Integrated Cost-effective Model for Augmenting Sustainable Water Supply through Surface and Groundwater Interaction in Hydrological and Hydro-geologically Critical Areas of West Bengal

Thesis Submitted by

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Dedication

"I dedicate my thesis work to my family and friends. A special feeling of gratitude to my loving parents and Elder Brother"

Title of Thesis

Integrated Cost-effective Model for Augmenting Sustainable Water Supply through Surface and Groundwater Interaction in Hydrological and Hydro-geologically Critical Areas of West Bengal.

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LIST OF PUBLICATION

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6	Development and hydraulic analysis of proposed drinking water distribution network using water gems and GIS	Research Res.	34	2	2015	371-379
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10.	A study to ascertain the optimum yield from groundwater source in the eastern part of Kolkata Municipal Corporation Area in West Bengal, India	European Journal of Sustainable development	1	2	2012	97-112

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CERTIFICATE FROM THE SUPERVISORS

This is to certify that the thesis entitled **"Integrated Cost-effective Model for Augmenting Sustainable Water Supply through Surface and Groundwater Interaction in Hydrological and Hydro-geologically Critical Areas of West Bengal.**" Submitted by **Sri Gourab Banerjee,** who got his name registered on 18.09.2012 for the award of Ph.D. (Engineering) degree from Jadavpur University is absolutely based upon his own work under the supervision of Prof. (Dr.) Asis Mazumdar, Prof. (Dr.) Arunabha Majumder, Dr. Pankaj Kumar Roy, and that neither his thesis nor any part of the thesis has been submitted for any degree/diploma or any academic award anywhere before.

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CHAPTER-1
INTRODUCTION

1. INTRODUCTION

1.1. General Background

Water is most precious and a universal asset. Water provides life supporting system of human beings, vegetation/plants and animals. It is also very important part of socio-ecological environment. It is evident from the fact that without water no civilization can sustain. That is why; ancient civilizations are developed along the water courses and near water bodies. Great importance of water and its development for all kind of needs including environmental functions can be found in ancient texts, local tradition and archeological remains. But these vital natural resources are depleted rapidly, since this life saving resources are always taken for granted in name of development. The twentieth century has seen phenomenal growth in the use of water and the easy accessibility of groundwater by even small-scale users, its local availability and the difficulty of coordinating and governing many users of the same aquifers across wide geographic spaces has frequently led to indiscriminate extraction of this precious natural resource for domestic, industrial and agricultural uses around the world. Groundwater exploitation, particularly in India, has increased by leaps and bounds over the last 50 years along with the expansion of shallow, mostly private, wells. The growth of groundwater abstraction from 1950 to 1990 clearly depicts the increasing use of ground water utilization across sectors. Groundwater is not an independent entity as it is only one of the phases of the Hydrologic cycle. This cycle is maintained by precipitation, runoff, evaporation, transpiration, ground water flow, each one of which is dependent on the other. Though precipitation is the main source of groundwater, all of the precipitated water does not form groundwater. Only a part which gets infiltrated into the ground reaches the groundwater aquifer underneath and the rest flows down to the sea as runoff and goes back to the atmosphere. It is, therefore, obvious that the occurrence, movement and storage of ground water would be intricately related to the climatological behaviour, the quantum and subsurface geological conditions. Ground water recharging has becoming an emerging new paradigm in water resource development and management. Through different type of groundwater recharging method this over exploitation can be mitigated to great extent. Apart from natural recharging areas it essential to adopt different method of reaching to rise the water level at a greater extent in the long run to meet the demand of water for all stakeholders for sustainable development of groundwater.

1.2. Importance of research in regional and global perspective

Groundwater in the area to the western part of West Bengal the rocky terrain and literate bearing undulating uplands occurs under water table condition in the near surface aquifers and under confined conditions in the deeper aquifers. Excepting some local recharge zones which replenish groundwater in the near surface aquifers, the deeper aquifers in the eastern parts separated by impermeable clay zones get replenished from infiltration of the precipitated water in the distantly located recharge zones. These recharge zones are located along the boundary of the rocky and the alluvial areas to the west and also in the northern portion of this part of the state located in the Murshidabad and Nadia districts where a thick granular zone from the surface down to 150 m depth occurs. The deeper aquifers below Kanchrapara in the 24 Pargana (North) to far south up to the coast are recharged from the north. The above discussions clearly depict the close relationship between the physical and geological factors with regard to groundwater occurrence and availability (A.B.Goswami). Over the past three decades, government policies that subsidize credit and rural energy supplies have encouraged a phenomenal increase in the growth of groundwater development as the ground water abstraction structure have increased from 4 million in 1951 to nearly 17 million in 1997 (Chadha, 1999). Groundwater extraction is convenient since it is available on demand. The status of ground water development for West Bengal state is in stress condition .At present, the groundwater contributes about 50 percent of water for irrigation, 80 percent for domestic use in rural areas and 50 percent in urban and industrial areas. Out of assessed 23.09 bcm of replenishable groundwater, a provision of 3.46 bcm has been made for domestic, Industrial and other uses, with remaining 19.63 bcm for irrigation. Rapid pace of groundwater development is resulting in over-draft and water table is declining at an alarming rate. Excessive extraction of groundwater by tube wells in the sweet water zone of West Bengal declined its water table at the rate of 23 cm per annum. An area is said to be overexploited when the level of groundwater utilization exceeds the annual recharge of groundwater in that area. Hydrological classification constitutes the first step of a new holistic framework for developing regional environmental flow criteria: the aim of this study was to develop a cost effective technology models for recharging in over-exploited groundwater resources through application of different groundwater conservation structures(Check dam, Collector, well, recharge shaft, Subsurface-Dam) in Hydrological and Hydro geological critical areas of West Bengal and assess the performance of different recharging technology units including economic viability, social acceptability and technical feasibility. Eco-friendly recharging technique are come out from the field study and prepared GIS based Map. To achieve the goal initially hydrogeological investigation done for classification of 90 stream sections of the Damodar, Kangshabati, and Darorokeswar river on 10 hydrological indices that characterize their natural flow regimes and also Identify Hydrological & Hydrgeological critical areas and to provide sustainable and Cost effective solution. The hydrological indices were calculated with 25 years of natural monthly flows (1990-2015) derived from a rainfall runoff model developed from historical data and field survey data. These indices included, at a monthly or annual basis, measures of duration of droughts and central tendency and dispersion of flow magnitude (average, low and high flow conditions. These classes can be simplified in 3 broader groups, with different seasonal discharge pattern: large rivers, , perennial seasonal streams and intermittent and ephemeral streams. They showed a high degree of spatial cohesion, following a gradient associated with climatic aridity from NW to SE, and were well defined in terms of the fundamental variables. Ecological point of view have been used for hydrologic classification to place individual streams and rivers into a broader spatial context with the goal of maximizing the transferability of knowledge among rivers of the same hydrologic class and to show broader application of intensely studied sites to unstudied sites .Hydrologic classes are expected to have similar biological responses to both natural and human-induced variability in patterns of magnitude, frequency, duration, timing and rate of change in flow conditions. Consequently, systems that show commonalities in their hydrologic characteristics have provided a basis for testing whether hydrology influences the structure and function of biological communities in a similar fashion (e.g. Poff and Ward 1989; Jowett and Duncan 1990; Poff and Allan 1995; Snelder et al. 2004). Recent efforts have also focused on how rivers in different classes vary with respect to the degree of human influence (e.g. land use, river regulation, construction of barrier or dam, with drawl of water for Industrial and agricultural uses , drinking water and pollution concentration) thus providing a benchmark against which the response of biological communities to these factors can be accessed through Detail survey. In hydrological classification within above mentioned three rivers Damodar river basin is only classify as large river catchment area but due to human influence construction of DVC project, Shantaldihi thermal plant and drinking water project with drawl rate was very high and riches to hydrologic critical condition. Other two river basins Kangshabati and Darokeswar are comprise of small catchment areas and short duration irregular rain fall with high bed slope. Therefore it helps to drain quickly to downstream and creates a hydro-geological critical condition in river basin. Hydro-geologically three river catchments area is underlain by granite gneiss and quartzite. The depth of weathering which is an important factor to locate the areas for good groundwater recharge is more in case of the former while the latter is less weathered. The northern part of the area falling along the course of Dwarekeswar river is underlain by quartzite while granitic rocks are available in the sub-surface along the catchment of Futiary stream. Depth to water level as observed from the wells varies from 7.5 to 11.0 m below ground level during pre-monsoon 4.8-7.5 m below ground level during post-monsoon. Although the weathered zone is having good proportion of sandy residue with relatively high porosity and permeability, moderate to high slope of the country facilitates quick recession of groundwater during lean period. Hence, the wells located in higher topography become dry during summer. As a result this two river also riches the critical condition. As the rivers are mostly gaining in nature, groundwater recharge activities are increased in the upper catchment along the river, the same would regularly enrich the base flow of the rivers consequently the rivers become more perennial. All these scientific observations and knowledge are utilized during framing the model of conservation and recharge in the area.Economic approach to the management of water resources is one of the balancing competing objectives. It is obvious that improvement in quality of living environment enhances our sense of well-being. Water constitutes a major part of our environment, and individuals quite naturally express their preferences for clean water and in adequate quantity. Supply of safe drinking water is a global concern. In the context of developing countries where issues of poverty and inequality deserves special attention, provision of safe and clean drinking water at an affordable price is also inextricably tied to efforts to erase gender inequality, alleviation of poverty, and enhancement of productivity (Jain 2012). The problem is particularly acute in the Southeast Asian countries where water resources have suffered both qualitative and quantitative degradation in a massive way under an enormous population pressure. Recent satellite-based estimates show that groundwater level is rapidly declining in many parts of India (Matthew et al. 2009). Rising demand for water by ever-growing population has forced over extraction of groundwater in many areas. Contamination of underground aquifers by leached out arsenic and fluoride has further aggravated the problem. In the arsenic contaminated areas, concentration of arsenic in groundwater has been found to reach up to 3 mg/L against the WHO-prescribed maximum permissible contaminant level (MCL) of 0.01 mg/L (Pal et al.2012). Concentration of fluoride in the groundwater of the surveyed block was 12–15 mg/L against the MCL of1.5 mg/L as set by WHO (Chakrabortty et al. 2013). High population density and ever-degrading groundwater quality have resulted in rapid decline in per capita availability of safe and clean drinking water (Roy 2010; Roy et al. 2008). Threats of climate change and inadequate rainfall have added other dimensions to the already stressed water resources. The state of West Bengal an eastern province of India is currently confronted with the challenge of supplying safe and adequate drinking water to millions of its people living in rural areas because of contamination of groundwater by arsenic and fluoride. Ten districts (out of a total of 18 districts) of West Bengal are arsenic affected (involving 30 million people) and the fluoride contamination problem has affected 43 blocks spread over seven districts (Bhattacharya and Chakrabarti, 2011). In most of these affected areas, alternate river water sources are often far away particularly from its fluoride-affected and water stressed regions. When bringing safe drinking water from faraway places is difficult, the other solution that remains is treating contaminated groundwater to the safe level using modern separation and purification technology. However, literature shows that such options have hardly been tried. Thus despite several studies on the groundwater contamination problems, people still continue to suffer as scale-up confidence is often limited for setting up such modern technology-based treatment plants. The common belief is that advanced water treatment schemes are likely to be very expensive and the affected people cannot think of such an advanced water treatment scheme. There has neither been any convincing techno-economic feasibility study. Thus developing a sustainable water resource management strategy remains a big challenge to the scientific community and the policy maker for such areas facing acute drinking water crisis. This necessitates thorough economic evaluation and comparison of the alternate technologies with respect to the affected region. This article aims to assess the current water status of such a critically water-stressed region of India both quantitatively and qualitatively and estimate benefits of water status improvement in order to develop a sustainable water management policy for a water scarce region. The selected study region as indicated by arrow in the map, presents a critically water-stressed region of West Bengal. This district is one of the most economically backward districts of West Bengal. Average annual rainfall in this district is only 1,200 mm. Rainfall in this region is below the state average, surface water resources are very scarce and groundwater in a number of blocks of the district is fluoride affected (Gupta et al. 2006; Ghosh et al. 2010). The district suffers from acute shortage of drinking water supply during the dry months and draws critical and immediate attention of the decision makers. It transpires from literature that the proposed research is the first of its. Keeping in view, to mitigate the acute water crisis not only in drinking water but also to provide irrigation water in Purulia district, an extensive water harvesting through recharging structures have been planned and subsequently few structures are selected and implemented with design based criteria. Under these circumstances, the overall objective of research work highlights a low cost technology model has been design and developed in the undulating rocky terrain. The pilot model cater the need for recharging in water scared particularly in valley and foot hills areas and for project scenario through the development and management of harnessing excess water for different stake holder. The pilot model implement at identified location, it will exhibit its efficacy in the rocky terrain and it would be a tool for policy maker for sustainable water recourse development in other similar terrain not only in West Bengal but also in the states like Jharkhand, Bihar etc. is to identify the suitable areas for recharging structures with low cost technology models through people's participation and their willingness so that the overall storage water could be augmented over implementation in the defined catchment and also the water table and the soil moisture would be raised as well to mitigate the water problem contaminated with fluoride and other metal substances.

1.3. The study area

The area under consideration in this research is the Purulia district which includes three river basins Kangshabati, Damodar, and Darokeswar catchment area and the Field model has been installed in Ghutlia village falling under Simla-Dhanera GP of Kashipur block In Purulia district. Location map of the study area shown in Figure 1.1 below.

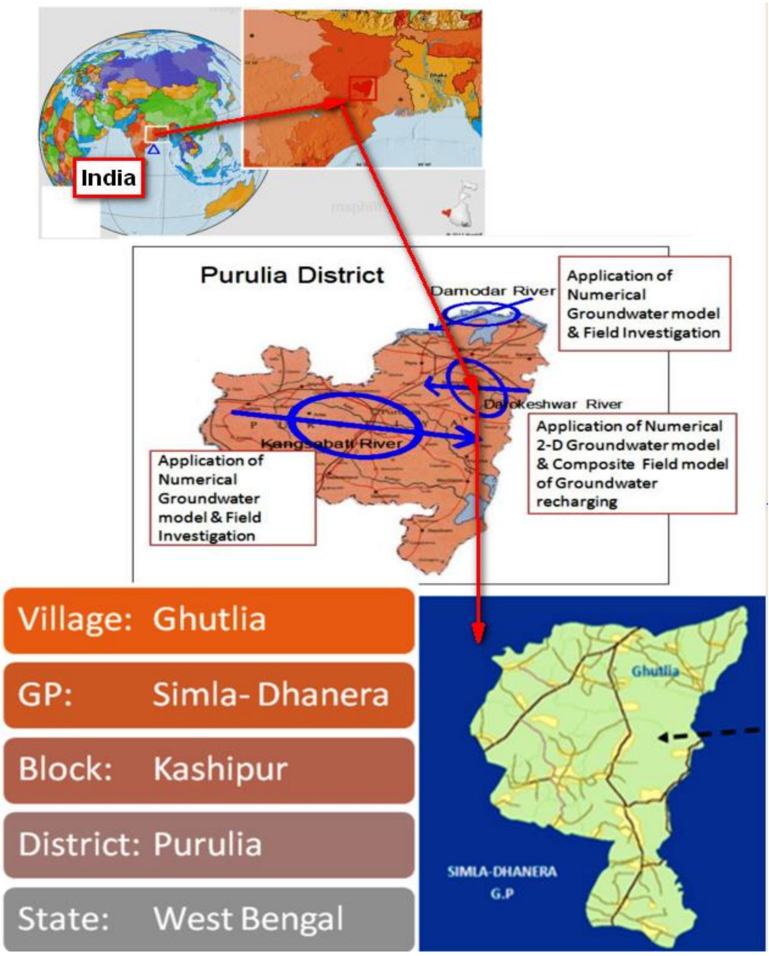


Figure 1.1 : Tangible road map of study areas

1.4. Research motivations

- Groundwater is an important resource for India and thus for sustainable use of it, a sustainable cost effective recharge technology is vital;
- Purulia is hydrogiologically critical area, drinking water scarcity problem in drought prone Purulia district is a major problem therefore mitigation of water scarcity is needed. Most of River and stream in the Purulia district are almost dried up in summer season and as well as in winter season. Therefore, groundwater is an important resource for the Purulia District and thus its recharge is vital ;

- Site-specific characterization is needed to identify areas of high potential of groundwater recharge and hence every care should be considered when developing land use categories and related practices.

- Literature survey reveals that application research was so far carried out to ascertain groundwater recharge potential and implementation model of utilizing that potential in hydro-geologically critical areas of West Bengal.

1.5. Research objectives

The main objective of the present research work is to develop cost effective technology models for recharging in over-exploited groundwater resources through application of different groundwater conservation structures in hydrological and hydrogeological critical areas of West Bengal and assess the performance of integrated model involving different recharging technology units including economic viability, social acceptability and technical feasibility.

1.6. Beneficiary stakeholders

The outcome of this research will be of great importance to the following:

- **Water resources development managers;** From this thesis work water resources managers will be apprised about composite model of cost effective technology for recharging in rural areas where ground water is over exploited as well the area is hydrogeologicaly critical. This study will also provide an insight of optimal withdrawal of sub surface from hydrological critical areas.

- **Decision makers**; This work will develop a database which culminates into GIS based map, out of VES survey pump testing and determine sustainable withdrawal aquifer response occurs. This will helps future planning for groundwater utility.

- **Education and Research sector**; There is a direct need to develop an environmental information system and a groundwater information system such that available data pertaining to the Ghutlia drained by river Darokeswar and its tributary locally known as futiary jhor(stream). It is also be helpful as a case study for planning cost effective ground water recharging structure in over exploited ground water resources in rural area will facilitate water management for future research. The present application research may be useful in in apprising the concerned department to replicate in the critical areas as well as technical input to the researcher developing ground water resource for an effective and sustainable use of this important natural resource.

- **Farmer & Villagers;** Direct impact on agricultural command area and drinking water availability purpose of the villagers in the adjacent area where field model is installed.

1.7. New research dimension

The present application research of this thesis work highlights the design, field study and deployment of low cost groundwater recharging model has in the undulating rocky terrain. The pilot model may cater the need for recharging in water hydrologically and hydrogeologically critical areas for future project scenario through the development and management of harnessing excess water for different stake holder. Once the pilot model could implement at identified location, it will exhibit its efficacy in the rocky terrain and it would be a tool for policy maker for sustainable water recourse development in other similar terrain not only in West Bengal but also in the states like Jharkhand, Bihar etc.

1.8. Methodology

The overall research methodology is mainly divided into four components: data collection, data analysis, model development and output and result analysis. A Gram panchayat in one district was chosen through participatory rural appraisal technique and with consultation of local community or panchayat. Accordingly critical water scared falling under

Simla-Dhanera Gram Panchayat Kashipur block in Purulia district was selected based on base line house hold survey carried out questionnaire format. Land use map includes drainage boundary topography etc. will be prepared based on GIS software (ERDAS), Surfer 8.0 and GPS technology. Scientific studies were carried out the project related data pedology, geology, geomorphology, surface groundwater identified locations. Finally a cost-effective composite model includes Check dams in series, Recharge wells with shaft, Percolation Tank with recharge shaft, sub-surface dam, Multipurpose collector well with side spillway and infiltration gallery in valley will be designed and developed for groundwater recharging in over exploited areas like Purulia district in west Bengal .Since the selected areas i.e. Purulia district in West Bengal is underlain by hard rock's with limited potential of groundwater, rainwater harvesting and recharge to sub-surface reservoir and optimal development of groundwater in the micro watershed is to be achieved for utilization of both surface and ground water potential to mitigate the acute crisis of drinking water as also to provide irrigation water in the study area, intensive rainwater harvesting and artificial recharge of ground water have been envisaged which can be achieved through proper application of watershed development or groundwater recharging techniques. For the up liftmen of the poor SC and ST population in the study area, watershed development is the prime factor to upgrade the economy of poor inhabitants.

1.9. Research outcomes

The cost-effective composite model includes Check dams in, Recharge wells with shaft, Multipurpose collector well with infiltration gallery in valley, and it has been designed based on households base-line survey, hydro-geological investigation through PRA exercise and with consultation of local communities, as well as collection of different designed based on hydro-geological and hydro-meteorological scientific data and site suitability. Organized collected data source of different designed based hydro-geological and hydro-meteorological scientific data for site suitability at that region. Assessment of per capita drinking water availability in the village.Per Capita drinking water availability will be mitigated to same extent as per standard norms to all the villagers in Selected Gram Panchayat of Purulia district and the irrigated area could be extended up to a greater extent so far their waterless land resources are concerned.

1.10. Thesis outline

The thesis consists of eleven Chapter 2 contains literature review. Chapter 3 describes the study area and its characteristics. Chapter 4 presents the research methodology. Chapter 5 survey and investigation. The process site selection with application of GIS and MCET approach described in chapter six Water Quality Scenario included in chapter -7and Hydrological data analysis and processing with numerical model discussed in Chapter 8 An overview of the development of the Cost-effective groundwater recharge model is discussed in Chapter 9. Chapter 10 demonstrates result and Impact analysis of the model. Finally, conclusions and recommendations are provide in chapter 11.

CHAPTER-2
LITERATURE REVIEW

2. LITERATURE REVIEW

2.1. State of the art work in the global perspective studies to developed rain water harvesting and groundwater recharging.

High undulating terrain, irregular monsoon, increasing industrial, irrigation and drinking water demand and overexploitation leads to exhaustion of groundwater resource and drop of groundwater layer in many parts of the world. This has create attraction to the researcher, government authority and policy maker understand, agree, and execute recharge of groundwater artificially in order to arrest such rapid lowering groundwater tables (Samadder et al., 2012). Therefore, artificial groundwater recharge for subsurface water storage has become an important aspect of studies and an effective measure to manage water-scarcity problem trough out the world (Barksdae and Debuchanne, 1946; Todd,1959) Alternative rain water harvesting methodology is playing crucial role for urban water scare area.(Wright and du Toit, 1996) Artificial recharge scheme can be established with help of regional study (CGWB, 2000) Hydrogeology is essential aspect for regional study (Bouwer, 2002; Asano and Cotruvo, 2004; Ong'or and Long-Cang, 2009). Rainwater harvesting and groundwater recharge of aquifers is affected by many factors such as :population growth, socioeconomic need and demand, economic feasibility (capital investment, operation cost, cost-effectiveness, skilled operational staff availability etc.)physical and technical feasibility (water quality, source water, site suitability, compatibility between source water and native water, etc.), and expected benefit or result (quantity of replenishment, fulfillment of demand, sustainable use of the extracted water, etc.). Groundwater recharge through recharge shaft in surface water recharge method collect fresh water from rivers, reservoir ,harvested rain-water into an aquifer via single well or a group of wells during periods of monsoon when water surplus, and is withdrawn at the time of summer and winter when water availability is less . Long-term groundwater monitoring data is help full to reveal scenario of the groundwater system (Turner et.al 2015) Subsurface water source creation has many advantages over surface water source creation such as less chance of contamination and evaporation loss, non-occupancy of space over the land and little maintenance cost. Arid and semi-arid regions hard rock areas, groundwater aquifers is already over-exploited, groundwater recharge has capacity to increase storage water and soil moisture also reduce drought (Dillon, 2005). Sometimes aquifer productivity is also measured and mapped using kringing a geo-statistical interpolation techniques such as, but these techniques tend to be unsuitable for mapping at the larger scale due to requirement of the more data and time (Corinna Abesser and Melinda Lewis 2015). To estimate changes in groundwater storage from changes in gravity over time. Maps of other key hydrologic budget components (Wolock 2003)However, recharge of water in the subsurface formations is a critical method, and depends on many factors, like survey for site selection to determination of the suitability of the source water, quality assessment of water, sedimentation load calculation, contamination control and its interaction with the groundwater, from collection of rainwater, recharge methods to recovery sustainable water quantity, all are important for the success of a recharge model and required reassessment process. Water quality is one of the most important aspect of groundwater recharge to aquifers because if any contamination like pesticides, industrial waste recharged to the aquifer then whole groundwater aquifer will be contaminated. Hydro-chemical criteria such as TDS, chloride concentration, isotopes, pHis also important (Sukhija et al., 1997; Pérez-Paricio et al., 2001).But normally anomaly between the aquifer water and recharged water is extremely high, water quality generally imposes no stop to water harvesting project. Sometimes recharge of portable water improves water quality of the concerned aquifer through dilution (Gale et al., 2002). From qualitative aspect, artificial recharge of aquifers has many advantages as listed by Stuyfzand (2001) which includes removal of of heavy metals suspended fine particles and pathogens. Due to the high quality of the groundwater, geochemical compatibility is required between the native aquifer-water and the harvested water. Above mentioned issues very vast and are beyond the arena of scope of discussion in this article as the present research study focuses on selection of intake well sites for sustainable subsurface water-storage. Selection of methods and suitable location is very important for the success of a Sustainable subsurface water recharge model. The selection of sites and techniques for water storage requires detailed survey of hydrological, hydraulics and geological, conditions which is more critical in hard rocks, where of infiltration rate and recharge rate calculation is very difficult. Water storage through Integrated model(Check Dam, Recharge shaft, collector well gives an emphasis on recovery of recharged water. Several indices such as recovery ratio k (Bear andJacobs, 1965) and regular Recovery Efficiency Sheng et al (2007) had been evaluated for assessment of efficiency of well in terms of percent of recovery of harvested or recharged water for a recharge-withdrawal cycle. Groundwater (GW) recharge is defined as the fraction of total precipitation falling into a drainage basin, which eventually reaches the water table in the saturation zone of an aquifer (Jukić and Jukić, 2004). It is a fundamental component of GW systems (Sanford, 2002), because information on GW recharge rates is often necessary for water resource management, inputs to regional GW models and predictions of climate change impacts (De Silva and Ruston, 2007). Thus, GW recharge is a critical hydrological parameter, which may need to be estimated at a variety of spatial and temporal scales depending on the application. Since GW recharge cannot be measured directly, it is often estimated by using the results of hydrogeologic and geologic investigations, hydro meteorological data, observed discharges or GW level hydrographs (Jukić and Jukić, 2004). However, aquifer scale recharge estimation is often required for water resource assessment and management, whereas local-scale recharge is critical to assessment of GW contamination from point sources. Estimation of GW recharge may be required on temporal scales ranging from days to thousands of years. As aquifers are depleted, recharge estimation have become more vital in determining appropriate levels of GW withdrawal. In addition, recharge estimation is becoming more important for contaminant transport, as aquifer management expands from cleanup of existing contamination to aquifer protection by delineation of areas of high recharge (Scanlon and

Cook, 2002). Thus, understanding GW recharge and its accurate estimation is essential for the successful management of water resources and modeling fluid flow and transport of contaminants within the subsurface (Healy, 2010; Healy and Cook, 2002). Increasing demand for recharge estimation is forcing the researchers to develop approaches for building a more thorough understanding of aquifer recharging process and quantifying recharge rates that reduce uncertainties and increase confidence in recharge estimates (Scanlon and Cook, 2002).Sub surface water is natural resource in most countries, particularly for those in arid and semi-arid areas, due to its relatively low susceptibility to pollution in comparison to surface water (Jamrah et al., 2008). But the existence of groundwater is under great stress of degradation both in quality and quantity. Due to increase in population, lack of awareness among the people, and improper management of this valuable resource lead to depletion of potential and deterioration of the quality. Unfortunately the excessive use and continued mismanagement of water resources to supply ever increasing water demands to profligate users have led to water shortages, increasing pollution of fresh water resources and degraded ecosystems worldwide (Clarke, 1991; Falkenmark and Lundqvist, 1997; de Villiers 2000; Tsakiris, 2004). It presents a clear fact that water is finite and vulnerable resource and it must be used efficiently for present and future generations GIS has emerged as an effective tool for handling spatial data and decision making in several areas including engineering and environmental fields (Stafford, 1991; Good child, 1993).GIS provides a means of representing the real world through integrated layers of constituent spatial information (Corwin, 1996). Most GIS, can easily access overlay and index operations but cannot model groundwater flow and transport processes. However, coupling a GIS to process-based groundwater models can provide an effective tool for data processing, storing, manipulating, visualizing and displaying hydro geological information. Data used in groundwater modelling consists of four categories: (1) the aquifer system stress factor, (2) the aquifer system geometry, (3) the hydro geological parameters, and (4) the main measured variables (Gogu et al., 2001). A well designed GIS database can significantly reduce the time required for data preparation, processing and presentation during the modelling process. Use of groundwater models in hydrogeology mainly includes the simulation of steady or transient sate groundwater flow, advection, hydrodynamic dispersion, and multi-component chemical reaction. In recent years, the use of GIS has grown rapidly in groundwater assessment and management research works. Lasserre et al. (1999) developed a simple GIS linked model for ground water nitrate transport in the IDRISI GIS environment. Visual MODFLOW is also an user friendly software that has ability to generate 3D visualization graphics and import GIS data. Xu et al. (2009) used MODFLOW 2000 (Harbaugh et al. 2000) coupled with GIS to simulate the groundwater dynamics. All of them vary both in space and time, thus adopting a Geographic Information System (GIS) in association with a model is helpful. Coupling GIS technology with a process-based groundwater model may facilitate hydro geological and hydrologic system conceptualization andcharacterization (Hinaman1993; Kolm1996; Gogu et al. 2001), thus also a proper adaptation of the groundwater flow model to the area under study (Brodie1998). In most of groundwater modelling softwares such as FEFLOW, MODFLOW, GMS (Groundwater Modelling System) there is an interface that links vector data through compatible GIS formats i.e. .shp, .lin, .dxf etc. and raster data formats i.e. .tif, .bmp, .img etc. Unlike surface water hydrology, the applications of GIS techniques in groundwater hydrology have received only cursory treatment and are less documented. In the present paper an attempt has been made to highlight coupling of GIS technology with MODFLOW as well as to present a state of the art review on the application of GIS techniques along with MODFLOW package in ground water hydrology However, these criteria's can be calibrated with post-project evaluation of site-efficiency or impact assessment, but are very limited helpful in decision-making for site selection for integrated model of groundwater recharging. The selection criteria for groundwater recharge also concerned with surface runoff variation (Anbazhagan et al., 2005). Practically low surface runoff or lean flow can ensure neither good amount of recharge nor sustainable productivity. Other important are hydraulic parameters such as hydraulic gradients, transmissivity, porosity and (Mukhopadhyay and Fadlelmawala, 2009). Some time in a particular region above mention parameters at different well-location may be similar but recharge, storage quantity , and yield may be found different. Therefore, it is most likely that these parameters will give different response with different recharge percentage. Since subsurface water recharge aims maximum possible availability of harvested water during the lean period, the water budget equation or inflow–outflow ratio of water will be more appropriate criterion. A hypothetical aquifer-stream water interaction system can developed using GMS software (groundwater modeling system) not only to depicted the hydraulic head changes in the groundwater system around well fields but also able to quantify the flow rate due to change in river stage, harvested due to rainfall, and also to determine how the surface water could correlate with groundwater due to with drawl(Roy et.al, 2015). Rationality of design methods of sub-surface well, dependent on image theory and numerical model study, adequately supported with geophysical investigations is combined approach for planning. Integrated -management models related with numerical simulation model with linear optimization can established to evaluate relation between groundwater withdrawals and stream flow depletions.

- Sun et.al.(2011) discuss about reservoir drawdown, the groundwater table in the adjacent aquifer falls down correspondingly. that is useful to calculate the groundwater table variation as a function of time during reservoir drawdown for hydraulic and hydrological purposes. The Boussinesq equation with a moving boundary is applied to analyze the groundwater table variation in the unconfined aquifer during reservoir drawdown. This approach assumes a negligible seepage face. Since the moving boundary condition in the mathematical formulation precludes analytical solutions even for the linear zed Boussinesq equation, it has transformed the Boussinesq equation into an advection-diffusion equation to address the negligible seepage face and the moving boundary condition. Based on the Laplace transformation, they yield analytical.

- Okoro et.al. discuss about a cost effective and efficient alternative in aquifer parameter estimation. They have using Vertical Electrical Sounding (VES) at 40 sites in the study area for the identification of the vertical variation in subsurface lithology and for the characterization of the aquifer system of the study area. The interpretation of the VES data revealed 3 to 5 geoelectric units with the depth to aquifer varying from 7 to 108m and the resistivity of the saturated layer varying between 42.9 and 8829 ohm-m. Using the geoelectric data, aquifer depth, hydraulic head and aquifer resistivity maps they prepare using appropriate software. Aquifer characteristics in the form of transmissivity and hydraulic conductivity calculated from interpreted VES results varied from 0.48 to 19.50m2/day and 0.06 to 3.75m/day respectively. This coupling has proved useful and most valuable in areas of scarce data such as the study area.

- Barthold et.al. (2008) States about a major research challenge in un gauged basins is to quickly assess the dominant hydrological processes of watersheds. In this paper they present a top-down approach from first field reconnaissance to perceptual model development, model conceptualization, evaluation, rejection and eventually, to a more substantial field campaign to build upon the initial modeling. This approach led them from an initial state where very little was known about catchment behavior towards a more complete view of catchment hydrological processes, including the preliminary identification of water sources and an assessment of the effectiveness of our sampling design.

- I W. Welby described a methodology is for estimating a "Ground Water Working Inventory" for areas of the eastern Piedmont Province drawing ground water from crystalline rocks. The approach can improve efficiency in water resource and land-use planning and use of surface and ground water resources. Calculation of the 7-day, 10-year and 7-day, 1-year flows from stream gage data together with application of geologic data and information calculated from 24- hr pumping tests allow the outlining of areas more favorable and less favorable for ground water development. The Ground Water Working Inventory based on the 7-day low-flow calculations allows estimates of available water to be made and permits land-use planners to make rational decisions about residential densities and other water-related issues. Political bodies and planners can use the methodology to guide development according to how large a role ground water is to play in local water supply planning and management. High density development can be discouraged in those areas where ground water is limited and encouraged where commitments for surface water supplies have been made. Additionally, the inventory provides information which strengthens planning for use of ground water as a supplement to surface water supplies.

- Sanz et.al.(2011) discuss about the Mancha oriental System(MOS,7,260km2)which is one of the largest aquifers within Spain, and is encompassed by the Jucar River Basin. Over the past 30years, socio economic development within the region has been largely due to intensive use of groundwater resources for irrigating crops (1,000km2). Groundwater pumping (406 million m3) has provoked a steady drop in groundwater level and reduction MOS discharge to the Jucar River. This study aims to characterize the river aquifer relationship, to determine the influence that ground water abstraction has on the river discharge. This research has advanced a three dimensional large scale numerical groundwater flow model(MOD-FLOW2000) in order to spatially and temporally evaluate, quantify the river aquifer inter actions that are influenced by ground water abstraction in MOS. It is demonstrated that although ground water increased considerably from the early 1980s to 2000, the depletion of water stored in aquifer lower than might be expected. This is mainly due to aquifer recharge from the Jucar River, induced by groundwater abstraction. The area of disconnection between river and water table (i.e. where ground water head is lower than the river bed) is found to have spread 20 km downstream from its position before pumping started.

- Dijksmaet.al.(2010) emphasized about the ground water level in basalt aquifer around the world have been declining for many years. They stated that understanding for water path ways is needed for solution like artificial drainage water supply in in the Palouse Basin,Washington and Idaho, USA, primarily relies on basalt aquifers. This study presents a combination of modeling and field observation to understand the spatial distribution of recharge path ways in over lying pleistosence sediment. Aspatially distributed model was used to quantify potential recharge rates. This model shows clearly that the recharge predominantly occurs trough non- argilic soils, and soils that are not under lain by fine grained sediments, i.e. upper area of the watershed. From their field survey they get 83percent perinial springs.Drilling near spring showed conection of coarse grained layers within the fine grained sediment of Bovill tpo this springs. Such layers,with streame like fractures , act as paleo chanels.water from one of this coarse-grained layers had similar electrical conductivity (200µS cm-1) to water from a downstram perennial spring, also suggesting the existence of a lateral conduit for deep percolation water.

- Oh Kimet.al.(2006) State that the various combining methods that have been commonly used in economic forecasting, and examined their applicability in hydrologic forecasting. The following combining methods were investigated: The simple average, constant coefficient regression, switching regression, sum of squared error, and artificial neural network combining methods. Each method combines ensemble streamflow prediction _ESP_ scenarios of the existing rainfall-runoff model, TANK, those of the new rainfall-runoff model that has been developed using an ensemble neural network for forecasting the monthly inflow to the Daecheong multipurpose dam in Korea. In addition to the combining, the ESP scenarios were adjusted using correction methods, such as optimal linear and artificial neural network correction methods. Among the tested combining methods, sum of squared error _SSE_, a combining method using time-varying weights, performed best with respect to the root mean square error. When SSE was coupled with optimal linear correction _OLC_, denotes/OLC, its bias became sufficiently close to zero. SSE/OLC also considerably improved the probabilistic forecasting accuracy of the existing ESP system.

- Lall(1995) discuss about an optimization model which is developed for selecting between surface water reservoirs and ground-water development. The model may be used to perform a preliminary screening of alternatives for water-supply development, and to identify storage capacities at reservoir sites, as well as aquifer pumping at candidate locations. A hybrid simulation-optimization strategy is used to consider monthly operation of the reservoir and aquifer system using historical or synthetic hydrologic data. A modified sequent peak algorithm is used for reservoir sizing, and a unit response matrix approach is used to model the ground-water subsystem. A target aggregate firm yield from the surface water-ground water system may be specified. Decisions on the failure of firm yield at each reservoir site during a critical period for stream flow, with a corresponding increase in ground-water pumping such that the system firm yield is maintained are considered. Example applications with data from the Jordan River basin in Utah demonstrate the utility of the model relative to a disjunctive analysis of surface water-ground water development.

- Barlow et.al.(2003) states about Conjunctive-management models that couple numerical simulation with linear optimization were developed to evaluate trade-offs between groundwater withdrawals and stream flow depletions for alluvial-valley stream-aquifer systems representative of those of the northeastern United States. A conjunctive-management model developed for a hypothetical stream-aquifer system was used to assess the effect of inters annual hydrologic variability on minimum monthly stream flow requirements. The conjunctive-management model was applied to the Hunt-Annaquatucket Pettaquamscutt stream-aquifer system of central Rhode Island. Results show that it is possible to increase the amount of current withdrawal from the aquifer by as much as 50% by modifying current withdrawal schedules, modifying the number and configuration of wells in the supply-well network, or allowing increased stream flow depletion in the Annaquatucket and Pettaquamscutt rivers. Alternatively, it is possible to reduce current rates of stream flow depletion in the Hunt River by as much as 35%during the summer, but such reductions would result in smaller increases in groundwater withdrawals.

- Watkins and McKinney(1996) describe that Geographic information systems (GISs) offer data management and spatial analysis capabilities can be useful in ground-water modeling. Many regional ground-water models require large, unwieldy datasets, and calibrating them has traditionally been a trial-and-error, hit-or-miss process. GIS provides automatic data collection, systematic model parameter assignment, spatial statistics generation, and the visual display of model results, all of which can improve and facilitate modeling. To utilize these abilities, however, GIS and ground-water models must be able to communicate. Researchers and practitioners have achieved this interface in three ways: (1) linking a GIS to a ground-water model through data-transfer programs; (2) integrating a model with a GIS database; and (3) embedding modeling capabilities within a GIS. This paper emphasizes the usefulness of GIS in ground-water modeling and evaluates these three methods of GIS-model interfacing. Current needs are identified, and suggestions for future work are made.

- Cosgrove and Johnson(2005) discuss that Conjunctive management of surface and ground water requires an understanding of the degree to which surface water resources will be impacted by aquifer stresses at different locations. Response functions quantify the rate of depletion or accretion of a surface-water body relative to a unit stress on an aquifer under simplified, but often realistic, conditions. Response functions for four reaches of the Snake River have been determined for each cell of the eastern Snake River Plain aquifer model grid in southern Idaho. Mapping the response functions for each reach creates a visual image of how effects of aquifer stresses at different locations are distributed among the four hydraulically connected river reaches. The mapped distribution of response functions reflects aquifer properties such as the distribution of aquifer transmissivity. Cluster analysis of the response functions for each cell and each river reach has proven useful for subdividing the aquifer into zones for the conjunctive management of ground water and surface water. The response function variance within each zone is minimized in this procedure. In most situations, zone boundaries defined through cluster analysis will likely be modified to better conform to existing political and administrative units. The selection of the number of zones, and the degree to which boundaries based on response functions are modified, reflect a balance between the exactness provided by the science and administrative convenience and efficiency. The approach of managing an aquifer through response function based zones has gained initial acceptance in Idaho, as evidenced by the citation of these zones in Idaho's draft Water Management Rules. How management zones based on response functions will ultimately be used in Idaho water policy and procedures has not yet been determined.

- **Marcela et.al(2001)** describe that open-check dams are built in mountain streams to control sediment transport during a flood. Sediment passes through them at the lowest discharges, whereas deposition occurs during the highest discharges. Open-check dams are currently designed based mainly on construction experience. Modeling of hydraulics and bed morphology in check dams involves mixed flows (supercritical and subcritical) as well as discontinuities such as hydraulic jumps. In this paper an unsteady coupled numerical mobile-bed model that can tackle rapid varying flows and discontinuities is applied. The numerical technique is based on the classical staggered grids and implicit integration schemes, together with a proper mass and momentum balance. The 1D numerical models successfully verified with experimental data of slit-check dams. The applicability of the model in the design of open-check dams is also illustrated.

- **Michele Catella et.al.(2005)** discuss that Slit-check dams are widely employed in mountain river control. However an analysis of their performance in the field is still lacking. In the present literature a field verification to evaluate the interaction between solid discharge regime and four slit-check dams built in two sub catchments of the Versilia River in Tuscany, Italy is presented. The analysis is based on a relatively detailed field knowledge consisting of hydrological, topographical, and sedimentological data, together with a recent model proposed by Armanini and Larcher.Slit-check dam efficiency is analyzed in terms of deposit formation during major floods and its influence on long-term sediment transportregime. Results suggest that the design efficiency is affected by the high sediment trapping capacity associated with the relatively minor floods. A comparison between the deposit geometry predicted by the theory and the field measurements gathered during a systematic monitoring activity shows good agreement.

2.2. Sate of the art study in regional perspective of groundwater and surface water interaction

- **Singhal1 et.al. (2010)** In their paper attempts to delineate aquifers in the piedmont zone of Himalayan foothill region in Pathri Rao watershed, district Haridwar, Uttarakhand, India by using integrated hydrogeological and geophysical techniques. The geophysical techniques included vertical resistivity soundings, two-dimensional resistivity image profiling and electromagnetic surveys. Nuclear isotope studies have been carried out to estimate groundwater recharge and its relative age. An assessment of groundwater availability and stage of groundwater development has also been made from the available and generated field data. On the basis of the study, it was found that the rate of recharge into the aquifers is of the order of 19% and the stage of groundwater development in the watershed is 164% indicating critical over-exploitation of groundwater. Based on the findings, possibilities of artificial recharge of groundwater have been looked into in the study area for augmentation of groundwater resources by proposing a few check dams at the suitable sites in the upstream areas of the watershed.

- **Sahu and Sikdar (2010)** discuss about the demand for groundwater for drinking, agricultural, and industrial purposes has increased due to rapid increase in population. Therefore, it is imperative to assess the groundwater potential of different areas, especially in a fragile wetland ecosystem to select appropriate sites for developing well fields to minimize adverse environmental impacts of groundwater development. This study considers East Calcutta Wetlands a freshwater peri-urban inland wetland ecosystem located at the lower part of the deltaic alluvial plain of South Bengal Basin and east of Kolkata city. This wetland is well known over the world for its resource recovery systems developed by local people through ages, using wastewater of the city. The subsurface geology is completely blanketed by the Quaternary sediments comprising a succession of silty clay, sand of various grades, and sand mixed with occasional gravels and thin intercalations of silty clay. Groundwater occurs mostly under confined condition except in those places where the top aquitard has been obliterated due to scouring action of past channels. The groundwater in the study area is being over-extracted at the

- **Jha et.al.(2004)** suggested that Water-harvesting structures have the potential to increase the productivity of arable lands by enhancing crop yields and by reducing the risk of crop failure in arid and semi-arid regions, where water shortages are common because of scanty rainfall and its uneven distribution. In semi-arid regions of Rajasthan, India, existing practice of harvesting rainwater is through anicut and earthen embankments. Because of higher costs and higher technical skills involved in the construction of these structures, these structures have not been accepted by the resource-poor local people. Therefore, in their paper, they done detailed design of some low-cost water-harvesting structures using locally available materials and adaptable to the socio-economic conditions of the beneficiaries is discussed. Two types of cost-effective water-harvesting structures, which include dry stone masonry and upstream-wall cement masonry of heights 1, 2, and 2.5 m for catchments of less than 10, 10 to 20, and 20 to 30 ha, respectively are proposed and designed. The analysis of costs involved in constructing dry stone masonry and upstream-wall cement masonry water-harvesting structures, emergency spillway, anicuts and earthen embankments revealed that the earthen embankments have the least cost of construction whereas the anicuts have the highest construction costs for all the selected heights. However, based on the past experiences, earthen embankments are not suitable for the semi-arid regions of Rajasthan. The economic evaluation of the proposed structures indicated that the dry stone masonry structures are very cost-effective for the region with a benefit-cost ratio of 3.5:1 and the net present worth value of Rs. 102978. Although the economic indicators ranked the upstream-wall cement masonry structures lower than the dry stone masonry structures, the former has greater stability and strength compared to the latter. In practice, both the cost effective water harvesting structures (i.e.,dry stone masonry and upstream-wall cement masonry) are gaining wide acceptance and popularity in the region through some nongovernmental organizations, which have adopted the design presented in this paper.

- **Banerjee (2012)** performed pumping tests in the Kangsabati River bed in Purulia District of West Bengal to find the transmissivity and strorativity of the aquifer system. With those results the Radial Collector Well (RCW) is designed. As there is lack of rational method of design, the RCW designed, often are found having a huge difference between anticipated yield and observed yield. Thus, geo-physical study using Vertical Electro Sounding (VES) and Bore logs were performed to select the site for the pumping test and long-term pumping tests were performed. The radial collector well was designed for the site having excellent hydraulic parameters i.e large values of transmissivity and storativity.

- **Hashemi (2013)** in his paper presented an inverse modeling approach to quantify the recharge contribution from both an ephemeral river channel and an introduced artificial recharge system based on floodwater spreading in arid Iran. The study used the MODFLOW-2000 to estimate recharge for both steady and unsteady-state conditions. The model was calibrated and verified based on the observed hydraulic head in observation wells and model precision, uncertainty, and model sensitivity were analyzed in all modeling steps. Uncertainty analysis revealed that the river channel recharge estimation represents relatively more uncertainty in comparison to the artificial recharge zones.

- **Ye and Du (2011)** proposed that in the groundwater Artificial Recharge (GAR) process, complex physical, chemical and water–rock reactions will take place in the recharge water, native groundwater and the aquifer matrix. It is difficult to forecast these complex reactions by traditional research approaches. Groundwater geochemical modelling of GAR supposes chemical equilibrium based on injection water, native groundwater and aquifer matrix; the modelled results may overestimate the environmental effects of water–rock interactions. In fact, because of the slow reaction rate, the storage duration of mixed water and the dissolution rate, the dissolved quantity of minerals and other factors, some chemical reaction processes may not actually be completed. If kinetic chemical reactions could be considered, the accuracy of the model would undoubtedly improve, but this would also cause a great increase in complexity and uncertainty of the model, as many kinetic reactive processes are not very clear and it is quite difficult to obtain kinetic parameters.

- **Daher et al. (2011)** described Managed Aquifer Recharge (MAR) is an emerging sustainable technique that has already generated successful results and is expected to solve many water resource problems, especially in semi-arid and arid zones although it may be difficult to quantify the risks of a MAR system not meeting the project expectations, as the performance of a MAR system depends considerably on the hydrogeologic conditions, sound characterization before construction followed by testing through the deployment of a pilot MAR system to validate the parameters of the study is essential before deployment of a MAR system at full operational scale. The lack of a rational approach to assess the recharge ability of a given karst aquifer and identify the best locations for conducting MAR operations has also certainly played an important role. Natural recharge is, to an increasing extent, impaired by various types of land use. MAR is thus no longer considered as a mere water storage technique but rather as an integrated water resources management (IWRM) tool to be adopted as a balancing process for safe and sustainable water resource development, allocation and monitoring.

- **Kumar et al.(1995)** described how the Hard-rock zones can be used efficiently for recharging if proper planning of integration of groundwater and surface water use is done. The difference in this case is the methodology. In hard rock jones along with direct recharge there should be some process to impound the surface water which will further recharge the groundwater storage. The limited Indian research on rainwater harvesting (RWH)/artificial recharge so far had focused on the engineering performance of individual structures. One of the reasons for little or lack of empirical research on the hydrological and economic aspects of water harvesting systems is the lack of ability to generate accurate scientific data on various parameters, mostly hydraulic, hydrological and meteorological, governing the performance and impact of water harvesting. The problem mainly stems from the fact that these systems are very micro in nature, thereby making it difficult to obtain data on the variables from conventional sources. The analysis of water harvesting systems also misses the influence of the 'scale factor'.

- **Badiger et al. (2002)** in their paper described how farming communities in Rajasthan have found groundwater exploitation as a security against drought in areas where irrigation with surface water resources has been deficient or non-existent. Due to frequent dry spells and unreliable surface water supply, farmers in Rajasthan have discovered several means of maximizing subsurface storage and its availability at times of need. In order to store water, they raised their field boundaries to heights equalizing 1-2 m to tap the rainfall runoff originating with their fields. The technology directly influences water availability in the areas immediately surrounding the Nala. This stored water infiltrates into the soil mantle and gradually percolates down to recharge the groundwater, which is extracted using open wells and tube wells.Paal systems are attractive low cost water harvesting Structures well suited to semi-arid regions. The natural recharge by rainfall during 2000-2001 is roughly 6.5 percent of annual rainfall whereas the additional recharge induced by nalas is estimated to vary between 3 to 8 percent of annual rainfall, depending on assumptions made with regard to specific yield of encountered aquifer.

- **Jasrotia et al. (2009)** described the importance of water balance equation for implementation of Rain Water Harvesting System (RWHS) using Remote Sensing and GIS techniques. After collection of different satellite imagery, land use map, soli pattern map, slope area map, drainage map all the maps were crossed. With the help of these maps the water balance was calculated using Thornth waite and Mather's (TM) Model. The suitable site for rainwater harvesting was prepared by integrating different thematic maps in the GIS environment. The guideline accordingly to Integrated Mission for Sustainable Development (IMSD) was applied over moderate runoff potential zones which are suitable for rainwater harvesting sites and desirable rainwater harvesting structures were delineated.

- **Rao et al.** (**2003**) showed that geological and geo-hydrological studies followed by geo-physical studies made easy to locate feasible groundwater pockets like valley fills, weathered zones and fractured zones, which in terms also made possible to demarcate soil pockets as well as aquifer thickness. These pockets are identified by the apparent values of resistivity. Lower values of apparent resistivity imply the aquifer is saturated or there are some weathered pockets. Once the zones are demarcated the structures may be constructed as per morphological condition.

- **Machiwal et al.** (**2003**) said water harvesting structures have the potential to increase the water table and thus the crop yields. But the problem in some semi-arid zones and arid zones of India is the availability of funding and skilled people in the village areas. So some alternative low-cost recharge structures were installed, which were invented by the local farmers. The structures were Dry-Stone masonry (DSM) and Up-stream wall Cement Masonry (UCM). These structures were not scientifically correct. After some modification these low-cost structures produced good results.

- **Busnelli et al. (2001)** proposed check dams as one of the structural measures for sediment control. They play an important role in the management and development of a river basin. Modeling of hydraulics and bed morphology in check dams involves mixed flows (supercritical and subcritical) as well as discontinuities such as hydraulic jumps. The experiment was done on an unsteady coupled numerical mobile-bed model which can take rapidly varied flows. The 1-D numerical model is successfully verified with experimental data of slit-check dams. The model is quite applicable for design of check-dams.

- **Batte et al. (2010)** described that the most common in-situ tests for calculation of real water supply and the indirect estimation of hydraulic conductivity in a borehole are the pumping tests performed on wells which involve measurement of the rise and fall of water level with respect to time. The water-level fluctuations with time are then interpreted to arrive at aquifer parameters. They showed in this paper how aquifer parameters vary with resistivity. The application of surface resistivity provide valuable information for flow modeling and recharge of groundwater, and to find suitable sites for groundwater exploration before generalizing this approach, application of resistivity and aerial photographic techniques for groundwater prospecting must be taken into consideration to understand the groundwater potential of the area to be studied in terms of lithologic units and fracture traces. The good agreement between aquifer hydraulic conductivities obtained from the resistivity soundings interpretation and those deduced from pumping test analysis emphasizes the potential of the method.

- **Scanlon et al. (2002)** used some techniques for estimating recharge which were subdivided into various types, on the basis of the three hydrologic sources, or zones, from which the data are obtained, namely surface water, unsaturated zone, and saturated zone. This subdivision of techniques is somewhat arbitrary and is probably not ideal. The different zones provide recharge estimates over varying space and time scales. Within each zone, techniques are generally classified into physical, tracer, or numerical modeling approaches. The range of recharge rates for different techniques is based on evaluation of the literature and general evaluation of uncertainties and should be considered only approximate.

- **Noori and Ismaeel (2010)** used a finite element method through a computer program, named SEEP2D to determine the free surface seepage line, the quantity of seepage through the dam, the pore water pressure distribution, the total head measurements and the effect of anisotropy of the core materials of Duhok zoned earth dam. The finite element program (SEEP2D) can be used to analyze the homogeneous and non-homogeneous (zoned) earth dams. The Kx/Ky ratio has a significant effect on the location of the free surface line. The free surface is higher by increasing the ratio and it is lower by decreasing the ratio. The quantity of seepage increases as the Kx/Ky ratio increases, therefore, the loss of water through the dam increases.

- **Ahilan et al. (2010)** identified aquifer zones by the Geophysical Electrical Resistivity Survey method in part of Mangalore block, Cuddalore District, Tamil Nadu, South India. As Electrical Resistivity Survey method is useful to delineate the subsurface formations, weathered zone, fracture pattern, etc. So, he made an attempt to identify the subsurface lithology and aquifer zones by geo-electrical resistivity method. The study area consist of charnockite and charnockitic gneiss rocks. He conducted Resistivity profiling in a grid pattern. The total study area of 100 Square km. had been divided into square grids of 2.0 Sq.km and 6 resistivity profiles were conducted with station interval of 2 Km by Schlumberger configuration using Microprocessor based signal stacking digital resistivity meter SSR-MP-AT-S of IGIS, Hyderabad, resistivity profiling with *AB/2 10*, 20, 30, 40 and 50 m have been carried out. VES had been conducted at 36 locations with *AB/2* 50 m, which had been qualitatively and quantitatively interpreted using software packages. Pseudo-sections had been generated by IPI 2 WIN ver.3.1 and is resistivity maps were prepared. Geo-electrical parameter and sections had been prepared based on quantitative interpretation. Considering the hydro-meteorological and hydrogeological conditions the VES interpretation was done. From the interpretation result, he marked VES no. 21 (Alambadi) was a prospective zone for groundwater exploration target.

- **Jha et al. (2008)** carried out a study to assess groundwater condition in the Salboni Block of West Midnapore district, West Bengal, India using surface resistivity method during November 2004 to January

2005. Vertical electrical sounding (VES) surveys were carried out at 38 sites using the Schlumberger array by the Signal Stacking Resistivity Meter (Model: SSR-MP-AT) manufactured by the Integrated Geo Instruments and Services Pvt. Ltd., Hyderabad, Andhra Pradesh, India with maximum current electrode spacing used in the study was about 400 m. The apparent resistivity-depth datasets (henceforth called 'VES data') thus obtained was interpreted by the genetic algorithm (GA) optimization technique. He developed GA-based stand-alone computer program for optimizing subsurface layer parameters (true resistivity and thickness) from the VES data. The optimal layer parameters were then correlated with the available well logs to identify aquifer and confining layers. Moreover, he created groundwater potential map by integrating the thematic layers of aquifer resistivity and thickness in a GIS environment. He also explored the spatial variation of layer resistivity at a particular depth, resistivity contour maps of the study area for different depths using ArcView software. The GA technique yielded layer parameters with reasonably low values of root mean square error (0.36–9.75 Ω m) for most VES datasets. He found that shallow aquifers existed at depths ranging from 4 to 19 m and relatively deep aquifers from 24 to 60 m below the ground surface.

- **Bouwer (2010)** said that Factors affecting the availability of water resources include increasing demands for water while water resources remain finite; potential climate changes; lack of availability of good dam sites for surface storage; and increasing difficulty of building dams because of social, environmental, costs, and other objections. Also, dams are not effective for long-term storage of water because of evaporation losses. Artificial recharge, resulting in conjunctive use of surface water and groundwater and long-term underground storage or water banking, is preferred where possible. Artificial recharge also plays an important role inwater reuse, because it gives quality benefits (soil-aquifer treatment) and storage opportunities to absorb seasonal differences between supply and demand for reclaimed sewage effluent.

Where sewage effluent is used for potable purposes, recharge and recovery breaks the objectionable toilet-to-tap connection of water reuse and enables blending with natural groundwater.

- **Singh et al. (2009)** cited that water harvesting structures are extremely important to conserve precious natural resources like, soil and water, which is depleting day by day at alarming rate. The good cultivable areas are being converted into waste and degraded lands due to over exploitation of land and water resources. In this paper they managed to plan the sites for water harvesting structures like check dam and percolation tank using the emerging techniques of Remote Sensing (RS) and Geographic Informatics System (GIS). They did not select the sites for nala bunding and farm ponds in that area due to steep slope, less soil thickness and high runoff velocity. The water balance study for the study area was calculated using Thronthwaite- Mather's Model (TM).

CHAPTER 3
DESCRIPTION OF THE STUDY AREA

3. Description of the study area

3.0. General Description

Jaina Bhagavati-Sutra of circa 5th century A.D. mentionaed that Purulia was one of the 16 Mahajanapadas and was a part of the country known as Vajra-bhumi in ancient times. However, little is known about Purulia before the East-India Company obtained the 'Diwani' of Bengal, Bihar, Orissa in 1765. By Regulation XVIIII of 1805, a Jungle Mahals district composed of 23 parganas and mahals including the present Purulia (known as 'Purulia' those days) was formed. By Regulation XIII of 1833 the Jungle Mahals district was broken up and a new district called Manbhum was constituted with headquarters at Manbazar. The district was very large in size and included parts of Bankura, Burdwan of present West Bengal and Dhanbad, Dhalbhum, Saraikela and Kharswan of present states of Jharkhand and Orissa. In 1838 the district headquarters was transferred to Purulia of today. Since the formation of the district it was withdrawn from regular administration and placed under an officer called Principal Assistant to the agent to the Governor-General for South-Western Frontier. Finally in 1956 Manbhum district was partitioned between Bihar and West Bengal under the States Reorganization Act and the Bihar and West Bengal (Transfer of Territories) Act 1956 and the present district Purulia was born on 1st November, 1956. Purulia is the westernmost district of West Bengal with all-India significance because of its tropical location, its shape as well as function like a funnel. It funnels not only the tropical monsoon current from the Bay to the subtropical parts of north-west India, but also acts as a gateway between the developed industrial belts of West Bengal and the hinterlands in Orissa, Jharkhand, Madhya Pradesh and Uttarpradesh. For its convenient location, this place has acquired an important place in the tourist map in India.

3.1. Social profile of study area

Purulia district is one of the most populated districts of the Adivasi (Scheduled Tribe) and backward class community in the Western region of West Bengal. As per 2001census total population of the district is 2535516, out of which 89.93 % are residing in rural areas and 10.07% are in urban areas. About 51.18 % of the populations are males and 48.82% are female. The percentage of Scheduled Caste and Scheduled Tribes are 18.29% and 18.27%. Total no of BPL families in rural areas of this district are 197381 (43.65 %). Out of which SC families are 40645 (20.5%) and ST families are 47666 (24.15 %). Total no. of BPL families in Purulia and Jhalda Municipality are 2573 (11.31 %) and 571(15.98 %) respectively . These tribes are concentrated in different parts of the district. A long-term bottom-up approach is visualized for the development of cost effective technology for recharging in over exploited ground water resources at Ghutlia. It is expected to generate the conditions and provide lessons for follow-up in the region in other villages. In order to have a holistic documentation of the project implementation process at Ghutlia, it was decided to conduct a baseline survey for evaluation of project benefits to the inhabitants of Ghutlia in the long-run. For the same purpose, semi-structured questionnaires have been administered among all the villagers along with Focused Group Discussions and personal interviews with key informants. The data analysis of this survey delineates the social and economic background of Ghutlia, and provides a careful examination of strengths and vulnerabilities associated with this particular intervention. The following sections describe the major findings of the survey in relevant categories. A striking feature of the OBC community in Ghutlia hamlet is the relatively high status of the women. The demographic features of Ghutlia show some interesting trends. For instance, the number of children in the age group of 0 to 12 years is higher in comparison to adults in Ghutlia. Children constitute 29 per cent of the total population of Ghutlia followed by the working population in the age groups 31-45 (22%) and 46-60 (12%) Figure 3.1.

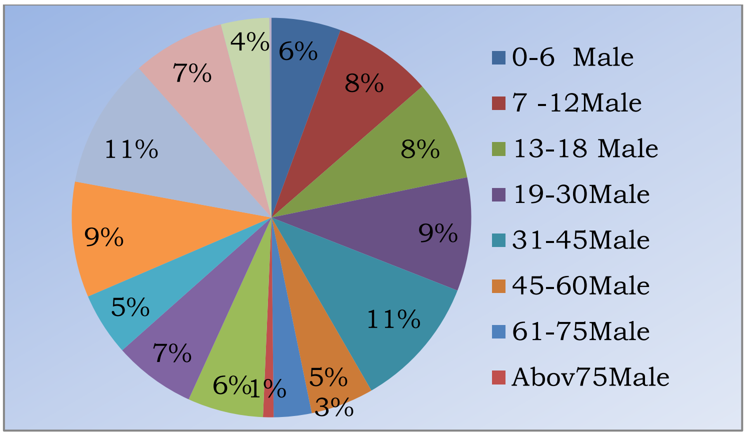


Figure 3.1: Age wise distribution of population in study area

The status of education in the village is still below the satisfactory level. Only 21 percent of the total population is educated, of which secondary levels consist almost 19 per cent higher secondary consist 3 %, graduation consist 2%. In absolute numbers 140 persons out of a total of 448 persons are completing secondary education depicted in below Figure 3.2.

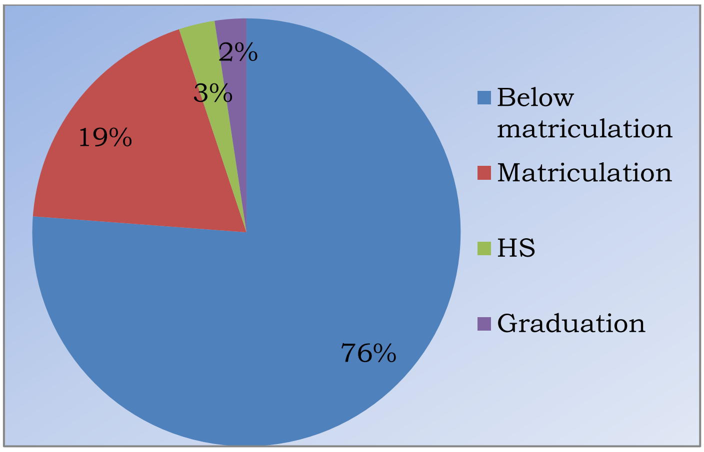


Figure 3.2 : Indicates education level in study area

Ghutlia community suffers from a high degree of economic deprivation. Single season subsistence agriculture is the norm, with the second season being taken up by migration to brick-kilns and other sorts of daily wage work. This is despite the fact that the region receives plentiful rainfall in the monsoon. Unfortunately, the absence of rainwater harvesting methods means that rainwater goes waste and water sources dry up by January

February. Lohar are landless community and depend upon laboring in the farms of land owner tribal as well as those of non-tribals in nearby villages. Most of the households in Ghutlia do not own any piece of fertile cultivable land. In other words, 15 per cent of households in the village are landless which indicates the existence of high level of poverty in the village (See Figure 3.3 and Figure 3.4). The households having land are basically owner-cultivators most being marginal peasants. The marginal farmers constitute 39per cent of the farming community having cultivable land less than 3 bigha per household. It can be said that the distribution of land in is well distributed but the possession of less than 3 bigha does not assure diversified agriculture in the village to serve as a potential source of livelihood. In the absence of alternative employment opportunities, especially non- farm activities, it shows the existence of structural vulnerabilities in the social and economic systems.

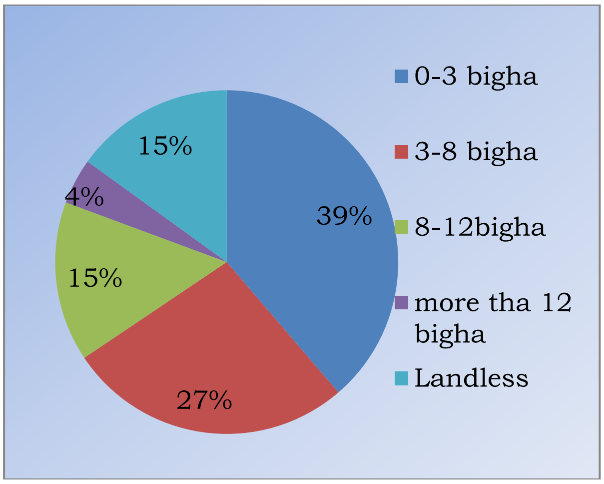


Figure 3.3 : Distribution of cultivable Land

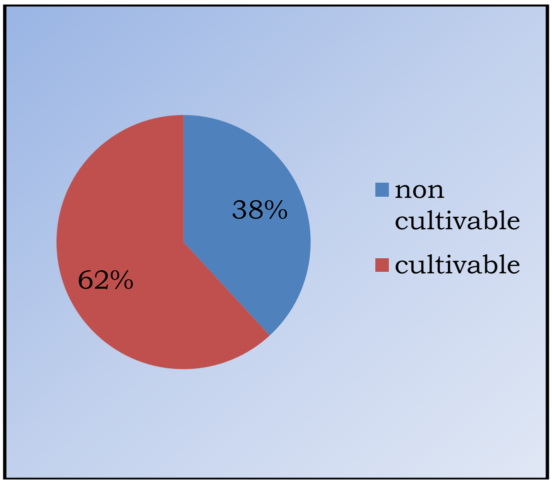


Figure 3.4 : Distribution of land in study area

Ghutlia is characterized by the greater dependence of inhabitants on agriculture for their livelihood. The total income of most of the households varies around Rs.6000 per annum with a few families having higher income in the range of Rs.12000 to Rs.24000 per annum. Another few family have income more than Rs 100000 per annum despite marginal land holdings cultivation has been the primary source of livelihood for the people in Ghutlia. Even though agriculture is rain-fed and subsistence in nature, it provides the staple diet to the villagers. Mostly, people cultivate paddy and branzyl as main crops. Besides cultivation, daily labour is the only source of income for the villagers. Only one person is found employed in government service. The annual income of the households in Ghutlia from different sources is given in Figure 3.5. In the village 59% of women are doing only house hold work 0.39% is also engaged with self help group.

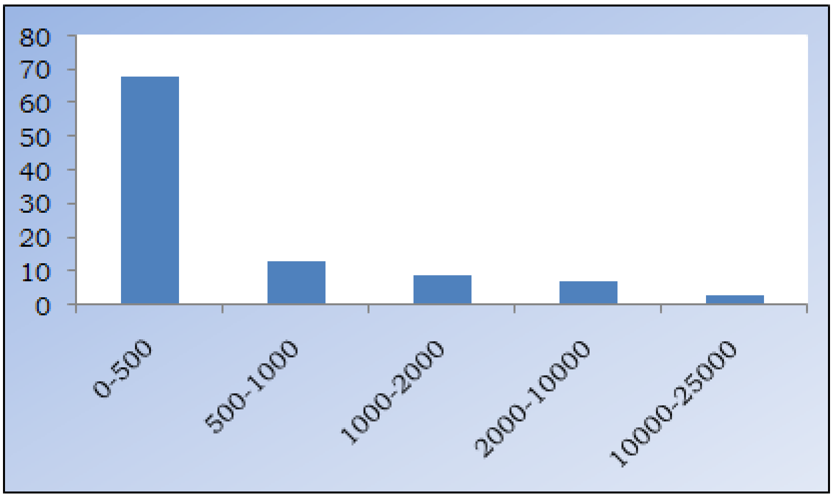


Figure 3.5 : Distribution of households in different income Groups

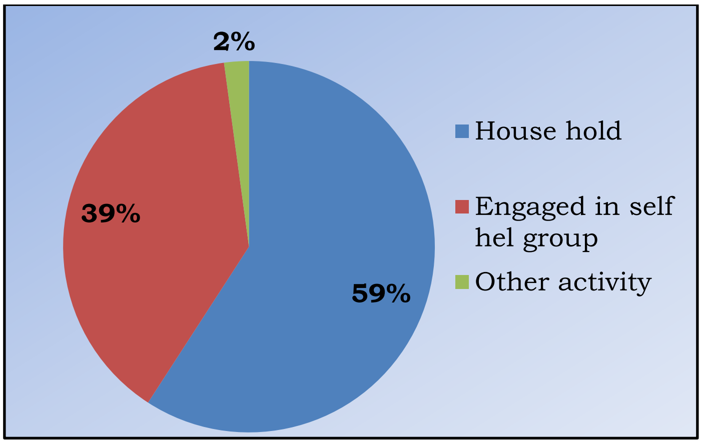


Figure 3.6 : Women activity in study area

Health Condition

Health condition and attainment in the community is low. This is linked to the poverty levels in the area. The diet typically comprises rice, yellow dal and vegetables like bottle gourd or brinjal. There is a deficiency of proteins and carbohydrates, which is manifested in the form of Protein Energy

Malnutrition (PEM). Thus, the incidence of marasmus and kwashiorkor is common. Poor health is also reflected in low educational attainment, something discernible among the older generation in the community. Water scarcity also leads to skin diseases, with scabies as one of the most common problems, apart from abdominal pains, and respiratory-track infections. The village population places considerable faith in the traditional healer, the ojha. The Ojha's knowledge is not to be belittled but it is to be accepted that he is unlikely to have superhuman curative powers, or that illnesses are attributable to spirits that have crept into one's body. The trend of seeking Ojha'shelp has been declining but is still prevalent in Ghutlia people prefer to go to doctors in nearby village and towns (Kashipur). A third reason for poor health rests in the relative inaccessibility of health services. Private medical services are generally clustered around towns. Primary Health Centers (PHCs) and their sub-centers may be better accessed but only relatively. These tend to be poorly equipped and bribes are charged for injections. Inaccessibility implies travel, which adds on to the cost of health care. They are mainly depended on Kashipur block Hospital. Due to using without purified water 51% of house hold affecting from water born decease like jaundice, diarrhea, etc. shown in Figure 3.18.

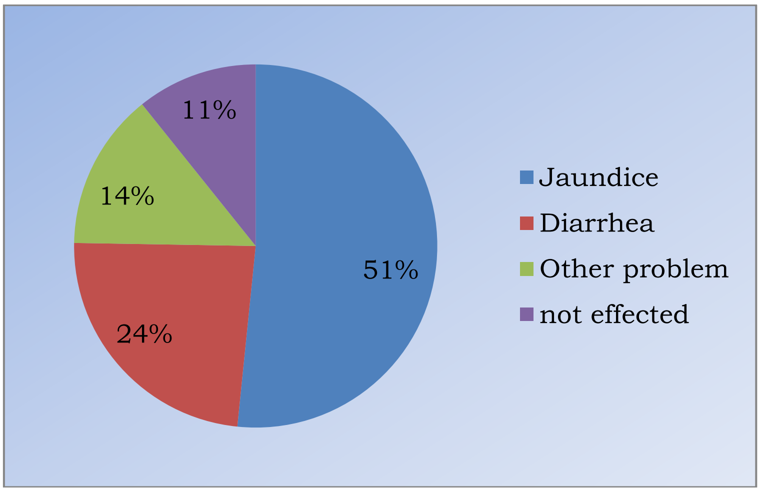


Figure 3.7 : Effects of waterborne disease in study area

3.2. River profile of study area

3.1.1. Kangsabati Rive and it’s catchment Area

The Kangsabati river system (also variously known as the Kasai and Cossye) rises from the Chhotanagpur Plateau in the State of Jharkhand, India and passes through the district of Purulia, Bankura and Paschim Medinipur in West Bengal before draining in the Bay of Bengal. The Kangsabati project is a major irrigation scheme located in the western part of state of West Bengal, India. The study area is located between latitudes 22°15′ and 22°30′N and longitudes 87°15′ and 87°22′E. The river Kangsabati and its main tributaries originate from the Chhotanagpur hills.

Land Use Pattern and Contour of the of the Kangsabati catchment Area

The land use map has been prepared based on satellite imageries.

Figure 3.8 depicts the classification of land use pattern of Kangsabati catchment and the classification scale is highlighted by different colors. This map has been validated based on the collected GPS data. All the blocks under study are shown.

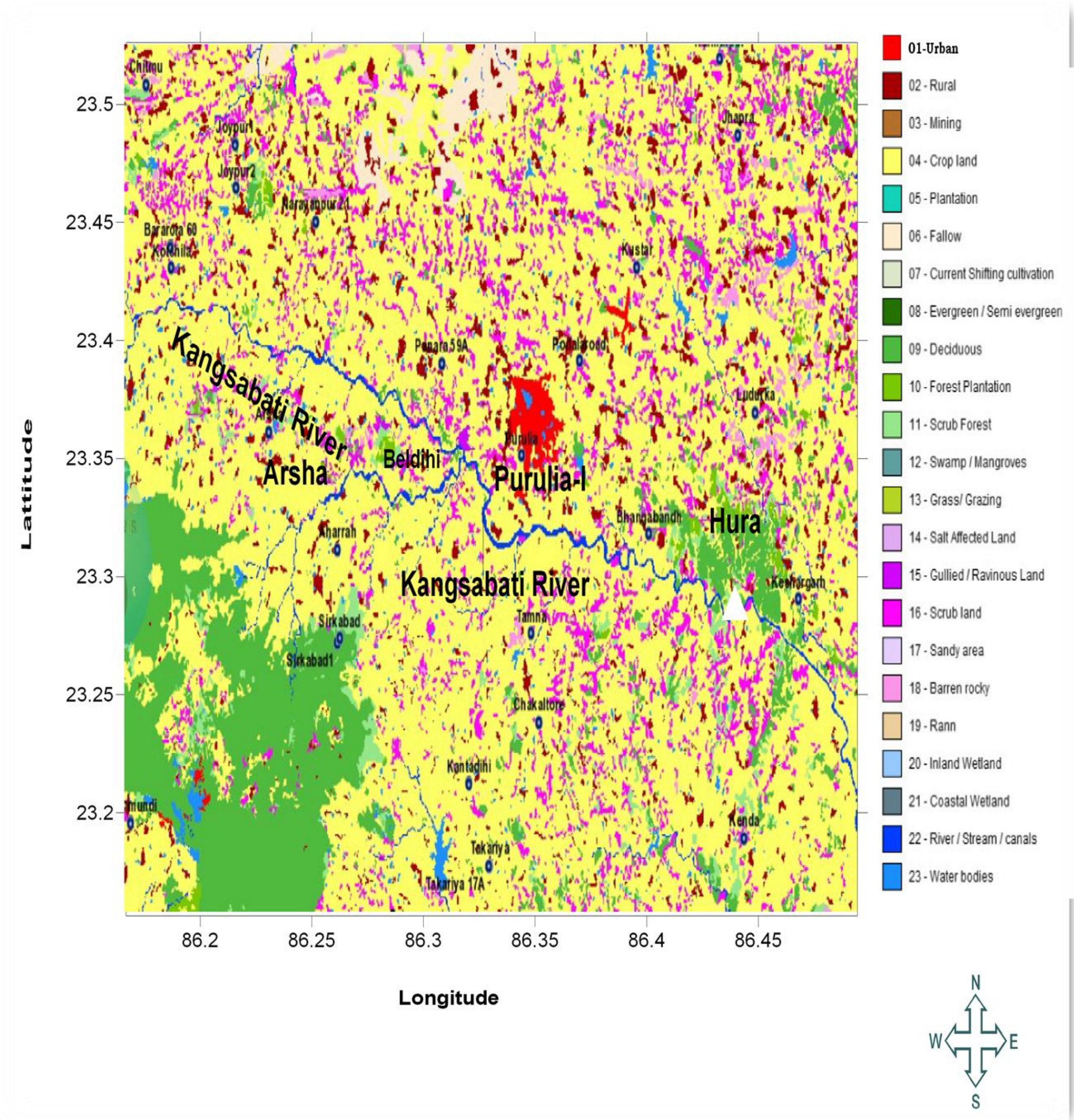


Figure 3.8 : Land Use Map of the Study Area

The contour map has been prepared by SURFER and GIS software based on Satellite imageries as well as GPS data. The GPS locations have been demarcated by red and green colour as shown in Figure 3.9. From the confluence point of Bandu, a tributary of Kangsabati river further upstream of both Bandu and Kangsabati river have been brought under study.

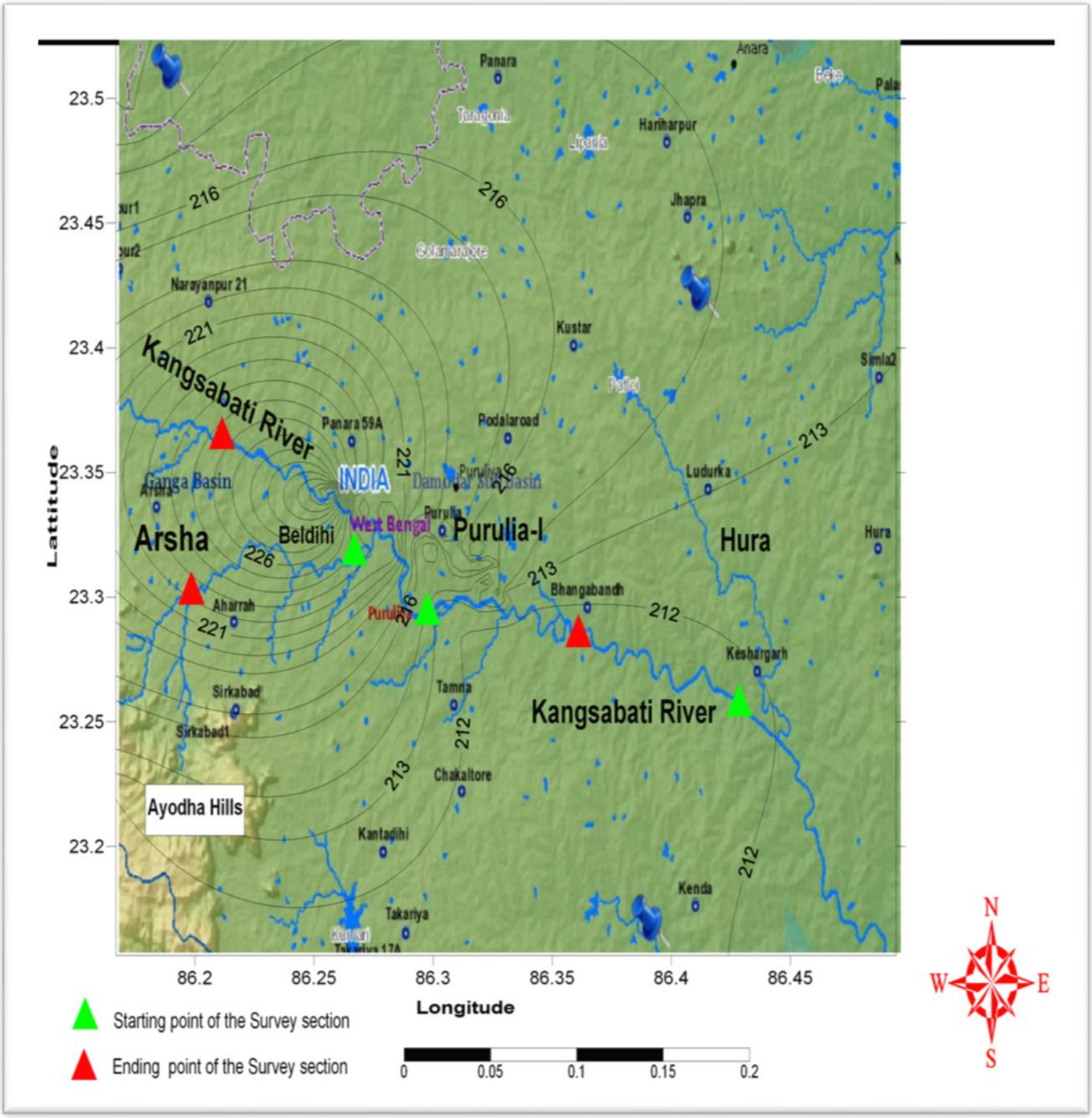


Figure 3.9 : Contour Map of the Study Area

3.1.2. Description of Damodar River catchment area

The Damodar river basin, which lies between the latitudes 23.5 N and 24.32 N and longitudes 85.516 E and 87.35 E, was selected due to its widely varied land type, variable elevations and ground cover. The river originates from the Palamu Hills of Chhota Nagpur at an elevation of about 610 m above mean sea level. The river flows in a south easterly direction and enters the deltaic plains below Raniganj in Burdwan district of West Bengal, India. Near Burdwan the river abruptly changes its course to southerly direction and joins the Hugli river about 48 km below Kolkata. The slope of the river bed during the first 241 km is about 1.89 m per km and in the next 161 km it is about 0.57 m per km which is a 3.31 times reduction than the initial slope, while the subsequent 145 km has a slope of 0.19 m per km. Details of soil condition is given below. The land use of the selected catchment was a mixture of arable, deciduous Forest (F) and non-agricultural land. The adjacent lands of Tilaiya reservoir were dominated by deciduous forest but the major portion of land in Konar and Panchet was used for cultivation.

Soil Condition, Land use and Contour of Damodar catchment area

The basin has six types of soils. At present alluvium soil has been brought under the plough so that apart from water intensive plants in the marshes, no trace of the original vegetation remains. Open Sal forest thrive mainly on Lateritic soil, and dense Sal forests on red and yellow loams, in the upper valley region. The types of soil are highlighted by different colours mentioned in Figure 3.10. The contour map has been prepared by SURFER and GIS software based on Satellite imageries as well as GPS data. The GPS locations have been demarcated by as shown in Figure 3.11.

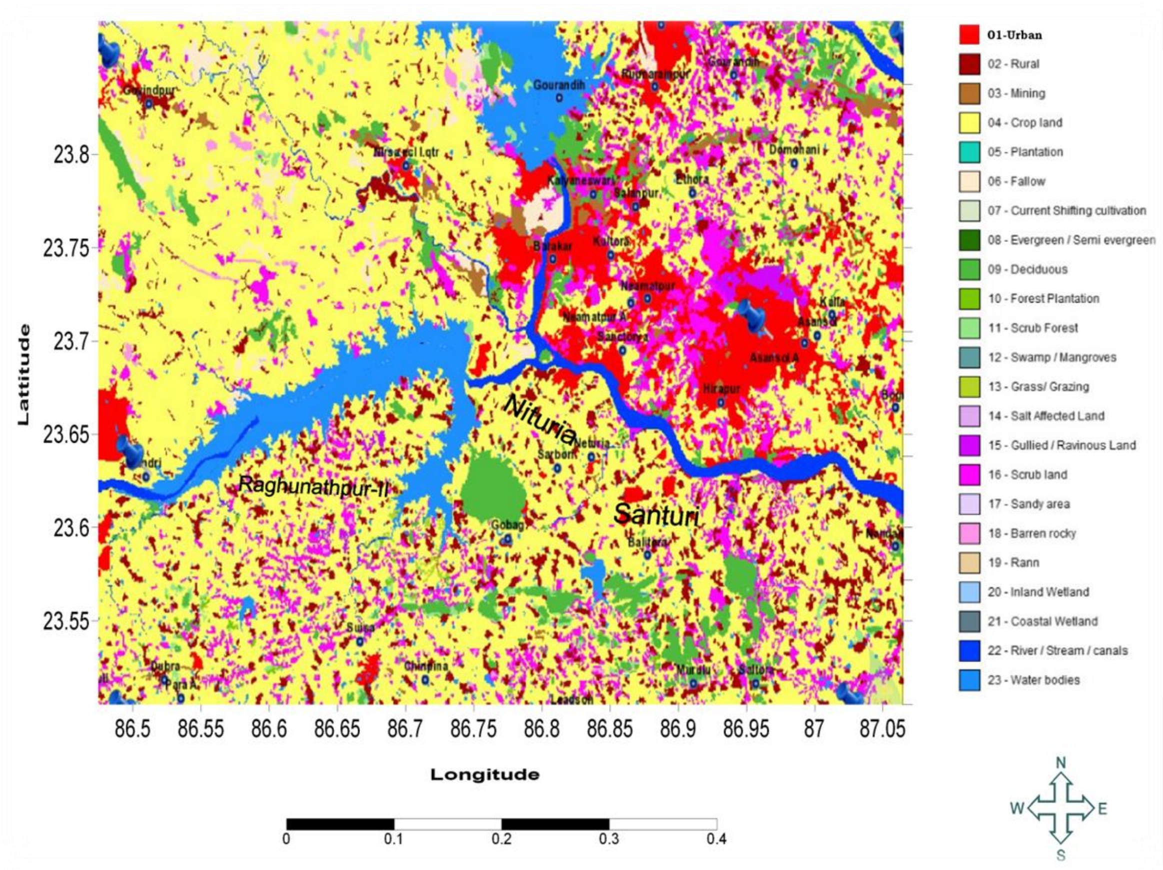


Figure 3.10 Land Use Map of the Study area of Damodar river

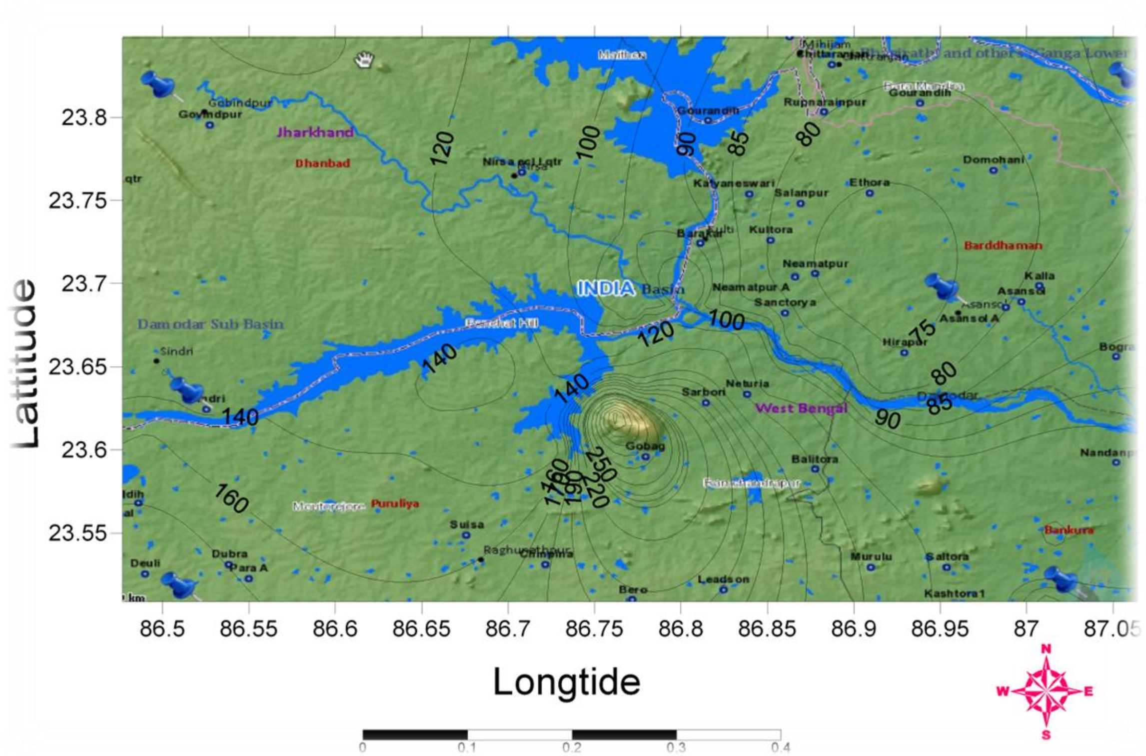


Figure 3.11 : Contour Map of the Study area of Damodar river

3.1.3. Darokeswar and Futiary Catchment Area

Pedology

The study area comprises of sandy to loamy soil with red coloration due to high iron content. However, the soils have good porosity and permeability hows in Figure 3.12 below.

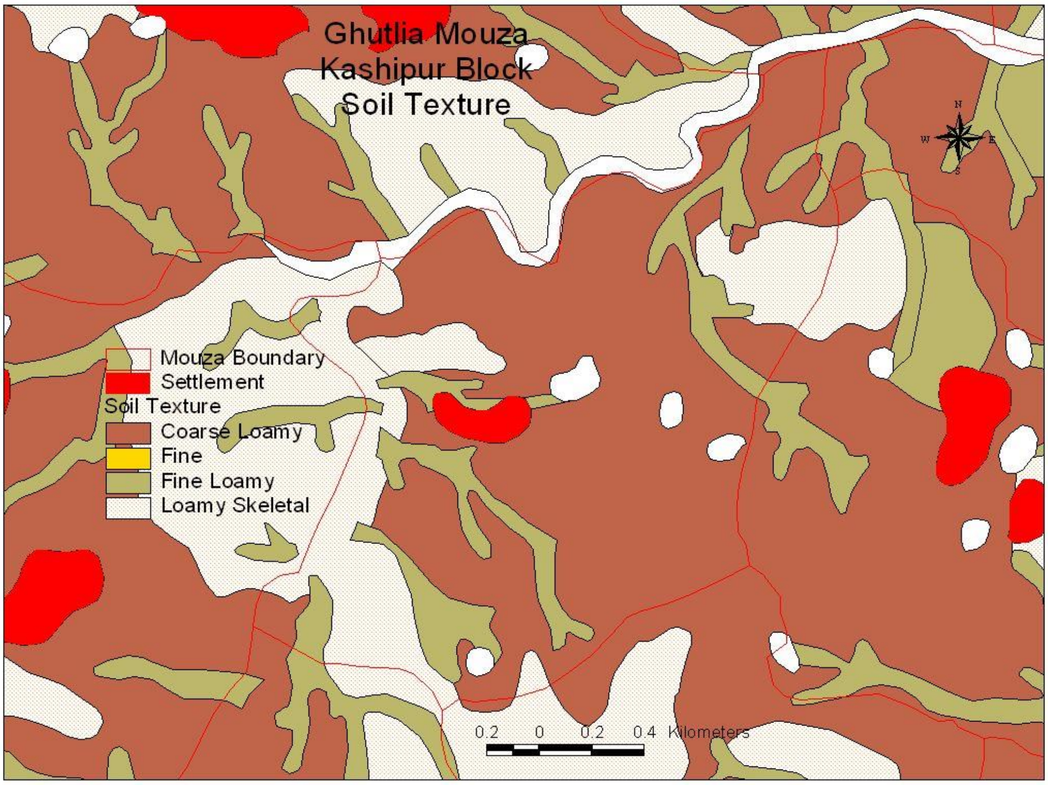


Figure 3.12 : Soil Map of The study Area

Geomorphology

Geo-morphologically of the study area represents moderately undulating terrain which has a general slope towards East-Northeast direction Figure 3.13 Figure 3.13. The area is drained by the river Dwarekeswar and its tributary locally known as Futiary jhor (stream). Both the rivers are perennial, although the flow in the rivers declines in summer.

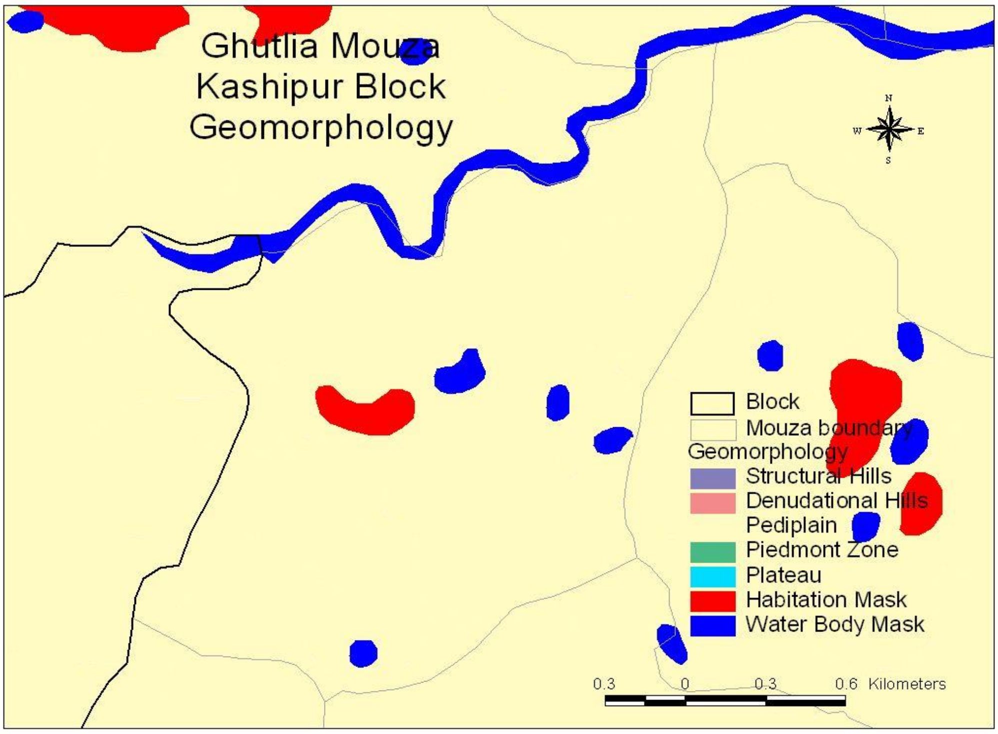


Figure 3.13 : Geomorphology of the study area

Drainage pattern

In Darokeswar and Futiary catchment drainage pattern is funnel/palmate shaped sub watershed with dendratic drainage pattern Figure 3.14. In the Village drainage is imperfect to moderately well.

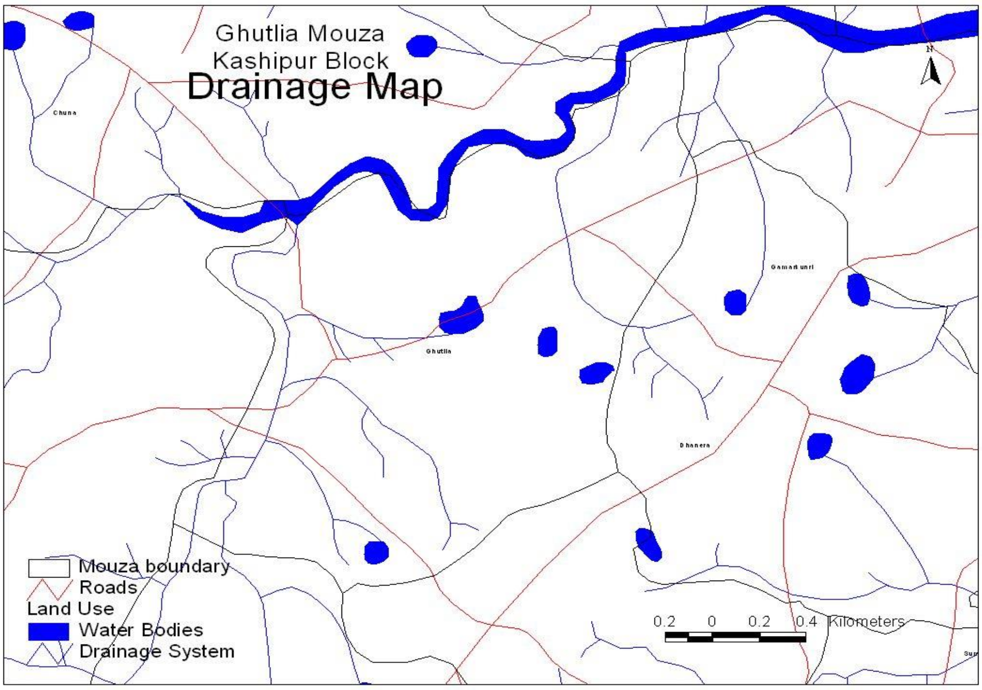


Figure 3.14 : Geomorphology of the study area

Geology and Hydrogeology

Geologically the study area is underlain by granite gneiss and quartzite. The depth of weathering which is an important factor to locate the areas for good groundwater recharge, is more in case of the former while the latter is less weathered. The northern part of the area falling along the course of Dwarekeswar river is underlain by quartzite while granitic rocks are available in the sub-surface along the catchment of Futiary stream Figure 3.15. Depth to water level as observed from the wells varies from 7.5 to 11.0 m below ground level during pre-monsoon 4.8-7.5 m below ground level during post-monsoon. Although the weathered zone is having good proportion of sandy residue with relatively high porosity and permeability, moderate to high slope of the country facilitates quick recession of groundwater during lean period. Hence, the wells located in higher topography become dry during summer. As the rivers are mostly gaining in nature, if groundwater recharge activities are increased in the upper catchment along the river, the same would regularly enrich the base flow of the rivers consequently the rivers become more perennial. All these scientific observations and knowledge are utilized during framing the model of conservation and recharge in the area.

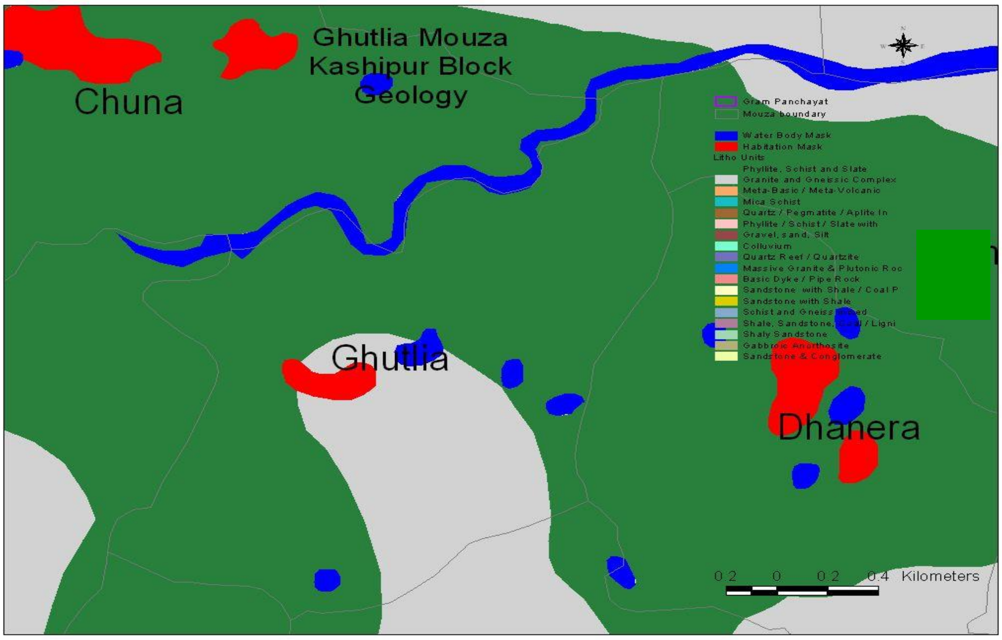


Figure 3.15 : Geology of the study area

Land-use pattern

Most of the village in Simla-Dhanera GP do not own any piece of fertile cultivable land. In this GP 56% of land is cultivable,6% is Forest Land ,20% irrigable land,19% use for other uses and 17% is cultivable waste land. Distribution of land is showed in Figure 3.16 and Figure 3.17.

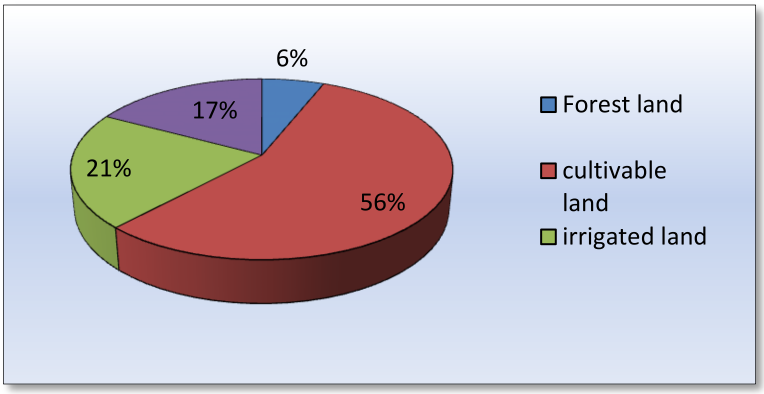


Figure 3.16 : Land distribution of Ghutlia Village.

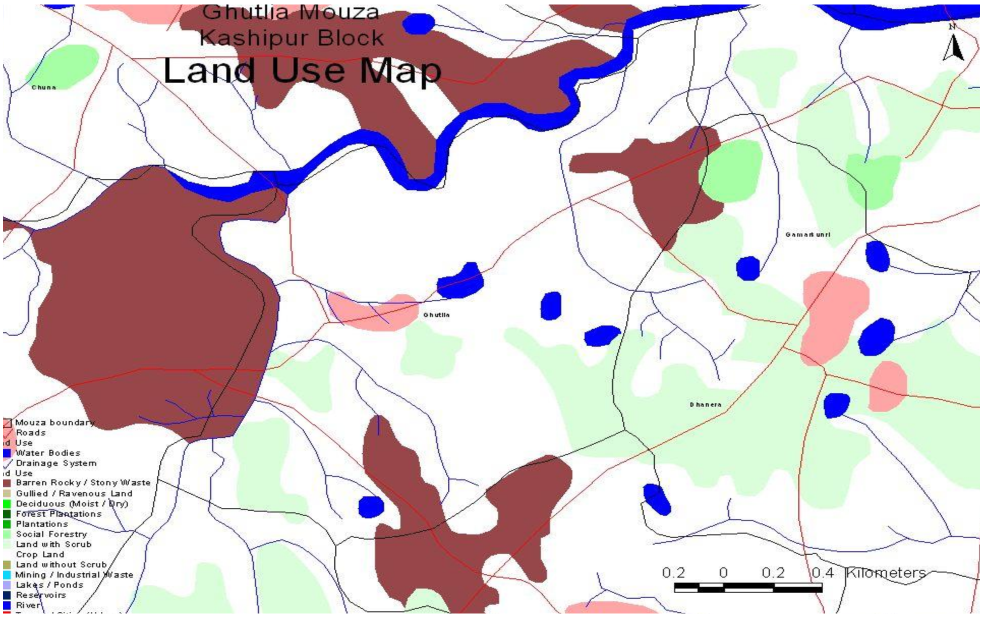


Figure 3.17 : Land Use pattern Map of the study area

3.1.4. Drinking Water

The water sources in the village are not perennial except for one tube well and 3 wells for drinking water. Basically villagers dependent on two river Darokeshawar and Futiary for drinking purpose which is not well treated water. Half of the villagers taking water from Futiary and half from Darokeswar. The daily water requirements in different seasons are given inFigure 3.18. As per the opinion of 93 households, less than 5 Harri (One Harri is approximately equivalent to 4-6 liters) of water are required in rainy season (June-September) whereas in summer 14 to 15 Hari of water are necessary for daily consumption. The village women and children travel 1 kilometer a day during June to January for fetching water from river. The situation is worse in winter and summer seasons (during February to May). On an average they cover 4 km to and fro to fetch water for daily domestic requirements. Men do not participate in rainy season for fetching water; however, a thin participation is there during the months of February and May. It is obvious from Figure 3.18 that the water requirement is very high during February to May. It reveals that the cost of collecting water in terms of time and energy is pretty high as compared to other villages. Therefore, necessary interventions from government and other development agencies are necessary to minimize the drudgery associated with the daily lives of the people in Ghutlia. There are some effort are shown by local panchyate samity they have dug few wells modified few ponds in village but that are not sufficient and appropriate for drinking purpose.

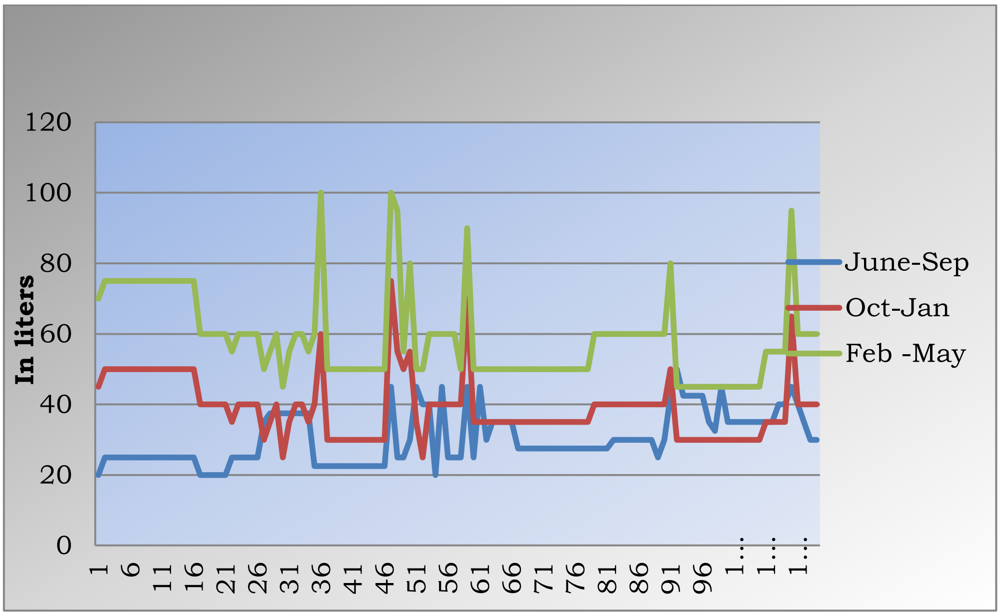


Figure 3.18 : Daily water requirement per household in different season

3.3. Description of existing water harvesting structure like Pond/Bundh

There are ten ponds in a Ghutlia village use as alternative source for drinking water and house hold activates. Water levels of are fluctuating in different season in summer season most of pond are dried up maximum water depth found was 1.526 m. In monsoon water level were increased and maximum depth found 5.48 m and in winter maximum water depth found in pond was 3.04 m. The detail description of the ponds and area were given in Table 3.1.

Photo 3.1: Water scare condition of pond in study area

Photo 3.2 : Water level of well in study area

Table 3.1 : Description of existing water harvesting structure like Pond/Bundh

Sl No	Name of the pond	Location		Elevation (m)	Water Level In Summer (m)	Water Level In Monsoon (bgl) (m)	Water Level In Winter (bgl) (m)	Area of the Pond (bgl) (m^2)
		Latitude	Longitude					
1.	Mahanto Garia	23°24'.455"	86°37'.536"	261	DRY	1.524	0.914	1145.91
2.	Kandan Garia	23°24'.455"	86°37'.536"	260	DRY	0.914	0.3048	1673.11
3.	Master Bandh	23°24'.353"	86°37'.381"	252	DRY	1.524	0.609	2549
4.	Mahato Bandh	23°24'.354"	86°37'.379"	181	DRY	1.524	0.609	3712.7
5.	Thakur Bandh	23°24'.358"	86°37'.436"	182	DRY	1.219	0.609	21187.9 7
6	Banser bandh	23°24'.796"	86°37'.244"	208	1.374	3.048	3.048	13351
7	Sonar bandh	23°24'.182"	86°37'.749"	183	1.524	3.96	3.35	13351
8	Gobordhan Garia	23°24'.270"	86°37'.611"	178	0.304	3.65	2.43	10760
9	Bania Garia	23°24'.349"	86°37'.643"	173	1.526	5.48	3.04	890.
10.	Tentul Garia	23°24'.39"	86°37'.363"	175	0.609	2.43	1.524	8903.4

3.4. Water Resources of study area and Problems of Water Scarcity

Kashipur (block) of Purulia district in West Bengal is known for tourist destinations as well as natural beauty. Many of the non-tribal hamlets located in the area face the problem of shortage of drinking water, Ghutlia is one of them. The water shortage in the village has led to various socio-economic problems related to villager daily life. For water villagers are depended upon two rivers Darokeshar and its tributary Futiary; which becomes dried up just after monsoon before implementation of the project. Depicted in Photo 3.3,Photo 3.4 and Photo 3.5 .

- The lack of water availability in the region during pre and post – monsoon, has resulted in the acute shortage of drinking water in the village.
- The lack of drinking water has resulted in increased burden on women in the dry seasons; They have to walk for 1 to 2 Km daily to fetch drinking water. The parents usually engage their daughter's in the water fetching work, instead of study to school.
- The lack of clean drinking water also resulted in the spread of various waterborne diseases among the villagers like jaundice, dehydration etc. Due to non-availability of primary health care facilities in the village, the patients have to suffer a lot and have to bear heavy medical expenses in treating themselves.
- The lack of water availability also resulted in low agriculture production due to no availability of water for irrigation purposes.
- Shortage of water also resulted in various changes in the social condition of the village. The people from outside villages are generally reluctant to establish martial relationship with daughter in the village. They felt that if they send their daughter to the village through marriage then her entire life would be devoted to the task of fetching the water.
- The low agricultural production and poverty in the village has forced the villagers to take money on high interest rate from outside people and become indebted. The increased indebtedness resulted in increased psychological and poor health status of the villagers.

Photo 3.3 : Village women collecting drinking water by digging small ditch in river bed

Photo 3.4: Source of drinking water from dry river bed

Photo 3.5 : Village women's carrying water from 3-4 km

CHAPTER-4
METHODOLOGY

4. Methodology

Literature (National and International level) has been reviewed for last thirty years. All historical data with related maps are collected from different Govt. organization reconance survey with primary data has been surveyed from field site with different time period. Socio-economic survey has been carried out through Participatory Rural Appraisal (PRA) technique based on questionnaires. All collected data is analyzed and tools with different software models are developed based on design parameters through optimization technique with output results are satisfied based with multi criteria decision technique. Finally, concluding remarks are highlights in terms of optimum cost-benefit outcomes and good numbers of recommendations are also described in this thesis.

4.1. Methodology step

Pictorial view stated in Figure 4.1is describing all points raised to understand the entire algorithm. Six main headings with different inputs have been considered for calibrating, simulating and validating the total research work to optimum the cost-effective technology of said model with social acceptability.

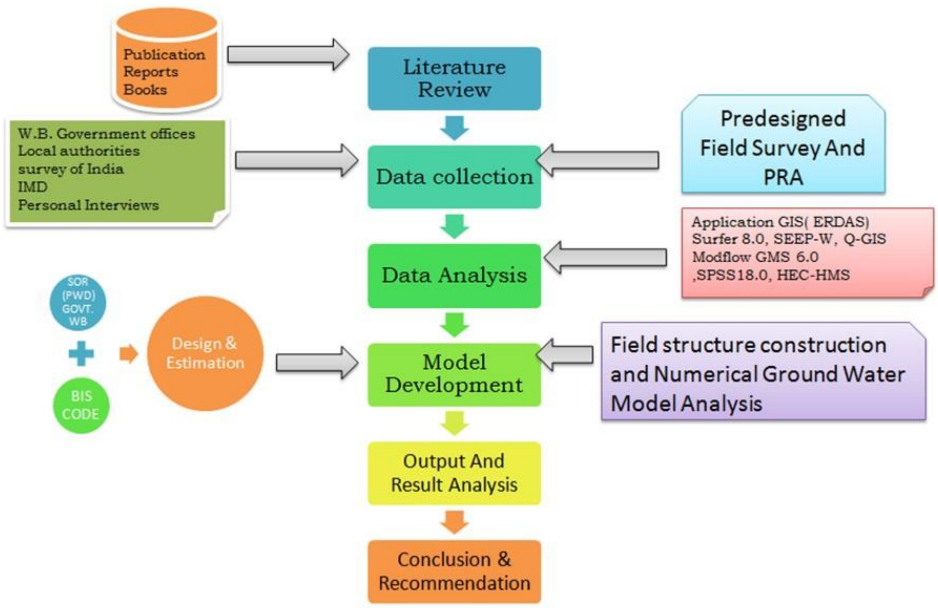


Figure 4.1: Description of the overall research methodology

4.2. Pre-Design survey and data collection, and PRA technique

Data collection and field survey was done in following stages

- Simla-Dhanera Gram Panchayat, Kashipur Block Purulia District, was selected through household baseline survey based on questionnaires, focused group discussion and village personal with interviews of individual community. PRA exercises also done in each and every hamlet followed by net planning along with families and focused discussion with SHG member.

- PRA (Participatory Rural Appraisal) techniques can be used in any situation, urban or rural, with both literate and illiterate people. PRA is intended to enable local communities to conduct their own analysis and to plan and take action PRA involves learning together with villagers about the village. The aim of PRA is to help strengthen the capacity of villagers to plan, make decisions, and to take action towards improving their own situation. Participatory Rural Appraisal (PRA) is considered one of the popular and effective approaches to gather information in rural areas. This approach was developed in early 1990s with considerable shift in paradigm from top-down to bottom-up approach, and from blueprint to the learning process. In fact, it is a shift from extractive survey questionnaires to experience sharing by local people. PRA is based on village experiences where communities effectively manage their natural resources. PRA is a methodology of learning rural life and their environment from the rural people. It requires researchers / field workers to act as facilitators to help local people conduct their own analysis, plan and take action accordingly. It is based on the principle that local people are creative and capable and can do their own investigations, analysis, and planning. The basic concept of PRA is to learn from rural people. By definition PRA is an approach and methods for learning about rural life and conditions from, with and by rural people. It is further stated that PRA extends into analysis, planning and action. PRA closely involve villagers and local officials in the process. Similarly, Rapid Rural Appraisal (RRA) reflects the new thinking about development, needs, and people oriented responsibilities. It is a process that is highly systematic and structured, relying on interdisciplinary teamwork and special strategies for data collection and analysis such as triangulation, probing, and iteration. Some critics consider RRA to be a quick and dirty technique. In the present study PRA technique is adopted for demand analysis and site suitability studies.

Photo 4.1: Discussion with villager

Photo 4.2 : Community dialogue with villager

- Topo-sheets (nos 73E/14, E/15 and E/16) were collected from survey of India, Govt of India. Watershed map which includes contour lines, drainage boundary, slopes, land-use pattern, soil cover, existing hydrology and hydrogeologcal condition based on secondary data and the same maps were geo-referenced using GPS (Global Positioning System). Vertical Electrical Resistivity (VES) was used to find out the soil profile and depth of water tables/peizometer and it was validated with known litholog data collected from Gram- Panchayat office depth of existing tube wells, are measured using water level indicator and also per capita drinking water was quantified.

Photo 4.3 : GPS reading measurement of well

Photo 4.4: Measurement of well depth

- Secondary data related to the thesis work, collected from Agricultural Department Government West Bengal. Hydro-meteological data from concerned division of Indian Meteorological Department(IMD). And Industry DVC. Local Got. bodies

4.3. Model development

Numerical groundwater modelling is a process in which computer programmes (model codes) are used to solve the equations controlling groundwater flow to determine groundwater levels (also referred to as aquifer heads or potentials) at all points in an aquifer system. The process begins by representing the aquifers and aquitards in the subsurface as a series of interconnected blocks. Values are assigned to represent the hydrogeological properties of each block. Initial estimates for the values may be obtained from subsurface borehole information and aquifer testing data. These estimates are usually refined in the process of model calibration. By using many small blocks, or cells, an accurate representation of the variation in aquifer and aquitard geometry and property values has been constructed to develop the model.

4.3.1. Conceptual model

The conceptual model has been developed from the lithological logs and pumping test data represented by Groundwater Modelling System (GMS 6.0) software for each pumping site. Seasonal variations of stream flow can lead to change of groundwater flow fields in an aquifer-stream water system. For assessing spatial and temporal variations of flow fields due to the seasonal variations of stream flow, a hypothetical aquifer-stream water interaction system has been modelled using GMS software (Groundwater Modelling System) for each pumping sites. GMS software is one of the most sophisticated groundwater modelling environment software available today. The GMS software is a comprehensive package which provides tools for every phase of a groundwater simulation including site characterization, model development, post processing, calibration, and visualization. GMS is the only system which supports TINs, solids, borehole data, 2D and 3D geo statistics, and both finite element and finite difference models in 2D and 3D environment. It supports many other models within a single environment. Due to the modular nature of GMS, a custom version of GMS with desired modules and interfaces can be configured. Unlike other models, this software combines the functionalities of different groundwater models and the powerful GIS (Geographic Information Systems) interface. The aquifer-stream water interaction system depends on several factors, namely, location, geometry, elevation and physical setting and other inherent properties. This involves use of large database, which can be easily handled using the GIS functionalities available within GMS.

Component of Conceptual model

Here an attempt is made to model the effect of stream flow on groundwater flow fields by using of MODFLOW simulation in GMS 6.0. Two approaches can be used to construct a MODFLOW simulation in GMS: the grid approach or the conceptual model approach. The grid approach involves working directly with the 3D grid and applying sources/sinks and other model parameters on a cell-by-cell basis. The conceptual model approach involves using the GIS tools in the Map module to develop a conceptual model of the site being modelled. The location of sources/sinks, layer parameters such as hydraulic conductivity, model boundaries, and all other data necessary for the simulation can be defined at the conceptual model level. Once this model is complete, the grid is generated and the conceptual model is converted to the grid model and all of the cell-by-cell assignments are performed automatically. MODFLOW is a finite-difference method model written in FORTRAN by the United States Geologic Survey. MODFLOW solves a series of finite difference equations in as many as iterations it requires to converge to a desired solution, which is simplified through graphical user interface (GUI). GIS and MODFLOW would be beneficial in furthering our knowledge of surface/subsurface interactions and our accuracy in analysing and predicting these two systems. Geographically referenced layers of data manipulation when linked with MODFLOW give better result than when only MODFLOW is used. The following objectives were envisaged for the modelling:

I. To generate and study the pattern in the variations of hydraulic heads in the groundwater system due to change in river stage.
II. To find out flux (flow rate) through the rivers and flux through recharge polygon.
III. To find out the flux for a set of selected cells.

The governing partial differential equation used in MODFLOW is:

$$\frac{\partial}{\partial x}\left[K_{xx}\frac{\partial h}{\partial x}\right]+\frac{\partial}{\partial y}\left[K_{yy}\frac{\partial h}{\partial y}\right]+\frac{\partial}{\partial z}\left[K_{zz}\frac{\partial h}{\partial z}\right]+W=S_S z \quad \ldots\ldots\ldots\ldots 4.1$$

Where,

K_{xx}, K_{yy} and K_{zz} are the values of hydraulic conductivity along the x, y, and z coordinate axes (L/T)

h is the potentiometric head (L) , W is a volumetric flux per unit volume representing sources and/or sinks of water, where negative values are extractions, and positive values are injections (T^{-1}),S_S is the specific storage of the porous material (L^{-1}) and, t is time (T).The right hand side of the above equation reduces to zero for steady-state flow (flow remains constant with respect to time).Required modules or interfaces to model MODFLOW are accordingly Grid, geostatic map,MODFLOW. Flow chart Figure 4.2 used for the modeling of MODFLOW conceptual model is as follows.

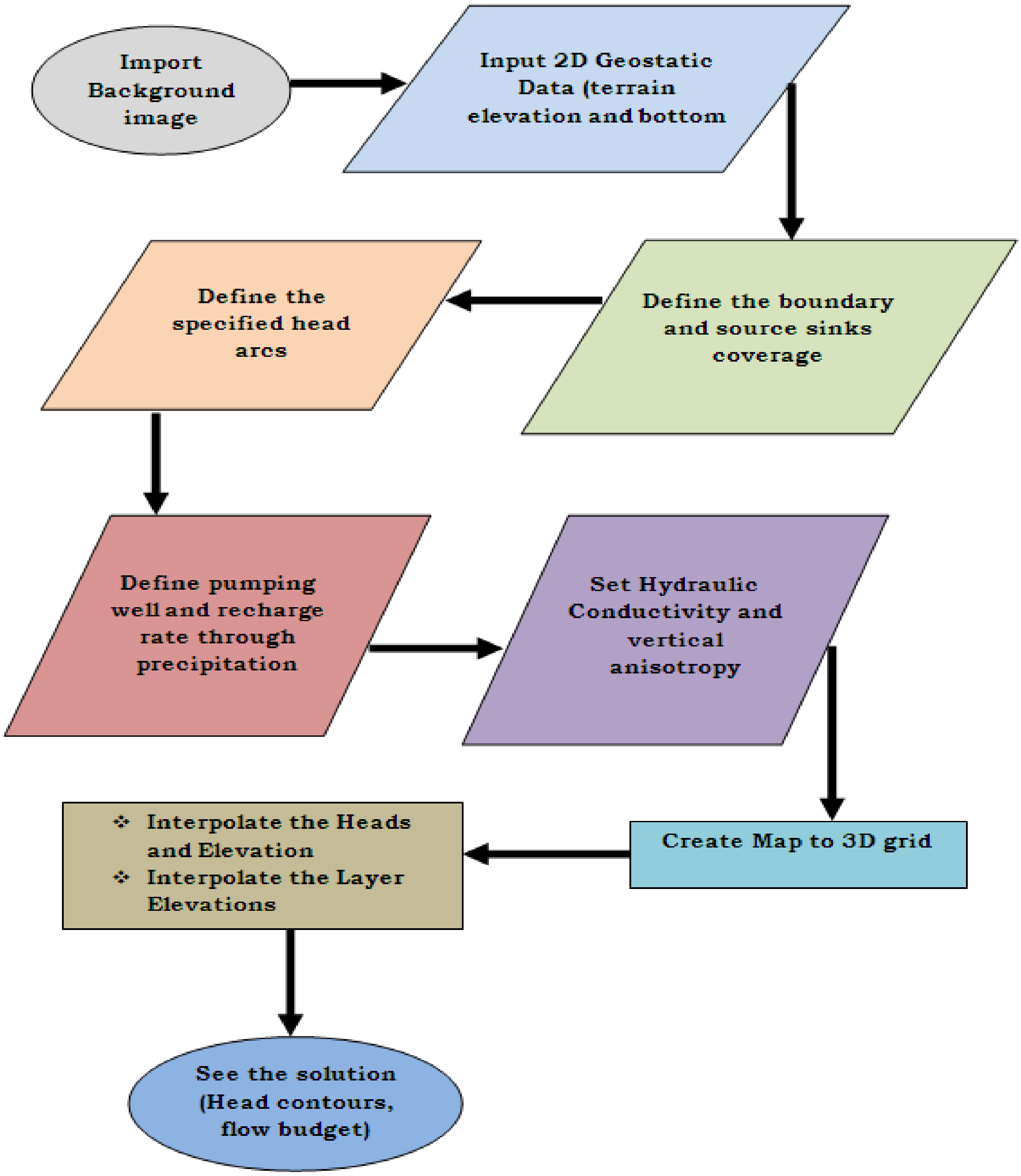


Figure 4.2 : Flow chart for modeling the effect of stream flow by GMS 6.0.

Output of Conceptual model

- Output of the MODFLOW conceptual model simulation shows the hydraulic head fluctuation around the well fields for each pumping site individually.
- The model output actually estimates the flow rate under steady state condition in groundwater system through balancing recharge and discharge for the entire zone.

4.3.2. Process Based model

The US Army Corps of Engineers (USACE) Hydrologic Engineering Center (HEC) has long been recognized as one of the most respected centers for hydraulic modelling software in the water resources community. In the mid 1960's, the HEC began development of models that soon became the water surface profile program HEC-2. For nearly 30 years,HEC-2 was probably the most widely used and accepted program worldwide for determination of water surface elevations. In 1993, the HEC introduced HEC-RAS(River Analysis System), the first version of their Windows based software for water surface profile calculation. The current version of HEC-RAS can be obtained from HEC's website. The IDNR encourages the use of HEC-RAS for regulatory and flood plain management purposes within India. The HEC-RAS steady state model uses the standard step-backwater method for calculation of water surface profiles. The HEC-RAS manual, along with many basic hydraulic engineering texts, describes this computational methodology.

Component of HEC-RAS

a) Graphical User Interface (GUI): HEC-RAS is an integrated system of software, designed for interactive use in a multi-tasking environment. The system is comprised of a graphical user interface (GUI), separate hydraulic analysis components, data storage and management capabilities, graphics and reporting facilities.

b) Analyze river components: The HEC-RAS system will ultimately contain three one-dimensional hydraulic analysis components for: (1) steady flow water surface profile computations, (2) unsteady flow simulation, and (3) movable boundary sediment transport computations. A key element is that all three components will use a common geometric data representation and common geometric and hydraulic computation routines. In addition to the three hydraulic analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed.

c) Data storage and management: Data storage is accomplished through the use of "flat" files as well as the HEC-DSS. User input data are stored in flats files under separate categories of project, plan, geometry, steady flow, unsteady flow, quasi-steady flow, sediment data, and water quality information. Output data is predominantly stored in separate binary files. Data can be transferred between HEC-RAS and other programs by utilizing the HEC-DSS.Data management is accomplished through the user interface. The modeler is requested to enter a single filename for the project being developed. Once the project filename is entered, all other files are automatically created and named by the interface as needed. The interface provides for renaming, moving, and deletion of files on a project-by-project basis.

d) Graphics and reporting: Graphics include X-Y plots of the river system schematic, cross-sections, rating curves, hydrographs, and inundation mapping. A three-dimensional plot of multiple cross sections is also provided. Tabular output is available. Users can select from pre-defined tables or develop their own customized tables. All graphical and tabular output can be displayed on the screen, sent directly to a printer, or passed through the Windows clipboard to other software, such as a word- processor or spreadsheet. Reporting facilities allow for printed output of input data as well as output data. Reports can be customized as to the amount and type of information desired.

Program capabilities and applications of HEC-RAS

1. Steady flow water surface profiles:

This component of the modeling system is intended for calculating water surface profiles for steady gradually varied flow.

2. Unsteady flow simulation:

This component of the HEC-RAS modeling system is capable of simulating one-dimensional unsteady flow network of open channels. The hydraulic calculations for cross-sections, bridges, culverts, and other hydraulic structures that were developed for the steady flow component were incorporated into the unsteady flow module.

Boundary condition of HEC-RAS:

This component of the modeling system is intended for the simulation of one-dimensional sediment transport/movable boundary calculations resulting from scour and deposition over moderate time periods (typically years, although applications to single flood events are possible. Following aspects can also utilize as boundary parameters.

- Water quality analysis (temperature, algae, DO, BOD, etc.)
- Inline Structures (dams, weirs and gated spillways), bridge/culvert modeling:
- Flood Management (floodplain encroachment, flood insurance Studies)
- Sediment transport and management
- Flow variance

COMPUTATIONAL PROCEDURES

Steady Gradually Varied Flow water surface profile calculations. Basic Equations called energy equation Figure 4.3 and energy loss described in Figure 4.5, Conveyance calculation from Manning's equation described in pictorial views in Figure 4.4.

- **1D Energy Equation:**

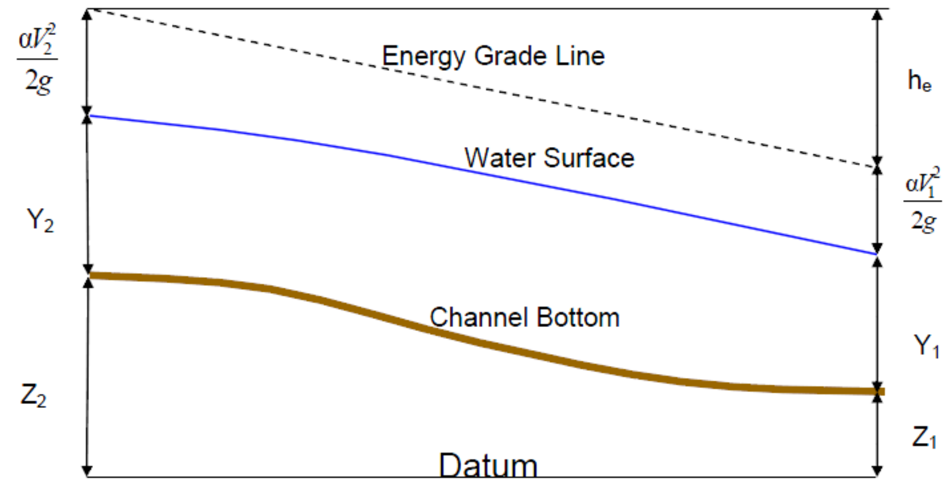


Figure 4.3 : 1D Energy Equation

$$\mathbf{Z_2+Y_2+\frac{a_2V_2{}^2}{2g} = Z_1 + Y_1 + \frac{a_1V_1{}^2}{2g} + h_e} \quad \text{..........4.2}$$

Where: Z_1, Z_2 =elevation of the main channel inverts

Y_1, Y_2= depth of water at cross sections

V_1, V_2= average velocities (total discharge/ total flow area)

a_1 ,a_2= velocity weighing coefficients

G = gravitational acceleration

h_e= energy head loss

- **Conveyance calculation from Manning equation**

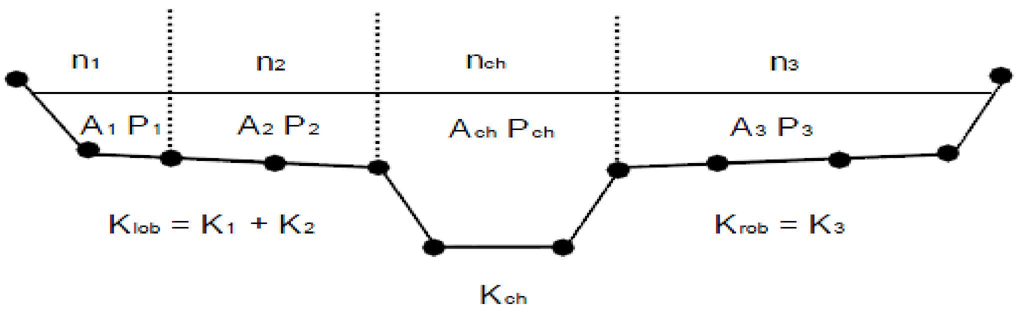


Figure 4.4 : Conveyance calculation from Manning equation

$$Q = \frac{1}{n} R_h^{\frac{2}{3}} A S_f^{\frac{1}{2}} \quad \text{..........4.3}$$

$$K = \frac{1}{n} R_h^{\frac{2}{3}} A \quad \text{..........4.4}$$

$$Q = K S_f^{\frac{1}{2}} \quad \text{..........4.5}$$

Q = Flow

n = Manning's coefficient

A= Cross sectional area

R_H= Hydraulic radius

S_f= Fictional force

K= conveyance

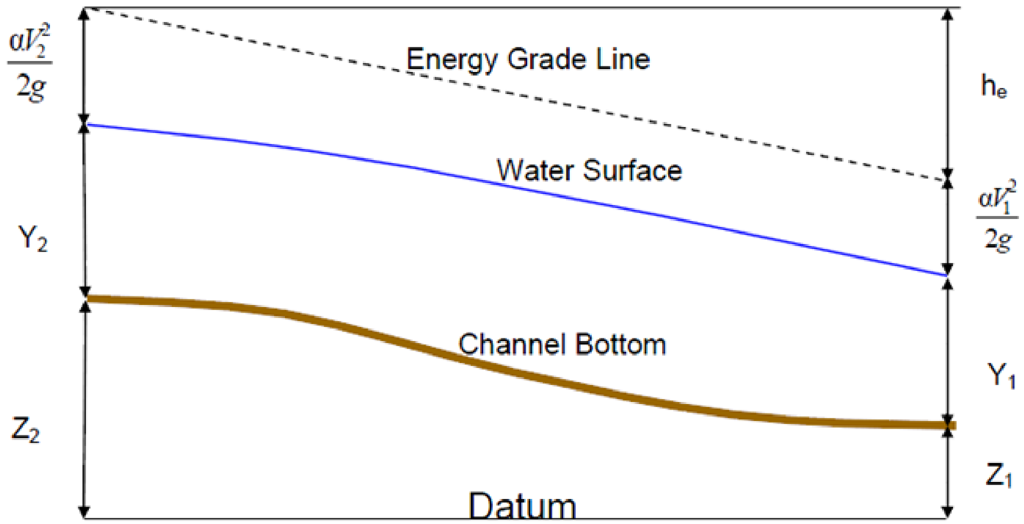


Figure 4.5 : Energy loss, h_e

- **Energy loss, h_e**

h_e= $L\bar{S}_f$ + $C[\frac{a_1 V_1{}^2}{2g} - \frac{a_2 V_2{}^2}{2g}]$..4.6

Where: L = reach length

A = cross sectional area

C= Contraction / Expansion coefficient

a = Velocity weighing coefficients

$\bar{S_f}$ = friction slope

h_e = head loss

g =gravity

V =flow velocity

The 1D Saint Venant equations is commonly used for open channel flow and surface runoff. It can be derived from the Navier stokes equation for one dimensional flow that describe the motion. The 1D Saint Venant Equation in the x direction,

$$\frac{\partial Q}{\partial x} + \frac{\partial A_t}{\partial t} = 0 \quad \ldots\ldots 4.7$$

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\alpha_B \frac{Q^2}{A_f}\right) + gA_f \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 R A_f} = 0 \quad \ldots\ldots 4.8$$

Where Q is discharge in m3/s, x is distance along river in m, tis time in sec,$\boldsymbol{\alpha_B}$is momentum distribution coefficient, $A_f$is the cross section of flow in m^2, $A_t$is total cross section area in m^2, is acceleration due to gravity in m^2/sec, h is water depth above datum level in m, C is the Chezy's coefficient in $m^{1/2}$/s and R is the hydraulic radius in m.

4.4. Data analysis and process:

GIS technology with help of ERDAS software enables the ease of data processing, visualization, sorting, assessment, computation and map preparation when compared with other technologies. Another software Surfer 8.0 is also used for development of contour map. MS Excel is used for analysis of socio economic survey data.

4.5. Design and Estimation of Field Model

There are three general steps followed in the design of composite model of cost effective technology for recharging in over exploited groundwater resources, after carrying out in situ investigation and site selection. These are hydrologic design, hydraulic design, and structural design

4.5.3. Hydrologic design

Hydrologic design involves estimation of peak rate of runoff required to be passed safely through the given structure and runoff volume and water yield required to be stored in ponds/reservoir. In hydrologic design following steps are taken.Here Rational method is used for the estimation of peak rate of discharge from small watershed .It is expressed by the equation.

$$Q = \frac{\mathbf{CIA}}{\mathbf{360}} \quad \ldots\ldots 4.9$$

Where, Q = Peak rate of runoff in m^3/s for given design frequency of rainfall

C = Rational runoff coefficient having value ranging from 0 to 1 depending up on Watershed condition

I = Rainfall intensity (mm/hr) for design frequency and duration equal to time of concentration.

A= Area of watershed(ha)

Here, area is calculated under various land use and soil and the area weighted average value of 'C' is computed as given below .

$$C=\frac{A1C1+A2B2+A3C3+\cdots\ldots\ldots AnCn}{A} \quad \ldots\ldots\ldots 4.10$$

The time concentration T_c of the watershed computed from Nomograph. Deter mining the 1 hour rainfall intensity in deferent frequency in India from the map. And also use the chart for relation of one hour rain fall intensities to intensities at other duration. Water yield is computing from Strange tables. The giving percentage of runoff with reference of total rain fall have been developed based on data from Maharastra. (Sivanappan,1992)

4.5.4. Hydraulic design

Hydraulic design includes determination of storage capacity and storage dimensions of the water harvesting structure. Height of dam or depth. Fixing of spillway dimension for safe disposal of peak flow. Different formula used in hydraulic design for different purposes are given below The length of spill way determined using standard weir formula

$$Q = \frac{2}{3} C_d L\sqrt{2g}\, H^{3/2} \quad \ldots\ldots\ldots 4.11$$

Q = peak discharge

C_d = coefficient of discharge

L =length crest

H= depth of flow over the crest

4.5.5. Structural Design

The objective of structural design is to fixing the dimension of structural body component and their material of construction to with stand static and dynamic forces acting on the structure the force include hydrostatic water pressure, weight of dam and uplift pressure. The structure is designed for safety against overturning, sliding tension, compression.

4.5.6. Estimation

After completion of design step get the cross section of structure and estimate the total cost for construction of the structures on the basis of Schedule of Rate (PWD) Govt. of West Bengal. Locally available materials are used for construction therefore the cost of construction will be less.

CHAPTER-5

SURVEY AND INVESTIGATION

(Kangsabati River, Damodar and Darokeswar River Basin)

5. Survey and Investigation

Water resource management of near-channel ground water and surface water to maintain stream health and flood plain ecological function requires researchers to refocus their conceptual models of water exchange between the aquifer and stream. The high hydraulic conductivity fluvial plain directs ground water flow down-plain where it exchanges with the stream channel creating gaining, losing, flow-through, and parallel-flow reaches. The resulting complex flow system requires consideration when profiles representing ground water flow paths are constructed. In addition to interaction at the scale of the fluvial plain, exchange of ground water and surface water within and immediately adjacent to the stream channel creates hyporheic zones. The physical and bio-geochemical extent of these zones depends on the head distribution and ground water flow directions, stream hydraulics, and channel bed conditions, and magnitudes and distributions of hydrogeologic parameters. Simulated conceptualizations of flow dynamics caused by slight increases in hydraulic potentials at the surface water-stream bed interface indicate stream-ground water mixing could occur to a depth of 1.7 m below the channel. Rescaling of traditional hydrogeologic approaches to include the fluvial plain and channel scale will need the proper survey and investigation. Outcomes of survey and investigation suggest the appropriate location and boundary condition for the cost-effective model of ground water recharging and supply the data's for numerical models. In this research survey and investigation carried out in two parts reconnaissance survey and detail survey or investigation. The total survey is conducted in three river basin Kangshabati River basin, Damodar River basin and Daorkeswar River Basin.

5.1. Reconnaissance Survey

Reconnaissance survey is done to examine the general character of the area for determining the most feasible location for further more detailed investigations. Data collected from reconnaissance survey is used for feasibility study of all different location, preparation of approximate estimates of quantities and costs. This helps in selection of most suitable alternatives. This survey also helps in determining any deviations required in the basic geometric standards to be adopted for the water resource planning.

The reconnaissance survey methods conducted in the following sequence:

- Study of topographical survey sheets, agricultural, soil, geological and meteorological maps.
- Ground reconnaissance including another round of serial reconnaissance for inaccessible and difficult stretches, where required.

Study of Survey Sheets and Maps:

Reconnaissance survey starts with a study of the all available maps. After study of the topographical features on the maps, a number of economical location feasible in a general way are selected keeping in view the following points:

(i) The location to take into account all the control points and to be shortened and more economical compatible with requirements of site suitability.

(ii) Shape of the alignment.

(iii) Avoidance as far as possible of earthen bank, steep terrains, unstable hill features and areas subject to severe climatic conditions, flooding and inundation.

(iv) Need of connecting important villages and localities.

(v) Bridging problems, and

(vi) Need to preserve environment and maintain ecological balance.

The satellite imageries taken to a scale of 1:20000 to 1:50000 to supplement the information from topographic maps. If stereoscopic techniques are applied, photographs can yield quantitative data, and if studied by a skilled photo interpreter, significant soil and subsoil information. Ground reconnaissance consists of general examination of ground by walking or riding along the probable locations and collecting all available information necessary for evaluating the same. In the case of hill sections, it may be advantageous sometime to start the reconnaissance from an obligatory point situated close to the top. If an area is inaccessiblefor ground reconnaissance, aerial method should be used to clear the doubts. Whilereconnoitring on the ground, it is advisable to leave reference pegs to facilitate further survey operations. Instruments for ground reconnaissance include compass, GPS, clinometers, water quality field test kit, current meter etc. Use of instruments mentioned above to obtain ground slopes, maximum gradients, elevation of critical summits or stream crossings and location of obligatory points, serves as a check on the maps being used.The area has been surveyed by the technical experts of SWRE before detail survey. GPS locations have been identified and to quantify the surface water during pre and post monsoon period at different stretches of Kangsabati and Damodar River at six different blocks of Purulia district (Figure 5.1,Figure 5.2 and Figure 5.3). Accordingly, the water samples have been collected at different hot spots area during reconnaissance survey.

Photo 5.1: Reconnaissance Survey at Hura block

Figure 5.1: Reconnaissance survey at Arsha and Purulia I Block

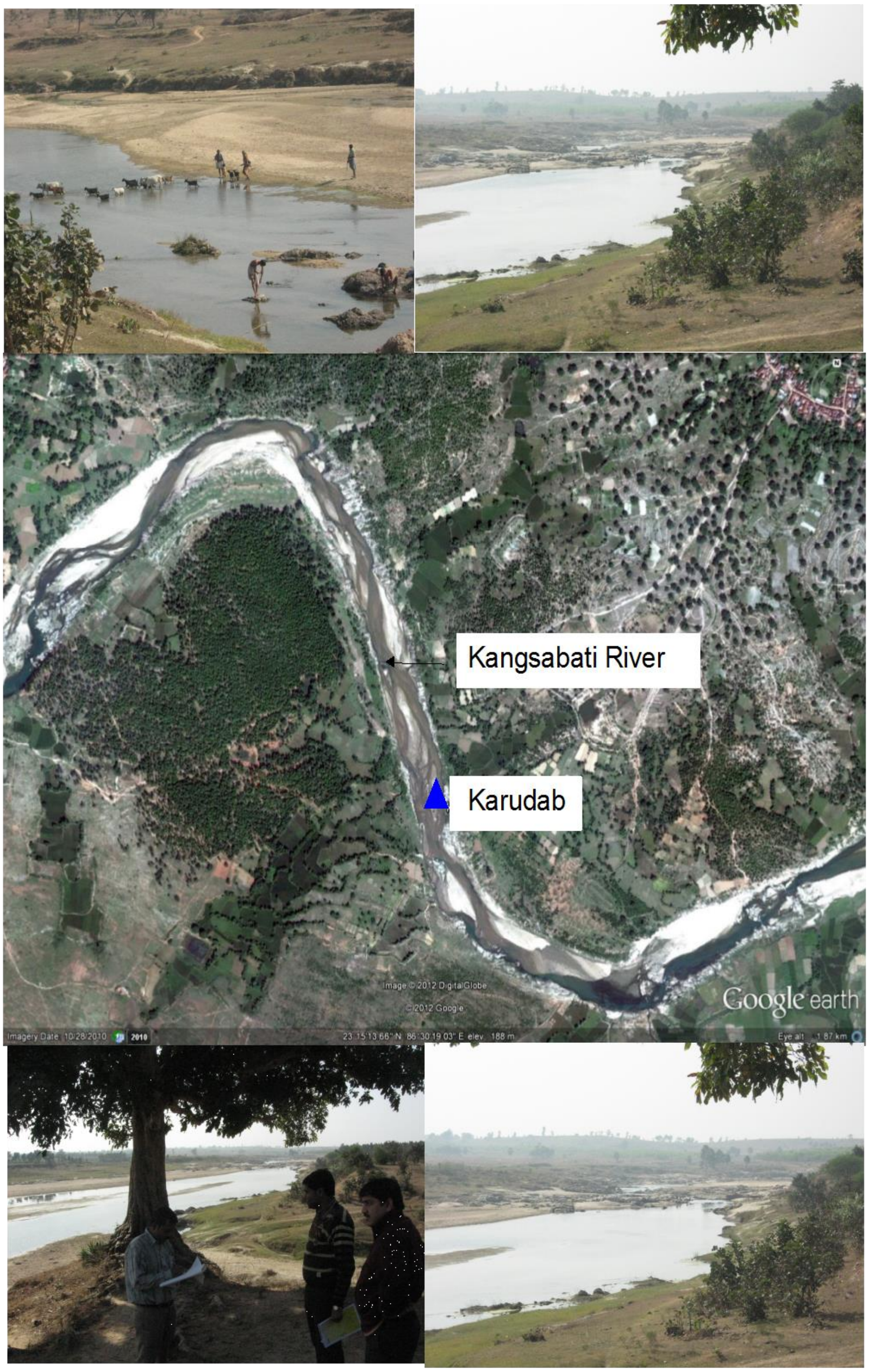


Figure 5.2 : Reconnaissance survey at Hura Block

Figure 5.3 : Reconnaissance survey at Nituria Block

5.2. Detail Survey and Investigation

5.2.1. Hydro-Meteorological survey

Hydrometeorological survey ensures integrated study of hydrometeorological conditions at the construction territory and predicting potential changes in such conditions due to interaction with the facility under design. A detailed hydro-meteorological study for the study area has been carried out using data obtained from the different Government agencies. Data has been collected from seven stations in and around the sub-basin. In Purulia district the twenty years' rainfall data have been collected and analysed given in depicts that the highest and lowest annual rainfall has occurred in 1999 and 2005 respectively. The result also highlights about 80% of total rainfall has predominant during monsoon. Figure 5.4 and Figure 5.5 describes the highest rainfall occurred during monsoon in different periods and the highest magnitude has found to be 316 mm in 2006 followed by 276 mm in 1995 and so on. Thus, the study area represents the intensity of rainfall has very high compared to the total annual rainfall found an average of 900 mm in the same area and hence the study area defines as zone of shot-pockets of scarcity so far, the quantity of water is concerned.

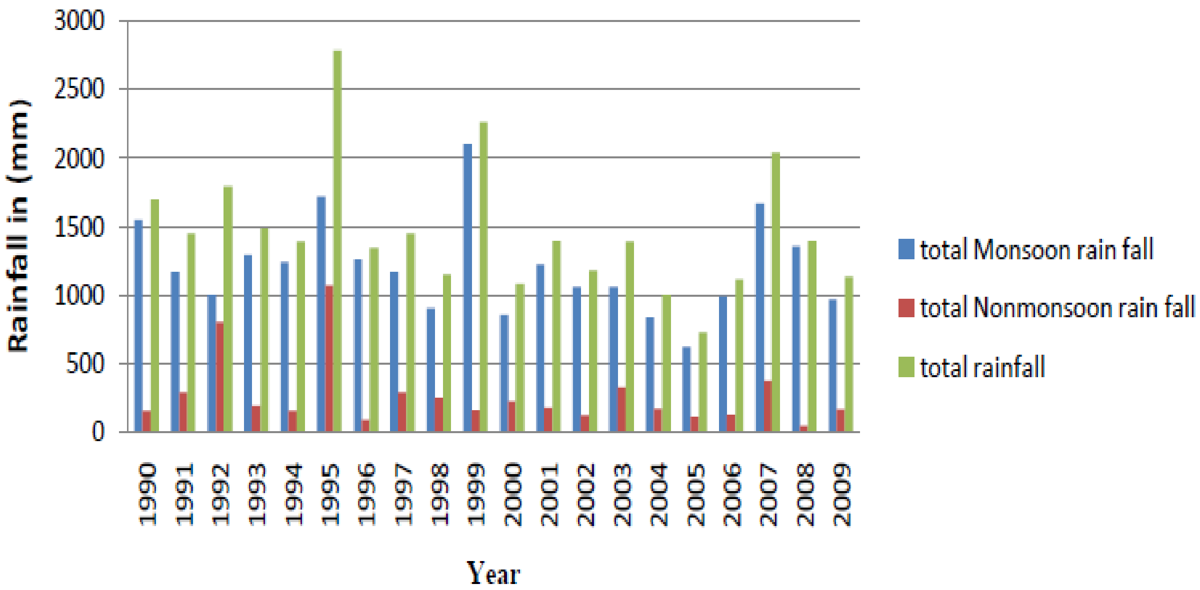


Figure 5.4 : 20 years monsoon non monsoon and total rainfall in study area

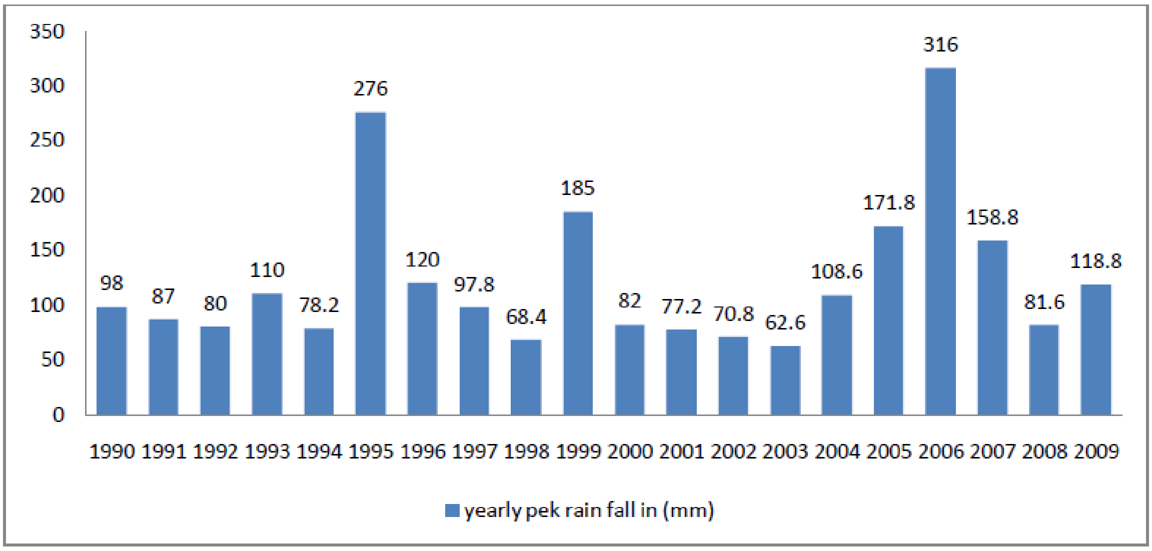


Figure 5.5 : Comparison of 20 years peak rain fall data in study area

The Hydrologic Modelling System (HEC-HMS) was used to calculate runoff generation. From the daily precipitation total runoff is calculated for each year 1993,1994,1995,1996,1997,1998,1999,2000,2001,2002 for Hura, Bandu and Kangsabati block which is shown in above Table 5.1andTable 5.2 considering evapotranspiration and other major losses like surface runoff. The model is calibrated and validated using observed monthly streamflow data collected at the outlet of the basin. Runoff Volumes have increased to different degrees during the study period (1993–2002) for the 2 blocks of river Kangsabati due to varying area of the catchment. The objective of the model calibration was to match simulated runoff with the observed data with different meteorological conditions.Total runoff observed was 3797.47 mm and 3077.56 mm for Hura and Bandu Kangshabati respectively. Total runoff was about 32% of the total precipitation from the year 1993-2002 which was represented and matched from the model runoff. Total runoff volume was 234.12 Mm^3 and 271.12 $Mm^3$for Hura and Bandu Kanshabati respectively. To calibrate and verify the HEC-HMS model, 10-year (1993–2002) streamflow and precipitation data were used for the study catchment.Initial abstraction, time of concentration, storage coefficient, recession constant, baseflow, kinematic wave and travel time were considered as HEC-HMS calibration parameters. However, the model estimated volume of runoff and observed volume of runoff did not match accurately but the difference was also till a very meager extent.

Table 5.1: Runoff volume (Mm^3) of Hura catchment area

Year	∑P (mm)	∑ET (mm)	Loss (L) (mm)	Base flow (mm)	Surface runoff (mm)	Total runoff (mm)	Runoff Volume (Mm^3) =Area x DR Area=61.73 km^2
1993	1150.6	680.36	30.51	115.06	314.66	429.72	26.48
1994	1160.99	698.59	34.83	114.09	313.46	427.55	26.35
1995	970.86	582.52	26.13	97.09	265.13	362.22	22.34
1996	1070.65	642.39	32.12	107.07	289.08	396.15	24.45
1997	1090.47	650.28	30.72	109.05	300.43	409.48	25.25
1998	1060.38	636.23	34.82	106.04	283.31	389.35	24.01
1999	1120.93	674.56	33.63	112.09	300.65	412.74	25.43
2000	790.56	470.33	22.72	79.06	218.45	297.51	18.33
2001	830.46	498.28	23.92	83.05	225.22	308.27	19.01
2002	990.49	596.29	29.72	99.05	265.43	364.48	22.47

Table 5.2 : Runoff volume (Mm^3) of Bandu-Kangsabati catchment area

Year	∑P (mm)	∑ET (mm)	Loss (L) (mm)	Base flow (mm)	Surface runoff (mm)	Total runoff (mm)	Runoff Volume (Mm^3) =Area x DR Area=65.4 km^2
1993	1150.6	665.44	20.02	126.58	338.56	465.14	30.41
1994	1160.99	676.37	26.22	127.71	330.39	458.1	29.95
1995	970.86	560.10	14.42	106.79	289.55	396.34	25.89
1996	1070.65	612.98	21.41	117.77	318.49	436.26	28.51
1997	1090.47	632.47	15.81	119.95	322.24	442.19	28.91
1998	1060.38	613.02	21.21	116.64	309.51	426.15	27.86
1999	1120.93	650.13	17.42	123.30	330.08	453.38	29.62
2000	790.56	448.52	15.81	86.96	239.27	326.23	21.32
2001	830.46	472.66	16.61	91.35	249.84	341.19	22.30
2002	990.49	574.48	12.81	108.95	294.25	403.2	26.35

When simulated runoff, and observed runoff was calibrated approximately 2% error was observed for Santuri catchment area which is supposed to be negligible error so observed runoff was used for further calculations and forecasting. Similar results were observed when calculations were done for Raghunathpur and Nituria catchment.

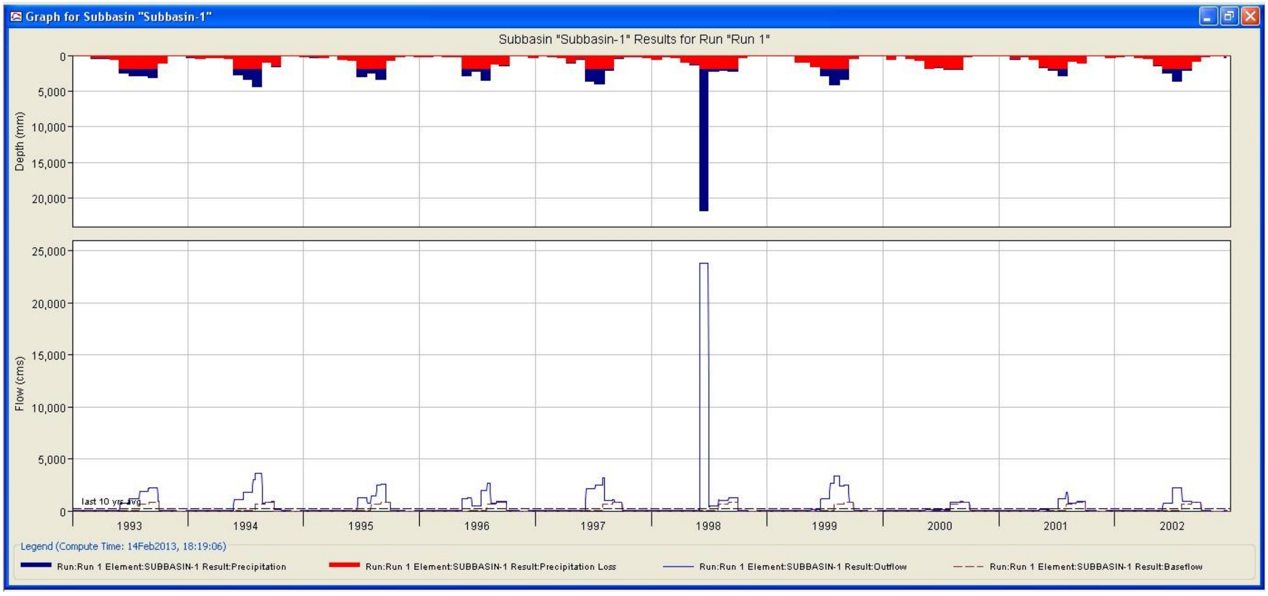


Figure 5.6 : Representation of total runoff and losses (Damodar river)

Table 5.3 : Runoff volume (Mm³) of block Raghunathpur

Year	∑P (mm)	∑ET (mm)	Loss (L) (mm)	Base flow (mm)	Surface runoff (mm)	Total runoff (mm)	Runoff Volume (Mm³) =Area x DR Area=96 km²
1993	1159.58	699.34	11.59	115.95	332.70	448.65	43.07
1994	1172.98	710.52	17.72	120.29	324.45	444.74	42.69
1995	994.63	601.72	11.94	102.46	278.51	380.97	36.57
1996	1002.32	602.42	11.02	100.23	288.65	388.88	37.33
1997	1078.52	650.89	12.78	112.85	302.00	414.85	39.82
1998	1051.91	635.67	12.51	110.19	293.54	403.73	38.75
1999	1115.21	672.28	11.15	111.52	320.26	431.78	41.45
2000	791.92	480.07	10.91	75.19	225.75	300.94	28.89
2001	820.83	500.71	14.20	80.08	225.84	305.92	29.36
2002	991.22	602.65	10.91	107.12	270.54	377.66	36.25

Table 5.4: Runoff volume (Mm³) of block Nituria

Year	∑P (mm)	∑ET (mm)	Loss (L) (mm)	Base flow (mm)	Surface runoff (mm)	Total runoff (mm)	Runoff Volume (Mm³) =Area x DR Area=103.46 km²
1993	1159.58	704.34	11.59	111.95	331.70	443.65	45.90
1994	1172.98	710.52	17.72	120.29	324.45	444.74	46.01
1995	994.63	605.72	10.94	97.46	279.51	376.97	39.00
1996	1002.32	610.42	10.02	96.23	285.65	381.88	39.50
1997	1078.52	650.89	14.78	112.85	300.00	412.85	42.71
1998	1051.91	650.67	10.51	105.19	285.54	390.73	40.42
1999	1115.21	675.28	11.15	108.52	320.26	428.78	44.36
2000	791.92	484.07	7.91	79.19	220.75	299.94	31.03
2001	820.83	502.71	10.20	82.08	225.84	307.92	31.85
2002	991.22	605.65	10.91	104.12	270.54	374.66	38.76

Table 5.5 : Runoff volume (Mm³) of block Santuri

Year	∑P (mm)	∑ET (mm)	Loss (L)mm	Base flow (mm)	Surface runoff(mm)	Total runoff(mm)	Runoff Volume (Mm³) =Area x DR Area=66.44 km²
1993	1159.58	680.34	12.59	125.95	340.7	466.65	31.00
1994	1172.98	695.52	20.72	127.29	329.45	456.74	30.34
1995	994.63	581.72	15.94	113.46	283.51	396.97	26.37
1996	1002.32	590.42	12.02	100.23	299.65	399.88	26.56
1997	1078.52	652.89	17.78	107.85	300.00	407.85	27.09
1998	1051.91	635.67	10.51	115.19	290.54	405.73	26.95
1999	1115.21	660.28	11.15	111.52	332.26	443.78	29.48
2000	791.92	473.07	7.91	90.19	220.75	310.94	20.65
2001	820.83	490.71	12.20	82.08	235.84	317.92	21.12
2002	991.22	605.65	10.91	104.12	270.54	374.66	24.89

The Hydrologic Modelling System (HEC-HMS) was used to calculate runoff generation. From the daily precipitation total runoff is calculated for each year for Nituria, Santuri and Raghunathpur-2 block which is shown in Table 5.3, Table 5.4 and Table 5.5considering evapotranspiration and other major losses like surface runoff. The model is calibrated and validated using observed monthly streamflow data collected at the outlet. Runoff Volumes have increased to different degrees during the study period (1993–2002) for calibration and the calibrated model has been used to estimate the runoff volumes for the period of 2011 for validation for the 3 blocks of river Damodar due to varying area of the catchment. The objective of the model calibration was to match simulated runoff with the observed data with different meteorological conditions. Total runoff calculated was 3898.12 mm, 3862.12 mm and 3981.12 mm for the catchments Raghunathpur-2, Nituria and Santuri respectively. Total runoff observed was about 32% of total precipitation which is represented and matched from the model runoff. Total runoff volume calculated was 374.18 Mm3, 399.54 Mm3 and 264.45 for Raghunathpur-2, Nituria and Santuri catchments respectively. To calibrate and verify the HEC-HMS model, 10 year (1993–2002) streamflow and precipitation data were used for the study watershed. Initial abstraction, time of concentration, storage coefficient, recession constant, baseflow, kinematic wave and travel time were considered as HEC-HMS calibration parameters However, the model estimated volume of runoff and observed volume of runoff did not match accurately but the difference observed was also till a very meagre extent.

5.2.2. Geophysical investigation

The Resistivity method is used to determine resistivity of subsurface geology layers, identify clay or saline aquifers, determine depth to hard rock of various Vertical Electrical Sounding (VES) and isolated various individual geo-formational layers with their thicknesses. After determining the resistivity of various layers from the interpretation of VES field data, soil texture of the layers can be determined and commencement on availability of water can be done, according to the standard scale given below in Figure 5.7.

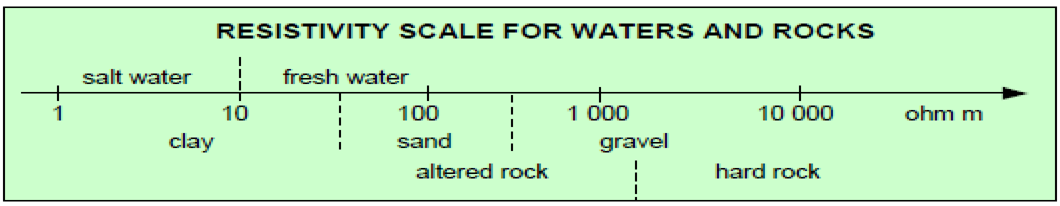


Figure 5.7 :Standard scale

For this survey, a total of 15 and 20 VES soundings are obtained using the SSR-MP-ATS resistivity meter with the Schlumberger configuration A-MN-B (Figure 5.7) in the sites of Kangsabati and Damodar River. Three sites were chosen to take 15 VES sounding Point readings and in case of Damodar River 20 VES sounding Point readings were taken in four sites, out of which two were carried out in Damodar river bed of Santuria Block and other two in Damodar river bed of Nituria Block. All the points are shown using Google Earth image.

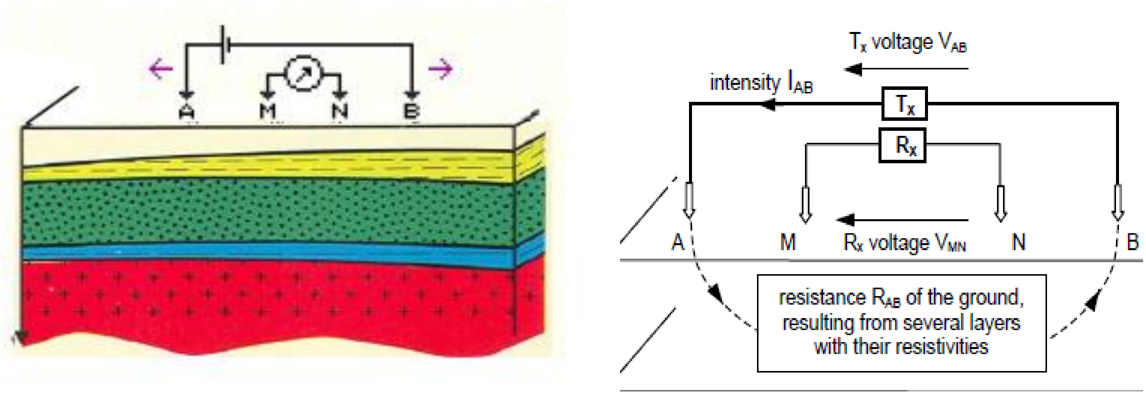


Figure 5.8 : Diagram of Schlumberger configuration array

For each VES Sounding, the distance between potential electrodes MN was kept fixed at 2 m and the distance between current electrodes was increased gradually from 10 m to 100 m (AB/2) with a step increase of 10 m or 20 m.The output resistivity data for each AB/2 distance was collected. The collected data are given in tabular form below as Table 5.6. In case of Damodar River the distance between current electrodes was increased gradually from 5 m to 50 m (AB/2) with a step increase of 5 m or 10 m and the collected data are given in Table 5.6 and Table 5.7.

Table 5.6 : Collection of VES data at site (Kangsabati River)

Location	VES Station	Latitude(N)	Longitude(E)	Elevation (m)	Sl. No.	AB/2(m)	Apparent Resistivity (Ω-m)	MN(m)
Kotloi-Barsimulia Village	1	23⁰18.137'	86⁰19.815'	220	1	10	111.6	2
					3	30	246.2	2
					5	50	342.6	2
					7	70	394.1	2
					9	90	418.5	2
					10	100	414.4	2
	2	23°18.151'	86°19.782'	221	1	10	107.3	2
					3	30	236.5	2
					5	50	360.2	2
					7	70	462.6	2
					9	90	458	2
					10	100	486.4	2
	3	23°18.141'	86°19.795'	219	1	10	112.8	2
					3	30	184.4	2
					5	50	360.2	2
					7	70	458	2
					9	90	570.9	2
					10	100	656.7	2
	4	23°18.155'	86°19.766'	220	1	10	117.4	2
					3	30	289.4	2
					5	50	387.34	2
					7	70	464.78	2
					9	90	704.4	2
					10	100	843.5	2
	5	23°18.158'	86°19.759'	222	1	10	112.8	2
					3	30	256.3	2
					5	50	427	2
					7	70	631	2
					9	90	631	2
					10	100	895.7	2

Location	VES Station	Latitude(N)	Longitude(E)	Elevation (m)	Sl. No.	AB/2 (m)	Apparent Resistivity (Ω-m)	MN(m)
Beldihi-Patrabera Village	1	23°18.425'	86°17.415'	244	1	10	112.92	2
					2	30	266.7	2
					3	50	501.2	2
					4	70	600.2	2
					5	90	758.49	2
					6	100	886.8	2
	2	23°18.389'	86°17.413'	247	1	10	103	2
					2	30	216.5	2
					3	50	387.8	2
					4	70	402.1	2
					5	90	558.3	2
					6	100	609.3	2
	3	23°18.367'	86°17.411'	244	1	10	124.6	2
					2	30	297.8	2
					3	50	511.3	2
					4	70	588.3	2
					5	90	718.7	2
					6	100	802.3	2
	4	23°18.347'	86°17.409'	243	1	10	127	2
					2	30	329.2	2
					3	50	462.6	2
					4	70	656.7	2
					5	90	697.4	2
					6	100	860.6	2
	5	23°18.325'	86°17.406'	245	1	10	142	2
					2	30	346	2
					3	50	516.5	2
					4	70	697.4	2
					5	90	690.5	2
					6	100	810.4	2

Location	VES Station	Latitude (N)	Longitude (E)	Elevation (m)	Sl. No.	AB/2 (m)	Apparent Resistivity (Ω-m)	MN (m)
Jhoriyadih-Hensla Village	1	23°18'21.91"	86°16'40.16"	235	1	10	131.9	2
					2	30	325.9	2
					3	50	482.6	2
					4	70	636.5	2
					5	90	755.6	2
					6	100	778.7	2
	2	23°18'21.39"	86°16'50"	235	1	10	110.70	2
					2	30	241.3	2
					3	50	340.1	2
					4	70	475.6	2
					5	90	554	2
					6	100	643.7	2
	3	23°18'20.82"	86°16'50.52"	235	1	10	112.4	2
					2	30	243.3	2
					3	50	360.6	2
					4	70	526.7	2
					5	90	612.3	2
					6	100	656.7	2
	4	23°18'20.22"	86°16'51.18"	234	1	10	129.6	2
					2	30	273.6	2
					3	50	438.9	2
					4	70	576.3	2
					5	90	760	2
					6	100	807.8	2
	5	23°18'19.63"	86°16'51.76"	236	1	10	127.6	2
					2	30	265.4	2
					3	50	388.5	2
					4	70	560.2	2
					5	90	672.7	2
					6	100	807.8	2

Table 5.7 : Collection of VES data at site (Damodar River)

Location	VES Station	Latitude (N)	Longitude (E)	Elevation (m)	Sl. No.	AB/2(m)	Apparent Resistivity (Ω-m)	MN(m)
Santuria Block	1	23°39.010'	86°53.298'	87	1	5	136.31	2
					2	10	116.6	2
					4	20	124	2
					6	30	149.1	2
					8	40	179.2	2
					10	50	229.2	2
	2	23°39.009'	86°53.067'	93	1	5	62.99	2
					2	10	54.12	2
					4	20	67.09	2
					6	30	80.66	2
					8	40	88.44	2
					10	50	91.2	2
	3	23°38.981'	86°53.321'	72	1	5	127.94	2
					2	10	113.1	2
					4	20	78.22	2
					6	30	71.48	2
					8	40	73.57	2
					10	50	67	2
	4	23°38.95'	86°53.274'	75	1	5	109.62	2
					2	10	94.05	2
					4	20	88.44	2
					6	30	96.98	2
					8	40	120.2	2
					10	50	144.5	2
	5	23°38.991'	86°53.296'	74	1	5	80.93	2
					2	10	79.54	2
					4	20	98.37	2
					6	30	131.84	2
					8	40	179.29	2
					10	50	190.57	2

Location	VES Station	Latitude (N)	Longitude (E)	Elevation (m)	Sl. No.	AB/2(m)	Apparent Resistivity (Ω-m)	MN(m)
Saturia Block	1	23°40.536'	86°51.087'	80	1	5	112.92	2
					2	10	98.85	2
					3	20	293.46	2
					4	30	578.96	2
					5	40	758.49	2
					6	50	45.89	2
	2	23°40.550'	86°51.074'	78	1	5	4258.50	2
					2	10	4357.69	2
					3	20	6574.89	2
					4	30	5128.78	2
					5	40	5589.51	2
					6	50	159.83	2
	3	23°40.508'	86°51.091'	77	1	5	185.96	2
					2	10	234.79	2
					3	20	1846.83	2
					4	30	2597.75	2
					5	40	1957.34	2
					6	50	269.78	2
	4	23°40.534'	86°51.085'	75	1	5	1578.56	2
					2	10	2386.74	2
					3	20	6184.62	2
					4	30	7254.98	2
					5	40	7486.49	2
					6	50	1967.53	2
	5	23°40.527'	86°51.02'	83	1	5	5487.62	2
					2	10	5246.54	2
					3	20	5672.28	2
					4	30	5937.73	2
					5	40	158.94	2
					6	50	6587.56	2

Location	VES Station	Latitude (N)	Longitude (E)	Elevation (m)	Sl. No.	AB/2(m)	Apparent Resistivity (Ω-m)	MN(m)
Nituria Block	1	23°40.955'	86°48.209	86	1	5	159.58	2
					2	10	198.64	2
					3	20	96.48	2
					4	30	84.34	2
					5	40	6348	2
					6	50	1984.87	2
	2	23°40.963'	86°48.185	89	1	5	187.53	2
					2	10	264.78	2
					3	20	45.67	2
					4	30	76.97	2
					5	40	987.12	2
					6	50	2384.67	2
	3	23°40.919'	86°48.232'	90	1	5	176.92	2
					2	10	264.78	2
					3	20	78.96	2
					4	30	67.54	2
					5	40	1578.64	2
					6	50	2457.68	2
	4	23°40.917'	86°48.195'	89	1	5	213.76	2
					2	10	198.64	2
					3	20	97.64	2
					4	30	576.35	2
					5	40	967.64	2
					6	50	6587.94	2
	5	23°40.911'	86°48.179'	88	1	5	259.46	2
					2	10	286.64	2
					3	20	184.67	2
					4	30	157.64	2
					5	40	4587.69	2
					6	50	5183.49	2

Location	VES Station	Latitude (N)	Longitude (E)	Elevation (m)	Sl. No.	AB/2(m)	Apparent Resistivity (Ω-m)	MN(m)
Nituria Block	1	23°40.731'	86°46.910'	76	1	5	95.86	2
					2	10	93.74	2
					4	20	124.68	2
					6	30	76.94	2
					8	40	89.47	2
					10	50	467.39	2
	2	23°40.732'	86°46.889'	79	1	5	102.95	2
					2	10	101.49	2
					4	20	59.38	2
					6	30	93.69	2
					8	40	186.94	2
					10	50	378.95	2
	3	23°40.718'	86°46.938'	77	1	5	105.47	2
					2	10	102.48	2
					4	20	99.53	2
					6	30	67.39	2
					8	40	156.98	2
					10	50	397.86	2
	4	23°40.729'	86°46.977'	78	1	5	87.59	2
					2	10	86.74	2
					4	20	164.38	2
					6	30	67.83	2
					8	40	82.35	2
					10	50	346.83	2
	5	23°35.574'	86°43.521'	81	1	5	157.38	2
					2	10	155.58	2
					4	20	86.43	2
					6	30	153.37	2
					8	40	267.38	2
					10	50	146.25	2

Analysis of Data:

The interpretation of the VES data was conducted using the Resist software IPIwin which produces the resistivity model (resistivity, thickness and depth) fitting the aquifer field data with the least RMS-error between the observed and calculated resistivity. The iteration was performed until the fitting error between field data and theoretical model curve fell less than 10%. Using the software, the variation of resistivity of both the observed and the modelled resistive layer with depth at each point is plotted and shown below. With the variation of depth, the variation of conductivity up to 50m depth for Kangsabati and 25m depth for Damodar at each point is also shown below. The maximum layer information below ground level found up to 1/3 to ½ of the observed AB/2 distance.

Discussion:

- Pseudo cross-sections: Five points have been selected for sounding in each site to take sounding data. The GPS data to locate the points are given above in the tables. The [oint has been denoted here by S_i/P_j (Where, is site number and j is point number). Using the data's of five sounding points of each site a cross-section of resistivity variation with depth is created, is called Pseudo cross-section. Here it has been taken the 1st point as reference and the distance between the 2nd, 3rd, 4thand 5th point from the 1st point is 30m, 50m, 90m and 120m respectively. In the pseudo cross-sections shown below, the top horizontal ruler represents the name of the sounding points, while the bottom represents the co-ordinates of the sounding points. Colour scale columns are displayed beside the cross-sections for apparent resistivity. In Figure 5.9 and Figure 5.10all the four Pseudo cross-sections for the four sites are shown below.

- Resistivity Variation Curve: The variation of resistivity in each 20 points of 4 sites with the variation of current electrode separation distance AB/2 have been plotted during the software interpretation of VES field data. The curves shows both the observed resistivity variation with AB/2 in field (shown using black line) and the interpreted model resistivity variation with vertical depth layer wise (shown using red line). The blue line shown in the curves can be used to customize the layer wise resistivity variation during the software interpretation. All the 20 curves for 20 points are shown in Figure 5.11, Figure 5.12.

- Resistivity Cross-Section and Conductance Variation: The four resistivity cross-sections shown below in Figure 5.11 shows the vertical layer wise resistivity variation of each point. Each cross-section for each site shows the vertical layer wise resistivity variation for the five points taken at that site. To clarify the proper values the table of vertical layer wise resistivity created by the software is also shown for each point. Figure 5.13, Figure 5.14, Figure 5.15, Figure 5.16, Figure 5.17, Figure 5.18 shows the variation of conductance up to vertical depth 30m below the ground level for each site.

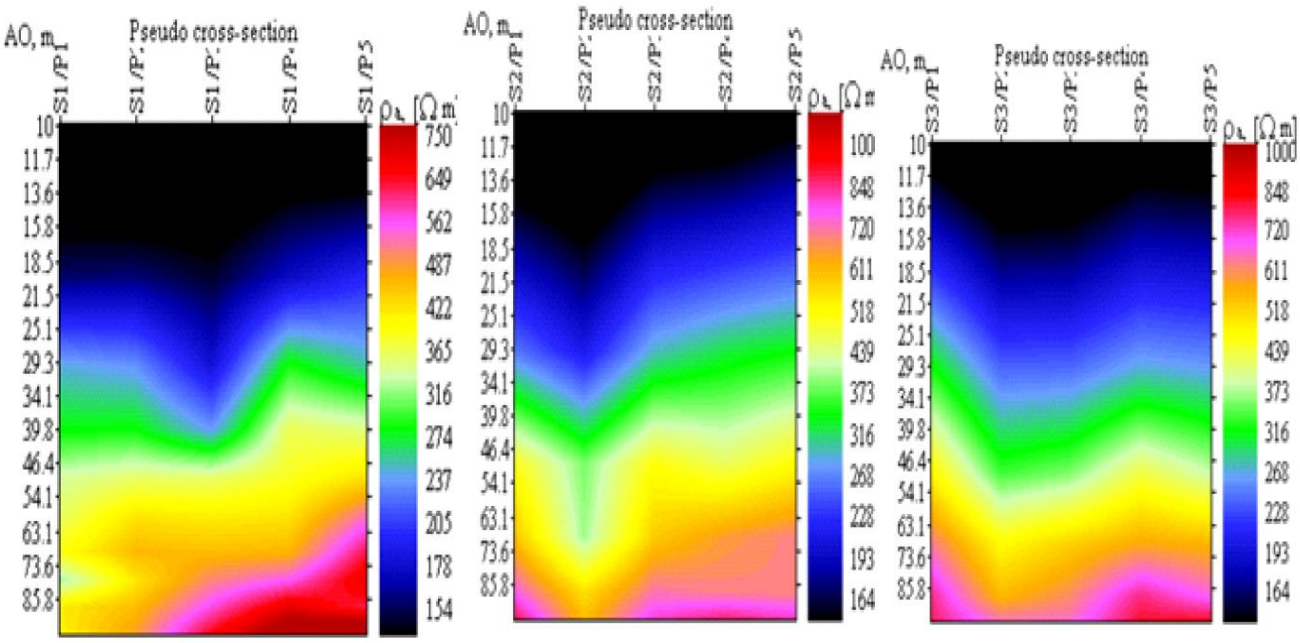


Figure 5.9: Pseudo cross-sections of the Soundings I to III (Kangsabati river)

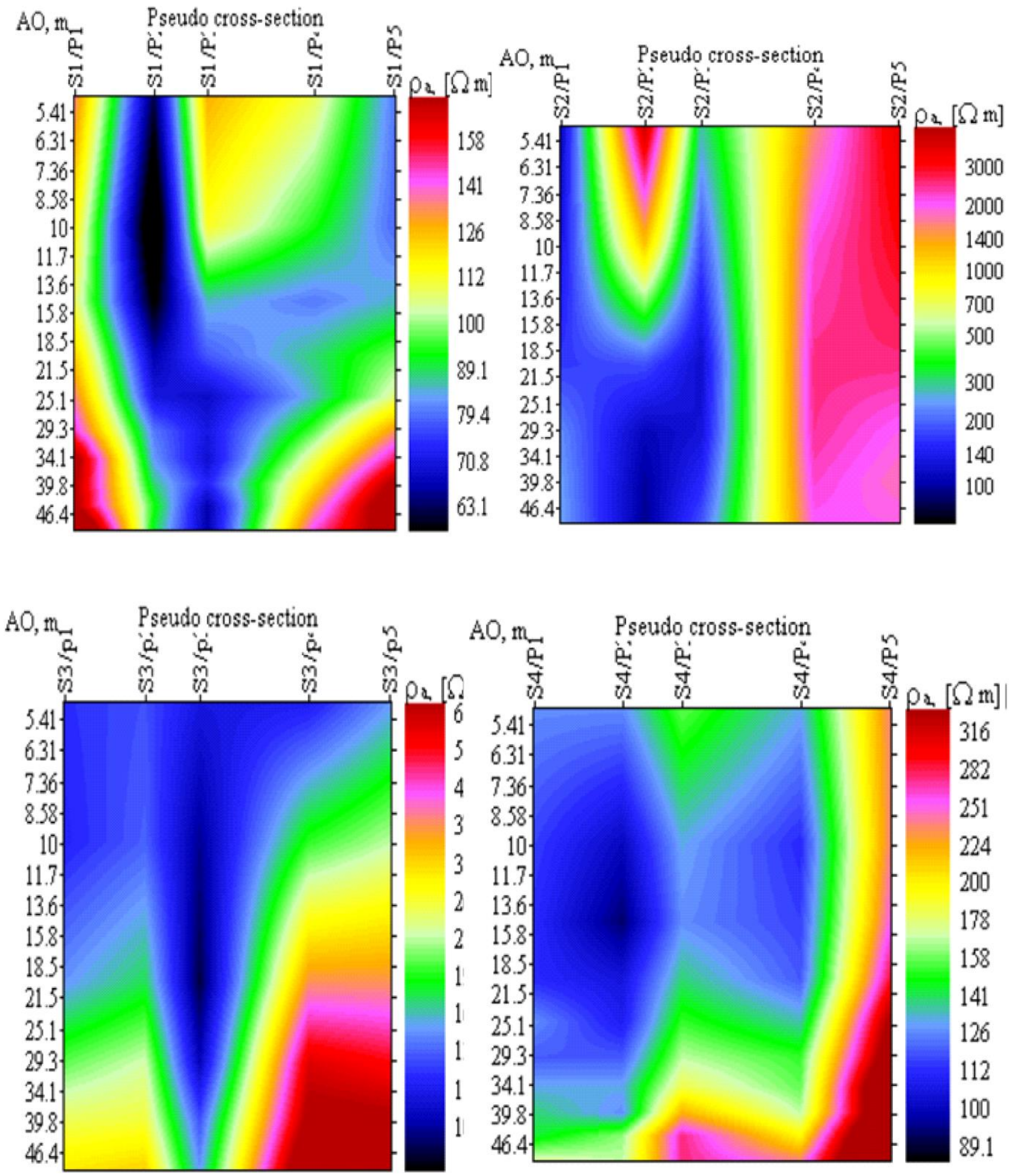


Figure 5.10: Pseudo cross-sections of the Soundings I to IV

(Damodar river)

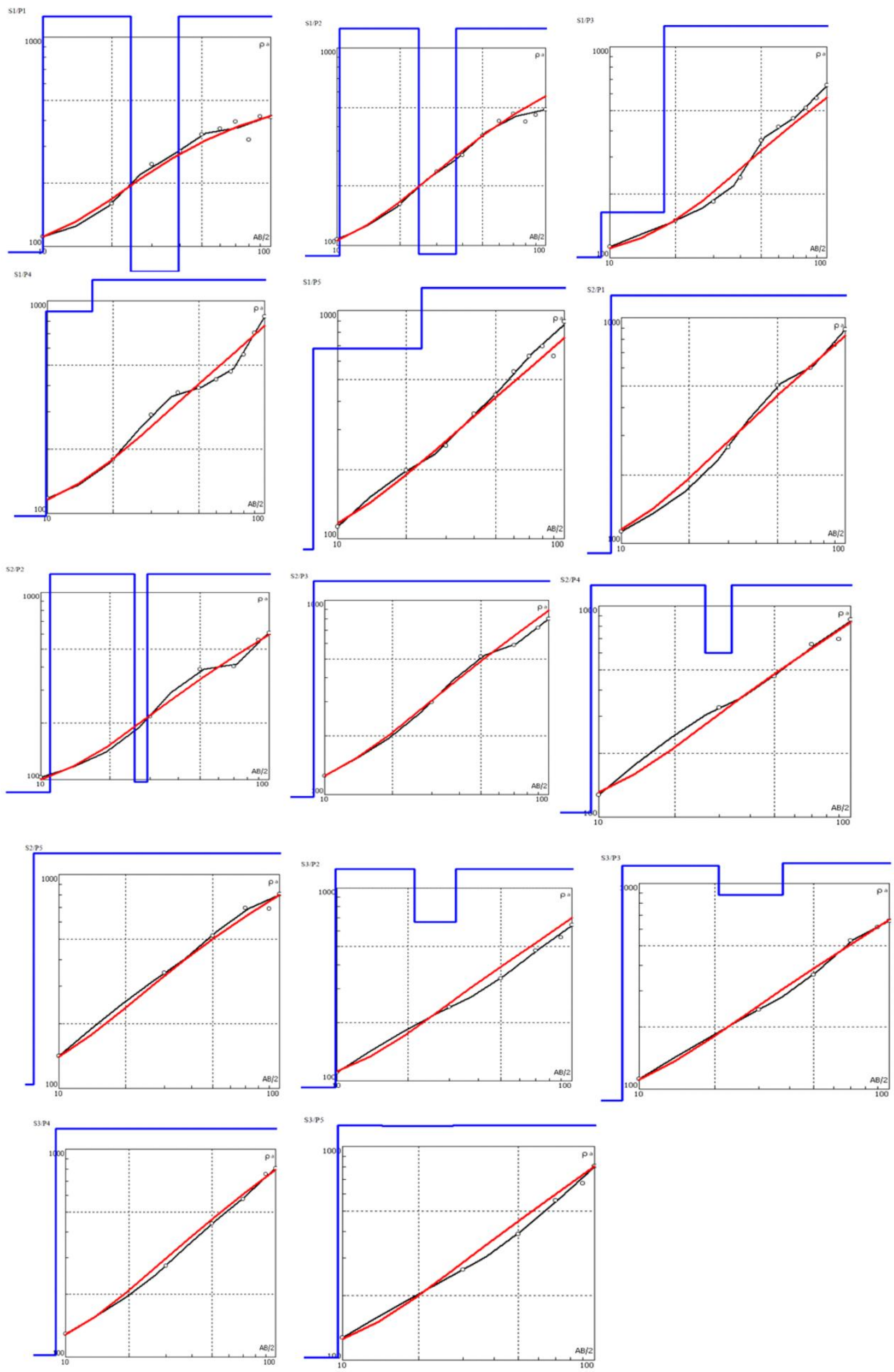


Figure 5.11: Resistivity Variation of the Points in Kangsabati river

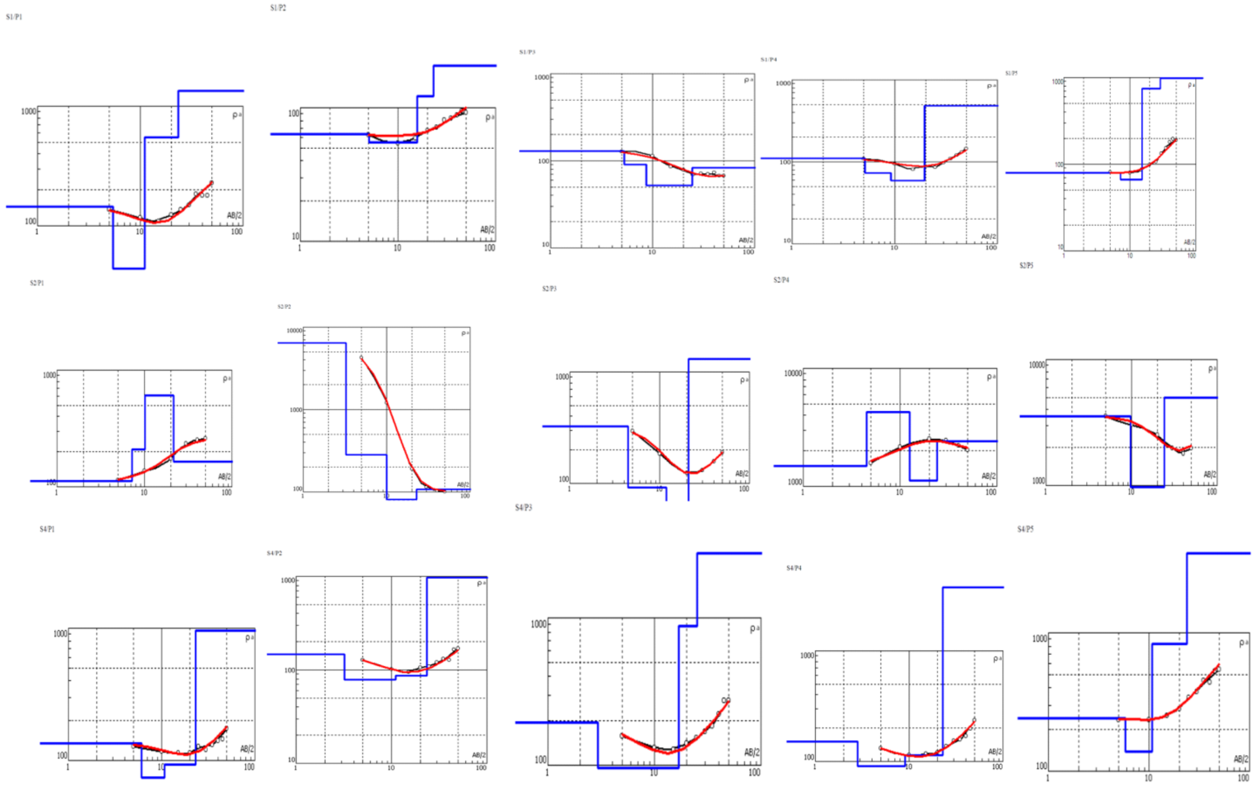


Figure 5.12: Resistivity Variation of the points in Damodar river

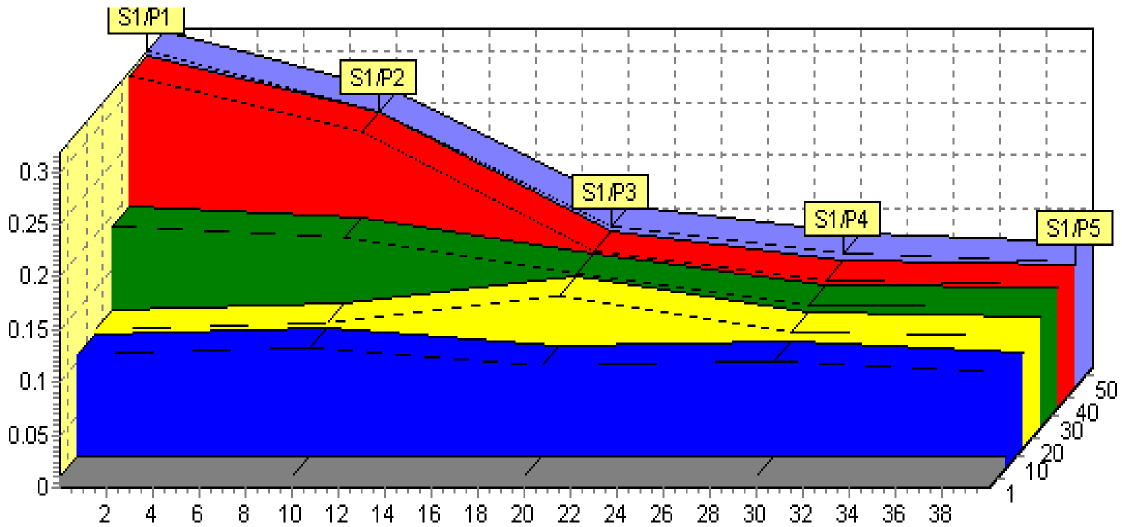


Figure 5.13: Variation of conductance up to 50 m depth in Site 1-(Kangsabati)

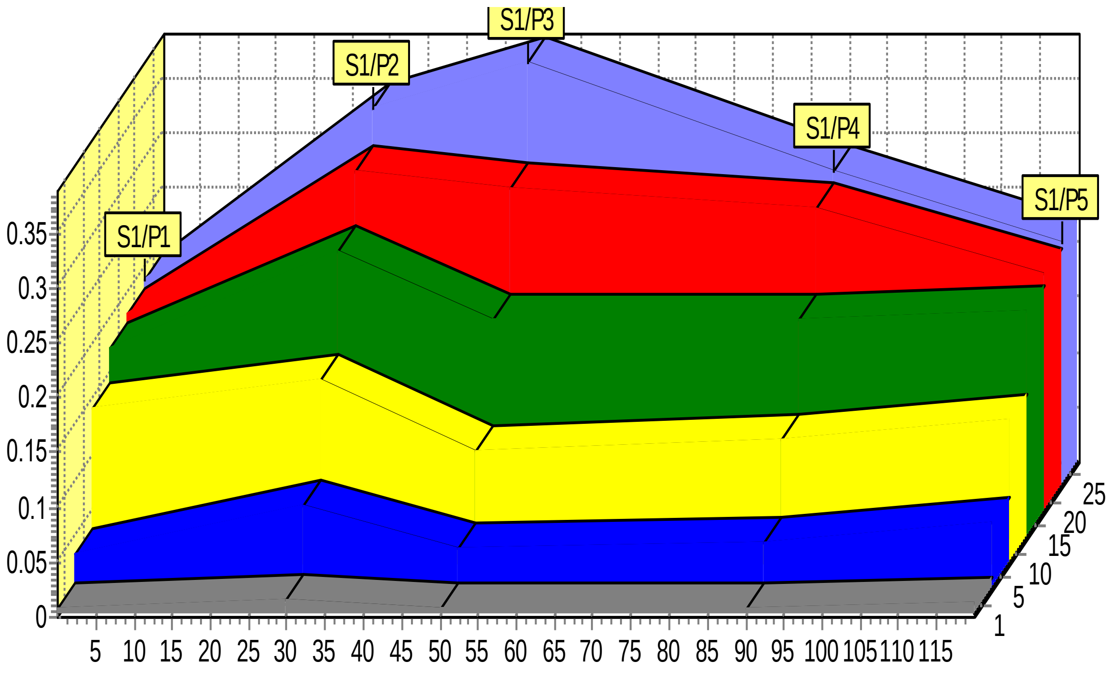


Figure 5.14: Variation of conductance up to 25 m depth in Site 1-Damodar

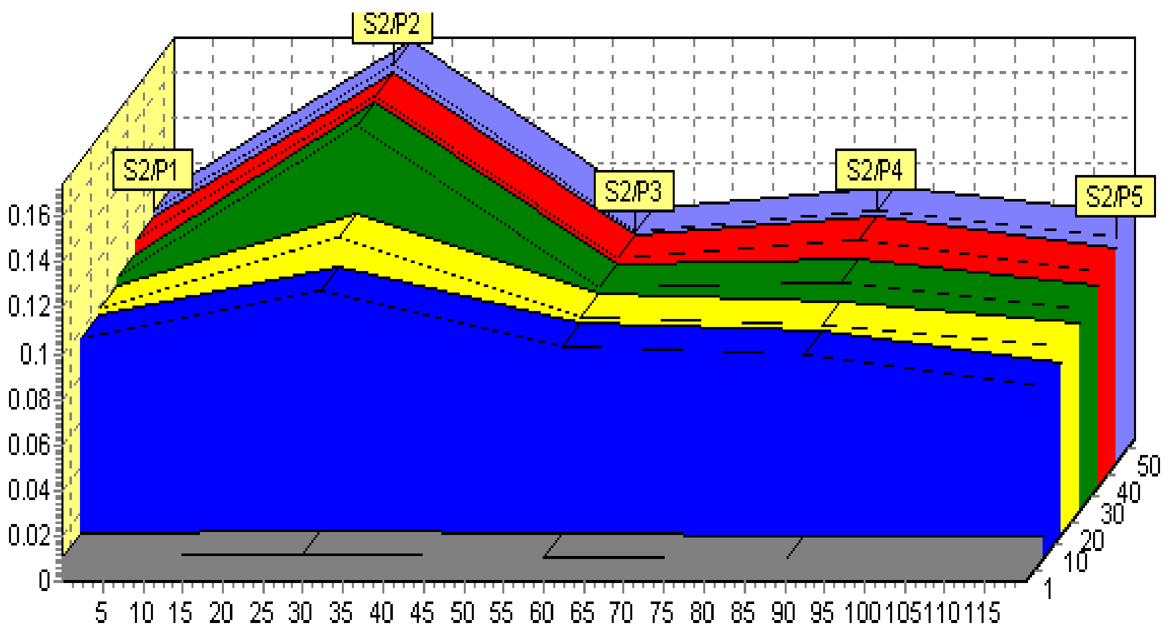


Figure 5.15: Variation of conductance up to 50 m depth in Site 2-Kangsabati

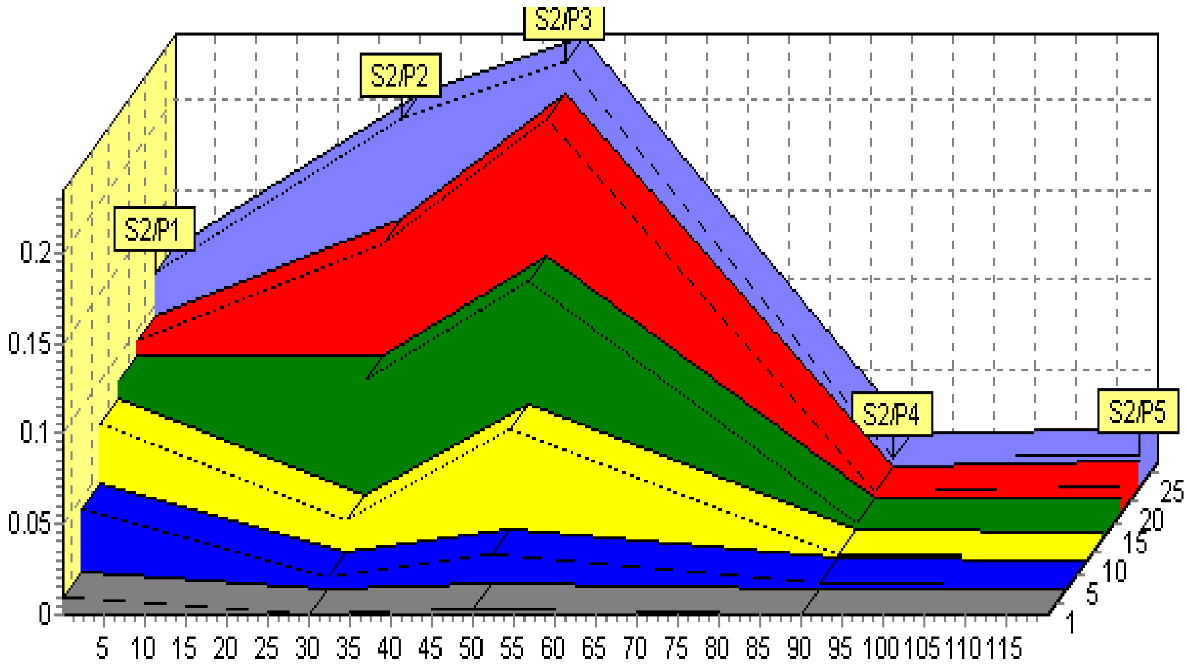


Figure 5.16: Variation of conductance up to 25 m depth in Site 2-Damodar

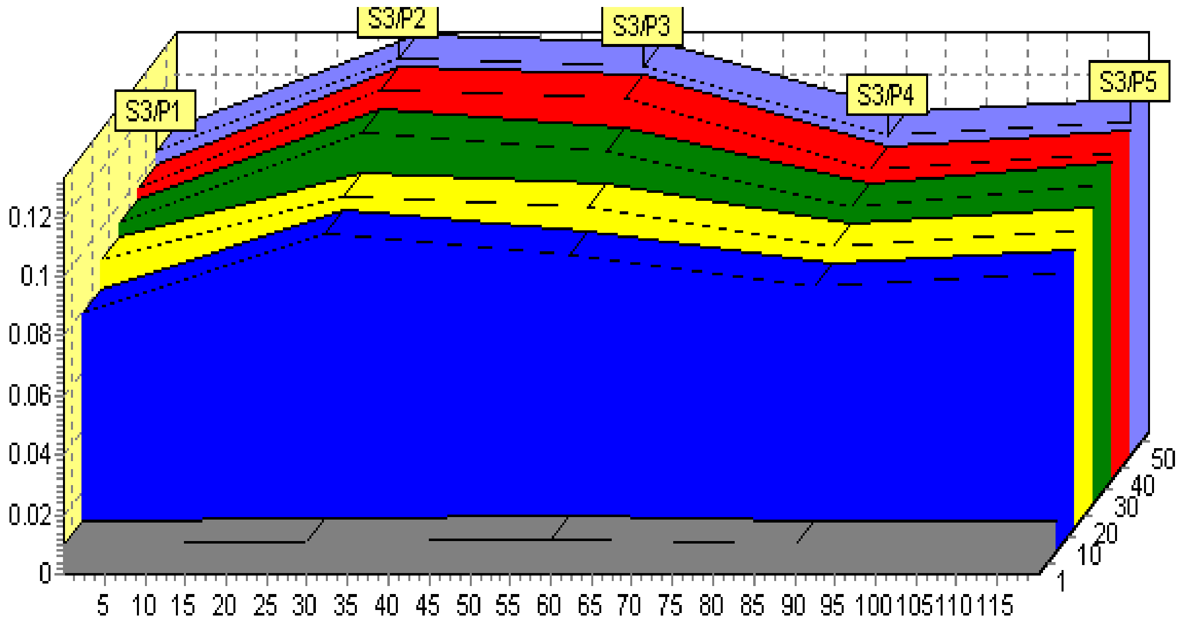


Figure 5.17: Variation of conductance up to 50 m depth in Site 3-Kangsabati

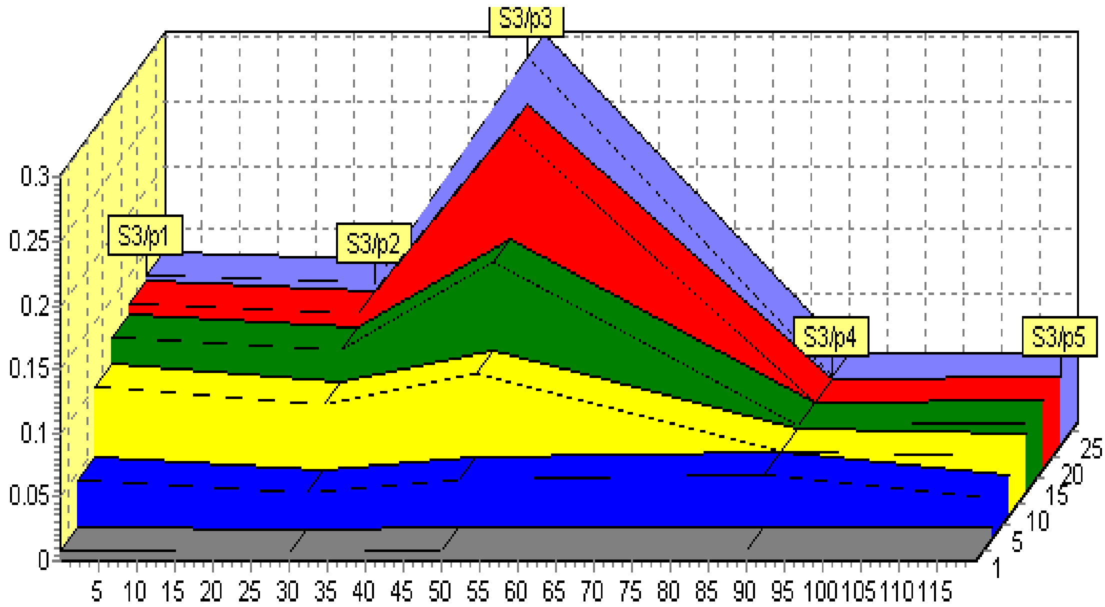


Figure 5.18: Variation of conductance up to 25 m depth in Site 3-Damodar

- **Total Transverse Resistance:**

The total transverse resistance (T) is one of the parameters used to define target areas of good groundwater potential. It has a direct relation with transmissivity and the highest T values reflect most likely the highest transmissivity values of the aquifers or aquiferous zones and vice versa. The total transverse resistance (T) for each geo-electric sounding (VES) stations iscomputed below in tabular form from the relation:

$\mathbf{T = \Sigma h_i \rho_i = h_1 \rho_1 + h_2 \rho_2 + ...+ h_n \rho_n}$........................5.1

Where T is the total transverse resistance, h_i is the thickness of the ith layer and ρ_i is the resistivity of the ith layer.

- **Total Longitudinal Conductance:**

The total longitudinal conductance (S) is also a parameter used to define target areas of groundwater potential. High S values usually indicate relatively thick succession and should be accorded the highest priority in terms of groundwater potential and vice versa.The total longitudinal conductance (S) for each of geo-electric sounding (VES) stations is computed below in tabular form from the relation:

S = Σ (hi / ρi) = h1 / ρ1 + h2 / ρ2 +. . ...+ hn / ρn..............................5.2

Where S is the total longitudinal conductance, h_i is the thickness of the ith layer and ρ_i is the resistivity of the ith layer.

Table 5.8: Transverse resistance and longitudinal conductance for Kangsabati

Site Location	Points	LAYER RESISTIVITY (Ω-m)	Thickness (m)	Total Transverse Resistance of Layers (Ω-m²)	Total Transverse Resistance of Each Point (Ω-m²)	Total Longitudinal Conductance of Layers (Mho)	Total Longitudinal Conductance of Each Point (Mho)
Kotloi-Barsimulia Village	S1/P1	94	10	940	52831.88	0.10638	0.31897
		1716	14.3	24538.8		0.00833	
		75.8	15.1	1144.58		0.19921	
		2279	11.5	26208.5		0.00505	
	S1/P2	88.7	10.3	913.61	144202.79	0.11612	0.26205
		5062	14.3	72386.6		0.00282	
		90.4	12.7	1148.08		0.14049	
		5167	13.5	69754.5		0.00261	
	S1/P3	101	9.1	919.1	121935.05	0.09010	0.15103
		165	8.63	1423.95		0.05230	
		5448	13.2	71913.6		0.00242	
		2772	17.2	47678.4		0.00620	
	S1/P4	97.4	9.83	957.442	66035.342	0.10092	0.12497
		892	6.3	5619.6		0.00706	
		2018	14.8	29866.4		0.00733	
		1751	16.9	29591.9		0.00965	
	S1/P5	90	7.79	701.1	101193.2	0.08656	0.11801
		679	15.6	10592.4		0.02297	
		1522	2.9	4413.8		0.00191	
		3607	23.7	85485.9		0.00657	

Site Location	Points	LAYER RESISTIVITY (Ω-m)	Thickness (m)	Total Transverse Resistance of Layers (Ω-m^2)	Total Transverse Resistance of Each Point (Ω-m^2)	Total Longitudinal Conductance of Layers (Mho)	Total Longitudinal Conductance of Each Point (Mho)
Beldihi-Patrabera Village	S2/P1	90.5	8.99	813.595	118068.275	0.09934	0.11255
		4157	19.9	82724.3		0.00479	
		1799	10.9	19609.1		0.00606	
		2512	5.94	14921.28		0.00236	
	S2/P2	85.7	11	942.7	148340.525	0.12835	0.17369
		5752	14.7	84554.4		0.00256	
		97.5	3.59	350.025		0.03682	
		3238	19.3	62493.4		0.00596	
	S2/P3	97.4	8.93	869.782	190574.262	0.09168	0.10100
		3556	11.3	40182.8		0.00318	
		4086	19.6	80085.6		0.00480	
		7188	9.66	69436.08		0.00134	
	S2/P4	105	9.29	975.45	117492.99	0.08848	0.10912
		3346	17.2	57551.2		0.00514	
		599	7.26	4348.74		0.01212	
		4016	13.6	54617.6		0.00339	
	S2/P5	104	7.69	799.76	80882.18	0.07394	0.09855
		1594	18.8	29967.2		0.01179	
		1763	17.1	30147.3		0.00970	
		2534	7.88	19967.92		0.00311	
Jhoriyadih-Hensla Village	S3/P1	97.6	7.57	738.832	76951.182	0.07756	0.10020
		2608	1.28	3338.24		0.00049	
		929	9.59	8909.11		0.01032	
		2326	27.5	63965		0.01182	
	S3/P2	92.5	9.84	910.2	72619.9	0.10638	0.13214
		2504	11.6	29046.4		0.00463	
		669	10.7	7158.3		0.01599	
		2630	13.5	35505		0.00513	
	S3/P3	87.7	8.55	749.835	79851.635	0.09749	0.13016
		1217	12.3	14969.1		0.01011	
		877	16.7	14645.9		0.01904	
		3749	13.2	49486.8		0.00352	
	S3/P4	102	8.99	916.98	92318.16	0.08814	0.10664
		2091	6.48	13549.68		0.00310	
		1849	20.7	38274.3		0.01120	
		3068	12.9	39577.2		0.00420	
	S3/P5	103	9.57	985.71	113186.32	0.09291	0.11126
		2567	4.83	12398.61		0.00188	
		1249	13	16237		0.01041	
		3714	22.5	83565		0.00606	

Table 5.9: transverse resistance and longitudinal conductance for Damodar

Site Location	Points	LAYER RESISTIVITY (Ω-m)	Thickness (m)	Total Transverse Resistance of Layers (Ω-m^2)	Total Transverse Resistance of Each Point (Ω-m^2)	Total Longitudinal Conductance of Layers (Mho)	Total Longitudinal Conductance of Each Point (Mho)
Saturi Block	S1/P1	144	5.42	780.48	7938.012	0.03764	0.18914
		43.6	5.62	245.032		0.12890	
		553	12.5	6912.5		0.02260	
	S1/P2	63.1	5.07	319.917	1800.057	0.08035	0.33754
		54.7	10.8	590.76		0.19744	
		122	7.29	889.38		0.05975	
	S1/P3	130	5.24	681.2	1804.056	0.04031	0.37973
		90.4	3.39	306.456		0.03750	
		52	15.7	816.4		0.30192	
	S1/P4	110	5.13	564.3	1477.596	0.04664	0.27835
		73.2	4.08	298.656		0.05574	
		59.1	10.4	614.64		0.17597	
	S1/P5	79.7	7.2	573.84	11385.606	0.09034	0.23056
		66.9	8.14	544.566		0.12167	
		744	13.8	10267.2		0.01855	
Saturi Block	S2/P1	112	7.2	806.4	8456.6	0.06429	0.09725
		210	2.97	623.7		0.01414	
		611	11.5	7026.5		0.01882	
	S2/P2	6414	3.27	20973.78	23904.6	0.00051	0.18299
		282	6.73	1897.86		0.02387	
		80.7	12.8	1032.96		0.15861	
	S2/P3	327	4.47	1461.69	2743.833	0.01367	0.23146
		92.1	7.32	674.172		0.07948	
		66.3	9.17	607.971		0.13831	
	S2/P4	1484	4.54	6737.36	53822.88	0.00306	0.01545
		4252	7.96	33845.92		0.00187	
		1122	11.8	13239.6		0.01052	
	S2/P5	3548	9.66	34273.68	48575.309	0.00272	0.01778
		968	14.5	14036		0.01498	
		1807	0.147	265.629		0.00008	

Site Location	Points	LAYER RESISTIVITY (Ω-m)	Thickness (m)	Total Transverse Resistance of Layers (Ω-m^2)	Total Transverse Resistance of Each Point (Ω-m^2)	Total Longitudinal Conductance of Layers (Mho)	Total Longitudinal Conductance of Each Point (Mho)
Nituria Block	S3/P1	134	3.05	408.7	8613.316	0.02276	0.13041
		90.6	8.36	757.416		0.09227	
		696	10.7	7447.2		0.01537	
	S3/P2	141	6.52	919.32	7410.273	0.04624	0.12615
		87.7	4.89	428.853		0.05576	
		501	12.1	6062.1		0.02415	
	S3/P3	120	4.2	504	1718.728	0.03500	0.30211
		76.9	9.92	762.848		0.12900	
		57.2	7.9	451.88		0.13811	
	S3/P4	124	1.75	217	118810.552	0.01411	0.05307
		95.2	3.26	310.352		0.03424	
		5012	23.6	118283.2		0.00471	
	S3/P5	122	3.16	385.52	59041.9	0.02590	0.05362
		349	9.02	3147.98		0.02585	
		5442	10.2	55508.4		0.00187	
Nituria Block	S4/P1	134	6.12	820.08	2327.424	0.04567	0.24078
		74.4	4.66	346.704		0.06263	
		93.6	12.4	1160.64		0.13248	
	S4/P2	146	3.21	468.66	2156.902	0.02199	0.26713
		78.1	7.82	610.742		0.10013	
		86.2	12.5	1077.5		0.14501	
	S4/P3	196	2.96	580.16	9095.59	0.01510	0.17150
		95.2	14	1332.8		0.14706	
		877	8.19	7182.63		0.00934	
	S4/P4	151	2.82	425.82	2552.964	0.01868	0.20773
		90.6	6.24	565.344		0.06887	
		114	13.7	1561.8		0.12018	
	S4/P5	243	5.82	1414.26	13111.8	0.02395	0.07472
		139	4.86	675.54		0.03496	
		835	13.2	11022		0.01581	

Remarks: From the interpreted results and according to the standard resistivity scale can say that there are many layers in the selected points which have layers of resistivity having value between 20-100 Ω-m. That means those layers probably be the source of fresh water. All the observed points consist of layers having resistivity value between 20-100 Ω-m except S2/P4, S2/P5, S4/P5. But out of these points S1/P2, S1/P3, S3/P3 have the highest ground water potential as these points have longitudinal conductance greater than 0.3, which are highest among all the other points. From the transverse resistance value, it can be commented that S2/P4, S2/P5, S3/P4, S3/P5 are the points having aquifer layers of highest transmissivity value. In case of Kangsabati River all the observed points have 20-100 Ω-m layers up to depth 8m-10m starting from the ground level and in some points, there are rocky layers after that. As the value of longitudinal conductance of points S1/P1, S2/P2 are the highest, that means those are the points having highest ground water potential. And S1/P2, S2/P2, S3/P3 have the highest transverse resistance value which indicates these points consists having aquifer of good transmissivity value.From the conductance variation curve depict that all the the points have conductance less than 0.5 dS/m upto 5o m depth of Kangsabati river and upto 25m depth of Damodar river which is under desirable limit of conductivity.

VES Survey in Ghutlia Village

For each VES Sounding, the distance between potential electrodes MN was kept fixed at 2 m and the distance between current electrodes was increased gradually from 10 m to 50 m (AB/2) with a step increase 20 m. The output resistivity data's for each AB/2 distance was collected. The collected data are given in tabular form below as Table 5.10. the field survey work is shown in Photo 5.2.

Photo 5.2: Geophysical Data collection using (VES) Instrument

Table 5.10 : Collection of VES data at site (near Ghutlia village)

Location	VES Station	Latitude (N)	Longitude (E)	Sl. No.	MN/2 (m)	AB/2 (m)	Apparent Resistivity (Ω-m)
Site 1	1	23°24'37.97"	86°37'00.36"	1	2	10	120.1
				2	2	30	234.9
				3	2	50	290.8
	2	23°24'35.64"	86°37'12.58"	1	2	10	109.6
				2	2	30	180
				3	2	50	211.1
Site 2	1	23°24'34.17"	86°36'49.22"	1	2	10	115.6
				2	2	30	214.4
				3	2	50	295.3
	2	23°24'48.29"	86°36'53.19"	1	2	10	111.3
				2	2	30	227.8
				3	2	50	286.4
Site 3	1	23°24'20.15"	86°36'59.44"	1	2	10	133.6
				2	2	30	238.5
				3	2	50	304.4
	2	23°24'26.48"	86°36'61.79"	1	2	10	125.7
				2	2	30	214.4
				3	2	50	245.9
Site 4	1	23°24'14.50"	86°36'56.14"	1	2	10	109.6
				2	2	30	181.3
				3	2	50	221
	2	23°24'26.76"	86°36'53.47"	1	2	10	114.7
				2	2	30	180
				3	2	50	214.4
Site 5	1	23°24'08.96"	86°36'54.68"	1	2	10	116.5
				2	2	30	214.4
				3	2	50	304.4
	2	23°24'19.79"	86°36'58.94"	1	2	10	107.9
				2	2	30	207.9
				3	2	50	286.4

Analysis of Data :

The interpretation of the VES data was conducted using the Resist software IPI2win which produces the resistivity model (resistivity, thickness and depth) fitting the aquifer field data with the least RMS-error between the observed and calculated resistivity. The iteration wasperformed until the fitting error between field data and theoretical model curve fell less than5%. Using the software, the variation of resistivity of both the observed and the modelled resistive layer with depth at each point is plotted and shown below. The maximum layer information below ground level found up to 1/3 to ½ of the observed AB/2 distance.

Results and Discussion:

- Pseudo cross-sections: Similarly it has been selected five points for sounding in each site to take sounding data. The GPS data to locate the points are given above in the tables. Again denotation of the points here Si/Pj (Where, is site number and j is point number). Using the data's of five sounding points of each site a cross-section of resistivity variation with depth is created, is called Pseudo cross-section. Here it was taken the 1st point as reference and the distance between the 2ndpoint from the 1st point is 10m. In the Pseudo cross-sections shown below, the top horizontal ruler represents the name of the sounding points, while the bottom represents the co-ordinates of the sounding points. Colour scale columns are displayed beside the cross-sections for apparent resistivity. In Figure 5.19all the five Pseudo cross-sections for the fivesites are shown below.

- Resistivity Variation Curve: The variation of resistivity in each 10 points of 5 sites with the variation of current electrode separation distance AB/2 have been plotted during the software interpretation of VES field data. The curves shows both the observed resistivity variation with AB/2 in field (shown using black line) and the interpreted model resistivity variation with vertical depth layer wise (shown using red line). The blue line in the curves shows corresponding resistivity and thickness of layers. All the 10 curves for 10 points are shown in Figure 5.16below.

- Resistivity Cross-Section and Conductance Variation: The five resistivity cross-sections shown below in Figure 5.20shows the vertical layer wise resistivity variation of each point partly for each site. Each cross-section for each site shows the vertical layer wise resistivity variation for the two points taken at that site. To clarify the proper values the table of vertical layer wise resistivity created by the software is also shown for each point. Figure 5.22 shows the variation of conductance up to vertical depth 20m below the ground level partly for each five site.

Photo 5.3 : Conduction of VES survey in River bed

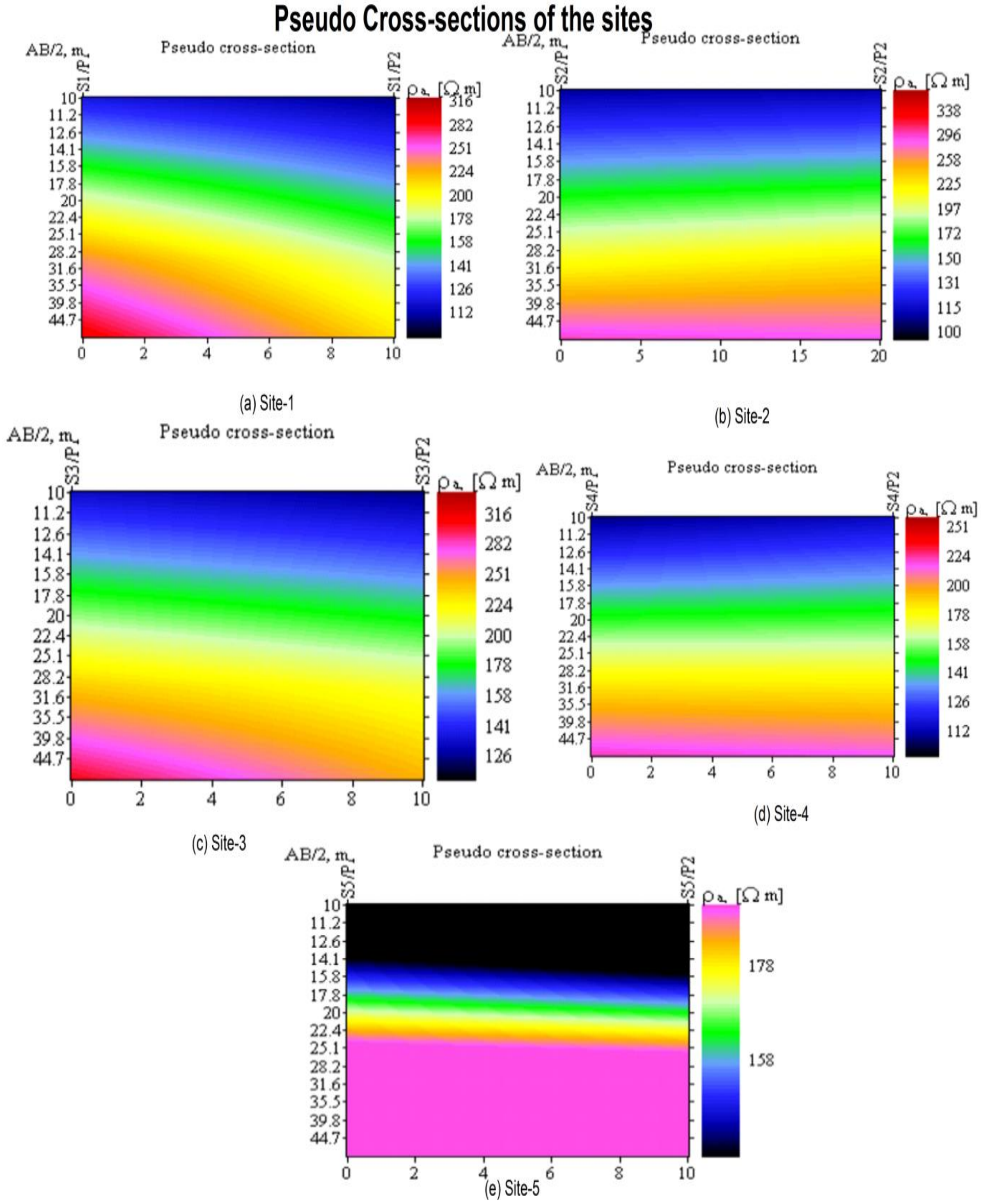


Figure 5.19 : Pseudo Cross-sections of the five sites

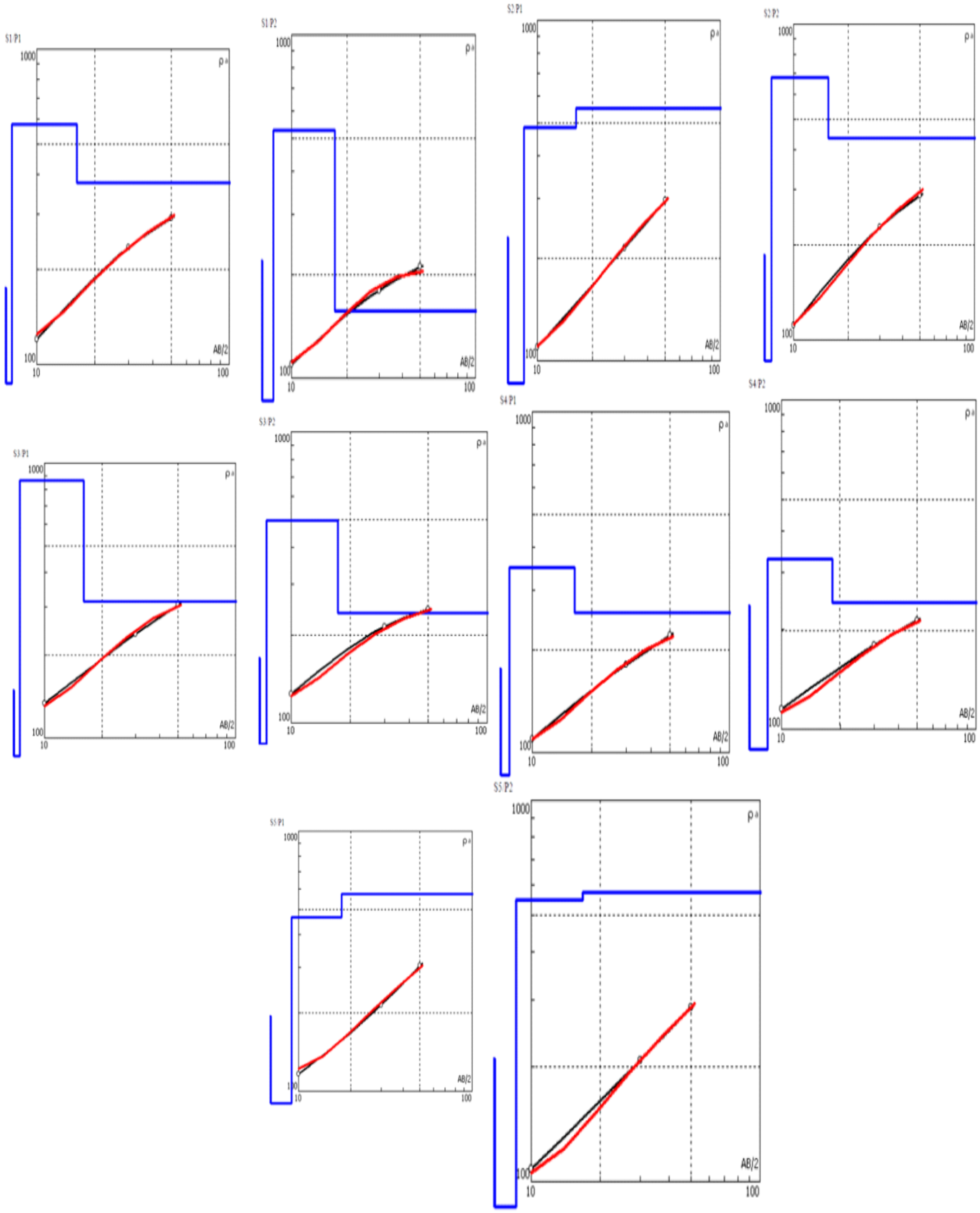
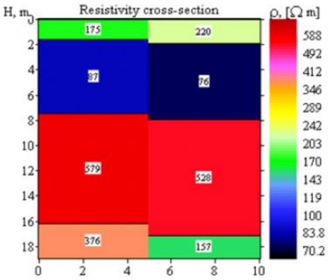


Figure 5.20 : Resistivity variation Curves



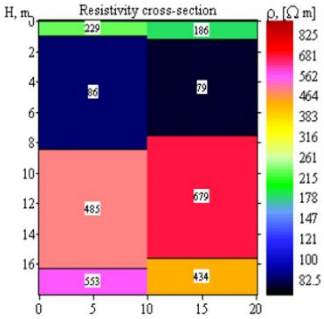
RMS=1.65%

N	1	2	3	4
ρ	175	87	579	376
h	1.6	5.84	8.77	
d	1.6	7.44	16.2	
Alt	-1.6	-7.445	-16.22	

RMS=2.4%

N	1	2	3	4
ρ	220	76	528	157
h	1.9	6.05	9.22	
d	1.9	7.95	17.2	
Alt	-1.9	-7.95	-17.17	

(a) Site-1



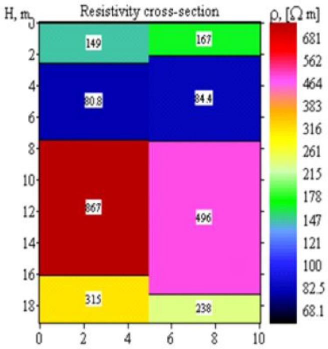
RMS=1.39%

N	1	2	3	4
ρ	229	86	485	553
h	1	7.45	7.86	
d	1	8.45	16.3	
Alt	-1	-8.449	-16.31	

RMS=2.47%

N	1	2	3	4
ρ	186	79	679	434
h	1.2	6.34	8.07	
d	1.2	7.54	15.6	
Alt	-1.2	-7.54	-15.61	

(b) Site-2



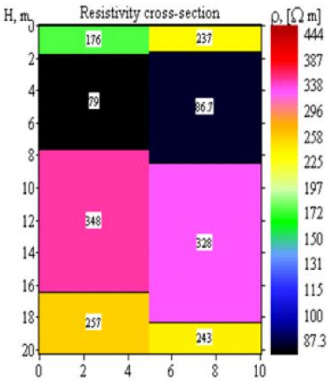
RMS=2.56%

N	1	2	3	4
ρ	149	80.8	867	315
h	2.6	4.85	8.64	
d	2.6	7.45	16.1	
Alt	-2.6	-7.45	-16.09	

RMS=3.02%

N	1	2	3	4
ρ	167	84.4	496	238
h	2.1	5.41	9.81	
d	2.1	7.51	17.3	
Alt	-2.1	-7.51	-17.32	

(c) Site-3



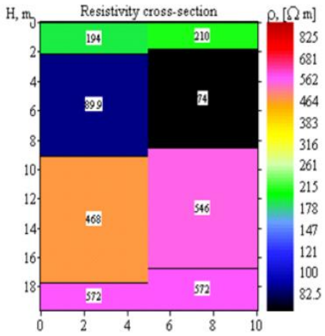
RMS=1.43%

N	1	2	3	4
ρ	176	79	348	257
h	1.8	5.86	8.78	
d	1.8	7.66	16.4	
Alt	-1.8	-7.66	-16.44	

RMS=2.96%

N	1	2	3	4
ρ	237	86.7	328	243
h	1.6	6.89	9.85	
d	1.6	8.49	18.3	
Alt	-1.6	-8.49	-18.34	

(d) Site-4



RMS=2.6%

N	1	2	3	4
ρ	194	89.9	468	572
h	2.1	7.04	8.61	
d	2.1	9.14	17.7	
Alt	-2.1	-9.138	-17.75	

RMS=3.48%

N	1	2	3	4
ρ	210	74	546	572
h	1.8	6.76	8.2	
d	1.8	8.56	16.8	
Alt	-1.8	-8.558	-16.76	

(e) Site-5

Figure 5.21 (a-e) :Resistivity Cross-Sections and Resistivity Attribute

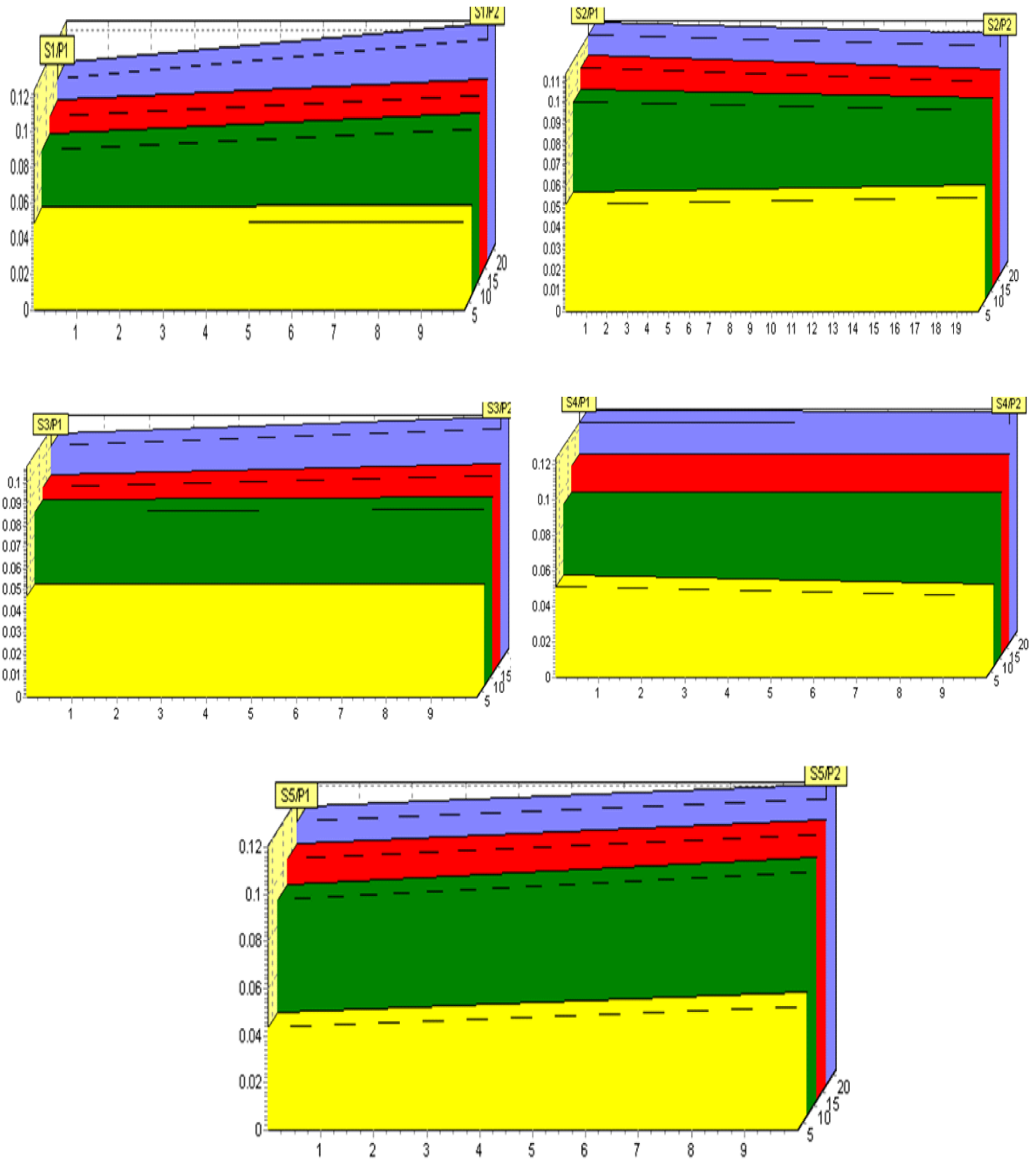


Figure 5.22 :Variation of conductance up to 20 m depth at Site 1to site V

- **Total Transverse Resistance:**

The total transverse resistance (T) is one of the parameters used to define target areas ofgood groundwater potential. It has a direct relation with transmissivity and the highest T valuesreflect most likely the highest transmissivity values of the aquifers or aquiferous zones and vice versa. The total transverse resistance (T) for each geo-electric sounding (VES) stations is computed below in tabular form from the relation:

$$\mathbf{T = \Sigma h_i \rho_i = h_1 \rho_1 + h_2 \rho_2 + ...+ h_n \rho_n} \text{.. } \mathbf{5.3}$$

Where T is the total transverse resistance, h_i is the thickness of the i^{th} layer and ρ_i is the resistivity of the ith layer.

- **Total Longitudinal Conductance:**

The total longitudinal conductance (S) is also a parameter used to define target areas ofgroundwater potential. High S values usually indicate relatively thick succession and should beaccorded the highest priority in terms of groundwater potential and vice versa. The total longitudinal conductance (S) for each of geo-electric sounding (VES) stations iscomputed below in tabular form from the relation:

$$S = \Sigma (hi / \rho i) = h1 / \rho 1 + h2 / \rho 2 +. . ...+ hn / \rho n \text{............................} 5.4$$

Where S is the total longitudinal conductance, h_i is the thickness of the ith layer and ρ_i is the resistivity of the ith layer.

Table 6.2: Calculation of Total Transverse Resistance & Total Longitudinal Conductance

Site Location	Points	LAYER RESISTIVITY (Ω-m)	Thickness (m)	Total Transverse Resistance of Layers (Ω-m^2)	Total Transverse Resistance of Each Point (Ω-m^2)	Total Longitudinal Conductance of Layers (Mho)	Total Longitudinal Conductance of Each Point (Mho)	Curve Type
Site 1	S1/P1	175	1.6	280	5866.924	0.00914	0.09148	H type
		87	5.845	508.515		0.06718		
		579	8.771	5078.409		0.01515		
	S1/P2	220	1.9	418	5745.96	0.00864	0.10570	H type
		76	6.05	459.8		0.07961		
		528	9.22	4868.16		0.01746		
Site 2	S2/P1	229	1	229	4682.8676	0.00437	0.10720	H type
		86	7.449	640.614		0.08662		
		484.9	7.864	3813.2536		0.01622		
	S2/P2	186	1.2	223.2	6203.59	0.00645	0.09859	H type
		79	6.34	500.86		0.08025		
		679	8.07	5479.53		0.01189		
Site 3	S3/P1	149	2.6	387.4	8270.16	0.01745	0.08744	H type
		80.8	4.85	391.88		0.06002		
		867	8.64	7490.88		0.00997		
	S3/P2	167	2.1	350.7	5673.064	0.01257	0.09645	H type
		84.4	5.41	456.604		0.06410		
		496	9.81	4865.76		0.01978		
Site 4	S4/P1	176	1.8	316.8	3833.788	0.01023	0.10962	H type
		79	5.86	462.94		0.07418		
		348	8.776	3054.048		0.02522		
	S4/P2	237	1.6	379.2	4207.363	0.00675	0.11625	H type
		86.7	6.89	597.363		0.07947		
		328	9.85	3230.8		0.03003		
Site 5	S5/P1	194	2.1	407.4	5069.776	0.01082	0.10753	H type
		89.9	7.04	632.896		0.07831		
		468	8.61	4029.48		0.01840		
	S5/P2	210	1.8	378	5355.44	0.00857	0.11494	H type
		74	6.76	500.24		0.09135		
		546	8.2	4477.2		0.01502		

Remarks: From the interpreted results and according to the standard resistivity scale can represent that the formation of layers in each point are more or less same and the resisistivity value in each point are less at the middle layer (below 100 Ω-m) and first and third layer have resistivity value more than the middle layer i.e., formation of resistivity curve types are H type ($\rho_1 > \rho_2 < \rho_3$). As the first layer resistivity value at each point is below 250

Ω-m, from the standard resistivity scale of rocky and sandy area revealed that at each point the first layer consist of top clay with coarse sand mixture having pores filled with some amount of water. As the middle layer of each VES point has resistivity value less than 100 Ω-m, it means that this layer consists of sand and gravel mixture with pores fully saturated with water. And at each VES point the third layer resistivity value is high it may consists rock or hard stone. Among the 10 VES points S2/P1, S4/P1, S4/P2, S5/P1, S5/P2have the highest ground water potential as these points have longitudinal conductance near 0.11 Mho, which is the highest among the 10 VES points. From the transverse resistance value, it can be commented that S2/P2, S3/P1 are the points having aquifer layers of highest transmissivity value among the 10 VES points. From the conductance variation curve depicted can say that all the points have conductance less than 0.5 ds/mupto 20 m depth in each point, which is under desirable limit of conductivity of fresh water.

5.2.3. Hydrogeological Investigation

Kangsabati River

Lithological survey

In this section, the important role of geophysical investigation carried out under this study has been demonstrated as a basis for selection of probable suitable sites adjacent to river as well as underneath the river bed. The detailed positions of litholog covering three blocks been shown in Map. 7.1. The individual block wise of slim bores of three blocks of Purulia district of Kangsabati river delineated in Map Figure 5.23, have been represented by the symbol “B”. The borehole module of GMS 6.0 (Groundwater Modelling System) was used to visualize boreholes depicted by chainage covering longitudinal and transverse section with left and right bank of the river created from drilling logs including plan view, 3-D view and hydro-geological features (see Figs. 7.1 (A) to 7.1 (U), 7.2 (A) to 7.2 (U) and 7.3 (A) to 7.3 (U).

Along the river Kangsabati, slim bores in different cross sections 0.5 km apart covered 10.5 km consists of 10 km upstream and 0.5 km downstream of each block namely Purulia I, Arsha, and Hura of Purulia district were drilled to visualize the soil strata and to identify the thickness the sand layer. The respective cross sectional slim bores are represented graphically below where the color coding of the borehole logs are shown along with the cross sections. While naming the bores a particular pattern has been followed e.g.: B1-3 stands for slim bore no. 3 from Purulia side at cross section no. 1 which in terms represents the cross section 0.5 km apart from the starting cross section (B0) of each block. In case of Kangsabati river, three sites of Purulia district have been considered and depicted in Table 5.11.

Table 5.11 : List of site of slim bores of Kangsabati river

Sl. No.	Name of the site	Name of Block	Distance covered during survey
Site-1	Kotloi to Bhool village	Purulia-I	10.5 km
Site-2	Jeledi to Hensla village	Arsha	10.5 km
Site-3	Karodoba to Jambad village	Hura	10.5 km

The first catchment area was chosen from Kotloi to Bhool village situated of Purulia I block with a distance of 10.5 km covering 10 km towards upstream and 0.5 km towards downstream. Similarly the other two sites were chosen for two other blocks namely Arsha and Hura of Purulia district and highlighted in Table 7(a). Based on these selected sites survey steps carried out the physical survey, velocity measured at different cross-section, slim bored at 0.5 km interval covering a total distance of 10.5 km of each site. Each site was covered for three pumping wells along with three observations wells of each pumping well. Pumping test of main well with three observation wells has been carried out of three sites where the thickness of sand layer received the higher value. The pumping test was conducted for a shorter and longer period having 24 hrs and 72 hrs respectively. The drawdown data have collected from different wells and the data's were plotted in the semi-log paper to estimate the aquifer parameters. Since the aquifer is semi-confined or unconfined in nature, so the aquifer parameters such as Transmissivity (T), Specific yield (S_y), Confined storativity (S), co-efficient of permeability (K) have been determined by using Neuman's Straight Line and the same value has been obtained by using internet based software with the same input. In this case, two types of straight line were observed, the first one is early response time and the second one is late response time. In the late response time region it usually found T and S_y whereas in early response time region it could obtain confined storativity (S). River bed condition and monitoring stations also depicted in below photograph Photo 5.4 and Photo 5.5 .

Photo 5.4: River bed condition in Kangshabati River

Photo 5.5 : Montoring well set up at Kangshabati river

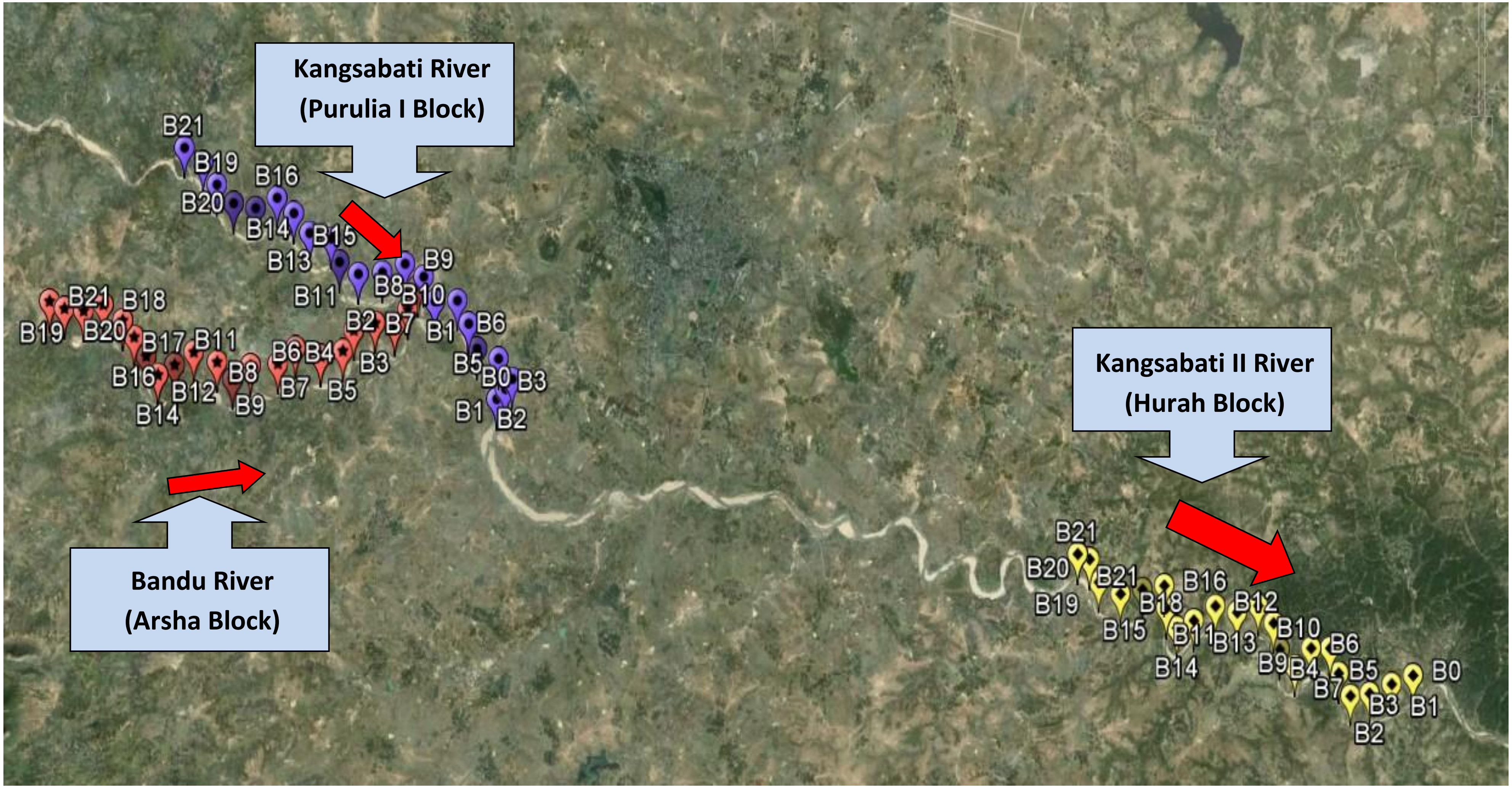


Figure 5.23: Slim bore operation in the study area of river Kangsabati

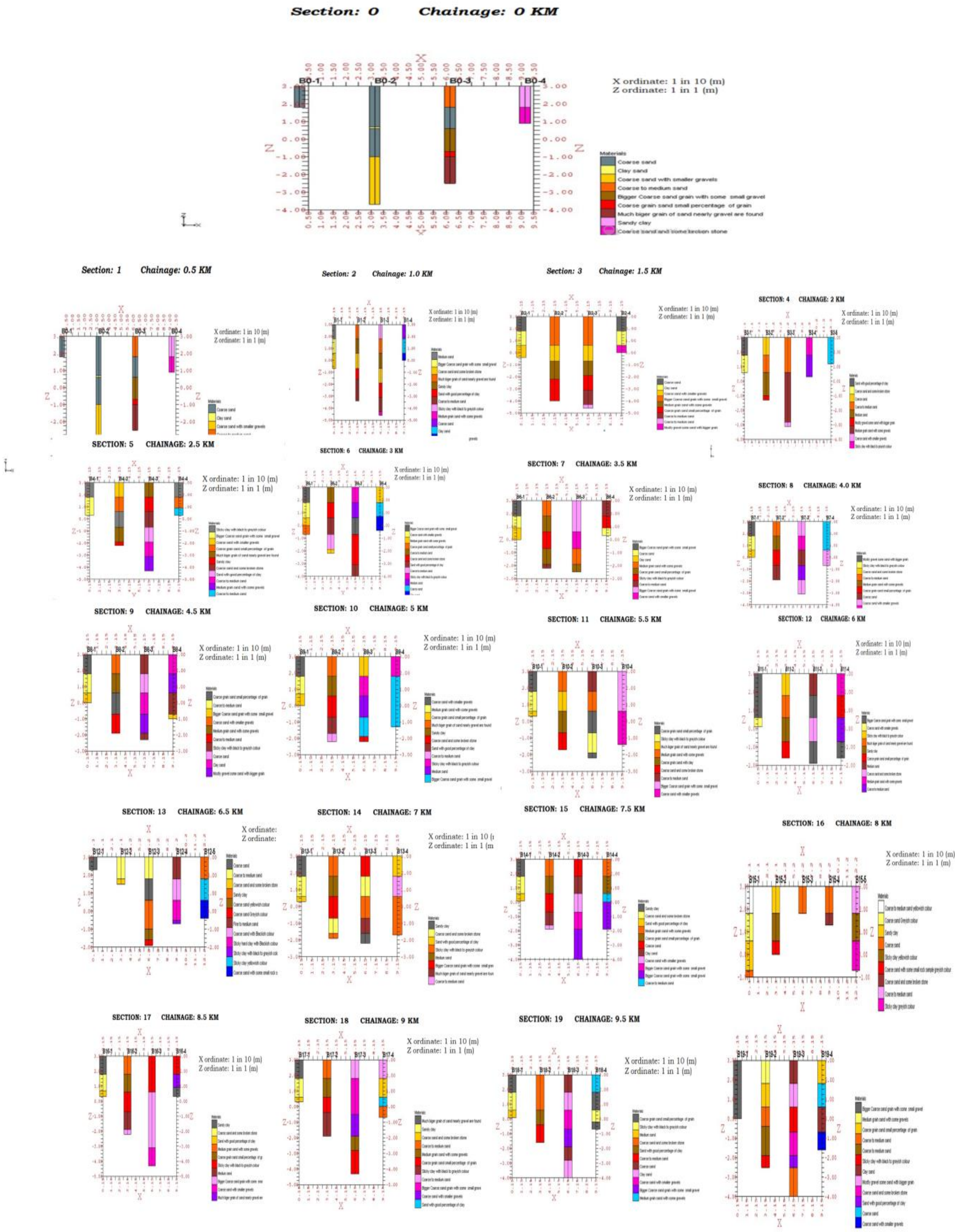


Figure 5.24 : Litholog of slim bores of Kangshabati River Purulia –I Block

Figure 5.25 :Slim Bore at Arsha Block, Bandu River

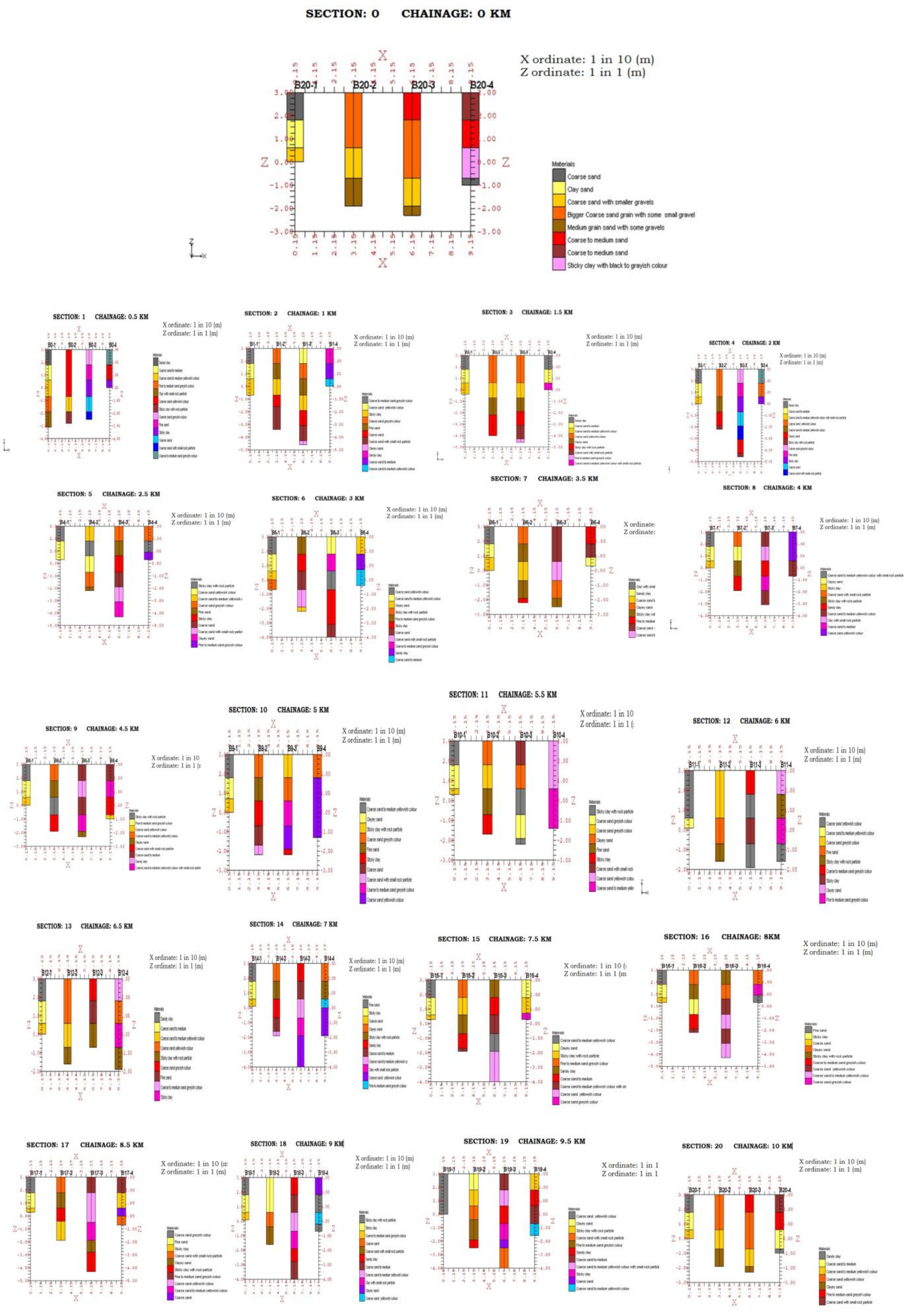


Figure 5.26 : Litholog details of the Kangshabati River in Arsha Block

Figure 5.27 : Slim Bore locations at Hurah Block, Kangsabati River

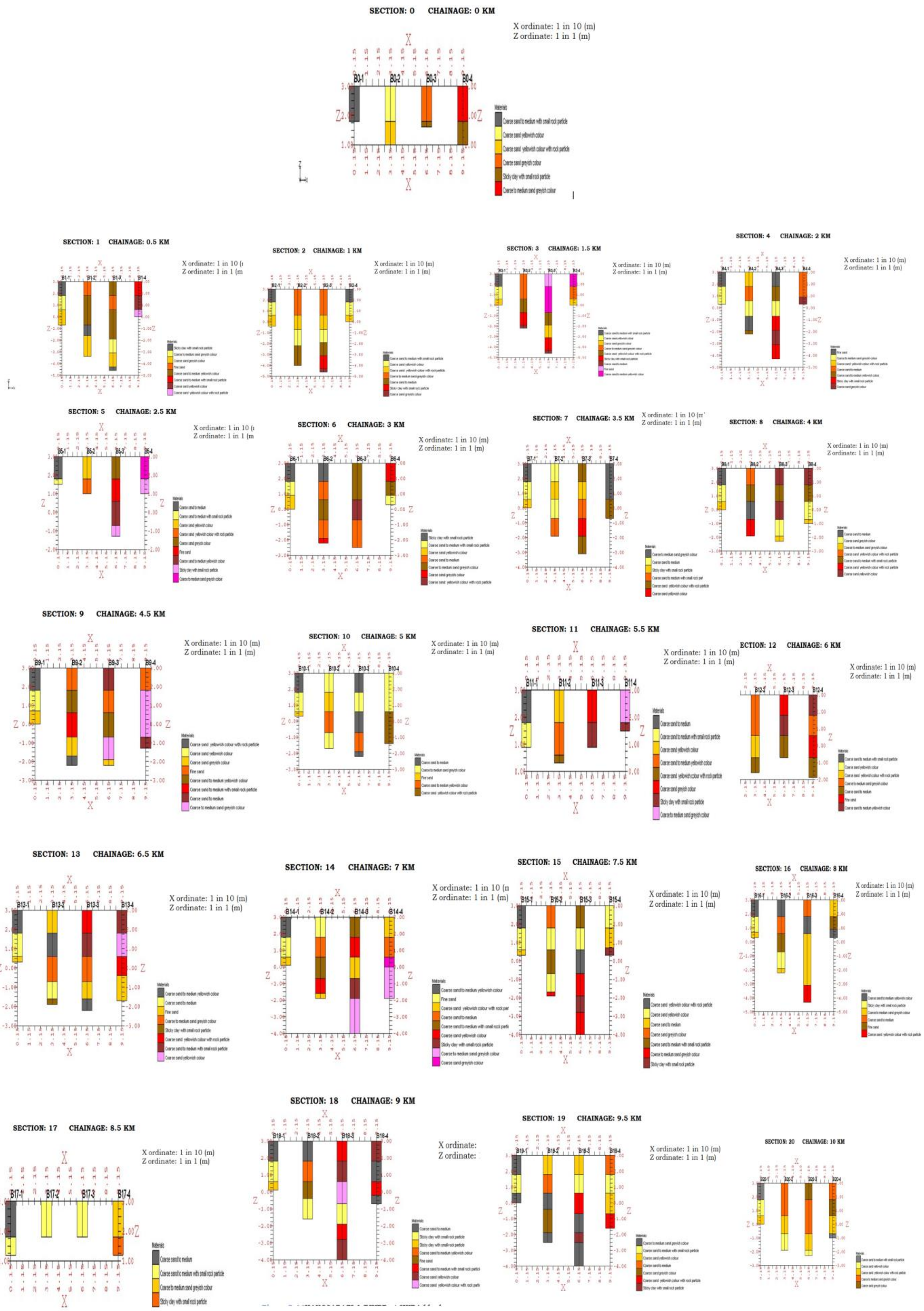


Figure 5.28 : Slim bore details of Kangshabati River at Hura Block

Photo 5.6Slim Boring operation at Kotloi

Photo 5.7 Slim Boring at Baligara

Photo 5.8 Slim Boring at Dhatkiri

Photo 5.9Soil Sample Collection after slim boring

Hydrological Tests of Kangsabati river

In order to adjudge various hydrological parameters, long duration pumping test were carried out 48 hrs. and 72 hrs. respectively. In the first occasion step drawdown test was conducted at variable discharges of 10, 18, 25, 30, 42, 54 and 65 m3/hrs. respectively. During the variable discharges, unsteady conditions of aquifer were noticed. After cessation of pumping, recuperation test was also carried out in the main as also from two and three in some cases observation wells. Values of drawdown versus time are plotted on semi-log paper. The plots reflect the three types of drawdown response that described in the introduction. At early time, the aquifer behaves as though it is confined with data falling along a straight line. At some intermediate time, it deviates from this straight line due to delayed gravity drainage. At late time, data again follow a straight line as the aquifer exhibits a Theis-type response with S equivalent to Sy. The transmissivity (T) and specific yield (Sy) of the aquifer are determined by fitting a straight line through the late-time data. The early-time response provides another estimate of transmissivity (T), and a confined storativity (S) value for the aquifer are obtained. The data plots are presented in figures (see below). The cone of depression with respect to the maximum discharge of 65 m3/hr for which testing was carried out in the area is found to be 250 m. The drawdown versus yield are shown in Table 5.12,Table 5.13, ,Table 5.15,Table 5.16,Table 5.17,Table 5.18 and Table 5.20. The position of the pumping well is represented in Fig. 7.7. From the above pumping studies, it was seen that a long duration pumping tests with another or more observation wells would have been more appropriate to adjudge the actual potentiality of the aquifer and its influence on the static water levels in the perspective of future projected scenario.

Determination of Hydraulic parameters using Distance Drawdown Data for Unconfined Aquifer

The response of an unconfined aquifer to pumping is complicated. Once a cone of depression forms, it naturally decreases the aquifer thickness and transmissivity because the upper boundary of the aquifer is the water table. Also the way in which water comes out of storage in the aquifer changes with time. At early time, when the well is first turned on, water is released from storage due to compression of the matrix and expansion of the water. This response is the same Theis behavior that applies to confined aquifers. Thus, early-time/ drawdown data, plotted on log-log graph paper, follow the Thesis-type curve. Storativity values would be comparable to those for confined aquifers, 10^{-4} or 10^{-5}.

As pumping continues, water comes from the slow gravity drainage of water from pores as the water table falls near the well. The pattern of drawdown depends on the vertical and horizontal hydraulic conductivity and the thickness of the aquifer. Once this delayed drainage begins, drawdown data deviate from the Theis curve. The drawdown is less than expected, resembling the pattern of drawdown in a leaky aquifer. Eventually, the contribution of water from delayed drainage ceases. Flow in the aquifer is mainly radial, and drawdown/time data again fall on a Theis-type curve. The storativity of the aquifer now is the same as the specific yield (S_y). The specific yield is the ratio of the volume of water that drains from a rock or sediment by gravity to the volume of the rock or soil.

Calculation of drawdown by correcting estimate for a confined aquifer

$$S' = S - S^2/2b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(1)$$

Where S is the drawdown for the unconfined aquifer and s' is the drawdown for the equivalent confined Aquifer

Following methodology is adopted to determine of hydraulic parameter using distance drawdown method taking following steps

I. Calculate the corrected drawdown using Eq.(1)
II. Plot corrected drawdown s' against r on semi-log graph paper
III. Approximate the curve s' versus log(t) with straight line
IV. Determining hydraulic parameter using following Equation

a) Transmissivity

$$T= \frac{2.3\,Q}{4\,\pi\Delta s}$$

b) Specific Yield

$$S_y = = \frac{2.25\,T\,t0y}{r^2}$$

c) Storativity

$$S= = \frac{2.25\,T\,t0y}{r^2}$$

Values of drawdown versus time are plotted on semi-log paper in Figure 5.30 and Figure 5.31 The plots reflect the three types of drawdown response that has been described in the introduction. At early time, the aquifer behaves as though it is confined with data falling along a straight line. At some intermediate time, it deviates from this straight line due to delayed gravity drainage. At late time, data again follow a straight line as the aquifer exhibits a Theis-type response with S equivalent to S_y Figure 5.29. The transmissivity and specific yield of the aquifer are determined by fitting a straight line through the late-time data. The early-time response provides another estimate of transmissivity, and a confined storativity value for the aquifer can be obtained.

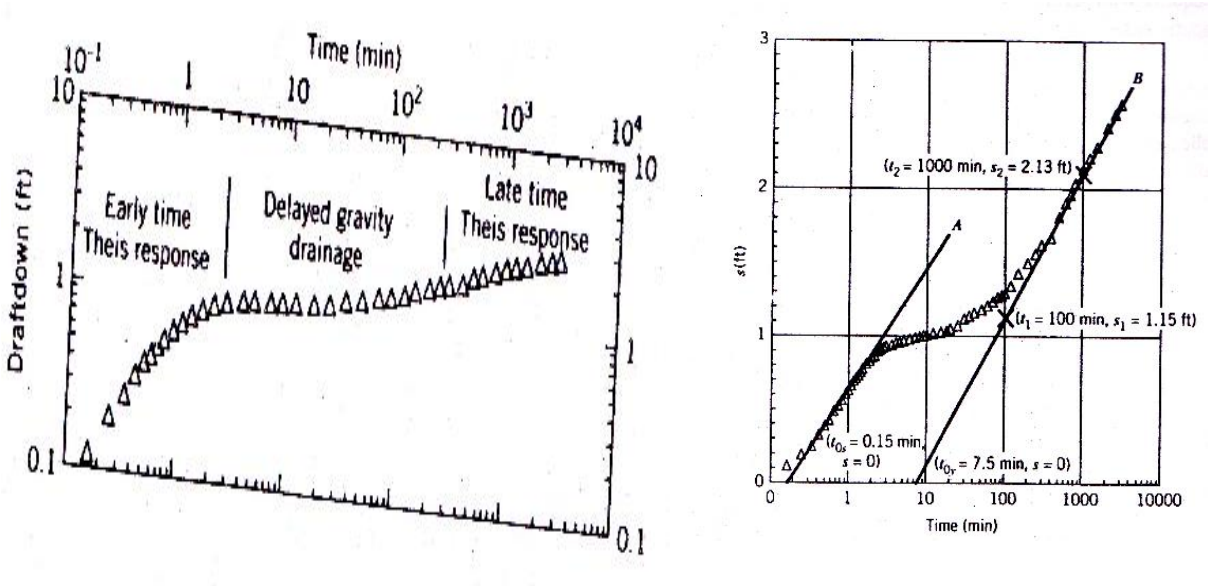


Figure 5.29 : Drawdown (s)versus time data (t) for aquifer test

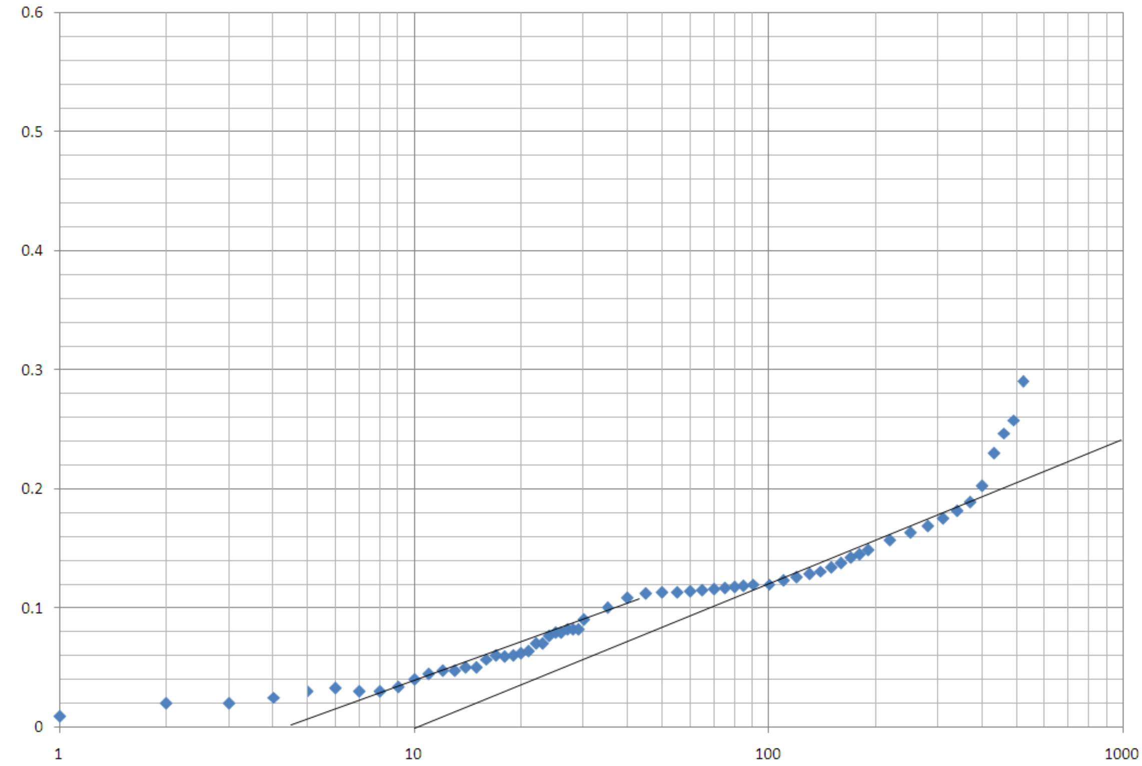


Figure 5.30 : Drawdown vs time curve for observation well-1 at Beledi

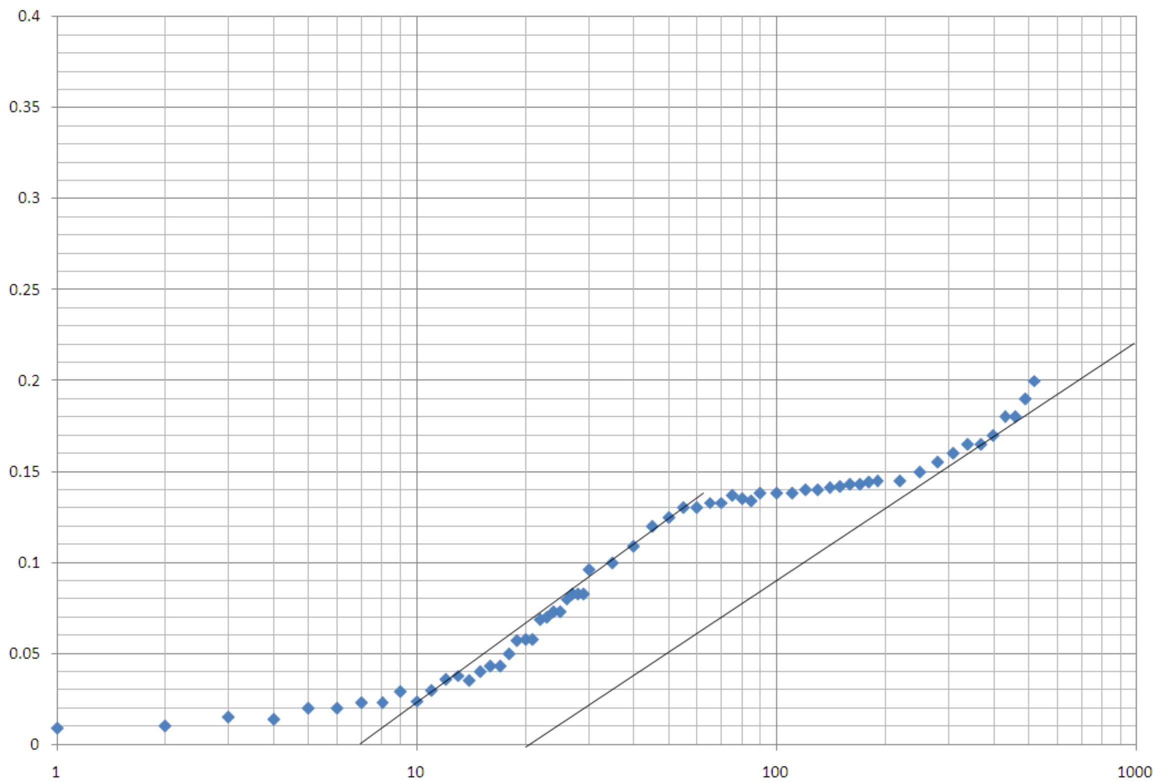


Figure 5.31 : Drawdown vs time curve for observation well-2 at Beledi

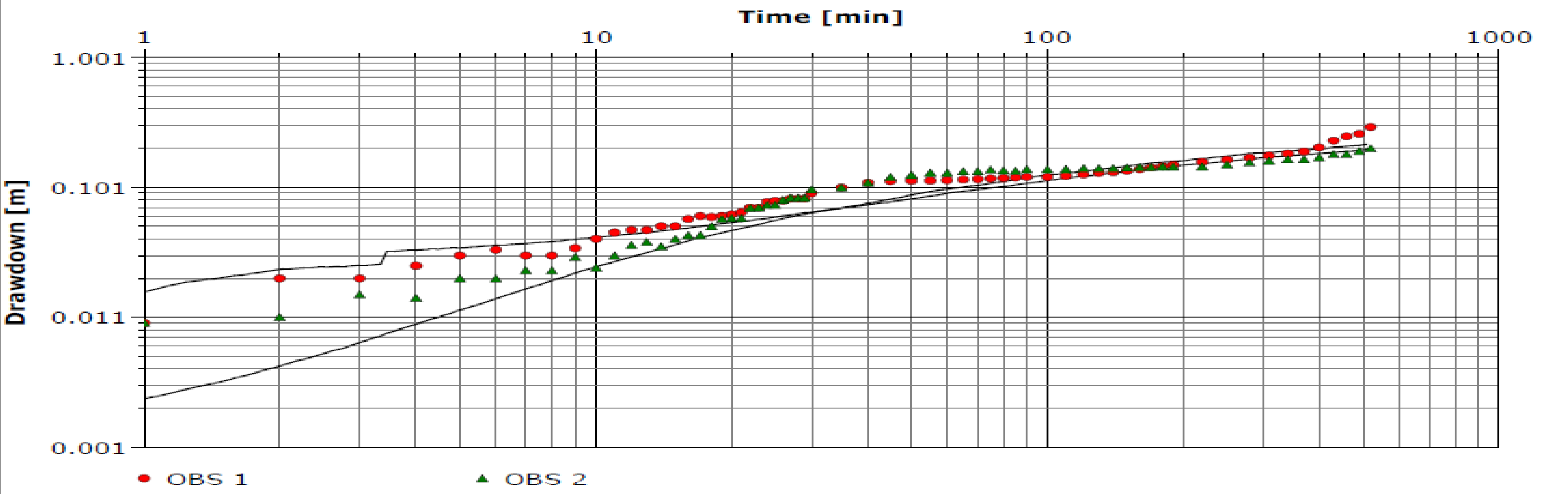
Table 5.12: 1st Pumping Test Result of Kangsabati River

BLOCK: PURULIA I; RIVER: KANGSABATI								
SL NO.	**LOCATION**	**MAIN WELL**				**OBSERVATION WELL**		
		PARAMETER	**GPS Location Of the Main Well**	**Radius Of Influence (m)**	**DISCHARGE (m3/hr)**	**OBSERVATION WELL NO.**	**PARAMETERS**	
		Transmissivity (m^2/day)					Transmissivity (m^2/day)	Storativity
1.	Near Beldi Bridge (Purulia-I block)	400	N 23°19'29.50" E 86°17'34.19" Elevation- 223	119	10	Obs-1	366	1.25×10^{-3}
						Obs-2	338	4.98×10^{-4}
						Obs-3*	324	4.05× 10-4
						Obs-4*	305	3.81× 10-4
		719		141	18	Obs-1	610	2.28×10^{-3}
						Obs-2	603	7.36×10^{-4}
						Obs-3	588	7.35× 10-4
						Obs-4	580	7.25× 10-4
		1098		155	25	Obs-1	1372	4.28×10^{-3}
						Obs-2	844	1.23×10^{-3}
						Obs-3*	732	9.15× 10-4
						Obs-4*	649	8.11× 10-4

*Observation well-3 constructed in left Bank of the River and Observation well-4 constructed in Right bank of the River

Name Of the Well	**Maximum Drawdown(m) At different discharge**			**Recovery Time for different discharge(Minutes)**			**Distance from Main Pumping Well(m)**
	10(m^3/hr)	**18(m^3/hr)**	**25(m^3/hr)**	**10(m^3/hr)**	**18(m^3/hr)**	**25(m^3/hr)**	
Main Well	0.59	0.62	0.70	192	203	206	-
Observation Well-1	0.29	0.36	0.37	182	183	186	50
Observation Well-2	0.20	0.34	0.38	162	183	185	80
*Observation well-3	0.21	0.25	0.29	167	179	187	110
*Observation well-4	0.18	0.22	0.26	169	184	191	120

School Of Water Resources Engineering Jadavpur University Jadavpur, Kolkata West Bengal	**Pumping Test Analysis Report** Project: ABC Number: 1 Client: XYZ	
Location: Beledi	Pumping Test: Pumping Test 1	Pumping Well: MW
Test Conducted by: SWRE,JU		Test Date: 5/31/2013
Analysis Performed by: SWRE,JU	Neuman	Analysis Date: 5/31/2013
Aquifer Thickness: 6.00 m	Discharge Rate: 10 [m³/h]	



Calculation using Neuman

Observation Well	Transmissivity [m²/d]	Hydraulic Conductivity [m/d]	Specific Yield	Ratio K(v)/K(h)	Ratio Sy/S	Radial Distance to PW [m]
OBS 1	3.50×10^{2}	5.83×10^{1}	3.00×10^{-3}	8.87×10^{-3}	1.52×10^{1}	50.0
OBS 2	3.30×10^{2}	5.50×10^{1}	1.00×10^{-3}	3.94×10^{-2}	2.56×10^{1}	80.0
Average	3.40×10^{2}	5.67×10^{1}	2.00×10^{-3}	2.41×10^{-2}	2.04×10^{1}	

Figure 5.32: Pumping test analysis through Aqua-pro Software

Table 5.13 : 2nd Pumping Test Result of Kangsabati River

BLOCK: PURULIA I; RIVER: KANGSABATI AND BANDU

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER	GPS Location of the Main Well	Radius Of Influence(m)	DISCHARGE (m3/hr)	OBSERVATION WELL NO.	PARAMETERS	
		Transmissivity (m^2/day)					Transmissivity (m^2/day)	Storativity
2.	Teledi and Satsimulia Confluence point of Bandu River and Kangsabati River (Purulia-I block)	316	N-23°19'05.23" E-86°18'35.09 Elevation-224	55	18	Obs-1	988	0.001236
						Obs-2	659	0.000322
						Obs-3	878	0.000227
						Obs-4	569	0.000699
		424		92	32	Obs-1	1035	0.001301
						Obs-2	1025	0.000501
						Obs-3	1538	0.000397
						Obs-4	1134	0.001418

*Observation well-3 constructed in left Bank of the River and *Observation well-4 constructed in Right bank of the River

Name Of the Well	Maximum Drawdown(m) At different discharge			Recovery Time for different discharge (Minutes)			Distance from Main Pumping Well (m)
	18(m^3/hr)	30(m^3/hr)		18(m^3/hr)	30(m^3/hr)		
Main Well	0.64	1.17		221	315		-
Observation Well-1	0.27	0.40		191	285		50
Observation Well-2	0.28	0.42		202	271		80
Observation well-3	0.24	0.37		207	277		110
Observation well-4	0.19	0.28		193	259		120

Table 5.14 : 3rdPumping Test Result of Kangsabati River

BLOCK: PURULIA I; RIVER: KANGSABATI

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER Transmissivity (m^2/day)	**GPS Location Of the Main Well**	**Radius Of Influence (m)**	**DISCHARGE (m3/hr)**	**OBSERVATION WELL NO.**	**PARAMETERS** Transmissivity (m^2/day)	Storativity
3.	Kotloi (Purulia-I block)	389		70	10	Obs-1	440	0.00055
						Obs-2	347	0.000169
						Obs-3	389	0.0001
						Obs-4	301	0.000155
		676		82	18	Obs-1	629	0.000786
						Obs-2	609	0.000297
						Obs-3	587	0.000152
						Obs-4	571	0.000221
		1064		169	25	Obs-1	1352	0.00169
						Obs-2	849	0.000415
						Obs-3	719	0.000186
						Obs-4	612	0.000158

*Observation well-3 constructed in left Bank of the River and Observation well-4 constructed in Right bank of the River

Name Of the Well	Maximum Drawdown(m) at different discharge			Recovery Time for different discharge (Minutes)			Distance from Main Pumping Well (m)
	18(m^3/hr)	**30(m^3/hr)**	**25(m^3/hr)**	**18(m^3/hr)**	**30(m^3/hr)**	**25(m^3/hr)**	
Main Well	0.51	0.67	0.73	195	215	222	-
Observation Well-1	0.31	0.38	0.41	182	195	203	50
Observation Well-2	0.22	0.34	0.39	167	182	193	80
Observation well-3	0.20	0.23	0.28	173	188	201	110
Observation well-4	0.17	0.23	0.29	169	187	195	120

Table 5.15 : 4th Pumping Test Result of Kangsabati River

BLOCK: ARSHA; BANDU RIVER:

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER	GPS Location of the Main Well	Radius Of Influence(m)	DISCHARGE (m3/hr)	OBSERVATION WELL NO.	PARAMETERS	
		Transmissivity (m^2/day)					Transmissivity (m^2/day)	Storativity
4.	Joradihi (Bandu) (Arsha Block)	128	N 23°18'27.48" E 86°17'06.74" Elevation- 230	50	10	Obs-1	732	0.000916
						Obs-2	744	0.000364
						Obs-3	878	0.000227
						Obs-4	891	0.00046
		226		85	18	Obs-1	1340	0.001676
						Obs-2	1361	0.002328
						Obs-3	1583	0.000409
						Obs-4	1602	0.000621
		296		149	25	Obs-1	1862	0.002328
						Obs-2	1831	0.000666
						Obs-3	2073	0.000535
						Obs-4	2079	0.000537

Observation well-3 constructed in left Bank of the River and Observation well-4 constructed in Right bank of the River

Name Of the Well	Maximum Drawdown(m) At different discharge			Recovery Time for different discharge (Minutes)			Distance from Main Pumping Well (m)
	10(m^3/hr)	18(m^3/hr)	25(m^3/hr)	10(m^3/hr)	18(m^3/hr)	25(m^3/hr)	
Main Well	1.26	1.29	1.43	223	231	329	-
Observation Well-1	0.24	0.26	0.32	240	249	261	50
Observation Well-2	0.25	0.27	0.39	245	256	269	80
Observation well-3	0.19	0.22	0.23	248	251	252	110
Observation well-4	0.15	0.18	0.20	242	246	249	120

Table 5.16 : 5thPumping Test Result of Kangsabati River

BLOCK: ARSHA; RIVER: KANGSABATI

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER Transmissivity(m^2/day)	GPS Location of the Main Well	Radius of Influence(m)	DISCHARGE(m3/hr)	OBSERVATION WELL NO.	PARAMETERS Transmissivity (m^2/day)	Storativity
5.	Goradag (Arsha Block)	1867	N-23°19'38.68" E-86°16'54.80" Elevation-229	159	42	Obs-1	2490	0.003113
						Obs-2	2134	0.001042
						Obs-3	1867	0.000482
						Obs-4	1756	0.000907
		1825		183	54	Obs-1	2498	0.003123
						Obs-2	2636	0.001042
						Obs-3	2636	0.000681
						Obs-4	2581	0.001
		2596		230	65	Obs-1	2528	0.00316
						Obs-2	2856	0.001395
						Obs-3	2837	0.000738
						Obs-4	2712	0.0007

*Observation well-3 constructed in left Bank of the River and *Observation well-4 constructed in Right bank of the River

Name Of the Well	Maximum Drawdown(m) At different discharge			Recovery Time for different discharge (Minutes)			Distance from Main Pumping Well (m)
	42(m^3/hr)	54(m^3/hr)	65(m^3/hr)	42(m^3/hr)	54(m^3/hr)	65(m^3/hr)	
Main Well	1.23	1.51	1.93	245	284	324	-
Observation Well-1	0.29	0.31	0.36	210	260	289	50
Observation Well-2	0.26	0.29	032	200	241	273	80
Observation well-3	0.22	0.25	0.28	203	245	279	110
Observation well-4	0.19	0.22	0.24	208	251	272	120

Table 5.17 : 6th Pumping Test Result of Kangsabati River

BLOCK : ARSHA; RIVER: KANGSABATI

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER Transmissivity(m²/day)	**GPS Location of the Main Well**	**Radius of Influence (m)**	**DISCHARGE (m³/hr)**	**OBSERVATION WELL NO.**	**PARAMETERS** Transmissivity (m²/day)	Storativity
6.	Belchari (Arsha Block)	208	N-23°18'52.64" E-86°14'23.83" Elevation-248 m	32.49	10	Obs-1	737	0.000921
						Obs-2	749	0.000366
						Obs-3	871	0.000225
						Obs-4	896	0.000463
		312		13.23	18	Obs-1	1356	0.001695
						Obs-2	1368	0.000668
						Obs-3	1589	0.00041
						Obs-4	1611	0.000832
		416		18.65	25	Obs-1	1868	0.002335
						Obs-2	1839	0.000898
						Obs-3	2077	0.000536
						Obs-4	2083	0.000538

*Observation well-3 constructed in left Bank of the River and Observation well-4 constructed in Right bank of the River

Name Of the Well	**Maximum Drawdown(m) At different discharge**			**Recovery Time for different discharge (Minutes)**			**Distance from Main Pumping Well (m)**
	42(m³/hr)	**54(m³/hr)**	**65(m³/hr)**	**42(m³/hr)**	**54(m³/hr)**	**65(m³/hr)**	
Main Well	1.19	1.23	1.36	221	23	305	-
Observation Well-1	0.25	0.28	0.32	240	245	263	50
Observation Well-2	0.23	0.27	0.30	225	249	261	80
Observation well-3	0.19	0.21	0.26	234	246	258	110
Observation well-4	0.14	0.18	0.21	242	249	253	120

Table 5.18 : 7th Pumping Test Result of Kangsabati River

BLOCK: HURAH; RIVER: KANGSABATI

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER	GPS Location of the Main Well	Radius of Influence (m)	DISCHARGE (m3/hr)	OBSERVATION WELL NO.	PARAMETERS	
		Transmissivity (m^2/day)					Transmissivity (m^2/day)	Storativity
7.	Rahamda Karadoba (Hurah Block)	158	N 23°15’38.06” E 86°29’40.25” Elevation- 207 m	70	10	Obs-1	576	0.00072
						Obs-2	612	0.000765
						Obs-3	637	0.000165
						Obs-4	649	0.000168
		219		153	18	Obs-1	636	0.000311
						Obs-2	644	0.000314
						Obs-3	687	0.000177
						Obs-4	697	0.00018
						-	-	-
						-	-	-

*Observation well-3 constructed in left Bank of the River and Observation well-4 constructed in Right bank of the River

Name Of the Well	Maximum Drawdown At different discharge(m)			Recovery Time for different discharge (Minutes)			Distance from Main Pumping Well (m)
	10(m^3/hr)	18(m^3/hr)		10(m^3/hr)	18(m^3/hr)		
Main Well	1.70	1.89		301	346		-
Observation Well-1	0.29	0.33		240	267		50
Observation Well-2	0.21	0.25		249	258		80
Observation well-3	0.19	0.22		231	242		110
Observation well-4	0.17	0.20		229	240		120

Table 5.19 8th Pumping Test Result of Kangsabati River

BLOCK : HURAH; RIVER: KANGSABATI								
SL NO.	**LOCATION**	**MAIN WELL**				**OBSERVATION WELL**		
		PARAMETER	**GPS Location Of the Main Well**	**Radius Of Influence(m)**	**DISCHARGE (m3/hr)**	**OBSERVATION WELL NO.**	**PARAMETERS**	
		Transmissivity (m^2/day)					Transmissivity (m^2/day)	Storativity
8.	Kudlung (Hurah Block)	395	N- 23°16'04.36" E- 86°27'50.69" Elevation-195 m	120	18	Obs-1	841	0.001052
						Obs-2	878	0.001962
						Obs-3	912	0.000236
						Obs-4	889	0.000193
		615		181	30	Obs-1	1569	0.001099
						Obs-2	1448	0.001811
						Obs-3	1453	0.000375
						Obs-4	1389	0.000301

*Observation well-3 constructed in left Bank of the River and Observation well-4 constructed in Right bank of the River

Name Of the Well	**Maximum Drawdown At different discharge(m)**			**Recovery Time for different discharge (Minutes)**			**Distance from Main Pumping Well (m)**
	25(m^3/hr)	**34(m^3/hr)**		**25(m^3/hr)**	**34(m^3/hr)**		
Main Well	0.90	1.07		240	271		-
Observation Well-1	0.24	0.29		198	216		50
Observation Well-2	0.21	0.23		196	210		80
Observation well-3	0.18	0.22		201	213		110
Observation well-4	0.16	0.20		204	216		120

Table 5.20 : 9th Pumping Test Result of Kangsabati River

BLOCK: HURAH; RIVER: KANGSABATI

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER Transmissivity (m^2/day)	**GPS Location of the Main Well**	**Radius of Influence (m)**	**DISCHARGE (m3/hr)**	**OBSERVATION WELL NO.**	**PARAMETERS** Transmissivity (m^2/day)	Storativity
9.	Jambad (Hurah Block)	420	N- 23°16'04.36" E- 86°27'50.69" Elevation-195 m	141	10	Obs-1	689	0.00086
						Obs-2	719	0.00035
						Obs-3	840	0.00022
						Obs-4	829	0.00036
		538		158	18	Obs-1	1204	0.00151
						Obs-2	1188	0.00058
						Obs-3	1486	0.00038
						Obs-4	1537	0.00067
		1036		199	25	Obs-1	1335	0.00167
						Obs-2	1401	0.00068
						Obs-3	1373	0.00036
						Obs-4	1416	0.00062

*Observation well-3 constructed in left Bank of the River and Observation well-4 constructed in Right bank of the River

Name of the Well	Maximum Drawdown(m) At different discharge			Recovery Time for different discharge (Minutes)			Distance from Main Pumping Well (m)
	10(m^3/hr)	**18(m^3/hr)**	**25(m3/hr)**	**10(m^3/hr)**	**18(m^3/hr)**	**25(m3/hr)**	
Main Well	0.96	1.19	1.29	219	263	302	-
Observation Well-1	0.27	0.3	0.36	181	196	210	50
Observation Well-2	0.25	0.28	0.32	179	193	203	80
Observation well-3	0.2	0.25	0.3	183	199	209	110
Observation well-4	0.17	0.22	0.27	191	202	219	120

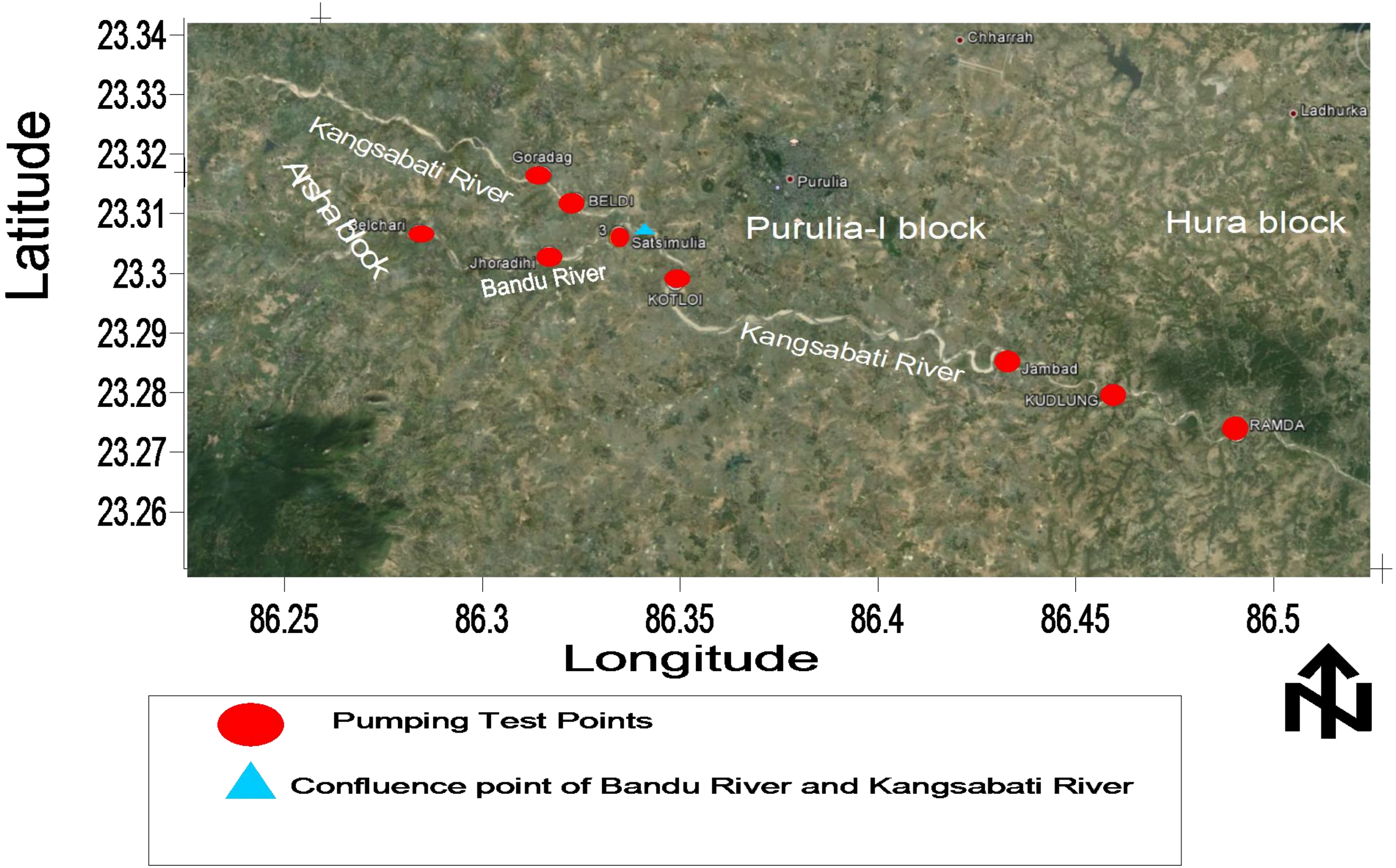


Figure 5.33 : Location pumping test in Kangsabati river at three different blocks

Photo 5.10 :Lowering of Casing pipe near Beldihi bridge

Photo 5.11 : Boring operation

Photo 5.12 : Pumping test photographs in Near Beldihi bridge

Photo 5.13: Monitoring observation well

Photo 5.14 : Lowering of Casing pipe during Boring operation at Joradihi

Photo 5.15 : Lowering of strainer pipe during at Joradihi

Photo 5.16 : Measuring discharge and sensor setup

Photo 5.17 : Measuring Discharge Trough Ultra sonic flow meter

Photo 5.18 : Observation of water table by the help of TLC meter Karadoba

Photo 5.19 : Lowering Strainer pipe in Kudlung

Photo 5.20 : Water table measurement by the help of TLC meter at Teledihi

Photo 5.21 : Boring operation in kudlung

Damodar River

Litho logical Survey

In this section, the important role of geophysical investigation carried out under this study has been demonstrated as a basis for selection of probable suitable sites adjacent to river as well as underneath the river bed. The individual block wise of slim bores of three blocks of Purulia district of Damodar river delineated in Map 7.5, Map 7.6 and Map 7.7 have been represented by the symbol “B”. The borehole module of GMS 6.0 (Groundwater Modeling System) was used to visualize boreholes depicted by chainage covering longitudinal and transverse section with left and right bank of the river created from drilling logs including plan view, 3-D view and hydro-geological features (see Figs. 7.8 (A) to 7.8 (V), 7.9 (A) to 7.9 (V) and 7.10 (A) to 7.10 (B). In the river Damodar, slim bores in different cross sections 0.5 km apart with a total distance of 10.5 km for both upstream and downstream covering three blocks namely Santuri, Neturia, and Raghunathpur-2 of Purulia district were drilled. The respective cross sectional slim bores are represented graphically below where the colour coding of the borehole logs are shown along with the cross sections. While naming the bores, a pattern has been followed e.g. B1-3 stands for slim bore no. 3 from Purulia side at cross section no. 1 which in terms represents the cross section 0.5 km apart from the starting cross section (B0) at each block. In case of Raghunathpur 2 block, due to presence of water over the full stretch of the river throughout the year, slim boring was not possible to carry out along the cross sections. In this regards only at the right bank side of the river a few boreholes were drilled.

In case of Damodar river, three sites of Purulia district have been considered and depicted in Table 5.21.

Table 5.21 : List of site of slim bores of Damodar river

Sl. No.	Name of the site	Name of Block	Distance covered during survey
Site-1	Deoli to Chinakuri village	Saturi	10.5 km
Site-2	Chinakuri to Narayanpur village	Neturia	10.5 km
Site-3	Sethpalash to Ichar village	Raghunathpur-2	10.5 km

The first catchment area was chosen from Deoli to Chinakuri village situated of Saturi block with 10.5 km covering 10 km towards upstream and 0.5 km towards downstream. Similarly, the other two sites were chosen for two other blocks namely Neturia and Raghunathpur-2 of Purulia district and highlighted in Table 7(b). Based on these selected sites the physical survey, velocity measured at different cross-section, slim bored at 0.5 km interval covering a total distance of 10.5 km of each site. Each site was covered for three pumping wells along with three observations wells of each pumping well. Pumping test of main well with three observation wells has been carried out of two sites where the thickness of sand layer received the higher value. The third site namely Ragunathpur 2 block could not be considered for pumping test as because the entire catchment area has sufficient surface water volume and thus result no pumping test was conducted. The pumping test was conducted for a shorter and longer period having 24 hrs and 72 hrs respectively. The drawdown data have collected from different wells and the data were plotted in the semi-log paper to estimate the aquifer parameters. Since the aquifer is semi-confined or unconfined in nature, so the aquifer parameters such as Transmissivity (T), Specific yield (Sy), Confined storativity (S), co-efficient of permeability (K) have been determined by using Neuman's Straight Line and the same value has been obtained by using internet based software with the same input. In this case, two types of straight line were observed, the first one is early response time and the second one is late response time. In the late response time region, it usually found T and Sy whereas in early response time region it could obtain confined storativity (S).

Figure 5.34 : Slim Bore at Santuri block, Purulia (Site-1 Santuri block)

Santuri block

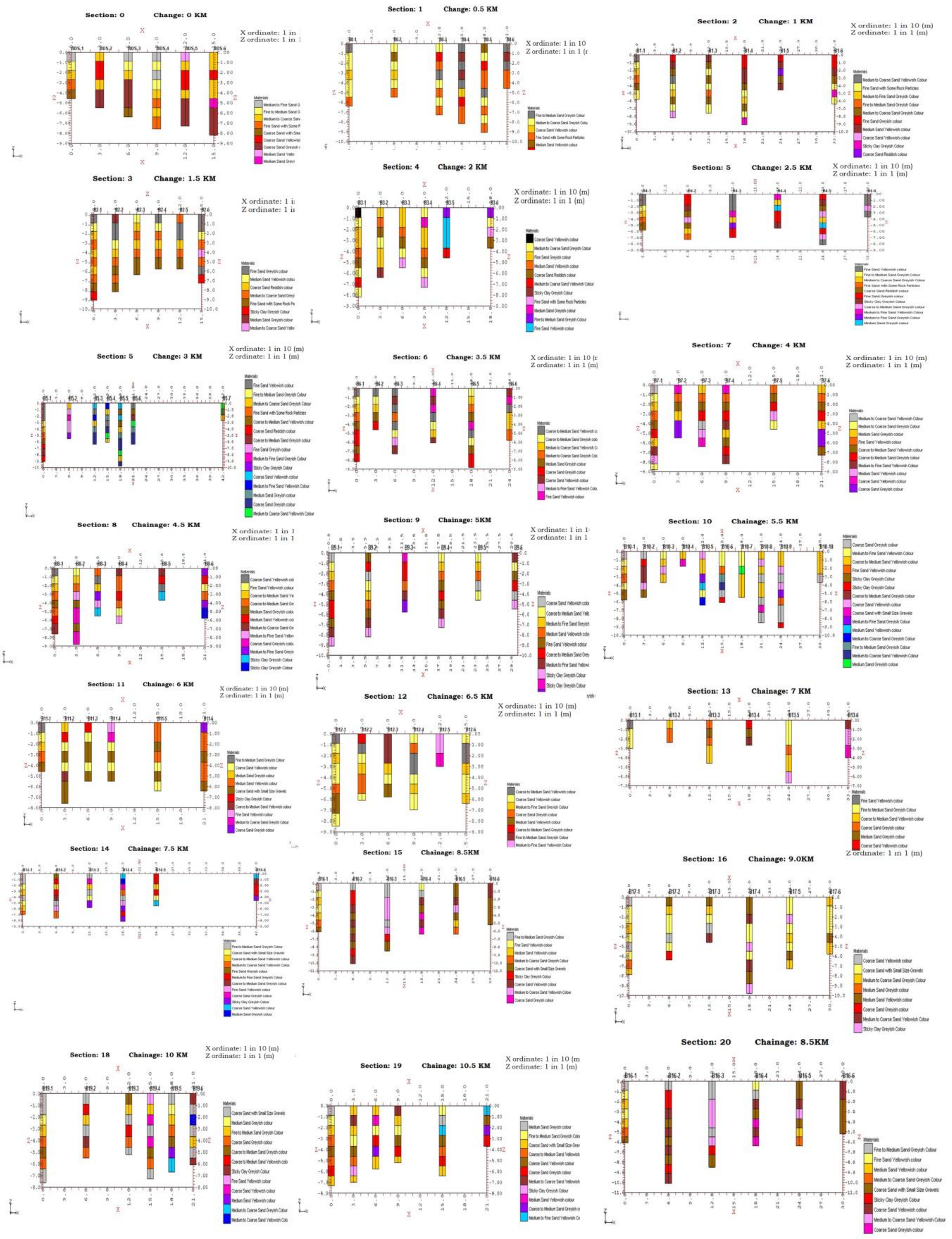


Figure 5.35 : Slimbore Litholog

Figure 5.36 : Locations of Slim boring at Neturia Block, Purulia (Site-2)

Neturia Block

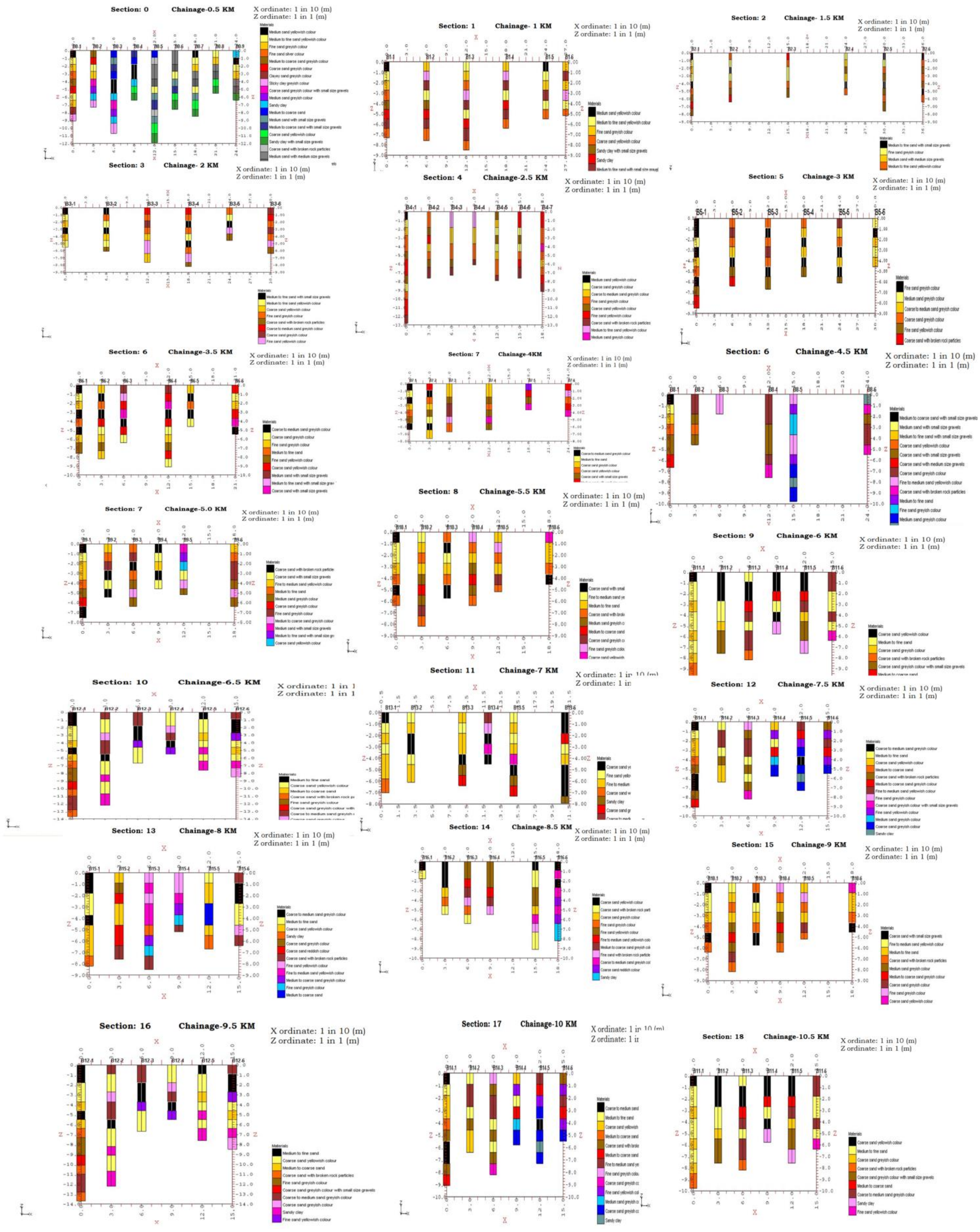


Figure 5.37 : Slim boring at Nituria Block in Damodar River

Figure 5.38 : Locations of slim bore at Raghunathpur 2 block, Purulia (Site-3)

Y ordinate: 1 in 100 (m)
Z ordinate: 1 in 1 (m)

Section:1 Chainage-0-10 KM (Left Bank)

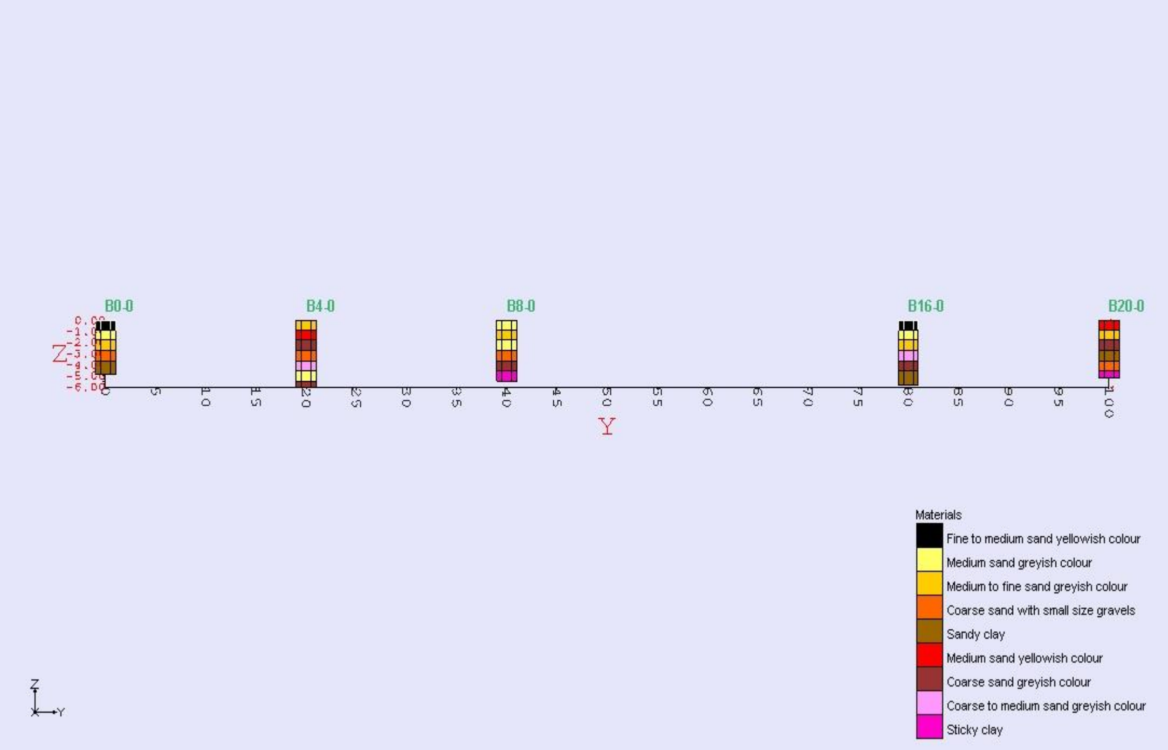


Figure 5.39 : Litholog structures at Raghunathpur 2 block (site 3)

Section:2 Chainage-0-10 KM (Right Bank)

Y ordinate: 1 in 100 (m)
Z ordinate: 1 in 1 (m)

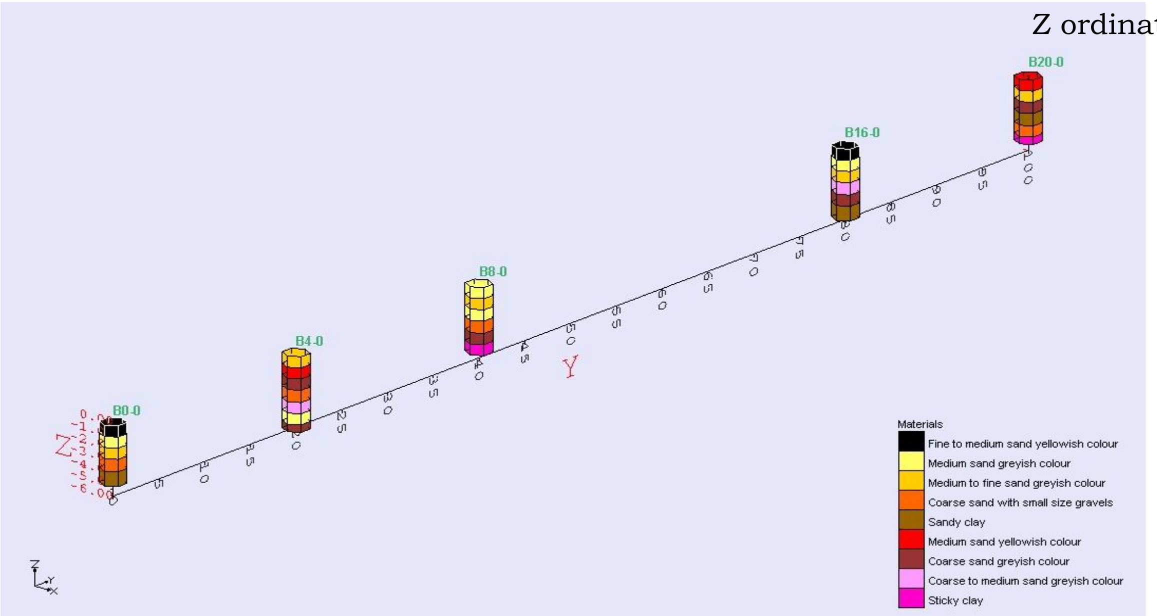


Figure 5.40 : Litholog structures at Raghunathpur 2 block (site 3)

Hydrological Testing for Damodar River

To adjudge various hydrological parameters, long duration pumping test were carried out 48 hrs and 72 hrs respectively. In the first occasion step drawdown test was conducted at variable discharges of 25, 30 and 44 m^3/hr respectively. During the variable discharges, unsteady conditions of aquifer were noticed. After cessation of pumping, recuperation test was also carried out in the main has also from two and three in some cases observation wells. Accordingly, the piezometric level data as obtained from the main as well as the observation wells were plotted in semi-log and log-log papers as opined by various proponents in groundwater hydrology like Thies, Cooper and Jacob and Chow. The data plots and their matching with the master curves has facilitated the determination of various aquifer parameters like hydraulic conductivity (K), Transmissivity (T), Storativity (S)/specific yield (S_s) etc. The data plots are presented in Figure 5.41 (below).

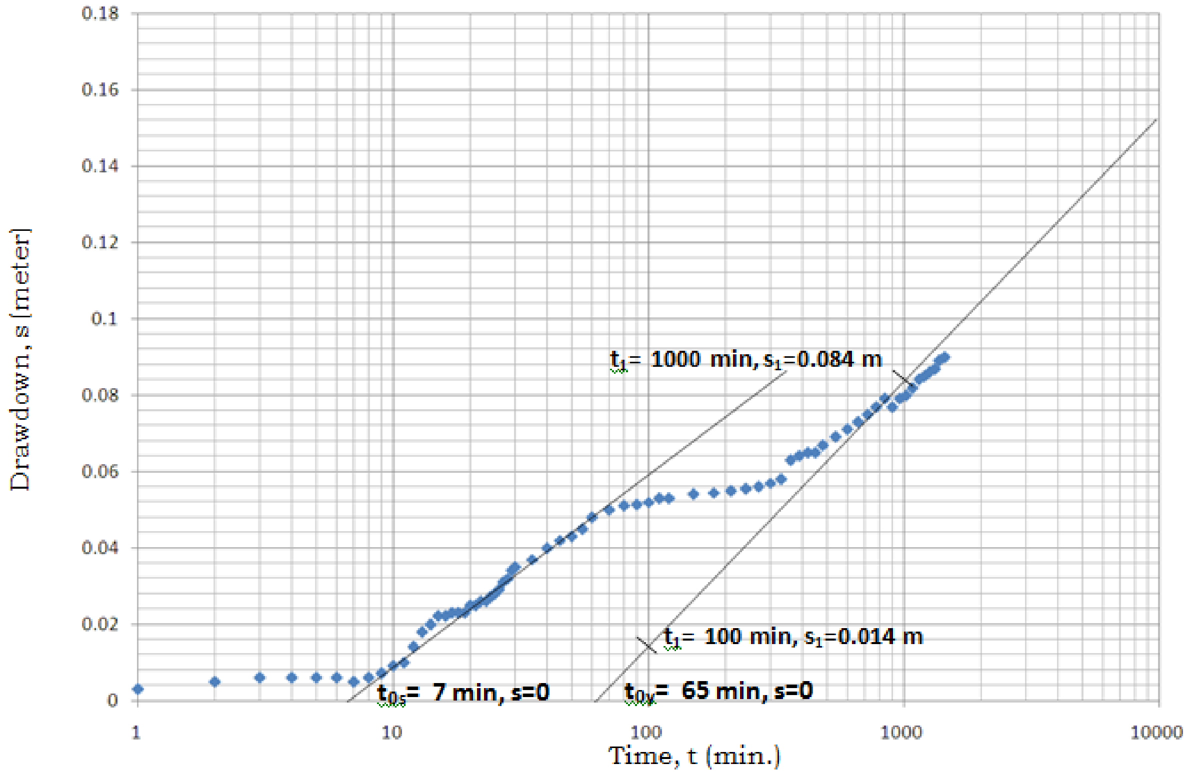


Figure 5.41: Time vs duration curve

The cone of depression with respect to the maximum discharge of 44 m^3/hr for which testing was carried out in the area is found 257 m. The drawdownversus yield are shown in Table 5.23,Table 5.24,Table 5.25,Table 5.26, and Table 5.27, From the above pumping studies, it was seen that a long duration pumping tests with another or more observation wells would have been more appropriate to adjudge the actual potentiality of the aquifer and its influence on the static water levels in the perspective of future projected scenario.

1st pumping test

Here Neuman's Straight-Line Method is used to determine the hydraulic parameters. The transmissivity and specific yield of the aquifer are determined by fitting a straight line through the late-time data. The early-time response provides another estimate of transmissivity, and a confined storativity value for the aquifer can be obtained in Pumping rate, Q= 25 m^3/hr. Observation well-1 is 35 m away from the pumping wellFigure 5.42.

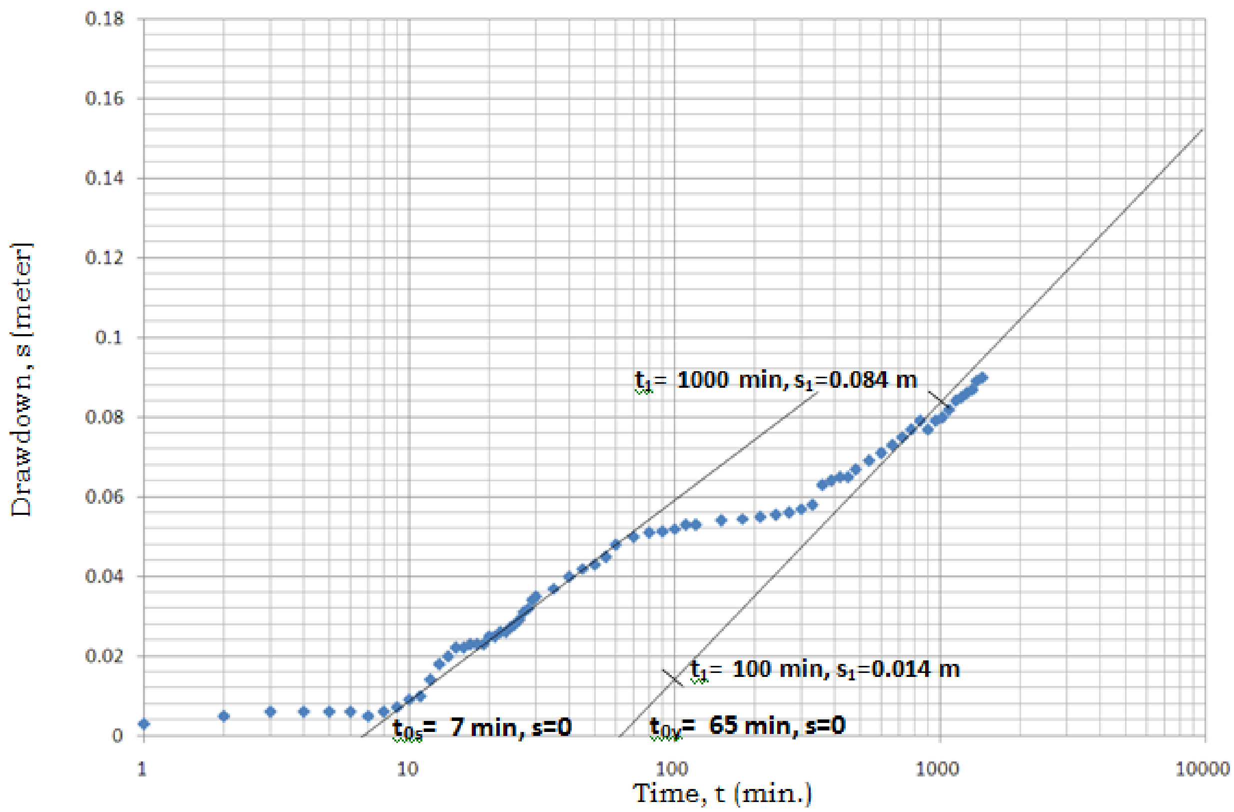


Figure 5.42 : Drawdown(s) versus time (t) data plotted on a semi-log scale.

Straight line fitted through the data as shown from the basis for the calculation of hydraulic parameters.

The transmissivity of the aquifer,

$$T = \frac{2.3\ Q}{4\Pi\Delta s} = \frac{2.3\times(25\times24)}{4\Pi\ (0.084-0.014)} = 1569.609\ \text{m}^2/\text{day}$$

The specific yield is

$$S_y = \frac{2.25Tt_{oy}}{r^2} = \frac{2.25\times1569.609\times(65\times\frac{1}{1440})}{35^2} = 0.13$$

The confined storativity,

$$S = \frac{2.25Tt_{os}}{r^2} = \frac{2.25\times1569.609\times(7\times\frac{1}{1440})}{35^2} = 0.014$$

The calculation for the rests observation well for different discharges and different distances from the pumping well are same as above.

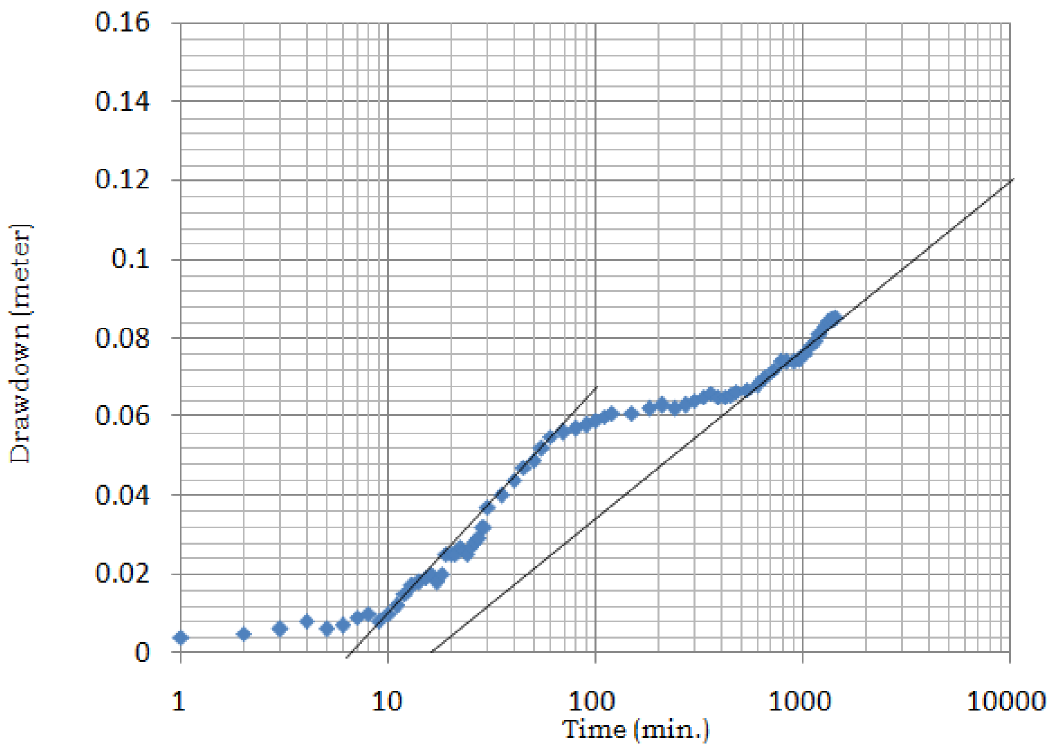


Figure 5.43 : Drawdown vs. time for observation well 2 in 1st pumping test

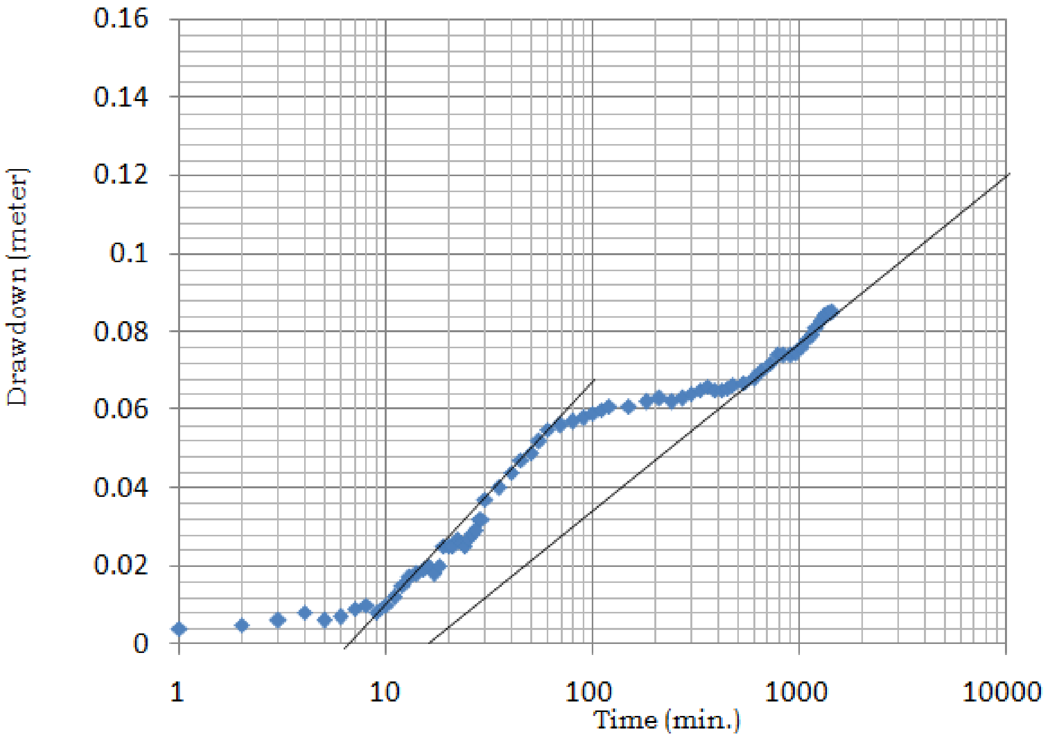


Figure 5.44 : Drawdown vs. time for observation well 3 in 1st pumping test

For pumping rate, Q= 30 m^3/hr

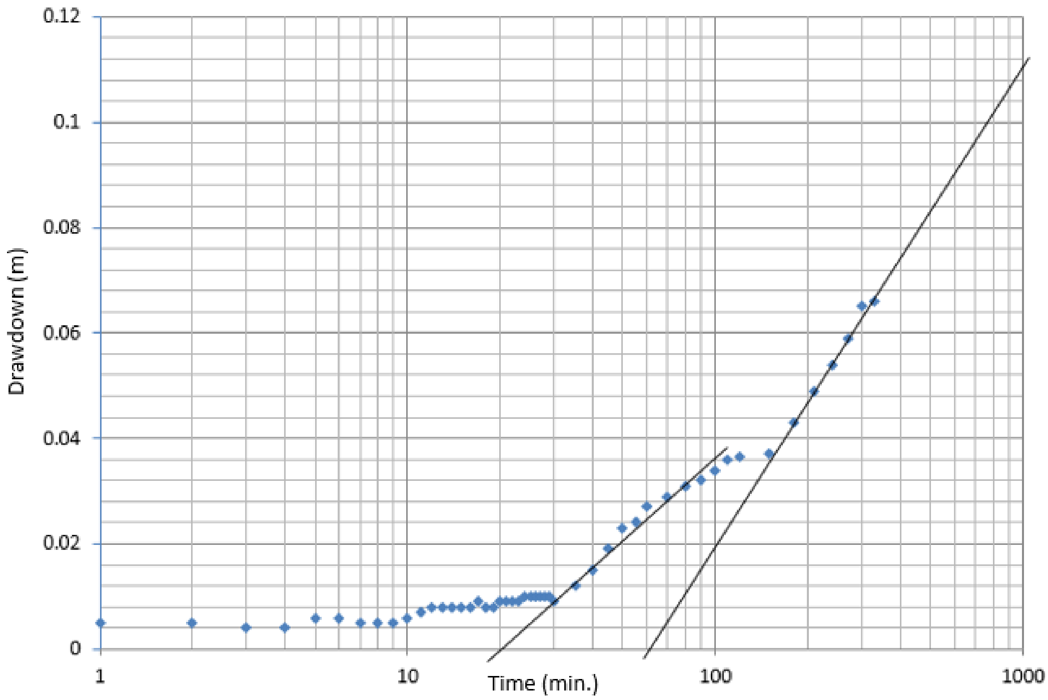


Figure 5.45 : Drawdown vs. time for observation well 1

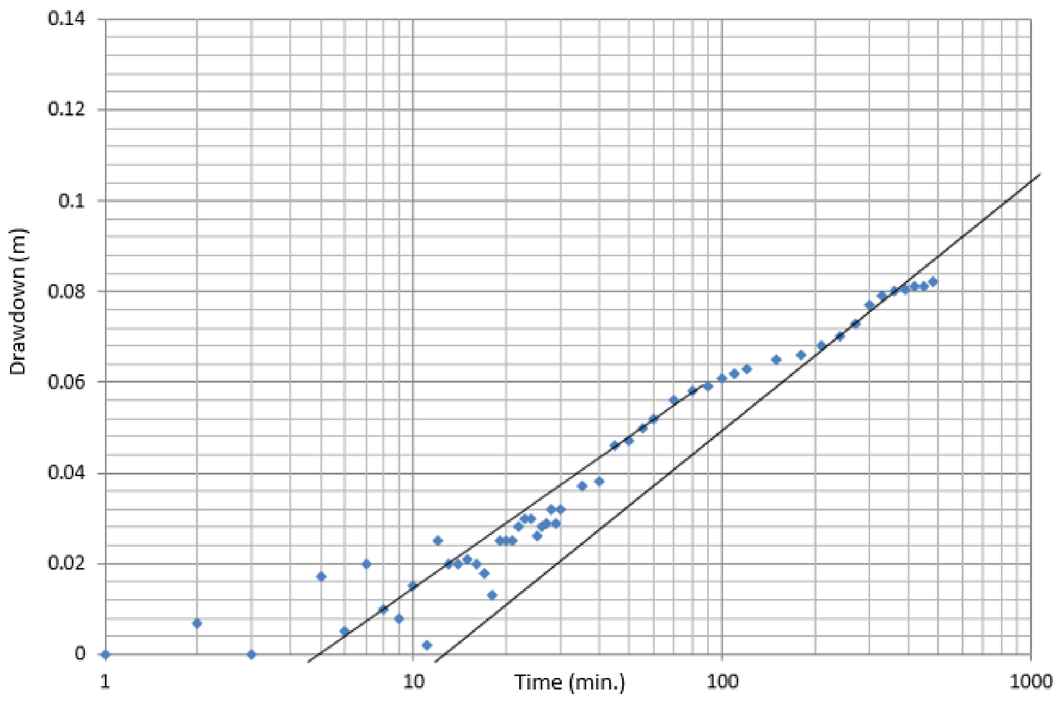


Figure 5.46 : Drawdown vs. time for observation well 2 in 1st pumping test

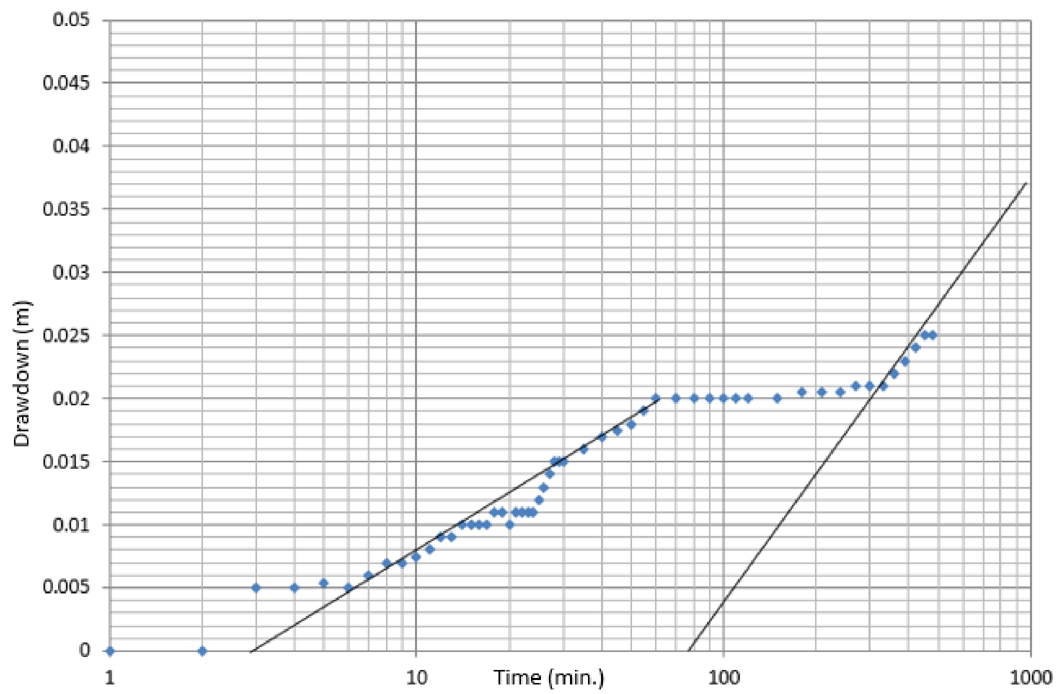


Figure 5.47 : Drawdown vs. time for observation well 3 in 1st pumping test

For pumping rate, Q= 44 m^3/hr.

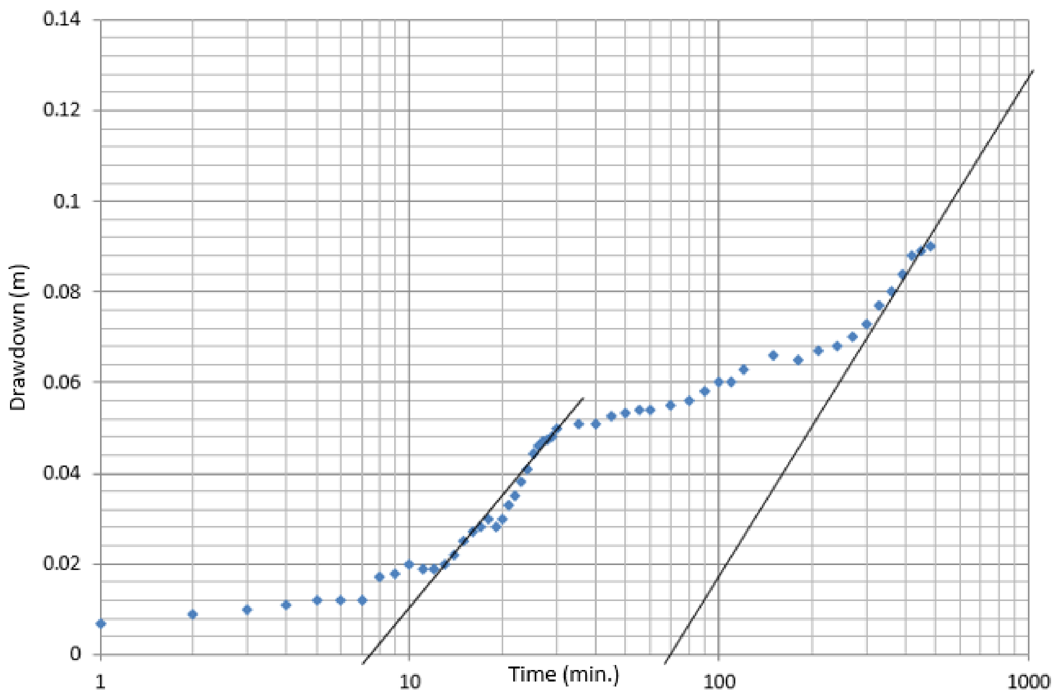


Figure 5.48 : Drawdown vs. time for observation well 1 in 1st pumping test

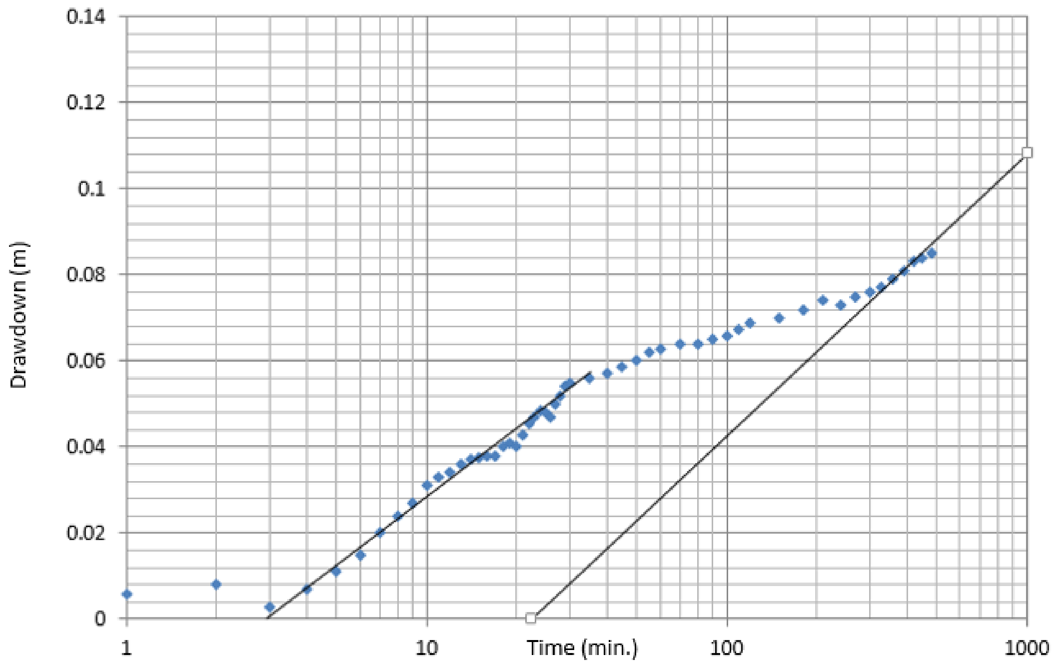


Figure 5.49 : Drawdown vs. time for observation well 2 in 1st pumping test

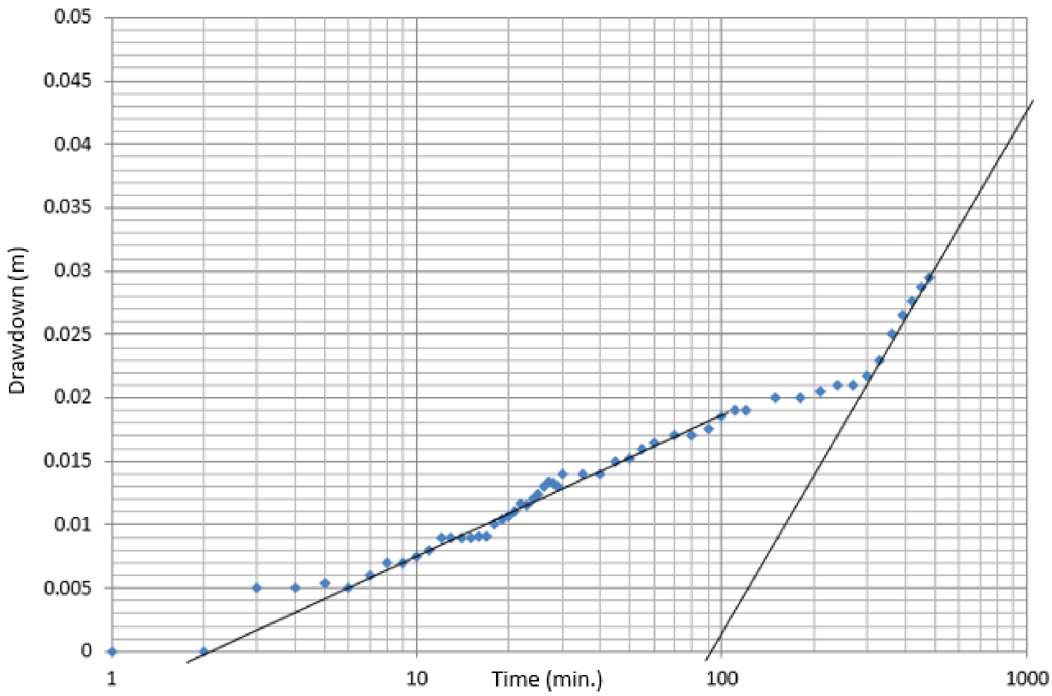


Figure 5.50 : Drawdown vs. time for observation well 3 in 1st pumping test

Table 5.22 : Summarized results for 1st pumping test in Damodar river

Name of the well	Radial distance from the pumping well (m)	Discharge rate (m^3/hr.)	Maximum Drawdown (m)	Transmissivity (m^2/day)	Specific yield	Confined storativity
Obs-1*	35	25	0.09**	1569.609	0.130	0.014
Obs-2	65		0.085	1308.007	0.007	0.003
Obs-3	90		0.04	1716.759	0.029	0.003
Average values for discharge rate 25m^3/hr.				**1531.458**	**0.056**	**0.006**
Obs-1	35	30	0.084	1433.121	0.11	0.035
Obs-2	65		0.082	2354.413	0.011	0.004
Obs-3	90		0.025	3877.857	0.06	0.002
Average values for discharge rate 30m^3/hr.				**2555.13**	**0.06**	**0.014**
Obs-1	35	44	0.09	1790.517	0.157	0.017
Obs-2	65		0.085	3021.497	0.026	0.003
Obs-3	90		0.029	4716.483	0.0819	0.002
Average values for discharge rate 44m^3/hr.				**3176.165**	**0.088**	**0.007**

2nd Pumping Test in Damodar river

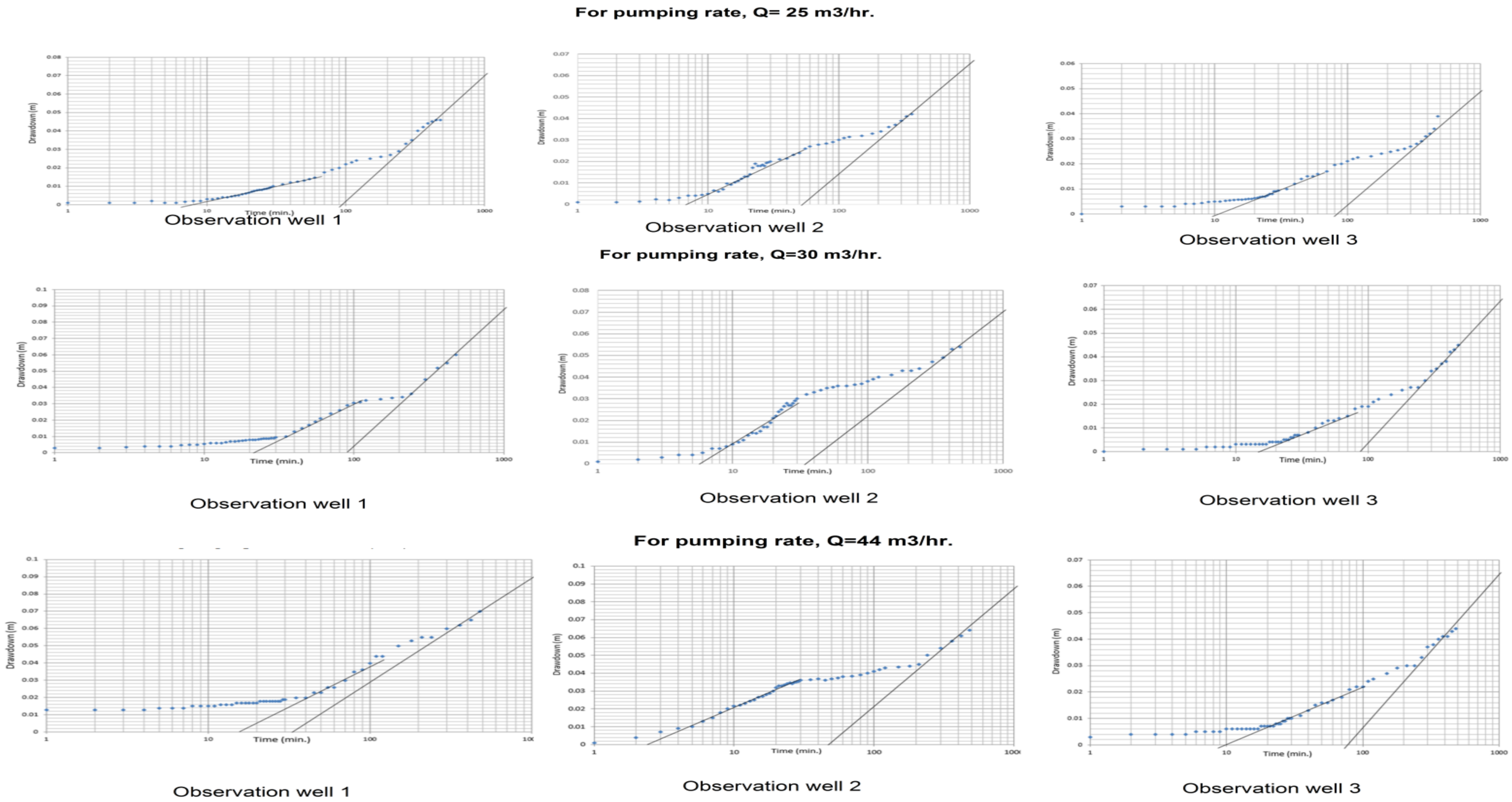


Figure 5.51 : Drawdown vs. time for 2nd Pumping Test

Table 5.23: Summarized results for 2nd pumping test in Damodar river

Name of the well	Radial distance from the pumping well (m)	Discharge rate (m^3/hr.)	Maximum drawdown (m)	Transmissivity (m^2/day)	Specific yield	Confined storativity
Obs-1*	35	25	0.046	1649.739	0.189	0.015
Obs-2	65		0.042	2154.365	0.041	0.006
Obs-3	90		0.039	2441.614	0.038	0.004
Average values for discharge rate 25m^3/hr.				**2081.906**	**0.089**	**0.008**
Obs-1	35	30	0.060	1569.609	0.18	0.040
Obs-2	65		0.054	2690.758	0.035	0.006
Obs-3	90		0.045	2197.452	0.038	0.007
Average values for discharge rate 30m^3/hr.				**2152.606**	**0.084**	**0.018**
Obs-1	35	44	0.070	2197.452	0.174	0.045
Obs-2	65		0.064	2929.936	0.053	0.028
Obs-3	90		0.044	3170.095	0.049	0.007
Average values for discharge rate 44m^3/hr.				**2765.828**	**0.091**	**0.026**

*Obs- Observation well

3rd Pumping Test

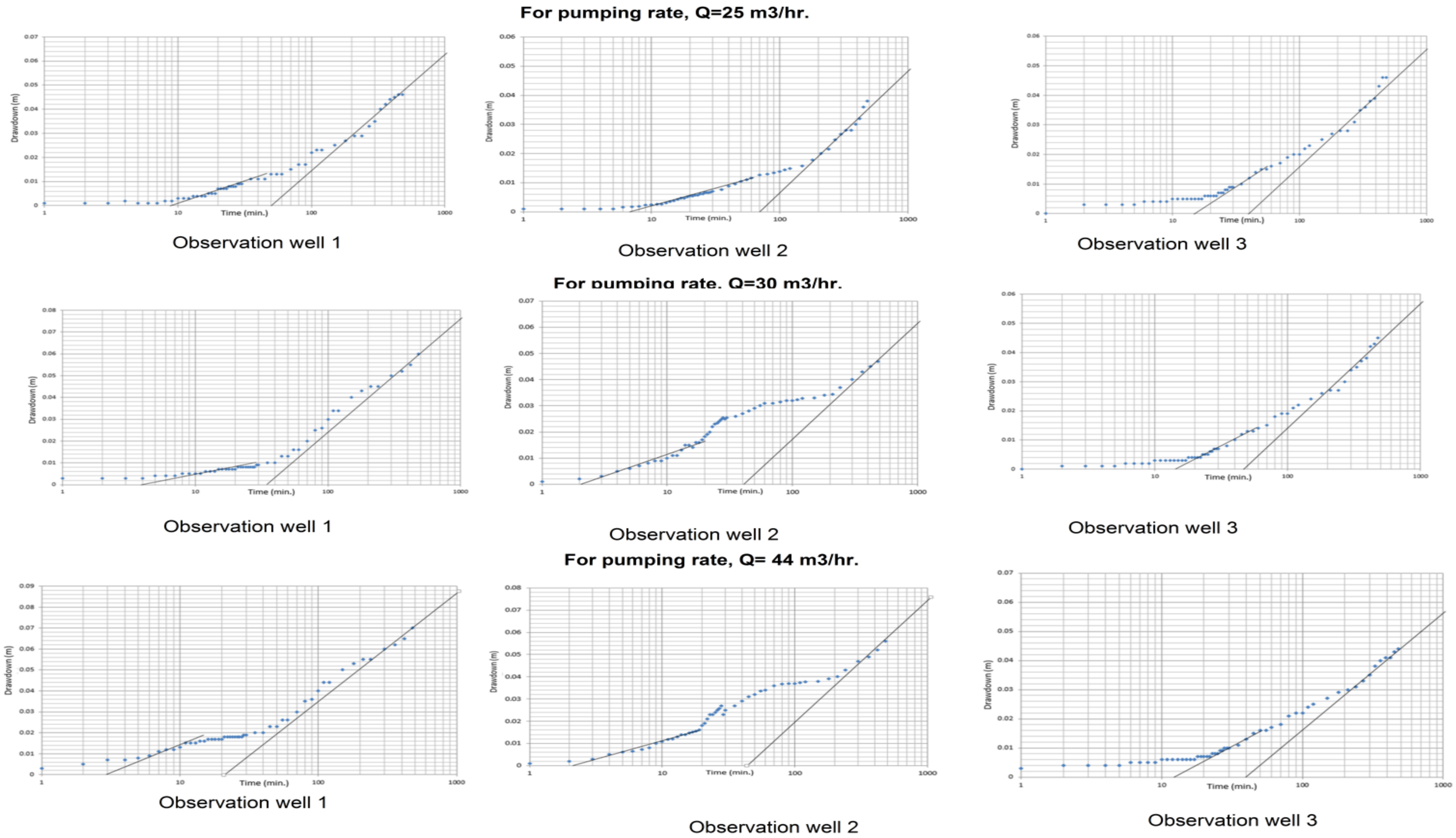


Figure 5.52 : Drawdown vs. time for 2nd Pumping Test

Table 5.24 : Summarized results for 3rd pumping test in Damodar river

Name of the well	Radial distance from the pumping well (m)	Discharge rate (m^3/hr.)	Maximum drawdown (m)	Transmissivity (m^2/day)	Specific yield	Confined storativity
Obs-1*	35	25	0.046	2242.298	0.143	0.026
Obs-2	65		0.038	2616.015	0.068	0.007
Obs-3	90		0.045	2746.815	0.021	0.008
Average values for discharge rate 25m^3/hr.				**2535.043**	**0.077**	**0.014**
Obs-1	35	30	0.060	2535.522	0.113	0.012
Obs-2	65		0.047	2996.526	0.044	0.002
Obs-3	90		0.045	3066.212	0.028	0.009
Average values for discharge rate 30m^3/hr.				**2866.087**	**0.062**	**0.008**
Obs-1	35	44	0.070	3718.765	0.104	0.014
Obs-2	65		0.056	3515.923	0.057	0.003
Obs-3	90		0.044	4716.482	0.036	0.012
Average values for discharge rate 44m^3/hr.				**3983.724**	**0.066**	**0.010**

*Obs- Observation Well

4th Pumping Test

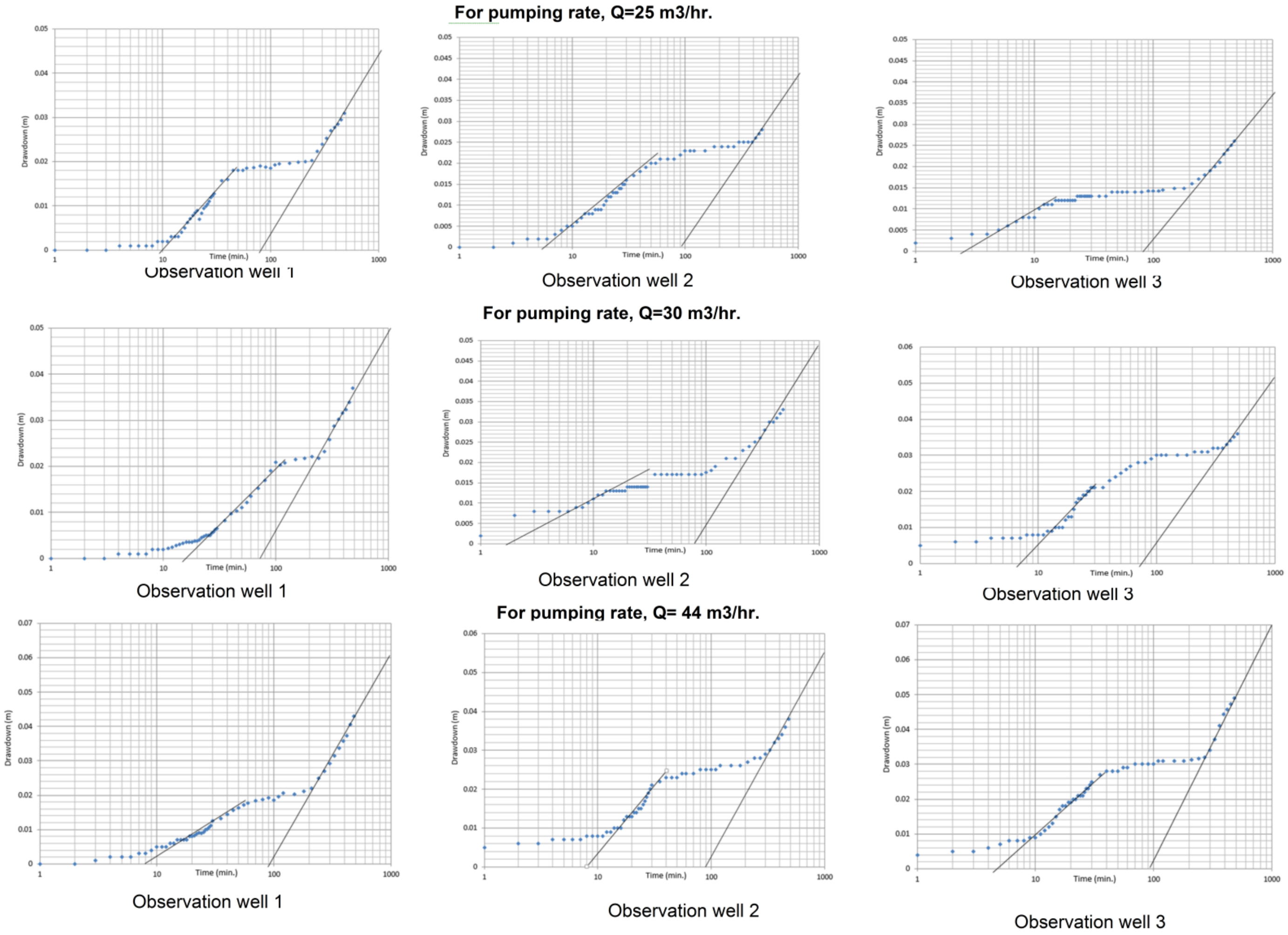


Figure 5.53 : Drawdown vs. time for 2nd Pumping Test

Table 5.25 : Summarized results for 4th pumping test in Damodar

Name of the well	Radial distance from the pumping well (m)	Discharge rate (m^3/hr.)	Maximum drawdown (m)	Transmissivity (m^2/day)	Specific yield	Confined storativity
Obs-1*	35	25	0.031	2746.815	0.280287	0.035036
Obs-2	65		0.028	2712.904	0.090296	0.00602
Obs-3	90		0.026	3052.017	0.052986	0.001472
Average values for discharge rate 25m^3/hr.				**2837.245**	**0.14119**	**0.014176**
Obs-1	35	30	0.037	3066.212	0.273769	0.062576
Obs-2	65		0.033	2996.526	0.088655	0.002216
Obs-3	90		0.036	2866.242	0.040915	0.00387
Average values for discharge rate 30m^3/hr.				**2976.327**	**0.134446**	**0.022887**
Obs-1	35	44	0.043	3222.93	0.369979	0.032887
Obs-2	65		0.038	3069.457	0.102164	0.009081
Obs-3	90		0.049	3021.497	0.052457	0.002914
Average values for discharge rate 44m^3/hr.				**3104.628**	**0.174867**	**0.014961**

*Obs- Observation Well

Table 5.26 : 5th Pumping in Damodar river

: BLOCK : NITURIA; RIVER: DAMODAR								
SL NO.	**LOCATION**	**MAIN WELL**				**OBSERVATION WELL**		
		PARAMETER Transmissivity (m^2/day)	**GPS Location Of the Main Well**	**Radius Of Influence (m)**	**DISCHARGE (m3/hr)**	**OBSERVATION WELL NO.**	**PARAMETERS** Transmissivity (m^2/day)	Storativity
5		989.8433		30.35	25	OBS-1	1569.609	0.009162
						OBS-2	1464.968	0.000715
						OBS-3	6867.038	0.001774
		1187.812		40.03	30	OBS-1	5273.885	0.006592
						OBS-2	1757.962	0.000858
						OBS-3	8789.809	0.00227
		1742.124		100.01	44	OBS-1	7735.032	0.009669
						OBS-2	2578.344	0.001259
						OBS-3	12085.99	0.003121

***Observation well-3 constructed in left Bank of the River**

Name Of the Well	**Maximum Drawdown(m) At different discharge**			**Recovery Time for different discharge (Minutes)**			**Distance from Main Pumping Well (m)**
	25(m^3/hr)	**30(m^3/hr)**	**44(m^3/hr)**	**25(m^3/hr)**	**30(m^3/hr)**	**44(m^3/hr)**	
Main Well	0.569	0.57	0.575	225	248	302	-
Observation Well-1	0.051	0.058	0.07	221	246	307	35
Observation Well-2	0.107	0.155	0.162	243	253	261	65
Observation Well-3	0.045	0.047	0.049	229	237	259	90

Table 5.27 : 6th Pumping test in Damodar river

BLOCK : SATURI; RIVER: DAMODAR

SL NO.	LOCATION	MAIN WELL				OBSERVATION WELL		
		PARAMETER	GPS Location Of the Main Well	Radius Of Influence (m)	DISCHARGE (m3/hr)	OBSERVATION WELL NO.	PARAMETERS	
		Transmissivity (m^2/day)					Transmissivity (m^2/day)	Storativity
6		1831.21		186	25	OBS-1	1569.609	0.001962
						OBS-2	1426.917	0.000697
						OBS-3	6867.038	0.001774
		976.6454		191	30	OBS-1	5732.885	0.006592
						OBS-2	1734.831	0.000847
						OBS-3	8789.809	0.00227
		1137.505		205	44	OBS-1	7735.032	0.009669
						OBS-2	1807.25	0.000882
						OBS-3	12085.99	0.003121

***Observation well-3 constructed in left Bank of the River**

Name Of the Well	Maximum Drawdown At different discharge(m)			Recovery Time for different discharge (Minutes)			Distance from Main Pumping Well (m)
	25(m^3/hr)	30(m^3/hr)	44(m^3/hr)	25(m^3/hr)	30(m^3/hr)	44(m^3/hr)	
Main Well	0.569	0.574	0.581	209	233	293	-
Observation Well-1	0.052	0.08	0.081	231	244	249	35
Observation Well-2	0.157	0.162	0.163	236	239	250	65
Observation Well-3	0.044	0.048	0.05	260	271	273	90

5.3. Recharge Dynamics and River Profile Survey

Stream geomorphology or river profile that is, the topography and sediment distribution within the stream channel, the occurrence of pools, riffles, channel-bars, alcoves, etc., exerts a major control not only on the stream flow itself, , and also on the spatial distribution and dynamics of groundwater flows underneath and near the stream. Stream geomorphology also is a critical factor in understanding stream ecology. A complete map of the current Stream River System geomorphology and a monitoring system to record its dynamics are therefore essential for understanding groundwater – surface water interactions. To evaluate cross-sections of the River main stem have been mapped as part of an effort to create a geomorphic indexing system. GIS maps depict the Kangsabati and Damodar River basins, along its main stem and Bandu river tributary of Kangsabati river basin. The following field work is carried out support of not only better understanding Yield of the river System, but especially to better understand groundwater – surface water interaction, recharge dynamics of these rivers. Table 5.28,Table 5.29,Table 5.30and show the details location of river bed with respect to GPS reference and also its elevation, left bank and right Banks side slopes, river width and chainage point from survey starting point to ending points in Kangsabati and Bandu River in Purulia-I, Hura, Arsha Block in Purulia District. Figure 5.54. show map of Study area where different hydro-geological investigation has been done. Figure 5.56 Maps also indicating the starting point, confluence point of Kangsabati and Bandu river with respect to GPS location. Figure 5.55,Figure 5.57, and Figure 5.58shows the 3-d Surface model of River bed profile of Kangsabati and Bandu river in different blocks. Table 5.31 shows the cross-sectional discharge calculation in Kangsabati river in different blocks.

Figure 5.59 and **Error! Reference source not found.** shows the graph of mean velocity along the distance of Purulia-I block Starting from Kotloi and mean velocity along the distance in Bandu River. In Hura block continuous surface flow had not been observed during survey period. Similarly in Damodar River Table 5.32, Table 5.33,and Table 5.34 Show the details location of river bed with respect to GPS reference and also its elevation, left bank and right Banks side slopes, river width and change point from survey starting point to ending point in Damodar river Santuri, Nituria and Raghunathpur – II block Figure 5.60,Figure 5.62 and Figure 5.64 shows the Maps of different cross section of 500 meter interval in Damodar River in Santuri, Nituria, and Raghunathpur-2 Block. Figure 5.61, Figure 5.63 and Figure 5.65shows the 3-d Surface model of River bed profile in above mention Three blocks in Raghunathpur sub-division. Table 5.35 shows the sectional discharge calculation in different yield testing point of Damodar river basin. In Table 5.36 Section shows the Velocity and DO profile data of different cross-section of Damodar River Basin in Santuri, Neturia and Raghunath Purulia-I blocks of Purulia district. Figure 5.66 depict the location of the sampling point and Figure 5.67 and Figure 5.68 shows the graphs of DO and Velocity profuile.

River profile of Kangsabati River

The purpose of this survey is to provide information on how the flow and chemistry of the main rivers change along their length. This, in turn, provides information regarding groundwater recharge processes. The survey should be timed such that the rivers are not in flood and yet such that there is discernible flow along the majority of the length of the rivers. Following observation are taken in for river profile survey.

- GPS location (digital latitude and longitude), and marking manually on map and village and district name. Digital photographs of the river channel (upstream and downstream views), and record date and time of photo. Description of the river at this point – single deep channel, many shallow channels etc.
- Width of flowing river channel and its depth at the centre of the channel and velocity of water in centre of channel (m/s). Estimate flow rate of river
- Water temperature, electrical conductivity and pH, using field meters.
- use of the river water (human drinking water, livestock, irrigation) • Record land use in area. • Record any major changes in flow since the previous (upstream) station • Take water samples, as indicated be

Table 5.28 : The cross-sectional River Profile of Kangsabati in Hura Block

C/S No	GPS ID			Chainage (km)	Width (m)	Left bank side slope (Indirection Of River Flow i.e. Hura Block)	Right bank side slope (Indirection Of River Flow i.e.Puncha Block)
	Latitude (N)	Longitude (E)	Elevation (m)				
ds-500	23°15'09.83"	86°31'13.99"	182	Ds-500	83	0.04	0.06
cs-0	23°15'04.50"	86°30'57.65"	185	0	108.67	0.0148	0.0297
cs-1	23°14'58.80"	86°30'40.87"	183	0.5	114.34	0.0466	0.0555
cs-2	23°14'57.35"	86°30'26.26"	187	1	84.43	0.0669	0.0206
cs-3	23°15'11.10"	86°30'17.20"	186	1.5	80.05	0.0456	0.1231
cs-4	23°15'26.38"	86°30'10.68"	188	2	87.35	0.0925	0.0889
cs-5	23°15'26.44"	86°29'56.12"	186	2.5	94.48	0.0671	0.0416
cs-6	23°15'15.38"	86°29'43.64"	190	3	133.61	0.0869	0.0223
cs-7	23°15'25.75"	86°29'32.11"	188	3.5	98.61	0.023	0.0591
cs-8	23°15'41.68"	86°29'27.42"	190	4	87.83	0.0516	0.1169
cs-9	23°15'52.24"	86°29'15.70"	192	4.5	98.57	0.0445	0.0524
cs-10	23°15'48.18"	86°28'59.66"	191	5	128.43	0.1	0.0418
cs-11	23°15'52.07"	86°28'43.03"	192	5.5	79.66	0.0449	0.076
cs-12	23°15'43.33"	86°28'26.47"	191	6	118.33	0.1158	0.0792
cs-13	23°15'38.86"	86°28'13.54"	191	6.5	123.38	0.0956	0.0496
cs-14	23°15'50.94"	86°28'05.48"	195	7	93.89	0.0892	0.1017
cs-15	23°16'04.59"	86°28'03.92"	197	7.5	128.32	0.0636	0.053
cs-16	23°16'02.24"	86°27'47.33"	195	8	91.87	0.0406	0.0342
cs-17	23°15'59.50"	86°27'30.46"	194	8.5	125.9	0.0735	0.057
cs-18	23°16'06.67"	86°27'13.88"	194	9	137.65	0.0506	0.0659
cs-19	23°16'20.54"	86°27'06.81"	196	9.5	102.06	0.0747	0.0215
cs-20	23°16'23.72"	86°26'57.81"	195	10	91.56	0.0503	0.0196

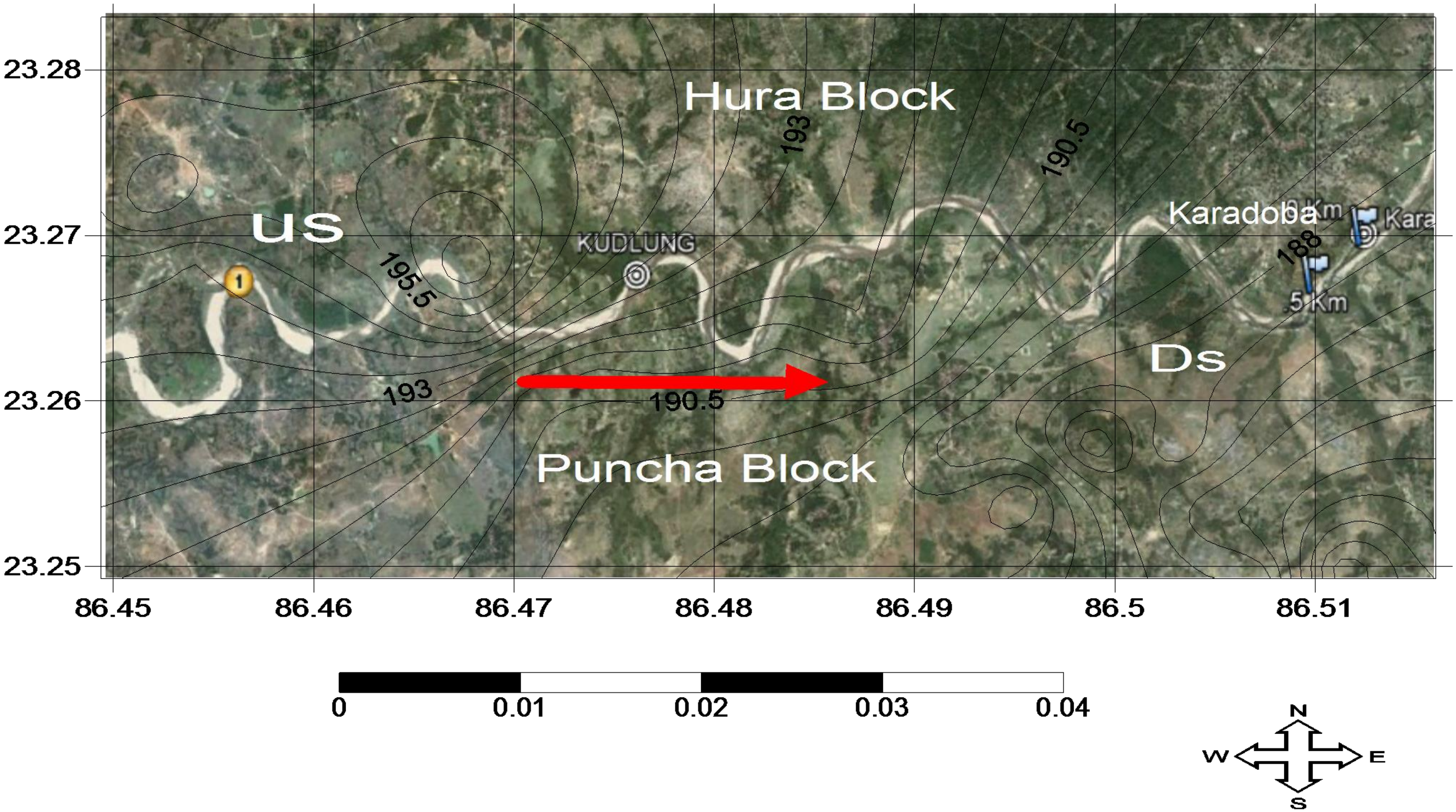


Figure 5.54 : The cross-sectional Map of Kangsabati River In the Hura Block

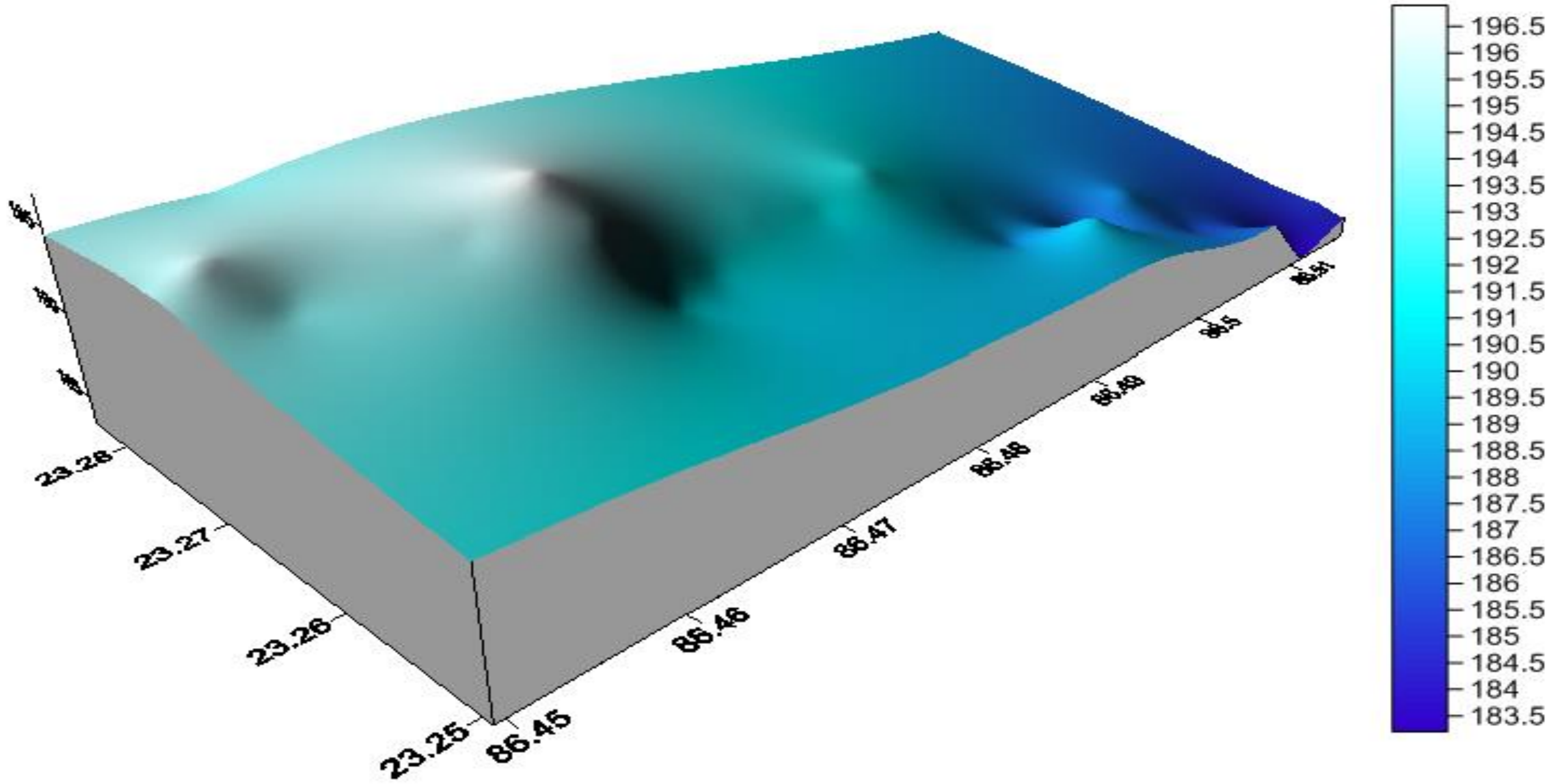


Figure 5.55 : Kangsabati river bed profile in Hura Block

River profile of Kangsabati in purulia Block

Table 5.29 : The cross-sectional river profile of Kangsabati in Purulia-I Block

C/S	GPS ID			Chainage (km)	Width (m)	Left bank side slop (Indirection Of River Flow i.e. Arsha Block)	Right bank side slop (Indirection Of River Flow i.e.Arsha Block)
	Latitude (N)	Longitude (E)	Elevation (m)				
ds-500	23°17'57.72"	86°19'34.57"	216	Ds-500	159	0.03	0.01
cs-0	23°17'57.72"	86°19'34.57"	216	0	185.94	0.0867	0.0121
cs-1	23°18'09.89"	86°19'46.33"	221	0.5	104.8	0.0298	0.0259
cs-2	23°18'22.17"	86°19'35.95	217	1	160.7	0.0507	0.0219
cs-3	23°18'28.19"	86°19'19.48"	218	1.5	169.87	0.0298	0.0394
cs-4	23°18'43.41"	86°19'13.12"	220	2	167.78	0.0276	0.0256
cs-5	23°18'57.46"	86°19'04.56"	221	2.5	121.97	0.0099	0.0276
cs-6	23°18'56.97"	86°18'47.38"	221	3	158.25	0.0449	0.0385
cs-7	23°19'11.93"	86°18'38.80"	223	3.5	82.4	-0.0269	0.0182
cs-8	23°19'19.77"	86°18'24.64"	225	4	81.55	-0.0158	0.0525
cs-9	23°19'14.54"	86°18'07.31"	222	4.5	139	0.0327	0.0308
cs-10	23°19'13.54"	86°17'49.02"	225	5	138.27	0.0148	0.0414
cs-11	23°19'20.85"	86°17'34.60"	224	5.5	119.5	0.008	0.0917
cs-12	23°19'34.92"	86°17'28.33"	225	6	109.58	-0.0068	0.0269
cs-13	23°19'38.33"	86°17'11.78"	226	6.5	76.39	-0.0266	0.0285
cs-14	23°19'50.35"	86°16'59.80"	226	7	90.65	0.0236	0.0152
cs-15	23°19'59.66	86°16'47.00"	224	7.5	102.6	0.0153	0.0181
cs-16	23°19'53.46"	86°16'30.61"	230	8	199.96	0.0179	0.0668
cs-17	23°19'56.35"	86°16'13.86"	229	8.5	100.12	0.0234	0.0169
cs-18	23°20'07.69"	86°16'01.03"	231	9	115.97	0.0696	0.0259
cs-19	23°20'20.91"	86°15'50.62"	230	9.5	181.73	0.0243	0.0207
cs-20	23°20'29.99"	86°15'36.33"	234	10	39.44	0.0139	0.0326

River profile of Bandu River

Table 5.30 : The cross-sectional River Profile of Bandu River in Arsha Block

C/S	GPS ID			Chainage (km)	Width (m)	Left bank side slop (Indirection Of River Flow i.e. Arsha Block)	Right bank side slop((Indirection Of River Flow i.e. Arsha Block)
	Latitude(N)	Longitude(E)	Elevation (m)				
1	23°19´05.89´´	86°18´34.24´´	222	DS 500	83.22	0.032	0.023
2	23°18´52.24´´	86°18´26.65´´	224	0	50.45	0.02	0.032
3	23°18´42.55´´	86°18´16.85´´	226	0.5	115	0.02	0.016
4	23°18´43.87´´	86°18´01.78´´	229	1	50	0.05	0.04
5	23°18´39.43´´	86°17´45.64´´	228	1.5	68.57	0.01	0.03
6	23°18´26.92´´	86°17´37.29´´	228	2	213	0.01	0.03
7	23°18´23.60´´	86°17´20.34´´	230	2.5	111	0.014	0.00854
8	23°18´28.59´´	86°17´01.17´´	231	3	92.39	0.017	0.01
9	23°18´19.13´´	86°16´47.75´´	236	3.5	103	0.028	0.01
10	23°18´17.87´´	86°16´26.87´´	235	4	126.80	0. 013	0.015
11	23°18´09.58´´	86°16´13.29´´	239	4.5	75.16	0.012	0.03
12	23°18´20.03´´	86°16´01.42´´	236	5	248	0.00746	0.0076
13	23°18´26.02´´	86°15´43.62´´	239	5.5	102.88	0.00925	0.011
14	23°18´18.29´´	86°15´29.10´´	240	6	101	0.021	0.02
15	23°18´12.19´´	86°15´16.36´´	242	6.5	104	0.00595	0.00558
16	23°18´23.49´´	86°15´07.38´´	242	7.0	262	0.0124	0.0245
17	23°18´35.15´´	86°14´58.17´´	246	7.5	130	0.0210	0.0408
18	23°18´45.24´´	86°14´49.53´´	245	8.0	87	0.027	0.045
19	23°18´54.70´´	86°14´34.77´´	246	8.5	82	0.027	0.027
20	23°18´50.99´´	86°14´19.73´´	248	9	84	0.012	0.011
21	23°18´52.34´´	86°14´05.16´´	246	9.5	68	0.017	0.027
22	23°18´57.93´´	86°14´56.57´´	250	10	71	0.027	0.047

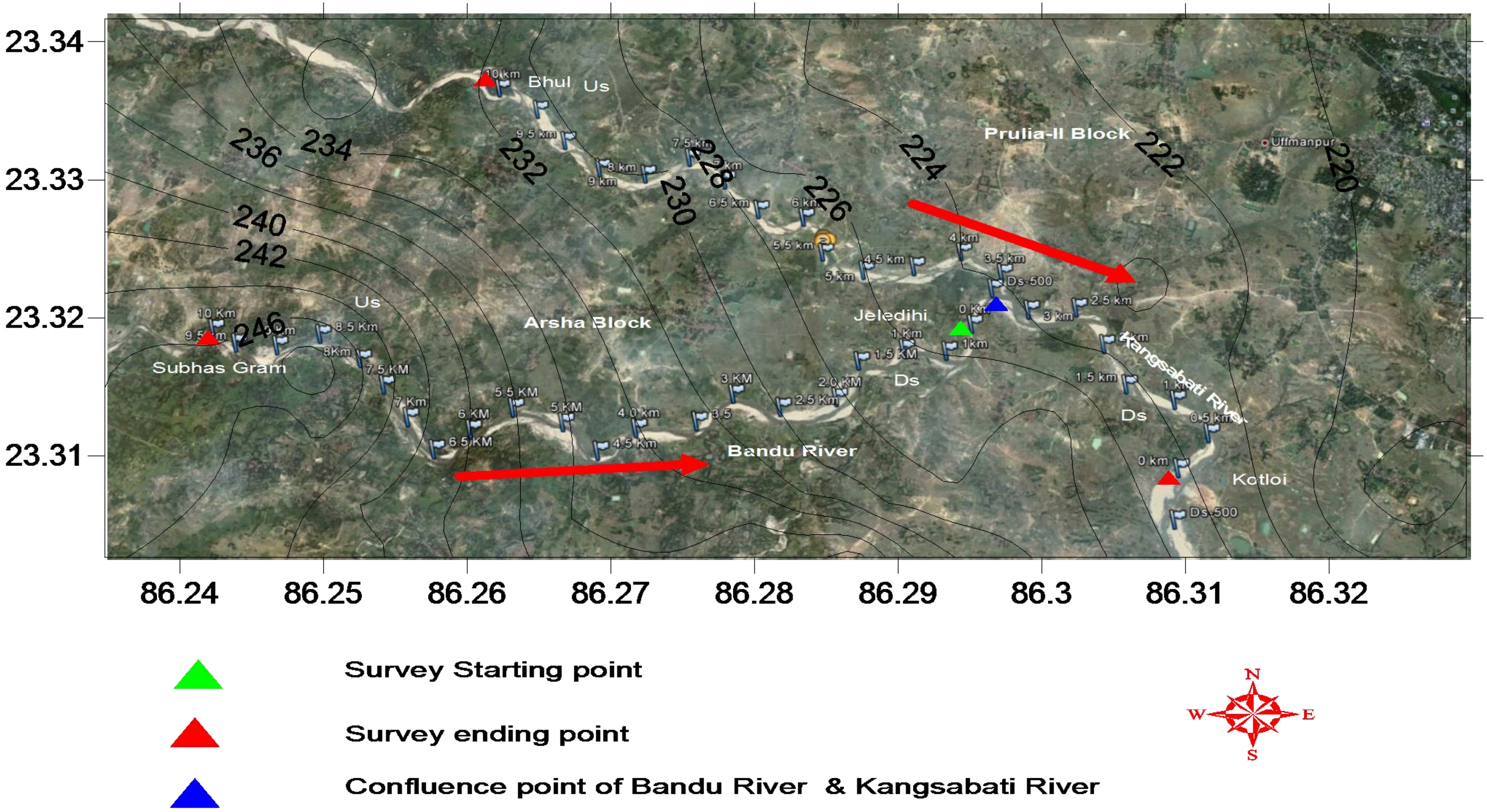


Figure 5.56 : River cross section Of Kangsabati River and Bandu River

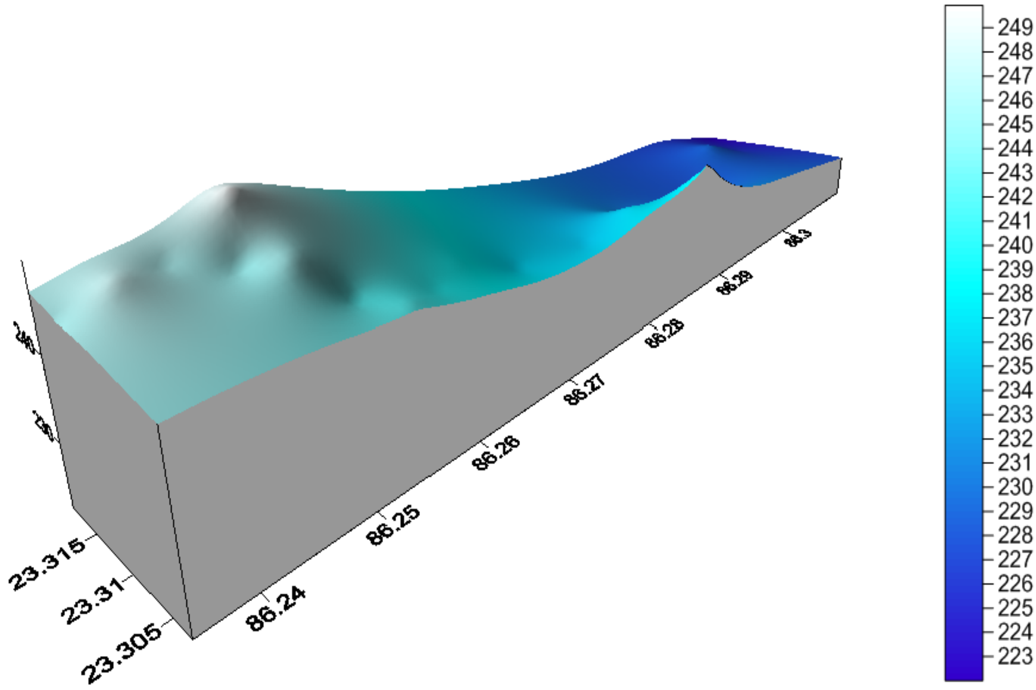


Figure 5.57 : River Bed profile of Bandu River in Arsha Block

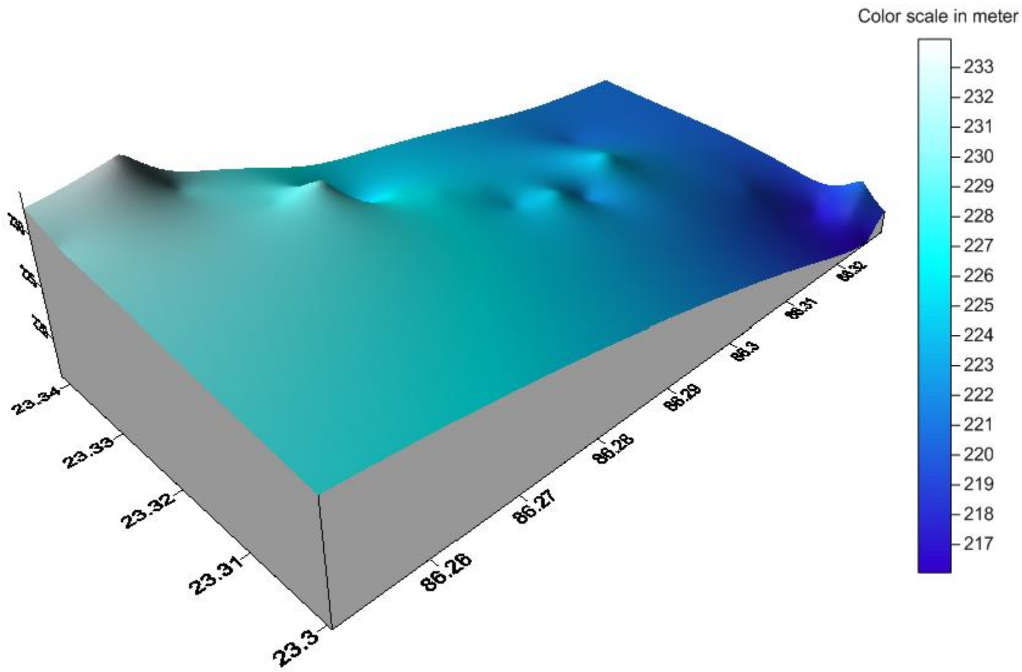


Figure 5.58 : River Bed profile of Kangsabati River in Purulia-II Block

Recharge Dynamics of Kangsabati River

The sectional discharge has been calculated based on hydraulic head, Transmissibility and the length of the section for each production well as depicted in table 5.31 The transmissibility has been found by running 72 hrs pumping operations for different wells.

Table 5.31 : Sectional discharge calculation of Kangsabati River

WELL	Discharge (Pump) (m^3/hr)	DRAWDOWN (m)	Transmissivity (T) (m^2/day)	Hydraulic gradient (i)	Length of the section (m)	Discharge (Sectional) (m^3/Day)	Discharge (Sectional) (m^3/hr)
K-1	25	0.7	1372	0.012182	160	2674.153	111.423
K-2	32	1.17	1538	0.005697	130	1139.058	47.46076
K-3	25	1.43	2073	0.004308	152	1357.434	56.55973
K-4	65	1.93	2856	0.008154	240	5589.078	232.8782
K-5	18	1.89	1205	0.006727	155	1256.486	52.3536
K-6	30	1.07	1569	0.012364	185	3588.731	149.5305

Section Wise Velocity and DO Profile

Velocity profiles of 10 km upstream from 0-0 Reference point set at Kotloi Village shown in Figure: 5.59.maximum velocity found 0.215 m/s.

Velocity profile in different cross-section

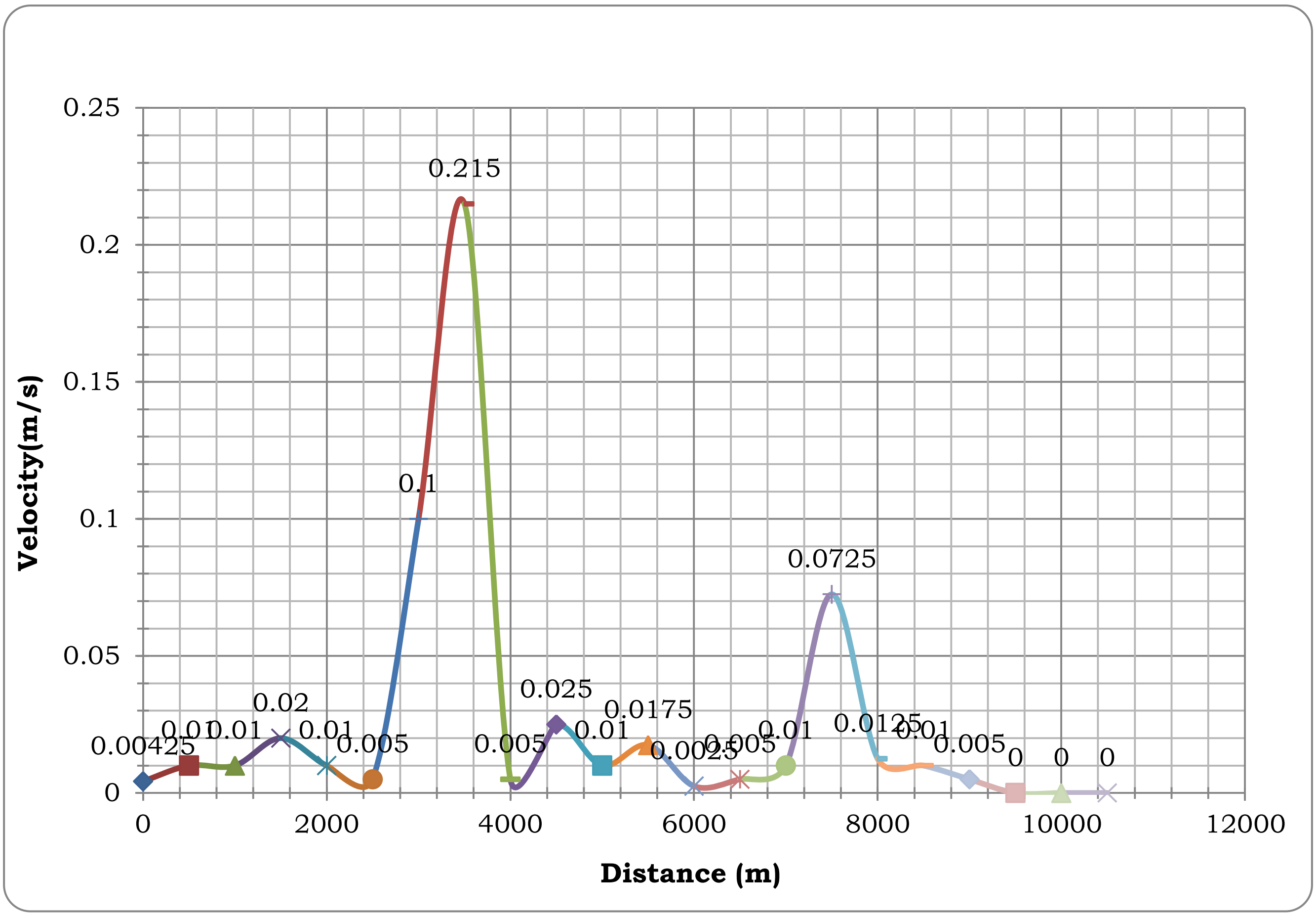


Figure 5.59 : Velocity profile for Kangshabati river

Photo.5.22 (a & b) : River Flow Measurement

Photo 5.23 : Measurement of Velocity Near Jeledihi

Photo 5.24 : Mesurement of Velocity in betwwen Beldihi bridge and Baligara

Photo 5.25 : River Bed Condition

Photo 5.26:River Bed condition at Kudlung(Hura Block)

Damodar RiverRiver profile Survey

Table 5.32 : Details information's for river profile at Santuri Block

Latitude (N)	Longitude (E)	Elevation (m)	Cross section	Chainage (km)	Width (m)	Left bank side slop (Indirection Of River Flow i.e. Bardhman and Jharkhand)	Right bank side slop (Indirection Of River Flow i.e.Purulia)
23°37'38.11"	86°56'7.95"	82	ds-500	-500	596	0.08	0.064
23°37'38.20"	86°55'49.93"	82	cs-0	0	573	0.09	0.025
23°37'37.97"	86°55'50.18"	82	cs-1	0.5	583	0.06	0.028
23°37'35.67"	86°55'14.82"	82	cs-2	1	432	0.03	0.06
23°37'37.50"	86°54'56.96"	83	cs-3	1.5	494	0.021	0.04
23°37'43"	86°54'39"	88	cs-4	2	579	0.018	0.038
23°37'48.46"	86°54'24.35"	82	cs-5	2.5	638	0.06	0.05
23°37'58.47"	86°54'10.68"	82	cs-6	3	523	0.035	0.032
23°38'10.57"	86°53'59.54"	84	cs-7	3.5	480	0.021	0.04
23°38'25.54"	86°53'51.19"	82	cs-8	4	468	0.01	0.02
23°38'40.59"	86°53'41.85"	84	cs-9	4.5	474	0.03	0.04
23°38'52.60"	86°53'34.00"	87	cs-10	5	573	0.02	0.09
23°39' 2.91"	86°53'21.44"	89	cs-11	5.5	575	0.028	0.028
23°39'10.80"	86°53'35.72"	85	cs-12	6	647	0.03	0.03
23°39'13.54"	86°52'48.16"	85	cs-13	6.5	452	0.08	0.037
23°39'34.31"	86°52'37.71"	108	cs-14	7	676	0.109	0.03
23°39'16.20"	86°52'12.20"	92	cs-15	7.5	897	0.08	0.05
23°39'23.85"	86°51'58.13"	88	cs-16	8	960	0.01	0.03
23°39'31.99"	86°51'42.20"	86	cs-17	8.5	743	0.057	0.04
23°39'44.76"	86°51'32.26"	87	cs-18	9	598	0.10	0.10
23°39'59.36"	86°51'23.02"	87	cs-19	9.5	545	0.05	0.06
23°40'13.70"	86°51'13.60"	88	cs-20	10	542	0.02	0.08

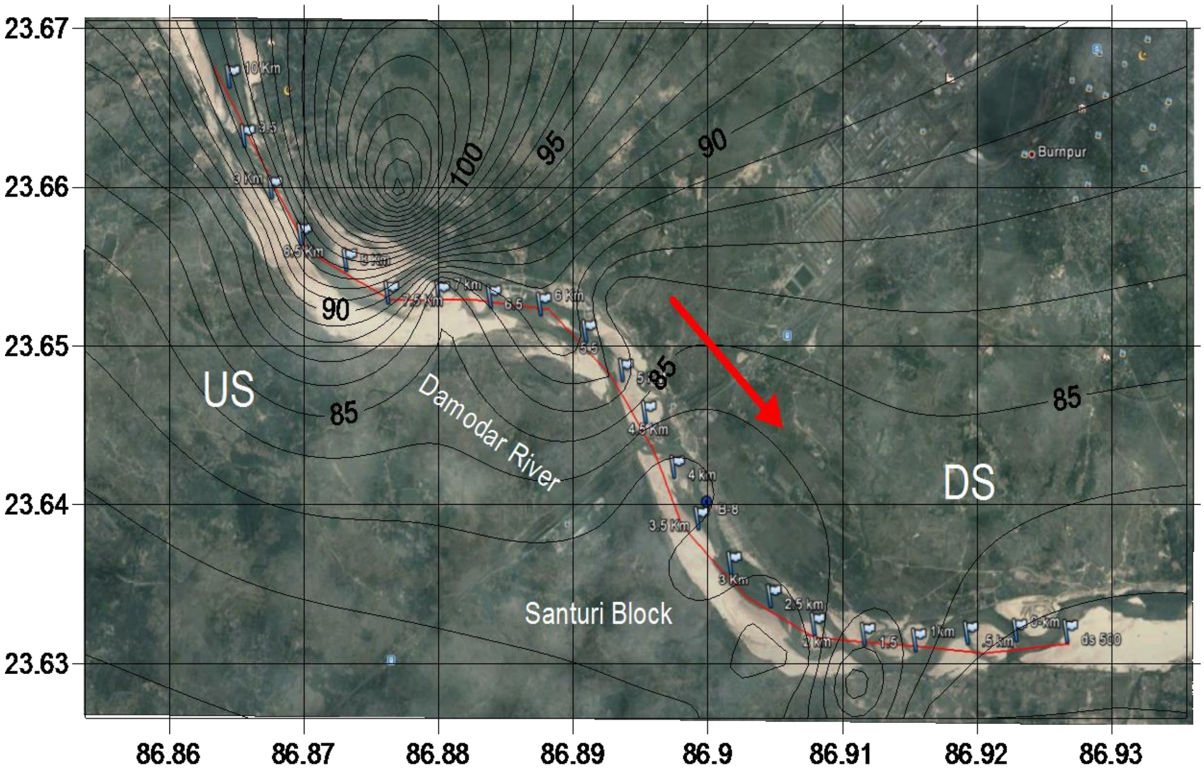


Figure 5.60 : The Profile Map of Santuri Block

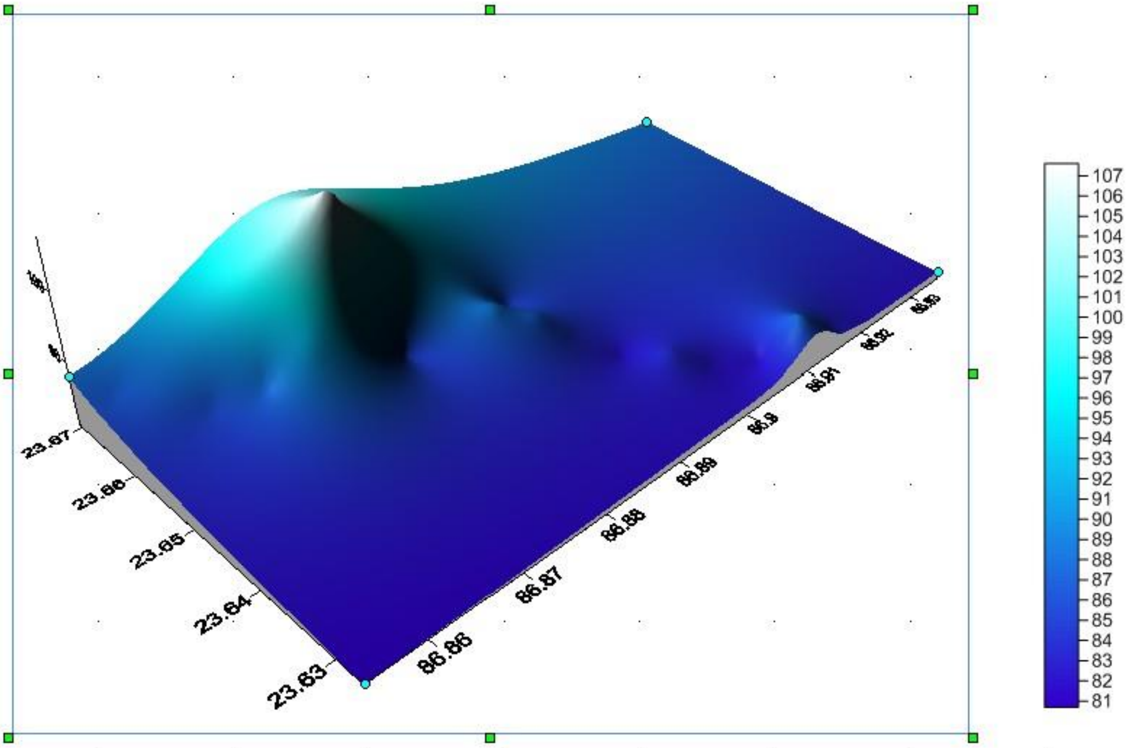


Figure 5.61 : River profile of Santuri block

Table 5.33 : Details information's for river profile at Nituria Block

Latitude(N)	Longitude(E)	Elevation (m)	Cross section	Chainage (km)	Width(m)	Left bank side slop (Indirection Of River Flow i.e. Bardhman)	Right bank side slop((Indirection Of River Flow i.e.Purulia)
23°37'37.11"	86°56'7.95"	82	ds-500	-500	596	0.08	0.064
23°37'38.20"	86°55'49.93"	82	cs-0	0	573	0.09	0.025
23°37'37.97"	86°55'50.18"	82	cs-1	0.5	583	0.06	0.028
23°37'35.67"	86°55'14.82"	82	cs-2	1	432	0.03	0.06
23°37'37.50"	86°54'56.96"	83	cs-3	1.5	494	0.021	0.04
23°37'43.36"	86°54'39"	88	cs-4	2	579	0.018	0.038
23°37'48.46"	86°54'24.35"	82	cs-5	2.5	638	0.06	0.05
23°37'58.47"	86°54'10.68"	82	cs-6	3	523	0.035	0.032
23°38'10.57"	86°53'59.54"	84	cs-7	3.5	480	0.021	0.04
23°38'25.54"	86°53'51.19"	82	cs-8	4	467.89	0.01	0.02
23°38'40.59"	86°53'41.85"	84	cs-9	4.5	474	0.03	0.04
23°38'52.60"	86°53'34.00"	87	cs-10	5	573	0.02	0.09
23°39' 2.91"	86°53'21.44"	89	cs-11	5.5	575	0.028	0.028
23°39'10.80"	86°53'35.72"	85	cs-12	6	647	0.03	0.03
23°39'13.54"	86°52'48.16"	85	cs-13	6.5	452	0.08	0.037
23°39'34.31"	86°52'37.71"	108	cs-14	7	676	0.109	0.03
23°39'16.20"	86°52'12.20"	92	cs-15	7.5	897	0.08	0.05
23°39'23.85"	86°51'58.13"	88	cs-16	8	960	0.01	0.03
23°39'31.99"	86°51'42.20"	86	cs-17	8.5	743	0.057	0.04
23°39'44.76"	86°51'32.26"	87	cs-18	9	598	0.1	0.1
23°39'59.36"	86°51'23.02"	87	cs-19	9.5	545	0.05	0.06
23°40'13.70"	86°51'13.60"	88	cs-20	10	542	0.02	0.08

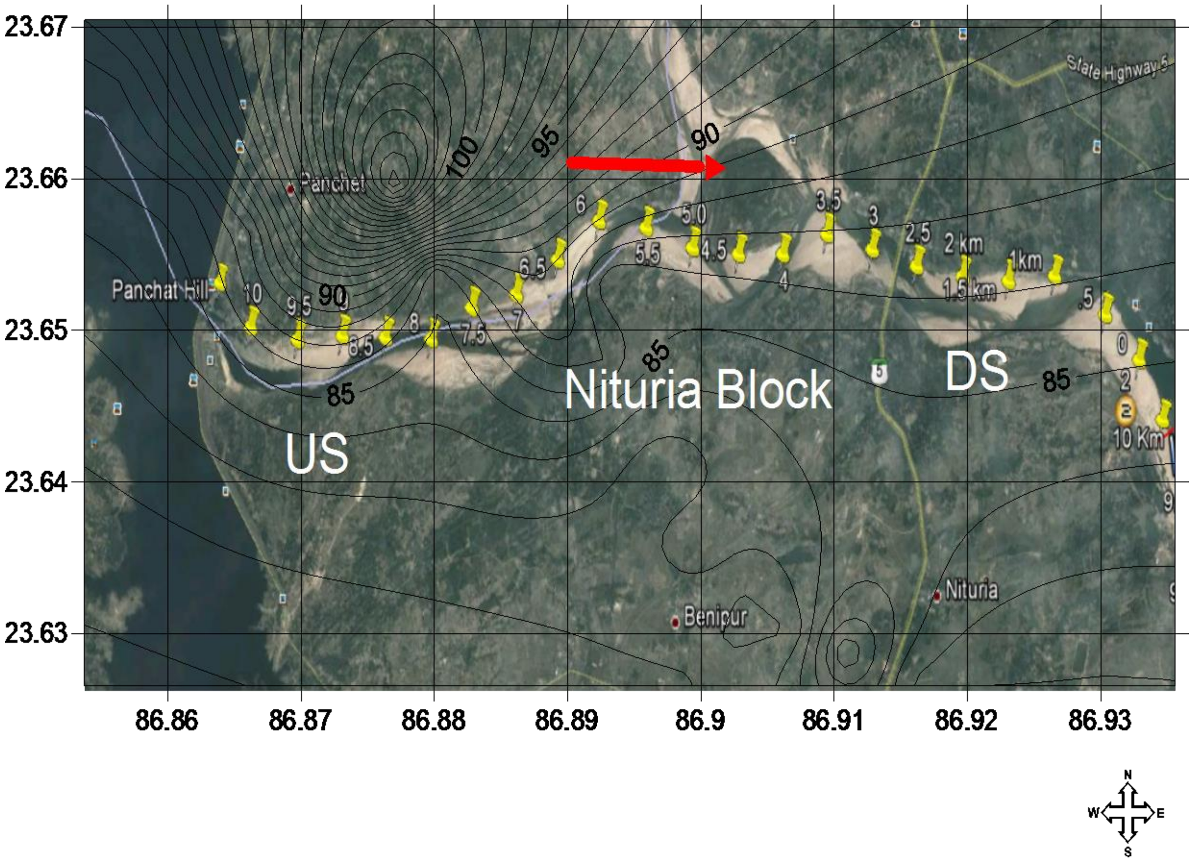


Figure 5.62: The Cross section Map of Nituria Block

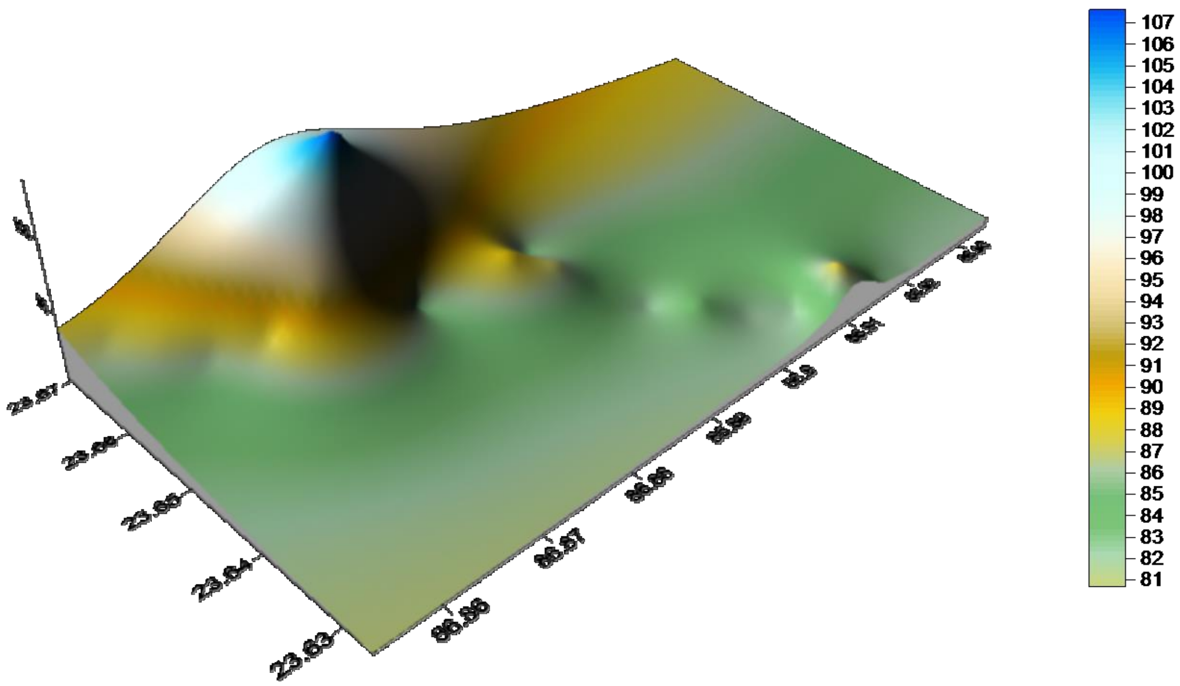


Figure 5.63 : Nituria River Bed Profile

Table 5.34 : Details information's for river profile at Raghunathpur-2 Block

Latitude(N)	Longitude(E)	Elevation (m)	Cross section	Chainage (km)	Width(m)	Left bank side slop (Indirection Of River Flow i.e. Bardhman)	Right bank side slop((Indirection Of River Flow i.e.Purulia)
23°38'09.36	86°34'00.33	129	ds-500	0	119.96	0.03	0.013
23°38'24.63	86°33'53.97	127	cs-0	0.5	1031.41	0.037	0.008
23°38'38.13	86°33'41.57	130	cs-1	1	2024.7	0.025	0.007
23°38'31.31	86°33'24.92	130	cs-2	1.5	1695.23	0.041	0.011
23°38'27.28	86°33'07.35	129	cs-3	2	1471.55	0.01	0.008
23°38'20.68	86°32'51.71	127	cs-4	2.5	939.39	0.04	0.011
23°38'11.12	86°32'37.00	128	cs-5	3	925.11	0.024	0.041
23°38'02.52	86°32'21.97	127	cs-6	3.5	666.72	0.016	0.004
23°37'58.03	86°32'04.82	126	cs-7	4	984.06	0.021	0.003
23°37'54.79	86°31'46.86	128	cs-8	4.5	1153.41	0.026	0.0074
23°37'54.73	86°31'29.17	127	cs-9	5	778.64	0.047	0.0036
23°37'59.31	86°31'11.91	127	cs-10	5.5	597.45	0.025	0.032
23°38'03.8	86°30'55.15	127	cs-11	6	619.98	0.013	0.059
23°38'09.40	86°30'38.15	128	cs-12	6.5	591.6	0.048	0.057
23°38'14.36	86°30'21.95	127	cs-13	7	707.35	0.03	0.022
23°38'19.8	86°30'05.03	127	cs-14	7.5	688.19	0.011	0.045
23°38'22.7	86°29'47.94	127	cs-15	8	516.36	0.032	0.062
23°38'25.45	86°30'30.34	127	cs-16	8.5	419.92	0.024	0.066
23°38'22.38	86°29'13.65	128	cs-17	9	511.28	0.031	0.075
23°38'23.43	86°28'56.32	128	cs-18	9.5	784.06	0.038	0.054
23°38'21.85	86°28'39.15	128	cs-19	10	911.87	0.028	0.082

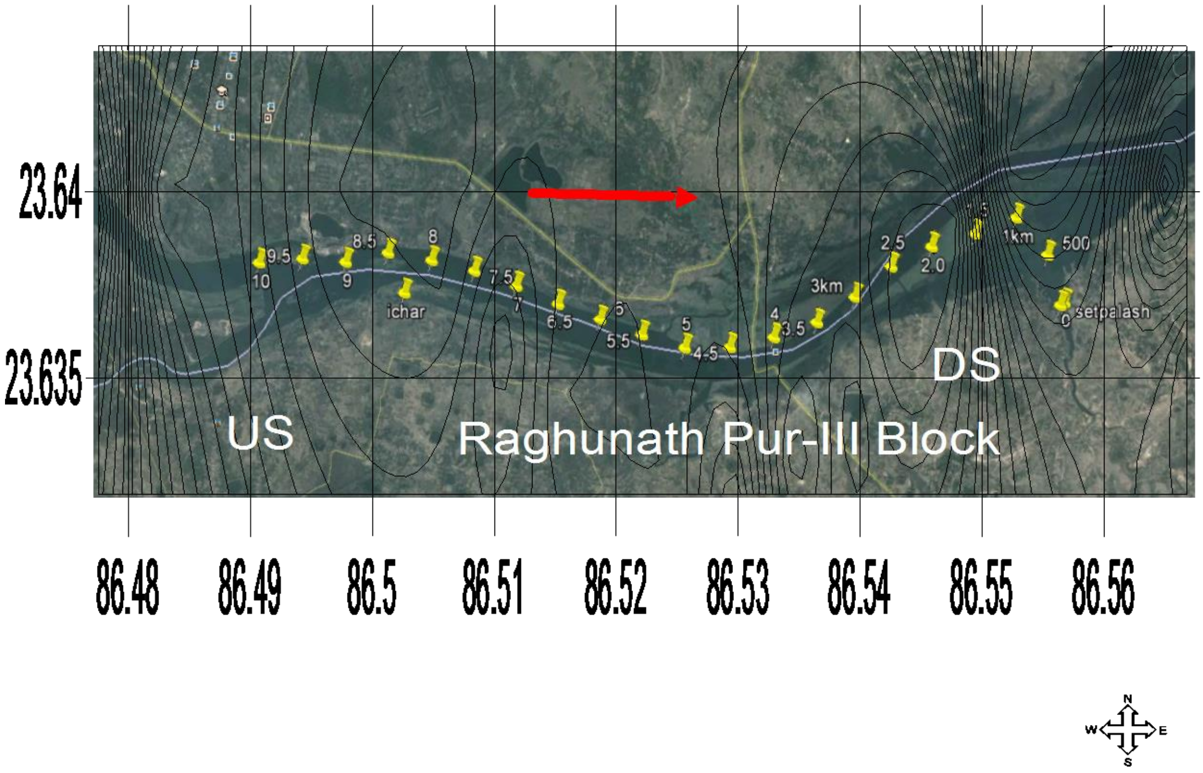


Figure 5.64 : Raghunathpur-II River Bed Profile

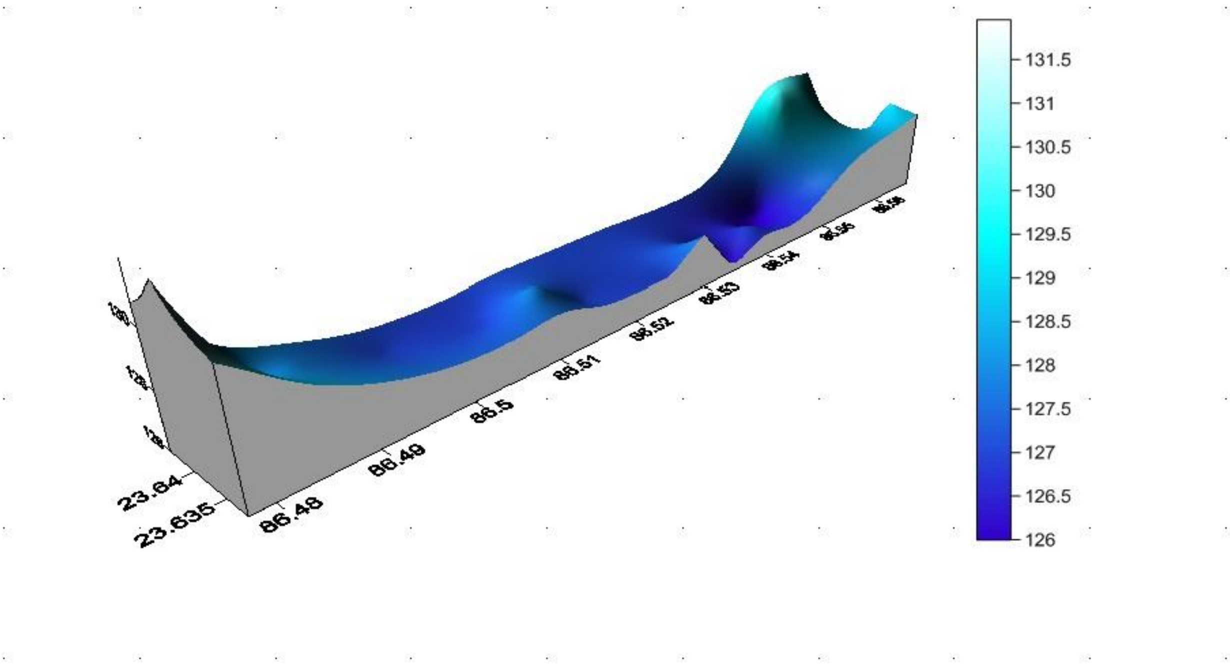


Figure 5.65: River Profile In Raghunathpur-II

Recharge Dynamics of the river bed

The sectional discharge has been calculated based on hydraulic head, Transmissibility and the length of the section for each production well as depicted in Figure 5.39. The transmissibility has been found by running 72 hrs pumping operations for different wells.

Table 5.35 : Sectional discharge calculation of Damodar River

WELL	Discharge (Pump) (m^3/hr)	DRAWDOWN (m)	Transmissivity (T) (m^2/day)	Hydraulic gradient (i)	Length of the section (m)	Discharge (Sectional) (m^3/Day)	Discharge (Sectional) (m^3/hr)
PWD-1	44	0.85	2975	0.02	180	10710	446.25
PWD-2	44	0.85	1831	0.012	260	5712.72	238.03
PWD-3	44	0.78	2578	0.0174	165	7401.438	308.3933
PWD-4	44	0.435	2396	0.009	180	3881.52	161.73

Table 5.36 : Velocity and DO at different Cross-Sections of River Damodar

Location	Cross Section	GPS Data		ADC Reading		Depth (m)
		Latitude(N)	Longitude(E)	Depth below surface Where velocity is measured(m)	Velocity (m/s)	
Swet-Palash	RB	23°38'08.4''	86°38'08.4''	0.05	0.01	1.3
	MID-I	28°38'09.5''	86°34'00.3''	0.06	0.08	2.5
	LB	23°38'10.0''	86°34'01.6''	0.12	0.09	1.4
Baghardanga Ghat	RB	23°38'5.6''	86°44'26.4''	0.01	0.68	1.2
	LB	23°41'4.8''	86°48'19.8''	0.06	0.08	1.5
Deuli Ghat	RB	23°41'7.4''	86°47'43.8''	0.24	0.97	1.4
	LB	23°41'14.7''	86°47'55.6''	0.1	0.87	2.2
Makgeu Ghat ,Saltore	RB	23°41'7.4''	86°47'43.8''	0.01	0.33	2.3
	LB	23°41'14.7''	86°47'55.6''	0.03	0.47	1.3
Ramnagar(Point-1)	RB	23˚38'17.4"	86˚33'19.1"	0.12	0.06	2.4
	MID-I	23˚38'19.8"	86˚33'19.1"	0.05	0.15	1.4
	MID-II	23˚38'22.0"	86˚33'19.1"	0.03	0.1	1.4
	LB	23˚38'22.7"	86˚33'16.7"	0.04	0.28	1.4
Ramnagar(Point-2)	RB	23˚38'14.4"	86˚33'0.4"	0.01	0.19	1.3
	MID-I	23˚38'14.7"	86˚32'57.8"	0.3	0.14	1.9
	LB	23˚38'16.33"	86˚32'57.6"	0.08	0.07	1.5
Korgali Ghat	RB	23˚37'57.9"	86˚32'30.8"	0.03	0.1	1.5
	LB	23˚37'58.2"	86˚32'30.9"	0.01	0.05	1.4
Ichor	RB	23˚37'54"	86˚31'05.5"	0.01	0.59	4.1
	LB	23˚37'54.3"	86˚31'05.5"	0.02	0.48	3.7
Ichor(Point-2)	RB	23˚38'17.3"	86˚29'5.01"	0.01	0.3	1.6
	MID-I	23˚38'17.3"	86˚29'5.01"	0.06	0.02	2.9
	LB	23˚38'17.3"	86˚29'5.01"	0.16	0.07	1.8
	RB	23˚37'47.8"	86˚32'3.5"	0.42	0.18	2
	MID-I	23˚37'53.6"	86˚31'42.9"	0.01	1.61	2.8
	MID-II	23˚37'56.5"	86˚32'1.5"	0.01	1.62	5.5
	LB	23˚37'58.3"	86˚31'59.5"	0.01	0.28	1.6

DO Profile with respective depths at some different Cross-Sections

The method used in this research work the for measuring discharge, velocity is the current-meter method. In this method, the stream channel cross section is divided into numerous vertical subsections. In each subsection, the area is obtained by measuring the width and depth of the subsection, and the water velocity is determined using a current meter (left-side picture below). The discharge in each subsection is computed by multiplying the subsection area by the measured velocity shown in Figure 5.66. The total discharge is then computed by summing the discharge of each subsection. Other types of equipment and methods are used by for calibration to make current-meter measurements because of the wide range of stream conditions throughout the study area . Subsection width is generally measured using a cable, steel tape, and by application of GIS . Subsection depth is measured using a wading rod, if conditions permit, or by suspending a sounding weight from a calibrated cable and reel system off a bridge, cableway, or boat or through a hole drilled in sand bed. From Boat measurements are taken in Damodar river showed in Photo 5.27 below and location of sampling shown in Figure 5.66. maximum velocity is found 0.4 m/s near Deuli Ghat. Average velocity found between 0.05 to0.15 m/s at depth of 0 to 4 m deph. Deatil measurement values are depicted in Figure 5.67. DO values in surface water varies from 2 mg/l to 7.9 mg/l shown in Figure 5.68.

Figure 5.66: Location of velocity profile and DO profile survey

Velocity Profiles Measurement :

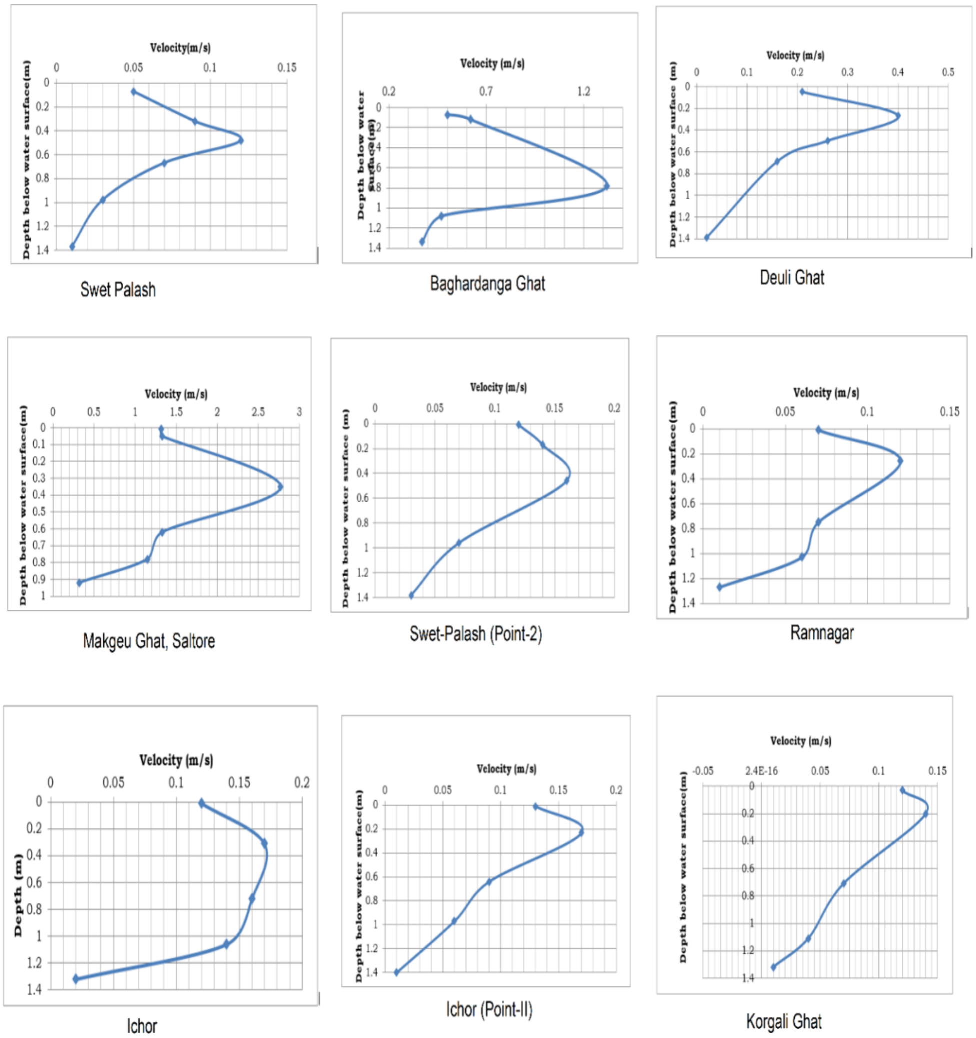


Figure 5.67 : Velocity profile measurement of Damodar river

Photo 5.27 : Velocity and DO measurement at Damodar river

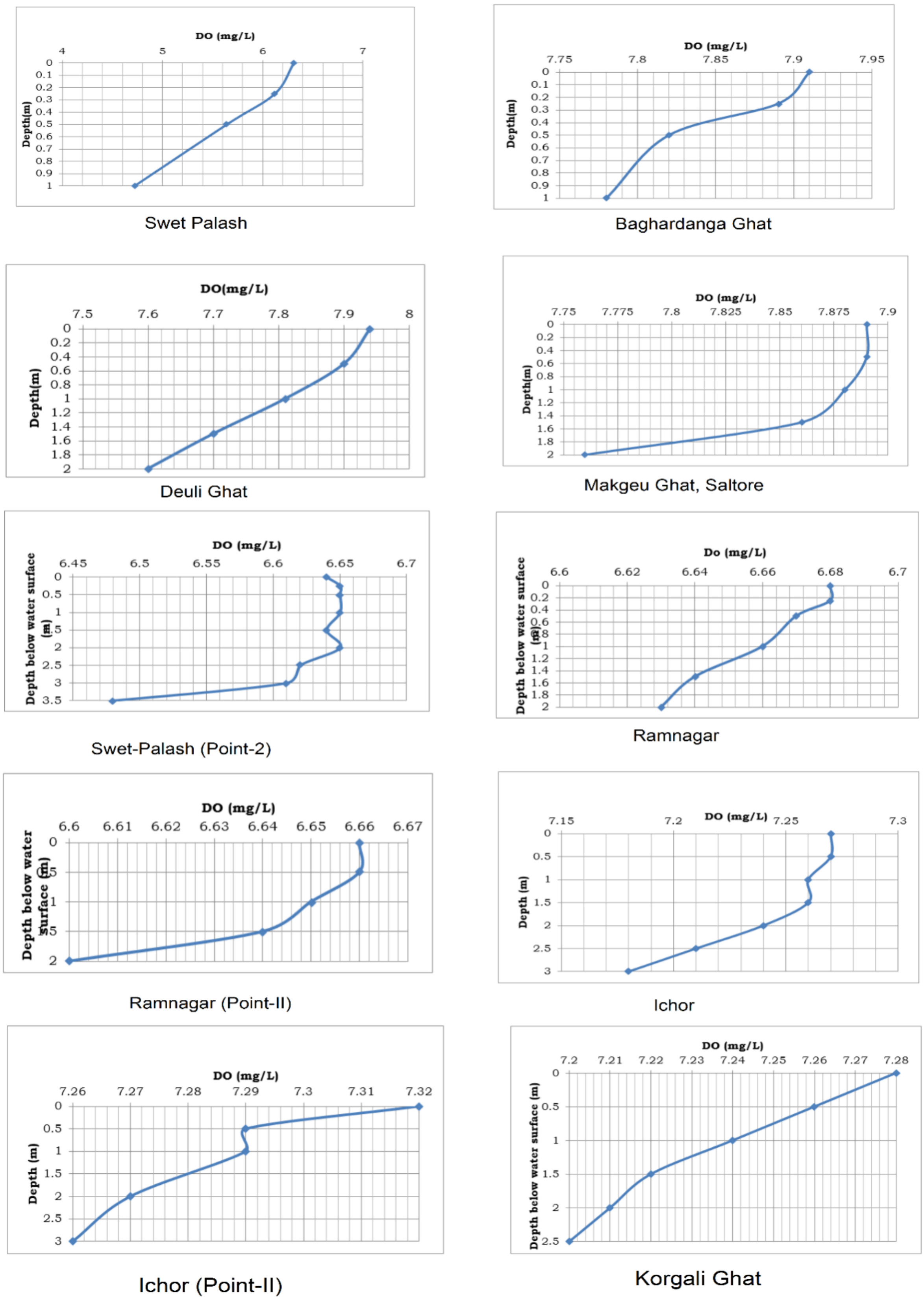


Figure 5.68 : DO profile measurement at Damodar River

5.4. Remarks

The survey and investigation chapter includes resonance survey, hydrological investigation, VES survey, hydrogeological investigation for aquifer parameter determination and river profile survey. Sustainable yield were also determined in this chapter through pumping tests. Results of different investigation are also summarized in this chapter which plays key inputs in numerical and field model development. The detail survey and experiment results of hydrological and hydrogeological critical areas of three river catchment plays key inputs for development of the integrated cost-effective technology model. It also depict the clear water resource scenario in the study area for future policy making and development planning.

CHAPTER-6

APLICATIONOF GIS WITH MCET FOR SITE SUITABILITY ANALYSIS

6. Application of GIS and MCET for site suitability analysis

Water provides life supporting system of human beings, vegetation/plants and animals. It is also very important part of socio-ecological environment. But this important asset is depleted rapidly, as this life saving resources always taken for granted in name of development. Water intake-point structure (WIS) is the collection tool of water for future sustainable use of water. It has been used to provide drinking water, water for livestock and water for irrigation or to give treated in natural filtration process for the remote area. However, rainwater provides affordable water for household use, agriculture, environmental flows and prevention of flood damage. Rainwater Harvesting techniques includes macro-catchment technologies that handle large runoff flows diverted from surfaces such as roads, hillsides, pastures, as well as micro-catchment technologies that collect runoff close to the growing crop and replenish the soil moisture. Sustainable WIS can be used to minimize water loss and to augment water supplies in watershed systems. WIS are important in managing scarce rainfall, and success of these systems mainly depends on identification of suitable potential sites and technologies. various approaches have been chosen by different authors to identify potential WIS in remote and data scare area; in most of these methods, thematic maps are derived from Remote Sensing (RS) data and integrated in GIS to evaluate suitable sites for rainwater harvesting. Remote Sensing and Geographic Information Systems (GIS) can help in the determination of areas suitable for water collection. GIS has been recommended as a decision-making and problem-solving tool in WIS during the decision-making process. As water collection is an ancient tradition and has been used for millennia in most dry lands of the world, many different techniques have been developed. However, the same techniques sometimes have different names in different regions and others have similar names but, in practice, are completely different. Kahinda et al. identified six key factors to be considered when identifying WIS sites, this includes; climate (rainfall), hydrology (rainfall–runoff relationship and intermittent water courses), topography (slope), agronomy (crop characteristics), soils (texture, structure and depth) and socioeconomic criteria (population density, work force, people's priority, experience with RWH, land tenure, water laws, accessibility and related costs). Rao et al. identified land use, soil, slope, runoff potential, proximity, geology, and drainage as criteria to identify suitable sites for WIS . The approach had been used in some studies in other countries. However, in West Bengal the approach was new in the field of WIS construction, because it had not been used to identify suitable areas for WIS technologies. That the combination was useful to evaluate multiple criteria and expert opinion in a consistent way in order to obtain suitability maps and tabular data. It was shown that MCE–GIS combination has potentiality to provide a rational, objective and non-biased approach to making decisions in identifying potential sites for WIS and technologies. Although effective, their methodology could be enhanced with, for example, the consideration of other factors such as social economic. One of the identified major inhibitors to the progress and economic development of most urban settlements is the lack of access to adequate water supplies for drinking and domestic uses, irrigation, and industrialization. Rural Purulia is currently facing a problem of acute shortage of fresh water, and if sufficient measures are not taken up immediately inhabitants may face a crisis which will be detrimental to the very survival of mankind. One of the most logical steps towards solving this problem would be acknowledging the importance of WIS sites and natural filtration methodology. Thus, development of methodology for identifying potential sites for WIS is an important step towards identifying areas suitable for certain techniques of water collection from the foregoing, it is evident that the identification of water collection site is a spatial problem and can be addressed by exploring GIS as a decision support tool based on site selection and criteria and a remote sensing technique as an alternative form of data capture for GIS. Hence the overall objective of this study is to identify the potential point to construct water intake structure and also suitable technologies are to justify for optimize water availability using Geographical Information System (GIS), and Remotely Sensed Satellite Imagery and calibration through some field experiment.

6.1. Materials and methods for identifying possible intake structure and technologies

The primary objective was to identify and decide on which intake well and Technology technologies to be mapped. This was to aid in collection the relevant data. The field survey revealed that any sub -surface interventions in the Kashipur Block should focus on the following

- Surface runoff collection from open areas and recharge to groundwater and river basin.
- In-situ RWH and storage in the soil profile for water supply.
- Other ground water recharge methods such as check dams, subsurface dams (dam walls and dam reservoirs.
- Sustainable yield of that location.

Criteria for selection and assessment of suitability level

From the information obtained from field survey supported by expert judgment, literature review and most importantly available data, five criteria were selected for the identification of potential areas for Intake structure i.e., rainfall data, Lithologs, topography (slope and aspect), drainage and land cover and soil information and aquifer parameters.

Data acquisition

- Secondary data is collected from literature and Government reports and primary data collected from detailed field survey, Hydro-geological survey and Socio-economic survey

- Stream map, road map, land use, open space map was extracted from SPOT imagery which was interpreted visually by digitizing with Arc-GIS 9.3.

- Slope, aspect map and Drainage map was derived from Shuttle Radar Topographic Mission (SRTM 30). SRTM data is available at SRTM Plugin Q-GIS 1.7.0.
- Rainfall data were obtained from both Agricultural department Govt of West Bengal , IMD.
- Aquifer properties are determined by Pumping Test.

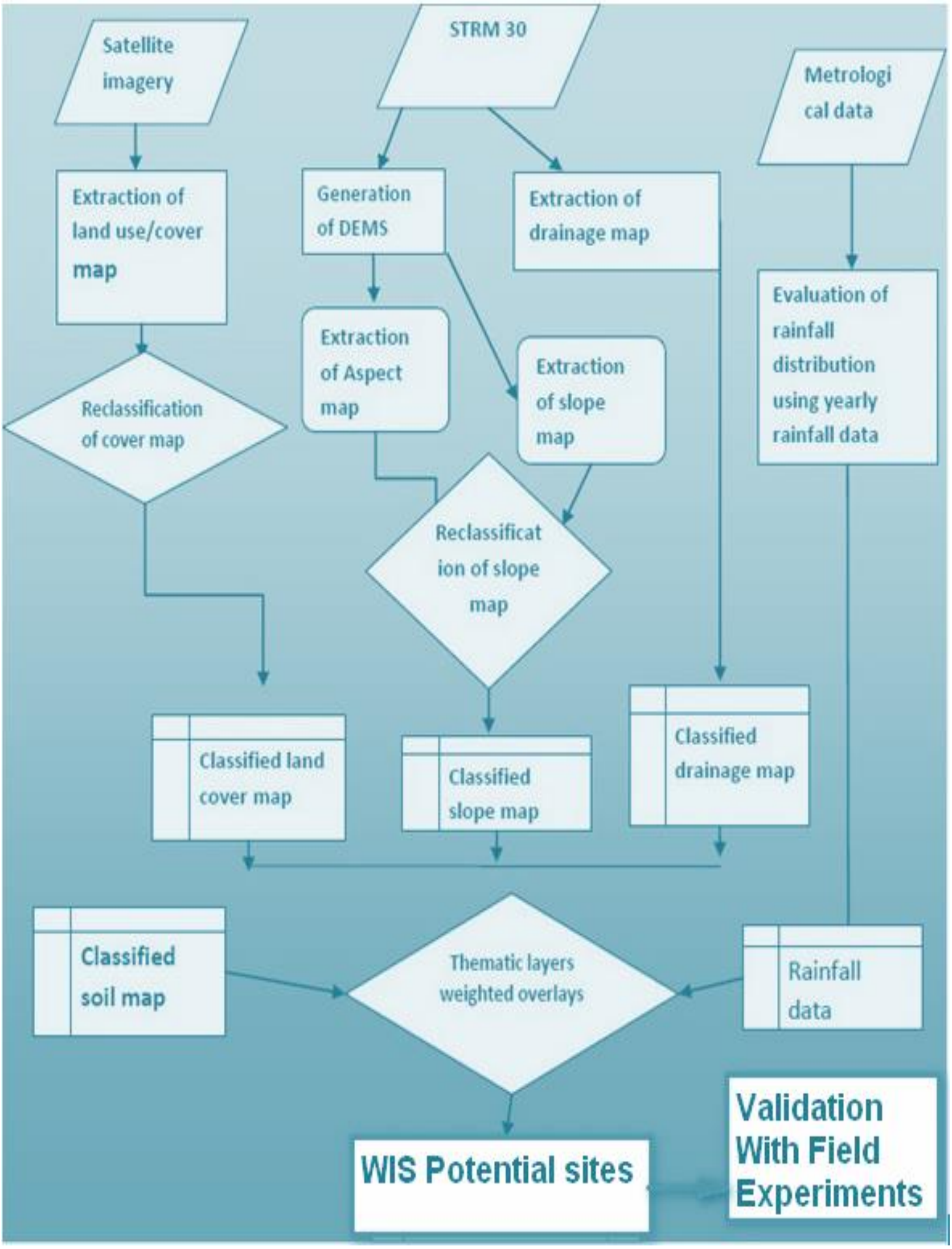


Figure 6.1.: Cartography model for the study

Data analysis

Land use pattern

As this study focused on intake structure point selection for agricultural, development both in-situ and ponds should be located close to agricultural areas. Therefore, in Figure 6.2 the land cover map was used as one criterion for in-situ and pond water recharging to identify potential areas on agricultural land. Details for suitability ranking showed in Table 6.1 as generated from Google satellite imagery

Table 6.1: Suitability rank for land use patte

Land cover class	Land use pattern		Pond	In-situ	Dam
1	Very high	Intensively cultivated	5	5	_
2	High	Moderately cultivated	5	5	_
3	Medium	Open surface	2	1	_
4	Low/ restricted	Settlement/water body	2	1	_
5	Verylow/restricted	Rock/drainages	Restricted	Restricted	_

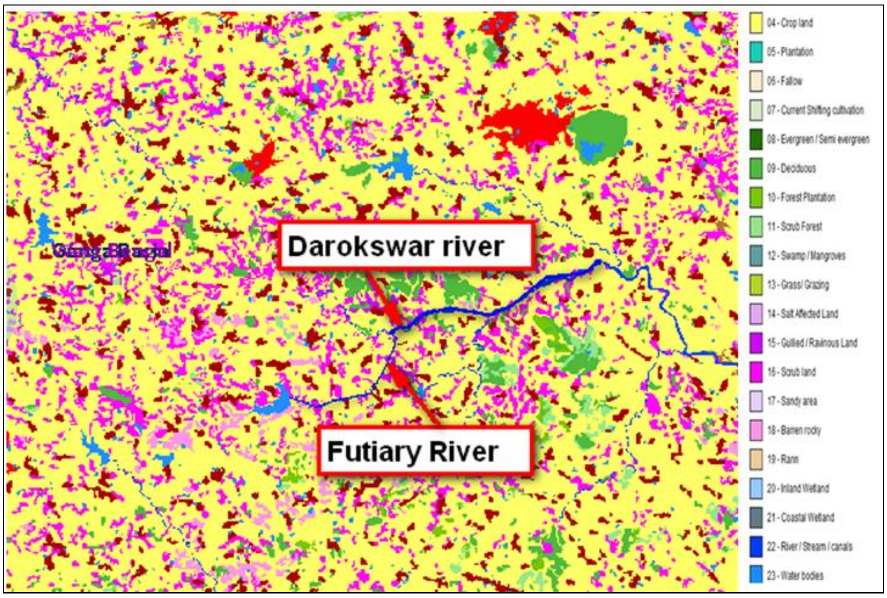


Figure 6.2 : Land Use map of the Study area West Benga

Topography

The contour Map is important in site selection and implementation of all river/subsurface base intake structure systems, especially hilly areas are not recommended where slopes are greater than 15% due to uneven distribution of run-off and large quantities of earthwork required which is often costly however a slight slope is needed for better harvesting of the runoff. Therefore flat areas with a slope less than 2% were assigned a higher suitability rank for in-situ Intake structure whereas for ponding areas with slope ranging from 2 to 8% were given higher suitability rank Table 6.2. and Figure 6.3 depicts the classified contour of the study area.

Table 6.2 : Suitability rank for slope

	Slope Class	Slope (%)	Pond	In-Situ
1	Flat	<2	3	5
2	Undulating	2-8	5	4
3	Rolling	8-15	4	3
4	Hilly	15-30	2	2
5	Mountainous	>30	1	1

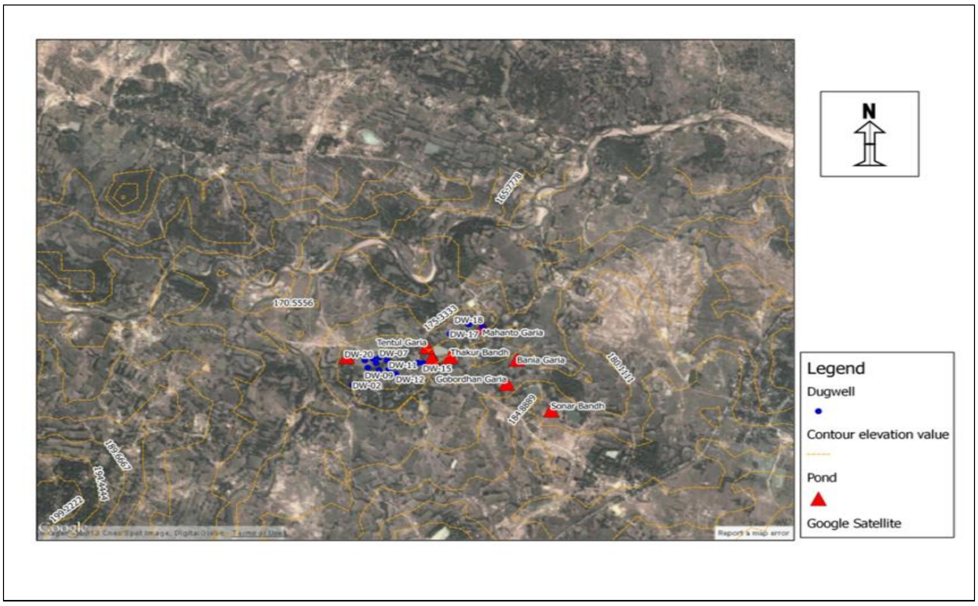


Figure 6.3 : the contour map of the area

Drainage pattern

The drainage system was classified according to the density (average channel/flow length) on the Google satellite image and the areas with high drainage density were ranked for lower suitability area of a water harvesting system. By determination of the location of the furthest point contributing to runoff, the runoff contributing areas of the various catchments were determine and ranking was done as in Table 6.2.

Table 6.2 : Suitability rank for drainage map drainage class

		Flow Length	Pond	In-Situ	Dam
1	Very high	0-125	5	5	3
2	High	125-250	4	4	4
3	Medium	250-350	3	3	5
4	Low/ restricted	350-500	2	2	2
5	Very low/restricted	<500	1	1	1

The site suitability analysis has helped in identifying areas generally suitable for Water intake structures. The suitability maps was classed into five comparable units i.e. suitability classes namely; 5 (optimum suitability), 4 (high suitability), 3 (moderate suitability), 2 (low suitability), and 1(restricted areas). The suitability classes are then used as base to generate the sites generally suitable for each Intake well construction techniques. Considering that Ghutlia has an average rainfall of 998-1100mm yearly and heavy rainfall ranging from July to August shows clearly that RWH is most needed and all RWH should be carried out during the raining season. Secondly, soils in Kashipur are mainly Coarse loamy, Fine loamy and Loamy skeketal which depicts that the soils are suitable for sitting all the identified catchments. From the results open space, intensively cultivated and moderately cultivated are generally suitable for most ponds, dams and in-situ catchments except for rock catchments. Areas 500m away from settlement and major roads and area 10m away from minor roads, secondary road and 13drainages are restricted area for all the identifies potential catchment. Slope within 2-8% are generally most suitable for pond, in-situ and dam except for rock catchment that is suitable at very high/rocky areas. Figure -6 depicts the selected spot for proposed structure for Water Intake point.

Satellite imagery

Satellite imagery obtains from Open layer plug Inn of Q-GIS Google Map, and Public Health Engineering Department Govt. of West Bengal and Web GIS, Portfolio of Drinking Water and Sanitation Govt. of India. Satellite image was classified in with the following domains moderately cultivated, intensively cultivated, settlement, drainage, open space and rock. Then imported back to Arc Map with the necessary layers such as drainage (flow length, slope, and aspect were picked and weighted overlay was done to identify areas generally suitable for rainwater harvesting. The rainfall data as well as soil information were also considered. Therefore, the criteria maps was re-classed into five comparable units i.e. suitability classes namely; 5 (very high suitability), 4 (high suitability), 3 (medium suitability), 2 (low suitability), and 1 (very low suitability). The suitability classes are then used as base to generate the criteria maps (one for each criterion). The general description is given in following flow diagram in Figure 6.2.

Lithological Survey

In this section, the important role of subsurface investigation carried out under this study has been demonstrated as a basis for selection of probable suitable sites adjacent to river as well as underneath the river bed. The borehole module of GMS 6.0 (Groundwater Modelling System) can be used to visualize boreholes created from drilling logs. The detailed positions of litholog have been shown in Figs. and 3-D view and features of Kashipur block in Purulia district. Along the river Futiary, slim bores in different cross sections 500 m apart in the Kashipur Block of Purulia district were drilled. The respective cross sectional slim bores are represented graphically below where the colour coding of the borehole logs are shown along with the cross sections. While naming the bores a particular pattern has been followed in Figure 6.4 e.g: B0-1 stands for slim bore no. 1 from from the right bank of the river. Its also help to revealed the unconfined aquifer material for the calibration of proposed structure map.

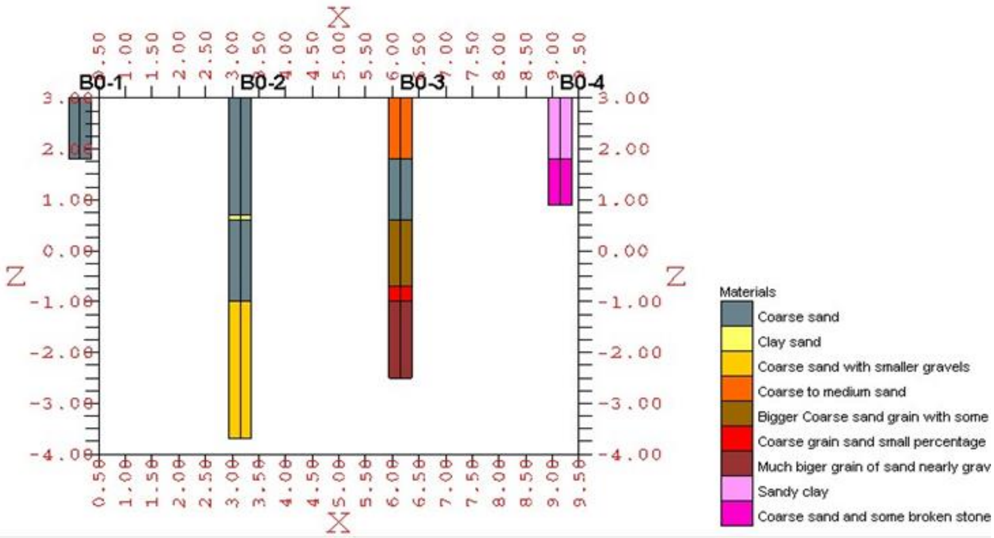


Figure 6.4: Lithologs of the study area

Aquifer parameter estimation

In order to adjudge various hydrological parameters, long duration pumping test were carried out 48 hrs and 72 hrs respectively. In the first occasion step drawdown test was conducted at variable discharges of 25, 30 and 44 m3/hr respectively. During the variable discharges, unsteady conditions of aquifer were noticed. After cessation of pumping, recuperation test was also carried out in the main as also from two and three in some cases observation wells. Accordingly the piezometric level data as obtained from the main as well as the observation wells were plotted in semi-log and log-log papers as opined by various proponents in groundwater hydrology like Thies, Cooper and Jacob and Chow. The data plots and their matching with the master curves has facilitated the determination of various aquifer parameters like hydraulic conductivity (K), Transmissivity (T), Storativity (S)/specific yield (Ss) etc. The data plots are presented in Figures (see below)Figure 6.5 and Table 6.1. The cone of depression with respect to the maximum discharge of 44 m3/hr for which testing was carried out in the area is found 257 m. The drawdown versus yield are shown in Table 5.0 From the above pumping studies, it was seen that a long duration pumping tests with another or more observation wells would have been more appropriate to adjudge the actual potentiality of the aquifer and its influence on the static water levels in the perspective of future projected scenario.

Table 6.3 The observed values from pumping test

Location ID	Transmissivity (m2/day)	Storativity
CS-1	1569.609	0.001962
CS-II	1426.917	0.000697
CS-III	6867.032	0.001774
CS-IV	5273.885	0.006592

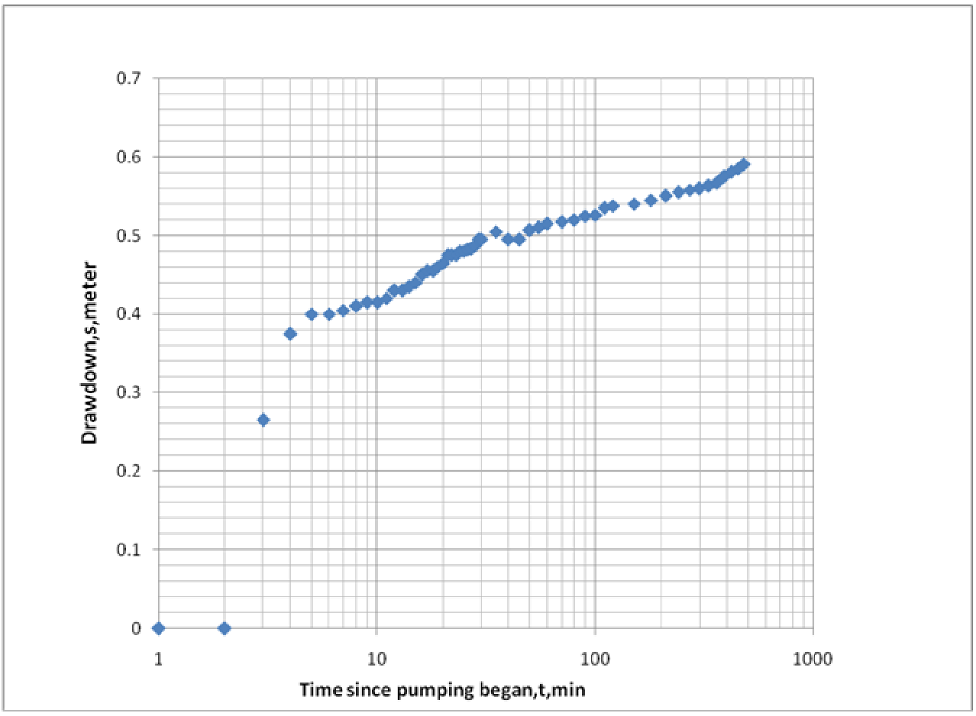


Figure 6.5 :12 hour pumping test graph in the study area

By carrying out the analysis of time-draw down data at study area the average values of storage coefficient(S) and transmissibility (T) were estimated to be 0.002377 and 1900 m^2/d respectively. The depth of the porous medium was found to be around 6 m which is optimum for introduction of a radial collector well at this point. Here, the quality of porous media and lack of obstruction in all directions facilitates the physical pushing of strainers conveniently. Also, good value of aquifer storage and high values of transmissivity makes the selection of these points a natural and obvious choice for construction of a radial collector well, which would consequently lead to the erection of a pumping station. However, the infiltration gallery may be installed at the similar locations but the designed capacity will be less with single gallery due to less width of the river in these areas. Although both the design for Collector Well as well as Infiltration Gallery are provided for the same areas, it may be stated that the collector well is more sophisticated and expensive but has higher capacities than the Infiltration Gallery. Hence choice should be made by the required yield and demand followed by economic aspects. After calibration Proposed points are found potential site for construction of WIS. The study has demonstrated the capabilities of using MCE supported in GIS for identifying potential sites for water intake structure construction sites and technologies that may be used for development and water resource management. The MCE approach was used in this study to identify areas for RWH technologies within a GIS context. The weighted linear combination was useful to evaluate multiple criteria in a consistent way in order to obtain suitability maps. Therefore, it has shown that MCE–GIS combination has potentiality to provide a rational, objective and non-biased approach to making decisions in identify potential sites for water intake structure technologies. The rainfall data as well as soil information were also considered. The criteria factors taken into consideration for the MDSS include; rainfall data, slope map, soil information, drainage map, and land use/cover from the SPOT 5 and LANDSAT satellite imagery. These criteria maps were re-classified into numerical figures ranging from 1 to 7. Buffer operation was done for each layer on the digitized SPOT 5 imagery i.e drainage, settlement and roads and overlaid with the classified weighted overlay for each catchment. Thus subjective numbers in the suitability levels and weights of the criteria can be changed according to the study area characteristics. The suitability maps was classed into five comparable units i.e. suitability classes namely; 5 (optimum suitability), 4 (high suitability), 3 (moderate suitability), 2 (low suitability), and 1(restricted areas). The suitability maps shows location of rock, check dams, pond and in-situ catchments. In this study, MCE-GIS were proved to be a flexible, time-saving and cost-effective tool to screen large areas for their suitability of two types of WIS intervention. The suitability maps provide an easy to understand source of information to quickly identify areas that are more promising than other areas for sustainable water supply intervention. Such information is helpful for decision-makers and planners and justification also done field experimental calibration.

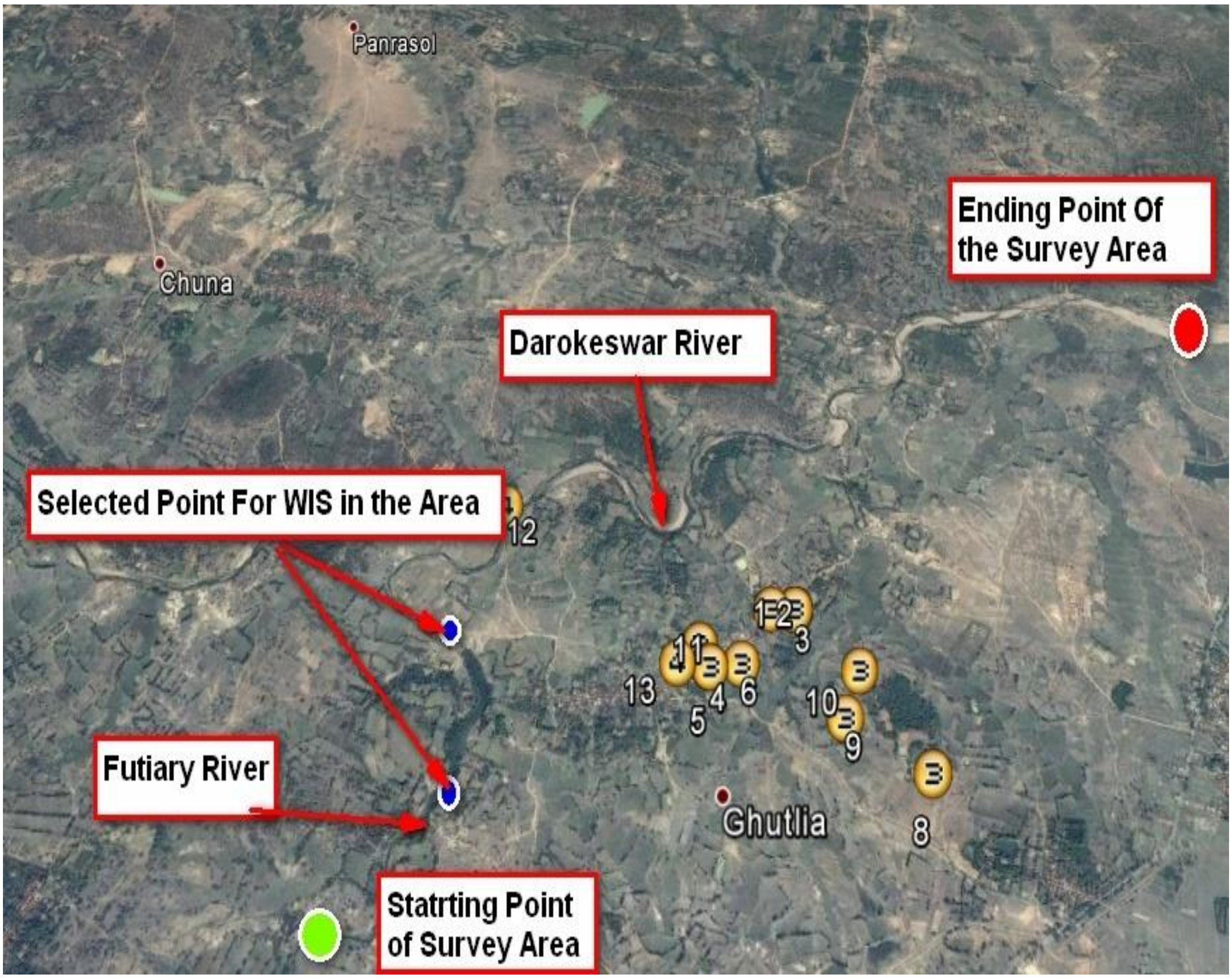


Figure 6.6 : Proposed structure map

CHAPTER-7
WATER QUALITY SCENARIO OF THE STUDY AREA

7. Water quality scenario of study area

Water is essential for all physiological activities associated with humans, animals, and the plant kingdom. However, the nature and the quality of surface and ground water are widely variable and are determined by the local geological history, including the rocks and hidden ore deposits surrounding the sites for the collection of the water, and other factors, such as the movement of elemental particles and pollutants by lentic and lotic waters and underground aquifers. The quality of water is poorly understood due to the variety in the interactions between water and soluble minerals, sparingly soluble minerals, and salts, both natural and anthropogenic. In spite of the complex hydro and biogeochemical factors, the number of dissolved constituents in ground water is limited. In addition to the trace constituents in water, the major constituents are Na^+, Ca^{2+},Mg^{2+}, SO_4^{2-} (as acid and salts), Cl^-, HCO_3^-, etc., and the secondary constituents are Fe^{2+}, Sr^{2+}, K^+, CO_3^{2-}, NO_3^-, and F^- (fluoride).Fluoride (F) becomes toxic when it occurs in drinking water beyond the maximum permissible limit of 1.5 ppm. Chronic exposure to fluoridated ground or drinking water creates a health problem not only in human beings but also in diverse species of domestic animals in the form of osteo-dental fluorosis. Recently, bio-indicators of endemic fluorotoxicosis due to fluoridated ground water have been reported. In the Purulia District (the present study area), instances of fluorosis are on the rise. The rural population is the worst affected, because of the absence of centralized water-treatment system in these areas. Outbreak of media reports on this issue has become a matter of grave concern for the Government in view of the strategic location of Purulia, its poor socio-economic status and the tribal-dominated demography. Geological and hydrogeological studies were carried out by drilling boreholes in 18 different locations of Purulia spread over the three worst-affected blocks, viz. Purulia I, Purulia II and Hura, Arsha, Ranghunathpur, Kashipur, Santuri. A total of 180 samples from these blocks were collected in the summer, post monsoon (winter) period of 2012–2016, which included samples collected from three different sources: boreholes, dug wells and tube wells and river, Ponds. Water quality analysis has been carried forward for the health safety of the villagers. They were using river, tube well, wells for drinking, agriculture and other daily purposes. TDS Meter and DO Meter was used for analysis of quality of water on this site. Other parameters of quality of water (Chemical and Bacteriological) were measured in laboratory. By Field kit Total Dissolved Solids, Electrical Conductivity, Salinity of has been measured for Groundwater and surface water analysis results are depicted in. Majorly Darokeswar, Kangshabati and Damodar river catchment area covered in this research study, The results and detail survey of each segment is discussed in this chapter.

7.1. Darokeswar and Futiary River

The Futiary represents a typical rural tributary of Darokeswar river under stress from continuing drinking water and agricultural development and an ever-changing climate. Sourced in the western most district, the river flows for 12.35 km through agricultural and rural environments, covering a catchment of around 60 km^2 . As the river passes between these different environments a water quality survey has been conducted to allow the implications of rural development on the catchment was understood, centring on the impacts of faecal pollution from underperforming open defecation. Through targeted sampling, laboratory analysis and the reviewing of long-term data an understanding of the current and long-term water quality situation in the Ghutlia village has been gathered. Results provide strong evidence that point-source faecal pollution is occurring at two locations along the sampled stretch; The effect of this pollution on a downstream bathing site has been analysed, concluding at faecal contamination levels are unacceptable but generally pollution is heavily localised. From reviewing long-term water quality data faecal pollution has become significantly worsened despite lower rainfall levels indicative of further damage to the malfunctioning storm overflow. Potential solutions to combat the effects of the pollution have been analysed concluding soft-measures are a preferred option until an ecological impact assessment can be undergone to understand the impacts of the pollution to the aquatic and riparian environments where large-scale projects may become a necessity. Water samples are taken from River water, surface water bodies and open well and tube wells Table 7.1 describe the locations of in-situ measurements with help of field test kit. Figure 7.2 and

Figure 7.3 depict variation of salinity and TDS values in the catchment area. Maximum TDS value found 1600 mg/l which above the permissible limit of using groundwater as drinking water in CM 12. Detail of location also depicted in the Figure 7.1 below.

Table 7.1: Description of sampling points

ID	Latitude (N)	Longitude (E)	Depth of Well(m)	SW Table(m)	Purpose of Use
KPM 1	23˚24'19.0"	86˚37'17.2"	10.17	7.17	Drinking
AM2	23˚24'18.1"	86˚37'16.6"	10.40	8.22	Not Used
GG3	23˚24'18.7"	86˚37'15.7"	10.88	8.49	House hold activity
GW4	23˚24'19.9"	86˚37'16.3"	10.47	8.35	Drinking
NKM5	23˚24'20.9"	86˚37'14.5"	10.22	7.82	House hold activity
SM6	23˚24'19.2"	86˚37'14.0"	10.13	8.33	Drinking
JM7	23˚24'21.3"	86˚37'14.8"	10.29	7.63	House hold activity
PL8	23˚24'21.5"	86˚37'12.9"	9.23	7.3	Drinking
SM9	23˚24'21.1"	86˚37'12.0"	9.33	7.2	House hold activity
GW10	23˚24'20.2"	86˚37'12.3"	8.69	7.34	Drinking
KPM 11	23˚24'18.9"	86˚37'13.0"	11	8.2	Agriculture
CM 12	23˚24'18.9"	86˚37'12.4"	10.12	7.95	drinking
IM13	23˚24'18.7"	86˚37'12.3"	9.91	7.15	House hold activity
PM14	23˚24'18.8"	86˚37'11.7"	10.25	7.52	Drinking
JM15	23˚24'19.3"	86˚37'11.0"	10.85	7.19	Drinking
SA16	23˚24'19.0"	86˚37'10.5"	9.66	7.39	Agriculture
SS17	23˚24'18.7"	86˚37'09.6"	10.78	7.19	House hold activity
EA18	23˚24'20.8"	86˚37'10.3"	9.94	6.50	House hold activity
NA19	23˚24'18.5"	86˚37'08.4"	9.89	6.62	Drinking
KS20	23˚24'18.9"	86˚37'08.4"	8.96	6.0	House hold activity

Figure 7.1: Location of Sampling well

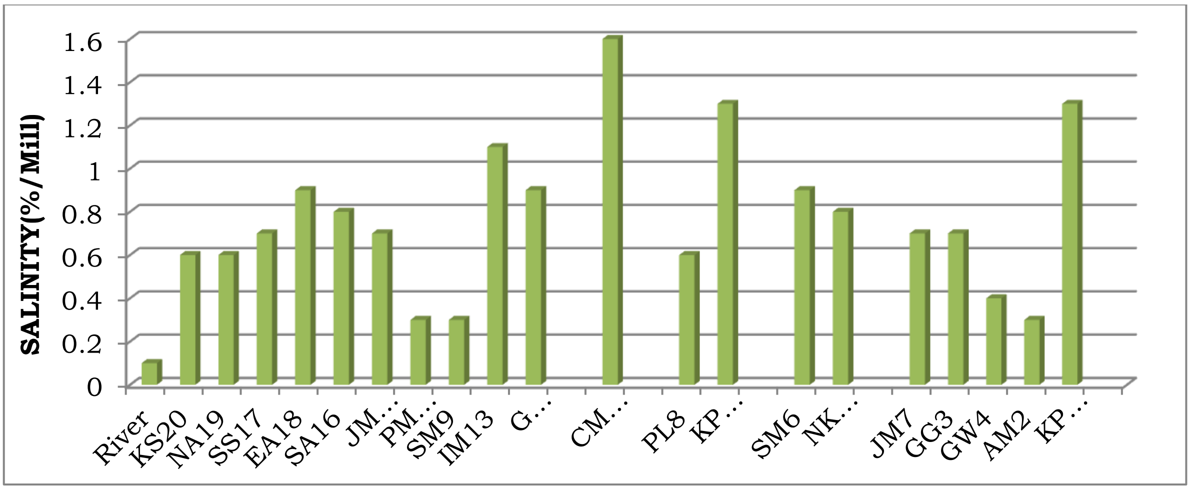


Figure 7.2 : Salinity Value of well in incremental distance from the River

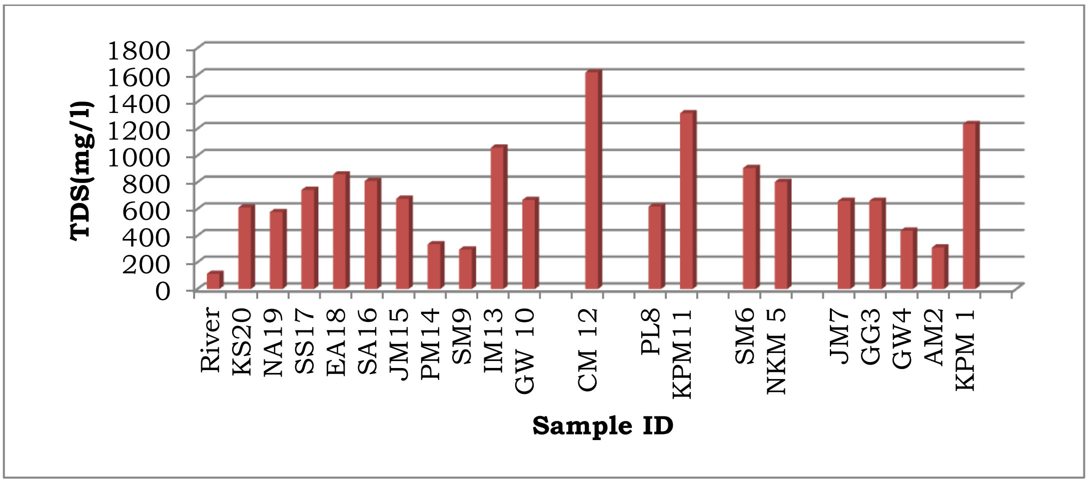


Figure 7.3 : TDS value of well in incremental distance from the river

Groundwater samples also collected to laboratory and analysis of different water quality parameter pH, Turbidity, Fluoride Iron, Nitrates , Total arsenic etc given in Table 7.2, and also compared BIS-10500 standard for assessing the status of the water quality in study region. In groundwater survey its found that there is no arsenic contamination values of arsenic is lower than permissible limit. Iron values ranges from 0.15 to 5.30 which is higher in range .Fluoride values are exceeding the permissible limit. Therefore detail survey on fluoride contamination and its done and described in below. Sample 1, 2, 3, 4is collected from well of the village Between these water quality of Sample 4 is much poor than others. Concentration of iron is high for some regions which iron contamination depending upon soil strata characteristics. Other Parameters are between the Permissible range which is more about satisfactory for health suggestion. These limits are taken from IS 10500:2012.

Table 7.2 : Groundwater quality results (Chemical parameters)

SL No.	Parameters	S1	S2	S3	S4	Desirable Limit*	Permissible Limit*
1	pH value	7.1	7.14	7.25	6.65	6.5-8.5	6.5-8.5
2	Turbidity	<1.0	9.3	<1.0	123	1	5
3	Fluoride(as F) mg/l	0.39	1.39	0.43	1.7	1.0	1.5
4	Iron(as Fe) mg/l	0.15	1.07	0.20	5.30	0.3	No relaxation
5	Nitrate(as NO_2)mg/l	0.036	0.03	0.14	0.19	45	No relaxation
6	Total Arsenic(as As)mg/l	<0.01	<0.01	<0.01	<0.01	0.01	No relaxation
7	Total alkanity(as $CaCO_3$)mg/l	222.2	189.9	177.8	167.3	200	600
8	Total Hardness(as $CaCO_3$)mg/l	597.5	240.6	252.2	523.8	200	600

Surface water samples also collected from different location in river and ponds and detail water quality values are given in Table 7.3 Sample 1 is taken from 0.4m from top surface of check dam Sample 2 is collected from river bed, by boring of 1m depth by immediate hand boring. For sample 2 as the collection of water is occur under immediate boring of soil layer, therefore TDS and turbidity value become high than usual. High value of TC, FC may reflected of human and animal excreta contamination.

Table 7.3 : Results for Surface water quality

SL No.	Parameters	S1	S2	Acceptable limit*	Permissible Limit*
1	pH value	6.63	7.8	6.5-8.5	6.5-8.5
2	Turbidity(NTU)	<1.0	2255	1	5
3	Total Suspended Solids (mg/l)	<2.0	1838	100 for Inland Surface water	No relaxation
4	Chemical Oxygen Demand(mg/l)	9.0	25	Nil,<250 mg/L for Inland surface disposal	Nil <250 mg/L for Inland surface disposal
5	BOD(3days at 27˚c)(mg/l)	1.6	4.5	Nil< 30 mg/L for Inland surface disposal	Nil < 30 mg/L for Inland surface disposal
6	Total Coliform Count(MPN/100ml)	330	2.2×10^5	Shall not be detected 100 ml sample	Shall not be Detected 100 ml sample
7	Fecal Coliform(MPN/100ml)	140	3.3×10^4	Shall not be detected 100 ml sample	Shall not be detected 100 ml sample

Photo 7.1: Water quality measurement in Futiary river

7.2. Kangsabati river catchment area water quality status

Water samples were collected every month from 2nd Feb to 8th June 2012 (Table 9.2). Sampling was conducted between 10 am and 6 pm based on weather conditions. Samples also collected according to different GPS location wise which were basically DW (Dug well), TW and PW of Kangsabati river, the sampling site was located by GPS and the boat was anchored to avoid drifting and 3 stratified water samples were collected according to the water depth. A total of 19 samples were collected at different GPS location in respective Block of Kangsabati River. All the parameters measured and water quality parameter was analyzed according to APHA method. The results of water quality parameter in Kangsabati River are listed in Table 7.3. Statistical analysis also done of the determined result and different statistical analysis and their result discuses below. Collection of water sample from pumping well and tube well depicted in Photo 7.2 and Photo 7.3

Photo 7.2 : River water sample collection in Kangsabati river

Photo 7.3 : Water Sample Collection from Tubewell

Table 7.3 : Water quality analysis of some tubewells, dugwells and pond water at selected locations (Samples collected by the team of SWRE, JU from different sites)

Name of the Village (Block)	**GPS location (Type of sample)**	**pH**	**Fe**	**TDS**	**TH**	**Ca**	**Mg**	**HCO_3**	**Cl**	**NO_3**	**F**	**SO_4**	**Na**	**K**	**As**
			(mg/L)												
Bhool (Arsha)	23°21.070’ 86°16.983’ (POW)	8.11	0.12	643	153.6	49.3	7.50	180.8	51.1	1.34	0.62	8.8	23.3	0.79	BDL
Kotloi (Purulia-I)	23°17.386’ 86°19.945’ (TW)	8.13	1.7	762	186.2	46.2	17.30	200.6	67.5	<0.5	0.47	15	21.3	4.4	BDL
	23°17.723’ 86°19.720’ (DW)	7.87	0.52	663	117.1	29.2	10.7	158.5	46.3	0.87	1.12	5	26.7	5.5	BDL
Joradihi (Arsha)	DW	7.89	<0.1	1016	398.4	123.1	22.2	368.2	188.00	23.46	0.58	38.5	81.70	6	BDL
	TW	7.39	0.2	1312	205.4	69.3	7.9	327.6	28.90	6.64	0.54	11	42.60	1.2	
Karadoba (Hura)	TW	7.93	0.77	1119	326.4	90.4	24.5	133.1	168.70	113.46	0.40	63.5	108.9	11.1	<0.01
	23°15.615’ 86°30.750’ (DW)	7.81	0.17	975	126.7	46.2	2.8	176.9	15.40	<0.50	0.32	17	20.3	4.2	<0.01
	23°16.776’ 86°27.563’ (DW)	-	<0.1	816	449.3	144.7	21.5	239.1	254.60	-	0.47	-	-	3.76	<0.01
	23°16.749’ 86°27.619’ (TW)	7.45	1.9	784	458.9	141.6	25.7	263	232.40	-	0.36	-	-	2.38	<0.01
Goradag (Arsa)	(TW)	7.29	<0.1	1253	873.6	223.2	77	349.2	475.00	-	0.41	-	-	3.4	<0.01
	(DW)	7.71	<0.1	993	835.2	196.2	84.1	451.9	522.50	-	0.65	-	-	10	<0.01

Name of the Village (Block)	GPS location (Type of sample)	pH	Fe	TDS	TH	Ca	Mg	HCO_3	Cl	NO_3	F	SO_4	Na	K	As
			(mg/L)												
Joradihi (Arsha)	PW	7.61	0.15	125	117.1	30.8	9.8	176.9	28.9	3.39	0.24	10.80	33.70	2.3	-
Karadoba (Hura)	23°15'38.06" 86°29'40.25" (PW)	7.68	<0.1	135	92.2	26.2	6.5	126.6	30.9	2.54	0.27	11.30	28.70	2.7	<0.01
Kudlung (Hura)	23°16'04.36" 86°27'50.69" (PW)	7.32	<0.1	162	88.3	24.6	6.5	124.3	27	-	0.72	-	-	1.68	<0.01
Teledihi (Purulia-I)	23°19'05.23" 86°18'35.09" (PW)	7.78	<0.1	129	86.4	24.6	6.1	454.3	10.5	-	0.43	-	-	2.2	<0.01
Goradag (Arsa)	23°19'38.68" 86°16'54.80" (PW)	7.55	<0.1	141	78.7	23.1	5.1	119.6	15.2	-	0.54	-	-	1.7	<0.01

One way ANOVA analysis:

ANOVA analysis is an important technique for analyzing the effect on categorical factors on a response of variation of a parameter. An ANOVA decomposes the variability in the response variable amongst the different factors. Depending upon the type of analysis, it may be important to determine: (a) which factors have a significant effect on the response, and/or (b) how much of the variability in the response variable is attributable to each factor. Bothe parameter determined in laboratory and field anova analysis has done and depicted in Table 7.4. and Table 7.5.

One-Way ANOVA

A one-way analysis of variance is used when the data are divided into groups according to only one factor. It has been used the test when questions of interest are usually:

(a) Is there a significant difference between the groups and

(b) If so, which groups are significantly different from which others?

The F-test in one-way analysis of variance is used to assess whether the expected values of a quantitative variable vary within several pre-defined groups differ from each other or not. Lesser the value in the F column lesser will be the variance between the samples. Thus, from the values in the F column, it can be say that the samples K show the least variance whereas the Cl samples show the highest degree of variance also revealed that if the value in the Sig. column goes below a previously assigned (0.05) critical value, it indicates that the variances in the sample are significant. From this knowledge, it can be revealed that the data Fe, TDS, HCO_3, NO_3, F, SO_4, Na, K show no significant variances. The below table (Table 7.4) is one way ANNOVA analysis of Table 7.3. The values in the F column in above table show that the conductivity has significant variance whereas other two sample have no significant variance as $\alpha > 0.05$.

Table 7.4 : ANOVA for Water quality parameter characteristics

Parameters		Sum of Squares	df	Mean Square	F	Sig.
Fe	Between Groups	4.052	6	0.675	2.376	0.085
	Within Groups	3.978	14	0.284		
	Total	8.030	20			
pH	Between Groups	0.812	6	0.135	2.880	0.048
	Within Groups	0.658	14	0.047		
	Total	1.471	20			
TDS	Between Groups	883801.238	6	147300.206	0.931	0.503
	Within Groups	2214141.333	14	158152.952		
	Total	3097942.571	20			
TH	Between Groups	7577478.412	6	1262913.069	3.371	0.029
	Within Groups	5245370.733	14	374669.338		
	Total	1.282E7	20			
Ca	Between Groups	788449.580	6	131408.263	3.385	0.028
	Within Groups	543515.847	14	38822.560		
	Total	1331965.427	20			
Mg	Between Groups	18177.520	6	3029.587	3.059	0.040
	Within Groups	13864.146	14	990.296		
	Total	32041.666	20			
HCO_3	Between Groups	148360.419	6	24726.737	2.316	0.092
	Within Groups	149497.953	14	10678.425		
	Total	297858.372	20			
Cl	Between Groups	5074273.360	6	845712.227	3.501	0.025
	Within Groups	3381641.020	14	241545.787		
	Total	8455914.380	20			
NO^3	Between Groups	84386.734	6	14064.456	2.322	0.091
	Within Groups	84803.888	14	6057.421		
	Total	169190.622	20			
F	Between Groups	0.865	6	0.144	2.347	0.088
	Within Groups	0.860	14	0.061		
	Total	1.726	20			
SO_4	Between Groups	96442.301	6	16073.717	1.498	0.249
	Within Groups	150226.941	14	10730.496		
	Total	246669.242	20			
Na	Between Groups	17689.663	6	2948.277	1.087	0.416
	Within Groups	37971.014	14	2712.215		
	Total	55660.677	20			
K	Between Groups	43.006	6	7.168	0.791	0.592
	Within Groups	126.908	14	9.065		
	Total	169.915	20			

Table 7.5: Water Quality parameters measured in Kangabati river

Name of the Village/Block	GPS Location		DO (mg/L)	pH
	N	E		
Kotloi	23°17.386'N	86°19.945'E	2.80	7.87
Kotloi	23°17.406'N	86°19.92' E	0.39	7.53
Kotloi	23°17.433'N	86°19.901'E	5.14	7.15
Kotloi	23°17.723'N	86°19.720'E	2.55	8.13
Sat-simulia	23°18.296'N	86°19.279'E	2.95	8.09
Sat-simulia	23°18.259'N	86°19.292'E	3.86	7.9
Sat-simulia	23°18.314'N	86°19.236'E	4.62	7.56
Hensle	23°20.46'N	86°21.637'E	3.66	8.33
Hensle	23°17.347'N	86°15.668'E	0.21	7.76
Hensle	23°18.748'N	86°15.824'E	2.89	8.15
Hensle	23°18.758'N	86°15.825'E	1.20	8.33
Bhool	23°21.050'N	86°16.991'E	2.73	8.23
Bhool	23°21.070'N	86°16.983'E	3.21	8.11
Bhool	23°21.070'N	86°16.983'E	3.29	8.35

Table 7.6 : One way ANOVA for Water quality parameter analysis.

Parameters	Details	Sum of Squares	df	Mean Square	F	Sig.
DO	Between Groups	14.963	8	1.870	0.785	0.622
	Within Groups	42.912	18	2.384		
	Total	57.875	26			
Conductivity	Between Groups	1854814.29	8	231851.787	6.339	0.001
	Within Groups	658314	18	36573		
	Total	2513128.29	26			
pH	Between Groups	2.539	8	0.317	1.377	0.271
	Within Groups	4.150	18	0.231		
	Total	6.689	26			

Hierarchical cluster analysis

Cluster analysis is an unsupervised pattern recognition technique that uncovers intrinsic structure or underlying behaviour of a data set without making a priori assumption about the data, in order to classify the objects of the system into categories or clusters based on their nearness or similarity. Hierarchical clustering is the most common approach in which clusters are formed sequentially, by starting with the most similar pair of objects and forming higher clusters step by step. The Euclidean distance usually gives the similarity between two samples and a 'distance' can be represented by the 'difference' between analytical values from both the samples (Hierarchical agglomerative CA was performed on the normalized data set by means of the Ward's method, using Euclidean distances as a measure of similarity). This method uses the analysis of variance approach to evaluate the distances between clusters, attempting to minimize the sum of squares of any two clusters that can be formed at each step. Cluster analysis was applied to the river water-quality data set with a view to group the similar sampling sites (spatial variability) spread over the river stretch and in the resulted dendogram. In this case each of the ten villages were numbered from 1 to 16 and plotted along the ordinate of the dendogram (Figure 7.4) and the distance plotted along the abscissa denotes the range of agreement of the responses obtained from water quality test on parameter DO, Conductivity, pH, of this villages.

```
                          Rescaled Distance Cluster Combine

C A S E                0         5        10        15        20        25
                       +---------+---------+---------+---------+---------+

PW(Teledihi)          13   -+
PW(Goradag)           14   -+
DW2(Bhool/Arsha)       2   -+
DW(Kudlung)           12   -+
DW(Joradihi)           5   -+
DW(Goradag)           16   -+
DW(Karadoba /Hura)    10   -+
PW(Kudlung)           11   -+
PW(Joradihi)           7   -+
PW(Karadoba /Hura)     9   -+
TW(Joradihi)           6   -+-+
TW(Karadoba /Hura)     8   -+ |
TW(Goradag)           15   -+ +---------------------------------------------+
DW1(Bhool/Arsha)       1   -+ |                                             |
DW(Kotloi purulia-1)   4   ---+                                             |
PW(Bhool/Arsha)        3   -----------------------------------------------+
```

Figure 7.4: Regression anlysis

Number of sample which is placed higher along the ordinate will have higher degree of correlated responses than groups placed lower along the ordinate even if they are connected by the same length of lines along the abscissa.

According to dendogram analysis, it showed that the water quality parameter of number 13,14,2,12,5,,16,10,11,17,9,6,8 are same, also 6,4 nearly same but there is a large difference between sample no 3,6,and 4 according to dendogram.

Table 7.7: Bacteriological parameter for of Kangsabati river

Name of the Village (Block)	Type of sample with GPS location	Total Coliform Count (MPN/100 ml)	Fecal Coliform Count (MPN/100 ml)
Bhool (Arsha)		79	23
	DW	>23	9.2
	POW	>1600	920
Kotloi (Purulia-I)	TW	-	Absent
	DW	430	130
Joradihi (Arsha)	DW	>23	16
	TW	2.2	<1.1
Karadoba (Hura)	TW	1.1	<1.1
	DW	>23	>23
Kudlung (Hura)	DW	>23	9.5
	TW	-	Absent
Goradag (Arsha)	TW	5.1	1.1
	DW	23	2.2

*DW-Dug well, POW-pond water, TW- Tube well Water

Table 7.8 : Bacteriological parameter for Pumping Well of Kangsabati

Name of the Village (Block)	Type of sample with GPS location	Total Coliform Count (MPN/100 ml)	Fecal Coliform Count (MPN/100 ml)
Beldi	PW	>23	>23
Karadoba (Hura)	PW	>23	>23
Joradih	PW	>23	>23
Kudlung (Hura)	(PW)	>23	>23
Teledihi (Purulia-I)	(PW)	>23	12
Goradag (Arsha)	(PW)	>23	12

Table 7.9 : ANOVA for Bacteriological Parameter

Parameters	Detail	Sum of Squares	df	Mean Square	F	Sig.
Total Coliform Count (MPN/100 ml)	Between Groups	1577655.546	6	262941.091	1.828	0.275
	Within Groups	2014074.440	14	157412.460		
	Total	2545720.986	20			
Fecal Coliform Count (MPN/100 ml)	Between Groups	367566.903	6	56412.150	1.321	0.421
	Within Groups	296566.227	14	42477.588		
	Total	935443.130	20			

In bacteriological parameter analyzed by One Way ANNOVA method. As it is known that if the value in the Sig. column goes below a previously assigned (0.05) critical value, it indicates that the variances in the sample are significant. From this knowledge represent can say that the data samples Total Coliform and Fecal Coliform show no significant variances. Table 7.8 and Table 7.9 shows the stastical analysis .Photo 7.4 depict the in-situ DO measurement.

Photo 7.4 : Water Quality testing in field with help of DO probe

7.3. Damodar river water quality scenario

Sample Sites Based on the basic information and the characteristics of topography, size, shape, depth, flow velocity, etc. 7 sampling sites in Damodar river were selected (Table 7.10, Table 7.11, 7.12 Table 7.13 Table 7.14 .Water samples were collected every month from 14 August to 25 January 2013 (Table 7.10.) Sampling was conducted between 10 a.m. and 6 p.m. based on weather conditions. Sample also collected according to different GPS location wise which were basically DW (Dug well), TW (Tube Well) and PW (Pond water) of Damodar river. The sampling site was located by GPS and the boat was anchored to avoid drifting and 3 stratified water samples were collected according to the water depth. A total of 53 samples were collected for different water quality parameter testing and all water quality parameter also analyses in School of Water Resources Engineering according to APHA guidelines.

Photo 7.5 : Water quality sampling from tube well

Table 7.10 : Water quality analysis of selected locations near of river Damodar

Name of the Village (Block)	GPS location (Type of sample)	pH	Fe	TDS	TH	Ca	Mg	HCO_3	Cl	NO_3	F	SO_4	Na	K
			(mg/L)											
Ichar	23°38'11.16' 86°29'37.42"' (DW)	8.10	0.15	586	146.7	39.8	10.6	170.9	48.5	2.14	0.68	7.6	20.3	0.67
Deoli	23°38.899' 86°52.939' (TW)	8.12	1.6	660	176.5	42.2	15.5	188.6	65.5	<0.5	0.45	12.9	20.7	4.2
	23°33.889' 86°52.939' (DW)	7.85	0.55	641	110.5	25.7	12.5	150.4	44.4	0.82	1.15	4.7	22.8	4.9
Hijli	23°40.756' 86°48.933 (DW)	7.92	<0.1	996	358.5	120.2	25.3	354.2	182.2	22.46	0.55	35.6	78.5	5.4
	23°40.515' 86°48.933 (TW)	7.48	0.2	1215	212.4	62.3	6.7	320.2	18.8	5.40	0.52	10.6	39.4	1.0
Swet-palash	23°38'05.57" 86°33'52.94" (TW)	7.98	0.72	1012	320.5	85.4	20.6	125.1	160.8	112.5	0.32	60.6	102.1	10.4
	23°38'02.14" 86°33'49.60" (DW)	7.68	<0.1	132	89.5	20.4	6.3	122.4	28.5	1.95	0.29	10.4	25.6	2.2
	23°38'01.32" 86°30.750' (DW)	7.81	0.15	960	120.4	45.6	1.7	170.8	16.5	<0.5	0.34	16.5	18.4	3.8
	23°38'43.27" 86°52'11.67" (DW)	-	<0.1	789	430.7	140.8	18.7	230.4	249.8	-	0.45	-	-	2.94
	23°38'42.40' 86°52'12.16' (TW)	7.48	1.7	779	450.7	139.4	26.3	259	230.7	-	0.32	-	-	1.78
Asanbani	23°40.261" 86°50.205" (DW)	7.86	<0.1	120	85.2	20.7	5.7	450.2	11.2	-	0.40	-	-	2.30
	23°37'15.94" 86°55'57.40" (TW)	7.25	<0.1	1240	870.2	220.4	76.7	330.8	445.2	-	0.40	-	-	2.46
	23°37'14.10" 86°55'57.80" (DW)	7.78	<0.1	987	840.1	190.5	86.5	462.3	520.7	-	0.62	-	-	8.64

Note: DW-Dug Well, TW-Tube Well

Table 7.11: Water quality characteristics analysis of some Pumping Wells (PW)

Name of the Village (Block)	GPS location (Type of well)	pH	Fe	TDS	TH	Ca	Mg	HCO_3	Cl	NO_3	F	SO_4	Na	K	As
			(mg/L)												
Hijli	23°40.519' 86°48.935' PW	7.65	0.17	120	118	29.8	8.7	175.9	22.7	3.34	0.25	9.5	32.4	1.68	-
Deuli	23°38.949' 86°52.612' (PW)	7.35	<0.1	157	85.4	20.7	6.2	120.8	25.6	-	0.70	-	-	1.62	<0.01
Chinakuri Ghat	23°40.637' 86°49.488' (PW)	7.68	0.38	112.5	62.7	15.4	4.95	94.3	10.7	1.95	0.15	4.67	8.87	1.24	-
Dumdumi	23°37'14.95" 86°55'57.40" (PW)	7.64	<0.1	137	75.8	22.1	4.87	115.7	14.9	-	0.52	-	-	1.54	<0.01
Shaltore	23°40.995' 86°48.209' (PW)	7.85	0.22	134.2	84.6	20.7	7.4	150.8	22.7	3.12	0.42	10.4	20.5	1.97	-
Narayanpur	23°40.731' 86°46.910' (PW)	7.54	<0.1	164	74.1	20.4	4.9	140.8	21.7	-	0.75	-	-	1.62	<0.01

Table 7.12 : One way ANOVA for water quality analysis characteristics

Parameters		Sum of Squares	df	Mean Square	F	Sig.
pH	Between Groups	1.659	7	0.237	8.527	0.000
	Within Groups	0.389	14	0.028		
	Total	2.048	21			
Fe	Between Groups	3.567	7	0.51	2.078	0.116
	Within Groups	3.433	14	0.245		
	Total	7	21			
TDS	Between Groups	1191229.621	7	170175.660	1.047	0.444
	Within Groups	2276195.333	14	162585.381		
	Total	3467424.955	21			
TH	Between Groups	7830291.805	7	1118613.115	2.964	0.04
	Within Groups	5282969.233	14	377354.945		
	Total	1.311E7	21			
Ca	Between Groups	816584.478	7	116654.925	3.013	0.038
	Within Groups	542075.000	14	38719.643		
	Total	1358659.478	21			
Mg	Between Groups	22441.713	7	3205.959	3.251	0.029
	Within Groups	13805.960	14	986.14		
	Total	36247.673	21			
HCO_3	Between Groups	210450.898	7	30064.414	3.539	0.021
	Within Groups	118940.913	14	8495.78		
	Total	329391.811	21			
Cl	Between Groups	5173330.941	7	739047.277	3.058	0.036
	Within Groups	3383624.393	14	241687.457		
	Total	8556955.335	21			
NO_3	Between Groups	102084.408	7	14583.487	2.871	0.044
	Within Groups	71105.215	14	5078.944		
	Total	173189.623	21			
F	Between Groups	1.035	7	0.148	2.984	0.039
	Within Groups	0.693	14	0.05		
	Total	1.728	21			
SO_4	Between Groups	120777.656	7	17253.951	3.008	0.038
	Within Groups	80306.013	14	5736.144		
	Total	201083.67	21			
Na	Between Groups	22594.712	7	3227.816	0.837	0.575
	Within Groups	53966.947	14	3854.782		
	Total	76561.659	21			
K	Between Groups	85.654	7	12.236	1.886	0.148
	Within Groups	90.811	14	6.487		
	Total	176.466	21			

The F-test in one-way analysis of variance is used to assess whether the expected values of a quantitative variable within several groups differ from each other or not. From the values in the F column, it showed that the samples show Na least variance whereas the show pH highest degree of variance. The values in the Sig. column indicate the degree of variances. It is known that if the value in the Sig. column goes below a previously assigned (0.05) critical value, it indicates that the variances in the sample (pH, TH, Ca, Mg, HCO_3, Cl, NO_3, F, SO_4) are significant and Fe, TDS, Na, K have no significant variances.

Table 7.13 : Bacteriological Parameter of some according to different location

Name of the Village (Block)	Type of sample	Total Coliform Count (MPN/100 ml)	Fecal Coliform Count (MPN/100 ml)
Ichor	DW	79	23
	DW	>23	9.2
Deuli	TW	0	Absent
	DW	430	130
Hijli	DW	>23	16
	TW	2.2	<1.1
Swet-Palash	TW	1.1	<1.1
	DW	>23	>23
	DW	>23	9.5
Asanbani	TW	5.1	1.1
	DW	23	2.2
Saltore	PW	>23	>23
Deuli	PW	>23	>23
Chinakuri	PW	>23	12
Narayanpur	PW	>23	12
Hijli	PW	>23	>23
Dumdumi	PW	>23	12

DW=Dug well and TW=Tube well

Table 7.14 : One way ANOVA for Bacteriological Parameter Analysis

Parameters		Sum of Squares	df	Mean Square	F	Sig.
Total Coliform Count (MPN/100 ml)	Between Groups	1577646.546	6	262941.091	1.828	0.165
	Within Groups	2014074.440	14	143862.460		
	Total	3591720.986	20			
Fecal Coliform Count (MPN/100 ml)	Between Groups	337566.903	6	56261.150	1.321	0.311
	Within Groups	596366.227	14	42597.588		
	Total	933933.130	20			

In bacteriological parameter also analyzed by One Way ANOVA method. And above table is output of one way ANOVA analysis. Also the value in the Sig. column goes below a previously assigned (0.05) critical value, it indicates that the variances in the sample are significant. From this knowledge revealed that the data samples Total Coliform and Fecal Coliform show no significant variance

Table 7.15 : Surface Water Quality of river Damodar during field survey

Name of the Village/Block	**GPS Location**		**DO**	**pH**
	Latitude(N)	Longitude(E)	(mg/L)	
Shaltore (SW)	23°40.756’	86°48.065’	5.13	7.65
Hijli (SW)	23°40.515’	86°48.933’	3.24	7.61
Amdhara (SW)	23°40.659’	86°49.585’	3.13	7.77
Parbelia (SW)	23°40.261’	86°50.205’	6.27	7.79
Deoli (SW)	23°38.889’	86°54.961’	4.87	7.56
Kuribari (SW)	23°38.949’	86°52.612’	6.88	7.24

In Table 7.15, Table 7.16 , water quality parameter (during field measurement) measured and their SPSS analysis (one way ANNOVA) also given in Table 7.14 From the values in the F column, it showed that the Conductivity is least degree of variance whereas the pH highest degree of variance. Each of the ten villages were numbered from 1 to 16 and plotted along the ordinate of the dendrogram and the distance plotted along the abscissa denotes the range of agreement of the responses obtained from water quality test on parameter DO, Conductivity, pH, of this villages. Also groups of villages which are placed higher along the ordinate will have higher degree of correlated responses than groups placed lower along the ordinate even if they are connected by the same length of lines along the abscissa

Table 7.16 : ANOVA Table of Water quality measurement during field survey

Parameters		Sum of Squares	df	Mean Square	F	Sig.
DO	Between Groups	41.439	15	2.763	1.678	0.563
	Within Groups	.000	0			
	Total	41.439	15			
Conductivity	Between Groups	1570894.884	15	104726.326	0.576	0.249
	Within Groups	.000	0			
	Total	1570894.884	15			
pH	Between Groups	1.325	15	0.088	10.543	0.089
	Within Groups	.000	0			
	Total	1.325	15			

In Table 7.16, water quality parameter (during field measurement) measured and their SPSS analysis (one way ANNOVA) also given in above table. Since every case value of sig. columns is greater than 0.05, represents that there is no significant variance of the parameter.

```
Dendrogram using Average Linkage (Between Groups)

                          Rescaled Distance Cluster Combine

    C A S E         0         5        10        15        20        25
  Label        Num  +---------+---------+---------+---------+---------+

  Shaltore (SW)   3   -+
  Hijli (TW)      4   -+-+
  Hijli (SW)      5   -+ +--------------------+
  Deoli (TW)      9   -+ |                    |
  Kuribari (SW)  11   -+-+                    |
  Parbelia (SW)   8   -+                      +--------------------------+
  Narayanpur (TW) 1   -+                      |                          |
  Shaltore (TW)   2   -+-----+                |                          |
  Amdhara (SW)    7   -+     +----------------+                          |
  Amdhara (TW)    6   -------+                                           |
  Deoli (SW)     10   -----------------------------------------------------+
```

Figure 7.5 : Regression analysis

Each of 11 samples collected from different sources, numbered from 1 to 11 and plotted along the ordinate of the dendogram and the distance plotted along the abscissa denotes the range of agreement of the responses obtained from water quality test on parameter DO, Conductivity, pH, of this villages. Also groups of villages which are placed higher along the ordinate will have higher degree of correlated responses than groups placed lower along the ordinate even if they are connected by the same length of lines along the abscissa. According to dendogram analysis,(Figure 7.6) it showed that the water quality of sample no 3,4,5, 9 and 7,2,1,8 are having same, group between 2,6 and 4,11 nearly same but group between 10,4, 11, 6 are found to be different.

7.4. Hydro-geochemical framework

The chloride mass-balance method is one of effective method to assemble time and aerial distribution of ground water recharge in regional boundary of semi-arid area. Fig.10 (a)(b)and (C) presents distribution of chloride concentration throughout the year in the study area. The fluctuations in water table depth with Chloride concentration in different season also depicted in fig 11. From the study it was found that with the increase in water table depth , the weighted average chloride concentration increases, Thus from the clearly conforming to an inverse relationship between the change in chloride concentrations and the water table level fluctuations. With help of GPS reading at sampling point the chloride concentration distribution maps were developed and analyze to Chloride concentration scenario of study area. From map revealed that the chloride concentration is found more in well far from the river and less the area near the river. Chloride concentration is less in monsoon period rather than more in pre monsoon and post monsoon period. Water table depth also less in monsoon period with comparison to pre Monsoon and a post monsoon period. The Interrelation seen between water table and chloride concentration fluctuation also Recharge quantity was also estimated. from equation no (14)(Sharda et.al.,2006)

$$\Delta WT = -0.0169 \times \Delta Cl,\ R^2 = 0.608 (N=20) \quad \text{...7.2}$$

Where

ΔWT is the changes in water table depth

ΔCl,is Change in water table concentration and R is the recharge.

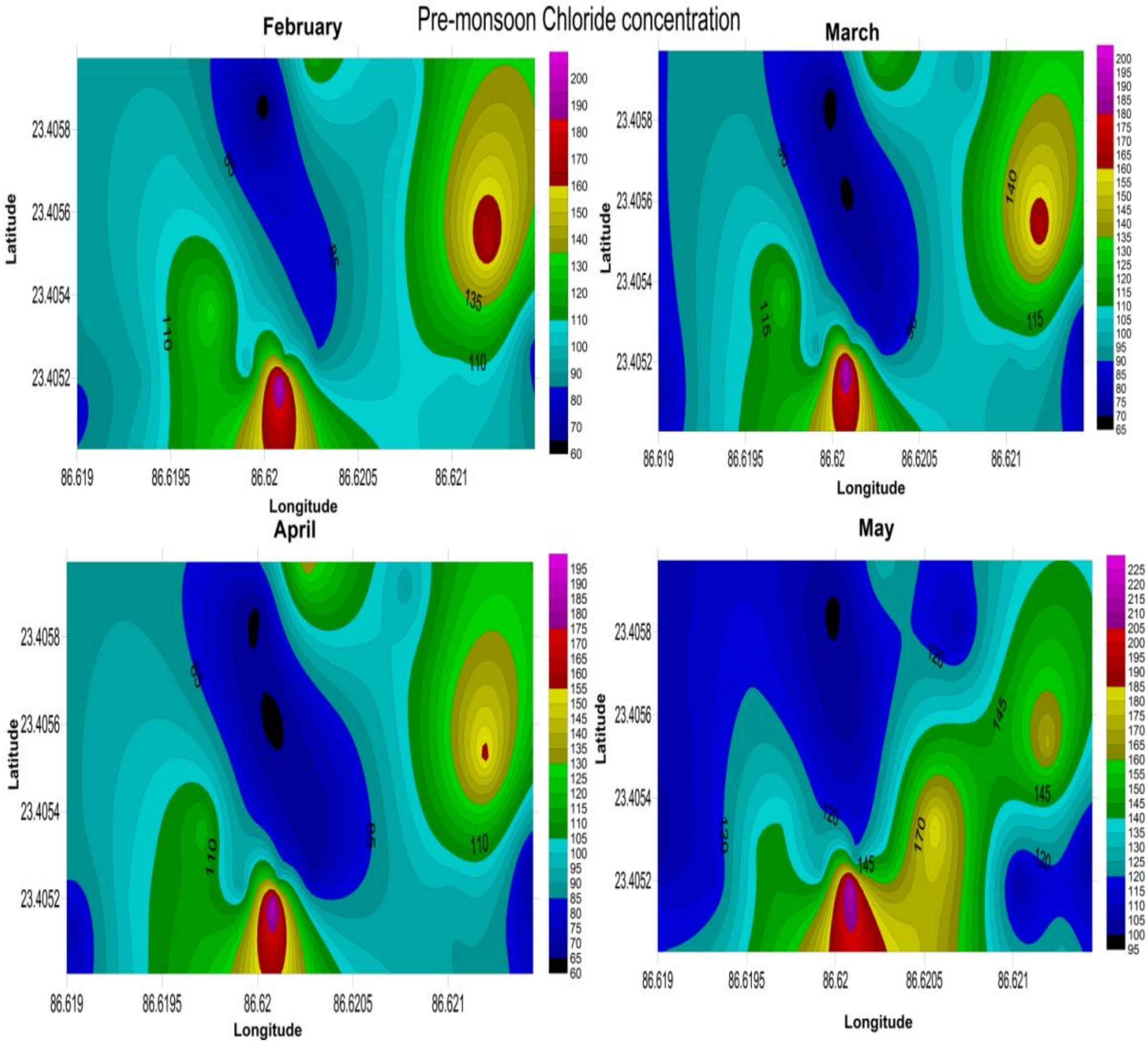


Figure 7.6 : Chloride concentration distribution in Pre monsoon Period

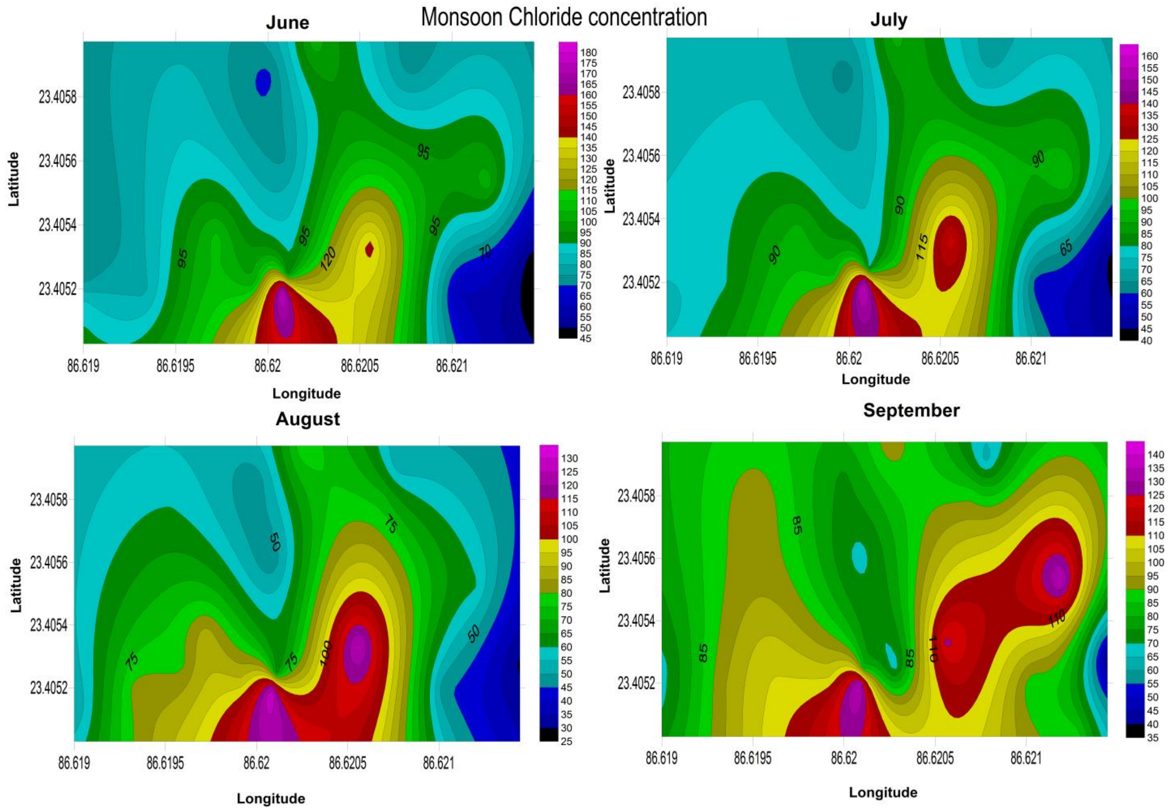


Figure 7.7 : Chloride concentration distribution in Monsoon Period

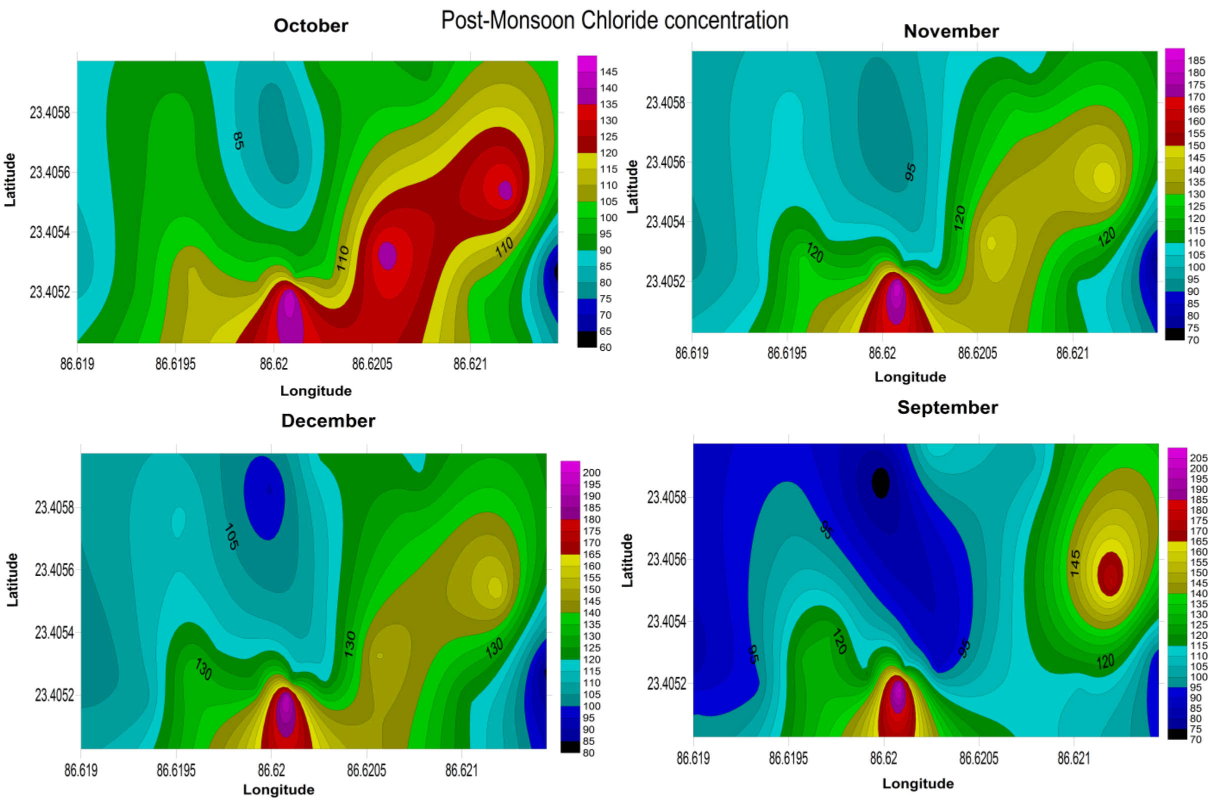


Figure 7.8 :Chloride concentration distribution in Post-Period

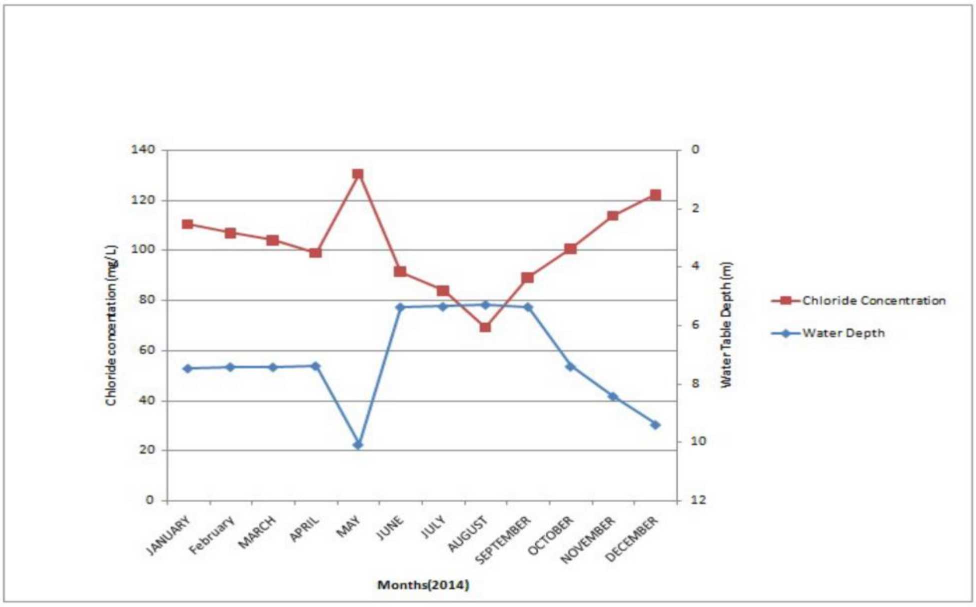


Figure 7.9 : Chloride concentration with respect to water level fluctuation.

7.4. Identification of critical water quality condition of the study Area

Fluoride contamination in drinking water is a burning environmental issue of the world today. A cursory glance at this problem reveals that the people of nearly 29 countries are affected with 'fluorosis' due to intake of fluoride-rich water. India also figures in this list, as the menace of this deadly poison is being reported from many parts of the country1,2. Ingestion of excess fluoride in the human body may cause dental, skeletal and non-skeletal fluorosis. Again, low fluoride less than 0.5 mg/l causes dental caries. Proper safeguards are, therefore, required to be taken to ensure safe fluoride level in drinking water. According to BIS the highest desirable limit of fluoride is specified at 1.0 mg/l and the maximum permissible limit is 1.5 mg/l. In spite of this, fluoride contamination of groundwater is reported to be endemic in as many as 15 states in India. In West Bengal, excess fluoride in groundwater has been detected so far in 43 blocks spread over seven districts, viz. Purulia, Birbhum, Bankura, Malda, South Dinajpur, North Dinajpur and South 24-Parganas5 . As the problem spreads day by day, a scientific inquest to find out the source and cause of fluoride in groundwater of Purulia has become the need of the hour. For example, in the Purulia

District (the present study area), instances of fluorosis are on the rise. The rural population is the worst affected, because of the absence of centralized water-treatment system in these areas. Outbreak of media reports on this issue has become a matter of grave concern for the Government in view of the strategic location of Purulia, its poor socio-economic status and the tribal-dominated demography. Geological and hydrogeological studies were carried out by drilling boreholes in 17 different locations of Purulia spread over the three worst-affected blocks, viz. Purulia I, Purulia II and Hura . A total of 401 samples from these blocks were collected in the post monsoon (winter) period of 2009–2010, which included 242 rock-chip samples from boreholes and 159 water samples collected from three different sources: boreholes, dug wells and tube wells.

These water and rock samples were analysed according to the standard procedures 3, 6 and correlated to find out the plausible reason for fluoride enrichment. The results gave a dismal picture of contamination with fluoride above the prescribed limit detected in a majority of the rock and water samples. For example, in Purulia I block, out of a total of 39 rock-chip and 30 borehole water samples, the maximum fluoride value in borehole water and the enclosed host rock was found to be 10.75 mg/l and 11,400 mg/kg respectively. In Purulia II block, out of 34 borehole water samples, the maximum fluoride value was found to be 6.25 mg/l and 9000 mg/kg respectively. In Hura block, out of 49 borehole water samples, the maximum fluoride value obtained was 6.25 mg/l and 10,200 mg/kg respectively (Table A closer scrutiny of water quality results (of all boreholes, dug wells and tube wells taken together) of these analytical findings further reveals that nearly 54% of the water samples have fluoride content more than 1 mg/l, out of which over 17% is infested with fluoride over 1.5 mg/l (Table 2). In case of rock samples, more than 90% reported fluoride content more than 4000 mg/kg. Study of the borehole rock-cutting samples has revealed that the area is principally underlain by Pre-Cambrian metamorphic represented by granite gneiss (Chotanagpur granite gneiss), biotite granite gneiss, calc-granulites, ultra basic and meta-basic rocks, metasedimentaries, including crystalline limestone, hornblende schist, biotite gneiss, pegmatite and quartz-vein. Microscopic studies of drill cuttings of granite gneiss and pegmatite have revealed fluoride bearing minerals, viz. apatite and fluorite. Two other fluoride-bearing minerals, biotite mica and hornblende, were also noted that might have added fluoride into the groundwater. Fluoride from the hydrothermal fluids usually gets adsorbed into the sheet structure of these silicate minerals and stays there until the conditions are congenial for leaching to take place. Abnormal level of fluoride is also observed within the pegmatite veins. Fluoride mineralization is found to have been favoured by the presence of structurally weak planes like shear/fracture zones, joints and contacts of host rock and vein quartz. Incidentally, all these conditions are prevalent in the present project area

7.5. Development of Prediction Model of Fluoride Contamination

Groundwater is the source of drinking water for many people around the world. Groundwater could be contaminated naturally or because of numerous types of human activities. Residential, municipal, commercial, industrial, and agricultural activities can all affect groundwater quality. Groundwater contamination is the reason of low quality drinking water, loss of water supply, high clean-up cost, high cost for alternative water supplies, and/or potential health problems. A wide variety of materials have been identified as contaminants in groundwater. The data analysis was carried out by combining GIS and statistical methods to test hypotheses about the spatial relationship between measured. Fluoride contamination in drinking water is a burning environmental issue of the world today. A cursory glance at this problem reveals that the people of nearly 29 countries are affected with 'fluorosis' due to intake of fluoride contaminated water. India also affected severely. Quantity of of excess fluoride in the human body may cause dental, skeletal and non-skeletal fluorosis. Again, low fluoride less than 0.5 mg/l causes dental caries. Proper safety measurement are, therefore, required to be taken to ensure safe fluoride level in drinking water. As per IS-10500 desirable limit of fluoride is specified at 1.0 mg/l and the maximum permissible limit is 1.5 mg/l. In West Bengal Purulia, Bankura, Birbhum, Malda, South Dinajpur, North Dinajpur and South 24-Parganasare districts fluoride affected. Purulia, Birbhum, Bankura, are more affected. Based on the review of available in literature, variables affecting the release of fluoride into groundwater, namely: fluoride concentration, surface elevation, soil reaction (pH), organic matter, iron content, surface geology and proximity to channels, were selected to predict the level of fluoride concentration. Q-GIS was used to generate process and analyze spatial data from the study area, District digital maps of rivers and drainage networks, are collected from District website and PHED Govt. of West Bengal website and the digital elevation model and surface geology map of the country were collected from DDWS from Govt of India website The point map of Fluoride concentration, prepared by School of Water Resources engineering Jadavpur University, were used to create fluoride concentration distribution, surface analyze the geospatial relationship of fluoride and validate degree and distribution pattern of fluoride in the predicted spatial. The 'Proximity toolset' in the 'Analysis toolbox' and Proximity Plugins of ArcGIS and Q-GIS was used to establish proximity relationships with feature data. The toolset can produce output information with buffer features. Multi ring buffer tool was employed to create a cover of channel proximity classifying the area into Grid on the basis of latitude longitude data from the GPS measurement with help GPS (Gramin) tool. In the study area fluoride levels are in range of 0.25 mg/l to 1.65 mg/l . Specially in geographical location in between of N-23.25° E-86.45 to N 23.3,E 86.55° . Experimental values gives 0.55to 1.15 % error with prediction model for fluoride level determination. Univariate statistics also given below in Table 7.17 Unvariate statistics an Figure 7.10 the prediction map.

Table 7.17 : Unvariate Statistics

Univariate Statistics

	X	Y	Z
Minimum:	86.28188889	23.24268333	0.27
25%-tile:	86.3012	23.26057222	0.43
Median:	86.45938333	23.2796	0.62
75%-tile:	86.5125	23.31811944	1.19
Maximum:	86.56123333	23.36166667	1.59
Midrange:	86.42156111	23.302175	0.93
Range:	0.27934443999999	0.11898334	1.32
Interquartile Range:	0.21130000000001	0.05754722	0.76
Median Abs. Deviation:	0.10185	0.019349999999999	0.3
Mean:	86.410203519333	23.289838703333	0.80466666666667
Trim Mean (10%):	86.408456197692	23.287940811538	0.78538461538462
Standard Deviation:	0.10235069793686	0.03238814987069	0.4455239113174
Variance:	0.010475665368162	0.0010489922520463	0.19849155555556
Coef. of Variation:			0.55367511762726
Coef. of Skewness:			0.4368866756313

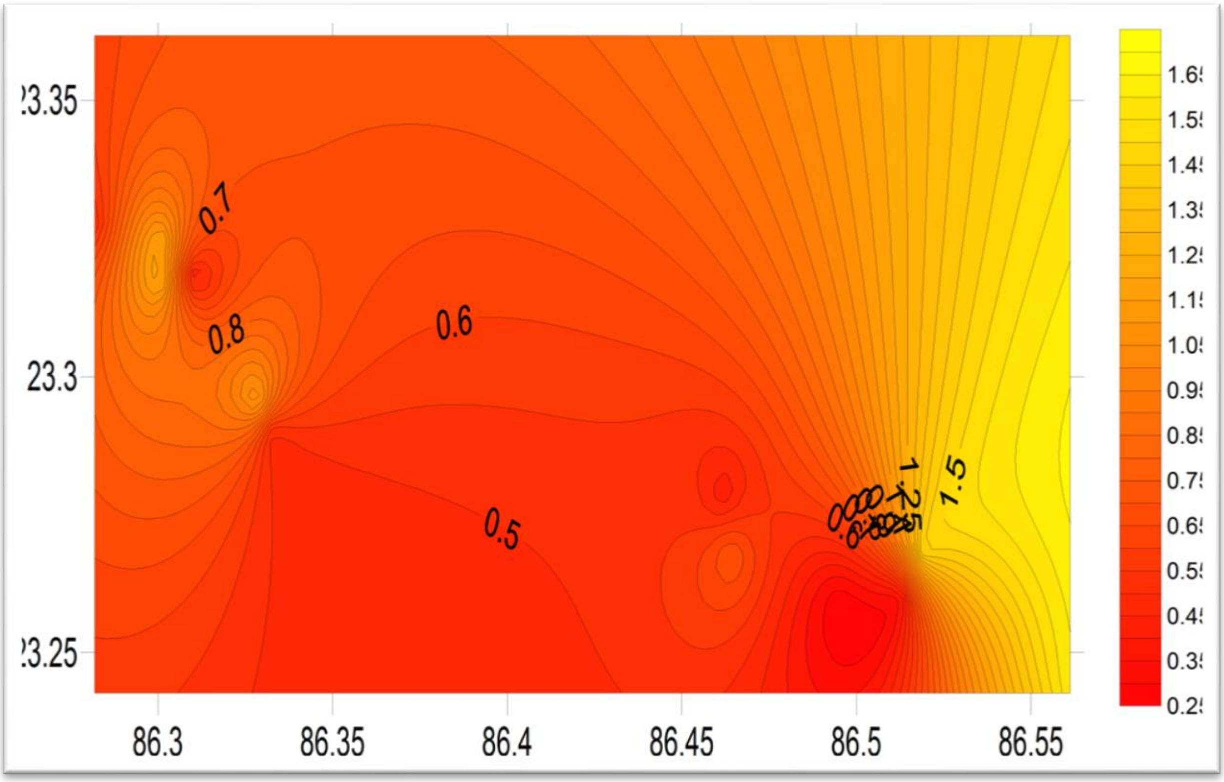


Figure 7.10 :Fluoride prediction map of the study area

Remarks

Geographic Information System (GIS) has become an indispensable analysis and modelling tool for many real-world problems involving spatial data distribution. In this study, the GIS capabilities were used to extract data from various cover maps, namely: Fluoride, Arsenic, iron content, pH, organic matter content, drainage and DEM maps, and fed into a three layered to predict the level of Fluoride values. The multi-layered perception predicted the level of contaminant with reasonable accuracy. Finally, the IK can suitably derive the probability maps of fluoride concentration under the threshold drinking water standard in the study area. Therefore it is very useful for the local and state Govt. authorities to manage water resources and to prevent villagers from drinking fluoride -contaminated groundwater as well as controlled in regard of agricultural water use. The pattern of variability in measured station can able to estimate and validate for the rest of study area by probability risk mapping tool. This model will significantly reduce the manpower cost of monitoring Fluoride concentration and effectively provide reliable estimation of fluoride concentration at regional basis. In summary, the proposed modelling approaches to the estimation of fluoride concentration using on-site measurement data of other water quality variables can be an alternative way to quantify the fluoride contamination and to provide predictive information for better human health management.

CHAPTER -8
HYDROLOGICAL DATA ANALYSIS AND PROCESSING

8. Hydrological data analysis and processing

The goal of the hydrological data analysis is the estimation of water availability and its reliability. At first, the rainfall data are analysed in different time scales: year, day and decade. The annual data offer a general climatic picture, the daily time step is the original time scale of the data and the seasonal interval the actual scale that is used for the reliability analysis. Due to their quality, the data had to be checked thoroughly. For all data distribution, independence, periodicity and trend were investigated and further tests of data consistency were necessary. The basic statistical investigation and interpretation was done set by set. Only then further processing was conducted.. Those data are supplied to the precipitation – evaporation model that evaluates the different geomorphological components of the catchment. Basic two models Groundwater modelling system and HEC-RAS model are developed and out put results are discussed in this chapter.

8.1. Modeling the effect of stream flow on groundwater flow fields

The study area has been demarcated by using Google Earth Tool with available GPS data recorded during field survey. Accordingly it defined all hydrologic and hydraulic features in the MODFLOW conceptual model. From lithologic logs which traced all soil types including depth and placed as input parameter in the model. Aquifer characteristics including yield of well has been given in the specified locations. The river head along a specified head arc is assumed to vary linearly along the length of the arc. The ground elevation of the different points in the site has been derived using GPS information available to us based on scatter point techniques. There are two layers in the model. Both the layers are modeled as unconfined layers. The upper layer and lower layer are made of coarse sand with gravels and coarse sand respectively. The influx to the system is primarily made through recharge due to rainfall. Since then groundwater level data of the catchment area was not available at regular basis. So the actual groundwater recharge could not be estimated with the help of water level fluctuation method suggested by (G.E.C) Ground Water Estimation Committee, 1982. Keeping in view 20% of total rainfall suggested by G.E.C. has been considered as recharge input in the catchment area. MODFLOW conceptual model has been simulated on the basis of hydraulic parameters collected during pumping test and subsequently analyzed by Neuman's Straight Line Method.

The following assumptions were made for the modeling:

I. The river is not separated from the aquifer by any confining material.
II. The influx to the system is primarily through recharge due to rainfall.
III. The flow in the aquifer system is steady-state.

Table 8.1: Input to the model for Pumping Test-1

Location: Deuli, Block: Santuri, Pumping test ID: PT-1

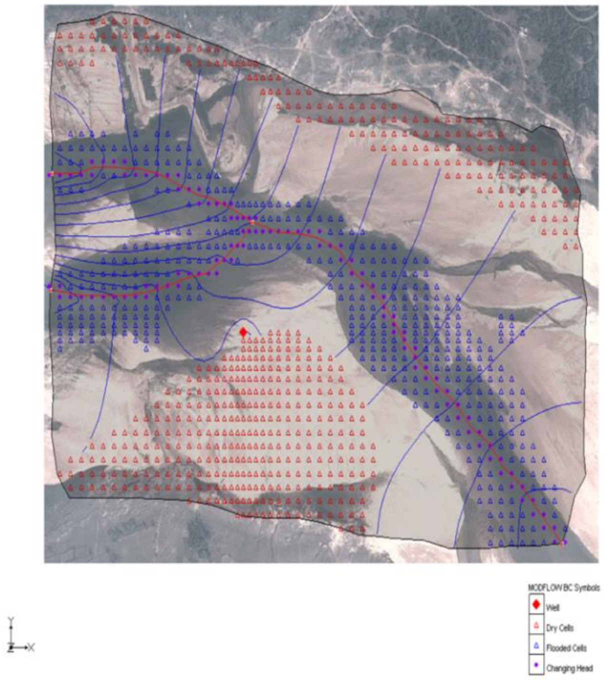
Area under study m^2	Recharge rate m/day	No of pumping well	Discharge rate m^3/hr	No of layers
566,021	0.000679	1	44	2

Output of the model for Pumping Test-1

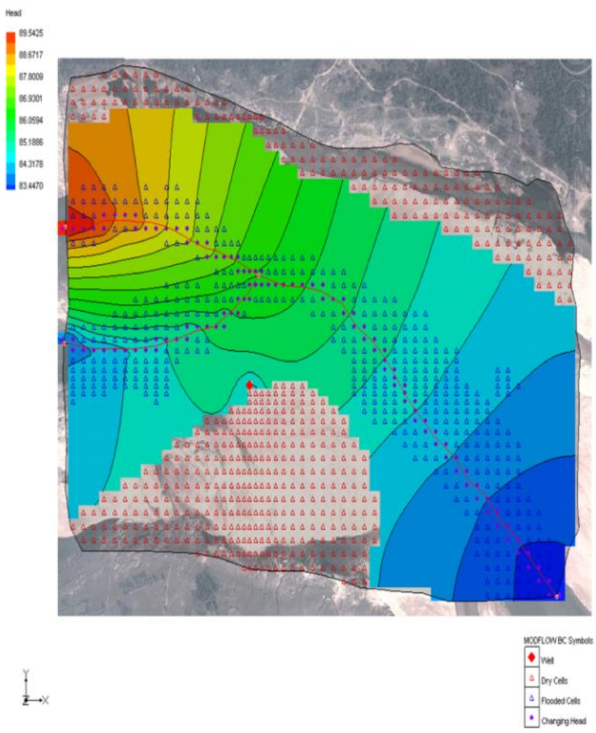
The head distribution obtained for the river stage in both the layers is shown in Figure 8.1. In layer 1 the red marked cells indicate that the cells are inactive due to elevation difference. The inactive cells are present in this model due to the fact that the groundwater tables at those points are nearly 1 m bgl (below ground level). The red line addresses the river which represents a changing head boundary. The blue cells are flooded cells which are due to the obvious effect of the presence of the river and the elevation differences of those points. The head distribution shows that the water extraction is mainly from the river because the other side of the pumping well is surrounded by no flow boundary. In layer 2, every cell is active. The contour line converged around the pumping well and the specified head boundary shows the active effect of river in layer 2. There is no dry zone present here. The contour diagram shows that the water withdrawal due to pumping is from both the no flow boundary and the specific head boundary. The steady state flow budget of the zone after MODFLOW conceptual model simulation is shown below Table 8.2.

Table 8.2 : Zone Budget calculated from MODFLOW conceptual model for PT-1

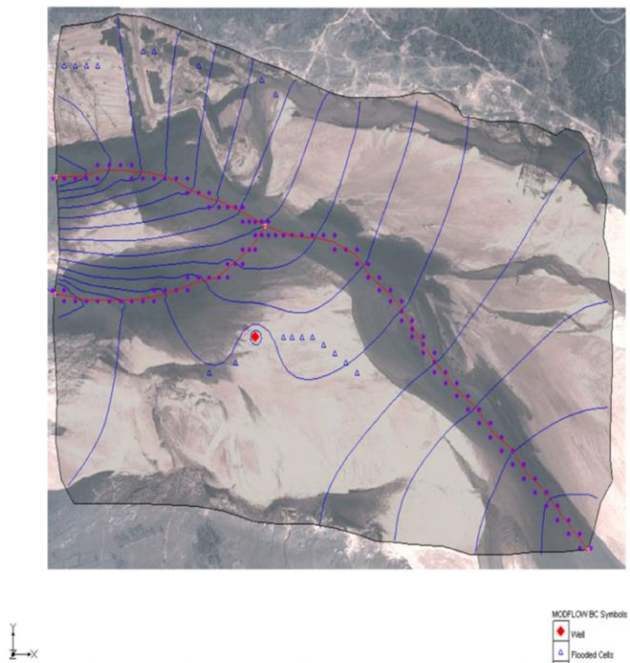
Budget Term	Flow (m^3/d)
Flow Budget for Zone 1	
IN:	
Constant heads	15190.038924456
Wells	0.0
Recharge	377.24616873264
Total IN	15567.285093188
OUT:	
Constant heads	14511.284964129
Wells	1056.0
Recharge	0.0
Total OUT	15567.284964129
SUMMARY:	
IN - OUT	0.0001290589571
Percent Discrepancy	8.29039594e-007



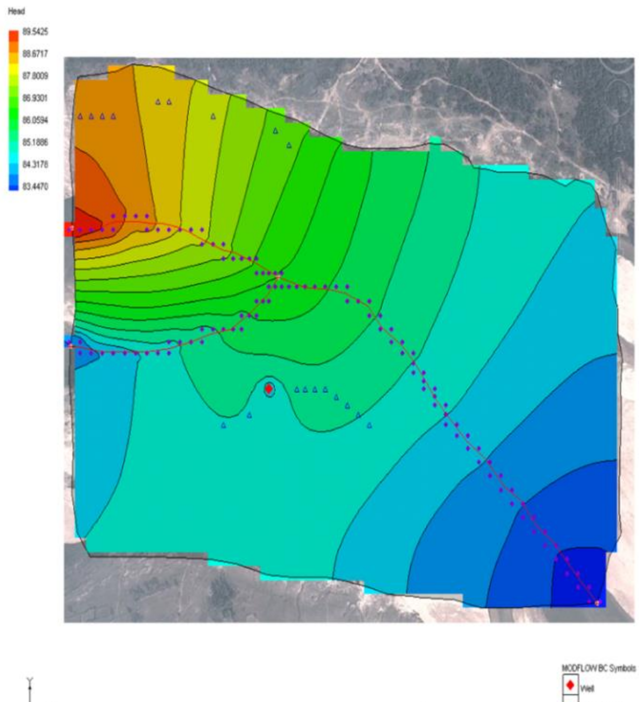
Contour line showing head distribution for layer 1 for PT-1



Contour line with color ramp showing head distribution for layer 1 for PT-1



Contour line showing head distribution for layer 2 for PT-1



Contour line with color ramp showing head distribution for layer 2 for PT-1

Figure 8.1: Head distribution in different layer for pumping test-1

Input to the model for Pumping Test-2

In this site both the upper and lower boundary are hard rock region and assumed as no-flow boundary and the river flowing through the middle of the site is assumed as a specified head boundary corresponding to the average stage of the river. The conditions and constraints are almost same as above case. The input details for this model are given in the Table 7.3.

Table 8.3 : Input to the model

Location: Chinakuri Ghat, Block: Santuri, Pumping test ID: PT-2

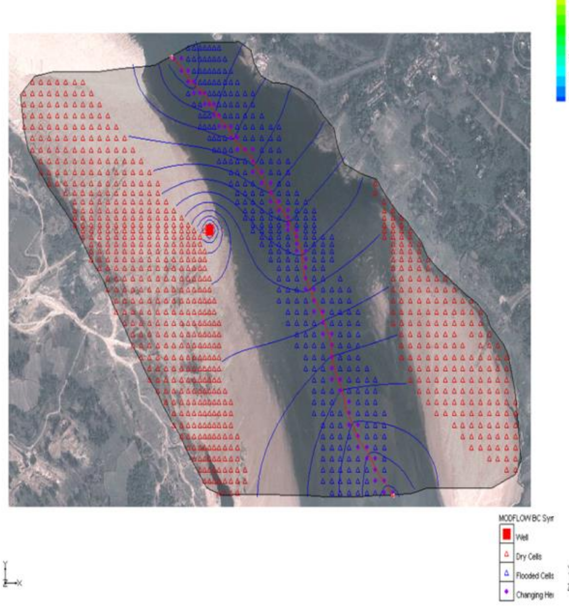
Area under study m^2	Recharge rate m/day	No of pumping well	Discharge rate m^3/hr	No of layers
652141.91	0.000679	1	44	2

The head distribution for the river stage in both layers is shown in Figure 8.2. In layer 1 the dry cells are closed to the no-flow boundary zone because of the elevation differences (Fig.8.8). The contour diagram (Fig. 8.7) shows that due to pumping there is a depression zone near the pumping well and as the pumping well is guided by no-flow boundary at one side, the whole contribution comes from the adjacent part of the river. In layer 2, there is no presence of dry zones . The contour diagram shows that the water extraction due to pumping is from both the no-flow boundary and the river. There is a flooded zone in layer 2 due to the affect of pumping well and changing river stage. The steady state flow budget of the zone after MODFLOW conceptual model simulation is shown below

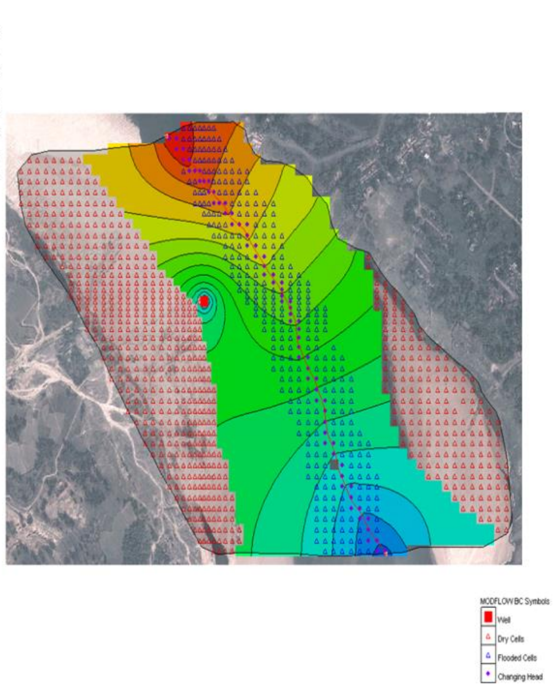
Table 8.4 : Zone Budget calculated from MODFLOW conceptual model for PT-2

Budget Term	Flow (m^3/d)
Flow Budget for Zone 1	
IN:	
Constant heads	1362.4847457632
Wells	0.0
Recharge	361.42553706467
Total IN	1723.9102828279
OUT:	
Constant heads	667.91140493285
Wells	1056.0
Recharge	0.0
Total OUT	1723.9114049328
SUMMARY:	
IN - OUT	-0.001122104935
Percent Discrepancy	0.0000650906806

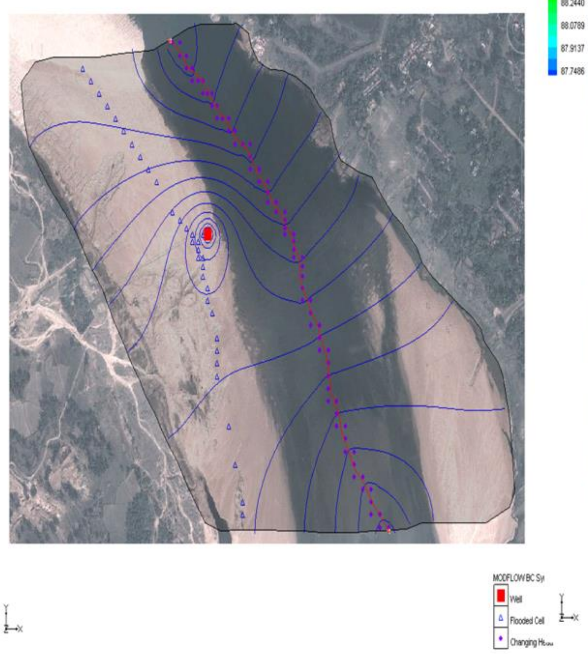
Photo 8.1: Withdrawal rate measurement at 2nd pumping test in Damodar river



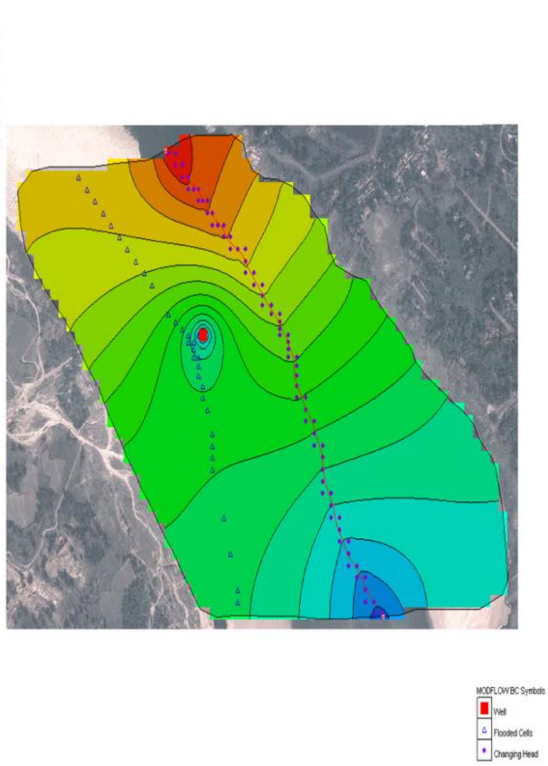
Contour line showing head distribution for layer 1 for PT-2



Contour line with color ramp showing head distribution for layer 1 for PT-2



Contour line showing head distribution for layer 2 for PT-2



Contour line with color ramp showing head distribution for layer 2 for PT-2

Figure 8.2 : The head distribution for the river stage in both layers

Input to the model for Pumping Test-3

In this site model boundaries are same as the previous site. Input details of the model are given in the Table 8.5

Table 8.5 : Input to the model

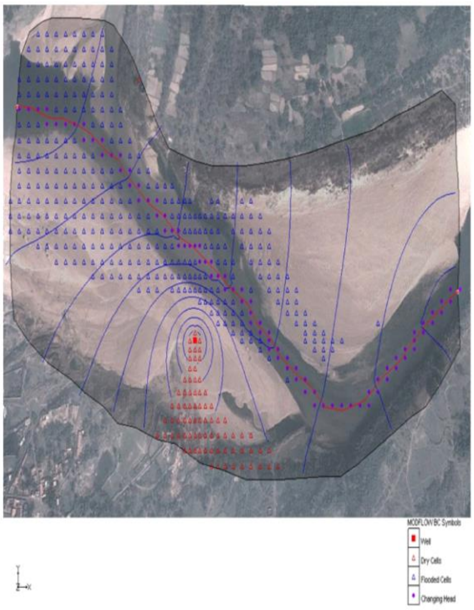
Location: Saltore, Block: Neturia, Pumping test ID: PT-3

Area under study m^2	Recharge rate m/day	No of pumping well	Discharge rate m^3/hr	No of layers
618400	0.00071	1	44	2

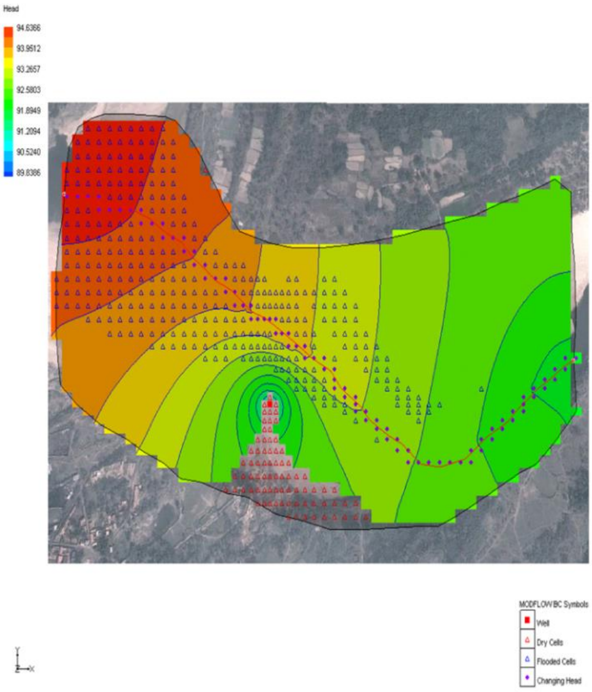
The head distribution for the river stage in both layer 1 and layer 2 were obtained Figure 8.3 . In the first layer dry cells are present around the pumping well and extend up to the no-flow boundary. The flooded cells are almost adjacent to the river. The contour diagram nearing the pumping well shows that the water is extracted from the river head boundary and as well as from the no-flow boundary. The layer 2 indicates that few amounts of cells are flooded around the pumping well and extends up to no-flow boundary.The steady state flow budget of the zone after MODFLOW conceptual model simulation is shown below

Table 8.6 : Zone Budget calculated from MODFLOW conceptual model for PT-3

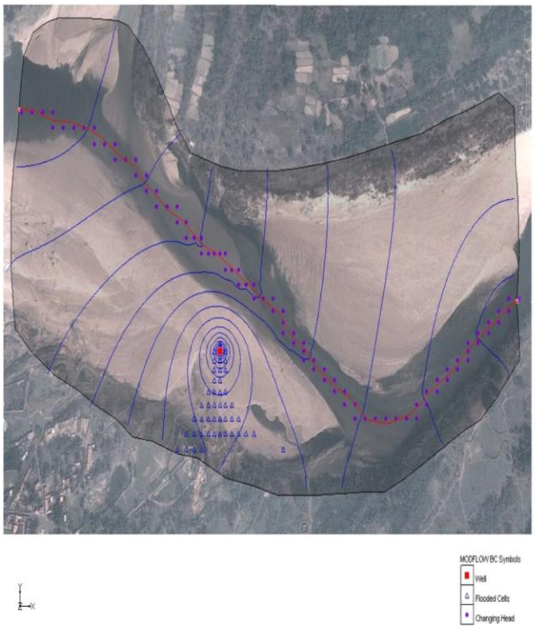
Budget Term	Flow (m^3/d)
Flow Budget for Zone 1	
IN:	
Constant heads	1080.4206252713
Wells	0.0
Recharge	253.02674528211
Total IN	1333.4473705534
OUT:	
Constant heads	277.4475412555
Wells	1056.0
Recharge	0.0
Total OUT	1333.4475412555
SUMMARY:	
IN - OUT	-0.000170702115
Percent Discrepancy	0.0000128015637



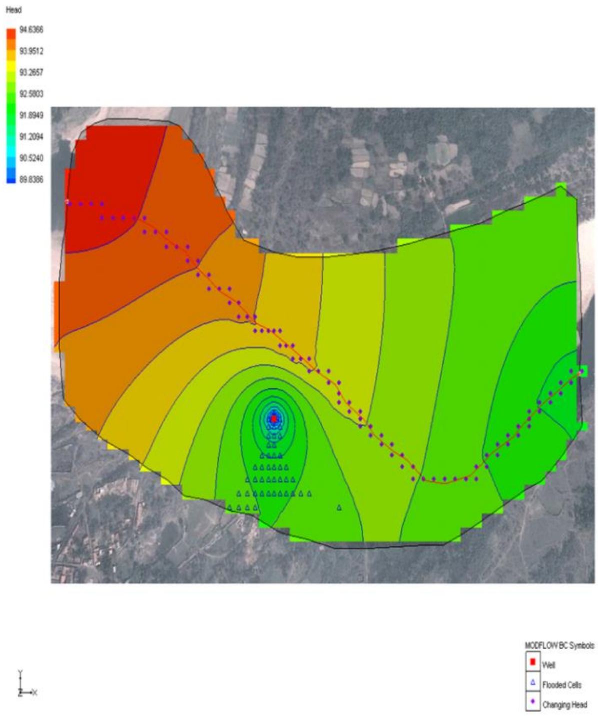
Contour line showing head distribution for layer 1 for PT-3



Contour line with color ramp showing head distribution for layer 1 for PT-3



Contour line showing head distribution for layer 2 for PT-3



Contour line with color ramp showing head distribution for layer 2 for PT-3

Figure 8.3: The head distribution for the river stage in both layers

Input to the model for Pumping Test-4

In this site model boundaries are same as the previous site. The conditions and constraints are same as the previous model. The pumping well is placed on a high elevation zone in the river, surrounded by the specific head boundary. Input details are given in the Table 8.7.

Table 8.7 : Input to the model

Location: Narayanpur, Block: Neturia, Pumping test ID: PT-4

Area under study m^2	Recharge rate m/day	No of pumping well	Discharge rate m^3/day	No of layers
838,983	0.00071	1	1056	3

The head distribution in all the layer 1 and layer 2 and layer 3 are shown in Figure 8.4.layer 1, dry cells are asymptotic to the no-flow boundary on the upper boundary side due to the elevation differences. The flooded cells are mostly around the specific head boundary. In layer 2 there are some flooded cells around the pumping well and contour line converged near the pumping well. There are no dry cells in layer 3. The curvy nature of contour line near to the pumping well indicates that the water is extracted mainly from the island where the pumping well was placed. The steady state flow budget of the zone after MODFLOW conceptual model simulation is shown below

Table 8.8 : Zone Budget calculated from MODFLOW conceptual model for PT-4

Budget Term	Flow (m^3/d)
Flow Budget for Zone 1	
IN:	
Constant heads	1037.9935208168
Wells	0.0
Recharge	271.3604805842
Total IN	1309.354001401
OUT:	
Constant heads	253.3531634016
Wells	1056.0
Recharge	0.0
Total OUT	1309.3531634016
SUMMARY:	
IN - OUT	0.000837999396
Percent Discrepancy	0.0000640009803

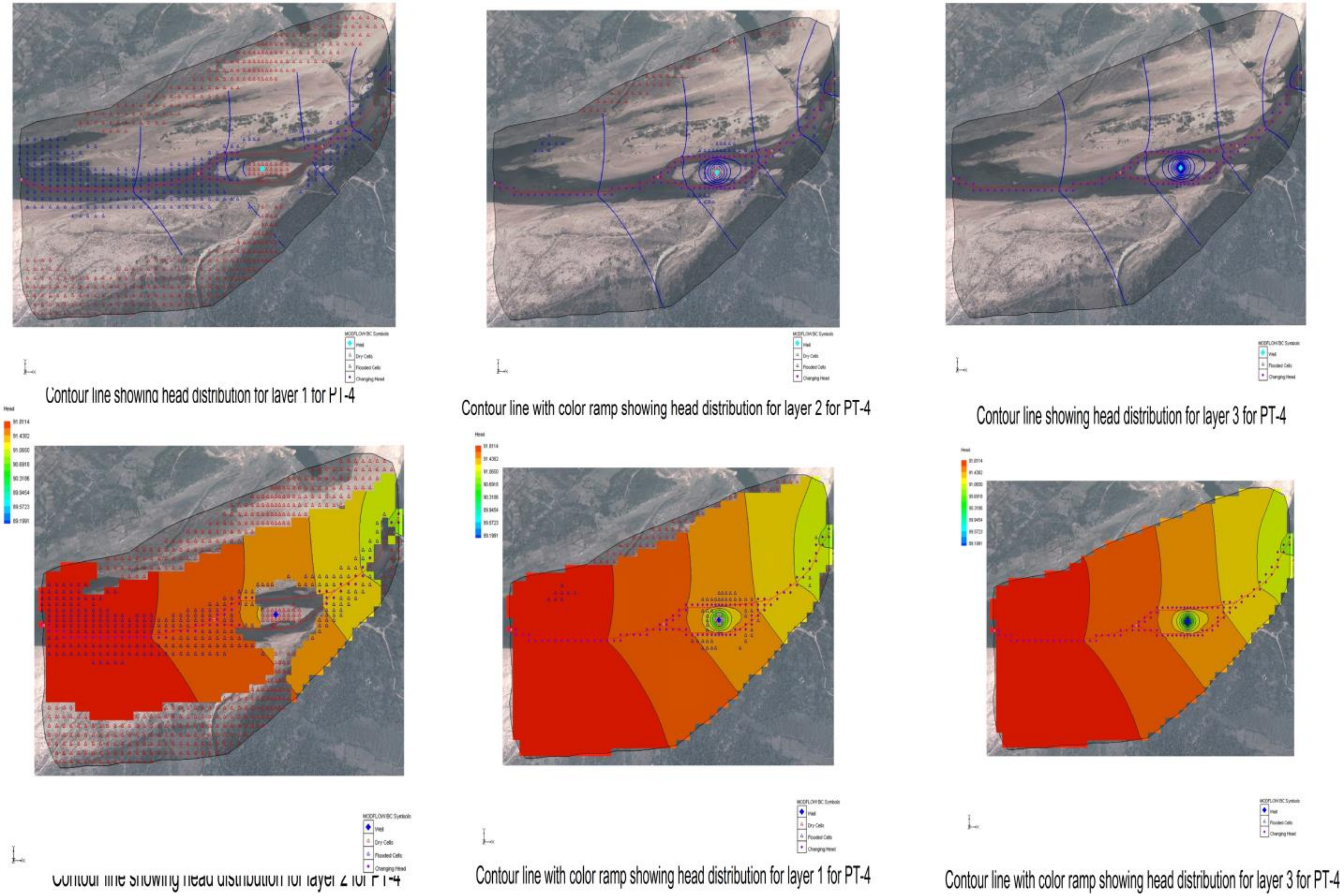


Figure 8.4 : The head distribution for the river stage in both layers

8.2. HEC-RAS data analysis and result

Rural development in the context of rain fed rivers and hydrological and hydraulic conditions regardless of the watershed and river, an incensement risk of flooding and loss of the other side can make investment in these areas. The purpose of this study is to evaluate the behaviour of river flooding and the spread of Futiary catchment area . combining hydrological model HEC-HMS and HEC-RAS hydraulic model was used for this purpose, meteorological and hydrometric stations using rainfall data and runoff area HEC-HMS model was calibrated and river cross sections taken at intervals residential areas, flood zones showed with return periods of 10, 20 and 50 years in GIS using HEC-GeoRAS amendments in Arc view is displayed The results indicate the ability of HEC-HMS and HEC-RAS models in simulated the behaviour of the hydrological and hydraulic basin is Karaj. The ability to use geographic information systems in providing inputs to the accuracy of model and view, combining performance shows. If the present composition of hydrometric and rainfall accompanied with appropriate data accurately convenient to be able to simulate floods on the other hand it is an accurate survey maps of the river bed and residential areas floodplain could be the efficiency and the accuracy of the maps as increase output. Thus, therapies studied in thw water shed management it is very helpful . Development of model includes collection of field data to get the out results and graph for sustainable management. Different steps of model discuses below with input data, intermediate interfaces out put results in this chapter. Filed data collection includes collection of rainfall and evaporation data given in Table 8.9 and Table 8.10 . In this current mode more than 30 years hydrologicatal data are incorporated. Analysis of all the data i.e year wise list of precipitation, evaporation to find out runoff, area, and the Discharge shown in Table 8.11 . Also calculating $\bar{x}$ for the Gumbel's method.

Table 8.9: Year wise monthly Rainfall Data from 1970-2002

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1970	17.01	35.59	42.66	8.04	43.71	249.4	261.3	277.71	239.6	93.02	8.15	0
1971	36.63	66.58	2.68	95.61	132	407.6	471.2	521.27	247.4	121.3	5.88	0
1972	6.84	50.43	1.23	42.18	4.39	64.57	261.6	457.54	197.2	141.5	16.1	0.88
1973	14.51	5.12	24.16	8.79	81.17	264.9	174.3	345.88	380.4	166.6	14.95	26.3
1974	2.6	1.57	57.72	4.23	51.41	84.87	400.3	289.48	284.5	150.4	3.9	0.04
1975	18.79	9.94	25.37	3.72	39.18	168.3	402	223.46	217.1	110.8	10.96	0
1976	1.28	14.02	5.45	55.24	55.39	64.44	387.2	287.25	359.6	6.96	2.99	0
1977	2.98	27.88	0.23	69.11	201.4	280.2	466.3	333.13	205.1	57.49	35.08	23.5
1978	3.09	81.96	54.71	76.97	55.25	252.9	312.3	338	414.4	80.99	12.33	18.6
1979	28.96	45.19	4.75	37.87	12.13	97.48	334.2	212	184.3	27.48	15.61	23.1
1980	45.48	30.11	52.67	2.16	65.55	285.6	321	332.2	101.3	56.64	0.22	2.49
1981	22.16	35.67	60.88	48.92	97.53	78.27	380.4	212.68	229.4	1.31	6	7.43
1982	16.56	24.95	64.72	59.91	53.22	113.9	189.7	440.27	209.5	5.51	12.86	1.87
1983	3.92	43.58	37.99	37.32	108.6	169.1	266	280.19	126.4	112.5	0.46	2.56
1984	42.46	22.19	0.54	44.75	33.36	507.3	303	370.32	137.4	58.3	0.02	0
1985	24.44	6.04	0.37	17.52	26.87	145.1	421.9	297.46	154.8	115.5	0.05	0.58
1986	22.21	31.81	1.92	26.93	58.16	168.1	377.4	261.93	414.1	37.92	37.04	5.31
1987	2.98	5.85	11.41	31.93	46.57	60.37	336.9	491.86	180.8	63.79	25.72	3.9
1988	6.05	31.08	34.64	18.33	67.35	126.6	277.5	296.6	175.1	64.32	20.61	0.54
1989	5.12	0.4	9.22	1.49	106.9	220.3	254.6	262.09	213.7	116.3	7.41	16.8
1990	0.16	39.99	41.06	14.45	138.5	278.8	667.7	234.67	240	102.7	9.73	0.9
1991	8.61	13.41	51.46	16.38	36.21	227.5	264.3	510.22	222	63.9	4.63	29
1992	22.31	8.28	0.29	8.22	48.8	165.7	290.4	152.39	141.1	70.13	1.05	0
1993	4.07	0.58	38.89	39.2	60.8	244.7	282.7	290	308.3	112.4	7.57	0
1994	26.61	42.56	32.46	38.02	46.54	277.8	337.7	349.81	97.13	153.1	2.24	0
1995	17.85	25.4	29.38	8.28	58.67	68.56	293.7	246.83	336.8	71.32	17.61	0
1996	13.9	18.6	5.14	20.42	17.26	287.4	226.2	434.98	123.1	144.2	0.66	0
1997	35.56	12.08	24.17	37.98	103.9	52.31	363.1	395.38	213.5	38.21	21.33	16.1
1998	30.24	62.41	17.31	27.48	96.36	128.9	218	222.38	214.7	224.7	34.18	0
1999	11.04	7.12	0.23	0.31	101	162	280.8	410.01	330.2	42.41	10.11	0
2000	5.84	55.33	0.35	45.16	73.09	188.5	166.3	197.05	196	19.74	6.35	1.05
2001	0.87	3.74	52.55	23.93	63.03	168.4	205.5	282.68	80.77	110.2	9.86	0
2002	35.73	17.37	5.85	34.67	40.61	139.9	245.3	367.75	206.9	84.34	15.5	0

Table 8.10: Year wise monthly Evaporation Data from 1970-2002

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1970	5.11	5.87	6.93	8	8.36	6.92	5.42	4.81	4.84	5.46	5.81	5.37
1971	4.91	5.89	7.43	7.21	7.77	6.14	5.06	4.98	5.33	5.15	5.22	5.03
1972	5.46	5.58	7.58	8.55	9.16	8.22	6.55	5.94	5.93	6.16	5.8	5.34
1973	5.55	6.3	7.78	9.04	7.98	7.51	6.13	6.14	5.75	5.54	5.7	5.17
1974	5.47	6.15	6.95	8.09	9.22	7.97	6.15	6.36	6.08	5.74	5.77	5.21
1975	5.25	5.39	6.67	8	8.66	7.17	5.71	5.46	4.88	4.99	5.23	4.93
1976	5.15	5.78	7.09	8.18	8.18	7.14	5.45	4.93	4.93	5.76	5.45	5.36
1977	5.11	6.05	7.4	7.74	7.71	6.54	4.77	4.87	5.11	5.33	5.16	4.78
1978	4.89	5.51	6.98	7.72	8.03	6.34	4.88	4.77	4.52	5.08	5.25	4.81
1979	5.15	5.31	7.08	7.85	8.75	7.41	5.25	5.16	5.29	5.51	5.21	4.62
1980	4.94	5.66	6.69	8.14	8.22	5.81	4.35	4.99	5	5.28	5.91	5.11
1981	4.71	5.56	6.56	7.52	7.8	6.91	5.26	4.89	5.17	6.03	5.98	4.83
1982	5.28	5.62	6.59	7.85	8.58	7.09	5.59	4.94	5.48	5.78	5.62	5.23
1983	5.06	5.65	6.7	7.93	8.13	7.46	5.94	4.91	5.09	5.26	5.64	5.13
1984	5.06	5.65	7.36	8.17	8.34	6.6	5.46	5.09	5.19	5.82	5.89	5.12
1985	4.9	5.77	6.86	8.11	8.3	7.2	5.34	5.07	5.02	5.75	5.94	5.3
1986	5.19	5.78	7.2	8.08	8.36	7.11	5.87	5.4	5.22	5.46	5.4	4.94
1987	5.16	5.92	6.91	8.05	8.11	7.08	5.32	4.76	4.97	5.76	5.48	5.15
1988	5.18	5.87	6.75	7.93	7.9	6.59	5.15	4.78	5.26	5.64	5.7	5.04
1989	5.18	5.8	7.09	8.26	8.33	6.87	5.64	5.02	5.03	5.43	5.56	5.1
1990	5	5.37	6.3	7.35	7.93	6.81	5.3	4.88	4.95	5.34	5.28	5.14
1991	5.11	5.91	7.11	8.03	8.36	6.94	5.49	5.1	5.21	5.59	5.58	5.08
1992	5.14	5.73	7.04	8.12	8.32	6.99	5.51	5.12	5.2	5.56	5.66	5.1
1993	5.19	5.95	7.07	7.99	8.3	7.04	5.52	5.13	5.18	5.58	5.61	5.08
1994	5.21	5.85	7.11	8.02	8.35	7.04	5.47	5.08	5.18	5.57	5.65	5.17
1995	5.1	5.82	6.97	8.13	8.31	7.08	5.39	4.86	4.88	5.48	5.42	5.11
1996	5.34	5.8	6.58	8.05	8.29	7	5.38	4.64	5.23	5.77	6.04	5.11
1997	5.09	5.92	6.78	7.92	8.37	7.39	5.26	5	5.03	5.8	5.6	4.67
1998	5.14	5.53	6.74	7.61	8.02	7.24	5.32	4.74	5	5.34	5.33	5.14
1999	5.17	5.83	7.22	8.39	8.35	7.1	5.44	4.68	4.53	5.45	5.55	4.8
2000	5.32	5.08	6.81	8.25	8.26	6.83	5.26	5.27	5	5.69	5.76	5.25
2001	5.31	5.83	6.98	8.08	8.39	6.46	4.9	5.17	5.32	5.44	5.38	5.01
2002	5.12	5.86	7.14	8.21	8.46	7.24	5.65	5.09	5.23	5.73	5.73	5.1

Table 8.11: Calculation of total, average precipitation, evaporation and runoff

Sl.no.	Ppt total	Pptavg	eva. total	eva.avg	Runoff(mm)	R in m
1970	1276.09	106.341	72.9	6.075	100.266	0.10027
1971	2108.11	175.676	70.12	5.84333	169.833	0.16983
1972	1244.49	103.708	80.27	6.68917	97.0183	0.09702
1973	1507.11	125.593	78.59	6.54917	119.043	0.11904
1974	1331.04	110.92	79.16	6.59667	104.323	0.10432
1975	1229.52	102.46	72.34	6.02833	96.4317	0.09643
1976	1239.8	103.317	73.4	6.11667	97.2	0.0972
1977	1702.29	141.858	70.57	5.88083	135.977	0.13598
1978	1701.57	141.798	68.78	5.73167	136.066	0.13607
1979	1023.09	85.2575	72.59	6.04917	79.2083	0.07921
1980	1295.45	107.954	70.1	5.84167	102.113	0.10211
1981	1180.64	98.3867	71.22	5.935	92.4517	0.09245
1982	1192.95	99.4125	73.65	6.1375	93.275	0.09328
1983	1188.58	99.0483	72.9	6.075	92.9733	0.09297
1984	1519.68	126.64	73.75	6.14583	120.494	0.12049
1985	1210.66	100.888	73.56	6.13	94.7583	0.09476
1986	1442.8	120.233	74.01	6.1675	114.066	0.11407
1987	1262.1	105.175	72.67	6.05583	99.1192	0.09912
1988	1118.68	93.2233	71.79	5.9825	87.2408	0.08724
1989	1214.43	101.203	73.31	6.10917	95.0933	0.09509
1990	1768.66	147.388	69.65	5.80417	141.584	0.14158
1991	1447.67	120.639	73.51	6.12583	114.513	0.11451
1992	908.6	75.7167	73.49	6.12417	69.5925	0.06959
1993	1389.12	115.76	73.64	6.13667	109.623	0.10962
1994	1403.97	116.998	73.7	6.14167	110.856	0.11086
1995	1174.37	97.8642	72.55	6.04583	91.8183	0.09182
1996	1291.87	107.656	73.23	6.1025	101.553	0.10155
1997	1313.68	109.473	72.83	6.06917	103.404	0.1034
1998	1276.6	106.383	71.15	5.92917	100.454	0.10045
1999	1355.26	112.938	72.51	6.0425	106.896	0.1069
2000	954.8	79.5667	72.78	6.065	73.5017	0.0735
2001	1001.54	83.4617	72.27	6.0225	77.4392	0.07744
2002	1193.95	99.4958	74.56	6.21333	93.2825	0.09328

CALCULATION OF AREA USING DEM

Q-GIS software was used for Calculation of area surrounding station 20, 6, 0.4, and at junction point 10.25 of Futiari and Darokeswar River using QGIS Method by placing several contour point and elevation point. Using DEM Method for understand it visually by the colour change. Figure 8.5 shown below. After development of Dem probable flow estimation was done and represented in Table 8.12 and out result shown in tabular form in Table 8.13. and seasonal variation is depicted in Table 8.14.

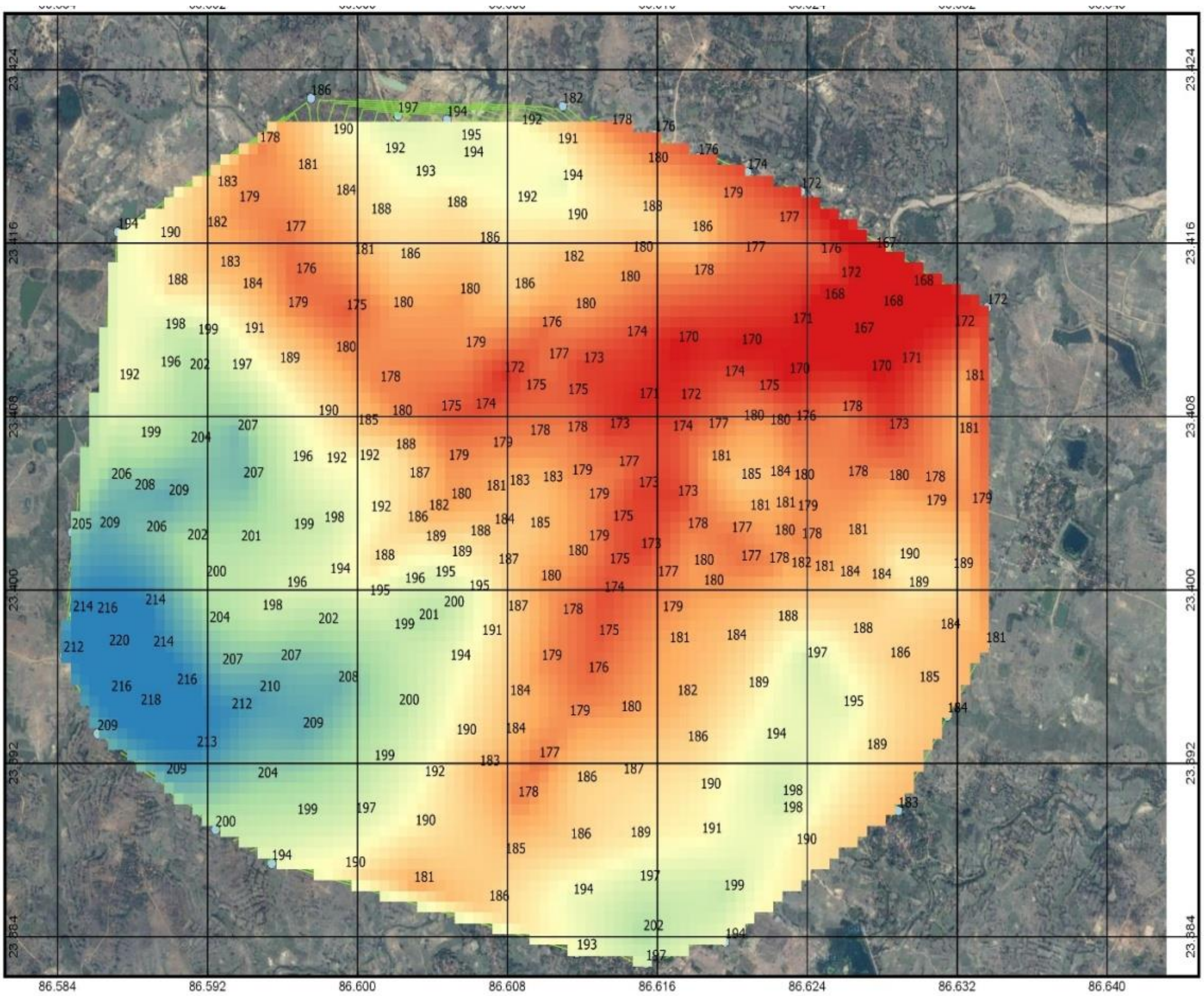


Figure 8.5: DEM of the Catchment area contributing all the station point

Table 8.12 : Discharge, order no (m),$\bar{x}$ at station point20 of Futiari River

area for st. 20 (m²)	Q=m³/month	Q=m³/s	order m	$\bar{x}$	x- $\bar{x}$	(x-$\bar{x}$)²
3138000	314634.185	0.121387	0.205607	0.125521	0.080086	0.0064138
3138000	532934.385	0.205607	0.171409	0.125521	0.045888	0.0021057
3138000	304443.53	0.117455	0.164728	0.125521	0.039207	0.0015372
3138000	373557.98	0.14412	0.16462	0.125521	0.039099	0.0015287
3138000	327366.62	0.126299	0.145876	0.125521	0.020355	0.0004143
3138000	302602.57	0.116745	0.14412	0.125521	0.018599	0.0003459
3138000	305013.6	0.117675	0.138635	0.125521	0.013114	0.000172
3138000	426694.78	0.16462	0.138094	0.125521	0.012573	0.0001581
3138000	426974.585	0.164728	0.134207	0.125521	0.008686	7.545E-05
3138000	248555.75	0.095893	0.132715	0.125521	0.007194	5.175E-05
3138000	320429.025	0.123622	0.129413	0.125521	0.003892	1.515E-05
3138000	290113.33	0.111926	0.126299	0.125521	0.000778	6.053E-07
3138000	292696.95	0.112923	0.125186	0.125521	-0.000335	1.122E-07
3138000	291750.32	0.112558	0.123622	0.125521	-0.001899	3.606E-06
3138000	378110.695	0.145876	0.122945	0.125521	-0.002576	6.636E-06
3138000	297351.65	0.114719	0.121615	0.125521	-0.003906	1.526E-05
3138000	357938.585	0.138094	0.121387	0.125521	-0.004134	1.709E-05
3138000	311035.945	0.119998	0.119998	0.125521	-0.005523	3.05E-05
3138000	273761.735	0.105618	0.117675	0.125521	-0.007846	6.156E-05
3138000	298402.88	0.115125	0.117455	0.125521	-0.008066	6.506E-05
3138000	444291.115	0.171409	0.116745	0.125521	-0.008776	7.702E-05
3138000	359342.84	0.138635	0.115125	0.125521	-0.010396	0.0001081
3138000	218381.265	0.084252	0.114719	0.125521	-0.010802	0.0001167
3138000	343998.02	0.132715	0.112932	0.125521	-0.012589	0.0001585
3138000	347865.605	0.134207	0.112923	0.125521	-0.012598	0.0001587
3138000	288125.93	0.11116	0.111926	0.125521	-0.013595	0.0001848
3138000	318674.36	0.122945	0.112558	0.125521	-0.012963	0.000168
3138000	324482.275	0.125186	0.11116	0.125521	-0.014361	0.0002062
3138000	315225.175	0.121615	0.105618	0.125521	-0.019903	0.0003961
3138000	335439.125	0.129413	0.095893	0.125521	-0.029628	0.0008778
3138000	230648.23	0.088985	0.093752	0.125521	-0.031769	0.0010093
3138000	243004.105	0.093752	0.088985	0.125521	-0.036536	0.0013349
3138000	292720.485	0.112932	0.084252	0.125521	-0.041269	0.0017031

Table 8.13: Estimated Probable Flow Shown in Tabular form

Sl. No.	River	Reach	River station	10 Yrs (m^3/s)	50 Yrs (m^3/s)	100 Yrs (m^3/s)
1.	Futiari	Upper	20	0.16324	0.20	0.215
2.	Futiari	Lower	10.25 (Jun)	0.45	0.89	1.07
3.	Futiari	Lower	6	0.307	0.602	0.73
4.	Darokeswar	Upper	0.4	0.08	0.10	0.102

T

Table 8.14: Seasonal variation in Water depth of Futiari River(approximately)

SL.NO.	SEASON	Water depth before dam construction	Water depth after Construction
1	Monsoon	3.5 m	3.5 m
2	Summer	0.05 m	2.06 m
3	Winter	1.00 m	2.06 m

Entering all the found data and inputs in HEC-RAS and develop a model:

Generating hydraulic model using HEC-RAS with incorporating all the inputs follows some steps, which are described as below:

Entering Geometric Data

The initial step is developing a steady flow model of Futiari and Darokeswar River of Purulia District with the help of HEC-RAS. To enter the geometric data, select the Geometric Data option from the Edit menu on the HEC-RAS main window. Then Geometric data window will appear where draw the river system schematic by following the steps one by one. First click on River Reach button on the geometric data window. Move the mouse pointer over the drawing space by pressing the left mouse button and draw the schematic. By double clicking the left mouse button to end the drawing of the reach. Once the reach is drawn a prompt box will appear where give the reach name and the name of the river say River: Futiari, Reach: Upper. Thus by the same process draw the upper reach of Darokeswar river. And split the meeting point of two rivers by entering a junction point named as "Futi-

Dara”. The downstream portion of the Fuiari River after the junction point will be named as Lower reach.

Entering the cross section data

The next step is to enter the cross-section data. This is accomplished by clicking the cross section button on the Geometric Data Window. Once the button is pressed the cross section data editor will appear. Where it has to select the River and the Reach name first. Then go to options menu of Cross Section Data window there select the Add a new cross section option, and give the river station name of the schematic .After that enter the Description of the river station. Now, give the cross section coordinates i,e Station vs Elevation value ,Downstream Reach Lengths of Left over bank(LOB), Right over bank(ROB) and the channel length. Specify the Main Channel Bank Stations, contraction and Expansion coefficients, and the manning’s n values of the LOB, ROB, and Channel. Values of Manning’s roughness coefficients (“n”) applied in all new flood modelsrequire supporting documentation. Also, any modification of “n” values from published or accepted flood models requires supporting documentation. Many Hydraulic engineering texts include tables of “n” values and, in some cases, photographs showing representative values. Many of the sources listed in the bibliography include discussions of applying “n” values. These values are typically representative for streams and rivers in India. In this problem manning’s value 0.015 was taken for both Futiari and Darokeswar River. River profile is plotted in Figure 8.6.

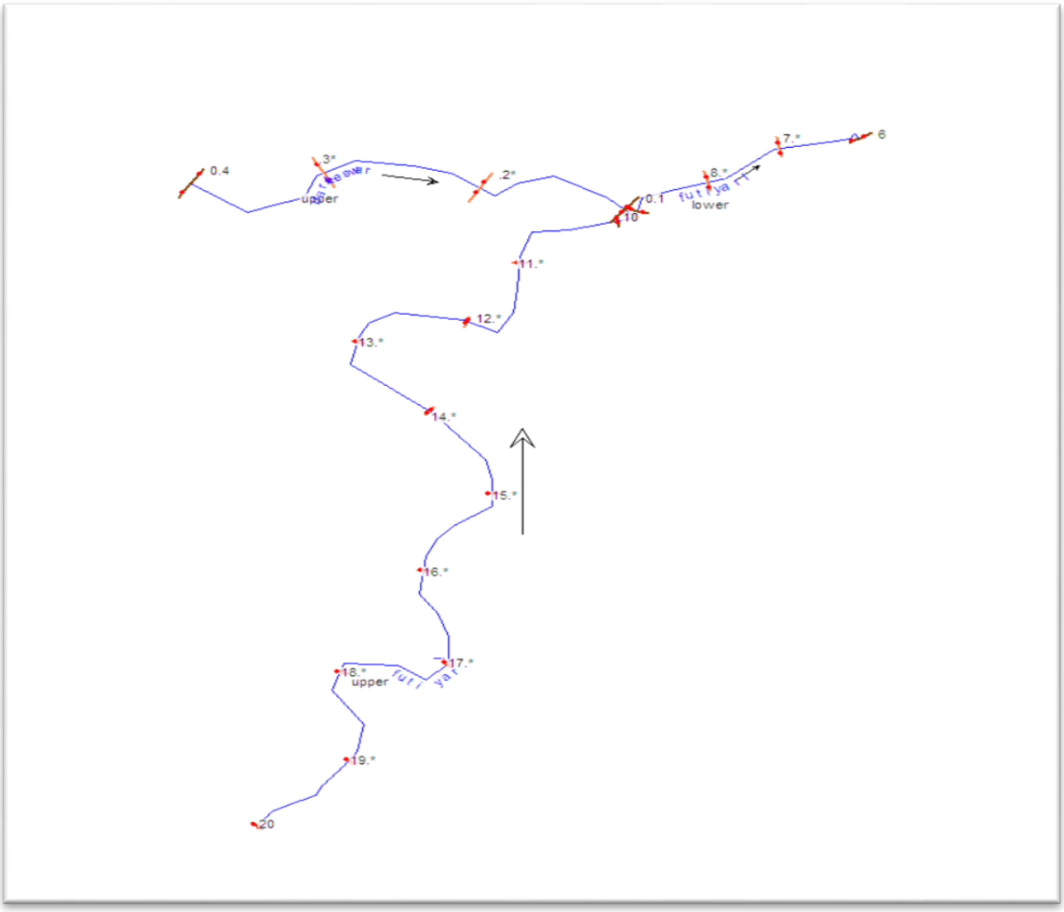


Figure 8.6 : Geometric data of the river system schematic

6

X-S interpolation

The next step is to interpolate the station and elevation value in between the river station. This can be obtained by clicking the Tools button of Geometric Data Window, then go to XS interpolation and then between 2 XS's interpolation. This will give the Cross Section Coordinates of all the stations of the river system schematic of Futiari and Darokeswar River.

Entering junction data

The next step is to enter the junction data. This is accomplished by clicking the junction button on the Geometric Data Window. Then a window will appear as shown below, enter the name of the junction and give the length across junction from Futiyari lower to Futiyari upper and Darokeswar River.

Entering the steady flow data

To perform the water surface profile calculation, First step is to enter the steady flow data by pressing the Steady Flow Data from the Edit menu on the HEC-RAS main window. Here enter all the flow change location and their corresponding flow rates, add the boundary conditions. Finally apply the Data.

Performing the hydraulic calculations

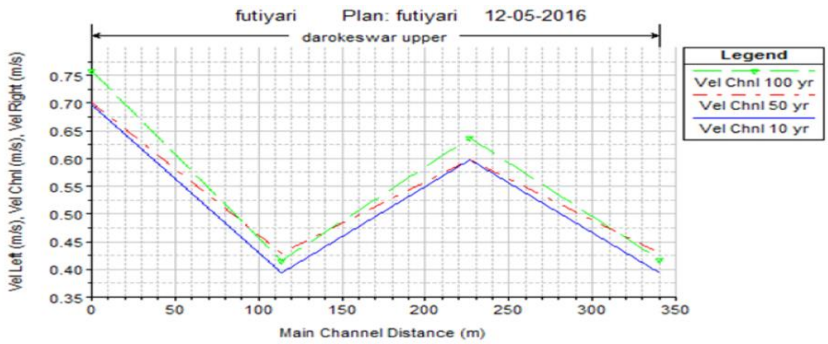
To perform the simulations, go to the HEC-RAS main window and select the Steady Flow analysis from the Run menu.Here put the Plan name which defines the geometry and flow data. Also put a short identifier for the plan. Select the flow regime i.e Subcritical/Supercritical/Mixed flow. Finally go to the Compute button for steady flow simulation.

Viewing graphical and tabular results

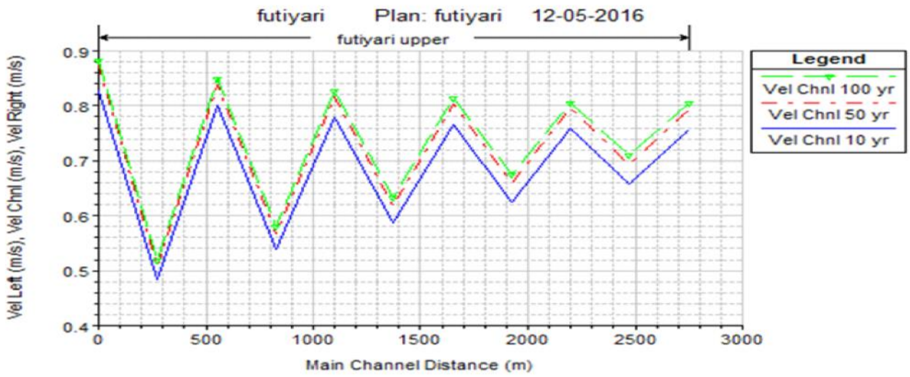
Once the model is developed all the computations successfully, the results can be obtained from the view menu bar on the hec-ras main window.

Result and output

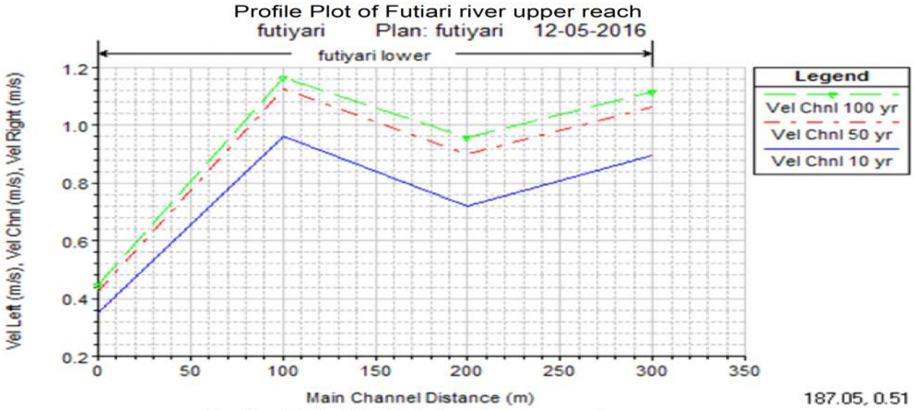
In hydrology, a rating curve is a graph of discharge versus stage for a given point on a stream, usually at gauging stations, where the stream discharge is measured across the stream channel with a flow meter.Numerous measurements of stream discharge are made over a range of stream stages. The rating curve is usually plotted as discharge on x-axis versus stage (surface elevation) on y-axis. The development of a rating curve involves two steps. In the first step the relationship between stage and discharge is established by measuring the stage and corresponding discharge in the river. And in the second part, stage of river is measured and discharge is calculated by using the relationship established in the first part. Stage is measured by reading a gauge installed in the river. If the stage-discharge relationship does not change with time, it is called permanent control. If the relationship does change, it is called shifting control. Shifting control is usually due to erosion or deposition of sediment at the stage measurement site. Bedrock-bottomed parts of rivers or concrete/metal weirs or structures are often, though not always, permanent controls. To determine the rating curve initially profiles of various parameters such as velocity, flow and depth in the longitudinal direction were determined with help of HEC-RAS model and depicted in Figure 8.7 and Figure 8.8. and then rating curve was generated in model shown in Figure 8.8 and Figure 8.9 also predicted for different time scale .



Profile Plot of Darokeswar river upper reach

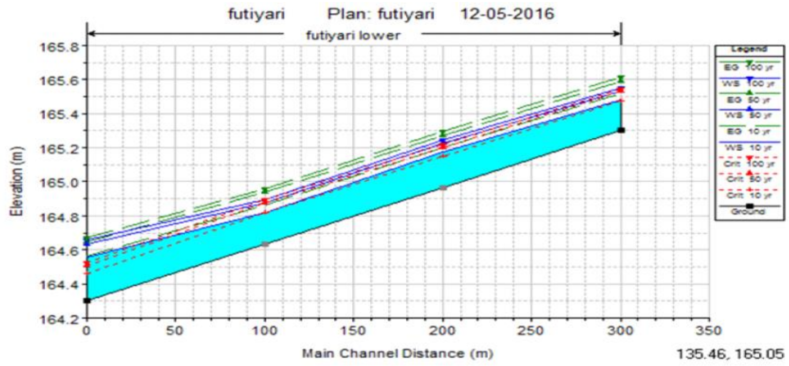


Profile Plot of Futiari river upper reach

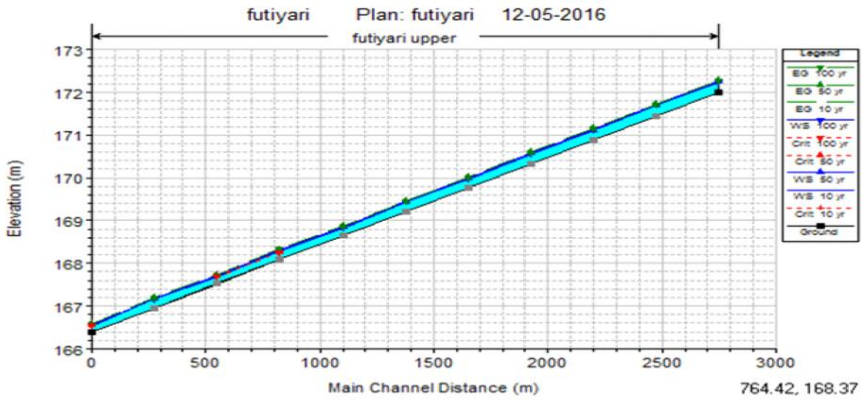


Profile Plot of Futiari river lower reach

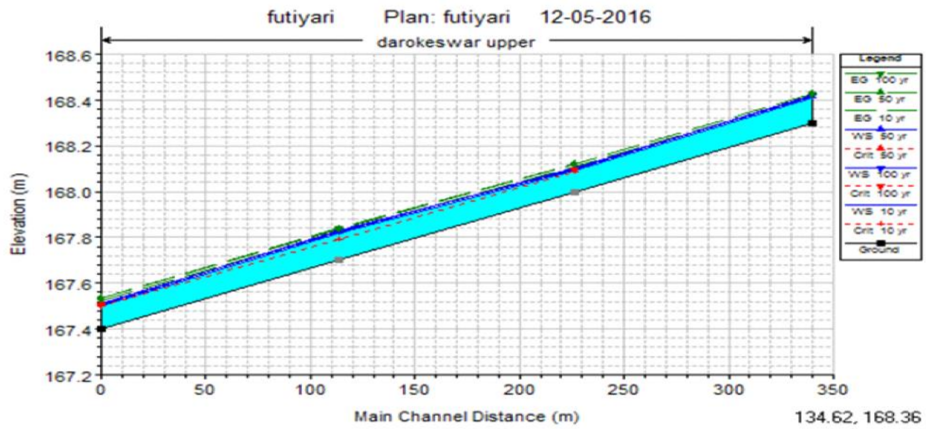
Figure 8.7 River profile plot stage-I of study area



Futiari river lower reach

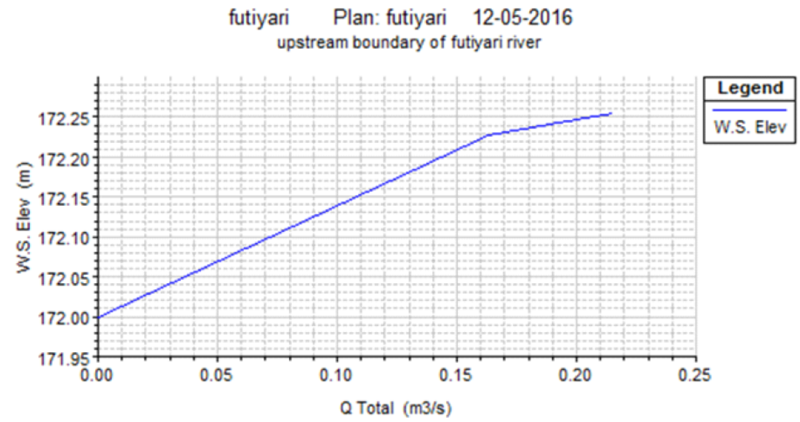


Futiari river upper reach

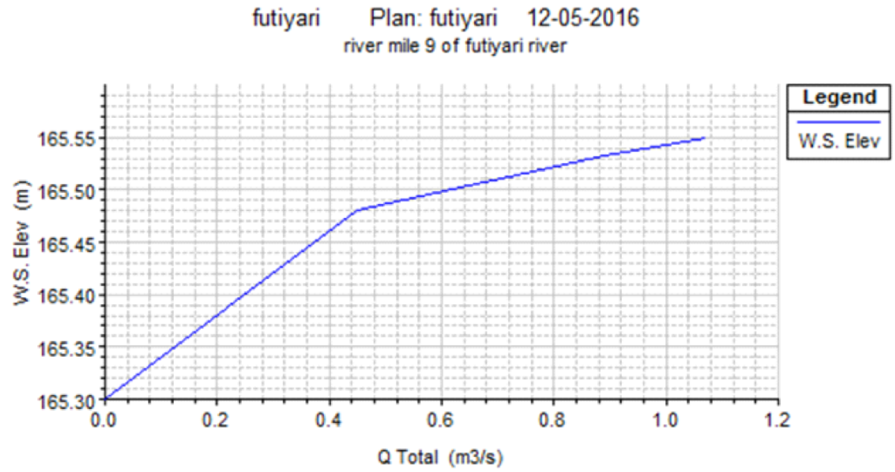


Darokeswar river upper reach

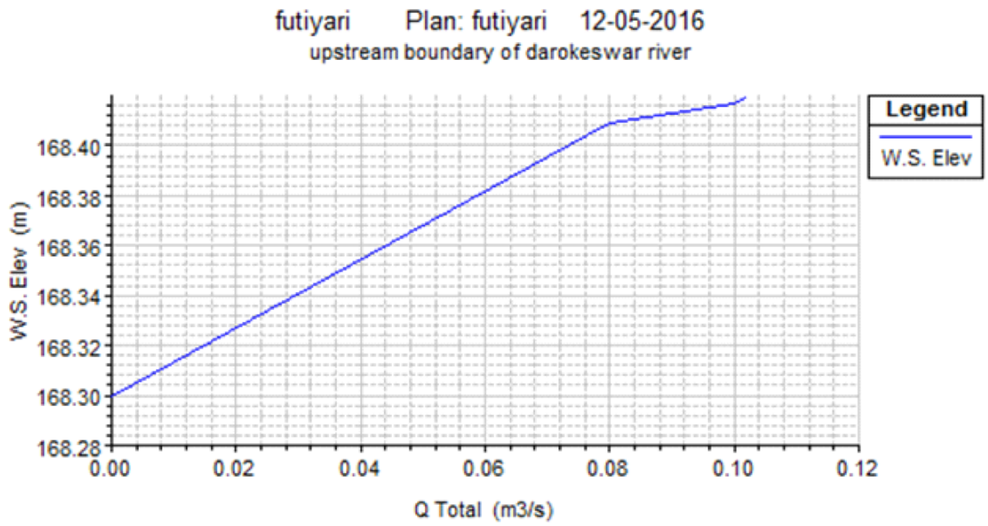
Figure 8.8 Profile plot stage-II study area



Futiaririver lower reach



Futiari river lower reach



Darokeswar river upper reach

Figure 8.7 : Rating Curve of different river stretch

Remarks

From the study it reveals that the application of GIS and HEC RAS model can save an enormous amount of time and money in comparison with traditional methods of hydraulic analysis, mapping flood prediction, and generating a water surface elevation. Steady flow simulation of the river Futiari of Purulia district by entering geometric data such as: the river system schematic, cross section geometry, reach length, Hydraulic structures or obstructions, energy loss data: Manning's n, contraction/ expansion coefficient, flow data i.e flow rate, profile, boundary conditions in software HEC-RAS which calculate the water surface profile from the 1D energy equation, head loss formula. As a result it gives the cross section plot, profile plot, to compare visually the various situations. The model output can also linked with the early alarming system and draught management systems. From the above study it is observed that the river being non-perennial the water availability round the year is insufficient for meeting drinking water requirement of Purulia district. Also the river course is very irregular. The google satellite image shows the river banks. As the district falls in a hilly region from geological point of view where the seepage loss and evaporation losses are very significant. This is essential, as in near future, it will help us make water security plan for sustainable and river basin management. But as the situation of water scarcity is very high in that region, such small construction like check dam considered as very useful. And the HEC-RAS software is also proved very user friendly for such river modelling.

CHAPTER -9

COST EFFECTIVE INTEGRATED MODEL DEVELOPMENT

9. Cost effective integrated model development

9.1. A General over view of cost effective integrated model development

Development of cost effective integrated model depends on hydro economic environment. Sustainable development of hydro economic model includes method, design estimation, material and site specific construction technology. Hydroeconomic models represent regional scale hydrologic, engineering, environmental and economic aspects of water resources systems within a coherent framework. The idea is to operationalize economic concepts by including them at the heart of water resource management models. These models have emerged as a privileged tool for conducting integrated water resources management (IWRM) (Global Water Partnership, 2000; Mariño and Simonovic, 2001; Cardwell et al., 2006). Hydro economic models are solution-oriented tools for discovering new strategies to advance efficiency and transparency in water use. The goal is to look at a system in a fresh way to investigate promising water management schemes and policy insights. Recent hydroeconomic modeling research has been described by McKinney et al. (1999), Jakeman and Letcher (2003), Lund et al. (2006), Heinz et al. (2007), Cai (2008), Pulido-Velazquez et al. (in press), Brouwer and Hofkes (2008) and Ward (2009). Engineers traditionally evaluate costs of building, operating and maintaining water supply, conveyance, storage, sewerage, drainage, and waste-water reuse infrastructure and estimate water requirements. In non-economic system models, water demands are commonly represented by fixed water “requirements” or delivery targets. The profession has often relied on a static view of water demands which can lead to over-design of infrastructure, waste, and slow adaptation to new conditions. In a mature water economy (Randall, 1981) with rapidly rising incremental costs of new supplies (aquifers already heavily exploited, best dam locations taken and other rivers protected) and increased conflicts among water users, a wider view is needed to face water scarcity problems. Economics helps water managers move from a static view of water demand, defined through water rights, priorities and projections of population growth and agricultural and industrial water requirements to a view of demand related to the economic concept of value. Water value changes with the quantity and type of use. Hydroeconomic models represent all major spatially distributed hydrologic and engineering parts of the system. Representations include water balance components such as river flows, evaporation from surface water bodies, natural groundwater recharge and discharge, and return flows. Relevant water supply infrastructure and operations may include canals, reservoirs, groundwater or pipeline pumping stations, artificial recharge basins and other groundwater banking infrastructure. In this current research study integrated model developed and implemented as hydro economic model for sustainable development. Scientific description design, numerical modelling aspect, cost-estimation described thoroughly in this chapter.

9.2. Technology description of cost-effective integrated model

Based upon the topographical and hydro-geological survey aided by GPS, Sokkia Total Station Survey and with the help of some field and historical data, some suitable areas were demarcated along the Futiary River catchment for construction of a composite model comprising a Cement Concrete Check dam followed by a Collector Well or an Infiltration Gallery and a sub-surface Dyke. The infiltration gallery or collecor well will be used to collect the water from the river after disinfection process. The potential recharge from a groundwater recharge structure has been defined as the volume of water retained and percolated from upstream of the hydraulic structure (Scanlon et al., 2002). The potential recharge was estimated using water balance equation considering the pertinent parameters of hydrologic cycle. The schematic diagram of the site instrumentation and recharge process has been depicted in Figure 9.1. The generic form of the water balance equations may be given as follows.

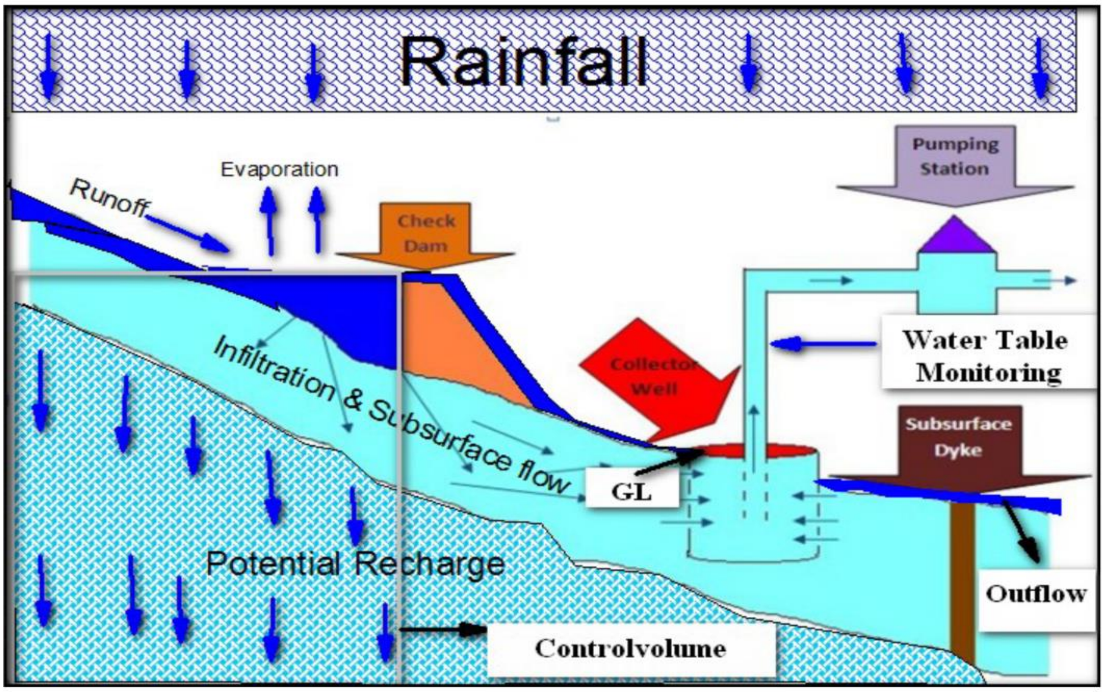


Figure 9.1: Site instrumentation

9.3. Design of water retaining structure

9.3.1. Check dam Design

Cement Concrete Check Dam along is to be done to restrict the quantum of rain water flows mainly in monsoon. In this regard, the Cement concrete check dam is much durable against the flash floods at those regions during the monsoon. The longevity and effectively of such type of check dam is also more than the boulder check dams or earthen check dams. This is recharge cum conservation structure recommended in all type of slope. Here is a detailed design of the check dam depending upon the real field data.

Design of Channel Section:

By using Dicken's formula

Discharge, $Q=CA^{3/4}$ 9.1

As per site condition

1 $mile^2$= 258 ha

Given data, Area, A= 10 ha

A= $(1/258)\times258^2$=0.99 ha

$Q=2400*(0.99)^{3/4}$ = 0.9924ha/s =6 m^3/s

Observed flood discharge

Hydraulic Radius, R= $\frac{24\times2.5}{10+24}$...9.2

$=\frac{120}{34} = 2.07$ m

Velocity, $V=1/n \times R^{2/3} \times S_0^{1/2}$ [Where S_0= Bed Slope on pre available survey data=18/1000m =0.018 and [n=0.04]=$1/.004 \times (2.07)^{2/3} \times (0.018)^{1/2}$ m/s = 7

Q= (24×2.5)× 0.5(Assuming Velocity 0.5 m/s)= 60m^3/s

Fixation of crest of check Dam:

$H^{3/2}$=60/(1.84× 24)=1.36 m

H=1.23 m

H= Head over the crest

Keeping upstream bed level as survey = 96.0 m (Crest level)

Full drainage level =B.L + F.D.D=(96+2.5) = 98.5 m

Velocity head, $h_a=(v^2/2g)$=0.05 m (say)

U/s TEL=(98.5+0.05)=98.55 m

Top level of the weir crest= U/s TEL – H=(99.23-96.0)=98.23 m

U/S bed level= 96.0 m

Height of the weir= (98.23-96.0)=2.32 m

A provision of 0.50 m assumed

D/S TEL =(98.55-0.5)=98.05 m

D/S F.D.L.=(98.05-0.5)=97.55 m

Calculation of scour depth:

q=Q/b =60/24=2.5/m^3/s/m

Lacy's Scour depth, R=1.35 $(q^2/f)^{1/3}$ = 1.35 × $(2.5^2/1)^{1/3}$= 2.49 m

Assuming factor of safety 1.5

Maximum scour depth= (1.5× 2.49)=3.74m (say)

R.L. of d/s deepest scour = (97.55-3.74)=93.81 m

D/S depth of cut-off = (96.0-93.81)= 2.19m

RL of U/S deepest Scour = (98.50-3.74)=94.76 m

U/S depth of cut-off = (96.0-94.76)=1.24 m

Provide 1.25 depth of upstream cut-off

Cistern level and D/S Cistern length:

We have, Q=60 m^3/s

Q $=\frac{60}{24}$ =2.5 $m^3/s/m$

H_L = 0.50 m(assumed)

Critical Depth, $d_c = (\frac{q^2}{g})^{1/3} = (\frac{2.5^2}{9.81})^{1/3}$ =0.86 m

Pre-jump hyper critical depth,

$d_1 = 0.183 \times q^{0.89} \times (H_L)^{-0.35} = 0.183 \times 2.5^{0.89} \times (0.50)^{-0.35} = 0.53$ m

Pre-jump hyper critical depth,

$d_2 = 0.98 \times q^{0.52} \times (H_L)^{0.2} = 0.98 \times 2.5^{0.52} \times (0.50)^{0.2} = 1.37$ m

Pre-Jump Velocity,

$v_1 = \frac{q}{d_1} = \frac{2.5}{0.53} = 4.72$ m/s

Post-Jump Velocity,

$V_2 = \frac{q}{d_2} = \frac{2.5}{1.37} = 1.82$ m/s

$E_2 = d_2 + \frac{v_2^2}{2g} = 1.37 + \frac{1.82^2}{2\times9.81} = 1.54$ m

$E_1 = E_2 + H_L$ = 2.04 m

D/S cistern level = d/s TEL-1.25E_2 = (98.05-1.25×1.54) = 96.125 m

D/S cistern level provided at R.L 95.60

Length of cistern = 5× E_2 = 5× 1.54 = 7.7 m

Cistern length provided as 8.0 m

Floor length in consideration of Exit gradient:

Maximum static head (H) = Top level of weir crest – D/S cistern level

= (98.23-95.60) = 2.63 m

D/S cut-off depth, d = 2.19 m

Assuming a exit gradient = 1/5 (For course sand to fine sand)

We have, $G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$

$\frac{1}{\pi\sqrt{\lambda}} = \frac{d}{H} \times \frac{1}{6} = \frac{2.19}{2.63} \times \frac{1}{5} = 0.167$

Value of α from plate VII/5 of C.B.I.P-12 comes 5

$\alpha = \frac{b}{d}$

$b = \alpha \times d = 5 \times 2.19 = 10.95$

Distribution of Floor Length and calculation of uplift pressure:

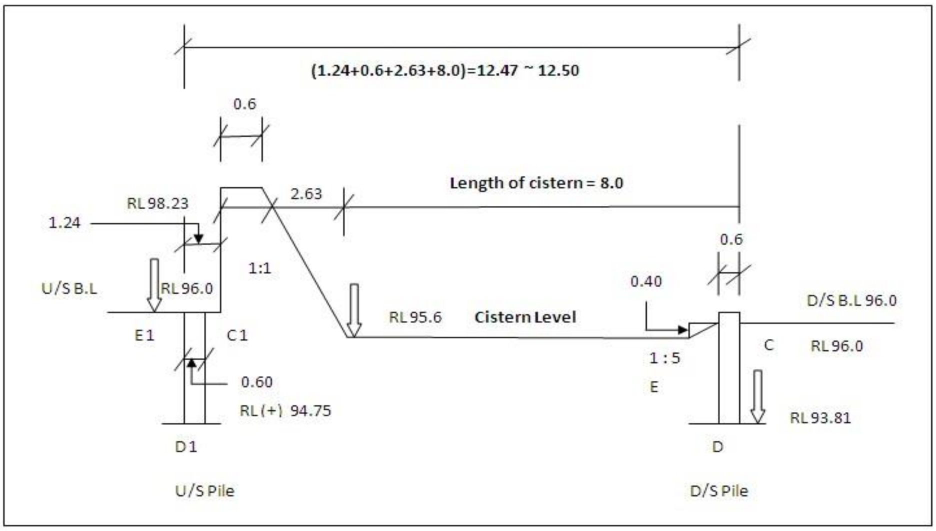


Figure 9.2: Distribution of Floor Length and calculation of uplift pressure:

Uplift Pressure:

We have, length of floor, b = 13.0 m

Depth of d/s cut off, d = 2.19 m

$\alpha = \frac{b}{d} \therefore \frac{1}{\alpha} = \frac{d}{b} = \frac{2.19}{13} = 0.168$

Correction at E:

(a)Due to mutual interference of pile at d/s

$$C = 19\sqrt{\frac{D}{b'}} \times \frac{d+D}{b} = 19\sqrt{\frac{1.25}{12.5}} \times \frac{1.25+2.19}{12.5} = (-)\ 1.65\ \%$$

(b) Due to thickness of the floor (Assume 0.60 m. thick floors)

Correction = $\frac{54-29}{96-93.81} \times 0.6$ = (-) 6.85 %

ΦE_1 = 100%, ΦD = 29%, ΦE = 54%

ΦD_1 = 100 - ΦD = 100-29 = 71%

ΦC_1 = 100 – ΦE = 100-54 = 46%

Correction at C_1:

(a)Due to mutual interference of pile at d/s

$$C = 19\sqrt{\frac{D}{b'}} \times \frac{d+D}{b} = 19\sqrt{\frac{2.19}{12.5}} \times \frac{1.25+2.19}{12.5} = (+)\ 2.19\ \%$$

Corrected pressure of E= 54-1.65-6.85 = 45.5 ≈ 46 %(Say)

Where, C = Correction as percentage of head.

D = D/S pile whose effect to be determined on U/S pile = 2.19 m

d = U/S pipe line =1.25 m

b'= distance between two piles = 12.5 m (Say)

b = total length of floor =12.5 m

(b) Due to thickness of the floor (Assume 0.60 m. thick floors)

Correction = $\frac{71-46}{96-94.75} \times 0.6$ = (+) 12%

Corrected pressure of C_1 =46+2.19+12 = 60.19 ≈ 61 %(Say)

Table 9.1 : Correction for slope has not been considered

Flow Condition	U/S Water Level	D/S Water Level	Head Of Water (m)	Height/Elevation of Sub-Soil H.G line above datum					
				U/s Pipe Line			D/S Pipe Line		
				ΦE_1 100%	ΦD_1 79%	ΦC_1 77%	ΦE 23%	ΦD 21%	ΦC 0
U/S Water Level at Pond Level(No Flow)	98.23	96.0	2.23	2.23 98.23	1.76 97.76	1.72 97.72	0.51 96.51	0.47 96.47	0 96.0
Flow at Maximum U/S W.L	98.50	97.55	0.95	0.95 98.50	0.75 98.30	0.73 98.28	0.22 97.77	0.20 97.75	0 97.55

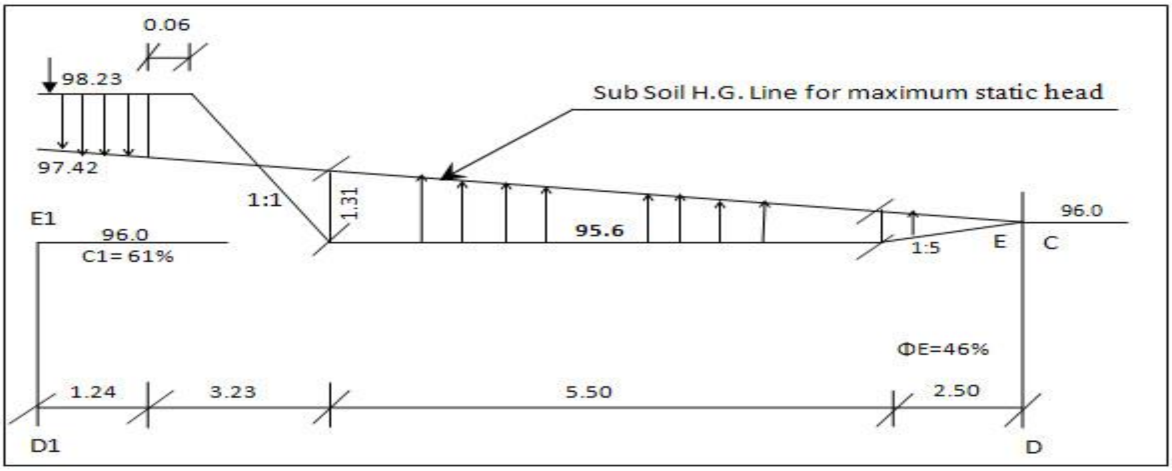


Figure 9.3 : No Flow Condition

For No Flow Condition:

(a) Level of H.G linefor static head of 2.63 m at the toe of D/S glacis

$= 96+(\frac{97.42-96}{12.5}) \times 8.0$

= 96.91 m

Unbalanced head at the toe of the glacis due to static head = (98.36-97.27) = 1.09 m

(b) Level of H.G line for static head of 2.63 m at 2.5 m from d/s

$= 96.0+(\frac{97.42-96}{12.5}) \times 2.5 = 96.28$ m

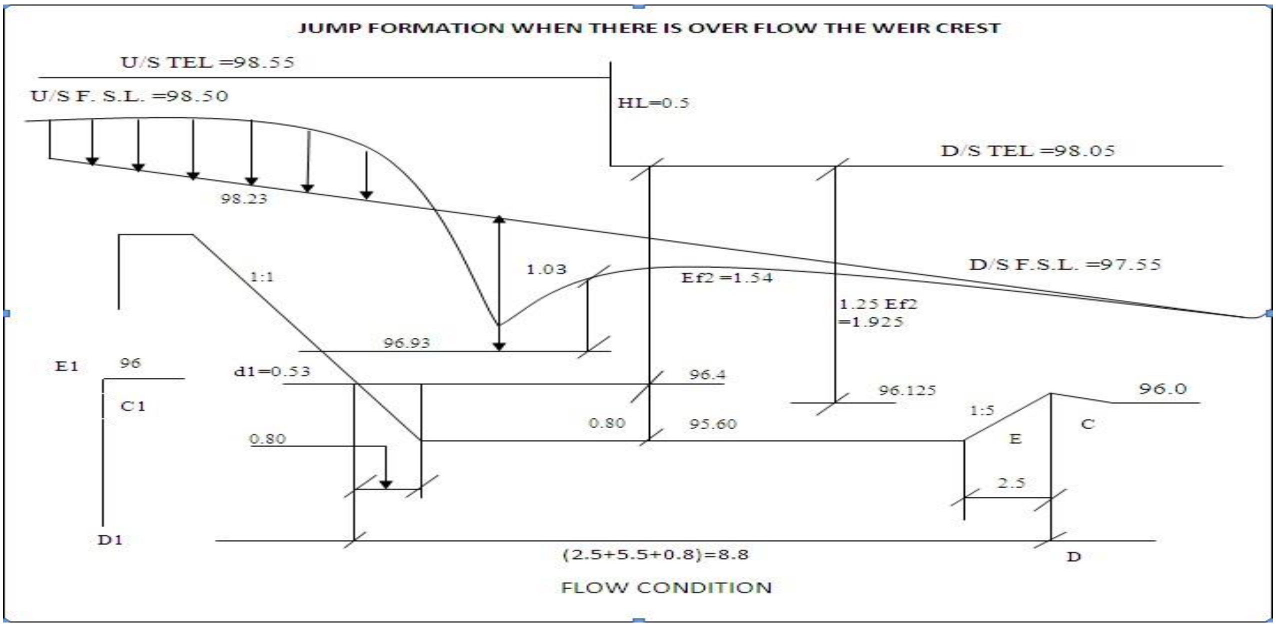


Figure 9.4: Jump Formation when there is a over flow

Unbalanced head = (96.26-95.60) = 0.66 m

Critical Depth, d_c = 0.86 m

Super critical Depth, d_1 = 0.53 m

Super critical Depth, d_2 = 1.37 m

E_2 =1.54 m

R.L of H.G line at the point of jump(Flow Condition)

$= 97.55+ (\frac{98.31-97.55}{12.5}) \times 8.8 = 98.09$ m

Maximum ordinate of unbalanced head at the point of jump

= 98.09-(95.6+0.80+0.66) = 1.03 m

2/3 of this ordinate

$= \frac{2}{3} \times 1.03 = 0.69$ m which is less than the unbalanced head of 1.31 m due to static condition.

Thus floor thickness at point of jump must not be less than (1.31/1.24) = 1.06 m≈ 1.10 m

Provided thickness of floor as 1.1 m

Level of H.G line at 2.0 m beyond the toe(due to static head)

$= 96 + \left(\frac{97.42 - 96}{12.5}\right) \times 6.0$

= 96.68 m

Unbalanced Head = (96.68- 95.6) = 1.08 m

Thickness required = (1.08/1.24) =0.87 m

Provided thickness of 1.0 m

In working out the floor thickness some amount of margin of safety is kept over the calculated thickness for the following reasons:

(a) Pressure P.C given by C.B.I.P-12 are accurate within a margin of $\pm 2\%$
(b) Due to limitations in manufacture of concrete the unit wt. of concrete with sand, cement and stone aggregate may vary.
(c) Floor thickness provided at different sections has been shown in drawing.

Design of Protection works: Protection works are required on the upstream as well as downstream in order to obviate the possibility of scour hole travelling close to the pucca floor of the check dam and to relieve any residual uplift pressure through the filter.

<u>D/S Block Protection</u>

Length of protection = 2×D = 2×2.19 = 4.38 m≈ 4.5 m

Provide C.C (1:3:6) block of size (0.45 m×0.45 m ×0.15 m) with 7.50 m gap filled with jherries/bajri and blocks laid over. 45 cm Filter (bottom 15 cm coarse sand, middle 15 cm layer made up of 20 mm down stone chips and top 15 cm layer made up of 37.50 mm to 20 mm down stone metal and chips.)

N.B. – Same block protection would be provided on U/S protection also.

Return Wall/Wing Wall:

Vertical cut-off both at U/S and D/S end of impervious floor has been extended into the bank for a length of 2.00 m and the return wall has also extended for this length.

Design of abutment, wing wall and return wall:

The pressure of the back fill has been considered and submerged. The pressure of this submerged soil shall be considered as made up of two components:

a) Active pressure due to earth in accordance with standard formula, the unit weight of earth being reduced by buoyancy.
b) Active pressure due to full hydrostatic pressure of water (weep holes though provided shall be considered as inoperative) shown in Figure 9.5. and load distribution given in
c) Table **9.2**.

In calculating active earth pressure for submerged back fill the following data has been taken into consideration.

1) Weight of plain concrete(1:3:6) = 230 kgf/m^3
2) Weight of wet or saturated compact earth = 2100 kg/m^3
3) Angel of internal fraction of soil, Φ= 30°
4) Weight of fresh water = 1000 kg/m^3
5) Ka = co-efficient of active earth pressure $(\frac{1-Sm\emptyset}{1+Sm\emptyset}) = \frac{1}{3}$

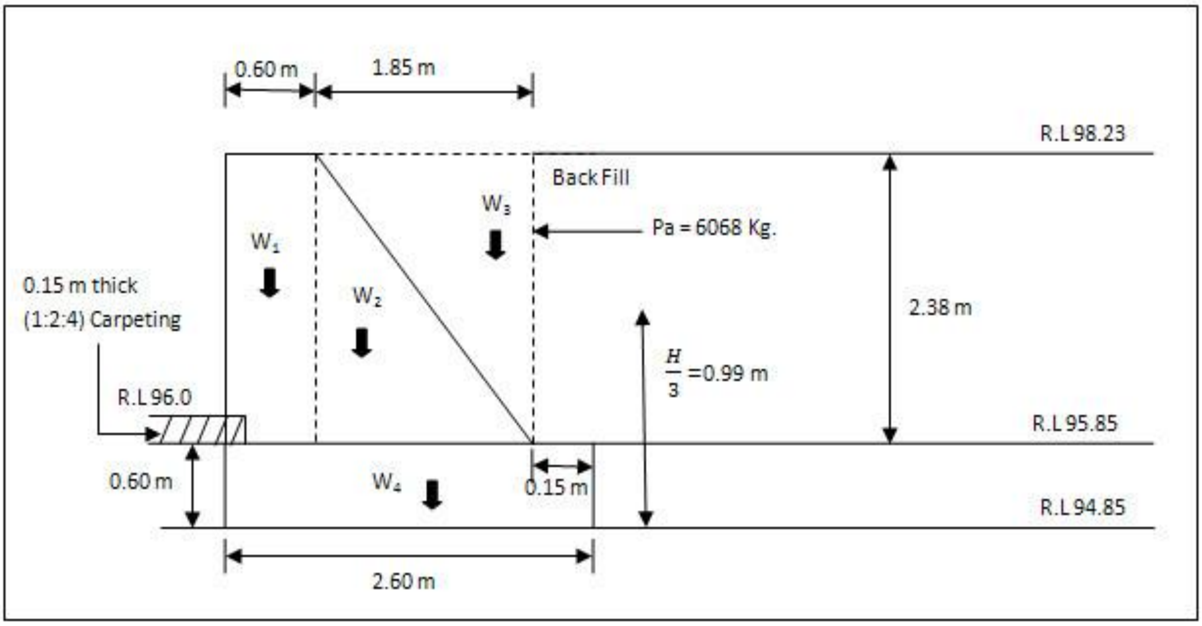


Figure 9.5 : Load distribution in Wing Wall

Table 9.2 : Load distribution in Wing Wall

Sl No.	Dimension	Load (Kg.)	Distance of C.G from the end of toe (m)	Moment about Toe(Kg-m)
W_1	$1 \times 0.60 \times 2.38 \times 2300$	3284.40	0.30	985.32
W_2	$1 \times 0.5 \times 1.85 \times 2.38 \times 2300$	5063.45	1.22	6177.41
W_3	$1 \times 0.5 \times 1.85 \times 2.38 \times 2100$	4623.15	1.84	8506.60
W_4	$1 \times 2.6 \times 0.6 \times 2300$	3588.00	1.30	4664.40
		$\sum W$ = 16559		$\sum M$=20334

Moment of horizontal force about toe = $\frac{6068 \times 2.98}{3}$ = 6028 kg-m

Distance of resultant from toe, $\bar{X}$ = $\frac{20334-6028}{16559}$ = 0.86 m

Eccentricity = $\frac{2.6}{2}$ - 0.86 = 0.44 m

Section Modulus, Z= $\frac{bd^2}{6}$ = $\frac{1 \times 2.6 \times 2.6}{6}$ = 1.13 m^3

Moment,($\sum W \times e$) = 16559×0.86 = 14241 kg-m

Bending Stress, $f_b = (\pm) \frac{M}{Z} = \frac{14241}{1.13} = (\pm)$ 12603 kg/m^2

Direct stress, $f_d = (+) \frac{\Sigma W}{A} = \frac{16559}{2.6 \times 1} = (+)$ 6369 kg/m^2

Maximum stress, $f = f_b + f_d$ = 12603+6369 =18972 Kg/m^2 compressive

Hence O.K

Minimum stress, $f = f_b - f_d$ = 12603-6369 =6234 Kg/m^2 compressive

Hence O.K

Check for sliding:

Frictional resistance $= \mu \times \sum W = 0.55 \times 16559 = 9108$ kg

Horizontal force Pa =6068 Kg.

Factor of safety = $\frac{9108}{6068}$ = 1.501 > 1.50Hence O.K

Check for overturning:

Stability moment about the Toe = 20334 Kg-m

Over turning moment about the toe = 6028 Kg-m

Factor of safety for overturning = $\frac{20334}{6028}$ = 3.37 > 2 Hence O.K

N.B – Same sections of wing wall and return wall will be provided at U/S and D/S of weir in continuation of abutment with top R.L same as that of abutment.

Stability of Check Dam at U/S Floor level:

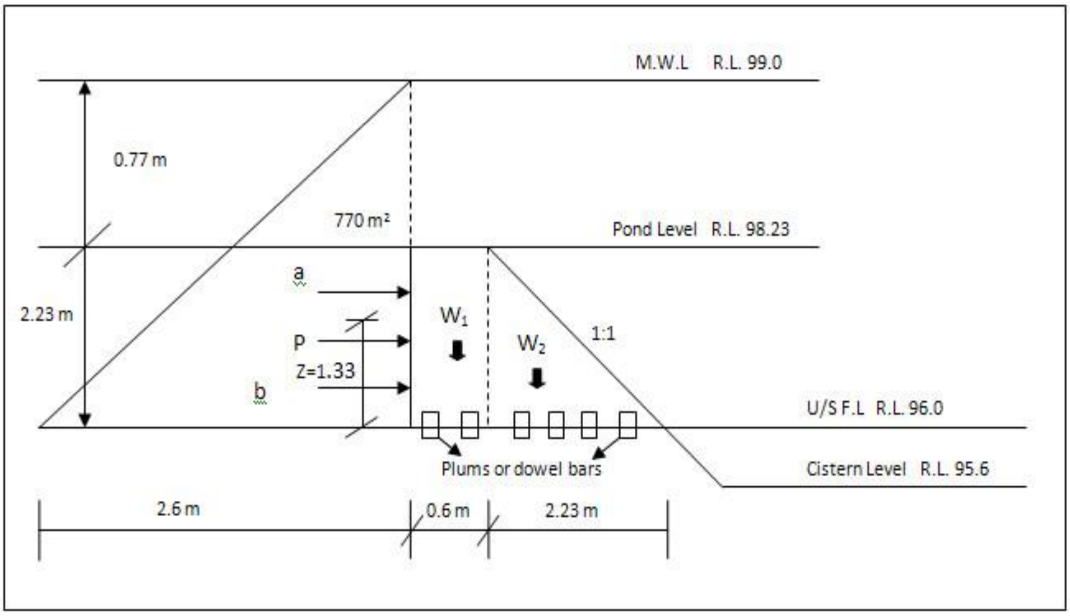


Figure 9.6: Stability of Check Dam

The horizontal Force, P = $^1/_2$(770+2600) × 2.23 =3757.5 Kg.The distance of point of application of horizontal force from the base is given by shown in Figure 9.6 and moment distribution given in Table 9.3:

$Z = \frac{H}{3} \times \frac{2a+b}{a+b} = \frac{2.23}{3} \times \frac{2\times770+2600}{770+2600} = 0.91$ m

Moment of horizontal force about toe = 3757.5×0.91 = 3419.3 kg-m

Table 9.3 : Moment of horizontal force about toe

Sl. No.	Dimension	Load (Kg.)	Distance C.G from toe (m)	Moment about toe (Kg-m)
W_1	1 × 0.60 × 2.23 × 2300	2303	2.53	5826.5
W_2	1 × 0.5 × 2.23 × 2.23 × 2300	5719	1.49	8502.2
		∑W =8022		∑M=14328.7

Distance of resultant from toe, $\bar{X} = \frac{14328.7-3419.3}{8022} = 1.36$ m

Eccentricity, $e = \frac{2.83}{2} - 1.36 = 0.055$m

Section Modulus, $Z = \frac{bd^2}{6} = \frac{1\times2.83\times2.83}{6} = 1.33\ m^3$

Moment = ∑W×e = 8022×0.055 = 441.2 Kg.

Bending Stress, $f_b = (\pm)\ \frac{M}{Z} = \frac{441.2}{1.33} = (\pm)\ 331.7\ kg/m^2$

Direct stress, $f_d = (+)\ \frac{\sum W}{A} = \frac{8022}{2.83\times1} = (+)\ 2834.6\ kg/m^2$

Maximum stress, $f = f_b + f_d$ = 2834.6+331.7 =3166.3 Kg/m^2 compressiveHence O.K

Minimum stress, $f = f_b - f_d$ = 2834.6-331.7 = 2502.9 Kg/m^2 compressive Hence O.K

Check for Sliding:

Frictional resistance = μ∑W = 0.70×8022 = 5615.4 Kg.

Horizontal force = 3757.5 Kg.

Factor of Safety (F.S) = 5615.4/3757.5 = 1.49 < 1.5

Since the weir body will cast monolithically with U/S floor and cut-off, factor of safety will be much more than 1.5, more over some plums or dowel bars would be inserted at the junction during casting of weir body.

Check for over turning:

Stabilizing moment about the toe = 14328.7 kg-m

Over turning moment about the toe = 3419.3 Kg-m

Factor of Safety (F.S)= 14328.7/3419.3 = 4.1 > 1.5 Hence O.K

Detail drawing of check Dam shown in Figure 9.7. Plan and elevation both shown in the Figure 9.7.

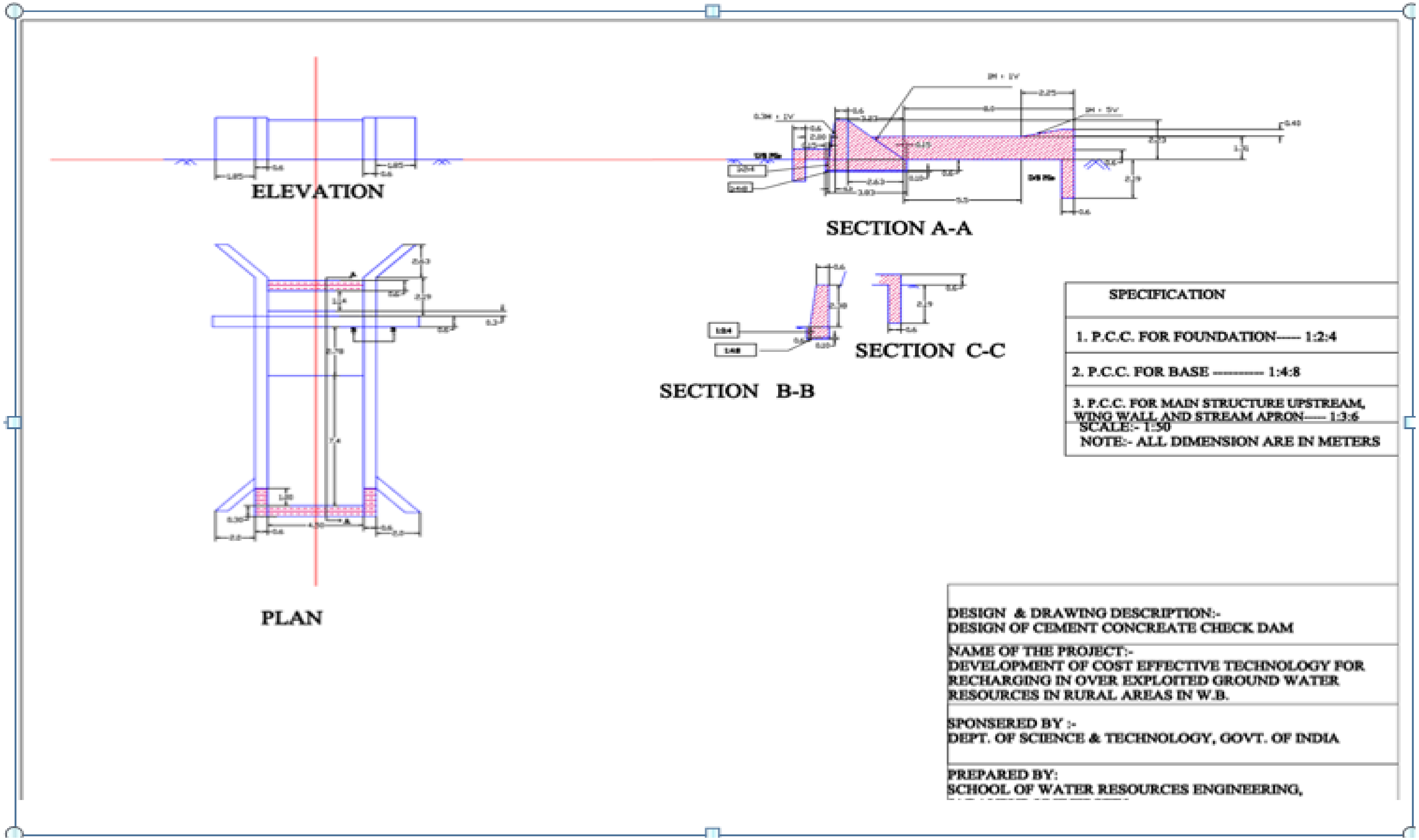


Figure 9.7 : Detail drawing of Check dam

9.3.2. Design of Infiltration Gallery:

An Infiltration Gallery is a sub-surface groundwater collection system, typically shallow in depth, constructed with open-jointed or perforated pipes that discharge collected water into a watertight chamber from which the water is pumped to treatment facilities and into the distribution system. The Infiltration Gallery is usually located close to streams or ponds.

In this study the Infiltration Gallery is not a rainwater harvesting/artificial recharging structure. Rather it is a water collecting structure. The Infiltration Gallery in this study collects the water from the river after natural river bank filtration and this way may fulfill the water demand of the area.

Let us consider that the maximum permissible drawdown at the Futiary River bed be 2 m after a pumping of 12 hrs/day i.e 183 days/year. The Transmissivity and Storage co-efficients at that point are 919.2 m^2/day and 0.0002 respectively.The distance between the drain and the oservation point is 3 m.

The drawdown pattern is given by Ferris (1950) as,

$$s = \frac{Qx}{2T} D(u)$$

$$u^2 = \frac{x^2 S}{4Tt}$$

Where x=distance from drain to the point of observation of drawdown and D(u)= Drain function, values of D(u) for values of u^2 are given in Table 8.1

Now for x=3m, $u^2 = \frac{3^2(0.0002)}{4\times 919.2\times 183} = 2.67\times 10^{-9}$

For u^2= 2.67x10^{-9}, D(u)=2291(approx) where D(u) = Drain Function

For an allowable drawdown of 2 m,

$$2 = \frac{Q \times 3}{2(919.2)} \times 2291$$

Q = 0.53 m^3/day/m of gallery

Now for a length of 50 m of gallery, the maximum supply that can be obtained from the infiltration gallery,

$= 50 \times 0.53 m^3/day$=26.75m^3/day

To increase the capacity of Infiltration Gallery more than one Gallery can be constructed in parallel. Values of D(u) and u^2 for drain function shown in Table 9.4

Table 9.4 : Values of D(u) and u^2 for drain function

u^2*	D(u)*	u^2‡	D(u) ‡
0.0025	10.32	1×10^{-7}	1783
0.0036	8.468	2×10^{-7}	1261
0.0049	7.109	4×10^{-7}	891.1
0.0064	6.13	7×10^{-7}	673.2
0.0081	5.331	1×10^{-6}	563.1
0.010	4.714	2×10^{-6}	398.0
0.013	4.008	4×10^{-6}	281.1
0.016	3.532	7×10^{-6}	212.1
0.020	3.079	1×10^{-5}	177.4
0.025	2.657	2×10^{-5}	125.2
0.030	2.354	4×10^{-5}	88.2
0.035	2.109	7×10^{-5}	66.42
0.040	1.943	1×10^{-4}	55.42
0.050	1.658	2×10^{-4}	38.92
0.060	1.441	4×10^{-4}	27.2
0.070	1.282	7×10^{-4}	20.36
0.090	1.049	1×10^{-3}	16.9
0.110	0.881	2×10^{-3}	11.67
0.130	0.7598	4×10^{-3}	7.99
0.160	0.6284	7×10^{-3}	5.84
0.190	0.5324	1×10^{-2}	4.698
0.230	0.4384		
0.280	0.3517		
0.330	0.2895		
0.380	0.2434		
0.440	0.2008		
0.500	0.1837		
0.580	0.1345		
0.660	0.1094		
0.760	0.0864		
0.900	0.623		
1.000	0.0507		

*(Ferris J.G,1950)
‡(Shanmugam C.T., 1968)

9.3.3. Design of Subsurface Dam

Subsurface dams are groundwater dams, which are structures that intercept or obstruct the natural flow of groundwater and provide storage for water underground. They have been used in several parts of the world, notably India, Africa and Brazil. They are used in areas where flows of groundwater vary considerably during the course of the year, from very high flows following rain to negligible flows during the dry season. A subsurface dam intercepts or obstructs the flow of an aquifer and reduces the variation of the level of the groundwater table upstream of the dam. It is built entirely under the ground. In this study, a sub-surface dam is to be designed after the Check dam to keep the river bed after the check dam saturated. During the monsoon the excess water will spill over the check dam. But during dry season there will be no flow or very less flow downstream of the check dam. The sub-surface dam will intercept the natural flow during the monsoon and during the non monsoon period the intercepted water will keep the river bed saturated. According to the field conditions, a sub-surface dam is designed. The design diagram of the sub-surface dam is given below. The dam will be constructed using Plain cement Concrete (P.C.C) shown in Figure 9.8.

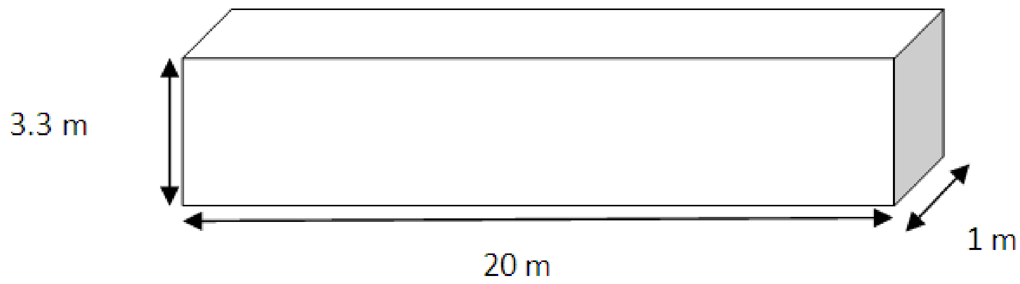


Figure 9.8: Design diagram of Sub-surface Dyke

9.4. Cost-Estimation of the Designed Hydraulic Structures:

Cost –Estimation of individual structure of integrated model has been done with help of PWD- Shedule of rate Govt. of West Bengal and detail cost estimations of check dam, collector well, subsurface-dam are given in

Table **9.5**,Table 9.6 and Table 9.7 correspondingly.

Table 9.5 : Cost- estimation of Check dam

S. No.	**Description of Items**	**Qnty.**	**Unit**	**Rate**	**Amount**
1.a	E/W in excavation in foundation including mixed soil but excluding laterite and sand stone including removing	246.20	m^2	53.01	11,799.99
1.b	In Sandy & Slurry Soil	375	m^2	61.00	22,899.99
2	Stone Boulder soling work 200mm thick on foundation wall and bed	792	m^2	86.40	68,428.80
3	Cement Concrete with graded stone chip, sand and cement. (6:3:1)	180	m^2	3618.00	6,51,240.00
4	Rubble Masonry for weir structure with sand, cement (4:1) including coast of Stone boulder (225mm)	51	m^2	1557.00	79,407.00
5	Rubble Masonry for weir structure with cement mortar (6:1) including cost of stone boulder (225mm)	400	m^2	1332.00	5,32,800.00
6	M-20 Cement Concrete (4:2:1) including cost of all materials, tools & plants (with concrete mixture machine)	10.40	m^2	4214.21	43,827.79
7	Supply of M.S. round, cutting, bending in proper position including cost of binding wire complete	818	Kg	53.00	43,354.00
8	Providing form work and staging including hire and labour charges for the same as required for concrete work of depth not exceeding 500mm or concrete works in vertical faces, to give good concrete finish including stripping off and removing after construction of work as per specification	214	m^2	84.00	17,976.00
9	Supply and laying of 30 cm dia, 2 nos. (Np3)	5	Mtr	1200.00	6,000.00
10	hume pipe for silt clearance of bottom of the weir with provision of Gate	339	m^2	31.71	10,749.69
11	Labour for earth work in embankment in the back filling of both wing with moorum and mixed soil with a lead of 25 m and left of 1.5 mtr.	600	m^3	48.90	29,340.00
12	Labour for line dressing and chilchalling work including breaking clods complete	325	m^2	5.00	1,625.00
13	Supplying and laying Stone Boulder pitching including hand packing and filling the interstices with smaller variety of same all complete 20 cm thick	200	m^2	150.52	30,105.00
14	Supply and fixing of M-5 sate of size 1'9" x 1'9" for salt clearance	2 Nos.	Each	15,000.00	30,000.00
					17,92,960.47

Table 9.6 : Cost-estimation of collector well

S. No.	Description of Items	Qnty.	Unit	Rate	Amount
1.a	Up to 3 meter below G.L. mtr.	3	Mtr	1,452.80	4,358.40
1.b	Beyond 3 meter up to 6 meter	3	Mtr	1,548.80	4,646.4O
2	lst class Brick works for steining wall of well in cement mortar 1:4 below G.L. including cutting bricks if required				
a)	Up to 6 meter below G.L.	15	m^3	5,416.50	81,247.50
b)	Above.6 meter to 10 meter above G.L.	11	m^3	5,266.60	57,932.60
3	Cement Plaster (1:4) to internal Brick wall				
a)	20mm thick internal area of Brick work	94	m^2	192.70	18,113.80
b)	15 mm thick external area above the G.L (1:6) Cement Mortar	44	m^2	166.70	7,334.80
4.a	1:2:4 Cement Concrete (R.C.C.) for cover slab (100 mm thick) on top of well exclude reinforcement, Complete	0.31	m^3	549.80	1,702.76
4.b	-- Do-- Reinforcement cutting, bending & fixing in proper position complete	26	Kgs	53.00	1,378.00
5	Excavation on river bed up to required depth as per drawing, or direction of site in charge for filter bed, including dewatering, Shoring, if necessary Complete a) Up to 2 meter depth below river bed level by mechanical means using excavation of suitable capacity.	36	m^3	69.40	2498.40
6	Supplying stone gravels of approved size (25 mm to 20 mm down), including carriage cost, and spreading the same in proper position by keeping properly line and level complete as per direction of site in charge.	18	m^3	1412.40	25, 423.20
7	Supply and laying of fibre glass strainer pipe in proper position and inserted one end to the newly constructed well as per direction of site in charge. a)200 mm dia and 07 mm thick	36	RMT	4700.00	1,69,200.00
8	Supplying of M.S. angle of 25mm×25mm×3 mm and fabricating of steel frame of approved size as per the direction of site in charge and fixing the proper position on filter bed complete. Supplying, fitting , fixing core mesh 6	92.4	Kg	70.00	6,468.00
9	mm ×6mm or approved size by site in charge to cover all site of steel frame of 18 m×1m×1m to protect from splitting out of filter bed complete Back filling of excavated trenches by river sand obtained from excavated	54	m^2	359.31	19403.01
10	Position properly and as per direction of the site in charge. Complete.	18	m^3	39.00	702.000
					4,00382.00

Table 9.7 : Cost-estimation of subsurface dyke

S. No.	Description of items	Qnty.	Unit	Rate	Amount
1	Excavation of foundation on river bed, including dewatering, shoring, cutting sand as per required depth for foundation complete (in ordinary rock)	99	m^3	137.80	13,642.20
2	Form work for foundation concrete	138.6	m^2	84.00	11,642.40
3	P.C.C. (1:2:4)	66	m^3	4214.21	2,78,137.86
					3,03,422.46

9.5. Development of numerical model

Application of numerical model give analysis of SEEP2D is a 2D seepage analysis program written by Dr. Fred Tracy of the United States Army Corps of Engineers. The program is used to analyze water seepage, typically through dams and levees or under sheet piles. "The model is internationally known in the engineering community as a model for complicated seepage analysis of dams and levees. It has been shown to have acceptable accuracy compared with experimental results for calibration simulation of 2 D Finite Element Model (FEM) Analysis

Seepage Analysis:

An earth fill dam's body prevents the flow of water from dam's back to downstream. However, with the most impermeable materials used in the dam's body, some amount of water seeps into dam's body and goes out from downstream of body slope until it meets an impermeable barrier. So if the water level at the upstream side is rapidly lowered, the water-soaked material may become unstable. This has to be considered in the design of earthfill dams. Earthfill dams are usually designed pervious, and some seepage flow through the dam body must be expected. Seepage flow which occurs in the dam's body has a top surface. This surface is called as phreatic line or zero pressure curves. However the upper zone of phreatic line can be wet or saturated because of capillarity. There is a pore water pressure under the phreatic line. Seepage in the dam's body is important due to two reasons. First one is that, phreatic line cuts downstream slab. The higher cutting of the dam slab because of phreatic line is the more dangereous condition for the slab, because the soil under that point will be saturated, when the soil saturation increases, pore water pressure increases too and due to the quantity of saturation, collapse probability increases. Second reason is maximum reservoir position that contains the body's maximum saturation degree is the most critical condition for the downstream slab's stability after the construction. The most critical condition for upstream slab's stability is the sudden drop in the water level in the reservoir. Both the cases make the dam unstable.

The Seepage through a pervious soil material, for 2-D flow, is given by Laplacian Equation,

$$\frac{\delta^2 \phi}{\delta x^2} + \frac{\delta^2 \phi}{\delta y^2} = \mathbf{0} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 9.3$$

Where, Φ = K. h = Velocity Potential

K = Permeability of Soil

h = Head causing flow

The above equation is based on the following assumptions:

1. Water is incompressible.
2. The soil is incompressible and porous. The sizes of the pore space do not change with time regardless the water pressure.
3. The quantity of water entering the soil in any given time is the same as the quantity flowing out of the soil.
4. Darcy's law is valid for given soil.
5. The hydraulic boundary conditions at the entry and exit are known.

The solution of Laplace equation gives two sets of curves as solution. This two sets of curves, known as equipotential line and flow lines (or stream lines), are orthogonal to each other. These two sets of lines together form Flow Net.

The properties of Flow Net are as follows:

- The flow lines and equipotential lines meet at right angles to one another.
- The fields are approximately square, so that acircle can be drawn touching all the four sides of the square.
- The quantity of water flowing through each flow channel is the same. Similarly same potential drop occurs between two successive equipotential lines.
- Smaller the dimensions of the field, grater will be the hydraulic gradient and velocity of flow through it.
- In a homogeneous soil, every transition in the shape of the curves is smooth, being either elliptical or parabolic in shape.

Description of the GMS SEEP2D Simulation Model:

SEEP2D is a 2D finite-element flow model designed to compute seepage on profile such as for earthen dam and levee cross sections. Finite element method (FEM) is a numerical technique for finding approximate solutions to boundary value problems. It uses variation methods (the Calculus of variations) to minimize an error function and produce a stable solution. Analogous to the idea that connecting many tiny straight lines can approximate a larger circle, FEM encompasses all the methods for connecting many simple element equations over many small sub-domains, named finite elements, to approximate a more complex equation over a larger domain. SEEP2D can be used to model confined, partially confined, and unconfined flow situations. For partially confined and unconfined flow situations, both the saturated and unsaturated flow is simulated and the phreatic surface determined. SEEP2D can model complicated 2-D seepage problems involving complex model geometries and soils that are non-homogeneous and anisotropic.SEEP2D is a steady-state flow model and will compute the flow value at each node of the finite-element mesh. From these values, flow lines and equipotential lines are plotted showing the resulting seepage flow net. The SEEP2D Interface Module provides a user-friendly graphical interface to the SEEP2D flow model. This Interface is used to assign boundary conditions and analysis parameters to the finite-element mesh to be modeled. During the model simulation the unit weight of water must be assinged. SEEP2D uses this value to compute pore pressures. The weight and length units defined in this value should be consistent with the units used elsewhere in the model.As our model is unconfined in nature, two options are available for dealing with the unsaturated zone: (1) deforming mesh and (2) saturated/unsaturated flow modeling.For both types of problems, exit face boundary conditions should be applied along the boundary of the mesh where the free surface is expected to exit. With the deforming mesh option, SEEP2D iterates to find the location of the phreatic surface and the mesh is deformed or truncated so that the upper boundary of the mesh matches the phreatic surface. The solution files from this type of simulation include a geometry file containing the deformed mesh. With the saturated/unsaturated option, the mesh is not modified and the flow in both the saturated and unsaturated zone is modelled. The hydraulic conductivity in the unsaturated zone is modified (reduced) using either the linear frontal method or the Van Genuchten method. Type of materials and hydraulic conductivity are also inserted into the model. The most common type of SEEP2D boundary conditions is nodal boundary conditions. Three types of nodal boundary conditions can be assigned: head, exit face, and flow rate.

Head Boundary Condition – Specified head boundary conditions represent boundaries where the head is known. They typically are found where water is ponding or at the boundary of a region where the water table is known to remain constant. Since the head along such boundaries cannot change, they represent regions of the model where flow enters or exits the system (flow lines are always orthogonal to constant head boundaries).

- **Exit Face Boundary Condition** – Exit face boundary conditions imply that the head is equal to the elevation (assuming that the datum is 0). They are used when modeling unconfined flow problems and should be placed along the face where the free surface is likely to exit the model. This boundary condition must be used if the option for deforming the mesh to the phreatic surface has been selected in the analysis options. It may also be used with a saturated/unsaturated flow model. In this case, if the head at a node on the boundary becomes greater than the node elevation during the iteration process, the head at the node is fixed at the nodal elevation and the node acts as a specified head boundary. Thus, water is allowed to exit the boundary above the tail water. If an exit face boundary is not used with a saturated/unsaturated flow model, all of the flow will be forced through the tailwater.

- **Flow Rate Boundary Condition** – Flow rate boundary conditions are used to specify nodes at which a certain flow rate is known to exist. They are used primarily when modeling wells and the flow specified represents the pumping rate. Negative values represent extraction of fluid from the system whereas positive values represent injection.After simulating the model phreatic surface, equipotential lines and flow lines are displayed in the output window.

Description of the Problem:

A typical earthen check dam having dimensions is delineated in figure 7.1. The height of the dam is found to be 1.8 m where as the bottom width and top width is highlighted as 6.3 m and 0.9 m respectively. The height of the head water is 1.7 m with a gradient of 2H: 1V in the upstream side. Whereas 1H: 1V is considered in the downstream side.

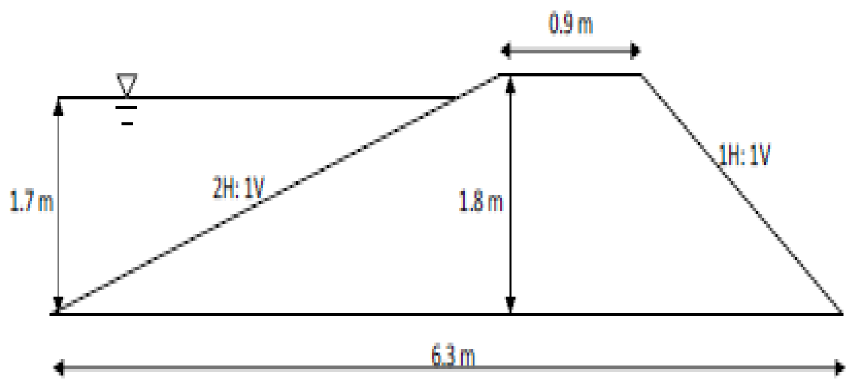


Figure 9.9: Diagram of the Earthen Dam

An earthen check dam has been considered to quantify the flow rate in each field by making flow net using SEEP2D finite element model as depicted in Figure 9.10. In this case, the material is considered homogeneous and there was no provision so far drainage gallery is considered. The permiability of the sample material was considered with a value of 0.31 m/yr and also there was no tailwater end.

8.4 Output of the model:

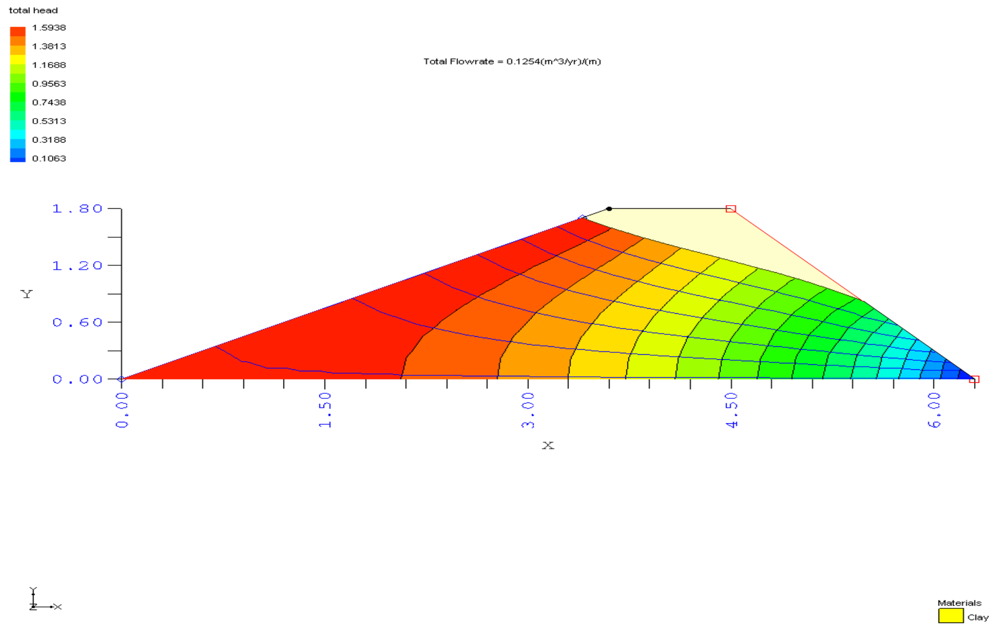


Figure 9.10: Flow net through the Dam

The flow net has been constructed with the help of flow line and equipotential line using SEEP2D model. The equipotential lines are generally perpendicular to the flow lines owing to flow net property. Figure 7.2 depicts there is no flow occurred on the top of the dam section beyond the phreatic line. Figure 9.11 also represented that the total head consist of piezometric head and velocity head was observed as maximum higher value in the upstream side where as it decreases in the tailwater end. Thus it may conclude that the minimum head is observed near the tail end.

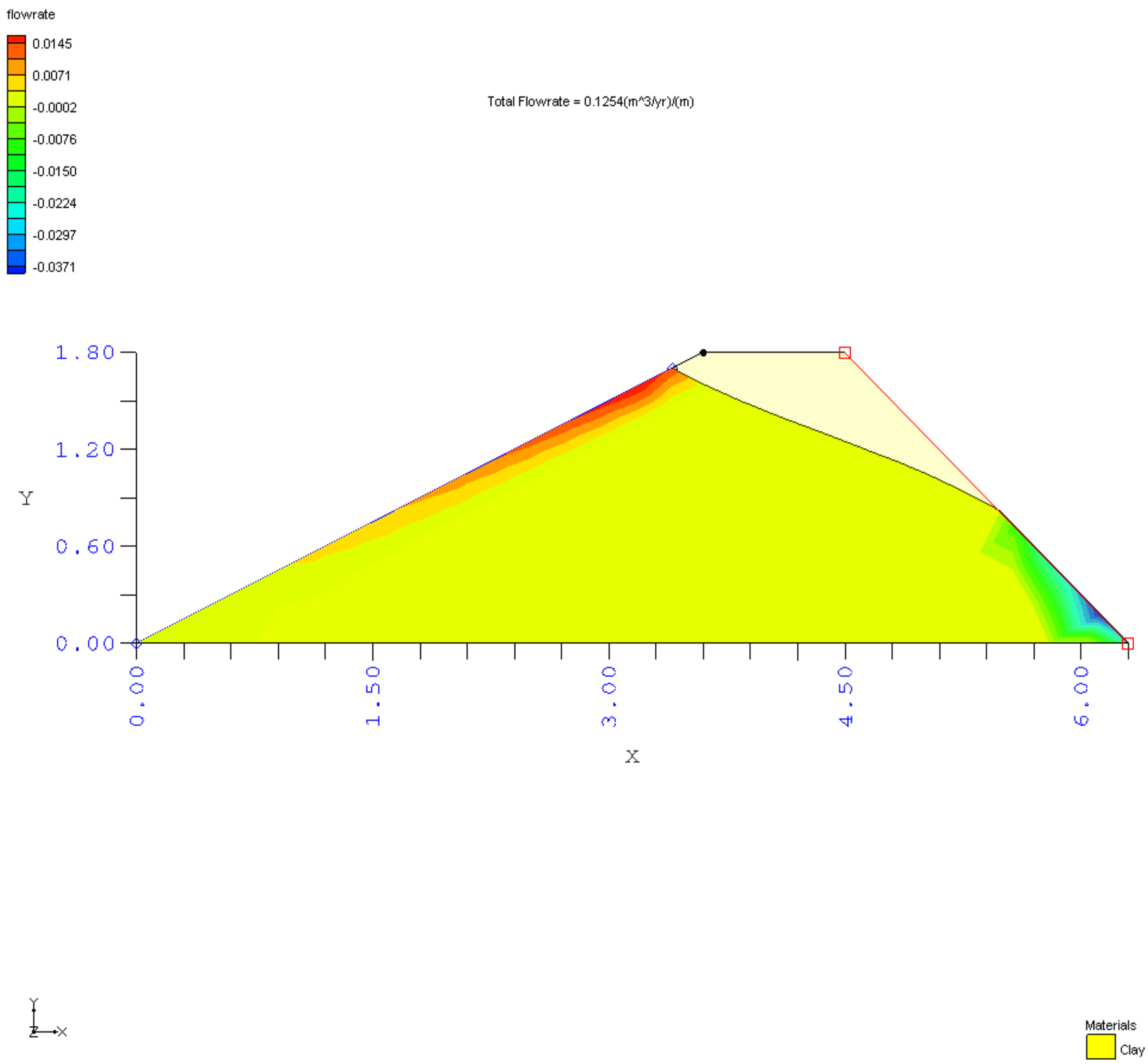


Figure 9.11: Variation of flow rate through the Dam

The total seepage flow rate along the dam section has been observed 0.1254 m^3/yr/m whre as the maximum and minimum flow ranged between 0.0145 m^3/yr/m and -0.0371 m^3/yr/m at the upstream and downstream end respectively resulted there is no flow occurred at the downstream end. Thus it may be opined that if the cross-section of the dam increases, the seepage flow will decrease.

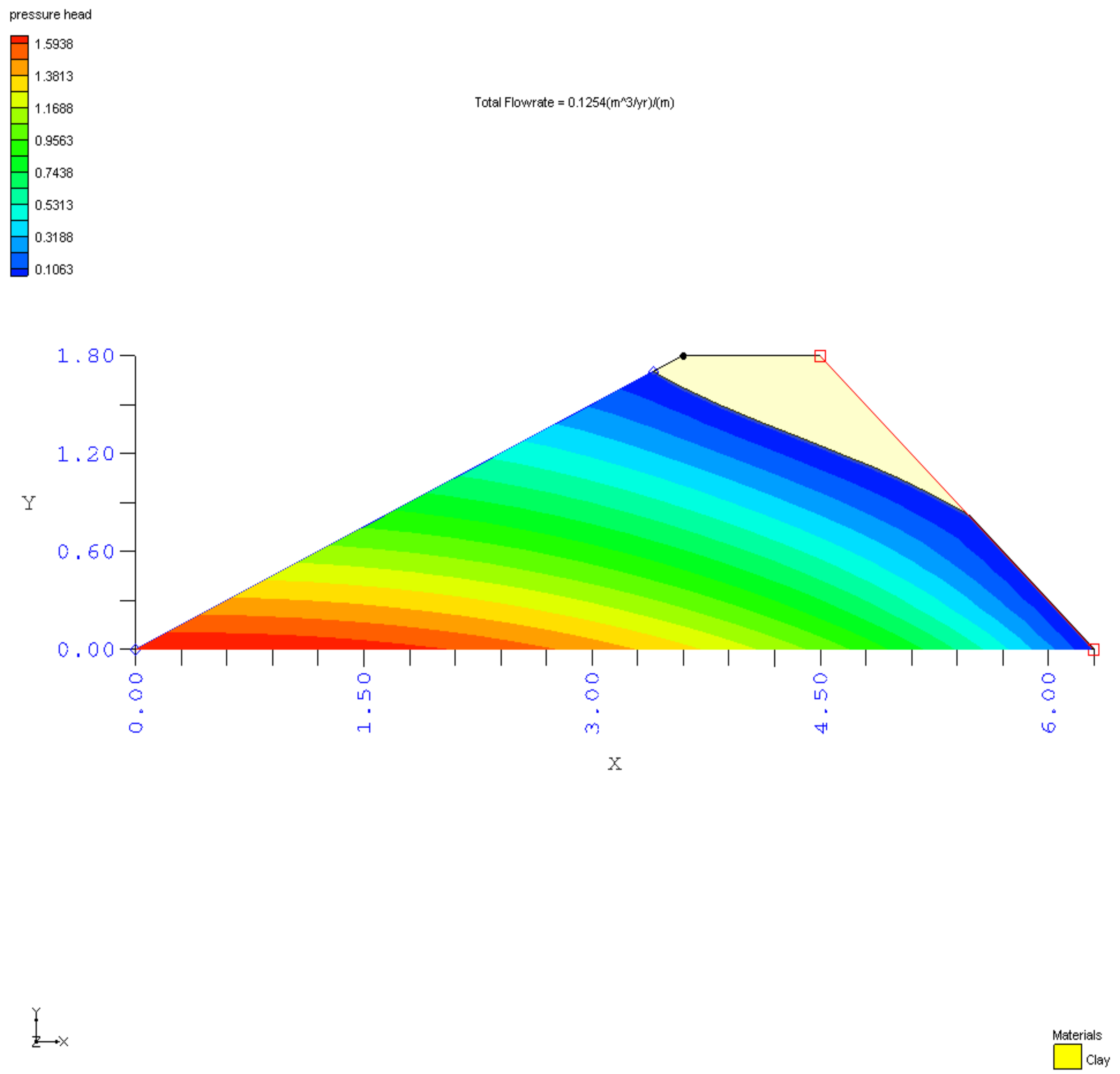


Figure 9.12: Variation of pressure head through the Dam

The Figure 9.12 shows the variation of pressure within the dam due to the seepage flow. In this figure this is clearly evident that near the up-stram toe of the dam, the pressure is maximum (about1.6 N/m^2). At the phreatic line the pressure is zero. The pressure at the upstream toe of the dam can cause the dam to failureif it rises to a very high value.This SEEP2D simulation Model is based on some predictions. So, it might have differed to the original phenomenon. Thus, to validate the SEEP2D Simulation Model, A hand calculation was done to check the phreatic line and the toatal seepage flow of the dam. The comparison between the two models is given below.

Comparison of SEEP2D Simulation and Analytical Calculation for Seepage Flow and Phreatic Line:

Data collected from the field:

Co-efficient of permeability = 0.31 m/yr.

Top level of the dam = 1.8 m

Level of the deepest river =0 m

High flood Level = 1.7 m

Top width of the dam = 0.9 m

Up-stream slope = 2H: 1V

Down-stream slope = 1H: 1V

Taking the Focus F at the down-stream toe of the dam as the origin, the equation of the base parabola is given by

$$\sqrt{x^2 + y^2} = x + s$$

Where,'s' is the distance point of (x, y) from the directrix, called focal length.

Selected the point in such a way that,

AB = 0.5HB

AB = 0.5×1.7 =0.85 m

The point A is plotted 0.34 m away from B. This is the starting point of the parabola. Now the co-ordinates of point A w.r.t F as origin are (2.75, 1.7).

Substituting this ordinate in base parabola we get,

$$\sqrt{2.75^2 + 1.7^2} = 2.75 + S$$

This gives, S = 0.48

The vertex(C) of the parabola shall be situated at a distance equal to $\frac{S}{2}$ or 0.24 m from F, beyond the down-stream toe of the dam.

Now, $x^2 + y^2 = x^2 + S^2 + 2xS$

$$\text{Or, } y=\sqrt{S^2 + 2xS}$$

X	0	0.18	0.36	0.54	0.72	0.817	1.09	1.275	1.455
Y	0.48	0.635	0.759	0.865	0.96	1	1.13	1.206	1.276
X	1.632	1.806	1.991	2.206	2.21	2.459	2.692	2.9	
Y	1.34	1.401	1.463	1.532	1.534	1.61	1.678	1.736	

With these values of ordinates, the base parabola is drawn.

Now this parabola has to be corrected at entry and exit. At entry the phreatic line is started from point B such a way that it becomes at right angles to the upstream face GB of the Dam.At exit, the point K at which the phreatic line intersects the downstream face can be obtained by using the equation, $\Delta a = (a + \Delta a)\left(\frac{180-\alpha}{400}\right)$

Where α is the angle of the downstream face = 45°

(a+Δa) = distance FJ i.e the distance of the focus from the point at which the base parabola intersects the downstream face.

From the calculation we get, a= 0.9 m from toe

Now, knowing 'a' the phreatic line is plotted and at the same graph the phreatic line obtained from SEEP2D model is also plotted.

The ordinates of the phreatic line for both the cases are tabulated below.

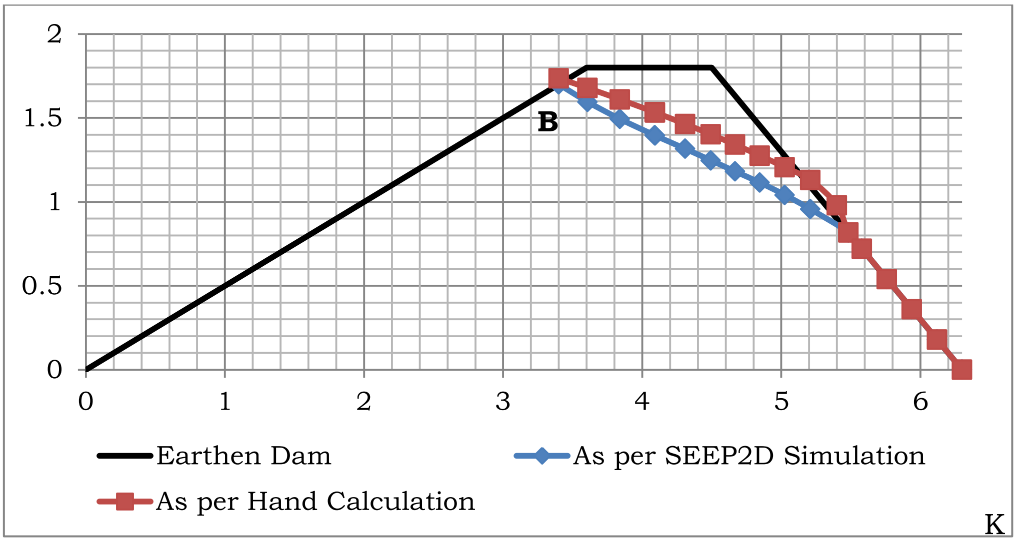


Figure 9.13 : Phreatic Line of the Earthen Dam for both the Cases

Using SEEP2D Simulation and Hand Calculation)

Now the flow through the Dam, q= K.S

Where, permiability of the soil, K = 1×10^{-6} m/sec = 0.31 m/year

S = 0.48 m

So, total seepage flow = 0.31×0.48 = 0.1488 m^2/yr/m

The same problem when simulated using SEEP2D Model,

Seepage flow using SEEP2D simulation = 0.1254 m^2/yr/m

The SEEP2D simulation of flowrate differs from the hand calculation by

$(\frac{0.1488-0.1254}{0.1488})$ = **15.7%**

9.6. Description of Field Model Implemented at the Site

Based upon the topographical and hydro-geological survey aided by GPS, Sokkia Total Station Survey and with the help of some field and historical data, some suitable areas were demarcated along the Futiary River catchment for construction of a composite modelcomprising a 30 m wide Cement Concrete Check dam followed by a Collector Well or an Infiltration Gallery and a sub-surface Dyke. Groundwater recharging structures are placed in series. Check dam will retain the rainwater then from 1.8 m to 2.06 m up to the height of check dam and then overflow water passes through natural river bed at length of 179 m and natural filtration process don trough river sand layer. Collector well is constructed in the down stream at right bank of the river depth of collector well is 7 meter 4 meter below the river bed and rest three meter above. the bed level. Two Strainers are installed across the river up to 18 m. Two valves also fitted in well two control the river water inflow during the time of disinfection. At 109 m below the collector well subsurface dam was constructed to for sustainable water availability in sand zone up to three meter below the sand bed level. Schematic diagram and map locations are shown in Figure 9.14 and Figure 9.15.

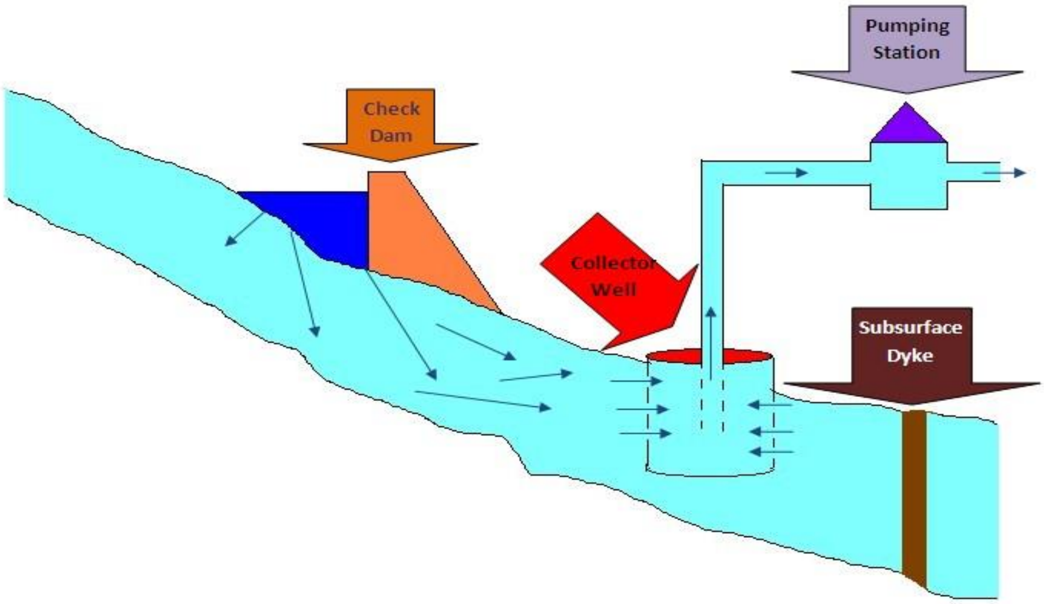


Figure 9.14 : Schematic diagram of the composite model

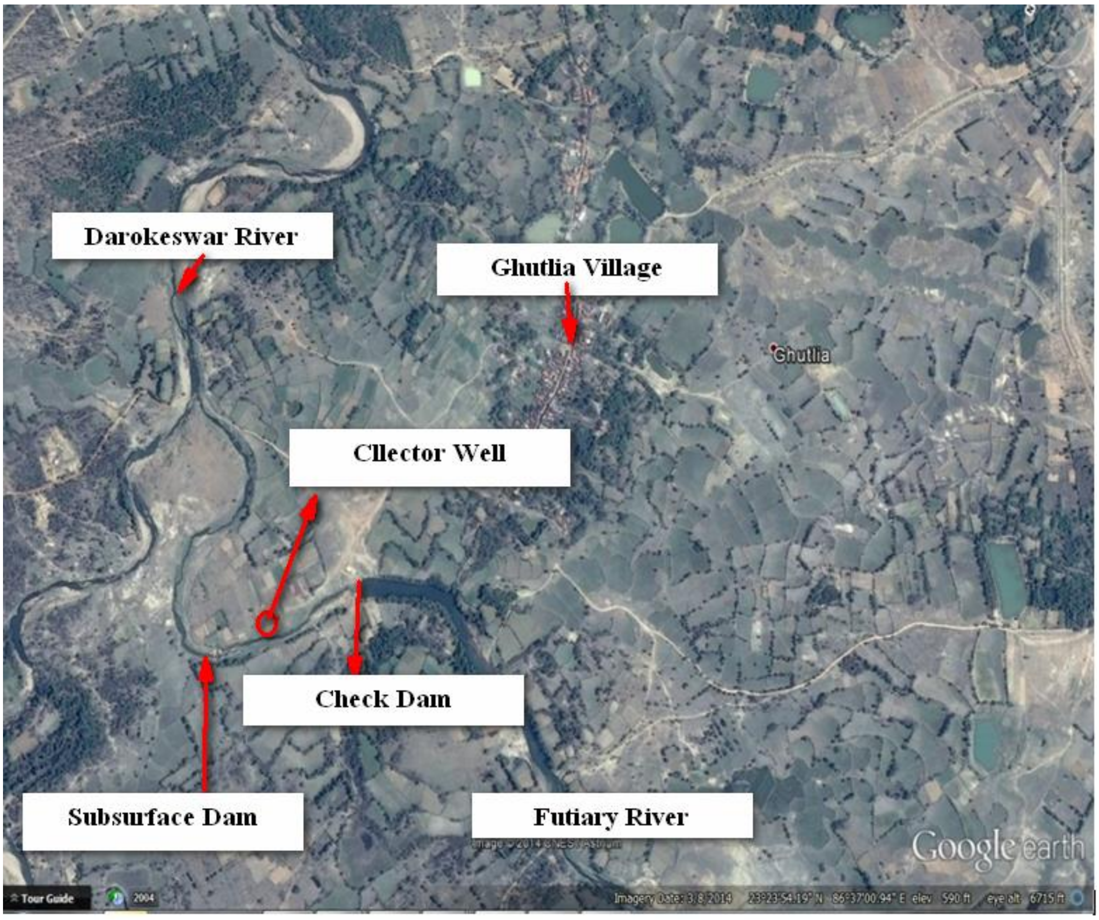


Figure 9.15 :Satellite imagery shows the Location of the structures in field

9.6.1. Check Dam with Recharge Shaft/Pit

Cement Concrete Check Dam along is to be done to restrict the quantum of rain water flows mainly in monsoon. In this regard, the Cement concrete check dam is much durable against the flash floods at those regions during the monsoon. The longevity and effectively of such type of check dam is also more than the boulder check dams or earthen check dams. This is recharge cum conservation structure recommended in all type of slope. In Ghutlia village 30m wide and 2.06 height check dam was constructed. Construction process includes layout of the structures, earthwork for foundation and diversion work, construction of cutoff wall and mentoring and construction of weir section and retaining wall. Layout of construction is given Photo 9.1 and Photo 9.2.

Photo 9.1: The length measurement for check dam layout of prior begin the construction work

Photo 9.2 : Layout marking of construction site for check dam

Earth excavation for the foundation of different units of composite model i.e. check dam and sub-surface dam has been done with the help of earth removal machinery and manual labor of the village. Initially coffer dam and diversion channel was constructed for the proper construction work in river bed depicted in Photo 9.3 and Photo 9.4

Photo 9.3: Earth work excavation for the foundation work of the check dam

Photo 9.4 : Constructed Diversion channel

For the check dam construction first cut-off shown in Photo 9.5 wall has been constructed in Upstream side and Downstream then boulder soling and weir section construction was carried

Photo 9.5 : Construction of Cutoff wall in upstream side

Construction work has been carried out under regular supervision of the research team members of SWRE, JU, to ensure that construction work carried out properly as per drawing and design shown in Photo 9.6.

Photo 9.6 Construction of weir section of Dam site

Quality monitoring and compressive strength test of concrete material also done by SWRE, JU at the time of concreting and other construction work. Abrasion value test also carried out for securing good quality boulder and stone chips material .Two hume pipes are also installed in weir section for future silt clearance shown in Photo 9.7,Photo 9.8,Photo 9.9, and Photo 9.10.


Photo 9.7 : Preparation of concrete test cube and its compressive strength for quality monitoring of the material section of Dam structure

Photo 9.8: Installation hume pipe in check dam construction

Photo 9.9 : The constructed weir section in Check Dam

Photo 9.10 The construction of wing wall in Check Dam Unit

Constant supervision done and quality monitoring and the overall performance of construction work regularly done by the Investigators and other eminent experts engaged by School of Water Resources Engineering, Jadavpur University at regular interval depicted in Photo 9.11.

Photo 9.11 : Site Inspection by SWRE, JU Research Team

9.6.2. Collector Well

A collector well is a sub-surface groundwater collection system, typically shallow in depth, constructed with open-jointed or perforated pipes that discharge collected water into a watertight chamber from which the water is pumped to treatment facilities and into the distribution system. The Infiltration Gallery is usually located close to streams or ponds. In this study the collector is not a rainwater harvesting/artificial recharging structure. Rather it is a water collecting structure. The Infiltration Gallery in this study collects the water from the river after natural river bank filtration and this way may fulfill the water demand of the area sustainable quantum of water. Construction of different steps of collector well shown in Photo 9.12Photo 9.13, Photo 9.14, Photo 9.15 and Photo 9.16.

Photo 9.12 : Earthwork excavation for collector well

Photo 9.13: Constructed earthen well

Photo 9.14 : Installation of Sluice valve

Photo 9.15 : Laying out of the strainer Pipe in the River

Photo 9.16 : Photo-Gravel packing in River Bed

Photo 9.17: Constructed Collector Well

9.6.3. Sub-surface Dam

Subsurface dams are groundwater dams, which are structures that intercept or obstruct the natural flow of groundwater and provide storage for water underground. They have been used in several parts of the world, notably India, Africa and Brazil. They are used in areas where flows of groundwater vary considerably during the course of the year, from very high flows following rain to negligible flows during the dry season. A subsurface dam intercepts or obstructs the flow of an aquifer and reduces the variation of the level of the groundwater table upstream of the dam. It is built entirely under the ground. In this study, a sub-surface dam is to be designed after the Check dam to keep the river bed after the check dam saturated. During the monsoon the excess water will spill over the check dam. But during dry season there will be no flow or very less flow downstream of the check dam. The sub-surface dam will intercept the natural flow during the monsoon and during the non-monsoon period the intercepted water will keep the river bed saturated. According to the field conditions, a sub-surface dam will be design.

Photo 9.18 : Construction of Subsurface Dam

Photo 9.19 : Constructed Subsurface Dam and Its Effect

9.6. Remarks

Although the benefits of integrated cost-effective models are numerous, particularly in the context of safe and sustainable water supply and other purpose as well. Predict groundwater recharging techniques with different with water retaining structures could be an alternative solution with respect to water storage to a great extent. As, know past literature revealed that many dams have serious negative environmental, human, and political consequences. Despite an extensive literature documenting the benefits and costs of dams from a single disciplinary perspective, few studies have simultaneously evaluated the distribution of socio-economic and technical implications of dams. To meet the simultaneous demands for recharging, and environmental protection well into the future, a broader view of integrated model is very much essential to overcome water stress condition. Therefore, a new tool is developed for evaluating the relative cost and benefits of water retaining structures based on multi-objective planning techniques. The SEEP-2D tool is designed to integrate biophysical, socioeconomic, and structural perspectives into a single prediction way of dam design. SEEP2D is a 2D finite-element flow model designed to compute seepage on profile such as for earthen dam and levee cross sections. Finite element method is a numerical technique considering some boundary condition for finding approximate solution and also gives more reliability, stability and durability to the adopted constructed model of integrated cost-effective technology and gives a output scenario of earthen dam construction suitability analysis. FEM gives best location for site specific project against output scenario of water retaining structure where is liked to be implemented or constructed at site. Numerical model also justify the advantage of adopted methodology of integrated model construction without construction of earthen dam in terms of stability and durability of dam. Integration of different structures in series gives simultaneous output in agricultural water supply, groundwater recharging, and drinking water supply than a single individual structure.

CHAPTER-10
RESULTS AND IMPACT ANALYSIS

10. Results and impact analysis

10.1. Hydrological and groundwater recharge impact

The twenty four years rainfall data have been collected and analyzed given in Figure 10.1 . The highest and lowest annual rainfall has occurred in 1999 and 2005 respectively. The result also highlights about 80% of total rainfall has predominant during monsoon. Figure 10.1 describes the highest rainfall occurred during monsoon in different periods and the highest magnitude has found to be 316 mm in 2006 followed by 276 mm in 1995 and so on. Thus the study area represents the intensity of rainfall has very high compared to the total annual rainfall found an average of 900 mm in the same area and hence the study area defines as zone of shot-pockets of scarcity so far the quantity of water is concerned.

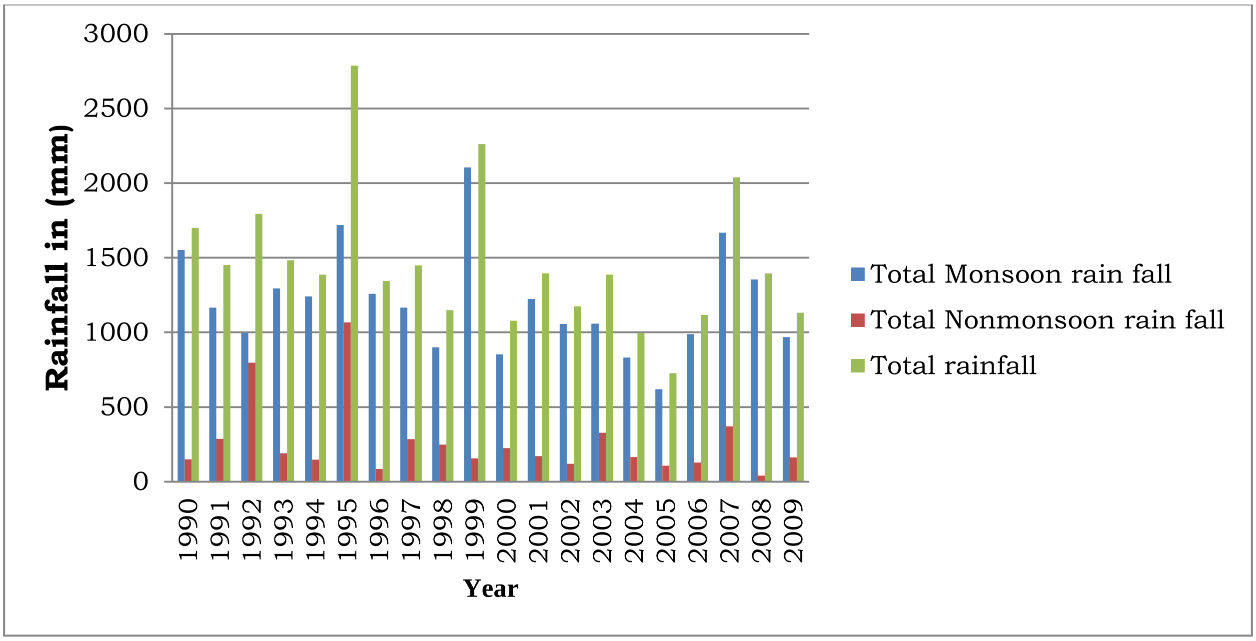


Figure 10.1 : Comparison of 20 years peak rain fall data.

Stage level of water storage structures

Regular water level records of five major water storage structures, check dam, sub surfaced am and Dugwells and three pond done regular basis from 2010 to 2014 . Check dam and Subsurface dam included in 2013 more precisely after completion of all kind of construction work completed for purpose of studying their impact on groundwater recharge. The stage level gauges were installed upstream of groundwater recharging structures to measure daily fluctuations in the water level of the storage structures during the water availability period.

Groundwater level monitoring network

Twenty two numbers of tubwell depicted in Figure 10.2 are available for providing drinking water as well as irrigation water. In addition to the existing wells, one more collector well and a piezometer were also installed in the study area. The details of the spatial distribution of wells/piezometers are shown in Figure 10.2.The water quality analysis were done regularly with help of field test kit and water samples were also taken to the laboratory for the chemical analysis to estimate chloride concentration in the water. Monthly twenty well monitored monthly basis and in situ TDS, Salinity, Conductivity, fluoride this test were done with help of Hach field test kits. simultaneously water levels were measured with TLC meter . The data was collected during from August 2010 to December 2014 and the interval of data collection varied from a maximum of 30 days to a minimum of seven days depending upon the alteration of in the water table.

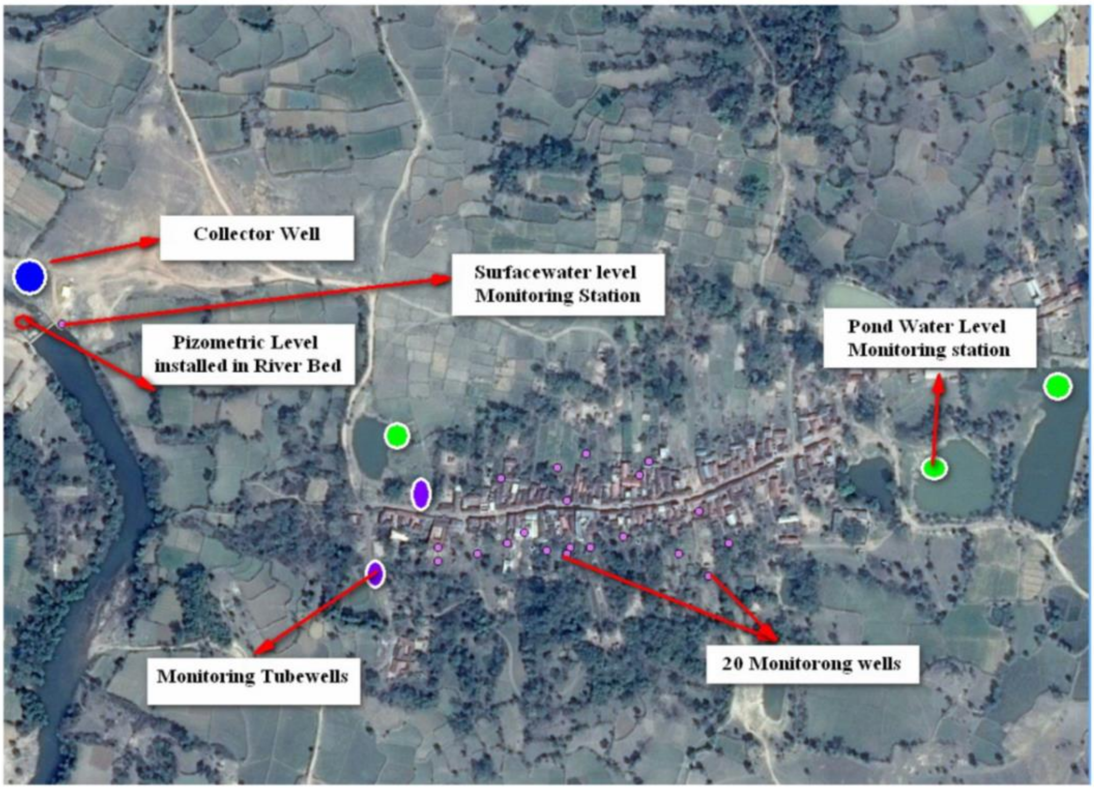


Figure 10.2 : Water Quality and Water level monitoring Network

Hydrological framework

The annual average annual rainfall of the area has been recorded as 1100 mm (1990–2014). The water storage and recharge structures are effective mostly during the high intensity and monsoon period. The distribution of rainfall events since 2005 exhibit low rainfall situation. Assessment of potential recharge from Recharging structures. An empirical equation conforming to power function was found to best describe the relationship between depth of storage and potential recharge expressed as

$$\mathbf{Rep=ah^{b}_{av}}\text{..}(10.1)$$

In above equation 10.1 , Rep is the potential recharge from groundwater recharging structures (m3),The relationship expressed in equation no . (10.2) is specific to a ground recharging structure for which the empirical constants have evaluated. The earlier day storage depth ($\mathbf{h^{b}_{av}}$) in the estimation of potential recharge volume (Rep) is incorporated so that the instantaneous response to water level i.e. level up or down in the in the storage depth due to runoff generated from high intensity storms may not cause any remarkable change in the recharge volume and is averaged estimation with 'hbav' to account for variable related to storage depth on a given day. Non-linear estimation technique was adopted for the fitting of the equation instead conventional log transformation technique due to overcome strong non-linearity in between to parameter Rep and hav Temporal variations in potential recharge volume from different groundwater recharge structures as influenced by rainfall patterns are depicted in Figure 10.3. It is Justified that the storage reservoir of the recharging structure get filled up during high intensity events or monsoon period as discussed earlier.

10.1.1. Relationship between potential recharge and rainfall

After estimation of the potential recharge from ground water recharging structure during 2012–2014 (Table 10.1 and Figure 10.3) a functional relationship between the cumulative recharge depth to cumulative rainfall depth was analysed . A extensive study of study of three structures (Check dam, subsurface dam , and two ponds) was carried out to find out their characteristics individually as well as integrated . The evaluating equation fitted to obtain the relationship between rainfall (P) and: is cumulative recharge (Re) of the form

$$\mathrm{Log}_{10}(\mathrm{Re})=a[b-e^{-cp}] \quad [10.2]$$

The equation is fitted using non-linear regression technique involving Rosenbrock–Quasi-Newton method (R–QN). R–QN method is a combination of non-linear estimation techniques such as Rosen brock and Quasi-Newton method and can be advantageously used where appropriatestart values of the fitting parameters are not known. This is the fastest method of convergence and is less sensitive to local minima. The statistical details of the regression analysis are given below table. During the Monsoon period, which experiences intense recharge and withdrawal activities with higher fluctuations in water table, Seasonal fluctuation of water table is monitored and calculated in Figure 10.3 and recharge function for measuring days calculation describe in Table 10.1.

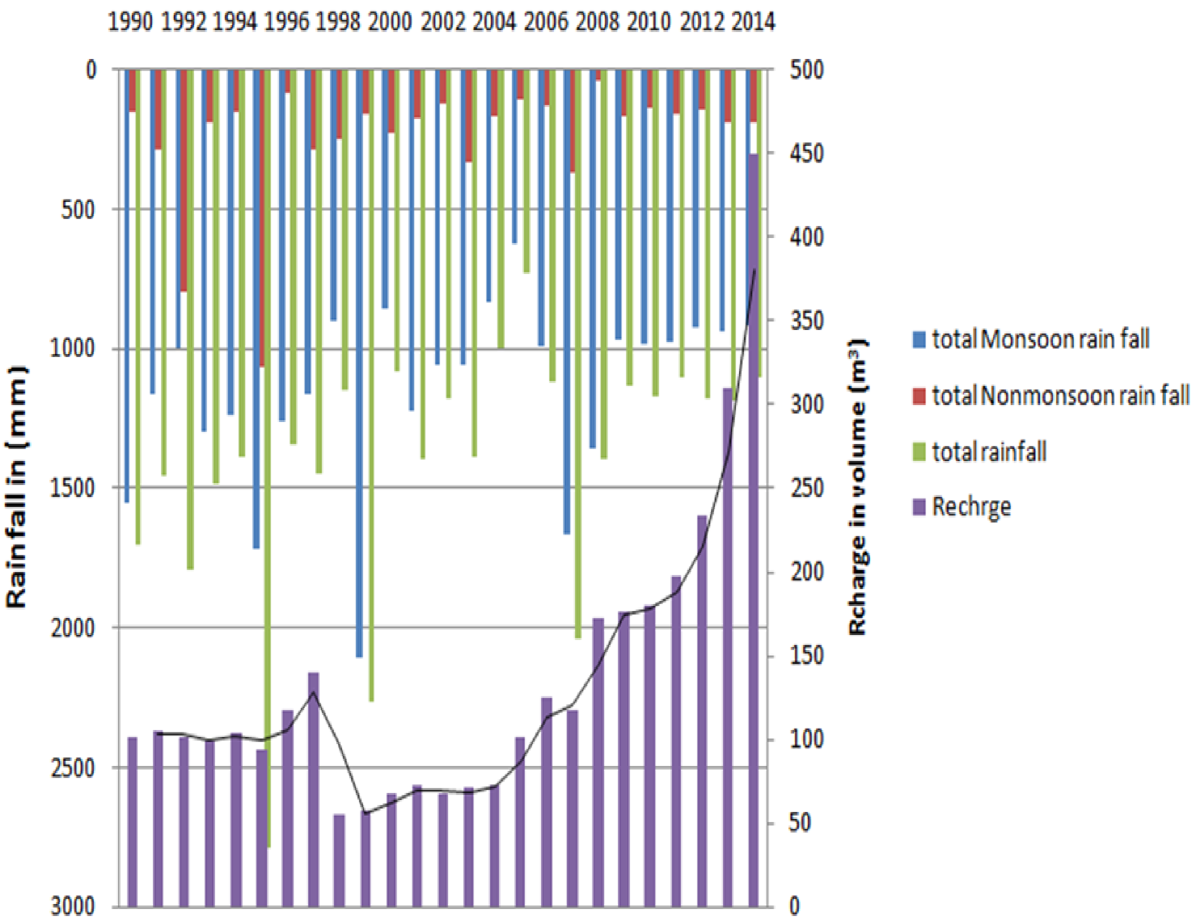


Figure 10.3 : Comparison of rainfall and Recharge volume in the

Table 10.1: Statistical parameter of recharge function for measuring days

Sl No	Rainwater Harvesting structures	Coefficients		
		a	b	R^2
	Check Dam	27.28	3.26	0.822(N=96)
	Sub-surface Dam	28.22	3.13	0.866(N=96)
	Mahatogaria Pond	76.77	1.64	0.633(N=120)
	Thakur bandh	74.88	1.53	0.655(N=120)
	Banser bandh	78.2	1.66	0.710(N=120)

R_g= Recharge(m^3) and h_{av}=successive days of average depth of impounding in m a Data for year 2010 to 2014 ,b selected data from year 2010-2014,

N= number of data pairs.

Impact of potential recharge on groundwater table

The surface water and groundwater interaction in connection to the potential recharge from surface water bodies to the groundwater storage was studied with importance, both under natural and groundwater recharging systems. The primary approach was to study the interaction of all the surface water bodies on the groundwater table fluctuation under the influence of groundwater recharging structures and methods. While the secondary objective was to determine the time required for a drop of water infiltrated from the storage under natural recharge conditions to reach the groundwater storage. The seasonal variation of water table maps (Figure 10.4) reveals that the in the areas having both natural storage structures and ground recharge structure, the groundwater table increased significantly. Based on the water table data collected during July 2012 to August , 2014 (rainfall during the period is 1106-1186 mm), it was established that 21 tube wells/open wells (80%) out of the designated 26 tube wells and open wells has direct impact on recharged water. The net rise in water table during this period was 1.5 m. For a short period, most of the water recharged from Figure 10.4 reveals the value of groundwater table depth before and after construction of groundwater recharge structure, in 2010 same monitoring well shows net annual rise in 1.4 meter.

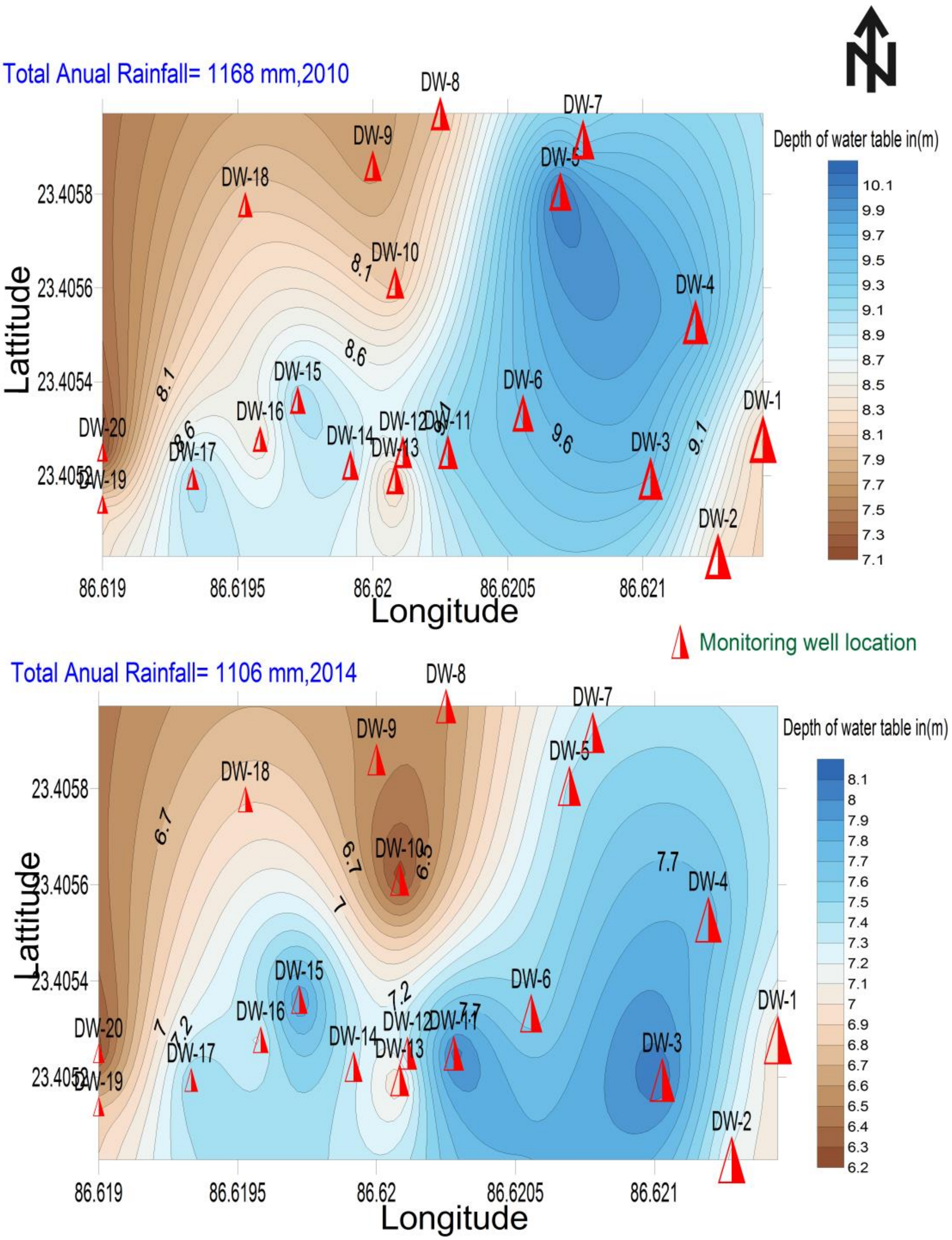


Figure 10.4: Water table depth in Pre and post construction of Model

Storage Scenario

After immediate construction of check dam it has been found the average water level to be 1.07 m in summer season whereas the back water length was measured as 1.07 km. But during rainy season the water level rose to 2.06 m near the dam and the average water level has found to be 1.59 m depth throughout back water length in upstream side covered 1.5 km. After construction of check the storage volume is estimated to be 60221.25 m^3 during monsoon and during summer storage it has calculated as 26937.25 m^3/day and average storage throughout the year is 37510.93 m^3/day. Seasonal Storage due to check dam construction shown in Figure 10.5 So the average recharge from the check dam is found to be as 18775.47 m^3/day. In Figure 10.6 satellite imagery also shows the difference in water condition of river bed.

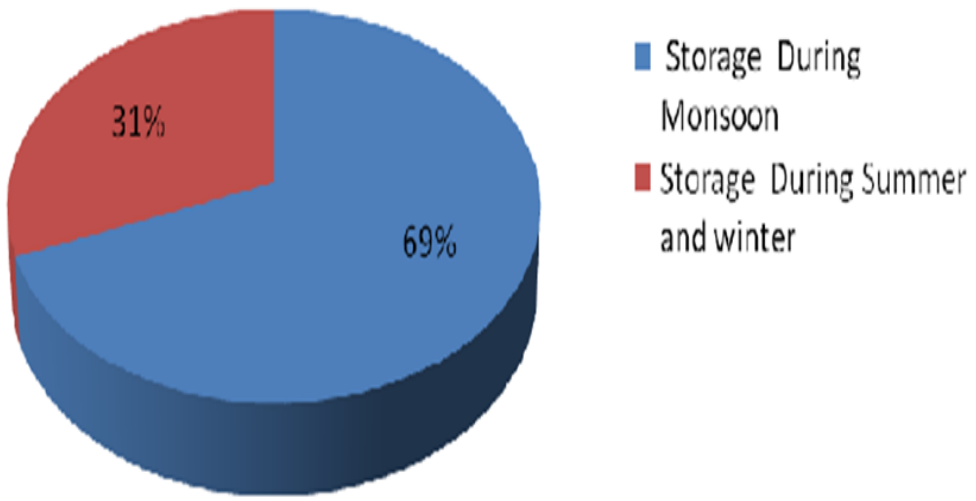
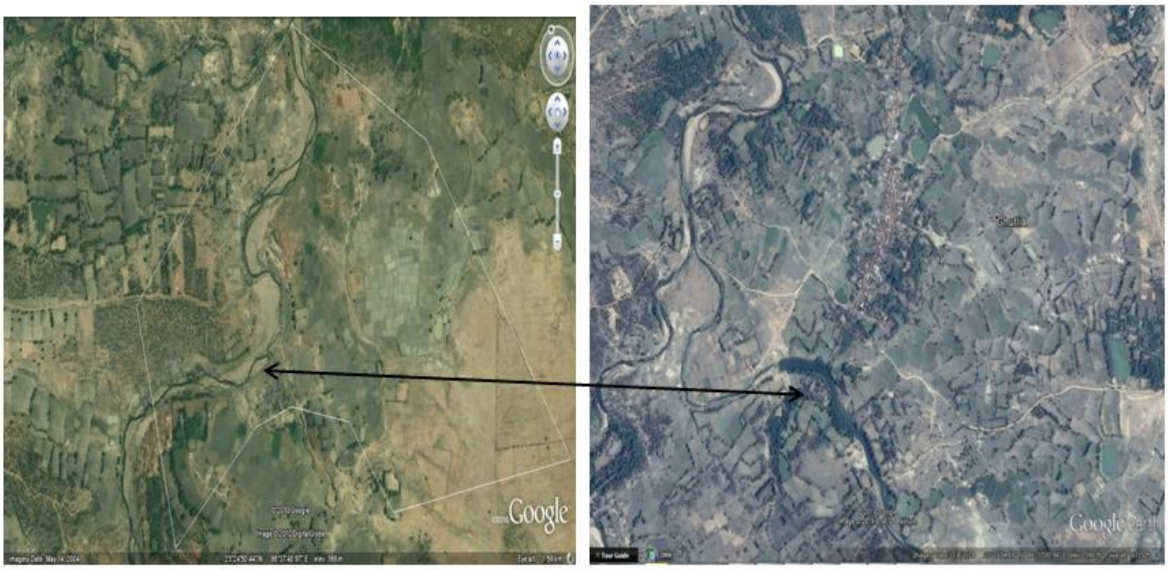


Figure 10.5 : Seasonal storage variation after construction of Check Dam



Pre-Construction **Post-Construction**

Figure 10.6 : Comparison of river condition with the help of satellite imagery

The water level of tube wells and dug wells situated nearby the catchment area are being monitored by weekly. The average water level in summer season rose up to 0.80 m to 1 m after construction of all recharging structures. In monsoon it has increased up to 2.5 m to 3 m and in winter it has increased up to 1.0 to1.5 m. Figure 10.7 reveals the actual water level in wells and in show the comparison water level in different season. Photo 10.1, Photo 10.2,Photo 10.3, shows the different surface water level condition in Futiary river and Photo 10.4 depict the comparative senerio of surface water in pre and post construction time. Finally Photo 10.5 reveals the current perennial condition of the river.

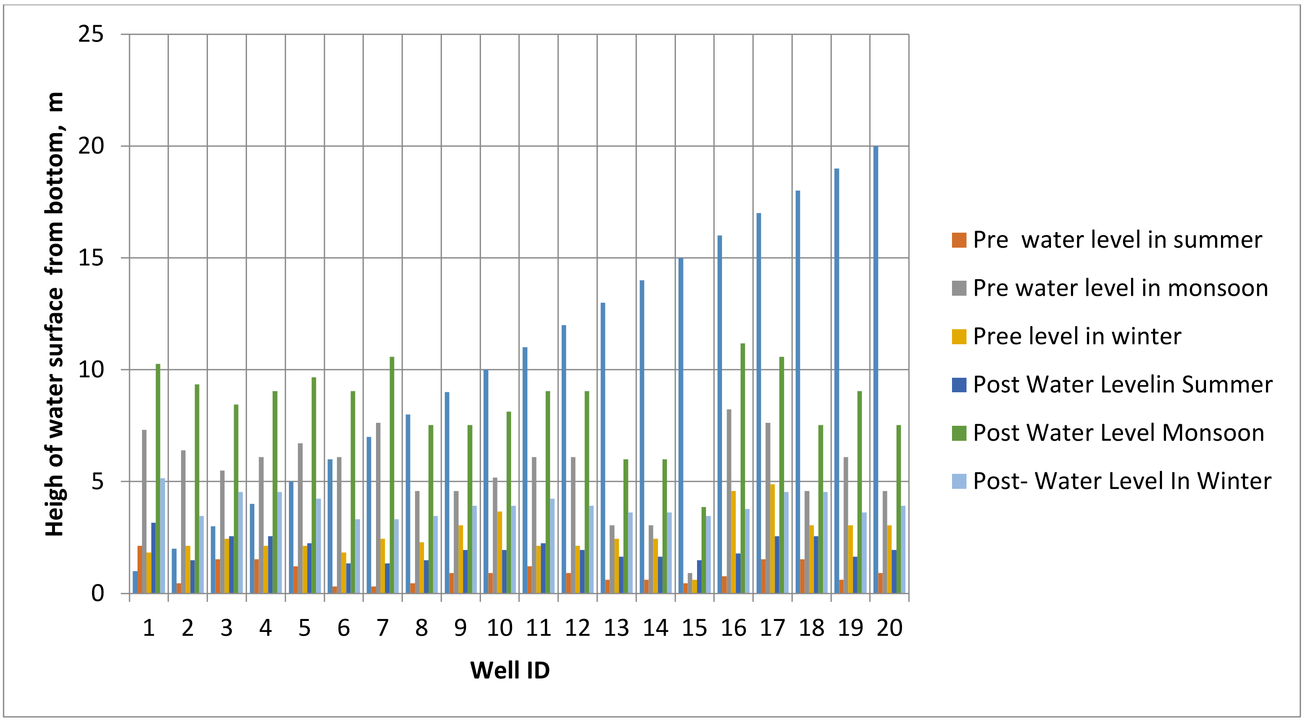


Figure 10.7: Comparison of actual water level in wells in different season

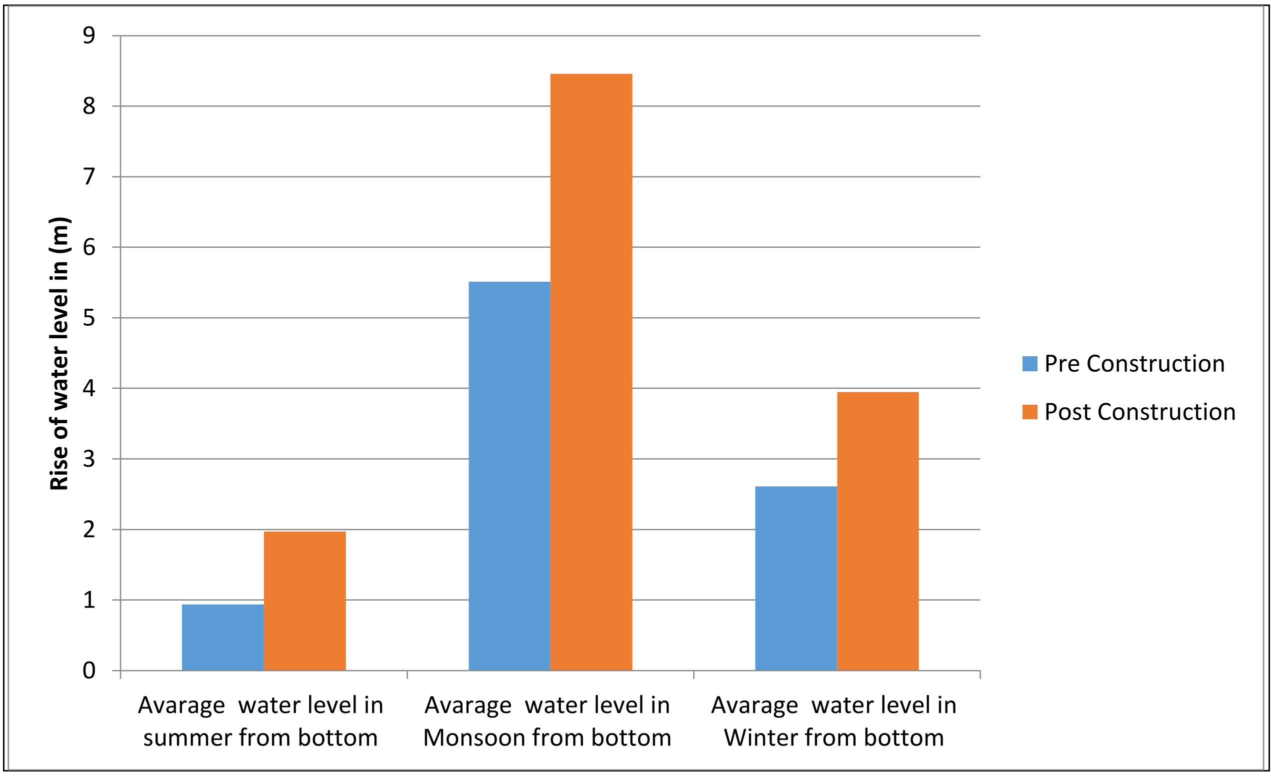


Figure 10.8: Comparison to average water depth change in wells of village

Photo 10.1 : Initial river water condition just after construction of Check Dam in summer, 2014.

Photo 10.2 : Increase in river water condition just after week of construction of Check Dam in summer, 2014.

Photo 10.3: Increase in river water condition after construction of check dam in Monsoon, 2014

River condition after construction of check dam

River condition before check dam construction

Photo 10.4 : Pre and Post river water condition

Photo 10.5 : Perineal water Source

10.2. Socio economic impact

A long-term bottom-up approach is visualized for the development of cost effective technology for recharging in over exploited ground water resources in at Ghutlia. It is expected to generate the conditions and provide lessons for follow-up in the region in other villages. In order to have a holistic documentation of the project implementation process at Ghutlia, it was decided to conduct a baseline survey for evaluation of project benefits to the inhabitants of Ghutlia in the long-run. For the same purpose, semi-structured questionnaires have been administered among all the villagers along with Focused Group Discussions and personal interviews with key informants. The data analysis of this survey delineates the social and economic background of Ghutlia, and provides a careful examination of strengths and vulnerabilities associated with this particular intervention. Post construction of Integrated Cost effective model monthly household income of the village increased significantly due to perennial availability of water for irrigation most of villagers who had the agricultural lad cultivated three time in a year, previously it was on time in a year. Majority of villager shifted monthly income group from Indian rupees 0-500 to 2000-10000.depicted in Figure 10.9

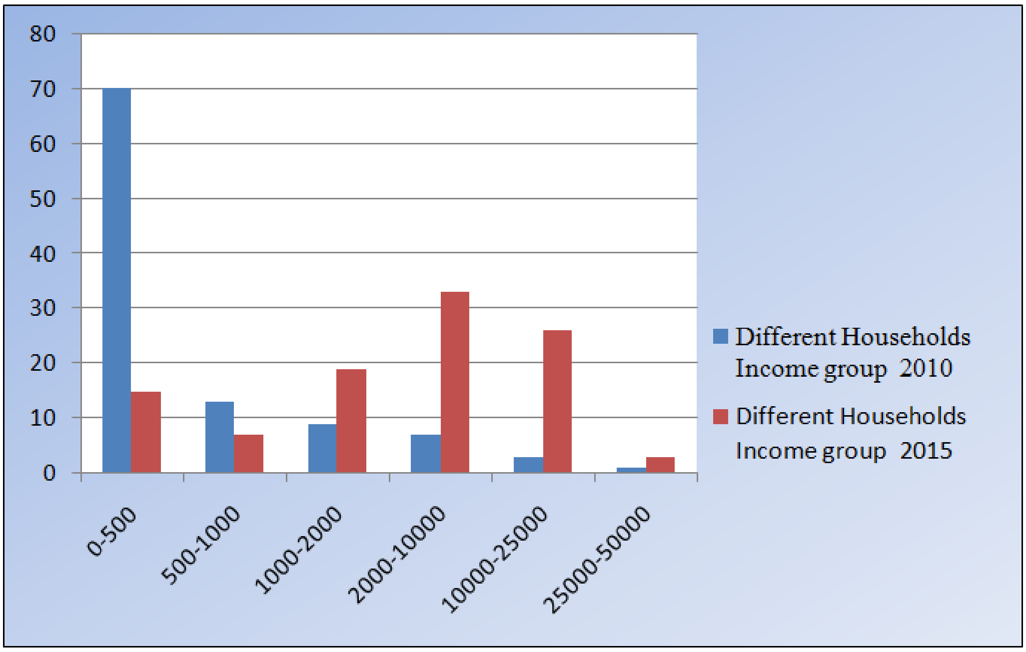


Figure 10.9 : Household monthly income variation in 2010 and 2015

10.3. Economic analysis of Integrated model of cost effective technology for groundwater recharging in over exploited groundwater resources.

Economic analysis for water harvesting structure is essential for selection planning, execution, evaluation and decision making of groundwater recharging system. This is very much important under the existing and future competing scenario. Justification of this model on a financial basis alone may be unreasonable, because of flow of environmental and social benefit of deferent magnitude over a period of time. However it is better to subject the model the rigidities of economic rational. Economic evolution of water harvesting alternative gained in view of making investment decisions for soil and water conservation method.

Measurements of cost are easier than the complex benefit consequences resulting from this composite model designed in this research study for groundwater recharging. Since this model is basically used for recharging particularly to implement in a water scarce areas in the different terrain conditions. Keeping in view, some tangible (market) and intangible (non-market) benefits could be obtained so far the above mentioned composite model is concerned. The following tangible benefits are summarized below:

i. Ground water table will be increased at over exploited area.
ii. Drinking water problem will be solved to a great extent at.
iii. It will give protection to the eroding uplands.
iv. More water will be available for irrigation in kharif and rabi season.
v. Land development in command area structure.
vi. If model is constructed employment will be generated.
vii. Flood moderation and draught alleviation will happen.

Although intangible benefit (extra market) benefit describes consequences which cannot be assigned a monetary value but should be considered while evaluating the cost-benefit ratio after implementation of cost effective composite model. However, so far the designed composite model is concerned, the tangible and non-tangible benefits have been considered for different low cost units suggested in different selected locations to analyze the cost-benefit ratio without hampering environmental restoration and ecological diversity.

Cost benefit analysis

Benefit and Cost estimate are based on future projection and subject to uncertainty. The likely hood of uncertainty become in groundwater recharging systems where water yield, crop production etc. largely depend on climatic features. If benefit-cost ratio (BCR) is more than 1 then the designed ground water recharging structure is consider feasible. BCR for the different unit of composite structure is given below.

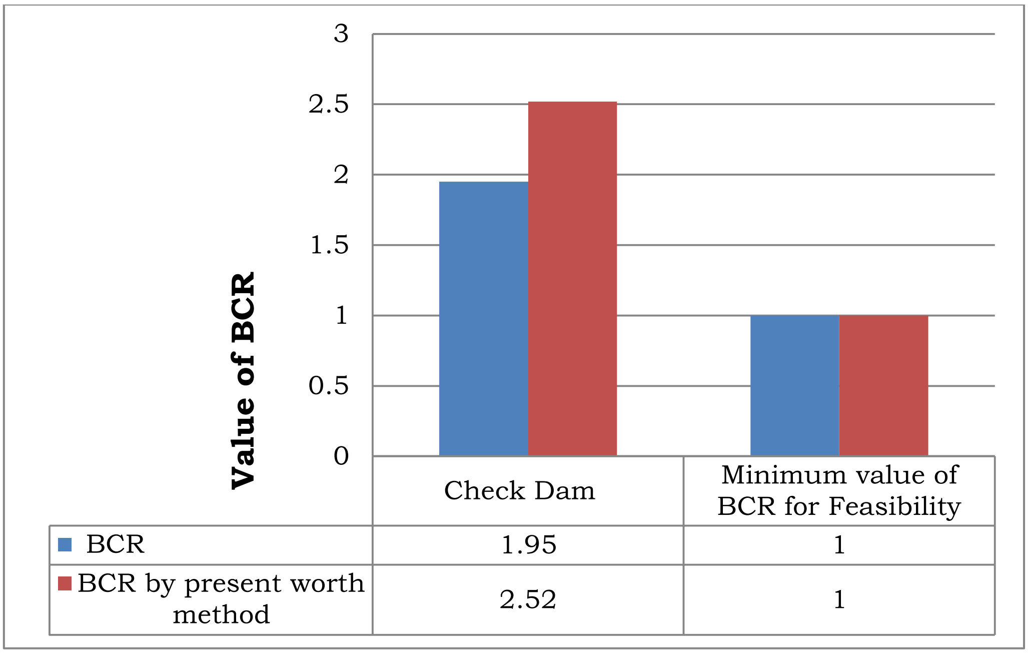
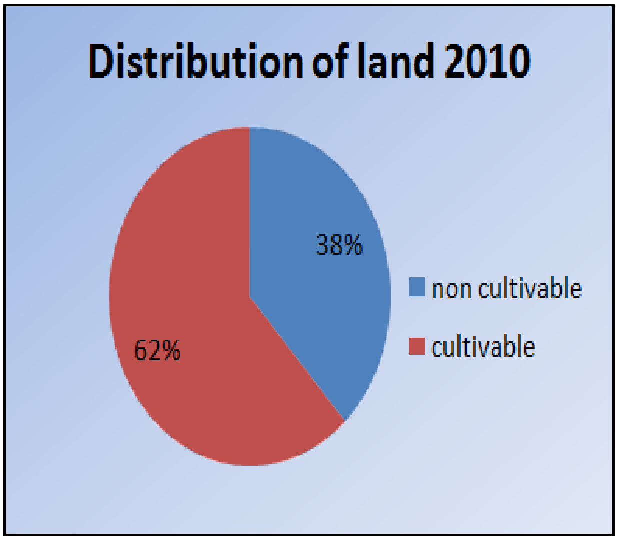


Figure 10.10: Comparison of BCR of different unit of composite structure

Figure 10.10 shows that in both methods the BCR values for both unit of composite model have more than 1. The results indicate that the BCR values have found to be 1.95 and 2.52 whereas the same ratios have been decreased for the same units while it has been calculated by present worth method. Howver, it concludes that the BCRs value may be decreased due to considering the discount rate against estimation for present worth method. The results also suggest that in both cases BCRs value have found greater than 1. Thus both the structures have economically viable.

10.4. Agricultural Impact

Initially in 2010 Most of the households in Ghutlia do not own any piece of fertile cultivable land. In other words, 15 % of households in the village are landless which indicates the existence of high level of poverty in the village. But after implementation of the field model new agricultural land were developed and three time cultivation was performed which increase the income of the villager as discussed earlier section. In 2015 increase of agricultural land shown in the graph below previously it was 62% and increased to 68% presently 6 % increment happen due construction of the Model.



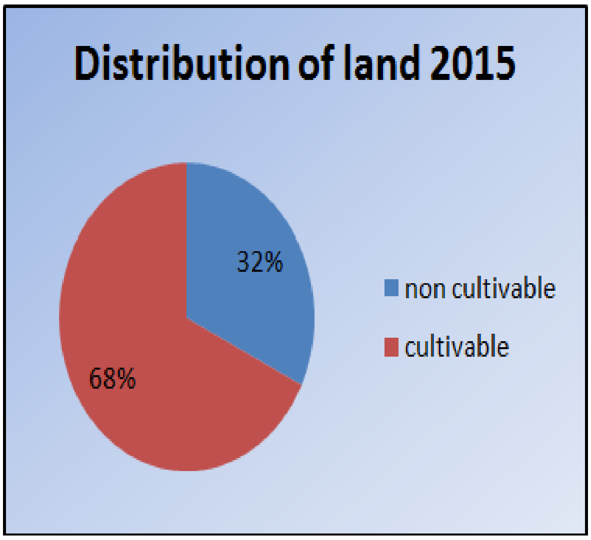


Figure 10.11 :Pre and Post measurement of agricultural Land

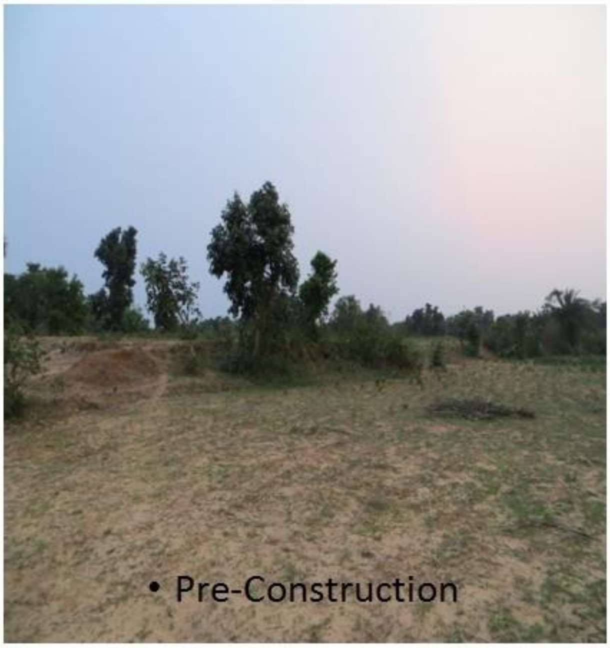


Photo 10.6 : Change in Land use pattern in Study area

10.5. Health impact against drinking water

It is a well-known fact that clean water is absolutely essential for healthy living. Adequate supply of fresh and clean drinking water is a basic need for all human beings on the earth, yet it has been observed that millions of people worldwide are deprived of this. Freshwater resources all over the world are threatened not only by over exploitation and poor management but also by ecological degradation. The main source of freshwater pollution can be attributed to discharge of untreated waste, dumping of industrial effluent, and run-off from agricultural fields. Industrial growth, urbanization and the increasing use of synthetic organic substances have serious and adverse impacts on freshwater bodies. It is a generally accepted fact that the developed countries suffer from problems of chemical discharge into the water sources mainly groundwater, while developing countries face problems of agricultural run-off in water sources. Polluted water like chemicals in drinking water causes problem to health and leads to water-borne diseases which can be prevented by taking measures can be taken even at the household level. Water-borne diseases are infectious diseases spread primarily through contaminated water. Though these diseases are spread either directly or through flies or filth, water is the chief medium for spread of these diseases and hence they are termed as water-borne diseases. Most intestinal (enteric) diseases are infectious and are transmitted through faecal waste. Pathogens – which include virus, bacteria, protozoa, and parasitic worms – are disease-producing agents found in the faeces of infected persons. These diseases are more prevalent in areas with poor sanitary conditions. These pathogens travel through water sources and interfuses directly through persons handling food and water. Since these diseases are highly infectious, extreme care and hygiene should be maintained by people looking after an infected patient. Hepatitis, cholera, dysentery, and typhoid are the more common water-borne diseases that affect large populations in the tropical regions. Previously the study area highly affected by water-borne diseases and fluoride contamination but after implementation of the model it decreases in Ghutlia village. The decreasing Pattern Shows in below shown Figure 10.13. Shows the monthly decrease of fluoride content due to recharging Impact of the model.

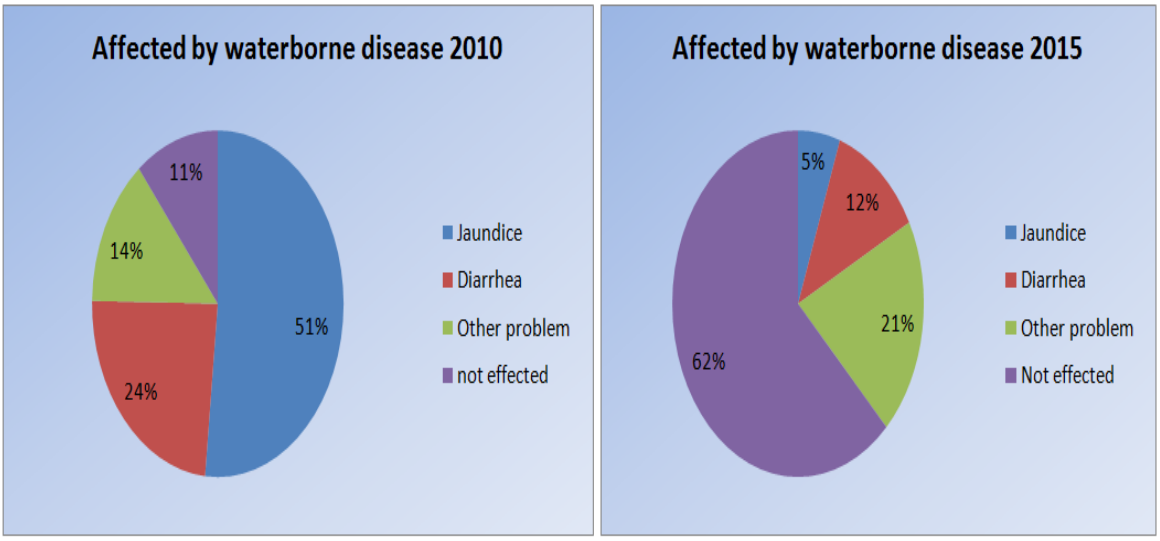


Figure 10.12 : Effect of water-borne disease

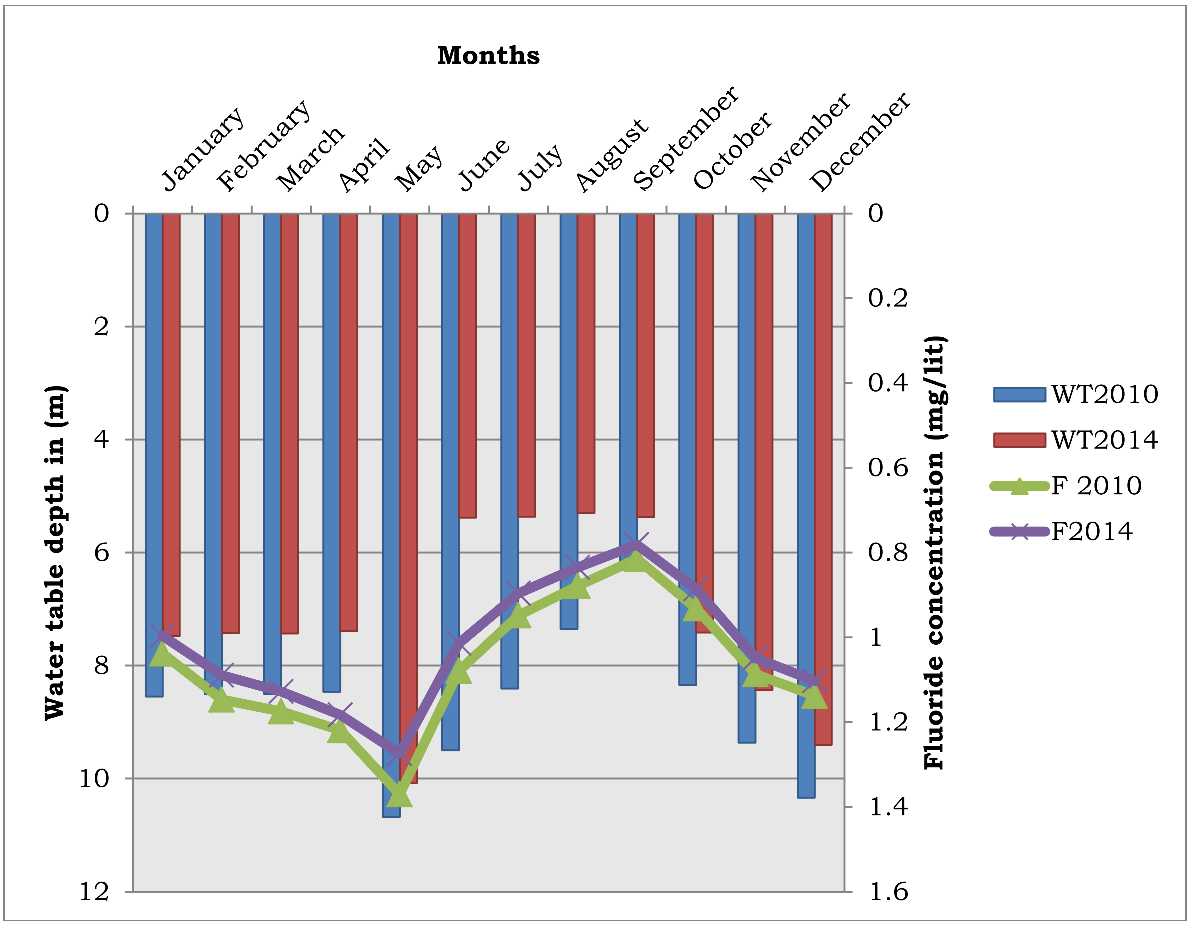


Figure 10.13 : Variation of fluoride concentration pre and post construction

10.6 Impact analysis on numerical model

In FEMWATER Conceptual modeling several varible data has been inputed and extracted. The data progessing and simulation of the model is quite hard to solve for asscessing different parameters which may not be available for all study areas. The Data processing is gone through different steps.

Output parameters:

Nodal Velocity: After FEMWATER Simulation it gives nodal velocity, pressure head, total head, moisture content data all over the study areas.In the model nodal Velocity is distributed from 0.0003 m/day to .0026 m/day. For downwards movement of the water Figure 10.15shows the velocity decreases when the depth increases, where the occurrence of nodal velocity range between 0 .0005 m/day to 0.001 m/day takes place.

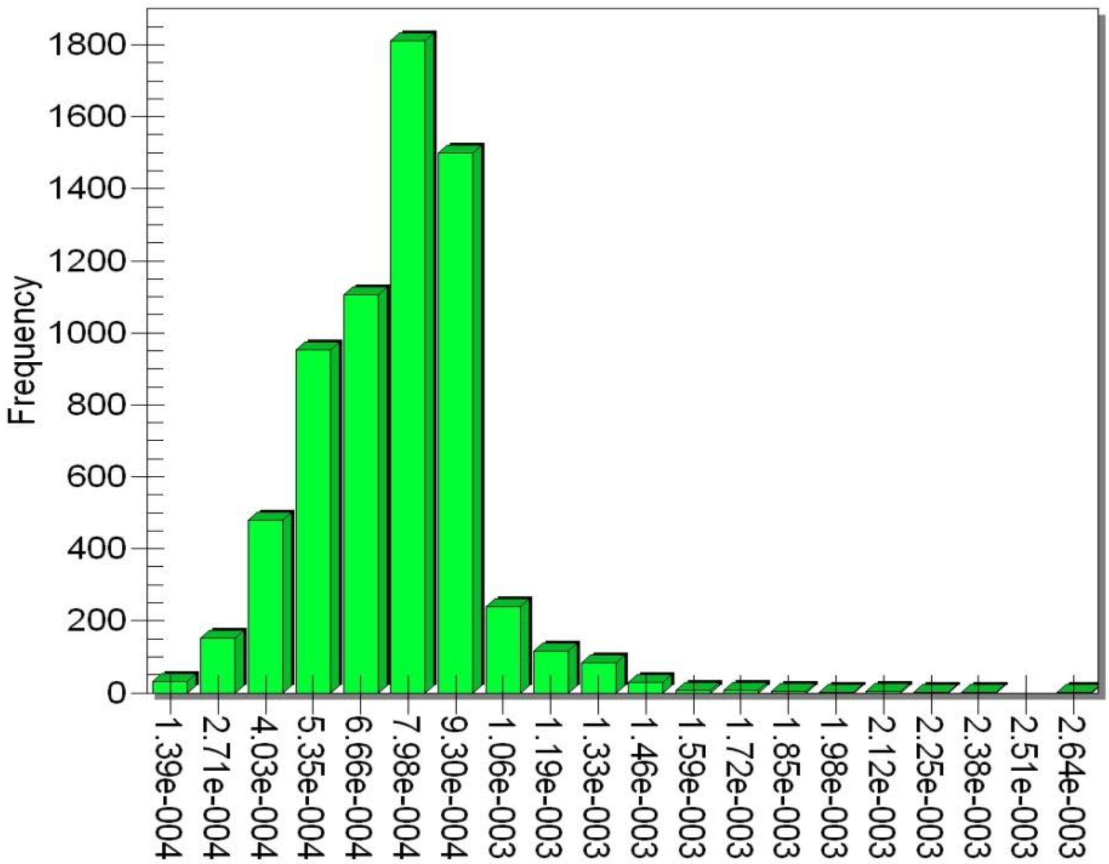


Figure 10.14 : Frequency distribution of nodal velocity (m/day)

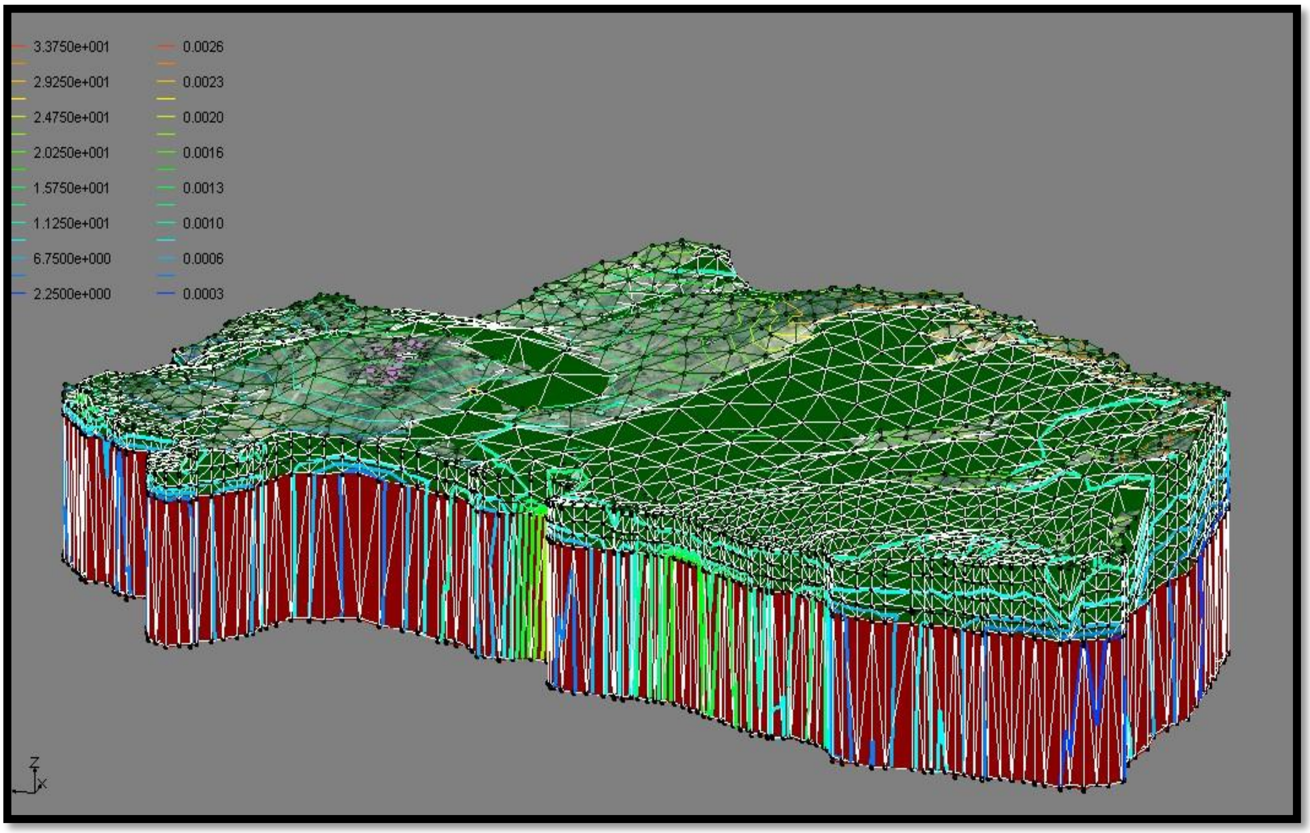


Figure 10.15 : Nodal velocity distribution of model of Ghutlia village

Pressure Head: The main purpose of FEMWATER modelling is to find out the pressure head of every nodes of the model within boundary. Pressure head holds the data for static water table depth, aquifer depth, available water pressure under soil or subsurface etc. In this model the Pressure head value range from 44m to 119 m. Figure 10.17show the pressure head will increased linearly to depth (color Index) depending upon layer. Again if the pumping rate and soil characteristics from well will change, the pressure head will increases or decreases. Frequency distribution shows (Figure 10.16) show the occurrence of pressure head of116 m and 72 m is distributed much more areas and depth than others.

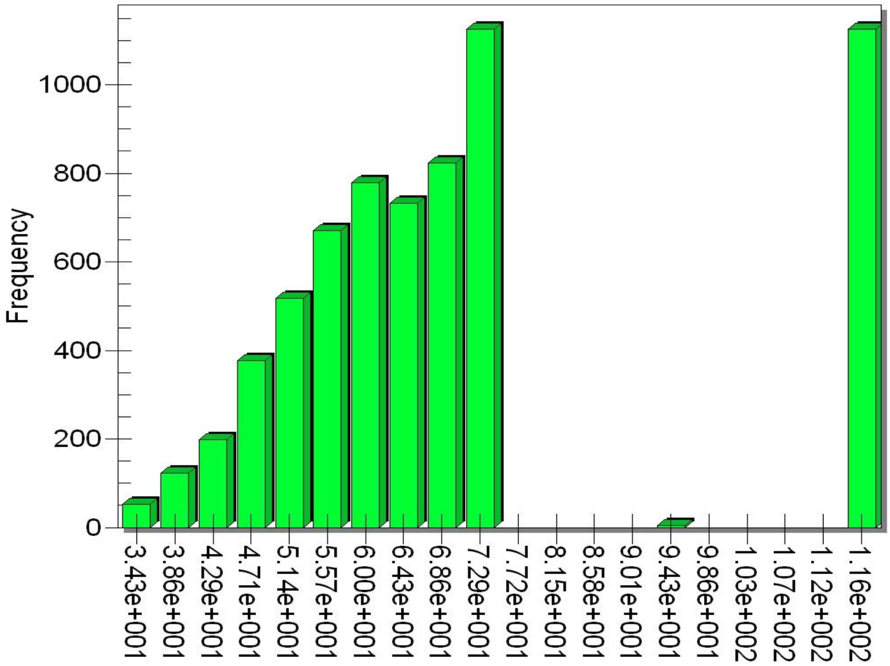


Figure 10.16 :Frequency distribution of pressure head (m)

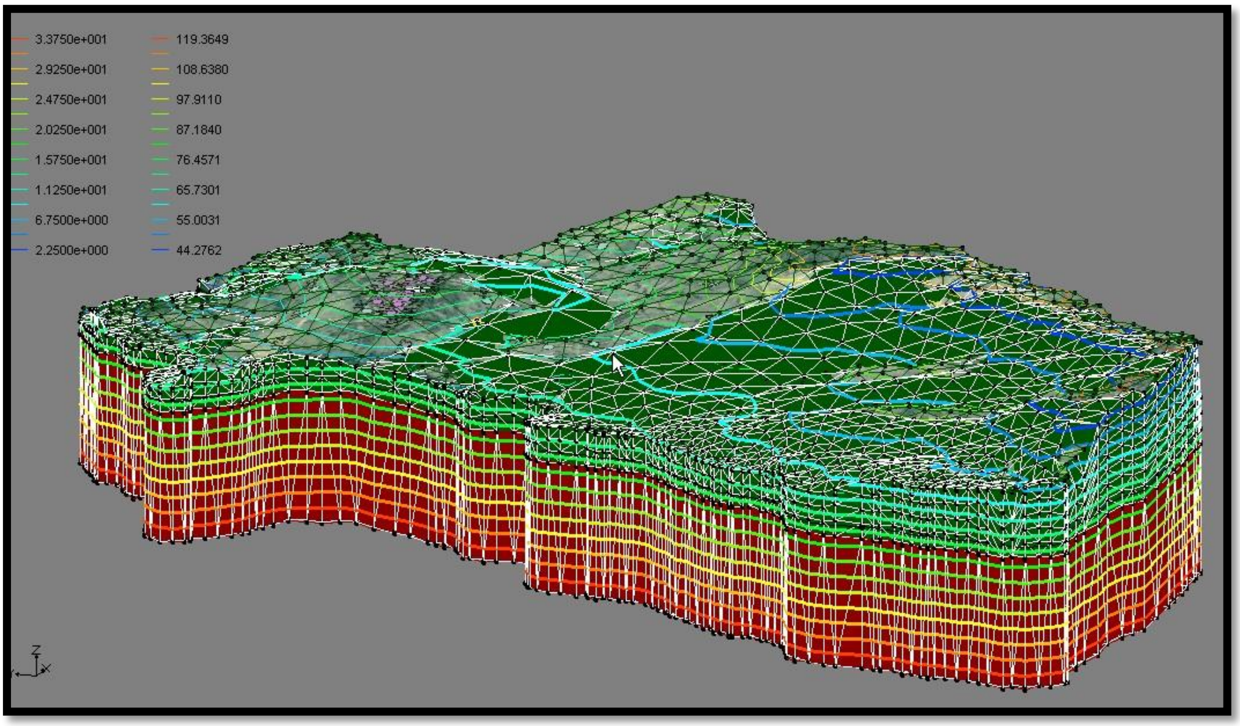


Figure 10.17: Pressure head distribution of the model of Ghutlia village

Total Head: Total head can be measured from FEMWATER modelling. Figure 10.19shows that the value of total head ranged between 74 m to 75.2 m. This result indicates that almost same values have found in calculated model for all runs. The distribution of total head is shown in the Figure 10.19

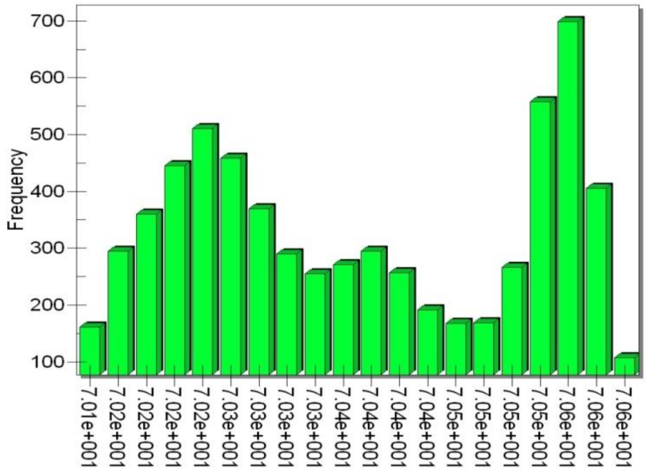


Figure 10.18 Frequency distribution of total head (m)

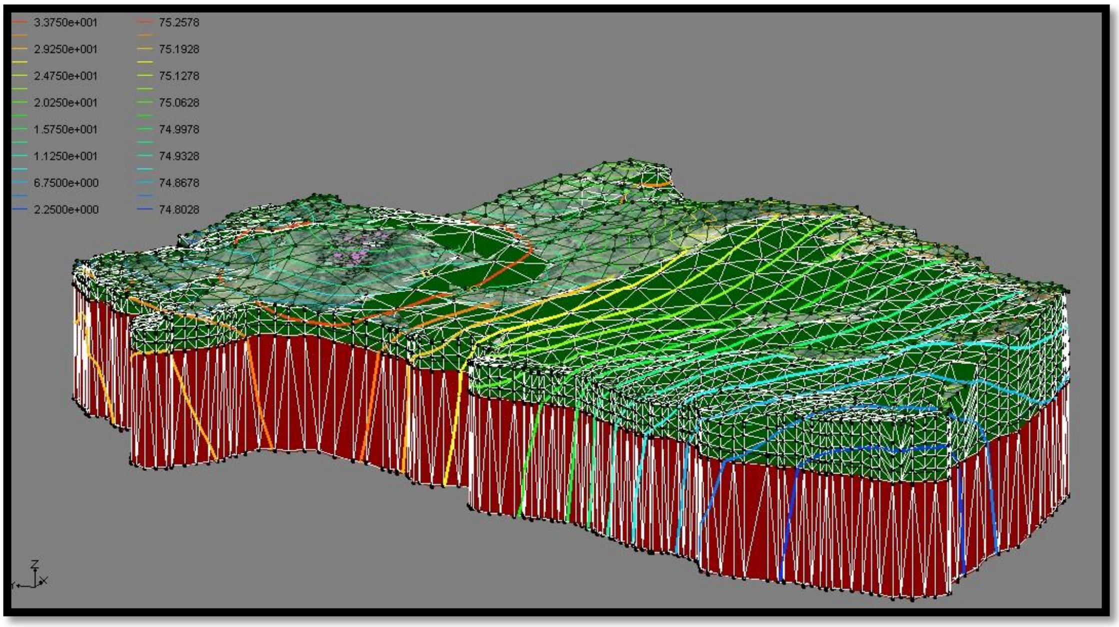


Figure 10.19 :Total head distribution of the model of Ghutlia village

10.7. Impact analysis of hydro geological investigation

Based on the facts, pertaining to site selection, design considerations and site constraints with respect to the pumping sites at river Kangsabati and Damodar, following recommendations may be drawn in order to facilitate the sustainable water supply.

- Hydro-geological survey along with the fence diagram resulting from borehole logs at all the sites show the existence of unconfined aquifer of a fairly good depth (average 6 m and above) except Ramda village on the bank of Kangsabati river. So, radial collector well or infiltration gallery may be established anywhere.
- Analysis of pump test data show that, for Damodar river transmissivity at different pumping site vary from 1138-3119 m^2/day with a storativity variation from 0.002378-0.003899. For Kangsabati river transmissivity varies from 606-1311 m^2/day with storativity variation ranging from 0.000758-0.00164.
- For Beldi, Kudulng, Joradihi, at Kangsabati and Narayanpur and Chinakuri Ghat at Damodar river, Radial collector wells are designed with a maximum safe yield ranging from 7.68-14.36 MLD at different sites, with a maximum allowable drawdown of 2.5 m for Kangsabati and 3.5 m for Damodar River (Figure 10.20 and Figure 10.21). Similarly the Infiltration Gallery are designed with a maximum yield ranging from 0.07 – 0.528 MLD at different sites having different lengths of gallery mentioned in Table 10.4.

- Another Infiltration Gallery has also been designed at Goradag under Arsha Block having capacity 0.253 MLD. This site has been selected as because the river width as well as the sectional discharge is found to be higher compared to the other sites At Deuli in Saturi block, Damodar River one Infiltration gallery may be designed with a maximum safe yield of 1.2 MLD (each capacity 0.53 MLD)with drawdown limitation of 3.5 m,may be extended to another parallelgallery at 0.26 m apart if laid across the river.

- In case of Kangsabati River, the river width is very less compared to Damodar River. Keeping in view, the Infiltration Gallery may not be preferred as because the supply water would not meet up the future demand so far the population pressure of the adjoining area of the catchment is concerned.
- Also from lithological data, the effective sand depth of Damodar River is comparatively higher (i.e. 10-15 m) than the depth of Kangsabati River (average 6 m).
- Analysis of pump test data show that there is good value of transmissivity (T) and storativity (S) of the aquifers at all five sites (three from Kangsabati catchment and two from Damodar catchment) as depicted in Table10.3. Hence these sites have been selected for design of radial collector well in a single layer as well as infiltration gallery.
- According to Table 10.2 and 10.3, the Method I based design approach seems to be much less conservative suggesting more yield as against that the same based on Method II (image well concept). The expected conservative safe yield have been delineated in Table 10.2 and 10.3.
- Infiltration galleries may be designed with a maximum safe yield ranging between 0.07 MLD and 0.53 MLD for both the rivers mentioned in Table 10.10 showing different lengths of gallery.
- It may also be stated that the total volume of recharge may be available for the range between 449.4 ML and 775.2 ML per year for both the catchments by considering 2% of net available surface runoff estimated by HEC-HMS mentioned in Table 10.4. Thus it may be concluded that the recommended safe withdrawals (Fig. 10.9 and Fig. 10.10) are ranging between 7 and 38% which is much below the average percentage of 40%, as mentioned by Ponce, 2007. Hence the recommended values are well within the sustainable yield for both the cases.

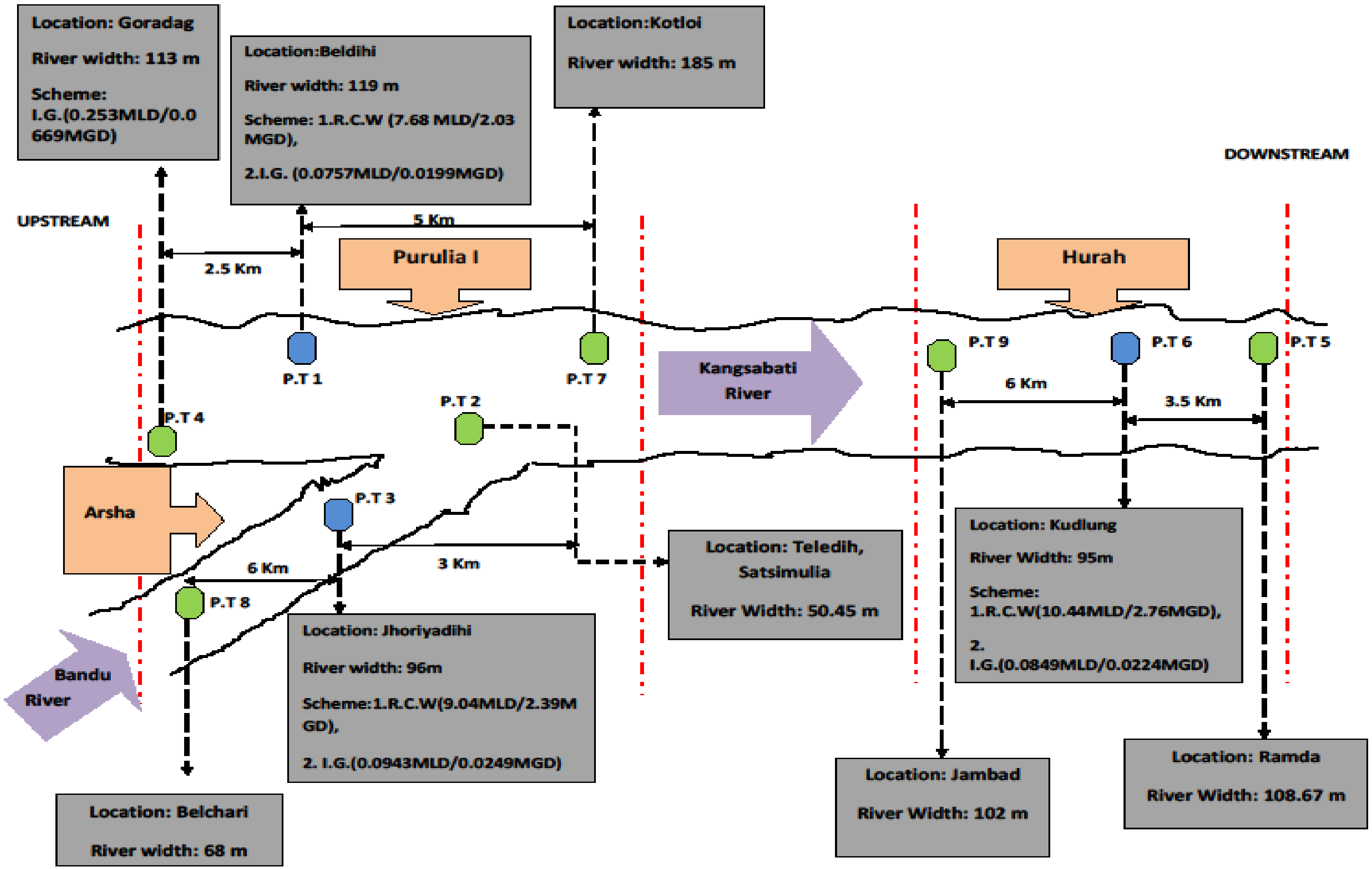


Figure 10.20 : Schematic diagram showing different locations of radial collector well in Kangsabati River

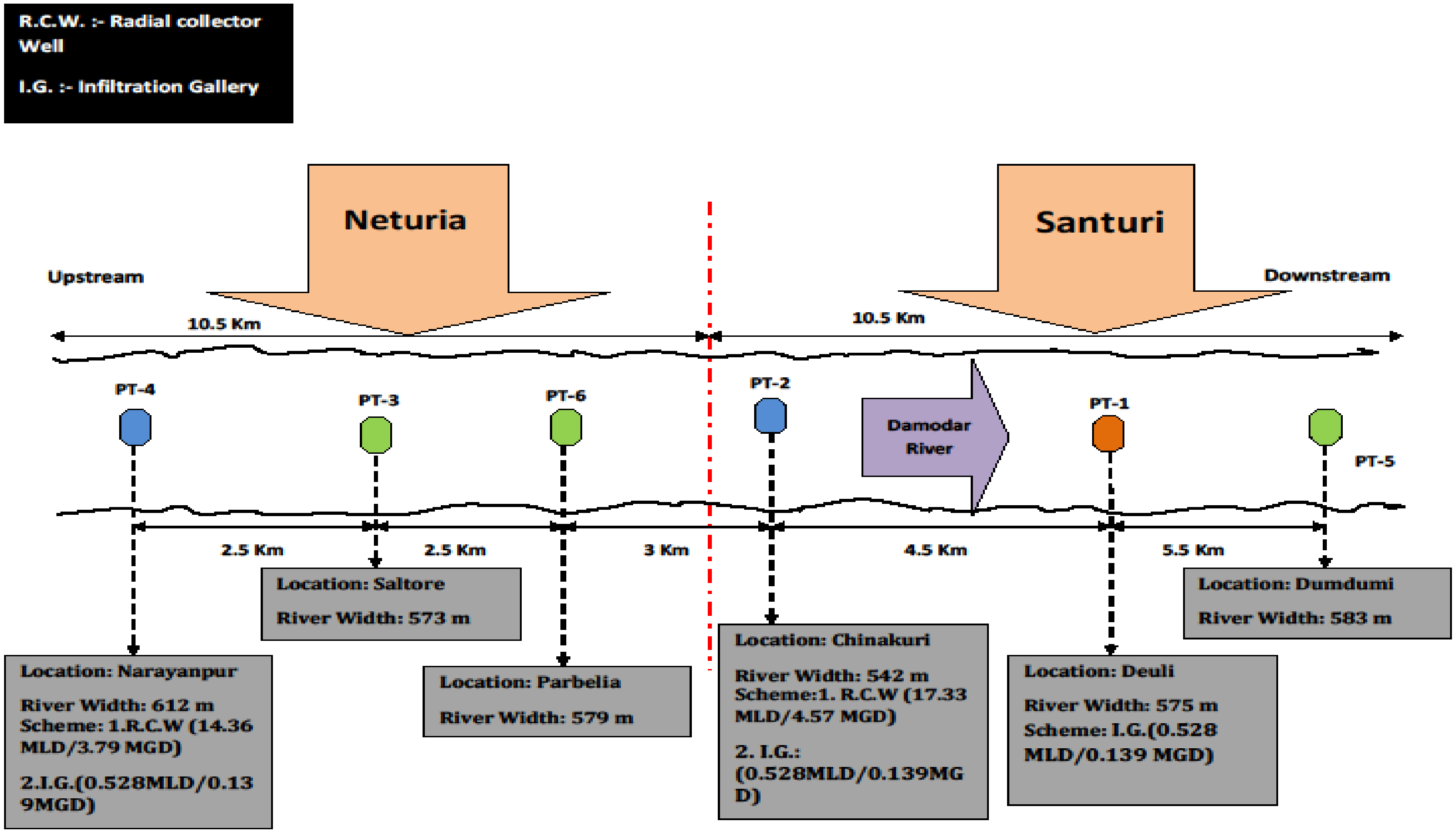


Figure 10.21 : Schematic diagram for different locations of radial collector well and Infiltration gallery in Damodar River

Table 10.2: Design summary of radial collector well for Kangsabati River

Method	Estimated yield (MLD)	No. of laterals	Diameter (Length) of each lateral, mm (m)	Central shaft of radial collector well (RCW)				Slot size		Slot type	Screen materials
				Inner diameter (m)	Outer diameter (m)	Depth (m) b.g.l.	Maximum drawdown (m)	Width (mm)	Length (cm)		
PTW-1: Place-Beldihi, Purulia-I block											
Model I	8.0	5	300 (30)	5	6	5.3	2.5	2	5	V type continuous	Stainless Steel/ Brass
Model II Image theory)	0.8	5	300 (30)	5	6	5.3	2.5	2	5		
PTW-3: Place-Jhoriyadihi, Arsha block											
Model I	9.0	5	300 (30)	5	6	5.3	2.5	2	5	V type continuous	Stainless Steel/ Brass
Model II	1.2	5	300 (30)	5	6	5.3	2.5	2	5		
PTW-6: Place-Kudlung, Hura-I block											
Model I	10.0	6	300 (30)	5	6	5.3	2.5	2	5	V type continuous	Stainless Steel/ Brass
Model II	1.0	6	300 (30)	5	6	5.3	2.5	2	5		

Table 10.3: Design summary of radial collector well and Infiltration Gallery for Damodar River

Method	Estimated yield (MLD)	No. of laterals	Diameter (Length) of each lateral, mm (m)	Central shaft of radial collector well (RCW)				Slot size		Slot type	Screen materials
				Inner diameter (m)	Outer diameter (m)	Depth (m) b.g.l.	Maximum drawdown (m)	Width (mm)	Length (cm)		
PTW-2: Place-Chinakuri, Santuri block											
Model I	17.0	8	300 (30)	5	6	9.46	3.5	4	50	V type continuous	Stainless Steel/Brass
Model II Image theory)	3.0		300 (30)	5	6	9.46	3.5	4	50		
PTW-4: Place-Narayanpuri, Neturia block											
Model I	14	8	300 (30)	5	6	9.46	3.5	4	50	V type continuous	Stainless Steel/Brass
Model II	3.0	8	300 (30)	5	6	9.46	3.5	4	50		

Design for IG: PTW-1: Place-Deuli, Santuri block (Ref. Fig. 10.10)

Estimated yield (MLD	Length of Gallery (m)	Maximum drawdown (m)	No. of Gallery	Spacing between two gallery (m)	Manholes	Depth of river bed (m)	Width of the river (m)
1.2	161	3.5	2	0.26	Reqd.	6.0	559

Table 10.4 : Design summary of Infiltration Gallery for Kangsabati & Damodar River

Design for IG: PTW-1: Place-Beldi, Purulia-I block (Kangsabati River) (Ref. Fig. 10.9)							
Estimated yield (MLD	**Length of Gallery (m)**	**Maximum drawdown (m)**	**No. of Gallery**	**Spacing between two gallery (m)**	**Manholes**	**Depth of river bed (m)**	**Width of the river (m)**
0.08	119	2.5	1	-	Reqd.	6.1	119
Design for IG: PTW-3: Place-Jhoyriadihi, Arsha block (Kangsabati River)(Ref. Fig. 10.9)							
0.09	96	2.5	1	-	Reqd.	6.1	96
Design for IG: PTW-4: Place-Goradag, Arsha block (Kangsabati River)(Ref. Fig. 10.9)							
0.25	113	2.5	1	-	Reqd.	6.0	113
Design for IG: PTW-6: Place-Kudlung, Hura block (Kangsabati River)(Ref. Fig. 10.9)							
0.08	95	2.5	1	-	Reqd.	5.91	95
Design for IG: PTW-2: Place-Chinakuri, Saturi block (Damodar River)(Ref. Fig. 10.10)							
0.53	264	3.5	1	-	Reqd.	7.6	466
Design for IG: PTW-4: Place-Narayanpur, Neturia block (Damodar River) (Ref. Fig. 10.10)							
0.53	260	3.5	1	-	Reqd.	9.4	540

CHAPTER-11

CONCLUSION AND RECOMMENDATION

11. Conclusion & Recommendation

11.1. Conclusion

- Any excessive use of natural resource for development will affect the environment and will be empowered to a critical situation where the ensuing damage will be hampered the fruit of development. Groundwater is precious natural resource, and extremely needed for human survival, but it also need preserve for sustainable use in the future. In semi-arid and water scare region like Purulia district extraction of groundwater is more and due to topography of undulating rocky terrain runoff is more than recharge Therefore the present study was carried out with the objectives to plan and design composite model with cost-effective groundwater recharging structures and to evaluate their economic feasibility for the semi-arid regions in India. Four types of water recharging structures, i.e., Check Dam with Recharge Multipurpose Collector well with Infiltration gallery has been considered in this study. The detailed cost analysis of proposed composite model of cost effective technology revealed that the integration of different groundwater recharging structure using local material is more cost effective rather than installation of single structure. It is also very much efficient than single recharging structure. The cost benefit analysis also stated that above designed structures are feasible because they have BCR value more than 1.

- The present study has formulated method for proper selection and evaluation of rain water harvesting site selection for recharging through artificial recharge. The criteria compare the input and output value recharge and withdrawal. Also the groundwater quality of studied wells and the source water is not of graet concern, the proposed criteria can efficiently suitability of sites for artificial recharge and recuperation. As the criteria is NSWRC since normalized can cover all arenas (local to regional) and both for active and passive techniques of artificial recharge.

- The structures constructed and designed in this study could be implemented in other semi-arid regions of India having almost the same climatic, terrain and socio-economic conditions.

- In this research study, attempt has been made to evaluate the effect of groundwater structure on recharge in a semi-arid area of West Bengal. The surface water groundwater interaction and potential recharge efficiency studied thoroughly to determine the actual groundwater recharge within the catchment area. The water retaining structures mainly check dam and sub-surface dams towards recharging through rainwater has also been estimated to assess the potential efficiency. Using the water level fluctuation as indicator or recharge function developed for each constructed structure. These functions calculated the potential recharge reasonably well.

- An spatial relationship developed between the rainfall, recharge, withdrawal and geographical location. Area and other attribute data's are collected from field survey and GIS application with GPS and total station survey. The analysis revealed that a minimum of 111.5 mm cumulative rainfall is required to incorporated 1 mm of potential recharge in the catchment area. It was also studied that the potential recharge from a groundwater recharging structure can avail at a lower rainfall than the available average rainfall in the study area.

- Findings of the study are very much helpful in designing future individual groundwater recharging structure and integrated models more efficiently and sustainably recharging ground water. In this study another alternative method i.e. chloride mass balance (CMB) method was used to evaluate its efficacy in determining the annual potential recharge in comparison to Water Table Fluctuation method. Combined measurements in both the methods were found to justified and comparable results for calibration and estimation of actual groundwater recharge. In this study it was observed that there is a inter relationship between the changes in chloride concentration and the water table fluctuations . A comparable relationship was found to fit the data reasonably well. Though more precise data base is needed for fine tuning of reliable equation development in particular region.

- For assessing spatial and temporal variations of flow fields due to interdependency of surface water and groundwater, a hypothetical aquifer-stream water interaction system has been modeled using GMS software (Groundwater Modelling System). Some assumptions have been considered such as steady state condition, homogeneous soil to develop a model to quantify flow rate of groundwater well fields due to changes of river stage.

- The governing equations that describe flow in unconfined aquifers have been used to determine the aquifer characteristics. The important thing is that in case of unconfined aquifer it gives calculated transmissivity, hydraulic conductivity, specific yield etc. Since it is an unconfined or semi-confined zone, so the aquifer is usually late time response and the time value is considered by making a best fit line from the second cycle. The transmissivity and specific yield has been calculated based on empirical formulae. The confined storativity is calculated by making a best fit line from the first cycle referred as early response time.

- Lithological survey was conducted on the Damodar River bank in order to ascertain the soil characteristics. The soil samples are mainly coarse, medium and fine sand in nature. The maximum drilling depths are found at 10 m and 13 m for Santuri and Neturia Block respectively. The pumping test sites were chosen depending on the maximum depth of porous medium.

- Step drawdown test was performed at variable discharges of 25, 30 and 44 m3/hr respectively. Continuous monitoring of the piezometric levels was performed during the pumping tests at the observation wells. Analysis of drawdown values using Neuman's straight line method generated the aquifer parameters such as transmissivity and storativity which are found to be ranged from 2082 – 3983 m^2/d and 0.043 – 0.175 respectively. The maximum drawdown due to pumping has been observed within the ranged between 0.023 m and 0.09 m of three observation wells.

- The analysis of drawdown data using Aquifer Test Pro v2013.1 software provided a significant similarity with the Neuman's straight-line method in terms of calculating transmissivity and storage co-efficient. There is a small variation of results in these two methods, which is due to the reason that the Aquifer Test Pro software follows Neuman's curve-fitting method and the theoretical analysis employs the Neuman's straight-line method to analyse the drawdown data in order to determine the hydraulic parameters of the aquifer.
- Hydraulic conductivity obtained from the software simulation are ranged between of 219 and 603 m/day.
- It is known that recharge and discharge in the groundwater system has been divided into three components namely Prsitine, Developed and Depleted zone. Here it has been try to build up a model to represent hydraulic head variations and also how the surface water could affect on the groundwater system due to changes in the river stage so that in near future it will help to make water security plan for sustainable withdrawal and also whether there could be any further development without disturbing unconfined or semi-confined zone in the peripheral region so far the groundwater system concerned.

- Modelling an aquifer system accurately involves a large amount of data collection and/or data interpretation. This study is hypothetical, though the site chosen is actual one. The objective is not to build accurate model but just to understand how surface water could affect groundwater flow fields due to changes in river stage.

11.2. Recommendation and Future Scope of Work

- Although the present study has been carried out in hard rocks, the purpose methodology also can be applied on alluvial zone. As per research out comes NSWRC may be obtained at places due to high rainfall but low recharge-coefficient, high undulating terrain. NSWRC and there is ample scope of research on pre-construction site evaluation for groundwater recharge. Efficiency evaluation and proper calibration need validation through pilot project field based experiments, at different geologic and hydro geologic which was beyond the arena of this study future scope of work.

- Feasibility study for Integration of River Bank Filtration (RBF) with the integrated model can show good results in future for water scare region.

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Appendix-A

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ESTIMATING GROUNDWATER VOLUMETRIC MASS BALANCE WITH HYDRAULIC HEAD USING GROUNDWATER MODELING SYSTEM IN TRIPURA, INDIA

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AUTHORS' CONTRIBUTIONS

This work was carried out in collaboration between all authors. Author PKR designed the study, wrote the protocol and interpreted the data and produced the initial draft. Author JD anchored the field study, gathered the initial data and performed preliminary data analysis. Authors SH, GB and MP managed the literature searches including preparation of maps and models runs. All authors read and approved the final manuscript.

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ABSTRACT

In this study, a three-dimensional finite-difference groundwater flow model has been developed to investigate the variety of hydro-geological conditions and simulating volumetric flow budget under different stress conditions in the aquifer of Agartala and Khowai city under Agartala and Khowai district respectively in the state of Tripura. MODFLOW package using groundwater Modeling System (GMS) was applied for this study to find out the volumetric flow budget and hydraulic heads for groundwater under steady state analysis method. Lithologs (soil strata), aquifer parameters during pumping test, hydro-meteorological data, catchment characteristics etc. were used to calibrate and validate the conceptual model. The objective of this paper is to develop conceptual models run by GMS MODFLOW for estimating the groundwater volumetric mass balance and hydraulic head at different point with varied withdrawal condition of three seasons. The results conclude the hydraulic heads decrease against discharge wells and it increases towards recharge wells and this model will help to estimate the groundwater volume for water budgeting.

Keywords: Aquifer; river head; GMS; discharge well; recharge well; MODFLOW.

1. INTRODUCTION

Many studies have been carried out to estimate hydraulic parameters for aquifers and to model the effect of stream flow on groundwater flow fields locally and worldwide. To build up a conceptual model and to calculate the aquifer parameters, different types of literature review has been done, stream-aquifer connectedness, GIS technology, MODFLOW conceptual model approach etc. are studied here. Some case studies are being described given below to overview an idea about conceptual Modeling and aquifer parameters.

Lautz and Siegel [1] used a three-dimensional MODFLOW model, paired with MT3D, to simulate hyporheic zones (mixing of shallow groundwater and surface water) both advective transport and

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Integrated, Sustainable and Eco-friendly Surface and Groundwater Management in an Arsenic affected Rural Area of West Bengal, India

P.K.Roy, S. Pal, G. Banerjee, A. Majumder, R. Chakraborti, M. Banik, A. Mazumder

Abstract— **The aim of the paper is to present cost-effective solutions, arsenic removal treatment system from arsenic prone groundwater source and ecological surface water treatment system as an alternative source of arsenic free water were developed for rural areas at Jyot Sujan village, Murshidabad District, West Bengal, India. 92-94% turbidity removal is possible from HRF 1 and 2, 99.22% turbidity removal is also possible from SSF1 and 2. And pH and bacteria are reducing from the combined surface water treatment system. A dual treatment method for groundwater comprising of oxidation-coagulation-filtration and adsorption by activated alumina is proven to be more economic having more capacity and superior reliability in terms of water quality prescribed by IS 10500, 2012 as compared to other arsenic removal processes using various other media.**

Keywords—ARU, cost-effective, HRF, integrated, SSF, sustainable

I. Introduction

In Department of Science and Technology (DST) sponsored “Energy Efficient Community based Water and Wastewater Treatment Systems for Deployment of India” (ECO-India) project, Indian-European Consortia, were responded to the challenge by delivering substantial innovation using conventional and leading Indian European novel technologies, together with advanced research[1]. In order to deploy reliable and cost-effective solutions, arsenic removal treatment system from arsenic prone groundwater source and ecological surface water treatment system as an alternative source of arsenic free water were developed for rural areas at Jyot Sujan village, Murshidabad District, West Bengal, India [2].

II. Material and Method

A. *Study Area*

The village Jyot Sujan were located at Dahapara GP, Jiagunge-Murshidabad Block, Murshidabad District, W.B having latitude 24˚9’11.23’’, longitude 88˚15’24.06’’. The JyotSujan settlement has a population of approx. 2000 out of which approx. 1500 are above 18 years belonging to about 250 households. There are 200 pupils studying in primary school, and 50 children in ICDS (integrated child development service) JyotSujan has a community Mosque where about 100 people offer prayers.

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The Malpara settlement has a population of approx. 400. The main occupation of both communities is farming, 40% of the surveyed people are farmers, and women are mostly house wife amounts to 45 % of the surveyed population. Apart from farmers, there are some people doing labour jobs on daily basis, survey shows about 14 % are labours. Fishery and animal husbandry are also a source of income for the villagers. The average monthly income for each household ranges from 1500 to 4500 INR.

The entire village was categorized as North (N), West (W), South (S), South East (SE), South West (SW) for site selection of Integrated scheme. Based on the preliminary results from evaluation of Socio economic survey, 98% of the respondents are interested in being covered integrated scheme and several reasons indicate the selection for location of the Integrated scheme in favour of Malpara part of the village for maximum monetory contribution toward infrastructure maintained and evident water transport and sanitation problems. The next possible location for the integrated scheme could be SE and/or SW part of the village for higher open defecation practices and water transport problems. In regards of the catchment area protection from contamination through open defecation Malpara and the south part of Jyot Sujan also provide the highest risk potential as the local population chooses locations in the catchment area of the pond for their defecation.

The general needs identified by the interview partners are provision of safe drinking water, improvement of sanitation facilities and increase of awareness on hygienic practices regarding water contamination. Further solid waste and waste water management would be welcome. Finally all important locations were agreed upon in the water committee.

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ORIGINAL PAPER

Study of impact on surface water and groundwater around flow fields due to changes in river stage using groundwater modeling system

P. K. Roy · S. S. Roy · A. Giri · G. Banerjee · A. Majumder · A. Mazumdar

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Abstract Excessive ground water withdrawal can lead to ground water depletion, and this may have serious social and economic consequences. Attempts to limit groundwater pumping have been commonly based on the concept of safe yield. In this study, in order to evaluate safe yield, short- and long-duration pumping tests have been performed at four different sites (Purulia District, India) on Damodar River bed. The lithological survey was carried out along the river bed at regular intervals from 500 m to 10.5 km, covering a total stretch of 10-km up-stream and 0.5-km down-stream, to determine the soil profile and other soil characteristics. The points, where the maximum depth of porous medium was obtained, were selected for carrying out the pumping yield tests. The appropriate hydrogeologic approaches were employed to estimate the values of aquifer parameters, viz., specific yield (S_y), transmissivity(T), and hydraulic conductivity (K) for the aforesaid sites under study, which were found to be in the range of 0.043–0.175, 2082–3983, and 219–603 m/day, respectively. A hypothetical aquifer-stream water interaction system has been developed using GMS software (groundwater modeling system) not only to represent the hydraulic head fluctuations in the groundwater system around well fields but also to quantify the flow rate due to change in river stage, recharge due to rainfall, and also to determine how the surface water could affect groundwater due to pumping.

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Keywords Safe yield · Sustainable yield · Lithological survey · Hydrogeologic approaches · GMS

Introduction

The most basic interpretation of surface water–groundwater interaction can be described by the direction of flux between a surface water body and the underlying aquifer. Stream reaches may be defined as losing, gaining, or parallel-flow depending on the elevation difference between the stage in the stream and the head in the aquifer. It should be noted that many in-stream processes are affected by these interactions such as ecological and geochemical processes (Valerio 2008). In this study, a conjunctive-management model is developed for a hypothetical stream–aquifer system which has been used to assess the effect of interannual hydrologic variability on minimum monthly streamflow requirements and to evaluate trade-offs between groundwater withdrawals and streamflow depletions for alluvial-valley, stream–aquifer systems (Barlow et al. 2003). Groundwater and surface water are not isolated components of the hydrologic system, but instead interact in a variety of physiographic and climatic landscapes. Thus, development or contamination of one commonly affects the other. Therefore, understanding of the basic principles of interactions between groundwater and surface water is needed (Sophocleous 2002).

Recently, the emphasis has shifted to sustainable yield (Alley and Leake 2004; Maimone 2004; Seward et al. 2006). Sustainable yield reserves a fraction of safe yield for the benefit of the surface waters. Bevan et al. (2005) discussed about an aquifer test involving 7 days of pumping at a constant discharge rate and 5 days of recovery was performed in the unconfined aquifer at Canadian Forces Base

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ORIGINAL PAPER

Removal of arsenic from drinking water using dual treatment process

Pankaj Kumar Roy · Arunabha Majumder · Gourab Banerjee · Malabika Biswas Roy · Somnath Pal · Asis Mazumdar

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Abstract This paper focuses on determining an efficient and simple method to remove arsenic from groundwater. Arsenic is a naturally occurring element widely distributed in the earth's crust. Arsenic is very toxic when found in large quantities in drinking water. This report documents the selected treatment method and laboratory experimentation of arsenic removal from drinking water in small water delivery systems and domestic water systems. The objective is to expand upon research of new and existing arsenic removal technologies or promote a new, alternative process. Several treatment technologies have been considered to perform this function, but cost and reliability concerns prompted the decision to analyze small-scale, community-based filtration units, specifically. Based upon initial test data, the use of dual treatment method comprising of oxidation-coagulation-filtration and adsorption by activated alumina has proven to be more economic having more capacity and superior reliability as compared to other arsenic removal processes using various other media.

Keywords AAS · Arsenic · Adsorption · Co-precipitation · Filtration

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Introduction

Arsenic is a natural element mostly found in the earth crust in both of its oxidation state which are trivalent arsenite (As III) and pentavalent arsenate (As V). Arsenic in both the forms is poisonous in nature. Arsenic contamination of groundwater has led to endemic condition of arsenic poisoning in Eastern India, Bangladesh and neighbouring countries. Chronic arsenic pollution is now recognized as a worldwide problem, with 21 countries experiencing arsenic groundwater contamination. It is a particularly important issue in rural areas of developing countries, where untreated groundwater is generally the preferred drinking source as piped water supply schemes are not in place. Technologies to remove or mitigate arsenic contamination of groundwater include pre-oxidation, adsorption, biological removal, deep tube wells or the substitution of the groundwater source by surface waters not containing arsenic. Whilst removal technologies such as these may be effective in stable conditions (for example, at a laboratory scale), their effectiveness in real-world circumstances is limited (Hossain et al. 2006) and need to be assessed further to inform policy making (Pearson et al. 2011). The largest population currently at risk is in Bangladesh, followed by West Bengal in India where groundwater concentrations frequently exceed the WHO guidelines (0.01 mg/L) more than tenfold (Rahman et al. 2002; Sarkar et al. 2008), thus also exceeding the Indian drinking water standard IS 10,500 which requires 0.05 mg/L. In India a discussion has started to lower the level to 0.01 mg/L similar as in the United States, where the Environmental Protection Agency has recently announced to lower the maximum contamination level for arsenic in drinking water from 0.05 to 0.01 mg/L (US-EPA 2002).

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Research Article

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Study of impact on developed groundwater system around flow fields by using numerical approach in Purulia District

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ABSTRACT

Seasonal variations of river stage, infiltration through rainfall, influence of pumping or recharging wells can lead to change of groundwater flow fields in an aquifer stream flow system. The aquifer stream water interaction system depends on several factors such as location, geometry elevation and physical setting and other inherent properties. In this present work collected data such as location by latitude and longitude, slope of the field, pumping rate etc. are used to develop the programming. The outputs of the programming are further developed by MATLAB to get the contour and 3D image of the flow field. The ground water flow field of that bounded area is to be changed due to impact of water withdrawal during pumping and slope variation. In this model recharge, soil properties such as storativity, hydraulic conductivity etc. is not considered. The contour map and 3D view model have been developed using numerical approach to show the variation of ground water flow field under the steady state and homogeneous condition.

Keywords: Finite Difference Method, Contour view, 3D view, groundwater system

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INTRODUCTION

Ground water is a precious and the most widely distributed resource of the earth and unlike any other mineral resource, it gets its annual replenishment from the meteoric precipitation. The pervasive and seemingly abundant supply of groundwater has led to its indiscriminate and sometimes excessive use. However, this use can have diverse and often wide ranging effects on the local and regional hydrology and ecology. These interdisciplinary aspects of groundwater utilization have brought into question the concept of safe yield, defined as the maintenance of a long-term balance between the amount of withdrawal and the amount of recharge [1]. Thus, the issue of groundwater sustainability has arisen [2].

The management of the groundwater resources is a challenging task worldwide against the backdrop of the growing water demand for industrial, agricultural, and domestic uses and shrinking resources. Singh [3] simulate modeling applications used for the management of groundwater resources. Groundwater resources are also essential to underpin the daily life of communities in remote regions. Hydro geological studies in these areas are complex, largely due to the absence of long water table records and to the difficulties involved in accessing some relevant locations. Rodriguez et.al [4] used numerical groundwater models to explore groundwater resources in desert areas. Modeling results suggest that the aquifer has been depleted by almost 30% due to groundwater extractions in less than 50 years. The average drawdown was found to be in the order of three meters. This corresponds to a total depletion in the order of 20–60 Mm^3.

The use of groundwater flow models is prevalent in the field of environmental hydrogeology. Yang et al [5] developed models have to investigate a wide variety of hydrogeological conditions. Recently, groundwater models have been applied to predict the fate and transport of contaminants for risk evaluation purposes. The model results help to identify the aquifer properties and to analyze the groundwater flow dynamics, the changes of groundwater

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DEVELOPMENT AND HYDRAULIC ANALYSIS OF A PROPOSED DRINKING WATER DISTRIBUTION NETWORK USING WATERGEMS AND GIS

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ABSTRACT

Safe and potable drinking water is a prerequisite for sustainable living. After achieving much success in distributing drinking water in cities and towns in India, rural areas have become a major concern by the state governments. In the present paper a sustainable, innovative, environmental friendly and cost effective water supply system for arsenic-affected water-stressed regions in Murshidabad District, West Bengal in India as proposed is analyzed using WaterGEMS software to study its dynamic behaviour in an Extended Period Simulation tool. The water supply network model was established using a GIS map obtained from the QuantumGIS software. The objective of this paper is to design safe drinking water networks through gravity cum pumping and hydraulic analysis of pipe networks using the WaterGEMS software for virtual-time realization of pressure head and average daily flows through the house connections and stand-posts. Hydraulic simulation has been carried out for a continuous supply of 24 hours daily to achieve the desired results with suggestive conclusions for practical implementations.

KEY WORDS : Water GEMS, GIS, Pressure head, Average daily flows, Extended period simulation.

INTRODUCTION

In any rural area, ponds and wells are used as the main sources of domestic water supply. Conventionally people carry water in cans or buckets from the ponds and wells for their day to day activities (Wagner and Lanoix, 1959). With the advent of modern technologies, it has become convenient to distribute water to every household and standposts (feeders) from the water bodies (source) nearby through pipes (distributing channels). Any type of distribution system involves: (1) the determination of storage; (2) the location and size of feeders; (3) the location and sizes of distribution pipes, valves, and hydrants; and (4) the determination of the pressure required in the system (Wagner and Lanoix, 1959). A typical water distribution system consists of network of pipes, nodes linking the pipes, storage tanks, reservoirs, pumps, additional appurtenances like valves (Mahapatra *et al.*, 2013).

For any water supply project, water distribution usually account for 40% to 70% of the capital cost. For this proper layout and design of the system is very important. The geometrical configuration of pipes, reservoirs and boosters, etc. is important for the functioning of the system. Adequate residual pressure at maximum demand depends upon the hydraulic characteristics of the system. For the present study, the average daily consumption is assumed to be 40 litres per person per day (lpcd) and the peak factor is assumed to be three (CPHEEO, 2005). The construction at the same point of time is efficient from the point of view of having to do earthen works only once.

Softwares like the LOOP, EPANET, and WaterCAD has been already used for hydraulic simulation of the water distribution system though the latest software WaterGEMS is the most advanced and powerful tool and used in the present

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Qualitative and Quantitative Assessment of Pollutional Load in River Ganga in West Bengal Using Statistical Technique

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Abstract: The Ganges, or Ganga, is a trans-boundary river of India and Bangladesh. It is the largest water resource available to India. As a result of the increasing anthropogenic activities in the Gangetic plain, Ganga water quality has deteriorated over years. Major efforts to clean the Ganga, named Ganga Action Plan (GAP), in two phases, were instituted by the Government of India. Phase-I was launched in 1985 and completed in March 2000 and Phase-II started in 1993. The main aim of GAP was the reduction of organic load on the river through interception, diversion and treatment of wastewater reaching the river, thus maintaining the biochemical oxygen demand (BOD) and dissolved oxygen (DO) levels of river within the acceptable limits. A major criticism of GAP is that the significance of river ecology has not been addressed adequately during its conception and implementation. So the water quality of the river remains a major concern. This present study aims at the assessment of the pollution load on the river in the geographical state of West Bengal. The total organic and inorganic load has been calculated. Statistical analysis is performed on the water quality of 5 specific locations using one way analysis of variance.

Key words: River Ganga · West Bengal · Pollution load · Water quality management · SPSS

INTRODUCTION

Rivers constitute the main inland water resources for all civilizations on earth. They are used by people all over the world for domestic, industrial and irrigation purposes. So, it is imperative to prevent and control the rivers pollution and to have reliable information on the quality of water for effective management [1]. Since most rivers carry off the municipal and industrial wastewater and run-off from agricultural land in their vast drainage basins, they are among the most vulnerable water bodies to pollution. The surface water quality in a region is largely determined both by the natural processes (precipitation rate, weathering processes, soil erosion) and the anthropogenic influences viz. urban, industrial and agricultural activities and increasing exploitation of water resources [2, 3]. The municipal and industrial wastewater discharge constitutes the constant polluting source, whereas, the surface run-off is a seasonal phenomenon, largely affected by climate in the basin. Seasonal variations in precipitation, surface run-off, ground water flow and water interception and abstraction have a strong effect on river discharge and subsequently on the concentration of pollutants in river water [4].

Coming to India, Ganga is the longest river basin here. The basin lies between East longitudes 73°30 and 89°0 and North latitudes of 22°30 and 31°30, covering an area of 1,086,000 sq km, extending over India, Nepal and Bangladesh. About 79% area of Ganga basin is in India. The basin covers 11 states viz., Uttarakhand, Uttar Pradesh, Madhya Pradesh, Rajasthan, Haryana, Himachal Pradesh, Chhattisgarh, Jharkhand, Bihar, West Bengal and Delhi.

It is the lifeline to millions of Indians who live along its course as they depend on it for their daily needs. Millions of Ganga devotees and lovers go to the river just to have a holy dip, Aachman (mouthful with-holy water) and absolve themselves of sins. However, the river itself is under threat from pollution that is being generated from many uncontrolled activities over the whole Ganga Basin.

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Study on Application of Conventional and Non-Conventional Methods for Defluoridation of Ground Water

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Abstract: Fluoride, as a dissolved constituent of drinking water, is perhaps the only substance providing divergent health effects on the consumer depending upon the relative proportions they are present in. While a fluoride concentration in the range of 0.8 to 1.20 mg/L is considered to be beneficial, concentration above 1.5 mg/L are reported to be harmful to the teeth and bone structure of human and animals. As excess fluoride (>1.5 mg/L) in drinking water is harmful to the human health, necessary removal of excess fluoride is required. This has led to the development of several defluoridation methods. In this backdrop several different mechanisms that is conventional and non-conventional methods have been developed for defluoridation particularly of ground water. This research work highlights a bench scale model, to experiment different defluoridation techniques such as Nalgonda Technique, Bone Charcoal and Activated Alumina adsorption process, cement, sand-cement, concrete granule filtration process and herbal coagulation flocculation adsorption process. This study deals with the working principle of the processes mentioned above and comparative study of the individual processes.

Key words: Defluoridation, bone charcoal, activated alumina, cement pallet, sand-cement pallet, concrete granule, moringa oleifera seed extract (MOE), Nalgonda technique.

Introduction

Water is one of the most essential requirement for the survival of mankind. The very existence of mankind depends on the availability of water as it is fundamental to all vital processes of value to mankind. Though the supply seems abundant, water is not a limitless resource; particularly the fresh potable water is available in very less quantity. As we know almost 70% of our earth's surface is covered with water. However, out of this 70%, 97% is ocean water which contains high percentage of chloride and salinity, hence not suitable for human usage. Out of the rest 3%, 2% is frozen in polar caps and hence cannot be put to use. Out of the 1% left only 0.35% is accessible. This small portion of the water supply consists majorly of underground reservoirs of water called groundwater, and the small amount in freshwater lakes and rivers.

Unprecedented population growth, a changing climate, rapid urbanization, expansion of infrastructure, migration, land conversion and pollution translate into changes in the fluxes, pathways and stores of water – from rapidly melting glaciers to the decline of ground water due to over-exploitation. Population density and per capita resource use have increased dramatically over the past century, and watersheds, aquifers and the associated ecosystems have undergone significant modifications that affect the vitality, quality and availability of the resource. In addition to this, water quality problems are mounting and in the process

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CASE STUDY

Variation of Water Quality Parameters with Siltation Depth for River Ichamati Along International Border with Bangladesh Using Multivariate Statistical Techniques

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Abstract River is considered as one of the main sources of freshwater all over the world. Hence analysis and maintenance of this water resource is globally considered a matter of major concern. This paper deals with the assessment of surface water quality of the Ichamati river using multivariate statistical techniques. Eight distinct surface water quality observation stations were located and samples were collected. For the samples collected statistical techniques were applied to the physico-chemical parameters and depth of siltation. In this paper cluster analysis is done to determine the relations between surface water quality and siltation depth of river Ichamati. Multiple regressions and mathematical equation modeling have been done to characterize surface water quality of Ichamati river on the basis of physico-chemical parameters. It was found that surface water quality of the downstream river was different from the water quality of the upstream. The analysis of the water quality parameters of the Ichamati river clearly indicate high pollution load on the river water which can be accounted to agricultural discharge, tidal effect and soil erosion. The results further reveal that with the increase in depth of siltation, water quality degraded.

Keywords Water quality parameter · Depth of siltation · Dendrogram · Cluster analysis · Multiple Linear Regressions

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Introduction

River Ichamati traverses a course of about 216 km and finally discharges into the river Kalindi at Hasnabad in the district of North 24 Parganas and ultimately finds its way into Bay of Bengal near New Moore Island as a part of Kalindi-Raimangal estuary in the deltaic southern part of West Bengal. The river Ichamati brings silt and soil from upstream and gradually fills up the riverbeds, and so, as time progresses, the river is unable to cope with larger volumes of water, which spills over and floods the surroundings. The rate of accumulation of soil and silt on the riverbeds increases as more and more trees are cut (deforestation) in the Himalayan Mountain range thereby loosening the soil which is then carried downstream by the rapidly flowing water stream. This is then aggravated by the seasonal heavy downpours report of SWRE [1]. River also plays an important role in the assimilation and transport of domestic water, which form constant pollution sources, and also agricultural runoff, which is temporal and commonly affected by climate [2].

A river is a system comprising both the main course and the tributaries, carrying the one-way flow of a significant load of matter in dissolved and particulate phases from both natural and anthropogenic sources [3]. The quality of a river at any point reflects several major influences, including the lithology of the basin, atmospheric inputs, climatic conditions and anthropogenic inputs [4, 5].

The multivariate statistical methods have been employed to extract significant information from Physico-chemical parameters and depth of siltation datasets in compound systems. The present study attempts to establish a mathematical model between water quality parameter and depth of siltation. Since the data obtained in this study had multivariate nature and several of the variables were correlated, cluster analysis and multiple regression analysis

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A Study to ascertain the Optimum Yield from Groundwater Source in the Eastern Part of Kolkata Municipal Corporation Area in West Bengal, India

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ABSTRACT

The increasing stress on fresh water resources due to ever-rising demands and profligate uses as well as growing population and industrial establishment of Kolkata is an issue of great concern. The purpose of this study is to make a quantitative estimate of the available groundwater resources in the eastern part of Kolkata for efficient utilization and management of groundwater resources. The methodology involved the investigation, drilling, lowering, collection and analysis of main well and observation wells data and groundwater quality as well. Based upon the study of lithological logs as also the electrical log, the sub-surface deposition of the assembly pipes have been determined. The results indicate that the aquifers are composite and composed of sands and overlying silts/clay beds. Long term Tests pumping indicate that the main well may be capable of a long term discharge rate of 120 m^3/hr restricted at 120 m and 156.65 m. The aquifer parameters from the study area are estimated from the analysis of short and long durations pumping test data. For the alluvial aquifer, transmissivity of 1491 m^2/d, hydraulic conductivity of 49.7 m/d and storage coefficient of 0.0064 are recommended found by using different methods. Long duration pumping tests have indicated that the maximum drawdown in water table by 4.89 m may be achievable by radius of influence about 682 m. After the closure of pumping operation, recuperation test was also carried out in the main wells as well as also from observation wells. Recovery test was monitored for 20 hrs after closure of pumping. Slope of the residual drawdown from t/t' indicated aquifer transmissivity of 1322 m^2/d and therefore hydraulic conductivity of the alluvial sand aquifer is 44 m/d having an aquifer thickness of 30 m. The physico-chemical and bacteriological analysis of groundwater of two pumping wells were tested and the results showed the groundwater must be disinfected before supplying to the consumers.

Keywords: optimum yield, sustainable source, groundwater, KMC, pumping wells

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[OP-22]

Development of Combined Site Suitability Analysis System for Riverbank Filtration& Groundwater Recharging

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Abstract

Site selection plays a most important role in riverbank filtration(RBF) and groundwater recharging for sustainable (GWR) water supply. Selection of most appropriate location among different sites is complex procedure, involving many variables. In this study, the Proximity analysis and analytic hierarchy process (AHP), used for analysis of attribute value attributes value and then quantifying the importance of each attribute values, This procedure was used to prioritize Identified location for RBF and GWR structure. A GIS-based computer analysis was developed to automate the assessment process. The developed model was applied to the Kangsabati River in Purulia District West Bengal. Detail analysis of sites reveals that a site that has sustainable availability to the neighboring collector well is more suitable than other locations.

Key Word: Groundwater, GIS, River Bank filtration, analytic hierarchy process.

[OP-23]

FRICTION STIR WELDING OF DISSIMILAR METALS; ALUMINIUM ALLOY A356 AND MAGNESIUM ALLOY AE42

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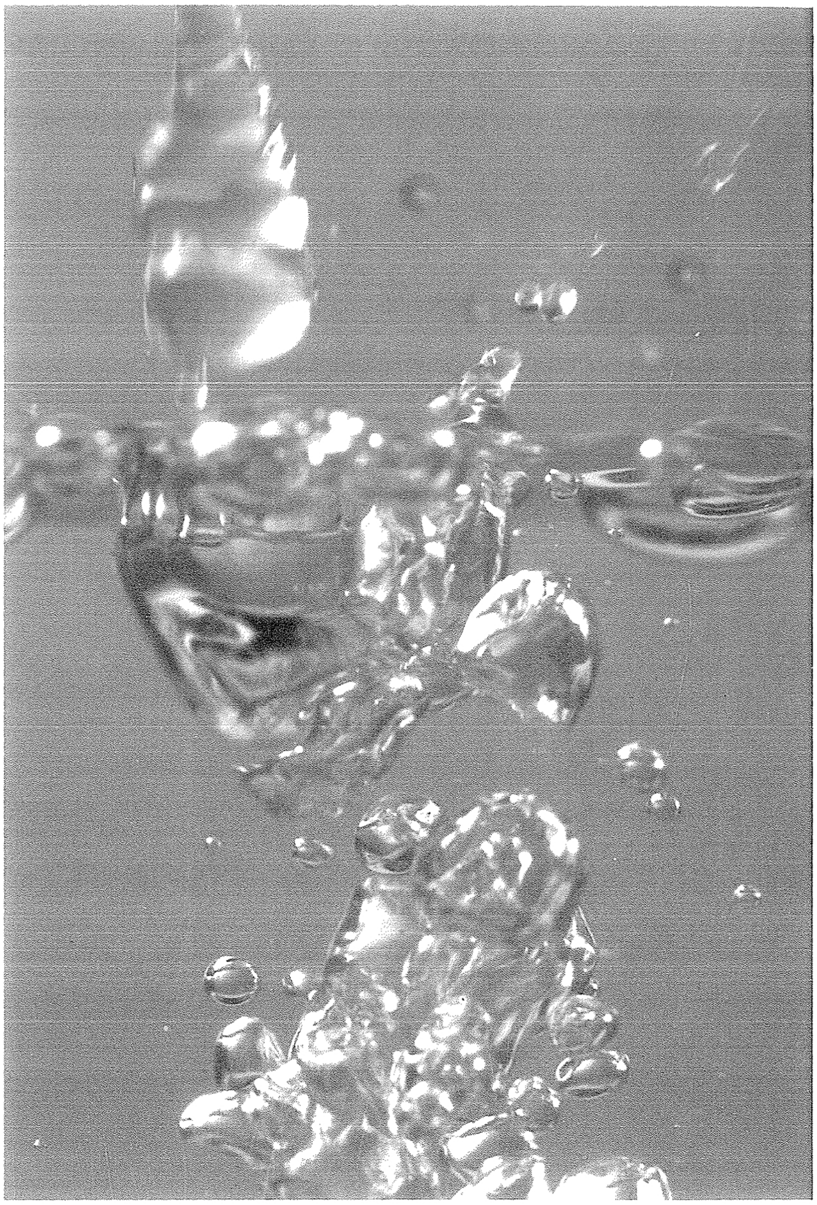
Abstract

In the present work, an attempt has been made to join Magnesium alloy AE42 (Mg-Al-rare Earth) with Aluminium alloy A356 (Al-Si alloy) using friction stir welding. These alloys were



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An Integrated Approach for Ground water Recharging: Key to sustainable Water Resource Management (Groundwater management)

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Keywords
Groundwater management ;Overexploitation ; Recharging ;GIS; Aquifer

Introduction

Groundwater is not an independent entity as it is only one of the phases of the Hydrologic cycle. This cycle is maintained by precipitation, runoff, evaporation, and transpiration and groundwater flow, each one of which is dependent on the other. Surface water and groundwater system are linked component of Water resource management related to sustainable issue. Groundwater is renewable resource, but not inexhaustible because the renewable quantity of water is finite and is exploitable up to a safe limit. Groundwater occurs in porous formation both under water table and confined condition. In Nadia, Murshidabad (except Kandi Sub-division) districts down to 150 m there is absence of any significant clay beds making the entire aquifer upto 150 m depth to occur under water table condition. In the Bhabar zone (foothills of Himalayan trench) aquifers are having very deep water table and are characterized by high seasonal variation of water table to the tune of 10-12 m in this part occurring in parts of Birbhum, Barddhaman, Bankura and Medinipur districts, individual aquifers being of limited thickness and discontinuous nature. The potentially of this aquifer is very poor. By and large yield of the tube well (down to 100-400 mbgl) varies from 80-100 m3/hr.
In Purulia districts groundwater occurs in the fractured zone within 100 mbgl generally discharging 10 m3/hr with maximum discharge of 22 m3/hr. Groundwater occurs both under unconfined and confined condition within the explored depth of maximum 600 mbgl. Aquifers are fairly thick and regionally extensive with large yield prospect of about 150 m3/hr. In Birbhum and Bankura districts aquifers beyond 136 m bgl upto the drilled tertiary formation are found autoflow condition. The occurrence of Arsenic in groundwater in the depth span of 20-80 mbgl restricted mainly in the eastern part of Bhagirathi River has posed a serious problem. With this back ground, the present Paper has been form Ghutlia village in Simla-Dhanera Gram Panchayat, Kashipur Block Purulia District, was selected through household baseline survey which was based on questionnaires, focused group discussion and village personal with interviews of individual community. PRA exercises also done in each and every hamlet followed by net planning along with families and focused discussion with SHG member. Based on the Primary and secondary data as well as the willingness of the local community /Panchayat a low cost composite model which include Multipurpose collector well with infiltration gallery in valley depended on site suitability has been evolved in the selected location of Purulia district. Accordingly the composite model was designed for installation in the selected location in Purulia district as demonstration unit. Water availability scenario both for drinking and irrigation will be augmented through application of watershed development technique and this will improve the economic status of the local inhabitant, drinking water infrastructure for rural tourism.

In the first part of the paper , the concept of groundwater overexploitation and groundwater recharge through surface water groundwater interaction presented with salient points followed by its importance in Water Resource Management .There after sustainable management and people participation together with an overview of IWRM(Integrated water Resource Management)concept.

Study of variation for arsenic contaminated groundwater using Multivariate Statistical Technique in the rural area of West Bengal

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ABSTRACT

In this paper, arsenic contaminated groundwater, tubewell depth and physico-chemical parameters in Jyot Sujan village, Murshidabad district were assessed by laboratory experiment results using statistical techniques. These techniques were applied to the arsenic concentration in groundwater, depth of tubewell and physico-chemical parameters dataset. Experimental result and Statistical analysis shows a pearson correlation between among concentration, tubewell depth and Physico-chemical parameters. Arsenic has strong positive correlation with Hardness and Conductivity. It clearly indicates that groundwater of the study area has highly affected with arsenic and iron contamination and arsenic contamination in groundwater is depend on tubewell depth that means arsenic contamination may be reduced in case of depth of tubewell will be more. So, it concluded that arsenic concentration will be reducing in deeper aquifer.

Keywords:
Groundwater; Arsenic; Tubewell depth; Physico-chemical parameters; Statistical analysis; Pearson correlation.

INTRODUCTION

Groundwater resources all over the world are threatened not only by over exploitation and poor management but also by ecological degradation. One of the main problems of global concern is the presence of Arsenic in ground water. Murshidabad district is one of the major arsenic affected regions in West Bengal. It also noticed that Dahapara Gram Panchayet in Murshidabad District is highly Arsenic contaminated area. Arsenic contamination level is found more than above permissible level of IS standard at Jyot Sujan village, Dahapara Gram Panchayet, but there has no alternative resources of drinkingwater.

The laboratory experimented results of arsenic, other water quality parameters and statistical methods have been implemented to extract significant information from physico-chemical parameters, tubewell depth and arsenic concentration datasets in compound systems. The present study attempts to establish a correlation between arsenic concentration, depth of tubewell and other related water quality parameters. Since the data obtained in this study had several of the variables were correlated and Pearson correlation analyses were used for the interpretation of the data using IBM SPSS model.

MATERIALS AND METHODS:-

Study area

The study area Jyot sujan village, Daharara GP, Murshidabad Jiaganj Block, Murshidabad district is situated at the right bank of Ganges (Bhagiroti) and has been formed primarily by the deposition of alluvial to recent sediments carried by the Ganges. In Jyot Sujan village, villagers mainly use tubewells (groundwater) as source of drinking water purpose but most of tubewells have been affected with high levels of arsenic contamination.

Sample collection and statistical analysis

Water samples of tube well were collected (after pumping for few minutes) without filtration in standard samples bottles and kept it with preservative for arsenic test purpose. Then samples also