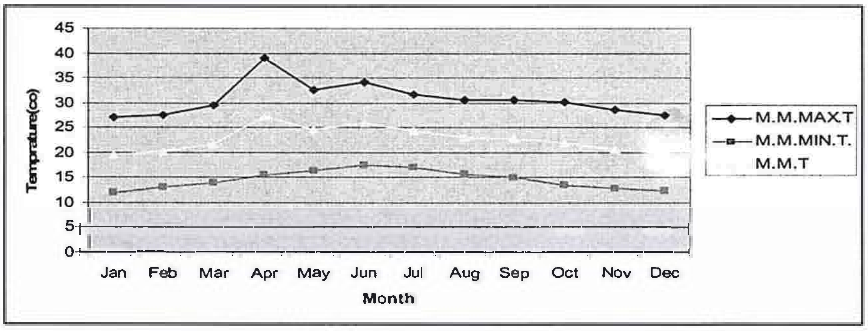


Figure 1. 4 Mean monthly Temperature of Alamata station

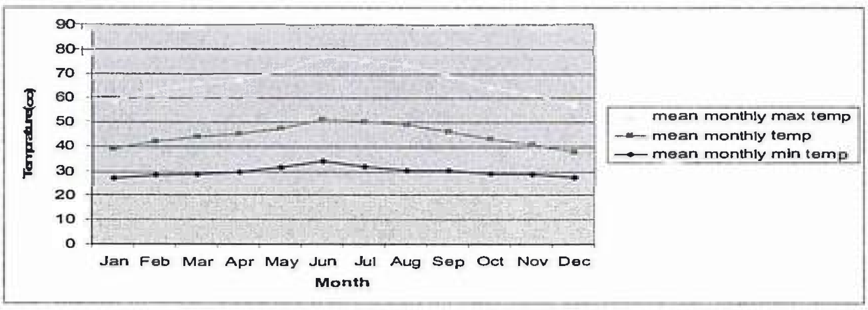


Figure 1. 5 Mean monthly temperature of Waja station

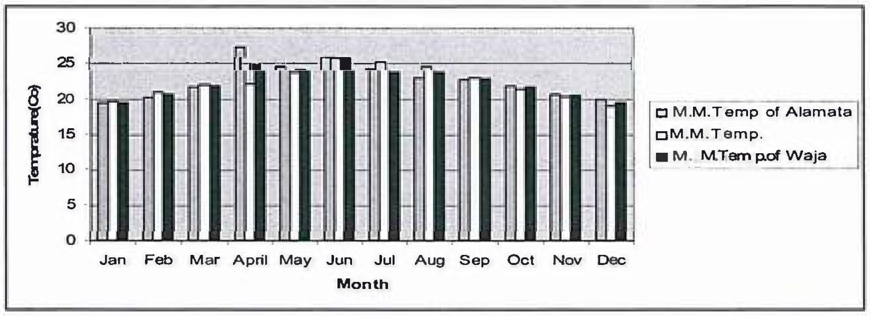


Figure 1. 6 Mean monthly temperature of Alamata basin

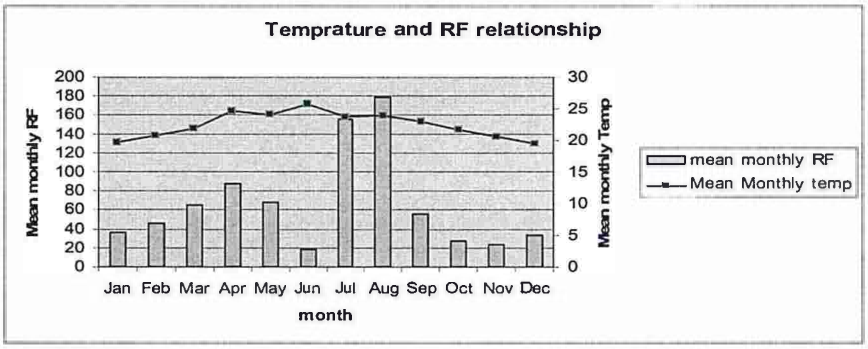


Figure 1. 7 Temperature and RF relationship

Figure 1.7 shows the temperature and rainfall relations for the basin. Even though the maximum air temperature occurs in June, high temperature values are observed during the rainy seasons. Months in the rainy seasons are warmer than months in dry seasons. The minimum temperatures as well as rainfall are also occurring almost in a similar range of months.

1.2.4 Wind speed

Wind speed data was taken from Kobo meteorological station. The data was a 30 years record from 1976 to 2005. The mean monthly values were computed and are given in the Table1.5 below. The maximum and minimum wind speed value is obtained in January (249.7 km/d) and September (103.7 km/d), respectively. In general the highest wind speed values are found in dry months whereas the lowest occur in very highly rainy months.

1.2.5 Humidity

The relative humidity data(%) was taken from Kobo meteorological station. The data was recorded from 1959 to 2004. The mean monthly values were computed and are given in the Table1.5 below. The maximum and minimum relative humidity value is found in August and July (67 %) and June (33 %), respectively. This maximum and minimum value is also within very highly rainy month and dry month, respectively. In general the highest humidity values are found in the rainy months whereas the lowest values are in dry months.

1.2.6 Sun Shine Duration (hours)

Monthly sunshine hours were collected from Kobo meteorological station. The data was recorded from 1976 to 2004. The mean monthly sunshine hours of the area are given in the Table 1.5. The maximum sunshine hour is recorded in November (9.13 hours) and October (8.6 hours) whereas the minimum one is in July (5.7 hours) and August (6.2 hours). Generally, the maximum sunshine hours are found in dry months whereas the minimum are in rainy months

Table 1. 5 Mean monthly wind speed, relative humidity, and sunshine Hours

Month	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
S(hrs)	7.9	7.7	8.05	8.14	8.4	6.62	5.3	5.4	6.67	8.6	9.13	8.3
RH(%)	60	57	51	52	42	33	67	67	61	53	46	54
W.S(km/day)	249.7	178	182.3	164.2	145.2	167.6	104.4	105.2	103.7	112.3	126.1	139.1

Where
W. S; is Wind speed;
R.H; is Relative humidity; and,
S; is Sunshine hours

1.2.7 Soils

As we can see from Fig 1.8, the soils do not show extensive variation, and are limited to six main classes of chromic vertisol, eutric cambisol, eutric nitisols, eutric rigosols, Haplic xerosols and Leptosols, No sever soil constraints to agriculture were identified on the basis of the field survey, but the particular properties of vertisols should be recognized in any agricultural development plan. These soils are characterized by a dominance of the clay mineral, montmorillonite, which expands when wet and contracts when dry, giving rise to wide surface cracks. It is deep to very deep, black to very dark greyish colours. And it is suitable for agriculture.Well drained vertisols are identified around the agricultural area of Alamata basin. These soils are more dominant to the adjacent sides of Alamata to Bala around Selenwiha in association with eutric cambisols.

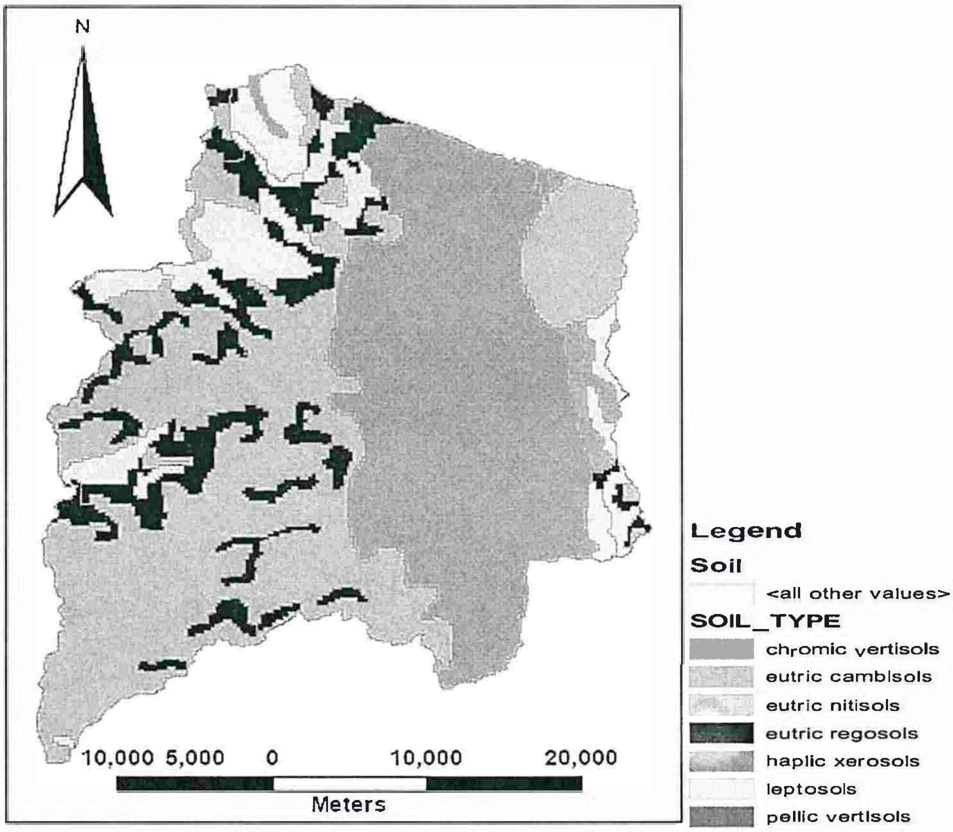


Figure 1. 8 Soil map of Alamata basin

1.2.8 Drainage

The proposed irrigation area is located in the flood plain of the Raya valley. The irrigation area is divided into two sub basins, Alamata and Mehoni sub basins. Several streams that drain uplands traverse the project area, particularly the Alamata sub basin, at shallow depths and flat slopes (see Figure 1.9). The Alamata sub basin is dissected into four major parts by Harosha, Oda, Hara, and Daya Rivers that originate in the western highland parts of the Raya valley. The

25-year return period discharge of these rivers is given in Table1.6 (adopted from MoWR final draft of hydro meteorological report for Raya valley, 2008)

Table 1. 6 Rivers that dissect the Alamata Sub-basin and their Q_{25} (discharge in 25 years return period)

Catchment/River	Area, km^2	Q_{25}, m^3/s
Harosha	33.9	57.3
Oda	18.2	38.7
Hara	63.6	85.2
Dayu	72.3	92.4

Moreover the rivers transport a large amount of sediment from the upslope areas. Consequently, the rivers have exposed the project area to flash floods and sedimentation problems. Farmers of the area, however, have the practice of diverting the flash floods for spate irrigation purposes.

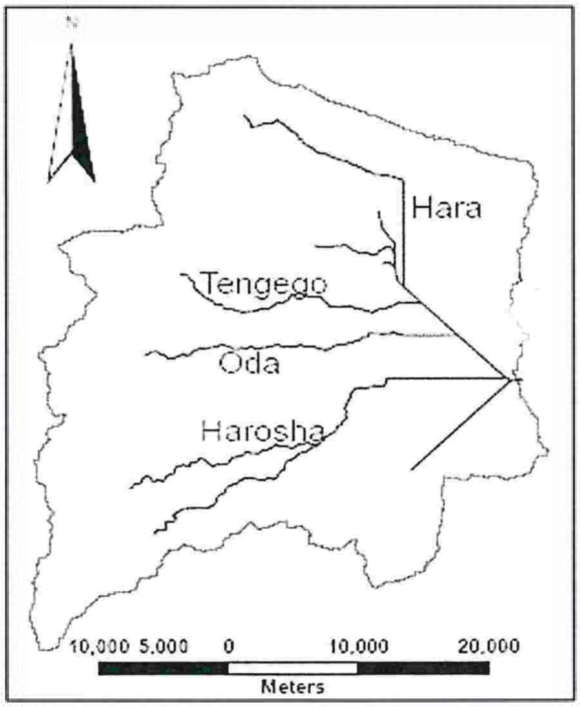


Figure 1. 9 Drainage map of the study area

1.2.9 Land Use

Appropriate land use policy and system of its practical application is an essential part of land use management. However, the study woreda has no systematic and technically supported land use system. According to the data taken from woreda's rural development and agriculture offices, the land use pattern for Alamata woreda includes; - cultivated land 34,503ha, grazing, 5,214ha, forest land 7,816ha and others, 1,709ha. But the actual land use pattern of Alamata wereda can be shown in fig 1.10 below

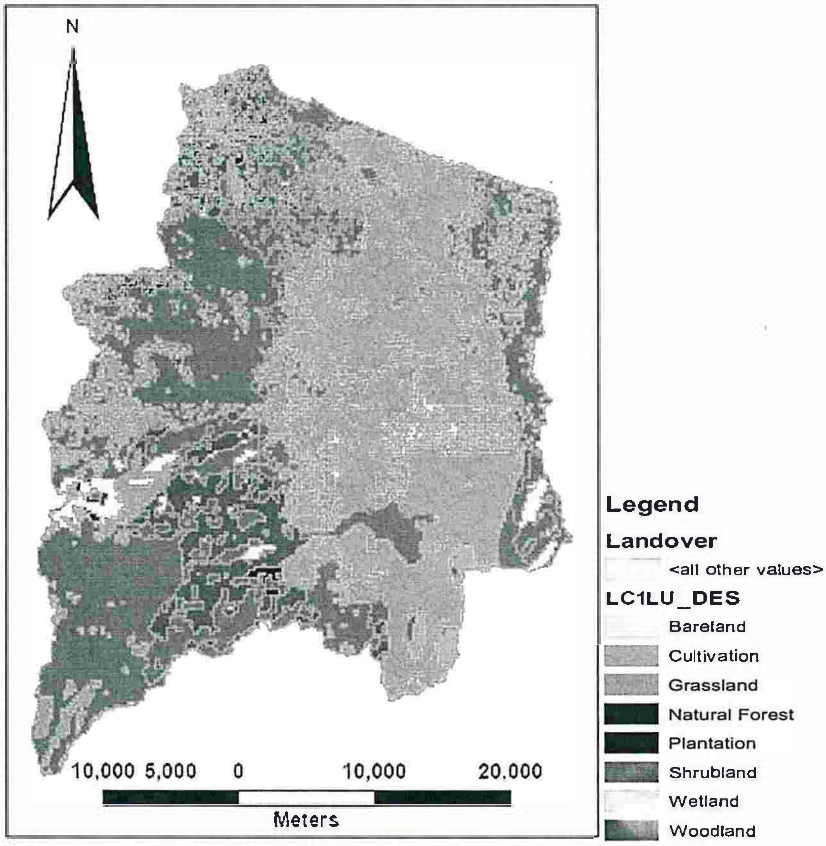


Figure 1. 10 Land use map of Alamata wereda

1.2.10 Geology of the Area

According to the 1998 study of the site by the Relief Society of Tigray (REST) [1] and the 1996 second edition Geological Map of Ethiopia, the project area is a flat plain dominated by deep to very deep undifferentiated alluvial, lacustrine, and beach sediments bounded on the East and West by Ashangi formation, which is a series of volcanic rocks characterized by deeply weathered alkaline (olivine) and transitional basalt flows with tuff intercalations, rare rhyolites from fissures and dissected by dikes and sills.

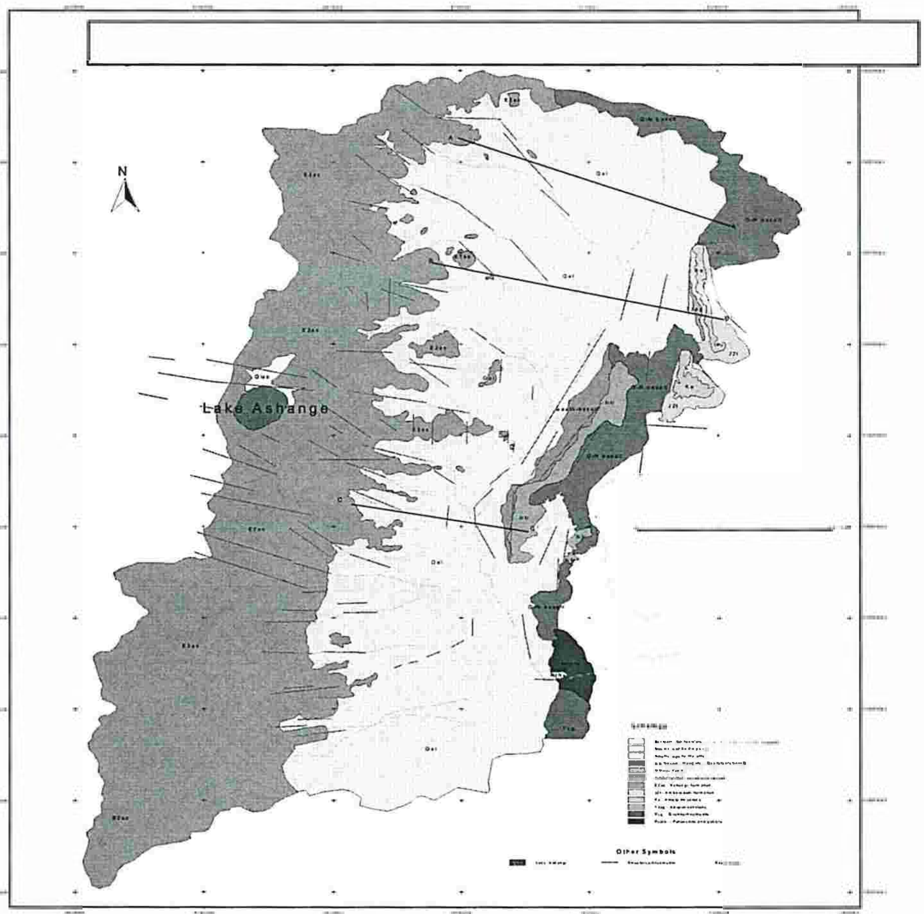


Figure 1.11 Geological map of Raya Valley (Source, RVADP geological feasibility report)

1.2.11 Crop Production and Farming System

Crop production in Tigray region in general and study area (Alamata woreda) in particular has more or less similar trend of development. In that farmers are gradually changing their culture into modern ways of production systems. The farming system and crop management including cropping pattern and cropping system is almost similar. The main difference observed within the region is rainfall variation, which is erratic in some zones and relatively reliable in others.

Mixed farming is the major economic base of Alamata woreda in which farmers practice both crops and livestock production giving more emphasis for crop production.

Diversified variation in Agro-climatic zones, soil types and socio-economic conditions of the farming communities, has contributed to the evolution of different cropping practices in the region in general and study area in particular.

Therefore, cropping practice-based approach of the study may contribute much to accelerate future agricultural development through optimized and well-judged use of resources.

There are two major cropping practices that have been identified during field survey. These are the only practices existing in the study area and include,

Cereal based smallholder single cropping practice (Meher &Belg)

Small-scale traditional irrigated agriculture and start of Pressurized irrigation at micro level

Table1.7 Existing meher cropping pattern & operational calendar (Alamata Woreda)

No	Crop type	Area	Yield qt/ha	Total Production (qts)	Operation calendar			
		Ha			Ploughing	Sowing	Weeding /hoeing/	Harvesting
1	Sorghum	10,699	22	235378	Jan -Mar	Apr-may	June-Aug	Nov. -Dec
2	Teff	15,262	11	167882	Feb-may	June- july	July-aug	Oct-nov
3	Barley	2,088	16	33408	Apr-may	June	July-aug	Oct
4	Wheat	1,710	15	25650	Apr-may	June	July-aug	Oct
5	Maize	1,465	19	27835	Jan -Mar	Apr-may	June-Aug	Nov. -Dec
6	Broad bean	826	11	9086	Apr-may	June	July-aug	Oct-nov
7	Field pea	889	10	8890	"	"	"	Oct
8	*Abysinicum pea*	69	8	552	"	"	"	"
9	Sesame	125	6	750	Feb-may	June	July	Oct
10	Lentil	105	4	420	-	June	-	Sept-oct
11	F.millet	500	16	8000	Feb-may	June	July-aug	Nov-dec
12	Millet	306	19	5814				
13	Nigger seed	25	5	125	-	June	July-aug	Oct
14	Line seed	35	5	175	-	"	"	Nov-dec
15	Chick pea	180	6	1080	July-aug	End Aug	-	Dec
	Total	34,284		525,045	-	-	-	-

Source: - Woreda rural development & Agricultural office

Table 1. 8 Existing Irrigated Cropping Pattern & operational calendar (Alamata Woreda)

No	Crop type	Area	Yield qt/ha	Total Production (qts)	Operation calendar		
		Ha			Ploughing	Sowing	Harvesting
1	Onion	673	90	60570	Dec-Jan	Feb-mar	End may-june
2	Tomato	76.25	120	9150	"	"	"
3	Cabbage	11.9	80	952	"	"	"
4	Pepper	186.9	7	1308	"	"	"
5	Total	948.05	-	71,980	-	-	-

Sources: - Alamata Woreda Rural Development & Agriculture Office

1.2.12 Population

According to demography report, the total population is 97,579 for Alamata woreda (male-47,817, female-49,762) and the total estimated household is about 23,000.

1.3 Statement of the Problem

Throughout the world, regions that have sustainable ground water balance are shrinking by the day. Three problems dominate ground water use; one, depletion due to over abstraction. In such a situation serious problems is created resulting in drying of shallow wells (Shire and Mekele), increasing of pumping cost for deeper well, and conflict for water between owners of neighbor wells and finally deprive future generation from using that resource; second, water logging and salinization are mostly due to inadequate drainage and insufficient conjunctive use. (Waja and gerjele swamp); and third pollution due to agricultural, industrial and other human activities

Although surface water is the main source of water for agriculture, the study area depends on ground water for domestic purposes and small scale irrigation and water supply schemes. Generally the water resource of the catchment area is spatially as well as temporally unevenly distributed, because rainfall is highly variable. During the dry season it is subject to water scarcity problems related not only to agriculture but also for human consumption.

Ground water use is rapidly increasing in the study area, bringing several benefits to small farmers by allowing them to grow more crops and minimizing the impact of water shortages that occur during dry season. Over the last five years groundwater usage through tube wells has increased sharply in the area but groundwater is not properly managed as an additional source. Government organizations as well as several private parties are engaged in construction of tube wells in the study area. At the same time there is no any authorized organization to monitor or register ground water data. Because of that, clear records on this resource cannot be obtained.

As a result of the above problem the sustainable utilization of ground water for irrigation and domestic use has often been in questione.

In order to utilize the ground water resource properly; understanding of the groundwater occurrence and distribution in space and time, delineation of aquifer systems, water balance and determination of aquifer characteristics, proper management and efficient exploitation are necessary. And then plan their use in such away that full crop water requirement are met and there is neither water logging nor excessive lowering of groundwater table. By dong so the groundwater reservoir will be maintained in state of dynamic equilibrium over a particular range of time.

Therefore this thesis is going to be prepared on ground water flow modeling on the study area (Alamata basin) to give answer for the problem mentioned above.

1.4 Objective

1.4.1 General Objective

The general objective of the study is to make a quantitative estimation for sustainable use of ground water resource for domestic, live stock and agricultural supply in the sub basin.

1.4.2 Specific Objectives

The specific objectives of the study are

- To establish a rainfall recharge coefficient of the basin
- To assess the current annual volume of ground water withdrawal for domestic, livestock and agriculture use and annul volume of water replenished
- To assess the present status of ground water in the basin whether the annul withdrawal is equal to the annual replenishment.
- To delineate the groundwater potential zones in the area.
- Constructing a numerical groundwater flow model of the Alamata basin to gain a better understanding of the aquifer system and to develop a tool for evaluating aquifer responses to various water management alternatives
- To determine groundwater flow directions in the basin.

1.5 Significance of the study

- Sustainable utilization of groundwater for irrigation, domestic and livestock supply.
- The ground water reservoir will be maintained in a state of dynamic equilibrium over a period of time.

2 LITERATURE REVIEW

2.1 Groundwater Flow Modeling

A model is any device that represents an approximation of a field situation (Anderson and Woessner, 1992). A problem or issue is defined, data is analyzed and an idealized explanation of the real behavior, the conceptual model, is formulated in terms of the major physical processes that appear to be operating. These processes are then represented mathematically, and the resulting mathematical model is used to test the initial understanding and then often to make predictions. Therefore, the preliminary conceptual understanding provided by drawing some geological cross-sections and calculating crude water balance estimates constitutes a model, as does the application of analytical solutions such as Theis. It is important to distinguish among three terms we use to discuss the modeling process: conceptual model, computer model program, and model.
A "conceptual model" is the hydrologist's concept of a ground-water system.
A "computer model program" is a computer program that solves ground-water equations. Computer model programs are general purpose in that they can be used to simulate a variety of specific systems by varying input data.
A "model" is the application of a computer model program to simulate a specific system. Thus, a model incorporates the model program and all of the input data required to represent a ground-water system.
It is this broad definition of modeling that is considered here

2.1.1 Derivation of the Finite-Difference Equation

The main equation that describes transient flow in an anisotropic porous medium when the coordinate system is oriented along the principal axes of anisotropy in three dimensions is(McDonald and Harbaugh, 1988):

$$\frac{\delta}{\delta x}(K xx\frac{\delta h}{\delta x}) + \frac{\delta}{\delta y}(K_{yy}\frac{\delta h}{\delta y}) + \frac{\delta}{\delta z}(K_{zz}\frac{\delta h}{\delta z}) - W = S_s\frac{\delta h}{\delta t} \qquad 1$$

Where:
K_{xx}, K_{yy} and K_{zz} = values of hydraulic conductivity along the x, y and z coordinate axes, which are assumed to be parallel to the major axes of hydraulic conductivity (Lt^{-1}),

h = potentiometric head (L),

W = volumetric flux per unit volume and represents sources and/or sinks of water

S_s = specific storage of the porous material (L^{-1}), and

t = time (t).

The derivation of Equation 1 can be found in Fetter (1999). Equation 1 along with the specification of flow and/or head conditions at the boundaries, and initial head conditions constitute a mathematical representation of the aquifer system. Analytical solutions to Equation 1 are rarely possible, and therefore a variety of numerical solutions have been developed to obtain approximate solutions.

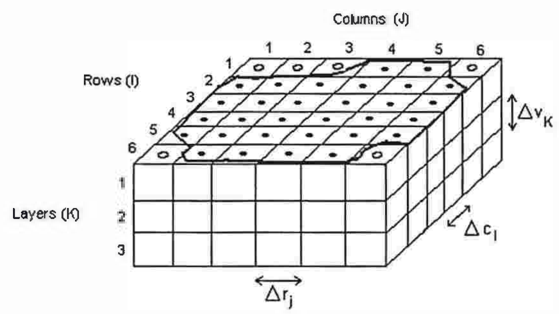
The finite-difference approach is a numerical approximation of the analytical solution to Equation 1. In this approach, the continuous system is replaced by a finite set of discrete points in space and time. The partial derivatives are replaced by terms calculated from the differences in head values at these points (McDonald and Harbaugh, 1988). The solution of the resulting multiple linear algebraic difference equations yields values of head at specific points in time, representing the time-varying head distribution that would be given by an analytical solution of the partial-differential equation of flow (McDonald and Harbaugh, 1988).

2.1.2 Discretization

The ground-water system is divided into a mesh of blocks called cells. The cells are defined by rows (i), columns (j), and layers (k). The model was developed to represent a Cartesian coordinate system, and therefore the k index indicates changes along the vertical z axis, rows would be considered parallel to the x axis, and columns are parallel to the y axis.

Figure 2.1 illustrates this discretization by showing the grid cells lined up in the coordinate system.

As stated above, MODFLOW uses a block-centered formulation of the finite difference equation. Thus, the discrete nodes at which heads are calculated are located at the center of each cell. The spacing of the nodes should be chosen so that the hydraulic properties of the system are uniform over the extent of the cell, because in assigning such properties, it is assumed that they are assigned to the whole cell (McDonald and Harbaugh, 1988).



Explanation

- Aquifer Boundary
- • Active Cell
- o Inactive Cell
- Δr_j Dimension of Cell Along Row Direction.
- Δc_I Dimesion of Cell Along Column Direction.
- Δv_K Dimension of Cell Along Vertical Direction.

Figure 2. 1 Discretized hypothetical aquifer system in MODFLOW (McDonald and Harbaugh, 1988).

2.1.3 Finite Difference Equation

The finite-difference equation for ground-water flow is developed based on the continuity equation: the sum of all flows into and out of a cell must equal the rate of change in storage within the cell. Figure 2.2 shows the three dimensional representation of a cell and its adjacent cells, and the corresponding subscript notation. The flow into cell i,j,k in the row direction from cell i,j-1,k, is given as:

$$q_{i,j-1/2,k} = KR_{i,j-1/2,k} \Delta c_i \Delta v_k \frac{(h_{i,j-1,k} - h_{i,j,k})}{\Delta r_{j-1/2}} \tag{2}$$

where:

$h_{i,j,k}$ = head at node i.j.k.
$h_{i,j-1,k}$ = head at node i.j-1.k.
$q_{i,j-1/2,k}$ = volumetric fluid discharge through the face between cells i.j.k and i.j-1.k (L^3t^{-1}).

$KR_{i,j-1/2,k}$ = hydraulic conductivity along the row between nodes i,j,k and i,j-1,k (Lt^{-1}).

$\Delta c_i \Delta v_k$ = area of cell faces normal to the row direction, and

$\Delta r_{j-1/2}$ = distance between nodes i,j,k and i,j-1,k.

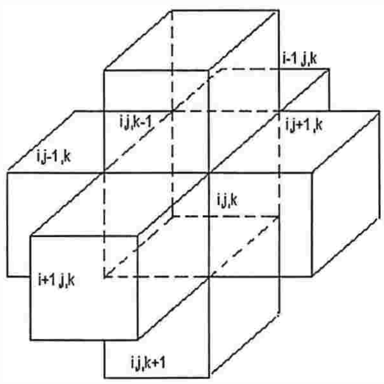


Figure 2. 2 Cell i,j,k and indices for the six adjacent cells in MODFLOW (McDonald and Harbaugh, 1988).

The subscript ½ does not represent a specific point between the nodes, rather it is meant to be the effective parameter for the entire region between the nodes, or the harmonic mean as described by Collins (1961) (McDonald and Harbaugh, 1988). Similar equations can be written for approximating flow into the cell through the remaining five faces. The expressions can be simplified by defining the "hydraulic conductance" or "conductance" as:

$$CR_{i,j-1/2,k} = KR_{i,j-1/2,k} \frac{\Delta c_i \Delta v_k}{\Delta r_{j-1/2}} \tag{3}$$

Where:

$CR_{i,j-1/2,k}$ = conductance in row i and layer k between nodes i,j-1,k and i,j,k ($L2t^{-1}$),

Thus, conductance is the product of hydraulic conductivity and cross-sectional area of flow divided by the length of the flow path (or distance between the nodes) (McDonald and Harbaugh, 1988).

The application of continuity, and taking into account any external flow rates from any other sources and defining them as QS, yields:

$$q_{i,j-1/2,k} + q_{i,j+1/2,k} + q_{i-1/2,j,k} + q_{i+1/2,j,k} + q_{i,j,k+1/2} + q_{i,j,k-1/2} + QS_{i,j,k} = SS_{i,j,k} \frac{\Delta h_{i,j,k}}{\Delta t} \Delta r_j \Delta c_i \Delta v_k \qquad (4)$$

where:

$q_{x,y,z}$ = cell-to-cell flow.

$QS_{i,j,k}$ = external flow rate from any other source.

$\Delta h_{i,j,k}/\Delta t$ = finite-difference approximation for the derivative of head with respect to time (Lt^{-1}).

$SS_{i,j,k}$ = specific storage of cell i.j.k (L^{-1}), and

$\Delta r_j \Delta c_i \Delta v_k$ = volume of cell i.j.k (L^3).

Substituting equation (2) for each cell into equation (4) yields:

$$\begin{aligned} & CR_{i,j-1/2,k}(h_{i,j-1,k} - h_{i,j,k}) + CR_{i,j+1/2,k}(h_{i,j+1,k} - h_{i,j,k}) \\ & + CC_{i-1/2,j,k}(h_{i-1,j,k} - h_{i,j,k}) + CC_{i+1/2,j,k}(h_{i+1,j,k} - h_{i,j,k}) \\ & + CV_{i,j,k-1/2}(h_{i,j,k-1} - h_{i,j,k}) + CV_{i,j,k+1/2}(h_{i,j,k+1} - h_{i,j,k}) \\ & + P_{i,j,k} h_{i,j,k} + Q_{i,j,k} = SS_{i,j,k}(\Delta r_j \Delta c_i \Delta v_k) \frac{\Delta h_{i,j,k}}{\Delta t} \end{aligned} \qquad (5)$$

Where:

CR, CC, and CV = the hydraulic conductivities along rows, columns, and layers,

$P_{i,j,k}$ $h_{i,j,k}$ = all head-dependent fluxes entering or leaving the cell due to external sources, and

$Q_{i,j,k}$ = all external fluxes that are not head dependent.

In equation (5), the term QS from equation (4) is divided into fluxes that are head-dependent, and those that are not (McDonald and Harbaugh, 1988).

It is necessary to also represent the time derivative of head in terms of specific heads and times to transform it into the finite difference form. Figure 3 shows a hydrograph of head values at node i,j,k. The values for time are shown on the horizontal axis and the head values associated with these times are also shown. The variable t_m is the time at which the flow terms of Equation 5 are evaluated, and t_{m-1} is the time that precedes t_m. The notation associate with the head terms is consistent with this convention. Then, an approximation to the time derivative of head at time t_m is given by:

$$\left(\frac{\Delta h_{i,j,k}}{\Delta t}\right)^{m} \cong \frac{h^{m}_{i,j,k} - h^{m-1}_{i,j,k}}{t_{m} - t_{m-1}} \tag{6}$$

Where:

$h^{m}_{i,j,k}$ = the head in the cell i,j,k at the end of time step m,

$h^{m-1}_{i,j,k}$ = the head in the cell i,j,k at the end of time step m-1,

t_m = the time at the end of time step m, and

t_{m-1} = the time at the end of time step m-1.

Therefore, the derivative is approximated using the change in head at the node over a time interval over which the flow is evaluated. This is termed a "backward difference" approach to the finite difference equation, because it extends backwards in time. MODFLOW uses this method as opposed to a "forward difference" or any other approach because a "backwards difference" approach is always numerically stable-that is, "errors introduced at any time diminish progressively at succeeding times" (McDonald and Harbaugh, 1988). Equation 5 can be written in backward-difference form in the following way:

$$\begin{aligned} & CR_{i,j-1/2,k}(h^{m}_{i,j-1,k} - h^{m}_{i,j,k}) + CR_{i,j+1/2,k}(h^{m}_{i,j+1,k} - h^{m}_{i,j,k}) \\ & + CC_{i-1/2,j,k}(h^{m}_{i-1,j,k} - h^{m}_{i,j,k}) + CC_{i+1/2,j,k}(h^{m}_{i+1,j,k} - h^{m}_{i,j,k}) \\ & + CV_{i,j,k-1/2}(h^{m}_{i,j,k-1} - h^{m}_{i,j,k}) + CV_{i,j,k+1/2}(h^{m}_{i,j,k+1} - h^{m}_{i,j,k}) \\ & + P_{i,j,k}h^{m}_{i,j,k} + Q_{i,j,k} = SS_{i,j,k}(\Delta r_j \Delta c_i \Delta v_k)\frac{(h^{m}_{i,j,k} - h^{m-1}_{i,j,k})}{(t_m - t_{m-1})} \end{aligned} \tag{7}$$

Equation 7 represents one equation and seven unknown heads at time m, and therefore cannot be solved independently. However, an equation of this type can be written for each active cell in the domain. Since there is only one unknown head for each cell, a system of "n" equations with "n" unknowns is created and can be solved simultaneously (McDonald and Harbaugh, 1988).

The initial head and boundary conditions defined by the user prior to the simulation become the first values of head for the first iteration (McDonald and Harbaugh, 1988). As the time progresses, the head distribution for time m in the preceding time step becomes the head distribution for m-1 in the next time step. Therefore, the set of finite-difference equation is reformulated at each time step (McDonald and Harbaugh, 1988).

2.1.4 Iteration

MODFLOW utilizes iterative methods to solve the system of finite-difference equations for each time step. The calculation of head values at the end of a given time step is arbitrarily assigned a trial estimate for the head at each node at the end of that time step (McDonald and Harbaugh, 1988). A calculation procedure is then initiated that alters the estimated values and results in a new set of values which are closer in agreement with the solution to the equations. These interim head values take the place of the initial estimates and the procedure is repeated. Ultimately, the interim heads approach values that would exactly satisfy the set of equations (McDonald and Harbaugh, 1988).

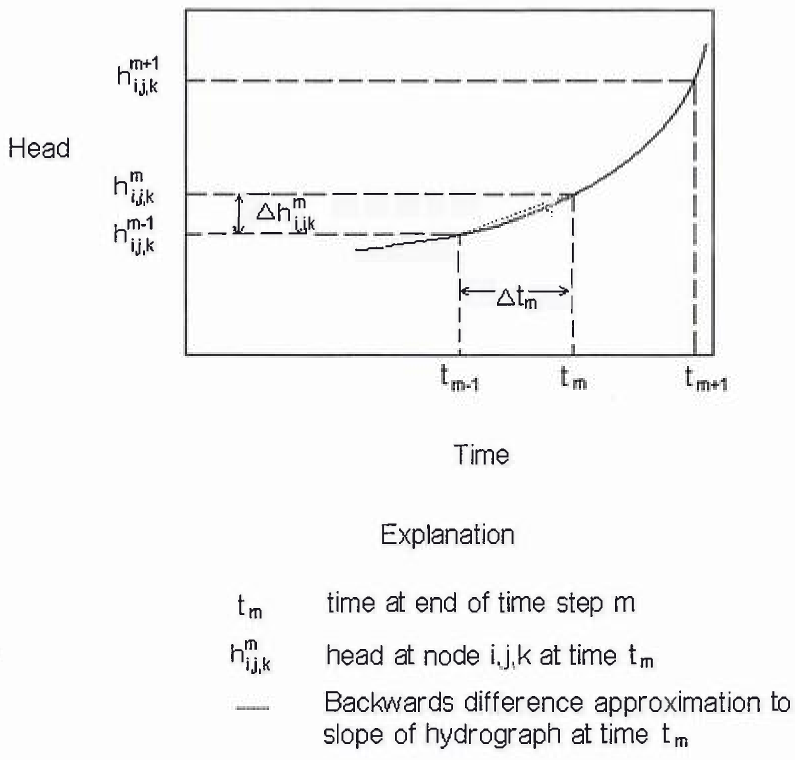


Figure 2. 3 Hydrograph for cell i,j,k in MODFLOW (McDonald and Harbaugh, 1988).

Ideally, the iteration procedure should stop when the calculated heads are suitably close to the approximate solution. However, the actual solution is unknown, and instead, the changes in the computed heads occurring from one iteration to the next must be less than a certain “closure criterion” or “convergence criterion”, specified by the user (McDonald and Harbaugh, 1988)

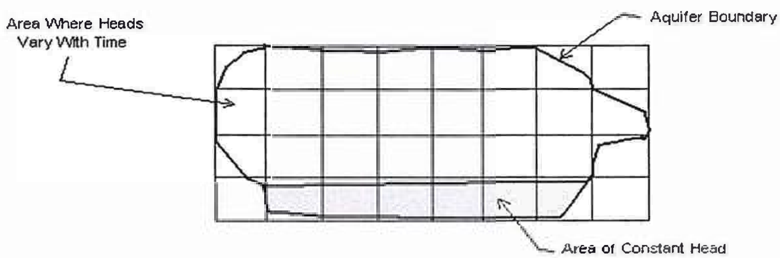
2.1.5 Boundary Conditions

MODFLOW uses boundary conditions to represent the physical and hydraulic boundaries within a groundwater system. Physical boundaries are formed by the physical presence of an impermeable body of rock or large body of surface water. Hydraulic boundaries are invisible boundaries dependent upon hydrologic conditions. These may include groundwater divides and flow lines and can change with the changes of stresses on the system at a given time. MODFLOW can incorporate specified-head boundaries, specified-flux boundaries, and general head boundaries during a simulation.

Specified head boundaries represent an inexhaustible supply of water. This means that the aquifer can potentially pull an infinite amount of water from this source without changing its head value. Therefore, they can accurately represent a large surface body of water that does not undergo a significant change in head, but they cannot accurately represent a body of water that does experience a large fluctuation in head.

Typically, specified-flux boundaries represent no-flow boundaries, but they can also represent constant flux boundary conditions where flow can be measured or estimated to be nonzero. No-flow boundaries can represent both physical conditions such as impermeable bedrock or impermeable fault zones, and hydraulic conditions like groundwater divides and streamlines. Care must be taken, however, in using this type of boundary condition for hydraulic conditions because they can change in time with the stresses on the system.

Finally, general-head boundaries are used whenever the head of a surface-water body or other known head is separated from the aquifer by material or deposits having different Hydro geologic properties than the aquifer. A separate package within the model simulates this type of boundary condition, and is discussed in the explanation of individual packages (McDonald and Harbaugh, 1988).



1	1	1	1	1	1	1	0
1	1	1	1	1	1	1	1
1	1	1	1	1	1	1	0
0	-1	-1	-1	-1	-1	-1	0

IBOUND Codes

< 0 Constant Head
= 0 No Flow
> 0 Variable Head

Figure 2. 4 Typical example of the boundary layer (IBOUND) for a single layer in MODFLOW (McDonald and Harbaugh, 1988).

2.2 Steady-State vs. Transient Models

MODFLOW can be run for “steady-state” or “transient” simulations. A steady-state simulation represents a cross-section in time, and produces one array of hydraulic head values for every cell. The model does not run for a given length of time, but it runs until the system reaches equilibrium, and the residuals have converged given the criterion specified in the Solver Package (McDonald and Harbaugh, 1988). A steady-state simulation is usually performed during the calibration procedure to develop an optimal parameter set. The optimal parameter set is then used for a transient simulation to solve a time-dependant problem, and the data are verified with available data for that time period. Sometimes the transient model will need to be re-calibrated to better match the observed data available for the transient simulation. However, if the steady-state model is adequately calibrated, usually, the re-calibration procedure just requires fine-tuning of the calibrated parameters, if necessary.

2.3 Applications of MODFLOW

MODFLOW has been applied to numerous systems in many different geologic settings for a variety of reasons. MODFLOW simulations are sometimes used to develop a better understanding of the groundwater flow system. For instance, in performing a modeling study, sometimes lack of data in a specific area can be identified, and effort can be made to collect more data to add to the knowledge of the flow system. The primary output from MODFLOW is a resulting hydraulic head distribution for a given simulation. This information can be used to manage existing problems in a groundwater system, evaluate the effects of proposed withdrawal rates from a system, or to evaluate the effects of urbanization on the flow system.

2.3.1 Managing Existing Problems in a Groundwater System

MODFLOW has also been applied to evaluate possible management strategies to alleviate existing problems in groundwater flow systems. In such an application, modeling offers an inexpensive way to evaluate whether or not preventative measures or management decisions will be effective.

Groundwater withdrawals in the Las Vegas Valley, Nevada totaled more than 2.5 million acre-ft between 1912 and 1981 (Morgan and Dettinger, 1994). Effects of heavy pumping are evident over large areas of the valley but are more pronounced near the major well fields.

Secondary recharge from lawn irrigation and other sources is estimated to have totaled more than 340,000 acre-ft during 1972-81. Resulting rises in water levels in shallow, unconfined aquifers have caused widespread water-logging of soils, increased groundwater discharge, and potential for degradation of water quality in deeper aquifers by accentuating downward vertical hydraulic potential. MODFLOW was applied to the study area to evaluate possible management alternatives aimed at alleviating problems related to overdraft and water-logging while maximizing groundwater resources. The use of the model as a predictive tool was demonstrated by simulating the effects of using most municipal wells only during the peak demand season for the period 1972-81. The model determined that long-term rates of water level decline near the well fields would be less, but the magnitude of seasonal fluctuations would increase.

MODFLOW was applied to the New Jersey Pinelands to determine alternate withdrawal strategies on groundwater flow patterns in the area (Modica, 1995). The model was developed to define groundwater flow patterns and residence times in a aquifer system typical of the New Jersey Coastal Plain, and to demonstrate the effects of alternative withdrawal strategies. Results of withdrawal simulations indicate that well-location strategies can alleviate the adverse effects of withdrawals on streams and that large scale regional withdrawals in confined aquifers can adversely affect streams although the effects are dispersed over numerous streams.

2.3.2 Evaluating Effects of Proposed Water Withdrawal

MODFLOW has also been frequently applied to determine the effects of increased water withdrawal, which is one of the primary results of urbanization. Since groundwater is relied on by an increasing number of people as a safe drinking water source, it is important that this increase in water demand does not deplete or lead to the contamination of the groundwater system. Therefore, using MODFLOW to simulate proposed increases in withdrawals serves as a key management practice to managing groundwater effectively.
An existing MODFLOW model was used to simulate changes in groundwater flow caused by hypothetical pumping in an area near the south-eastern part of Carson City, Nevada (Maurer, 1988). A total of five hypothetical pumping patterns were used in the model simulations. The simulations indicate that a maximum of 140 gal/min of induced flow from the Carson River could occur as a result of projected total pumpage of 1,700 gal/min after 10 years; the induced flow could increase 320 gal/min after 50 years. However, river losses were projected to decrease after 10 years and after 50 years when the locations of the pumping centers were moved farther away from the river

A similar study was performed in East Carson Valley, Douglas County, Nevada (Maurer, 1988). An existing MODFLOW model was used to simulate changes in groundwater flow in response to hypothetical increases in groundwater pumpage. Pumpage scenarios that reflect State groundwater permits and pending applications were used in four different simulations to estimate the effect of hypothetical development of groundwater levels, storage, groundwater flow to the Carson River, and groundwater flow to the Carson River, and groundwater consumed by evapotranspiration over a 45-year period. The highest pumping rate caused water level declines as much as 15 ft, decreased water storage by 27,000 acre-ft, decreased groundwater to the Carson River by 4.3 ft_3/sec, and reduced evapotranspiration losses by about 1,200 acre-ft.

A study was performed using MODFLOW to produce simulated effects of proposed groundwater pumping in 17 basins of East-Central and Southern Nevada (Schaefer and Harrill, 1995). The simulations indicate that the proposed pumping would cause water-level declines in many groundwater basins, decreases flow at several regional springs, and decreases discharge by evapotranspiration from the basins. Groundwater levels could ultimately decline several hundred feet in the basins scheduled to supply most of the pumped groundwater. The overall purpose of the study was to estimate potential effects of implementing the proposed water-rights applications filed by the Las Vegas Valley Water District.

MODFLOW was also applied in the Pullman-Moscs Area, Washington and Idaho (Lum, et al. 1990). The model was used to assess the effects of changes in the rate of withdrawal of groundwater on the water levels in the hydrogeologic units and on streamflow in the area. The model results indicate that groundwater levels would stop declining if the pumpage rates were stabilized at a constant level. Levels will continue to decline into the foreseeable future, however, as along as groundwater pumpage continues to increase.

The public water supply in the Rockaway River Valley, Morris County, NJ, depends almost entirely on groundwater from wells in the valley-fill deposits (Gordon, 1993). A steady state groundwater flow model was developed to quantity the effects of groundwater withdrawals on water levels. The purpose of the investigation was to examine aquifer response to current and predicted groundwater withdrawals in areas of proposed well sites and the effect of these withdrawals on groundwater discharge to the river. It was found that the anticipated increases in withdrawals of 11.5 million gallons per day by the year 2000, and 14.6 million gallons per day by the year 2040 would significantly affect water resources in the area. The baseflow to the Rockaway River above the Boonton Reservoir may not be sufficient to meet the minimum required reservoir outflow during extended periods of decreased recharge.

2.3.3 Evaluating the Effects of Land Development

MODFLOW has been applied to different sites to determine the combined effects of increased demand on groundwater as a water source and the decrease in recharge associated with urbanization. These studies include both the effects of increased withdrawal, an impact of urbanization, and sometimes of reduced recharge. The combination of these two effects of urbanization can be detrimental to the management of groundwater, and modeling these effects can aid in the planning of management procedures.

MODFLOW was applied to Carson Valley, a river-dominated basin in Douglas County, NV, and Alpine County, CA to simulate the effects of development on the groundwater system.
The simulations show that surface water flow is the ultimate source of about 75% of pumped water for six scenarios of possible future groundwater development. Model simulations also indicate that changes from agricultural to urban land uses could decrease the loss of Carson River outflow to pumpage when streamflow is not used for flood irrigation in that area.

Intense development of the Miocene aquifer systems for water supplies along the Mississippi Gulf Coast has resulted in large water level declines that have altered the groundwater flow pattern in the area (Sumner, 1987). This study used MODFLOW to investigate the system response to increased withdrawal due to an increase in urban development. The simulation showed that drawdowns caused by large groundwater withdrawals along the coast have resulted in the gradual movement of the saltwater toward pumping centers.

MODFLOW was also applied to streams and springs in small basins typical of the Puget Sound Lowland, WA (Morgan and Jones, 1995). The model was calibrated to predevelopment conditions and then was used to simulate the effects of pumping on natural discharge to streams and springs. Simulations showed that increased well density caused greater water-level decline locally, but, at equilibrium, did not impact the extent of the area affected by reduction of natural discharge to streams and springs. The model also showed that decreased recharge in areas where development had created impervious surfaces had a direct effect on natural discharge rates to streams and springs. Increased recharge, however, increased natural discharge and offset the effects of water withdrawals from wells. The investigators concluded that further analyses of the time-dependent effects of withdrawals would provide additional insights.

MODFLOW was applied in the Portland Basin, Oregon and Washington (Morgan and McFarland, 1994). The model was used to test and refine the conceptual understanding of the flow system and estimate the effects of past and future human-caused changes to the groundwater system. A Recharge\under predevelopment condition was 180 cu ft/sec more than in 1987-88 (post development conditions) due to greater pervious areas in the basin, as compared to post-development conditions. Simulation of the effects of the increased recharge and no well discharge indicates that water levels could have declined as much as 50 feet in response to municipal pumping. The combination of reduced recharge and increased pumpage could have reduced discharge to large rivers by 25 percent, and discharge to small rivers and streams by 16 percent, compared to the predevelopment conditions.

A study was performed on the upper Santa Cruz basin, AZ which used MODFLOW to simulate groundwater flow and potential land subsidence (Hanson and Benedict, 1993). The study evaluated predevelopment conditions, and potential water-level declines and land subsidence as a result of development. The results of projected simulations indicate that a maximum potential subsidence for 1987-2024 ranges from 1.2 feet for an inelastic specific storage of 0.0001 ft to 12 feet for an inelastic specific storage of 0.0015 ft. The simulations were made on the basis of pumpage and recharge rates from 1986 and by using a preconsolidation stress threshold of 100 feet. A permanent reduction in aquitard storage can range from 1 to 12 percent of the potential loss of 3.9 million acre-feet in aquifer storage, which could significantly impact future groundwater supplies.

2.4 Previous Works Around the Study Area

The study by the German consult in 1977 under the Kobo-Alamata Agricultural Development Project (KAADP) undertaken in the area between Kobo and Alamata towns covering part of the Alamata sub basin has concluded that the geological conditions in the east and west of the valley floor are similar and hence the sub surface water inflow and outflow into and from the valley are equal. The KAADP (1977) also reported that the flattening of ground water level in the eastern part of the valley indicates wide-spread ground water losses by evapotranspiration in this part of the valley.

But the reconnaissance study of the Raya Valley Agricultural Development Project (RVADP, 1998) has disproved the above conclusions by the fact that the western plateau and mountains and the valley fill basin are hydraulically independent ground water systems because the western plateau groundwater is discharged on the escarpments via springs rather than as sub-surface inflow into the valley fill basin where as the eastern mountains act as sub surface dam.

According to RVADP (1998), the well inventory in the eastern margins of the valley fill basin shows that the ground water level depth ranges 18-20 meters and the electrical conductivity values did not differ much from the western and central part of the valley (960 µs/cm). More over no direct groundwater discharge (as springs, swamp etc) and no ground water level fluctuation in the inventoried wells was observed. In general the conceptual ground water model of the KAADP is modified in the study of the RVADP (1998).

In the hydrogeology report the RVADP (1998), the Raya Valley is sub-divided into two sub-basins called Alamata sub-basin and Mehoni sub-basin. The average annual groundwater recharge and the static ground water reserve for the whole valley were estimated to be 85.6 Mm^3/year and 7150m^3 respectively. They have tried to show that the regional ground water flow directions of the Raya valley are in the west-east and north-south directions. The fan-like fields along the foot of the western escarpment were considered as groundwater potential sites. They also concluded that the groundwater is suitable for drinking and irrigation purposes but it is hard and needs softening to be used for industrial purposes.

A shallow examination and verification of the RVADP (1998) feasibility report was done by Isaak Gershanovich in the year 2000. He supposed the existence of large size fault of regional extent in the eastern part of the valley accompanied by ramified fracture systems that can be one possible ground water potential zone for future development. As proposed in this report, the deep position of the aquifers in the fractured rocks will be compensated by significant discharge wells. He assumed the valley fill sediments as fans from the western highlands that interweave each other and the existence of buried sediments of predominantly Alluvium origin. He also commented that the density of information used to produce transmissivity map in a scale of 1:50,000 is not sufficient to map at this scale. He estimated the exploitable groundwater potential of Raya valley to be 130 Mm^3/year rather than 162 Mm^3/year (estimate of RVADP). His conceptual model for the Raya valley was a fan-like field where water bearing layers extend to a large distance under the plain sinking deeper and deeper towards the central and eastern parts of the valley.

According to Dessie Nedaw (2003), (the main purpose of his work was to study the groundwater of Raya valley from geological, hydrological and chemical point of view) the mean annual rain fall and mean actual evapotranspiration of the valley are found to be 779 mm and 695 mm respectively. The annual evaporation from Lake Ashange was estimated to be 1204mm and surface water outflow 60 $Mm^3$from the Raya Valley. The annual recharge to the whole Raya valley was computed to be 129.3 $Mm^{3.}$ Based on calculated transmissivity values potentiality of the aquifers of the valley are classified from high (> 500 m^2/day) to weak (< 0.5 – 5 m2/day). As in the other previous works, he also concluded that high potentiality aquifers are found in the western margin of the valley floor. He indicated that Ca – Mg – HCO3 or Mg – Ca – HCO3 type groundwater is common in the Raya valley. Different from other previous studies, he concluded that there are two separate ground water flow systems named as the Alamata sub-basin flow system and the Mehoni sub-basin flow system.

Generally, all the stated previous works has recommended that their works are just giving a general out look of the regional hydrogeological setting of the Raya valley and further more detailed local studies are mandatory so that a better hydrogeological knowledge of the area will be attained and there by a sustainable development and usage of the water resources of the Raya valley will be achieved.

2.5 Estimation of Ground Water Recharge

2.5.1 Soil Moisture Balance Approach

Quantification of the rate of natural ground water recharge is a basic pre-requisite for efficient ground water resource management. It is particularly important in regions with large demands for ground water supplies, where such resources are the key to economic development. However, the rate of aquifer recharge is one of the most difficult factors to measure in the evaluation of ground water resources. The main techniques used to estimate ground water recharge rates are the Darcian approach, the soil water balance approach and the ground water level fluctuation approach. Estimation of recharge, by whatever method, is normally subject to large uncertainties and errors.

Rainfall is the principal means for replenishment of moisture in the soil water system and recharge to ground water. Moisture movement in the unsaturated zone is controlled by capillary pressure and hydraulic conductivity. The amount of moisture that will eventually reach the water table is defined as natural ground water recharge. The amount of this recharge depends upon the rate and duration of rainfall, the subsequent conditions at the upper boundary, the antecedent soil moisture conditions, the water table depth and the soil type.
Water balance models were developed in the 1940s by Thornthwaite (1948) and revised by Thornthwaite and Mather (1955). The method is essentially a book-keeping procedure which estimates the balance between the inflow and outflow of water. In a standard soil water balance calculation, the volume of water required to saturate the soil is expressed as an equivalent depth of water and is called the soil water deficit. The soil water balance can be represented by:

$$G_r = P - E_a + \Delta S - R_o \qquad (8)$$

Where, Gr= recharge;

P = precipitation;

E_a= actual evapotranspiration;

ΔS = change in soil water storage; and

R_o = run-off.

2.5.1.1 SCS Rain fall – Runoff Relationship

The runoff curve number method for the estimation of direct runoff from storm rainfall is well established in hydrologic engineering. Its popularity is rooted in its convenience, its simplicity, and its responsiveness to four readily grasped catchment properties: soil type, land use/treatment, surface condition, and antecedent moisture condition. The method was developed by the USDA Soil Conservation Service (SCS, 1985).

In developing the SCS rainfall-runoff relationship, the total rainfall was separated into three components: direct runoff (Q), actual retention (F), and the initial abstraction (I_a).

Conceptually, the following relationship between P, Q, I_a, and F was assumed:

$$\frac{F}{S} = \frac{Q}{P - I_a} \qquad 9$$

in which S is the potential maximum retention. The actual retention is

$$F = (P - I_a) - Q \qquad 10$$

Substituting equation (10) into equation (9) yields the following:

$$\frac{(P - I_a) - Q}{S} = \frac{Q}{P - I_a} \qquad 11$$

Rearranging equation (11) to solve for Q yields

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad 12$$

Equation (12) contains one known, P, and two unknowns, I_a and S. Before putting equation (12) in a form that can be used to solve for Q, it may be worthwhile examining the rationality of the underlying model of equation (9). The initial abstraction is the amount of rainfall at the beginning of a storm that is not available

for runoff; therefore, ($P-I_a$) is the rainfall that is available after the initial abstraction has been satisfied. Letting K_1 equal the ratio of Q to ($P-I_a$),
K_1 represents the proportion of water available that directly runs off. If S is the amount of storage (e.g., depression, interception, subsurface) available to hold rainfall, $K_2 = F/S$ is the proportion of available storage that is filled with rainwater. Equation (9) indicates that $K_1 = K_2$; in other words, the proportion of available storage that is filled up equals the proportion of available water that appears as runoff. Given equation (10), there are two unknowns to be estimated, S and I_a. The retention S should be a function of the following five factors: land use, interception, infiltration, depression storage, and antecedent moisture. Empirical evidence resulted in the following equation:

$$I_a = 0.2\,S \qquad 13$$

If the five factors above affect S, they also affect I_a. Substituting equation (13) into equation (12) yields the following equation, which contains the single unknown, S:

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \qquad 14$$

Equation (14) represents the basic equation for computing the runoff depth, Q, for a given rainfall depth, P. It is worthwhile noting that while Q and P have units of depth (e.g., mm), Q and P reflect volumes and are often referred to as volumes because we usually assume that the same depths occurred over the entire watershed.

In order to use equation (14) to compute the runoff for a given P, it is necessary to provide a means for estimating the one unknown, S. For this purpose, the SCS runoff curve number (CN) was developed. A curve number is an index that represents the combination of a hydrologic soil group and a land use and treatment class. Empirical analyses suggest that the CN is a function of three factors: soil group, the cover complex, and antecedent moisture conditions.

SCS developed a soil classification system that consists of four groups, which are identified by the letters A, B, C, and D. The SCS cover complex classification consists of three factors: land use, treatment or practice, and hydrologic condition. There are approximately 35 different land uses that are identified in the tables for estimating runoff curve numbers. Agricultural land uses are often subdivided by treatment or practices, such as contoured or straight row; this separation reflects the different hydrologic runoff potential that is associated with variation in land treatment. The hydrologic condition reflects the level of land management; it is separated with three classes: poor, fair, and good. Not all of the land uses are separated by treatment or condition. Antecedent soil moisture is known to have a significant effect on both the volume and rate of runoff. Recognizing that it is a significant factor, SCS developed three antecedent soil moisture conditions, which were I, II and III.

As indicated previously, the CN was developed for use with equation (14). Thus, there was a need to relate S, which was unknown of equation (14), and the runoff CN. An empirical analysis led to the following relationship:

$$S = \frac{25400}{CN} - 254 \qquad 15$$

Equations (14) and (15) can be used to estimate Q when the values of P and CN are available. It is important to note the following constraint on equation (14):

If $P \geq 0.2S$ use equation 14

If $P \leq 0.2S$ it is necessary to assume that $Q = 0$

3 METHODS AND MATERIALS

3.1 Methodology

3.1.1 Data Collection

The following data was collected

- Topo-map & aerial photograph from REST which is required for the preparation of land use map, soil map, and drainage map.
- Meteorological data from office of Meteorological agency
- Pumping test data of drilled wells from office of Tigray water works enterprise and REST.
- Recent literatures related to the research topic was collected from internet & libraries

3.1.2 Data Processing and Analysis

3.1.2.1 Ground Water Flow Model

Groundwater flow system of the study area was simulated numerically by using a computer code, the U.S. Geological Survey modular tow-dimensional finite difference groundwater flow model, MODFLOW (Harbough et al., 2000). MODFLOW uses input arrays (grided data) that describe hydraulic parameters such as hydraulic conductivity and recharge, top and bottom elevation of the aquifers and boundary conditions.

Prior to the construction of the two dimensional groundwater flow model, a conceptual model of the system was developed on the basis of the secondary data collected from different institutions, previous works, field observations and groundwater literature. By analyzing the collected data, the input parameter for the numerical model was estimated. Different boundary conditions were estimated from physical and hydrogeologic boundaries. Hydraulic parameters such as hydraulic conductivity and storage coefficient were calculated from pumping test data whenever it is available and from literature for the others depending on the aquifer type. Besides, hydraulic head map was established from the water level measured during field data collection and drilling reports. Groundwater recharge was determined using soil moisture balance method and withdrawal was obtained from daily record of the Town Water Supplies, and irrigation practices.

After the aquifer parameters and hydrologic stresses were put in to the numerical model, the model was calibrated using trial and error method by rearranging the parameters and stresses within plausible range to get the best fit between the observed and simulated heads. The calibration was done under steady state.

Prediction for future groundwater management condition was done by assigning different aquifer system conditions, hydrologic stresses, and pumping conditions under different scenarios. Finally, the best pumping condition was selected under normal hydrologic stress.

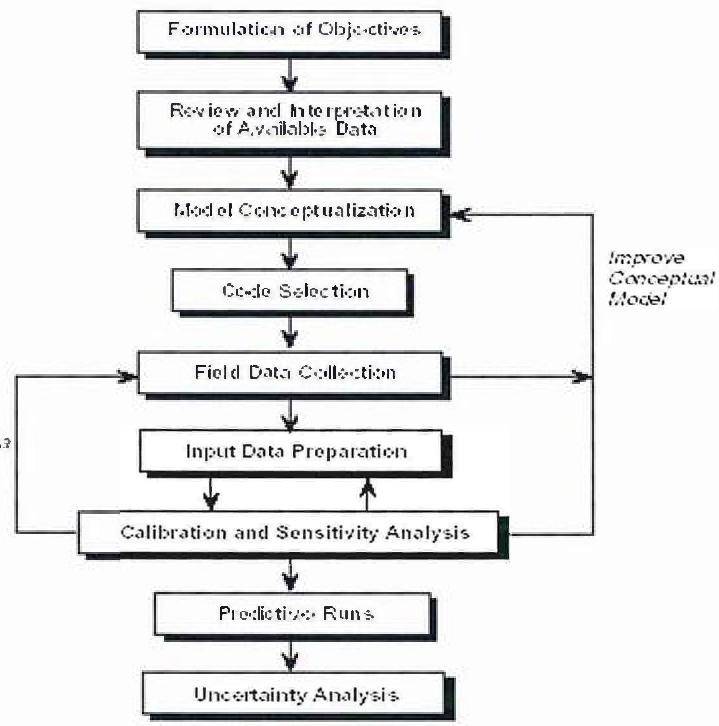


Figure 3. 1 Model application process.

3.1.2.2 Ground Water Recharges from Rainfall Using Water Balance Simulation Techniques.

Assuming that the change in soil moisture storage (ΔS), surface runoff (Ro), actual Evapotranspiration(AET) and precipitation (P) ,Ground water recharge can be computed by the revised Thornthwaite and mather(1955) water balance equation.

$$Gr = P - AET - Ro + \Delta S \qquad 16$$

Where;

AET= PET if P>PET

AET= P + ΔS if P<=PET

PET can be calculating using FAO Penman Montheith method

3.1.2.3 Runoff Estimation

- using curve number (CN) method,

3.1.3 Data Analysis

After gathering the data, editing, coding and enter in to the computer, analysis using arc GIS 9.1 to gather with arc hydro, Globalmaper, Surfer, Aquifer test and MODFLOW software sequentially has been made and managed by the researcher.

3.2 Materials

In this study the following materials has been used

- GPS; to determine the position of drilled well and hand dug well in the basin
- Topographic map and aerial photo; to prepare land use, soil, and drainage map.
- Arc GIS 9.1, Arc hydro, Global maper, Surfer, MODFLOW and Aquifer test soft wares; to manipulate and analyze the data.

Arba Minch

4 RESULTS AND DISCUSSIONS

4.1 Groundwater Recharge Estimation Using Soil Moisture Balance Approach

Water balance models were developed in the 1940s by Thornthwaite (1948) and revised by Thornthwaite and Mather (1955). The method is essentially a bookkeeping procedure which estimates the balance between the inflow and outflow of water. In a standard soil water balance calculation, the volume of water required to saturate the soil is expressed as an equivalent depth of water and is called the soil water deficit. The soil water balance can be represented by:

$$Gr = P - AET + \Delta S - Ro \quad \ldots\ldots \qquad 17$$

Where,

Gr= recharge;

P= precipitation;

AET= actual Evapotranspiration;

ΔS= change in soil water storage; and

Ro= run-off

This method is valuable for helping to phrase precise questions about the chance of success, mode of operation, and environmental impact of proposed changes.

And this method answers the following questions

What is the magnitude of Evapotranspiration demand that is not satisfied by rainfall during the dry part of the year?

When would one expect run off derived from excess of moisture over the demand of Evapotranspiration , and what is the magnitude of this excess?

4.1.1 Computation of PET

Climatic variables required for the computation of PET using the Modified Penman Montheth method in the CROP-WAT package include, minimum and maximum temperature, relative humidity, wind speed and sunshine duration. These variables are computed from the recorded time series data of Kobo and Alamata stations. For Alamata sub basin temperature is adapted from the average of Waja and Alamata station. Where as there is no wind speed and sunshine duration data of Alamata station these variables are adapted from Kobo station which is located at very close distance from the project area.

The climatic input data used for the estimation of monthly potential evapotranspiration and the estimated potential evapotranspiration of the basin is presented in Table 4.1

Table 4. 1 Computed Potential Evapotranspiration for Alamata Sub-basin

Month	Max Temp.(c)	Min Temp(c)	Humidity (km/d)	Wind speed (km/d)	Sunshine (hours)	Solar Ra ($MJ/m^2/d$)	ETo(mm/d
Jan	27.0	12.0	60.0	249.7	7.9	21.0	5.2
Feb	27.5	13.0	57.0	178.0	7.7	21.4	4.9
Mar	29.5	14.0	51.0	182.3	8.1	22.3	5.5
April	39.0	15.6	52.0	164.2	8.1	21.6	5.2
May	32.5	16.5	42.0	145.2	8.4	20.8	5.4
June	34.2	17.5	66.0	106.6	5.0	17.5	5.7
July	31.6	17.0	67.0	105.2	5.3	15.5	3.8
Aug	30.5	15.7	67.0	105.2	5.3	16.8	3.9
Sept	30.5	15.0	61.0	103.7	6.7	19.7	4.4
Oct	30.1	13.6	53.0	112.3	8.6	22.7	4.9
Nov	28.5	12.9	46.0	126.1	9.1	22.8	4.9
Dec	27.5	12.3	54.0	139.1	8.3	21.2	4.6
Average	30.7	14.6	51.0	156.0	7.6	20.5	4.9

4.1.2 SCS Rainfall-Runoff Relation

The following step-by-step procedure has been followed to find surface runoff using curve number method

1. Find hydrologic soil groups in the watershed, as per the following criteria:

Soil Group Infiltration Capacity (Cm/hour)

A................................ 7.5 - 11.5

B .... 4.0 - 7.5

C................................ 0.13-4.0

D................................ 0-0.13

Group A: Sand, loamy sand or sandy loam. Soils having a low runoff potential due to high infiltration rates. These soils primarily consist of deep, well-drained sands and gravels

Group B : Silt loam, or loam. Soils having a moderately low runoff potential due to moderate infiltration rates. These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C: Sandy clay loam. Soils having a moderately high runoff potential due to slow infiltration rates. These soils primarily consist of soils in which a layer exists near the surface that impedes the downward movement of water or soils with moderately fine to fine texture.

Group D: Clay loam, silty clay loam, sandy clay, silty clay or clay. Soils having a high runoff potential due to very slow infiltration rates. These soils primarily consist of clays with high swelling potential, soils with permanently-high water tables, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious parent material.

Data from direct field measurements on soil permeability and infiltration rates for Ethiopian soils are very limited. Data is generally available only for soil types located near major irrigation projects and agricultural research stations. The hydrological soils groups presented in Table 4.2 are based on limited field measurements and from profile morphology and physical characteristics, and are subject to further review and refinement.

Table 4. 2 Hydrological soil groups

Soil Types	*Hydrologic Soil Group*
Ao Orthic Acrisols	B
Bc Chromic Cambisols	B
Bd Dystric Cambisols	B
Be Eutric Cambisols	B
Bh Humic Cambisols	C
Bk Calcic Cambisols	B
Bv Vertic Cambisols	B
Ck Calcic Chernozems	B
E Rendzinas	D
Hh Haplic Phaeozems	C
Hl Luvic Phaeozems	C
I Lithosols	D
Jc Calcaric Fluvisols	B
Je Eutric Fluvisols	B
Lc Chromic Luvisols	B
Lo Orthic Luvisols	B
Lv Vertic Luvisols	C
Nd Dystric Nitosols	B
Ne Eutric Nitosols	B
Od Dystric Histosols	D
Oe Eutric Histosols	D
Qc Cambric Arenosols	A
Re Calcaric Regosols	A
Re Eutric Regosols	A
Th Humic Andosols	B
Tm Mollic Andosols	B
Tv Vitric Andosols	B
Vc Chromic Vertisols	D
Vp Pellic Vertisols	D
Xh Haplic Xerosols	B
Xk Caloic Xerosols	B
Xl Luvic Xerosols	C
Yy Gypsic Yermosols	B
Zg Gleyic Solonchaks	D
Zo Orthic Solonchaks	B

Source: Ministry of Agriculture

2. Find antecedent moisture condition (AMC) from 5-day antecedent rainfall.

AMC Group	Dormant Season (cm)	Growing Season (cm)

I	< 1.3	< 3.6
II	1.3 to 2.8	3.6 to 5.4
III	> 2.8	> 5.4

3. Areas of hydrologic soil groups determined in step (1) and superimpose on the land use map showing fallow land, row crops, small grain, pasture, meadow, roads etc.

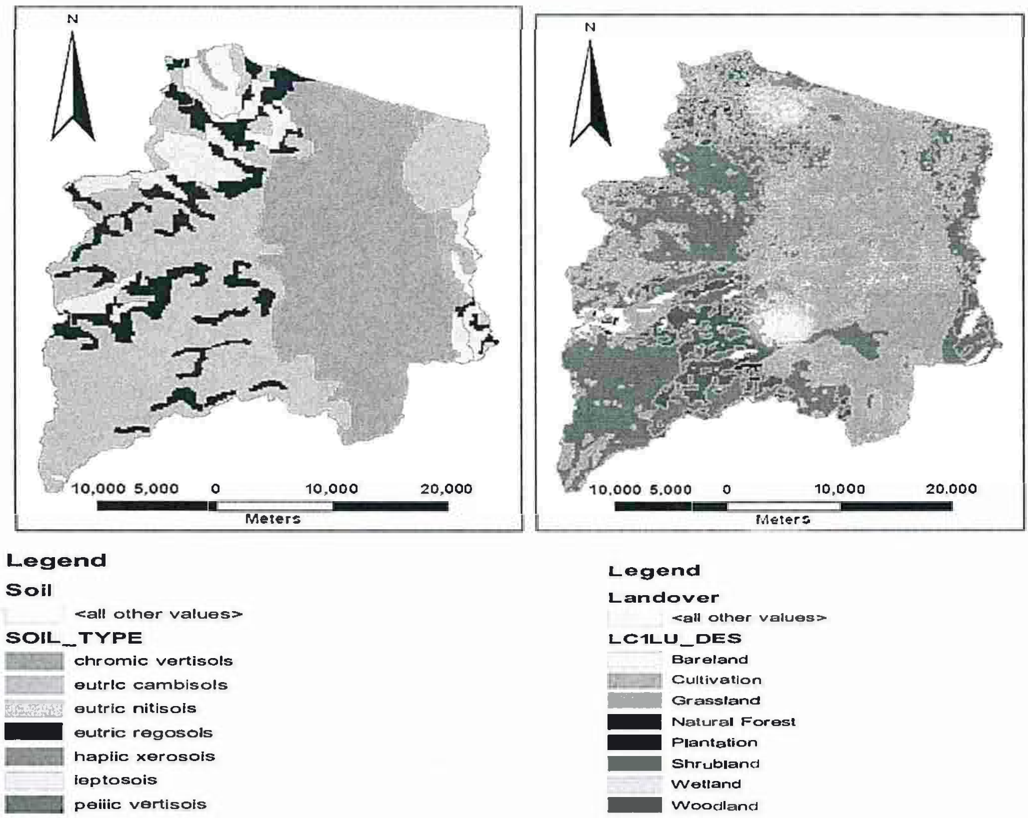


Figure4. 1 Soil and land cover map to be superimposed

4. For each of the demarcated area (A1, A2..., B1, B2...., C1, C2..., D1, D2...,) obtained in step (3), find the runoff curve number, CN from SCS method tables depending upon the land use, treatment or practice, hydrologic soil group and antecedent Moisture condition group(AMC).

Table 4. 3 Hydrological soil group of Alamata basin

	Hydrological Soil Group			
LANDUSE	A	B	C	D
Shrubland; Open (20-50% woody cover)	43	65	76	82
Cultivated Land; Rainfed; Cereal Land Cover System; unstocked (woody p!)	30	55	70	77
Shrubland; Dense (>50% woody cover)	35	56	70	77
Woodland; Open (20-50% tree cover)	55	69	78	83
Grassland; unstocked (woody plant)	57	73	82	86
Cultivated Land; Rainfed; Cereal Land Cover System; lightly stocked	45	58	72	79
Forest; Montane coniferous; Open (20-50% crown cover)	58	72	81	85
Bare land; Exposed sand / soil	77	86	91	94
Cultivated Land; Irrigated	72	81	88	91
Bare land; Exposed rock	59	74	82	86
Grassland; lightly stocked	49	69	79	84
Bare land; Exposed rock	59	74	82	86

Table 4. 4 Curve Number generated using Hec Geohms

Sub Basin	CN	Ia	Lag Time(min)	Sub Basin	CN	Ia	Lag Time(min)
W1000	62.773	30.12651	92.108	W890	79.519	13.0841	293.78
W1010	76.043	16.00431	212	W900	59.086	35.17637	147.3638949
W1020	64.043	28.52171	116.49	W910	58.957	35.36449	105.3526737
W1030	78.527	13.89113	125.15	W920	64.772	27.62895	117.34
W1040	79.936	12.75084	155.63	W930	75.076	16.86477	144.22
W1050	69.355	22.44634	124.22	W940	76.345	15.74005	217.4080035
W1060	76.248	15.8247	105.0323295	W950	77	15.17403	32.621
W1070	75.132	16.81433	105.98	W960	77.902	14.41014	181.7365539
W560	78.081	14.26064	183.93	W970	68.4	23.46901	94.292
W570	77	15.17403	35.048	W980	79.157	13.37626	211.0427208
W580	77	15.17403	13.8002409	W990	78.656	13.78503	168.66
W590	78.111	14.23565	684.2015148	W1000	62.773	30.12651	92.108
W600	77	15.17403	25.341	Total	3889.429		
W610	79.305	13.25649	471.8052105				
W620	74.382	17.49609	453.19				
W630	77.835	14.46627	696.11				
W640	78.287	14.08945	617.4751842				
W650	67.293	24.69076	126.59				
W660	77.272	14.9418	410.64				
W670	78.485	13.92574	605.56				
W680	77	15.17403	32.132				
W690	77.736	14.54939	357.66				
W700	78.119	14.22899	491.71				
W710	77.43	14.80765	277.62				
W720	77	15.17403	26.038				
W730	62.623	30.32036	220.7847792				
W740	78.244	14.12511	436.03				
W750	74.711	17.19534	121.91				
W760	77	15.17403	12.886				
W770	77.367	14.86107	357.98				
W780	77.443	14.79663	250.72				
W790	77.4	14.83307	190.89				
W800	76.346	15.73918	73.598				
W810	60.902	32.6127	191.36				
W820	64.232	28.2883	159.3897585				
W830	69.005	22.81785	223.81				
W840	77.303	14.91543	291.14				
W850	65.783	26.4236	115.97				
W860	60.727	32.85307	154.1				
W870	65.453	26.81294	293.24529				
W880	77.854	14.45034	231.14				

i For normal antecedent moisture condition AMC (II) the equivalent CN is computed by

Weighted curve number CN $= \frac{3889.429}{53} = 73.4$ =

ii. For AMC (I) the equivalent CN is computed by;

$$\text{CN (I)} = \frac{4.2CN(II)}{10-0.058CN(II)} = 53.7$$

ii. For AMC (III) the equivalent CN is computed by;

$$\text{CN (III)} = \frac{23CN(II)}{10+0.13CN(II)} = 86.4$$

The runoff curve number (CN) of 73 for antecedent moisture condition three (dry condition) is considered representative for all the catchments. (Ethiopian road authority)

Conversion from average antecedent moisture conditions to dry and wet conditions

CN For Average Conditions	Corresponding CN's For	
	Dry	Wet
100	100	100
95	87	98
90	78	96
85	70	94
80	63	91
75	57	88
70	51	85
65	45	82
60	40	78
55	35	74
50	31	70
45	26	65
40	22	60
35	18	55
30	15	50
25	12	43
15	6	30
5	2	13

The soil conservation method is used to determine the excess runoff

$$R=\frac{\left[P-0.2\left(\frac{25400}{CN}-254\right)\right]^2}{P+0.8\left(\frac{25400}{CN}-254\right)}$$

Where: R_O = Runoff, mm

P = Rainfall

CN = Curve number = 73

Table 4. 5 Monthly runoff

Month	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Avg
MMRF (mm)	35.5	45.5	65.5	87.5	67.5	18.5	156	179.5	55.5	27.5	23	33.5	60.7
CN	73	73	73	73	73	73	73	73	73	73	73	73	73
Runoff (mm)	0.0	0.0	0.0	0.0	0.0	0.0	76.9	101.4	0.0	0.0	0.0	0.0	0.0

4.1.3 Determination of the Soil Moisture Storage

Each type of soil has its own specific capacity to store moisture in its pores. Hence determination of soil moisture content at different depth and places may be needed to understand the available moisture storage capacity in the soil.

The major textures of the investigated soils vary from loam to silty loam. The result of laboratory analysis of surface soil showed that the content of silt varied from 22 % to 68% with an average of 47.40%. The dominant soil texture of the project area is silty loam (RVPIP, Soil survey final feasibility report, 2008)

The available water capacity of the soils in the study area is given in the table below and the water retained in the soil can be read from figure below.

Table 4. 6 Moisture characteristics value and soil textural relationship. (Source RVPIP, Soil survey final feasibility report, 2008)

Soil Texture	AWC mm/m (Actual Measured)	FAO standard
Clay	140	200
Loam	170	140
Loamy Sand	120	60
Silt	190	140
Silty Clay	160	200
Silty Loam	190	140
Clay Loam	150	200

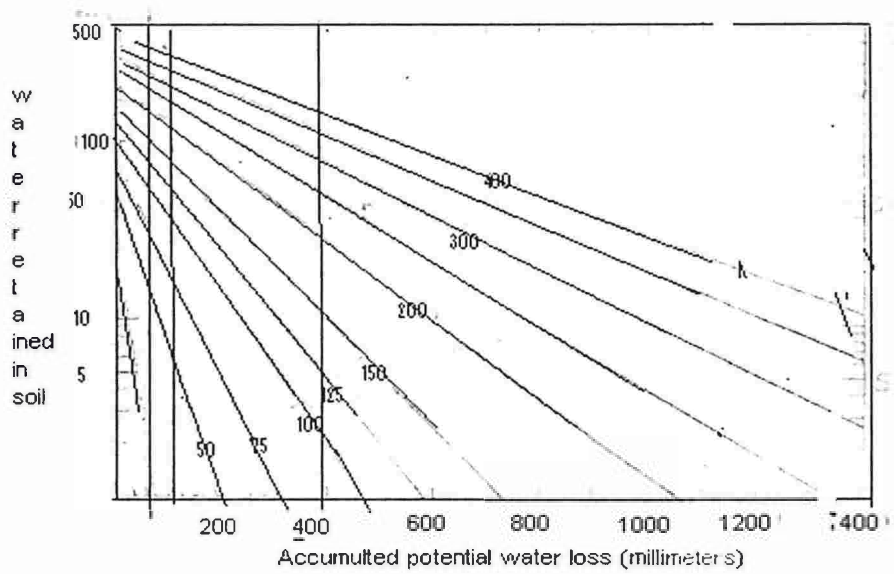


Figure 4. 2 Water retained in the soil against accumulated potential water loss. The number on each curve is the available water capacity for the soil in millimeter (Data from Thornthwaite and Mather)

4.1.4 Calculation of the Water Balance

Soil moisture budgeting, taking in to account Evapotranspiration abstraction from precipitation, provides a measure of moisture availability for runoff and ground water recharge. This can be done by Thornthwaite (1945) book keeping method of moisture balance, which has gained wide application. The available water capacity is obtained from table 15. The difference between the moisture surplus and runoff gives ground water recharge

If a soil with an available water holding capacity of 190mm is subjected to a potential water loss of 424.5mm on the month of December, the amount of water that will be retained by the soil is 30mm. This value of soil moisture is entered in the fifth row of table 15.and the process is repeated for other dry season months. Soil moisture values for the other wet-season months are obtained by adding the excess precipitation from row 3 to the moisture level at the end of the dry seasons. The detail calculation is given in table 4.7

Table 4. 7 Monthly water balance at Alamata basin

For a soil with available water holding capacity of 190mm.The soil is silty loam. All values in this table are in mm

item	Month												
	Jan	Feb	Mar	April	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Anual
P	35.5	45.5	65.5	87.5	67.5	18.5	156	179.5	55.5	27.5	23	33.5	795
PET	156	147	165	156	162	171	114	117	132	147	147	138	1709
P-PET	-120.5	-101.5	-99.5	-68.50	-94.5	-152.50	42.0	62.5	-76.5	-119.5	-124	-104.5	
Acc.pwl	-545.0	-646.5	-746	-814.0	-909	-1061.5			-76.5	-196.0	-320	-424.5	
SM	12	7.5	5.2	2.8	1.3	0	42.0	104.5	140	71	45	30	
ΔSM	-18	-4.5	-2.3	-2.4	-1.5	-1.3	42.0	62.5	35.5	-69	-26	-15	0.0
AET	17.5	41	63.2	85.1	66	17.2	114	117	91	-41.5	-3	18.5	586
R_O	0.0	0.0	0.0	0.0	0.0	0.0	76.8	101.4	0.0	0.0	0.0	0.0	178.2
Gr	0.0	0.0	0.0	0.0	0.0	0.0	7.2	23.6	0.0	0.0	0.0	0.0	30.8

In order to calculate the AET then a water balance method originally developed by Thornthwaite and Mather (1957), are used. This requires constructing a table that consists of Precipitation (P) , PET, P-PET, and accumulated potential water loss (ACPWL) as the summation of the successive, P-PET value starting from the dry month. Then soil moisture storage will be calculated using maximum available water and the ACPWL using figure 4.2 next to this raw the change in soil moisture storage will be calculated for successive months. After this the AET will be PET when P > PET but if P < PET the AET will be the summation of the change in soil moisture storage plus the PPT. Ro= surface run off, Gr= P-AET+ΔSM-Ro

Note; For the purpose of water-balance accounting, it is good to focus on the soil moisture available at the end of one month, which for the next month becomes the available initial soil moisture.

4.2 Hydraulic Properties of the Aquifer

The ability of an aquifer to transmit, store and release water are described by hydraulic conductivity / transmissivity and specific yield / storage coeficient of the aquifer(s).

These parameters are collectively known as the hydraulic properties of the aquifer. For the Aquifer parameter calculation basically two techniques are used based on the type of data available and the kind of aquifer under investigation.eleven wells with pump test data has been used. The techniques employed are Cooper Jacob straight line method and Theis and Jacob recovery method..

4.2.1 Cooper and Jacob straight line method.

Constant discharge pumping test analysis method uses a continuous reading of drawdown with successive time under constant pumping rate and this test should be conducted in an observation well but for our case the distance(r) from the pumping well to the observation well is taken one meter

The steps are given below:

Step 1: plot the data of one of the observation wells in single logarithm paper (t in logarithm scale) and draw a line through the points .

Step 2: find the interception point of the line with the time axis where the drawdown (S) is zero. Read the value of t_o

Step 3: determine the geometric slop of the line, that is, the drawdown difference ΔS per log cycle of time.

Step 4: introduce the values of Q, ΔS and $\Delta(\log t)$ in to the equation below to determine the value of T .for one log cycle of time, $\Delta(\log t) = 1$ and the corresponding ΔS should be taken from the graph. More than one log cycle can also be taken. :

$$T = \frac{2.303Q}{4\prod \Delta S}\Delta \log(t)$$

Step 5: with the known values of T, to, r calculate the storage coefficient (S) using equation below.

$$S = \frac{2.25Tt_o}{r^2}$$

4.2.2 Theis and Jacob Recovery method.

This method is mainly used to counter check the reliability of aquifer parameter results calculated from pumping test data. When the pump is shut down after a pumping test, the water levels in the well will start to rise. This rise in water levels is known as residual drawdown, s'. Wheres' is the drawdown expressed as the difference between the original water level before the start of pumping and the water level measured at a time t' after the cessation of pumping. Residual draw down data are more reliable than pumping test data because recovery occurs at a constant rate, where as a constant discharge during pumping is often difficult to achieve in the field. The Theis recovery method is applicable to data from single well recovery tests conducted in confined, leaky or unconfined aquifers (Kruseman et.al, 1990). The Theis and Jacob recovery method is applied by using the Aquifer Test program in all wells which has a recovery data after the constant pumping test.

Q=36.3l/sec=3136.32m^3/day

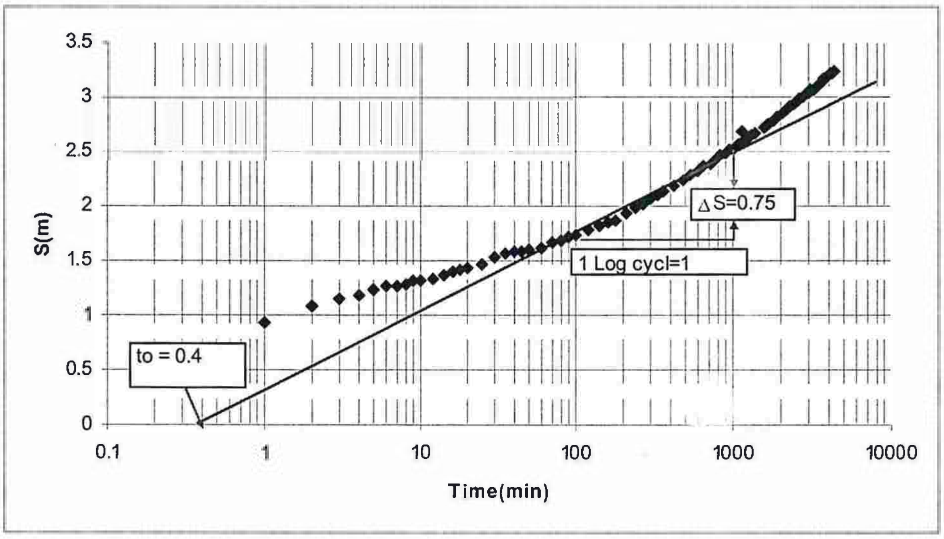


Figure 4. 3 Cooper and Jacob straight line method for well PZ3

Using Cooper and Jacob straight line method using time drawdawn data of well number PZ-3 with constant flow rate of 36.3l/sec transmitivity and storotivity of the aquifer is obtained as follows

$$T = \frac{2.303Q}{4\prod \Delta S}\Delta(\log t) = \frac{2.303 * 3136.32}{4 * 3.14 * 0.75} = 766.77 \text{m}^2/\text{day}$$

$$S = \frac{2.25Tt_o}{r^2} = \frac{2.25 * 766.77}{1^2} * \frac{0.4}{60 * 24} = 0.48$$

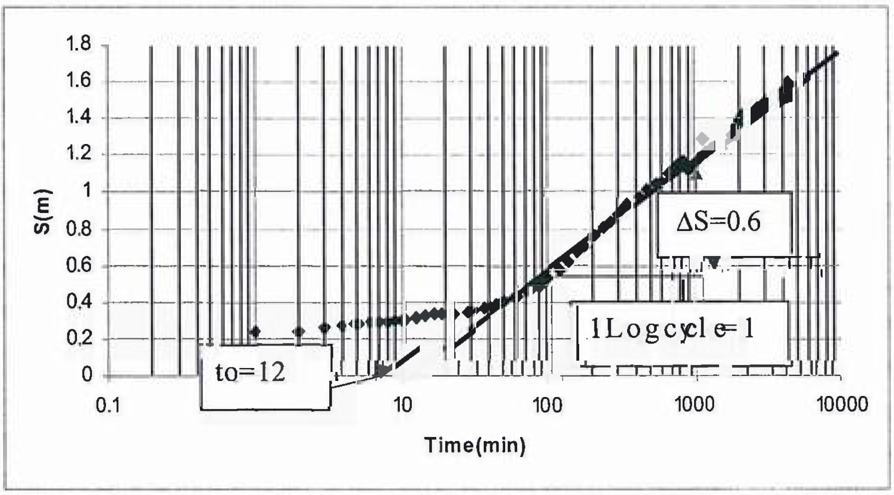


Figure 4. 4 Cooper and Jacob straight line method for well PZ5

Using Cooper and Jacob straight line method using time drawdown data of well number PZ-5 with constant flow rate of 32l/sec transmitivity and storotivity of the aquifer is obtained as follows

$$T = \frac{2.303Q}{4\prod \Delta S}\Delta(\log t) = \frac{2.303 * 2764.8}{4*3.14*0.6} = 844.92 m^2/day$$

$$S = \frac{2.25Tt_o}{r^2} = \frac{2.25 * 844.92}{1^2} * \frac{12}{60*24} = 15.84$$

Q=7l/sec

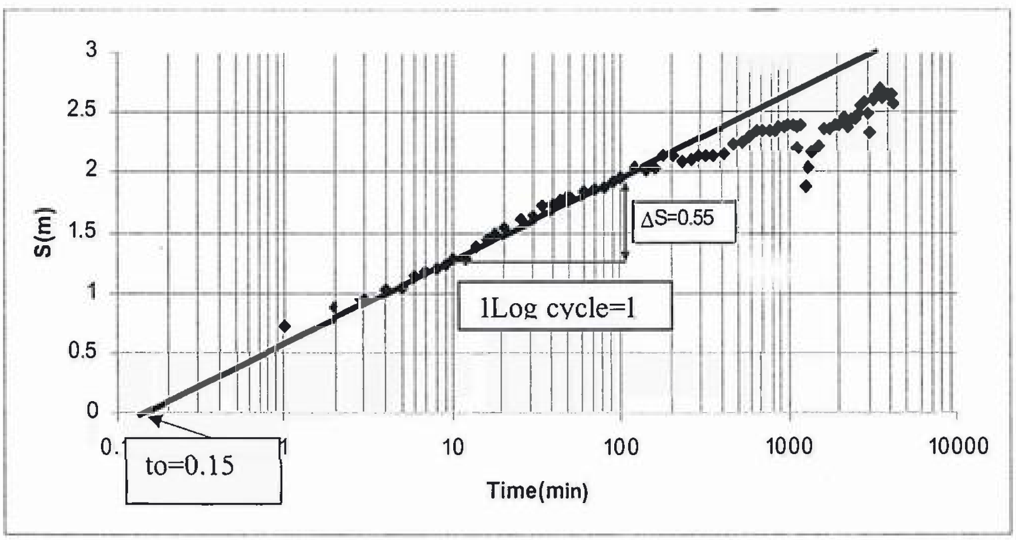


Figure 4. 5 Cooper and Jacob straight line method PZ-9

Using Cooper and Jacob straight line method using time drawdown data of well number PZ-9 with constant flow rate of 7l/sec transmitivity and storotivity of the aquifer is obtained as follows

$$T = \frac{2.303Q}{4\prod \Delta S}\Delta(\log t) = \frac{2.303 * 604.8}{4*3.14*0.55} = 201.63\text{m/day}$$

$$S = \frac{2.25Tt_o}{r^2} = \frac{2.25 * 201.63}{1^2} * \frac{0.15}{60 * 24} = 0.0473$$

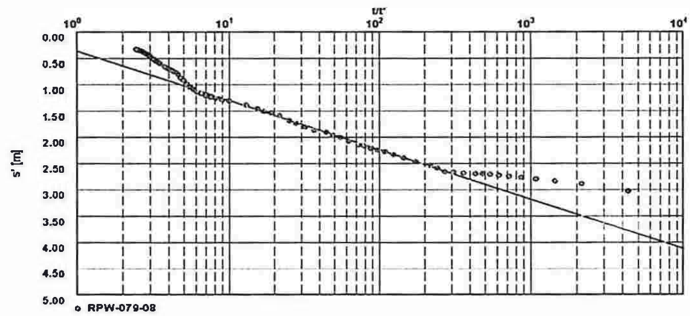


Figure 4. 6 Recovery test analysis curve after THEIS&JACOB for well RPW-7

The transmisivity and hydraulic conductivity values are $4.98\times10^{-1}m^2/min$, 8.49×10^{-3} m/min respectively.

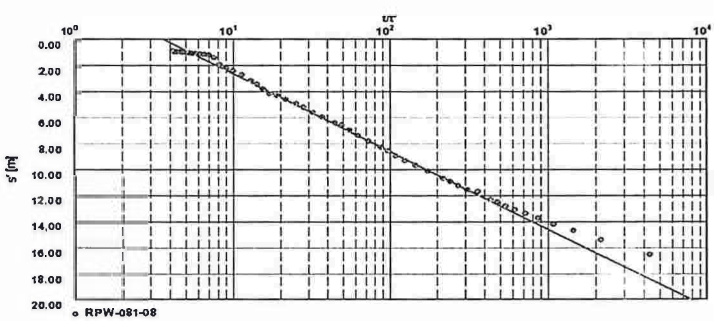


Figure 4. 7 Recovery test analysis curve after THEIS&JACOB for well no RPW- 81

The transmisivity and hydraulic conductivity values are $7.44\times10^{-2}m^2/min$, $1.27\times10^{-3}m/min$ respectively

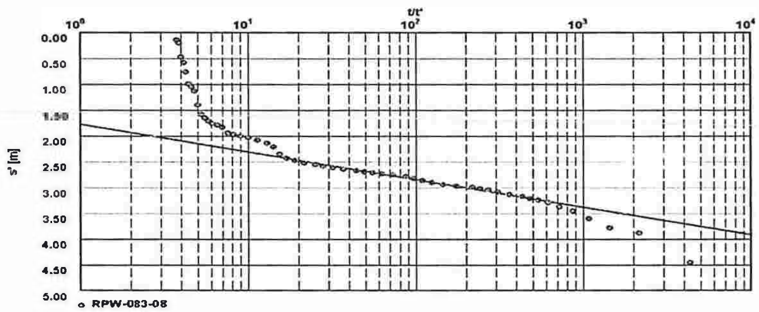


Figure 4. 8 Recovery test analysis curve after THEIS&JACOB for well RPW-83

The transmisivity and Hydraulic conductivity are $1.02\times10m^2/min$, 1.94×10^{-2} m/min respectively

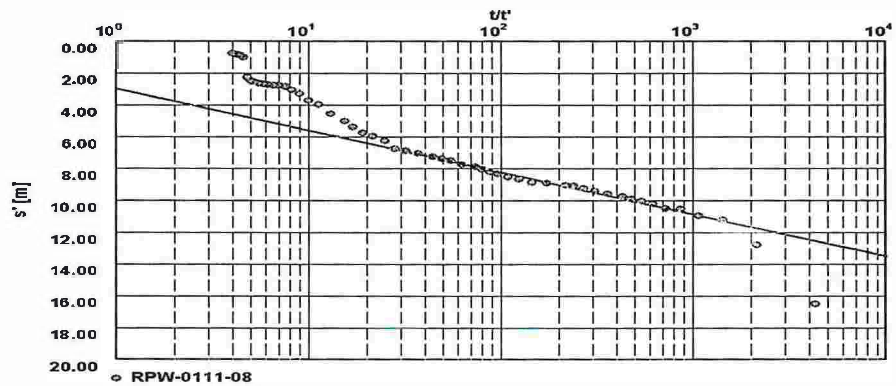


Figure 4. 9 Recovery test analysis curve after THEIS&JACOB for well RPW-111

The transmisivity and hydraulic conductivity values are $1.25\times10\text{-}1m^2/min$,$3.02\times10\text{-}3$ m/min respectively

Table 4. 8 Aquifer classification and Transmissivity values for different aquifers in the study area (Source Dessie Nadew, 2003)

Borehole Id	Transmissivity (m^2/day)			Aquifer Type	Preferred Value T,(m^2/day)	Productivity
	Straight line	recovery	Eden-Hezel			
BH92	2577	2649		Sand & Gravel	2649	High
BH94	627	519		Gravel & sand	519	High
BH93	155	155	199	Gravel & sand	155	Moderate
BH21	201.6			Medium to coarse sand.	201.6	Moderate
BH64	1.37	1.24	1.35		1.32 (avg. value)	Weak

Table 4. 9 Summery of the transmistivity and head values of the selected wells

Well Id	coordinate		Elevation	Total depth (m)	SWL (m)	Water column (m)	T (m2/day)	K (m/day)	Q (m^3/day)
	x	y							
PZ-3	569257	1377425	1428	170	9.58	160.4	766.77	4.78	3136.23
PZ-5	572491	1373604	1403	184	15.81	168.2	844.92	5.02	6764.8
PZ-9	562943	1365069	1510	146	19.46	126.5	201.63	1.60	604.8
RPW-079-08	568001	1367873		210	20.6	189.4	717.12	12.23	3689.0
RPW-081-08	566599	1367047		190	16.7	173.3	107.14	1.83	3456.0
RPW-083-08	565386	1365580		160	11.42	148.6	1468.8	27.94	4320.0
RPW-0111-08	568853	1368737		146	21.60	124.4	180	4.35	2617.92
BH-21	574000	1361697	1394	63	19.25	43.75	201.6	4.61	362.88
BH-64	563336	1363248	1580	120	74.66	45.34	1.32	0.03	51.84
BH-92	562769	1357377	1512	154	14.98	139.02	2649	19.05	1987.2
BH-93	563900	1364400	1580	111.3	40.03	71.27	155	2.17	1296.0

According to Sen (1995) aquifer potentiality is classified as table below

Table 4. 10 Aquifer potentiality

Transmisivity (m^2/day)	Potentiality
T>500	High
50<T<500	Moderate
5<T<50	Low
0.5<T<5	Weak
T<0.5	Negligible

5. NUMERICAL SIMULATION OF THE GROUND WATER FLOW SYSTEM

5 .1 General

A numerical groundwater flow model of the Alamata sub basin in Raya valley is developed to better understand the aquifer system of the basin and to determine the long-term availability of groundwater by simulating groundwater condition at present and predict the future condition under different hydrologic and pumping scenarios for various groundwater management alternatives. The model was developed using assumptions and approximations to simplify the actual aquifer system. The model idealizes the complex geo-hydrologic relations of the actual system, thus, it is a simplification based up on the data and the assumptions used to develop it.

5.2 Modeling Approach

The ground water flow system was numerically simulated in two dimension using MODFLOW, a widely used modular finite –difference model that simulates the flow of ground water of uniform density(McDonald and Harbaugh,2000).MODFLOW _96 is a modified version of MODFLOW(McDonald and Harbaugh,1988) that incorporates the use of parameters to define model inputs, the calculation of parameter sensitivities and the modification of parameters value to match observed heads ,flows or advective transport using the observation, sensitivity and parameters estimation process described by Hill and others (2000).

The model was calibrated in the steady-state mode with drilling report static water levels. Hydraulic parameters were iteratively adjusted by trial and error between steady state of the model until satisfactory match is gained.

5.2.1 Discretization

The numerical method used to approximate equation (1) requires that the modeled domain be divided into discrete volumes, called *cells*. The three-dimensional array of cells is known as the *model grid*. The center of each cell defines the point for which hydraulic head is determined. The head is taken to represent the average head within the cell. For transient models, time must also be divided into discrete intervals known as time steps. Heads and flows are calculated at the end of each time step.

5.2.2 Spatial discretization

The numerical ground water flow modeling of Alamata sub basin in raya valley, which has an aerial extent of the modeled area is 47.5km easting by 54.9 km northing and contains the entire study area shown in figure 5.1. The model uses a uniform grid size of 450 m by 450 m and contains 1 layer, 97 columns, 113 rows and 6757 active cells. The finite-difference model grid must be regular spacing, in order to facilitate data input from DEM & SURFER files; certain cells may represent areas outside the modeled area (which can have any shape). Such cells are considered inactive and the ground-water flow equation is not solved for them.

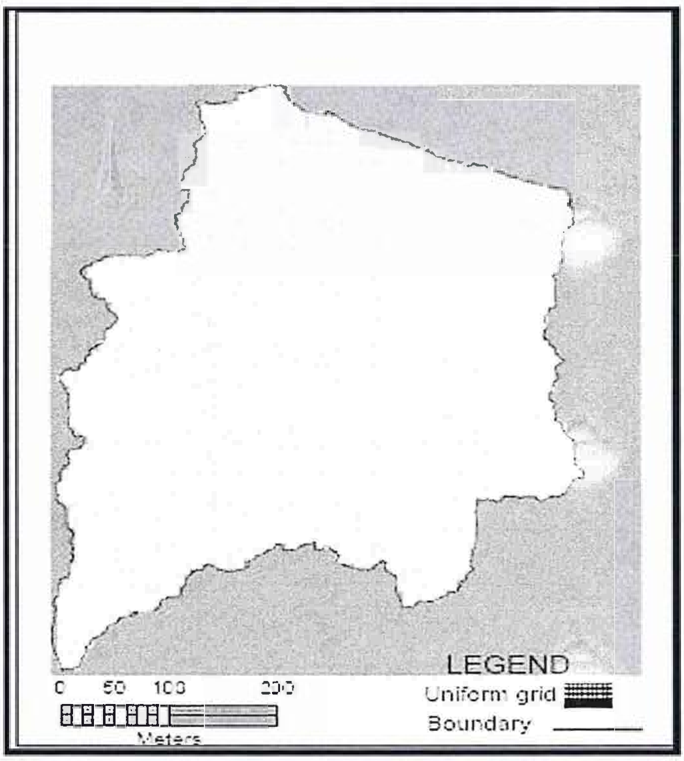


Figure 5. 1 Model Area Descritization

The modeled aquifer system was considered to be single layer and unconfined. Generally, the top layer elevation was considered to be the elevation of ground surface and in this case nodal values of ground surface elevation were interpolated from USGS digital elevation data. The interpolation was done at a resolution of 450m by 450m and then loaded into MODFLOW top elevation array. In this study, the aquifer thickness lies within the range from few meters to more than 200m in the valley fill except along the boundaries where ridges with high elevation are found. Elevated zones were simulated by giving relatively higher thicknesses at the cells in order to avoid drying of cells during simulations. Hence bottom elevation was set at 1350m.a.m.s.l. In fact, the thicknesses of the aquifers are very rough as it has not yet been determined exactly for the aquifers and was modified a bit in few areas during model calibration process. The relative thickness variation in most parts of the aquifer can be clearly observed in figurer 5.2 below.

5.2.3 Boundary Condition

Boundary conditions are mathematical statement specifying the dependent variable (head) or the derivative of the dependent variable (flux) at the boundaries of the problem domain (Anderson and Woessner, 1992). The choice in the type and location of model boundaries is important, as this may affect the simulation results. Ideally, model boundaries represent actual hydrologic boundaries, but this objective can not always be met. If model boundaries do not represent actual hydrologic boundaries, it is important that they are located far enough away from the area of interest so they do not affect the simulation results (Marijke and Smith, 2002).

The ground-water flow system in the Alamata sub-Basin is controlled, to a large extent, by the hydrologic and geologic boundaries of the system. Boundary conditions define the geographic extent of the flow system as well as the movement of ground water into and out of the system, such as flow to or from streams.

Boundary conditions may be three types: no-flow boundaries, specified-flux boundaries, and head-dependent flux boundaries. Geologic or hydrologic barriers to ground-water flow were simulated using no-flow boundaries. The contact between the permeable ground-water flow system and nearly impermeable bedrock is an example of a no-flow boundary. Known or estimated hydrologic fluxes, such as recharge and well discharge, are represented using specified-flux boundaries. A head-dependent flux boundary is one across which ground water moves at a rate proportional to the hydraulic-head gradient between the boundary and the ground-water system. Streams are usually represented as head-dependent flux boundaries because the movement of ground water to or from a stream is proportional to the difference between the head in the aquifer and the stage of the stream (which was not used in this study)

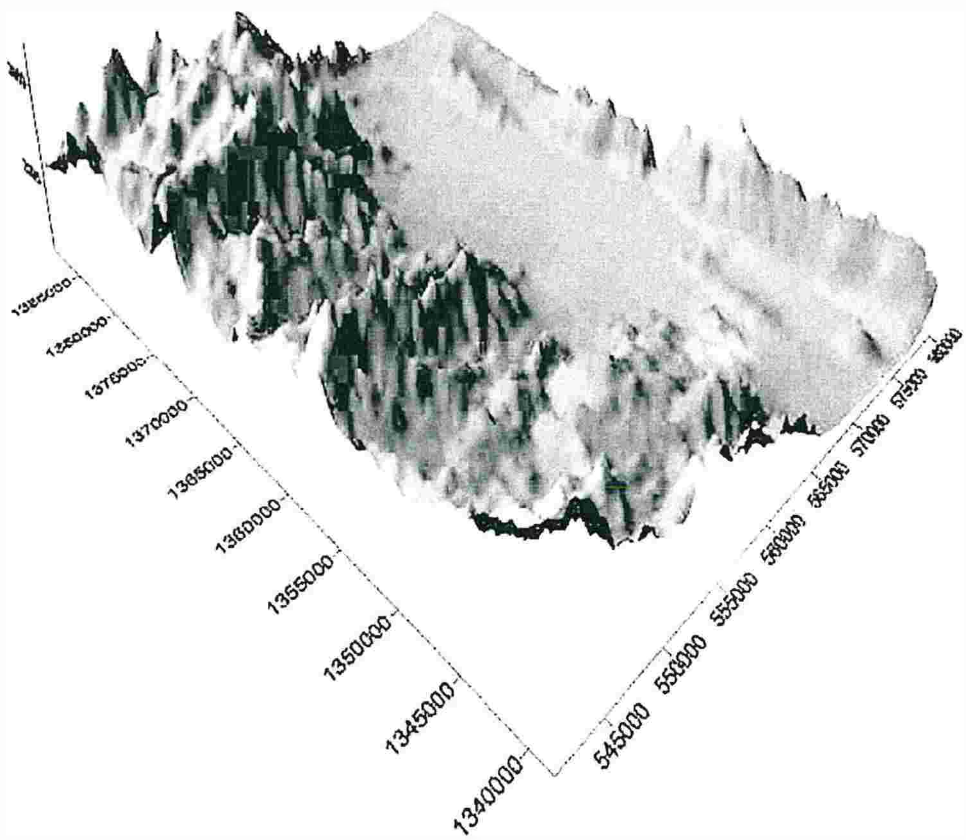


Figure 5. 2 Three dimensional view of the study area

The aerial boundaries of the model area (Fig 5.4) were either no flow boundaries or head dependent flux boundaries. The no flow boundaries of the study area were chosen to correspond as closely as possible with natural hydrologic boundaries across which ground water flow can be assumed negligible, such as ground water divides, or can be reasonably estimated. Major topographic divides are often considered no flow boundaries because topographic divides are typically coincident with ground water divide. Ground water on either side of a ground water divides flows away from the divide and not across it, so the divide itself acts as no flow boundary.

The lateral boundaries of the study area flow model generally represented as no flow boundaries, with the exception of a narrow Northerly surface inlet and easterly surface outlet (see figure5.2), where they were represented as general head boundary, and also the Waja swamp was assumed as constant head boundary.

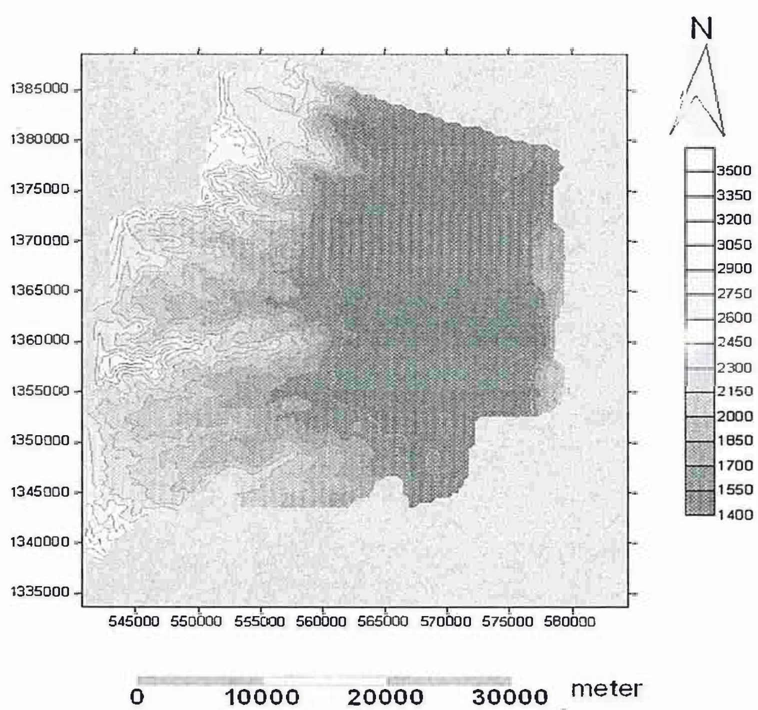


Figure 5. 3 Contours showing surface elevation

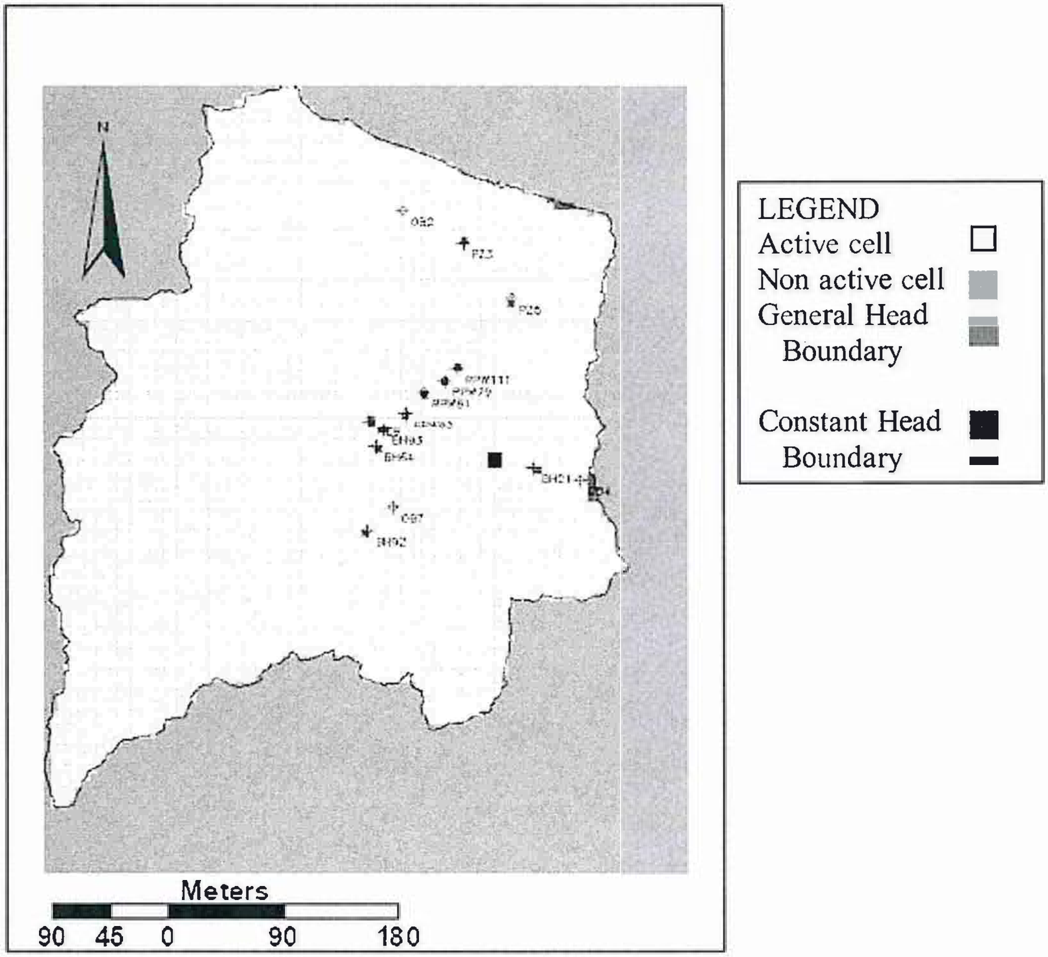


Figure 5. 4 Boundary conditions of the study area

5.3 Model Input Parameters

Simulation of groundwater flow and fluxes requires specifying aquifer system properties and stresses. Aquifer system properties can vary considerably both horizontally and vertically and thus, cannot be precisely represented in a numerical model. The aquifer system properties specified for each active cell in the model are estimates of the average conditions in the area represented by the cell. Similarly, stress applied to the system (recharge and discharge) are estimates for the area represented by each cell. The initial aquifer system properties of the study area were conceptually modeled from different groundwater literatures and analysis of pumping test. Aerial recharge estimation was obtained from water balance calculation, and pumpage was taken from the pumping wells. Selected properties and stresses were modified during model calibration.

5.3.1 Initial and Prescribed Hydraulic Heads

Water table surface used to define initial head in the model are based on the DEM land surface data source for each model cell. The land surface value of the DEM data are adjusted to the water table depth by subtracting the depth to the water table from land surface at measured points of water level from drilling reports. By analyzing the elevation difference between the land surface and the water level for 14 wells, it was assumed that the level varies depending on the topography of the area. The difference increases as attitude increases. Based on this, the initial and prescribed hydraulic head for the model was assigned for each cell of the model layers by the following methods:

- areas where land surface elevation ranges from 2700m – 3600m hydraulic head was assigned by land surface minus 54m,
- areas where land surface elevation ranges from 2700m – 2000m hydraulic head was assigned by land surface minus 39m,
- Areas where land surface elevation ranges from 2000m – 1400m hydraulic head was assigned by land surface minus 37m.

5.3.2 Hydraulic Conductivity and Transmissivity

Groundwater flow within the model layer was assumed to be horizontal. Hydraulic conductivity and transmissivity are properties that, in conjunction with the horizontal hydraulic gradient, control horizontal flow of groundwater.

Hydraulic conductivity is a measure of the water transmitting properties of aquifer material; coarse and well sorted materials have a higher hydraulic conductivity than fine and poorly sorted materials. Transmissivity is the product of hydraulic conductivity and saturated thickness and represents the water transmitting properties of the saturated section of the aquifer.

The hydraulic conductivity of the study area was estimated from aquifer test data of 11 boreholes drilled in the study area at different times. Based on the pumping test analysis, the aquifer hydraulic conductivity area ranges from 0.03 m/d-27.94m/d.Based on the geological map of Raya valley and the hydraulic conductivities obtained from pumping test data of the 11 wells, hydraulic conductivity map is developed. See figure 5.5

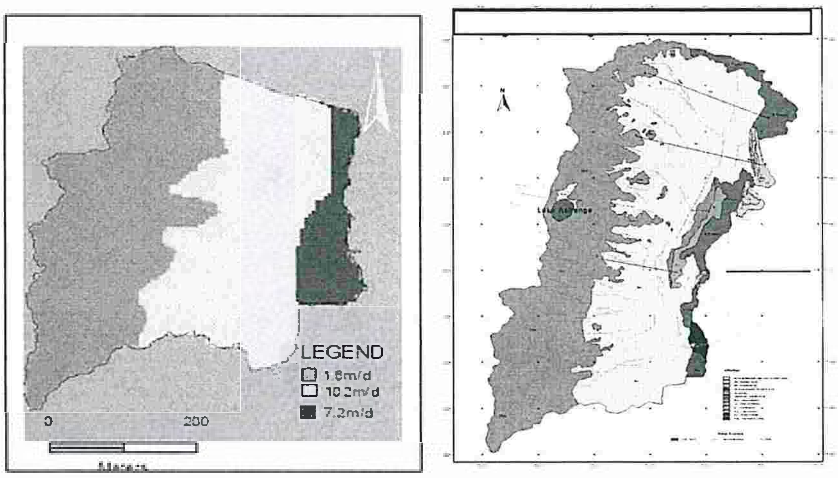


Figure 5. 5 map showing similar hydraulic conductivities and geologic map of Raya valley (MoWR Raya valley geological survey, 2008)

5.3.3 Model Stresses

5.3.3.1 Recharge

Hydraulic heads in the groundwater flow system respond to stresses on the system, which correspond to recharge and discharge. Recharge to the model consisted of infiltration from direct precipitation. Recharge was applied to the top grid layer area as a spatially varying. In general, precipitation recharge was obtained 8.44E-5 m/day using soil water balance method.

5.3.3.2 Ground Water Pumping

Ground–water pumping rates were specified in the model by wells for irrigation and rural water supply purpose withdraw 26,066.88m^3/d were assigned to the appropriate location and hydrgeological units. Ground water pumpage for the model was simulated using well package of the MODFLOW depending on the geographic coordinates of the wells.

5.3.3.3 Ground Water Inflow

There is a ground water inflow of 7806.7710 m^3/day to Alamata basin from Mehoni sub basin (Muez Amare, 2007) and assignd at apropraite place using general head boundary of the MODFLOW package

6 MODEL CALIBRATIONS AND SENSITIVITY ANALYSIS

6.1 Model Calibration

Model calibration is the process where by model parameter structure and parameter values are adjusted and refined to provide the best match between measured and simulated values of hydraulic heads and flow parameters are adjusted with reasonable limits from one simulation to the next to achieve the best model fit. Model fit is commonly evaluated by visual comparison of simulated and measured heads and flows or by comparing root mean square (RMS) errors of head and flow between simulations.

Basically, calibration can be achieved in two ways. These are the forward and inverse problem solutions. In an inverse solution method one determines values for a given parameter structure and hydrologic stress using a mathematical technique, such as nonlinear regression (Cooley & Naff, 1990; Hill, 1992, 1998) from information about head distribution (Anderson and Woessner, 1992). This technique is sometimes called parameter estimation & it finds the set of parameter values that minimize the difference between simulated and measured quantities such as hydraulic heads and flows; where as in the forward problem system parameters such as hydraulic conductivity and hydrologic stresses are specified and the model calculates the head distribution.

The Alamata sub basin ground-water flow model was calibrated using trial-and error method in adjusting initial estimates of aquifer properties (hydraulic conductivities) to get the best match between simulated hydraulic heads and measured water levels and selected water budget items. The study area ground-water model was calibrated steady-state simulation of ground-water flow.

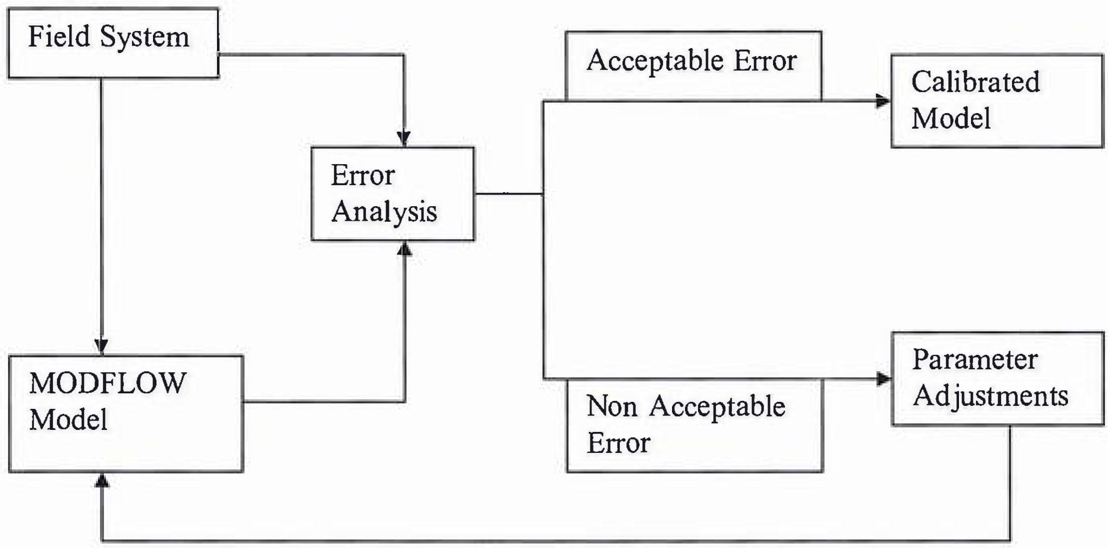


Figure 6. 1 Steps in Trial and Error Calibration

6.1.1 Steady-state Simulation

Steady-state flow condition exists when inflow is equal to out flow and aquifer storage does not exist. Hydraulic heads for steady-state conditions are sensitive to the amount of water that recharges to and discharges from the groundwater system; the hydraulic conductivity of the aquifer system and aquifer thickness.

The steady-state flow simulation of the study area based on the water level measurement of 14 wells during the construction was made to provide initial conditions.

Prior to the calibration of the model, there were criteria established to assess the simulation results in relation to measured data.

The calibration criterion takes account of matching more than 95 percent of all wells to within ±15 meter of the observed hydraulic heads. This criterion was considered satisfactory because 15 meter is less than 15 percent of the difference between the maximum and minimum hydraulic head of the estimated average of wells in the model area. Simulated hydraulic heads matched observed values within ±15 m difference for 100% of the 14 observed wells

Table 6. 1 List of Head Observation Wells Used For Calibration Process

Observation wells	Easting(UTM)	Northing(UTM)	Observed head value(m)
PZ-3	569257	1377425	1443.42
PZ-5	572491	1373604	1449.19
PZ-9	562943	1365069	1453.04
RPW-079-08	568001	1367873	1443.9
RPW-O81-08	566599	1367047	1442.33
RPW-083-08	565386	1365580	1449.08
RPW-0111-08	568853	1368737	1439.4
BH-21	574000	1361697	1437.75
BH-64	563336	1363248	1449.09
BH-92	562769	1357377	1486.02
BH-93	563900	1364400	1452.72
OB4	577138	1360890	1447.25
OB7	564496	1358992	1482.25
OB2	565180	1379685	1489.25

To provide an overall indication of the quality of the calibration, summary statistics on the difference between simulated and measured water levels were calculated after model calibration. The root mean square error (RMSE), the mean absolute error (MAE) and the mean error (ME) are common ways to provide ways to determine the overall goodness-of-fit between the simulated and the measured hydraulic heads. (Anderson and Woessner,1991).

The mean error (ME) is the mean difference between measured heads (h_m) and simulated heads (h_s).

$$ME = \frac{1}{n}\sum_{i=1}^{n}(h_m - h_s)_i \quad \text{--1}$$

The mean absolute error (MAE) is the mean of the absolute value of the differences in measured and simulated heads.

$$MAE = \frac{1}{n}\sum_{i=1}^{n}\left|(h_m - h_s)_i\right| \quad \text{----------------------------------2}$$

The root mean squared error (RMSE) or the standard deviation is the average of the squared differences in measured and simulated heads.

$$RMSE = \left[\frac{1}{n}\sum_{i=1}^{n}(h_m - h_s)_i^2\right]^{0.5} \quad \text{------------------------------3}$$

The summary of statistics, after calibration for residual heads between simulated and observed values was calculated, for 14 water levels taken from drilling water level reports. Based on this, the mean error (ME), the mean average error (MAE) and the root mean square error (RMSE) were computed for the wells (Table 6.2). The mean error of the calibration results for Alamata sub-basin groundwater flow model is -1.86. This indicates that, in the overall calibration of head levels, calculated heads were greater than observed heads by about1.86m. The root mean square error shows whether the calibration criteria set prior to or during calibration has been met or not. The mean absolute error and the root mean squared error for the calibration are 3.2m and 4.024 m respectively. The statistical computation for residual errors is summarized in table 6.2.

Table 6. 2 Simulated Head and Measured Heads in Observation Wells

Observation wells	Observed headvalue(h_m)	Simulated value (h_s)	(h_m-h_s)	$(h_m-h_s)^2$	/h_m-h_s/
PZ-3	1447.441	1443.42	-4.021	16.16844	4.298
PZ-5	1447.389	1449.19	1.801	3.243601	1.667
PZ-9	1452.935	1453.04	0.105	0.011025	0.238
RPW-079-08	1445.616	1443.9	-1.716	2.944656	1.82
RPW-O81-08	1446.358	1442.33	-4.028	16.22478	4.176
RPW-083-08	1446.508	1449.08	2.572	6.615184	2.382
RPW-0111-08	1446.076	1439.4	-6.676	44.56898	6.791
BH-21	1438.881	1437.75	-1.131	1.279161	1.155
BH-64	1459.137	1449.09	-10.047	100.9422	8.531
BH-92	1486.188	1486.02	-0.168	0.028224	0.452
BH-93	1447.793	1452.72	4.927	24.27533	4.608
OB4	1453.027	1447.25	-5.777	33.37373	5.772
OB7	1483.79	1482.25	-1.54	2.3716	1.723
OB2	1489.664	1489.25	-0.414	0.171396	1.13
SUM			-26.113	252.2183	44.743
			ME=-1.86	RMSE=4.24	MAE=3.2

In addition to the above criteria, the quality of the calibration is evaluated by a linear regression analysis of simulated and observed hydraulic heads for all wells (Figure 6.2) yielded a coefficient of correlation equal to 0.9493. Observation points that lie on the straight line show exact fit between head points, observation points that are above the straight line show higher calculated heads and those points below the straight line show higher observed heads. Overall, it shows that the head differences are normally distributed and, high and low simulated values are evenly distributed through most parts of the area.

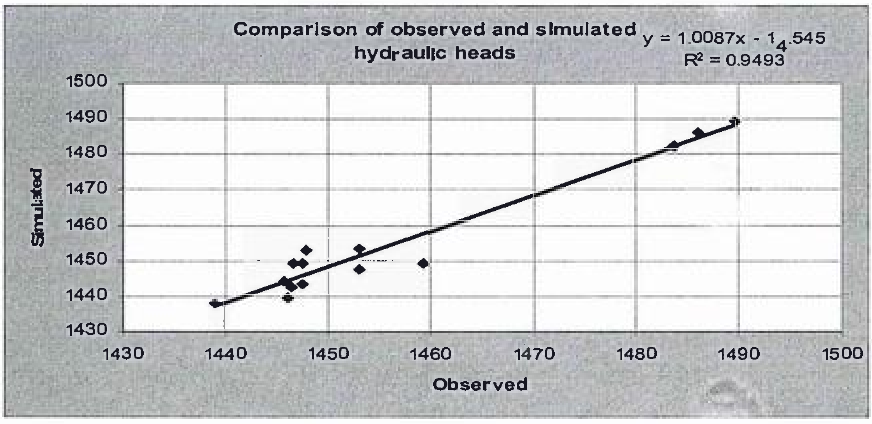


Figure 6. 2 Linear regression of observed and simulated hydraulic heads

6.1.2 Model-Derived Groundwater Budget

Using the calibrated model, water budget of the whole model domain was calculated with a percent discrepancy of 0.00.Table below shows inflow and outflow components of the water balance and steady-state hydrologic budget of the study area calculated by the model.

Generally, these values are somewhat different from the estimates made in the water balance of the conceptual model, which could be due to those parameters which are not considered in the model.

PMWBLF (SUBREGIONAL WATER BUDGET) RUN RECORD
FLOWS ARE CONSIDERED "IN" IF THEY ARE ENTERING A SUBREGION
THE UNIT OF THE FLOWS IS [L^3/T]
TIME STEP 1 OF STRESS PERIOD 1

==

WATER BUDGET OF THE WHOLE MODEL DOMAIN:

==

FLOW TERM	IN	OUT	IN-OUT
STORAGE	0.0000000E+00	0.0000000E+00	0.0000000E+00
CONSTANT HEAD	0.0000000E+00	7.7981914E+04	-7.7981914E+04
WELLS	0.0000000E+00	2.8286760E+04	-2.8286760E+04
DRAINS	0.0000000E+00	0.0000000E+00	0.0000000E+00
RECHARGE	1.0710892E+05	0.0000000E+00	1.0710892E+05
ET	0.0000000E+00	0.0000000E+00	0.0000000E+00
RIVER LEAKAGE	0.0000000E+00	0.0000000E+00	0.0000000E+00
HEAD DEP BOUNDS	4.6223425E+02	1.3059768E+03	-8.4374255E+02
STREAM LEAKAGE	0.0000000E+00	0.0000000E+00	0.0000000E+00
INTERBEDSTORAGE	0.0000000E+00	0.0000000E+00	0.0000000E+00
RESERV. LEAKAGE	0.0000000E+00	0.0000000E+00	0.0000000E+00
SUM	1.0757116E+05	1.0757465E+05	-3.4921875E+00
DISCREPANCY[%]	0.00		

6.1.3 Hydraulic Head Contours

The main output from a model such as MODFLOW is hydraulic head values for each cell in the model domain. A water table surface can be interpolated from these hydraulic head values. The water table is presented as contour lines representing an interpolated surface that indicates the hydraulic head of the model domain. This information is significant because the water table indicates a variety of important observations about the flow system. A reduction in the elevation of the water table can indicate a depletion of groundwater resources. A depletion of groundwater resources could significantly impact the people that rely on groundwater as a primary drinking water source. The location of the water table also dictates the interaction between surface-water and groundwater, which could potentially influence surface water supplies. This information is key to managing groundwater effectively, and assuring a safe and sufficient groundwater supply. Simulated Hydraulic head of the study area is shown in the figure 6.3

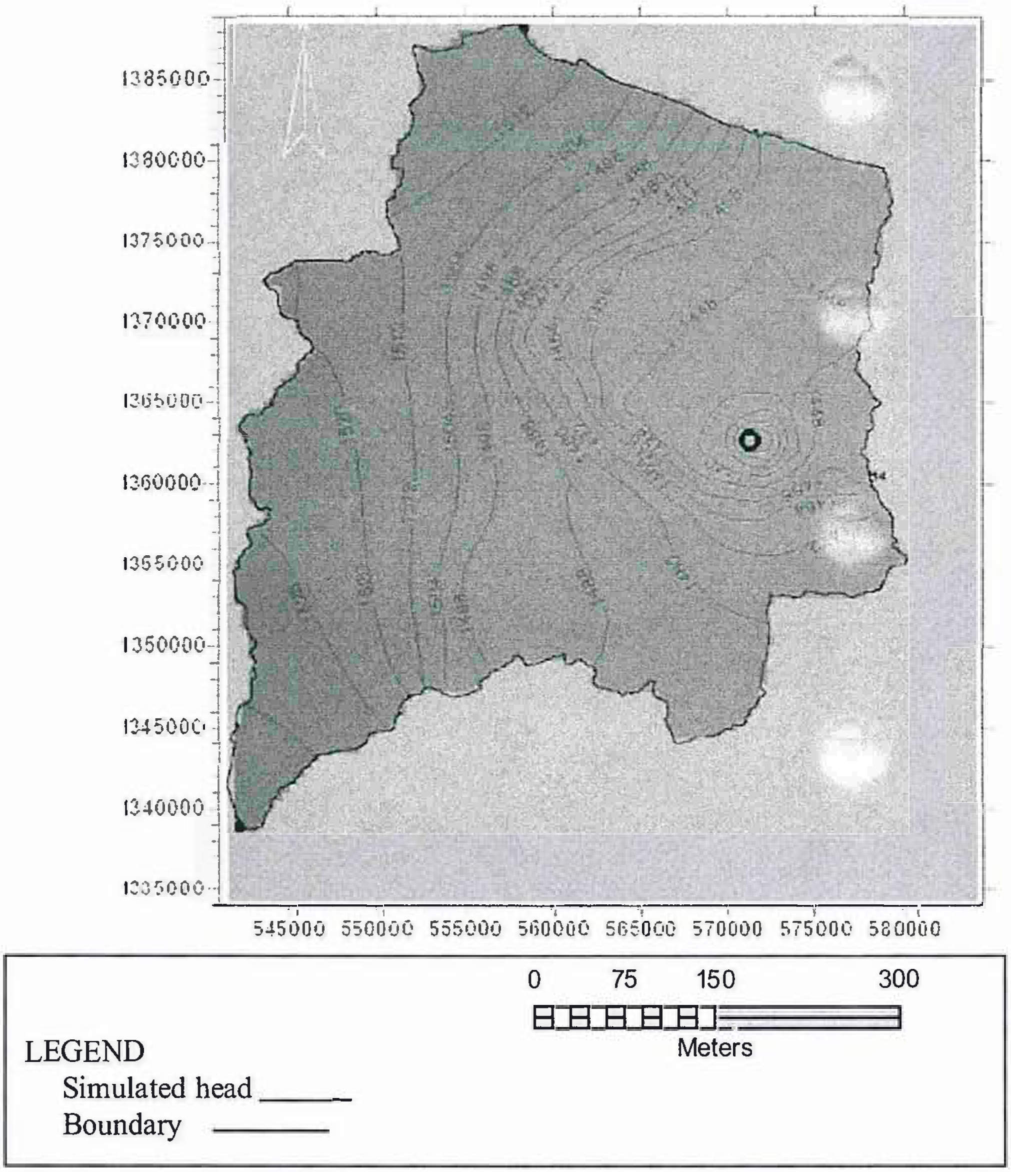


Figure 6. 3 Simulated ground water head contour

6.1.4 Interpretations of the Contour Map and Flow Nets

The spacing of the ground water contours gives a good indication of variation in aquifer permeability values. Those closely spaced groundwater contour indicates low permabilities, because a steep hydraulic gradient is needed to push the water through the aquifer. Where ground water contours are more widely spaced the converse is true, and the aquifer is likely to be much more permeable.

Ground water flow-lines indicate not only the over all direction but where flow is concentrating. This is also an indicator of variation in aquifer permeability.

In figure 6.4 the ground water contours spacing suggests that the southeast and north east of the area of the aquifer is more permeable than the northern west part. The flow lines show that there is a concentration flow towards southeast and northeast and this is an area of discharge to the surface streams. If this map were to be used to select favorable locations for a new well, the discharge area would have much to commend it as it is both in an area of high permeability values and where flow lines converge.

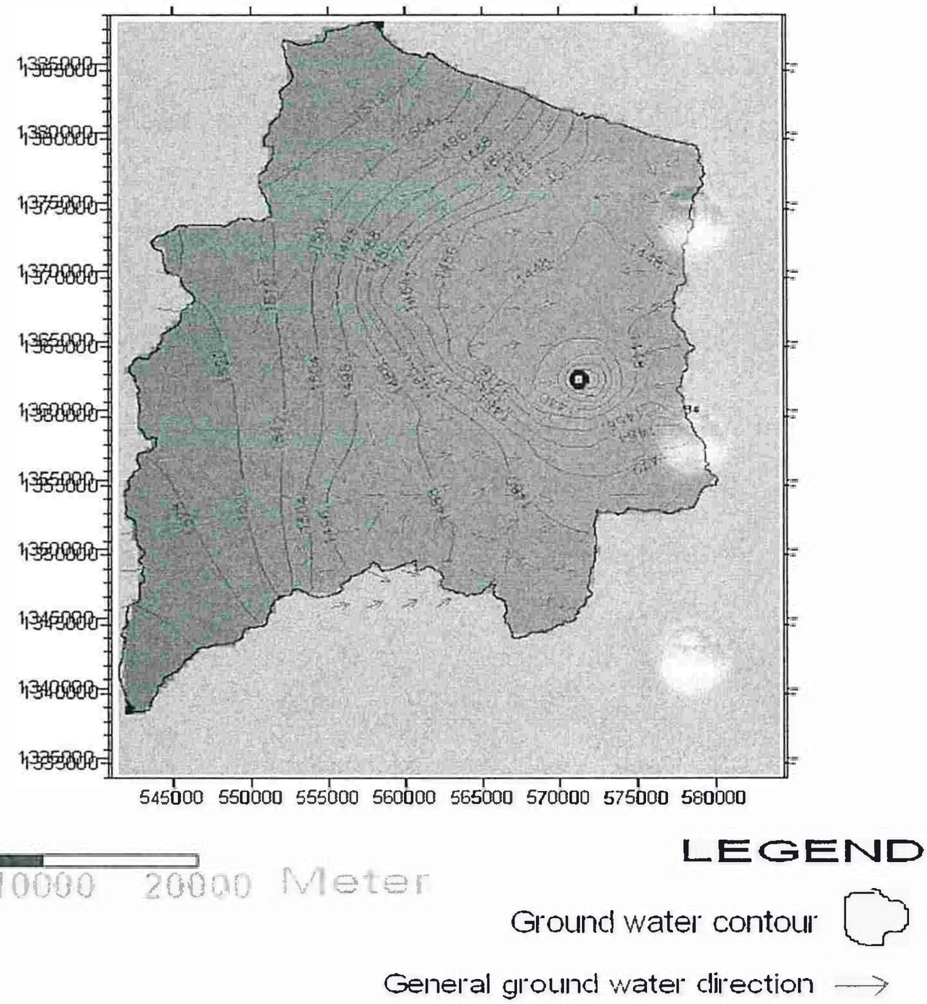


Figure 6. 4 Contour maps of groundwater elevation and ground flow direction

6.2 Model Sensitivity Analysis

The purpose of sensitivity analysis is to quantify the uncertainties in the calibrated model caused by uncertainty in the estimates of aquifer parameters, stresses and boundary conditions. During this analysis the calibrated value for the aquifer parameters and the boundary conditions are systematically changed with plausible range (Anderson and Woessner, 1992). Moreover sensitivity analysis can also be used to observe the model response to variation in uncertain parameters. The results can be used to identify sensitive input parameters for the purpose of guiding for the calibration.

Different methods are available to conduct such analysis, but there is no one method to conclusively determine model sensitivity. In this study, a traditional approach was used by adjusting the most important parameters by selected percentages and documenting the resulting change in simulated water levels and groundwater fluxes in different part of the model area.

The model is considered sensitive to parameter when a change of parameter value changes the distribution of the simulated hydraulic head. When the model is sensitive to an input parameter, the value of that parameter within the model is more accurately determined during model calibration because small changes to the parameter value cause large change in hydraulic head. If the change of parameter value does not change the simulated hydraulic head, the model is considered insensitive to that parameter.

Ground-water modeling results are affected by various model parameters and assumptions, including the 1) geometry of the hydrogeologic units 2) Vertical and horizontal spacing of the model grid, 3) types and locations of model boundaries, 4) magnitudes and aerial distribution stresses such as ground-water recharge and pumpage, 5) conductance of stream, drain and general head boundary cells, and 6) horizontal and vertical hydraulic conductivities of the aquifers and confining units.

The sensitivity of the model in steady-state mode for the study area was determined by systematical increasing or decreasing the model parameters and stresses from the calibrated values. The adjusted parameters and stresses include; horizontal hydraulic conductivity and recharge. The adjustment was done by increasing or decreasing the values by 25, 45 and 55 percents. Effects of the adjustments on the simulated water level and groundwater flux were calculated (table 6.3 and fig 6.5). As observed from the values, the model was sensitive for both parameters but more sensitive to horizontal hydraulic conductivity for both decreasing and increasing of the value. In general, it revealed that, the model is more sensitive for decreasing the scenarios than increasing for the hydraulic conductivity and recharge while small changes. Therefore, for further study of the sub-basin care has to be taken during hydraulic conductivity and recharge zonation.

Table 6.3 Sensitivity Analysis of the Effect of the Hydraulic Conductivity and Recharge on the RMS Value of Water Level

Percent change in HC & R from calibrated value(%)	RMS value for the selected parameters	
	HC	Recharge(R)
-55	76.31	53.3
-45	53.7	40.52
-25	25.43	23
0	0	0
25	24.5	22.17
45	45.6	39.9
55	51	47.8

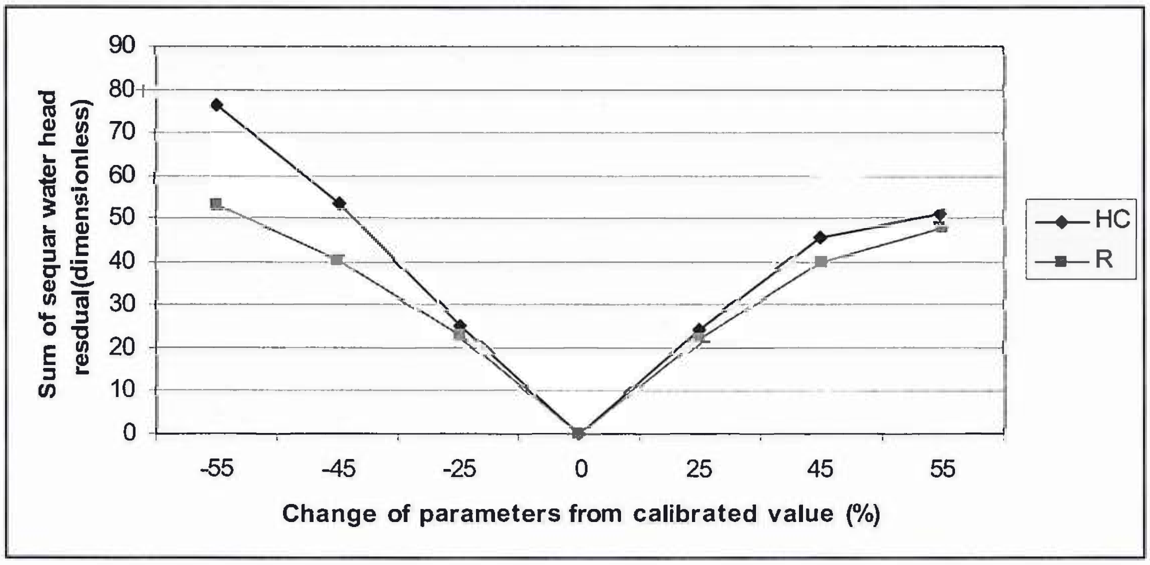


Figure 6. 5 Sensitivity Analysis of the Effect of the Hydraulic Conductivity and Recharge on the RMS Value of Water Level

6.3 Scenario Analysis

It is possible to use the model to simulate the resulting change in water levels and fluxes due to the new proposed scenarios and project the likely effect, if having a calibrated model that was tested for sensitivity as capable of simulating water level elevations or fluxes and given its associated limitations.

One of the objectives of numerical flow model simulated in this study was intended to test the response of the hydrologic system to different scenarios. So, alternative scenarios were developed to test the response of the hydrologic system to changes in water used or hydrologic stress under steady state condition. System response was evaluated by using fluxes and heads of the calibrated model as a baseline changes in water table elevation and changes in groundwater outflow in the new scenario simulation.

Changes in water levels and fluxes caused by groundwater withdrawal in the whole sub-basin and the effect of local increase in groundwater withdrawal of irrigation activity takes place were simulated using the model. The effect of changes in water levels and fluxes caused by decreased recharge due to less than normal precipitation in the area that may result from weather modification was also simulates.

It should be noted that the results of the scenarios depend on future land use, population growth, weather conditions, hydrologic stress etc, and may not be used as predictive tool to generate absolute amounts in the future, but used primarily to test the response of the system. In general, the results of the scenarios or their accuracy depend on the validity of the assumptions behind the scenarios. Moreover, errors introduced due to limitations associate with the model also affect the results of the scenarios and should be taken into consideration during interpretation and application result.

The first scenario, which is increment of withdrawal by 25% of the existing average withdrawals for the whole sub-basin, other parameters was kept constant, was conducted to represent possible future changes in water use in the sub-basin. The heads calculated for this scenario show a maximum decline of the water level by 14.12 m at well PZ3 and a minimum of 3.75 m at observation well OB4. In the second scenario, increases of withdrawal with 50% results a maximum drawdown of 31.47m at well PZ3 and a minimum decline of 8.79m at well OB7.The third scenario simulated by the model is a decrease in recharge by 50 percent which could be the case if mild drought conditions were imposed on the aquifer system while water extraction is maintained at current rates. Simulated drought condition showed complete drying 0f well PZ3 and a minimum reduction of water level up to 41.16m at well RPW111.

Table 6. 4 Scenario analysis for different stresses

	Head(after calibration)	Simulated heads for different scenarios				H difference			
		Increase in pump		Reduction in recharge		25%incr in pump	50%incr in pump	25%redu. In recharge	50%redu. In recharge
		25%	50%	25%	50%				
PZ3	1447.441	1433.325	1415.969	1414.956	dry	14.116	31.472	32.485	dry
PZ5	1447.389	1439.649	1431.411	1422.89	1397.788	7.74	15.978	24.499	49.601
PZ9	1452.935	1446.192	1439.155	1428.869	1404.006	6.743	13.78	24.066	48.929
RPW79	1445.616	1438.497	1430.994	1422.128	1397.71	7.119	14.622	23.488	47.906
RPW81	1446.358	1439.337	1431.954	1422.94	1398.498	7.021	14.404	23.418	47.86
RPW83	1446.508	1439.542	1432.224	1423.185	1398.663	6.966	14.284	23.323	47.845
RPW111	1446.076	1438.978	1431.501	1422.548	1398.28	7.098	14.575	23.528	47.796
BH21	1438.881	1433.775	1428.573	1418.548	1397.717	5.106	10.308	20.333	41.164
BH64	1459.137	1453.26	1447.224	1434.068	1411.114	5.877	11.913	25.069	48.023
BH92	1486.188	1482.182	1478.2	1461.948	1435.505	4.006	7.988	24.24	50.683
BH93	1447.793	1440.937	1433.753	1424.588	1399.926	6.856	14.04	23.205	47.867
OB4	1453.027	1449.28	1445.042	1432.029	1410.903	3.747	7.985	20.998	42.124
0B7	1483.79	1479.028	1474.998	1456.566	1433.187	4.762	8.792	27.224	50.603
OB2	1489.664	1484	1478.239	1463.36	1439.091	5.664	11.425	26.304	50.573

6.4 Model Validation

Prior to finalized model it is necessary to validate the model. Model validation should be carried out using another data set which was not used for the model calibration. Due to the lack of data with the same duration, model validation has not been completed

6.5 Model Limitations

A groundwater flow model represents a complex, natural system with a set of mathematical equations that describe the system. Intrinsic to the model is the error and uncertainty associated with the approximations, assumption and simplifications that must be made. Hydrologic modeling errors typically are the consequence of a combination of: input data, representation of the physical processes by algorithms of the model and parameter estimation during the calibration procedure.

Data on types and thickness of hydrogeologic units, water levels, and hydraulic properties were taken from different phase studies of RVDP (1996, 1998), Dessie Nadow (2003) and Ermias Hagos (2005) and represent only an approximations of actual values especially the hydraulic conductivity and recharge estimation. Those data having full pumping test data were concentrated in small portion of the study area. Broad ranges of hydraulic property parameter values are possible. No short or longer term data exist that provide information about how the system responds to changing condition, and this wide time range data is used for the model construction as well as calibration with the assumption of no significant change occurred. However, with no log-term data, it is unknown if the simulated steady state response is truly representative of the real condition.

A wide range of parameter values such as hydraulic conductivity and recharges were used. The previously worked researches have not zoned this parameter. So, numbers of different trial and error model runs were made with various combinations of parameter values during calibration process to arrive at the calibration target; and various combinations can result in low residual error yet improperly represent the actual system. Acceptable degree of agreement between simulated and measured values does not guarantee that estimated model parameter values reasonably represent the actual parameter values.

7 CONCULUSION AND RECOMMENDATION

7.1 Conclusions

Alamata sub-basin which is found in the Raya Valley is located in the southern part of Tigray Regional State of the Northern Ethiopia. The area covers 51% of the Raya valley with area coverage of 1292.57km^{2}.

The study area is important water source for irrigation, and domestic water supplies. Development of water project in the area increases withdrawal of water from the aquifer. This would lead to the decline in water level in the aquifer. Water resource tools are needed to evaluate the management and environmental issue associated with the aquifer, such as planning for source water protection and estimating sustainable aquifer withdrawals. This report describes a conceptual model of groundwater flow in the aquifer and documents the development and calibration of a numerical model to simulate groundwater flow.

Arial recharge from annual precipitation (728.5mm) to the Alamata sub basin is estimated using soil water balance method .The accuracy of the method depends on the accurate estimation of Evapotranspiration, Soil moisture storage capacity and surface run off. In this study potential evapotranspiration is estimated using Penman Montheith method (Cropwat 4 soft ware), surface runoff is estimated using CN method and soil moisture storage is estimated from Thornthwaite and Mather. The obtained results are; actual Evapotranspiration 690.5mm, surface run off 178.3mm and ground water recharge 30.8mm.

In this study, a homogeneous, isotropic and single layer aquifer is considered. Geological, hydrogeological and other parameters were collected from different bureaus, institutions and previous works. From these collected data, 14 wells with static water level measurements were taken. Map of equipotential line and groundwater flow direction was developed and the flow direction was determined from West-East, East-West and finally south –east direction.

Groundwater recharge is from precipitation and ground water inflow from the upper sub basin of Raya valley called Mehoni sub basin. The out flow from the ground system includes well withdrawal and sub_surface outflow.

Numerical groundwater flow was used to study the response of Alamata sub-basin groundwater flow system to different scenarios. As numerical groundwater flow models represent the simplification of complex natural systems, different parameters were assigned into the conceptual model to represent the system in a simplified form. The numerical groundwater flow model was simulated using MODFLOW, 1996(McDonald and Harbaugh, 1998). The area was represented by a uniform grid cells arranged in 113 rows and 97 columns each cell with sides of 450m by 450m.A two dimensional profile model under steady state condition is developed to study the response of the system to different scenarios.

Estimated horizontal hydraulic conductivity used for the numerical model input ranges from 0.03 to 27.94m/d.

The model was calibrated in steady state condition using trial and error calibration techniques. This method involves determination of the set of optimal parameter values that best fit the observed hydraulic heads and flows by using number of trial and error of input parameters such as recharge and horizontal hydraulic conductivity and was adjusted and refined using calibration.

The result of calibration was evaluated using linear regression analysis of simulated and observed hydraulic heads for all wells and yielded a coefficient of correlation equal to 0.9493 and by statistical model fit analysis approach by calculating the average difference between simulated and measured heads using groundwater level data at 14 wells. Though the three statistical measuring parameters were calculated, the standard deviation (RMS) value was used to evaluate the calibration result. The calculated standard deviation of hydraulic head for the calibration result was 4.24m. This is considered good, since it is within the set of calibration target.

Model simulated heads were found to be more sensitive to horizontal hydraulic conductivity than recharge. In both cases, value decreased the sensitivity increased. This is done by changing of the parameters by ± 25, ± 45 and ± 55 percent.

As the model is intended to study the response of the hydrologic system the following four scenarios were assigned: In the first and second scenarios which apply for the sub-basin and, withdrawals were increased by 25 and 50% of the average existing withdrawals. In the third and forth scenarios, 25% and 50% reduction in recharge is applied. The effects of the scenarios are evaluated with respect to change induced groundwater heads to the steady state simulated heads. Accordingly, an increase of withdrawals by 25% over the whole area resulted in an average decline of steady state water level with minimum decline of 3.747m at well OB4 and a maximum decline of 7.74 at well PZ3. Again an increase of withdrawals by 50% results with a minimum drawdown of 7.985m at well OB4 and a maximum of 31.472m at well PZ3.

On the other way, reduction of recharge by 25% results with a minimum decline of 20.333m at well BH21 and a maximum decline of 32.485m at well PZ3. A reduction of 50% recharge resulted with a complete drying of well PZ3.

7.2 Recommendations

- The concerned bodies should collect the data and put it in appropriate place and make it accessible for researchers who are interested to conduct further studies which are beneficial for both the researchers and organizations in charge of water affairs.
- The distribution and rate of recharge and effects of development on recharge are poorly understood, so further work on recharge would give better result.
- The area should be zoned as per the value of hydraulic conductivity and recharge so that there would be a facilitated condition to estimate the groundwater potential of the area.
- The Alamata sub basin groundwater flow model could be improved with additional hydrologic and geologic research, data collection and interpretation, and the use of additional MODFLOW options and packages. As new data available, the model could be updated and recalibrated.
- Within the study area, there are no continuous measurements of ground water level which was very important in determining the groundwater table fluctuation due to some stress. Therefore I strongly recommend taking water level measurement at the end and beginning of both monsoon and non monsoon seasons and groundwater level monitoring wells should be placed in the whole sub basin in order to understand the general fluctuations in groundwater levels. This helps to carry out transient groundwater flow modeling, so that system response can be predicted with accuracy.
- The existing ground water abstraction rate is 26% of the recharge therefore we can utilize the aquifer till dynamic equilibrium between recharge and discharge reaches.

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9 APPENDICES

Appendix A; Pumping test data

Appendix (A-1) well no PZ3

Time since pumping started (min)	Water level (m)	Draw dawn s(m)	Time since pumping started (min)	Water level (m)	Draw dawn s(m)	Time since pumping started (min)	Water level (m)	Draw dawn s(m)
0	9.58	0	120	11.37	1.79	1140	12.26	2.68
1	10.52	0.94	140	11.40	1.82	1560	12.30	2.72
2	10.67	1.09	160	11.43	1.85	1680	12.33	2.75
3	10.73	1.15	180	11.45	1.87	1800	12.36	2.78
4	10.77	1.19	210	11.52	1.94	1920	12.40	2.82
5	10.82	1.24	240	11.56	1.98	2040	12.43	2.85
6	10.84	1.26	270	11.60	2.02	2160	12.46	2.88
7	10.85	1.27	300	11.64	2.06	2280	12.49	2.91
8	10.87	1.29	330	11.68	2.10	2400	12.51	2.93
9	10.89	1.31	360	11.71	2.13	2520	12.54	2.96
10	10.9	1.32	420	11.76	2.18	2640	12.56	2.98
12	10.92	1.34	480	11.81	2.23	2760	12.58	3.00
14	10.95	1.37	540	11.86	2.28	2880	12.61	3.03
16	10.98	1.40	600	11.90	2.32	3000	12.62	3.04
18	10.99	1.41	660	11.94	2.36	3120	12.64	3.06
20	11.01	1.43	720	11.97	2.39	3240	12.65	3.07
25	11.05	1.47	780	12.01	2.43	3360	12.67	3.09
30	11.11	1.53	840	12.04	2.46	3480	12.70	3.12
35	11.14	1.56	900	12.07	2.49	3600	12.72	3.14
40	11.16	1.58	960	12.10	2.52	3720	12.74	3.16
45	11.16	1.58	1020	12.12	2.54	3840	12.75	3.17
50	11.18	1.60	1080	12.14	2.56	3960	12.77	3.19
60	11.20	1.62	1140	12.17	2.59	4080	12.79	3.21
70	11.24	1.66	1200	12.19	2.61	4200	12.8	3.22
80	11.27	1.69	1260	12.21	2.63	4320	12.81	3.23
90	11.30	1.72	1320	12.23	2.65			
100	11.32	1.74	1380	12.25	2.67			

Appendix (A-2) well no PZ5

Time since pumping started (min)	Water level (m)	Draw dawn s(m)	Time since pumping started (min)	Water level (m)	Draw dawn s(m)	Time since pumping started (min)	Waterleve l(m)	Draw dawn s(m)
0	15.81	0	90	16.32	0.51	1320	17.05	1.24
1	16.05	0.24	100	16.34	0.53	1140	17.10	1.29
2	16.05	0.24	120	16.39	0.58	1560	17.12	1.31
3	16.07	0.26	140	16.43	0.62	1680	17.12	1.31
4	16.08	0.27	160	16.48	0.67	1800	17.13	1.32
5	16.09	0.28	180	16.51	0.70	1920	17.16	1.35
6	16.10	0.29	210	16.57	0.76	2040	17.20	1.39
7	16.10	0.29	240	16.62	0.81	2160	17.23	1.42
8	16.10	0.29	270	16.65	0.84	2280	17.24	1.43
9	16.11	0.30	300	16.69	0.88	2400	17.26	1.45
10	16.11	0.30	330	16.71	0.90	2520	17.27	1.46
12	16.12	0.31	360	16.74	0.93	2640	17.27	1.46
14	16.13	0.32	420	16.78	0.97	2760	17.29	1.48
16	16.14	0.33	480	16.82	1.01	2880	17.29	1.48
18	16.15	0.34	540	16.86	1.05	3000	17.30	1.49
20	16.15	0.34	600	16.87	1.06	3120	17.31	1.50
25	16.16	0.35	660	16.91	1.10	3240	17.32	1.51
30	16.17	0.36	720	16.94	1.13	3480	17.32	1.51
35	16.18	0.37	780	16.96	1.15	3600	17.33	1.52
40	16.19	0.38	840	16.98	1.17	3720	17.33	1.52
45	16.20	0.39	900	16.93	1.12	3840	17.34	1.53
50	16.22	0.41	960	16.95	1.14	3960	17.35	1.54
60	16.24	0.43	1020	16.97	1.16	4080	17.37	1.56
70	16.27	0.46	1080	16.99	1.18	4200	17.38	1.57
80	16.29	0.48	1200	17.02	1.21	4320	17.41	1.60

Appendix (A-3) well no PZ9

Time since pumping started (min)	Water level(m)	Drawdowns(m)	Time since pumping started (min)	Water level(m)	Drawdown s(m)	Time since pumping started (min)	Water level(m)	Drawdown s(m)
0	19.46	0	120	21.50	2.04	1140	21.65	2.19
1	20.18	0.72	140	21.48	2.02	1560	21.67	2.21
2	20.34	0.88	160	21.50	2.04	1680	21.82	2.36
3	20.41	0.95	180	21.59	2.13	1800	21.82	2.36
4	20.48	1.02	210	21.60	2.14	1920	21.85	2.39
5	20.50	1.04	240	21.55	2.09	2040	21.85	2.39
6	20.60	1.14	270	21.56	2.10	2160	21.92	2.46
7	20.63	1.17	300	21.59	2.13	2280	21.83	2.37
8	20.66	1.20	330	21.59	2.13	2400	21.90	2.44
9	20.69	1.23	360	21.60	2.14	2520	21.90	2.44
10	20.74	1.28	420	21.61	2.15	2640	21.94	2.48
12	20.74	1.28	480	21.69	2.23	2760	22.01	2.55
14	20.84	1.38	540	21.70	2.24	2880	22.04	2.58
16	20.90	1.44	600	21.75	2.29	3000	21.94	2.48
18	20.96	1.50	660	21.80	2.34	3120	21.79	2.33
20	21.00	1.54	720	21.81	2.35	3240	22.06	2.60
25	21.06	1.60	780	21.80	2.34	3360	22.09	2.63
30	21.10	1.64	840	21.81	2.35	3480	22.14	2.68
35	21.18	1.72	900	21.83	2.37	3600	22.16	2.70
40	21.2	1.74	960	21.84	2.38	3720	22.07	2.61
45	21.23	1.77	1020	21.85	2.39	3840	22.10	2.64
50	21.26	1.80	1080	21.85	2.39	3960	22.10	2.64
60	21.30	1.84	1140	21.83	2.37	4080	22.09	2.63
70	21.32	1.86	1200	21.85	2.39	4200	22.10	2.64
80	21.33	1.87	1260	21.33	1.87	4320	22.03	2.57
90	21.39	1.93	1320	21.50	2.04			
100	21.41	1.95	1380	21.63	2.17			

Appendix (A-4) well no RPW-79

Time Since Pumping Stopped (Min)	Water level(m)	Residual draw down	Time Since Pumping Stopped (Min)	Water Level(m)	Residual draw down
0	26.55	5.95	600	21.87	1.27
1	23.71	3.11	660	21.85	1.25
2	23.56	2.96	720	21.82	1.25
3	23.51	2.91	780	21.78	1.22
4	23.47	2.87	840	21.75	1.18
5	23.44	2.84	900	21.70	1.15
6	23.42	2.82	960	21.66	1.10
7	23.40	2.80	1020	21.60	1.06
8	23.38	2.78	1080	21.53	1.00
9	23.37	2.77	1140	21.48	0.93
10	23.36	2.76	1200	21.41	0.88
12	23.35	2.75	1260	21.38	0.81
14	23.33	2.73	1320	21.34	0.78
16	23.32	2.72	1380	21.32	0.74
18	23.25	2.65	1440	21.30	0.72
20	23.20	2.60	1500	21.28	0.70
25	23.12	2.52	1560	21.26	0.68
30	23.05	2.45	1740	21.20	0.66
35	22.99	2.39	1800	21.18	0.60
40	22.93	2.33	1860	21.16	0.58
45	22.9	2.30	1920	21.14	0.56
50	22.86	2.26	1980	21.12	0.54
55	22.81	2.21	2040	21.1	0.52
60	22.79	2.19	2100	21.08	0.50
70	22.72	2.12	2160	21.06	0.48
80	22.65	2.05	2220	21.04	0.46
90	22.60	2.00	2280	21.03	0.44
100	22.54	1.94	2340	21.01	0.43
120	22.50	1.90	2400	21.00	0.41
140	22.44	1.84	2460	20.99	0.40
160	22.37	1.77	2520	20.98	0.39
180	22.32	1.72	2580	20.97	0.38
210	22.21	1.61	2640	20.96	0.37
240	22.16	1.56	2700	20.96	0.36
270	22.13	1.53	2760	20.95	0.36
300	22.08	1.48	2820	20.95	0.35
360	22.01	1.41	2880	20.94	0.35
480	21.93	1.33	2940	20.94	0.34
540	21.9	1.30	3000	20.93	0.34

Appendix (A-5) well no RPW-81

Time Since Pumping Stopped (Min)	Water Level(m)	Residual drawdown	Time Since Pumping Stopped (Min)	Water Level(m)	Residual drawdown
0	39.36	22.66	140	22.31	5.61
1	33.24	16.54	160	21.92	5.22
2	32.10	15.40	180	21.61	4.91
3	31.38	14.68	210	21.29	4.59
4	30.89	14.19	240	21.03	4.33
5	30.42	13.72	270	20.86	4.16
6	30.07	13.37	300	20.51	3.81
7	29.77	13.07	330	20.13	3.43
8	29.50	12.80	360	19.87	3.17
9	29.20	12.50	420	19.41	2.71
10	29.02	12.32	480	19.11	2.41
12	28.40	11.70	540	18.89	2.19
14	28.25	11.55	600	18.63	1.93
16	27.94	11.24	660	18.09	1.39
18	27.64	10.94	720	17.90	1.20
20	27.37	10.67	780	17.87	1.17
25	26.82	10.12	840	17.84	1.14
30	26.40	9.70	900	17.80	1.10
35	26.01	9.31	960	17.78	1.08
40	25.65	8.95	1020	17.74	1.04
45	25.26	8.56	1080	17.70	1.00
50	25.00	8.30	1140	17.69	0.99
60	24.51	7.81	1200	17.68	0.98
70	24.11	7.41	1260	17.67	0.97
80	23.68	6.98	1320	17.66	0.96
90	23.27	6.57	1380	17.60	0.90
100	23.11	6.41	1440	17.58	0.88
120	22.65	5.95			

Appendix (A-6) well no RPW-83

Time Since Pumping Stopped (Min)	Water Level(m)	Residual Draw Down (m)	Time Since Pumping Stopped (min)	Water Level(m)	Residual Draw Down (m)
0	20.07	8.65	180	13.97	2.55
1	15.87	4.45	210	13.94	2.52
2	15.30	3.88	240	13.89	2.47
3	15.20	3.78	270	13.85	2.43
4	15.02	3.60	300	13.77	2.35
5	14.87	3.45	330	13.63	2.21
6	14.79	3.37	360	13.56	2.14
7	14.71	3.29	420	13.50	2.08
8	14.66	3.24	480	13.45	2.03
9	14.63	3.21	540	13.42	2.00
10	14.59	3.17	600	13.39	1.97
12	14.55	3.13	660	13.36	1.94
14	14.50	3.08	720	13.25	1.83
16	14.46	3.04	780	13.21	1.79
18	14.44	3.02	840	13.18	1.76
20	14.41	2.99	900	13.13	1.71
25	14.39	2.97	960	13.07	1.65
30	14.36	2.94	1020	13.00	1.58
35	14.32	2.90	1080	12.82	1.40
40	14.28	2.86	1140	12.55	1.13
45	14.24	2.82	1200	12.47	1.05
50	14.20	2.78	1260	12.41	0.99
60	14.17	2.75	1320	12.18	0.76
70	14.15	2.73	1380	12.00	0.58
80	14.13	2.71	1440	11.89	0.47
90	14.11	2.69	1500	11.62	0.20
100	14.09	2.67	1560	11.56	0.14
120	14.06	2.64	1620	11.40	0.02
140	14.03	2.61			
160	14.00	2.58			

Appendix (A-7) well no RPW-111

Time Since Pumping Stopped (Min)	Water Level (m)	Residual draw down(m)	Time Since Pumping Stopped (Min)	Water Level (m)	Residual draw down(m)
0	49.79	25.64	120	31.20	7.05
1	40.63	16.48	140	31.01	6.86
2	36.94	12.79	160	30.90	6.75
3	35.36	11.21	180	30.41	6.26
4	35.09	10.94	210	30.11	5.96
5	34.70	10.55	240	29.88	5.73
6	34.63	10.48	270	29.51	5.36
7	34.37	10.22	300	29.16	5.01
8	34.18	10.03	360	28.71	4.56
9	34.05	9.9	420	28.11	3.96
10	33.88	9.73	480	27.86	3.71
12	33.74	9.59	540	27.43	3.28
14	33.55	9.40	600	27.21	3.06
16	33.42	9.27	660	26.97	2.82
18	33.25	9.10	720	26.92	2.77
20	33.20	9.05	780	26.88	2.73
25	33.06	8.91	840	26.84	2.69
30	33.03	8.88	900	26.80	2.65
35	32.82	8.67	960	26.76	2.61
40	32.69	8.54	1020	26.69	2.54
45	32.49	8.34	1080	26.60	2.45
50	32.34	8.19	1140	26.39	2.24
55	32.19	8.04	1200	25.14	0.99
60	32.04	7.89	1260	25.04	0.89
70	31.90	7.75	1320	24.98	0.83
80	31.65	7.50	1380	24.94	0.79
90	31.53	7.38	1440	24.90	0.75
100	31.40	7.25			

Appendix B; Meteorological data

Appendix (B-1) Monthly Patched rainfall time series of Alamata station

Year	Ja	Fe	Ma	Ap	Ma	Ju	Jul	Au	Sep	Oc	No	De	Annual
1954	0	48	17	10	45	95	15	98	46	11	1	9	495
1955	10	1	11	35	146	0	14	445	90	8	15	18	1161
1956	93	26	50	36	1	37	26	22	30	2	60	19	1282
1957	15	50	25	66	54	0	27	256	57	22	0	4	961
1958	1	35	4	16	9	96	7	55	30	0	55	49	508
1959	42	22	81	92	27	1	55	300	81	45	1	61	808
1960	10	1	18	44	52	1	11	93	25	17	1	7	543
1961	2	51	22	5	51	2	38	173	67	14	0	3	428
1962	2	2	17	17	33	2	61	150	24	10	84	31	746
1963	37	63	40	31	41	54	8	43	30	1	14	17	1001
1964	11	0	15	87	54	0	23	193	61	36	0	0	728
1965	29	68	30	12	97	6	21	392	95	80	2	27	1055
1966	13	0	18	11	30	1	95	0	10	0	1	26	471
1967	4	2	98	60	16	4	41	279	35	0	14	0	553
1968	24	0	18	67	18	14	3	183	30	4	0	37	565
1969	10	24	93	70	40	0	11	96	22	0	0	0	559
1970	6	43	15	40	9	4	99	124	42	17	0	0	399
1971	1	2	67	38	11	33	33	279	53	36	0	1	858
1972	6	1	14	14	117	3	2	155	26	8	49	5	662
1973	62	1	10	14	12	3	42	71	31	19	4	1	498
1974	56	12	31	24	85	1	15	7	22	0	10	67	427
1975	98	3	96	11	53	19	4	344	75	3	0	36	742
1976	1	12	53	15	110	0	17	184	68	38	52	7	849
1977	33	18	10	70	123	15	14	239	55	11	2	12	934
1978	26	18	29	28	10	1	9	0	10	9	55	105	463
1979	11	6	84	82	67	0	0	171	48	49	0	32	655
1980	17	16	33	62	4	11	37	63	29	66	0	2	822
1981	4	64	43	81	13	36	95	134	49	4	0	11	534
1982	38	93	14	63	70	0	3	178	48	18	22	6	685
1983	38	93	14	63	13	1	98	200	36	7	5	0	700
1984	0	6	0	8	116	0	23	6	21	0	6	77	263
1985	12	0	17	12	90	11	59	139	28	5	0	13	497
1986	0	66	38	18	31	38	10	230	54	0	0	120	866
1987	0	27	22	70	123	10	4	136	52	0	0	22	670
1988	24	13	50	12	0	0	23	248	77	30	0	5	920
1989	25	46	13	12	19	0	86	65	36	9	0	6	553
1990	36	88	26	83	29	5	25	53	27	0	1	0	1165
1991	42	15	59	48	28	0	19	456	89	10	0	10	1188
1992	12	43	16	28	78	0	72	24	79	43	97	132	740
1993	49	99	65	18	97	6	90	53	113	37	0	6	800
1994	0	0	11	58	14	0	15	283	77	0	21	1	715
1995	0	69	14	21	93	13	12	201	52	11	0	97	1016
1996	13	0	69	12	115	25	77	280	36	8	58	0	924
1997	46	0	12	27	29	22	87	54	72	19	13	0	793
1998	17	23	26	35	20	0	34	272	64	18	0	0	985
1999	44	0	21	9	7	1	21	432	67	55	0	0	847
2000	0	0	10	44	74	6	24	449	68	15	83	73	1068
2001	0	0	15	9	30	17	22	244	25	10	10	3	731
2002	72	0	18	11	8	4	73	214	46	14	0	90	651
2003	76	70	42	94	25	13	15	234	23	0	0	108	700
2004	33	16	40	16	14	50	11	241	41	8	21	20	769

Appendix (B-2) Monthly Patched rainfall time series of Waja station

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1953	0	3	108	114	18	14	143	200	22	12	3	0	637
1954	13	121	28	32	1	1	234	140	27	0	0	0	597
1955	10	19	125	149	9	0	210	252	92	6	5	28	905
1956	8	19	27	128	1042	135	9	324	10	43	32	604	2381
1957	0	2	30	136	16	0	166	173	58	2	0	0	583
1958	5	2	25	51	111	315	25	141	19	14	5	33	746
1959	1	4	22	102	9	8	153	128	86	8	1	8	530
1960	1	1	75	110	3	0	193	49	23	0	2	1	458
1961	0	0	19	28	2	2	23	111	133	46	0	0	364
1962	12	22	94	146	164	15	40	219	11	29	6	53	811
1963	1	9	9	132	699	121	105	207	12	61	8	83	1447
1964	2	2	91	20	13	10	50	297	192	15	0	2	694
1965	35	1	38	69	3	4	201	125	181	17	0	4	678
1966	0	5	92	41	56	7	58	279	13	1	28	1	581
1967	30	12	36	72	8	12	88	194	33	4	140	4	633
1968	202	9	92	73	20	25	44	319	5	6	2	1	798
1969	2	0	57	7	16	2	103	311	30	1	3	0	532
1970	1	0	89	145	2	22	56	79	12	5	0	0	411
1971	371	687	122	128	1	3	234	87	92	0	0	0	1725
1972	341	102	122	147	35	2	7	123	48	414	9	1	1351
1973	0	2	36	33	89	8	70	169	7	4	1	7	426
1974	5	0	42	2	2	10	18	367	4	33	216	30	729
1975	0	4	31	145	16	7	23	260	31	47	0	7	571
1976	21	299	28	82	19	1	85	241	131	0	28	5	940
1977	0	30	21	135	108	6	34	292	82	157	0	0	865
1978	17	7	21	1	2	4	50	68	10	3	206	97	486
1979	1	0	101	7	1	0	180	124	77	0	1	8	500
1980	10	5	11	141	275	17	48	113	65	3	1	0	689
1981	0	0	1	132	29	47	183	176	35	3	1	0	607
1982	27	430	7	108	5	17	74	104	63	14	0	3	852
1983	54	30	37	23	164	10	13	53	42	0	18	1	445
1984	0	0	17	26	146	4	35	70	0	0	0	0	298
1985	3	0	32	128	69	45	101	85	80	60	0	8	611
1986	0	79	13	93	38	36	132	182	110	0	0	67	750
1987	0	66	260	20	114	0	18	125	453	8	0	18	1082
1988	2	0	18	56	2	12	260	127	79	19	0	0	575
1989	23	21	106	108	7	0	64	130	15	4	0	1	479
1990	0	0	23	29	158	2	118	200	86	2	18	0	636
1991	0	78	91	174	37	0	152	197	38	11	0	98	876
1992	20	0	83	142	166	26	96	194	55	13	48	5	848
1993	10	0	93	48	0	6	68	54	13	223	120	0	635
1994	76	41	37	32	34	4	308	380	55	8	0	0	975
1995	41	30	42	36	4	3	240	196	43	85	0	0	720
1996	5	0	0	56	19	7	161	252	7	37	31	0	575
1997	0	0	58	20	27	339	139	197	37	10	0	0	827
1998	63	0	22	64	0	0	84	139	55	0	0	88	515
1999	0	61	42	88	0	40	92	324	47	0	1	43	738
2000	74	1	34	0	6	23	0	113	25	4	43	8	331
2001	0	8	17	133	91	7	142	175	92	0	61	0	726
2002	8	1	31	27	9	19	91	160	86	2	1	5	440
2003	2	0	32	44	14	41	151	308	3	22	1	8	626
2004	0	0	87	97	91	51	35	193	13	9	0	14	590

Appendix (B-3) Patched Minimum Temperature Alamata

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1960	6	11	14	14	15	17	15	15	13	11	10	10
1961	9	10	11	12	12	13	14	15	13	11	13	13
1962	11	11	12	11	11	10	15	10	12	9	10	14
1963	12	14	16	17	16	18	15	13	12	10	7	9
1964	15	12	12	12	11	14	13	13	13	8	10	12
1965	14	11	13	15	14	17	17	16	15	13	12	12
1966	12	15	15	15	16	18	17	16	15	15	13	10
1967	14	13	15	16	15	17	17	15	14	13	14	11
1968	5	13	13	15	16	17	16	16	15	15	13	12
1969	13	15	16	18	19	20	19	17	14	15	14	14
1970	14	14	16	17	17	18	16	17	14	14	12	14
1971	10	11	9	16	15	16	16	16	14	14	14	13
1972	8	14	14	16	17	20	18	17	15	13	15	14
1973	12	13	14	15	15	16	16	15	14	13	12	12
1974	15	14	16	18	17	16	18	18	15	13	15	14
1975	11	14	14	16	17	15	15	16	16	16	15	13
1976	11	14	13	18	15	17	16	11	12	7	10	5
1977	15	13	12	11	15	17	16	18	18	15	15	14
1978	11	15	15	18	17	18	18	18	16	16	14	13
1979	13	14	14	16	16	17	16	17	15	15	15	14
1980	13	14	17	17	20	20	18	17	19	18	15	14
1981	15	15	16	16	18	18	18	16	18	16	15	15
1982	15	15	15	18	19	20	19	18	19	17	15	10
1983	12	15	16	17	17	17	17	15	16	16	14	12
1984	11	7	9	10	15	16	18	18	17	13	15	17
1985	13	14	16	17	17	18	18	17	16	16	14	14
1986	12	16	16	16	17	19	17	17	16	16	14	14
1987	13	13	17	17	17	18	19	18	18	18	15	14
1988	15	16	17	18	19	20	19	17	16	15	12	12
1989	13	15	16	16	16	18	18	16	14	16	13	13
1990	13	9	11	13	19	20	19	18	17	13	13	15
1991	15	11	14	17	19	20	18	16	16	13	14	12
1992	13	16	9	17	19	19	18	17	17	14	14	15
1993	14	14	15	16	16	17	17	16	15	16	14	12
1994	13	14	17	18	19	20	18	16	11	13	10	11
1995	11	14	16	17	17	17	17	12	13	10	12	15
1996	15	13	16	16	16	16	17	16	16	14	13	12
1997	13	13	16	16	18	19	18	18	17	16	16	13
1998	15	15	17	18	18	20	17	13	17	15	15	12
1999	12	12	13	17	20	21	19	17	10	6	4	2
2000	2	5	7	8	10	11	12	9	9	8	7	6
2001	4	6	9	10	12	14	12	11	10	11	8	7
2002	10	14	17	18	19	20	20	17	17	16	15	16
2003	14	16	17	18	20	20	19	17	17	16	15	13
2004	15	9	9	16	18	20	19	17	17	15	15	15

Appendix (B-4) Waja Patched Minimum Temperature

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1960	10	12	15	16	17	17	18	16	16	14	11	9
1961	9	16	19	17	16	18	18	17	18	15	11	8
1962	7	18	16	18	13	15	19	19	19	17	18	18
1963	11	14	16	17	15	17	18	17	15	14	12	10
1964	15	9	14	14	16	17	18	17	15	13	10	6
1965	10	14	15	16	16	16	18	16	14	12	10	9
1966	15	10	13	12	17	18	18	16	15	12	9	7
1967	13	13	16	16	17	19	18	16	14	13	10	12
1968	17	9	15	13	16	17	18	17	14	12	9	8
1969	12	12	16	14	16	18	18	16	14	10	8	7
1970	10	14	18	16	16	17	16	16	15	13	12	11
1971	10	15	17	17	14	17	19	19	16	13	12	10
1972	14	11	14	15	16	18	18	18	19	17	14	13
1973	15	9	11	14	17	17	17	16	15	12	11	11
1974	13	13	17	14	16	17	17	17	15	14	10	7
1975	9	15	15	16	17	18	18	17	15	11	12	8
1976	12	12	14	15	17	18	17	16	15	13	15	15
1977	11	13	14	14	16	18	19	19	16	13	17	12
1978	15	10	17	17	16	16	19	17	16	15	12	12
1979	11	16	13	17	16	18	19	79	15	12	9	9
1980	12	13	16	17	16	17	19	16	16	16	11	9
1981	11	14	16	14	14	17	19	18	19	18	14	14
1982	10	15	16	17	14	16	19	19	16	14	9	8
1983	12	15	17	17	17	17	19	18	15	13	12	8
1984	8	15	17	17	17	19	18	16	15	13	17	15
1985	11	13	15	17	16	16	19	19	16	14	12	15
1986	9	16	14	16	16	18	18	17	16	14	11	13
1987	12	13	17	16	16	17	18	17	15	14	15	12
1988	10	14	16	16	16	18	19	17	15	12	8	9
1989	12	13	15	15	14	15	19	20	14	11	8	7
1990	10	13	16	17	16	19	18	17	14	11	12	9
1991	8	16	17	18	16	16	18	17	15	13	10	13
1992	14	12	17	16	16	17	18	17	15	11	11	9
1993	13	12	15	16	16	17	17	18	16	16	16	12
1994	15	14	17	15	15	18	18	17	16	14	10	6
1995	10	9	14	14	16	17	18	19	15	15	12	12
1996	15	9	12	16	16	17	18	16	15	13	14	11
1997	10	11	12	12	13	16	18	16	15	12	9	8
1998	16	10	12	11	17	18	16	16	17	15	15	14
1999	13	12	16	16	17	19	19	17	14	12	11	7
2000	14	13	13	15	14	17	17	21	21	18	16	12
2001	13	11	15	13	16	17	19	18	19	17	13	13
2002	16	9	16	16	16	16	17	16	16	13	17	10
2003	16	8	11	14	14	16	18	17	15	12	11	9
2004	13	11	13	15	14	17	18	17	20	15	14	9

Appendix (B-5) Patched Monthly Maximum Temperature Alamata

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1960	29	29	31	32	33	37	33	32	32	32	33	30
1961	32	29	31	31	35	37	30	28	31	32	28	29
1962	29	32	31	33	33	38	36	32	32	30	30	27
1963	26	25	31	31	33	36	33	31	30	32	31	30
1964	29	29	32	32	34	36	31	30	31	32	29	23
1965	24	26	28	27	31	34	32	28	28	31	26	27
1966	27	25	29	28	30	34	32	29	29	29	27	27
1967	26	27	28	28	29	33	29	27	29	28	25	25
1968	27	24	28	26	31	31	30	29	32	29	27	25
1969	26	23	29	30	31	35	31	29	28	30	30	26
1970	26	28	26	30	33	36	28	31	29	29	25	28
1971	26	28	30	31	31	29	29	31	29	30	29	27
1972	27	27	30	31	34	35	34	31	30	29	29	24
1973	26	30	29	32	32	36	34	34	33	30	28	28
1974	27	23	27	27	34	36	34	33	33	31	29	29
1975	26	26	29	30	29	32	30	29	29	28	26	27
1976	28	26	30	29	30	35	33	30	29	27	25	27
1977	26	26	29	30	33	34	31	32	30	32	30	29
1978	28	29	31	33	34	29	28	31	29	27	25	28
1979	25	25	28	30	30	33	30	29	28	30	29	28
1980	28	28	30	31	37	36	31	31	31	31	30	27
1981	27	28	27	28	33	34	33	32	32	32	29	28
1982	29	30	29	32	36	36	31	32	31	30	31	29
1983	25	25	27	29	31	35	34	32	34	33	29	27
1984	26	27	31	33	30	33	33	33	31	30	28	25
1985	27	27	29	28	30	34	32	31	30	29	28	26
1986	27	27	29	27	32	33	31	29	30	30	29	25
1987	26	28	27	28	29	34	35	31	33	30	29	28
1988	27	28	29	30	33	35	30	30	30	30	28	29
1989	25	26	28	27	32	34	33	33	33	30	29	30
1990	28	27	30	32	36	36	32	30	29	29	29	28
1991	28	28	30	30	31	35	31	28	31	30	28	28
1992	25	28	30	30	31	33	32	28	28	28	25	27
1993	24	23	29	27	29	24	23	32	32	30	30	29
1994	30	31	30	32	34	35	32	30	31	31	30	30
1995	30	29	29	26	32	35	33	30	31	31	30	28
1996	30	29	31	31	31	33	32	31	32	31	29	28
1997	27	30	30	30	34	35	33	34	33	30	29	29
1998	27	28	31	35	35	36	32	29	31	31	30	30
1999	29	32	31	34	36	36	31	30	30	30	30	29
2000	30	32	33	34	35	37	33	31	31	29	29	27
2001	28	30	31	31	34	34	31	29	30	30	28	28
2002	25	28	30	30	34	34	34	31	29	31	29	26
2003	26	29	29	30	33	34	31	29	30	30	28	26
2004	27	22	30	29	33	33	32	30	30	29	28	26

Appendix (B-6) Waja Patched Monthly Maximum Temperature

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1960	27	27	27	30	30	31	28	30	31	27	27	28
1961	29	32	31	32	34	34	33	33	35	31	30	27
1962	26	28	30	30	33	35	33	32	30	29	25	27
1963	26	29	31	33	34	34	29	27	27	24	28	29
1964	29	27	26	27	30	34	33	32	31	31	28	28
1965	29	30	31	32	31	37	34	28	29	30	31	29
1966	29	27	30	30	32	38	34	29	27	30	29	29
1967	27	26	28	27	29	37	33	32	33	31	31	27
1968	26	31	31	32	32	35	34	28	27	26	29	28
1969	23	25	22	28	28	36	32	28	27	28	25	28
1970	29	29	30	31	32	34	34	31	30	30	29	27
1971	29	29	29	29	30	36	28	24	25	27	25	28
1972	29	29	31	30	34	33	33	26	25	30	30	27
1973	28	29	31	32	32	35	34	30	28	29	28	29
1974	28	30	26	27	31	35	34	33	33	30	30	28
1975	24	30	31	31	33	34	31	33	33	30	30	29
1976	28	25	28	30	34	33	31	32	33	31	28	29
1977	28	31	30	33	32	35	34	32	32	31	31	28
1978	29	31	30	31	33	34	34	32	32	31	29	28
1979	26	25	26	28	32	35	34	31	31	28	28	29
1980	29	30	30	28	36	31	31	31	30	31	30	28
1981	27	29	29	31	31	32	29	31	32	29	27	29
1982	28	31	30	30	31	32	30	32	32	30	30	27
1983	25	26	28	29	30	32	34	32	32	29	28	27
1984	27	28	30	32	31	32	34	26	26	28	31	28
1985	27	27	30	29	30	34	32	32	30	29	29	27
1986	27	27	30	28	31	33	31	30	29	30	29	26
1987	27	28	27	28	29	33	34	31	30	31	29	28
1988	28	27	30	30	33	34	31	29	29	28	27	27
1989	25	27	28	26	32	35	33	29	29	31	31	28
1990	30	30	29	30	33	36	34	31	32	31	32	28
1991	27	32	30	31	32	33	33	31	32	31	29	28
1992	28	30	30	31	34	34	33	34	35	30	29	29
1993	28	29	30	31	34	37	32	29	30	30	29	29
1994	28	31	29	33	34	32	29	30	30	28	29	27
1995	24	27	28	27	29	30	27	25	25	23	23	27
1996	21	23	23	25	27	36	34	32	32	31	29	29
1997	29	29	30	28	34	34	33	32	32	30	30	27
1998	28	31	31	32	32	36	32	31	30	28	29	28
1999	28	28	28	26	31	34	33	33	32	31	30	28
2000	29	32	30	28	32	34	32	33	31	30	29	27
2001	25	29	31	33	29	36	31	31	33	30	29	28
2002	27	28	30	31	32	31	30	31	31	29	28	27
2003	28	27	28	29	33	39	34	33	32	29	29	27
2004	27	27	25	31	32	33	32	30	31	31	31	27

Appendix (B-7) Patched Monthly Wind Speed (m/s) Kobo

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1976	1.8	2.1	2.1	2.1	1.6	2.6	2.3	2.0	1.7	1.8	1.8	2.1
1977	2.6	2.6	2.3	2.1	1.6	2.0	2.2	1.5	1.0	1.2	1.4	1.5
1978	1.7	2.0	2.0	1.7	1.6	1.9	1.6	1.2	1.1	1.3	1.8	1.7
1979	1.8	1.9	2.2	1.8	1.6	1.7	1.8	1.3	0.9	1.0	1.2	1.5
1980	1.7	1.6	1.6	1.5	1.3	1.4	1.0	1.0	0.1	0.0	0.0	0.1
1981	1.5	1.8	1.8	1.6	1.8	1.1	1.0	0.8	0.6	0.8	0.9	1.0
1982	1.7	2.2	2.1	1.3	1.0	0.9	1.0	0.8	0.7	0.7	1.2	1.8
1983	1.8	2.2	2.3	2.1	1.8	1.8	2.3	2.2	1.9	1.7	1.8	2.1
1984	2.0	2.0	2.5	2.4	2.4	2.3	2.5	2.4	2.2	2.1	2.0	2.1
1985	2.2	2.4	2.5	2.2	1.8	2.2	1.7	2.3	1.6	1.7	2.0	2.1
1986	1.5	2.1	2.1	2.0	1.7	2.1	2.1	1.7	1.2	1.5	1.6	1.6
1987	1.9	2.1	2.1	2.0	1.8	1.8	2.2	1.8	1.8	1.7	1.7	2.1
1988	2.0	2.2	2.2	2.3	2.2	2.3	2.4	1.6	1.1	1.4	1.5	1.7
1989	1.8	2.2	2.1	2.0	2.1	2.3	2.7	2.1	2.0	1.9	2.2	2.1
1990	1.9	1.8	1.6	0.9	0.5	2.0	0.7	0.8	0.0	0.0	0.0	0.0
1991	1.5	1.8	2.3	2.0	1.7	2.0	2.1	1.8	2.0	2.1	2.4	2.6
1992	2.1	2.2	2.4	2.3	2.2	1.7	2.3	1.7	1.6	1.6	2.1	2.1
1993	1.9	2.1	2.0	1.5	1.3	1.5	2.0	1.8	2.0	2.1	2.4	2.6
1994	33.4	3.7	2.5	2.3	1.8	1.8	2.3	1.9	2.0	2.1	2.4	2.6
1995	2.2	2.3	2.2	1.8	1.4	2.5	2.1	1.6	0.4	2.1	2.4	2.6
1996	2.6	2.0	2.1	2.0	1.6	1.8	2.1	1.7	1.2	1.3	1.5	1.6
1997	1.8	2.0	2.1	2.0	2.2	1.9	2.0	1.7	1.6	1.4	1.3	1.4
1998	1.6	1.7	2.0	2.2	2.0	2.5	2.2	1.6	1.1	1.3	1.4	1.5
1999	1.6	1.8	2.2	2.1	2.1	2.3	2.0	1.5	0.0	0.0	0.0	0.0
2000	1.6	1.8	2.2	1.8	1.7	2.2	2.2	2.0	1.5	0.0	0.0	0.0
2001	1.5	1.7	1.7	1.4	1.2	1.4	0.5	0.8	0.2	1.6	1.7	1.8
2002	1.8	1.8	1.9	1.8	1.4	2.3	2.1	1.8	0.9	1.0	1.3	1.5
2003	1.6	1.7	2.0	1.9	1.5	2.0	2.0	1.4	1.0	1.1	1.3	1.3
2004	1.7	1.9	2.0	2.0	1.8	2.1	2.2	1.5	1.5	1.2	1.3	1.6
2005	2.0	2.1	2.2	2.0	1.6	1.9	2.0	1.8	1.2	1.2	1.3	1.5

Appendix (B-8) Patched Sun Shine duration (hours) Kobo

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1976	7.8	6.5	8.4	8	9.2	8.2	5.6	7.1	7.6	8.4	6.4	6.9
1977	5.5	7.1	8.8	8.9	8	7.4	6.4	6.5	6.9	8.7	8.7	8.4
1978	9.1	7.4	8.1	9.3	9.8	5.5	4.2	7.1	6.6	8.4	8.4	7.9
1979	5.4	8.4	7.4	8.8	7.6	6.8	7.1	6.2	7.8	8.5	9.9	8.3
1980	7.9	7.2	8.1	9.1	9	7.2	5.8	6.5	6.8	8.7	8.8	9.1
1981	9	9.1	8.7	6.8	8.3	6.2	6.4	7	7	9.5	9.4	8.5
1982	6.1	5.2	8.2	7.5	8.5	7.9	5.2	4.6	5.6	8.1	8.6	7.5
1983	7.2	8.9	6	7.9	7.1	5.6	5.4	7.2	7.3	9.4	9	8.7
1984	8.8	10	9.2	10.4	5.1	5.6	7.3	5.9	6.7	9.8	9.3	8.3
1985	9.4	8.5	8.9	7	7.8	7.6	7.5	3.6	4.7	8.6	10	8.3
1986	9.5	6.8	5.3	6.6	9.3	4.8	4.1	4.6	5.7	8	10	7.8
1987	8.3	7.5	5	7.5	5.6	5.6	7	6.7	7.7	9.3	10	8.3
1988	7.6	6.4	9.3	7.7	9.4	6	3.3	5.4	5.7	7.2	9.9	9.5
1989	9.2	6.3	7.3	7.1	9.6	6.2	5.6	6.6	7.6	8.1	6.7	5.9
1990	4.2	9.3	4	7.1	7.2	5.9	6.3	7.3	6.8	8.7	10.3	8.9
1991	9.4	7.5	9.5	8.7	10	7.1	4.8	6.7	5.8	8.8	9.5	8.1
1992	5.9	8.8	8.6	9.5	8.4	8.2	6	7.1	5.9	8.9	7.3	8.5
1993	8.6	8	9.4	7.4	9	7.3	6.4	5.8	5.8	8.2	9.1	7.4
1994	9.8	9.2	8.2	9.3	7.8	5.9	7.4	6.1	7.7	8.7	9.9	9
1995	9.8	8	9.5	7.8	9.2	7.6	4.9	7.1	7.9	8.6	9.8	8.7
1996	6.8	5.3	7.9	9.6	9.9	8.5	4	5.1	6.8	9.6	9.9	8.8
1997	8	7.3	9.3	6.9	9.4	6.2	3.1	5.1	7.3	9.5	9.7	8.1
1998	6.3	7.5	8.9	9.5	9.2	6.8	6.7	7.4	6	7.1	9	7
1999	6.9	4.4	8.7	6.6	9.1	6.3	6.1	6.5	7	9.6	10.1	9.3
2000	9.3	7.2	6.1	7.6	9.1	5.1	5.2	5.1	5.1	8.2	9.3	8.7
2001	8	7.4	8.9	7.9	9.8	8.3	3.4	6.6	7.1	7.9	8.1	7.9
2002	7.8	8.6	8.7	9.7	6.5	5.7	4	5.8	7.7	8.9	9.1	8.5
2003	8.7	10.2	8.1	9	6.8	6.4	8.3	7.1	6.7	7.3	8.6	9
2004	8.1	8.7	9	6.8	7.2	6.1	7.3	5.6	6.2	8.1	9.9	9.4