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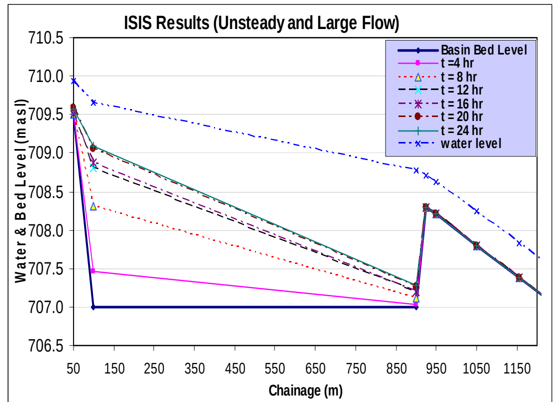

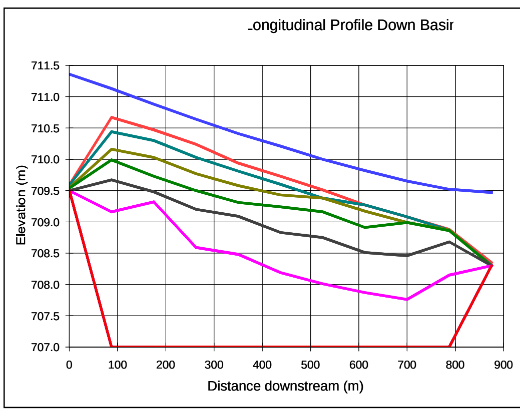

Analysis of Spate Irrigation Sedimentation and the Design of Settling Basins

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Delft
April 2009

UNESCO-IHE
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Dedicated to my
beloved wife
Tsegabrhan Haileselassie
and
my brother
Temesgen Gebreegziabher

Abstract

Spate Irrigation is a type of runoff farming where flash floods produced in the highlands are diverted by irrigation structures to irrigate fields. The farmers in the southern part of Tigray have long been diverting traditionally the flash floods in to their irrigated fields. However, the indigenous spate irrigation practices have many problems such as the destruction of headworks during large flood events. Hence, the regional government of Tigray has shouldered the responsibility to modernize these spate schemes. However, the majority of the schemes are not effective, the main problem being sedimentation of canals and irrigated fields. Sedimentation highly affects the operation of the scheme; coarser sediments reduce the diversion efficiency of the scheme, and raise the bed level of irrigated field. Contrary to this, fine sediments improve the fertility of spate irrigated fields. Therefore to compromise between these, effective sediment control and management, that enables the trapping of coarser sediments before reaching the irrigated field and allows the fine sediments to reach in to the irrigated fields, should be implemented.

Therefore the objective of the thesis study is to analyze the existing sediment control system on spate irrigation scheme, to review and test innovative sediment control and management systems and to recommend as necessary alternative sediment control and management systems and structures. The Fokisa spate scheme, which has a design capacity of 500 ha, but which currently irrigates only 150 ha of land mainly due to sedimentation has been chosen for this study. Three models have been used DORC (Design of Regime Canals) is used to generate data, DOSSBAS (Design of Sluiced Sediment Basins), a steady state model, used to design sand trap basin and ISIS sediments has been used to check how the sand trap basin performs under unsteady conditions.

Four options of sediment control and management, the sand trap basin, lengthening of the main canal, avoiding large floods using deflector weir at the upstream canal and modifying canal dimensions and slope have been tested and analyzed. Results show that, by modifying the canal cross sections and slope, a sand trapping efficiency of 28.7 % and 15.6 % can be achieved for medium and small floods respectively. The scheme performance is poor either with or without sand trap basin when large flows are diverted towards the irrigated field. A deflector weir has been shown to avoid this inefficiency. Furthermore, from the particle size and textural distribution of the scheme, D_{50} of the particles and the proportion of gravel and sand, decreases along the canal route, hence lengthening the main canal length helps in trapping coarse sediments. However, the application of settling basins is questionable. Sensitivity test results show that, bed level rise is highly sensitive to hydrograph shape changes, sequence of flow events, input sediment size and concentrations, selected sediment transport formula, and Manning's n. Therefore, a combination of deflector weir, modifying canal dimensions and lengthening main canal has been proposed as a solution. Furthermore, data collection efforts and data base systems should be improved, and designed schemes should be well monitored for the future.

Key words: Spate Irrigation, Sedimentation, modeling, DORC, DOSSBAS, ISIS sediments.

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List of symbols

ISO	International Organization for Standardization
Masl	meters above sea level
Mbsl	meters below sea level
θ_{cr}	critical mobility Shield's parameter
τ_{cr}	critical sheer stress for initiation of motion (N/m^2)
ρ_s	density of sediment (kg/m^3)
ρ	density of water (kg/m^3)
g	acceleration due to gravity (kg/m^2)
D	characteristics grain size
υ_*	local shear velocity (m/s)
R	hydraulic radius (m)
S	slope (m/m)
Δ	relative density
Ws	terminal fall velocity of a sphere in still fluid (m/s)
s	specific gravity
C_D	drag coefficient
$(q_s)_l$	quantity of sediment of particle size leaving the basin (m^3/s)
$(q_s)_e$	quantity of sediment of particle size entering the basin (m^3/s)
K	Strickler roughness coefficient
L	Length (m)
V	velocity (m/s)
η	settling efficiency
A	surface area (m^2)
Q	discharge (m^3/s)
wo	weight of sediment entering the basin (Kg)
Wo	weight of sediment leaving the basin (Kg)
W	weight of sediment leaving the basin
q	Discharge per unit width of settling basins (m^2/s)
υ	kinematics viscosity
D_{35}	sediment particle size
H	depth of flow
C	Chezy coefficient
q_s	Total sediment discharge per meter width (m^3/sm)
d_{50}	mean particle size
q_b	bed load transport rate (m^2/s)
q_{sus}	suspended load transport rate (m^2/s)
D_*	particle parameter
T	bed shear parameter
F	shape factor
Ca	reference concentration
C_f	coefficient for the transport rate
F_s	grain Froude number
F_{gcr}	critical grain Froude number
C_t	total sediment transport expressed in PPm by weight
I, J	coefficient in the total sediment transport of Yang's function
C_v	sediment capacity concentration (by volume),

τ_b	bed shear stress,
P	flow wetted perimeter (m)
n	Manning's roughness coefficient
S_o	water surface slope (m/m)
TWRMEB	Tigray Water Resources,Mines and Energy Bureau
DORC	Design of Regime Canals (model)
DOSSBAS	Design Of Sluiced Sediment Basins (model)
SHARC	Sediment and Hydraulic Analysis for Rehabilitation of Canals
RB1, 2....8	River bed sample number 1, 2...8
MC1, 2...6	Main canal bed samples 1, 2...6.
SC1, 2 and 3	Secondary canal samples 1, 2 and 3.
IF1	Irrigated field bed sample 1.
SF	smallest flood that farmers expect every year
MF	Medium flood that farmers expect every year, and
HF	Large flood that farmers expect every year
BBL	Basin bed level
Eq	Equation
IFAD	International Fund for Agricultural Development
FAO	Food and Agricultural Organization
HFC	Water surface level of the canal at large/higher flow
HF	Water surface level of the river at large/higher flow
MFC	Water surface level of the canal at medium flow
MC	Water surface level of the river at medium flow
LFC	Water surface level of the canal at small/low flow
LC	Water surface level of the river at small/low flow
1D model	One dimensional model

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1. INTRODUCTION

1.1 General

According to Tesfai and Sterk (2002), spate irrigation is a type of runoff farming that makes use of flash floods produced in the highlands, which are diverted by temporary irrigation structures to irrigate fields. Spate irrigation, or Wadi agriculture, also called Arroyo agriculture in America (Barrow, 1987) is the name for floodwater farming cited by (Mehari, 2007). As a result, for many centuries, floods from different highland parts of the world have been diverted to irrigate the lowland command areas. Ethiopia is one of these countries, which is engaged in spate irrigation practices.

Ethiopia is gifted with ample water resources having 12 river basins that can potentially be used for irrigation. However, the country is largely dependent on practicing rain fed agriculture. As a result, the country is exposed to food shortages during drought seasons, where rainfall is very low. In the southern part of Tigray, north Ethiopia, although the dominant agricultural practice is rain fed, there is a tradition of supplementing the rain fed agriculture by spate irrigation.

The farmers in the southern part of Tigray have long been practicing traditional spate irrigation by constructing simple diversion structures from boulders, bush-bunds and sand bags, to divert floods in to their agricultural fields. However, these indigenous traditional headworks are often destroyed by large floods. Hence, as a solution, the regional state has shouldered the responsibility to upgrade the indigenous spate irrigation schemes. Consequently, about 15 traditional spate irrigation diversion schemes have been upgraded and some are under construction.

However, most of these designed river diversion intakes are not efficient; the main problem is sedimentation in the intake structures, settling basins, canals and irrigation fields. This can be attributed to three main reasons. Firstly, lack of understanding and research on the design and operation of the schemes. Secondly, most of the schemes are located at the bottom of mountains which have high sediment yield. Lastly, the schemes operate by diverting high floods, which have the capacity to carry and move the coarser sediments and the sedimentation rate estimations have been under estimated.

To solve the problems associated with sedimentation, the most common practice is to provide sand trapping basins at the end of the intake structures. However spate irrigation schemes, where the designs and operations are different from conventional river diversion schemes, sand traps have not been effective. Therefore this thesis will analyze the sediment control and management of an existing spate irrigation scheme. Firstly, it investigates why the scheme does not operate efficiently, and then proposes and tests options based on this understanding, and finally provides recommendations.

1.2 Background

Ethiopia has been described as the water tower of Africa, because most of the rivers in Eastern Africa originate from its highlands. The country is endowed with 12 river basins and 22 natural and artificial lakes. The total annual surface runoff is about 123 billion cubic meters while usable water is estimated 2.56 billion cubic meters (Metu, 2002). Mean annual rainfall ranges from 2000 mm over some areas in the southwest highlands, to less than 250 mm in the lowlands. In general, annual precipitation ranges from 800 to 2200 mm in the highlands (elevation, greater than 1500 masl) and varies from less than 200 to 800 mm in the lowlands (Annual rainfall in Ethiopia, 2002).

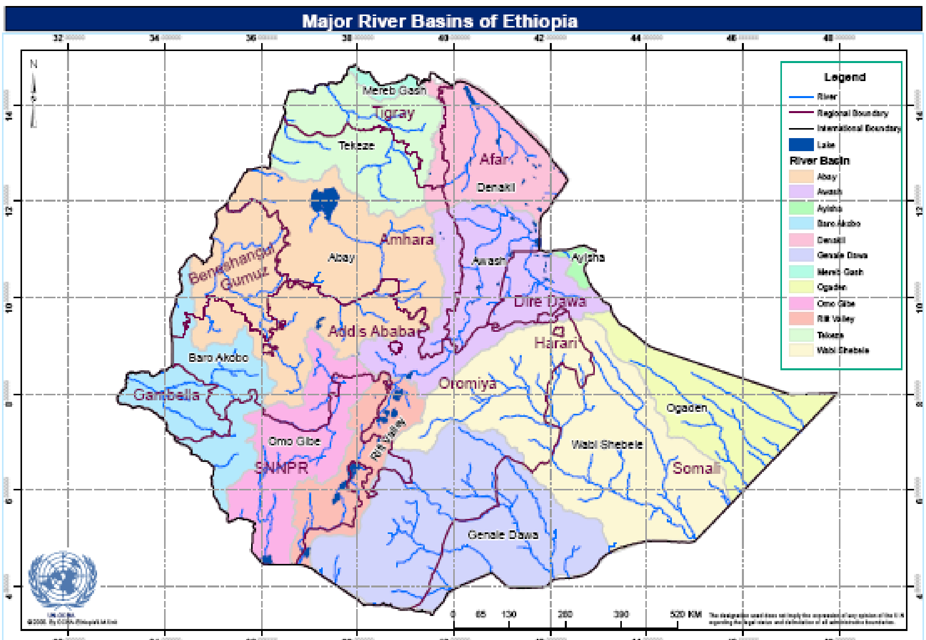

Figure 1. Major river basins of Ethiopia (map) (2006)
Source (http://www.google.nl/search?hl=nl&q=denakil+basin&meta=lr%3Dlang_en&rlz=1W1GPEA_en cited September, 2008).

The country also has different modes of rainfall. The Central, eastern and northern areas of the country experience a bimodal rainfall pattern, receiving the majority of their rainfall from the Atlantic, while the western and southern areas derive from the Indian Ocean. The main rain fall season is from June to September which comes mainly from the Atlantic, while the small spring rains between February and May, are derived predominantly from the Indian Ocean. (Ethiopia-climate (nd)).

Spate Irrigation is one means of increasing land productivity. Spate irrigation in Ethiopia has been practiced in the plains and valleys of southern Tigray, in Afsa close to Serda, Eastern Hararghe, around Nazareth and in the South, the Konso region and the area north of Lake Stephanie (near Jinka). The spate systems in Ethiopia rely more on rainfall. In some systems the spate flows are supplementary to the rainfall rather than the other way around, (Community spate irrigation (nd)). In such areas spate irrigation is used as a supplementary irrigation to the rain fed agriculture during periods of small rain fall.

Currently the country has given due attention to improve and modernize the indigenous spate irrigation practices. In addition, it is also doing its level best to disseminate these practices all over the country. An example is the Boro Dodota spate irrigation scheme, located 35 km from Adama (Nazareth). According to Aman (2007), the potential irrigable land on Boro Dodota spate diversion scheme can be as high as 5000ha.

Though the upgrading is very essential, it is not as effective as desired by the farmers. Especially in some schemes like Tirkie, 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.

Sedimentation has been a very serious problem in spate irrigation schemes in Ethiopia. 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 to design 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 historically and towards the settling basin design or for the irrigated fields.

1.3 Problem Statement

Due to shortage of rainfall and its increasing variability, moisture stress is identified to be one of the most critical factors affecting agricultural productivity in the dry lands of Ethiopia (Tamene et al., 2006). To supplement the moisture shortage farmers in the project area have long been using traditional flood diversions. Currently, they have access to what is called "modern "flood diversion. However the modern diversion schemes have many but inter related problems, the main problem being sedimentation of canals and irrigated fields.

Sedimentation of canals and irrigated fields is one of the major challenges in the modern spate irrigation diversion intakes. Sedimentation affects the operation of schemes by reducing discharge capacities and raising water levels (Lawrence et. al. 1998). However, in spate irrigation schemes, unlike the conventional river diversion schemes, sediments are helpful to improve the fertility of the irrigated fields. Tesfai and Sterk (2002), have shown that sedimentation has improved the chemical and physical characteristics of the irrigated fields, but the surface level of the irrigated fields is rising every year. Too large variations in the levels within fields lead to over watering and leaching of plant nutrients at lower levels and under watering at higher levels resulting in a poor water use efficiency and typically uneven crop growth and yields within the same field, Goldsworthy (1975), Williams (1979), WS Atkins (1984) and Mu'Allem (1987) cited by (Community spate irrigation (nd)). Crops in the low-lying flood irrigated fields do not grow well and suffer from nitrogen deficiency, Mu'Allem (1987) cited by (Community spate irrigation (nd)). In addition there is a change in river, irrigation fields and canal bed levels

Figure 2. Fokisa spate irrigation scheme main canal (capacity reduced as a result of sedimentation)

Provision of sand traps at the end of the intake structures of river diversion schemes tends to be common practice to minimize the effect of sedimentation. Similarly, spate irrigation diversion schemes have been designed with sand traps. The basins in spate irrigations have been designed and operated like the sand traps of the conventional river diversion intakes, where the flow is diverted at relatively lower sediment concentrations or flow volumes. However, sand traps for spate irrigation should be designed differently as these schemes are usually located at the bottom of mountains which carry coarser and high sediment loads and sediment supply fluctuates with the amount of diverted flow.

Sedimentation in spate irrigation has many advantages. One of the main advantages is fine sediments transported towards the command area will improve the fertility of the irrigation field. Tesfai and Sterk (2002) have shown that, sediments contain plant nutrients, which enrich the soil. In addition, sediments append plant nutrients and organic matter to the spate soils. Sedimentation also expands the irrigable land by developing deep alluvial soils with good water holding capacity on originally dry infertile sandy soils (Tesfai, 2001).

In contrast, sedimentation has many disadvantages. Firstly, coarser sediments that are dropped in the intake structure clog the diversion intakes or reduce the diversion design discharge, and these in turn reduce the diversion efficiency of the intake. Secondly sedimentation also clogs pipe structures in canal systems. Finally, it raises the surface level of the command area (Tesfai and Sterk, 2002). Hence, major increase in elevation due to sedimentation means that the floodwater cannot be conveyed into these higher fields. Therefore, there will be a need to shift the headwork upstream of its previous location or reconstruct and raise the existing intake.

Figure 3. Gabion headwork built over the weir in Fokisa spate scheme

Sedimentation depends on many factors such as, the amount, type, availability and composition of the sediment load that a river carries, which also depends on the rainfall pattern and the characteristics of the catchment area, its geology, morphology, amount of flow and its velocity, vegetation cover, slope etc. (Community spate irrigation (nd)). Hence to compromise between the merits and demerits, sand traps that are capable of trapping only coarser sediments must operate very well. Therefore the design and operations of the spate irrigation schemes requires a good understanding of the hydrology and hydraulics of the flows that are likely to arrive and the nature of the sediments and supply.

To negotiate between the fertility improvement and bed level rise, designing sand traps that are capable of trapping only coarser sediments is a challenge. This is mainly because, there is high variability between the flows passing through the basin and the basin does not need to trap fine sediments at all and should only trap the coarser sediment fraction. Most of the spate irrigation sand traps constructed have not been designed in such a way that they meet the above criteria. Moreover sand trap basins of spate irrigation schemes have not been addressed very well i.e., research to support the designs and the hydraulic operations have not been completed. Hence, the main focus of this research is to address the problems stated above and to propose and test alternative solutions.

1.3.1 Research Objectives

The objectives of this research work are:

- To analyze the existing sediment control and management systems of spate irrigation;
- To review and test sediment control and management options and structures.
- To recommend as necessary, innovative alternative sediment control and management systems and structures.

1.3.2 Research Questions

Based on the above definition of the problems, the following research questions are developed.

- How effective is the existing sediment control system of the Fokisa scheme?
- What if any, alternative sediment control and management as well as hydraulic structures, improvement measures can be recommended?
- Would a sand trap based sediment control and management system perform effectively in this scheme?

1.4 Thesis Setup

In the following chapter, literature on spate irrigation floods, sedimentation, headwork, sand trap basins and canals will be reviewed. Moreover, sedimentation estimates and, sediment control and management options will be assessed. Then it is followed by chapter three, which deals with theoretical background and literature review on basics of sedimentation and sediment transport. In chapter four, research methods and materials used is discussed. Chapter five analyses and discusses major research findings and results. Chapter six provides uncertainties associated with the research, limitations and assumptions of the research. Based on the research results, the conclusions and recommendations are in chapter seven.

2. LITERATURE REVIEW

2.1 Floods in Spate Irrigation

River floods used for spate schemes are unpredictable in timing, frequency and volume (Mehari et al. 2005). Moreover, unlike the other conventional irrigation schemes, spate irrigation is a pre-planting system, where the flood season precedes the crop production period. Hence it is a difficult and risky investment with many uncertainties. However, many of the traditional spate irrigation schemes have local rules and regulations which make them very useful for improving many livelihoods. Farmers in spate irrigation schemes have traditional working rules and regulations, which enable them to share the available floods equally and proportionally especially, in times of very low floods. Small floods tend to be diverted to the upper sections of the command area, because, small floods are not likely to travel that far, (Mehari et al. 2005).

Mul et al. (2008) have indicated that, during smaller flood events the flow contribution originates mainly from the upper catchment. The response time of the catchment at all scales is short. A comparison of measured flood runoff depths with areal rainfall derived from five rain gauges located in 597 km^2 catchment in western Saudi Arabia presented in (Wheater, 1996) is shown below (Wallingford, 2005). The plot shows no correlation between run off measured at the catchment outlet and rainfall events observed with rain gauge network, which has a density of around one per 120 km^2. In this example the storm with the largest run off appears to be generated by the smallest rainfall. Extreme rainfall intensities and short concentration times characterize spate schemes.

The Wadi Laba farmers categorize the spate floods into six types: very small, small, medium, moderately-large, large and very large based on the surface area the floods cover in the Wadi and on some natural height measuring elements such as huge trees and historical large stones (Mehari, 2005). All the measured spates displayed some common flow characteristics, namely a rapid increase of the discharge in the first half hour and a peak with a short duration of about 10 minutes (Mehari et al. 2005b). The peak was followed by a sharp decline in discharge for nearly half to one hour and a gradual decline and recession that extends from several hours to 3 to 4 days as shown in figure 4 below.

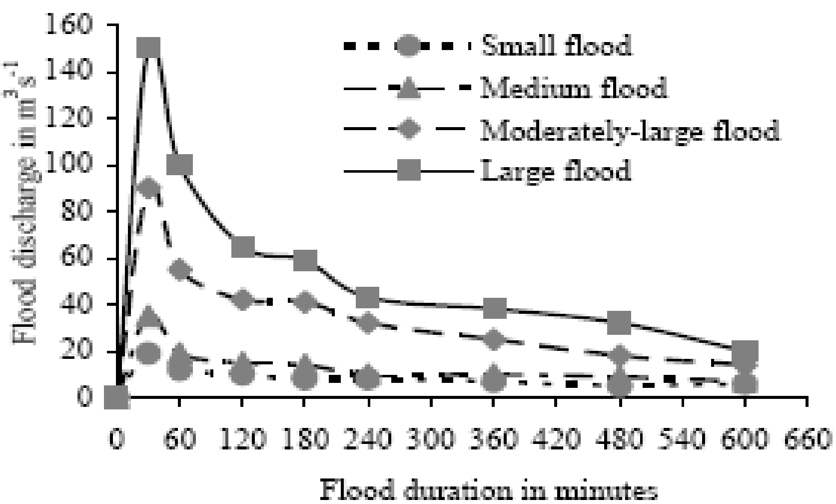

Figure 4. Hydrograph of small, medium, moderately-large and large floods at the Wadi Laba scheme (Mehari, et al, 2005b).

Attempts have been made to establish relationships between flood peak discharges and flood durations and flood volumes, but recent studies in Yemen and Eritrea, (Arcardis, 2004) and (Halcrow, 1997), have shown little or no correlation between peak discharges and flood volumes (Wallingford, 2005). More over, floods with a small peak discharge can have a long duration, and a large flood volume, while conversely floods with a large peak discharge can have a very short recession, and a small flood volume. Hence spate floods are chatacterized by a very rapid rising limb, therefore, they should not be represented using classic triangular hydrograph models such as those of the US Soil Conservation Service (Wallingford, 2005). These do not show the rapid rise to peak, the rapid initial recession, or the proportions of the flood volume occurring before and after the flood peak. These characteristics enhance the unpredictable nature of such big floods at the outlet of the catchment.

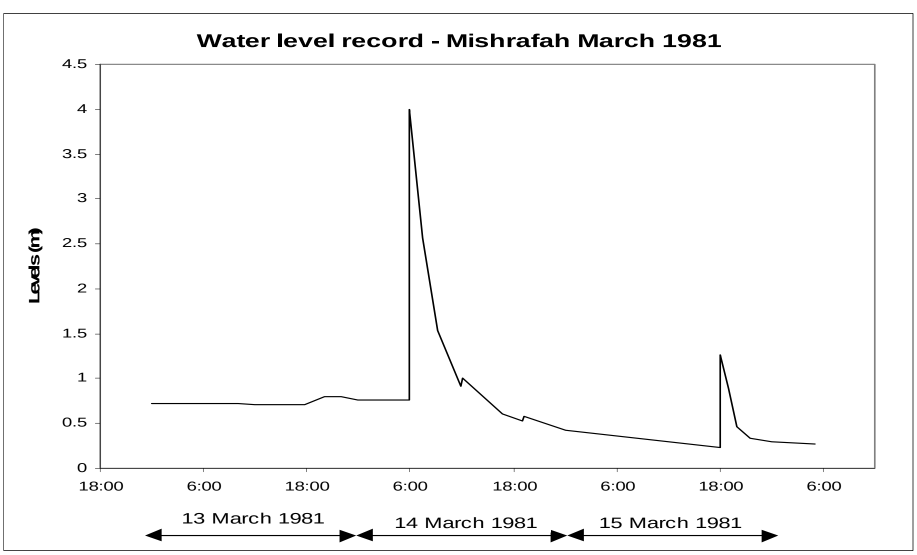

Figure 5. Spate irrigation flood hydrograph from Wadi Rima, Yemen

2.2 Spate Irrigation Sedimentation

In most of the spate irrigation schemes, floods from the mountainous area are the sources of sedimentation. Sediments of all sizes are transported in the large floods. Coarser sediments settle in the channels and canal system and finer sediments are deposited in the irrigation fields, (Lawrence, 2008). Therefore, the sediment fraction has its own advantage and disadvantages. In addition, sediment concentrations rising to and exceeding 100000 ppm or 10% by weight occur in floods in some wadis, and sediment concentration up to 5% by weight are common, (Lawrence, 2008).

Sedimentation has been one of the key issues in developing improved spate irrigation schemes. Deposition of coarser sediments clogs intakes and canals, reducing the discharges that can be diverted and conveyed. Sedimentation on the fields, wanted by farmers, progressively increases command levels over time. This is not a serous problem in traditional systems, where the simple intakes can easily be moved upstream to regain command, but has to be considered when “engineered” diversion weirs and intakes are constructed. Tesfai and Sterk (2005) has indicated that, the sedimentation rates in the Sheeb, Eastern Eritrea spate irrigation scheme, ranged from 8.3 - 31.6mm/y to 5.2 - 8.6mm/y from upstream to downstream in the command area.

Of course sedimentation in spate schemes has also some advantages. Fine sediments carry nutrients and improve the fertility of the irrigated fields. Compared to non-irrigated soils, sedimentation improves the physical and chemical characteristics of spate irrigated soils (Tesfai and Sterk, 2005). Sedimentation develops deep alluvial soils with good water holding capacity on what were originally dry infertile sandy soils (Tesfai, 2001).

Figure 6. Sedimented canal and settling basin of spate scheme in Tigray, Ethiopia

2.3 Spate Irrigation Headwork

Spate schemes are found in many countries, for example, Pakistan, Iran, Afghanistan, Yemen, Saudi Arabia, Morocco, Algeria, Tunisia, Ethiopia, Eritrea, Kenya, Sudan, Somalia, and soon. According to Steenbergen et al. (2008), spate irrigation is rapidly expanding in Ethiopia and Eritrea. Currently many countries are developing the traditional spate schemes in a modern way.

In traditional spate irrigation schemes, headwork structures are made from earth with tree trunks and bushes or from stone bunds mixed with wood and wadi bed materials (Gebremariam, (nd)). Some of the traditional spate schemes are made of gabion reinforced stone bund deflectors. In most of the traditional schemes, these headworks are constructed at an angle across the river. Contrary to this, the modern spate irrigation schemes are built from concrete and mostly at 90 degrees to the river flow.

Some of the larger spate irrigation schemes are categorized among the largest farmer managed irrigation systems in the world. The diversion structures are sometimes spectacular: earthen bunds, spanning the width of a river, or extensive spurs made of brushwood and stone bunds (Community spate irrigation (nd)). These traditional spate systems are very helpful in keeping the largest floods away from the command area. Very large floods would create considerable damage to the command area and canal structures. Moreover, they are helpful in avoiding sedimentation in cases of large floods (Anderson, 2007). This is possible because large floods will breach and take these traditional head work structures. However, sometimes they destroy flood diversion channels and cause rivers to shift.

Figure 7. Traditional spate irrigation headwork in Tigray

Anderson (2007) has indicated that, while traditional intakes may seem crude, they have enabled irrigation to be sustained for many years using only local materials and indigenous skills. They are very flexible, as the location and layout can easily be adjusted to suit the changing wadi conditions. As the level of the command area rises, they can easily be moved upstream. In addition they are appropriate and of low cost. Furthermore, they are relatively efficient in water use and sharing between users.

Though, these traditional schemes are very helpful, they have some drawbacks. Firstly, they are labour intensive, an enormous input of labor is needed to maintain and reconstruct intakes that are damaged or washed out by large floods and continual use (refer to figure 7). Secondly, with multiple intakes, it is not possible to divert water where it is needed, especially, towards the downstream plots. Thirdly, there are some environmental problems associated, resulting from the unsustainable use of trees and brushwood.

2.4 Spate Irrigation De-Silting Basins and Canals

In conventional river diversion schemes, the right choice of diversion angle is one of the most important parameters that influence intake sedimentation (Boeriu, 2008). Moreover, for given water course no definite angle of diversion can be established because, the angle of inflow varies with the ratio of diverted discharge. Any increase of the diverted discharge will increase the angle of diversion. Vanoni (2004) has also indicated that, there is no optimum diversion angle, because this angle would vary with the diversion ratio (the ratio of discharge in the diverted field to that of in the stream) and the position of the intake in a bend. Therefore, this diversion angle is site specific and should be fixed by modeling. In spate schemes, in practice however, the majority of the intakes divert 100% of the flow, and the diversion angle would have little effect, (except during flood peaks, where part of the peak flood is diverted).

In conventional river diversion schemes, settling basins are provided to trap and remove sediments entering the basins. The settled sediment is removed by sluicing, flushing or dredging. The sediment traps (of the conventional irrigation) in the canalized flow downstream of the intake are less subjected to unpredictable flow variation than in the river itself which, within certain limits ensures more reliable and efficient operation (Boeriu, 2008). Therefore, the sediment basins in the conventional irrigation schemes are not subjected to highly varying floods. In spate schemes, however, the aim is to trap coarser sediments only with variable floods. Effective sediment exclusion at an intake may not be possible, or will be insufficient to exclude all the larger sediments (Lawrence, 2008).

Canals for perennial schemes are often designed using maximum and minimum velocities set by "non scouring-non silting" criteria (FAO, 2002) as cited by (Lawrence, 2008). As concentrations of sand diverted to canals is far larger in spate schemes than is the case in perennial schemes, canals are operated at a fraction of their design capacity for most of the time, spate canals designed in this way rapidly silt up (Lawrence, 2008). Chang (1985) gives predictions of slopes and bed width which are similar to wide shallow canals observed in many spate canals (Lawrence, 2008). In fact, flow variations in spate canals are similar to those in wadi, except for the short periods when diversions exceed the wadi flow.

2.5 Sedimentation Estimates

Lawrence (2008) has reported that, according to the information partially derived from FAO data base, and supplemented with data collected during his project studies, the sediment yield from catchement yields in Ethiopian and Eritrean highlands is:

$$\text{Sediment Yield} = 3209\ A^{-0.21} \qquad \text{[eq. 2.1]}$$

Where, A = catchement area. Therefore, for this scheme, an area of 75 km^2, the expected sediment yield would be around 1300 tons/km^2/yr. According to Lawrence et al. (2001), all sediments larger than 63 microns are generally assumed to be sand. Particle size ranging from 0.063 mm to 2.0 mm are categorized by sand, after Jansen (1979), cited by (Foppen et.al. 2008). Furthermore, the specific gravity of sand is assumed to be 2.65 (Lawrence et .al. 2001). Therefore, assuming the entire sediment yield having a specific gravity of 2.65 and the whole sediment to be uniformly distributed in the irrigated field of 150 ha, the annual field rise rate of the area would be 25 mm/yr. However the sediment distribution is not uniformly distributed in the entire command area, rather it would be highly distributed near the canal route, raising the nearby fields significantly.

2.6 Sediment Control and Management Options

Effective sediment control and management in spate irrigation schemes is one of the main challenges. This is mainly because, it requires excluding the diversion of only coarser sediments and letting fine sediments to the fields through the canal system without settling in the canals and sediment trapping structures (Lawrence, 2008). There are many methods to control and manage canal and irrigated field sedimentation in spate schemes. Some of them are listed below.

- Provision of sediment settling basins, or other secondary sediment control structures, and making design provisions for the bed level rise of the irrigated fields.
- Avoiding floods carrying high sediment loads using deflector weirs.
- Changing canal cross sections to affect and vary the velocity of the flow so that the majority of fine sediments are transported through long main canals and coarser sediments are dropped immediately downstream of the off take in the main canal.
- Locating intakes at the outside of bends which is very helpful to manage sedimentation problems.
- To provide simple sediment sluice with low sill set at the existing wadi bed level, to align canal intake at a shallow angle to the axis of the Wadi and to consider arrangements for and sustainability of canal de-silting if needed (Lawrence, 2008).

Therefore, the first three options have been evaluated here. In addition the results from particle size distribution analysis have been analyzed to provide best management practices.

Figure 8. Farmers removing sediments from the main canal by dredging, at the Fokisa spate scheme

3. SEDIMENTATION and BASICS of SEDIMENT TRANSPORT

According to Depeweg et al. (2007), sediments are fragmented material, primarily formed by the physical and chemical disintegration of rocks from the earth's crust. They are formed as a result of weathering and are transported by liquid, gravity and wind. The principle of sediment transport in water is governed by the hydrodynamic nature of flow and sediment parameters. As the flow's sediment carrying capacity exceeds the actual sediment concentration of the river, erosion of the river banks and or beds takes place. However, when the flow's sediment carrying capacity is lower than the sediment concentration deposition or sedimentation will occur.

Three modes of motion can be distinguished in the sediment transport induced by flowing water: rolling and sliding, saltation, and suspension (Depeweg et.al. 2007). Based on the above three modes of motion two different ways of sediment transport can be defined, bed and suspended loads. The definition in accordance with the ISO-standards (ISO4363) is as given below (Lawrence et al. 2001 and Van Rijn, 2006) cited by (Wubneh, 2007).

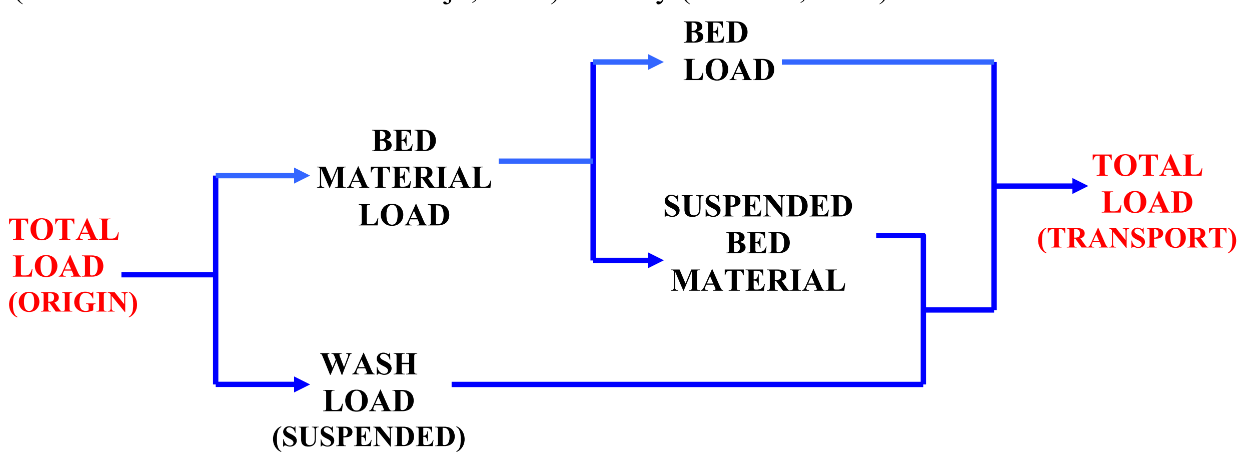

Figure 9. Classifications of sediment transport

Bed material load: The part of the load sediment transport which consists of the bed material and which rate of movement is governed by the transport capacity of the channel.

Suspended load: That part of the total sediment transport which is maintained in suspension by turbulence in the flowing water for considerable period of time without contact with the stream bed.
Bed load: The sediment transported in almost continuous contact with the bed, carried forward by rolling, sliding or hopping.

Wash Load: That part of the suspended load which is composed of particle size smaller than those found in appreciable quantities in the bed material. It is in near-permanent suspension and, therefore, is transported through the stream without deposition. The discharge of the wash load through a reach depends only on the rate with which these particles become available in the catchment area and not on the transport capacity of the flow.

Suspended bed material load: that part of the suspended load that is composed of particle sizes present in the channel bed. The concentration of suspended bed material is governed by the hydraulic parameters in the channel reach.

Total load: All the sediment in transport.

3.1 Initiation of Motion (Shield's Curve)

Any particle in flowing water will tend to move when the fluid forces imposed on it exceed the resisting forces as a result of the material property. Many studies have been under taken on the initiation of motion and most of them are based on critical bed sheer stress. The most accepted criteria are according to Shield's diagram. Shield's curve expresses the relation between the critical mobility Shield's parameter θ_{cr} and the dimension less particle Reynolds number R_e. Accordingly, initiation of motion will occur when the actual mobility Shield's parameter θ exceeds the critical mobility Shield's parameter θ_{cr}

$$\theta_{cr} = \frac{\tau_{cr}}{(\rho_s - \rho)gD} = \frac{\upsilon_*^{\,2}}{\Delta g D} \qquad \text{[eq. 3.1]}$$

and

$$\mathrm{Re} = \frac{\upsilon_* D}{\nu}, \qquad \text{[eq. 3.2]}$$

$$\upsilon_* = \sqrt{\frac{\tau}{\rho}} = \sqrt{gRS} \qquad \text{[eq. 3.3]}$$

Where, θ_{cr} = critical mobility Shield's parameter
τ_{cr} = critical sheer stress for initiation of motion (N/m^2)
ρ_s =density of sediment (kg/m^3)
ρ = density of water (kg/m^3)
g = acceleration due to gravity (kg/m^2)
D = characteristics grain size
υ_* = local shear velocity (m/s)
R = hydraulic radius (m)
S = slope (m/m)
Δ = relative density = $(\rho_s-\rho)/ \rho$

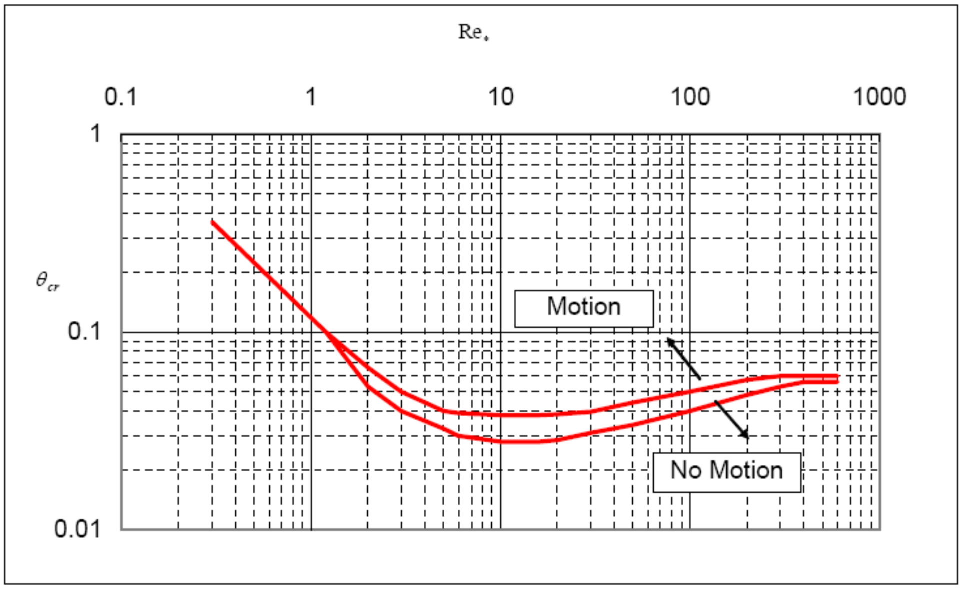

Figure 10. Shield's diagram for initiation of motion (θ_{cr} Vs Re)

According to Depeweg et al. (2007), the use of Shield's diagram is not practical , since the υ_* = appears on both axes of the diagram, and can only be solved by trial and error. This imperfection is eliminated by introducing the particle parameter D_* which is represented by (Yalin, 1977).

$$D_* = \frac{\mathrm{Re}^2}{\theta} = \left[\frac{\Delta g}{\nu^2}\right]^{1/3} d_{50} \qquad \text{[eq. 3.4]}$$

Van Rijn (1994) presented the relation ship between θ_{cr} and D_*, accordingly

$$\theta_{cr} = \frac{0.24}{D_*} \qquad \text{if } D_* \leq 4 \qquad \text{[eq. 3.5]}$$

$$\theta_{cr} = \frac{0.14}{D_*^{0.64}} \qquad \text{if } 4 < D_* \leq 10 \qquad \text{[eq. 3.6]}$$

$$\theta_{cr} = \frac{0.04}{D_*^{0.10}} \qquad \text{if } 10 < D_* \leq 20 \qquad \text{[eq. 3.7]}$$

$$\theta_{cr} = 0.013 D_*^{0.29} \qquad \text{if } 20 < D_* \leq 150 \qquad \text{[eq. 3.8]}$$

$$\theta_{cr} = 0.055 \qquad \text{if } D_* > 150 \qquad \text{[eq. 3.9]}$$

3.2 Fall Velocity

The primary variable defining the interaction of sediment transport with the bed, banks or suspended in the fluid is the fall velocity of sediment particles, (Simons et al. 1992).It has been shown that the bed configuration in a sand channel may change when the fall velocity of the bed material changes. Therefore, it is a very important parameter which determines the suspension and sedimentation of sediments.

The fall velocity of a sphere is the fall velocity of a particle when the drag force on the particle is in equilibrium with the gravity force (Gaweesh, 2006), cited by (Wubneh, 2007). Accordingly,

$$1/2\{C_D \rho W_s\} * 1/4(\Pi D^2) = [1/6(\rho_s - \rho) g D^3]$$

Resulting in $W_s = [\frac{4(s-1)gD}{3C_D}]^{0.5}$ [eq. 3.10]

Where, Ws = terminal fall velocity of a sphere in still fluid (m/s)
D = sphere diameter (mm)
s = specific gravity
C_D = drag coefficient
g= acceleration due to gravity (m/s^2)

The drag coefficient is a function of Reynold's number, and shape factor.

Van Rijn (1984) has indicated that, as cited by Crosato, (2007):

- For Stokes range (Re = WsD/υ < 1 or D≤ 100μm):

$$w_s = \frac{1}{18}\frac{\Delta g D^2}{\nu} \quad \text{[eq. 3.11]}$$

- For 100< D ≤ 1000μm :

$$w_s = \frac{10}{D}\left(\sqrt{1+\frac{0.01\Delta g D^3}{\nu^2}} - 1\right) \quad \text{[eq. 3.12]}$$

- For D > 1000 μm :

$$w_s = 1.1\sqrt{\Delta g D} \quad \text{[eq. 3.13]}$$

3.3 Sediment Trapping/Settling Efficiency

Most settling basins of irrigation schemes are designed to trap sediments as much as they can, however, most of them fail to trap desired sediment especially when the diverted flow fluctuates. In spate schemes, where the diverted flow is unpredictable like the river flow, designing efficient settling basins is a difficult task. Trapping efficiency is the ratio of volume of trapped sediment to the volume of sediment entering the basin. Many studies have been undertaken to estimate the trapping/ settling efficiency of a settling basin under regulated diverted flows. Some of them will be briefly discussed below.

3.3.1 Camp and Dobbins (1946)

Dobbin (1944) has studied the settling of discrete sediment particles having a single settling velocity in turbulent flow using the theory of turbulent sediment conveyance cited by (Simanjuntak, 2007). Camp (1946) extended Dobbins study and developed an expression for the removal of particles having a single settling velocity (Simanjuntak, 2007). It is obtained by integrating the equation for the transient concentration profile during settling. According to Camp (1946), trapping efficiency of a settling basin depends on an analysis of the effect of turbulence upon the rate of deposition.

$$1-\frac{(qs)_l}{(qs)_e} = f\left(\frac{w_s}{\upsilon_*}, \frac{w_s L}{Vh}\right) \qquad \text{[eq. 3.14]}$$

And $$\upsilon_* = \frac{V\sqrt{g}}{KR^{1/6}} \qquad \text{[eq. 3.15]}$$

Where, $(q_s)_l$= quantity of sediment of given particle size leaving the basin (m^3/s)

$(q_s)_e$ = quantity of sediment of given particle size entering the basin (m^3/s)

Ws = fall velocity of the given particle size (m/s)

K = Strickler roughness coefficient

R = hydraulic radius of the basin (m)

L = Settling length (m)

V = design flow velocity in the basin (m/s)

The above function has been evaluated analytically, as shown in figure 11. It is a practical and relatively easy way of estimating the efficiency of a settling basin in trapping suspended sediment.

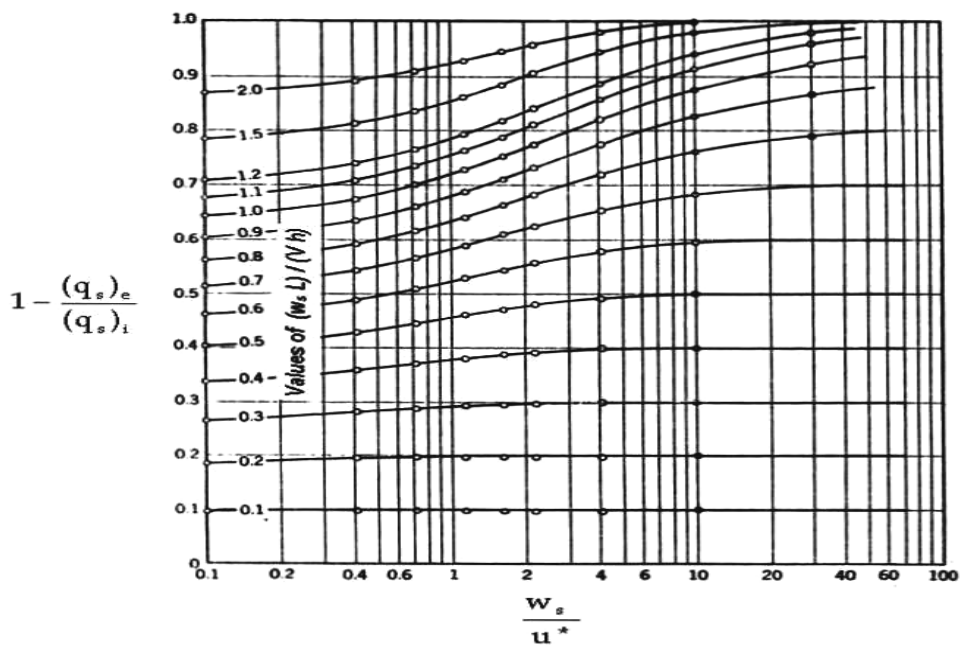

Figure 11. Camp (1946), relation between 1-$\frac{(qs)_l}{(qs)_e}$ and $\frac{w_s}{\upsilon_*}$

3.3.2 Vetter (1940)

Ellis et al.(1994) has indicated that, Vetter's (1940) trap efficiency formula, used for irrigation design, attempts to account more explicitly for the effects of turbulence on settlement and the equation corresponding to the turbulent side of Camp's (1946). According to Ellis et al. (1994), the Vetter formulation thus appears to derive a rather conservative estimate of the trap efficiency but may be best suited to the non uniform flow, short circuiting and inactive zones that occur in typical flood storage reservoirs and which cause the departure from ideal basin, through-flow conditions

$$\eta = 1 - e^{-(Vs * A/Q)} \quad \text{[eq. 3.16]}$$

Where η = settling efficiency
V_s = settling velocity
A = surface area of the basin
Q = discharge in the basin

A preliminary estimation of the trap efficiency of a sedimentation basin for sediments of mean size, $d_{(m)}$, may be obtained using the empirical equation (USBR, 1971, Barfield et al. 1981, Vetter 1940) cited by (Prakash, 2004).

$$\frac{w}{w_o} = \exp(-wL/q) = \exp(wA/Q) \quad \text{[eq. 3.17]}$$

Where, wo = weight of sediment entering the basin (Kg)
W = weight of sediment leaving the basin (Kg)
L = length of the basin (m)
w = fall velocity (m/s) of particles of size d (m)
A = basin surface area (m^2)
Q = basin inflow or outflow (m^3/s)

3.3.3 Vanoni (1975)

According to Vanoni (1975), an inherent and desirable characteristic of all settling basins is that they can trap a large percentage of the fine sediment fraction of the suspended sediment, along with the coarser fraction, which would have been carried through the canal system if a less efficient type of device, had been used, cited by (Simanjuntak, 2007). The settling basin can be designed to control the amount of suspended sediment removed by varying the dimensions of the structure and thus the time that the water is retained in the basin. The formula presented below can be applied for highly turbulent flow.

$$W = W_o e^{-w_s L/q} \quad \text{[eq. 3.18]}$$

Where, W = weight of sediment leaving the basin
W_o = weight of sediment entering the basin
w_s = settling velocity of a particle (m/s)

L = length of settling basins (m) and
q = discharge per unit width of settling basins (m^2/s)

3.4 Sediment Transport Predictors

Sediment can be transported in equilibrium or non equilibrium conditions, (Mendez, 1998). Equilibrium condition means sediments are transported without any change to the bed, with no erosion or deposition taking place. Sediment transport predictions are supposed to be under equilibrium conditions (Mendez, 1998). Mendez (1998) has also indicated that, there is no universally accepted equation to determine the transport capacity as a result their predictability of all of them is poor. The performance of sediment transport functions has been reviewed by ASCE (1975) and by White (1973), cited by (Lawrence, 1987). The former study showed that the mean ratio of observed to predicted transport rate was between 0.5 and 2 for only 64 percent of the comparisons for the best method that was tested. Prediction of wadi sediment transport rates are even more uncertain than indicated by White (1973), because of wide bed material sediment size grading, unpredictable flows and high Froude Numbers (Lawrence, 1987).

There are many sediment transport predicting formulas. Therefore, some of them will be briefly described below.

3.4.1 Ackers and White (1973)

This method is based on flume experiments with a uniformly or nearly uniform sediment size distribution, with an established movement including a range of bed forms, flow conditions for water depths smaller than 0.4m and a lower flow regime (Fr ≤ 0.8) (Mendez, 1998). This method describes the sediment transport in terms of three dimension less parameters, D_{gr} (grain size sediment parameter), G_{gr} (mobility parameter) and F_{gr} (sediment transport parameter).

$$D_{gr} = D_{35}\left[\frac{g\Delta}{\nu^2}\right]^{1/3} \qquad \text{[eq. 3.19]}$$

$$G_{gr} = 0.025\left[\frac{F_{gr}}{0.17}-1\right]^{1.5} \qquad \text{[eq. 3.20]}$$

$$F_{gr} = \frac{1}{\sqrt{\Delta g D_{35}}}\frac{\upsilon}{\sqrt{32}}\frac{1}{\log\frac{10h}{D_{35}}} \qquad \text{[eq. 3.21]}$$

$$X_* \frac{(\Delta+1)D_{35}G_{35}}{h}\left\{\frac{C}{\sqrt{g}}\right\} \qquad \text{[eq. 3.22]}$$

And Finally, $q = \frac{s\rho_s(1-\varepsilon)}{X_*\rho}$ [eq. 3.23]

Where, g = acceleration due to gravity
Δ= relative density
υ= kinematics viscosity

D_{35} = sediment particle size
h = depth of flow
C = Chezy coefficient
ε = constant, 0.4
ρ = density of fluid
ρ_s = density of sediment

3.4.2 Engelund and Hansen (1967)

The method of Engelund and Hansen is based on energy approach (Mendez, 1998).They established a relationship between transport and mobility. The formula of Engelund and Hansen (1967) total for total load (bed and suspended material load) is written as follows

$$q_s = \frac{0.05V^5}{(s-1)g^{0.5}d_{50}C^3} \qquad \text{[eq. 3.24]}$$

Where, q_s = total sediment discharge per meter width (m^3/sm)
V = mean velocity
s = relative density
g = acceleration due to gravity
d_{50} = mean particle size
C = Chezy coefficient

According to A. Crosato (2008), Engelund and Hansen formula is valid for situations in which,

- 0.19mm < d_{50} < 0.93mm
- $0.07 < \theta < 6$, where θ is Shield's parameter and,
- $\frac{w_s}{\upsilon_*} < 1$, where w_s = particle fall velocity and υ_* = shear velocity

3.4.3 Van Rijn (1984a and 1984b)

According to this formula, the total sediment load transported can be computed by summing the bed and suspended load transport (Mendez, 1998). The bed load transport q_s is computed by multiplying the saltation height, the particle velocity and bed load concentration. It assumes that, the motion of the bed particles is dominated by gravitational force. Van Rijn equation, which is prominent to the suspended sediment, the formula is limited to the particle size in range of 64µm to 2000 µm (Wubneh, 2007).

The bed transport rate is:

$$q_b = 0.053(s-1)^{0.5} g^{0.5} d_{50}{}^{1.5} D_*{}^{-0.3} T^{2.1} \qquad \text{[eq. 3.25]}$$

and the suspended load transport:

$$q_{sus} = FVhC_a \qquad \text{[eq. 3.26]}$$

Where, q_b = bed load transport rate (m^2/s)
q_{sus} = suspended load transport rate (m^2/s)
s = relative density

g = acceleration due to gravity
d_{50} = mean particle size
D_* = particle parameter
T = bed shear parameter
F = shape factor
h = water depth
V = mean velocity
Ca = reference concentration

3.4.4 Brownlie (1981)

This equation defined a method to compute the sediment transport rate, which is based on dimensional analysis and calibration of wide range of field and laboratory data where uniform conditions were present, (Mendez, 1998). The equation can be written as:

$$q_s = 727.6C_f (F_g - F_{gcr})^{1.978} S^{0.6601} \left(\frac{R}{d_{50}} \right)^{-0.3301} \qquad \text{[eq. 3.27]}$$

Where, q_s = total sediment transport rate per unit width (m^2/s)
C_f = coefficient for the transport rate
C_f = 1 for laboratory conditions, 1.268 for field conditions)
F_s = grain Froude number
F_{gcr} = critical grain Froude number
d_{50} = mean particle diameter (mm)
R = hydraulic radius (m)

3.4.5 Yang (1973)

Yang formula is based on the hypothesis that the unit stream power, VS, is the dominant factor determining the sediment concentration, X, in alluvial channels (Karamisheva et al. 2005). This method also called as the Yang stream power function, is based on the hypothesis that the rate of sediment transport in a flow should be related to the rate of energy dissipation of flow, (Mendez, 1998). The rate of energy dissipation is defined as the unit stream power and it can be expressed as the velocity times slope (V*S). By integrating turbulence energy production rate over flow depth, the suspended sediment transport can be expressed as a function of the unit stream power. Yang's formula can be expressed as:

$$\log C_t = I + J \log \left(\frac{VS - V_{cr} S}{W_s} \right) \qquad \text{[eq. 3.28]}$$

Where, C_t = total sediment transport expressed in PPm by weight
I, J = coefficient in the total sediment transport of Yang's function
Ws = fall velocity (m/s)
V = mean velocity (m/s)
S = bottom slope
V_{cr} = critical velocity

3.4.6 Westrich and Juraschek (1 985)

Westrich and Juraschek (1985) developed a sediment transport equation for silt-sized material (Yang et al. 2006). The equation is derived in the laboratory with particles having a settled velocity ranging from 0.06 to 9mm/s. The predicted transport capacities obtained from this formula do not depend on bed material composition, but only on the material in suspension. The formula is expressed as:

$$C_v = \frac{0.0018\tau_b V}{(s-1)\rho g D W_s} \qquad \text{[eq. 3.29]}$$

where C_v = sediment capacity concentration (by volume),
τ_b = bed shear stress,
s = specific gravity of silt,
ρ= fluid density,
g = acceleration due to gravity,
D = water depth, and
W_s = settling velocity of the sediment particles.

4. MATERIALS AND METHODS

In this chapter the methodology employed and materials used will be discussed. Therefore, the methods used will be discussed below.

4.1 Methods

In this section, general approach and modeling sections will be discussed.

4.1.1 General Approach

This study is undertaken on Fokisa spate irrigation scheme located in Tigray, Ethiopia. It is one of the largest spate schemes in Tigray. The scheme has a design capacity of 500 ha but, currently it irrigates 150 ha of land mainly due to sedimentation. It has a severe problem of sedimentation in the canals and irrigated fields. To solve this problem, the sediment control and management system of the scheme has been analysed.

The study is started by reviewing literatures which are helpful to achieve the research objectives (stated in section 1.3.1). Then, available primary and secondary data were collected from the field, interviews held with farmers and experts, from TWRMEB library, and literatures. The collected data include:

- Topographic surveys of the river, canal route, and part of the command area
- Particle size distribution
- Sediment concentration measured at flows not exceeding 60 l/s.
- Design report of the scheme
- Auto Cad drawing map of the scheme.
- Geological and watershed study report of the scheme
- Operational and dredging frequencies of the canal and
- Hydrological parameters such as flood marks, time to peak and time to end has been collected as a result of interviews held with farmers and experts around the scheme.

These data have been analysed and prepared in such a way that they can be used in the models and for further analysis. The details of the collected data and model inputs are in the data collection, field survey and analysis section (for more details refer to Section 4.3). As a result of the analysis, the following input data were developed

- River and canal cross sections, slopes along with the distance between each cross section
- Hydrographs (yearly expected small, medium, large flood and yearly long series hydrographs)

- Representative particle sizes, D_5, D_{15}...D_{95} have been estimated from plotting river bed grading sizes on the same scale.
- Sand concentrations of the river and canal route (estimated using DORC model).
- Silt concentrations have been adopted from the Wadi Laba Scheme,
- Fall velocities have been adopted from the Gezira scheme in Sudan available in the DOSSBAS model.

For this study, four sediment control and management systems have been tested and analysed. These include, settling basin, lengthening the main canal, deflector/ rejection weir at the upstream canal and a new approach which is to modify the canal cross sections and slope. To test the applicability of the settling basin, three models were used. DORC model is used to generate model inputs such as sediment concentrations. DOSSBAS steady state model is used to design a sand trap basin which would trap most of coarser sediments and allows the majority of the fine sediments in to the irrigated fields. ISIS sediments 1D hydrodynamic model with sediment transport capacity is used to check how the settling basin performs under unsteady conditions. Therefore, the results from DOSSBAS steady are further tested by ISIS sediments model. For details about the models refer to section 4.1.2. To check how the inputs affect the ISIS model results, sensitivity check has been carried out on the following parameters:

- Hydrograph shape changes
- Flow event order changes
- Particle size distribution
- Sediment concentration
- Sediment transport equations and
- Manning's roughness coefficient, n

For details about the sensitivity analysis refer to section 5.4.

Other options such as changing canal cross sections, lengthening main canal and avoiding high flows using deflector weir have been dealt in parallel with the modeling of the settling basin. Finally, all results have been analyzed in view of research questions. The strategy that improves the efficiency of the sediment control and management of the scheme has been recommended. Furthermore, future research foci have been recommended. A summary of the research approach is as indicated in figure 12.

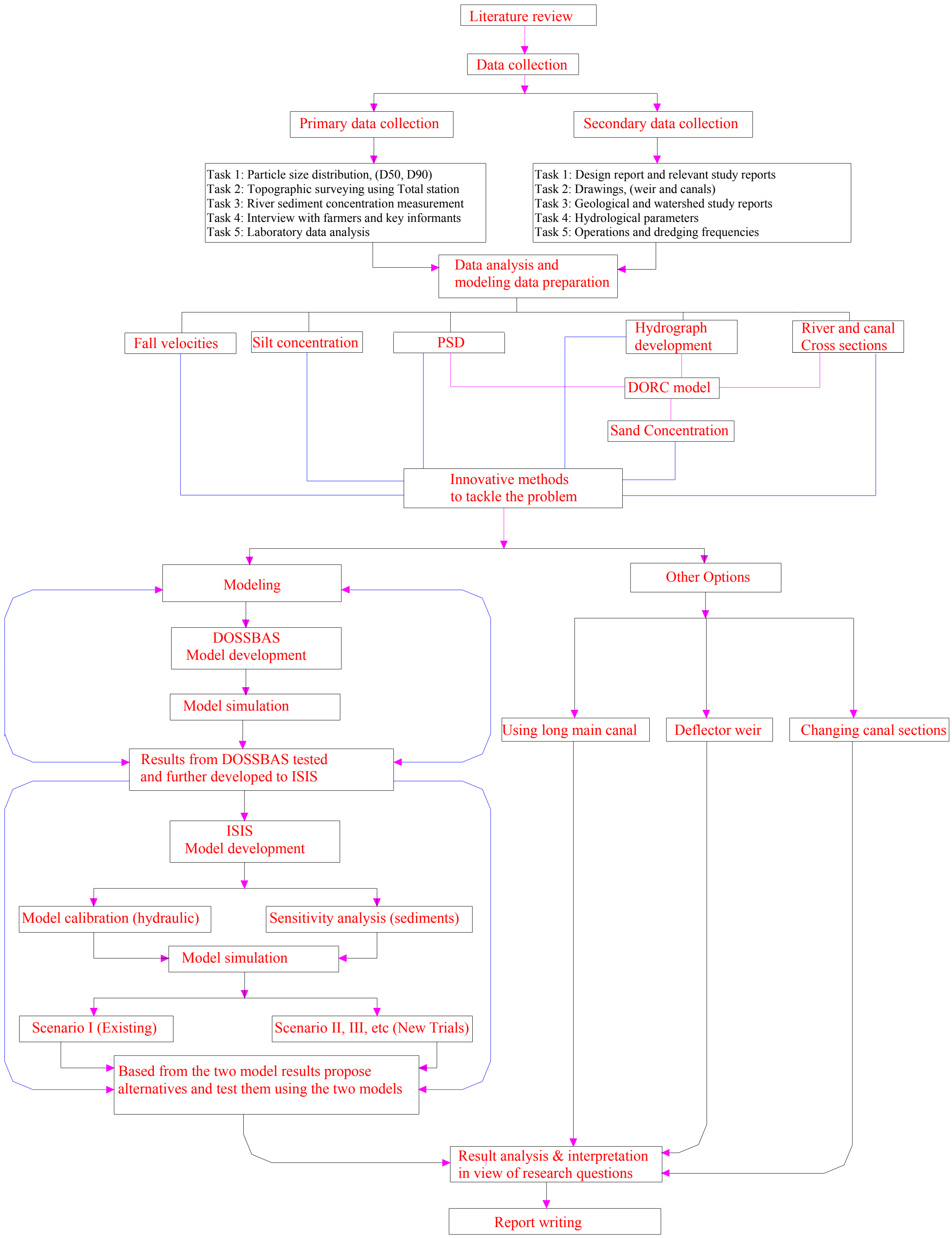

Figure 12. Research methodology flow chart

4.1.3 Modeling

For this study three models were used. These are DORC and DOSSBAS models, which are part of the SHARC model, and ISIS sediments. Why the three models DORC, DOSSBAS and ISIS sediments are used, is briefly discussed below. Before that, it is wise to briefly describe SHARC.

SHARC is a package of integrated programmes designed to assist in the identification and solutions of sediment problems in irrigation canal systems (Wallingford, 2002). SHARC stands for "*Sediment and Hydraulic Analysis on Rehabilitation of Canals*". It is a package composed of six components:

- Problem Diagnosis and Initial Options
- Preliminary Economic Screening
- Design Tools such as DOSSBAS, DORC
- Hydraulic Simulation
- Environmental Impact and
- Economic Analysis

DORC is a tool which is developed to assist the design alluvial canals. It provides a variety of options such as the Manning's equation, regime equation and rational method. In this study DORC is used to generate potential sediment concentrations transported by the river through its sand transport estimating options. The alluvial friction option enables to calculate the alluvial roughness and the hydraulic characteristics such as depth, velocity, slope discharge and shear velocity, which are inputs to the sand transporting estimating option.

For the Fokisa scheme sand concentration data for the large, medium and small floods, is not available. Hence, DORC model was used to generate the sand concentrations. For a given discharge, canal side and longitudinal slopes, canal bed width, specific gravity and sediment sizes (D_{16}, D_{35}, D_{50}, D_{65}, D_{84} and D_{90}), the alluvial friction option enables the prediction of the depth of flow, mean velocity and Manning's roughness n which are input to the sand concentration prediction option. Known output values such as depth of flow can be used as an indirect check or calibration unit, as is the case in this thesis, to compute mean velocity. Therefore, mean velocity (computed from alluvial friction option), sediment sizes, depth of flow, sediment size D_{50}, longitudinal slope, and specific gravity were used to estimate the sediment concentration.

DOSSBAS is a steady state model which helps when designing a proposed sand trap basin. DOSSBAS has an option which estimates sand trap and silt trap efficiencies of the basin separately. It is this option which enables to maximize the fine sediment trap efficiency and minimize the sand trap efficiency of the basin. It also predicts the deposition level (bed level rise) of the basin and volume of sediment deposited in the basin. It is a tool which helps to design both sluiced and mechanically excavated basins. In this study, DOSSBAS deposition model is used, in which the proposed sand trap is to be excavated mechanically. The main reason is farmers are not willing to let a drop of water for purposes other than irrigation.

ISIS sediments is 1D hydrodynamic model which is used to study the morphology of rivers and alluvial canals. It solves both steady and unsteady states. It is used to check how the proposed sand trap basin (as a result of DOSSBAS model computations) in an unsteady states. Though it does not predict the trapping efficiency, it enables to predict sediment transport rates, changes in bed elevation and amounts of erosion and deposition in the river and canal systems. Therefore, the proposed basin has been designed using DOSSBAS steady and tested using ISIS sediments. The mathematical formulations, inputs required, results weaknesses and strengths of the three models, DOSSBAS, DORC and ISIS sediments is discussed below.

DOSSBAS

Mathematical Formulation

DOSSBAS stands for "*Design of Sluiced Settling Basins* ".DOSSBAS model, which is part of the SHARC models, is a steady state model developed by the Hydraulic Research Wallingford. It also solves unsteady state solution but treats them as a series of steady flows. The initial lay out option in DOSSBAS is used to make an initial estimate of the required sediment trap efficiency using Vetters, 1940 trap efficiency equation and Vanoni, 1975. For details of the sediment trapping efficiency equations refer to section 3.3.The transport and deposition of fine sediments is derived using Westrich-Jurashek, 1985 predictor, while concentration change in sand sized sediments is computed from a turbulent diffusive equation.

Input Required

The input required to the DOSSBAS model are discharge, downstream water level, sand and fine concentrations, duration of run, geometry of the basin (length, width, side slope and bed elevations),width of the channel upstream of the basin, bed material size upstream of the basin settling velocities of sediments and so on.

Results

The results include adaptation length (a proportion of the basin length over which turbulence generated at the basin entrance, hinders sediment deposition), sand and silt trap efficiency, percentage of silt in the trapped material, mean sand and silt concentrations leaving the basin, total volume of sand and silt deposited in the basin, longitudinal deposition profile of the basin at different times, particle size distribution of the materials in transport, in transport and bed materials grading curves,

Strengths and Weaknesses

This model is not a widely tested model in research. However, it has extensively been used in the design of settling basins. One of the strengths of this model is it predicts the variations in trapping efficiencies as the basin fills with sediments. The other strength is, it displays separate efficiencies for the fine and sand sized particles which are very helpful in optimizing fine sediment efficiency and minimizing sand trapping efficiency in spate schemes. The weakness is it is based on steady state computations.

DORC

Mathematical Formulation

DORC stands for "*Design of regime canals* ". DORC model, which is part of the SHARC models, is developed by the Hydraulic Research Wallingford. It is a tool which provides many design methods to predict both alluvial friction and sediment transport in canals. The prediction options for designing canals are Regime equation (Lacey method and Simons and Albertson method), Tractive force method, Rational method (White et al, Chang, Rational method and Lacey width and Rational method and Lacey width), Manning's calculation, Irregular cross section calculations and Alluvial friction predictor method (Brownlie, Engelund and Hansen, Van Rijn and White et al). The sand transport prediction is based on six sediment transport predictor equations which are, Brownlie, Engelund and Hansen, Van Rijn, Ackers and White, 1973, Revised Ackers and White and Yang. For details of the sediment transport predictors refer to section 3.4. Silt transport prediction has three options namely, Westrich - Jurasech, Arora- Raju - Garde and Sediment delivery.

Input Required

The inputs required for each option depends on the nature and development of the equations. Therefore, discussing the inputs of every equation is not worth good. Hence some of the inputs which are required by the equations are discussed below. Canal parameters (side slope, longitudinal slope, depth of flow and bed width), discharge, silt factor, water temperature, particle size distributions (D_{16}, D_{30}, D_{35}, D_{50}, D_{65}, D_{84}, and D_{90}) specific gravity, sand concentrations, Manning's roughness and Settling velocities are the main inputs.

Results

The results from the DORC model for the canal design are depth of flow, mean flow velocity, Manning's roughness n, side slope, and shear velocity. The sand concentration predictors give sand concentrations in PPm and the silt concentration predictors are useful to make estimation of canal parameters as silt concentration are also dependent on catchment properties.

Strengths and weaknesses

The strength of this model is it has a wide range of methods to design canals. It also enables predictions of sand concentrations where bed material load measurements are not available. The weakness is, it is not possible to determine the sediment transport under non-equilibrium conditions (Wallingford, 1992) cited by (Mendez, 1998).

ISIS sediment

Mathematical Formulation

ISIS sediment is a hydrodynamic model developed by the Hydraulic Research Wallingford. The mathematical formulation of ISIS model is based on 1D St. Venant equation. The St. Venant or shallow water equations describe the motion of a body of water flowing in an open channel. These equations express conservation of mass and momentum. They arise from applying Newton's second law to fluid motion. They assume that the fluid stress is the sum of a diffusing viscous term, proportional to the velocity gradient, and a pressure term. Conservation of mass leads to the continuity equation which establishes a balance between the rate of rise of water level and wedge and prism storages. Conservation of momentum leads to the dynamic equation which establishes a balance between inertia, diffusion, gravity and friction forces.

$$\frac{\partial Q}{\partial t} + \frac{\partial A}{\partial t} = 0 \qquad \text{[eq. 4.1]}$$

and $$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + g\frac{\partial h}{\partial x} = g(S_o - S_f) \qquad \text{[eq. 4.2]}$$

Where: S_f is friction slope.

$$S_f = \frac{Q|Q|}{K^2} \qquad \text{[eq. 4.3]}$$

The friction slope S_f can be obtained from a uniform flow, such as Manning or Chezy.
K = channel conveyance calculated according to Manning's equation.

$$K = \frac{AR^{2/3}}{n} \qquad \text{[eq. 4.4]}$$

Where, R is hydraulic radius, $R = \frac{A}{P}$ [eq. 4.5]

A = flow cross sectional area
P = flow wetted perimeter
n = Manning's roughness coefficient
u = velocity (m/s)
t = time (s)
g = gravitational acceleration (m/s^2)
h = depth of flow (m)
S_o = water surface slope (m/m)
x = distance along the flow (m)

The St. Venant equation can be simplified to

$$\underbrace{\frac{1}{g}\frac{\partial u}{\partial t}}_{\text{dynamic wave}} + \underbrace{\frac{u}{g}\frac{\partial u}{\partial x}}_{\text{dyn. quasi steady wave}} + \underbrace{\frac{\partial h}{\partial x}}_{\text{diffusive wave}} = \underbrace{S_o - S_f}_{\text{kinematic wave}} \qquad \text{[eq. 4.6]}$$

To compute the sediment transport, ISIS sediments has four options. The first option is the Engelund and Hansen predictor, which predicts the bed material load. The second and third options are the Ackers and White, 1973 and the revised Ackers and White. The last option is the Westrich-Jurashek, 1985 predictor, in which its applicability is limited to fine sediments. For more details on the mathematical formulation of ISIS sediments please refer ISIS sediments manual.

Input Required

The inputs required for the ISIS 1D hydraulic model are river and canal cross sections, distances between each cross section, Manning's roughness coefficient, water surface slope, weir parameters such as discharge coefficient, breadth, upstream and downstream weir crest elevations, discharge versus time rating curve as an upstream boundary, and water surface elevation versus duration as a downstream boundary condition. For the 1D hydrodynamic model, in addition to the hydraulic model inputs, an input is required as a "filename.sed" data. In this data active layer thickness factor (ά), sediment transport calibration coefficient, bed porosity, erosion/deposition depth change, method to update channel geometry, sediment transport calculation method, sediment diameter, sediment density, sediment transport equation, proportion of sediment size proportion, sediment transport rate/ concentrations, cohesive sediment data, bed material grading options, hard bed options and so on are input. Furthermore time step of simulations, starting and finishing times are also input to both models.

Results

The hydrodynamic model results are expressed using graphs, text files and spreadsheet files and the hydrodynamic results include:

- Graphs expressed at any cross section are stage, velocity, discharge, Froude number, water depth, total energy, bed profile, maximum and minimum stage.
- Graphs expressed at any time t and along the longitudinal section, are, stage, velocity, discharge, Froude number, water depth, total energy, bed profile, maximum and minimum stage, sediment transport rate, bed level, bed change, sediment concentrations, net bed change and user defined output like, cumulative sediment passing each node, bed material size in the active and deposit layers, cumulative sediment deposition, cumulative dredged volume and so on.
- Graphs expressed in time intervals at any cross section are, stage, velocity, discharge, Froude number, water depth, total energy, sediment transport rate, bed level, bed change, sediment concentrations, net bed change and user defined output like, cumulative sediment passing each node, bed material size in the active and deposit layers, cumulative sediment deposition, cumulative dredged volume and so on.
- Combinations of the above variables can also be expressed in time intervals, for instance, flow versus stage in any time interval.

Strengths and Weaknesses

ISIS sediment is not specifically designed to design settling basins. Therefore, it definitely has some drawbacks and strengths. Some of the strengths and drawbacks of the model with respect to the objective of the thesis are presented below.

Strength

- It is possible to run both in steady and unsteady state flow conditions.
- There are many structures such as spills which enable to model bifurcation/ canal flow from off takes.
- It enables to divert variable discharge proportional to the flow in the river, which is best

to represent schemes where there is no head or flow regulating structure at the off take, or where the off take is an open gate.

Weaknesses

- There is no provision of diversion angle which directly affects sedimentation and its rate.
- There is no direct way to control diversion ratio and diverted flow.
- Simulations with zero flows are not possible for the sediment module.
- Specifically, for ISIS sediments, challenges are encountered when modeling extremes such as very low flows.

4.2 Description of the Study Area

4.2.1 Location of the Area

Tigray is the northern most region of Ethiopia, extending from 12°15’ to 14°50’N latitude and from 36°27’ to 39°59’E longitude. (Wubneh, 2007). It covers an area of about 80,000 km^2, most of which are highlands between 1,500 m and 3,900 masl (Solomon. et al 2006, and Mintesnot et.al. 2006). In the southern part of Tigray especially in Enda Mekoni (Raya Azebo) and Alamata weredas, there is an extensive tradition of diverting floods to supplement irrigated fields.

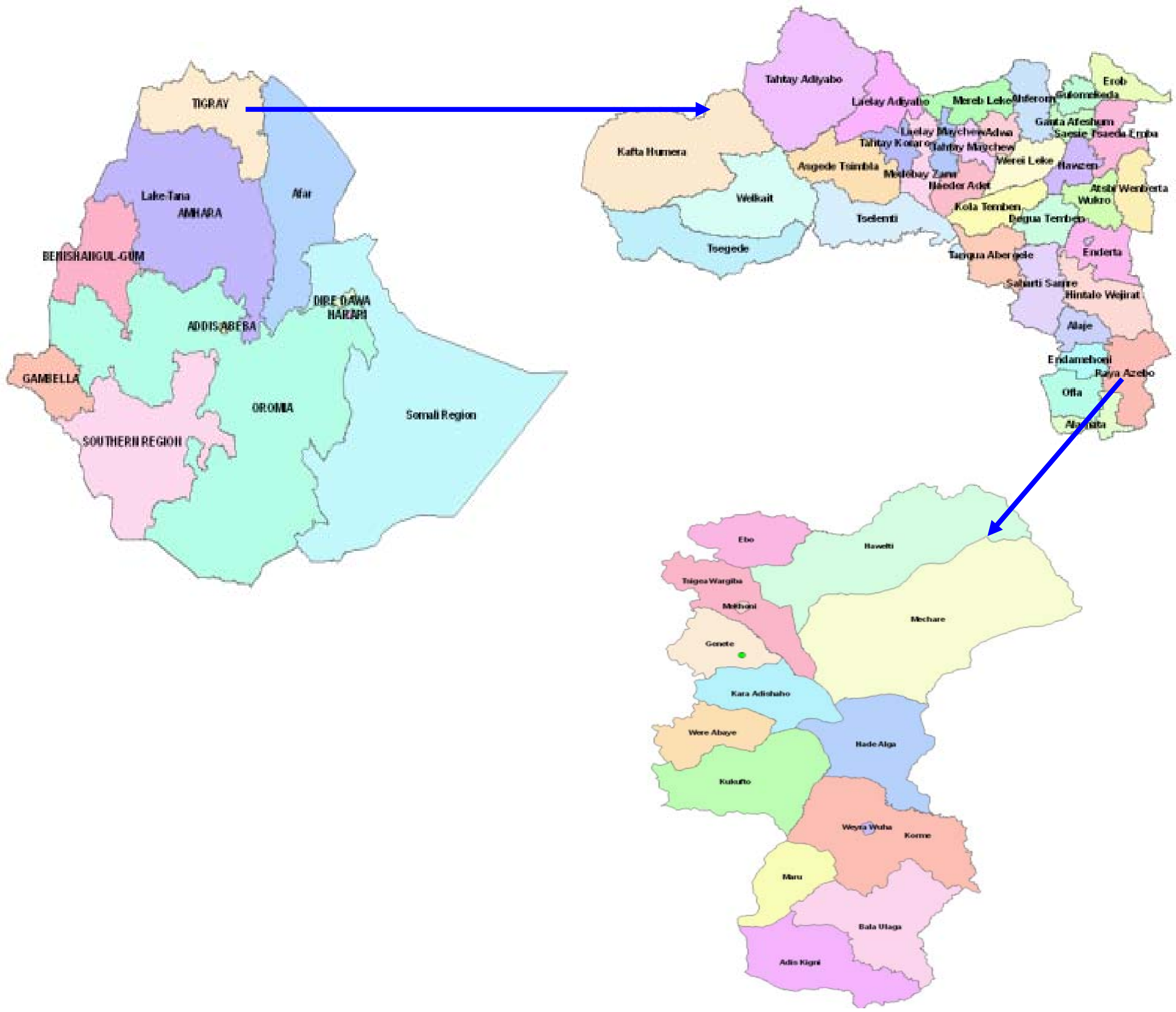

Figure 13. Location of the Fokisa spate scheme

Fokisa spate irrigation scheme, the study area of this research, is geographically located at Latitude: $12^0$42'13" - $12^0$46'21'' N Longitude: $39^0$30'20'' - $39^0$38'47''E and at an elevation of 1720 masl. Administratively the area is found in Tigray region, Southern zone, Enda Mehoni wereda, Genete Tabia and Hujera, and Genete kushets (according to the regional administrative hierarchy). The study area is also located 15 Km south of the town of Mekoni, which is 700 km North of Addis Ababa.

Figure 14. Fokisa spate irrigation scheme diversion headwork

The Fokisa spate irrigation scheme is located in the Danakil basin, one of the 12 big Ethiopian basins. According to Awulachew et al. (2007), Denakil river basin has an area of 74,000 km^2, which covers Tigray, Amhara and Afar regional states. The basin has no major river draining out of it. The basin has a lowest elevation of 197 mbsl at Denakil depression, the lowest altitude of the country, and a highest elevation of 3,962 masl. The total mean annual flow from the river basins is estimated to be 0.86 billion cubic meters.

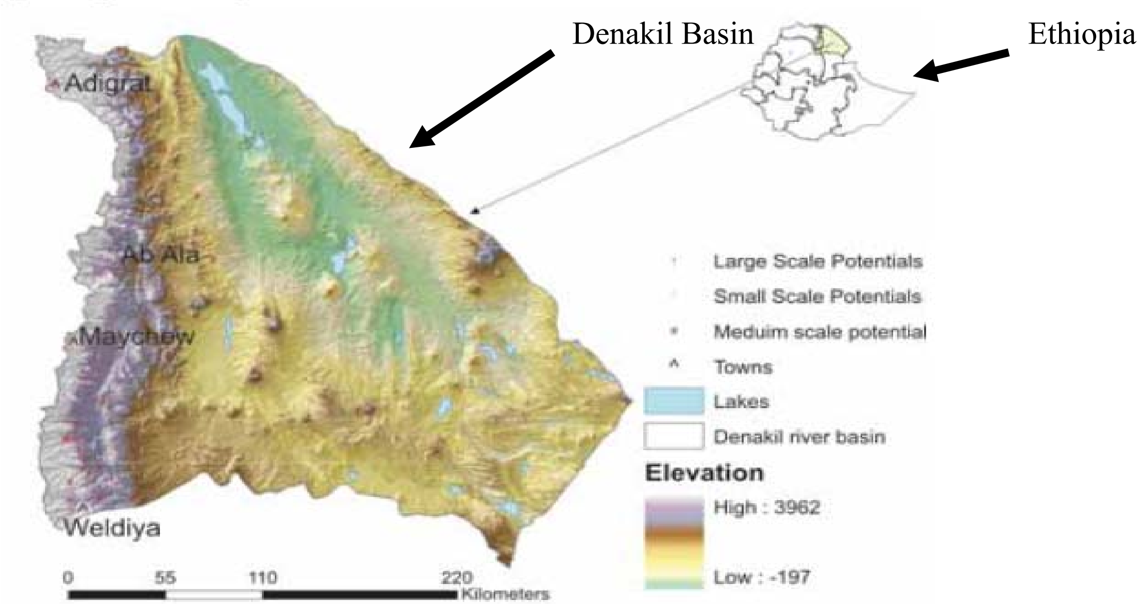

Figure 15. Irrigation potential of the Denakil basin (map) (2007)

In the Fokisa scheme, it is not only traditional irrigation but also modern spate irrigation has long been practiced. The first modern spate irrigation practice dates back 40 years using masonry headwork, however, it did not last long as it was breached immediately by the first high flood event. In 1994 gabion headwork was constructed. Then the gabion headwork was improved by implementing a concrete masonry weir in 2005/2006.

Currently the scheme is operating with gabion reinforced headwork structure built over the weir to overcome the level rise in the river and command area as a result of sedimentation. The gabion headwork was built over the masonry in July, 2008. The gabion structure was measured and is 35 meters long, the same size as the weir, 1.5 meters wide and 0.6 m high at the center of the structure and 0.8m high towards the left and right sides of the structure.

The weir is 35 m long and 1.5 m high. The scheme has 1100 m long main canal, in which 450 m is lined by masonry. The masonry lining starts at the off take and extends 450 m long towards the downstream and the remaining main canal and all the secondary canals are earth lined. In the canal system there are three long secondary canals and two short secondary canals. The canal system has been designed with division structure, turnouts in the secondary canals. However, farmers do not use these turnouts; they breach the secondary canals to irrigate their fields. The lay out of the canal system is as presented in figure 16.

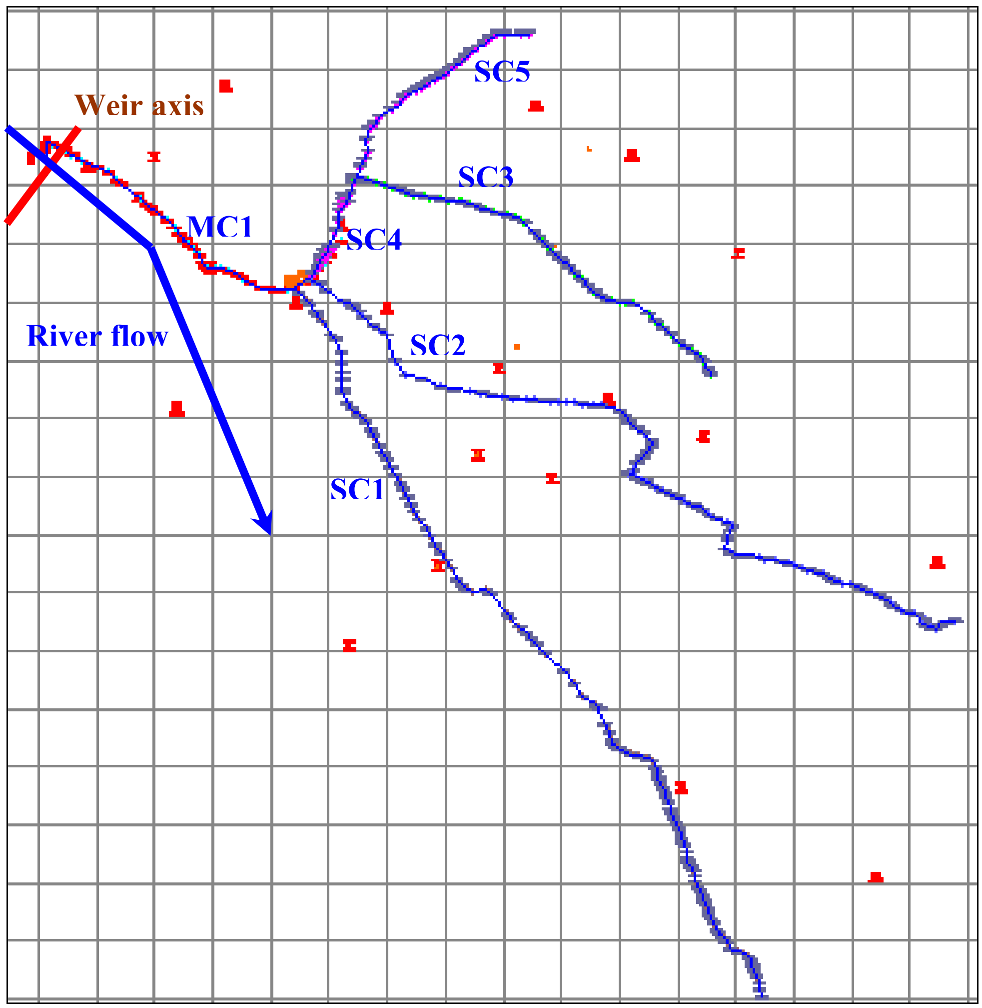

Figure 16. Fokisa irrigation scheme lay out

4.2.2 Climate, Land Use and Topography

Tadese and Abate (2005) indicated that, the rainfall of the project area is bimodal type, where small rainfall occurs during the month of February to April, and large amount of rainfall occurs during the month of July to September. The size of the watershed that contributes runoff to Fokisa spate diversion site is 75 km^2 (Tadese and Abate, 2005). The river is predominantly dry and only wet in the months specified above. The scheme has a design capacity to irrigate 500 ha of land, (Mekonen, 2005). Currently the scheme only irrigates 150 ha of land mainly due to sedimentation of canals and irrigated fields.

The land use study of the scheme catchment indicates that, the watershed area is characterized by undulating plain to very steep sloped terrain (Gebrehiwot, 2005). The land use features have been classified based on slope ranges.

- The dominant topographic feature in the area, which is very steep landscape, has slope greater than 50%, covering 46% of the total catchment area, and having a land use category of free grazing and forestland.
- The second dominant topographic feature is, slope ranging 8% to15% and covers 20 % of the total area. The commonly practiced land use pattern is cultivated land and homesteads.
- The third dominant topographic feature is slope ranging from 15% up to 30%, covering 17% of the total area. This area is used as cultivated land and grazing land.
- The fourth feature, gentle sloping topography with slope ranges of 3% to 8% has been occupied commonly with cultivated land.
- The last feature, slope ranging 30% up to 50% covers 5% of the total area and is covered with free grazing land and some forestlands.

4.2.3 Geology

The geological and geotechnical study of the area indicates that, the dominant geological formations found in the basin are poorly compacted sedimentary basis fill deposits. The floor of the valley is mostly a flat plain and it is filled with quaternary alluvial deposits derived largely from basaltic mountain ranges that are standing high on the shoulders of the valley. Further more, the Raya plain is covered by very thick and very young Holocene sediments. These are gravely silty-sand with cobbles towards the foot of the slopes and the central part is dominated by fine alluvium (clay, silts and fine sands), (Berhane, 2005).

4.3 Data Collection, Field Survey and Analysis

4.3.1 Topographic Surveying of the Area

The river course, the canal route and part of the command area were surveyed using total station and the data was used for schematization of the diversion system. In addition, about 14 river cross sections have also been re-taken so that they would be utilized for calibrating the hydraulics of the system. Further more, more than 50 canal cross sections have also been taken at an average interval of 15 meters. 14 cross sections were utilized to calibrate the ISIS model. Moreover longitudinal profiles of the river and main canal route have also been taken during the topographic surveying.

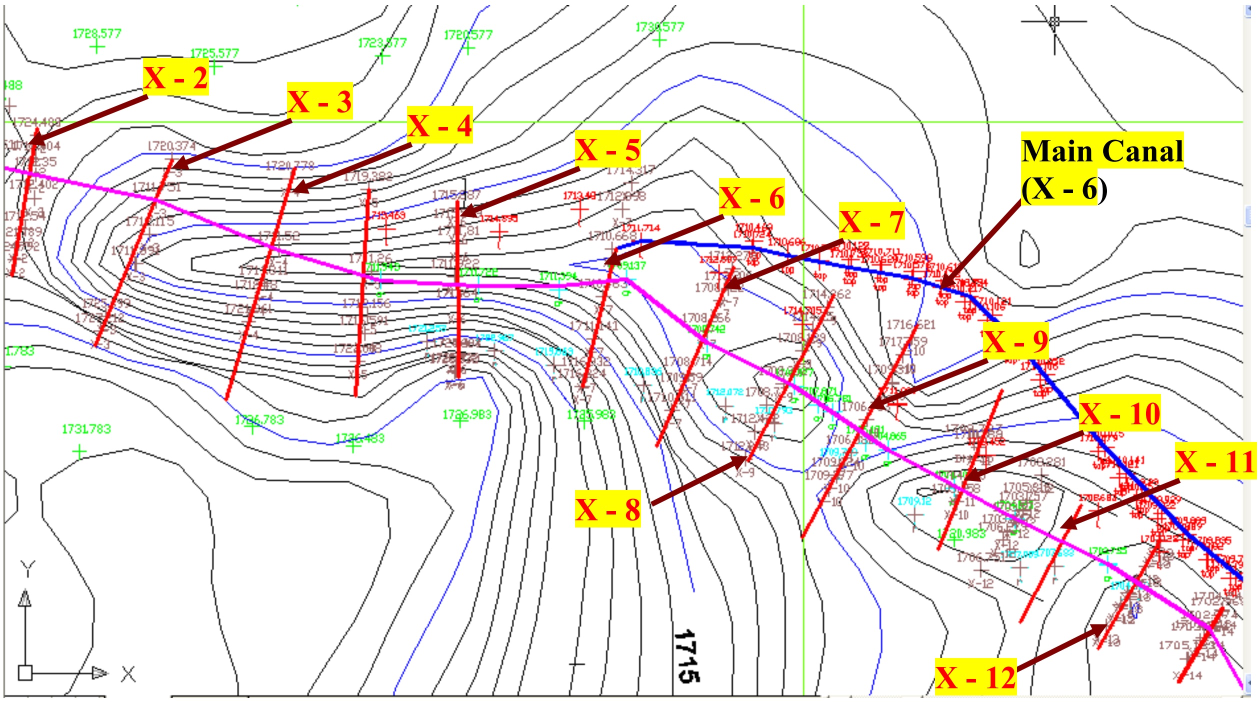

Figure 17. Contours and river cross sections as a result of the topographic survey using Surfer and Auto-Cad

Figure 18. Pictures taken during topographic surveying of the scheme

To improve the accuracy of the ISIS model, river cross sections have been extended. Extending of the cross section is made after plotting the contours using Surfer software and re-plotting that in Auto Cad. Elevations and distances are then measured from the plots to extend the sections. The river is 35 m wide at the weir location. According to the topographic surveying prior to the weir construction, upstream slope is estimated to be 0.95 % and the downstream slope is estimated 2.4 %. These slopes are very steep that they are playing a major role in making the sedimentation of the canals and irrigated fields severe. One of the cross sections is as in figure 19 below. Summary of the river cross sections collected during design period and for the thesis study are presented in ANNEX-G.

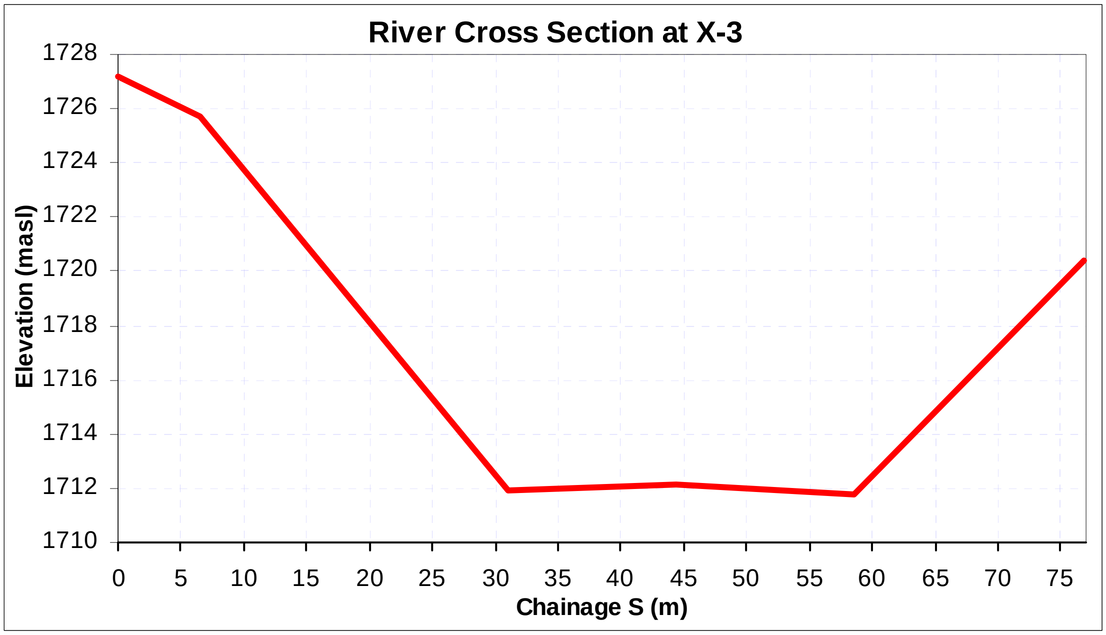

Figure 19. River cross section at X-3 (river cross section 3), 240m upstream of the weir.

Figure 20. Picture taken upstream of the weir showing the river flow and flood plains (looking downstream)

The lined main canal section is 2.34 m wide and 1.5 meters deep. In addition, the earthen main canal is 1.5m deep, having a bottom width of 1.75 m and a side slope of 1:1. The slope of the canals is 0.4%. Refer to figures 21 and 22 below.

Figure 21. Picture of a lined main canal section filled with some sediment

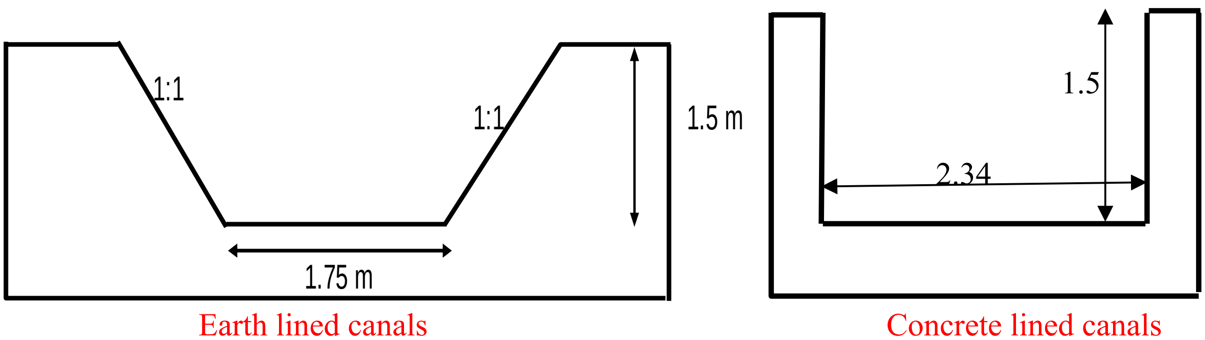

Figure 22. Schematics of the earth Lined and concrete lined main canal sections (not to scale)

Longitudinal profile along the main canal shows that the sediment level rise in the canal is higher at the most upstream of the main canal than is at the downstream. According to the cross section from the topographic survey, the sediment volume is estimated to be 1434 m^3 with 15 % of the sedimentation with in 50 meters downstream of the off take. Estimation has been made by multiplying the sediment cross sectional area with lengths along the canal. If the canal is completely filled with sediments, its capacity is 4750 m^3. The bed level rise of the main canal is as shown in figure 23.

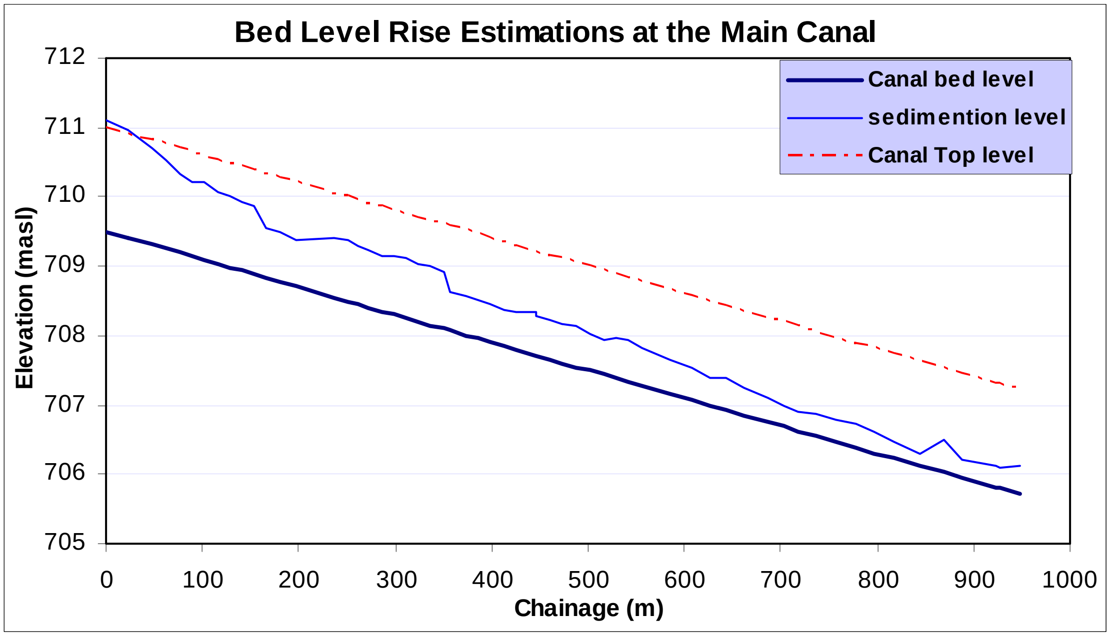

Figure 23. Bed level rise in the main canal compared with canal bed level

Similarly, the level rise in the river is plotted as below in figure 24. As it can be observed from figure 24, the highest deposition is at the weir location and immediate upstream of it, deposition being 1.8 m and 1.3 m above the original bed level (at river cross section 7 and 6 respectively). Furthermore, there is a continuous deposition of 0.4-0.6 m towards upstream up to river cross section 1. Contrary, the area downstream of the weir is eroded to a depth of 0.8 m at cross section 8, and 0.1-0.2 m at river cross sections 12, 13 and 14. However, river cross sections, 9-12 have shown a respective deposition of 1.1, 1.2 and 0.3 m. Summary of the results is tabulated below in table 1.

Table 1. Summary of bed level changes at the river cross section

Chainage (m)	River bed level before Weir construction, Dec 2004 (masl)	River bed level in Nov, 2008 (masl)	Bed level change (m)	Remark
0	1712.51	1713.06	0.55	X1
40	1711.91	1712.35	0.44	X2
86	1711.35	1711.75	0.40	X3
137	1711.06	1711.52	0.46	X4
186	1710.69	1711.26	0.57	X5
225	1710.35	1711.64	1.29	X6
281	1709.50	1711.30	1.80	X7
317	1709.03	1708.27	-0.76	X8
361	1707.02	1708.08	1.06	X9
406	1704.98	1706.18	1.20	X10
444	1704.12	1704.41	0.29	X11
473	1703.77	1703.57	-0.19	X12
529	1703.45	1703.34	-0.11	X13
566	1702.95	1702.81	-0.14	X14

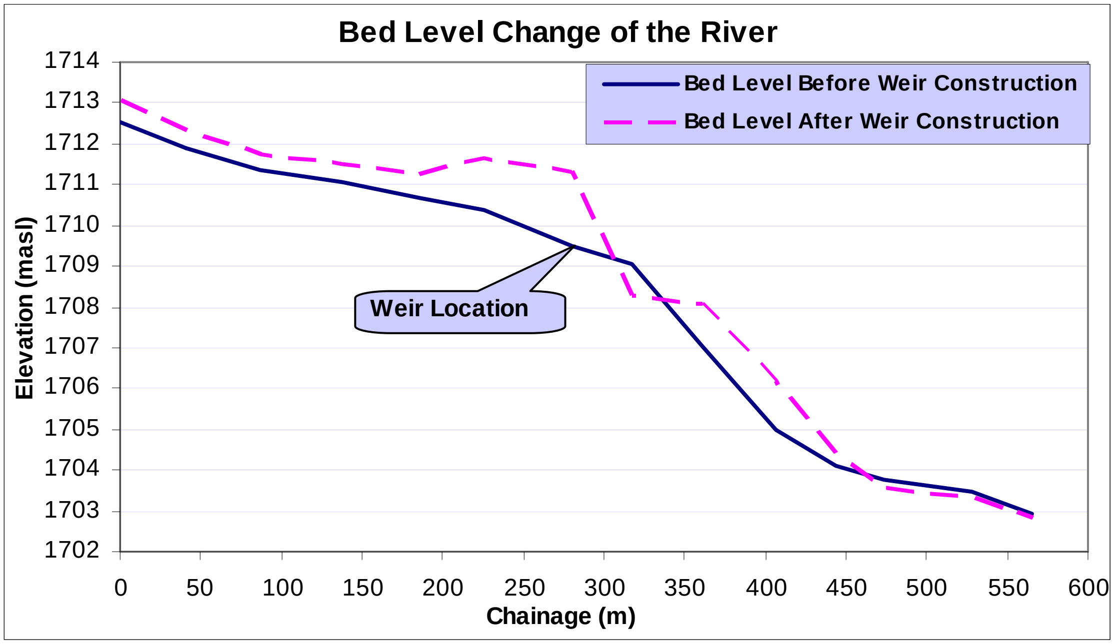

Figure 24. Sediment level rise in the river route of Fokisa scheme

4.3.2 Discharge

Discharge is one of the main inputs which is essential when building an **ISIS** or **DOSSBAS** model. However, there is no hydrological gauging station to measure, or even undertake study using the flow in the river Fokisa scheme. As a solution, therefore, the long term hydrological experience and knowledge of the farmers and rainfall measurements from near by stations were integrated to develop the design hydrographs for testing methods.

An interview format was prepared which was completed in the field through discussion with the farmers. Accordingly, the yearly expected average small, mean and high flows have been marked at one of the river cross sections and over the weir. In addition, farmers indicated the time to peak and time to end for the yearly average small, mean and high floods. Furthermore, the respective frequency for the floods was indicated by the farmers.

From the interview three hydrographs were developed (for details refer to section 4.3). The Manning's equation was used to develop the hydrographs; cross sectional area, and perimeter has been measured from the results of the topographic surveying after plotting the cross sections in Auto Cad, (refer to figure 16). The Manning's equation can be written:

$$Q = \frac{1}{n} AR^{2/3} S_o^{\ 1/2} \qquad \text{[eq. 4.7]}$$

Where, Q = discharge (m^3/s)
n = Manning's roughness coefficient
A = cross sectional area of flow (m^2)
R = hydraulic radius (m), and
S_o = slope of the water surface (m/m)

The respective Manning's roughness coefficient was estimated to be 0.035 (Arcement et al. (nd)). The slope of the river has been measured from the longitudinal profile taken during the topographic surveying and to minimize the effect of the weir on the slope, the longitudinal profile between cross section 2 and cross section 4 has been used, where cross section 2 and cross section 4 are located immediate upstream and downstream of cross section 3 respectively. For details on cross section 3 refer to figure 19 and results are reported in Table 2.

Table 2. Summary of discharges developed from Manning's equation

Elevation (masl)	Area (m^2)	Perimeter (m)	S_o (%)	Manning's n	Hydraulic Radius, R (m/m)	Discharge (m^3/s)	Remark
1711.9	0.0	0.0	0.3	0.035	0	0.0	RBL
1712	1.6	17.1	0.3	0.035	0.1	0.5	
1712.9	26.6	32.2	0.3	0.035	0.8	35.5	SF
1713	31.3	32.8	0.3	0.035	1.0	45.8	
1713.8	56.5	36.1	0.3	0.035	1.6	115.3	MF
1714	65.5	37.2	0.3	0.035	1.8	144.3	
1715	103.5	41.6	0.3	0.035	2.5	287.5	HF
1716	145.4	45.9	0.3	0.035	3.2	474.2	
1717	191.2	50.3	0.3	0.035	3.8	704.4	
1718	240.9	54.7	0.3	0.035	4.4	979.4	

In addition, discharges were estimated using the broad crested weir formula. The broad crested weir equation can be written as

$$Q = CLH^{3/2} \qquad \text{[eq. 4.8]}$$

Where, Q = discharge (m^3/s)
C = weir coefficient
L = weir crest length (m)
H = head above the weir (m)

The weir coefficient, C =1.7 is used for the computations adopted from the design document (Tadese and Abate, 2005). Results are reported in Table 3.

Table 3. Summary of discharges developed using weir equation

Discharge estimated using weir equation (m^3/s)	Time to Peak (hrs)	Time to end (hrs)	Frequency of floods per year			Remark
			dry year	medium year	wet year	
24.2	2	8	2-3	7	15-20	**SF**
62.14	2	8	1-3	7-8	10	**MF**
138	4	24	1	3-4	7-8	**HF**

Remarks: **SF** = Small flood that farmers expect every year,
MF = Medium flood that farmers expect every year, and
HF = Large flood that farmers expect every year

The 50 year return period design flood, which was utilized for the design of the scheme, was collected from the design report of the scheme. According (Tadese and Abate, 2005), the 50 year return period design flood is 220 m^3/s and the time to peak is 6.7 hr. These results were derived from soil and water conservation curve number method, which gives the composite hydrograph, developed by U.S. corps of engineers. According to this method, slope, land use, length of the main drainage and curve numbers are the main inputs. The results from the SCS method are as reported below in figure 25. However, the results of this method are not applicable to spate schemes, as spate floods are characterized by a very rapid rising limb they should not be represented using classic triangular hydrograph models such as those of the US Soil Conservation Service (Walingford, 2005).

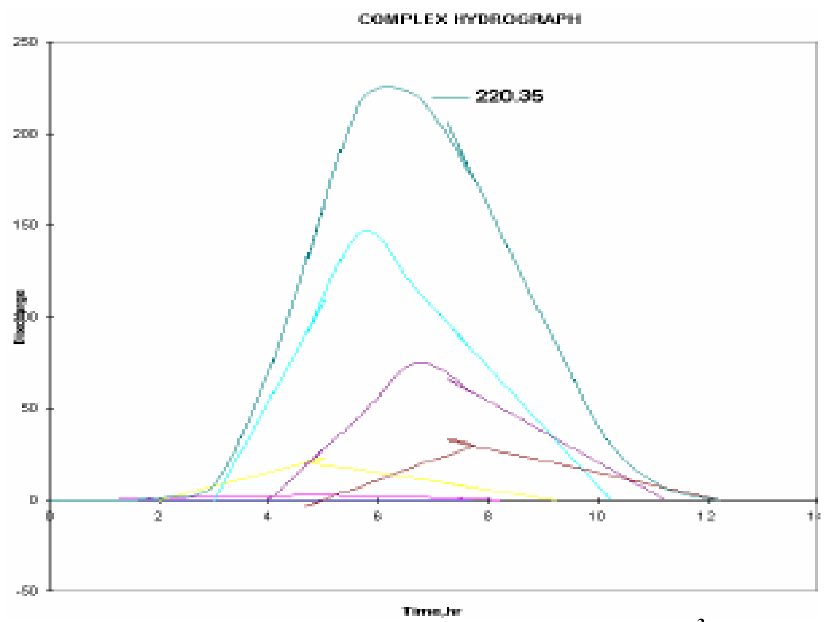

Figure 25. 50 yr return period design flood (discharge is in m^3/s and time is in hours). (Source: headwork design document)

As there is no measured flow data for the scheme, there is no easy way to prove which of the flow data collected is most reliable, however, an indirect check has been made using sensitivity analysis to the weir coefficient and Manning's n as presented in figures 26 and 27. Accordingly, the sensitivity analysis results indicate that, discharges computed are less sensitive to the weir coefficient than to the Manning's n. Therefore, the discharges computed using the weir formula are less likely to have large errors as compared to that computed by the Manning's n. Furthermore, the upstream river cross sections are subjected to relatively higher changes than the weir, as a result of erosion and deposition. As a result, the discharges computed by the weir formula are used for further analysis. Summary of the discharge estimations is presented in Table 4 below.

Table 4. Summary of the discharge estimations

Discharge estimated using SCS (m^3/s)	Discharge estimated using Manning's equation (m^3/s)	Discharge estimated using Weir equation (m^3/s)	Time (hr)		Frequency of floods			Remark
			to Peak	to end	dry year	medium year	wet year	
-	35.5	24.2	2	8	2-3	7	15-20	**SF**
-	115.3	62.1	2	8	1-3	7-8	10	**MF**
-	415.8	138.0	4	24	1	3-4	7-8	**HF**
220	-	-	-	-	-	-	-	**HF (50 year RP)**

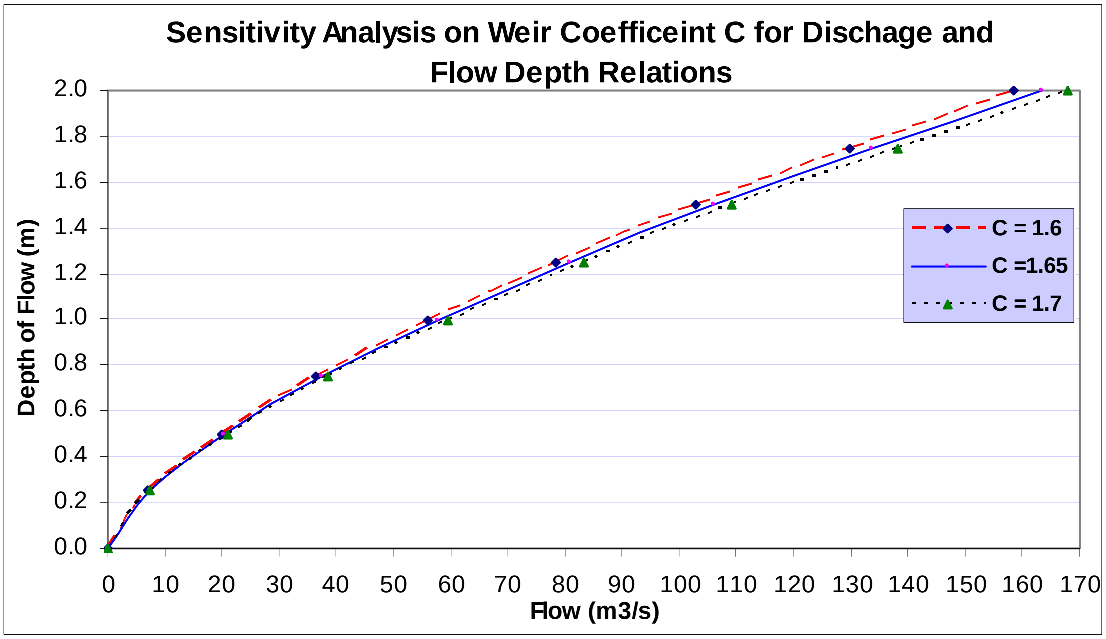

Figure 26. Sensitivity check on weir coefficient, C

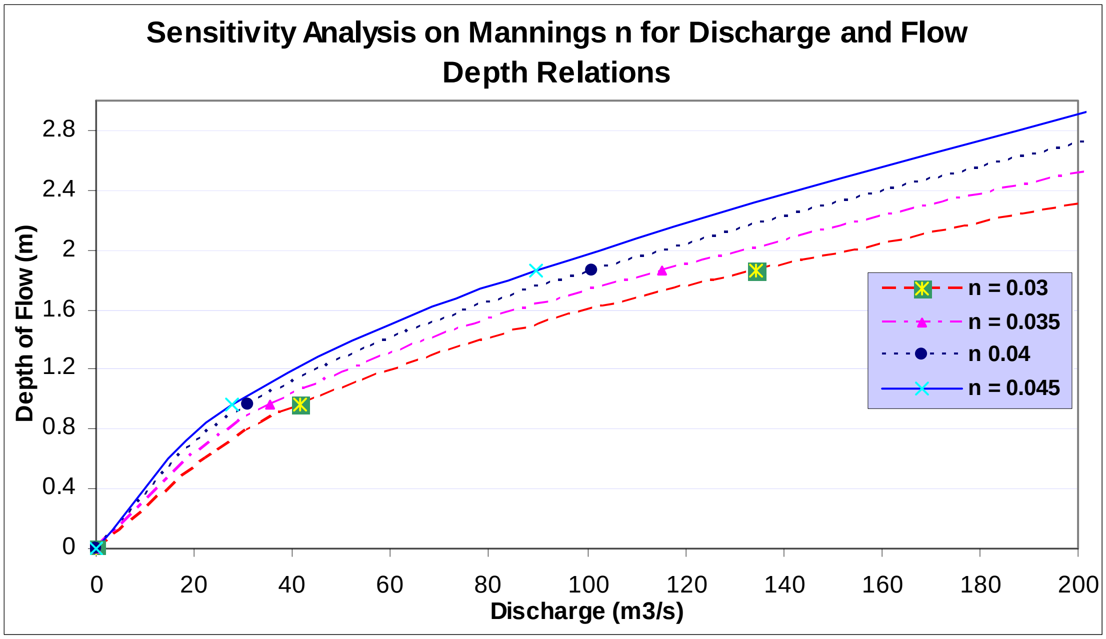

Figure 27. Sensitivity check on Manning's n.

4.3.3 Sediment Data

Sediment data collected both during the field survey and from literature, include particle size distribution of the bed material, particle size distribution of the suspended sediments, sand and silt concentrations and sediment fall velocity. All will be discussed in details below.

Particle Size Distribution of Bed Materials

Sediment data is also an important input to both the ISIS and DOSBASS models. The only sediment data available for the river prior to the study was the particle size distribution at the weir location, available from the geological report. However, during the field work 18 sediment samples were taken from the following locations as illustrated in figure 28**.** Sediment samples collected from the canal route and command area were grab samples collected by shovel. Sediment samples from the river RB6 and RB8 were armoured layers and samples were taken after removing the top layer boulders. In addition two samples, from the irrigated field and secondary canal 2, were taken even without removing any of the top layers.

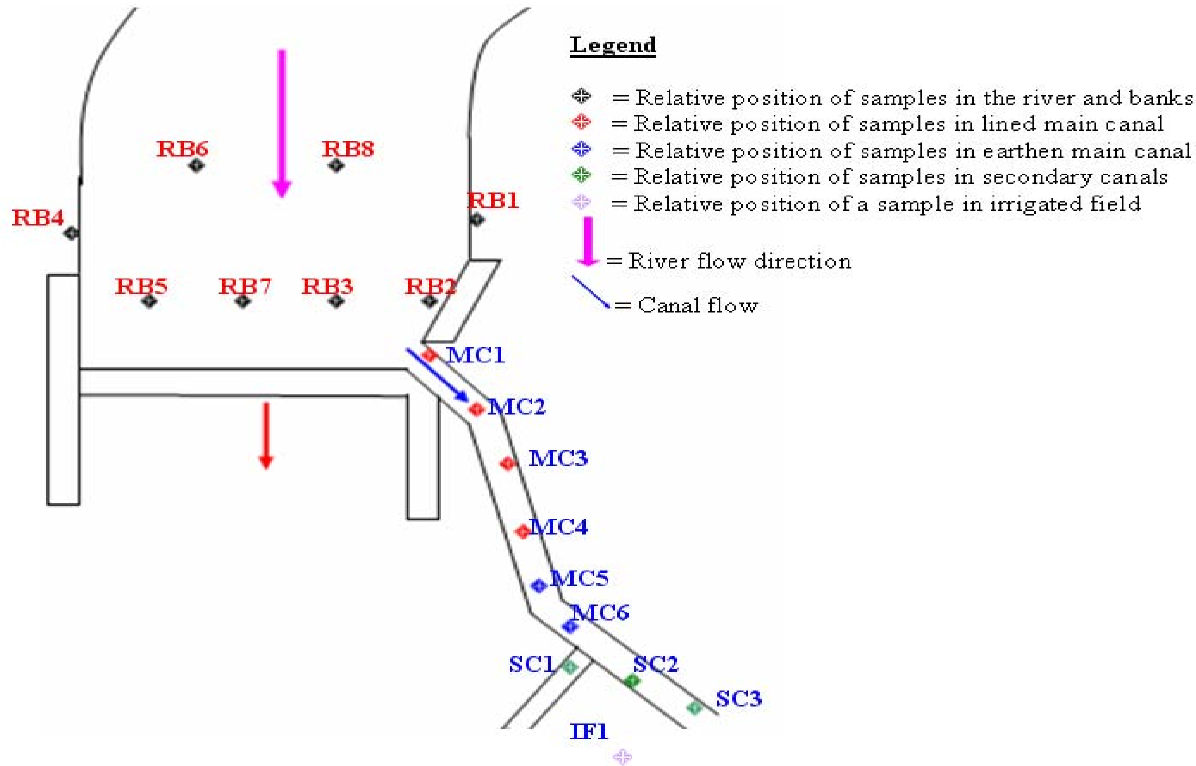

Figure 28. Rough sketch indicating the relative location of sediment samples

Remarks **RB1, 2....8 = river bed sample number 1, 2...8**
MC1, 2...6 = Main canal bed samples 1, 2...6.
SC1, 2 and 3 = secondary canal samples 1, 2 and 3.
IF1 = Irrigated field bed sample 1.

- 6 bed samples from the river, up to a depth of 0.6 m (RB2,3,5,6,7and 8)
- 1 sample each from the right and left banks, up to a depth of 0.6 m (RB1 and 4)
- 4 samples from the main lined canal, up to a depth of 0.6 m (MC 1,2 3, and 4)
- 2 samples from the main earthen canal, up to a depth of 0.6 m (MC 5 and 6)
- 3 samples from the secondary canals, up to a depth of 0.6 m (SC1,2 and 3)
- 1 sample from the command area, up to a depth of 0.6 m (IF1)

Majority of the bed samples taken were composed of fine sediments and therefore, were taken in plastic containers having a carrying capacity of 2 kg of sediment samples. Then, all samples were taken to the Tigray Water Resources, Mines and Energy Bureau soil laboratory and they were oven dried before sieve analysis was held. Respective sieve analysis was done for each sample using the United States sieve opening standard sieves. Accordingly, results for D_{10}, D_{30}, D_{60} and USCS classification of the particles is given by the software developed by Spears Engineering and technical Services (PS, 1996-2005). Furthermore, the test results are plotted in a semi-log paper to refer for the likes of D_{50} and D_{90}. The test results are summarized as below in figures 29 and 30. The relative location of the areas of sampling is as presented in figure 28.

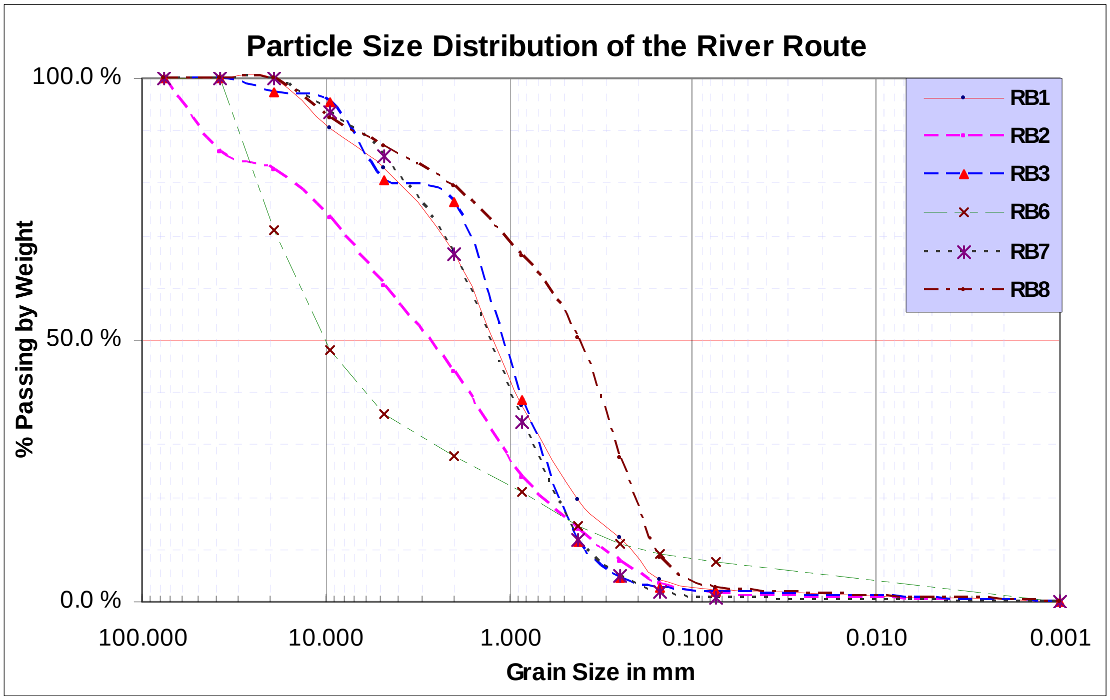

Figure 29. Summary of particle size distribution in the river bed

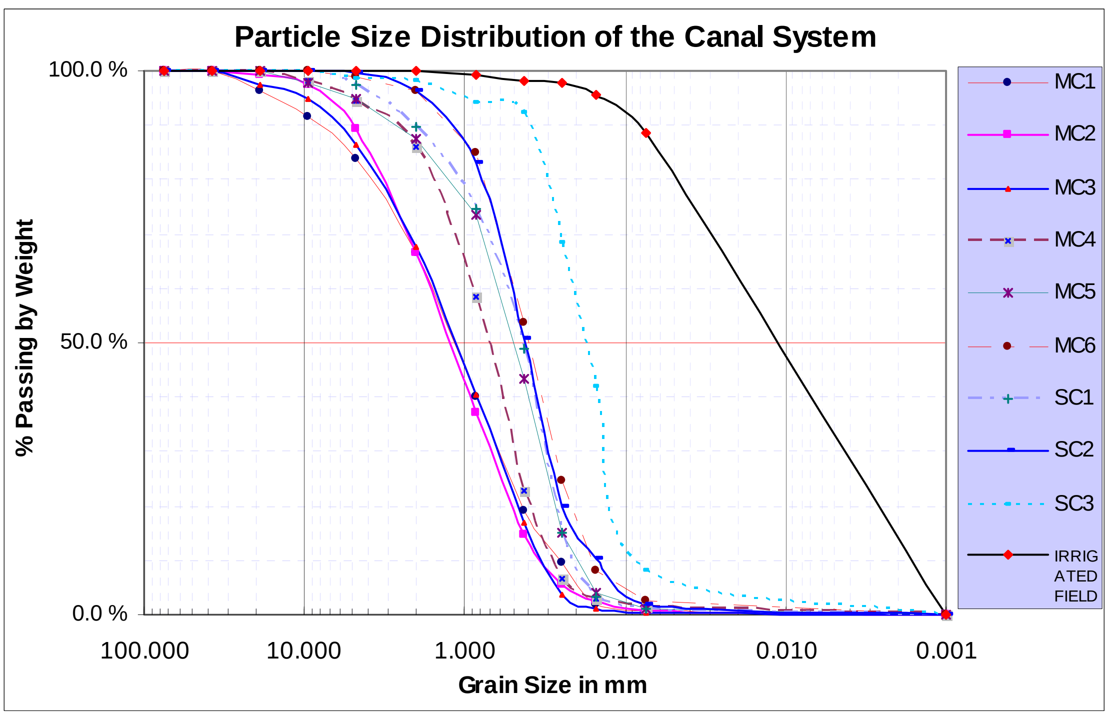

Figure 30. Summary of particle size distribution in the canals and irrigated field

The river and canal bed grading size results have been plotted on the same scale as shown above figure 29 and figure 30. From Figure 30, it can be observed that, the size of sediment particle decreases as the canal length increases from the off take and further decreases as it reaches the irrigated field. Therefore, long main canal routes are helpful to trap coarser sediments. Furthermore, plotting on the same scale enables to select representative sediment particle size distributions. As a result ten particle size distribution results, D_5, D_{15}...D_{95} have been estimated from the plots, as per the requirements of the two models ISIS sediments and DOSSBAS. Results are summarized in ANNEX E.

Figure 31. Pictures showing sediment data collection by the method of grab sampling taken by shovel

Particle Size Distribution of Suspended Sediments

Particle size distribution of the suspended sediments has also been measured at the TU Delft/ Deltares hydraulic laboratory. This is mainly because, the data is very important to determine the falling velocity of the suspended particles held in suspension, which will be utilized in both the models. Data collection was held at different discharges. As a result the D_{50} of the particles is 9µm, 8.5µm, 14µm and 9.5µm, for discharges 12m^3/s, 38m^3/s, 47m^3/s and 58 m^3/s respectively. Furthermore, respective measurements show that 90%, 85%, 80% and 85% of the suspended sediments have a mean particle size diameter less than 63 µm. Therefore, these suspended sediments constitute both wash load and suspended bed material load. In engineering practices, wash load is often defined as sediment particles smaller than 63 µm, as sediments finer than this are not usually found in appreciable quantities in canal or river beds (Lawrence et al. 2001).

It can be seen that, the sediment particles collected by the bottle sampler were very fine as compared to that of the parent bed material. It was not possible to use these results. This is because, DOSSBAS model requires fall velocity fractions by weight from a cumulative settling velocity at 10%, 20% ...100 % of the weight. (V_0, V_{10}....V_{100}). However, it was possible to use the fall velocities from the DOSSBAS model as these results are obtained from the measurements held in Gezira scheme, of the Nile basin, Sudan. Results are shown in figure 32.

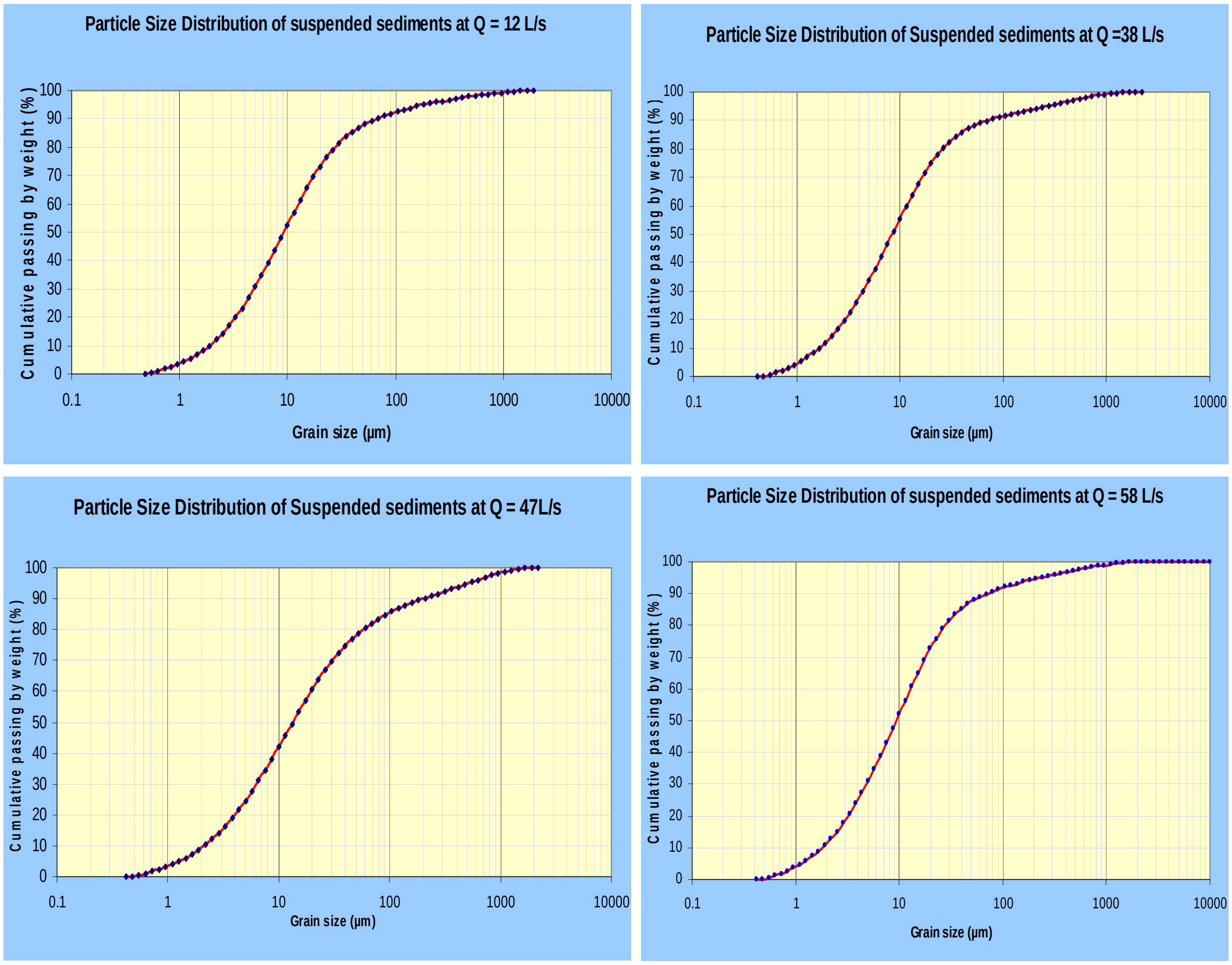

Figure 32. Particle size distribution of the suspended sediments.

Sediment Concentration Measurements

Sediment concentration samples have also been taken during the field work. Bottle sampling technique was used to collect the samples. Four bottles of one liter each have been taken at discharges of 12l/s, 38l/s, 47 l/s and 58 l/s. The samples were taken near to the bed, as the depths of the flows were very small. Then these samples have been taken to IHE laboratory to estimate the PPm of sediments and total dissolved solids. Accordingly, two 50ml samples from each bottle, have been weighed and dried in oven, and then also weighed to know the existing proportion of sediments. In addition, the samples have been put in to furnace for two hours to determine the amount of organic sediment concentrations. In addition, the samples were also analyzed for particle size distribution. Table 5 shows the summary of the results from the analysis.

Table 5. Summary of sediment and organic sediment concentrations

Sample. no	I	II	VII	VIII	III	IV	V	VI
Can weight	2.103	2.0973	2.0964	2.096	2.0996	2.0859	2.0841	2.0991
Oven Dry weight	3.13	3.0066	3.1107	3.168	3.3454	3.2508	3.293	3.3361
Furnace Dry weight	3.036	2.9291	3.0277	3.064	3.2289	3.1442	3.1995	3.1925
Volumes (ml)	50		50		50		50	
Discharges during sampling	**12 l/s**		**38 l/s**		**47l/s**		**58 l/s**	
PPm (organic concentrations)	**1716**		**1872**		**2228**		**2368**	
PPm (sediment concentrations)	**19131**		**20602**		**23751**		**24092**	

The result of the bottle sampling is quite very large than was expected. An objection to bottle sampling techniques is Bolton (1983) that the samples are too small to be analyzed to yield size grading curves of the suspended material cited by (Lawrence, 1987). Therefore, sediment concentration estimations for both sand and silt are estimated using the Engelund and Hansen and Van Rijn predictors from DORC respectively.

Sand Concentration Estimation

Sand concentration or bed material load is one of the inputs to both the DOSSBAS and ISIS hydrodynamic models. There is no measured bed load data in the scheme; therefore it is necessary to make estimation using DORC model. With the available stage discharge relationship on the weir, particle size distribution and longitudinal slope, river section properties such as depth and velocity are computed using the alluvial friction predictors (formulae). The main purpose of these computations is to estimate shear velocity of the flow which is an input to estimate the concentrations. This helps to identify which of the canal parameters are used as an input to estimate the bed material load.

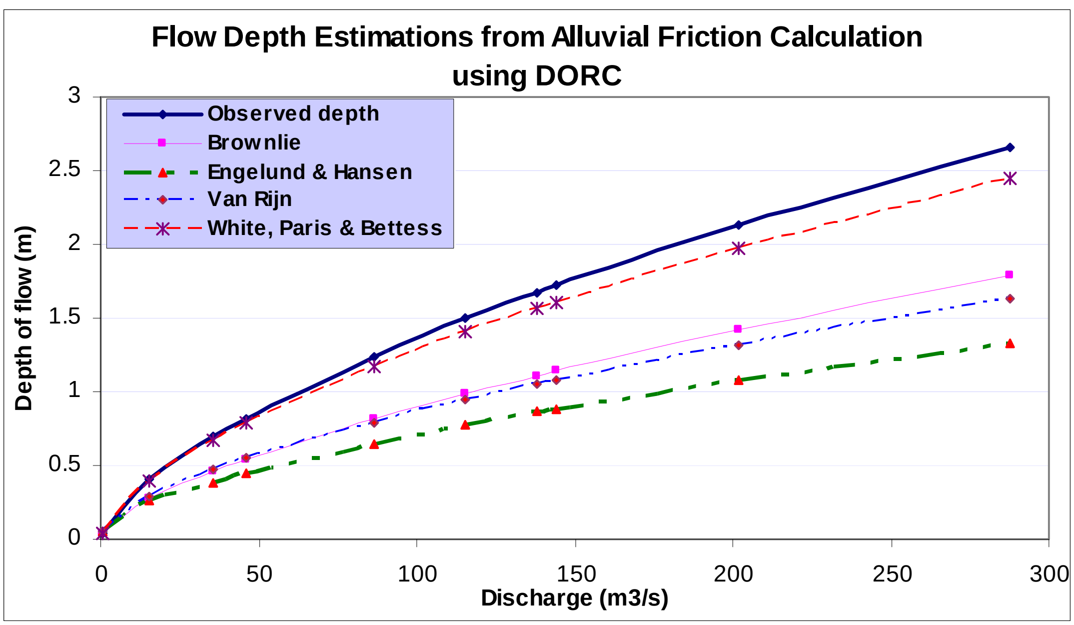

Figure 33. Comparison of flow depth estimations from Alluvial friction predictor using DORC model

As a result of the computations made in DORC, White et. al. formula gives relatively close values to the observed depths as indicated in the figure 33 above. The velocities were also computed to check the Froude number of the flows. The results reveal that the flows are all sub critical with values ranging from 0.02 to 0.44. Therefore, river parameters such as mean velocity, shear velocity and Manning's n are estimated from the White et al. (1980) predictor method as an input to the Engelund and Hansen bed material load predictor method. The Engelund and Hansen bed material load predictor is ranked first in the performance evaluation of some transport predictors "Sediment transport in Wadi systems "(Lawrence, 2008).

For each depth and discharge relations, estimations of the concentration of the sand load using the Engelund and Hansen sediment transport predictor of DORC has been applied to 10 representative sediment sizes. The concentrations are weighed by the proportion of the bed material each represents. This enabled to derive the concentration for 1 mm sediment, and then derive the concentrations (C, ppm) for each size fraction using the relationship C proportional to 1/D (In the EH relationship). Plotting C and a function of discharge, Q yields a sediment rating relationship that can fit with a simple power relationship ($C = a\,Q^n$), that is used to derive in incoming sediment concentration. Hence, the computations indicate that, the relation between sediment concentrations and discharge are $\mathbf{C = 2113\ Q^{0.62}}$**.** Therefore this relation is used to generate the concentrations at the required discharges.

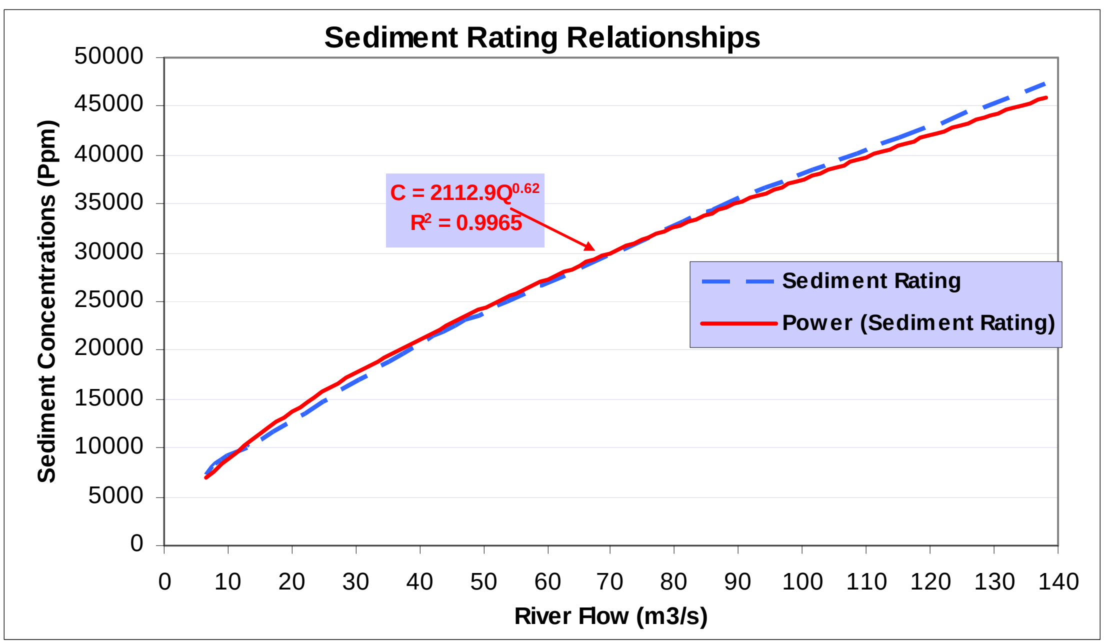

Figure 34. Sediment rating estimation for the Fokisa spate scheme

Bed material load estimate of the canal route is also important data to consider, because the out put bed load concentration from DOSSBAS model should be equal to the estimated sediment transport capacity of the canal. Therefore the only observed value to compare and select the best alluvial friction predictor is depth of flow in the main canal.

Table 6. Depth comparisons from alluvial friction results for the main canal from the four predictors

Depth of Flow (m) Estimations from Alluvial friction predictors						
Q, Discharge (m^3/s)	Observed/ Generated depth (m)	Brownlie	Engelund & Hansen	Van Rijn	White et al	Remark
6.8	1.2	-	0.813	1.276	1.425	Earthen canal
4.35	0.95	-	0.64	0.937	1.084	Earthen canal

From the table 6, we can deduce that Van Rijn alluvial friction predictor method gives depth estimate close to the observed values. Therefore, results from this predictor are used to determine canal parameters such as average velocity, shear velocity and Manning's n. To estimate the sand transport in the canals, the Ackers and White transport formula is used. Based on the overall performance of the above methods, according to the evaluation criteria, the Ackers and White, and Brownlie methods appear to be the methods to predict sediment transport in irrigation canals, (Depeweg et. al. 2005). To select the best method from the above two, the results of the two methods are compared with the estimated river bed load material concentrations, as presented in table 7 below. As a result, some of the canal bed material load concentration results from Brownlie predictor are even larger than the estimated river bed material load indicating that there is no sediment deposition in the canal, which is not the case. Therefore, results from the Ackers and White predictor are selected for further analysis.

Table 7. Comparison of bed load estimations of the canal from the Ackers and White, Brownlie and estimated river bed load.

Discharge (m^3/s)	Concentration (PPm)		
	Ackers and White	Brownlie	Estimated River bed material load
1.5	2350	3165	2720
4.35	2520	5420	5260
6.8	2540	6560	6810
11.4	2555	8300	9560

Silt Concentration Estimation

Silt or fine sediment concentration is also an input to the DOSSBAS model. The only measured suspended sediment load concentration data available for the scheme is the data collected during the data collection period, by bottle sampling. These measurements were held at smaller discharges which do not exceed 60l/s. Therefore, to derive any estimate of concentration from the existing measured data does not give good results. Bolton (1983) states that, samples are often too small to be analyzed to yield size grading curves of the suspended material, cited by (Lawrence, 1987). Furthermore, Silts and clays are diverted via canals to the fields, "supply controlled" fine sediment concentrations do not correlate well with wadi discharge, (Walingford, 2005). Therefore, fine sediment concentration data from the wadi Laba, spate scheme from community spate irrigation (Wallingford, 2005) have been taken for further analysis as there is a topographic similarities between Eritrean and Tigray areas.

Sediment Fall Velocity

Similar to the sand concentration, there is no measured data of the fall velocity. There are ways to compute fall velocity of suspended sediment; however, these methods do not provide options to estimate the cumulative velocity curve profile of the fall velocity so that velocity fractions by weight. Furthermore, estimated results from these small suspended samples may not give good results. Therefore fine sediment settling velocities given in the DOSSBAS demonstration model are adopted for further analysis. These fall velocity measurements are for sediments from Ethiopian catchment, after being transported by the Nile to the Gezira scheme in Sudan. Results are as shown in figure 35.

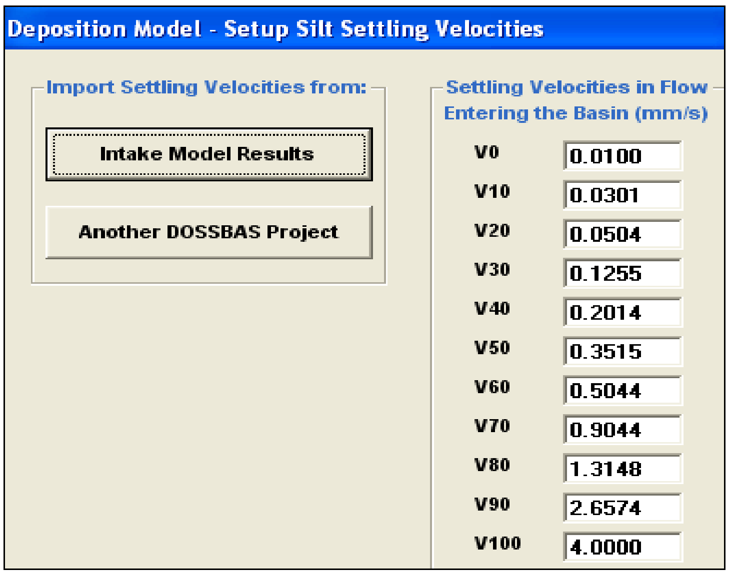

Figure 35. Fall velocity results from DOSSBAS

4.3 Hydrograph Development

There are many methods to develop hydrographs for unguaged catchment. Some of them are SCS unit hydrograph method, area slope method and weir formula using flood marks on the weir. SCS method is not applicable, for spate floods (refer to section 4.2.2). Therefore, flood marks both on the weir and in the river banks were used to develop hydrographs. The hydrographs computed using the weir formula are selected for further analysis as stated in section 4.2.2.

The hydrographs are developed as an input for the ISIS model, as the DOSSBAS model is a steady state model it does not require hydrographs, only discharge is an input. To develop hydrographs data collected from the interview of the farmers and rainfall data from the nearest meteorological station has been integrated. Based on the data collected from the interview, yearly expected small, medium and large floods for dry, average and wet year have been marked both in the river bank and on the weir wing wall. Then an average year with yearly expected small, medium and large floods have been selected to calibrate the ISIS hydraulic model. As a result of the computations using the weir formula, the following flood hydrographs were developed as in figure 36 and 37 below.

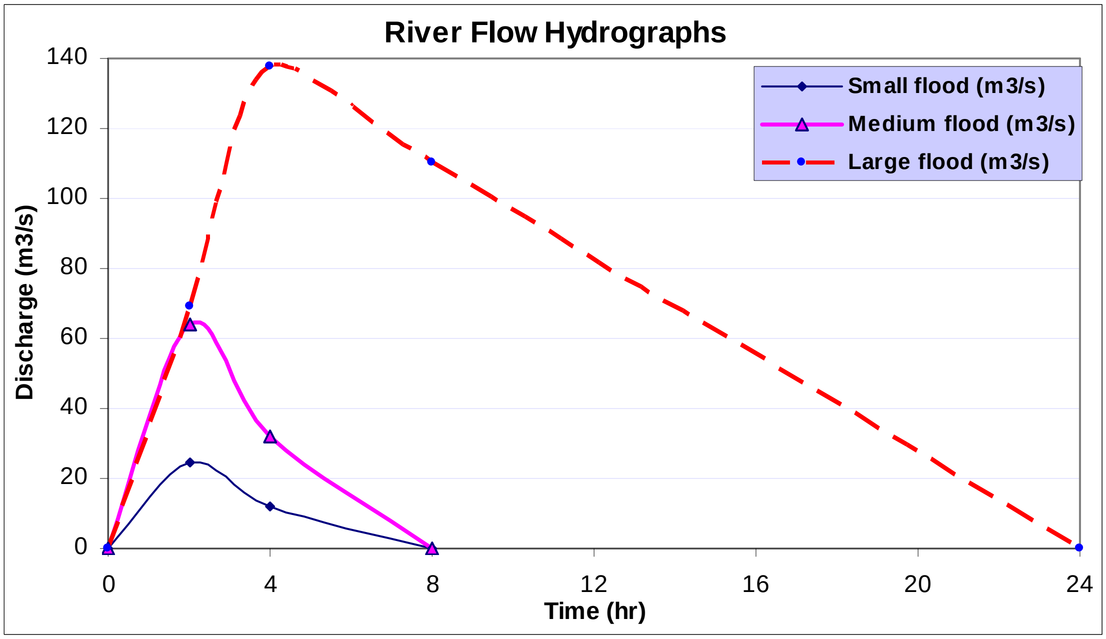

Figure 36. Average small, medium and large flood river flow hydrographs

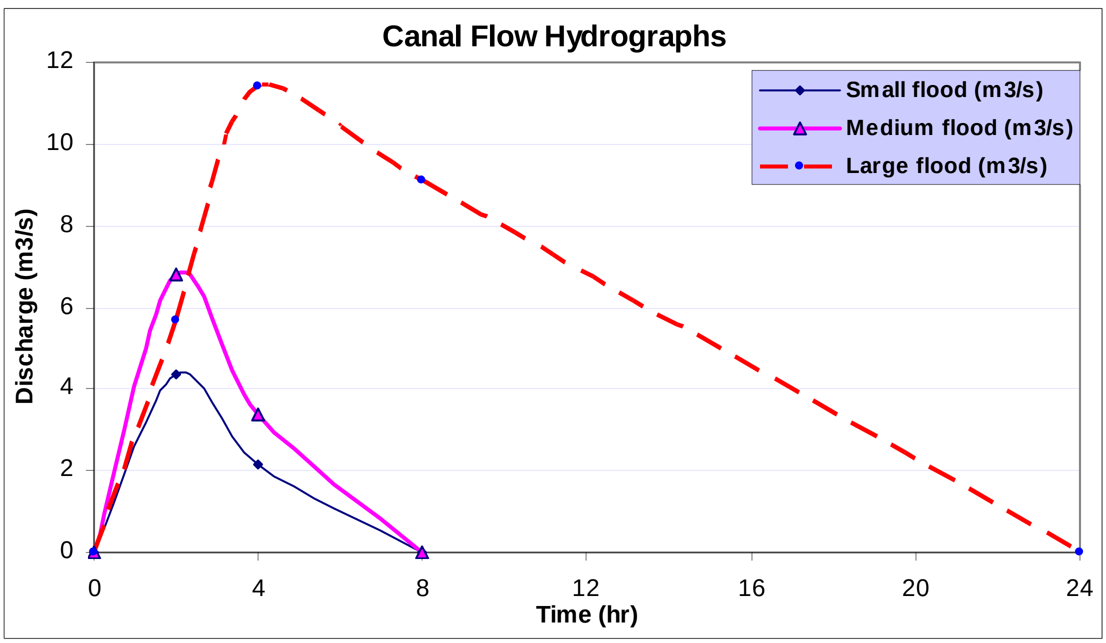

Figure 37. Average small, medium and large flood canal flow hydrographs

These hydrographs are not similar to that of Wadi Laba, Eritrea and Wadi. Rima, Yemen. Flow recession variations might have an effect on the performance of the settling basin. Therefore sensitivity check will be carried out using flash flood hydrograph. To develop flash flood hydrograph a shape factor with some adjustment was taken from the Wadi Laba and Wadi Yemen hydrographs, to fit the hydrograph, and the following hydrograph is generated as in the figure 38 below.

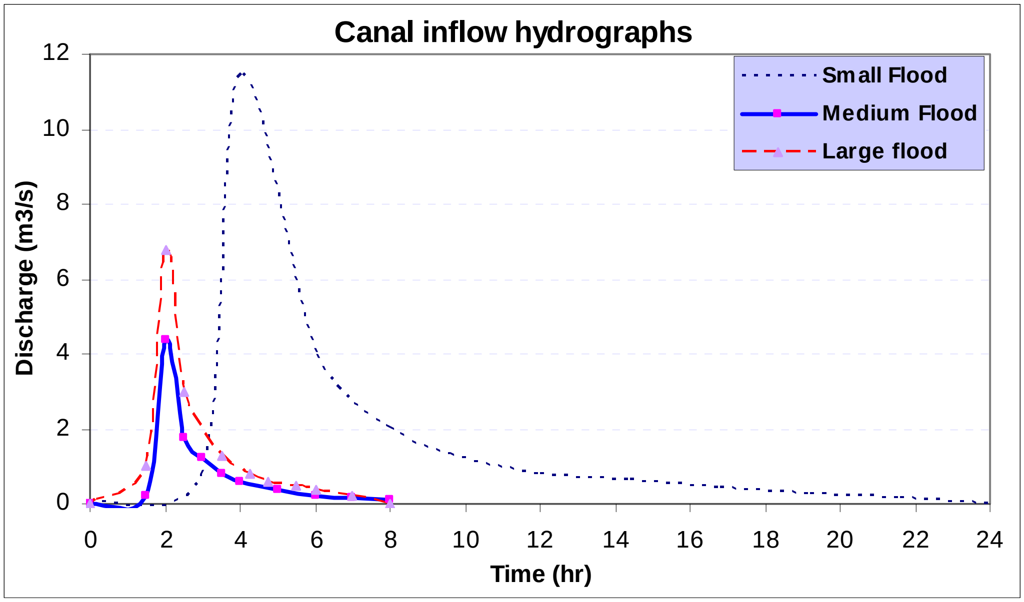

Figure 38. Modified river inflow hydrograph as a result of shape factors taken from Wadi Laba and Wadi Zibad river flow hydrographs.

To model a sand trap settling basin which will be dredged twice every year, a yearly series hydrograph is important. To develop a yearly series hydrograph for an average year, the frequency and magnitude of floods from the farmer's interview and the 25 years of rainfall data were utilized. From the rainfall data, daily rainfall records above zero have been averaged to get the monthly average rainfall, from the recorded rainfall data was plotted as in figure 39. This is because it gives an indication to the series of flows. Furthermore, the area is characterized by bimodal nature of rainfall, the first season being between February and April and the second and main season being between July and September. Spreadsheet data of the hydrographs are presented in ANNEX-F.

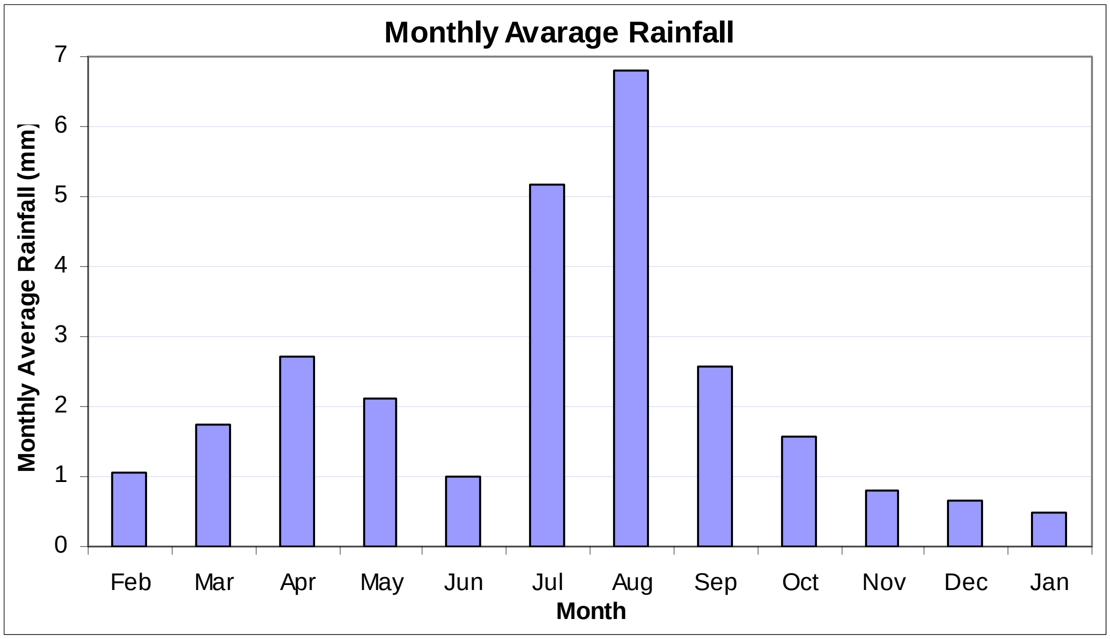

Figure 39. Monthly average rainfall of Maychew rainfall station

From figure 39, it can be seen that higher flows occur in the months between July and August, medium flows occur in both between March and May, and between September and October. Therefore, this information was vital to make the distribution of the floods with in a year. As a result, the flows have been distributed to give the following river yearly series hydrograph. From calibrated ISIS hydraulic model, and using the yearly series river inflow hydrograph, yearly series canal inflow is generated, as in the figure 40 below.

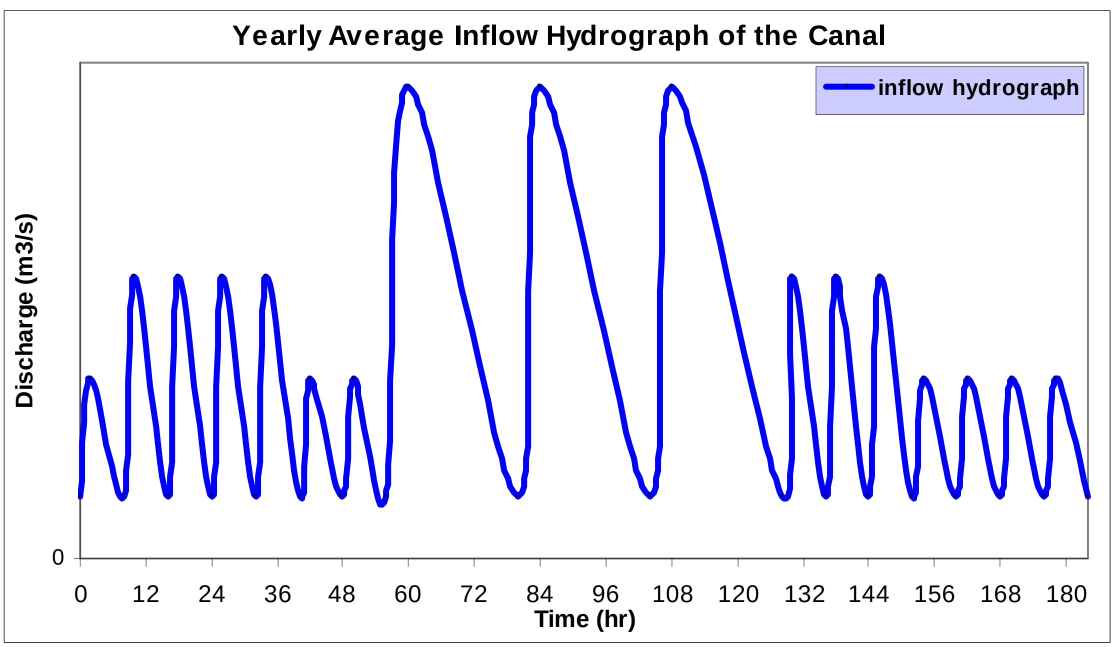

Figure 40. Yearly long series canal inflow hydrograph

For a scheme with no measured flow data, generating a yearly long series data as in figure 40 above is vital. However, it is obvious that the above hydrograph is one of the many possible hydrographs and the flow order might have a different sequence in a given year. Therefore, it is difficult to test all the possible hydrographs; some selected hydrographs will be tested as a sensitivity check. However, two hydrographs are tested to check how the bed level changes respond to the sequence of hydrographs.

- the worst scenario, such as to have all the large flows at the start, all medium flow at the middle and all small flows at the end, and
- The most representative situation as presented in figure 40 above.

4.4 Model Setup and Schematization

For this study, two models DOSSBAS steady and ISIS sediments were built. As indicated in section 4.1, DORC model is used to generate input data for both models, and will not be discussed here.

4.4.1 DOSSBAS

DOSSBAS model is 1D steady state model which predicts trapping performances and is very useful to develop a basin design which the sediment concentrations leaving the basin are kept very low. The determination of basin dimensions is a trial and error process, with the results of the trapped silt and sand efficiencies be displayed in percentages. The inputs to this model are geometry of the basin (height, width and length), discharge, upstream and downstream bed levels, downstream water level, sand and silt concentrations, particle size distributions of the bed material / materials in transport, fall velocities and upstream channel width.

To create the DOSSBAS steady model for the Fokisa spate scheme,

- Mean discharge at a medium year (6.8 m^3/s)
- Sand concentration (computed using DORC model)
- Silt concentration (adopted from Wadi Laba scheme)
- Width of upstream canal
- Downstream water level
- Sediment sizes and
- Fall velocities (adopted from Gezira scheme) were used.

Then from the irregular geometry option, it was computed to find the best combinations of sand trap length, width, height and, slopes, that would have a maximum fine sediment trapping efficiency and minimum sand trapping efficiency.

4.4.2 ISIS Sediments

ISIS 1D hydrodynamic model has been built both for the river and the canal flow. To create the model

- 30 river cross sections and 14 canal cross sections

- Slope of the river measured from the topographic survey
- Distance between each cross section
- Hydraulic parameters such as Manning's n and weir coefficient have been utilized.

Further more, the intake is built as a spill with a weir. 16 of the cross sections have been generated from the partly from the longitudinal profiles taken along the river and partly form the contours developed as a result of the topographic surveying. Flood plains of the 14 measured cross sections have also been further extended from the contour maps.

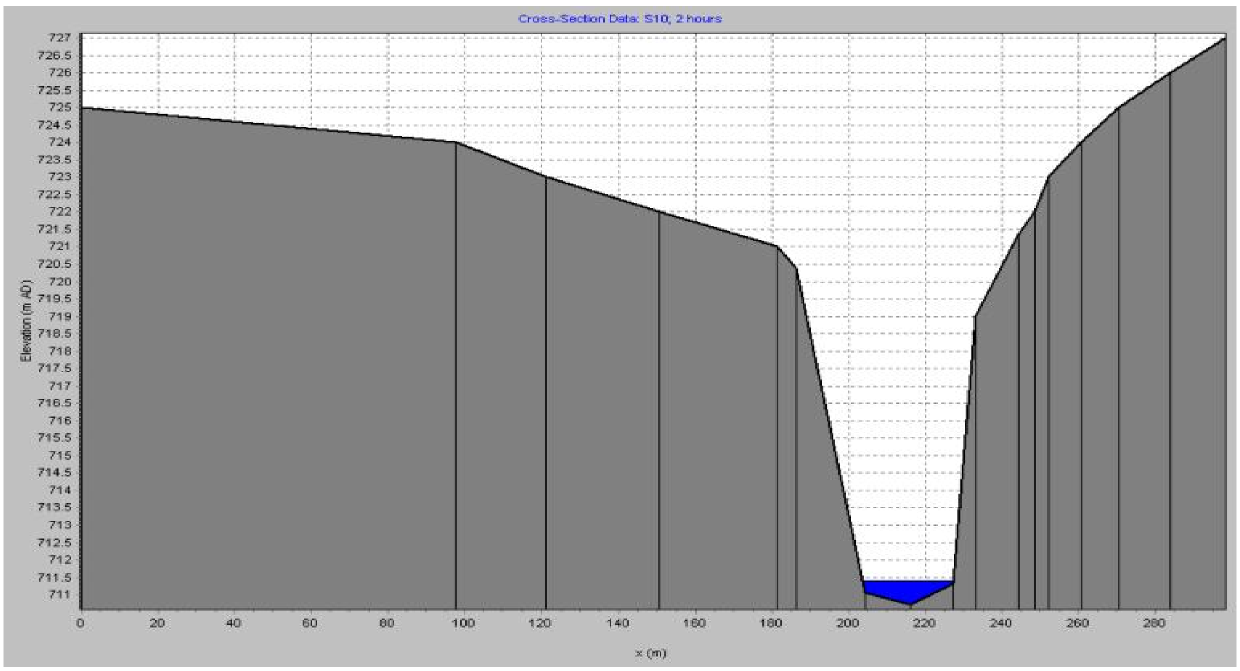

Figure 41. River cross section upstream of the weir

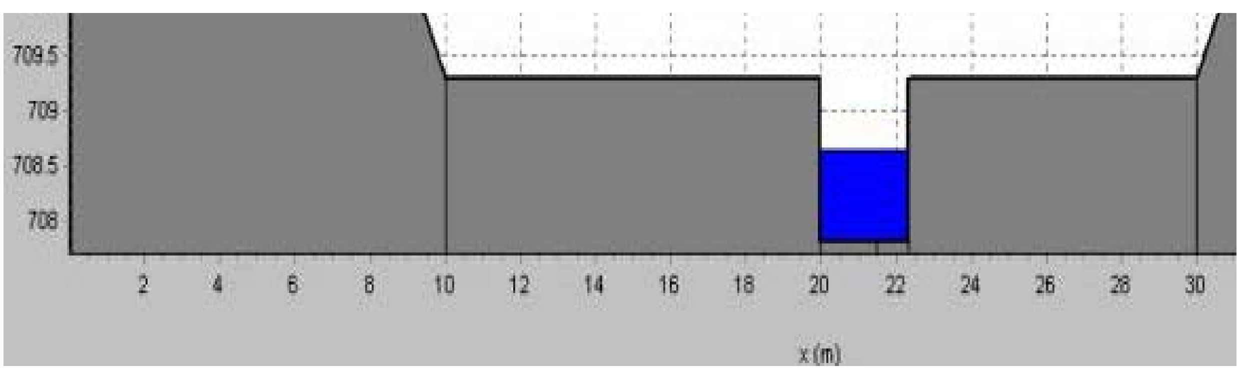

Figure 42. Lined canal section (main canal)

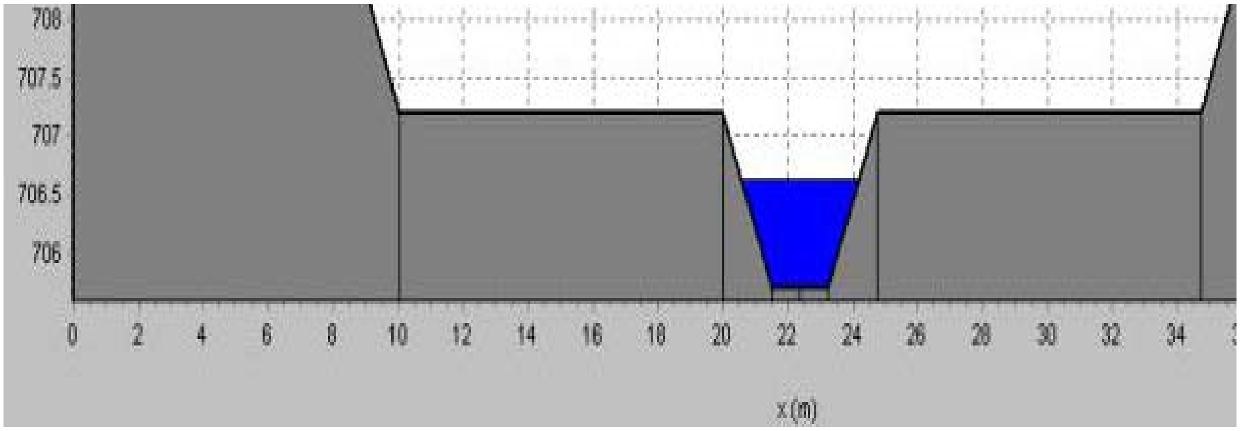

Figure 43. Earthen canal section (main canal)

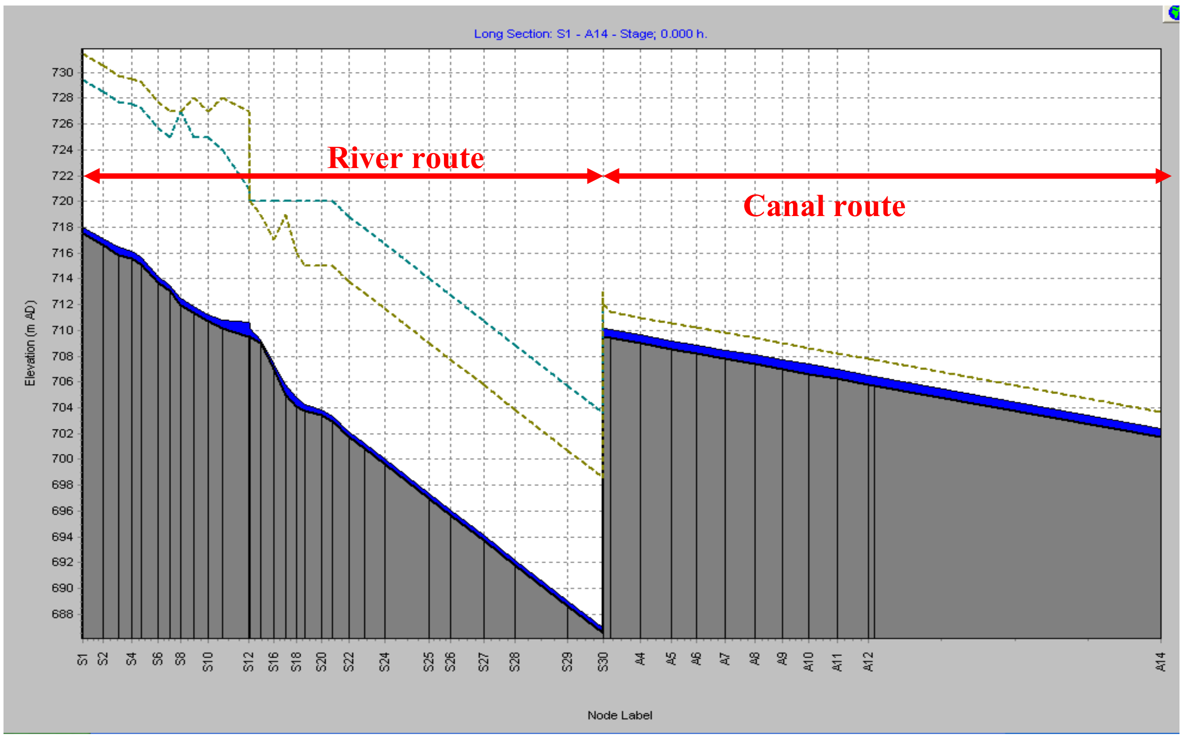

Figure 44. Longitudinal profile of the river and canal from ISIS model

4.5 Model Calibration

For model calibration and validations, long series of sediment concentrations and flow are necessary. However, for the Fokisa spate irrigation scheme, neither of them is readily available. Hence, the only possible calibration can be done on ISIS is, hydraulic calibration from the data collected from the farmers. Since there was no sand trap basin in the Fokisa spate scheme, DOSSBAS model is not calibrated. Furthermore, DOSSBAS is a steady state model; therefore it is not necessary to calibrate.

Before the process of calibration was started on ISIS flow, it is wise to carry out a sensitivity analysis on the model inputs like the time steps and hydraulic parameters, such as Manning's n and weir coefficient C, to determine the response of water depth and velocity. This gives an indication which parameters are sensitive to change and it would be good information in the process of calibration. The results of the sensitivity analysis are as presented in figures 45, 46 and 47. The values used to undertake the sensitivity tests are n = 0.035 and C = 1.7.

4.5.1 Sensitivity Analysis of the Input Parameters

The sensitivity analysis result indicate that, flow depth and velocity of flow are very sensitive to Manning's n, furthermore, the weir coefficient has an effect on the head over the weir, and hence flow depth and velocity are sensitive to the weir coefficient on the weir location and some distance upstream of the weir. However, the flow depth and velocity are not sensitive to time steps. Meanwhile, to avoid instability of the model, a time step of 1 sec has been utilized.

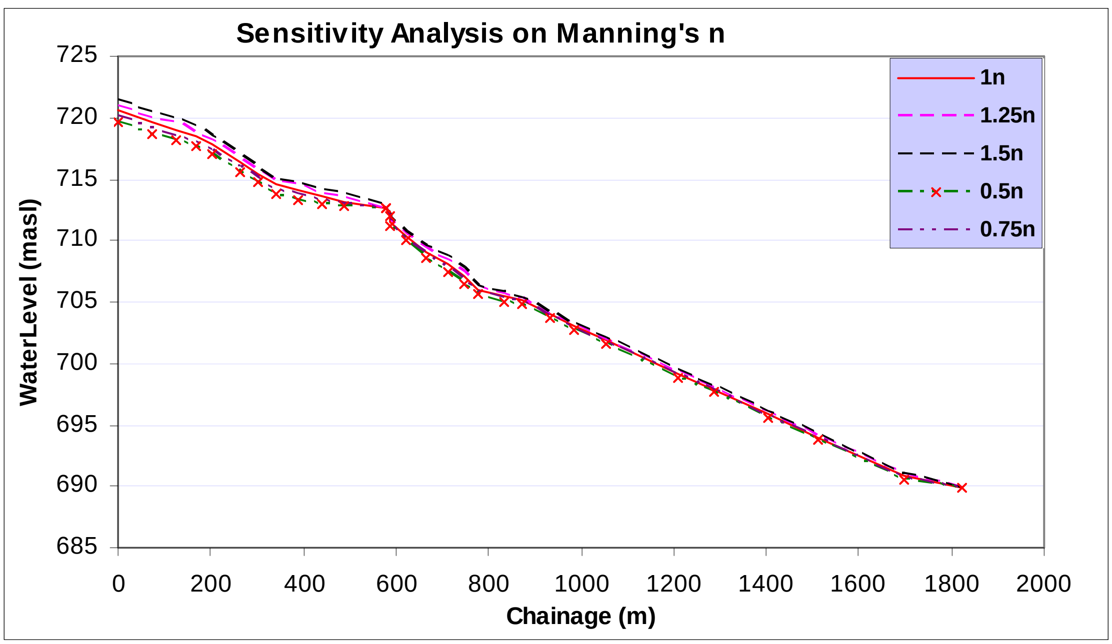

Figure 45. Sensitivity test on Manning's n

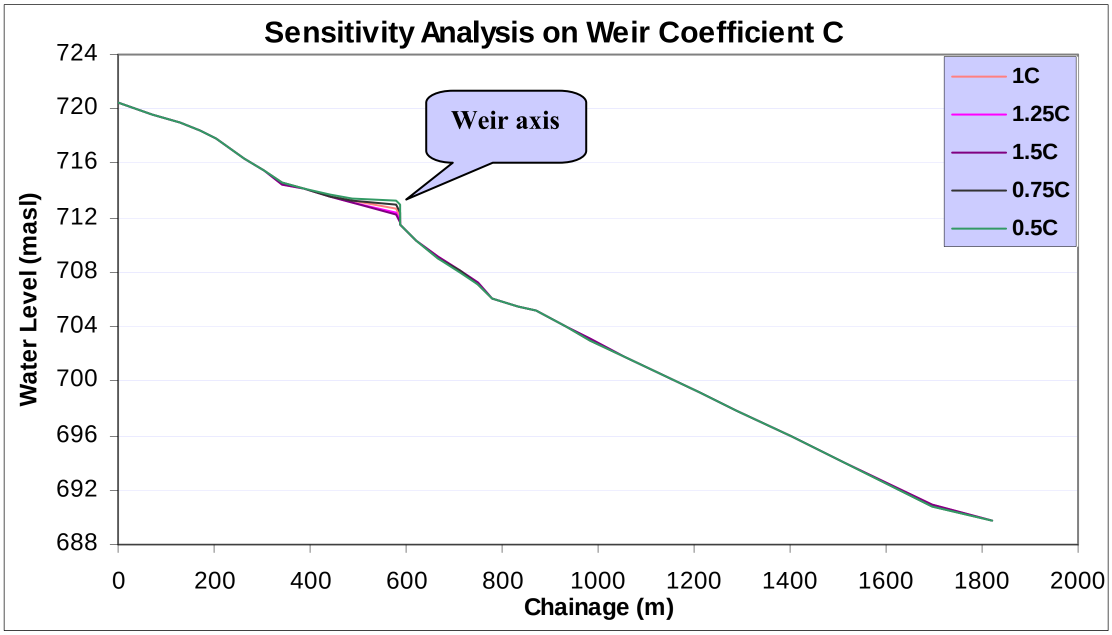

Figure 46. Sensitivity test on Weir coefficient C

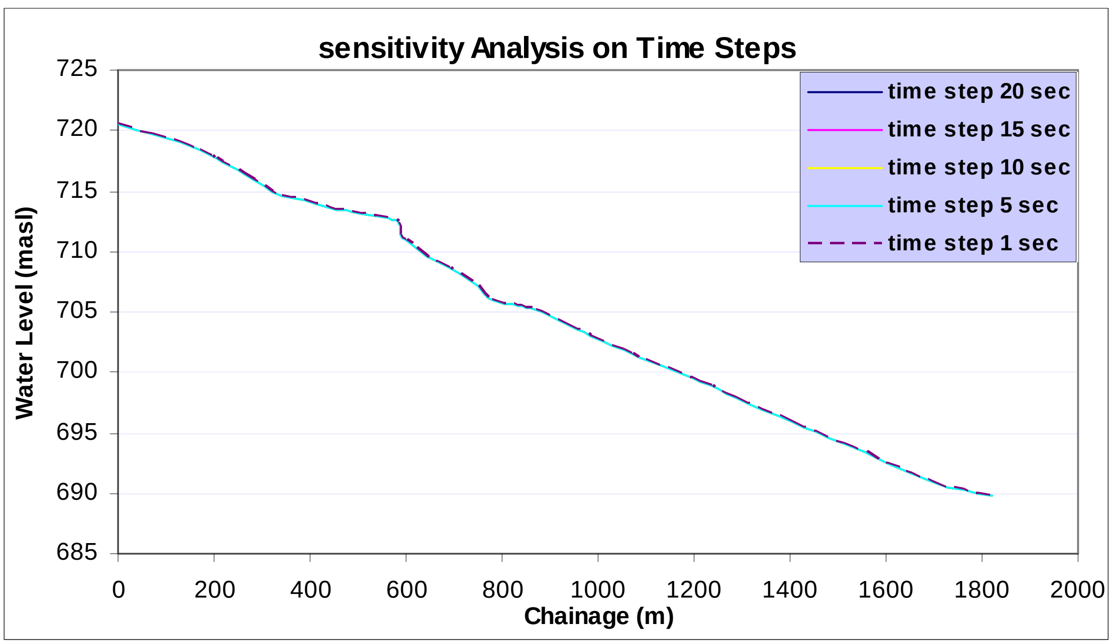

Figure 47. Sensitivity test on time steps

4.5.2 Boundary Conditions

The boundary conditions for the ISIS flow model have been collected during the data collection period. Accordingly, the upstream boundary conditions are discharge versus duration. In addition downstream boundary conditions have also been set to be water levels versus time (rating curve) at both, the downstream river cross section and canal. The downstream water levels at both the canal and the river for the medium flow were collected as a result of the interview held with farmers and experts around the area.

4.5.3 Model Calibration

First ISIS flow has been calibrated for the whole river and canal sections using observed water levels at the mean flow. The result of the calibration is shown in figure 48. And to improve the stability of the model at low flows, it was necessary to build a separate canal model with similar given flow properties. The comparisons are made based on the observed data (at medium flow) and generated data for the small and large flows. Hence the canal flow was recalibrated and checked as below (refer figure 49)**.**

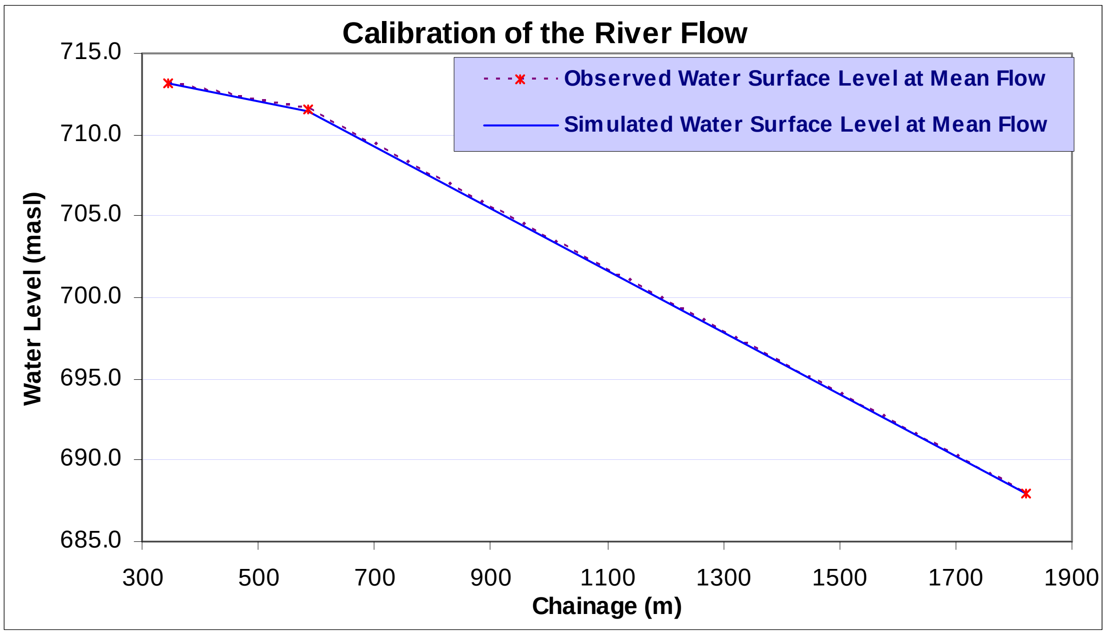

Figure 48. River flow calibration results

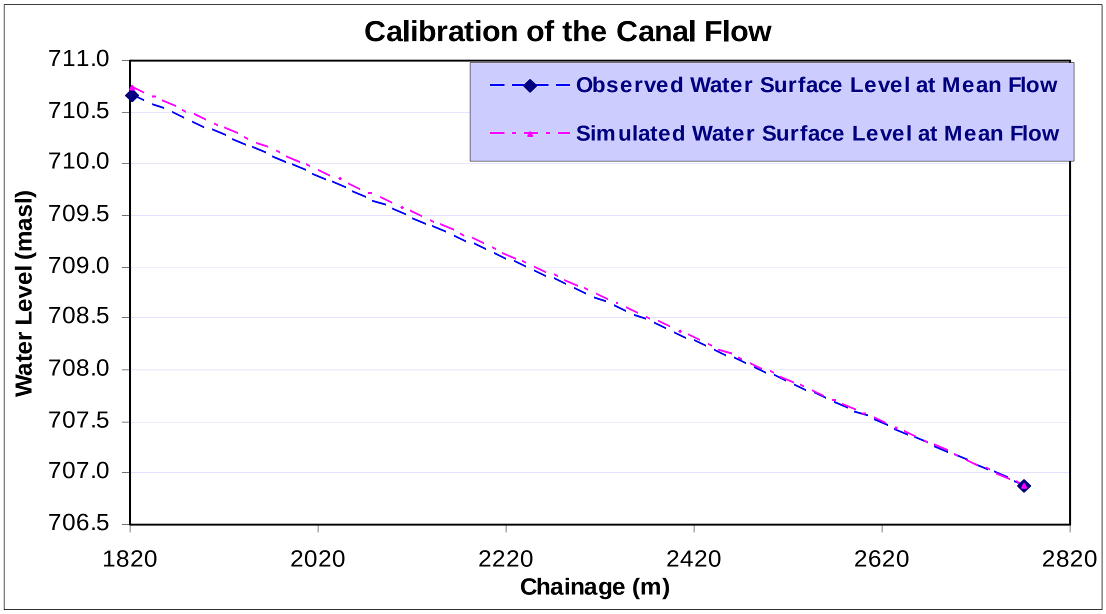

Figure 49. Canal flow calibration results

Summary of the calibration results are presented in ANNEX-B.

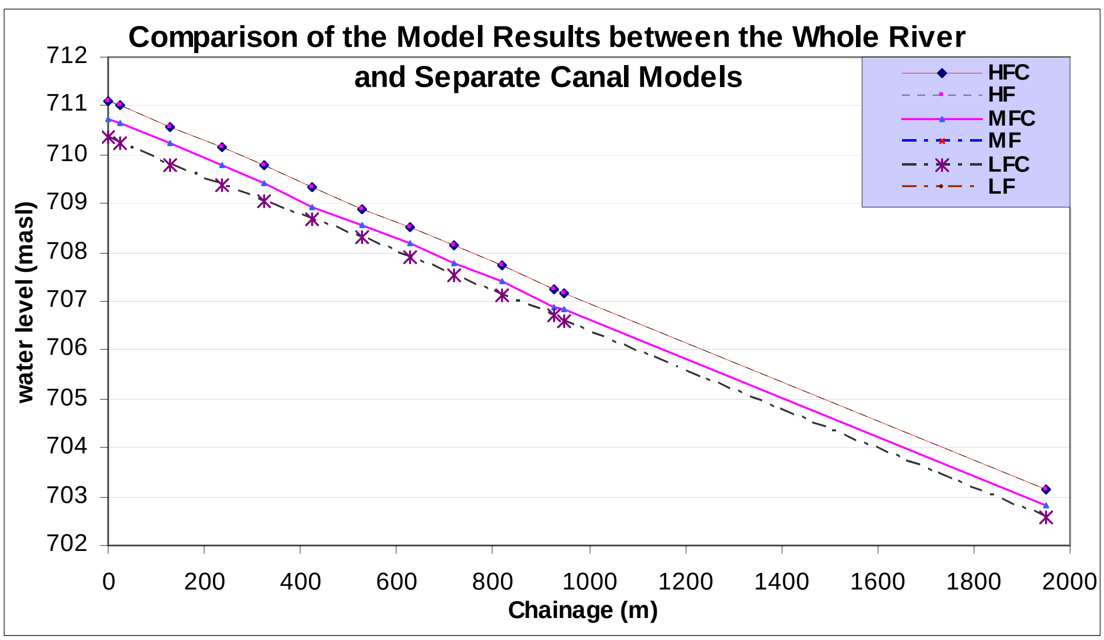

Figure 50. Separate canal and whole river model comparison results

As can be seen from figure 50 above, the calibration results of the whole river model and the separate canal flow model are very close to one another. The results of the high flows of the separate canal and the whole river are all equal. However, the results of the medium flow show a maximum difference of 3 cm and the results of the low flow show a maximum difference of 8 cm. Therefore, this little difference in water depth will not have a significant effect on the bed level changes of the canal sedimentation.

5. RESULTS AND DISCUSSION

This chapter deals with the major findings of the research and discusses selected scenarios. It focuses on existing condition of the scheme, improving the scheme performance with settling basins, improving the scheme performance without settling basins, and sensitivity tests to some of the model inputs.

5.1 Scenario I: Existing Condition

Before proposing solutions that would improve the performance of any irrigation system, it is wise to evaluate the performance of the existing situation. To do so, the canal system has been tested using both the DOSSBAS model, treating the canal as a long and narrow basin and using an ISIS sediment model, with the canal represented as a lined main canal section having 0 side slope at the beginning (chainage, 0 – 450 m), and the remaining section has1:1 side slope (chainage, 450 -1050 m). The results from the two models are discussed below.

5.1.1 DOSSBAS Steady Results on Existing conditions

In this case all the three discharges (large, medium and small discharges) have been tested. The results show that, for the large flows, 13.3 % (896m^3) of coarse sediment is deposited along the canal route. In addition a sediment level rise of 0.4m is observed. The trapping efficiency increases with an increase to the downstream water level and the downstream water level could increase with sediment deposition at the downstream canal. Therefore, the trapped sediment volume will increase with an increase to the volume and duration of diverted large flows, till the basin fills. The result is as shown in figure 51. Farmers show higher interest to divert all the flows, diverting large flows reduces the capacity of the canal flow due to sedimentation.

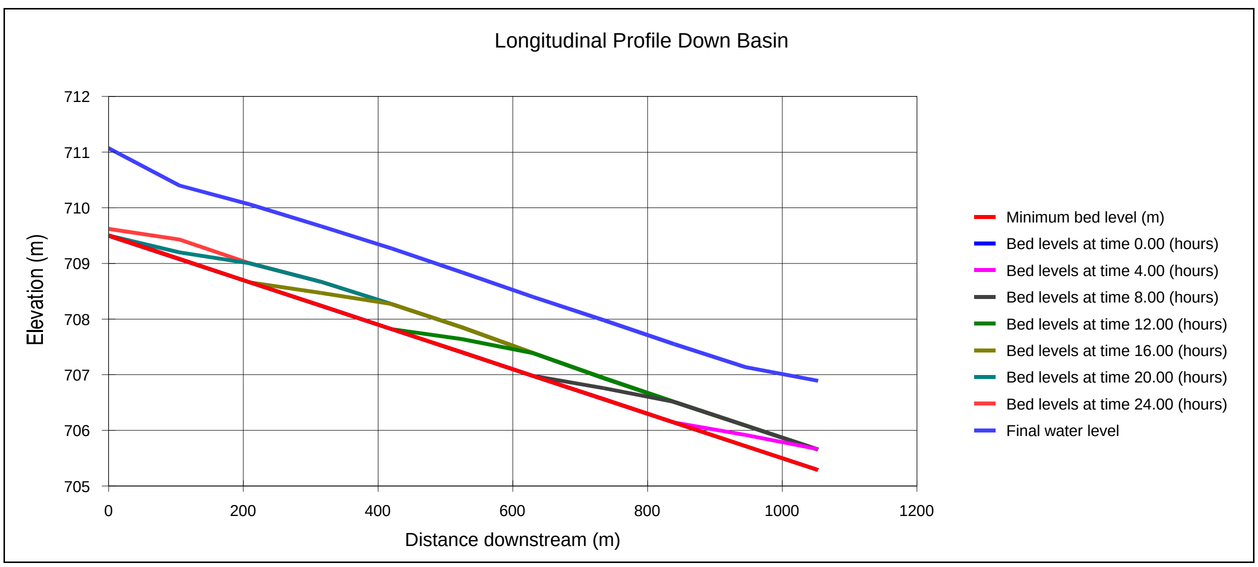

Figure 51. DOSSBAS simulation results of an existing condition with large flow.

In cases of medium and small flows all the fine and coarser sediments are not trapped in the canals, they all tend to deposit in the irrigated fields. The design report of the scheme indicates that, the canal route is designed based on "non-silting and non-scouring" criteria, which should not be for spate schemes (refer to section 2.4). Therefore, though with non-optimal design criteria for spate design (i.e. followed perennial system design criteria), canal design criteria has been met. The results of the medium and small flow are similar and only the result of a medium flow is presented as shown in figure 52. As indicated in figure 52, there is no bed level change.

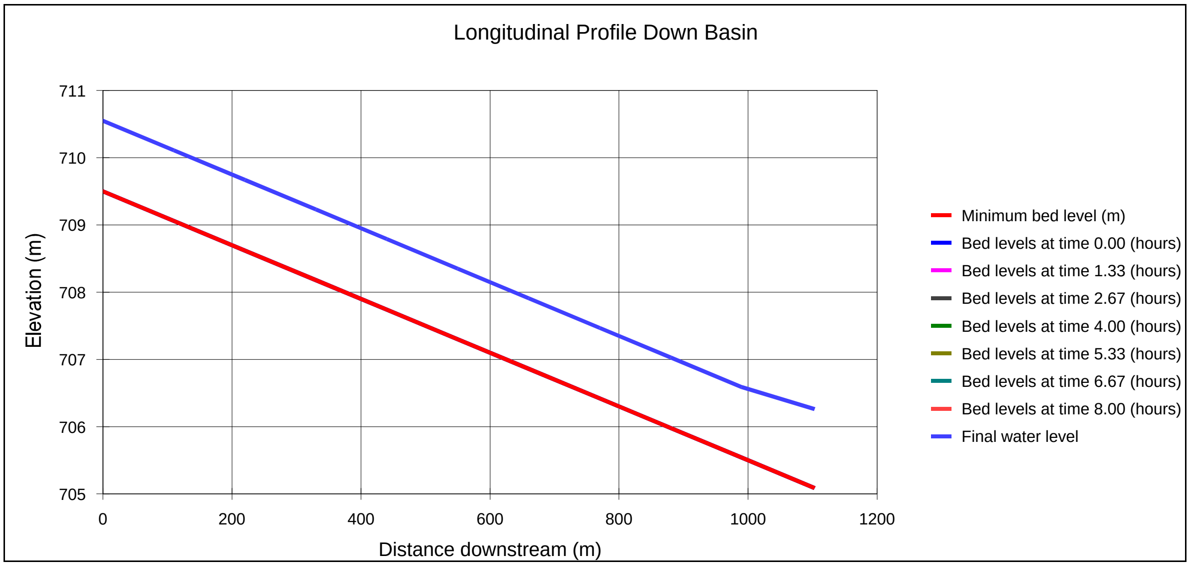

Figure 52. DOSSBAS simulation results of an existing condition with medium flow.

5.1.2 ISIS Results of the Existing Condition

Similar to the DOSSBAS results, the medium and small flow simulations of the canal route show that, there are no trapped sediments with in the canal. Both the fine and coarser sediments are all transported and deposited in the irrigated field. Though the entrance of fine sediments is very essential, the entrance of the coarser sediments is undesired. Coarser sediments here are simply contributing to the level rise of the command area. Results of the medium and small flow simulations are similar and the result of the small flow is as presented in figure 53.

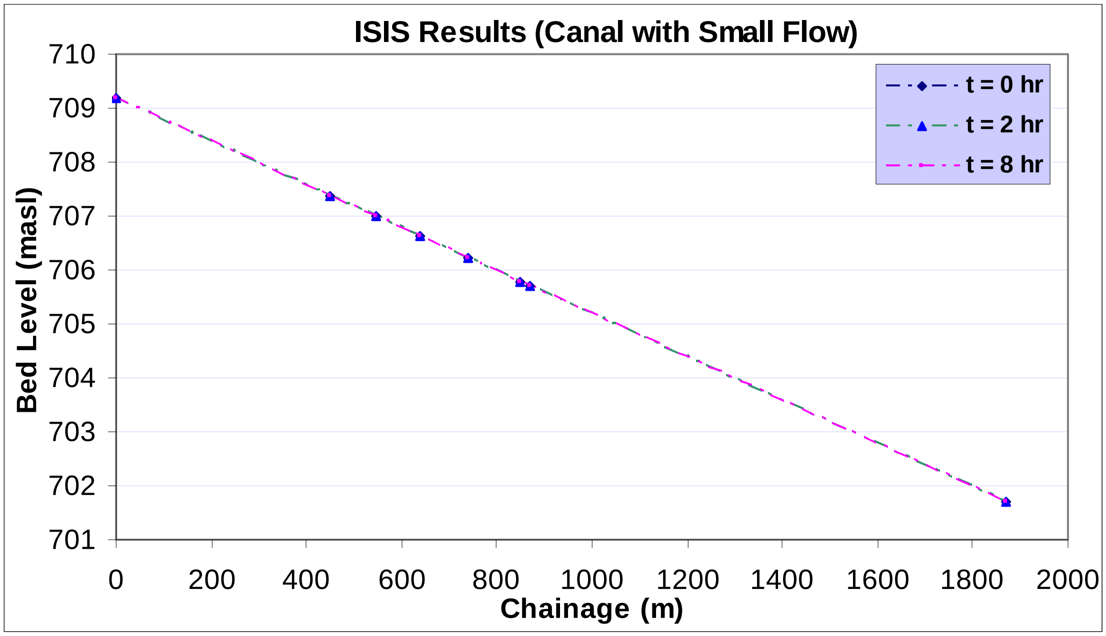

Figure 53. ISIS simulation results of an existing condition with small flow

The estimated sedimentation volume of coarser sediments in the canal during large flow simulation is 235 m^3. Furthermore, there is a sediment level rise of 0.4 m at the off take. Though trapping of coarser sediments is important, the diversion efficiency of the flow is reduced. Both results from DOSSBAS model and ISIS sediments for the large flow case show that, bed level rises during the run which would reduce the canal capacity significantly. Results of the existing condition simulations at large flow are presented in figure 54.

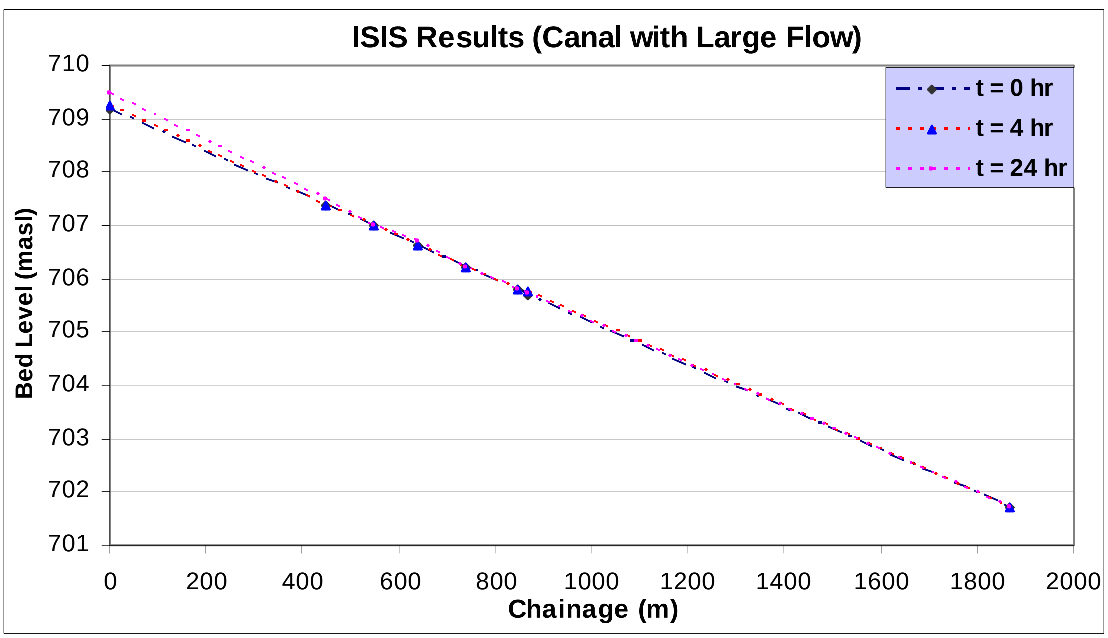

Figure 54. ISIS simulation results of an existing condition with large flow

Results of the yearly series flow in ISIS sediments show that, 426 m^3 of sediment is trapped at the end of the season. Volume calculations have been made by multiplying the deposition cross sections and canal length, as ISIS sediments presents the results of the deposition in m^3/s. Furthermore, a sediment level rise of 0.8 m is observed at the off take. For a canal having a design flow depth of 1.5 m sediment level rise of 53 % can be considered as significant. Results of the existing condition simulations at yearly long series flow are presented in figure 55.

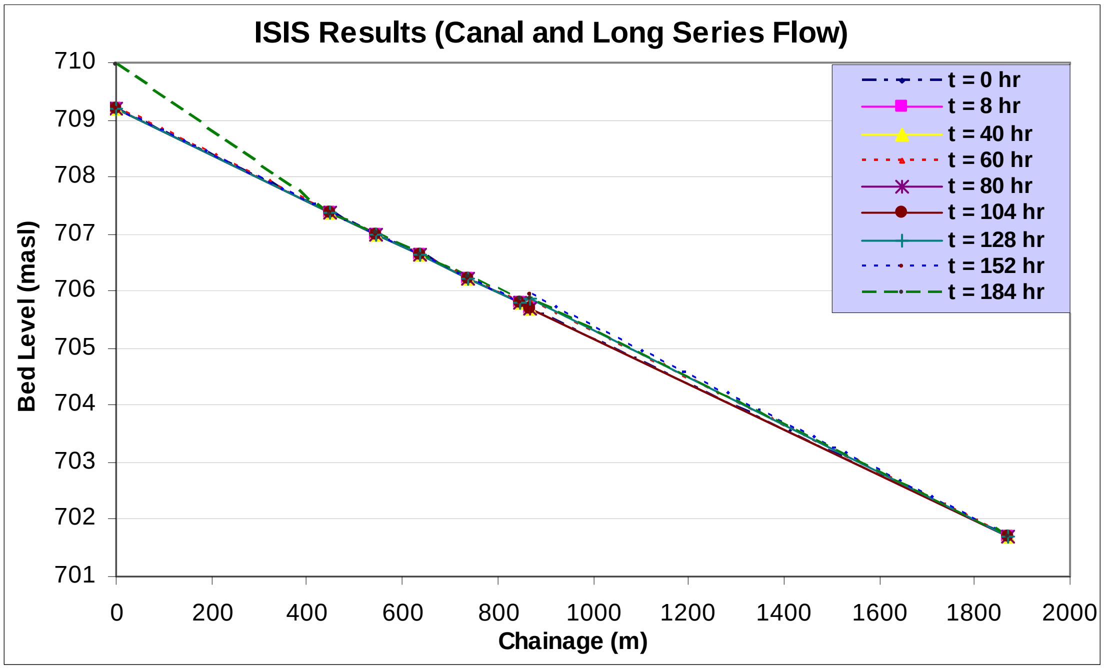

Figure 55. ISIS simulation results of an existing condition with yearly long series flow

Therefore, the above results indicate that, all the coarse and fine sediments are diverted to the irrigated field in cases of medium and small flow simulations, which the coarse sediments are not desired as they contribute to the command area bed level rise. However, the large flow and yearly long series flow simulation results reveal that, they trap 13.3% of the coarse sediments with respective bed level rise of 0.4 m and 0.8 m has been observed. Though trapping of coarser sediments is important, the bed level rises have a negative effect, if they are not frequently dredged, they reduce the diverting capacity of the scheme. For a scheme which is dredged twice a year, the existing sediment management practice is therefore, not effective.

5.2 Scenario II: Innovative Options to Improve the Performance of the Scheme

There are many ways to improve the sedimentation management of a spate irrigation scheme. Some of them are, providing settling basins, avoiding large flows using deflector weirs, changing canal cross sections and providing long main canals. To select the best option for the scheme, their performance is assessed and presented below.

5.1.1 Settling Basin

Minimizing the entrance of coarser sediment and maximizing that of fine sediments towards the irrigated field is an optimization process. Hence, it will be worth setting selection criteria, which would be helpful to see the efficiency of the basin under different operations. Before setting the selection criteria it is useful to assess the performance of previously implemented settling basins. Lawrence (2008) has indicated that, a disadvantage of settling basins in spate schemes is their high trap efficiency for fine sediments at low flows or when basins are empty. Therefore, in addition to those stated above, setting operational criteria would be the best option.

Therefore, the following selection criteria have been set.

- From economic point of view both in cost and energy required to dredge the basin, the basin should be manually cleaned once per season or twice a year, without interrupting irrigation or affecting the water distribution system of the scheme. (Fokisa spate scheme has two rainy seasons in a year).
- The majority of the sand should be trapped at a seasonal expected mean flow without affecting the bed material load transport capacity of the downstream canal, (the transport capacity of the canal should be equal to the mean sand concentration leaving the basin to avoid downstream erosion of earthen canals).
- The majority of the fine sediments should not be trapped using a seasonal expected flow.
- The basin should have lower trapping efficiency for fine sediments at low flows. (Q = 1.5 m^3/s is assumed to be low flow)
- The basin should have lower trapping efficiency for fine sediments when it is empty.

DOSSBAS Results

Optimizing the basin performance is a trial and error process. Many trials have been simulated in DOSSBAS model and the following basin has been selected, because it satisfies most of the above criteria. The results are as follows.

I. Dimensions of the basin

The basin is 875m long and has varying bottom widths. It is 2.34 m wide for the first 20 m at the entrance and 1m wide in the middle and 2.34 m wide for the rest 15 m at the exit. It is 5.0 m deep at the entrance and 3.5 m deep at the exit (including the flow depth {1.5m} and free board {0.5m}). Furthermore, the basin has a side slope of 1:1 and a longitudinal slope of 0. For details refer to the Auto Cad drawings of the basin in ANNEX-D. When it is compared to similar gravel trap basin from the Wadi laba scheme, which is 700 m long, 30 m wide and 4 m deep, the settling basin is of good dimension (discussion held with Dr. Abraham Haile Mehari). In addition Lawrence (2008) has indicated that, basins should be relatively narrow, with sediment storage obtained by increasing the length, rather than the width or depth of the basin. However, the basin should also be wider at the exit to accommodate for the area reduction due to change in longitudinal slope of the basin. The main purpose of having an irregular shaped basin is to create mean velocity which enables the trapping of coarse sediments and allows the passage of fine sediments by changing the cross sectional area.

II. Simulation results for Q = 6.8 m^3/s for 92 hours.

```
Summary Results

SUMMARY OUTPUT DATA.

Adaption length (% of basin length)    =       0.4 %
Sand trap efficiency                   =      63.7 %
Silt trap efficiency                   =       4.7 %
Trapped material that is silt          =      17.3 %
Volume of water at start of run        =    10534. m^3
Volume of water at end of run          =     3252. m^3
Total volume of sand deposited         =     6902. m^3
Total volume of silt deposited         =     1445. m^3
Mean sand concentration leaving basin  =     2520. ppm
Mean silt concentration leaving basin  =     6670. ppm
```

Figure 56. Summary of DOSSBAS steady model results (Q = 6.8 m^3/s, duration of 92 hrs)

- To protect the downstream canal from erosion, the canal dimensions should be set so that it should have its sand transport capacity in equilibrium. Therefore, the maximum achievable sand trap efficiency should be as computed below.

$$Sand\,\eta = \frac{Input.sand.conc - sand.conc.leaving.the.basin}{input.sand.conc.} * 100\% = \frac{6940 - 2540}{6940} * 100\%$$

$$= 63.4\ \%$$

Therefore, the basin sand trapping efficiency is excellent.

- The fine sediment trapping efficiency should be low as much as possible (0 %), hence a trapping efficiency of 4.7% can be appreciable.

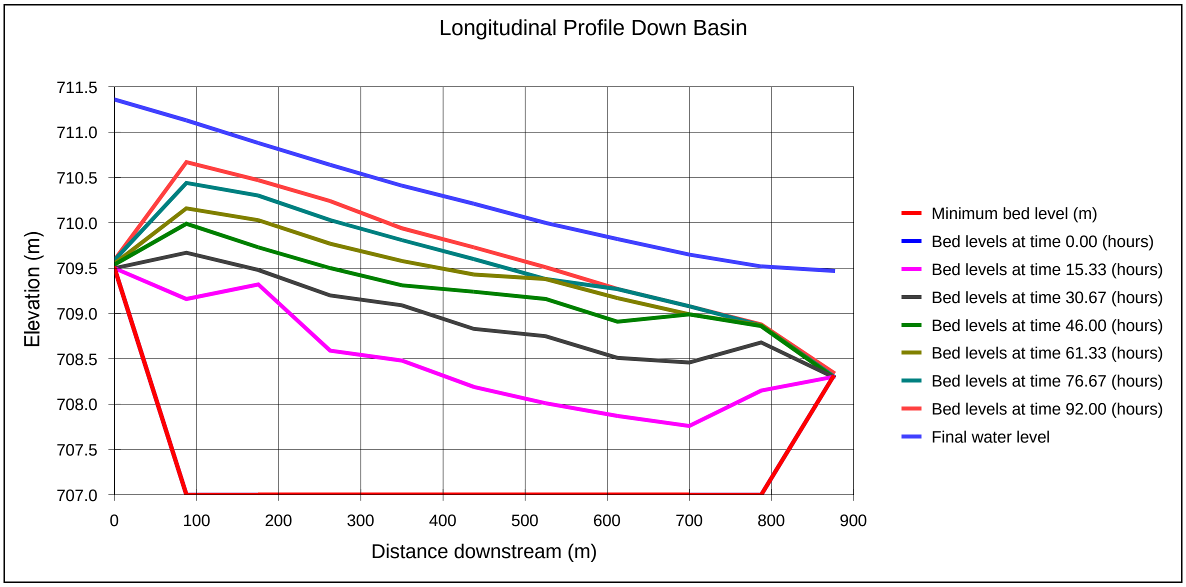

Figure 57. DOSSBAS model result showing the longitudinal profile of the basin at different durations

III. Simulation results for Q = 1.5 m^3/s for 92 hours (to see how it performs at low flows)

Summary Results

```
SUMMARY OUTPUT DATA.

Adaption length (% of basin length)   =      0.3 %
Sand trap efficiency                  =     99.9 %
Silt trap efficiency                  =     47.3 %
Trapped material that is silt         =     54.9 %
Volume of water at start of run       =    6280. m^3
Volume of water at end of run         =    3312. m^3
Total volume of sand deposited        =     964. m^3
Total volume of silt deposited        =    1176. m^3
Mean sand concentration leaving basin =       3. ppm
Mean silt concentration leaving basin =    1316. ppm
```

Figure 58. Summary of DOSSBAS steady model results (Q = 1.5 m^3/s, duration of 92 hrs)

At very low flows, the sand trap efficiency is high, it traps almost all the sand and this will cause downstream erosion of the canal. Moreover, its silt trapping efficiency is as high as 50%. This is the situation where the basin performs poorly.

IV. Simulation results for Q = 6.8 m^3/s for 8 hours (to see how it performs when the basin is empty).

Summary Results

```
SUMMARY OUTPUT DATA.

Adaption length (% of basin length)   =      1.0 %
Sand trap efficiency                  =     94.5 %
Silt trap efficiency                  =     16.8 %
Trapped material that is silt         =     33.4 %
Volume of water at start of run       =    9104. m^3
Volume of water at end of run         =    7337. m^3
Total volume of sand deposited        =     890. m^3
Total volume of silt deposited        =     447. m^3
Mean sand concentration leaving basin =     382. ppm
Mean silt concentration leaving basin =    5823. ppm
```

Figure 59. Summary of DOSSBAS steady model results (Q = 6.8 m^3/s, duration of 8 hrs)

- When the basin is empty, its fine sediment trapping efficiency is low which is good, however, its sand trapping efficiency is high in which it creates erosion at the downstream earthen canals.

ISIS Results

The small and medium flow ISIS sediment model simulation results show that, all the sediments are trapped by the basin and not in the upstream canal or off take. Therefore the basin is performing well by trapping sediments and it does not fill when a small flood or a medium flow is diverted towards the basin. The results of medium and small flow simulations are shown below in figure 60 and 61.

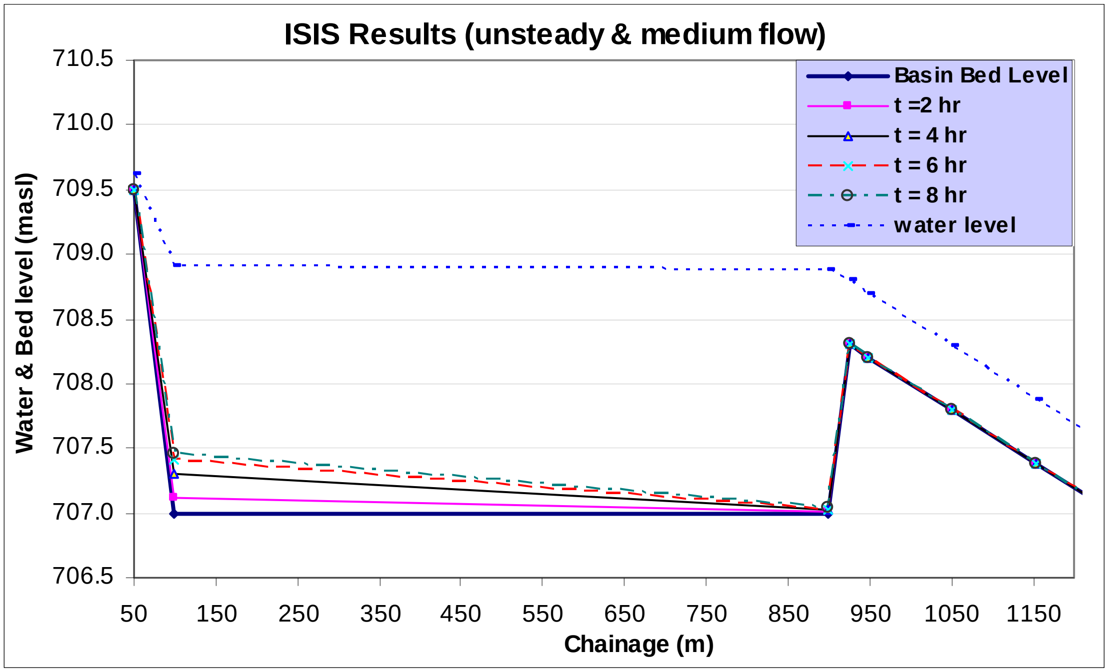

Figure 60. Medium flow ISIS sediments unsteady state simulation results

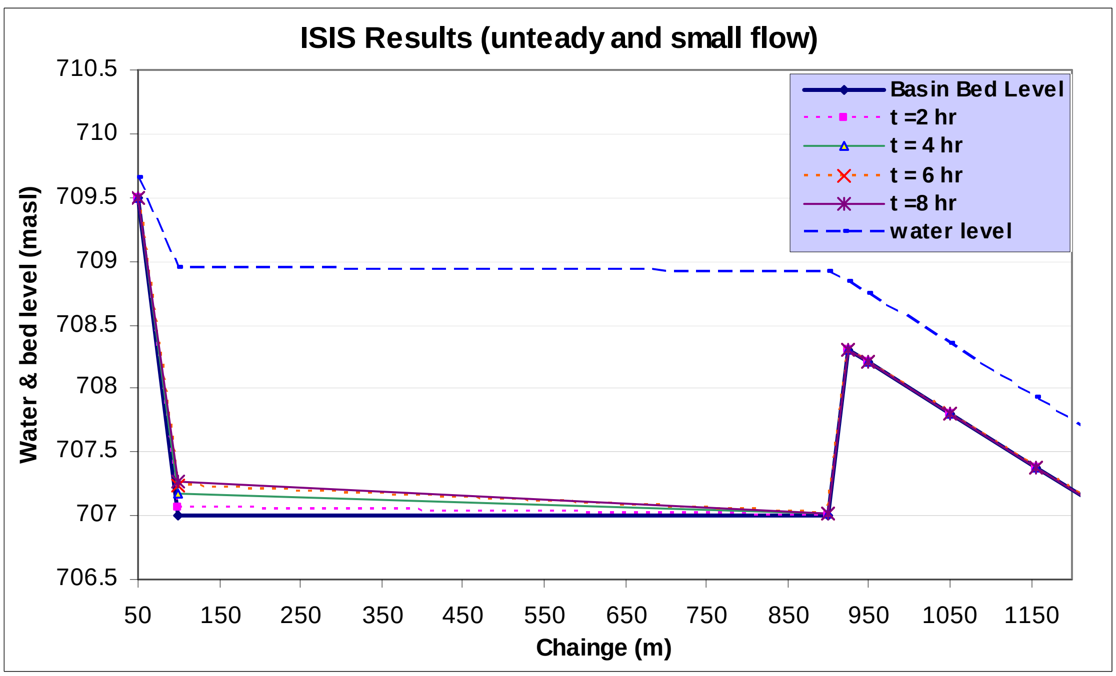

Figure 61. Small flow ISIS sediments unsteady state simulation results

The large flow ISIS sediments simulation results show that there is high sediment level rise around 1.5m at the end of the simulation period. This shows that diverting large flows to the basin has a large impact in lowering the basin performance. First it fills the basin after 1/3 of the large flow is diverted. Secondly it reduces the diversion capacity of the off take by raising the level of the upstream basin canal. The result of large flow simulations is shown below in figure 62.

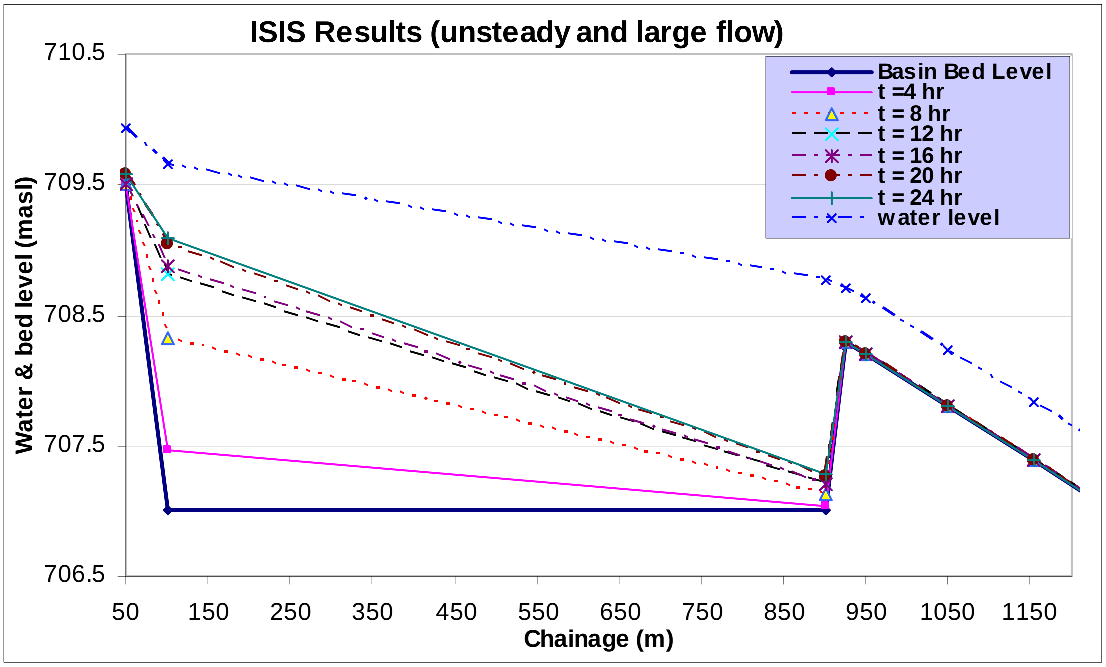

Figure 62. Large flow ISIS sediments unsteady state simulation results

To evaluate how the basin performs when it is not dredged for a year, a yearly long series simulation (for 184 hrs) has been run. The results show that, (as shown in figure 63), the sediment level rise at the off take of the canal is around 1m at the end of the medium flows (t = 61.33 hr) and is 3.5 m when large flow is diverted to the basin. Furthermore, the basin is filled after the entrance of a large flow (at t = 92 hrs). This indicates that a large flow lowers the performance of the basin significantly. If the basin is not dredged at the end of the season and large flows are diverted towards the irrigated scheme, the off take may be filled with sediments and the diversion capacity of the scheme is reduced. In addition, all coarser and fine sediments will tend to deposit in irrigated fields.

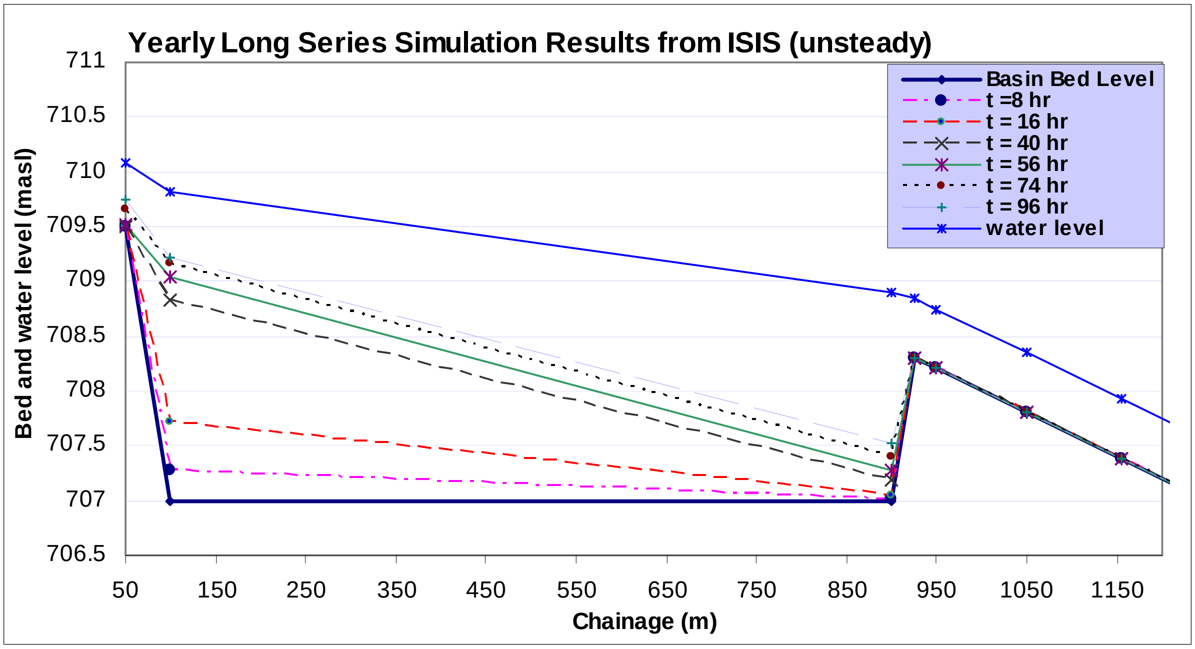

Figure 63. Yearly long series ISIS sediments unsteady state simulation results

The main problem with this basin is its high sand trapping efficiency when the basin is empty and when the basin operates at low flows. It causes downstream erosion of the canal route. Unlike Perennial River diversion schemes, spate schemes have unpredictable flow and operate under variable discharges, where the discharges vary even at a time of diversion. Therefore, compensating the sand transporting capacity of the canals at low flows and when the basin is empty is not an easy task. Moreover, the main canal of the Fokisa scheme is around 1km, providing such a long and large basin may not be economically justifiable.

Currently, the scheme is irrigating 150 ha of command area and approximately 2970 m^3 of sediment is dredged from the main canal every season in 2 to 3 days by approximately 120 to 130 farmers. The dredged volume per season from the basin is 8350 m^3. Assuming that the irrigated area would increase to 500ha (design capacity of the scheme), the number of serviced farmers would increase by three fold. Therefore, an increase to the volume of sediments by three fold would be handled by the increase in the number of spate irrigating farmers. Therefore, the increased volume of dredging is not going to be a problem.

5.2.2 Changing Canal Cross Sections and Slope along the Canal Route

To see the effect of modifying the canal cross sections and slopes many simulations have been run in DOSSBAS. It is a trial and error process to find out an optimum trapping efficiency. The purpose of changing the canal dimensions and slope is to bring about changes in the velocity of the flow, so that the coarser sediments will be settled around the off take in the upstream canal. Results show that without changing the slope of the canal route, sand trapping efficiency of 26.1 % for medium flow and 15.1 % for small flow can be achieved. But if changes to the slope of the canal route are integrated with the canal dimension modifications, sand trap efficiency of

28.7 % for medium flow and 15.6 % for small flow is achieved. Slope changes in the earthen canals can easily be made. Therefore, this can easily be implemented. Results of the DOSSBAS model simulation are presented below in figures 64 and 65. Auto cad drawing of the canal dimension changes is as presented in ANNEX-D.

Summary Results

```
SUMMARY OUTPUT DATA.

Adaption length (% of basin length)    =       1.2 %
Sand trap efficiency                   =      28.7 %
Silt trap efficiency                   =       1.3 %
Trapped material that is silt          =      11.2 %
Volume of water at start of run        =     6054. m^3
Volume of water at end of run          =     4226. m^3
Total volume of sand deposited         =     3212. m^3
Total volume of silt deposited         =      407. m^3
Mean sand concentration leaving basin  =     4958. ppm
Mean silt concentration leaving basin  =     6910. ppm
```

Figure 64. Summary of DOSSBAS steady model results for canal cross section and slope changes

Deposition Model - Setup Irregular Geometry

	Distance U/S (m)	Bed Elevation (m)	Bed Width (m)
1	0.0	705.300	1.75
2	150.0	705.900	1.75
3	200.0	706.100	20.00
4	250.0	706.300	20.00
5	300.0	706.500	1.75
6	450.0	707.100	1.75
7	500.0	707.300	20.00
8	550.0	707.600	20.00
9	600.0	707.700	2.34
10	1050.0	709.500	2.34

Figure 65. Summary of the canal geometry input to the DOSSBAS model

5.2.3 Deflector Weir

Deflector weir is implemented at the upstream of the main canal. It will be used to avoid the diversion of large flows in to irrigated fields. The advantage of avoiding large flows has been discussed in section 5.1.1 above, therefore it will not be discussed again here. The details of the deflector/ rejection weir is attached in ANNEX-D.

5.2.4 Long Main Canals

Particle size analysis of the scheme, which was collected during the data collection period, was held along the canal route starting from the off take towards the irrigated field. It is observed that, D_{50} of the particles decreases from 1.05 mm at the off take to 0.18mm at the secondary canal and it further decreases to 0.01mm at the irrigated field. In addition, the maximum diameter of a trapped particle at the off take is 19 mm, 9.5 mm at the secondary canal and 4.75 mm at the irrigated field.

The textural distribution of sediments at the off take shows that, 16.3 % of the particles are gravel, 82.7 % are sand and 0.9 % is silt and clay. Similarly the distribution at the secondary canal shows that, 1.3% of sediments are gravel, 90.7% are sand and 8.0 % of the particles are silt and clay. Furthermore, the distribution at the irrigated field shows that 11.3% of the sediments are sand, 88.7 % are silt and clay and with no gravel. Though long main canals might have implications on water distributions system of the scheme, the above results prove that long canals are very helpful in trapping most of the coarser sediments.

5.3 Comparison of DOSSBAS Steady with ISIS Sediments

Comparisons between predictions from ISIS sediments and DOSSBAS steady models were undertaken. ISIS sediment was run in steady states for discharges 6.8m^3/s and 1.5 m^3/s and for duration of 92 hrs (one season). Although the comparison of the two models was not simple, volumetric computations of the sediments and sediment level rises was used as a comparison method. The bed level rises in the basin are lower for ISIS sediments than for DOSSBAS. Similarly the volume of trapped sediments from the ISIS sediments (6850 m^3) is much lower than that of DOSSBAS simulation result (8350m^3).

The comparison of the two models ISIS sediments have been done with limited data. However, it can be concluded that, DOSSBAS which is a steady state model, is a useful tool and can be used the initial stages of design for indicative sizing. If further performance evaluation of the two models is needed, it has to be supported by field measurements on both input and output data.

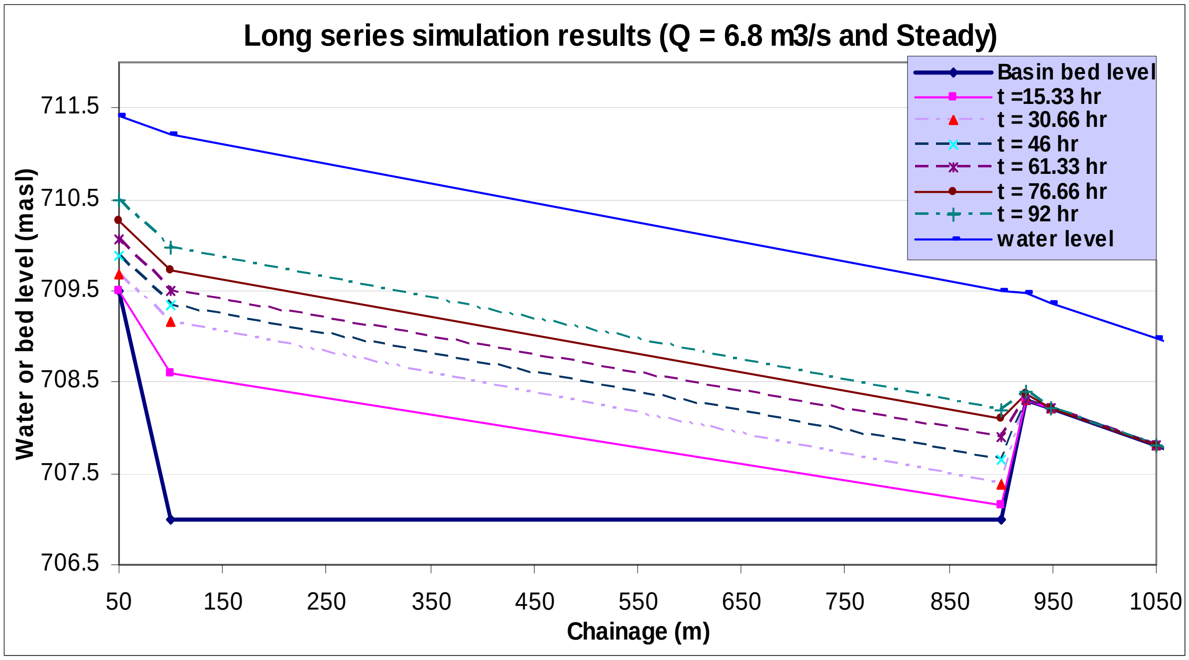

Figure 66. Long series steady simulation results from ISIS model for Q = 6.8 m^3/s

5.4 Sensitivity Analysis

Sensitivity analysis on some of the model inputs and hydraulic parameters has been carried out. The purpose is to see how these values affect the results. Therefore sensitivity tests have been completed on the following parameters.

- hydrograph shape
- flow sequences
- particle size distribution
- sediment concentration
- sediment transport formula and
- Manning's roughness coefficient n
- The results are shown in sections 5.4.1 to 5.4.6.

5.4.1 Sensitivity Check on Hydrograph Shape Changes

This sensitivity test is carried out so that the research output could be useful for other spate schemes. Therefore, the two hydrograph types, the Fokisa scheme hydrograph and the hydrograph developed from other spate schemes has been tested. The results are as follows.

Results from the Fokisa Hydrograph

As it can be seen from figure 67 below, there is a settling basin bed level rise during the 8hrs (whole simulation period). However, the relative bed level rise for the period of an increase in velocity (0-2hrs) is lower than the rest of the simulation period. This is because of a continuous decrease in velocity which has a direct effect on deposition of sediments (in which more than

80% of the deposition occurs during the period of decreasing velocity). However, deposition is not only related to the decrease in velocity, it is also related to the decrease in water surface slope. For example, high sediment level rise is observed at time 7.92 hrs where the slope of the water surface shows a significant decrease with a slight increase in the velocity as shown in figure 68.

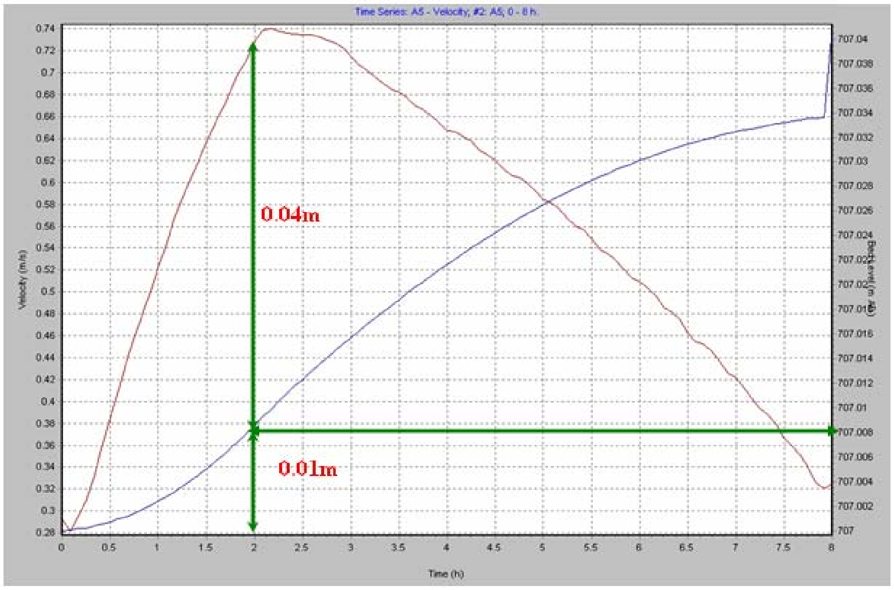

Figure 67. Velocity and sediment bed level rise comparisons from ISIS model on settling basin section 5 for the Fokisa scheme hydrograph

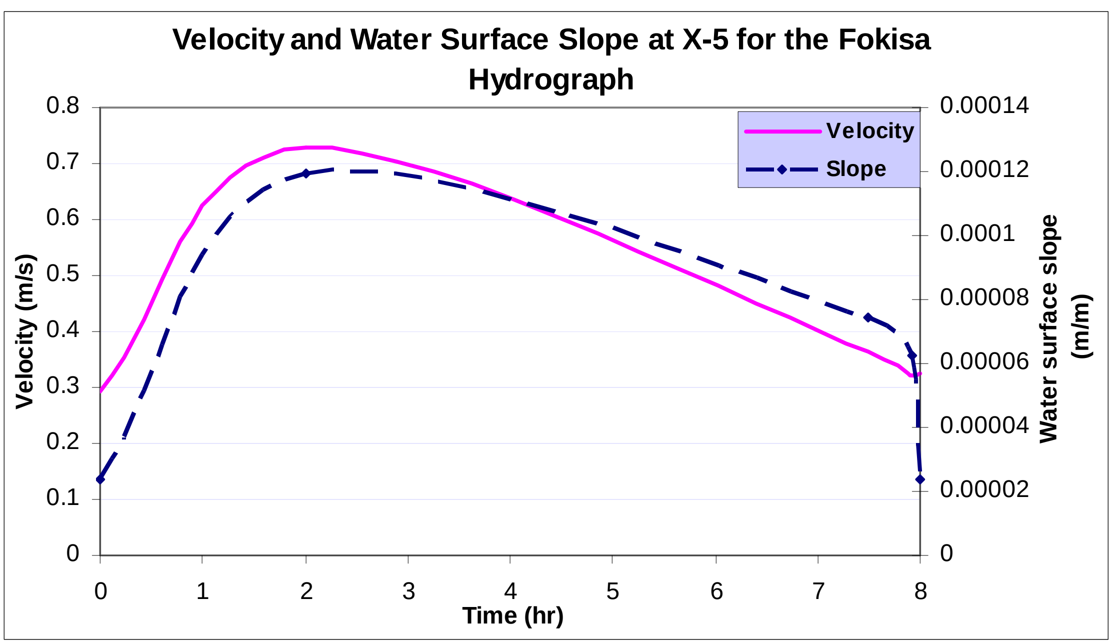

Figure 68. Velocity and surface slope change comparisons from ISIS model on settling basin section 5 for the Fokisa scheme hydrograph

Results of the Hydrograph Developed from Other Spate Schemes

As it can be seen from figure 69 and 70 below, similar to the above results, the majority of the bed level rise is during the periods of decreasing velocity and water surface slope reduction.

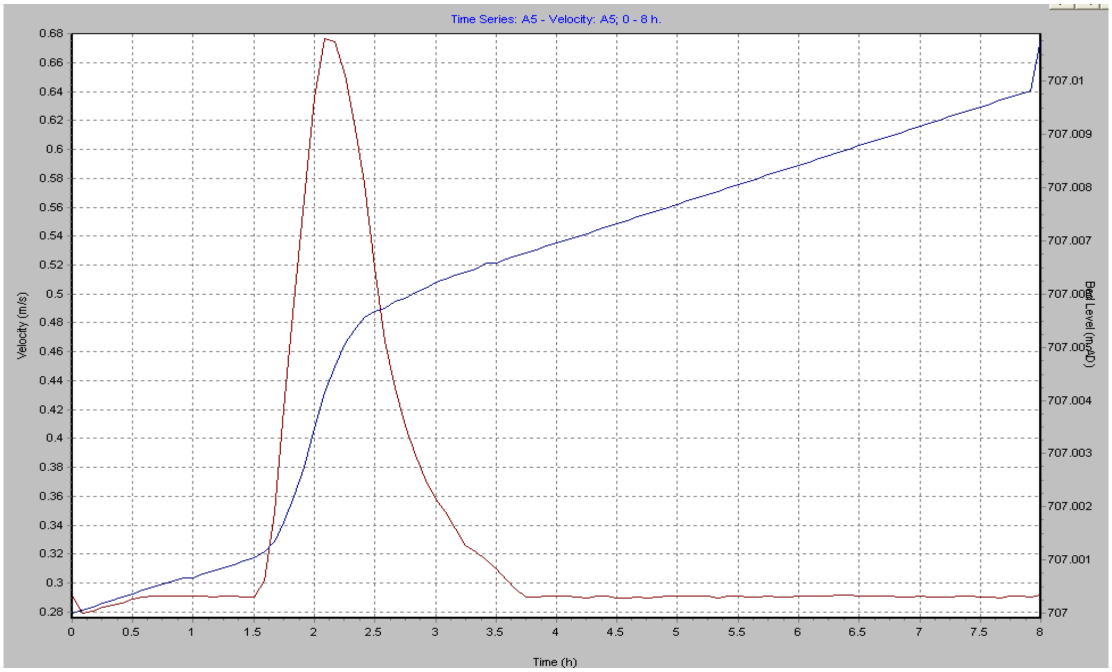

Figure 69. Velocity and sediment bed level rise comparisons from ISIS model on settling basin section 5 for the hydrograph developed from other schemes

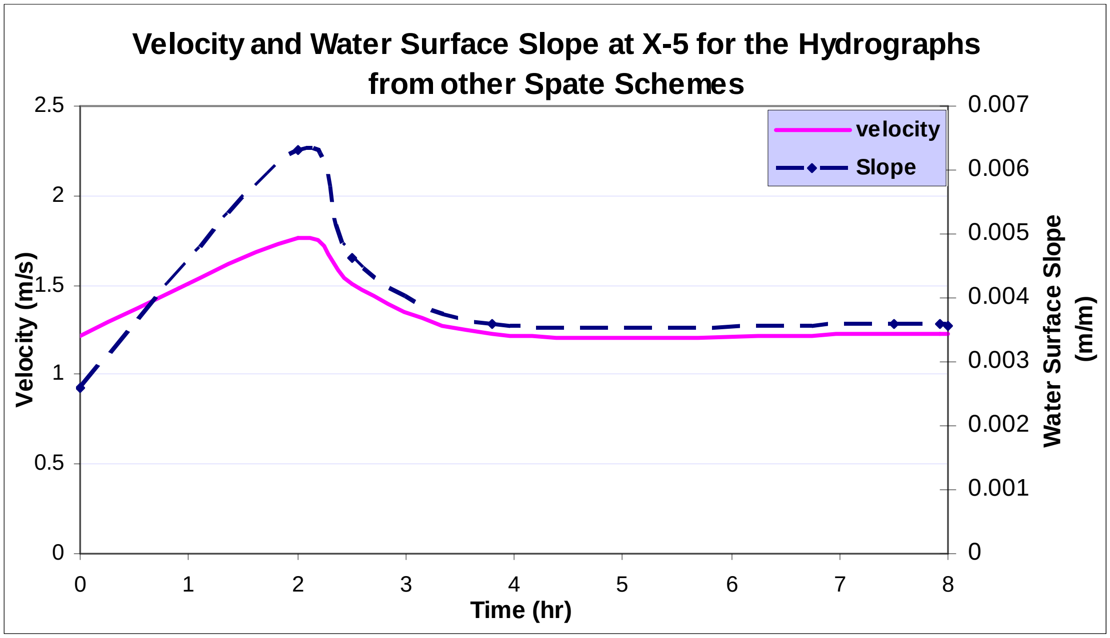

Figure 70. Velocity and surface slope change comparisons from ISIS model on settling basin section 5 for the hydrograph developed from other schemes.

The results of the two hydrograph types have been plotted on the same scale. Hence the results from the small, medium and large flow indicate that, the hydrograph of the Fokisa scheme yield higher sediment level rises and volumes. The result of the medium flow simulation is as presented in figure 71 below. The main reasons are,

- There is a continuous reduction in velocity from time, t = 2hr to time, t = 7.92 hrs for the Fokisa scheme hydrograph. However, in the other hydrograph, the continuous reduction in velocity starts at time, t = 2hrs and extends till time t = 3.8 hrs.

- Similar to the decrease in velocity, for the Fokisa scheme hydrograph, there is continuous reduction in the water surface slope from time, t = 2 hr to time, t = 7.92 hrs and significant reduction from time = 7.92 hrs to time = 8 hrs. However, in the other hydrograph, the continuous reduction in velocity starts at time, t = 2hrs and extends till time t = 3.8 hrs.

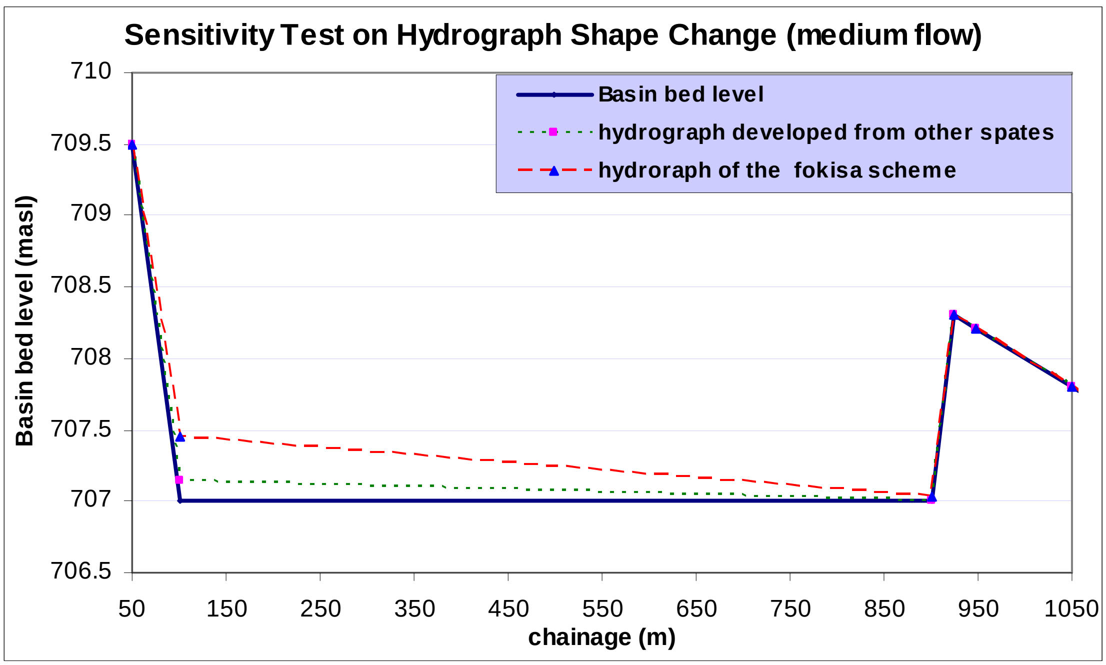

Figure 71. Sensitivity test results on changes to hydrograph shape changes

From the above results, it is clearly indicated that, sediment depositions and bed level changes are highly sensitive to the hydrograph shapes. Therefore, hydrological inputs should be collected carefully as it has an impact on the efficiency of the settling basin. The results/ charts of the small and large flow simulation are presented in ANNEX-A, and the summary tables of the water surface slope computations and velocities are also presented in ANNEX-B.

5.4.2 Sensitivity Check on Changes to Sequence of the Flows

To find out the effect of changes in the flow sequence, the following sequence of flow has been randomly selected and tested. The selected sequences are:

- All the large flows are at the beginning, all medium flows at the middle and all small flows are at the end. The result is as presented in figure 72.
- The sequence developed from the previous rainfall pattern of the area, and

Accordingly, the simulation results show that, 70% of the basin is almost filled by the first large flow (at t = 24 hr) for case I. While 75 % of the basin area is filled after 7 flows (t = 63hr) in which three of them are small flows and four of them are medium flows. Therefore, it can be seen that, the performance of the basin is highly dependent on the sequence of the flow. For spate schemes where the floods are unpredictable in nature, the application of sand trap basins is questionable.

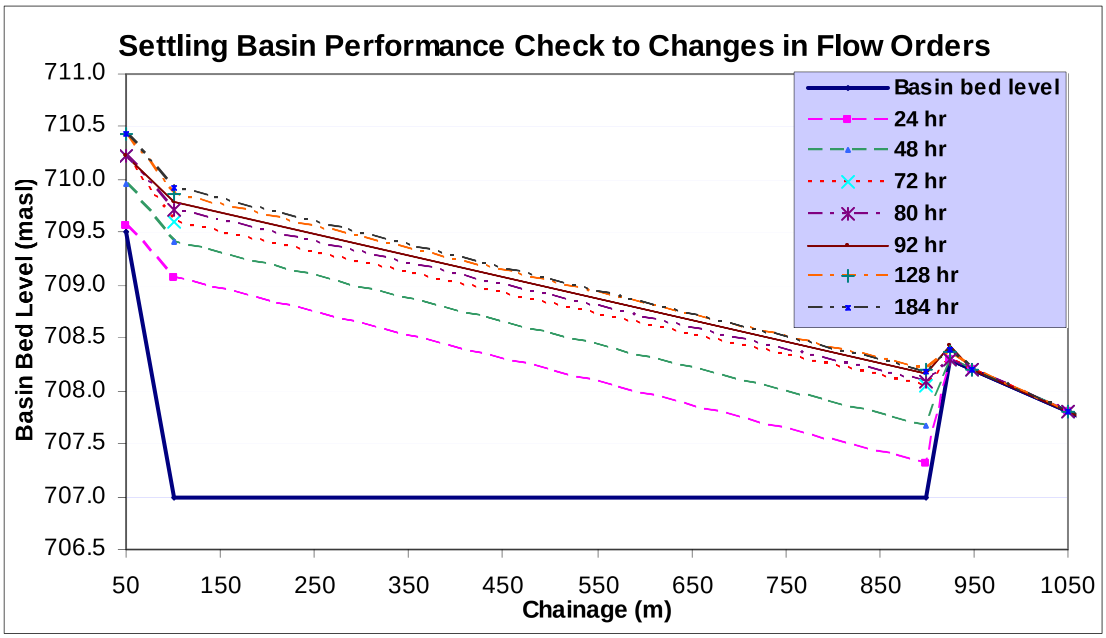

Figure 72. Sensitivity check to flow sequence changes

5.4.3 Sensitivity Check on Sediment Size Changes

Sensitivity check on the particle size change has also been undertaken. The particle size used was as in table 8 below.

Table 8. Particle size of the river

D_5	D_{15}	D_{25}	D_{35}	D_{45}	D_{55}	D_{65}	D_{75}	D_{85}	D_{95}
0.15	0.3	0.55	0.8	1.0	1.1	1.9	3.1	6.0	14 .0

Remark: all the particle sizes in table 8 above are in millimeters.

To see the effect of the changes to the particle sizes, it was multiplied by the factors 0.5, 0.75, 1.25 and 1.5. The input values used are summarized as in table 9 below. The results show that depositions in the basin increase with an increase to the particle sizes. Sherpa (2005), in his SETRIC and SOBEK Re model simulations has found similar results that given the sediment discharge constant, the bed level rise decreases with the decrease in sediment sizes for both the models. The results are as presented **in** figure 73 below. Therefore, care must be taken in selecting a representative particle size during studies and design periods.

Table 9. Summary of the particle size inputs used for sensitivity tests

Multiplier	D_5	D_{15}	D_{25}	D_{35}	D_{45}	D_{55}	D_{65}	D_{75}	D_{85}	D_{95}
0.5	0.08	0.15	0.28	0.40	0.50	0.55	0.95	1.55	3.00	7.00
0.75	0.11	0.23	0.41	0.60	0.75	0.83	1.43	2.33	4.50	10.50
1.25	0.19	0.38	0.69	1.00	1.25	1.38	2.38	3.88	7.50	17.50
1.5	0.23	0.45	0.83	1.20	1.50	1.65	2.85	4.65	9.00	21.00

Remark: all the particle sizes in* table 9 *above are in millimeters.

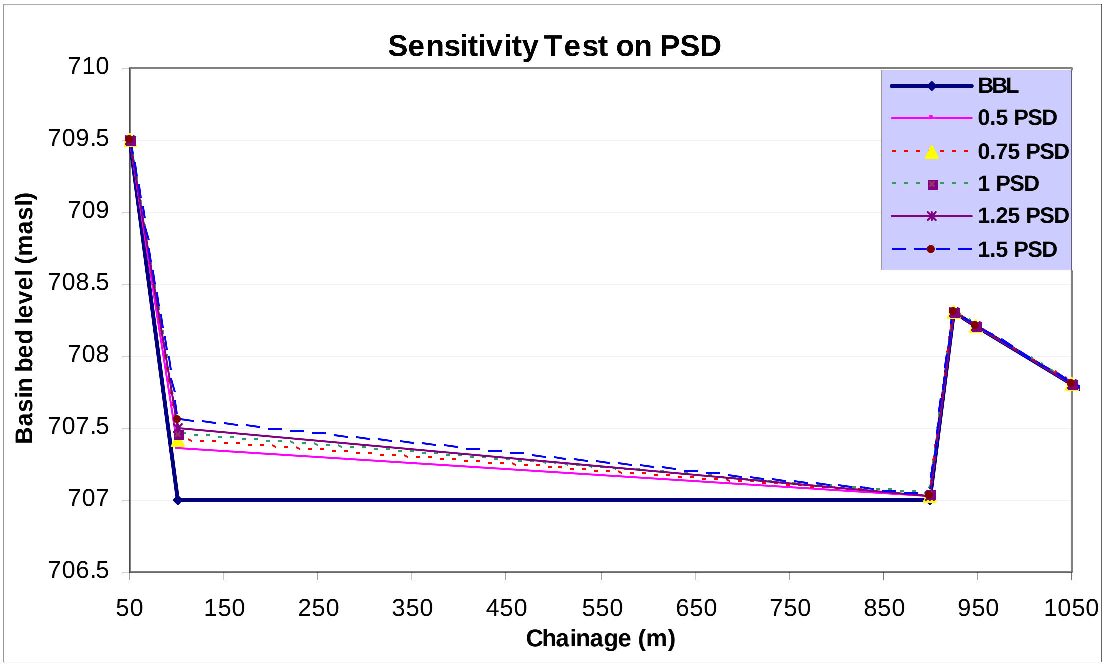

Figure 73. Sensitivity test on particle size distribution change

5.4.4 Sensitivity Check on Changes to Sediment Concentrations

Similarly, the effect of changes in the sediment concentrations has also been tested. The sediment concentration values used are as in table 10 below.

Table 10. Summary of the sand and silt concentrations

Discharge (m^3/s)	Sand conc. (ppm)	Silt conc. (ppm)	total (ppm)
1.5	2720	2500	5220
4.35	5295	6000	11295
6.8	6870	7100	13970
11.4	9555	12000	21555

To see the effect of the changes to the sand and silt concentrations, they were multiplied by the factors 0.5, 0.75, 1.25 and 1.5. The input values used are summarized as in table 11 below. The results show that depositions in the basin increase with an increase to the concentrations. Sherpa

(2005), in his SETRIC and SOBEK Re model simulations has found similar results that the bed level rise is high for higher concentration and low for lower concentration for the given sediment size. The results are as presented in figure 74 below. Therefore, for the scheme, where these values are not taken from field measurements, estimations have a significant effect.

Table 11. Summary of the sediment concentration inputs used for sensitivity tests

Discharge (m^3/s)	Multiplier				
	1	0.5	0.75	1.25	1.5
1.5	5220	2610	3915	6525	7830
4.35	11295	5650	8470	14120	16945
6.8	13970	6985	10480	17465	20955
11.4	21555	10780	16165	26945	32335

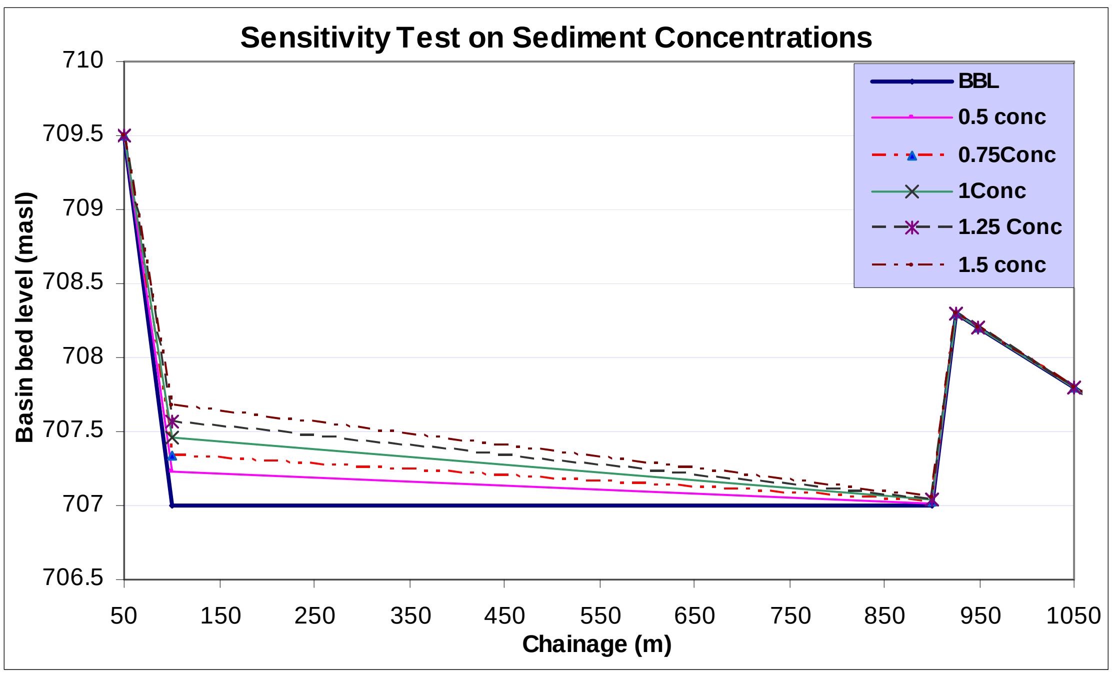

Figure 74. Sensitivity test on sediment concentration changes

5.4.5 Sensitivity Test on Changes to Sediment Transport Formulas

A sensitivity test on the changes to the sediment transport predicting equations has also been computed. The results show that, the Westrich- Jurashek equation, which is limited to fine sediments only, gives different result in terms of bed level rise. The level rise of the sedimentation in the basins is much lower for the Westrich- Jurashek equation, the other equations however, give almost similar results in which the differences in the level of sedimentation is in the range of centimeters. The results are as in figure 75 below. A combination of the Engelund and Hansen equation and the Westrich- Jurashek equation has also been tested. A combination has been computed, where Engelund and Hansen equation is applied for the coarser sediments and Westrich- Jurashek applied to the fine sediments. The result indicates that, the result of the combination is similar to that of the result from Engelund and Hansen equation.

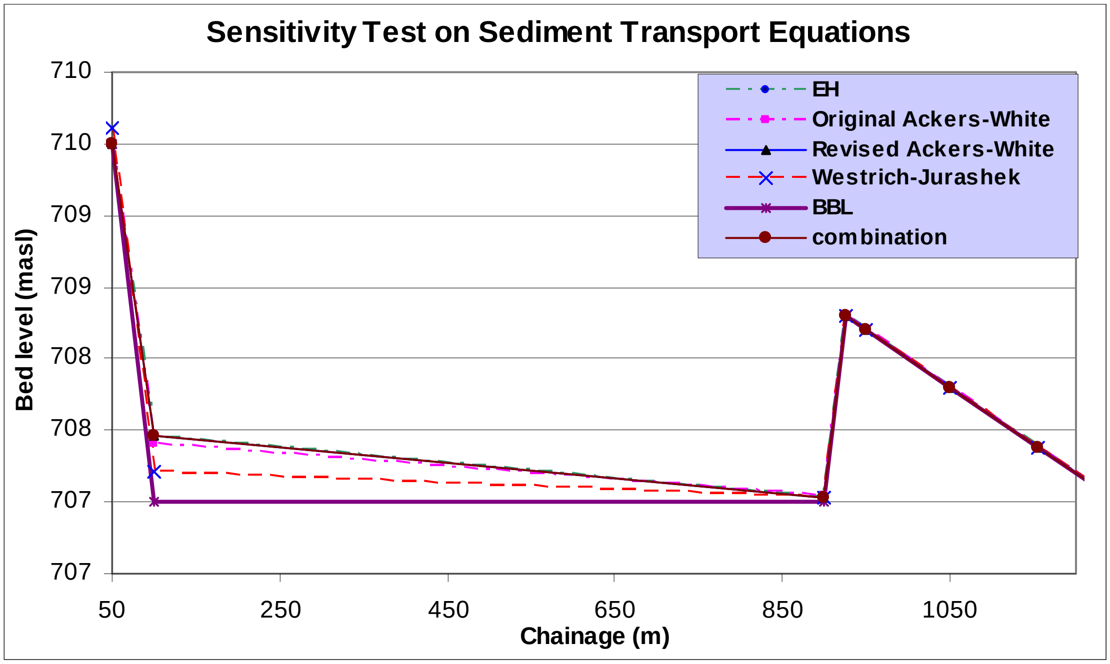

Figure 75. Sensitivity test on sediment transport equations

5.4.6 Sensitivity Check to Changes on Manning's n

Sensitivity analysis on the hydraulic parameter, Manning's roughness coefficient n has also been done. The calibrated Manning's n has been multiplied by factors 0.8, 0.9, 1.1 and 1.2. The results indicate that, though in the range of millimeters level of bed level rise increases with an increase to the Manning's n. The results are as in figure 76 below.

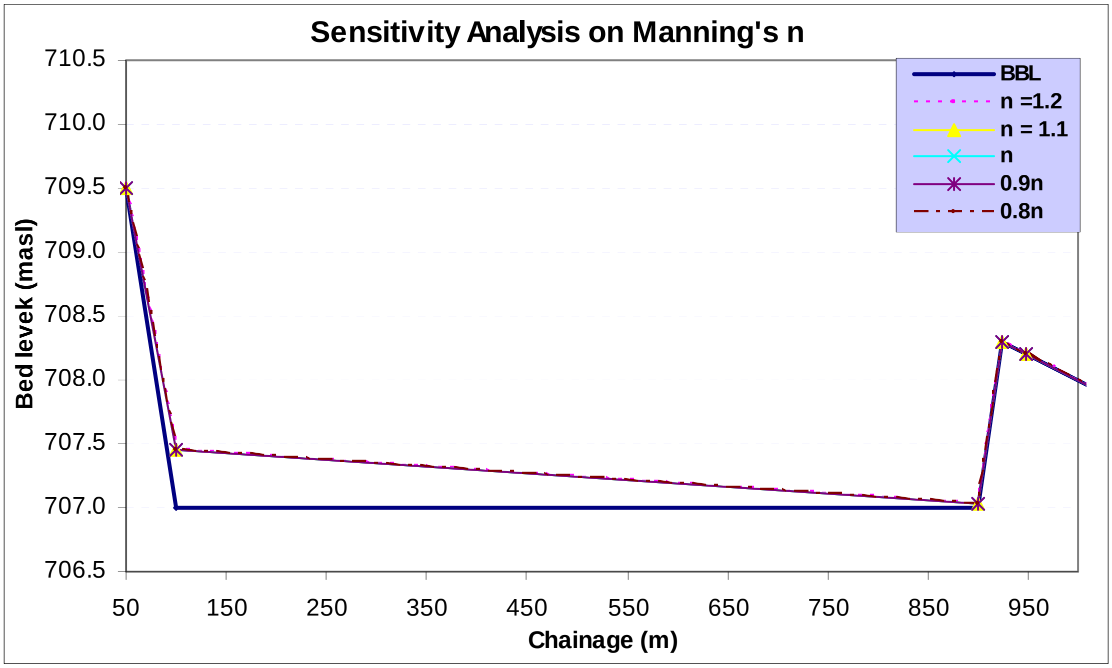

Figure 76. Sensitivity test on Manning's n

Summary of the Sensitivity Tests

The main findings of the sensitivity analysis are:

- The basin bed level rise is sensitive to hydrograph shapes: the hydrographs of the Fokisa scheme yield higher sediment level rises and volumes than the other spate hydrographs.
- The bed level rise and volume of depositions are highly sensitive to the sequence of the flood events.
- Given the sediment concentration constant, bed level rise increases with an increase to sediment size.
- The bed level rise is high for higher concentrations and low for lower concentrations.
- The bed level rise is much lower for Westrich- Jerushek equation and, results vary depending on the equations used.
- Bed level rise increases with an increase to the Manning's roughness coefficient n.

This type of analysis is very important in data scarce environments and can help ascertain the level of sensitivity of the results to their inputs. Therefore, data collection efforts or data base systems should be improved and designed scheme should be well monitored for the future.

6. AREAS of UNCERTAINTY, LIMITATIONS AND ASSUMPTIONS

This chapter deals with areas of uncertainties, limitations of the research, and assumptions made. It also discusses on future research areas of spate irrigation.

6.1 Areas of Uncertainties

Hydrodynamic models, which require significant input data, and which contain many sediment transport predicting equations also contain many uncertainties. To assess the uncertainties associated with hydrodynamic models and sediment transport predicting equations, research has been done. The performance of sediment transport functions has been reviewed by ASCE (1975) and by White (1973), cited by (Lawrence, 1987). The study results showed that the mean ratio of observed to predicted transport rate was between 0.5 and 2 for only 64 percent of the comparisons for the best method that was tested.

Furthermore, the sediment transport predicting equations of the Ackers and White, Brownlie, Engelund and Hansen, and Yang methods have been evaluated and compared to field and laboratory data, in which the flow conditions and sediment characteristics are similar to those prevailing in irrigation canals. The results indicated that, a prediction of the sediment transport in irrigation canals with an error factor smaller than 2 is often impossible, even in the case of the most reliable method, only 61 % of the measured values are predicted with an error factor of 2 (Mendez, 2005). For error factors less than 2, the predictability of the method is considerably less. Prediction of wadi sediment transport rates are even more uncertain than indicated by White (1973). Because of wide bed material sediment size grading, unpredictable flows and high Froude numbers (Lawrence, 1987).

ISIS sediments model may not be stable with extreme changes such as low flows or abrupt topographical features such as sharp slope changes in the basin. Therefore, efforts have been made to get the best results out of this model. Techniques such as constructing a separate canal model having similar hydraulic properties as the canal in the whole river system model, and using smaller time steps during simulation periods were used. The model is more stable with smaller time steps.

Despite the deterministic nature of the mathematical models, the inputs and outputs are also subjected to many uncertainties due to:

- Variability of the input and out put parameters in time and space,
- Associated human errors and due to clack of good understanding of the involved processes,
- The limitations both in input data and out put data to validate.

Therefore, the results of the simulation in models may prove efficient, the real performance of the system as actually operated may not be. Therefore, the model results have to be supported by physical assessments, to suggest improvements for the existing sediment control and management practices.

6.2 Limitations

This study has been conducted with some limitations. To further improve the results of this research and make it usable for future studies and designs it is worth discussing some of the limitations. The limitations have been categorized in to input data and other researches and references on spate irrigation

6.2.1 Input Data

- **Hydrological data**: For the Fokisa river, there is no hydrological station. All the discharge data and flow hydrograph used has been generated as a result of interviews held with the farmers from the area and neighboring rainfall station. Hydraulic calibration has been done with limited observed data that is collected during the interview of the farmers. Therefore, the reliability of the results could have increased if long series discharge data of the river were available.

- **Sediment concentration data**: similar to the hydrological data, sediment concentration data are not available both for the river and canal. Wash load data has been taken in the canal but it was taken only at low flows, (discharges not more than 60 l/s) and it is not possible to use it for larger flows. Hence, wash load data has been taken from the Wadi Laba river, and sand concentration data has been generated from DORC using sediment transport predicting equations. The predicted values have not been validated or compared with measured values from field. Therefore, reliability of the results could have increased if sediment concentration data of the river was available.

- **Sediment fall velocity data**: There is no data about the sediment fall velocity measurements of the scheme. Fall velocity data have been used from measurements made at the Gezira scheme in Sudan from the DOSSBAS demonstration data. Therefore, reliability of the results could have increased if fall velocity data of the river was used.

6.2.2 Limited Researches and References on Spate Irrigation

Though Spate irrigation has long been practiced, it is not well covered by scientific research. Spate irrigation has many features that need researching. Though not as required both in quantity and quality, much research is now being undertaking on spate irrigation. Therefore, the limitation of scientific research and references on spate irrigation can be stated as one of the research limitations. This is mainly because, some of the computed values and results can not be compared with that of other research findings.

6.3 Assumptions

During the simulations of the modeling, many assumptions have been made.

- Though part of the main canal (450 meters long) is lined, the whole canal system is assumed to be a hard bed that does not erode. Erosion and consolidation of the canal bed material is not included in the model, which is not the real situation. However, erosion of the deposited sediments is included.

- To avoid boundary condition issues of the ISIS model, the basin has been located at 50 meters away from the off take and it could have been more.

- In DOSSBAS model, there is no option to use different side slopes with in a basin, and when the canal sediment trapping performance is estimated, the whole canal section is assumed to have a side slope of 1:1, however, the lined canal has 0 side slope.

- In DOSSBAS model, the geometry of the sediment settling basin has been computed using the deposition option model. In this case the basin is to be dredged mechanically.

- Particle size distribution analysis has been carried out for both the canal and the river, which is an input to both models, has been selected after plotting them on the same scale. Therefore, the selected particle sizes are assumed to be representative sizes for all the scenarios.

- The bed load estimations have been made using the Engelund and Hansen equation and sand concentration data estimations have been made based on the generated sediment rating curve. Therefore, the curve $\mathbf{C = 2113Q^{0.62}}$, is assumed to represent the sand concentration of the river.

- A yearly series hydrograph has been developed based on interview held with farmers and rainfall data from a neighboring meteorological station. The yearly long series data has been developed based on the rainfall data sequence. Therefore, it is assumed that, the sequence of the rainfall from the nearest station is the same as the flow sequence of the river.

- The yearly long flow series is assumed to end at time equal to 184 hr. This is based on the summation of the durations of the individual events (summation of small flow + medium flow + large flow durations).

- To develop flash flood hydrographs similar to other spate schemes, a shape factor has been taken from the Wadi Laba hydrograph with some adjustments to fit the duration and peak flows to this study. This is hypothetical and it is developed to check the sensitivity of the results to hydrograph shapes.

- The seasonal dredged volume of sediment in the canal is computed assuming that the bed level rise at the downstream main canal is 1.0m.

- Bed level elevation data of the canal system and the basin were reduced by 1000 m in both the ISIS and DOSSBAS models for simplicity reasons during exporting data in to spreadsheets.

6.4 Future Research

The final goal of researches in spate irrigation is to improve the livelihood of the spate irrigators. Therefore simultaneously with research on sedimentation and hydraulics of spate irrigation systems, research focusing on agricultural improvements should be included. Research should address:

- Utilizing large flows that will not be diverted to the irrigated fields, for aforestation, and productivity of fodders for livestock production.
- Improving the yields of drought resistant spate-irrigated crops
- Maximizing the productivity of schemes and
- The effect of climate change on spate irrigation / is spate irrigation adaptive to climate change?

7. CONCLUSION AND RECOMMENDATIONS

This chapter deals with the conclusions drawn as a result of the research findings and further proposes suggestions.

7.1 Conclusions

After evaluating and analyzing the existing sediment control and management practices of the scheme, and testing proposed alternatives, the following conclusions were drawn:

- For the existing canal system of the scheme, the simulation result of the medium and small flows indicate that, coarse and fine sediments are diverted to the irrigated field, which the coarse sediments are not desired as they contribute to the command area bed level rise. However, the large flow and yearly long series flow simulation results reveal that the scheme traps 13.3% of the coarse sediments with a respective bed level rise of 0.4m and 0.8m. Since the canal has a depth of 1.5m bed level rise of 27 % and 53% can be considered as significant. Though trapping of coarser sediments is important, the bed level rises have a negative effect, in that it reduces the diverting capacity of the scheme. Therefore, the existing sediment management practice of the scheme is not effective.

For the Fokisa scheme, four options of sediment control and management systems have been tested and analyzed. These are sand trap basin, modifying earthen canal sections and slope, avoiding large flows using deflector weir and lengthening the main canal. The conclusions drawn from the analysis is presented below.

- From DOSSBAS simulation results for the Fokisa scheme, modified canal cross sections and slopes shows that without changing the slope of the canal route, a sand trapping efficiency of 26.1 % for medium flow and 15.1 % for small flow can be achieved. If a change to the slope of the canal route is integrated with canal dimension modifications, a sand trap efficiency of 28.7 % for medium flow and 15.6 % for small flow can be achieved. Therefore this could be easily implemented at Fokisa scheme where proposed changes are at earth lined canals.

- As indicated from the ISIS sediment simulations of large and yearly long series flows, the scheme performance is poor both with and without settling basin cases, if large flows are diverted towards the irrigated fields. Hence a deflector weir is proposed at the upstream of the off take. Results indicate that, the performance of the scheme could be improved if the diversion of large flows towards the irrigated field is avoided, by providing deflector weirs.

- From the particle size analysis, it is observed that, D_{50} of the particles decreases from 1.05 mm at the off take to 0.01mm at the irrigated field. In addition, the maximum diameter of a trapped particle at the off take is 19 mm and 4.75 mm at the irrigated field. The textural distribution of sediments at the off take shows that, 16.3 % of the particles are gravel, 82.7 % are sand and 0.9 % is silt and clay. Similarly, the distribution at the irrigated field shows that 11.3% of the sediments are sand, 88.7 % are silt and clay and no gravel. Though long main canals might have implications on water distributions system of the scheme, the above results prove that long canals are very helpful in trapping most of the coarser sediments.

- A well designed sand trap basin would trap most of the coarse sediments and allow the diversion of fine sediments towards the irrigated fields. A basin performs differently at different conditions. Firstly its performance highly depends on the sequence of flood events, the performance reducing if large floods are set at the beginning of a flood sequence. Secondly, its trapping efficiency is low if a large flood event or two medium events are diverted and it fills up early in the season. Thirdly, its performance also varies with variation to hydrograph shapes. Finally, the size of the basin may become prohibitively large for economic justification. For spate schemes, where the flows are unpredictable, designing a sand trap basin can be complex; therefore, the applicability of settling basins for spate schemes is questionable.

- From the above conclusions, a combination of lengthening the main canal, modifying the canal dimensions and longitudinal slope, and providing a deflector/ rejection weir at the upstream of the off take would be the best alternative to improve the scheme sediment control and management.

- To compare the results from the two models, DOSSBAS model being a steady state model, ISIS sediments is run at steady state. Though the comparisons are based on limited data, where model results are not validated, the bed level rise and the volume of trapped sediments for ISIS model are lower than that of DOSSBAS. However, DOSSBAS model is a useful tool and can be used for the initial stages of design for indicative sizing.

- Sensitivity test results have been carried out on many scenarios and input data changes. The results show that, the results are sensitive to hydrograph shape changes, sequence of flow events, input sediment sizes, input sediment concentrations, selected sediment transport formula and Manning's roughness coefficient n. Therefore, care must be taken during data collection and analysis. Furthermore, data collection efforts or data base systems should be improved and designed schemes should be well monitored for the future.

7.2 Recommendations

To improve the results of this study and other future studies, the following recommendations are suggested.

- To improve the sediment control and management of spate schemes, a combination of deflector weir, modifying the canal dimensions and slope, and lengthening the main canals could be proposed. Modifying the canal dimensions and slope and lengthening the main canals can easily be tested on earthen canals and should be tested on the ground at a test site.

- The study has been conducted with limited data, where the results of the modeling are not validated. To further strengthen the out put of this thesis, further studies, where model results can be validated, is recommended. Therefore, more field data in the following fields are necessary:
    1. Long term flow hydrographs
    2. Particle size distribution
    3. Sand and silt concentrations
    4. Sediment fall velocity
    5. Manning's roughness coefficient, n

- DOSSBAS model has unsteady state simulation options, however it treats the simulations as a series of steady flows. It was not possible to test it due to time constraints. Therefore, the performance of the scheme should be tested with the unsteady state option in DOSSBAS. DOSSBAS is freely available model and hence it can be proved to be by designers in poor countries.

- The basin sediment trapping efficiency varies with changes to the sequence of flooding event and hydrograph shapes. Therefore, care must be taken during hydrological data collection and analysis if studies and designs of settling basins are to be carried out.

- During the data collection period, very little data were found. Therefore, a data base system of all the schemes would improve data management. This data base should collect all data on schemes such as:

    1. Design and study documents of the schemes
    2. Auto Cad Drawing of the designs and topographic maps
    3. Operational and maintenance reports of the schemes
    4. Monitoring reports of the schemes
    5. Hydrological and Sediment concentration data and so on.

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Discussion with Dr. Abraham Haile Mehari

ANNEX-A. ISIS sediments and DOSSBAS Steady Results

Deposition Model - Setup Irregular Geometry

	Distance U/S (m)	Bed Elevation (m)	Bed Width (m)
1	0.0	708.300	2.34
2	20.0	707.000	1.00
3	50.0	707.000	1.00
4	120.0	707.000	1.00
5	440.0	707.000	1.00
6	540.0	707.000	1.00
7	860.0	707.000	1.00
8	875.0	709.500	2.34

Figure 77. Geometrical set up of the basin from DOSSBAS steady

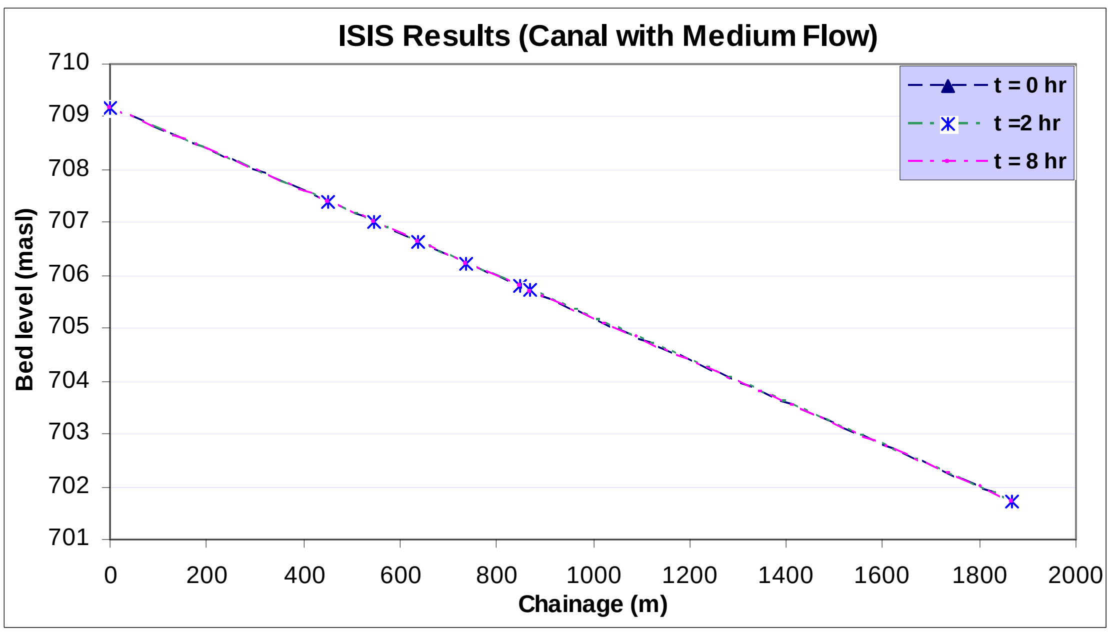


Figure 78. ISIS unsteady simulation result for an existing scheme with medium flow

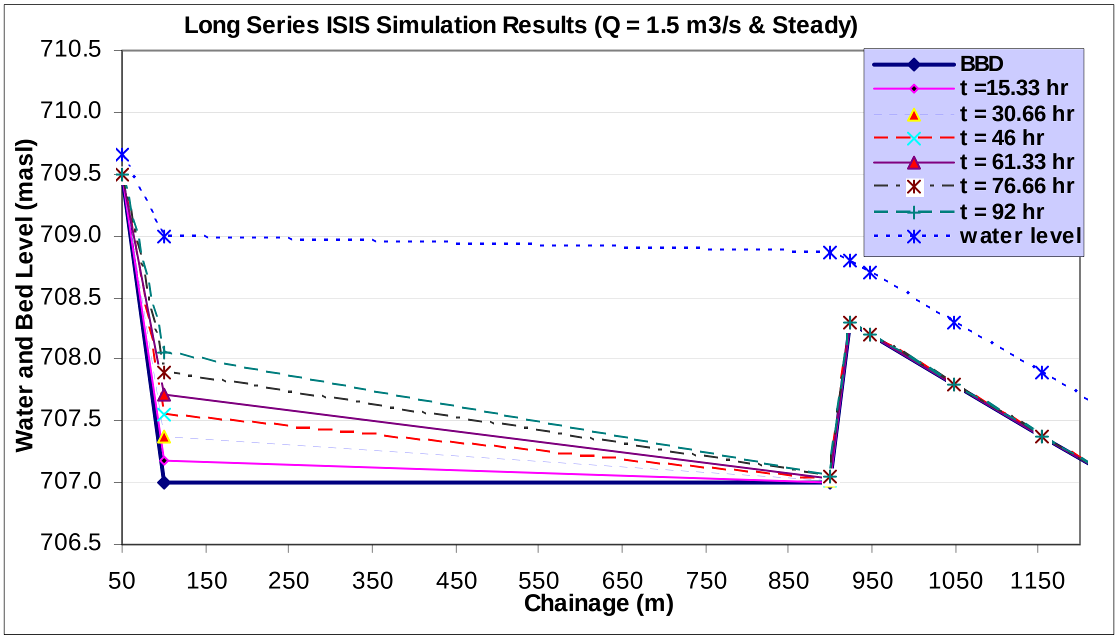


Figure 79. Long series steady simulation results from ISIS model for Q =1.5 m^3/s

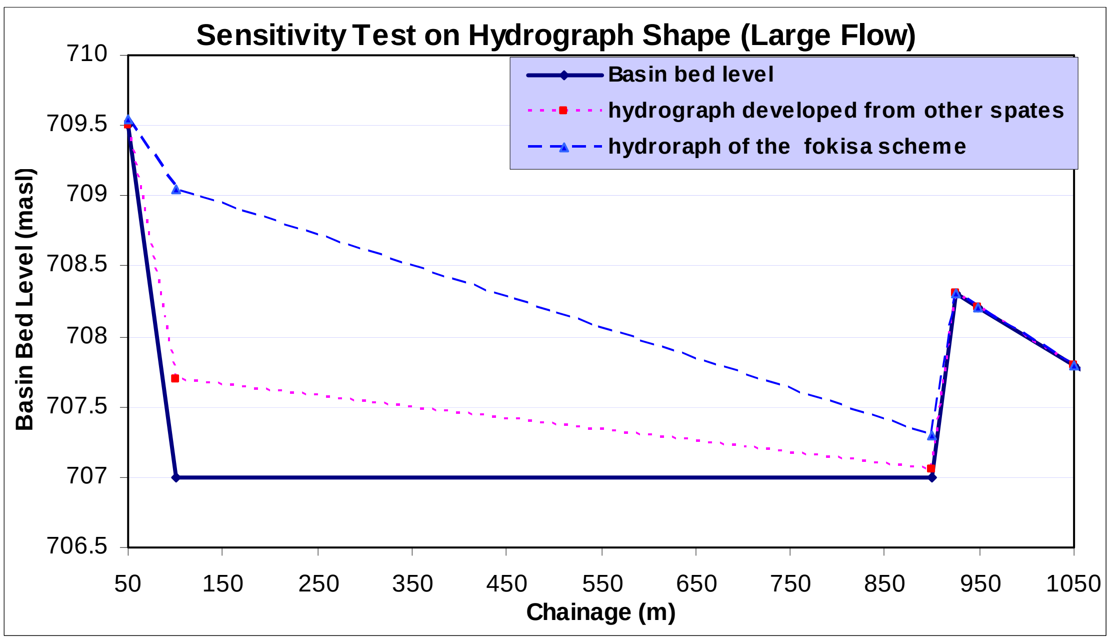


Figure 80. Sensitivity test results on changes to hydrograph shape changes (large flow)

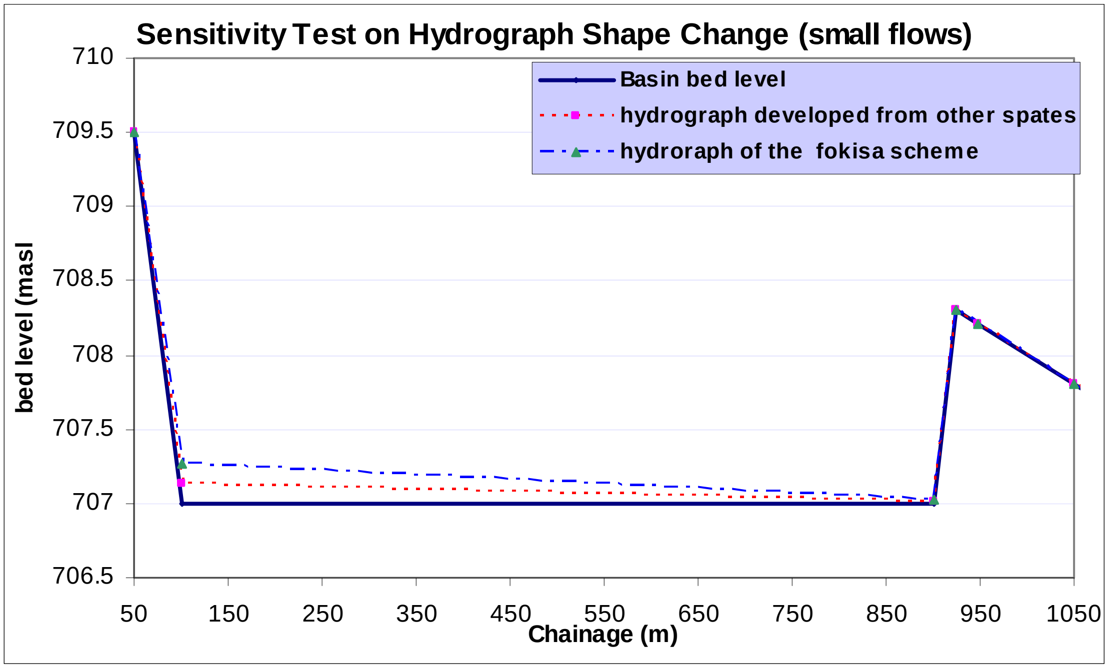


Figure 81. Sensitivity test results on changes to hydrograph shape changes (small flow)

ANNEX-B. Summary of calibration results (water levels)

Table 12. Summary of water level results (calibration)

Label	Chainage (m)	Medium flow levels (masl)	
		Observed values	Simulated values
S8	343	713.2	713.1
S14UP	586	711.5	711.4
S30	1821	687.9	688.0
A2DN	1822	710.7	710.7
A13	2770	706.9	706.9

Table 13. Separate canal and whole river model comparison results

Label	Chainage (m)	HFC (masl)	HF (masl)	MFC (masl)	MF (masl)	LFC (masl)	LF (masl)
A2	0	711.11	711.11	710.74	710.73	710.34	710.26
A3	23	711	711	710.65	710.65	710.23	710.18
A4	129.5	710.57	710.57	710.22	710.22	709.79	709.81
A5	237.1	710.14	710.14	709.79	709.79	709.4	709.41
A6	324.3	709.79	709.79	709.43	709.43	709.06	709.07
A7	425.3	709.35	709.35	708.92	708.92	708.67	708.68
A8	530.1	708.89	708.89	708.56	708.56	708.32	708.32
A9	626.7	708.5	708.5	708.17	708.17	707.92	707.91
A10	718.9	708.13	708.13	707.8	707.8	707.55	707.54
A11	818.9	707.73	707.73	707.41	707.41	707.14	707.15
A12	927.1	707.24	707.24	706.9	706.92	706.69	706.73
A13	949.1	707.17	707.18	706.83	706.86	706.61	706.65
A14	1949.1	703.15	703.15	702.81	702.79	702.57	702.55

Remark

HFC = Water surface level of the canal at high flow
HF = Water surface level of the river at high flow
MFC = Water surface level of the canal at mean flow
MC = Water surface level of the river at mean flow
LFC = Water surface level of the canal at low flow
LC = Water surface level of the river at low flow

ANNEX-C Summary of water surface slope and velocity change computation results

Table 14. Summary of the water surface slope and velocity change computations for the Fokisa scheme hydrograph

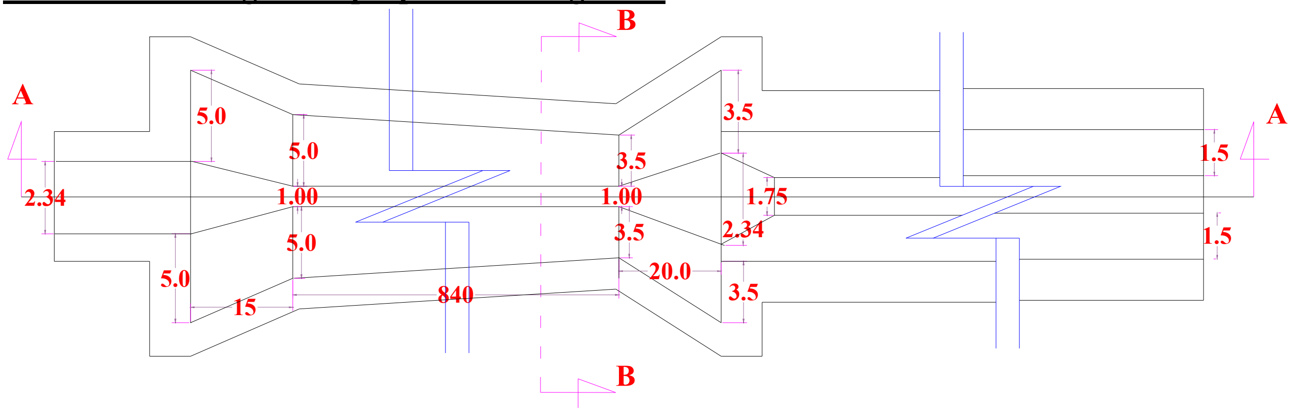
Label	Basin Chainage (m)	Water surface slope (m/m)							VELOCITY (m/s)				
		0 hr	2 hr	2.5 hr	3.8 hr	7.5 hr	7.92 hr	8 hr	0 hr	2hr	7.5 hr	7.92 hr	8 hr
A2	0								6.13	6.28	5.48	5.20	5.18
A3	50	0.021	0.019	0.019	0.019	0.021	0.021	0.021	8.18	4.77	5.74	5.95	5.97
A4	100	0.015	0.008	0.008	0.008	0.012	0.013	0.015	0.29	0.81	0.64	0.57	0.59
A5	900	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.29	0.73	0.37	0.32	0.32
A6	925	0.003	0.007	0.007	0.006	0.004	0.003	0.003	1.22	1.85	1.31	1.23	1.22

Table 15. Summary of the water surface slope and velocity change computations for the hydrograph from other spate schemes

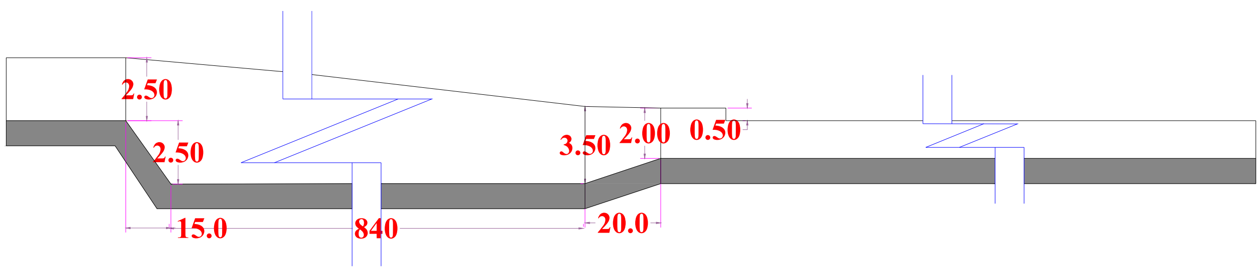
Label	Basin Chainage (m)	Water surface slope (m/m)							VELOCITY (m/s)						
		0 hr	2 hr	2.5 hr	3.8 hr	7.5 hr	7.92 hr	8 hr	0 hr	2 hr	2.5 hr	3.8 hr	7.5 hr	7.92 hr	8 hr
A2	0								6.1	6.1	6.0	4.8	4.8	4.8	4.8
A3	50	0.021	0.021	0.02	0.021	0.021	0.021	0.021	8.2	5.4	5.0	6.3	6.3	6.3	6.3
A4	100	0.015	0.011	0.009	0.015	0.015	0.014	0.014	0.3	0.8	0.3	0.3	0.3	0.3	0.4
A5	900	2E-05	1E-04	3E-05	2E-05	3E-05	3.1E-05	3E-05	0.3	0.6	0.5	0.3	0.3	0.3	0.3
A6	925	0.003	0.006	0.0046	0.0036	0.0036	0.00358	0.004	1.2	1.8	1.5	1.2	1.2	1.2	1.2

ANNEX-D. Auto Cad drawings of the proposed settling basin and canal dimension modifications

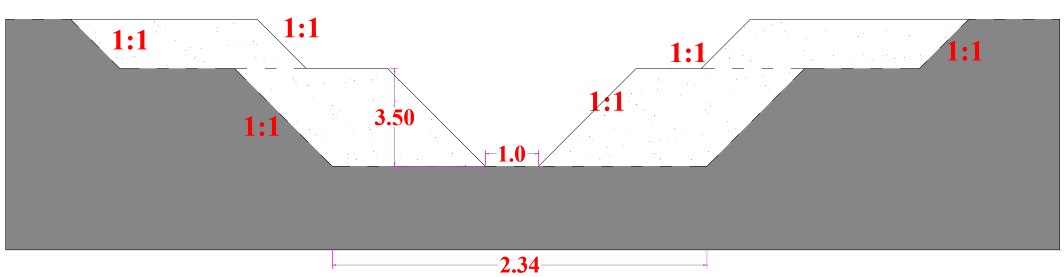
Auto Cad drawing of the proposed settling basin



PLAN VIEW OF THE PROPOSED SETTLING BASIN

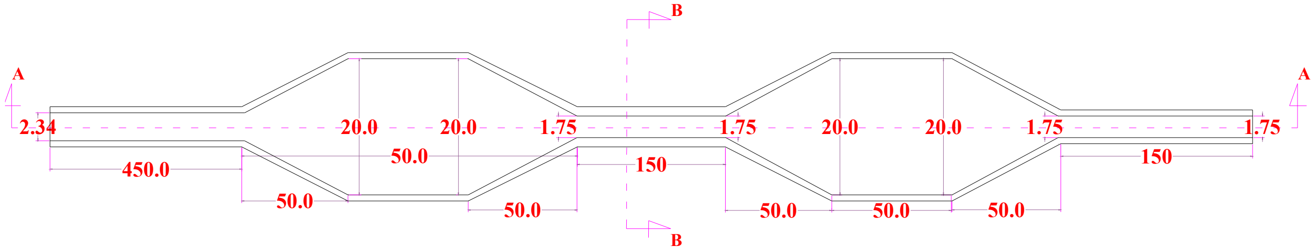


SECTION A-A

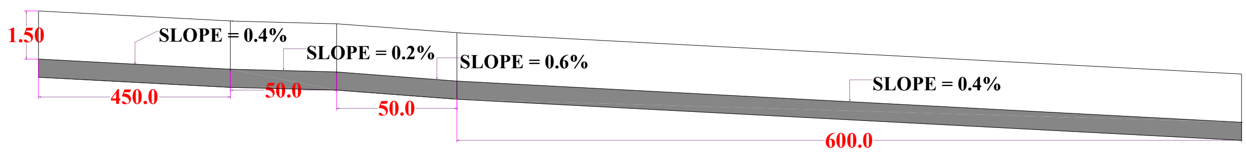


SECTION B-B

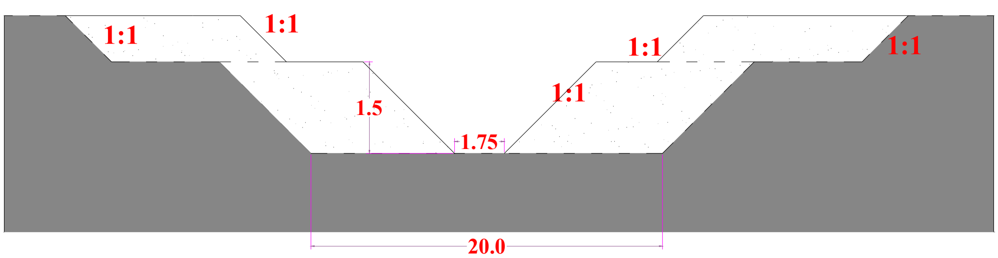
Auto Cad drawing of the proposed modifications on canal dimensions and slope



PLAN VIEW OF THE PROPOSED CANAL DIMENSION AND MODIFICATIONS

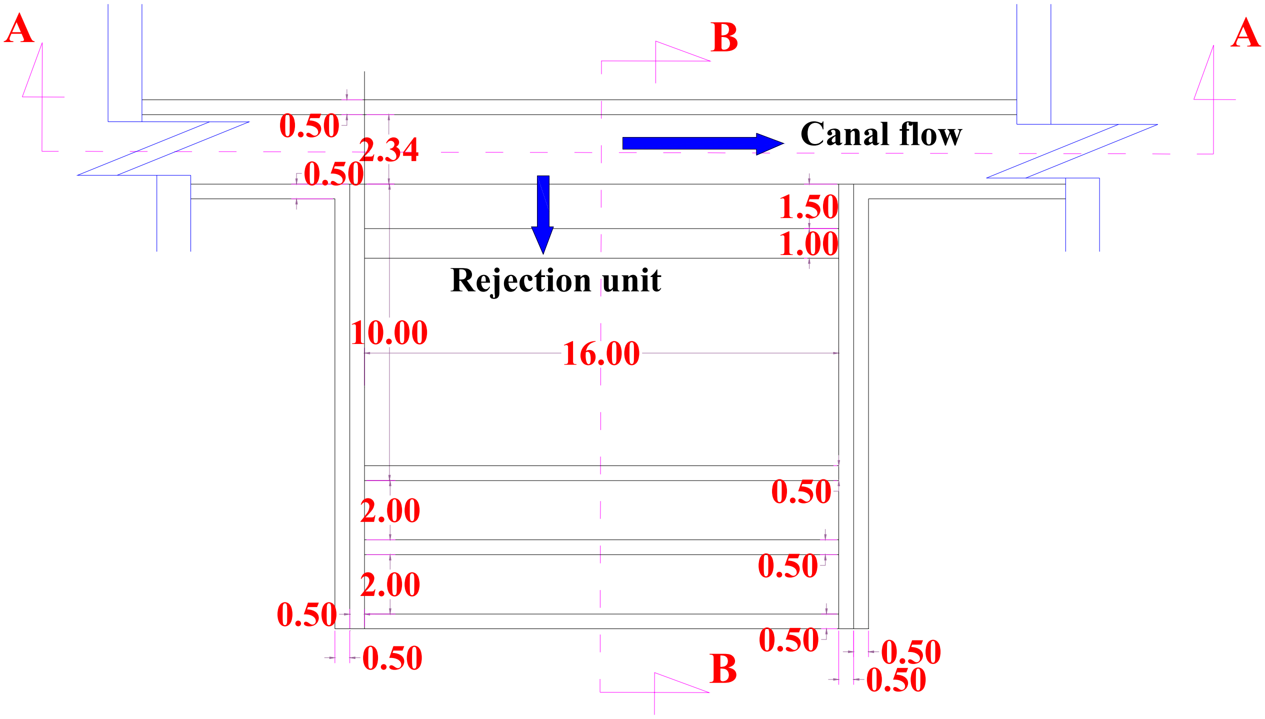


SECTION A-A

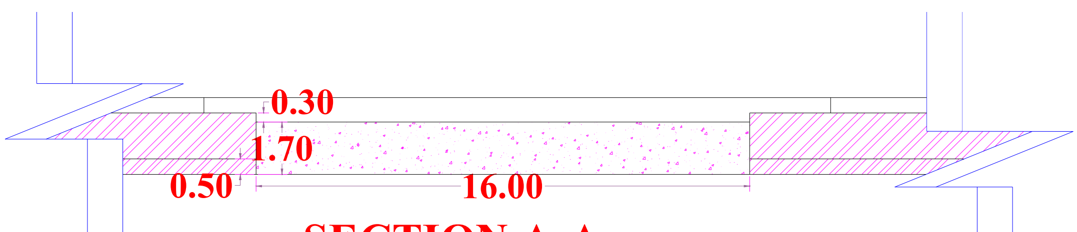


SECTION B-B

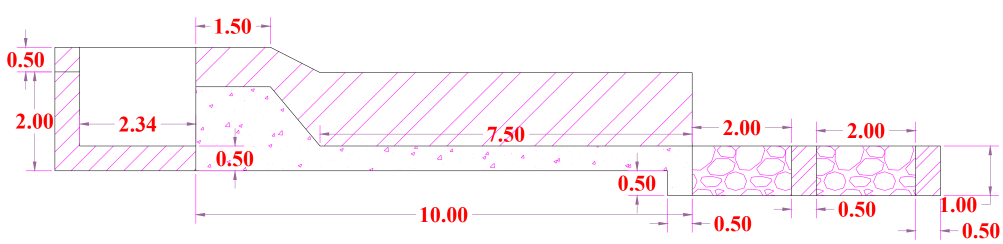
Auto Cad drawing of the proposed deflector weir



PLAN VIEW OF THE PROPOSED DEFLECTOR WEIR



SECTION A-A



SECTION B-B

ANNEX-E. Particle size distribution data

Particle size distribution data of the river route

Table 16. Particle size distribution data of the river route

	Percent Passing					
Sieve size	RB1	RB2	RB3	RB6	RB7	RB8
75.00	100.0 %	100.0%	100.0 %	100.0 %	100.0 %	100.0 %
37.50	100.0 %	86.0%	100.0 %	100.0 %	100.0 %	100.0 %
19.00	100.0 %	82.4%	97.2 %	71.0 %	100.0 %	100.0 %
9.50	90.5 %	73.5%	95.5 %	48.10%	93.7 %	92.3 %
4.75	82.7 %	60.4%	80.6 %	36.0 %	85.1 %	86.9 %
2.00	66.8 %	44.1%	76.2 %	27.8 %	66.4 %	79.5 %
0.85	37.4 %	23.6%	38.7 %	21.0 %	34.3 %	66.2 %
0.43	19.6 %	13.7%	11.3 %	14.6 %	12.0 %	50.4 %
0.25	12.1 %	7.6%	4.4 %	11.0 %	4.8 %	27.4 %
0.15	4.3 %	3.4%	2.5 %	9.0 %	2.0 %	8.4 %
0.08	2.1 %	1.6%	1.8 %	7.7 %	0.9 %	2.7 %
0.001	0.0 %	0.0 %	0.0 %	0.0 %	0.0 %	0.0 %

Particle size distribution data of the canal route

Table 17. Particle size distribution data of the canal system

	Percent Passing									
Sieve Size (mm)	MC 1	MC2	MC 3	MC 4	MC 5	MC 6	SC1	SC2	SC3	Irrigated Field
75.00	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
37.50	100%	100%	100%	100%	100%	100%	100%	100%	100%	100%
19.00	96%	99%	97%	100%	100%	100%	100%	100%	100%	100%
9.50	92%	98%	95%	98%	98%	100%	100%	100%	100%	100%
4.75	84%	89%	87%	95%	95%	99%	98%	100%	99%	100%
2.00	67%	67%	68%	86%	88%	96%	90%	97%	98%	100%
0.85	40%	37%	41%	58%	74%	85%	75%	83%	94%	99%
0.43	19%	15%	17%	23%	43%	54%	49%	51%	92%	98%
0.25	9%	6%	4%	7%	15%	25%	15%	20%	69%	98%
0.15	2%	3%	1%	3%	4%	8%	3%	10%	42%	96%
0.08	1%	1%	0%	1%	1%	3%	1%	2%	8%	89%
0.001	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%

ANNEX-F. Hydrograph Data

Canal flow hydrograph data developed as a result of interview with farmers

Table 18. Small and medium flow hydrographs developed as a result of interview with farmers

Time (hr)	Small flood (m^3/s)	Medium flood (m^3/s)
0	0	0
2	4.35	6.8
8	0	0

Table 19. Large flow hydrograph developed as a result of interview with farmers

Time (hr)	**Large flood (m^3/s)**
0	0
4	11.4
24	0

Canal flow hydrograph data developed from other spate schemes

Table 20. Small flow hydrograph developed from other spate schemes

Time (hr)	**Small Flood (m^3/s)**
0	1.5
1.5	1.5
2	4.35
2.5	1.75
3	1.5
3.5	1.5
4	1.5
5	1.5
6	1.5
7	1.5
8	1.5

Table 21. Medium flow hydrograph developed from other spate schemes

Time (hr)	**Medium Flood (m^3/s)**
0	1.5
1.5	1.5
2	6.8
2.5	2.1
3.5	1.5
4.25	1.5
4.75	1.5
5.5	1.5
6	1.5
7	1.5
8	1.5

Table 22. Large flow hydrograph developed from other spate schemes

Time (hr)	Large Flood (m^3/s)
0	1.5
3	1.5
4	11.4
6	4
8	2
9.6	1.5
12	1.5
14.4	1.5
16.8	1.5
19.2	1.5
21.6	1.5
24	1.5

ANNEX-G. Summarized topographic surveying data of river cross sections

Topographic surveying data collected during design period (Dec, 2004)

Remarks
X = Latitude
Y = Longitude
Z = Elevation (masl)
S = Relative distance (m)

X-1 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569653.4	1408485	1717.872				0.0
569650.1	1408483	1712.733	11.1	5.7	4.1	4.1
569644.1	1408482	1712.512	35.5	0.2	6.0	10.1
569639.7	1408480	1712.310	19.6	3.0	4.8	14.8
569630.2	1408477	1721.411	89.2	10.7	10.0	24.8

X-2 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569682.2	1408488	1722.100				0.0
569681.5	1408481	1712.524	0.4	50.5	7.1	7.1
569682.8	1408471	1711.914	1.6	100.2	10.1	17.2
569679.8	1408458	1712.329	9.0	164.7	13.2	30.4
569676.2	1408451	1722.956	13.2	46.5	7.7	38.1

X-3 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569731.3	1408485	1720.298				0.0
569728.9	1408468	1711.350	6.0	287.2	17.1	17.1
569726.9	1408458	1711.534	3.9	100.3	10.2	27.3
569722.1	1408444	1711.878	22.9	196.4	14.8	42.1
569722.8	1408433	1720.044	0.5	123.1	11.1	53.3
569714	1408421	1726.473	78.0	153.3	15.2	68.5

X-4 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569782	1408477	1721.450				0
569785.8	1408474	1720.383	14.4	9.8	4.9	4.9
569778.7	1408458	1711.060	50.6	271.7	18.0	22.9
569776.7	1408446	1711.091	3.9	138.5	11.9	34.8
569775.7	1408435	1711.319	0.9	123.6	11.2	46.0
569774.9	1408429	1719.017	0.6	33.2	5.8	51.8

X-5 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569828	1408465	1718.636				0
569826.5	1408452	1710.740	2.1	181.8	13.6	13.6
569824.6	1408441	1710.690	4.0	107.4	10.6	24.1
569822.9	1408428	1710.757	2.9	180.8	13.6	37.7
569819.8	1408414	1717.769	9.4	202.1	14.5	52.2

X-6 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569871	1408473	1715.269				0
569866.6	1408456	1714.578	18.8	275.4	17.2	17.2
569865.5	1408449	1710.621	1.4	46.4	6.9	24.1
569863.7	1408436	1709.500	3.0	170.3	13.2	37.2
569861.6	1408423	1710.353	4.6	183.7	13.7	51.0
569857.6	1408404	1724.683	15.9	348.2	19.1	70.0

X-7 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569926.4	1408471	1713.568				0
569924.1	1408455	1712.753	4.9	270.7	16.6	16.6
569922.7	1408449	1709.500	2.1	34.1	6.0	22.6
569918.4	1408426	1711.300	18.2	524.0	23.3	45.9
569913.9	1408414	1709.782	20.3	153.2	13.2	59.1
569910.7	1408402	1717.190	10.0	129.3	11.8	70.9

X-8 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569926.4	1408471	1713.568				0
569960.1	1408444	1710.124	1141.2	729.1	43.2	43.2
569960.1	1408442	1709.379	0.0	3.2	1.8	45.0
569959.8	1408438	1709.030	0.1	22.2	4.7	49.7
569959.1	1408435	1709.852	0.5	5.9	2.5	52.3
569954.2	1408418	1709.209	24.4	297.0	17.9	70.2
569951.4	1408398	1709.116	7.9	418.4	20.6	90.8
569949.8	1408390	1709.738	2.6	55.4	7.6	98.4
569945.9	1408367	1711.968	15.2	511.7	23.0	121.4

X-9 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570005	1408428	1713.396				0
570003.3	1408420	1712.516	2.9	62.7	8.1	8.1
570002.7	1408416	1708.587	0.4	14.0	3.8	11.9
570000.6	1408411	1708.485	4.6	22.0	5.2	17.0
569999.8	1408410	1709.170	0.5	1.3	1.4	18.4
569996.9	1408403	1707.190	8.8	48.2	7.6	26.0
569991.3	1408394	1708.545	30.7	79.9	10.5	36.5
569993.2	1408393	1707.021	3.5	1.2	2.2	38.6
569986.1	1408384	1707.527	50.7	92.3	12.0	50.6
569980.1	1408373	1712.364	35.2	112.3	12.1	62.7

X-10 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570044.4	1408393	1715.225				0
570043.8	1408389	1709.020	0.3	16.7	4.1	4.1
570043.5	1408386	1708.899	0.1	14.5	3.8	7.9
570042.2	1408383	1710.886	1.7	8.7	3.2	11.2
570038.2	1408377	1707.475	15.8	28.9	6.7	17.9
570037.3	1408374	1705.668	0.8	7.9	3.0	20.8
570029.6	1408371	1706.640	59.2	13.5	8.5	29.3
570027.9	1408367	1704.977	2.9	10.7	3.7	33.0
570023	1408364	1706.524	24.6	12.4	6.1	39.1
570012.9	1408354	1709.520	100.5	96.8	14.0	53.2

X-11 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570072	1408376	1710.673				0
570072.2	1408374	1708.935	0.07	5.72	2.41	2.41
570071.5	1408372	1709.409	0.53	4.26	2.19	4.59
570070.8	1408370	1710.240	0.48	2.62	1.76	6.36
570069.5	1408366	1709.863	1.70	15.95	4.20	10.56
570068.2	1408363	1707.718	1.72	9.55	3.36	13.91
570066.5	1408361	1707.320	2.87	3.47	2.52	16.43
570064.5	1408358	1705.115	4.17	12.78	4.12	20.55
570058.4	1408346	1704.115	37.19	131.79	13.00	33.55
570058.8	1408343	1706.527	0.23	8.25	2.91	36.46
570053.6	1408334	1708.730	27.25	96.83	11.14	47.60

X-12 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570099.3	1408366	1708.936				0
570098.7	1408365	1708.689	0.4	0.9	1.1	1.1
570097.3	1408362	1708.640	1.9	8.6	3.2	4.4
570095.6	1408359	1710.265	3.0	6.5	3.1	7.5
570093.3	1408353	1706.777	5.1	41.1	6.8	14.3
570089.3	1408346	1705.639	16.0	48.8	8.1	22.3
570088	1408344	1703.766	1.8	5.8	2.8	25.1
570083.2	1408332	1704.001	23.3	139.5	12.8	37.8
570081.6	1408329	1706.630	2.4	6.5	3.0	40.8

X-13 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570150.9	1408340	1708.807				0
570149.1	1408335	1708.809	3.3	32.1	5.9	5.9
570147.4	1408332	1710.200	2.9	5.4	2.9	8.8
570145.7	1408330	1709.631	3.2	7.7	3.3	12.1
570141.5	1408321	1704.866	17.4	72.8	9.5	21.6
570138.2	1408318	1703.449	10.7	12.4	4.8	26.4
570131.6	1408308	1703.615	44.4	90.7	11.6	38.0
570130.2	1408302	1706.324	1.9	33.1	5.9	44.0

X-14 design data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570181	1408316	1709.056				0
570179.7	1408314	1708.715	1.6	2.2	1.9	1.9
570177.1	1408311	1708.587	6.8	9.1	4.0	5.9
570175.2	1408309	1709.719	3.6	3.8	2.7	8.7
570171.6	1408299	1704.488	13.0	100.4	10.6	19.3
570170.4	1408297	1702.949	1.6	3.8	2.3	21.6
570163.6	1408290	1703.151	46.1	54.6	10.0	31.6
570162.5	1408288	1704.162	1.1	4.2	2.3	34.0

Topographic surveying data collected during thesis data collection period (Nov, 2008)

X-1 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569663.2	1408493	1719.511				0
569652.6	1408492	1718.076	112.0	0.2	10.6	10.6
569647.8	1408490	1713.244	23.0	4.6	5.3	15.8
569641.2	1408485	1713.064	43.8	22.8	8.2	24.0
569633.5	1408482	1713.625	58.2	12.3	8.4	32.4
569630.0	1408477	1720.418	12.4	23.7	6.0	38.4
569623.4	1408469	1724.693	44.3	59.6	10.2	48.6

X-2 data						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569685.2	1408495	1724.488				0
569684.8	1408485	1719.404	0.2	92.0	9.6	9.6
569684.5	1408479	1712.350	0.1	43.7	6.6	16.2
569684.1	1408469	1712.402	0.2	90.9	9.5	25.8
569680.2	1408456	1712.540	15.2	158.3	13.2	38.9
569677.3	1408450	1721.789	8.5	42.4	7.1	46.1
569676.5	1408444	1724.792	0.7	36.7	6.1	52.2

X-3 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569740.6	1408485	1720.374				0
569734.2	1408468	1711.751	41.1	289.3	18.2	18.2
569731.5	1408454	1712.115	7.4	194.4	14.2	32.4
569725.7	1408442	1711.891	33.6	146.7	13.4	45.8
569713.6	1408421	1725.699	145.2	454.2	24.5	70.3
569711.2	1408415	1727.212	5.8	36.4	6.5	76.8

X-4 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569789.3	1408477	1720.778				0
569784.4	1408448	1711.520	24.3	839.6	29.4	29.4
569778.4	1408434	1711.811	35.3	186.3	14.9	44.3
569775	1408428	1718.469	11.8	37.4	7.0	51.3
569775.1	1408428	1718.480	0.0	0.0	0.1	51.4
569772.1	1408418	1721.361	9.3	93.7	10.1	61.6

X-5 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569821.3	1408472	1719.382				0
569828.6	1408456	1713.463	53.6	254.1	17.5	17.5
569822.2	1408439	1711.260	41.0	311.1	18.8	36.3
569826.3	1408435	1711.943	16.3	11.9	5.3	41.6
569820.8	1408421	1712.156	30.3	217.1	15.7	57.3
569819.7	1408414	1716.591	1.1	50.7	7.2	64.5
569816.9	1408402	1722.008	7.8	134.0	11.9	76.4

X-6 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569857.6	1408465	1715.887				0
569858.1	1408457	1715.549	0.2	57.3	7.6	7.6
569858.5	1408450	1711.810	0.1	51.6	7.2	14.8
569856.8	1408437	1711.922	2.8	186.1	13.7	28.5
569866.4	1408433	1711.722	92.7	12.2	10.2	38.8
569857.7	1408425	1711.640	77.0	69.9	12.1	50.9
569857.1	1408398	1725.739	0.3	694.8	26.4	77.2
569857.6	1408404	1724.663	0.2	33.5	5.8	83.1
569856.5	1408398	1726.532	1.2	41.0	6.5	89.6
569857.7	1408404	1725.404	1.4	40.6	6.5	96.0

X-7 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569929.1	1408475	1714.317				0
569925.4	1408464	1712.898	14.2	117.6	11.5	11.5
569921.5	1408448	1711.668	15.2	260.3	16.6	28.1
569917.6	1408429	1711.299	15.3	380.1	19.9	48.0
569913.7	1408411	1711.310	14.7	302.3	17.8	65.8
569910.7	1408397	1716.932	9.4	212.0	14.9	80.6
569909	1408392	1716.824	2.7	24.9	5.3	85.9

X-8 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
569970.8	1408440	1712.278				0
569969.2	1408432	1713.206	2.4	65.7	8.3	8.3
569965.7	1408427	1708.722	12.7	24.1	6.1	14.3
569960.2	1408415	1708.266	29.4	155.6	13.6	27.9
569960.1	1408410	1708.742	0.0	23.8	4.9	32.8
569952.7	1408397	1708.714	55.6	168.1	15.0	47.8
569949.8	1408390	1709.590	8.2	45.0	7.3	55.1
569945.8	1408382	1710.511	15.9	66.1	9.1	64.1

X-9 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570009.6	1408424	1714.262				0
570003.6	1408415	1714.250	36.5	80.5	10.8	10.8
569999.5	1408417	1714.705	16.5	4.6	4.6	15.4
569999.3	1408406	1708.189	0.0	125.5	11.2	26.6
569991.7	1408393	1708.352	58.6	173.2	15.2	41.8
569996.2	1408392	1708.077	20.7	1.1	4.7	46.5
569984.7	1408384	1708.770	134.5	59.2	13.9	60.4
569987.8	1408376	1710.793	9.7	62.3	8.5	68.9
569980.1	1408373	1712.422	59.1	9.9	8.3	77.2
569976	1408362	1712.243	16.5	125.4	11.9	89.1

X-10 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570044.3	1408412	1716.621				0
570040.8	1408405	1717.459	12.4	51.9	8.0	8.0
570036.2	1408393	1709.319	20.6	132.9	12.4	20.4
570025.7	1408378	1706.783	110.4	221.9	18.2	38.6
570025.4	1408369	1706.181	0.1	93.3	9.7	48.3
570019.3	1408364	1706.988	36.9	16.7	7.3	55.6
570014.3	1408359	1709.739	24.6	28.7	7.3	62.9
570013.3	1408355	1709.824	1.0	15.6	4.1	67.0
570010.4	1408350	1709.377	8.3	28.5	6.1	73.1

X-11 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570071.8	1408367	1710.369				0
570071.8	1408367	1710.188	0.0	0.0	0.0	0.0
570075	1408363	1709.452	10.0	10.6	4.5	4.6
570061.6	1408350	1704.408	178.8	177.6	18.9	23.5
570064.9	1408350	1704.605	11.0	0.0	3.3	26.8
570062.6	1408344	1705.458	5.5	31.1	6.1	32.8

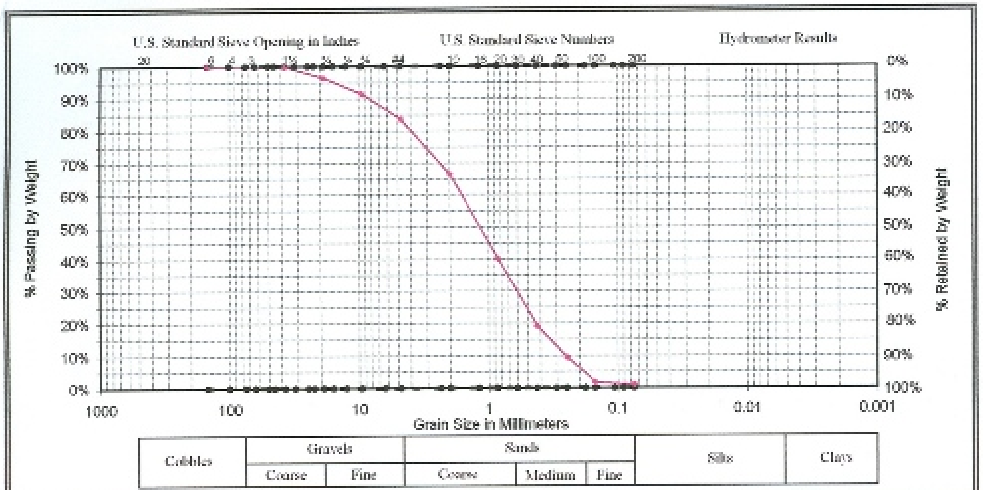
X-12 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570097.6	1408355	1708.281				0
570091.8	1408345	1705.316	33.5	105.5	11.8	11.8
570090.2	1408341	1703.757	2.7	16.6	4.4	16.2
570086.7	1408338	1704.063	12.2	12.0	4.9	21.1
570087.4	1408337	1703.572	0.6	0.3	1.0	22.1
570083.2	1408332	1703.847	18.0	25.7	6.6	28.7
570081.8	1408329	1706.279	2.0	10.0	3.5	32.1
570072.7	1408317	1706.751	82.8	149.2	15.2	47.4

X-13 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570146.5	1408329	1709.722				0
570144.8	1408325	1707.648	2.6	12.0	3.8	3.8
570140.6	1408318	1704.034	18.0	54.5	8.5	12.3
570133.9	1408304	1704.382	44.3	185.0	15.1	27.5
570136.4	1408312	1703.342	6.1	56.8	7.9	35.4
570133	1408307	1703.342	11.5	24.4	6.0	41.4
570130	1408302	1706.027	9.2	21.5	5.5	46.9
570124.2	1408293	1706.623	32.8	80.4	10.6	57.6

X-14 (Thesis data)						
X	Y	Z	$(X_2-X_1)^2$	$(Y_2-Y_1)^2$	$S=((X_2-X_1)^2+(Y_2-Y_1)^2)^{0.5}$	Chainage (m)
570171.7	1408300	1704.245				0
570170.6	1408298	1702.968	1.2	6.0	2.7	2.7
570166.7	1408292	1702.974	15.6	30.7	6.8	9.5
570164	1408289	1702.813	7.0	12.0	4.4	13.8
570162.9	1408287	1703.966	1.4	2.1	1.9	15.7
570157.5	1408280	1705.183	28.5	56.6	9.2	24.9

ANNEX-H. Selected textural and particle size distribution of the canal system and irrigated field.

Textural and particle size distribution at the off take



Date :	D_{10} = 0.26	USCS Classification	% Gravel: 16.3% / % Sand: 82.7%
Sample #: 1	D_{30} = 0.64	SP, Poorly graded Sand wi	
Sample ID:	D_{60} = 1.72	Specifications	
Source: FOKISA DIVERSION	C_C = 0.92	No Specs	
Project: DIVERSION	C_U = 6.57	Sample Meets Specs	% Silt & Clay
Location: OFF TAKE	Liquid Limit= n/a	n/a	0.9%
Boring #: TP-1	Plastic Limit= n/a	Fineness Modulus	
Depth: 0-30cm	Plasticity Index= n/a	3.70	

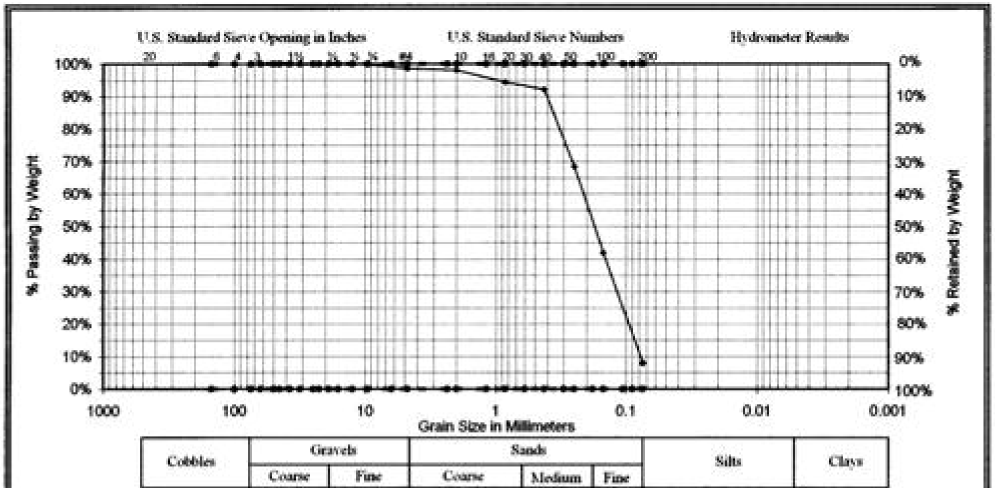
Course Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%		
4.00"	100.00		100.0%		
3.00"	75.00		100.0%		
2.50"	63.00		100.0%		
2.00"	50.00		100.0%		
1.75"	45.00		100.0%		
1.50"	37.50	100.0%	100.0%		
1.25"	31.50		98.8%		
1.00"	25.00		97.5%		
7/8"	22.40		97.0%		
3/4"	19.00	96.4%	96.4%		
5/8"	16.00		94.9%		
1/2"	12.50		93.1%		
3/8"	9.50	91.6%	91.6%		
1/4"	6.30		86.3%		
#4	4.75	83.7%	83.7%		

Fines Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
#4	4.750	83.7%	83.7%		
#8	2.360		68.7%		
#10	2.000	66.5%	66.5%		
#16	1.180		47.8%		
#20	0.850	40.2%	40.2%		
#30	0.600		27.9%		
#40	0.425	19.2%	19.2%		
#50	0.300		12.2%		
#60	0.250	9.4%	9.4%		
#80	0.180		4.0%		
#100	0.150	1.7%	1.7%		
#140	0.106		1.2%		
#170	0.090		1.1%		
#200	0.075	0.9%	0.9%		
#270	0.053				

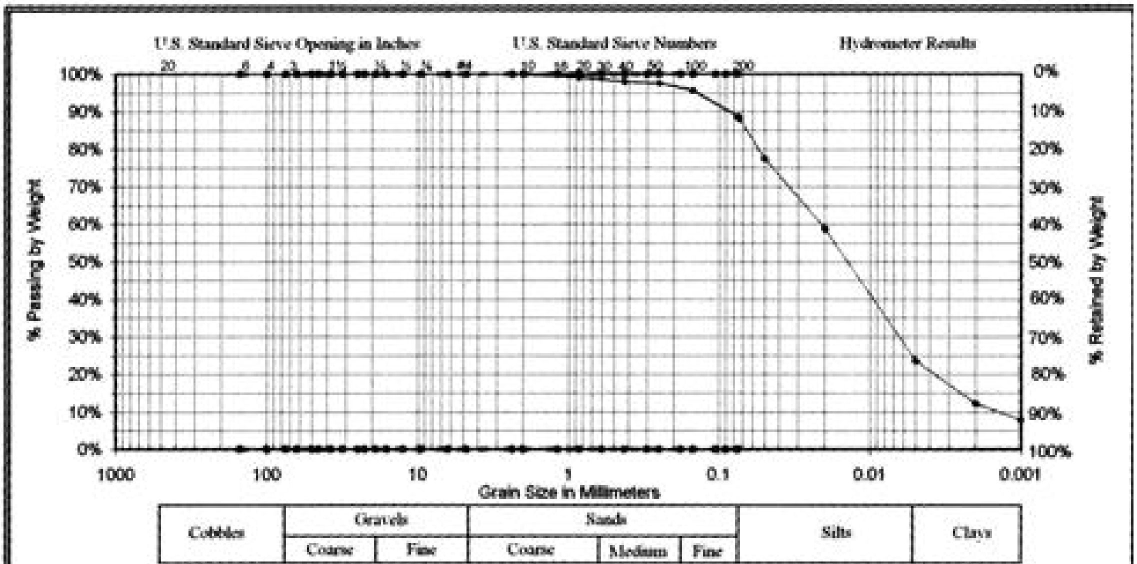
Textural and particle size distribution at the secondary canal



Date :	D_{10} = 0.08	USCS Classification	% Gravel / % Sand
Sample #: 9	D_{30} = 0.12	SP-SM, Poorly graded San	1.3% / 90.7%
Sample ID:	D_{60} = 0.22	Specifications	
Source: FOKISA DIVERSION	C_c = 0.88	No Specs	
Project: DIVERSION	C_u = 2.74	Sample Meets Specs	% Silt & Clay
Location: Secondary canal	Liquid Limit= n/a	n/a	8.0%
Boring #: TP-9	Plastic Limit= n/a	Fineness Modulus	
Depth: 0-30cm	Plasticity Index= n/a	0.98	

Coarse Section Sieve Size US	Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min	Fines Section Sieve Size US	Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%			#4	4.750	98.7%	98.7%		
4.00"	100.00		100.0%			#8	2.360		98.2%		
3.00"	75.00	100.0%	100.0%			#10	2.000	98.1%	98.1%		
2.50"	63.00		100.0%			#16	1.180		95.4%		
2.00"	50.00		100.0%			#20	0.850	94.3%	94.3%		
1.75"	45.00		99.9%			#30	0.600		93.0%		
1.50"	37.50		99.9%			#40	0.425	92.1%	92.1%		
1.25"	31.50		99.9%			#50	0.300		75.2%		
1.00"	25.00		99.9%			#60	0.250	68.5%	68.5%		
7/8"	22.40		99.9%			#80	0.180		49.9%		
3/4"	19.00		99.9%			#100	0.150	42.0%	42.0%		
5/8"	16.00		99.9%			#140	0.106		22.0%		
1/2"	12.50		99.9%			#170	0.090		14.8%		
3/8"	9.50	99.9%	99.9%			#200	0.075	8.0%	8.0%		
1/4"	6.30		99.1%			#270	0.053				
#4	4.75	98.7%	98.7%								

Textural and particle size distribution at the irrigated field



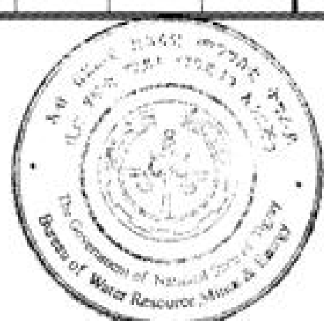
Date :	D_{10} = 0.01	USCS Classification	% Gravel: 0.0%
Sample #: 10	D_{30} = 0.03	#N/A	% Sand: 11.3%
Sample ID:	D_{60} = 0.05	Specifications	
Source: IRRIGABLE LAND	C_C = 1.50	No Specs	
Project: DIVESION	C_U = 6.00	Sample Meets Specs	% Silt & Clay
Location:	Liquid Limit= n/a	n/a	88.7%
Boring #: TP-10	Plastic Limit= n/a	Fineness Modulus	
Depth: 0-30cm	Plasticity Index= n/a	0.09	

Coarse Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
6.00"	150.00		100.0%		
4.00"	100.00		100.0%		
3.00"	75.00		100.0%		
2.50"	63.00		100.0%		
2.00"	50.00		100.0%		
1.75"	45.00		100.0%		
1.50"	37.50		100.0%		
1.25"	31.50		100.0%		
1.00"	25.00		100.0%		
7/8"	22.40		100.0%		
3/4"	19.00		100.0%		
5/8"	16.00		100.0%		
1/2"	12.50		100.0%		
3/8"	9.50		100.0%		
1/4"	6.30		100.0%		
#4	4.75	100.0%	100.0%		

Fines Section

Sieve Size US	Sieve Size Metric	Actual Cumulative Percent Passing	Interpolated Cumulative Percent Passing	Specs Max	Specs Min
#4	4.750	100.0%	100.0%		
#8	2.360	100.0%	100.0%		
#10	2.000	100.0%	100.0%		
#16	1.180		99.4%		
#20	0.850	99.2%	99.2%		
#30	0.600		98.5%		
#40	0.425	98.0%	98.0%		
#50	0.300		97.8%		
#60	0.250	97.7%	97.7%		
#80	0.180		96.3%		
#100	0.150	95.7%	95.7%		
#140	0.106		91.6%		
#170	0.090		90.1%		
#200	0.075	88.7%	88.7%		
#270	0.053				



ANNEX-I. Result of the Interview held with farmers and experts

Date 7-11-2008

I. Farmers

- **Seasonal discharge/ water levels associated with known rainfall in the upstream catchment from the rainfall stations**

1. Water level at known cross-section (u/s) marked on X-3_
2. Water level at known cross-section (d/s) marked on X-14_
3. Water level in the canal 140 Cm on 15 August 1999 E.C
4. Water level in the weir 70 cm on 15 August 1999 E.C
5. Irrigated field (ha) 150 ha

- **Extreme discharge observed (slope area calculations)**

1. Small flood average water level at known cross-section (weir) 40 cm above weir crest
2. Small flood average water level at known cross-section (u/s) (SF on X-3)
3. Mean flood average water level at known cross-section (u/s) (MF on X-3)
4. Mean flood average water level at known cross-section (weir) (marked on the weir crest using total station)
5. Large flood average water level at known cross-section (u/s) (HF = 1715 m.asl on X-3 marked using total station)
6. Large flood average water level at known cross-section (weir) (marked on the weir using total station (occurring once per 5 yrs)
7. Expected frequency of small floods in a dry year (2-3 times per year) and (0.5 m deep sedimentation per year)
8. Expected frequency of small floods in an average year (7 times per year) and (0.75 m deep sedimentation per year)
9. Expected frequency of small floods in a wet year (15-20 times per year) and (1m deep sedimentation per year)
10. Expected frequency of mean floods in a dry year (1-3 times per year) and (0.5 m deep sedimentation per year)
11. Expected frequency of mean floods in an average year (7-8 times per year) and (0.75 m deep sedimentation per year)
12. Expected frequency of mean floods in a wet year (10 times per year) and (1m deep sedimentation per year)
13. Expected frequency of large floods in a dry year (1 times per year) and (1 m deep sedimentation per year)
14. Expected frequency of large floods in an average year (3-4 times per year) and (1m deep sedimentation per year)
15. Expected frequency of large floods in a wet year (7-8 times per year) and (1m deep sedimentation per year)
16. Time to peak during dry year 2 hr and duration to end of flooding 8 hr (during August and during the night

17. Time to peak during average year 2 hr and duration to end of flooding 8 hr (during August and during the night
18. Time to peak during wet year 4 hr and duration to end of flooding 24 hr (during August and during the night

- **Sediment transport**

1. Upstream movement of tradition spate headwork schemes (m) to know historical level rise (after construction of the spate scheme, gabion headwork have been constructed over the masonry weir, the level rise has been estimated 0.6 -0.8 metres)
2. annual dredging volume (depth and frequencies) (2 -3 times a year, 0.5-0.8 m deep in the main canal per year)
3. types of sediment, i.e size (any cobbles) mostly sand and some cobbles
4. Perception on the scheme efficiency, has the transition from traditional to modern scheme showed up some improvements
5. Yes, there is an improvement with regard to reducing the labour required to maintain the head required to divert the required flood and it has also contributed by reducing the trees and shrubs cut for constructing traditional headwork. However, the improvement has also some problems, sedimentation being the most challenging one and insufficient canal sizing.
6. How much effort does dredging take? How many men and how many days 120-130 men for 3-4 days.

II. Experts

- **Seasonal discharge/ water levels associated with known rainfall in the upstream catchment from the rainfall stations**

1. Water level at known cross-section (u/s) __________
2. Water level at known cross-section (d/s) __________
3. Water level in the d/s canal for medium flow1.2m
4. Water level above the weir 0.8m
5. Irrigated field (ha) 150 ha in an average year

- **annual dredging volume (depth and frequencies)** full depth and mostly twice a year

Remark

- Farmers are not happy with some of the designs. For example, masonry lining of the main canal has been criticized. According to their interest, only the right wing of the main canal is important as it protects the flow from going back to the river. However, the canal bottom and left side masonry linings are not good as they enhance sediment trapping and also they are in sufficient.
- Spate irrigation has long been practicing before 40 years using masonry headwork, but it was breached immediately. It was in 1994 again gabion headwork had been implemented. Then the existing headwork was implemented in 2005/2006. Finally, gabion headwork had been added to the masonry to accommodate for the level rise of the scheme in July, 2008)

- Downstream protection of the weir has to be made by concrete.
- The four model farmers interviewed are,
    1. Nuemetela Siraj
    2. Nuru Abay
    3. Haji Dibicho and
    4. Jineydi Nurey