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Towards Improved Design of Diversion Structures in Spate Irrigation System: Case Study of Raya Valley, Ethiopia

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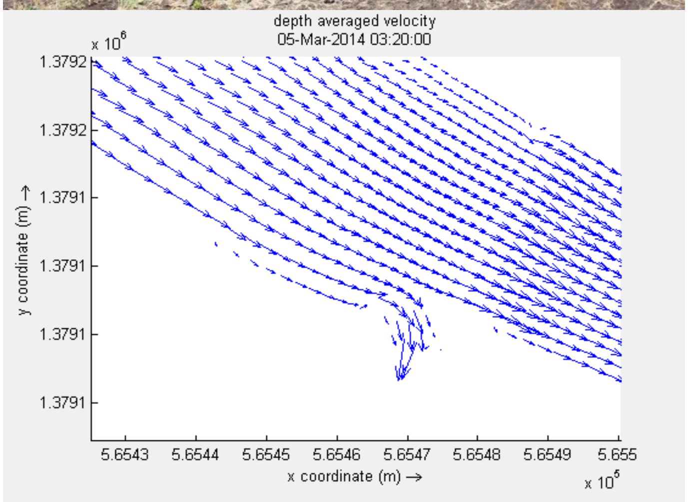

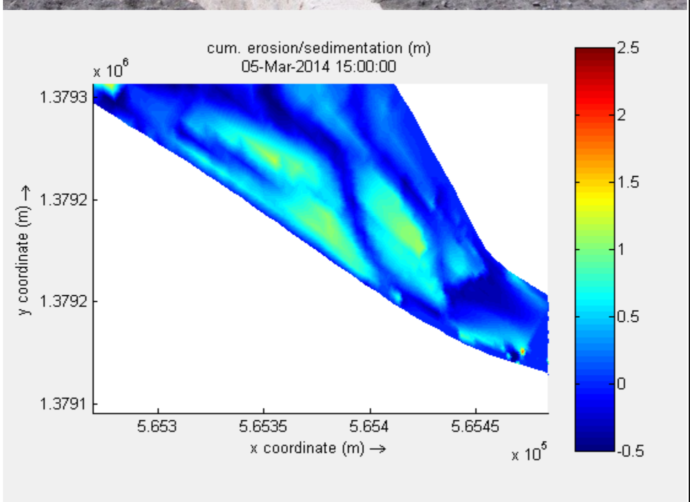

Towards Improved Design of Diversion Structures in Spate Irrigation System: Case Study of Raya Valley, Ethiopia

Master of Science Thesis
by
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Abstract

Spate irrigation is a resource system, whereby flood water is emitted through normally dry wadi and conveyed to irrigable fields. This is commonly used in arid and semi arid areas of the world where annual evapo-transpiration is greater than annual rainfall. This includes letting of flood water to irrigation land through temporary or permanent channel. Now a day's traditional spate irrigation are changing to improved and modern once. Modern spate irrigation schemes are mainly known for their ridge and permanent diversion structures constructed across the river. Weir, intake, sluice gate, retaining wall and main canals are main part of modern spate structures. Modernization has been taking place in Raya valley since 1998 while the efficiencies are not as intended. So far there are no appropriate design standards for spate scheme and this lead for poor scheme performance. The main objective of this study was to investigate spate schemes design development, identify current design problems, develop problem based design alternatives and to recommend the best alternative for better flood and sediment management in Raya valley. Design report collection, preliminary design assessment, field observation for selected schemes were made. Among the visited sites Dayu spate irrigation was found relatively best performing scheme and was selected for further study of design limitation. Primary data like river topography survey sediment analysis structure measurements, discussion with farmers and design experts was made. River bathymetry, cross section, slope, discharge estimation, hydrograph development and sediment concentration estimation was undertaken from the collected data. Based on the farmers and spate irrigation designs experts perception and current problems three design alternatives was developed in addition to the current design. Delft3D model was employed to simulate hydrodynamics and morpho-dynamic to evaluate the flow pattern, depth average velocity, water level and erosion/sedimentations for scenarios. The result of spate irrigation design development in Raya valley shows significant changes; for example widening of intake, increasing of deflection angle, excluding of rain fall during irrigation water requirement and reducing of irrigation time. The current problems of relatively best performing schemes are sedimentation around intakes, less spate flow and low overall low scheme performances. The simulation result of Delft3D showed improving of intake deflection angle from 120^0 to 150^0 for 3 meter wide intake can increase the irrigation area by 21%. Improving of intake width from 3 meter to 5 meter at 120^0 deflection angle can increase the total irrigation area by 81%. Improving of intake width from 3 meter to 5 meter and deflection angle from 120^0 to 150^0 can improve the irrigation area by 100%. These interventions in deflection angle and intake width did not result in any significant reduction of sediment deposition at the intake. However, the enhanced supply of water through the main spate flow gates may convince farmers not to block the scour sluice gates, which are primarily designed to remove coarse sediments. From purely design point of view an intake with 5 m wide and 150^0 deflected angle could be recommended. A detailed cost benefit analysis is required to make a final recommendation. Furthermore, before deciding and implementation before deciding and implementation of this design alternative it is worthy to make proper study on the structural and geotechnical stability and cost comparisons of the option.

***Key words**: Spate irrigation, sedimentation, intake, diversion structure, Delft3D, improved design, flood water*

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Abbreviations

AARC	Alamata Agricultural Research Centre
CFL	Courant (Friedrichs-Lewy) Number
Co-SAERT	Commission for Sustainable Agricultural and Environmental Rehabilitation in Tigray
CSA	Central Static's Agency
CWR	Crop Water Requirements
DORC	Design of Regime Canals (model)
D/S	Downstream
GIWR	Gross Irrigation Water Requirement
FAO	Food and Agriculture Organization
IFAD	International Fund for Agricultural Development
IWR	Irrigation Water Requirement
Masl	mean above sea level
MSRC	Mekelle Soil Research Centre
NIWR	Net Irrigation Water Requirement
Ppm	Parts per million
REST	Relief Society of Tigray
RVADP	Raya Valley Agricultural Development Program
SHARC	Sediment and Hydraulic Analysis on Rehabilitation of Canals
S1	Scenario one
S2	Scenario Two
S3	Scenario Three
S4	Scenario four
TRBARD	Tigray Region Bureau of Agriculture and Rural Development
TARI	Tigray Agricultural Research Institute
UNDP	United Nations Development program
U/S	Upstream
V-Weir	Weir parallel to the flow direction
WUA	water use association
X-1	Cross Section One
X-5	Cross Section Five
X-11	Cross Section Eleven

List of Symbols

mm	millimetre
m	metre
km	kilometre
m3	cubic metre
m2	square metre
m/s	metre per second
m3/s	cubic metre per second
Q	flow rate
Qs	Sediment load concentration
%	percent
A	area
ha	hectare
n	Manning's roughness coefficient
R	hydraulic radius
d	average water depth
S	channel slope
&	and
u	flow velocity in ξ direction
ν	fluid velocity in η or y direction
ζ	water level above horizontal reference level (datum)

CHAPTER 1

Introduction

1.1. General

According to UNDP and FAO (1987) spate irrigation define as "an ancient irrigation practice that involves the diversion of flashy spate floods running off from mountainous catchments where flood flows, usually flowing for only a few hours with appreciable discharges and with recession flows lasting for only one to a few days, are channelled through short steep canals to bunded basins, which are flooded to a certain depth". Mehari et al. (2007) also defines spate irrigation in the simple way as "a resource system, whereby flood water is emitted through normally dry wadi and conveyed to irrigable fields". Moisture stress resistant crops, often sorghum and maize are grown in the spate irrigated agricultures and planted after the first flood irrigation water has occurred. In many areas crops can get matured and give reasonable yield using two or more floods depending on the water holding capacity of the soil.

According to Van Steenbergen et al. (2010) rough estimates, global spate irrigation coverage extends up to 3.3 million hectares even though uncertainty is there. According to the reference made by Mehari et al. (2011) spate irrigation is frequently practiced in the Middle East, North Africa, West Asia, East Africa and parts of Latin America. Although spate irrigation is uncertain type of investment economically it is very important practice in countries such as Yemen, Pakistan, Eritrea and Ethiopia where agriculture is a vital component of their economy (Ratsey, 2011). Even though spate irrigation contributes a lot for food security enhancement in the drought prone areas little concern and emphasis had been given in its developments.

In Ethiopian spate irrigation is a common practice in midlands as supplementary and in lowland area used as dominantly full irrigation while both systems have different characteristics as shown in **Table 1.1**. According to Van Steenbergen et al. (2011) in Ethiopia both farmer's initiative and public investments are the driving forces for spate irrigation development. Currently the cultivated areas under spate irrigation estimates to be 140,000 ha of which 20,000 ha is modern spate irrigation and 70,000 ha still need improvements and other 50,000 ha are under design and construction phases (Van Steenbergen et al., 2011).

Spate irrigation system in Ethiopia is increasing in arid areas particularly; south Tigray (Raya valley), Oromia (Bale, Arusi, West and east Hararghe), Dire Dawa Administrative Region, Southern Nations, Nationalities and Peoples Region (Konso), Afar and Amhara (Mehari et al., 2011).

Table 1.1 Spate irrigation characteristics in mid land and lowland areas of Ethiopia

Spate system	Midland (1000 -1700 masl)	Lowland (below 1000 masl
Rainfall	Supplementary	Less important
Catchment area	Limited	Large
River bed materials	Coarse-cobbles, gravel and sand	Mostly sandy
Gradient	Steep	Gentle
Flow	Flash floods and semi-perennial flow	Short duration spate flow
Command area	Small	Can be large
Water diversion and distribution	Change of flood channel	Change of flood channels

Source: Van Steenbergen et al., 2011

Raya valley is one of the areas which are spate irrigation is being practiced for long times. Farmers were diverting flood water to their farm land using traditional spate irrigation system. During the past decades many governmental and non-governmental organizations were trying to improve and modernize the traditional spate irrigation systems. Many traditional spate schemes were modernized while they did not perform as expected due to several problems. Among this problems are over sedimentation in diversion and canal, failure of structures, inappropriate design and poor participation of farmers during design and construction.

1.2. Background of study area

The Raya Valley is located in the south-east part of the Tigray Regional State between 39^{0}22' to 39^{0}25' north latitude and 12^{0}17' to 12^{0}15' east longitude. It is bordered by Hintalo Wajerat Woreda to the north, Afar Region to the east, Endamekoni and Ofla woredas to the west and Amhara Region to the south. It comprises the total area of Raya Azebo and Alamata Woredas and some eastern high lands of Endamekoni and Ofla Woredas (REST, 1996). **Figure 1.1** shows the location map of Raya valley. The total of population of the Raya Valley Area is about 227,431(136,039 for Raya Azebo and 85,359 for Alamata woreda). From the total population in Raya Azebo 119,984 (88%) and from the total population of Alamata 80,796 (95%) are living in rural areas (CSA, 2007).

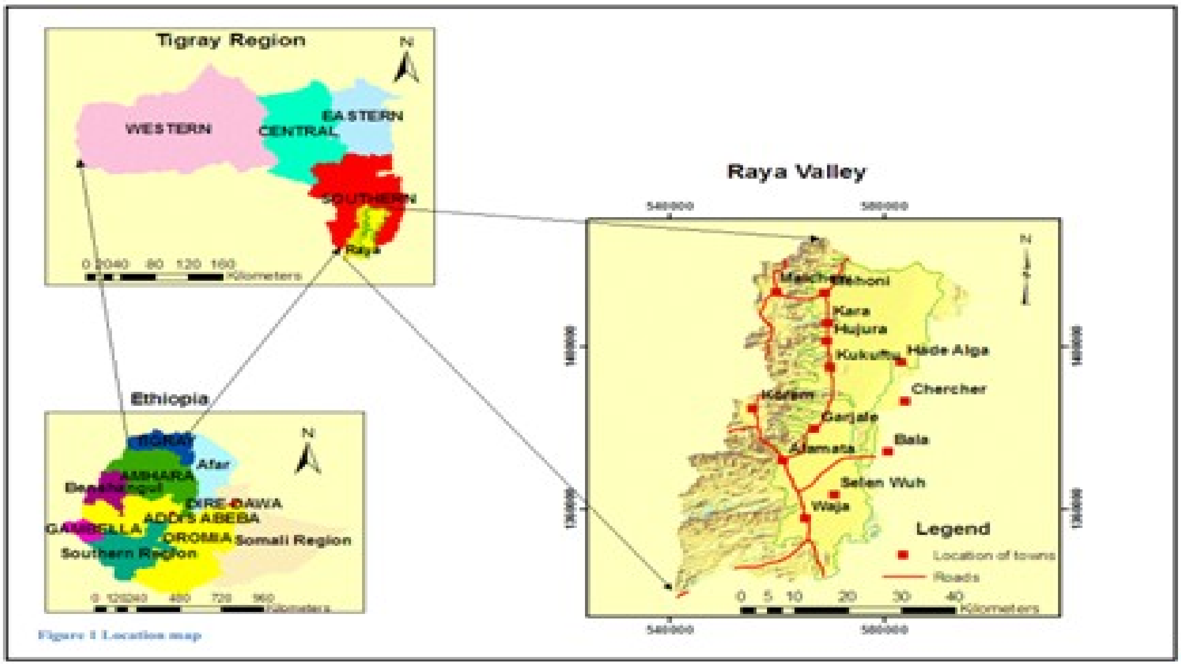


Figure 1.1 Location map of Raya valley (adopted from Gebreezgi A.H., 2010)

Topographically the Raya Valley is divided in to two major zones: low land areas with an altitude less than 1500 m.a.s.l which mostly covers large part of the central part of the valley; and the high land areas having altitude above 1500 m.a.s.l which covers the western and eastern edges of the valley. According to the moisture index criteria provided by REST (1996) the Raya Valley area is classified as dry climates of semi-arid and arid types.

Raya Valley has a bimodal rainfall pattern. Though diminishing from time to time, the area experiences a short rainy season locally known as Belg which runs from February to April followed by the main rain season called Kiremt which runs from June to early September (REST, 1996). The eastern and western highland of the valley experience better rainfall. For instance the Chercher highlands get average rainfall of 620 mm while the Mai-chew highlands get up to 775 mm of rainfall annually. The high fluctuation and unreliability becomes most common in the lowland valley of Mekoni and Alamata areas. The average annual rainfall collected from Mekoni and Alamata meteorological stations show that it is 486 and 693 mm annually respectively (Gebreezgi, A.H., 2010). This shortage of rainfall needs special attention on development of spate irrigation agricultures. Most of the rivers in Raya valley flow from western high lands to east. **Figure 1.2** show the drainage network of Raya valley.

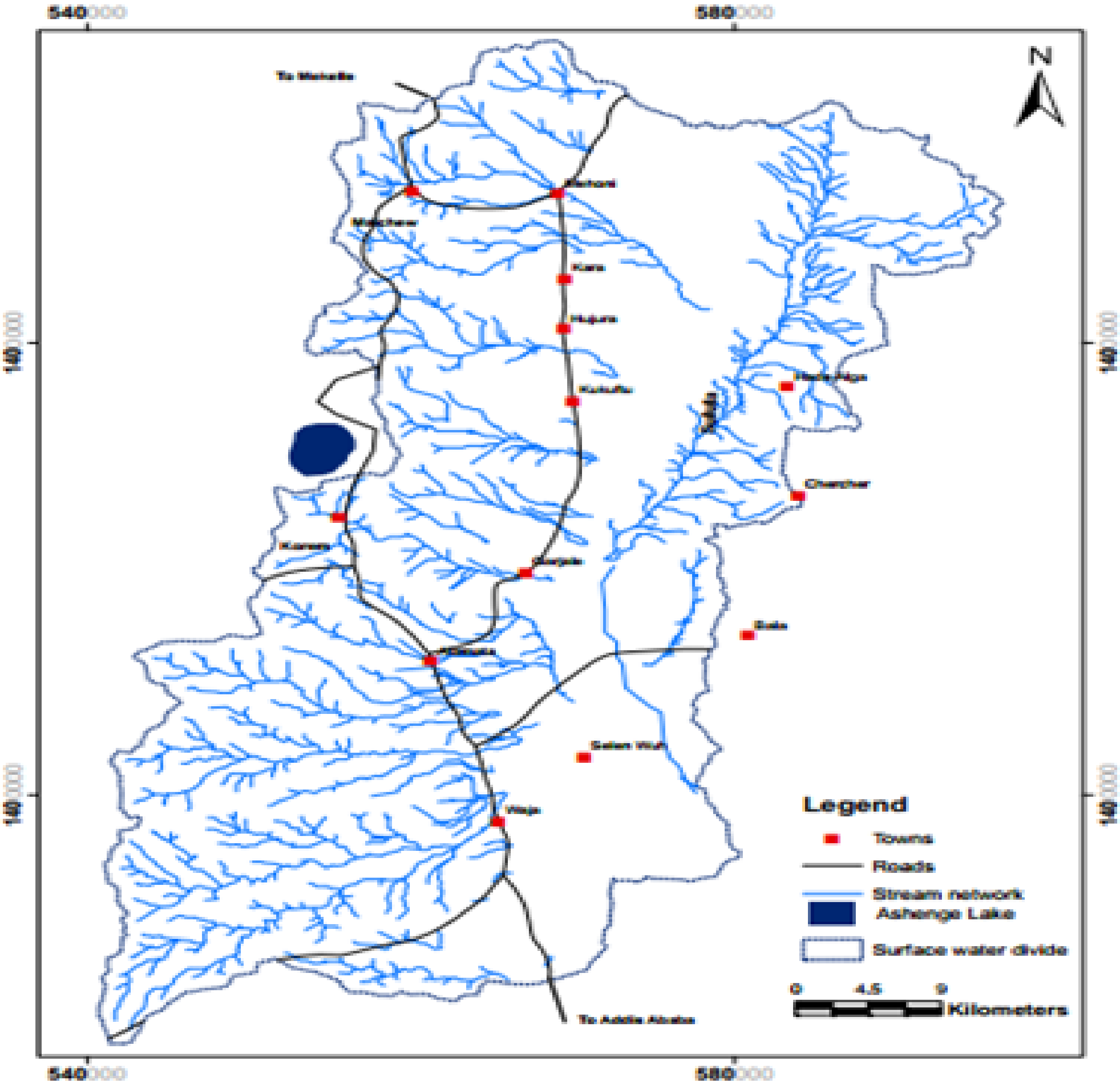


Figure 1.2 Drainage networks in Raya valley (sources: Adopted from Gebreezgi A.H., 2010)

Currently due attention has been given to improve and modernize the indigenous spate irrigation practices. Though the upgrading is very essential, it is not as effective as desired by the farmers. Especially in some schemes like Tirke, where the interest of farmers was not duly considered, failure is inevitable. According to IFAD (2005) “not all modern irrigation is an improvement over indigenous systems. Sometimes especially, when farmers’ views are not fully considered, the construction of modern engineered systems can worsen the operations for those farmers involved”. **Figure 1.3** shows unsuccessful modernized spaate irrigation scheme in Raya valley.

Figure 1.3 Unsuccessful modernized spate irrigation scheme in Hara, Raya valley

Sedimentation has been a very serious problem for spate irrigation schemes in the Raya valley. This is mainly because, most of the schemes are located at the foot of mountains characterized by high sediment yield. In addition sediment rate estimation methods are rarely used in design consideration of these spate irrigation schemes. For instance, sedimentation rate is not estimated during design of the schemes and it is simply fixed and assumed from available secondary data without any solid evidence from research. Land use and geological studies are carried out during the study and design processes of the schemes. This is for the reason that, they are helpful to understand and to provide good estimates of the flood, as the floods are specifically generated using empirical formula which is readily available. Although the land use data is used to generate design floods for both the weir and intake, these study outputs have not been used to generate sediment transport yield of the catchment during design of modern spate irrigation schemes.

1.3. Problem of Statement

The traditional and modern spate irrigation systems in Raya valley have been showing big problems in controlling flood water and sediment management. The problem of sedimentation is more sever in modern spate irrigation than the traditional ones. The main cause could be the existence of permanent diversion structures and lack of flexibility during high flood occurrence.

Traditional spate diversion structures include local materials stone, boulders, shrubs and logs of trees. These structures are mostly temporal and can be demolish when large floods occur and as a result most of the sediments will be transported easily. This is good opportunity for sediment management while there could be loss of flood water in addition to the structural failures. Traditionally, farmers have had good experience for years to rebuild the intakes and distribution structures with in short time period after the extreme flood to harvest the next flood water. When there are consecutive floods and hence not enough time to rebuild and maintain canals and structures, significant amount of floodwater is lost and crop failures occur. **Figure 1.4** shows the problems of modernised schemes such as intake clogging in Hara, weir breakdown in Oda, burying of whole structure by sediment in Tirke and canal over siltation in Fokissa.

Blocked intake, Hara

Burying of diversion by sediment, Tirke

Breakdown of weir structure, Oda

Over siltation of main canal, Fokissa

Figure 1.4 Different problems of modernization spate schemes in Raya valley

In the modern spate irrigation developments the diversion and canal structures are set to be permanent. These structures are set to accommodate all low, medium and high flood incidences. This means during high flood much of flood water which carries more sediment concentration will convey to the irrigation canal system. As the high flood mostly carries stones and boulders the structures will get damaged and a lot of segmentation was accumulating in diversion and canal structures. Many of the spate irrigation schemes in Raya valley did not consider the impact of sedimentation during design and constructions.

Lack of incorporation of sediment management structures in the design part structures are the leading to failures of structures. As can be observed from the scheme sites like Hara and Tirke spate irrigation schemes the problems of structural failures and sedimentation are sever and this needs further study to increase the productivity of spate irrigation schemes. But even in those schemes where sediment management structures such as scour sluice are incorporated, the sedimentations problems were not addressed. Not because that these structures are not effective, but they were not operated properly. Farmers completely block these structures to avoid further loss of irrigation flood water as the main irrigation gates fail to divert sufficient water.

1.4. Objective of the Study

To build up on the efforts over the last 15 years and further improve the design of main intakes for better spate flow diversion and sediment management.

Specific objectives

- Assess the historical evolvement of designs of spate irrigation structures over the past 15 years.
- Make an in-depth assessment of limitations and strengths of the latest main intake design with regard to spate flow diversion efficiency and sediment
- Develop alternatives of improved main intake design and recommend the most suitable design of diversion structure with regard to ease of operation and management requirement

1.5. Research Questions

- How has the design of spate irrigation structures evolved over the past 15 years?
- What are the main problems of the relatively best performing main intake design of the spate irrigation systems with regard to sediment management and reliable spate flow?
- What alternative of main intake designs could be recommended to address the existing problems?
- Which alternative design could be recommended to the Raya valley, the focal study area?

1.6. Research setup

The research work was prepared in seven chapters. The first chapter deals with an introduction of the thesis. The introduction part includes generals of spate irrigation systems, backgrounds of the study area, problem statements, objectives and research questions of the thesis work.

The second chapter deals about the research methodology and explains the overall steps followed to accomplish the study work. The third chapter also deals about the literature review and gives detail information about the spate irrigations systems, spate typology, hydrology of spate, structures of spate irrigation system, sediment transport and management, Delft3D and SHARC models.

Chapter four deals with the data collection and analysis part of the thesis work. The data collection includes secondary data collection, scheme visit and primary data like measurement of topography survey, flood water marks, sediment analysis. The collected data are analyzed accordingly so as to get the necessary information about the study area. Mainly the analysis emphasis on design development through time, profiles of the river, sediment grading analysis and discharge and hydrograph determinations. The fifth chapter explains about the model set up of Delft3D and gives detail information about the model parameters used for set up.

Chapter six elaborates about the result and discussions of the thesis work. The main purpose of the result and discussion part is to present the major finding of the study. This chapter emphasised on the design developments, problems and limitation of the current spate design standards, alternative designs and their importance in relation to sediment management and spate flow. Chapter seven deals about the conclusion and recommendation resulted from the study work.

CHAPTER 2

Research Methodology

Introduction

This chapter deals about the research methodology followed to achieve the scope of the study. This study was takes place in two parts which is field work and followed by data compiling and modelling. Literature review was also main component of the research work in all chapters. The detailed methodologies followed to answer the research question of the study are presented as follows:

1) How has the design of spate irrigation structures evolved over the past 15 years?

In order to answer this research question several data were collected from respective organizations. The collected data are mainly engineering feasibility study, design reports and head work structure drawing and specification of the modernized spate irrigation schemes. The collected reports of the schemes were reviewed and evaluation was made on the development of spate irrigation structure designs over the past 15 years in Raya valley. An intensive field visit was also undertaken to envision the modern spate irrigation schemes. During the field visit an exhaustive interview and discussion was held with local experts and farmers so as to get detail information about the construction, implementation and problems of modern spate irrigation schemes. Therefore the designs developments of spate irrigation structure were determined based on the collected data and field observation. The relatively best performing designs were identified and selected for further study.

2) What are the main problems of the relatively best performing main intake design of the spate irrigation systems with regard to sediment management and reliable spate flow?

Based on the current status of the modern spate irrigation schemes in Raya valley one best scheme was selected for further detail study of problems and limitations in relation to spate flow and sediments. An assessment was made to the diversion design problems and limitations of the relatively best performing. The current conditions and problems of the scheme related to spate flow and sedimentation was evaluated. Discussion was made with professional experts of spate irrigation designer, local experts and farmers to identify the current problems and limitation.

3) What alternative of main intake designs could be recommended to address the existing problems?

To answer this research questions an assessment of the traditional spate irrigation was undertaken so as to determine the reason for their sustainability. An alternative design of intake structures was developed based on traditional knowledge of farmers and the experience of design engineers. The alternatives designs include the farmer's perception, reflections of spate design expert and engineering point of view. Therefore the alternative designs or scenarios were developed based on intake width and deflection angle of the intake to the flowing water.

4) Which alternative design could be recommended to the Raya valley, the focal study area?

To recommend the best alternative design or scenario a simulation modelling was made using Delft3D software program. This software program simulates the hydrodynamics and morphodynamics of the river reach and it is well known model for river flow and sediment transports. Delft3D needs several inputs for simulations and some of these inputs were organized from secondary data, measured inputs, derived inputs from the measured once and from literatures. The main inputs of Delft3D were organized as follows.

- All the head work dimensions were measured using a measuring tape. This data were used to develop the detail size of the diversion structures like gate size, angle and weir axis length. This data are use full parameters for bathymetry generation and used as an inputs for delft 3D software program
- Topography survey was made using total station surveying instrument. The river was surveyed in eleven cross sections for 591 meter long river reach. These data was used to drive the slope, cross sectional areas, grid and bathymetry of the river.
- Flood marks of the river for minimum, medium, maximum and extreme flood events were collected from the farmers' interview. Suitable river cross sections were selected on the river reach around the diversion structure.
- Flood frequency and magnitude of the river was collected from interview held with farmers. The number of flood occurrence during dry, medium and wet seasons was estimated. The duration of minimum, medium, maximum and extreme floods were also collected from the farmers experience and observations.
- Discharge of the river was calculated from the river cross sectional area and flood marks. Manning and Bathurst formula was used for flood discharge calculation. Manning roughness coefficient was determined from literature based on the river bed materials.
- Sediment sample was collected from representative sample points on the river. Eight samples of $1m^3$(1*1*1 meter) each were taken from three river cross section and one sample from the intake of the diversion. Manual sieve was made in the field for sediment size greater than 5mm and for the sediment material less than 5 mm around two kilograms of sample were collected for laboratory sieve analysis and analysis was made in Mekelle Soil Research Centre.
- Sediment concentration was determined from DORC module of SHARC software. The main inputs of DORC model are bed material size and river hydraulic parameters like velocity and flow depth. The river hydraulic data of depth and velocity are estimated from DORC model alluvial friction predictor part. The alluvial friction predictor part of DORC has alternative methods of predictor of Brownlie, Engelund and Hansens, Van Rijn and White, Paris and Bettess. Comparison was made to all methods of alluvial friction predictor and observed or calculated data of the river and the best fitted was selected for generation of sediment concentration

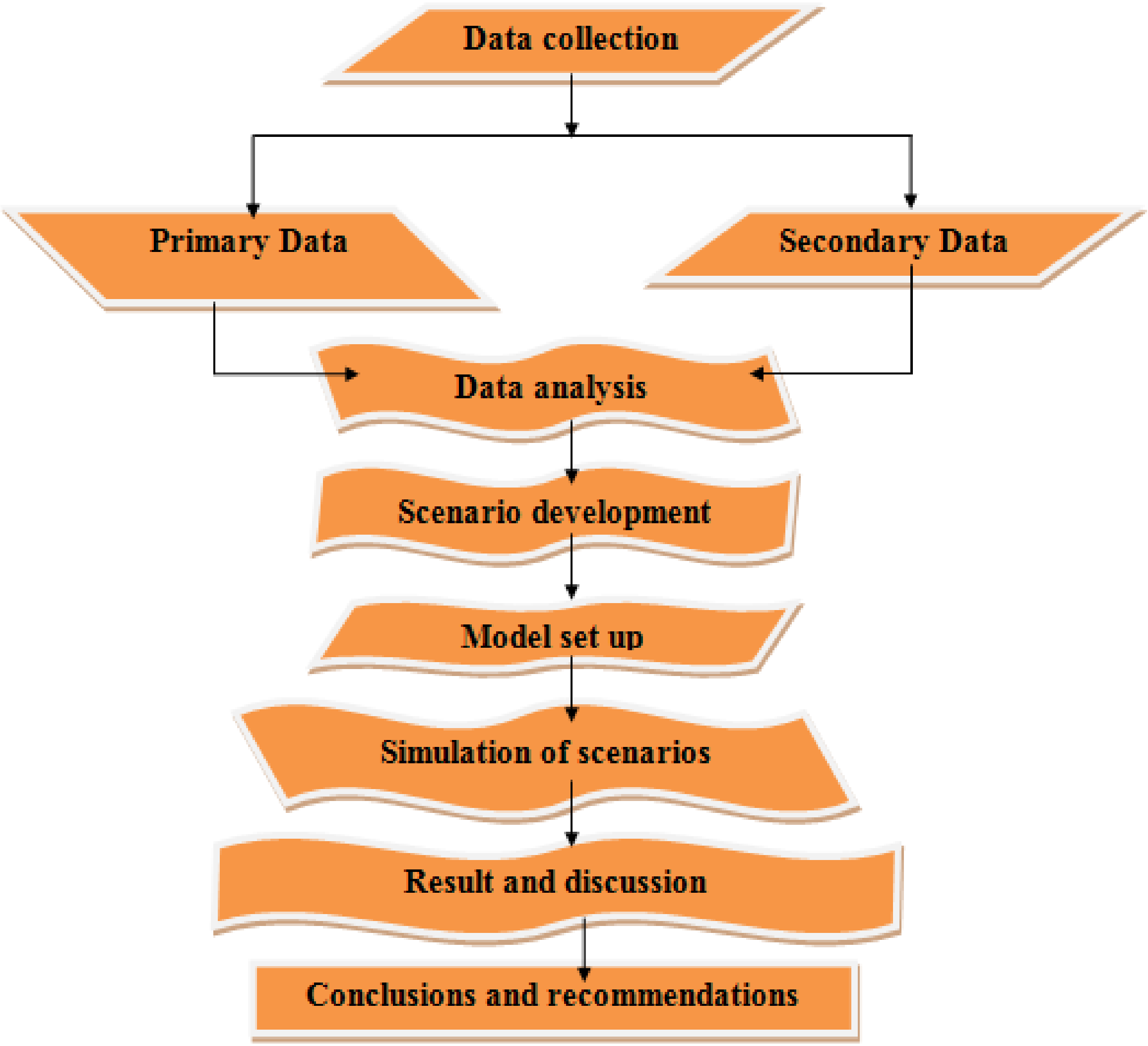


Figure 2.1 Research methodology flow chart

CHAPTER 3

Literature Review

3.1. Spate irrigation system

The evolution of spate irrigation dates back long time era. The remaining of some diversion dam structures in ephemeral rives of Iran and Balochistan dates back to 3,000 BCE. It is believes that spate irrigation was started in Yemen, when the climate becomes arid and it has been practicing for about five thousand years (Laurence and Van Steenbergen, 2005). Spate irrigation system is a sort of floodwater gleaning and managing, exclusive to arid regions nearby highland areas (Mehari et al., 2011). Notwithstanding its potential to sound contribution of livelihood improvement of poor people in the delicate ecosystems, it is hardly neglected and has been given less emphasis in many countries of developmental program and resource management (Mehari et al., 2011). Spate irrigation system is mainly carried out in hot arid and semi arid areas of the world, where evapo-transpiration is much more than available rainfall (Laurence and Van Steenbergen, 2005). **Figure 3.1** show the relationship between evapo-transpiration and rainfall in two spate irrigation schemes of Yemen (wadi zabid) and Pakistan (Jacobabad)

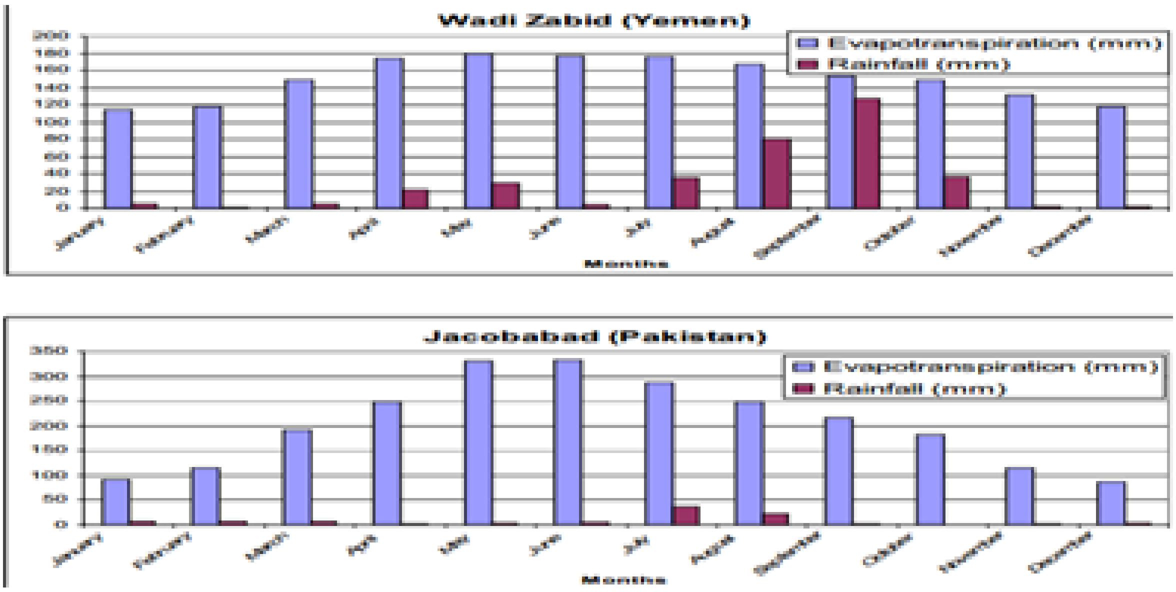


Figure 3.1 Monthly rainfall and evapo-transpiration in two spate irrigated areas. (Source: Laurence and Van Steenbergen, 2005)

3.2. Spate typology

According to Van Steenbergen et al. (2010) spate irrigation classified based on: scheme size (small, medium and large), infrastructure (traditional, improved and modernized), operation and maintenance (farmers managed, farmers supported from agencies and agency managed) and flow regimes (only spate flow, flow includes significant base flow and conjunctive use of spate and groundwater). Classification of spate in relation to infrastructure includes traditional intakes and canals, improved traditional systems and modernized and new systems.

3.2.1. Traditional intakes and canals

According to Van Steenbergen et al. (2010) traditional diversion/headwork includes deflecting spurs or in flatter plain areas, bunds that are constructed right across the flood channel. The construction of diversion is quite simple and temporal. The traditional canals are mainly short and sometimes can include secondary canals Usually water distribution between fields takes place through letting water to pass by breaking field bunds when the predetermined water depth reaches. When extreme flood occurs the structure can fail and much of the sediment moves along the river flow. There for small amount of sediment can enter to the canals and scheme and it can be easily maintain. The adverse effect of traditional spate irrigation; it needs frequent maintenance, labour intensive and uses a lot of bushes and trees for reconstruction of the structure. This is also main causes of flood water losses when harvestable flood followed by extreme flood in short time, which is before reconstruction of diversion structures (Van Steenbergen et al., 2010). This problems lead to an improvement of traditional spate irrigation.

3.2.2. Improved traditional systems

According to Van Steenbergen et al. (2010) improve traditional spate irrigation is diversion of flood water from the river and mainly the structures could have flow throttling structures, rejection spillways near canal heads, drop structures and flow division structures in main canals. In some area the construction of improved structures may need huge investment and farmers can hire bulldozers. During support for improvements from outside agencies, bulldozers may be provided at subsidized rates, and simple gabion masonry structures may be used at diversions. Improved water control structures may also be incorporated in the canal and field systems. Although this improvement solves some of the problems occurred in traditional system but there are many problem are still occurring like sedimentation of canals, siltation of diversion structures and this needs huge investment for operation and maintenance. Money traditional and improved spate irrigation schemes are modernized in the past decades so as to increase the overall efficiency of the irrigation systems.

3.2.3. Modernized and new systems

Modern spate irrigation is the diversion of spate to farm lands using strong structures, mainly cement masonry and concrete structures. The design concept is to divert significant amount of the flood, the amount varies through experience and practice, at a point to serve a large command area. In large systems, numerous traditional intakes are replaced with concrete diversion weirs, with sediment sluices. Owing to the high costs of permanent structures a single permanent weir often replaces many traditional intakes. In newer schemes, steep canals and sediment management structures are provided to minimize sedimentation. Even in new schemes, where farmers may not have the traditional practices needed to manage spate flows, a range of diversion types, including large semi-permanent soil bunds and small, simple diversion weirs, are used (Van Steenbergen et al., 2010).

3.3. Hydrology of spate

Spate hydrology is characterized by a great variation in the size and frequency of floods which directly influence the availability of water for agriculture. Wadi is also characterized by very high sediment loads and important groundwater recharge through seepage in the wadi bed. All these characteristics are specific to wadi hydrology. Management of floods and high sediment load therefore require a good estimate of the main hydrological characteristics of the wadi (Van Steenbergen et al., 2010).

Spate hydrology describes the runoff and sediment transport processes that influence spate irrigation practices and the design of improved spate irrigation schemes. This provides some simple methods that can be used to derive the hydrological information needed to design intakes and canals for spate irrigation systems (Van Steenbergen et al., 2010).

Understanding of spate hydrology has great implication for design of spate irrigation schemes. The high-intensity rainfall events that generate spate flows in wadi are characterized by a wide variability in space and time. In many areas information on the spatial characteristics of rainfall wadi watersheds is inadequate.

3.3.1. Flood estimation

Flood amount of a river is an important parameter for knowing of hydrograph and this play a vital role in design of spate irrigation structures. According to Ratsey (2011) there are several methods of flood estimations among this:

- Analysis of long term records of measured flood discharge
- Analysis of synthetic long term run off data derived from stochastic modelling
- Rational method
- Regional flood frequency relationships
- Slope area method
- Velocity area measurement of actual floods.

These methods use different input data and have different accuracy. In spate irrigation practices it is difficult to get long term and detail data inputs. Slope area method could be relatively best method for estimation of flood discharges in many wadi.

Slope area method

This method depends on the flood marks of the river banks. Knowing of the actual river cross section is and longitudinal river slops are also important parameters. According to Ratsey (2011) three to four river cross sections have to survey so as to develop independent estimation of discharge. Roughness coefficient of the river bed also needs estimation either from table or empirical formula. Slope area method can use either Manning or Bathurst formula.

Manning formula

Manning formula mainly uses bed roughness coefficient for calculation of the discharge

$$Q = \frac{1}{n} * A * R^{2/3} * S^{1/2} \qquad \text{Manning}$$

Where:

Q = Discharge in m^3/s
A = Cross section of river in m^2
R = Hydraulic radius which is area per wetted perimeter in m
S = River slope in m/m
n = manning roughness coefficient
g = acceleration due to gravity m/s^2

Bathurst formula

Bathurst formula used to estimate flood discharge and it mainly based on size of bed material

$$Q = A * D^* * (g * R * S)^{1/2} \qquad \text{Bathurst, 1985}$$

$$D^* = \left(5.62 * \log\left(\frac{d}{D_{84}}\right) + 4\right)$$

Where:

Q = Discharge in m^3/s
A = Cross section of river in m^2
R = Hydraulic radius which is area per wetted perimeter in m
S = River slope in m/m
d = mean flow depth, similar with R
D_{84} = size of bed material with 84 percent is finer in m
g = acceleration due to gravity m/s^2

According to Arcement and Schneider (1989) for area which have know median size bed material it is better to use Bathurst formula instead of Manning equation. Bathurst formula is independent of Manning roughness coefficient and mainly depend on the D_{84} of bed materials.

3.3.2. Shape of the spate hydrograph

Flows move down the channel network as a flood wave. Runoff from different parts of a catchment converges in the steep wadi channels, sometimes generating multi-peaked spate flows at the water diversion sites in the lower wadi reaches. Flood hydrographs are characterized by an extremely rapid rise in time, followed by a short recession, as illustrated in **Figure 3.2**. In this case, the discharge at a spate diversion site in Wadi Rima in Yemen increased from less than 1.0 m³/s to about 550 m³/s in around 30 minutes, with a second smaller peak occurring the next day. The lower water surface elevation after the flood is due to bed scour (Laurence and Van Steenbergen, 2005).

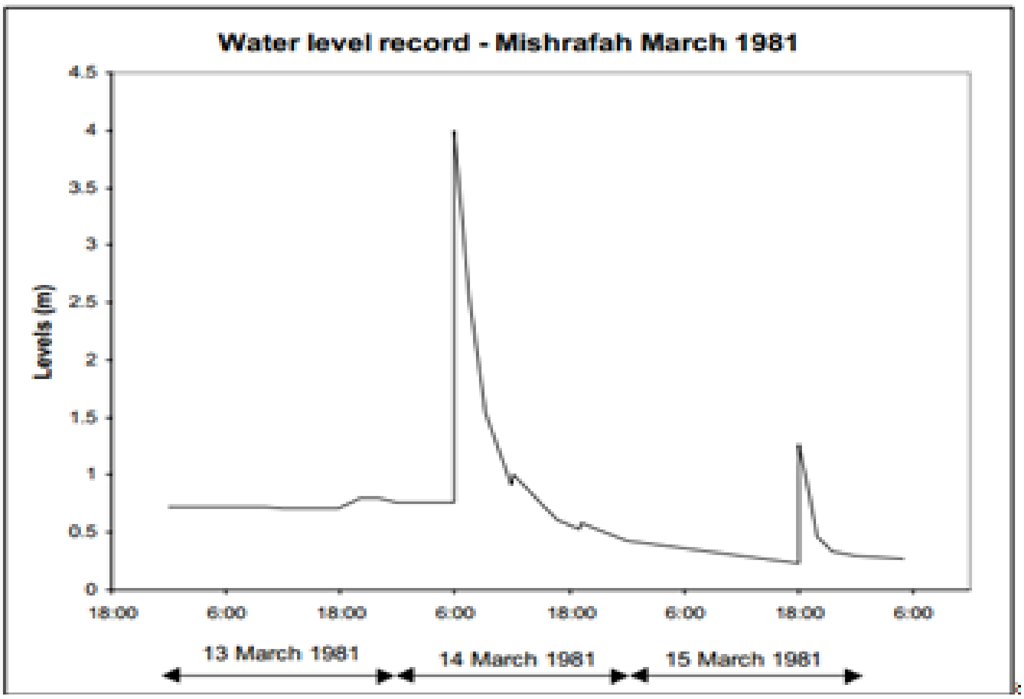


Figure 3.2 Spate flood hydrograph from wadi Rima, Yemen (Source: P Laurence and Van Steenbergen, 2005)

3.4. Structures in spate irrigation

According to Van Steenbergen et al. (2010) spate irrigation structures like any other conventional irrigation schemes spate irrigation also consists different structures like intakes, canals, water control structures, wadi bed retrogression and wadi training. Unlike conventional irrigation structures the spate structures requires special attention in construction, implementation and maintenance phases as the flood/water availability is different. Conventional irrigation has flow almost all over the year where as in spate the flood water occurs from few minutes to days with high flood discharge rate. The details of the spate irrigation structures are discussed below.

3.4.1. Diversion structures (Intakes)

The main purpose of diversion structures is to divert large amount flood water in unreliable levels to the canal system. These structures have to convey sufficient amount of flood water to guarantee commanded irrigated fields. They have to avoid delivering of uncontrolled flood flows to the canals, so as to reduce damage to channels and irrigation field and limit the entrance of the high concentrations of coarse sediments that occurs in large flood events. These functions need to achieve in unstable wadi, characterized by irregular lateral movements of low-flow channels within the wider wadi cross-sections, bank cutting and vertical movements of the wadi bed caused by scour and sediment deposition during floods.

Intake structures have to give function over the longer term with rising irrigation command levels caused by sediment deposition on the irrigated fields and degradation of wadi bed levels due to changing hydrological conditions, climate change and catchment deforestation. Traditional diversion structures have either Spur-type deflector or Bund type diversions while the modern spate diversion structures consists of weir, Spur-type deflector, scour sluice and canal head regulator (Van Steenbergen et al., 2010).

The traditional spur-type deflectors are commonly found in the upstream reach, just after the end of foothills and stare of flood plains. The main characteristics of this location are defined as steep longitudinal slops, coarse bed materials and fast water velocity of during flooding events. This structure consists of spur, usually built from wadi bed material and reinforced with brushwood and other local durable materials which are brought down by the flood events. This all materials will collect and organize in the main wadi bed level and aims to divided the incoming flood flow with the larger part of flood allows to continue to the downstream of the river. From the main deflector a bund will provide and extended up to the wadi bed level al relatively sharper angle to covey water to the canal system both in low, medium and high flow. All of the traditional spur-type deflectors are connected to the canal system un-gated intakes. A common example of traditional spur-type intakes is as shown in **Figure 3.3**

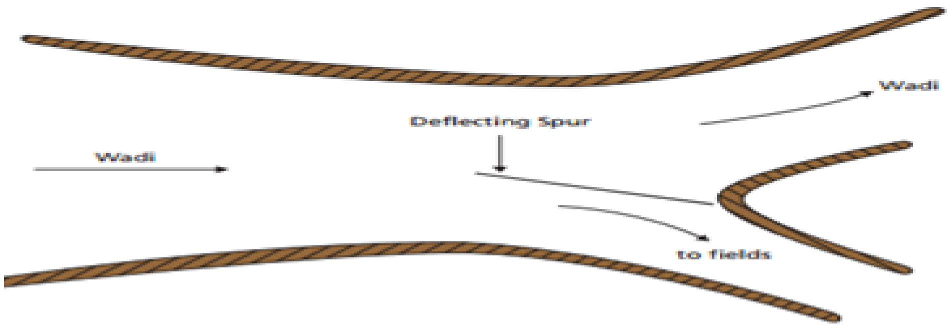


Figure 3.3 Deflecting spur type traditional intakes (Source: adopted from Van Steenbergen et al. 2010)

Bund type diversion structure consists of big bund built from wadi bed materials which constructed across the wadi bed as shown **Figure 3.4**. This type of structure can divert the entire incoming flood to the canal systems. mostly this kind of structure are constructed in the lower reach of wadi when the bed slope gets flatter, frequency of flow availability are less, velocity of water is too small and bed materials are fine. This structure can divert water until the flow starts overtopping above the bund and scouring happens by large flood events.

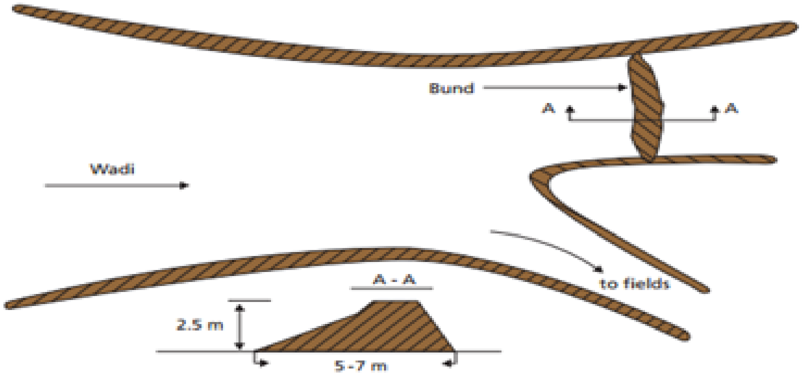


Figure 3.4 Bund type traditional intake (Source: adopted from Van Steenbergen et al., 2010)

3.4.2. Spate canals and water control structures

Canals need to convey large volumes of water to fields quickly in the short periods when flood flows occur. The timing, duration and maximum discharge of spate flows are unpredictable and thus canal capacities have to cope with a wide range of design conditions. Water distribution systems developed for perennial irrigation are thus not appropriate for spate irrigation systems as canal capacities are determined for a relatively narrow and predictable range of design conditions. Traditional intakes and their modern replacements can be adapted to meet spate design conditions, although the design parameters will be very different, resulting for large differences in cost and maintenance requirements

3.4.3. Wadi bed retrogression and wadi training

The bed level of wadi can be significantly lowered than the initial level during the existence of large flood events. In traditional spate schemes it is necessary to change place of intake or to extend a diversion spur further upstream to deliver the desired amount of water for the command area otherwise it will be impossible to divert the water. In some areas it is also necessary to construct small check dam structures to trap sediments and raise the bed levels.

Providing structures to control bed levels is an option but it is often difficult to justify in small spate schemes. The preferred material for bed sills is mass concrete, which can be cast into excavated trenches. Gabion bed sills have also been used.

3.4.4. Bank protection

High flow velocities during spates often erode wadi banks, particularly in the meandering middle and lower reaches. The sinuous flow alignments within the wider wadi channel result in scouring and undercutting of wadi banks at the outer curves and sedimentation at the inner curves. These causes Meander patterns to develop and migrate downstream. Bank erosion scours out valuable irrigated land and can threaten canals running parallel to the wadi banks.

3.5. Sediment transport

All floods carry numerous amounts of sediment and delivers to the diversion, canals and field. According to Xiaoqing (2003) sediments can classify based on laws and patterns of movements in to bed load and suspended load. It can also group in to bed material load and wash load depending to their origin, particle size and impact on fluvial developments. Wash load sediments remains in suspension for long time and gradually settle while coarser materials stop moving quickly as they are highly dependent on the flowing velocity (Ratsey, 2011). **Figure 3.5** shows the components of total sediment loads.

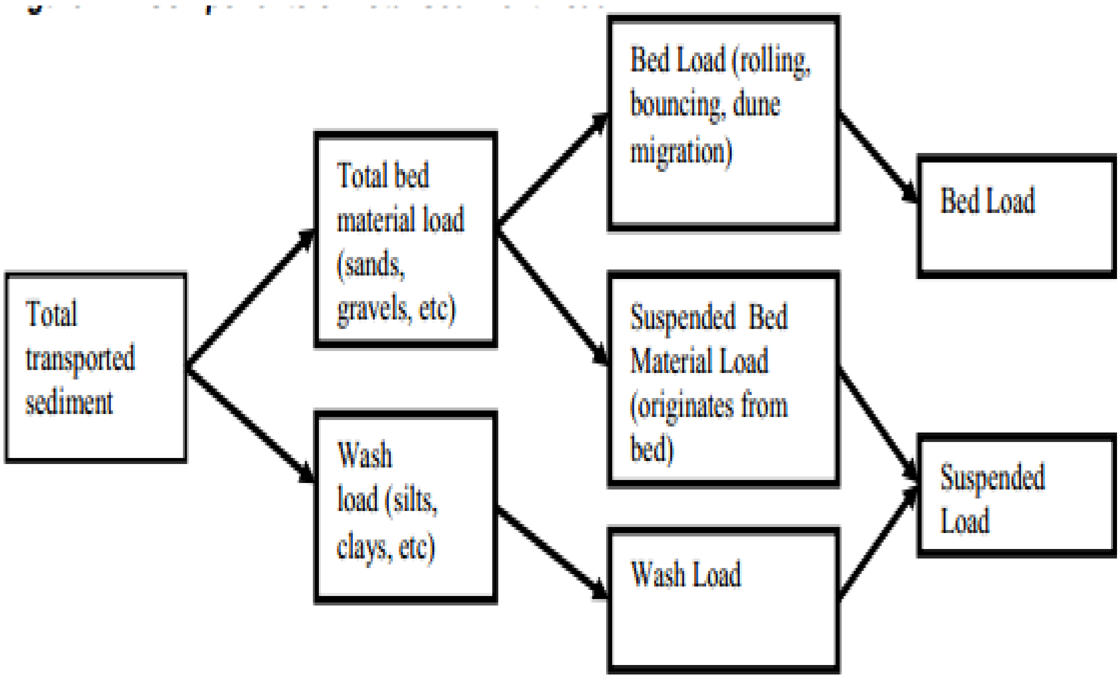


Figure 3.5 Components of total sediment load (source Ratsey, 2011)

In most spate irrigation farmers are happy to deliver fine sediment on their field so as to enhance soil fertility. Even though sediment management is the dominant factor in spate irrigation designer does have very little data to consider sediment management options.

3.6. Delft3D

The Delft3D program was a software program developed by Deltares. The model has many module components which can work in different water related modelling works. This model has a multidisciplinary approach and suitable for coastal, river and estuarine areas. Delft3D model is skilled program in simulation of flows, waves, sediment transports, water quality, morphological developments and ecology. Delft3D-FLOW is one of the modules of Delft3D software programs, which is also used for this research work.

Delft3D-FLOW is a multidimensional which is 2D or 3D hydrodynamic and transport simulation program which determines non-steady flow and transport phenomena that comes from either tidal or meteorological forces on a specific rectangular or curvilinear bounded grid. River flow simulation, online sediment transport, morphology, wave driven forces, simulation of tsunamis, hydraulic jump, tidal and flood waves are listed among the main areas of application of Delft3D-FLOW (Hydraulics, 2011).

3.6.1. Physical processes

Numerical hydrodynamic modelling arrangement of Delft3D-FLOW answers the unsteady shallow water equations in two dimensions (depth-averaged) or in three dimensions. The system of equations consists of the horizontal equations of motion, the continuity equation, and the transport equations for conservative constituents. The equations are formulated in orthogonal curvilinear co -ordinates or in spherical coordinate on the globe.

The model also includes a mathematical conceptualization which takes in to account the following physical phenomena:

- Free surface gradients (barotropic effects)
- The effect of the Earth's rotation (Coriolis forces)
- Turbulence induced mass and momentum fluxes (turbulence closure models)
- Space and time varying wind shear -stress at the water surface
- Space varying shear-stress at the bottom
- Space and time varying atmospheric pressure on the water surface
- Time varying sources and sinks
- Effect of secondary flow on depth-averaged momentum equations
- Lateral shear-stress at wall
- Vertical exchange of momentum due to internal waves
- Wave induced stresses (radiation stress) and mass fluxes
- Flow through hydraulic structure

3.6.2. Major assumptions of Delft3D-FLOW

- In the co-ordinate system the depth is assumed to be much smaller than the horizontal length scale. For such a small aspect ratio the shallow water assumption is valid, which means that the vertical momentum equation is reduced to the hydrostatic pressure relation. Thus, vertical accelerations are assumed to be small compared to the gravitational acceleration and are therefore not taken into account. When this assumption is not valid then Delft3D provides an option to apply the so-called Non hydrostatic pressure model in the Z -model.
- In Cartesian reference frame, the impact of the Earth's curvature is not considered.
- At the bottom a fall boundary condition is assumed, a quadratic underneath stress conceptualization is applied.
- The formulation for the enhanced bed shear-stress due to the combination of waves and currents is based on a 2D flow field, produced from the velocity close to the bed using logarithmic estimate.
- For a dynamic online coupling between morphological changes and flow the 3D sediment and morphology Add-on is available.
- The equations of DELFT3D-FLOW are capable of solving the turbulent scales (large eddy simulation), but usually the hydrodynamic grids are too coarse to resolve the fluctuations. Therefore, the basic equations are Reynolds-averaged introducing so-called
- Reynolds stresses. These stresses are related to the Reynolds-averaged flow quantities by a turbulence closure model.
- The boundary conditions for the turbulent kinetic energy and energy dissipation at the free surface and bottom assume a logarithmic law of the wall (local equilibrium).

3.6.3. Governing Equations in Delft3D

The main governing equations in Delft3D software program are Continuity, Momentum and transport equations.

Continuity Equation

The depth-averaged continuity equation is given by:

$$\frac{\partial \zeta}{\partial t} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\frac{\partial[(d+\zeta)U\sqrt{G_{\xi\xi}}]}{\partial \xi} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\frac{\partial[(d+\zeta)V\sqrt{G_{\xi\xi}}]}{\partial \eta} = Q$$

Where Q is representing the contributions per unit area due to the discharge or withdrawal of water, precipitation and evaporation:

$$Q = H \int_{-1}^{0} (q_{in} - q_{out}) d\sigma + P - E$$

With q_{in} and q_{out} are local sources and sinks of water per unit of volume [1/s], respectively,

P is the non-local source term of precipitation and

E is the non-local sink term due to evaporation. We remark that the intake of a power plant is, for example, a withdrawal of water and should be modelled as a sink.

Momentum Equations in Horizontal Direction

The momentum equation uses the formula given by:

$$\frac{\partial u}{\partial t} + \frac{u}{\sqrt{G_{\xi\xi}}}\frac{\partial u}{\partial \xi} + \frac{\upsilon}{\sqrt{G_{\eta\eta}}}\frac{\partial u}{\partial \eta} + \frac{\omega}{d+\zeta}\frac{\partial u}{\partial \sigma} - \frac{\upsilon^2}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\frac{\partial\sqrt{G_{\eta\eta}}}{\partial \xi} + \frac{u\upsilon}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\frac{\partial\sqrt{G_{\xi\xi}}}{\partial \eta} - f\upsilon = -\frac{1}{\rho_o\sqrt{G_{\xi\xi}}}P_\xi + F_\xi + \frac{1}{(d+\zeta)^2}\frac{\partial}{\partial\sigma}\left(\upsilon_V\frac{\partial u}{\partial\sigma}\right) + M_\xi$$

And

$$\frac{\partial \upsilon}{\partial t} + \frac{u}{\sqrt{G_{\xi\xi}}}\frac{\partial \upsilon}{\partial \xi} + \frac{\upsilon}{\sqrt{G_{\eta\eta}}}\frac{\partial \upsilon}{\partial \eta} + \frac{\omega}{d+\zeta}\frac{\partial \upsilon}{\partial \sigma} + \frac{u\upsilon}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\frac{\partial\sqrt{G_{\eta\eta}}}{\partial \xi} - \frac{u^2}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\frac{\partial\sqrt{G_{\xi\xi}}}{\partial \eta} + f u = -\frac{1}{\rho_o\sqrt{G_{\eta\eta}}}P_\eta + F_\eta + \frac{1}{(d+\zeta)^2}\frac{\partial}{\partial\sigma}\left(\upsilon_V\frac{\partial \upsilon}{\partial\sigma}\right) + M_\eta$$

P_ξ and P_η are pressure terms and represents the pressure gradient in ξ and η directions

F_ξ and F_η are forces and represents momentum equations represent the unbalance of horizontal Reynolds's stresses.

M_ξ and M_η represents the contributions due to external sources or sinks of momentum (external forces by hydraulic structures, discharge or withdrawal of water, wave stresses, etc.).

Transport Equation

The flows in rivers, estuaries, and coastal seas often transport dissolved substances, sediments, salinity, and/or heat. The transport equation is formulated in a conservative form in orthogonal curvilinear coordinates in the horizontal direction and σ coordinates in the vertical direction:

$$\frac{\partial(d+\zeta)c}{\partial t} + \frac{1}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\left\{\frac{\partial[\sqrt{G_{\eta\eta}}(d+\zeta)\upsilon c]}{\partial\xi} + \frac{\partial[\sqrt{G_{\xi\xi}}(d+\zeta)\upsilon c]}{\partial\eta}\right\} + \frac{\partial\omega c}{\partial\sigma}$$
$$= \frac{d+\zeta}{\sqrt{G_{\xi\xi}}\sqrt{G_{\eta\eta}}}\left\{\frac{\partial}{\partial\xi}\left(D_H\frac{\sqrt{G_{\xi\xi}}}{\sqrt{G_{\eta\eta}}}\frac{\partial c}{\partial\eta}\right)\right\} + \frac{1}{d+\zeta}\frac{\partial}{\partial\sigma}\left(D_H\frac{\partial c}{\partial\sigma}\right)$$
$$- \lambda d(d+\zeta)c + S$$

With

D_H the horizontal diffusion coefficient,

D_V the vertical diffusion coefficient, and

S the source and sink terms per unit area due to the discharge q_{in} or withdrawal q_{out} of water.

3.7. SHARC

SHARC (Sediment and Hydraulic Analysis for Rehabilitation of Canals) is a suite of integrated programs designed to assist in the identification and solution of sediment problems at intakes in rivers and canal systems. Main module components of SHARC model are problem diagnosis and analysis, preliminary economic screening, design tools, hydraulic simulations, environmental impact and economic analysis.

A design tool is one of the main module components of SHARC and deals on Intake models. Design tool has three components namely DORC used for design of alluvial canals, DACSE deals on sediment extractors system and DOSSBAS used for design of settling basins.

DORC is used for design of canals to transport sediments and to determine the existing canals conditions. DORC design appropriate canals to match with the incoming sediment concentration. This method has alternatives of sediment transport predictors like Brownlie, Engelund and Hansen, Van Rijn and White, Paris and bettess. For this study DORC was used for determination of the river sediment concentration (Lawrence et al., 2001).

CHAPTER 4

Data collection and analysis

Introduction

This chapter deals about the data collection and analysis methods followed to achieve the objective of the research study. This chapter includes collection of different design related reports from relevant offices, field observation and assessment of selected schemes and filed data measurements. The selection of schemes for field observation was decided by their year of construction at least one scheme was selected from each schemes which were designed and constructed in the same year with the same design standard. Primary data was collected and analysed from the relatively best performance scheme. The collected data was analysed, interpreted and prepared in the input form for the Delft3D model. DORC module of SAHRC software program was used to develop and estimate the bed material sediment concentration in the river reach.

4.1. Secondary data collection

Secondary data mainly study design report, design specification and scheme locations were collected from relevant organizations of Tigray Water Resources, Mines and Energy Bureau, Mekelle University, Raya Alamata and Raya Azebo weredas or districts. After having this secondary data rough evaluation on the design development in time was made and seven schemes namely Hara, Tirke, Fokissa, Beyru, Tengago, Dayu and Oda were selected for field observation and assessment. Hara, Tirke and Oda modern spate irrigation schemes did not have any report. Therefore analysis was made to this sites based on the current condition in the field and farmers perceptions.

4.2. Schemes visit and observation

An intensive scheme visit and observation was made for the seven selected schemes so as to envision the current situations in the ground. Headwork structures measurement was also made to Hara, Tirke, Fokissa, Beyru, Tengago, Dayu and Oda modern spate irrigation schemes. The field observation was aimed to measure the headwork structures and to observe the practical problems in the field. Structures like intake size, weir dimensions, sluice gates and main canals were measured. This data are used for comparison of design development with other scheme designs. Discussion with local farmers and experts were held in all visited schemes to determine the perception of the beneficiaries.

4.3. Primary data collection and analysis

4.3.1. Topography surveying

The river topography was surveyed using the surveying instrument which is called total station. Total station measures both X, Y, Z coordinates of a point. So as to know the detail topography level of the river a surveying was made for 385 meter long upstream of the diversion structure within seven cross section and 206 meter long of four cross sections downstream of the river. The selection of cross section sites were based on the availability of river bend or meanderings in either side of the river bank. These helps to have a the representative river bank sides. From the surveyed topographic data river cross section and slope were developed. **Table 4.1** shows the topography survey data for cross section five (X-5) and the details of survey data for all cross sections are included in Appendix A

Table 4.1 Topography survey data for cross section five (X-5)

X (m)	Y (m)	Z (m)	$(x_2-X_1)^2$	$(Y_2-Y_1)^2$	$((x_2-X_1)^2 + Y_2-Y_1)^2)^{0.5}$	Chainage
565,391.11	1,379,157.82	1,465.87	-	-	-	0.00
565,396.27	1,379,165.01	1,463.29	26.59	51.75	8.85	8.85
565,409.60	1,379,184.28	1,464.01	177.69	371.14	23.43	32.28
565,426.98	1,379,202.48	1,464.24	302.20	331.28	25.17	57.45
565,437.79	1,379,211.27	1,465.98	116.66	77.35	13.93	71.38

River cross section

The river cross section which was calculated from the survey topography data shows the detail behaviour the river in eleven reaches. This cross section data helps to develop representative topography level of the river cross section and also they are inputs for calculation of river discharges. The selected river reach has wide cross section in upstream and very narrow cross sections in the downstream. **Figure 4.1** shows the sample cross section of Cross section five (x-5). The whole cross section of the river is included in Appendix A

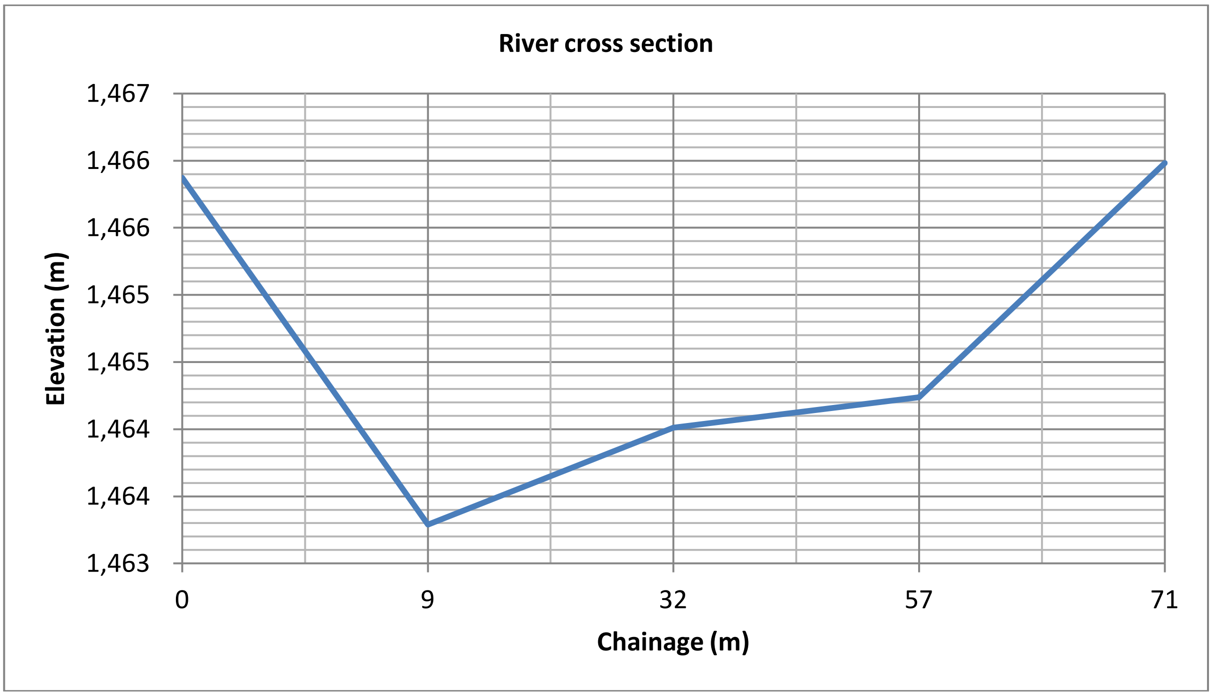


Figure 4.1 River cross section at cross section number five (X-5). (not in scale)

Slope

The slope of the river which was determined from the surveyed river topography shows that the average upstream part of the river diversion has a slope of 0.010 m/m and the average slope of the downstream of diversion was 0.013 m/m. Therefore the total average slope of river reach is 0.011 m/m. **Figure 4.2** shows the longitudinal river slope profile above and below diversion structure. Slope of the river is an important input for discharge calculation when using Manning and Bathurst formulas. It is also important parameter for grid and bathymetry developments.

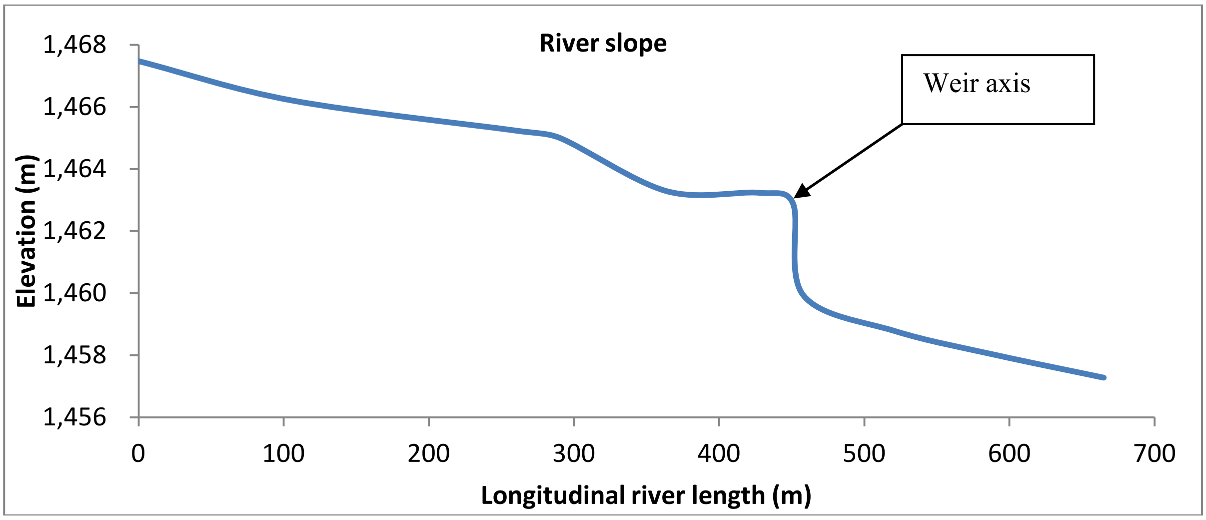


Figure 4.2 River bed slope in longitudinal direction (not in scale)

4.3.2. Head work measurements

All the head work dimensions were measured using a measuring tape. This data were used to generate the detail size and location of the diversion structures like gate size, angle and weir axis length. This data are use full parameters for bathymetry generation and used as an inputs for delft 3D software program.

The measured dimensions of diversion structures are:

- Total weir length 29 m
- Intake width 3 m in the right side of river
- Deflection angle of intake 120^0 to the river flow direction
- 2 Under sluice gate 0.9 m and 2.7 long
- Upstream retaining wall length 28 m in both sides
- Height of upstream retaining wall ranges 2.2 to 3 m
- Weir depth above apron level 3 m
- Apron length 20 m
- Downstream retaining wall length left side 25 m and 30 right side
- Height of downstream retaining wall 4.4 m

Figure 4.3 Measurements of diversion structure dimensions in Dayu scheme

4.3.3. Sediment Grading

To build up a sediment bed material grading eight bed sediment samples were collected from three river cross sections and one in the intake structure. The selection of sites for sediment sample was intentionally decided to include areas of depositions. The sediment samples are located in downstream of diversion, just upstream of weir axis, at the intake and far upstream of diversion. According to the Laurence et al. (2001) for sediment diameter greater than 5mm using of manual sieving in field and for sediments smaller than 5mm taking of around 2 kilogram sample to laboratory for mechanical sieve is the best way of sediment grading estimation method. This principle was used for sediment grading of the study area. Eight sediment samples were digging from 1m*1m*1m =1m^3 hole. As the sediment samples collected are too much it was unrealistic to take them to soil laboratory for mechanical sieves. Therefore it was decided to make manual or hand sieves in the field for the larger sediment sizes. The sieve sizes used in the field are 5mm, 25mm, 50mm and 80mm.

For the sediment size below 5 mm around two kilogram from each sample sites were taken to Tigray Agricultural Research Institute (TARI): Mekelle Soil Research Centre (MSRC) for mechanical sieve analysis. The samples were dried in oven dry for 24 hours at 105 degree cent grade of temperature. After 24 hours the samples was fully dried and prepared for mechanical analysis. Sieve sizes 4.75, 2.36, 2.00, 1.00, 0.50, 0.25, 0.106 and 0.053 mm was used for mechanical sieves. Each sediment samples were shacked for ten minutes at amplitude of 40. The amount of sediment remaining per sieve size was weighed. **Figure 4.4** show the process of manual and mechanical sieving in field and lab respectively.

Figure 4.4 Sediment bed material grading manual sieving at field (left) and mechanical sieving at laboratory (right)

The sediment grade data collected from laboratory and field was merged and prepared to create one sediment grading curve. **Figure 4.5** show the sediment grading graph curve for the study area. The sediment grading graph was drawn with a log scale in horizontal level and normal scale in the vertical direction.

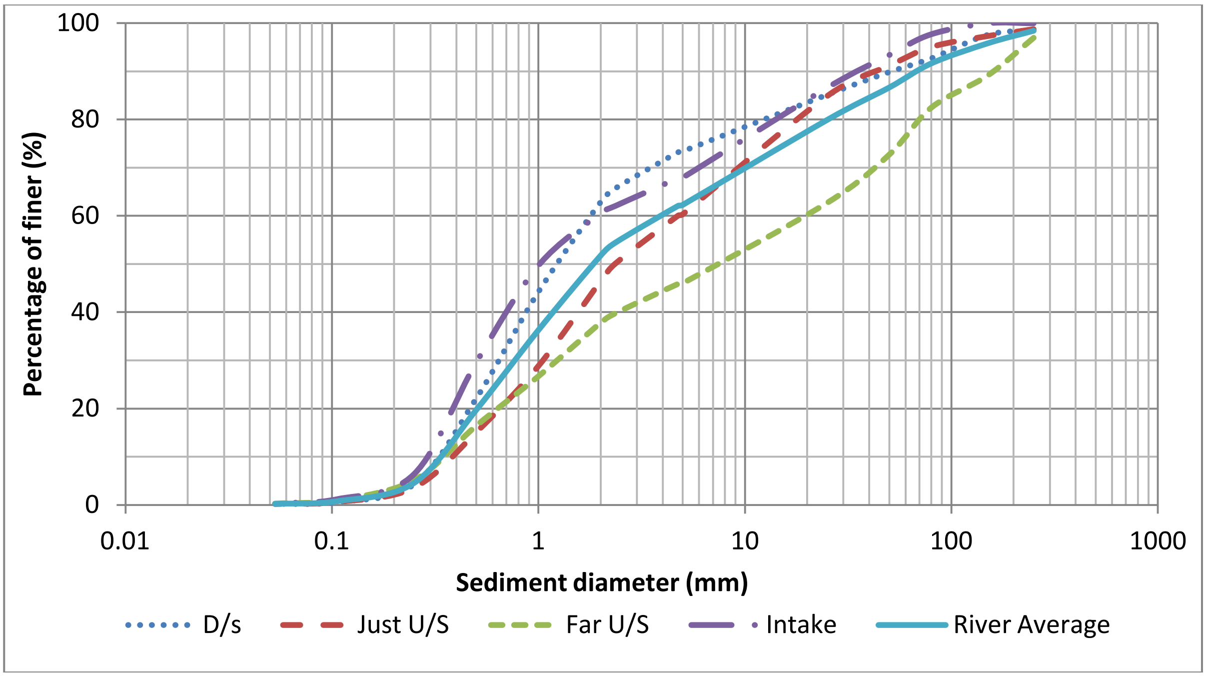


Figure 4.5 Sediment grading graph at different cross sections of the river

As we can see from **Figure 4.5** sediment sample collected from the far upstream are coarser than the others while the downstream and intakes are also relatively fine materials. The graph of sediment grading shows that the median diameter of the bed sediment material in the study area is 1.8 mm. According to the soil classification of American geographical union a soil which have a median diameter of 1 - 2 mm is grouped as very coarse sand. Therefore the sediment of the river Dayu will also considered as coarse sand.

4.3.4. Flood marks

The flood level marking on the bank of river was undertaken in the discussion held with experienced farmers and water use association leader so as to put the average depth of minimum, medium, average maximum and extreme flood flow conditions. Three representative cross sections were selected from the downstream, middle and upstream of the river reach. These floods marks are used for determination of river discharge **Table 4.2** show the flood mark levels at three cross sections marked by farmers.

Table 4.2 flood marks of the river at three cross sections.

X section	Bank side for marks	Flood type	Depth (cm)
X-1	Right bank	Low	52
		Medium	85
		High	145
		Extreme	166
X-5	Right bank	Low	55
		Medium	115
		High	182
		Extreme	213
X-11	Left bank	Low	60
		Medium	178
		High	335
		Extreme	375

4.3.5. River discharge

Since there is no discharge data on the study area a slope area method of Manning`s and Bathurst formula were employed to determination or estimate river discharge. The main inputs for these equations are manning roughness coefficient, river cross section and slope of the river reach.

$$Q = \frac{1}{n} * \ A * R^{2/3} * \ S^{1/2} \qquad \text{Manning}$$

$$Q = A * D^{*} * (g * R * S)^{1/2} \qquad \text{Bathurst, 1985}$$

$$D^{*} = \left(5.62 * \log\left(\frac{d}{D_{84}}\right) + 4\right)$$

Where:

Q = Discharge in m^3/s
A = Cross section of river in m^2
R = Hydraulic radius which is area per wetted perimeter in m
S = River slope in m/m
n = manning roughness coefficient
d = mean flow depth, similar with R
D_{84} = size of bed material with 84 percent is finer in m
G = acceleration due to gravity m/s^2

Using the flood marks of three different cross sections and other river parameters like slope, manning coefficients the discharge for each flow levels were calculated using the Manning`s and Bathurst formula. **Table 4.3** presents discharge calculation using Manning`s and Bathurst formula at cross section one. The discharge of cross sections 1, 5 and 11 are also included in Appendix C in details. And finally the average discharges of the three cross sections were used as the discharge of the river flow.

Table 4.3 Discharge calculations in cross section one (X-1)

	River cross Section one (X-1)									
Flow type	Head (m)	Area (m^2)	Perimeter (m)	R (m)	S (m/m)	n (-)	Manning`s Q (m^3/s)	D_{84} (m)	D* (-)	Bathurst`s Q (m^3/s)
Minimum	0.52	5.84	22.32	0.26	0.011	0.030	8	0.04	8.58	8
Medium	0.85	26.68	72.97	0.37	0.011	0.030	48	0.04	9.39	50
Maximum	1.45	75.85	88.33	0.86	0.011	0.030	239	0.04	11.48	265
Extreme	1.66	95.03	96.54	0.98	0.011	0.030	329	0.04	11.81	366

According to Arcement and Schneider (1989) the roughness coefficient (n) of gravel with median size of bed material 1-2 mm ranges from 0.026 - 0.035. As the median size bed sediment material of the study area are 1.8 mm an average roughness coefficient of 0.030 was used for the calculation of discharge using Manning formula and an analysis of discharge sensitivity analysis was made to different roughness coefficient in the range value of coarse sand bed material as presented in **Figure 4.6**

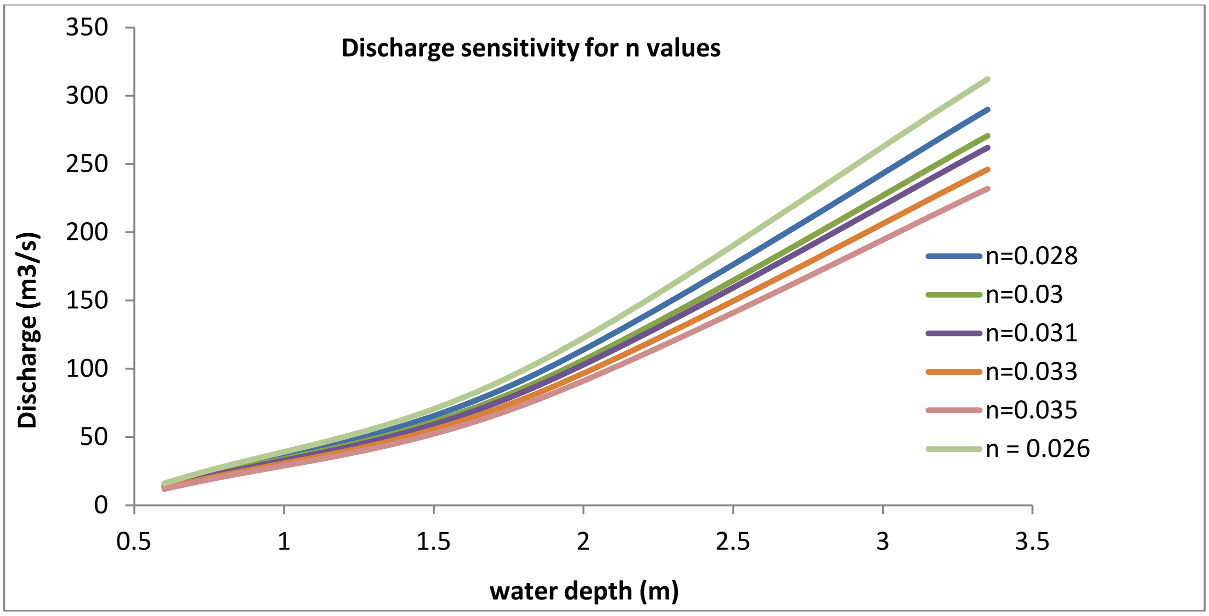


Figure 4.6 Discharge sensitivity at different values of roughness coefficient (n)

According to Arcement and Schneider (1989) for area which have know median size bed material it is better to use Bathurst formula instead of Manning equation. Bathurst formula is independent of Manning roughness coefficient and mainly depend on the D84 of bed materials. The result shown in the sensitivity analysis of roughness coefficient reveals that the discharges are significantly affected by the roughness values. Therefore it was decided to use the discharge value calculated from Bathurst formula for further inputs of discharge. Table 4.4 show the discharge calculated from three cross section using Bathurst formula.

Table 4.4 Average discharges of the river for different flow type

Flow type	River discharge in (m^3/s)			
	X-1	X-5	X-11	Average
Minimum	8	8	15	10
Medium	50	66	94	70
Maximum	265	272	306	281
Extreme	366	406	378	383

4.3.6. Hydrograph

Like the discharge data there is no data about the flow hydrograph for the study area as well. Therefore a hydrograph was generated based on the concept of Camacho (1987) the characteristics of and the discussions of the farmers. According to Camacho (1987) spate flow hydrograph is characterised by rapid rising limp and sharply decreasing recession limp with in very short period of time. As the main objective of this study is to develop and evaluate design alternatives in relation to spate flow and sediment management the flow hydrograph was developed for minimum, medium and maximum floods only. The extreme flood occurs rarely and it is better to not consider in spate flow and intake hydrograph estimation. But the structures of the diversion have to be design based on at least fifty year return period of the extreme flood events. The farmer's observation to the hydrograph was collected from the discussion held with elder and experienced local residents. The flood frequency and for minimum, medium and maximum flood events are presented in table **Table 4.5**.

Table 4.5 Farmers observation to river flow hydrograph

Flow type	Flood frequency per year (no.)			Duration time in (hr)		Remark
	Wet season	Average season	Dry season	To peak	end	
Minimum	30	19	11	1	5	
Medium	16	11	7	1	8	
Maximum	8	7	4	1	12	

As can be seen from **Table 4.5** the flood in Dayu river occurs for 5, 8 and 12 hours for minimum, medium and maximum floods respectively. Using the calculated river discharge, flood frequency and Camacho (1987) principles a flood flow hydrograph was developed for minimum, medium and maximum flood events as shown in **Figure 4.7**.

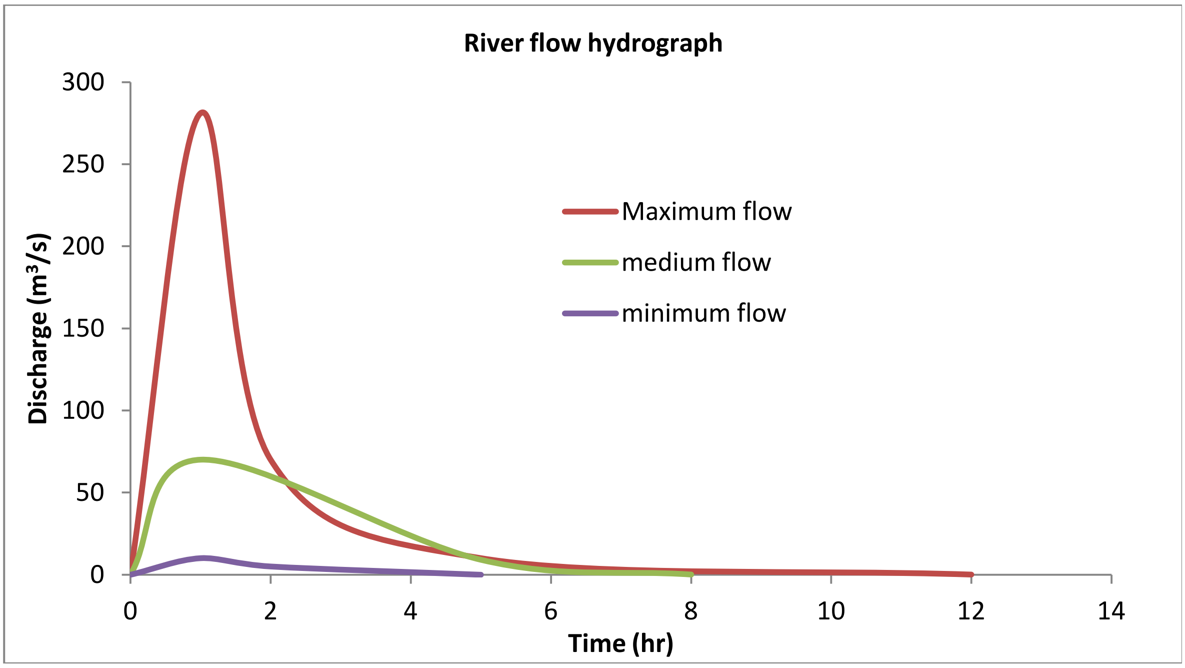


Figure 4.7 River flow hydrograph of dayu spate scheme

4.4. Irrigation water requirement

As the incoming flood is not certain farmer did not have a formal irrigation scheduling. They simply distribute the incoming flood to farms starting from the upstream up to the flood cease. In fact sometimes they are flexible and willing to give priority for sensitive crops. Most of the time farmers try to divert more water as much as the canal capacity. Depending on the time and magnitude of flood occurrence farmers grow different crops.

According to the discussion made with farmers main crops grown in Raya valley are sorghum and teff. The selections of crop for cultivation depend on the existence of flood. For example if the flood comes from end of April to June farmers prefer to grow sorghum whereas if the flood comes in late June to July they prefer to grow teff. During wet season which means when flood comes in April nearly 100% of the land cultivated by sorghum, in medium season 50% sorghum and 50% teff while in dry season 100% of the scheme could cultivated by teff.

According to Steduto et al. (2012) the crop water requirement ranges 500 mm -800 mm for sorghum and 450 mm - 550 mm for teff. The rain fall in the study area is uncertain therefore it is better to omit during calculation of irrigation water requirement. For this reason the irrigation water requirement for the spate irrigation scheme is assumed to deliver from the flood water and the calculation was based on full irrigation system.

The main factors which can affect irrigation water requirements application and conveyance efficiency. Research work conducted by Khaliq (1980) shows that the application efficiency of non perennial canals would reach 60%. According to Hatem (2007) as cited by Gebrehiwot K.A. (2013) the conveyance loses of an earthen canal system could reach up to 3.3% per kilometre. The total length of main canal in Dayu spate irrigation system is about 5 kilometres and this could result a total conveyance loss of 16.5%. Combining both efficiencies, the total system efficiency spate irrigation scheme will be 50%.

The total volume of water needed for sorghum and teff during the whole growing season was calculated based on the crops water requirements and the combined system efficiency. This was calculated for one hectare.

Table 4.6 Irrigation water requirement of sorghum and teff

Crop type	Net irrigation requirement (mm/season)	Average net irrigation (mm/season)	Total efficiency	Gross irrigation (mm/season)	Total volume (m^3/ha/season)
Sorghum	500 - 800	650	51%	1,275	12,750
Teff	450 - 550	500	51%	980	9,800

Table 4.6 show the amount of water needed for both sorghum and teff crops per hectare per season. The calculation for total volume of water needed for the scheme have based on this unit value.

4.5. Sediment concentration

As there is no data about the sediment concentration in the study area it was tried to generate sand sediment concentration from SHARC software program, DORC module. The main inputs of DORC model are bed material size and river hydraulic parameters like velocity and flow depth. The river hydraulic data of depth and velocity are estimated from DORC model alluvial friction predictor part. The alluvial friction predictor part of DORC has alternative methods of predictor of Brownlie, Engelund and Hansens, Van Rijn and White, Paris and Bettess. Comparison was made to all methods of alluvial friction predictor and observed or calculated data of the river.

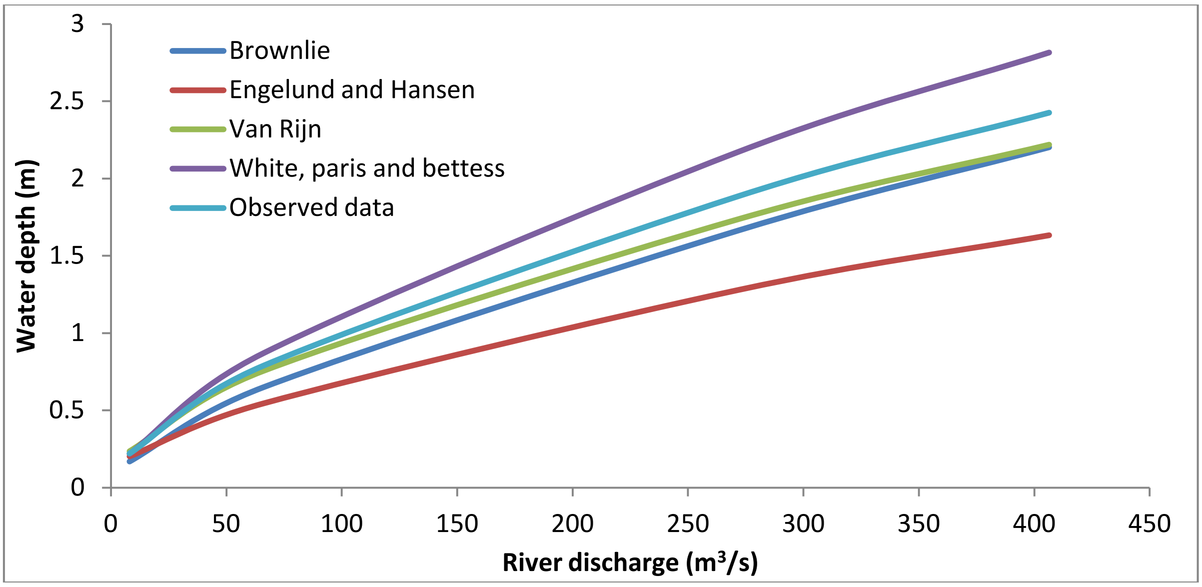


Figure 4.8 Comparisons of depth estimates from different alluvial friction predictor of DORC model

As can be seen in **Figure 4.8** Van Rijn method is relatively best fitted to the observed data. Therefore Van Rijn alluvial predictor method was used for generation of depth and velocity of the river to estimate the sand sediment concentration.

According to comparison made by Lawrence et al. 2001 to different predictors of sediment transport Engelund and Hansen methods is the best in areas which have not enough data of sediment concentrations. Hence, the sand transport prediction was made using the Engelund and Hansen`s of sediment transport predictor method to get sediment load concentration of the river in parts per million (ppm). A simple power relationship ($Qs = 1061.2Q^{0.7134}$) which have R^2 values of 1 was developed to estimate sediment transport concentration for different discharge levels. **Figure 4.9** Shows the estimated sediment concentration curve and power sediment concentration curves.

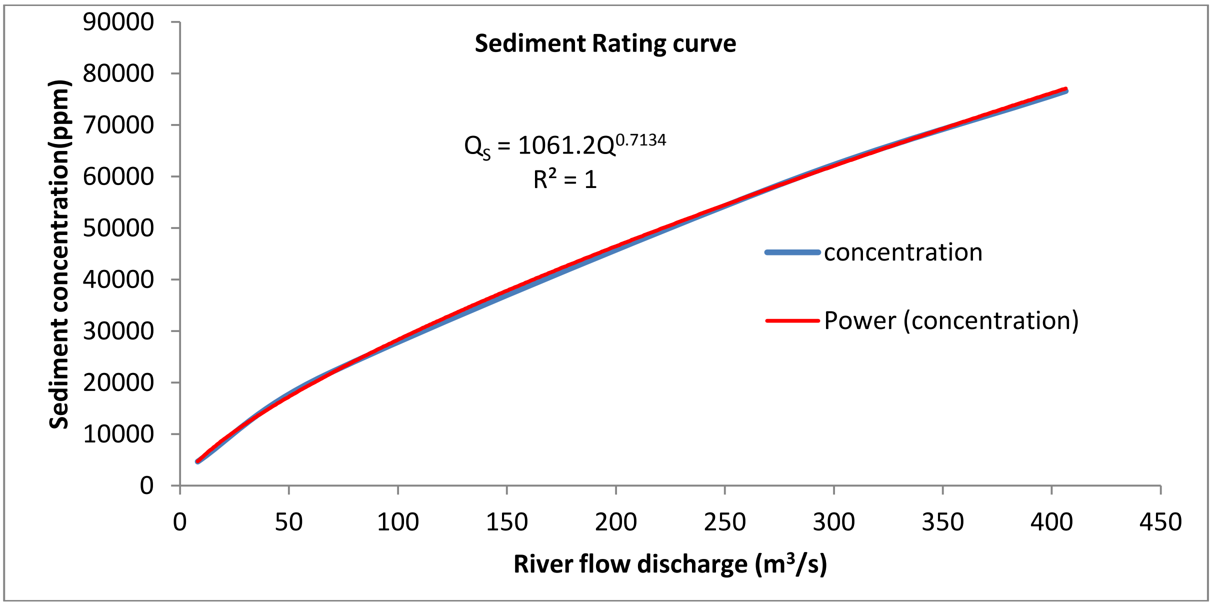


Figure 4.9 Estimation of sediment concentration

CHAPTER 5

Model setup

Introduction

This chapter deals about Delft3D model setup and input data preparation for simulation of river hydrodynamics and morphology. The main activities discussed in this chapter are Grid generation, bathymetry preparation, Delft3D model flow parameters, model calibration and description of the developed scenarios.

5.1. Grid generation

The grid of the river was developed based on the collected topography data. As discussed in chapter 4 the topography data was collected for 591 long river reach which is the 385 meter is in the upstream and 206 meter is downstream of the existing diversion structure. From the collected data of easting (X) and nothing (Y) land boundary of the river were created from each cross section. This data were organized in notepad and saved as *.ldb. In the file menu of Delft3D-RGFGRID window the land boundary was opened from attribute files.

Splines were building up along and across the land boundary so as to divide the river bed part in to small sample grids. From Delft3D-RGFGRID window, operation menu of the splines were changed into grid. In the areas which needs further refinement like in the weir and intake; refine and derefine grid locally were used and refined/derefined until the desired grid size delivered. After having this river grids an intensive smoozing and orthogonalise was made so as to make the river grid smooth and orthogonalise.

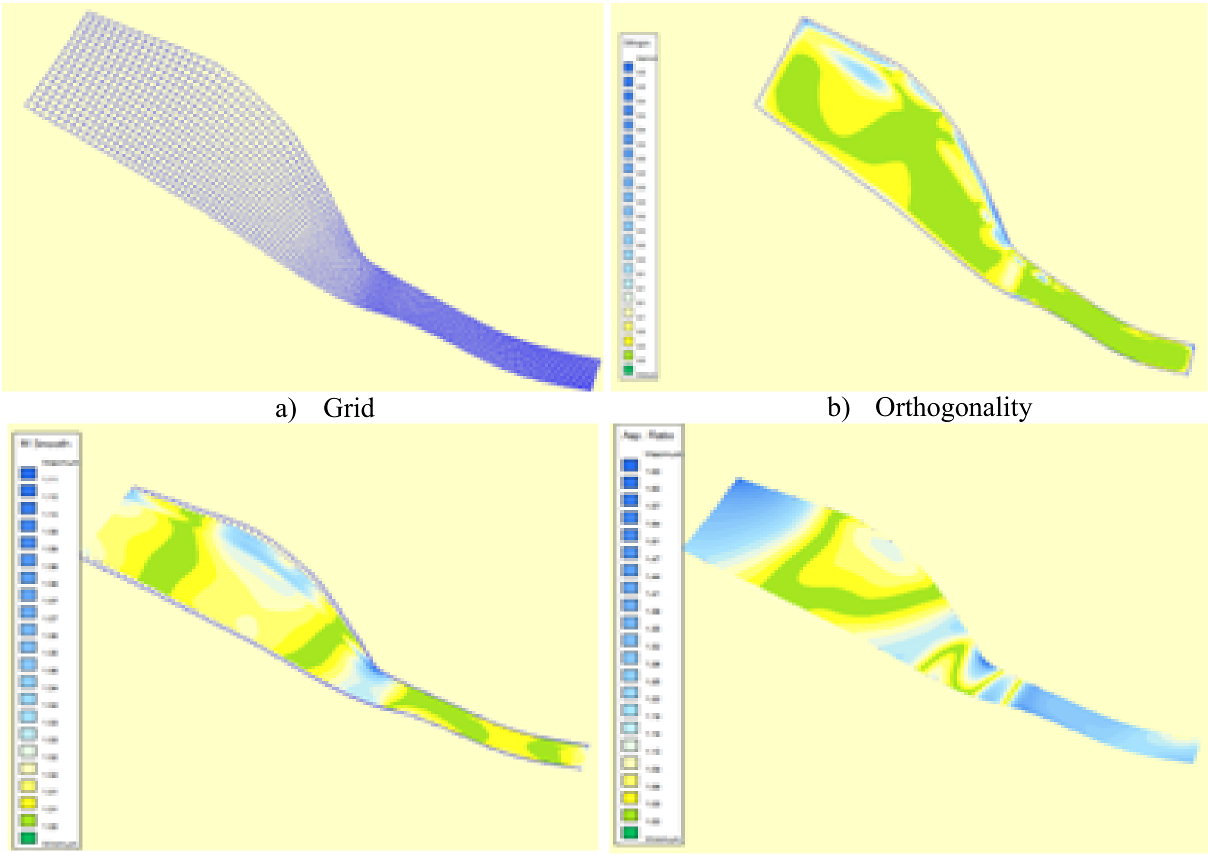
The grid properties of the developed grid mainly Orthogonality, N smoothness, M smoothness, and Aspect ratio parameters were checked to the standard and limit values. **Figure 5.1** shows grid generation and their properties.

The grid properties are describes as follows:

- Orthogonality ranges from 0 to 0.04 all over the grid
- M smoothness ranges from 1 to 1.11
- N smoothness ranges from 1 to 1.08
- M sizes ranges from minimum of 1.41m to maximum of 8.21m
- N sizes ranges from minimum of 1.62m to maximum of 8.15m
- Aspect ratio ranges from 1 to 1.63

Table 5.1 Mminimum requirement and achieved results of grid properties

Grid property	Minimum requirement	Achieved
Orthogonality	0 - 0.04	0 - 0.04
M smoothness	1 - 1.2	1 - 1.11
N smoothness	1 - 1.2	1 - 1.08
Aspect ratio	1 - 2	1 -1.63



a) Grid b) Orthogonality

c) M Smoothness a) Aspect ratio

Figure 5.1 Grid development and grid properties

5.2. Bathymetry

The bathymetry of the river bed was also prepared from the collected XYZ topographic data. The XYZ coordinates of the river cross sections were prepared in notepad and saved as samples file (*.xyz). The samples file was imported in to Delft3D-QUICKIN window so as to put the coordinate points on the river.

As the sample points are few in number a triangular interpolation method was used employed to interpolate the depth or elevation values to each grid cells. Internal diffusion features also used to interpolate grid cell points which are not interpolated in triangular interpolation. After the internal interpolation and internal diffusion the complete bathymetry or depth with reference to mean sea level of the river was delivered. In Delft3D model a depth above reference level considered as negative while depth below reference levels considers as positive value. In this study area the elevation of river bed level is above the reference level; therefore the bathymetry of the river considered and saved in negative sign. **Figure 5.2** shows the developed river bathymetry.

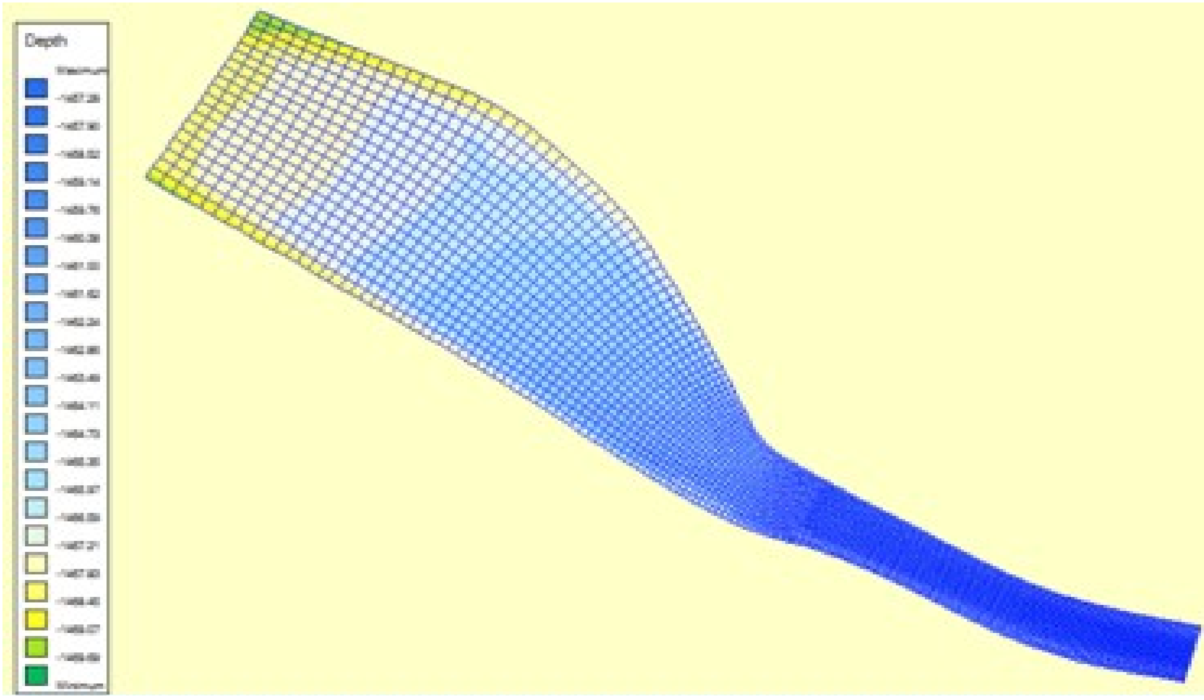


Figure 5.2 River bathymetry of Dayu

In the Delft3D-QUICKIN window, after having the complete bathymetry data the initial conditions were also developed accordingly. The main data developed are initial water levels for simulation, initial sediment thickness and initial velocities for U and V directions and all initial conditions were saved accordingly.

5.3. Delft3D flow module parameter

The flow module parameter of Delft3D was described as follows

Description

In the Description part of flow module a details of the model simulation can present. In this study the name of simulation for different scenarios was written as description. The description is used to give detail identification of the model simulation and does not have impact on the result of simulation. Different identification names were given for minimum, medium and floods during simulation of all developed scenarios.

Domain

The domain part of the module includes input information of grid parameter, bathymetry, dry points and thin dams. For this study the input files of grid and bathymetry was loaded accordingly from the folder where they were saved. As there is no need of dry points and thin dams for this river this parameters were left free.

Time frame

The model runs for high, medium and low flow rates. Reference date, simulation start and stop time were filled in this part. The time step for simulation was selected based on the recommended value of model which is Courant number. To certify table and accurate flow and sediment transport simulation applicable simulation time step was selected based on the Courant (Friedrichs-Lewy) number (CFL). This can define by:

$$\text{CFL}=\frac{\Delta t\sqrt{gH}}{(\Delta x,\Delta y)}$$

Where:
CFL= Courant (Friedrichs-Lewy) number
Δt= is time step in seconds
g= gravitational acceleration
H= total water depth
Δx or Δy = is the minimum value of the grid size in either direction (Hydraulics, 2011)

Based on the formula and recommended value ranges the time step used for simulation are 0.01, 0.008 and 0.002 minutes for low, medium and high floods respectively.

Processes

Under the process part different constituents, physical and man-made can be selected. The objectives of this study are simulation of spate flow and sedimentation. Therefore sediments and secondary flow was selected under process parameter. The box of sediment and secondary flow was ticked so as to make them active and inputs were introduced in the next parts.

Initial conditions

The initial conditions developed in Delft3D-QUICKIN were used. This initial condition includes initial water level, initial water velocity in U and V direction and initial sediment thickness.

Boundary conditions

The model has three boundary conditions, upstream with discharge in time serious, downstream and intake with discharge head relations. Flow hydrograph of the river was used for boundary conditions. A sediment concentration estimated using SHARC-DORC model was used in both upstream and downstream as transport boundary conditions.

Physical parameter

In the physical parameter some physical condition are defining like constants, roughness, viscosity, sediment and morphology. Parameters used for this study are:

- Roughness formula manning with uniform value of 0.03,
- Horizontal eddy viscosity = 1 m^2/s
- Horizontal eddy diffusivity = 0.1 m^2/s
- Median sediment diameter = 1.8 mm
- Initial sediment layer thickness = prepared file in QUICKIN
- Spin-up interval before morphological changes = 120 minute
- Morphological factor = 1
- The other parameters were kept as default.

There are so many sediment transport formula capable of estimating sediment transport in flowing water. These formulas have their own mode of motion and uses different parameters. The main formulas are Van Rijn (1993), Engelund and Hansen (1967) and Meyer-Peter-Muller (1948). This formulas has their own range of diameter sizes. For example Van Rijn (1993) works in the range of 64 to 2000 micrometer

The median diameter of sediment of the study area is 1800 micrometers or 1.8mm which is in the range Van Rijn 1993 sediment transport formula. It was decided to use the default sediment transport formula of Delft3D which is Van Rijn (1993) and this includes both bed load and suspended materials.

Numerical parameters

The following numerical parameters were used:

- Drying and flooding check at Grid cell centres and faces
- Depth specified at Grid cell centres
- Depth at grid cell centres Mean
- Threshold depth 0.01 meter

Operation

As there is no recorded discharge data for the intake nothing was introduced operation

Monitoring

In the monitoring parameter, 12 different observation points along the river especially in the upstream, river intake and downstream were selected.

Additional parameters

In the additional parameter group it is possible to add some information which is not incorporated on the GUI. This parameters can include sediment transport formula, weir location and dimension and others if applicable Delft3D manual 2011. For this study a weir structure was included. The weir location and dimension file was created and saved in the directory as 2D weir. Commands (in Keyword, Fil2dw and in value, #weir.2dw#) were given in the additional parameter so as to realize the model the existence of the structure during simulation.

5.4. Model calibration

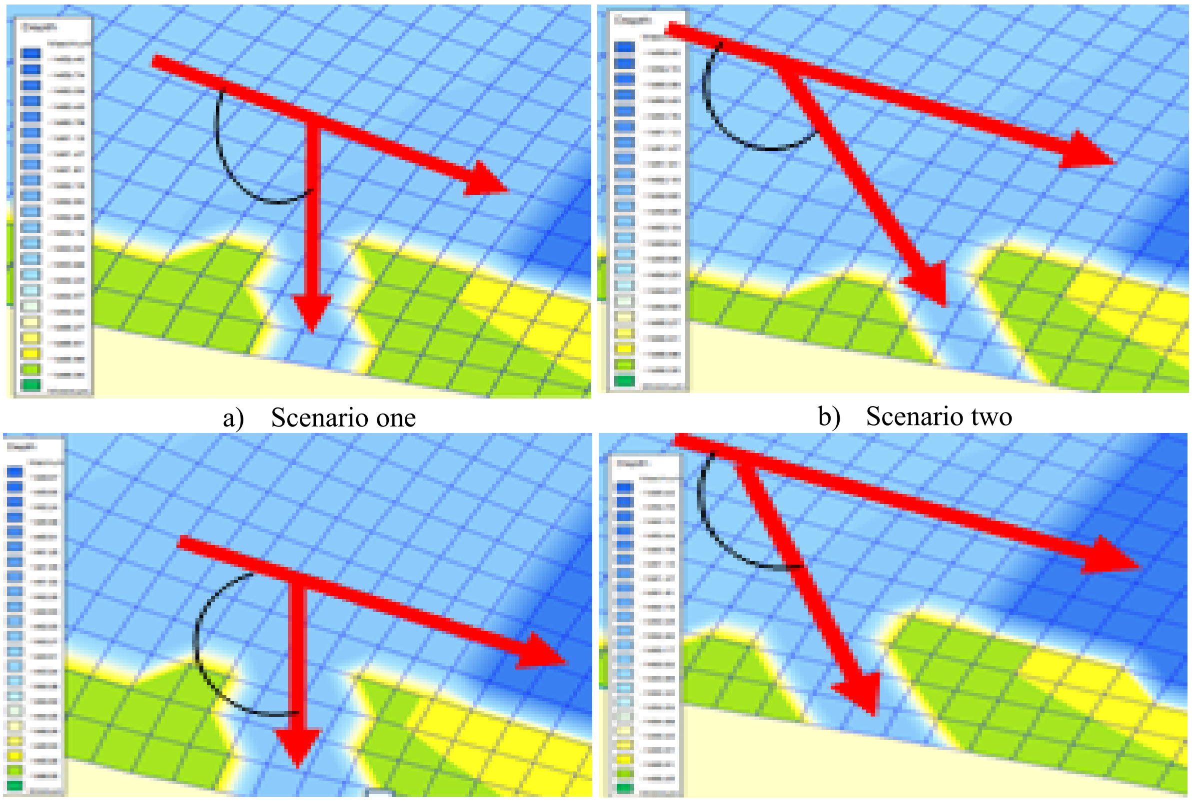
Like many spate irrigation schemes in the world, schemes of Raya valley are also have shortage of recorded data. There is no enough data for calibration and validation of Delft3D model for the specific location. Nevertheless, a research work conducted by Toska and Zenebe (2012) in Gash river of Sudan, shows that Delft3D model is very good tool for simulation of flow and sediment in spate irrigation systems at different magnitude of spate river discharges. As most of the spate irrigation systems shares common characteristics like high sediment concentration and high flood discharge in short period of time it was assumed the model will work for this study area as well.

5.5. Scenarios

As the major problems of low water abstraction and sediment accumulation around the intake structures three alternatives of intake designs were developed in addition to the existing condition. The detail development of scenario will discuss in 6.3. **Figure 5.3** show the developed layout of scenarios

The combinations of these two factors are describes as below:
Scenario one (S1) = Three meter gate size, 120^0, deflection angle = Current condition
Scenario two (S2) = Three meter gate size, 150^0, deflection angle
Scenario three (S3) = Five meter gate size, 120^0, deflection angle
Scenario four (S4) = Five meter gate size, 150^0, deflection angle

Each scenarios lay out was saved as bathymetry and simulated for hydrodynamics and morpho-dynamics



a) Scenario one

b) Scenario two

c) Scenario three

d) Scenario four

Figure 5.3 Layout of the developed scenarios

CHAPTER 6

Result and Discussion

Introduction

This chapter presents the results of the research work and discusses the current conditions and developed scenarios. To evaluate the developments of spate irrigation structure schemes in the past 15 years relevant data collected from respective offices. Preliminary assessment was made to the collected data; representative sites were also selected for field observation according to their year of construction and design standards. One relatively best performing scheme was selected for further problem investigation.

According to the data collected and filed observation Dayu spate irrigation scheme was found relatively good performing schemes. Therefore this scheme was selected for further research study to investigate the problems and limitation of the headwork design structures. Alternative headwork designs structures were developed as possible scenario. The scenario design is based on the current problems and includes idea and perception of farmers and experts.

6.1. Design development of spate irrigation structures

Modernization of spate irrigation schemes in Raya valley starts in 1998. Hara was the first modernized spate irrigation scheme in the area and that leads to many improvements in the designing and constructions of modern schemes in Tigray. Tirke spate irrigation scheme was also modernised in 2004 following to Hara scheme. In 2005 four schemes namely Fokissa, Beyru, Utu and Burka was designed and constructed while Ula-ula, Buffie, Tengago and Dayu schemes were constructed in 2006. According to Embaye et al. (2013) forty schemes were designed in 2010. While, 13 of them were constructed in the same year none of them performed well.

The design standard of Hara and Tirke was directly adopted from the conventional irrigation schemes which have low sediment concentration. The headwork of this two spate schemes has gated off take or intake with broad crested weir and all the structures were made up of concrete masonry. Hara and Tirke schemes were failed in one rainy season due to problem of sediment in both intakes and canal systems. **Figure 6.1** shows the modernized headwork structures of Hara and Tirke spate irrigations.

Intake structure of Hara scheme | Under sluice gate in Hara scheme

Crossing structure, Hara spate scheme | Orifice intake, Tirke (adopted from Embaye, 2013)

Figure 6.1 Headwork structures of Hara and Tirke

According to the farmers perception the main reason for failure of these schemes was the inappropriate design structure of intakes. During construction the farmers were complaining about the size, shape and deflation angle of the gate. According to field observation and a report made by Embaye et al. (2013) the intakes of Hara and Tirke has 90^0 deflection angles from the river flow direction and less than one meter diameter of gate.

In 2005 when Fokissa, Beyru, Utu and Burka was designed and constructed the design engineers took key lesson from the failure of Hara and Tirke. They came to realize that the incoming sediment or bed material load was too high. Hence, they decided that gated intake, narrow canal and siphons cannot work as structures of spate scheme. At that time the designers tried to know the indigenous knowledge of farmers for sediment managements and they observed some traditional irrigation schemes in the valley.

The major findings of farmer's knowledge were wide open gate intake with an angle of deflection greater than 90^0 and wide size of canals. To some extent experts tried to understand and incorporate farmers traditional knowledge during design and construction. They took good lesson on size and deflection angle of intake and they tried to give attention for sediment problems.

The major changes of the design include;

- To change the gated intakes to open gate
- To increase the width of the intakes and canals
- Improving of diversion angle from 90^0 to 120^0

- Avoiding of crossing structures

Weir axis of Fokissa scheme

Demolished intake Fokissa scheme

Weir axis Beyru scheme

Open intake Beyru scheme

Figure 6.2 Headwork structures of Fokissa and Beyru spate schemes

Figure 6.2 show the improved diversion structures of Fokissa and Beyru. Although these improvements are good and show better performance but the designs of Fokissa, Beyru, Utu and Burka still did not perform as the intended.

The main problems or limitations during these designs are:

- The crop water requirement (CWR) was calculated for 24 hours while flood occurrence is too short'
- Effective rainfall was considered during irrigation water requirement calculation (IWR) which leads to underestimation of net irrigation water requirement (NIWR) but rain fall is not reliable.
- As the width of the gates ranges from 1m to 3m depending to size of irrigable area but the farmers were still complaining as they were thinking even 3m gate is small.

A survey was made by Tigray water resources, mining and energy bureau aimed to monitoring and evaluation of spate irrigation in Raya valley. This study covers six spate irrigation schemes which were designed and constructed starting from 1998 up to 2005. The study gives the following major conclusion and recommendations.

- Understanding the experience, wisdom and tradition is necessary during design and construction of spate irrigation
- Inappropriate design parameters of intakes and canals are the main cause of failure.

- Reconsideration for appropriate diversion design have to made
- Frequent supervision and giving training to beneficiaries is necessary.
- Multiple intakes along the river reach could increases the efficiency of flood water management

In 2006 four spate schemes were designed and constructed namely Ula-ula, Buffie, Tengago and Dayu schemes. In addition to the design improvement takes place in 2005 some improvements were made based on recommendations of supervision. These improvements try to solve the limitations and problems occurred in the design of schemes made in 2005. **Figure 6.3** presents the headwork structures of Tengago and Dayu modern spate irrigation scheme. According to Embaye et al. (2013) the main design improvements for Ula-ula, Buffie, Tengago and Dayu schemes are;

- The calculation of crop water requirement was minimized to 4 hours
- Effective rainfall was neglected during net irrigation water requirement calculation
- The schemes design was limited to headwork and main canals.

Diversion structure of Tengago scheme

Sluice gate blocked by gabion, Tengago

Diversion weir in Dayu spate

Silted intake, Dayu

Figure 6.3 Headwork structures of Tengago and Dayu spate schemes

From 2006 - 2010/11 there was no design development in Raya valley spate irrigation system. In 2011 Oda spate irrigation was designed with some improvements to traditional spate system, it was designed as a simple intake using gabion and only cut offs built to reduce the risk of bed level lowering around the river bed and intake (Embaye et al., 2012).

During the field visit to Oda spate irrigation it was found that the weir or cut off structure was completely destroyed by flood hazard. According to the farmers perception Oda scheme was failed before handed over to users just during completion of construction work. Now farmers are using in traditional way using forest and shrub embankment. **Figure 6.4** shows the failed weir axis and reconstruction of scheme in traditional system

Broken diversion structure, Oda

Embankment using forest and shrub, Oda

Figure 6.4 headwork structure of Oda spate irrigation system

After 2011 there was no sound change in design development of spate irrigation schemes. Few schemes were constructing by the wereda or district of Raya Azebo and Raya Alamata bureau of water resources, mines and energy. Most of these schemes are simple and small structures and they are exposed to flood hazards.

The major design development made for spate irrigation system in Raya valley are summarised as shown in**Table 1.1**.

Table 6.1 Summery of spate structures design development

Parameters	Schemes				
	Hara	Tirke	Fokissa	Tengago	Dayu
Year of construction	1998	2004	2005	2006	2006
Design flood discharge	-	-	220.5	50.0	358.89
Weir length	35	34	35	23	29
Intake type	Closed gate	Closed gate	Open gate	Open gate	Open gate
Gate size	0.8X0.8	0.9m diameter	3 m	2.5 m right & 2.m left side	3 m
Deflection angle	90^0	90^0	120^0	120^0	120^0
Main canal system	concrete	Concrete	Concrete	Concrete	concrete
Crossing structures	Available		Avoided	Avoided	Avoided
Assumed irrigation time	24 hrs	24 hrs	24 hrs	4 hrs	4hrs
Effect of rainfall	Considered	Considered	Considered	Neglected	Neglected
Designed ha	400	380	500	500	320
Current ha	0	0	100-150	<50	150
Over all status	Failed	Failed	Poor	Poor	Good

The relatively best performed modern spate irrigation system in Raya valley was the one designed and constructed in 2006 namely Ula-ula, Buffie, Tengago and Dayu schemes. Renovations have been taking for these schemes to minimize the structural damages and sedimentation problems. Among this schemes Dayu spate irrigation scheme were found relatively best performing scheme. Therefore, this scheme was selected for further study.

Comparing Tengago and Dayu spate irrigation schemes Dayu is relatively best performing. The reason for this could be the difference of river flood discharge. As we can see from **Table 6.1** the design flood discharge for Tengago is 50.0m^3/s while 358.89 is flood discharge of Dayu. Even though the river discharge is small but Tengago was designed to irrigate 500 ha with two intakes in one diversion structure. The structures of Tengago are still in good conditions while there are accumulations of sediments around both intakes. Therefore designing of 500 ha to a river which has 50m^3/s is not optimum and this could be the reason for poor performance.

6.2. Problems of best schemes in relation to sediment management and spate flow

Dayu spate irrigation scheme is the relatively best performed scheme in Raya valley while it is irrigating about half of the designed area. The main structural problem of Dayu in relation to sediment management and spate flow are;

- Siltation problems both in intakes and main canal
- Diverted amount of water through in intake is small
- In small flood it is difficult to convey water through intake as too much sediments are accumulated in the intake than in weir.

6.2.1. Causes of the problems

Farmer's perception

According to the farmer's perception the main cause of the structural failure to modern spate irrigation systems are;

- Narrow intake and canal width
- Angle of intake deflection
- Existence of sluice gate; it is not good because it can lost many floods

Designers and experts perception

According to the discussion held with designers and professional experts of spate irrigation system the main cause could be lack of good operation and maintenance in addition to lack of inappropriate design. As there is no known standard for spate irrigation system most the decisions for all structural design are by trial and errors. The experts are still not confident on the size and angle of intakes which they have been designing for years. In the other way round the experts are not convinced by the farmers complaining about the existence and functionality of sluice gate. Sluice gate is important parameter for sediment control. Opening of sluice gate during high flood helps to erode the accumulated sediments around intake. In low flood is must be closed so as to rise the water level and divert more water. Therefore the existence of intake could not be a problem but it needs care full management and frequent supervision.

6.2.2. Remedial solutions for the problems

From farmers point of view

Based on the farmers indigenous knowledge most of traditional irrigation system are characterised as wide intake width up to 6 meter wide, the angle of deflection are greater than 120^0 in some area they can make it near to 180^0 which is parallel to the river flow and mostly they use temporary and small solid weir or barrage to clot the flow along the river and divert to earthen canal. For the modernised schemes the farmers put the following remedial actions;

- Width of intake have to be up to 5 meter
- Angle of deflection have to be more than 120^0 deflected
- The weir must be without sluice gate

From expert point of view

The design experts of spate irrigation system are keen to know the impact of different deflection angle and width length on sediment management and spate flow. Therefore the possible remedial solution in relation to sediment management and spate flow could be;

- Width of intake 3m or 5m
- Deflection angle 120^0 or 150^0

The improved design alternatives of spate irrigation system should have to incorporate the perception of farmers and experts.

6.3. Scenario development for alternative main intake design

The major problems are low water abstraction and sediment accumulation due to inadequate design standard. Three alternatives/scenarios of intake designs were developed in addition to the existing/current condition. This scenario was developed aiming to solve the problem of current condition. The main reasons for selecting the scenarios were based on the farmer's traditional knowledge's and engineer's experiences. The developed scenarios are related to deflection angle of the intake from the river and width of the open gate intake. **Figure 5.3** show the layout of developed scenarios.
Therefore the scenarios for the improved design of spate irrigation system are;

- Gate size: 3m and 5m
- Deflection angle: 120^0 and 150^0

The combination of the two factors will be 2*2 = 4 scenario

Table 6.2 Combination of design factors or scenario

		Gate size (m)	
		3	5
Angle (^{0})	120	S1	S3
	150	S2	S4

Scenario description

S1 = Three meter gate size, 120^0, deflection angle this is the existing design
S2 = Three meter gate size, 150^0, deflection angle
S3 = five meter gate size, 120^0, deflection angle
S4 = five meter gate size, 150^0, deflection angle

6.4. Simulation results of design alternatives

Delft3D model was run to simulate hydrodynamic and morphodynamics for low, medium and high flood events to all possible scenarios. The major hydrodynamic and morphodynamic parameters considered for result presenting and discussed in this study are flow patterns or depth average velocity, water level, and cumulative erosion/sedimentation. The result of hydrodynamics and morphodynamics are presented and discussed here under for all possible scenarios.

6.4.1. Scenario one (S1)

Flow patterns

The flow pattern around the intake in the existing design which is 3 meter wide and 120^0 deflection angle intake shows that the major flow stream lines are moving far from the intake location in low, medium and high flood conditions. As shown in **Figure 6.5** little and small magnitudes of flow patterns are moving towards the intake direction. This shows most of the flood discharge moves directly over the center of the weir and the sediments are dropping around river side's including intake structure. This could be the main cause for structural failure of spate irrigation structures.

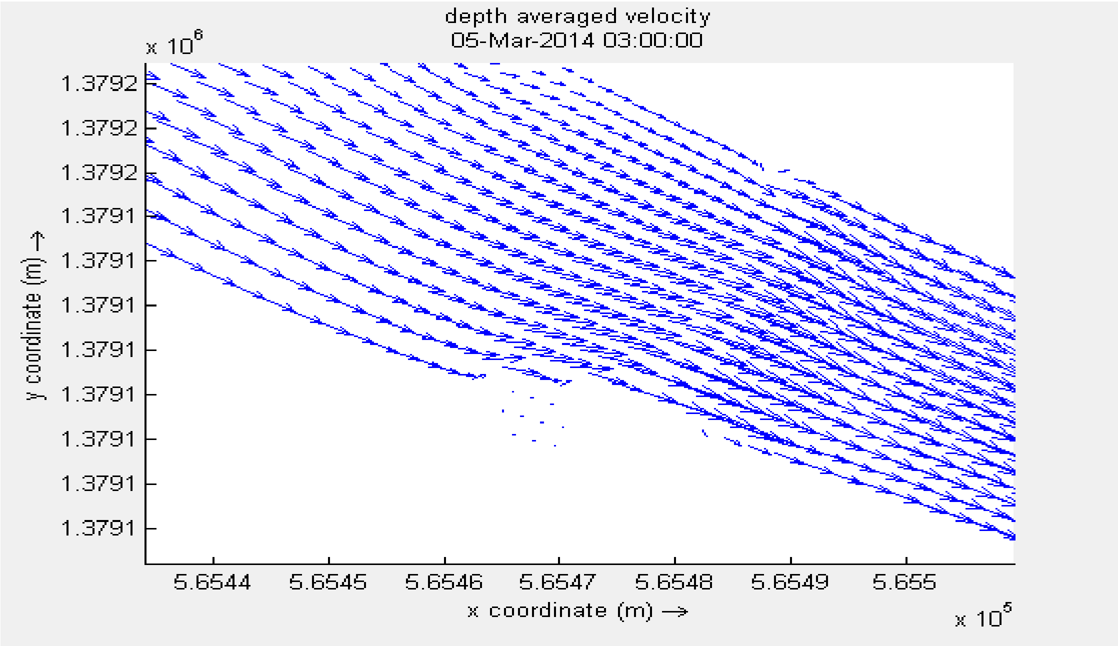


Figure 6.5 Flow patterns of the flood for scenario one

Intake discharge

Intake flood hydrograph was developed from the Delft3D simulation results of depth averaged velocity and water levels in the intake structure. The intake discharge was also calculated from the intake flood hydrograph. The discharge of the intake at a time was calculated by multiplying velocity (m/s) and area (m^2). **Figure 6.6** shows the graph of intake hydrograph which could be diverted from the river to the main canal for irrigation purposes. The intake inflow hydrograph shows at peak discharges it is possible to divert irrigation flood water up to 2.6, 6.6 and 13.5 m^3/s during low, medium and high floods incidences respectively.

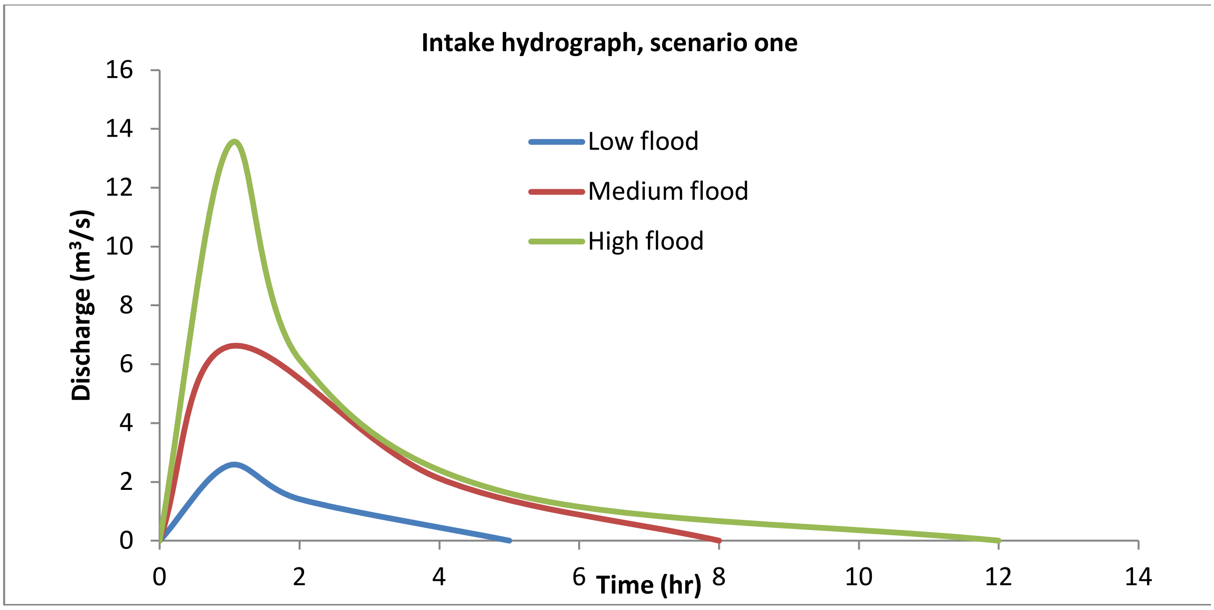


Figure 6.6 Intake hydrograph

Erosion and Sedimentation

The simulation result shows the cumulative sedimentation around the intake is 0, 26 and 41 centimeter (cm) depth for low, medium and high flood events respectively. In this condition, as the magnitudes of flow patterns towards the intake are small the incoming sediments cannot move and deposited around the intake. Gradually the main spate flow channel starts to move far from intake and more sediment is accumulated. The reality in most of the modernized spate irrigation headwork shows that after two and or three floods events the flood water didn't flow towards intake and the main channel starts moving far. a lot of sediments are also accumulated around the intake structures and blocks flow. **Figure 6.7** shows the cumulative erosion/sediment around the intake at high flood.

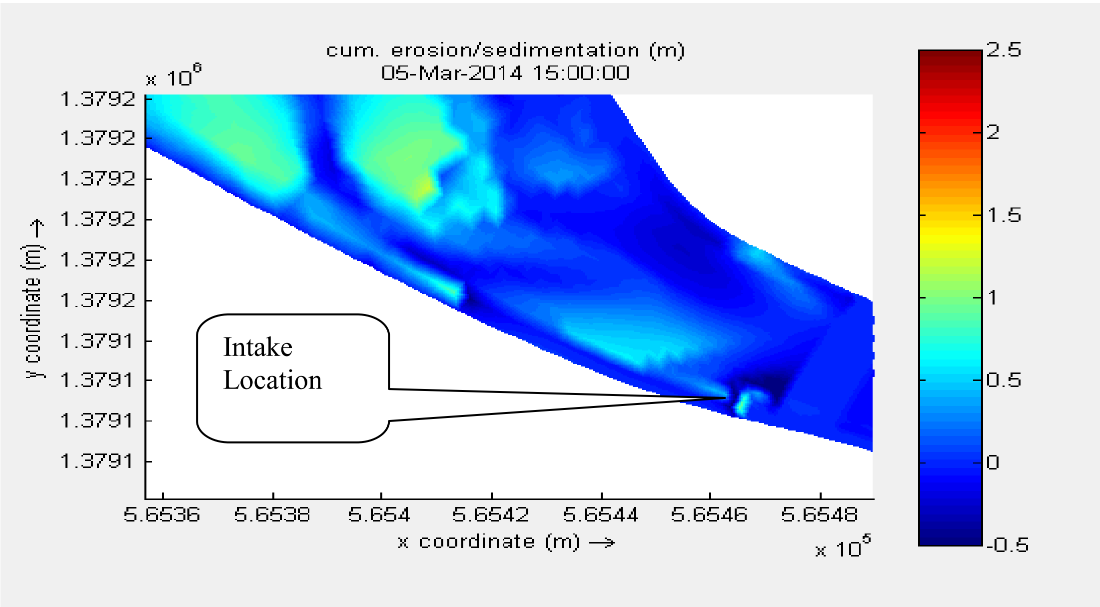


Figure 6.7 Cumulative erosion/sedimentation scenario one

6.4.2. Scenario two (S2)

Flow pattern

The flow patterns in the second scenario or alternative design of 3 meter intake with deflection angle of 150^0 shows that there are plenty of water velocities towards the intake structure. The more flow pattern to the direction of intake is the one which can divert more flood water for irrigation. **Figure 6.8** illustrates the flow pattern for 3 meter wide and 150^0 deflection angel of intake.

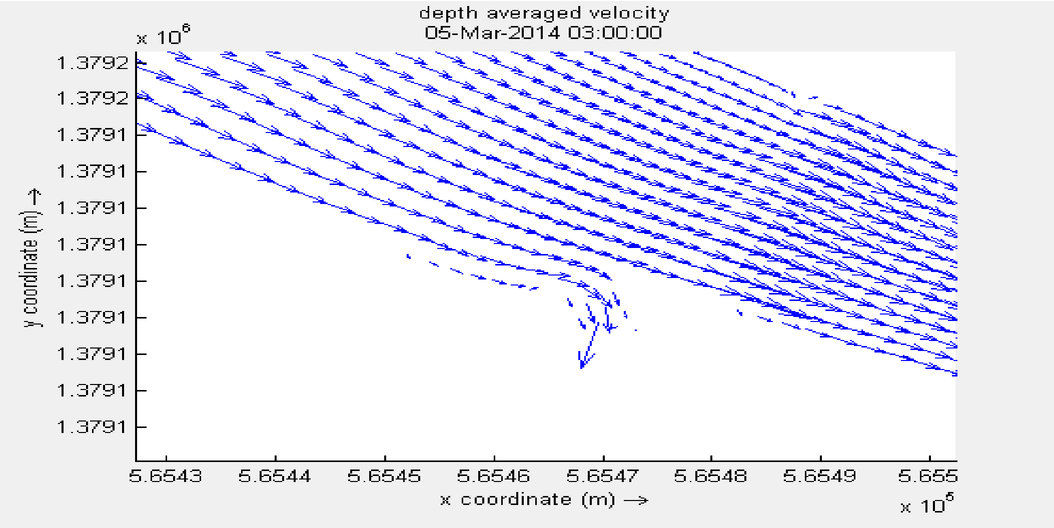


Figure 6.8 Flood pattern of scenario two

Intake discharge

Discharge hydrograph in the intake was developed from the result of depth averaged velocity and water levels in the intake structure. **Figure 6.9** shows the result of inflow flood hydrograph which could be diverted from the river to the main canal. In this alternative design significant amount of flood water can be diverted to main canal. The intake inflow hydrograph shows it is possible to divert flood water up to 3.1, 8.4 and 17.1 m^3/s for low, medium and high flood respectively.

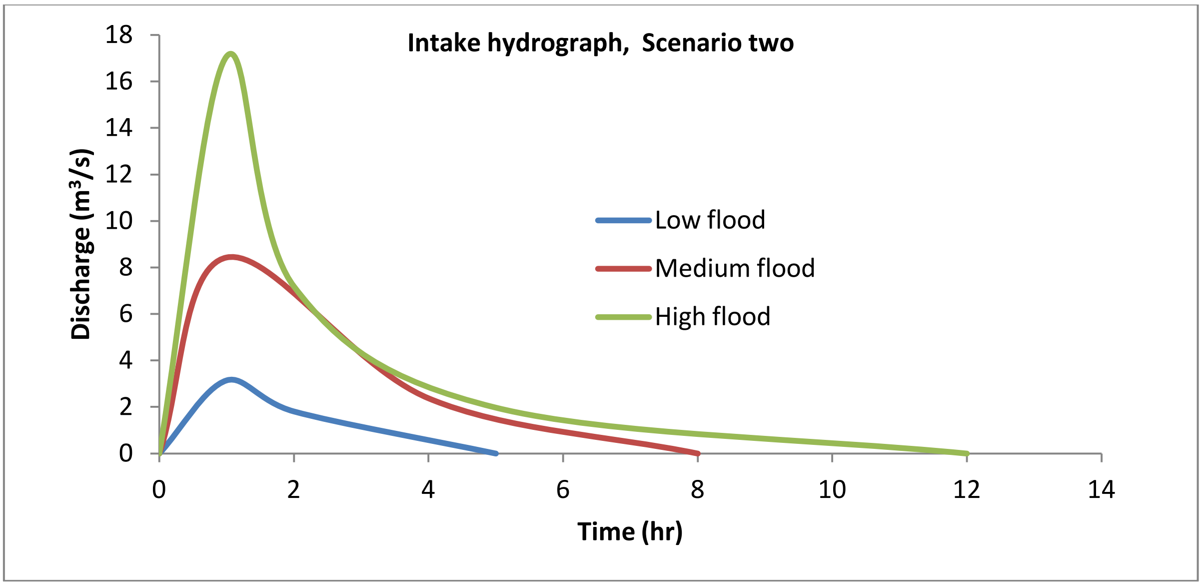


Figure 6.9 Intake hydrograph

Erosion and Sedimentation

The simulation result of morphodynamics for the second scenario shows the cumulative sedimentation around the intake reaches up to 0, 28 and 34 centimeter (cm) depth for low, medium and high flood events respectively. **Figure 6.10** shows the cumulative erosion/sediment around the intake at high flood. As can be seen from **Figure 6.10** the accumulations of sediments are widely spread above the diversion structure.

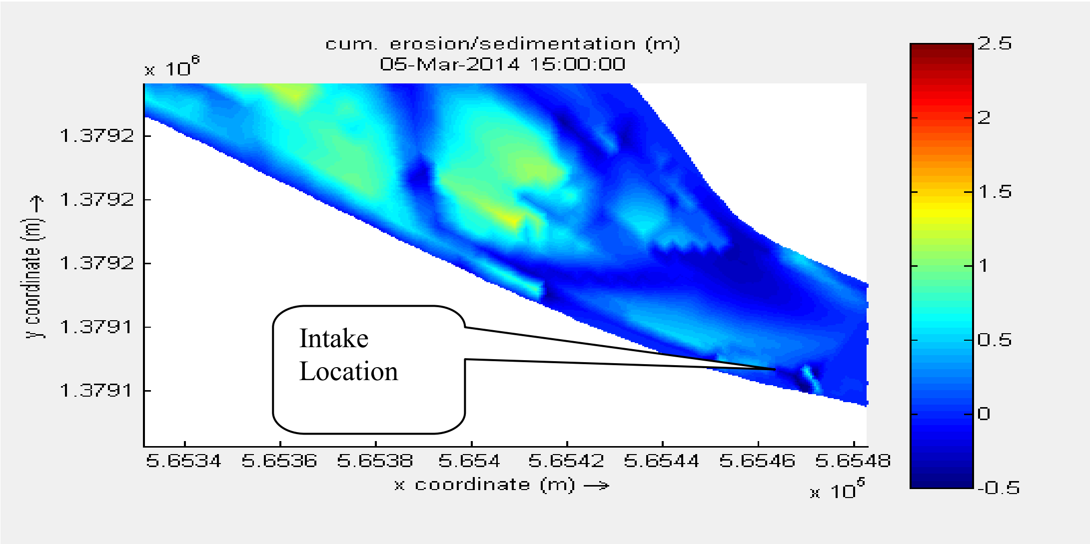


Figure 6.10 Cumulative erosion/sedimentation scenario two

6.4.3. Scenario three (S3)

Flow pattern

The flow patterns in the third scenario or alternative design thee shows that there are flows of water velocity water flow towards the intake structure. **Figure 6.11** illustrates the flow pattern for 5 meter wide and 120^0 deflection angel of intake. In this condition the velocity towards intake and water levels are high which could result to divert appreciable amount of flood water. Flow patterns in scenario three are more concentrated than scenario two but they are little bit deflected.

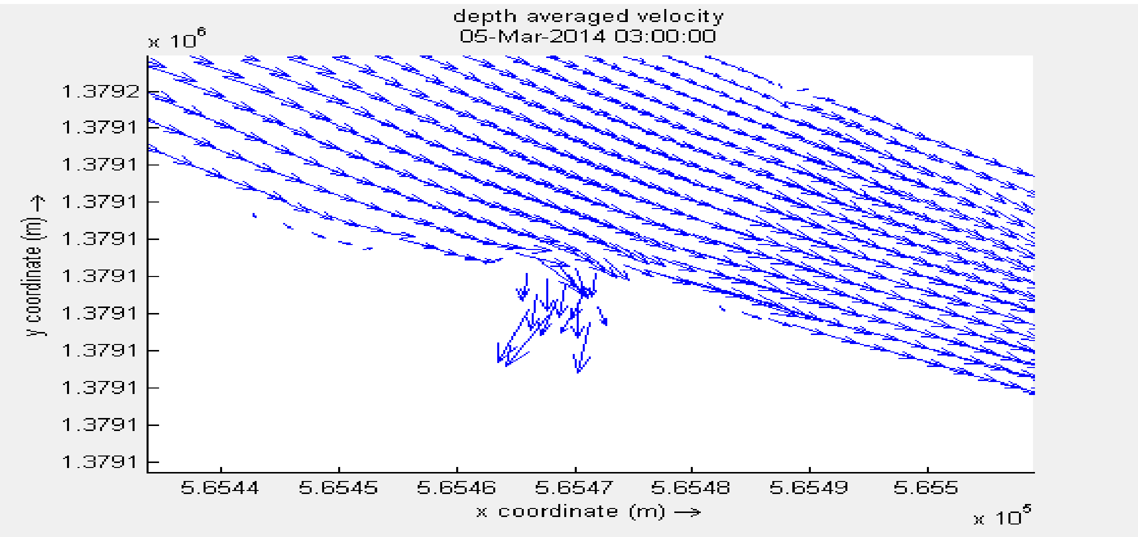


Figure 6.11 Flood patterns of scenario three

Intake discharge

Flood hydrograph in the intake was developed from the result of depth averaged velocity and water levels in the intake structure. **Figure 6.12** shows the result of inflow flood hydrograph which could be diverted from the river to the main canal. In this alternative design significant amount of flood water can be diverted to main canal. The intake inflow hydrograph shows it is possible to divert flood water up to 4.7, 12.6 and 22.8 m^3/s for low, medium and high flood respectively during the peak events.

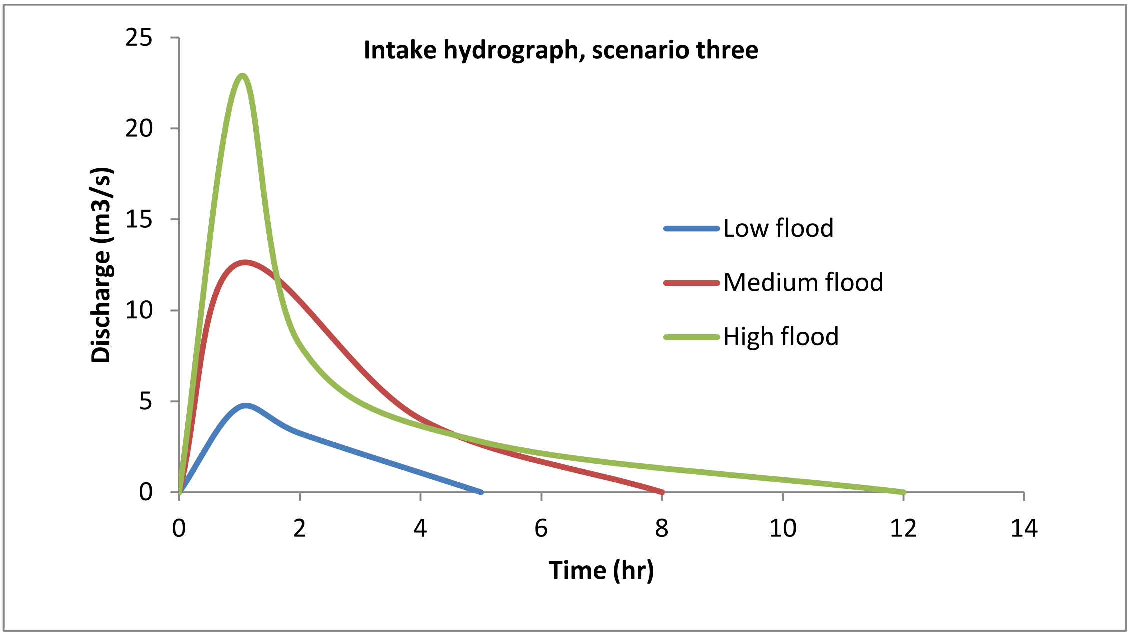


Figure 6.12 Intake hydrograph

Erosion and Sedimentation

The simulation result of morphodynamics of scenario three shows the cumulative sedimentation around the intake reaches up to 3, 24 and 48 centimeter (cm) depth for low, medium and high flood events respectively. **Figure 6.13** shows the cumulative erosion/sediment around the intake at high flood.

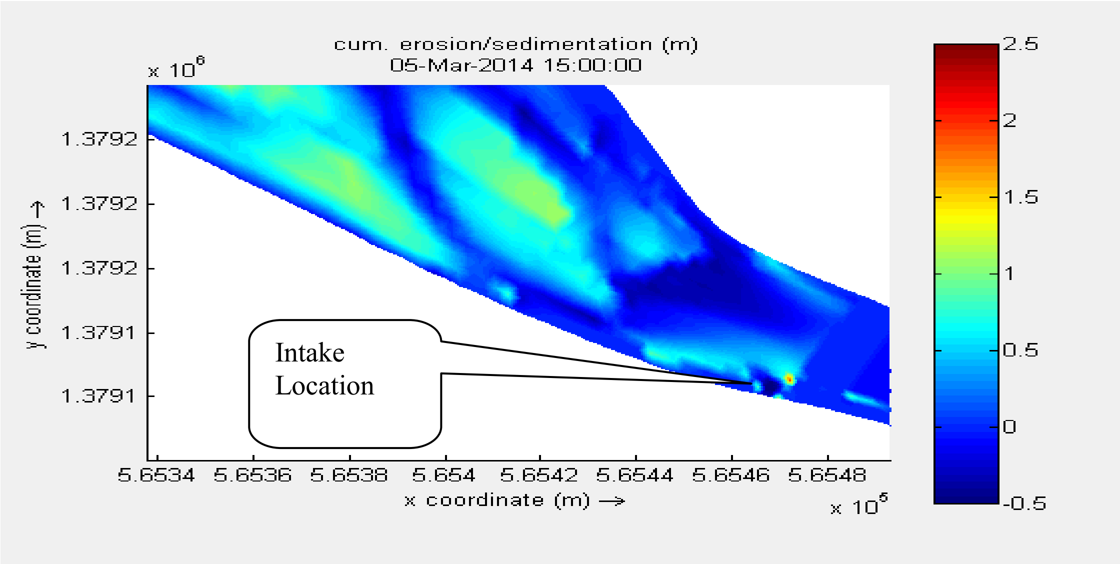


Figure 6.13 Cumulative erosion/sedimentation scenario three

6.4.4. Scenario four (S4)

Flow pattern

The flow patterns in the fourth scenario or alternative design four show that there are flows of water velocity water flow towards the intake structure. **Figure 6.14** illustrates the flow pattern for 5 meter wide and 120^0 deflection angel of intake. In this condition the velocity towards intake and water levels are high which could result to divert appreciable amount of water. Flow patterns in scenario four are directed forward to the intake with an angle greater than scenario three.

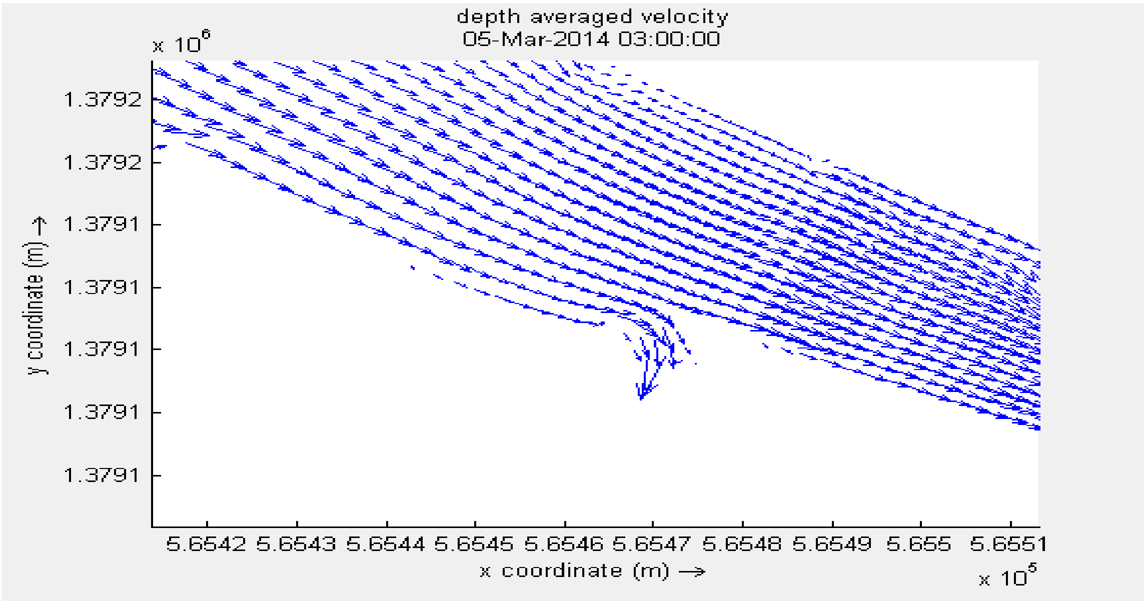


Figure 6.14 Flow patterns of scenario four

Intake discharge

Flood hydrograph in the intake was developed from the result of depth averaged velocity and water levels in the intake structure. **Figure 6.15** shows the result of inflow flood hydrograph which could be diverted from the river to the main canal. In this alternative design significant amount of flood water can be diverted to main canal. The intake inflow hydrograph shows it is possible to divert flood water up to 5.5, 13.7 and 24.4 m^3/s for low, medium and high flood respectively.

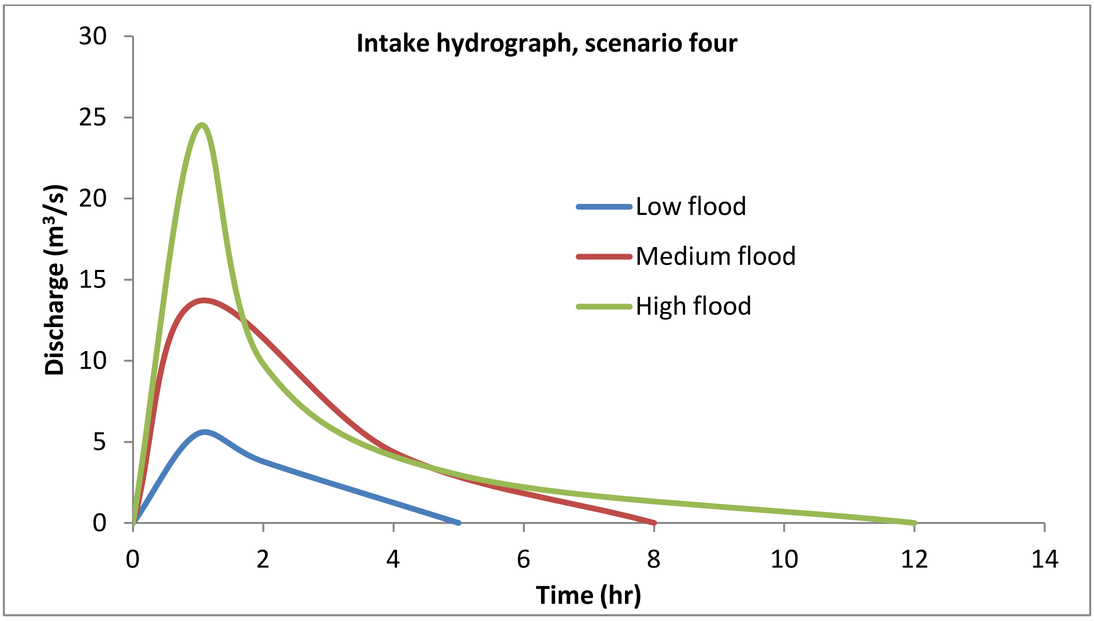


Figure 6.15 Intake hydrograph

Erosion and Sedimentation

The simulation result of morph dynamics of scenario four shows the cumulative sedimentation around the intake reaches up to 3, 27 and 46 centimeter (cm) depth for low, medium and high flood vents respectively. **Figure 6.16** shows the cumulative erosion/sediment around the intake at high flood.

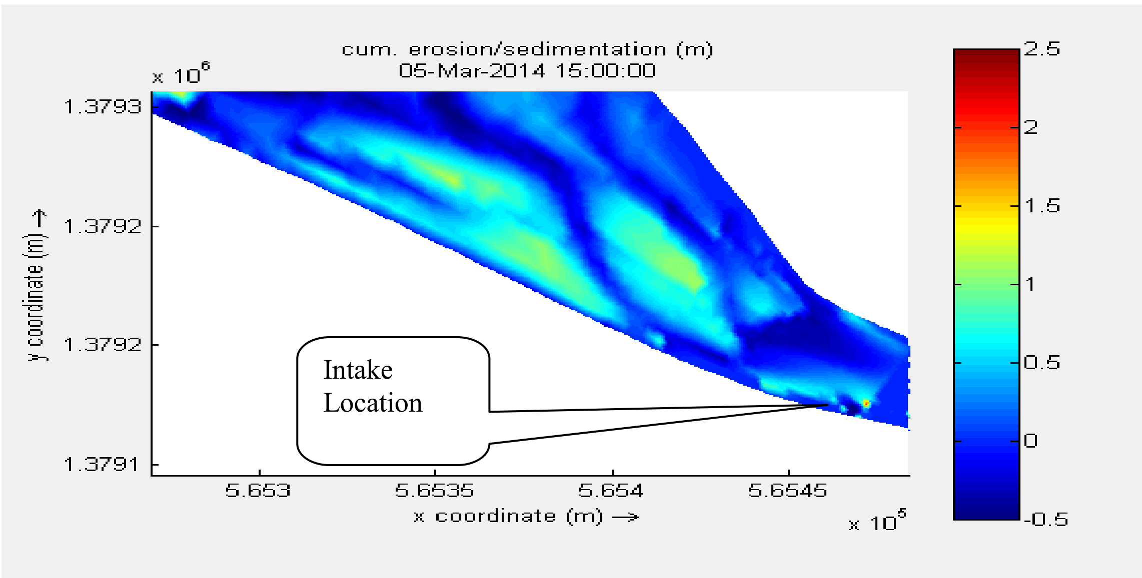


Figure 6.16 Cumulative erosion/sedimentation scenario four

Total water abstraction in one flood

The total water abstraction from the river to the main canal in one flood occurrence for all scenarios was calculated from the intake hydrograph. From the developed intake hydrograph a total volume to be diverted from the river was calculated by multiplying discharge and time. A comparison of the scenario was made based on the total volume of water.

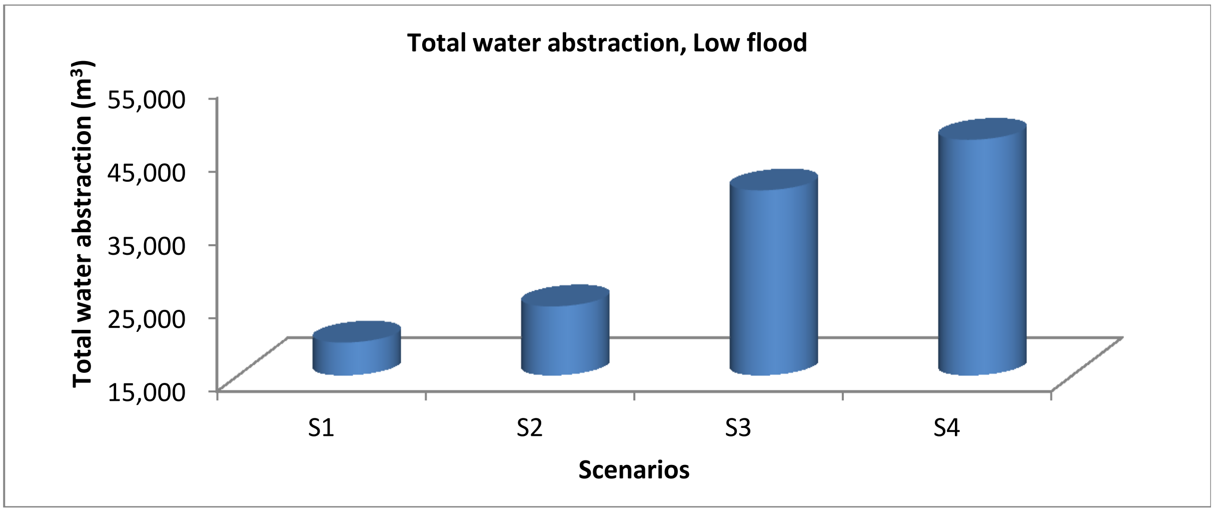


Figure 6.17 Total volume of water abstraction under low flood condition

As can be seen from **Figure 6.17** the amount of water that can be diverted from the river under low flood condition shows that there is small increase from scenario one to scenario two and scenario three and four too. The increase from scenario two to scenario three is quite large. Numerically, during one flood occurrence there is a possibility of abstracting water up to 19,442 m^3, 24,369 m^3, 40,203 m^3 and 47,101 m^3 from scenario one, two, three and four respectively. This shows that increasing intake width is more important than changing deflection angle.

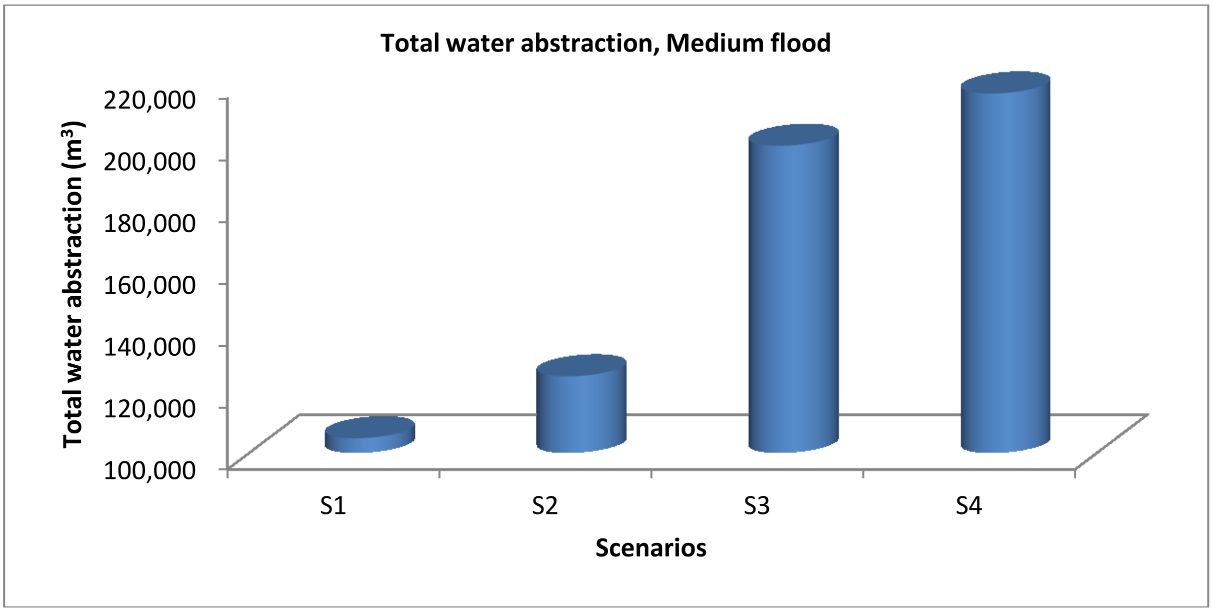


Figure 6.18 Total volume of water abstraction under medium flood condition

As shown in **Figure 6.18** the amount of water that can be diverted from the river under medium flood condition shows that there is an increase in amount of diverted water from scenario one to all other scenarios while their magnitudes are different. The increase from scenario two to scenario three is quite large. During one medium flood incidence there is a possibility of abstracting water up to 104,687 m^3, 124,740 m^3, 199,409 m^3 and 216,346 m^3 from scenario one, two, three and four respectively.

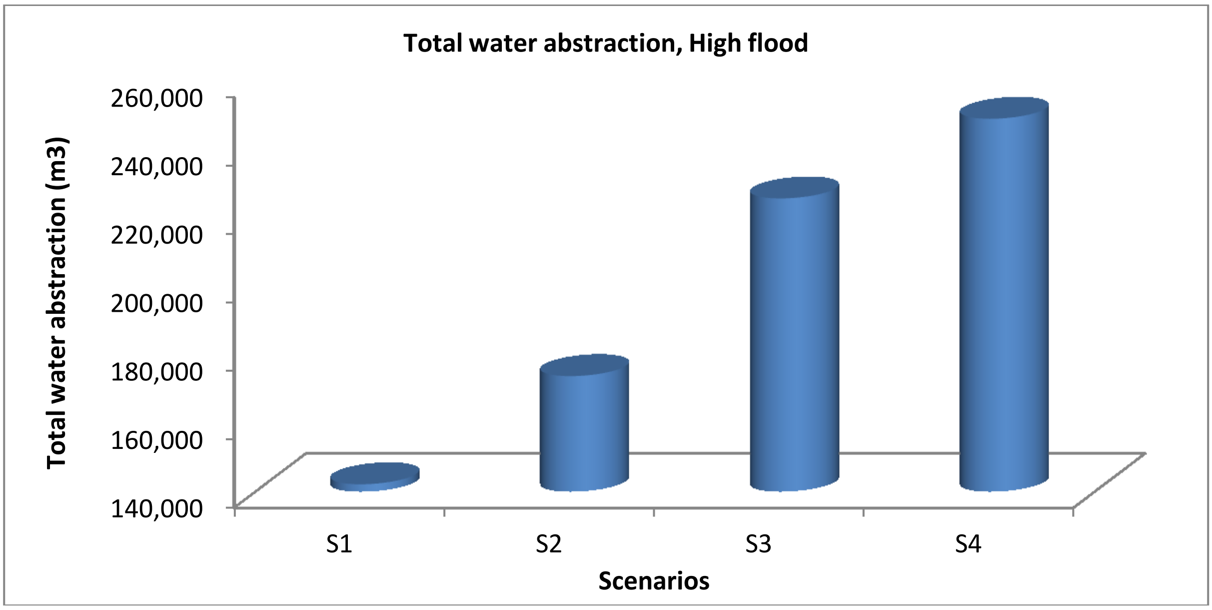


Figure 6.19 Total volume of water abstraction under high flood condition

Figure 6.19 shows the amount of water that can be diverted from the river under maximum flood condition. This shows that there is an increase in amount of diverted water from scenario one to all other scenarios while their magnitudes are different. The increase from scenario two to scenario three is relatively large. Numerically there is a possibility of abstracting water at amount of 142,076 m^3, 173,699 m^3, 225,644 m^3 and 248,936 m^3 from scenario one, two, three and four respectively per one high flood condition.

Total water abstraction per season

Based on the flood frequency which was collected during discussion with farmers to flood frequency in wet, medium and dry season's section 4.3 and the irrigation water requirement calculated in section 4.4 the total amount of water to be diverted from the river through the intake and the total area could be irrigated was calculated. **Table 6.3** shows the amount of water to be diverted and area to be irrigated per season.

Table 6.3 Total water to be diverted and area to be irrigated under all scenarios

Parameter	Season	Scenarios						
		S1	S2		S3		S4	
				%		%		%
Total water to be diverted (Mm^3)	Wet	3.39	4.12	21	6.20	83	6.87	103
	Medium	2.52	3.05	21	4.54	80	5.02	100
	Dry	1.52	1.84	21	2.74	81	3.03	99
Area to be irrigated (ha)	Wet	266	323	21	486	83	539	100
	Medium	227	275	21	409	80	453	100
	Dry	155	187	21	280	81	309	100

As shown in **Table 6.3** scenario four has the capacity to divert more flood water and can irrigate more lands and followed by scenario three and two respectively. Scenario one which is also the current or existing condition irrigates minimum area in all seasons.

Improving of intake deflection angle from 120^0 to 150^0 for 3 meter wide intake can increase the irrigation area by 21%. Improving of intake width from 3 meter to 5 meter at 120^0 deflection angle can increase the total irrigation area by 81% for both flood conditions. Improving of intake width from 3 meter to 5 meter and deflection angle from 120^0 to 150^0 can improve the irrigation area by 100%.

Sedimentation

The sediment accumulation around intake for all scenarios was determined from simulation result of morphodynamics of Delft3D. All scenarios shows huge amount of accumulation of sediment around the intake structure. Scenario two shows relatively smaller sediment accumulation during high flood than other scenarios. Scenario three and four are almost similar in sedimentation. **Table 6.4** show the sediment accumulation under different flood magnitudes for all scenarios.

Table 6.4 Sediment accumulation around intake for all scenarios

Scenario	Sediment per flood type in cm		
	Low	Medium	High
S1	0	26	41
S2	0	28	34
S3	3	25	48
S4	3	26	46

As can be seen from **Table 6.4** the sediment accumulation during low flood scenario one and two did not create sedimentation while scenario three and four could have sediment thickness up to 3 cm per flood. During medium flood condition the sediment accumulation could reach up to 25, 26, 26 and 28 cm for scenario three, one, four and two respectively. This shows that all scenarios could have similar amount of sedimentation while they are diverting different amount of flood water. During high flood condition the accumulation of sediment could reach to 34, 41, 46 and 48 cm for scenario two, one, four and three respectively.

CHAPTER 7

Conclusion and Recommendation

7.1. Conclusion

After analyzing the design developments of spate irrigation systems; evaluation of problems for the current best or relatively good performing scheme; developments of alternatives designs for better flood and sediment managements and evaluation of alternative design using Delft3D model simulation for hydrodynamics and morphology in relation to sediment and spate flow management, the following conclusions were given.

- The design of main intakes has significantly improved over the past years. The intake dimensions were changed from closed intake, 90^0 deflection angles and narrow (90 cm wide) gates to 3 meters wide open intake with 120^0 deflection angle and this improvement gives relatively good performance for modern spate irrigation schemes.
- The latest design of diversion structure is however, far below optimum. This design is irrigating about 50% of the intended area. The main reason for the poor performance could be lack of optimum intake designs.
- An alternative design of 5 meter wide intake with a deflection angle of 150^0 to the river flow diverts the highest amount of water at 0.99, 1.74 and 1.99 Mm^3 for wet, medium and dry season's respectively. This can cover an irrigation area up to 539, 453 and 309 ha for wet, medium and dry seasons respectively. Using of this scenario can increase the irrigated area by 100% from the current condition or scenario one.
- In case of sedimentation all scenarios are very sensitive but intake dimensions with 150^0 degree shows relatively uniform distribution along the cross sections around intake. Since the bed level increments' is distributed through the immediate upstream area of the diversion, the amount of water to be diverted may not minimized significantly.
- The accumulations of sediments are not significantly different for all scenarios and this creates difficulties for making decision. The availability of too much water for scenario four and three can compensate for the high sedimentation.

7.2. Recommendation

- For better use of flood water for spate irrigation the intake structure should be 5 m wide and 150^0 deflection angle. This diverts more water and can increase the current irrigated area by 100% while it has similar sediment accumulation to the current designs.
- As the sediment accumulation around intake is too much which can reach up to 46 cm per flood during high flood incidence frequent dredging should have to perform. The farmers have to be aware on the importance of dredging and sediment removals from intake and canals.
- It is not possible to solve the problem of sedimentation around intake by having only good intake design. This needs further investigation to introduce appropriate sediment control structures
- As this study focus only on the water to be diverted and sediment accumulation further detail study to the structural and geotechnical stability needs to be done before implementing the recommended alternative designs.
- Increasing the intake width and deflection angle can increase the construction cost and input materials, therefore it is recommended to make cost analysis before making decision and implementation of the selected alternative design.
- Simulations of the model was performed based on single flood event, therefore it is to perform in season wise for the future to get clear image of erosion and or sedimentation accumulation and spate flow
- For having better spate irrigation system participation of farmers during design and construction is necessary and needs to exploit the indigenous knowledge of traditional spate system. Construction and designs of spate irrigation structures have to incorporate the farmer's interest and traditional knowledge of spate irrigation designs.

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Appendices

Appendix A River cross section data taken by total station

Cross section one (X-1)

X (m)	Y (m)	Z (m)	$(x_2-X_1)^2$	$(Y_2-Y_1)^2$	$((x_2-X_1)^2 + (Y_2-Y_1)^2)^{0.5}$	Chainage
565151.434	1379311.003	1469.448	0.00	0.00	0.00	0.00
565165.694	1379334.728	1467.480	203.35	562.88	27.68	27.68
565176.743	1379355.997	1468.339	122.08	452.37	23.97	51.65
565189.613	1379381.788	1468.015	165.64	665.18	28.82	80.47
565194.066	1379388.289	1468.307	19.83	42.26	7.88	88.35
565201.473	1379400.766	1469.529	54.86	155.68	14.51	102.86

Cross section one (X-2)

X (m)	Y (m)	Z (m)	$(x_2-X_1)^2$	$(Y_2-Y_1)^2$	$((x_2-X_1)^2 + (Y_2-Y_1)^2)^{0.5}$	Chainage
565275.336	1379388.611	1467.831	0.00	0.00	0.00	0.00
565265.524	1379370.563	1466.209	96.28	325.73	20.54	20.54
565246.090	1379336.483	1467.359	377.68	1161.45	39.23	59.77
565235.345	1379319.752	1467.605	115.46	279.93	19.88	79.66
565221.125	1379296.601	1467.416	202.21	535.97	27.17	106.83
565224.322	1379266.409	1468.042	10.22	911.56	30.36	137.19

Cross section one (X-3)

X (m)	Y (m)	Z (m)	$(x_2-X_1)^2$	$(Y_2-Y_1)^2$	$((x_2-X_1)^2 + (Y_2-Y_1)^2)^{0.5}$	Chainage
565306.861	1379219.852	1466.858	0.00	0.00	0.00	0.00
565310.586	1379223.933	1465.243	13.88	16.65	5.53	5.53
565328.635	1379248.850	1465.387	325.77	620.86	30.77	36.29
565342.559	1379285.392	1465.891	193.88	1335.32	39.10	75.40
565353.101	1379326.027	1466.227	111.13	1651.20	41.98	117.38
565361.193	1379338.079	1466.410	65.48	145.25	14.52	131.89

Cross section one (X-4)

X (m)	Y (m)	Z (m)	$(x_2-X_1)^2$	$(Y_2-Y_1)^2$	$((x_2-X_1)^2 +(Y_2-Y_1)^2)^{0.5}$	Chainage
565393.854	1379310.625	1466.002	0.00	0.00	0.00	0.00
565369.386	1379273.869	1465.707	598.68	1351.00	44.16	44.16
565363.528	1379265.913	1465.567	34.32	63.30	9.88	54.04
565352.865	1379247.315	1465.089	113.70	345.89	21.44	75.47
565334.036	1379204.397	1465.008	354.53	1841.95	46.87	122.34
565329.859	1379197.246	1466.584	17.45	51.14	8.28	130.62

Cross section one (X-5)

X (m)	Y (m)	Z (m)	$(x_2\text{-}X_1)^2$	$(Y_2\text{-}Y_1)^2$	$((x_2\text{-}X_1)^2 + (Y_2\text{-}Y_1)^2)^{0.5}$	Chainage
565,391.11	1,379,157.82	1,465.87	-	-	-	0.00
565,396.27	1,379,165.01	1,463.29	26.59	51.75	8.85	8.85
565,409.60	1,379,184.28	1,464.01	177.69	371.14	23.43	32.28
565,426.98	1,379,202.48	1,464.24	302.20	331.28	25.17	57.45
565,437.79	1,379,211.27	1,465.98	116.66	77.35	13.93	71.38

Cross section one (X-6)

X (m)	Y (m)	Z (m)	$(x_2\text{-}X_1)^2$	$(Y_2\text{-}Y_1)^2$	$((x_2\text{-}X_1)^2 + (Y_2\text{-}Y_1)^2)^{0.5}$	Chainage
565465.234	1379160.413	1465.976	0.00	0.00	0.00	0.00
565465.209	1379159.815	1463.294	0.00	0.36	0.60	0.60
565456.803	1379145.233	1463.236	70.66	212.63	16.83	17.43
565449.461	1379130.156	1463.768	53.90	227.32	16.77	34.20
565449.295	1379129.840	1466.033	0.03	0.10	0.36	34.56

Cross section one (X-7)

X (m)	Y (m)	Z (m)	$(x_2\text{-}X_1)^2$	$(Y_2\text{-}Y_1)^2$	$((x_2\text{-}X_1)^2 + (Y_2\text{-}Y_1)^2)^{0.5}$	Chainage
565468.967	1379124.652	1466.005	0.00	0.00	0.00	0.00
565468.879	1379125.138	1462.892	0.01	0.24	0.49	0.49
565478.272	1379139.461	1462.900	88.23	205.15	17.13	17.62
565483.989	1379149.599	1463.052	32.68	102.78	11.64	29.26
565484.964	1379149.231	1465.913	0.95	0.14	1.04	30.30

Cross section one (X-8)

X (m)	Y (m)	Z (m)	$(x_2\text{-}X_1)^2$	$(Y_2\text{-}Y_1)^2$	$((x_2\text{-}X_1)^2 + (Y_2\text{-}Y_1)^2)^{0.5}$	Chainage
565488.589	1379147.179	1464.381	0.00	0.00	0.00	0.00
565488.648	1379146.875	1460.076	0.00	0.09	0.31	0.31
565481.861	1379135.350	1460.019	46.06	132.83	13.37	13.68
565475.202	1379121.693	1459.927	44.34	186.51	15.19	28.88
565474.493	1379121.006	1464.728	0.50	0.47	0.99	29.87

Cross section one (X-9)

X (m)	Y (m)	Z (m)	$(x_2\text{-}X_1)^2$	$(Y_2\text{-}Y_1)^2$	$((x_2\text{-}X_1)^2 + (Y_2\text{-}Y_1)^2)^{0.5}$	Chainage
565531.939	1379094.251	1464.446	0.00	0.00	0.00	0.00
565532.807	1379095.544	1461.881	0.75	1.67	1.56	1.56
565534.336	1379099.146	1458.774	2.34	12.97	3.91	5.47
565537.109	1379106.904	1458.783	7.69	60.19	8.24	13.71
565541.502	1379114.062	1458.804	19.30	51.24	8.40	22.11
565543.247	1379115.597	1463.174	3.05	2.36	2.32	24.43

Cross section one (X-10)

X (m)	Y (m)	Z (m)	$(x_2-X_1)^2$	$(Y_2-Y_1)^2$	$((x_2-X_1)^2 + (Y_2-Y_1)^2)^{0.5}$	Chainage
565602.633	1379086.684	1461.716	0.00	0.00	0.00	0.00
565602.081	1379083.745	1458.003	0.30	8.64	2.99	2.99
565599.265	1379076.808	1458.114	7.93	48.12	7.49	10.48
565597.251	1379070.529	1458.142	4.06	39.43	6.59	17.07
565587.979	1379061.033	1462.759	85.97	90.17	13.27	30.34

Cross section one (X-11)

X (m)	Y (m)	Z (m)	$(x_2-X_1)^2$	$(Y_2-Y_1)^2$	$((x_2-X_1)^2 + (Y_2-Y_1)^2)^{0.5}$	Chainage
565667.725	1379047.365	1462.975	0.00	0.00	0.00	0.00
565667.890	1379047.974	1459.601	0.03	0.37	0.63	0.63
565668.676	1379051.185	1457.275	0.62	10.31	3.31	3.94
565671.131	1379060.920	1457.298	6.03	94.77	10.04	13.98
565672.185	1379064.217	1459.999	1.11	10.87	3.46	17.44
565672.605	1379064.824	1463.097	0.18	0.37	0.74	18.18

Appendix B Sediment bed material grading analysis

Table B.1 Retained weight of sediment in kilo grams (kg) per sieve size at field by hand sieving

Sample no.	sieve size (mm)							Remarks
	<5	5	25	50	80	150	>250	
1	1287.27	150.35	20.72	7.24	0	0	0	D/S
2	734.78	172.58	108.99	71.56	134.13	26.95	40.14	
3	621.21	299.53	108.75	88.02	42.02	35.13	29.91	Just U/S
4	863.97	303.94	46.73	7.67	13.67	0	0	
5	367.85	175.07	116.64	126.31	72.33	90.18	38.94	Far U/S
6	546.96	156.57	82.76	67.99	60.36	65.04	21.68	
7	532.23	122.25	48.03	36.99	21.11	0	0	Intake
8	259.88	95.11	31.34	13.33	5.29	0	0	

Table B.2 Retained weight of sediment in grams (g) per sieve size in laboratory, mechanical sieving

Sample no.	Sieve size (mm)									Remarks
	4.75	2.36	2.00	1.00	0.50	0.25	0.106	0.053	pan	
1	0.51	98.07	43.04	314.44	420.57	340.26	62.80	5.75	1.83	D/S
2	5.02	108.18	35.94	198.72	188.54	150.99	38.63	5.93	2.82	
3	1.85	105.08	38.34	185.36	140.06	112.00	30.65	5.44	2.44	Just U/S
4	4.90	140.19	51.19	248.41	198.53	171.02	42.74	4.92	2.08	
5	0.45	52.36	17.06	78.59	69.31	98.47	40.52	8.02	3.07	Far U/S
6	2.14	69.10	22.43	141.97	135.04	126.45	41.89	6.17	1.75	
7	0.10	18.49	5.88	73.32	192.61	207.52	30.12	3.25	0.94	Intake
8	1.35	48.59	12.16	52.97	41.77	62.97	30.72	6.56	2.78	

Appendix C River discharge calculation

River discharge at cross section one (X-1)

flow type	Head (m)	Area (m^2)	Perimeter (m)	R (m)	S (m/m)	n (-)	Manning`s Q (m^3/s)	D_{84} (m)	D* (-)	Bathurst`s Q (m^3/s)
Minimum	0.52	5.841	22.322	0.262	0.011	0.031	8	0.040	8.58	8
Medium	0.85	26.684	72.973	0.366	0.011	0.031	46	0.040	9.39	50
Maximum	1.45	75.851	88.333	0.859	0.011	0.031	232	0.040	11.48	265
Extreme	1.66	95.033	96.547	0.984	0.011	0.031	318	0.040	11.82	366

River discharge at cross section five (X-5)

flow type	Head (m)	Area (m^2)	Perimeter (m)	R (m)	S (m/m)	n (-)	Manning`s Q (m^3/s)	D_{84} (m)	D* (-)	Bathurst`s Q (m^3/s)
Minimum	0.55	5.428	19.824	0.274	0.011	0.031	8	0.040	8.69	8
Medium	1.15	27.648	54.155	0.511	0.011	0.031	60	0.040	10.21	66
Maximum	1.82	66.488	61.942	1.073	0.011	0.031	236	0.040	12.02	272
Extreme	2.13	86.193	65.545	1.315	0.011	0.031	350	0.040	12.52	406

River discharge at cross section eleven (X-11)

flow type	Head (m)	Area (m^2)	Perimeter (m)	R (m)	S (m/m)	n (-)	Manning`s Q (m^3/s)	D_{84} (m)	D* (-)	Bathurst`s Q (m^3/s)
Minimum	0.60	6.037	11.295	0.534	0.011	0.031	13	0.040	10.32	15
Medium	1.78	20.282	15.773	1.286	0.011	0.031	81	0.040	12.46	94
Maximum	3.35	45.419	20.499	2.216	0.011	0.031	262	0.040	13.79	306
Extreme	3.75	52.313	21.316	2.454	0.011	0.031	323	0.040	14.04	378