AN EXPERIMENTAL STUDY OF FLOW AND SEDIMENT DISTRIBUTION AT RIVER OFFTAKE

A Thesis Submitted

by

Lamisa Malik

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CERTIFICATION OF APPROVAL

The thesis titled **"An Experimental Study of Flow and Sediment Distribution at River Offtake"**, submitted by Lamisa Malik, Roll No. 1014162008P, Session: October 2014, to the Department of Water Resources Engineering, Bangladesh University of Engineering and Technology, has been accepted as satisfactory in partial fulfillment of the requirement for the degree of **Master of Science in Water Resources Engineering** on 30th April, 2018.

Dr. Md. Abdul Matin — **Chairman** (Supervisor)
Professor
Department of Water Resources Engineering
BUET, Dhaka-1000, Bangladesh

Dr. Md. Mostafa Ali — **Member** (Ex-Officio)
Professor and Head
Department of Water Resources Engineering
BUET, Dhaka-1000, Bangladesh

Dr. Md. Sabbir Mostafa Khan — **Member**
Professor
Department of Water Resources Engineering
BUET, Dhaka-1000, Bangladesh

Dr. M. R. Kabir — **Member** (External)
Professor and Pro Vice-Chancellor
The University of Asia Pacific
Dhaka, Bangladesh

April, 2018

DECLARATION

This is to certify that this thesis work has been done by me and neither this thesis nor any part thereof has been submitted elsewhere for the award of any degree or diploma.

Dr. Md. Abdul Matin	**Lamisa Malik**
Countersigned by the Supervisor	Signature of the Candidate

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B	Width of channel
C	Concentration of sediment particle
C_s	Mean concentration of sediment particle
D	Diameter
D_*	Dimensionless particle parameter
D_{50}	Median diameter of bed material
d_g	Dimensionless grain diameter
Fg	Mobility number
Fr	Froude number
g	Acceleration due to gravity
g_s	Sediment transport per unit time per unit width
K	Conveyance
L	Length
n	Manning's Roughness Value
Q	Water discharge
Qs	Sediment discharge
q	Discharge per unit width
Q_b	Bed load transport
Q_t	Total load transport of bed material
$q_{s,c}$	Volumetric suspended load transport
S	Slope of the channel
T_m	Instantaneous shear stress parameter
U*	Shear velocity
$\bar{u}_{cr}$	Critical depth-averaged velocity according to Shields
$\bar{u}$	Depth-averaged velocity
V or U	Average flow velocity
x,y	Cartesian co-ordinated system
y	Water depth
γ	Specific weight of water
γ_s	Specific weight of sediment particles
τ_o	Bed shear stress
τ_b	Instantaneous critical bed shear stress

τ_b'	Instantaneous effective bed shear stress
ν	Kinematic viscosity
σ	Ratio of relative sediment distribution over relative flow distribution
ρ	Density of fluid
μ	Viscosity of fluid
θ	Angle of offtake

LIST OF ABBREVIATION

BanDuDeltAS	Bangladesh Dutch Delta Advisory Services
BRIC	Bangladesh River Information and Conservation Project
BWDB	Bangladesh Water Development Board
CEGIS	Center for Environment and Geographic Information Services
CPP	Compartmentalization Pilot Project
DWRE	Department of Water Resources Engineering
EKN	Embassy of Kingdom of The Netherlands
FAP	Flood Action Plan
HEC-RAS	Hydraulic Engineering Center- River Analysis System
IWM	Institute of Water Modelling
PPM	Parts Per Million
PWD	Public Works Datum
SSC	Suspended Sediment Concentration

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ABSTRACT

Off-take is a channel that bifurcates from the main channel. River offtake is one of the complex features in fluvial systems, like braiding, anabranching streams and deltas. In the process of developing offtakes, the distribution of flow and sediments along the branches are still a matter of research. Few works have been conducted in the past, however no notable experimental work has been carried out on bifurcation at off-take channel with special reference to Bangladesh. In this study, an experimental work has been conducted in the Hydraulics and River Engineering Laboratory of DWRE, BUET for understanding the flow and sediment distribution at river offtake by changing the discharge and offtake angles. A total of eighteen test runs have been conducted for three different discharge conditions with three different offtake angles i.e. 20^0, 40^0 and 60^0. Prior to test runs theoretical development has been made using dimensional analysis to obtain a functional relationship among the salient variables. For all the experimental runs, flow velocity, water depth and sediment concentration have been measured both for offtake and main channel. Along with the measurements, flow visualizations have also been carried out at the vicinity of the offtakes. It has been observed that turbulent eddies and zone of siltation were formed in the vicinity of offtake mouth(s). Sedimentation and erosion pattern of the main and offtake channel has also been observed. Water and sediment discharge ratios have been calculated for the offtake system. It has been found that, water and sediment discharge ratios increases with the increase of offtake angle. From the experiment, the measured flow velocity found to be increase with the increase of offtake angle. In order to estimate the sediment discharge three well known sediment transport formulas i.e. Ackers-White, Engelund-Hansen and Van Rijn formula have been used in this study. Comparison with the measured sediment discharge shows that the Ackers-White formula predicts reasonably better compared to other two. Based on theoretical development and experimental data, two relationships have been obtained in this study. These equations predict the water and sediment discharge ratios which are function of channel geometry and offtake angle. In addition, field data from selected river offtake systems of Bangladesh has also been used to verify the obtained relationships. The observation made in the study will help in understanding the flow and sediment behavior and it is hoped that proposed relationship developed in this study will be useful for the estimation the flow and sediment distribution of main and offtake channel providing a guideline for better offtake management of Bangladesh.

CHAPTER ONE

INTRODUCTION

1.1 General

In many rivers situation arise where one channel will split into two or more smaller channels. These are often called river channel bifurcations. When a main river channel bifurcates into two channels, the point of bifurcation is often called offtake of the bifurcated channel. Bifurcations occur, for example in river deltas, where a river flows into the sea or lake or in braided rivers where individual channels combine and then separate repeatedly. Over the past 25 years or so much concentration have been given on studying what happens to water flow or sediment (muds, sands and gravels) at river confluences, where two channels combine into one. However, much attention has not been paid to understanding what happens when one channel splits into two or more channels. As a result it is not very known to us about how the river is divided, the influence that this division of the channel has one on the water flow and fluid turbulences and, crucially, how sediments are moved through and around these complicated river divisions. Knowledge and understanding of the process of channel bifurcation is vital to researchers for better modeling, management of many of our natural waterways and better prediction of sediment transport and deposition phenomena. Although ongoing research is beginning to fill in some of these gaps in our understanding through the use of laboratory experiments and mathematical models, this has not been matched by sufficient progress in measuring and quantifying the bifurcation process in natural river channels, very often because natural rivers are much complicated to study and the technology required has simply not been available.

1.2 Background of the Study

The distribution of flow and sediments along the bifurcated channel undergoes complex hydro-morphological processes (Bertoldi, 2004) that need considerable attention. Off-take bifurcations are common features of different fluvial systems, like braiding and anabranching river networks and river deltas. The morphology and development of a bifurcation is mainly governed by the dominant sediment transport mechanism, which determine the distribution of sediment discharge into two bifurcated branches (Pittaluga, 2003). Hannan (1995) studied the sediment distributions at symmetrical channel bifurcation in laboratory experiments. It was reported that no general relation can be expected in case of sediment distribution over two downstream branches of bifurcation because the phenomena depend not only on the nose geometry but also on the of condition of branch channels. Obasi (2008) investigate the effect of off-take angles on the flow distribution at a concave channel bifurcation and derived an equations showing that the off-take discharge increased positively with increases in off-take angles as well as main channel discharges.

As field case, Gorai River is an off-take channel from Ganges. Morphological response of the Gorai had been studied by Clijncke (2001). Mathematical model result on Gorai restoration project found that hydraulic performance and morphology of the Gorai river are strongly affected by the morphological conditions locally near the off-take, both in the Ganges and in the Gorai (Garsdal, 1999) It has been observed by Hore (2013) that high rate of right bank erosion in the Ganges upstream of Talbaria is causes deterioration of the Gorai, as the incoming sediment tends to settle near the Gorai off-take.

The morphological behavior of a river at bifurcation is not as yet a properly understood phenomenon. Because the combined transport of water and sediment in rivers is a complex process. It is difficult to study the morphological behavior of the bifurcation in rivers both in the laboratory and in the field. However, no notable experimental works has been carried out on bifurcation at off-take channel with special reference of Bangladesh. Though Hannan (1995) and Islam (1996) have studied the morphology of symmetrical river bifurcation on an experimental setup built in the Hydraulics and River Engineering Laboratory of Bangladesh University of Engineering and Technology. But their study concentrates only on distribution of discharge and sediment with nose angle and shape of nose junction as a major variable for symmetrical case.

In this study an attempt has been made to investigate the hydraulics of flow and sediment transport in the main channel and in the vicinity of river off-take as proper understanding on the flow and sediment transport behaviors as it is very essential for river analysis and management. Laboratory experimentation for different off-take angles with varying discharges have been carried out for this purpose. Dimensionless groups have been formed to develop theoretical relationship among various hydraulic parameters. Experimental data have been used to obtain exponents and coefficients of the developed equations for predicting flow and sediment distribution in offtake. The developed relationship will provide a guideline for estimation of flow and sediment distribution at the channel offtake. The results obtained from this study may be utilized in possible similar field conditions with considerable modification.

1.3 Objective of the Study

The objectives of this study are as follows:

(i) To develop a theoretical relationship of flow and sediment discharge in main and offtake channel.

(ii) To design and develop an experimental setup for conducting laboratory experiment at the vicinity of offtake for various flow and sediment condition.

(iii) To compare the developed relationship with field observation.

1.4 Organization of Thesis Contents

This thesis has been organized under seven chapters. Chapter one describes the background and objectives of the study. In Chapter two the review of literature related to the offtake study has been described. In Chapter three, theoretical basis of dimensional analysis and governing parameters for discharge distribution and sediment transport at the offtake has been outlined. Chapter four summarizes the methodology of this study as well as test scenarios generated for completion of objectives. Chapter five illustrates the experimentation set-up of the laboratory, measuring techniques, test procedures followed during measurements and the observations noted at that time. In Chapter six, analyses, results and discussions of experimental results are presented. Finally, the main conclusions of this study and recommendations for further research are presented in chapter seven.

CHAPTER TWO
LITERATURE REVIEW

2.1 Introduction

Fundamental processes that causes chaotic morphodynamics of braided rivers arises because of rapid, extreme, and unpredictable changes in stream patterns due to bed and bank scour. In this chapter, a brief overview on mechanism of channel bifurcation process along with morphology and sediment transport at river offtake is discussed. Description and scenarios of major offtakes of Bangladesh as well as some important studies on offtakes are also reviewed in the chapter. In addition, the short overviews of relevant works conducted on the sand bed flume are presented.

2.2 Braiding Phenomena

Braiding is one of the natural river patterns that emerges over a wide range of scales, from small pro-glacial gravel-bed streams to large fluvial systems as the Brahmaputra River; it can be portrayed as a network of channels, splitting and rejoining around islands and bars (**Figure 2.1**)

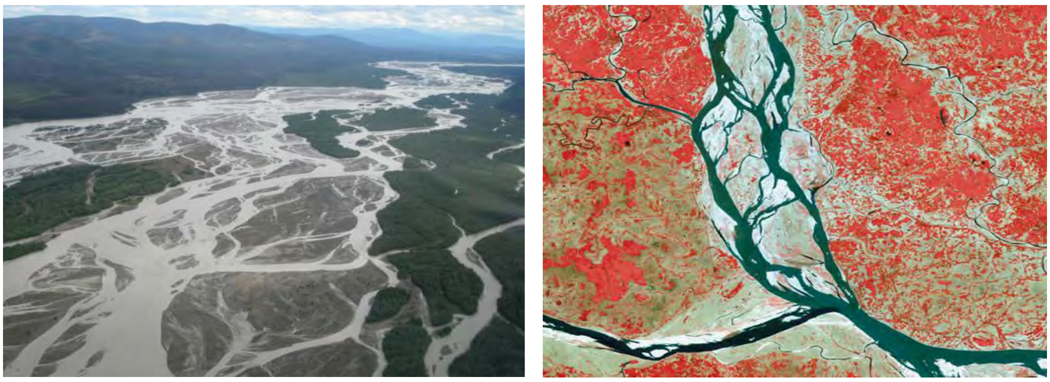


Figure 2.1: Braided network of Brahmaputra- Jamuna River

Murray and Paola (1994) defined the braided pattern as "the fundamental instability of laterally unconstrained free-surface flow over cohesionless beds". Braiding is a complex system of channels interconnected by nodes; namely, confluences and bifurcations.

At present, the prediction of braided rivers planform evolution is possible to some extent with enough accuracy only on a short time scale. But due to its deterministic system characterized by a somehow chaotic behavior, it apparently shows unpredictable features. Time evolution of the channel network is affected by rapid changes, due to the rearrangement of the branches and nodes and, in particular, caused by the modification of the flow and sediment distribution at the bifurcations. Braided rivers are characterized by several complicating features that must be taken into account during assessing its flow pattern. Strong non-linearity due to planform non-uniformities, unsteadiness flow field and sediment transport rate, partially sediment transporting in cross sections due to low sediment mobility and gravitational effects due to the fairly large bed gradients associated with local depositions and scours are among the complicated features.

2.3 Bifurcations and Offtakes

Braided rivers are generated and maintained through the mechanism of flow bifurcation. Bifurcation is the process that determines the distribution of flow and sediments along the downstream branches and adds difficulty to the system. A bifurcation occurs when a river or stream splits into two branches and naturally it occurs when a middle bar forms in a channel or a distributary carries flow from the main river. When a main river channel bifurcates into two different channels, the point of bifurcation is often called offtake of the bifurcated channel. Modeling a bifurcation and offtake is a challenge for existing mathematical models, hence understanding river bifurcation and offtake phenomena is one of the open issues of fluvial research.

Figure 2.2: Difference between river bifurcation (left) and river offtake (right).

2.3.1 Mechanisms of Channel Bifurcation and Offtake Process

A quantitative description of river offtake process is still lacking. The process has been identified primarily by Leopold & Wolman (1957) as the generating process of braiding; Ashmore (1991) describes in detail three possible mechanisms of bifurcation formation, as suggested by laboratory observations.

- *Central bar mechanism and dissection of transverse unit bar.* This mechanism involves the development of a central bar by the accumulation of the coarsest fractions which forces the flow to diverge and is eventually exposed. A similar mechanism, with the dissection of a transverse unit bar may occur, when the channel is characterized by higher sediment mobility.
- *Chute cutoff mechanism.* It is the most common bifurcation mechanism which is characterized by the modification of an alternate bar structures in low sinuosity channels. The bar is progressively transformed into a more complex bed form by lateral accretion, which determines more flow to be directed over the point bar. The steeper gradient near the head of the slough channel captures progressively larger volumes of water, leading to the bifurcation of the flow.
- *Multiple bars mechanism.* This mechanism applies only to channel with very high values of the width-depth ratio, documented by Fujita & Muramoto (1988) and can be roughly explained in terms of the results of the linear stability analysis (Fredsoe, 1978). The multiple rows bars, which characterize the initial bed configuration, are gradually converted into fewer larger bars which concentrate the flow and lead to braiding.

2.3.2 Physical Processes at Bifurcation

At a bifurcation both discharge and sediment are divided over the two branches. The discharge distribution is governed by the conveyance capacity and hydraulic gradient at the bifurcation with the condition that water levels at the bifurcation point must be the same (Jansen et al., 1979; Kleinhans et al., 2008). However, the division of sediment depends on the details of local conditions at the bifurcation point. In 1D computation, an empirical nodal point relation needs to be used to determine how sediment splits between the two branches Wang et al. (1995). In principle, 2D (and 3D) models are capable of calculating this division of sediment transport (van der Mark and Mosselman, 2013) without the need of a nodal point relation. But even in 2D and 3D models, a number of singular physical processes need to be taken into account at bifurcations for its effects on the distribution of water and sediment between the two branches. These processes are described in the following paragraphs.

a) Bulle Effect

In a laboratory experiment, Bulle (1926) investigated the sediment transport division at a bifurcation between a straight channel and a lateral offtake channel at different angles, with the same discharge flowing into both branches. Under these conditions it was found that more sediment was diverted into the lateral channel than continued in the straight channel. This phenomenon is often referred to as the Bulle effect and has been corroborated later on by multiple studies. This phenomenon is due to the curvature of the streamlines at the bifurcation, which induce a helical flow analogous to river bends this helical motion directs near-bed flow towards the offtake channel.

b) Flow separation

From his experiments, Bulle also describes the phenomenon of flow separation at the entrance of an offtake. Flow separation occurs at sharp edges, where flow can no longer follow the banks. In that situation, there is a region behind the sharp edge where flow velocities are smaller than the main flow. This difference in velocity creates a turbulent mixing layer that extends downstream of the sharp edge. In the case of an offtake, this sharp edge is located at the start of the bifurcation if the offtake angle is too large.

Further downstream in the offtaking channel there is a reattachment point, where the flow is again connected with the bank. An eddy is formed between the sharp edge and the re-attachment point. Because of the circulation inside this eddy, sediment near the bed moves towards the center of the eddy (Mosselman, 2014).

At the boundary of the eddy there is some exchange with the main flow due to turbulence. As this happens, sediment particles near the bed that enter the eddy are trapped in its center, while flow escaping the eddy at the water surface is relatively clear. With the mechanism described here, the formation of this eddy at the entrance of the offtake steers sediment transport into the bifurcated channel.

c) Transverse bed slope advantage

Bed topography can also play an important role on sediment distribution. Transverse bed slopes influence sediment transport direction. The presence of a transverse bed slope upstream of a bifurcation can therefore influence the distribution of sediment between the two downstream branches, favoring the branch located in downslope direction. This effect was studied by Pittaluga et al. (2003), who concluded that transverse bed slope upstream of the bifurcation point up to a distance of 2 to 3 times the channel width will affect the sediment distribution and can be a determining factor for the evolution of bifurcations, especially when sediment transport occurs mainly as bed-load.

d) Asymmetrical approach

While the transverse bed slope advantage describes a local effect near the bifurcation point, asymmetrical approach conditions refer to a larger scale upstream of the bifurcation. Sediment transport at the bifurcation can be altered if bed topography upstream of the bifurcation presents large differences in transverse direction (for example because of the presence of a river bend upstream of the bifurcation. Deeper parts of the channel will concentrate higher flow velocities and sediment transport, which can lead to a larger sediment input into one of the bifurcates on top of the local geometry effects of the bifurcation mentioned in the previous paragraphs) (Sloff and Mosselman, 2012).

The presence of a bend upstream of the bifurcation can also produce lateral sediment sorting (bend sorting), where coarser sediment is found at the outer bend due to helical flow. The branch bifurcating from the outer bend will then receive relatively coarser sediment than the branch at the inner bend.

2.3.3 Morphology of Offtakes

Offtakes are bifurcations where the one branch is minor compared to the other. The minor branch is the offtake of the main branch. As for example the Gorai River is the offtake of the Ganges Riiver near Talbaria.

2.3.3.1 Sediment Distribution in Offtakes

The parameters that influence the sediment distribution over the offtake are:

- The discharge ratios $Q_{offtake}/Q_{main}$
- The importance of bed load transport for the total sediment transport, expressed in w_s/u.
- The situation of the offtake related to the main river.
- The downstream boundary conditions.

Bulle, 1926 and Akkerman, 1993 stated that the offtake of a straight channel collects more sediment relative to the discharge than the ongoing channel because of the spiral current that is developed (**Figure 2.3**).

The bed load transport of directed toward the offtake. In a situation where the ongoing channel also has an angle with the main channel, the ongoing channel or offtake with

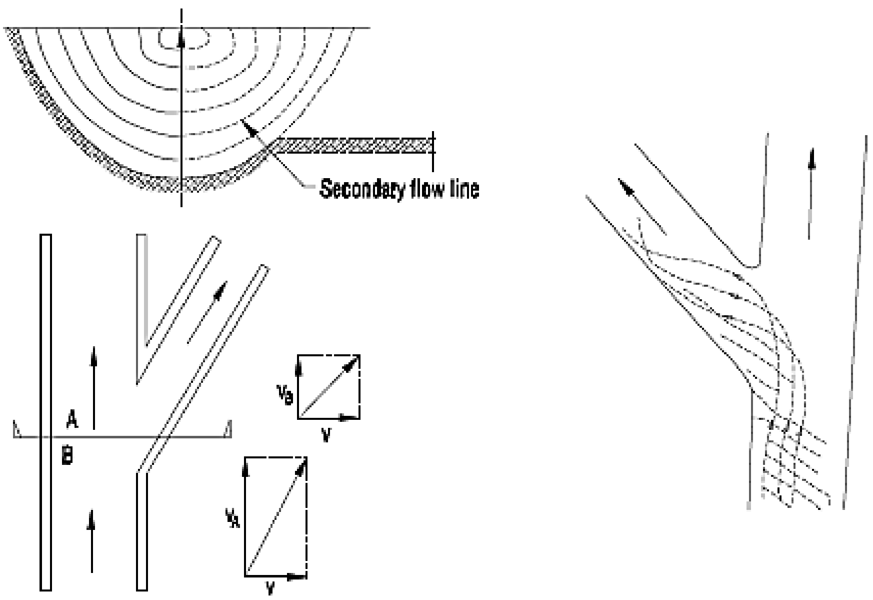


Figure 2.3: Offtake from the straight channel and Bulle effect (Akkerman, 1993)

the largest angle will collect relatively the most sediment. The shape and geometry of the offtake also determines the curvature of the flow lines entering the offtake and therefore the occurrence and magnitude of the secondary flow (DHV-Haskoning, 2000).

This effect is stronger when the offtake is situated in an inner bend. When the offtake is situated in an outer bend this is favorable for the sediment intake in the offtake, because less sediment will come into the offtake. Besides angle and shape of the offtake, the discharge ratio of the offtake and main channel is important for the sediment intake. The discharge is distributed over the two channels, dependent on downstream boundary conditions and therefore the slopes in the ongoing branches. When the suspended sediment consists mainly of wash load and will therefore be distributed equally over the vertical, the suspended sediment is equally distributed as the discharge. The ratio of the bed load transport is not equal to the ratio of the discharge of the two channels. So, it is stated that the suspended sediment is dominant for the situation of the Gorai River (Clijncke, 2001).

For a case study of Gorai small discharge ratios are favorable for the sediment intake in the offtake. However, angle of the main channel which is about 70^0 for Gorai, is also important for sediment intake. According to Indlekoffer, with a discharge ratio of 10% and an angle of 90^0, this gives a sediment intake of 35% of bed load of Ganges River. A different approach angle will result difference in sediment intake for Gorai River. A decrease in angle of offtake will result decrease in sediment intake in the Gorai River.

2.3.4 Closure of Offtakes

Sediment transport is larger and the rivers are more dynamic during monsoon season and the river bed morphology experiences the most significant changes, especially the large rivers. In some cases these changes produce a more favorable configuration for an offtake distributary, but they can also lead to unfavorable layout that reduces the conveyance capacity of the distributary, increasing the probability of closure.

A specific example of low parent channel water level can be found in the Ganges River with the operation of the Farakka Barrage (Mirza, 2004). After the construction of the barrage in 1975, water was diverted upstream of Farakka into the Hoogly river during the dry season, with a 40% reduction of the minimum mean monthly discharge (from

Table 2.1: Causes of offtake closures and their drivers

Causes	**Driver**
• Water level of parent river become too low in dry season	Natural hydrological variable
	Operation of hydraulic structure
Bed level at offtake is too high, because	
• Sediment input becomes too large	Offtake is located at inner bend
	Angle of approach becomes less favorable
	Main channel of parent river moves away from the offtake
	Presence of mid channel bars in front of offtake reduces flow velocities
• Erosion becomes insufficient	Recession period after monsoon is reduced
	Shallow bend crossing at a distributary reduced the hydraulic gradient
	Stronger sediment layer limits adoptability of the offtake

2000m^3/s to just 1200m^3/s) and minimum mean monthly water levels decreased around 1m (from 7 to 6m PWD). The situation got even worse between 1988 and 1996, when the agreement between India and Bangladesh was broken and more water was diverted to the Hoogly, reducing minimum mean monthly flows to only 550m^3/s and minimum water levels to 5.2m PWD (CEGIS, 2012b). During this period, flow in the Gorai River was discontinued each dry season for more than 100 days. From 1997 to the present, with a new treaty, minimum discharges increased to 935m^3/s, still 50% lower than before the construction of the barrage. Between 1998 and 2004, dry-season flow was maintained in the Gorai River through intense dredging for the Gorai River Restoration Project (GRC, 2002; CEGIS, 2012a) but since then this offtake has been closing again each year. Even though the effects of the Farakka barrage on water levels only affect the Gorai offtake, this is an issue of major concern in Bangladesh as there are ongoing studies to apply similar measures to the Brahmaputra River in the future (Misra et al., 2007; NWDA, 2014).

2.4 The Rivers and Offtakes of Bangladesh

The Ganges, Brahmaputra and Meghna Rivers flow through Bangladesh, where they confluence and discharge into the Bay of Bengal (**Figure 2.4**). Sediment brought by these Rivers has built up over thousands of years what is now one of the largest deltas in the World: the Bengal Delta (Sarker et al., 2013). Socio-economic development of Bangladesh largely depends on these three major rivers (The Ganges, Brahmaputra and Meghna Rivers) and their numerous tributaries and distributaries. This complex and extensive fluvial system brings

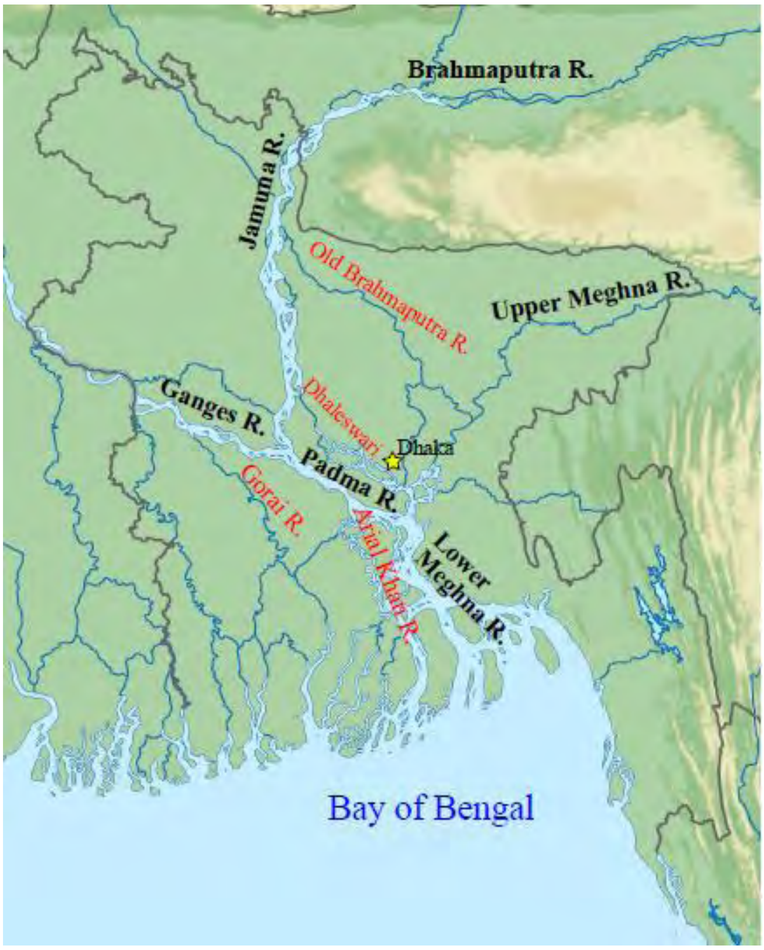


Figure 2.4: Major rivers of Bangladesh and their main distributaries (in red).

fresh water, fish and a means of transportation to the rural areas of the country. It plays an important role for urban areas as well, with an ever increasing water supply demand, an industrial sector heavily dependent on river resources and navigation being a key aspect for future economic growth (BanDuDeltAS, 2014, 2015; EKN, 2013).

2.4.1 Major Rivers

Brahmaputra-Jamuna River

The Brahmaputra-Jamuna enters Bangladesh from its north border and has a length of around 240 km within the country until its confluence with the Ganges River. A notable change is the last avulsion of the Brahmaputra River to the present Jamuna River. Two distinct reaches can be identified: the upstream reach from the border with India until the Old Brahmaputra offtake is called Brahmaputra; the downstream reach from there until its confluence with the Ganges is called Jamuna. About 250 years ago the Brahmaputra River was flowing along the east side of the Madhupur Tract, discharging into the Meghna River and then reaching the Bay of Bengal.

Ganges River

Around 200 years ago the Brahmaputra river started an avulsion process with a significant amount of its flow being diverted to the Jamuna River, which discharged into the Ganges. The Ganges enters Bangladesh from the west and from the country border it flows for about 220 km until it meets the Jamuna River. The river morphology is controlled by the presence of less erodible bank materials (clay outcrops) and human interventions, with a predominant meandering planform (CEGIS, 2012b).

Padma River

The Padma River results from the combined flow of the Jamuna and the Ganges. It flows for 110 km until the confluence with the Upper Meghna and then for another 120 km under the name of Lower Meghna until it reaches the Bay of Bengal. By the early 20th century the Jamuna River had taken most of the Brahmaputra flow, with the old branch, renamed Old Brahmaputra, which is a distributary of the Brahmaputra-Jamuna system. Later, the Padma River moved eastwards to join the Meghna River, leaving its former course as a distributary of the main flow similar to Old Brahmaputra. This distributary received the name of Arial Khan (Santamaria, 2017).

Table 2.2: Hydrological characteristics of the major rivers of Bangladesh

River Name	Total Length (Km)	Basin Area (Mkm^2)	Average Rinfall (mm/yr)	Average Discharge (m^3/s)	Bankfull Discharge (m^3/s)	Maximum Discharge (m^3/s)
Jamuna	2,900	0.55	1,900	19,600	48,000	100,000
Ganges	2,510	1.09	1,200	11,000	43,000	78,000
Padma	121	1.64	-	28,000	75,000	128,000

Table 2.3: Hydraulic and morphological characteristics of the major rivers of Bangladesh

River Name	Width (Km)	River Gradient	Average Water Depth (m)	Sasonal water level variation (m)	Planform
Jamuna	12	$7X10^{-5}$	5	6	Braided/ Anabranching
Ganges	5	$5X10^{-5}$	4.5	8	Meandering
Padma	7	$4X10^{-5}$	-	6	Meandering

Tables 2.2 to 2.4 (Santamaria, 2017) summarizes some of the general characteristics of the major rivers as observed and measured in the recent past. The Meghna River has been excluded from this study as it does not have any important distributary and its geographic setting, hydrological regime and morphologic evolution present major differences from the rest of the river system in Bangladesh. Although the Padma River originates from the confluence of both Jamuna and Ganges, the discharge of the Padma is lower than the sum of the other two rivers. This is because the Distributary Rivers from the Ganges and Jamuna are not taken into account and convey a significant flow, particularly during high water events.

Table 2.4: Sediment transport characteristics of the major rivers of Bangladesh

River Name	Suspended Bed Material Load (Mton/yr)	Wash Load (Mton/yr)	Grain size, D_{50} (mm)	Geometric Standard deviation, σ
Jamuna	125	277	0.2	1.6
Ganges	76	558	0.17	1.4
Padma	227	721	0.09-0.15	-

2.4.2 Main Distributaries and Offtakes

Old Brahmaputra

The Old Brahmaputra is the former course of the Brahmaputra River as it flowed east of the Madhupur tract. The Old Brahmaputra is one of the main distributaries of the Jamuna (Brahmaputra) that distributes part of Jamuna discharge over a large area of North Central region of Bangladesh. The old course of the Brahmaputra River, presently known as the Old Brahmaputra, takes off at Kholabarichar, approximately 10km upstream from Bahadurabad, and follows a south-easterly course via Mymensingh and Toke up to Bhairab Bazar - at the confluence with the Upper Meghna River. The total river length between the off-take and outfall is approximately 225 km, in accordance with the estimate by River Morphology and Research Circle, Bangladesh Water Development Board. The Old Brahmaputra River is at present reduced to a left bank spill channel of the Brahmaputra River and only active during the high stage of the Brahmaputra River. Since the avulsion of the Brahmaputra into the Jamuna, the Old Brahmaputra River is losing its conveyance capacity. The offtake is the most dynamic part of the river and its location has shifted large distances over the last decades. Some attempts and proposals have been made to stabilize this offtake and reopen the river for navigation (e.g. Boskalis-GRC, 2000) but none of them have completely succeeded and the Old Brahmaputra offtake is still silted up each year. From FAP 24 (1996) and IWM Database the Characteristics of the Old Brahmaputra are tabulated in **Table 2.5**.

Table 2.5: The Characteristics of the Old Brahmaputra

Parameters	Value
Length of offtake to outfall Bhairab Bazar (km)	225
Discharge at Mymensingh (m^3/s)	12-4.890
Seasonal water level variation at Kholabarichar (m)	5.3
Average channel sinuosity	1.24
Average bed slope upto Jamalpur (cm/km)	8.4
Median grain diameter, d_{50} (mm)	0.005-0.348

Gorai

The Gorai River is the main distributary of fresh water from the Ganges to the southwest region of Bangladesh. At Talbaria Gorai takes off from Ganges. After about

Table 2.6: The Characteristics of the Gorai

Parameters	Value
Average catchment area (Km^2)	15160
Average annual water depth (m)	7
Average width (m)	1200
Average flood flow of the (monsoon period) (m^3 /s)	4,500
Average flood flow of the (dry period) (m^3 /s)	1300
Annual average sediment transport (Mtons)	50
Mean annual flow volume (m3/s)	2000 to below
Median grain diameter (d_{50}) (mm)	0.14

190 km from the offtake Gorai flows into the Bay of Bengal. Final modeling report of Gorai Restoration Project, BWDB, 2010 has been used to tabulate the characteristics of the Gorai in **Table 2.6**.

During the last decades, the low flow characteristic of the Gorai has changed significantly. The Gorai has been virtually dry during almost the entire dry season since 1989 due to substantial sedimentation of the river at its offtake from the Ganges. Morphological developments in the Ganges river basin, the operation of the Farakka

Barrage on the Ganges further upstream on the Indian side of the border and other human interference on the rivers in the region have been cited as reasons for deterioration of the Gorai (FAP 24, 1996). One of the main causes of sedimentation in offtakes is an unfavorable angle of one of the branches with respect to the main flow.

Dhaleswari system

This river system feeds the metropolitan region of Dhaka, the capital of Bangladesh. The network of rivers comprises the Dhaleswari, Pungli, Bangshi, Turag and Buriganga—amongst other rivers—and has three active offtakes along the left bank of the Jamuna River. The most important offtake is called New Dhaleswari Spill Channel and is located just downstream of the Jamuna Bridge guide bunds. Even though the banks of the Jamuna have been stabilized at that location, the offtake receives a large amount of sediment and there is no flow at the distributary for 4 to 5 months each year (IWM, 2015).

Another offtake, the South Dhaleswari offtake is located 4 km downstream of the location of the first offtake. Finally, a smaller offtake developed further downstream and reopened an old channel of Dhaleswari. It has been named Old Dhaleswari offtake and has been a focus of interest recently because it silts up in the dry season, but presents high erosion rates during the rainy season, threatening villages along the river banks next to the offtake.

Arial Khan

The Arial Khan River was the main channel of the Padma River some 150 years ago until the Padma shifted towards the Meghna. After that, the discharge into the Arial Khan was significantly reduced and this river became very dynamic, adapting to the new flow conditions. According to CEGIS (2012a), until around 1980 there were two distinct offtakes to this distributary, in occasions being up to four depending mostly on the planform of the Padma River. This offtake is currently located around 50 km downstream of the confluence between the Ganges and the Jamuna rivers, but in the last 50 years it has moved within a range of 10km (reduced to 3 km in the last 15 years), also according to CEGIS (2012a). Despite the northern offtake being active throughout the last decades, the flow to the Arial Khan in the dry season has been oscillating and the conveyance capacity of the offtake has reduced over the last years (Mamun, 2012). The causes for this behavior can be the shifting of the main channel of the Padma away from the offtake location, and the formation of bars in front of the offtake.

2.5 Other Mentionable Rivers and Their Offtakes

There are some other offtakes of Bangladesh that are used during the analysis of the study were described below:

Korotoya and Dhepa

Karatoya River is an intriguing river, formerly the main channel of the Teesta. It is originated from India. The northern part, called the Dinajpur-Karatoya, is the main source of the Atrai River. Dhepa River is an offtake of the Karatoya-Atrai (Buri Tista) river. The river originated from the right-bank of the atrai near Mohanpur (25°53'N latitude, 88°43'E longitude) in the greater Dinajpur district and flow southeast direction. It has two distributaries Goveshwari and Dhepa. The characteristics and location of the Korotoya River have been tabulated in **Table 2.7** and **Figure 2.5** respectively after BWDB, 2011.

Table 2.7: The characteristics of the Korotoya River

Parameters	**Value**
Average Length (km)	187
Average width (m)	135
Average flood flow of the (monsoon period) (m^3/s)	4,570
Average flood flow of the (dry period) (m^3/s)	2.36

Dhepa originates from Birganj Upazilla neas Mohanpur Union at Korotoya River and ends at Farakka Barrage Union at Punarbhaba River. It is a seasonal river thus it becomes almost dry during dry season. Average length of the river is 5km where average width is 175 m. It is meander in nature. It has a tributary called ―Choto Dhepa”.

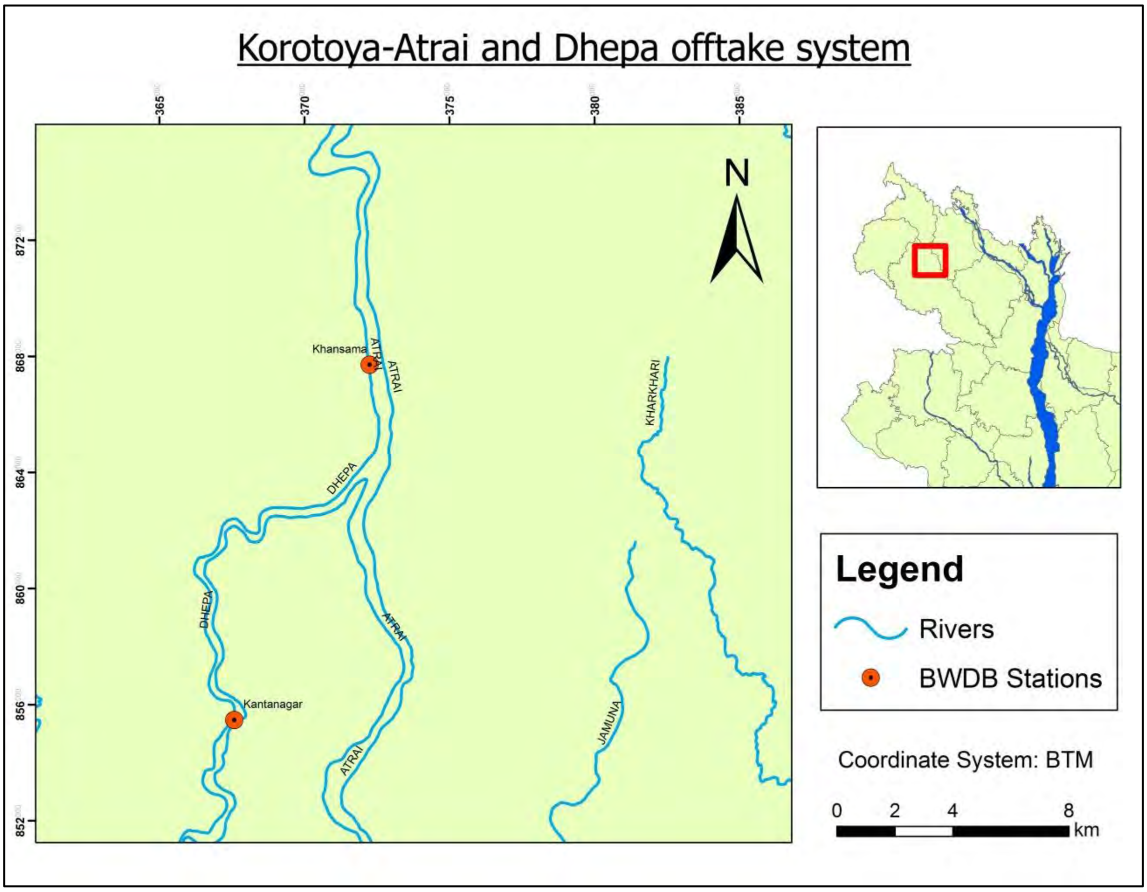


Figure 2.5: Location of Korotoya-Dhepa offtake system

Surma River and Botor Khal

The Surma River is a major river in Bangladesh, part of the Surma-Meghna River System. It starts when the Barak River from northeast India divides at the Bangladesh border into the Surma and the Kushiyara rivers. It ends in Kishoreganj District, above Bhairab Bazar. Water balance studies indicate that the annual contribution of the Surma to the Balui amounts to 89.7 km^3/year (Surface Water Resources of Northeast Region, FAP-2, 1993). This flow identifies Surma as the third largest river in the Meghna Sub region. These inflows, occur mainly in monsoon season as intense flash floods with peak flows ranging up to 30 or more times average flows. The Surma also collects 35% of the Barak inflows.

The upper Surma flows through Surma Flood Plain deposits until a point downstream of Chhatak where it enters the haor areas of the central basin. Between Amalshed and Chhatak, the Surma flows in a single irregularly meandering sand bed channel. The average annual suspended sediment flow of the Surma at Sylhet is estimated as 2.681

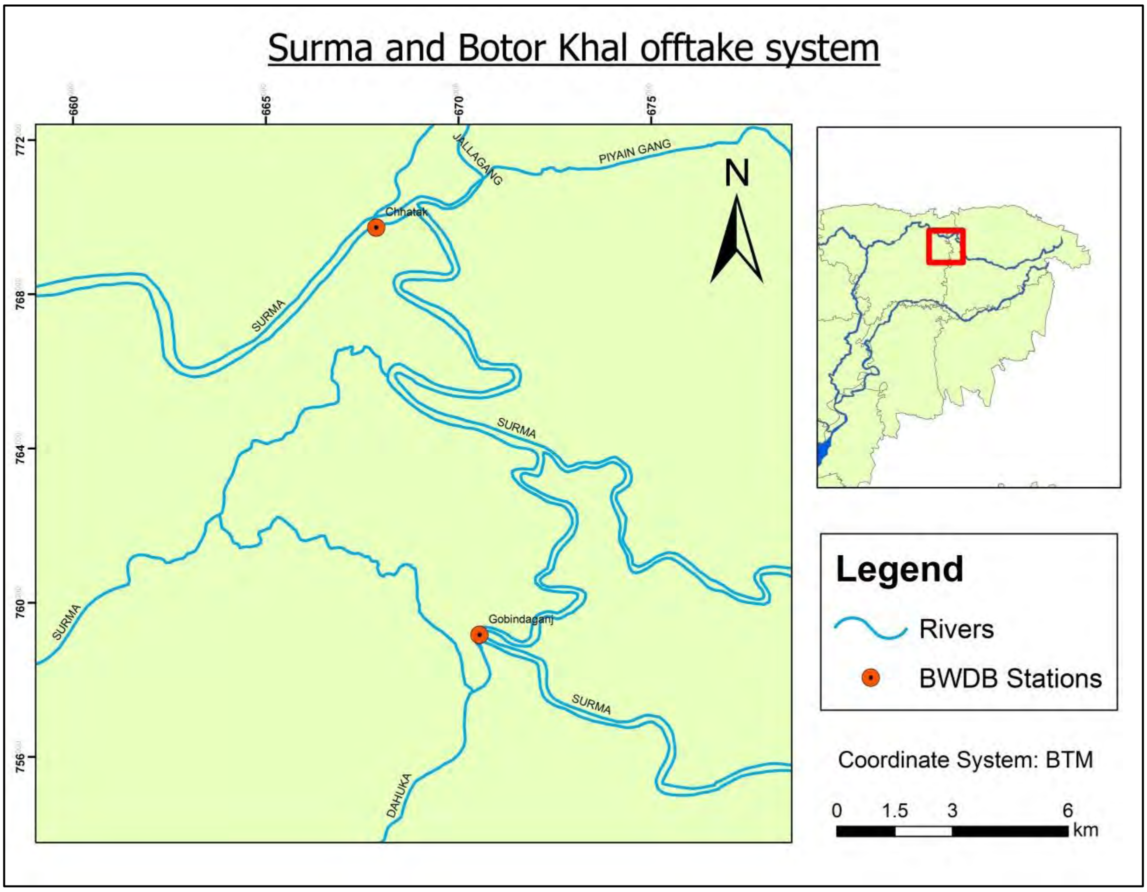


Figure 2.6: Location of Surma River and Botor Khal offtake system

million tons. The peak suspended sediment flow is estimated as 3.966 million tons in the year 1991 (Kamrunnesa, 1994).

Botor Khal River originates from Chattaka Upazilla of Sunamganj at Surma River and falls at Bukha River situated in Pagla Union South Sunamanj Upazilla. Average length of the river is 29 km where average width is 77 m. It is meander in nature. It has a distributary named Dauka River.

Old Brahmaputra and Jhenai Offtake

Old Brahmaputra River is a river that originates from the left bank of the Brahmaputra to the north of Bahadurabad. Flowing more or less to the southeast it passes by Jamalpur and Mymensingh towns and falls into the Meghna at Bhairab Bazar. River shifting has been a characteristic feature of the Bengal basin, affecting small sections or even the entire river. The most dramatic was the shifting of the courses of the Teesta, Brahmaputra and lower Ganges River channels as evident from maps prepared hundreds of years ago.

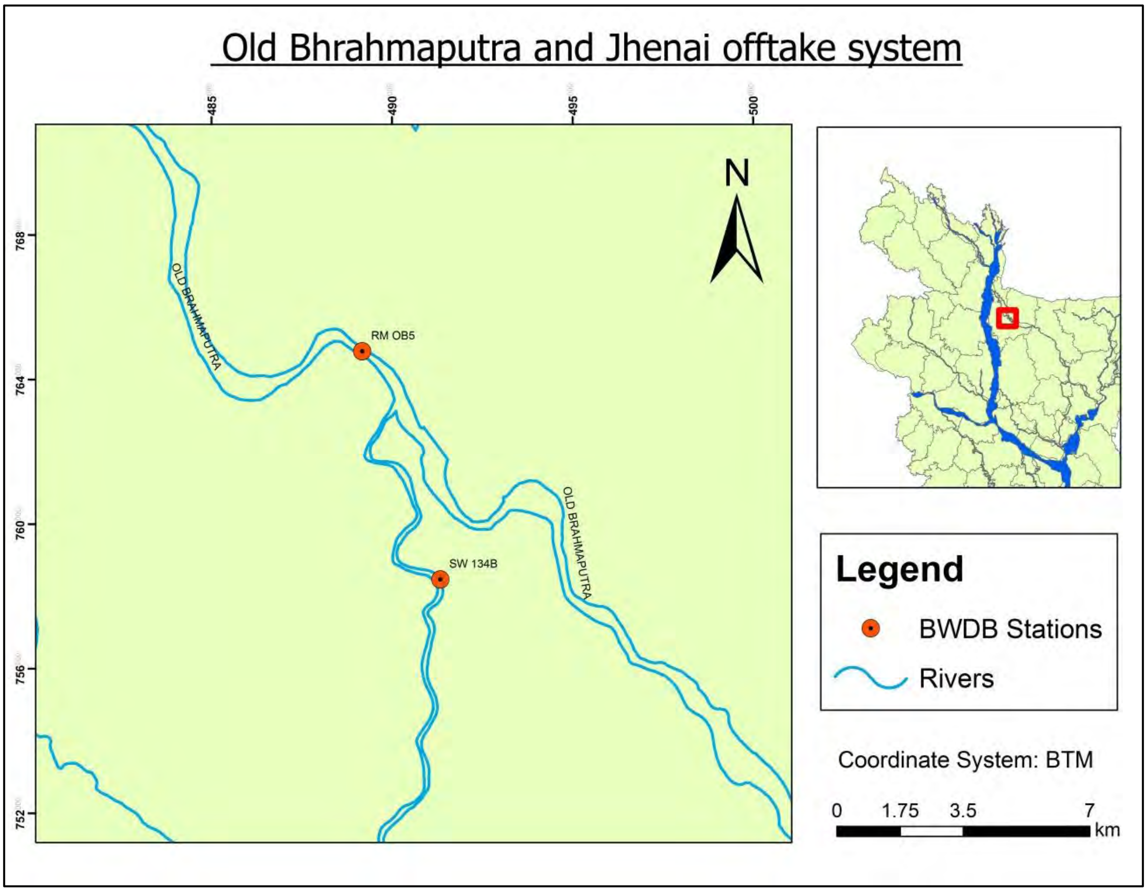


Figure 2.7: Location of Old Brahmaputra and Jhenai offtake system

It has seven tributaries among them Jhenai, Shitalakhya, Sutia, Arial Khan Rivers are some of the significant tributaries. Average length of the river is 283 km where average width is 200 m. It is meander in nature.

Jhenai River originates from Melandah Upazilla of Jamalpur at Old Brahmaputra River and falls at Bangshi River situated at Nagbari Union, Kalihati Upazilla in Tangail. This rivers flows through Sarishabari Upazilla of Jamalpur district and meets with Jharkata River. Average length of the river is 133 km where average width is 76 m. It is meander in nature with flood slope of 7 cm/km. It has two distributaries named Jharkata and Chapai River.

2.6 Previous Studies and Researches on Offtake

Wang et al. (1995) proposed a nodal-point relation at bifurcations based on model-technical as well as physical considerations. For one-dimensional (1D) network morphodynamic models: The ratio between the sediment transports into the downstream branches was proportional to a power of the discharge ratio for one-dimensional (1D) network morphodynamic models. He also analyzed the influence of the nodal-point relation on the behavior of the morphodynamic model theoretically. Using the nodal point relation

$$S_1/S_2=Q_1/Q_2 \tag{2.1}$$

Here, S is the sediment discharge and Q is the water discharge, subscripts indicate main and bifurcated channel. It was found that there are three equilibrium states. The first equilibrium state describes the situation in which both branches of the river remain open; the other two states describe the situation in which one of the branches closes. They also found that the first equilibrium state is unstable and the latter pair is stable. On the basis of their analysis, they proposed the nodal point relation

$$S_1/S_2= (Q_1/Q_2)^m \tag{2.2}$$

instead of Equation 2.1 where *m* is a constant. With this new nodal point relation, they found that there are three equilibria. The exponent in the nodal-point relation appeared to be crucial for the stability of the bifurcation in the model. For large values of the exponent, the bifurcation was stable, i.e. the downstream branches remain open. For small values of the exponent, the bifurcation was unstable: only one of the branches tends to remain open. The exponent also had a strong influence on the morphological time scales of the network. The conclusions from the analysis had been verified by numerical simulations using a package for one-dimensional network modeling.

Fokkink and Wang (1995) extended the analysis carried out by Wang et. al (1995). The analysis was extended in two ways, first by taking the hydraulic radius into account and second by considering a width-depth relation of bed. They proposed the following general nodal point relation

$$S_1/S_2=(Q_1/Q_2)^{m} (B_1/B_2)^{l} \tag{2.3}$$

S denotes sediment transport, Q denotes discharge and B denotes channel width. The indices denote the different channels and k is a positive exponent and l is equal to l-k. If B1 is equal to B2 the new nodal point relation is the same as the old one proposed by Wang et. al (1994). The width has been incorporated in this relation because the widths of channels 1 and 2 have a strong influence on the equilibrium position. With this new nodal point relation, they concluded that if k is smaller than 5/3, the nodal point relation unstable and if k is larger than 5/3, the nodal point relation is stable. If hydraulic radius is taken into account then the nodal point relation is stable if k is larger than 5; it can be stable or unstable if k is in between 5/3 and 5, depending on the geometry of the network; it is unstable if k is smaller than 5/3.

A literature survey on the sediment distribution at bifurcation points in natural rivers and artificial channel was carried out by Akkerman, 1993 (after Wang and Kaaij, 1994). It was found that the curvature effect at the bifurcation and immediately upstream of the bifurcation is very important for the sediment distribution. This indicates that the sediment distribution relation or the nodal point relation is different from bifurcation to bifurcation.

Richardson and Thorne (1995) studied the secondary currents and morphological evolution in a bifurcated channel. They tried to understand the factors which are important in determining the sediment transport distribution at bifurcation. They defined secondary currents as currents which occur in the plane normal to the axis of the primary flow. Sediment transport is strongly influenced by the secondary flow pattern.

This hypothesis of secondary flow pattern is consistent with the main morphological features of bifurcating channels. After the study they concluded that the pattern of secondary currents in a bifurcating channel is more complex than the hypothesis. It was suggested that curvature of flow at the point of hydraulic division of the two streams of water induces vertical flow that is clockwise in the left channel and counter clockwise in the right channel. Along the middle third of the divided reach, strong vertical flow exists with a counter clockwise rotation in the left channel and clockwise rotation in the right channel. Flow patterns, bed topography and morphological changes in this middle reach correspond to the hypothesized system.

Roosen et al. (1995) made an experiment on bifurcation using test rig. With the use of statistical analysis, nodal point relations were found for the specific types of bifurcations in the test rig, for three different upstream discharges and two different shapes of the bifurcation. It appeared from the experiment that the general nodal point relation proposed by Wang et al. was appropriate.

FAP 24 (1996b) studied the Gorai offtake in detail on the Special Report No 10. The distributary was selected because of the importance of the Gorai for the fresh water inflow to the Southwest region, but also because the Gorai a well-defined single channel without any tidal influence and therefore less complex. They mainly focused on three elements: the analysis of historical data in order to understand how the hydraulic conditions at Gorai offtake have changed over the last 30 years, the collection and analysis of detail data in special surveys of the RSP, the development of two-dimensional morphological modeling to improve the understanding of which processes are important for the development of the dry season flow to the Gorai.

Dey et al. (1998) have taken attempts to derive the exponent ―b” analytically for different total and bed load predictors with some assumptions. Seven total load and three bed load prediction formula were considered in this analysis. The relation between the exponent ―b” and Shields parameter was also analyzed. The analysis showed that the exponent ―b” varies between 3.1 to 8.5 for total load and 3.3 to 10.5 for bed load predictors. The sensitivity of the exponent ―b” for variation with the grain sizes (d_{50}) was found to be significant for Ackers and White and Van-Rijn formula. The water level slope had a small influence on the exponent ―b” as observed at low values of the Shields parameter.

Pittaluga et al. (2001) investigated the equilibrium configurations and the stability of river bifurcations in gravel braided networks within the context of a one-dimensional approach, the nodal point relationship pointed out by Wang et al. They proposed an alternative formulation of nodal point conditions based on a quasi two-dimensional approach. In the case of a simple channel loop the model predicts the existence of a threshold value of the Shields parameter in the upstream channel above which the loop only admits of one stable equilibrium solution with both branches open. For values of the Shields parameter below the above threshold two further equilibrium configurations appeared, which were characterized by different values of water discharge flowing into the two downstream branches. The model also provided useful information on the physical mechanism that governs the development of a bifurcation in gravel braided rivers, the transverse exchange of sediment which was induced by topographical effects close to the channel division.

Imteaz et al. (2001) developed a mathematical model for the Old Dhaleswari River as well as other offtakes from Jamuna River. Due to the construction and associated river bank protection works of Jamuna Multipurpose Bridge on Jamuna River at Bangladesh, water flow through the Old Dhaleswari River was reduced significantly. As a result effectiveness and usefulness of irrigation project named ―Compartmentalization Pilot Project (CPP)" area at Tangail located at the downstream end of Old Dhaleswari River became at a stake. A link canal was proposed with mouth at the Jamuna River connecting the Old Dhaleswari River. The model was simulated to check the effectiveness of the proposed link canal in terms of water availability to the downstream users. It was found that proposed link canal could augment water supply for the irrigators significantly.

In a series of experiments on flow in isolated, well defined bifurcations Federici and Paola (2002) found that although a central bar always develops, the divided flow may continue to flow on both sides of the bar (‗stable'' bifurcation) or may eventually be forced entirely to one side of the bar or the other (‗unstable'' bifurcation). They also found that an unstable bifurcation forms when the flow field is characterized by both a low Shields stress and a non-uniform incoming flow. We also found that divergences with erodible banks tend to an equilibrium configuration that depends mainly on the widening ratio of the channel.

Bertoldi (2003) provides a suitable description of the bifurcation process that can readily be implemented in predictive models for braiding evolution, for which the adoption of physically based nodal point conditions would be highly desirable. In the first part of the work the attention had been focused on the quantitative description of the evolution of a single laterally unconstrained channel and analyses had been carried out performing four different sets of experimental runs with both uniform and graded sediments. Two sets of experiments had been carried out on a "Y-shaped" symmetrical configuration, in which the upstream channel diverged into two branches. The experimental results showed the existence of an unbalanced configuration, when the Shields stress reaches relatively low values and the width to depth ratio was large enough. Later the dynamics of river bifurcation were also analyzed in the field for these two field campaigns were performed on the Ridanna Creek, Italy and on the Sunwapta River, Canada, joining an international research group. Theoretical analysis, laboratory and field investigations altogether had allowed a much deeper in sight in the bifurcation process, giving a quantitative detailed description of the phenomenon.

Federici (2003) found by conducting series of experiments on flow isolation phenomena of braided river that although a central bar always develops, the divided flow may continue to flow on both sides of the bar (‗stable'' bifurcation) or may eventually be forced entirely to one side of the bar or the other (‗unstable'' bifurcation). They stated that an unstable bifurcation forms when the flow field was characterized by both a low Shields stress and a non-uniform incoming flow. They established that divergences with erodible banks tend to an equilibrium configuration that depends mainly on the widening ratio of the channel.

Mosselman (2004) employed different categories of methods: theoretical analysis, field measurements, laboratory experiments (elementary process experiments as well as physical scale modeling) and mathematical modeling to define the morphology of river bifurcation. As a consequence, different categories of methods have been used in research on sediment transport and morphology at river bifurcations. A theoretical analysis revealed that the morphological development of bifurcated channels depends sensitively on the way in which sediment transport rates are divided over the two branches. A combination of field measurements and mathematical modeling provided an insight in the effects of grain sorting and alluvial roughness that was not given by previous physical modeling.

Kleinhans et al. (2006) studied about the effect of upstream meanders on bifurcation stability and sediment division in 1D, 2D and 3D models. According to them, at river bifurcations, water and sediment are divided over two branches. The dynamics of the division determine the long-term evolution of the downstream branches, which can be studied by 1D model. For such models, a relation describing the sediment division was necessary, but this was poorly understood. They studied the division of sediment and the morpho-dynamics on a time scale of decades by idealized 2D and 3D modeling of bifurcations with upstream meanders and dominantly bed load transport. They concluded that bifurcations are extremely sensitive to local conditions affecting the secondary currents and the sediment transport direction, and to the downstream boundary conditions. Although most combinations of parameters lead to the development of an asymmetrical discharge division, some combinations lead to a quasi-stable symmetrical division. Finally they discussed the limitations of the models and the applicability to natural meandering rivers.

Obasi et al (2008) constructed a physical model with meandering features and used to investigate the effect of off-take angles on the flow distribution at a concave channel bifurcation. Seven different off-take angles with varied main channel flow rates were used for the study. Predicting equations for the off-take discharge dependent on the off-take angles, main channel discharges, dispersion coefficients and Reynolds numbers were developed and calibrated statistically. Results of the study and predicting equations showed that the offtake discharge increased positively with increases in off-take angles as well as main channel discharges. The developed empirical predicting equations for the off-take discharge gave correlation coefficient values of 9.9974×10^{-1} for both model equations with corresponding standard errors of 9.754×10^{-5} and 9.42×10^{-5}, respectively. It was observed that the predicted off-take discharge values from the model equations compared closely with those of the study suggesting that off-take discharges for concave channel bifurcations could be fairly predicted with the established model equations.

Agunwamba et al. (2009) showed how different offtake angles influence velocity distribution around the canal entrance which would influence the quantity of sediments deposited along the canal bed. The problem of excessive siltation in canals (navigation, irrigation, water supply, etc.) was tackled by the Schwarz-Chrtoffel transformation, neglecting gravity and assuming a constant depth of flow. This implied that large off take angles will encourage more intake of sediments by the canal. In addition, it was also observed that large off take angles exhibit higher and lower (wider range) velocities. That was, near the stagnation point, a large off take angle will possess lower velocities than small off take angles thus encouraging siltation, while near the point of infinite velocity, a large offtake angle will possess higher velocities thereby increasing sediment intake by canal. It was therefore recommended that canals off take angles should be as small as possible but not too small.

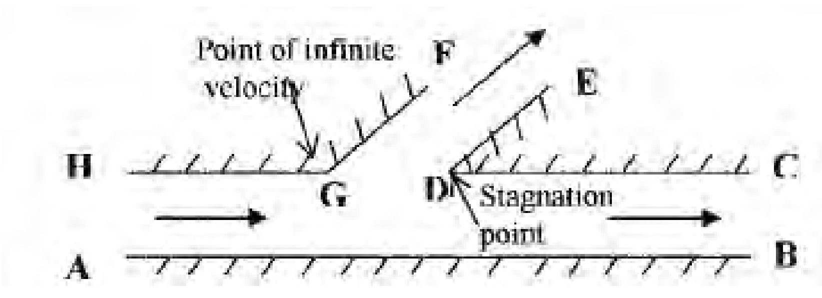


Figure 2.8: Channel with branch canal showing critical points (Agunwamba et al., 2009)

If the off take angle was too small, the bank between the branch canal and the main canal will be eroded gradually leading to flooding and eventual destruction of the canal. The results obtained can be applied to navigation, irrigation and water supply canals. The results obtained show that the larger the off take angle, the higher will be the off take discharge as well as the off take entrance velocity distribution. The results were found to agree with both laboratory data obtained using a model and field data, giving correlation coefficients of 0.76, 0.77 and 0.62.

Haskoning (2012) in collaboration with IWM carried out the Bangladesh River Information and Conservation Project (BRIC). The main objective of the project was to develop a new two-dimensional hydraulic and morphological model for The Gorai off-take area and its surroundings (including data collection and re-calibration), and using the model to assess the effectiveness of the engineering interventions of the selected option that could potentially help improve water flows into The Gorai sub-basin, and reduce salinity in the downstream areas. Based on the changes over the last ten years such as planform changes and river training works constructed in The Gorai and / or the Ganges, the structural interventions of the option might need to be modified. The model was used to analyze the effectiveness of the modified option. As a result suitable modification was made, for which the engineering designs will be updated.

Obasi et al. (2012) examined the effect of offtake angles on spatial distribution of silt material at concave bifurcation. For this purpose, a meandering physical channel was constructed. Four different off-take angles of 30^0, 45^0, 60^0 and 90^0 with varied main channel flow rates were used for the study. Predicting statistical equations dependent on the off-take angles and main channel discharges for the evaluation of the tributary channel sediment intake were developed.

$$\frac{S_2}{S_1} = 1/(1.582\,(\frac{q_2}{q_1})^{0.574}\,\theta^{-0.82}) \tag{2.4}$$

Where, S_2/S_1 is the sediment discharge ratio of offtake channel to main channel, q_1 is the specific discharge for upstream of main channel, q_2 is the specific discharge and θ is offtake angle in radian. It was observed that the predicting equation under estimated the tributary channel sediment yield for off-take angles between 30^o – 70^o and for those between 70^o- 90^o the sediment values were overestimated for all the main channel flow rates considered. The predicting tributary sediment values equaled the experimental values at the off-take angles of 50^o–70^o but varied differently for each of the main channel flow rates.

Ashok (2010) used a two-dimensional numerical model which employed the depth-averaged forms of continuity and momentum equations along with k-c turbulence closure scheme used to simulate the flow at the open channel bifurcation. Performance of the model in predicting discharge distribution, surface profiles, separation zone parameters and energy losses evaluated and discussed in detail. The results of the model generated using McCormack finite volume method were used to calculate the energy losses at the right-angled junctions. Model study showed that the energy losses in open channel bifurcation were similar to those observed in closed conduits, when the submerged flow condition in the branch channel prevails.

Mamun et al. (2012) studied the hydro-morphological analysis of the offtake of the Arial Khan River of Bangladesh to predict its sustainability. Analysis of historical hydrometric data and satellite images near the offtake and selected stations for both the parent Padma River and the bifurcated Arial Khan River had been carried out. It showed that water levels and discharges were in ring trend in both the rivers. In 1975, 2% peak flow of the Padma River was diverted to the Arial khan River which has been increased to about 3% in 2004. The offtake reach of the Arial Khan Upper River which concerned for the study was in the trend of losing conveyance due to aggradations resulted from long-term sedimentation. It revealed that heavy sedimentation had been occurring which leads to formation of sand bars. That eventually had an impact over the dynamics of the offtake of the Arial Khan River. Again, bed topography generated from bathymetric data of 2005 at the junction with the Padma clearly demonstrate that the main channel of the Padma aligned far north which opposite to the offtake and there was a char (sand bar) formed at the mouth of the Arial Khan hindering the flow and the channel has become narrowed. It was also observed from the satellite images of 1974 to 2006, that the char area in the vicinity of Arial Khan offtake was 60 square km in 1974, 72 sq km in 1985, 82 sq km in 1997 and 92 sq km in 2006. The yearly development rate of char area was 2%. The Arial Khan Upper offtake in the trend of abandoning stage and there must be a shifting offtake which was dynamically under development to keep the river morphologically active. Recent bed topography of the offtake, conveyance, trend analysis of recent inflows and stages in the Arial Khan River support that the river going to be the cause of sufferings in terms of flooding in the monsoon and navigability losses and other water resources activities would be hampered due to inadequate inflows in the river.

Hore et al. (2013) studied the off-take dynamics for restoring the Gorai River, a right bank distributaries of the Ganges River with the help of time series satellite images and

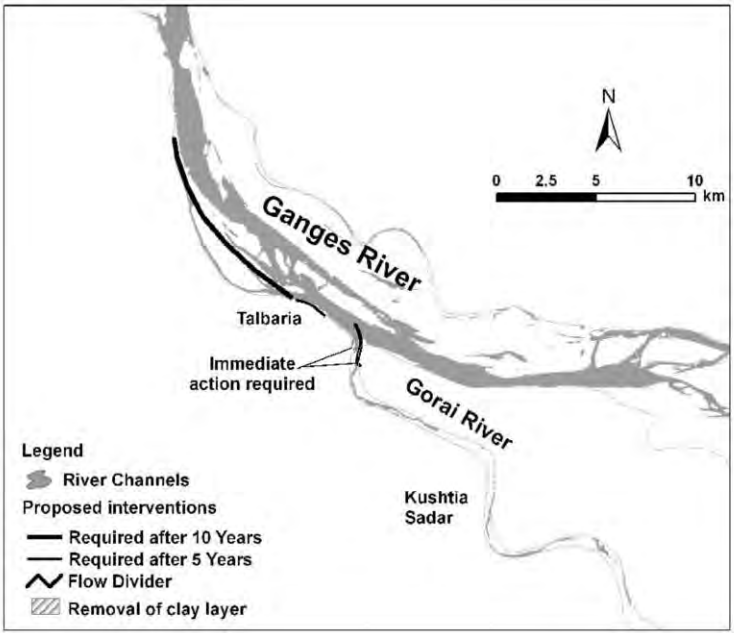


Figure 2.9: Proposed measures in the Ganges for restoring the Gorai River

hydrological data. Analyses of these revealed the causes of deterioration and also helped in deciding probable measures to restore the abandoned distributaries. The high rate of. right bank erosion upstream of Talbaria, and the angle between the Ganges main channel and the Gorai, had a significant influence on the sediment distribution at the off-take. Shortening of the flood recession period of the Ganges would have contributed to the reduction of both dry and monsoon flow in the Gorai. Study results showed that the river processes in the longer time-scale in the active deltas had made it possible to design effective and efficient interventions.

Noor (2013) studied 50 km reach of Jamuna River starting from the 25km upstream of the mouth of Old Brahmaputra offtake to 25km downstream of the offtake and setup these morphological model, MIKE21C. The model was calibrated and validated against the year of 2011 and 2012, respectively. Finally, assessment of morpho-dynamics at high, medium and low flood event of Jamuna for existing and design river bed condition of offtake was conducted. Four different options were chosen considering the dredging and river training works for the simulation. From Noor's observation on Jamuna- Old Brahmaputra Offtake, sediment rating curve of Jamuna and Old Brahmaputra was prepared from the available historical sediment data of Bahadurabad station and Mymensingh station. From 1968-2011, 862 data are obtained and trend analysis was conducted to obtain a sediment discharge equation of Jamuna River,

$$Q_s = 0.133\ Q^{1.474} \tag{2.5}$$

15 data was used for the period of 1993-1996 for Old Brahmaputra River and the obtained equation was the following

$$Q_s = 0.052\ Q^{2.029} \tag{2.6}$$

Where, Q_s and Q is the sediment discharge (ton/day) and water discharge (m^3/s).

Alim-uz-zaman (2017) conducted the trend analysis for same river and his obtained sediment discharge equation was,

$$\text{For Jamuna River, } Qs = 0.3093\ Q^{0.9} \tag{2.7}$$

$$\text{For Old Brahmaputra River, } Qs = 0.0035\ Q^{1.717} \tag{2.8}$$

2.7 Review on previous studies in sand bed flume

Buiyan (1991) investigated phenomenon of confluence scour to understand flow patterns in channel confluences by conducting laboratory tests on different parameters controlling confluence scour. Investigation was done in 50 feet long 20 feet wide and 3 feet deep bed flume of 0.016 longitudinal slope and 1:1 side slope. It was found that reasonable estimation of scour depth at the confluence could be obtained from knowledge of only the average depth of anabranches and angle of incidence. Scour depth increases as confluence angle increases and scour depth decreases as relative discharge and total sediment discharge were increased. The study also showed that, scour depth was maximum when the angle of incidence was symmetrical and discharges in the tributaries are equal.

Hannan (1995) made a laboratory study of sediment distribution at symmetrical 1 meter wide Y-shaped channel bifurcation. In these l-D models the bifurcation phenomena was represented by the help of nodal point relations. Experiments were carried out with two noses representing two different bifurcation conditions. For both cases, it was found that with the increase of discharges the value of m (nose geometry) did not remain constant rather it increased. This was occurred because the shape of the nose created an additional influence in the model. He plotted the data of the experiments to the following nodal point relation

$$\frac{S_2}{S_3} = k.\left(\frac{q_2}{q_3}\right)^m \qquad (2.9)$$

where, q_2, q_3, S_2 and S_3 are discharges and sediment transports per unit width respectively and k and m are constants. The subscripts 2 and 3 represent branch 2 and branch 3 respectively. He concluded that the value of k and m are not the same for the same nose for different discharges. For each nose he found that the value of m increases with increase in discharge. He also found that the value of m is greater than 5/3 for all the three discharges (20 l/s, 30 l/s and 40 l/s) and concluded that it fits well with the theoretical analysis.

Islam (1996) described the influence of nose angle (the angle between the tip of the nose and the symmetrical line of a bifurcation) on sediment distribution at channel bifurcation. It was done on same setup constructed by Hannan. A total of four different noses have been used to investigate the influence. For each nose, three upstream discharges have been used. He found that as the nose angle changes, the power and the coefficient of the nodal point relation (Equation 2.8) change to a great extent. The value of the exponent, k in the nodal point relation increases as the discharge increases when the nose angle is held constant. This study concluded that the distribution of sediment to the downstream branches is independent of upstream discharge. The nose angle is the major variable for the distribution of sediment.

Rahman (2002) made a study on bed level changes of alluvial river using hydraulic model which works on principle of McCormack explicit finite difference scheme. To verify the model experiment was carried out in 12 meter long 1 meter wide and 0.61 tall sand bed flume. With the help of test results coefficient ‗a' and ‗b' of sediment transport equation was determined. For each run, four transient bed profiles were plotted at one hour interval of flow. The computed results are compared with the experimental data which came out satisfactory.

Haque (2010) experimentally investigated and studied the flow behavior around launching apron in a laboratory channel of 11 meter long and 1 meter wide for both fixed bed and mobile bed. Apron materials were fabricated using sand-cement of varying sizes; geobags were placed on the toe of sloping revetment during test runs. The study showed that the presence of apron at the toe of revetment caused changes in the velocity distribution near bed. In most of the cases, shifting away of maximum velocity was observed that caused shifting of maximum discharge flux away from the revetment. It was also found that placing of apron in different arrangement not only reduced the maximum scour depth but also had changed the deposition pattern around revetment.

Hossain (2011) investigated the behavior of launching apron in a straight rectangular laboratory channel of 11 meter long and 2 meter wide. For scour study three vertical-wall of 0.5 inch thick particleboards and three sloping wall abutments of metal were used. The investigation showed launching apron was protect structure in two ways - one was by forming a protective layer on the sloping face of the developed scour hole and the other was by forcing the scour hole to be formed away from the structure.

Sadeque (2012) studied on bed scour around bridge foundations from the perspective of bridge hydraulics problems of Bangladesh. Study provided an indication of design modification for riprap protection around pier foundation considering the velocity amplification close to the structure.

Hasan (2003) experimentally analyzed local scour at toe protected embankment. It was found that for both vertical and slopping wall structure the scour value was closer to Lacey‘s method.

Hossain (2011) made an experimental study on settling behavior of toe protection elements of river bank protection works. The study dealt with the investigation of the settling phenomena and threshold condition for movement of toe protection elements in bank protection structures in the context of conventional underwater construction procedure followed in Bangladesh. For the purpose protection elements (e.g. CC blocks and geobags) were used to assess the fall velocity of these particles. Therefore, fall velocity of CC blocks, geobags and a combined outcome of the experiments were described. An empirical relationship was developed to estimate horizontal settling distance of a toe protective element after dumping. Verification of the proposed relationship using independent set of laboratory data had been done and showed satisfactory agreement with an error of 3.80%. Sixteen experimental runs with eight types of elements had been conducted for investigating incipient condition for discharges ranges from 0.033 m^3/s to 0.052m^3/s and an empirical equation was provided which could be used to generate values of the size of toe protection elements. These results are then utilized to study the horizontal settling distance of the elements when they are dumped in the flowing water.

Ahmed (2014) made an experimental study on placement of toe protection elements of river bank protection works under live bed condition. The study was undertaken to investigate experimentally two important aspect of underwater construction such as the placing behavior and incipient condition of toe protection elements under live bed condition.20 cm thick layer of sand was used in 12.75 m trapezoidal channel of bottom width 0.6m and top width 1 m. Three types of CC blocks and five types of geobags were used to conduct fifteen experimental runs with two different hydraulic conditions to investigate placing behavior. Experiment result showed that both CC block and geobags exhibit greater shear velocity and shear stress in case of live bed than that of fixed bed.

Above review of literatures reveal that most the studies were conducted on channel bifurcation with symmetrical or non-symmetrical nose etc. In sand bed experimental setup of DWRE, numbers of studies were conducted but most of these were related to the flow and scour analysis for structural measures in river and streams.

CHAPTER THREE
THEORITACAL CONSIDERATIONS

3.1 Introduction

Flow through an offtake bifurcation can be compared to a channel junction, which involves number of variables such as shape and slope of the channel, angle of intersection, direction and discharges of flow, rounding at the corner of the junction etc. The problem is so complicated that only a few simple and specific cases have been studied and most cases the conclusion indicates that generalization of the problem is not possible as dividing flow problem cannot be easily analyzed theoretically. So, offtakes or distributaries that usually bifurcated from the major rivers are the most uncertain part of the river. The purpose of this chapter is to give a brief description of the related theories regarding the offtakes.

3.2 Flow Distribution in Bifurcation

When flow in a stream is divided by a long island, the division of flow between the two channels may be determined approximately with the aid of flow-profile computations. If flow of the channel is considered to be subcritical calculation is done by assuming first a set of discharges Q_1 and Q_2 for the divided flows such that the sum of the discharges is equal to the total discharge Q (Chow, 1959). Then, flow profile is computed in the two channels past each side of the island to a point *A* (**Figure 3.1**) where the flow is divided. Since the flow is subcritical, the computation shouldproceed upstream from the downstream point *B* where the divided flows unit again

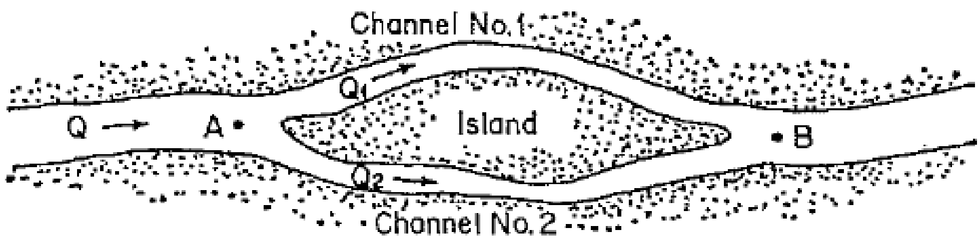


Figure 3.1: Flow through bifurcated channel

If normal flow condition prevails it may be assumed that the flow is uniform and division of flow is approximately estimated from the following equation

$$Q_1=K_1\sqrt{S_1} \tag{3.1}$$

$$Q_2=K_2\sqrt{S_2} \tag{3.2}$$

$$\text{and } Q= Q_1+Q_2 \tag{3.3}$$

Where, Q is discharge, K in conveyance, S is slope of the channel and 1, 2 subscript indicated channel number 1 and 2 as in the **Figure 3.1**.

For specific case of subcritical flow passing through a channel at a junction is investigated by Taylor for horizontal channel with equal width with few assumptions to simplify the calculation. The assumptions are a) channels 1 and 3 lie in a straight line; b) the flow is parallel to the channel walls, and the velocity is uniformly distributed immediately above and below the junction and c) ordinary wall friction is negligible in comparison with other forces involved. The application of the momentum principle is difficult because it involves some unknown quantities and assumptions were not sufficient to analyze the discharge distribution as determined for confluence. An experimental approach for specific case is shown in **Figure 3.2**.

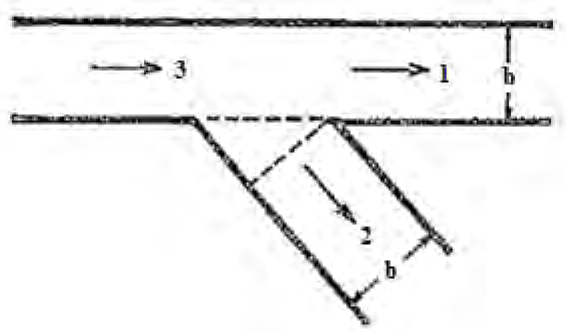


Figure 3.2: Dividing flow at simple channel junction

Basically, the division of the flow depends upon the backwater effects of the two branch channels and the dynamic conditions existing at the junction (Chow, 1959). If the divided flow is to be combined together again at a certain point downstream, a solution of the problem could be done using (Equations 3.1 to 3.3). But if the divided flow does not combine in at a single point (**Figure 3.2**) analysis could be done with an experimental approach as proposed by Taylor. For a given value of diverging angle (θ) dimensionless ratios Q_2/Q_3, y_3/y_2, y_1/y_2 and $k_3= V_3^2/2gy_3$ were derived from experimental data. For $\theta = 90^0$ curves (**Figure 3.3 and 3.4**) are plotted for these dimensionless quantities. These curves can be used to determine the division of flow of a given discharge Q_3. Assuming Q_1, Q_2 is determined from $Q_2 = Q_3 - Q_1$. The corresponding depths y_1, and y_2 can be obtained from the rating curves of channels 1 and 2. For y_1/y_2, the ratio y_3/y_2 can be determined from **Figure 3.3**. By assuming other values of Q_1, the corresponding ratios y_3/y_2 can be obtained. Thus, Q_2/Q_3 is plotted against y_3/y_2, as shown by the curve A in **Figure 3.4**. The intersection of this curve with K_3 curve gives all possible combination of the variables among which one value will correspond the actual flow. At next step intersected k_3 values is plotted against the corresponding y_3. From the rating curve of channel 2, the y_2 corresponding to the Q_2 just computed can be found. The plot of k_3 against y_3 can be constructed, as shown by the curve B in (**Figure 3.4 right**). The intersection of the two curves gives the required values of k_3 and y_3. With this k_3, the corresponding value of Q_2/Q_3 could be obtained from (**Figure 3.4 left**) and divided discharge Q_1 and Q_2 was divided accordingly. But due to some uncertainty and time consuming procedure this procedure was not put into practice.

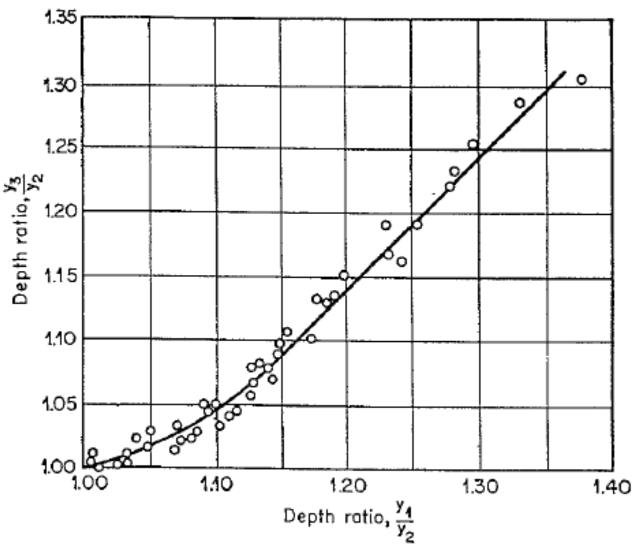


Figure 3.3: Relationship between depth ratio for $\theta = 90^0$

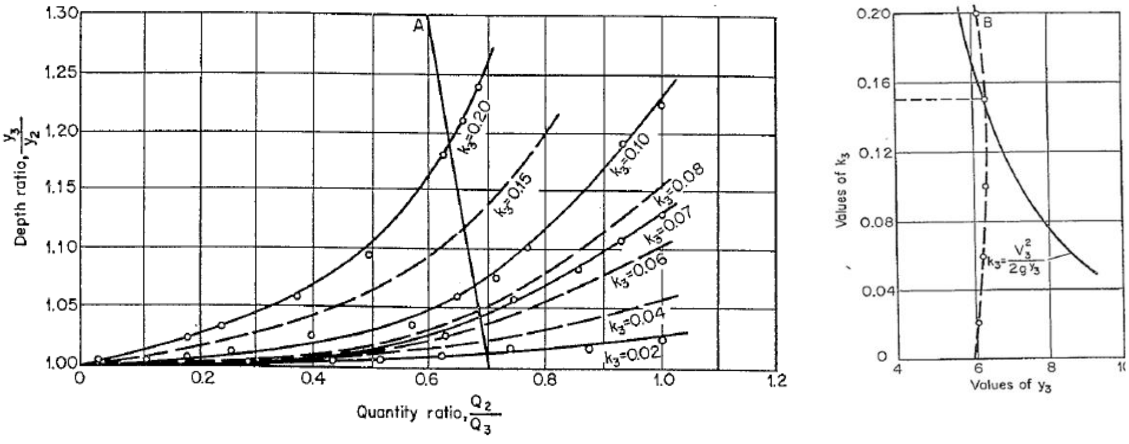


Figure 3.4: Characteristics of 90^0 flow division

3.3 Dimensional Analysis

One of the main objectives of engineering research is discover the governing relationships that may exist between various physical quantities. Hence the great importance that attaches to analytical and empirical methods and techniques to investigate possibilities for such relationships. Having defined the necessary variables and parameters which control a system, the ideal situation is to have a complete analytical solution to the problem under investigation. But there are often complexities in behavior. In many cases difficulties arise in quantifying the problem analytically. In order overcome such difficulties drastic simplification is often made. This may even lead erroneous solutions. This can be achieved by empirical and numerical methods which may give some information or approximate solution to the problem, although these may have a limited range of applicability.

For the purpose of solving several engineering problems a mathematical technique used to study the dimensions of involved parameter is called dimensional analysis. Each physical phenomenon can be expressed by an equation giving relationship between different quantities; such quantities are dimensional and non-dimensional. Dimensional analysis helps in determining a systematic arrangement of the variables in the physical relationship, combining dimensional variables to form non-dimensional parameters. It is based on the principle of dimensional homogeneity and uses the dimensions of relevant variables affecting the phenomenon.It is especially useful in presenting experimental results in a concise form.To plan model tests and present experimental results in a systematic manner; thus making it possible to analyze the complex fluid flow phenomenon dimensional analysis plays important role. It enables getting up a theoretical equation into simplified and dimensional form. It also provides partial solutions to the problems that are too complex to be dealt mathematically.

The various physical quantities used in fluid phenomenon can be expressed in terms of fundamental quantities or primary quantities. The fundamental quantities are mass, length, time, temperature, designated by the letters, M, L, T, θ respectively. Temperature is especially in compressible flow. The quantities which are expressed in terms of the fundamental or primary quantities are called derived or secondary quantities, (e.g. velocity, area, acceleration etc.). The expressions for a derived quantity in terms of the primary quantities are called the dimension of the physical quantity.

3.3.1 Methods of Dimensional Analysis

There are number of methods for analysis the dimensionless quantities are available in literature. Among these methods, Buckingham -π, Rayleigh and echelon matrix methods are discussed in the following.

Rayleigh Method

This method gives a special form of relationship among the dimensionless group, and has the inherent drawback that it does not provide any information regarding the number of dimensionless groups to be obtained as a result of dimensional analysis. Rayleigh's method is used for determining the expression for a variable which depends upon maximum three or four variables only. In case the number of independent variables becomes more than four, then it is very difficult to find the expression for the dependent variable.

In this method a functional relationship of some variables is expressed in the form of an exponential equation which must be dimensionally homogeneous. Thus if X is a variable which depends on X1, X2, X3,...Xn; the functional equation can be written as:

$$X=f\,(X1\ ,\ X2,\ X3,.......Xn) \qquad (3.4)$$

In the above equation X is a dependent variable, while X1, X2, X3,...Xn are independent variables. Equation 3.4 can also be written as:

$$X= C(X1^{a},\ X2^{b},\ X3^{c},.....Xn^{n}) \qquad (3.5)$$

Where, C is the constant and a, b, c,..., n are the arbitrary powers. The value of a, b, c are obtained by comparing the powers of the fundamental dimensions on both sides. Thus the expression is obtained for dependent variable.

Buckingham's -π Method

When a large number of physical variables are involved Rayleigh's method becomes increasingly laborious and cumbersome. Buckingham designated the dimensionless group by π, it is therefore often called Buckingham's-π method. The advantage of this method over Rayleigh's method is that it let us know, in advance of the analysis, as to how many dimensionless groups are to be expected.

The Buckingham's π-theorem states as follows:

"If there are n variables (dependent and independent variables) in a dimensionally homogeneous equation and are these variables containing m fundamental dimensions (such as M, L, T etc.) then the variables are arranged into (n-m) dimensionless terms. These dimensionless terms are called π -terms. "

Mathematically, if any variable X1, depends on independent variables, X2, X3, X4,...... Xn; the functional equation may be written as

$$X1=f\,(X2, X3, X4.......Xn) \tag{3.6}$$

Or,

$$f\,(X1,X2, X3, X4.......Xn) = 0 \tag{3.7}$$

It is a dimensionally homogeneous equation and contains n variables. If there are m fundamental dimensions, then according to Buckingham's π- theorem, Equation 3.7 can be written in terms of number of π- terms (dimensionless groups) in which number of π -term is equal to (n-m). Hence Equation (3.7) becomes

$$f1\ (\pi_1,\pi_2,\pi_3,.....\pi_{n-m})= 0 \tag{3.8}$$

Each dimensionless π term is formed by combining m variables out of the ot n variable one of the remaining (n-m) variables. Repeating variables are chosen from among the variables such that they together involve all the fundamental dimensions they themselves do not form a dimensionless parameter. Let, in the above case X2, X3 and

X4 repeating variables if the fundamental dimensions in (M,L,T)= 3, then each term is writen as

$$\left.\begin{aligned} \pi_1 &= X2^{a1}.\ X3^{b1}.X4^{c1}.X1 \\ \pi_2 &= (X2^{a2}.\ X\ 3^{b2}\ .X4^{c2}.X5) \\ &. \\ &. \\ \pi_{n\text{-}m} &= (X2^{a}{}_{n.m}.X3\ {}^{b}{}_{n.m}\ .X4\ {}^{c}{}_{n.m}.Xn) \end{aligned}\right\} \quad (3.9)$$

Where a1, b1, c1; a2, b2, c2 etc are const constants, which are determined by considering, dimesional homogeneity. The final general equation for the phenomena may then be obtained by expressing anyone of the π-terms as a function of the other as

$$\left.\begin{aligned} \pi_1 &= \varphi(\pi_2.\ \pi_3.\ \pi_4.......\ \pi_{n\text{-}m}) \\ \pi_2 &= \varphi(\pi_1.\ \pi_3.\ \pi_4.......\ \pi_{n\text{-}m}) \end{aligned}\right\} \quad (3.10)$$

Echelon Matrix Procedure

A basic procedure for dimensional analysis is Barr's analyses method. It was claimed by Barr that the procedure offers advantages in straight forward execution over the traditional procedures such as those ascribed to Rayleigh or to Buckingham method. Langhaar extended Buckingham procedure by showing necesseary form of pre analysis check. Barr (1985) also showed advantages of echelon matrix method on checking the rank of dimensionless matrix. The basic procedure of echelon matrix is as follows:

1. Arrange the variables in preferred of repeating variables from the left hand side, and so as to keep those which are considered dependent to the right hand side.
2. Complete the dimensional matrix in terms of the basic dimensions in which the variables are expressed.
3. Form unit matrix under the repearting variables. The necessary invariance of the net dimensions of the non-repeating variables allows the solution matrix to be completed.
4. If echelon matric does not arise naturally, first step is to force echelon in an intermediate matrix formation. In such cases, an intermediate matrix in reverse echelon formation is obtained in respect of the new row dimensions.

So a matrix in row echelon form has the following properties and Equation 3.11 is a matrix in row echelon form.

The first non-zero entry of every row is 1.
2. The first non-zero entry in every row is one position right to the previous row.
3. The row with all zero elements will be below the rows having a non-zero element.

$$\begin{bmatrix} 1 & 3 & 0 & 4 \\ 0 & 1 & 2 & 1 \\ 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} \tag{3.11}$$

3.4 Sediment Movement in Rivers

An important aspect of fluvial processes is the movement of sediment in rivers, to which river morphology and river channel changes are closely related. The term load, as used in sediment transport, may refer to the sediment that is in motion in a stream. It is also used to denote the rate at which sediment is moved, for example, cubic feet per second or tons per day. The latter usage is preferred in river morphology. Sediment transport consists of bed load transport and suspended transport.

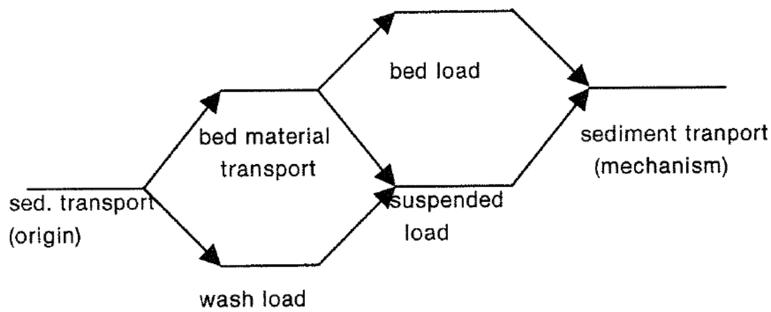


Figure 3.5: Classification of sediment transport (Jansen et.al., 1979)

3.4.1 Sediment Transport Equations

Sediment movement of any channel is controlled by stage, discharge, velocity, water surface and bed slope, shear stress, mean particle diameter and stream power. Only a single equation cannot incorporate all these variables and predicts the sediment load. For this reason, different equations have been put forward on the basis of different independent variables. From available empirical formulas following three well known equations have been selected for the present study.

Engelund-Hansen Equation (1967)

Engelund-Hansen's (1967) equation is based on the shear stress approach. In developing the equation Engelund-Hansen's relied on data from experiment in a specific series of test in a large flume. The sediment used in this flume had mean diameter of 0.19 mm, 0.27 mm, 0.45 mm and 0.93 mm. The equation can be written as:

$$g_s = 0.05\gamma_s V^2 \sqrt{\frac{D_{50}}{g\left(\frac{\gamma_s}{\gamma}-1\right)}} \cdot \left[\frac{\tau_0}{(\gamma_s-\gamma)D_{50}}\right]^{3/2} \quad (3.12)$$

where, $\mathbf{g_s}$ is sediment transport per unit time per unit width, $\boldsymbol{\gamma_s}$ and $\boldsymbol{\gamma}$ is specific weight of sediment particles and water respectively, $\boldsymbol{\tau_o}$ is the bed shear stress, $\mathbf{D_{50}}$ is median diameter of bed material, **V** is the average flow velocity and **g** is acceleration due to gravity. This equation is dimensionally homogeneous and any consistent set of units can be used.

Ackers-White Formula (1973)

Based on Bagnold's stream power concept, Ackers-White related the concentration of bed material load as a function of the mobility number Fg

$$C_s = cs\,\frac{d}{R}\left(\frac{U}{U_*}\right)^n \left(\frac{F_g}{A}-1\right)^m \quad (3.13)$$

Where, **n, c, A** and **m** are coefficients. The mobility number $\mathbf{F_g}$ is given by

$$F_g = \frac{U_*^n}{[gd(s-1)]^{0.5}}\left[\frac{U}{\sqrt{(32)}.\log(10R/d)}\right]^{1-n} \quad (3.14)$$

The first part of F_g reflects the power expenditure associated with turbulent intensity of the flow. The coefficient n is the transition exponent, which depends on the sediment size; it is zero for coarse sediments with bed load only. The coefficient A may be interpreted as the critical value for F_g.

They also expressed sediment size by a dimensionless grain diameter d_g

$$d_g = d\left[\frac{g(s-1)}{\nu^2}\right]^{1/3} \quad (3.15)$$

The coefficients are determined from best fit curves of almost over 1000 sets of data with sediment size greater than 0.04 mm and Froude number less than 0.8. Values of these coefficients are listed as follows:

Table 3.1: Values for Ackers- White formula coefficients

Coefficient	dg > 60	60≥ dg≥1
c	0.025	log$c$= 2.86 logd_g- (logd_g)^2 -3.53
n	0.0	1-0.56 log d_g
A	0.17	0.23/(d_g)$^{(1/2)}$ +0.14
m	1.50	9.66/d_g +1.34

Van Rijn's Equation (1984)

A simplified method was given by Van Rijn for calculating suspended sediment transport. This method is based on computer computations in combination with a roughnesspredictor. Using regression analysis, the computational results for a depth range of 1 to 20 m, a velocity range of 0.5 to 2.5 m/s and a particle range of 100 to 2000 μm were represented by a simple power function, as follows:

$$\frac{q_{s,c}}{\bar{u}h} = 0.012 \left(\frac{\bar{u}-\bar{u}_{cr}}{(s-1)gd_{50}{}^{0.5}}\right)^{2.4} \left(\frac{d_{50}}{h}\right)\left(\frac{1}{D_*}\right)^{0.6} \qquad (3.16)$$

Where, $q_{s,c}$ is volumetric suspended load transport in m^2/s, $\bar{u}_{cr}$ is the critical depth-averaged velocity according to Shields, h is the water depth and $\bar{u}$ is the depth-averaged velocity. For calculation of critical depth average velocity critical velocity of quartz material (National Engineering Handbook, Dept. of Agriculture, USA) has been used. Shields velocity can be determined if channel velocity and grain size is known. It requires $\bar{u}$, h and d_{50} as input data and can be used to get a first estimate of the suspended load transport.

It is assumed that the instantaneous bed-load transport rate is related to the instantaneous T parameter, as follows

$$q_b = 0.1\,(s-1)^{0.5}\, g^{0.5} d_{50}{}^{1.5}\, D_*{}^{-0.3} T_m{}^{2.1} \qquad (3.17)$$

In which, T_m is instantaneous shear stress parameter= ($\tau b'$−τb,) /τ,

τ_b' is instantaneous effective bed shear stress which is equal to $\mu\tau_b$, τ_b is the instantaneous critical bed shear stress and $\boldsymbol{D_*}$ is dimensionless particle parameter.

When the bed load transport and the suspended load transport are known, the total load transport of bed material can be determined by

$$q_t=q_b+q_s \tag{3.18}$$

Where, $\boldsymbol{q_t}$ is the total load transport of bed material, $\boldsymbol{q_b}$ is bed load transport and $\boldsymbol{q_s}$ is suspended load transport.

3.4.2 Sampling for Sediment

The methods and equipment used for sampling suspended sediment are different from those used for deposited sediments. For bottom sediments it may be necessary to collect deposited sediments with minimum disturbance in order not to lose the fine material on the sediment surface, or because thevertical distribution of the sediment components is important. In deep waters this necessitates the use of grabsor corers, but in shallow water a scoop or spatula may be used.

There are four main types of samplers (Ongley, 1996) for suspended sediments:

- Integrated samplers,
- Instantaneous grab samplers,
- Pump samplers, and
- Sedimentation traps.

In practice, depth-integrating samplers are lowered to the river bottom, then immediately raised to the surface; lowering and rising should be done at the same rate. The objective is to fill the sampler to about 90 per cent capacity; if the sampler is completely full when it emerges from the water the sample will be biased because the apparatus will have stopped sampling at the point at which it filled up.

Large, heavy samplers are usually only necessary when samples must be obtained from abridge, boat or similar situation. In shallow streams, where all points can be reached by wading, a bucket (if nothing else is available) or a small sampler attached to a metal rod can be used. It is possible to make a simple depth-integrating sampler for use in shallow streams, using a wide-mouth, one liter bottle, a rubber stopper and short pieces of rigid tubing. The tubing forms the water inlet and air outlet.

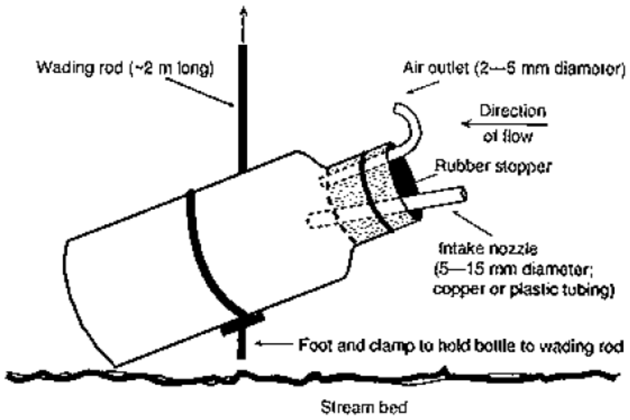


Figure 3.6: Home-made suspended sediment sampler

3.4.3 Measuring Suspended Sediment

Particle size distribution and concentration not only vary in the vertical section, but may alsovary considerably across a river section. Therefore, measuring suspended sediment concentration must take into account these variations. This becomes especially important when suspended sediment concentration is being measured for the purpose of calculatingsediment load in a river. Sediment-discharge measurements usually are available on a discrete or periodic basis. There are two generally accepted methods for measuring suspended sediment concentration for load determination. One is based on direct measurement of the quantities of interest, and the other on relations developed between hydraulic parameters and sediment transport potential.

Equal Discharge Increment Method

This method requires first that a complete flow measurement be carried out across the cross-section of the river. Using the results, the cross-section is divided into five (more on large or complex rivers) increments (i.e. vertical sections) having equal discharge. The number n of increments is based on experience. Depth integrated suspended sediment sampling is carried out at one vertical within each of the equal-discharge-increments, usually at a location most closely representing the centroid of flow for that increment. The sediment concentration for each equal-discharge-increment is measured and the mean discharge-weighted suspended sediment concentration (SSC) is obtained by taking the average of the concentration values C obtained for each interval *i*.

$$\mathrm{SSC} = \frac{\sum_{i=1}^{n} C_i}{n} \tag{3.19}$$

The discharge-weighted suspended sediment load (SSL), in tons per day, for the river cross-section is obtained by multiplying the concentration, **C**, in ppm (mg/l) by the discharge, **Q**, in m^3/s of each equal discharge increment, **i**, and summing for all increments.This method is very time consuming, but is that most used by sediment agencies.

$$S\mathrm{SL} = \sum_{i=1}^{n} (C_i\, Q_i) X 0.0864 \tag{3.20}$$

Equal Width Increment Method

This method is used without making flow measurements and is usually used in small to medium rivers and especially rivers that are shallow enough for wading. The operator marks off 10-20 equal intervals across the river cross-section. At the deepest point, the operator takes a depth integrated sample, noting the transit rate of the sampler (i.e. the uniform speedat which the sampler is lowered, then raised to the surface). Using that same transit rate, asuspended sediment sample is taken at each of the intervals. Because each vertical will have a different depth and velocity, the sample volume will vary with each vertical sampled. All samples are composited into a single container which is then agitated and sub-sampled, usually two or three times, and analyzed for suspended sediment concentration. The average of these analyses is the mean cross-sectional suspended sediment concentration. In this method, the results are corrected for differences in discharge at each section by virtue of using the same transit rate (and thesame nozzle diameter) at all sections i.e. a shallow section with less discharge will producea proportionally smaller suspended sediment sample than a deep section having greater discharge.

Though there are no universally accepted rules for sampling, many scientists will collect a grab sample from adepth of 0.5 m at the point of maximum flow in the cross-section. For larger rivers, or rivers where there is concern over cross-sectional variation, grab samples can be taken from several locations across the section and integrated. For more exacting work where accurate loads are required, especially for micro-pollutants, sampling should be carried out usingeither of the methods noted above. It is particularly important to avoid sampling near river banks (or lake shores) where elevated concentrations of suspended matter occur and whichare often contaminated by garbage and other anthropogenic materials.

Suspended Sediment Concentration Interpolation Method

The most commonly used method by USGS is based on the derivation of a temporal relation by interpolating between measured suspended sediment concentration values and using measured and estimated concentration values with time weighted water discharge values to calculate suspended sediment discharges (John, 2008). To derive daily suspended sediment discharges the following equation is used

$$Q_s = Q_w C_s k \tag{3.21}$$

Where, $\mathbf{Q_s}$ is suspended-sediment discharge, in tons per day; $\mathbf{Q_w}$ is water discharge, in cubic feet per second or cubic meters per second; $\mathbf{C_s}$ is the mean concentration of suspended sediment in the cross-section in milligrams/liter and **k** is a coefficient based on the unit of measurement of water discharge that assumes a specific weight of 2.65 for sediment which is equals to 0.0027 in inch-pound units, or 0.0864 in SI units.

Reliable suspended sediment records cannot be obtained unless all concentration values used in the computation are representative of the mean cross-sectional value.

Transport Curve Method for Suspended Sediment Load, Bed Load and Total Load

The empirical relation between water discharge and sediment concentration (or sediment discharge) at a site can be expressed graphically as a single average relation

$$Q_s = a{Q_w}^b \tag{3.22}$$

Where, $\mathbf{Q_s}$ is the suspended sediment discharge, in tons per day or tons per day; $\mathbf{Q_w}$ is the water discharge, in cubic feet per second or cubic meters per second; **a** is the intercept or coefficient and **b** is the slope or exponent.

The rating coefficient **a** contains information for converting discharge Q into sediment concentration C_s and information about the offset of the rating line in log-log space. Rating relationships provide an empirical method to convert discharge hydrographs into sediment load estimates (Syvitsi and Alcott, 1995) the statistical relationship between suspended load and stream discharge is called the rating curve and commonly takes the power law Equation 3.22.

For determining the sediment discharge of the experimental main and offtake channel, equal width interval method was used while for sediment discharge comparison in field condition suspended sediment concentration interpolation method was used.

CHAPTER FOUR
METHODOLOGY

4.1 Introduction

In order to achieve the objectives of the study six stepwise methodologies have been followed. In this chapter these steps followed to fulfill the objectives of the study have been elaborated and a flow diagram has been provided for better understanding of workflow. Steps include theoretical study, experimental channel design and data collection, observation and analysis.

4.2 Methodology

The following steps were adopted to accomplish the activities mentioned in the objectives:

- Theoretical analysis of governing parameters
- Design of experimental channel
- Test scenarios and experimental run
- Data collection and observations
- Data analysis and development of dimensionless equations
- Predictive performance of developed equation with field data.

The stepwise methodology can be better outlined in a flow diagram as shown in Figure 4.1.

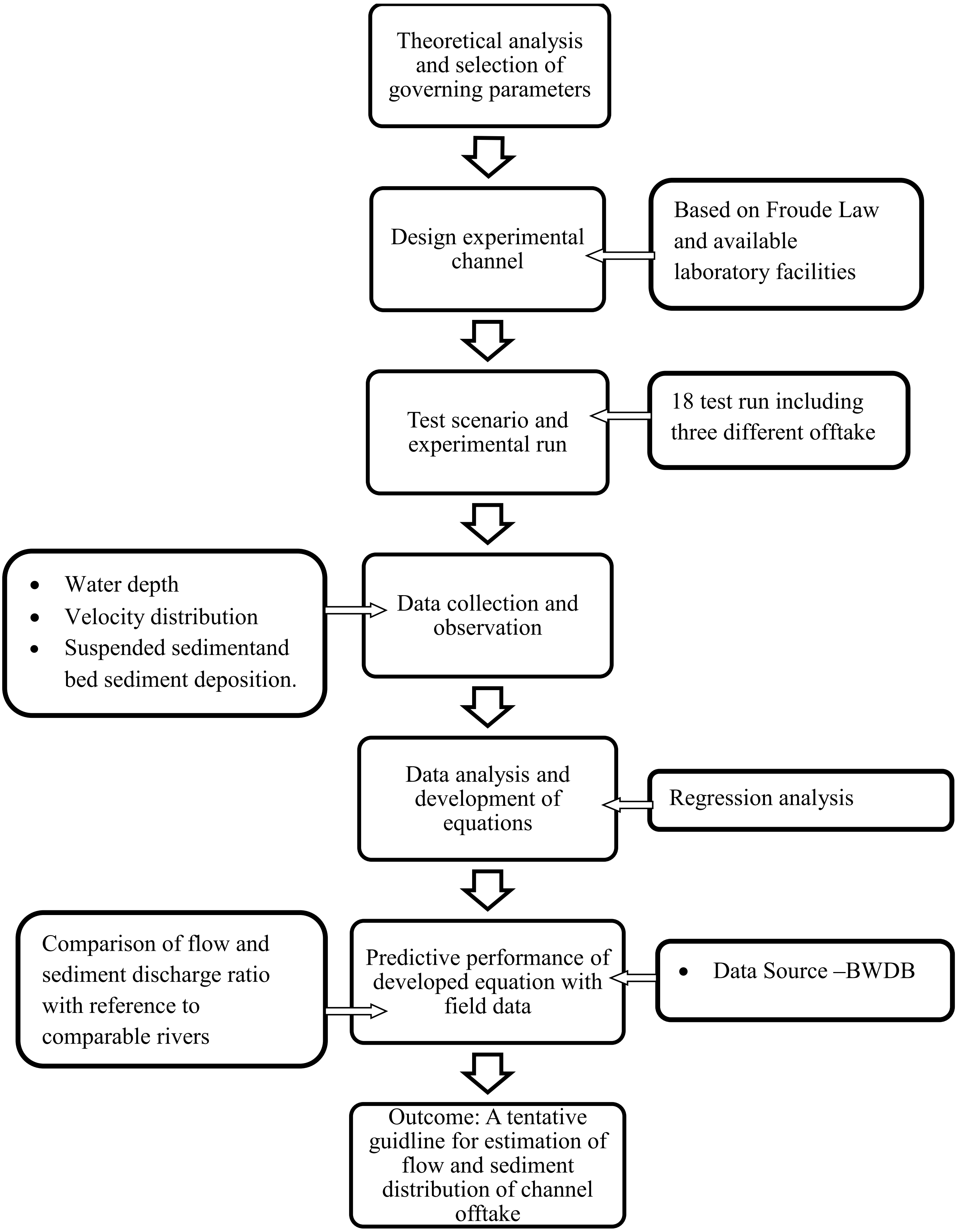


Figure 4.1: Flow diagram showing methodology of the study

4.2.1 Theoretical Analysis of Governing Parameters

Flow phenomena in bifurcated channel govern by water discharge, channel geometry, conditions of the approaching channel, sediment flow, angle of offtake etc. Literature reviews on various researches conducted on bifurcation study and sediment transport were used to determine governing parameters. Later, dimensional analysis has been performed to determine the relationships among the parameters.

4.2.2 Design of Experimental Channel

Experimental setup was developed in the Hydraulics and River Engineering Laboratory of the Department of Water Resources Engineering, BUET. Offtake channel is rectangular in cross-section also with a width of 40 cm. The main channel is a rectangular channel which is 11 m long and 1 m wide. Sediment used in the experiment has a median grain diameter of 0.23 mm. Sand feeder and sediment distributor have been used to distribute the sediment evenly at the inflow section of the main channel. To conduct test runs, main channel was designed and constructed from which an offtake channel was bifurcated. Experimental setup was designed considering the dimensional scale ratio using Froudian law. Test runs were done by changing the angle of offtakes to 20^o, 40^o and 60^o. For the experimentation three different discharges have been set for each of the angle of offtake.

4.2.3 Test Scenarios and Experimental Run

Considering different hydraulic parameters like velocity, bed shear stress, free board, water depth etc. discharge range 320 m^3/h to 120 m^3/h were maintained. During the experimental run flow velocities and depths were observed for all test discharges. Sediment discharge calculation was done for 320 m^3/h of discharge for each offtake angle. For determining sediment discharge, suspended sediment sample has been collected from selected cross sections. In addition to this two extra test runs have been conducted in mobile bed conditions for flow observation.

To accomplish the objectives 18 test scenarios have been set for the study. Experiments were conducted with three different angles of offtake and with three different discharge conditions. Test scenario table has been provided in **Table 4.1**.

Table 4.1: Test Scenarios

Test Run No.	Offtake angle	Sediment charge in main channel (Kg/h)	Discharge (m^3/h)
1	NO	--	320
2			220
3			120
4		100	320
5	20^{o}	--	320
6			220
7			120
8		100	320
9	40^{o}	--	320
10			220
11			120
12		100	320
13	60^{o}	--	320
14			220
15			120
16		100	320
17	60^{o} Mobile Bed	100	320
18		100	220

4.2.4 Data Collection and Observations

Experimental observation was carried out for various angles of offtake from main channel. First test was done to observe the flow and sediment behavior of channel without any offtake. Later data was collected for 20^{o}, 40^{o} and 60^{o} offtake angle. Observation was made within the test section of 5m near the junction, in such a way that the flow is stabilized and uniform within the section. For each test run velocity, depth of flow, flow distribution and sediment distribution was determined within the experimental reach. At last flow visualization was made which would help in understanding flow behavior and erosion-deposition behavior at the vicinity of offtake.

4.2.5 Data Analysis and Development of Dimensionless Equations

Experimental data were collected and analyzed to understand the changes of hydraulic and sediment parameters with the change of offtake angle. Powers of dimensionless parameters have been determined from the experimental data. Two dimensionless equations have been obtained for determination of water discharge and sediment discharge ratio. Additional to this, three well known sediment transport formulas i.e. Ackers-White, Engelund-Hansen and Van Rijn formula have been used to estimate and compare the experimental sediment discharge ratio.

4.2.6 Predictive Performance of Developed Equation with Field Data

Relationships obtained from this study were compared with the field data. For this purpose, field data for selected offtakes of Bangladesh has also been used to verify the obtained relationships. Used field data for comparison was obtained from BWDB and review of previous literatures. Thus, four offtake systems of Bangladesh, Upper Korotoya- Dhepa, Surma- Botor Khal (offtake of Surma), Old Brahmaputra- Jhenai offtake and Ganges- Gorai have been taken for relative comparison of proposed flow discharge distribution equation. For comparison of sediment discharge ratio sediment data of Ganges- Gorai River system has been used.

CHAPTER FIVE
EXPERIMENTAL SETUP AND OBSERVATION

5.1 Introduction

Experimental data from the present study was collected from a sand bed facilities located in the Hydraulics and River Engineering Laboratory of Water Resources Engineering Department (DWRE) of Bangladesh University of Engineering and Technology. Detailed experimental setup, experimental procedure and data collection techniques are discussed in this chapter. A brief description of the observations during experimentation is also presented.

5.2 Design of Experimental Channel

The experimental channel has been designed based on two main criteria. Firstly, the available space and the discharge capacity of the pump. Secondly, Shield criteria for the incipient condition of the bed material.

Determination of Channel Size

In addition to pump capacity, the extent of the tail gate movement has also been considered for the determination of main channel dimension. Thus, the dimension of main channel was considered to be 1.0 m and maximum depth 0.5 m of rectangular shape. Width of the offtake channel was considered 40% of main channel in order to commensurate with the width ratios similar situations in practice.

Selection of Discharge Range

Maximum discharge was selected by considering the existing pump capacity of the laboratory. Minimum discharge was selected based on capacity of propeller size of current meter.

5.3 Construction of Laboratory Channel

To carry out the experimental study, the existing sand bed trapezoidal channel located in the laboratory was reconstructed. A number of modifications in the test facility were required to address the requirements of the objective of the present study. Previous channel was modified into a fixed bed channel with rectangular cross section from which an offtake channel of different angle bifurcates.

Figure 5.1: Experimental channel before reconstruction (left), during reconstruction (right)

5.4 Experimental Setup

The experimental setup consists of two separate parts; the experimental reach and experimental facilities. Experimental reach consist of main channel and offtake channel similar to model of a small scale river offtake. The experimental facilities are necessary for the storage and regulation of the water, circulating through the model and act as guiding part. Schematic diagram of flow circulation system in the experimental setup is shown in **Figure 5.2**

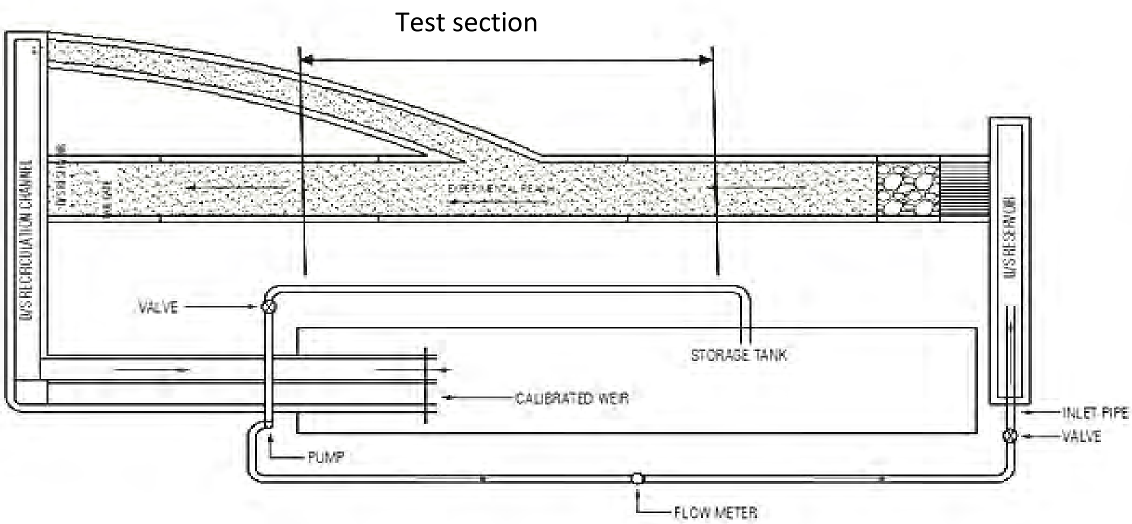


Figure 5.2: Layout of experimental setup

5.5 Experimental Reach

Experimental reach is major part of experimental setup which consists of these following components.

I. Main channel
II. Offtake channel
III. Inflow zone
IV. Test section
V. Outflow zone

Experimental reach was considered 4m from the intake point of the main channel. The distance between intake and the experimental reach required for the development of uniform flow. For this purpose, water depths were measured at various upstream sections and found to be constant for uniform flow.

Main Channel

This is a straight channel of length about 11 meters, width 1 meter and height 0.4 meter. Both of the channel banks are vertical and fixed. A mild bed slope of 0.00385 is maintained Bed throughout the main channel. Water is flowed from the upstream reservoir to the channel through the PVC tubes (D=2.7 cm; L=40 cm) placed over the width of the entrance to get rid of larger eddies present in the water coming from the upstream reservoir. After entering smoothly from an inlet reservoir a secondary reservoir was placed at the end which was connected by a tailgate. Through the tailgate water goes to a sump. From this sump, the water was re-circulated through a return system.

The main channel was made of plastered brick wall and brick soiling. Plastering was carried out with sand-cement mortar of ratio 5:1 (sand: cement) to ensure no water seepage and at the same time the wall can be easily broken for remodeling the channel for other tests. The channel is shown in **Figure 5.3**.

Figure 5.3: New rectangular main channel **Figure 5.4**: Offtake channel

Offtake Channel

Offtake channel is constructed with brick wall and brick soiling in slightly concave manner and narrower than the main channel. The width of offtake channel is 40 cm with both of the channel banks are vertical and fixed. Length of the offtake channel varies with the change of offtake angles. Higher the value of offtake angles shorter the length of the channel. To maintains consistent results bed slope is kept same as main channel. Water of upstream reservoir is flowed from the main channel to the offtake channel, which bifurcate nearly middle of the main channel. A tail gate of same width is placed at the end of the channel to control the tail water depth of flow. Through the tailgate water goes to a sump which is connected to secondary reservoir.

Inflow Zone

An inflow section and of considerable length is provided to ensure stable flow conditions before the water reaches the test section. Water flows from the upstream reservoir to main channel via the inflow section. PVC tubes (D=2.7cm; L=40 cm) are placed over the width of the entrance to get rid of the larger eddies present in the water coming from the upstream reservoir and thus the flow is stabilized. Just after the PVC tubes, about 2.8 m of the sand bed is filled with stone to ensure uniform flow in the test section.

Figure 5.5: Diffuser pipes **Figure 5.6:** Upstream stone bed

Test Section

The test section in the experimental reach of the setup is selected based on two considerations. Firstly, water approaching towards the channel from upstream reservoir, becomes stabilized within inflow zone. Thus, before reaching the test section the flow becomes uniform. Secondly, within the test section there should be no or minimum backwater effect from downstream of the test section. Test section in the middle portion of the main experimental channel and initial zone of offtake channel, in total around 5 meter of length.

Outflow Section

At the downstream end of the experimental setup, the water in the channel flows over a tail gate into the experimental facilities of the setup. Part of experimental channel downstream of test section is considered as outflow section for this experimental setup.

5.6 Experimental Facilities

The experimental facilities are usually the permanent part of the setup. It acts as a facility to conduct all types of experiment in the channel reach. The permanent part is the hardware of the setup. It acts as a facility to conduct all different types of experiment in the sand bed. The components of the permanent part are given below:

I. Downstream reservoir
II. Pump
III. Pipe line
IV. Upstream reservoir

V. The regulating and measuring system

The components of the permanent part of experimental facility are described below in brief.

Downstream Reservoir

The downstream reservoir (**Figure 5.7**) serves as storage reservoir. Its volume is 11.5 m^3. The maximum water level can be at 0.77 m elevation with respect to reservoir bottom. There is a spillway at the end of the downstream reservoir for excess water to spill out.

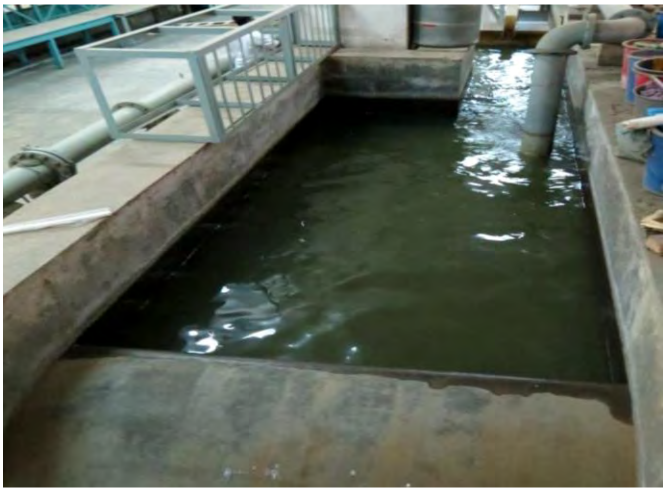


Figure 5.7: Downstream reservoir

In every one or two weeks the tank had to be cleaned and emptied. The fine particles of the sediment that are deposited at the bottom are removed through a valve placed at the lowest level of overflowing spillway.

Pump

The circulating pump in **Figure 5.8** near the measuring flume draws water from the downstream reservoir. The pump has a maximum delivery of up to 90 cusec and head of 7m.

Figure 5.8: Pump **Figure 5.9**: Pipe line

Pipe Line

The rate of water flow was controlled by the pipe line system. The layout of the pipe line is shown in **Figure 5.9**. The pipeline has three parts:

1. Suction pipe line
2. Delivery pipe line and
3. Excess discharge pipe line

The pump drags the water from the downstream reservoir into the pipe line. The T-joint on top of the pump divide the water over the excess pipe and the delivery or supply pipes, depending on the regulation of the valves in the respective pipes. As the pump delivers a constant discharge, the required discharge through the model must be regulated by these valves.

Upstream Reservoir

The pump draws water from the downstream reservoir and the discharge into this reservoir through the pipe line system. The volume of the upstream reservoir is 4.8 m^3. The maximum water depth in the upstream reservoir is 1.25 m. The reservoir has two chambers one big and the other is small. Water is dropped into the small chamber of the reservoir from the delivery pipe line. The main purpose of the small chamber is to dampen the delivery pipe line. The main purpose of making the small chamber is to dampen the turbulence in the water. This small chamber is separated by a wall (with a number of openings in it) from the large chamber of the upstream reservoir. This is done to create a smooth inflow into the big chamber. As undisturbed water is wanted in the channel a number of plastic pipes are placed in such a way that water passes through them before going into the main channel. For maintenance purpose the upstream reservoir can be emptied through a small regulated opening placed at the lower level of the reservoir wall. It also acts as a storage reservoir.

The Regulating System

The regulating system helps in regulating the flow rate, amount of water entry and depth in the channel. The regulating system of the model consists of the following:

I. The tail gates
II. The stilling basin and transition flumes
III. The guiding vanes and tubes
IV. The approach channel
V. Gate valve

The Tail Gate

At the downstream end of the sandtrap of the channel the tail gate is placed. There are two tailgates in the experimental setup. The one at the end of main channel and another one is at the end of offtake channel. The tail gate is shown in **Figure 5.10**. It is made of cast iron and encircled with rubber flaps, so that water flows only over the gates. It also has steel plates on both sides for guidance of flow. Ventilation tubes are provided under both tailgates. The ventilating tube has a valve at the middle of the tube so that if water gets inside the tube it can be drained out. The downstream regulation is performed by the tailgate. For a particular discharge, if the tail gate is raised it increases the water level and vice versa.

The tail gate has two major functions

a) They regulate the water level in the branch, and
b) They prevent the sand bed from running dry if a power failure occurs during experimentation or when it becomes necessary to stop the run for some reason.

Figure 5.10: Tail gate

Figure 5.11: Stilling basin and transition flume

Stilling Basin and Transition Flume

Behind the tail gate, water falls into a stilling basin (**Figure 5.11**). The width of the transition flume is equal to the width of the approach channel which is 0.50 m. Besides, transporting water to the measuring part of the permanent facility, the stilling basin as well as the transition flumes helps minimize turbulence.

Guiding Vanes and Tubes

To ensure a more smooth flow towards the approach channels, guiding vanes are placed between the transition flumes and the approach channels which are at rightangle to each other. These vanes guide the water around the corner. In order to prevent creation of extra unwanted turbulence in the approach channels, PVC tubes (diffuser pipes) are used on both the upstream and downstream side of the guiding vanes (**Figure 5.12**).

Figure 5.12: Guiding vanes and tubes

Figure 5.13: Approach Channel

Approach Channel

The water flows over the tail gate into the stilling basin before entering the approach channel. The approach channel is 5.27 m long and 0.50 m wide. The approach channel is designed according to ISO standards, thus avoiding an extra cumbersome calibration.

Valve

The flow in the channel is controlled with the help of two valves, one in the delivery pipe and another in the excess discharge pipe. When more discharge is required in the channel, the valve in the delivery pipe line had to be opened and the other valve had to be closed accordingly. In this way, flow of water is controlled in the channel. **Figure 5.14** shows the valve.

Figure 5.14: Valve

Figure 5.15: Electromagnetic flow meter

5.7 Measuring System

The measuring system involves all the instruments and structures used in measuring water depth and flow rate of the model. The measuring system of the model consists of the following:

I. Electromagnetic flow meter
II. Point gage
III. Current meter
IV. Electronic weighing scale

Electromagnetic Flow Meter

Discharge measurements are taken from the electromagnetic flow meter in the unit of m^3/h. The flow meter is 200 mm diameter. The flow through the pipe is controlled by the valve. **Figure 5.15** shows an electromagnetic flow meter.

Point Gage

The point gage is used to measure water level in the channel. This point gage was installed on top of wooden bar in a measuring bridge in different locations of channel.

Current Meter

The velocity of flow at any point in the open channel can be most accurately and conveniently determined by means of a mechanical device named current-meter. Current meter helps to know the speed of the water to be measured. Current meter which is used during the experiment is classic propeller type of current meter.

When the meter is lowered in water and when it faces the current of water in the channel the wheel attached to the end of the meter rotates. To keep the meter facing the direction of flow a tail is attached. This tail aligns the meter in the direction of flow.

The meter is also fitted with a streamlined weight which keeps the meter in a vertical position. The rate of rotation of the wheel depends on the velocity of flow. A commutation system is fixed to the shaft of the revolving wheel which helps to pass electrical signal due to revolution of the propeller. An automatic revolution counter is kept by the operator with the battery which registers the revolutions. The time taken for a required number of revolutions is noted. The velocity of flow can be read from a calibration equation.

A calibration equation is determined prior to the test for the particular type of current meter by operating the meter in a container of known velocity. For this purpose the current meter is lowered in the still water of the tank from the trolley by a suspension rod. The trolley is run over the tank at different known velocities for number of times. The number of rotations of the current meter propeller for various velocities is noted. From the readings a rating curve is prepared. It comes out to be a straight line and the equation is of the form

$$V = aN + b \tag{5.1}$$

Where V is velocity, N is revolutions of propeller per second and M and C are constants.

Figure 5.16: Current meter

For this case values of constants **a** and **b** are 0.1334 and 0.029 respectively. Thus Equation 5.1 becomes

$$V= 0.1334.\mathbf{N} + 0.029 \tag{5.2}$$

Electronic Weighing Scale

Electronic weighing scale is a device to measure weight or calculate mass. Spring balances measure weight (force) by balancing the force due to gravity against the force on a spring. It has been used to measure the weight of sediment during sediment run.

5.8 Sediments

The sediment that is specifically chosen for this experiment was bought from the market. Then it was washed with water so that there is no dirt in it. Sand feeder was used for distributing the sediment uniformly over the channel entrance. Sieve analysis was performed to determine median particle size and calculation has been done to determine equilibrium sediment load.

5.8.1 Selection of Particle Size

Several samples were taken from the washed sand for sieve analysis in order to find the grain size distribution. The grain size distribution can be seen from sieve analysis

Figure 5.17. Average particle size of the sand used in the experiment was determined by the value of the particle diameter at 50% in the cumulative distribution. Particle size distribution D_{50} is also known as the median diameter or the medium value of the particle size distribution. From the sieve analysis curve determined average particle diameter (D_{50}) is 0.23 mm.

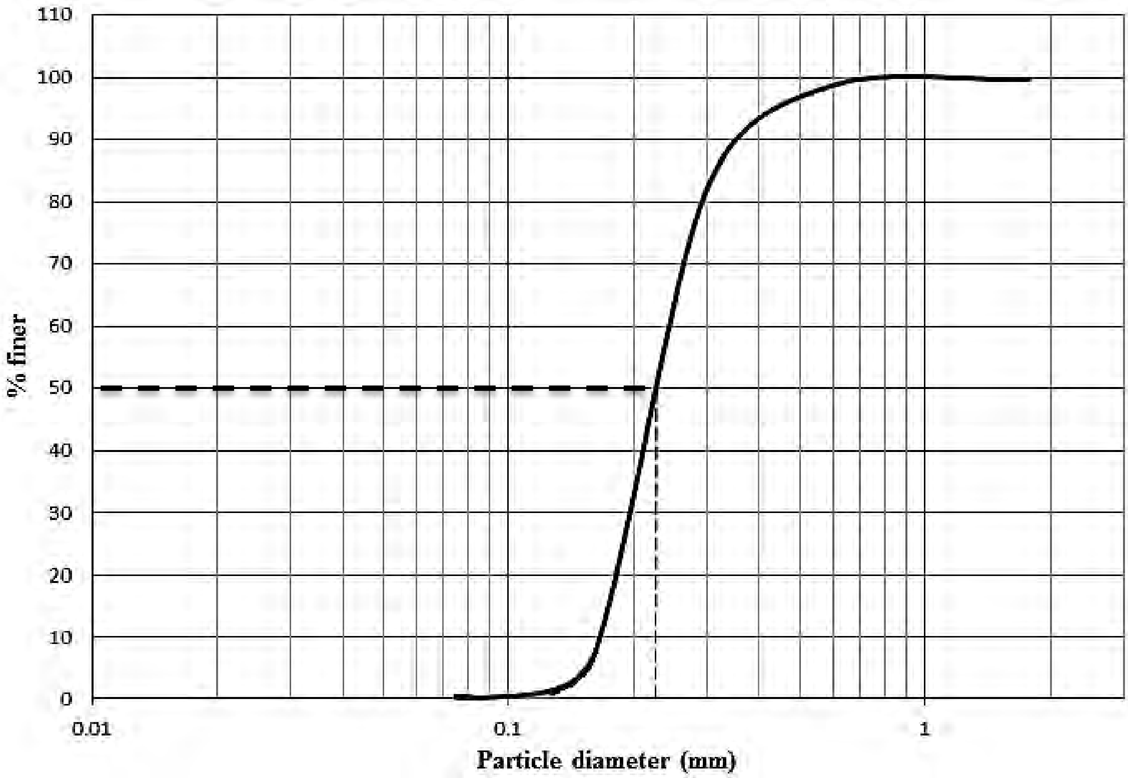


Figure 5.17: Grain size distribution curve

Sediment size 0.23 mm and slope of 0.00385 were chosen for the experimental work. Shield's diagram for incipient motion has been used to determine the critical shear stress for the channel. Bed shear stress for given sediment size and channel gradient was found to be greater than critical shear stress of the experimental channel. Critical velocity by Shield's was determined to be 0.25 m/s, which was not possible to obtain for minimum discharge thus only maximum discharge has been considered for sediment analysis.

5.8.2 Determination of Upstream Sediment Load

During the experiment, the sediment inflow rate to be supplied is needed so that the equilibrium condition can be achieved. Power law of sediment transport has been used for the estimation of inflow sediment load (Dey, 1998).

5.8.3 Sand Feeders

To feed sediment and to maintain equilibrium state in the main channel sediment hopper or sand feeder was installed. A sediment feeder **Figure 5.19** is a mechanical device run by electric power, which feeds sediment into streams of flow of water at measured rates and is used for model studies of rivers.

Figure 5.18: Sediment used in the experiment

Figure 5.19: Sand feeder

It is composed of a rectifier, a variac, a DC motor, a gearbox, a gear plate, a hopper and a sand bucket. The hopper just holds a large amount of sand within it. There is a narrow slit at the front base of the hopper through which the sand passes out and gathers at the rim of the gear plate.

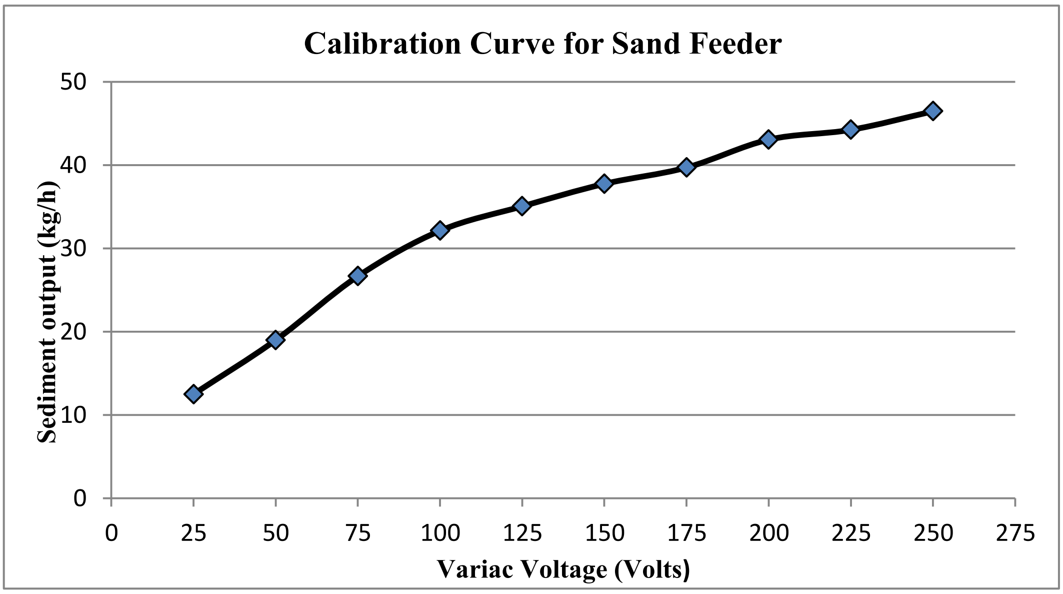


Figure 5.20: Sand feeder calibration curve

Figure 5.21: Wooden structure for sediment distribution

As the sand gathers and grow in amount they finally falls into the sand bucket at measured rates depending on the speed of the gear plate. The calibration curve of the feeder is present in **Figure 5.20**. The sediment falls from the sediment feeder into the wooden structure (**Figure 5.21**), which distribute the sediment uniformly over the main channel width.

Input sediment capacity was also limited by the capability of the sand feeder used during the experiment. Half of the sediment load was fed by sand feeder and other half was been fed manually.

5.9 Selection of Scale for Experimentation

Design of physical model is an iterative process because it is quite difficult to fulfill all scale conditions to the governing processes to obtain complete similitude between the model and prototype. Space, pumping capacity, availability and usefulness of the bed material, steepest slope in the river bank were considered in selecting scale factors. Selection of scale for experiment is based on two factors. They are

- Available laboratory flume facilities
- Froude law criteria.

Model scale has been fixed based on the available facilities (Ahmed, 2014) in the Hydraulics and River Engineering Laboratory. This scaling is needed to convert model data into prototype. The scale ratio shown in the table has been used to design the laboratory setup and also to correspond with the available field conditions.

Table 5.1: Scale ratio of model parameter

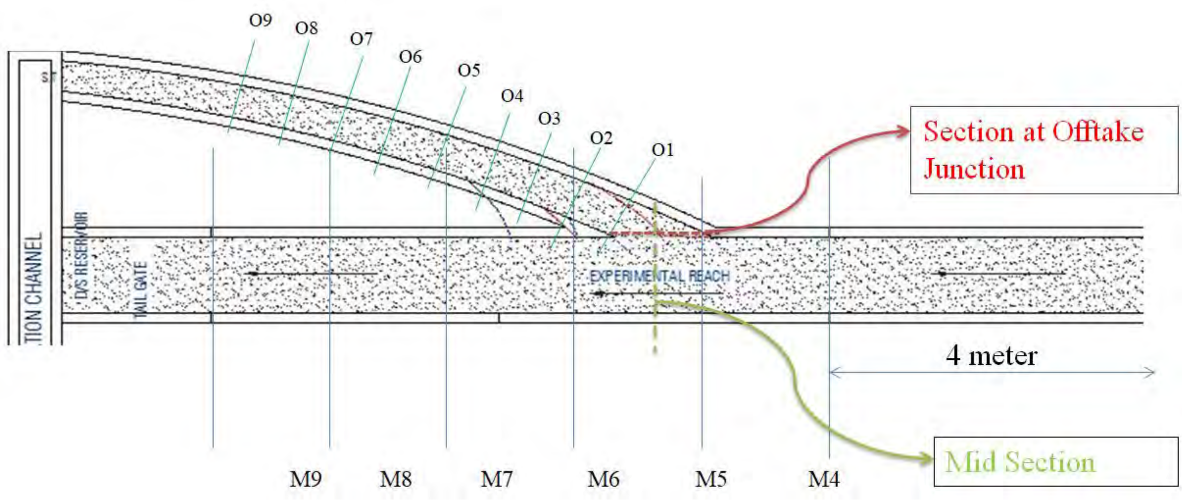
Quantity	Dimension	Scale Ratio
Length	L	1:20
Velocity	$L^{1/2}$	1:4.47
Discharge	$L^{5/2}$	1:1789

The name movable-bed implies that in this type of model due attention is to be paid to sediment transport, erosion, deposition and bed-forming processes. The research objective is to focus on angle of offtake so, much attention was given to the angle of offtake than the model bed condition.

5.10 Measuring Techniques

In this section the measurements to be made during experimentation are discussed. Measurements will have to be made of the parameters describing an offtake. Aims of the experiments are to study the distribution of the discharge and sediment at an offtake for different offtake angles**.** Flow phenomena in offtake bifurcation channel govern by water depth, discharge, sediment flow, angle of offtake. Dimensionless groups are formed to develop theoretical relationship between these parameters. Experimentation is conducted and data are collected to obtain exponents and coefficients of the developed equations for predicting flow and sediment distribution in bifurcated channel.

Measurement for water depths and velocity have been taken for total 17 cross sections including one at junction and another at midsection of the intersection. The cross sections of main channel have been marked as M4 to M9, placed each at one meter apart and offtake channel have been marked as O1 to O9, each half meter apart as illustrated in **Figure 5.22**.



**Sections of main channel (M4 to M9) are place 1 meter apart*

** * Sections of offtake channel (O1 to O9) are place 0.5 meter apart*

Figure 5.22: Location of the measurement points

Each cross section, both main and offtake channel has been divided into three segments; left side (LS), middle (M) and right side (RS) as illustrated in **Figure 5.23 and 5.24**. Data of velocity and water depth have been collected at midpoint for these three divisions for each section to get idea of velocity and depth variation due to side interruption. Again data of variation of velocity at nose and mid-section have enabled to understand the effect due to presence of offtake for parent channel.

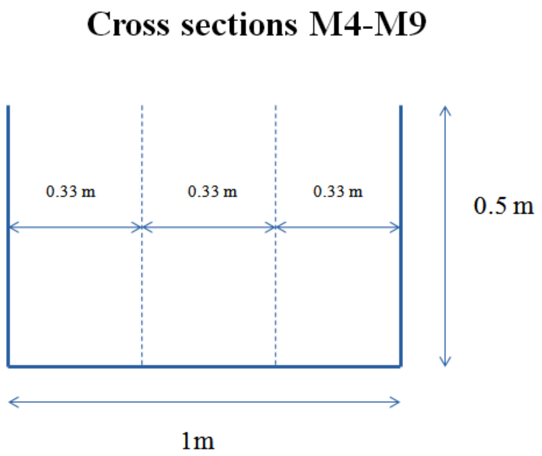


Figure 5.23: Division of cross sections in main channel

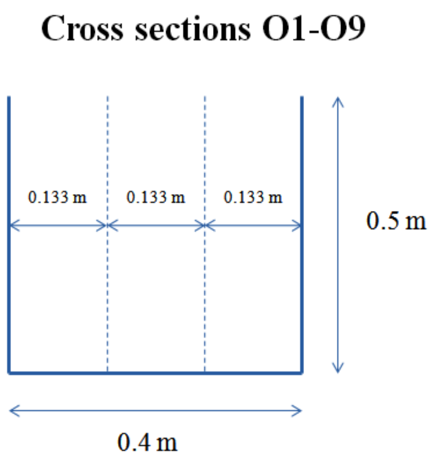


Figure 5.24: Division of cross sections in offtake channel

5.10.1 Water Level Measurements

The water level was measured at three locations (left, middle, right) across the cross sections.

5.10.2 Discharge Measurements

Discharge is measured using water flow meter fitted with the setup. But this meter provides reading of incoming upstream discharge only. For measurement of discharge in downstream of main channel and offtake channel Area-Velocity Method has been used.

5.10.3 Sediment Transport Measurements

Sediment transport is measured by collecting sample at upstream of main channel and downstream of main channel and offtake channel. Sediments are collected only after equilibrium ripples formation on bed. Percentage of sediment transferred in main channel and offtake channel is determined by oven drying collected samples.

Sediment inserted into the channel with the help of sand feeder. Released sand through the sand feeder falls into channel after passing triangular distributor flows with the stream, settling heavier particle. Cylindrical container was used to collect suspended sediment from the channel at pre stated cross sections. Samples are then weighted and oven dried at 110^{o} C for a period of time until it completely lost moistures. Later water was siphoned from the model to remove of stagnant water in the channel. Then settled sediment of previously mentioned cross sections were collected in a pan or beaker and weighted. Again they were oven dried and weighted to obtain percentage of sediment settled. Later total percentage of sediment transported in offtake and main channel was determined.

It is difficult to define accuracy for the sediment transport because the transport rate is an average for the time interval chosen. The transport rate will vary continuously, but it is not possible to measure these variations. The only way to get more detailed information on the change in transport rates is to shorten the time intervals for which the sediment transport rates are determined.

5.11 Assumptions and Limitations of the Study

The study is based on the physical model of bifurcation at river offtake and some assumptions have been made. In order to simulate the results of the experiments of the physical model of river bifurcation some additional assumptions were made when designing the model. Apart from these the model itself implies some limitations. The following are the assumptions and limitations of the model.

Assumptions

- Only one offtake is diverted from main channel.
- The upstream discharge is constant in time and upstream flow is uniform.
- Sediment is suspension is considered only.
- Deposition of sediment is considered individually.
- The downstream water levels (near the tail gate) are equal.
- The shape of nose of the offtake is kept same during every run and shape of nose effect is neglected.
- Possible influences of tides and salt water are neglected.
- The supply of sediment is constant during the run of the experiment.

Limitations

- The widths of the channels are fixed and selected considering space limitation.
- Small deviations of the upstream discharge, water levels at the end of the branches and the amount of sand fed upstream, which are unavoidable, are neglected.
- All the sediment transport is assumed to be suspended load and deposited which creates limitations for the upstream discharge and the ratio of the discharges in the downstream branches.
- The height of the model wall and capacity of the discharge pump is fixed which restricts the maximum water level and discharge.
- The sand is not uniformly fed over the width of the model; it is assumed that the water movements distribute the sediment equally over the width before the sediment reaching the bifurcation.
- Due to space limitation of the location of experimental setup, distributary channel for higher offtake angle may undergo slight flow curvature effect.

- Due to limitations of current meter in measuring low velocity of flow, measurement of near-bed velocity during low discharge is not possible.
- Discharges in downstream branches are determined using Area-Velocity Method. Inaccuracy may arise due to time fluctuation of velocity at a point.
- Due to space, time and budget constraint only three angle of offtake are taken into consideration.

5.12 Experimental Procedure

First the model was constructed as per requirement set by the objectives of the experiment. The flexibility needed in such experimental, setup to carry out further studies in future was also kept in mind. The construction period was nearly one year. For conducting the experiment the following procedure was followed. Running the experiment and collecting data required not only a great deal of physical work but also a careful observation.

5.12.1 Things Had Been Done Before Starting the Model for Experiment

1. Several test run were made on main channel to identify necessary changes to reconstruct the channel. For this experiment effective flow area of the channel was increased by reconstructing it into a rectangular channel.
2. Sand of grain size of D_{50}= 0.23 mm was selected from the market and washed with water so that there is no dirt in it. After that sieve. Analysis was done in order to find the grain size distribution.
3. Before running the model several runs were needed in order to find whether the Engelund-Hansen equation could be used for this model. This is required for estimating the amount of sand that should be fed from the sand feeder during the experiment.
4. All the items such as point gauges, tail gates, stop locks and other items were checked whether they were working well.
5. Current meter was calibrated and calibration equation was developed (Equation 5.2) so that no error would occur during velocity measurement.
6. The sand feeder has different speeds. At different speeds the rate of sand outflow was measured and calibration curve was developed (**Figure: 5.20**).

5.12.2 Things Had Been Done During the Experiment

1. The first step is the fixation of the discharge. Supply pipe line has valve for the regulation of the discharge. Before starting the pump the valves in the excess and the delivery pipe line should be closed. Sufficient depth of water in the downstream reservoir should be present before starting the pump. After that by adjusting both the valves, the desired flow rate through the model was achieved.
2. Tail gates were lifted to a position so that maximum depth of water flows into the channel without backwater effect.
3. Both of that tail gates were adjusted in such a position that downstream water depth at main and offtake channel remain same.
4. Point gauge was hoisted with wooden frame to measure water depths at middle point of the segments at different cross sections.
5. Current meter was placed carefully into the water so that it remains completely vertical and propeller facing parallel to the current flow.
6. Velocity was measured at three vertical points on each segment. Velocity profile was plot and velocity of 0.6Y depth from the surface of water was considered as depth average velocity of that segment. Average of depth average velocities of three segments was considered as average velocity of that cross section.
7. During test with sediment inflow care was taken so that the pump and the sand feeder were started more or less at the same time.
8. Sand feeder was kept in operation until equilibrium ripple bed was formed thoroughly throughout the channel.
9. Samples were collected, in two steps. At first for obtaining of suspended sediment samples were collected by suspended sediment collecting bottle and then transferred in to glass beaker. Deposited sediment was determined by obtaining bulk sample from the bed of the channel.
10. Samples were weighed after collection and kept in oven for drying at 110^{O} C until the bulk has lost its moisture completely. It is weighed again to obtain dry weight of sample.
11. Same procedure was repeated during observation for all the test run

Figure 5.25: Velocity measurement with current meter

Figure 5.26: Formation of ripple bed

Figure 5.27: Sample collection

Figure 5.28: Weighing of samples **Figure 5.29**: Oven drying in 110^{o} C

5.13 Experimental Observation

Experimental observation was made to understand the direction of flow velocity, siltation and erosion pattern at test section. During experiment following observations were made near the offtake region.

5.13.1 Velocity Observation

Color die has been used to observe surface velocity variability at test section. Turbulent eddies were observed for both downstream of main and offtake channel at the junction region. This turbulent eddies were formed due to the flow separation of streamlines at the nose. Flow separation causing stream line to form eddy that decrease linear flow velocity of the region and they penetrate the laminar sublayer formed along the bed. As a result of this erosion takes place at the region. Discrete particles resting on the bed are acted on by two components of the forces. One component force is exerted parallel to the flow (drag force) and the other is perpendicular to the flow (lifting force). Drag force results from the difference in pressure between the front and the back sides of a particle.

Figure 5.30: Flow visualization

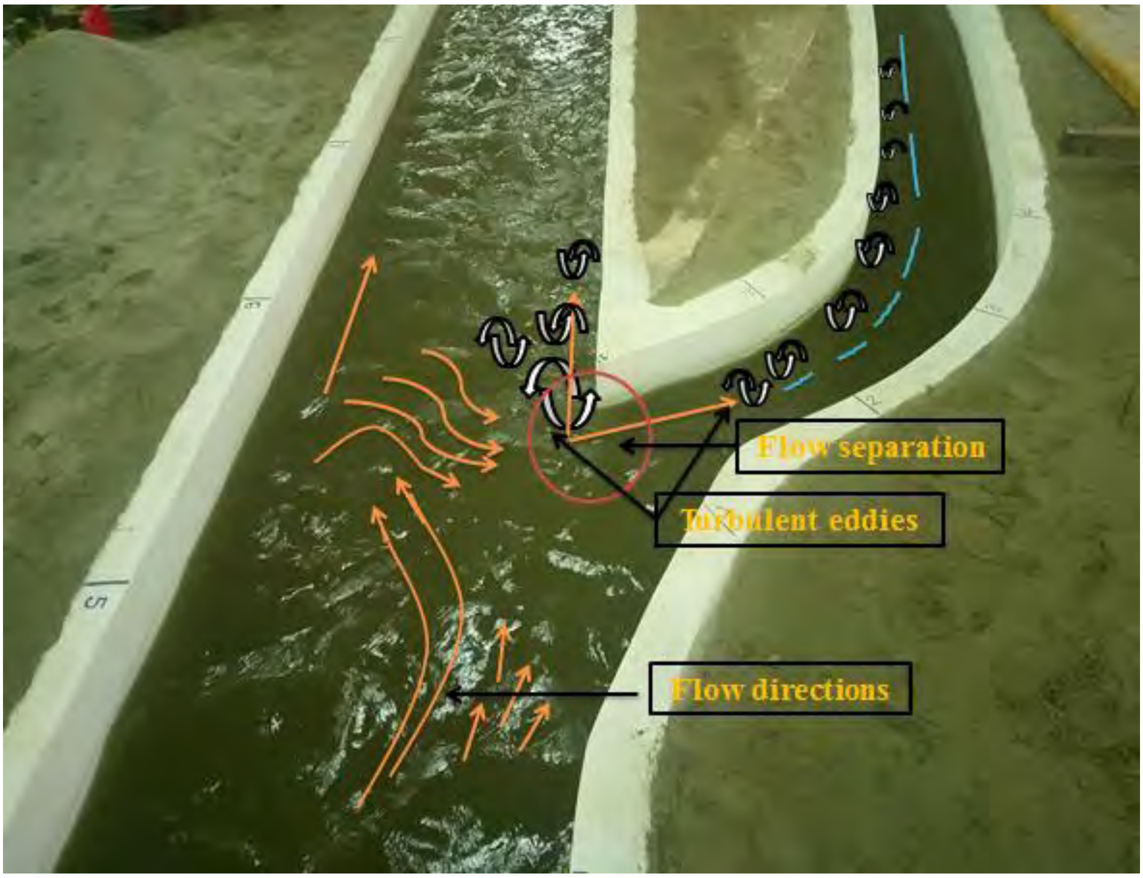


Figure 5.31: Observation of the flow behavior in the vicinity of the offtake section

Lifting force results from the difference in pressure on the upper and lower surfaces. If the lifting force exceeds the particle's immersed weight and the interference of neighboring grains, the particle goes into suspension. As streamlines approaches near the offtake they deviate from straight uniform configuration. As shown in the Figure 5.31 stream lines of middle to left segment at upstream parent channel bend toward left. Again near the junction zone streamlines bend toward offtake and obliquely entered into offtake channel.

Average velocity variation and contour lines were plotted with Surfer 15.0 from experimental data obtained for 60^0 offtake angle. It is observed that presence of offtake causes velocity drop at surrounding areas. In turbulence zone of main channel erratic velocity vectors cause to reduce average velocity of that area (blue zone, **Figure 5.32 and 5.33**).

In same manner average velocity was plotted for offtake channel and observed that velocity is low near the junction and high just at opposite end of junction. Due to the limitation of the plotting software curvature of the offtake was showed as linear scale.

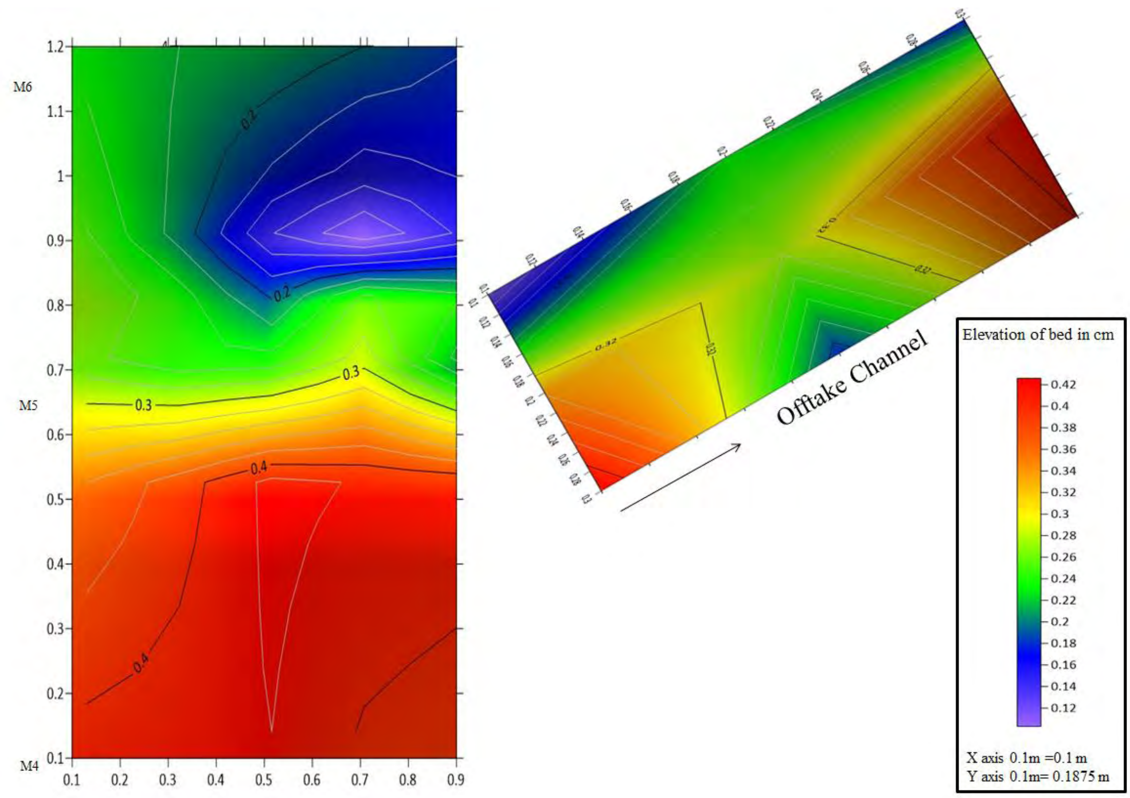


Figure 5.32: Average velocity variation at the vicinity of offtake for discharge 320 m^3/h

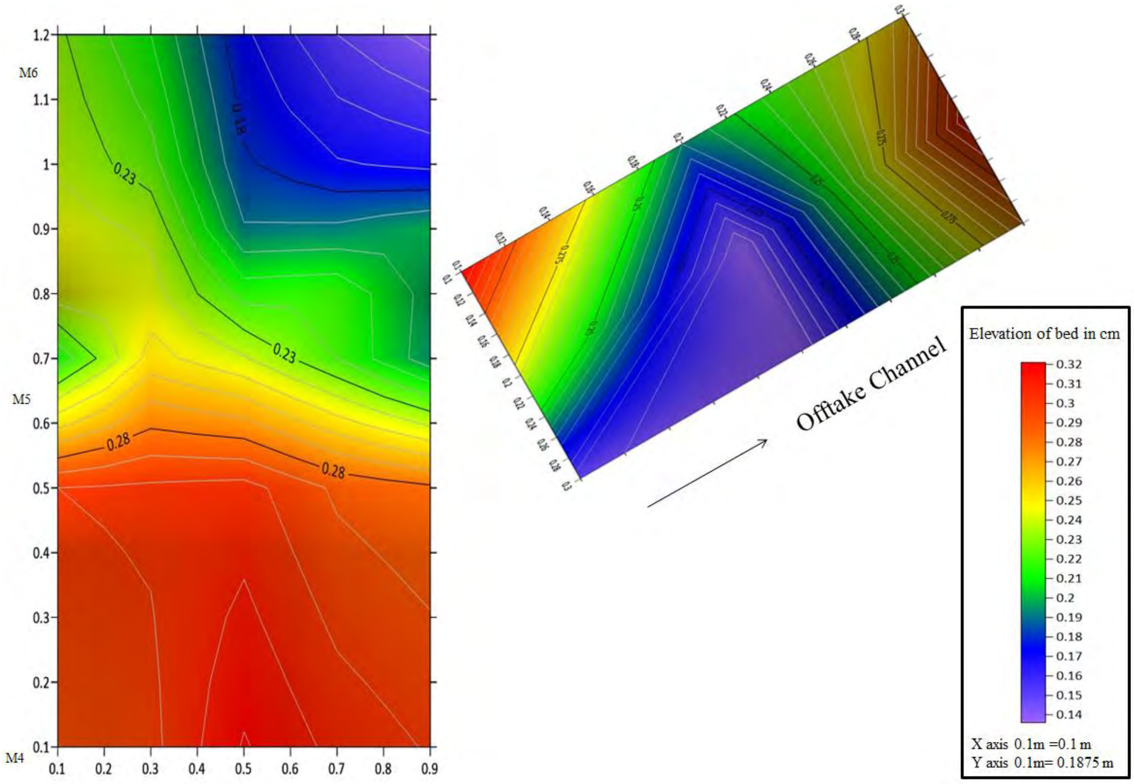


Figure 5.33: Average velocity variation at the vicinity of offtake for discharge 220 m^3/h

5.13.2 Model Run with Mobile Bed Condition

Bed elevation has been measured to determine the silting and erosion phenomena at surrounding offtake region. Cross sectional width is segmented and bed level height was measured with respect to channel bottom to determine erosion and deposition at test section.

Due to the configuration of the experimental channel sedimentation occur at outer side and erosion at the mouth of offtake. This allows us to rethink about configuration of offtake to prevent siltation at the mouth of offtake. According to Bulle effect more sediment was diverted into the lateral channel than continued in the straight channel. Bend in channel introduces helical flow which influences sediment transport. The shear stress near the bed responsible for bed load transport will have a direction slightly different than that of the depth averaged velocity during helical flow. Concentration of suspended sediment become higher near the bed and therefore helical flow will also change the direction of suspended sediment transport with respect to the depth averaged velocity direction.

a)

b)

Figure 5.34: Observed bed forms in the experimental reach for various discharge a) 320 m^3/s and b) 220 m^3/s

From the observation, junction zone of main channel and offtake can be said to have four zones as shown in **Figure 5.35**

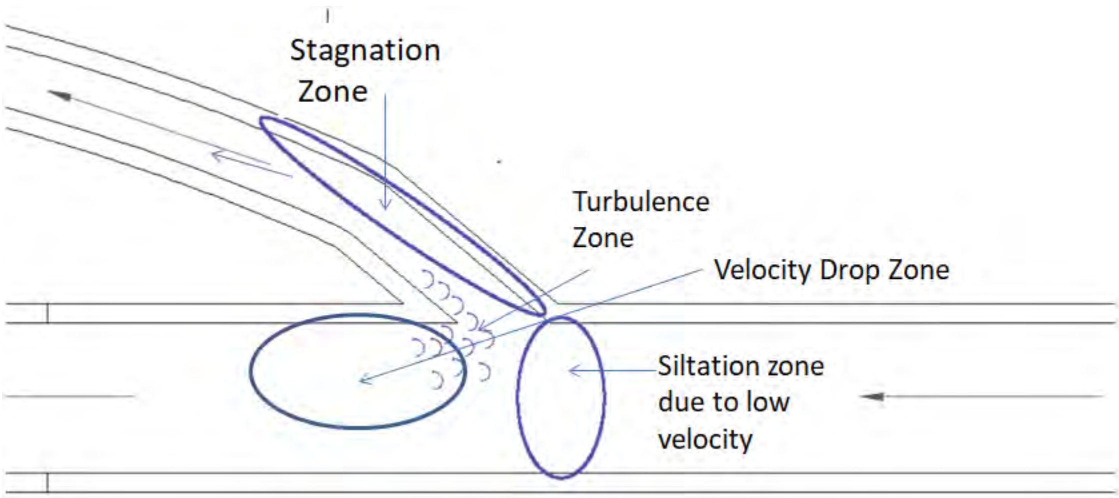


Figure 5.35: Observed flow zones near the main channel and offtake channel junction

Stagnation zone occur near the upstream of the offtake where reduction in velocity causes particle to stay in bed where turbulence zone at the nose of the offtake causes erosion at the region. Sudden velocity reduction near the downstream junction wall due to sudden transition in a channel creates velocity drop zone. Wall near the junction of downstream main channel and offtake channel help in formation turbulent eddies. Due to this high turbulence all particles present in the sand bed before the experimental run have been washed away creating erosion zone just after the siltation zone (**Figure 5.34 and 5.36**). Creation of turbulent and stagnation zone causes flow line to deviate from parallel path and starts to move in irregular direction which ultimately settles the sediment particle just at the outer edge of the mouth of the offtake channel.

The bed level elevation after the observational run with mobile bed is shown in **Figure 5.36**, for $\theta=60^0$. The effect of stagnation zone, siltation zone and turbulent eddies can be seen in the Figure.

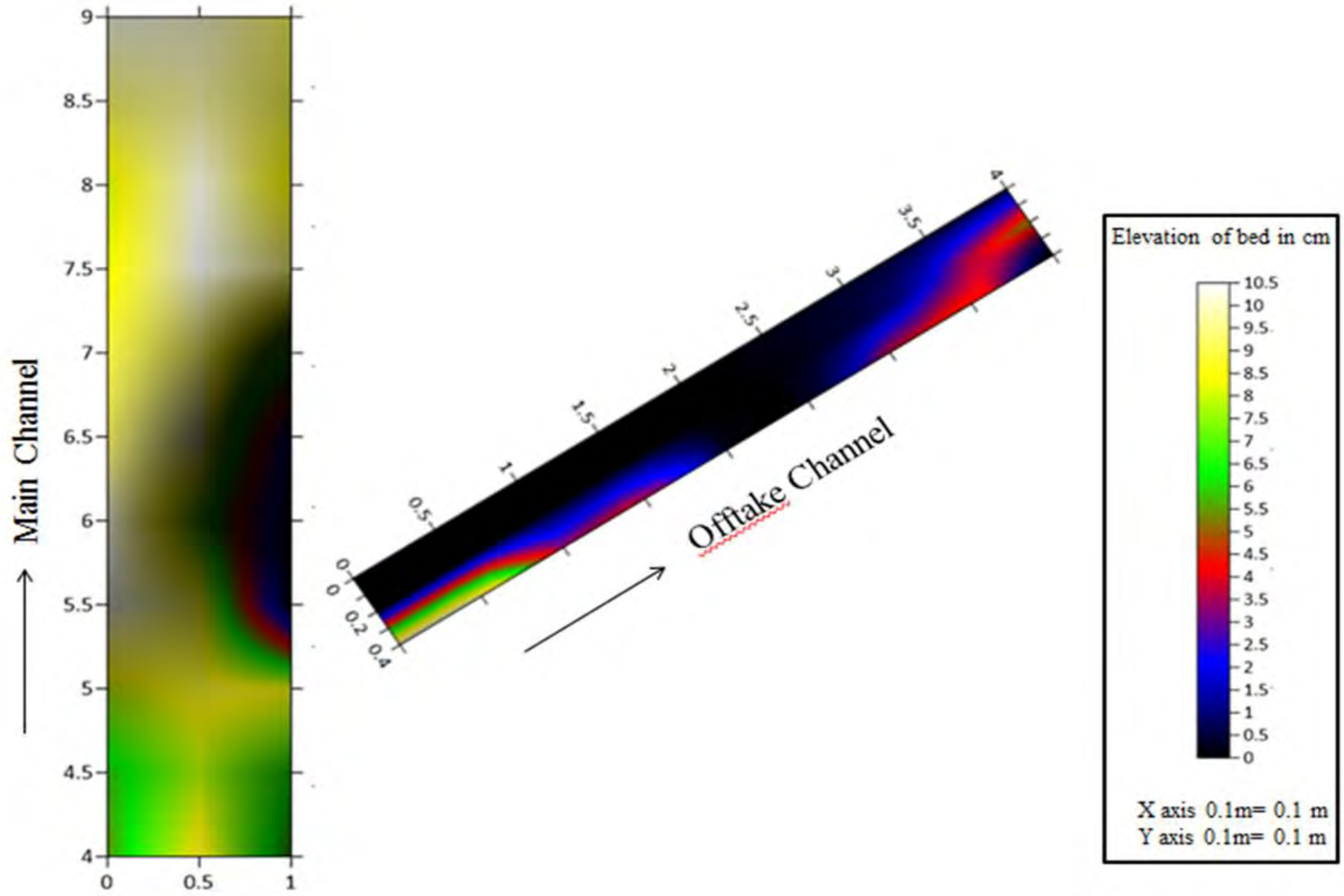


Figure 5.36: Bed elevation change within the experimental reach

CHAPTER SIX

DATA ANALYSIS, RESULTS AND DISCUSSIONS

6.1 Introduction

Hydraulic and sediment data were collected at upstream and two downstream branches from the offtake model which was developed for the study purpose. The intention was to understand the hydraulics and distribution of the phenomena of offtake at a bifurcation. The data were collected mainly relating to average water level and discharge data of all branches. Velocity variation data, average sediment discharges and amount of suspended sediment throughout the experimental reach has also been obtained.

These are all described in detail in Article 5.10. For the purpose of data collection, four sets of observation were carried out with varying offtake angle and each set with varying incoming discharge. Total 18 runs have been conducted as in **Table 4.1** and additional run has been made for flow visualization.

6.2 Data Analysis

In first set of observation, four experimental run was conducted in a rectangular horizontal channel without any offtake channel. Hydraulic data were collected for discharge of 320 m^3/h (0.09 m^3/s), 220 m^3/h (0.06 m^3/s), 120 m^3/h (0.03 m^3/s) and sediment data was collected for 320 m^3/h only. Second set of observation was carried out for angle of offtake of 20°. In this case, water level and velocity data were collected for discharge of 320 m^3/h, 220 m^3/h, 120 m^3/h. For sediment discharge calculation water level, velocity and suspended sediment concentration were obtained for maximum discharge 320 m^3/h for all test scenarios. Similarly third and fourth set of observation was made with offtake angle 40° and 60°. Discharges in the main branch were chosen according to the carrying capacity of the channel and sediment load in the main branch was selected for equilibrium condition, i.e. non-scouring, non-silting condition. Visual observations were made during the period with a view to study the pattern and the process in channel offtake bifurcation of model channels. Photographs were also taken during the experimental run.

Summary of data on discharges and sediment discharges in the main and offtake branches are presented in **Table 6.1.**

Table 6.1: Data on discharges and sediment discharges in the main and offtake channel

Test Runs	Offtake Angle (Degree)	Q_1 (m^3/s)	Q_2 (m^3/s)	Q_{s1} (ton/d)	Q_{s2} (ton/d)	Q_2/Q_1	Q_{s2}/Q_{s2}	σ
Run 1	NO	0.0879	..	..	.	..	..	..
Run 2		0.0554	..	..	..	..	..	..
Run 3		0.0293	..	..	..	..	..	..
Run 4		0.0990	..	7.3181	..	..	..	..
Run 5	20^0	0.0697	0.0304	..	..	0.4366	..	..
Run 6		0.0549	0.0188	..	..	0.3420	..	..
Run 7		0.0314	0.0123	..	..	0.3918	..	..
Run 8		0.0850	0.0432	7.1340	1.5523	0.5082	0.2176	0.4281
Run 9	40^0	0.0887	0.0360	..	..	0.4061	..	..
Run 10		0.0505	0.0194	..	..	0.3835	..	..
Run 11		0.0336	0.0107	..	..	0.3198	..	..
Run 12		0.0900	0.0370	6.3262	1.5150	0.4108	0.2395	0.5830
Run 13	60^0	0.0888	0.0363	..	..	0.4093	..	..
Run 14		0.0600	0.0347	..	..	0.5791	..	..
Run 15		0.0332	0.0218	..	.	0.6563	..	..
Run 16		0.0906	0.0486	5.3894	2.0505	0.5364	0.3805	0.7093

Where, Q_1 is discharge for upstream of main channel , Q_2 is the discharge for offtake channel, Q_{s1} is the sediment discharge for upstream of main channel, Q_{s2} is the sediment discharge for offtake channel and **σ** is change in relative sediment distribution over relative flow distribution; expressed in terms of $(Q_{s2}/ Q_{s1})/(Q_2/Q_1)$.

Discharge for each section was calculated using Area-Velocity method. For this purpose each cross section was divided into three segments (Article 5.10) and consecutive velocities and water depths were measured to obtain discharges using the method. For sediment discharge determination, at first suspended sediment concentration in ppm was determined using Equation 3.19 homemade sampler (Article 3.4.3) for collection of sediment at upstream of the main channel and downstream of the offtake. Obtained sediment concentration value was then converted into sediment discharge in tons/day using Equation 3.20. For the entire test runs same cross sections were considered for comparing water and sediment discharges. Upstream main channel discharge (Q_1 and

Q_{s1}) is determined for the cross section at the beginning of test section (section M4) and downstream offtake discharge (Q_2 and Q_{s2}) is considered at the cross section just after the curvature (section O5) so that the discharge is not affected by existing curvature present in offtake channel.

From **Table 6.1** it is observed that water and sediment discharge ratios of offtake to main channel, both increases with the increase of offtake angle. This means for greater offtake angle more water would flow and more suspended sediment will flush through the system. Value of $\boldsymbol{\sigma}$ indicates relative sediment input to a channel compared to discharge input to it. As the offtake angle increases value of $\boldsymbol{\sigma}$ increases which indicates the higher the value of offtake angle the more relatively larger amount of sediment enter into the offtake than discharge input. This parameter is very important for understanding siltation at river offtake. If $\boldsymbol{\sigma}$ value decreases with decrease in sediment transport capacity but an increase with sediment intake, siltation occurs at the mouth of offtake.

6.3 Variation of Hydraulic Parameter with Angle of Offtake

To obtain discharges, conveyance capacities and flow and sediment distribution observation hydraulic parameters (water depths, velocities and water surface slopes) were measured. It was observed that water depths and velocities vary in same discharge with different offtake angles. Though due to smaller experimental channel changes in water depth at various cross sections were small but velocity values vary to a large extent specially at the vicinity of offtake.

6.3.1 Average Velocity Variation at Test Section

Velocity was measured with the help of current meter at three vertical depths of each segment for every cross section off the main and offtake channel. Velocities at 0.6 of total depth are considered as depth average velocity of the cross section. Variation of depth average velocity for different angles of offtake is shown in **Figure 6.1 to 6.4**.

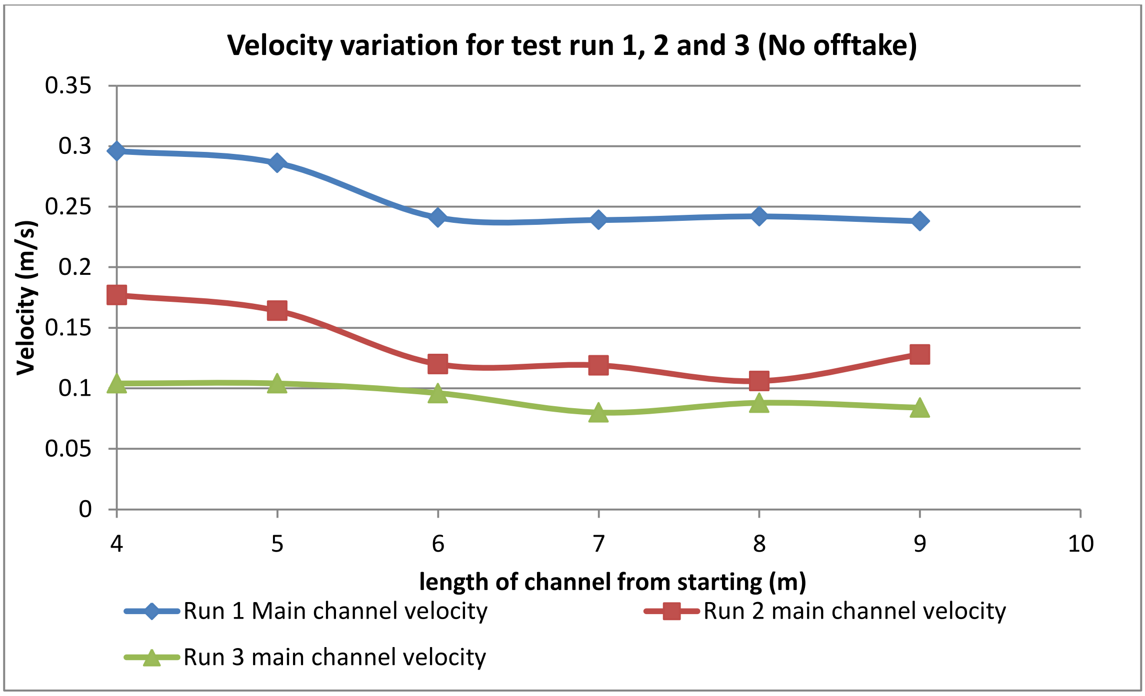


Figure 6.1: Velocity variation on main channel without offtake

Test sections were selected in such a way that velocity variation is minimized and uniform flow is maintained throughout the channel. Small lowering of average velocities has been observed in **Figure 6.1** was due to frictional loss over the length of the channel.

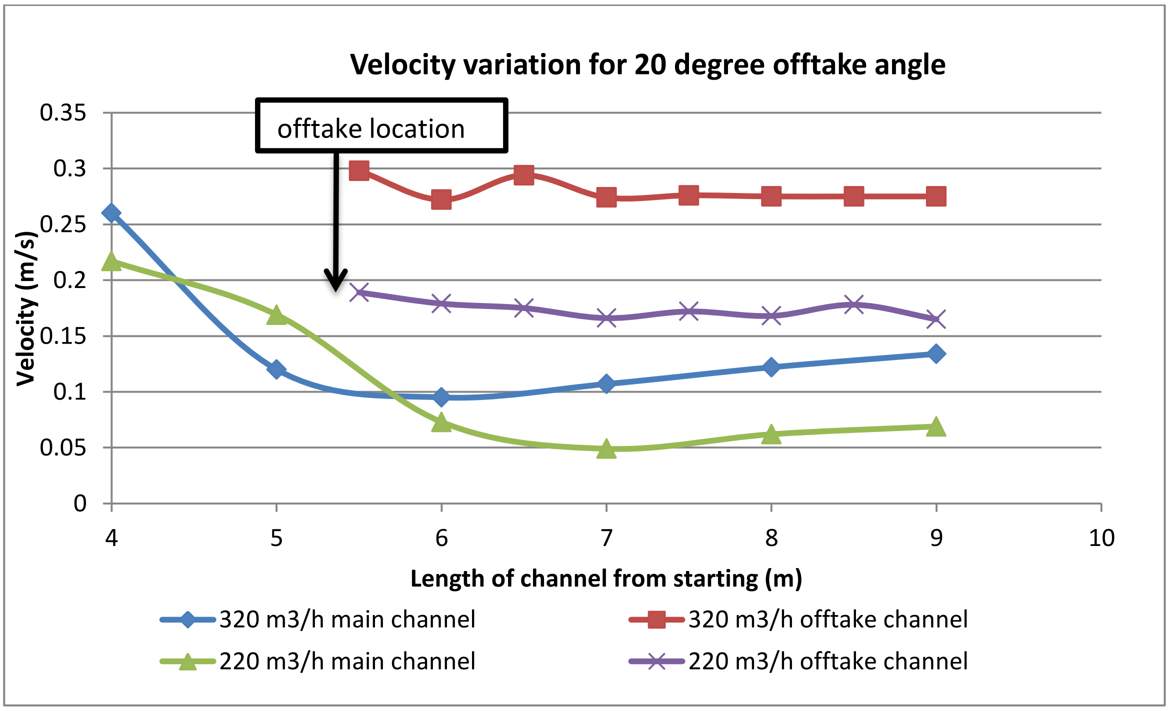


Figure 6.2: Velocity variation on main channel for 20^{o} offtake angle

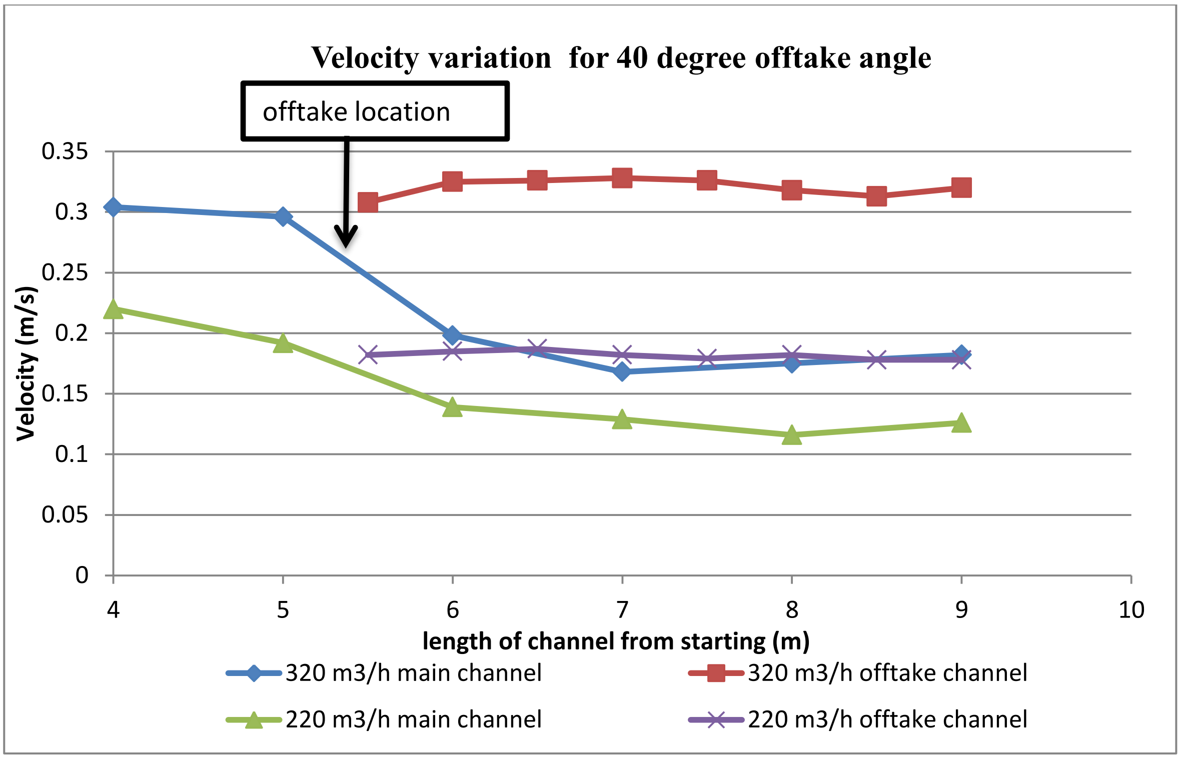


Figure 6.3: Velocity variation on main channel for 40° offtake angle

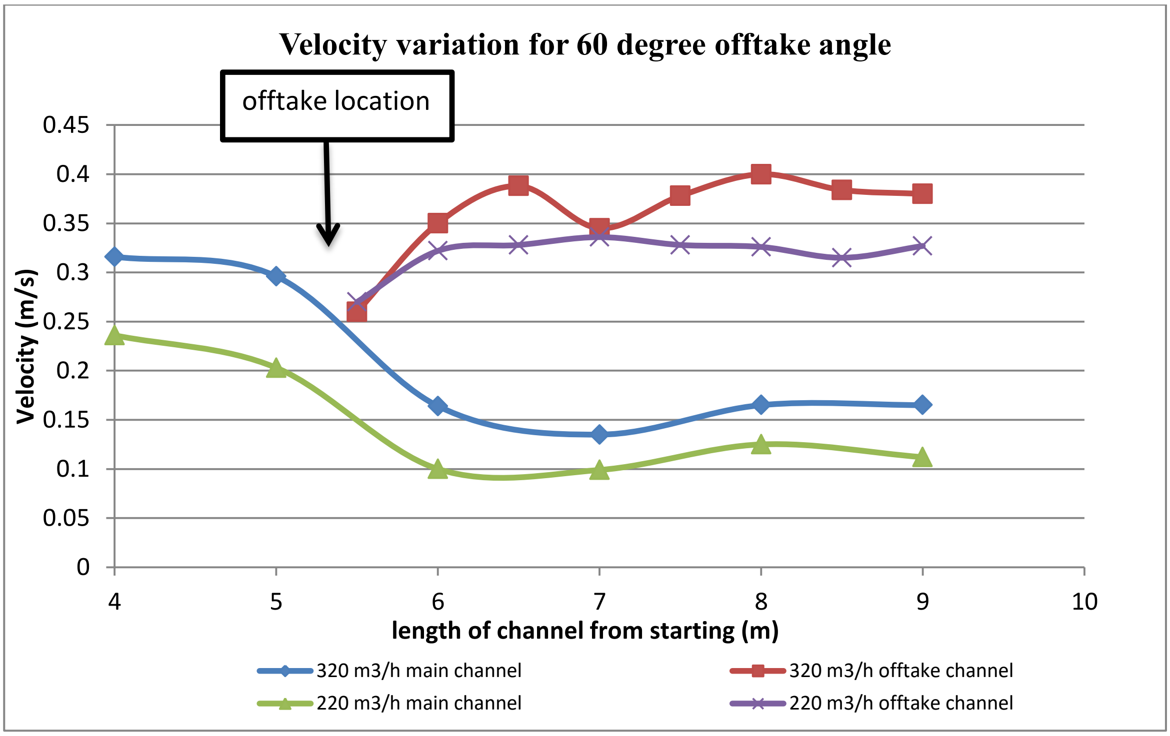


Figure 6.4: Velocity variation on main channel for 60° offtake angle

For all test observations, average velocities increase at downstream of offtake channel while velocities decrease at downstream of main channel. Here, upstream main channel inflow is divided and flown into the offtake and downstream of main channel as per the continuity of flow.

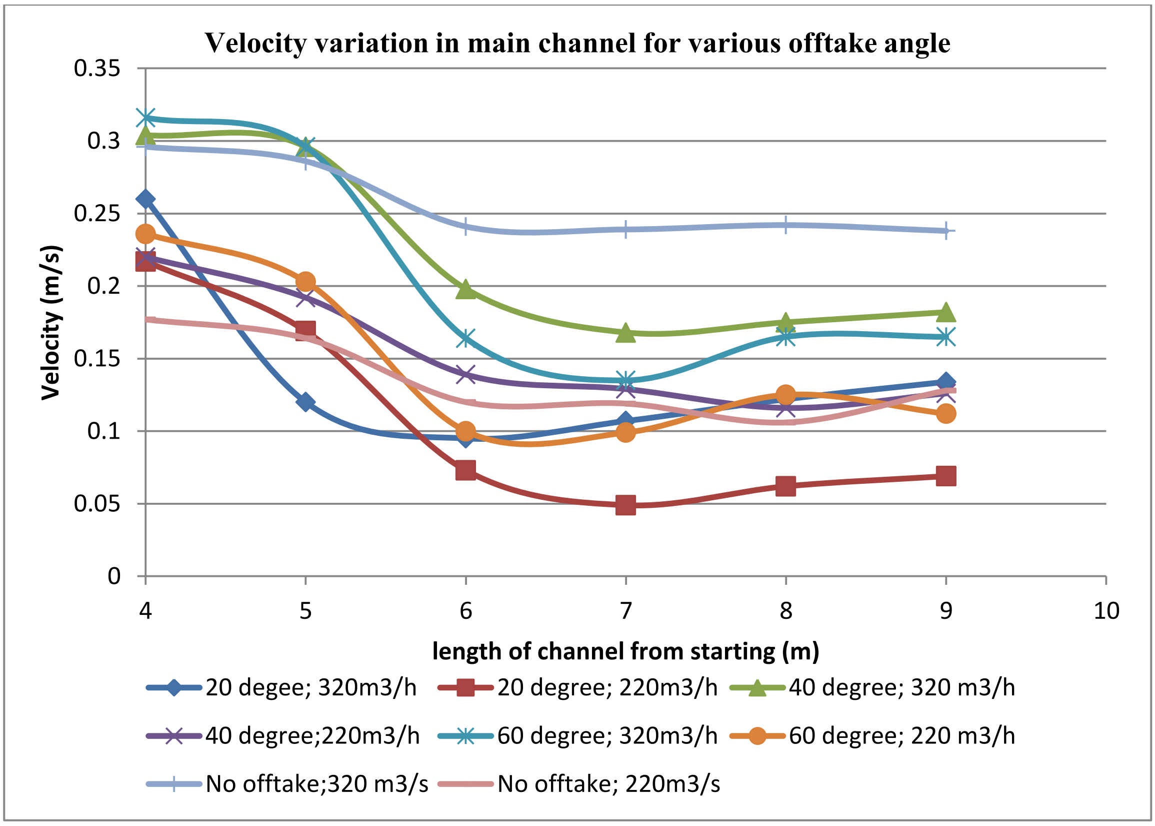


Figure 6.5: Velocity variation on main channel for various offtake angle

In **Figure 6.5 and 6.6** velocities variation in main channel for different offtake angle has also been presented. As the off take angle increases, the velocity in the offtake channel increases, while velocity of flow in the downstream end of the main channel decreases. It is also observed from the figures that the relative downstream velocity of main to offtake channel increases with the increase of offtake angle. Velocities at the mouth of offtake and the downstream of offtake channel showed discrepancies with the change in angle of offtake. At 20^0 offtake angle entrance velocity at the mouth was higher than downstream velocity, for 40^0 angle entrance velocity at the mouth was nearly same as downstream velocity but for 60^0 angle of offtake showed entrance velocity at the mouth was smaller than that of downstream velocity of the offtake channel.

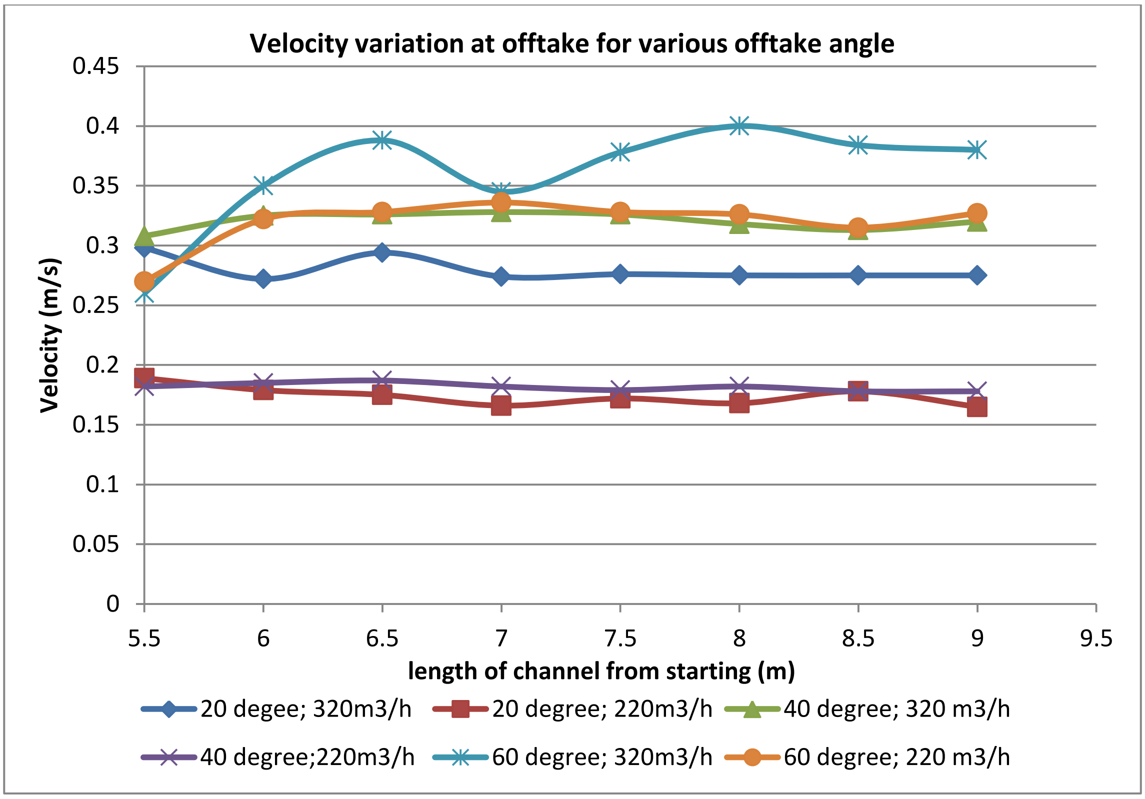


Figure 6.6: Velocity variation at offtake channel for various offtake angle

Further vertical velocity distribution at various cross sections of the test channel is given in **Table A1 to Table A6**.

6.3.2 Velocity Variation at the Offtake Junction

Junction of offtake channel to the main channel is the most critical point for a canal system containing offtake that bifurcates from main channel. This zone is vulnerable to siltation due to stagnation of flow, at the same time turbulence is experienced at the nose point due to geometrical difference between two channels. In this study velocity variation a junction has also been observed for different offtake angles and various discharge conditions.

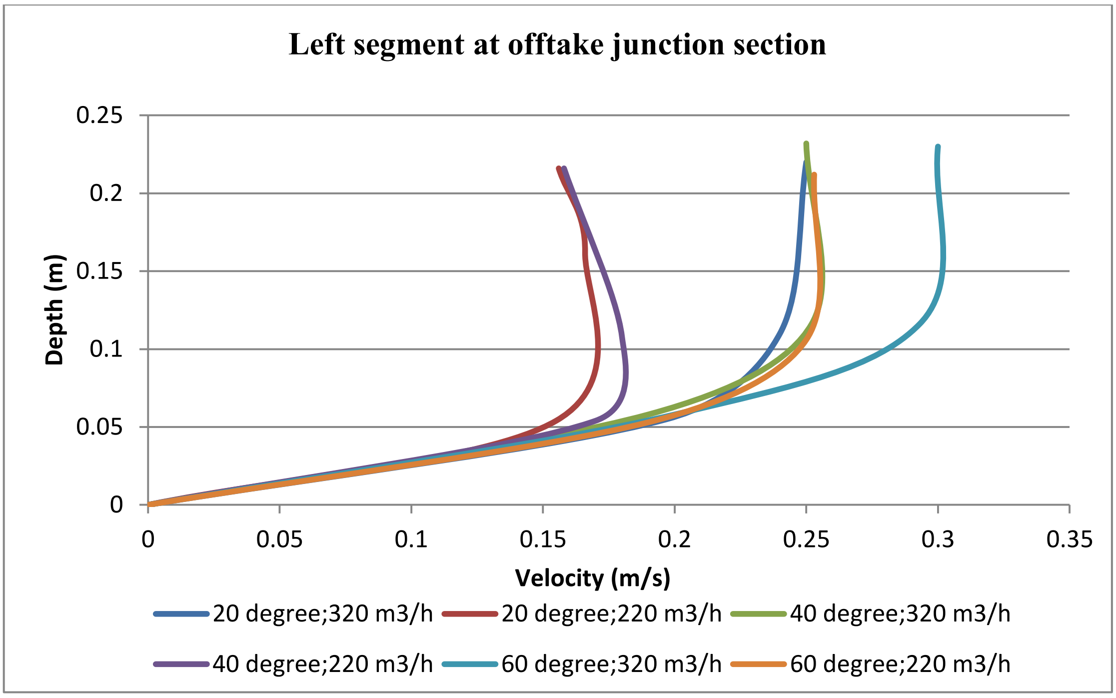


Figure 6.7: Velocity variation at offtake junction (left segment)

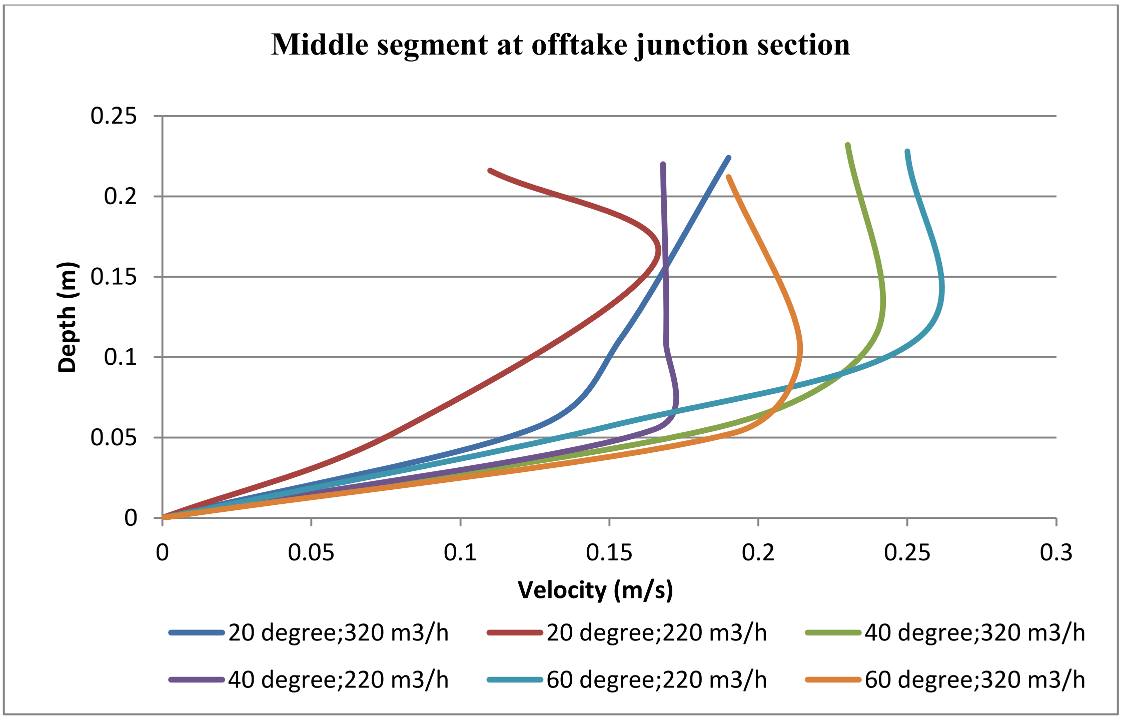


Figure 6.8: Velocity variation at offtake junction (middle segment)

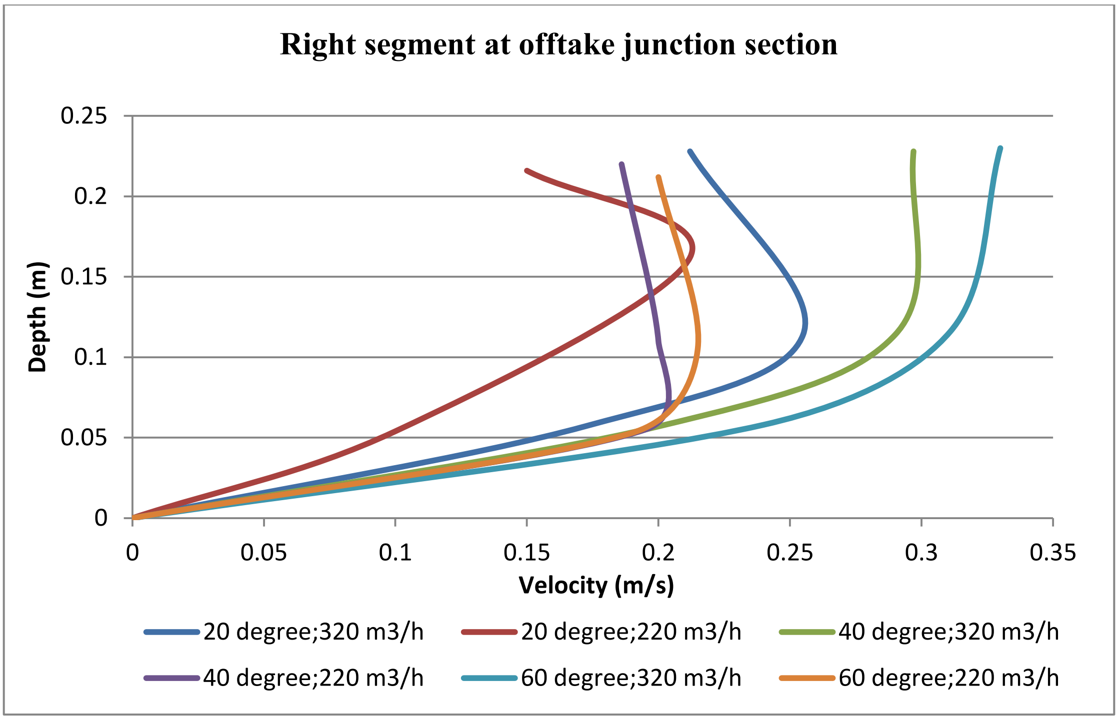


Figure 6.9: Velocity variation at offtake junction (right segment)

Velocities at the region generally depend on junction angle, shape and material of channel. In this study cross section at the junction of offtake and main channel has been shown in **Figure 5.22**. For analyzing velocity variation at the junction due to discharge and offtake angle velocity values of left middle and right segment are plotted in **Figure 6.7 to 6.9**. It is observed from the figures that angle of offtake does not have significant effect on left segment of junction section velocity. But for the middle and right segments an increase in offtake angle results increase in velocity at the mouth of the offtake. The velocity profiles in junction were found to be slightly irregular when compared to typical velocity distribution profile in a regular channel. This irregularity may be due to the development of secondary current and stagnation zone at the junction of the offtake and main channel.

6.3.3 Slope and Manning's n Value Adjustment with Conveyance and Discharge Plot

The conveyance of a channel section is a measure of the carrying capacity of the channel section because it is directly proportional to channel discharge. Using Manning's formula is used as uniform flow formula the conveyance (k) becomes a function of discharge and slope.

This equation can be used for computing the conveyance when the discharge and slope of the channel are known. In this section comparison has been made between measured and theoretical discharge to conveyance value for main and offtake channel. Measured or actual conveyance was determined by considering conveyance as a function of water discharge and slope, where value of slope was taken as water surface slope assuming that flow in uniform for each channel.

Theoretical line was obtained from equation of manning's **n** (Equation 6.1) and considering conveyance as a function of wetted area and hydraulic radius (Chang, 1992).

$$n=\frac{d_{50}^{\frac{1}{6}}}{25.6}.y^{-0.95} \tag{6.1}$$

Where, d_{50} (in meters) is average particle diameter of material used in bed of the channel. Plaster finishing with snowcem is used for our case. Thus assumed value of d_{50} for cement powder is considered to be 0.00001 meter from **Figure 6.10**. Depth of water surface from channel bottom is expressed as y (meters). Possible depths in both channels (0.1 m to 0.35 m) were used in Equation 6.1 to obtained depth dependent manning's coefficient values. For our case **n** values of varies from 0.017 to 0.026.

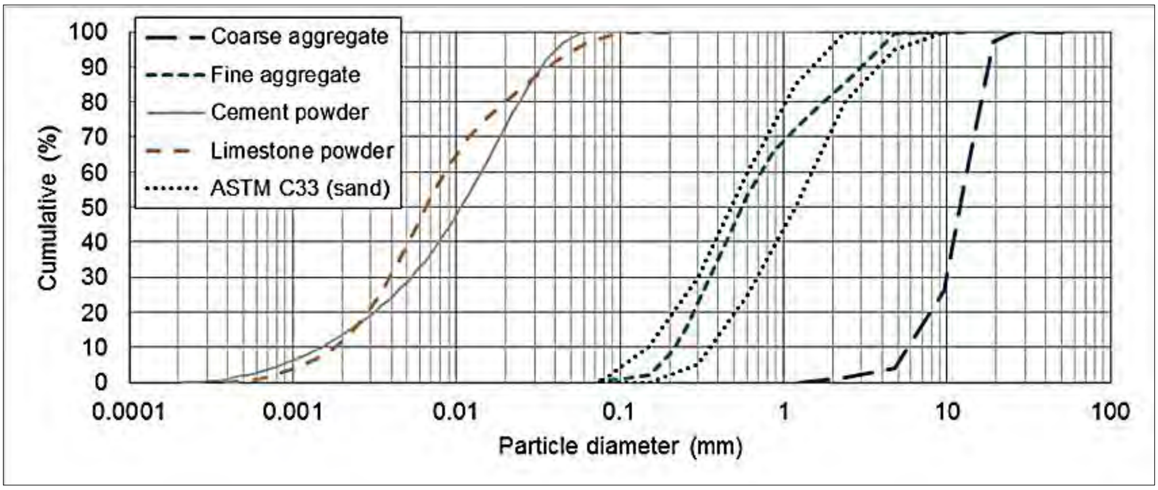


Figure 6.10 Cumulative particle size distributions for limestone powder, cement, sand, and coarse aggregate used in the three concrete mixtures. (Bentz D.P. et. al. Minimizing Paste Content in Concrete Using Limestone Powders – Demonstration Mixtures, 2016).

Hydraulic parameters, area and hydraulic radius were calculated with possible depths and properties of channel geometry. With those values, theoretical conveyance and discharge was calculated assuming constant uniform slope 0.00385.

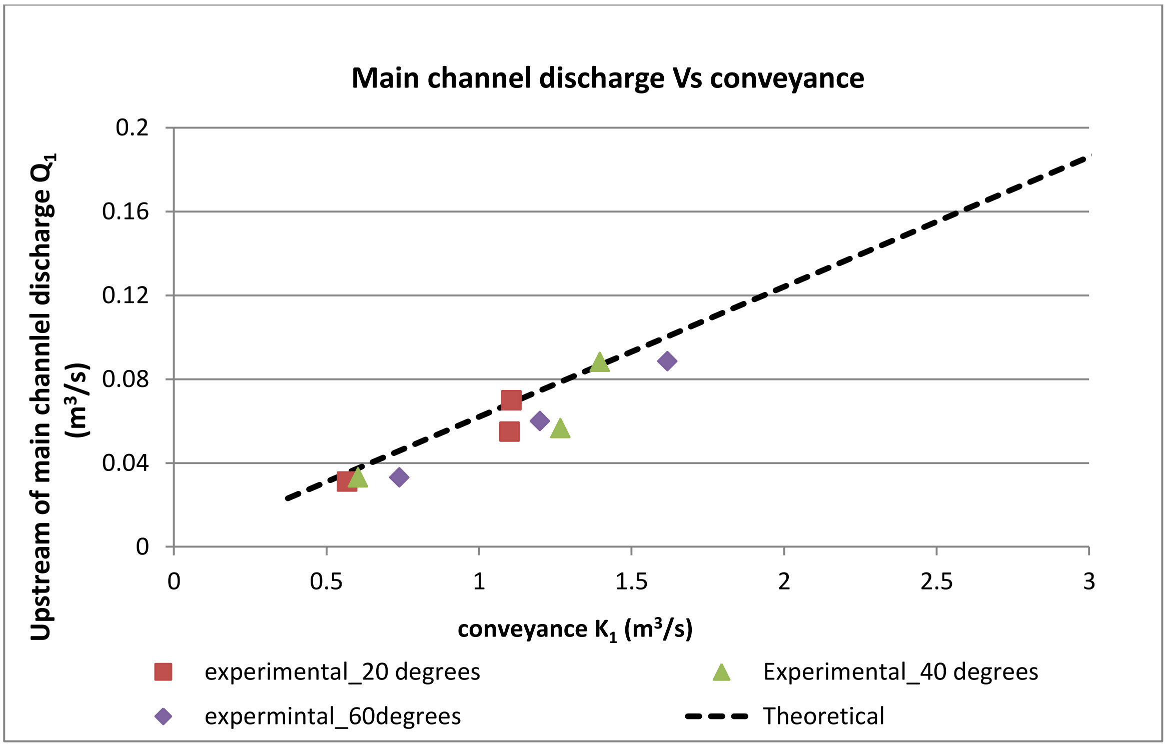


Figure 6.11: Plot of upstream discharge Vs. main channel conveyance

In **Figure 6.11** and **Figure 6.12** it is observed that measured experimental values fit into theoretical line and there exist a linear relationship between discharges and conveyances for both main and offtake channel. Conveyance capacity for both channels increase with the increase of discharge and vice versa. In the figure subscript 1 and 2 are used respectively for upstream main channel and downstream offtake channel. It has also been observed that main channel would have greater conveyance capacity for a particular discharge than offtake channel with the same discharge. As sediment flow is a direct function of water discharge it can be conclude that more water and sediments will flow through main channel for a particular discharge compared to offtake channel.

Relative values of discharge and conveyance have been plotted in **Figure 6.13** and an increasing proportional relationship has been obtained from the graph. Slope of the line indicate a ratio between offtake and main channel slope which is positive.

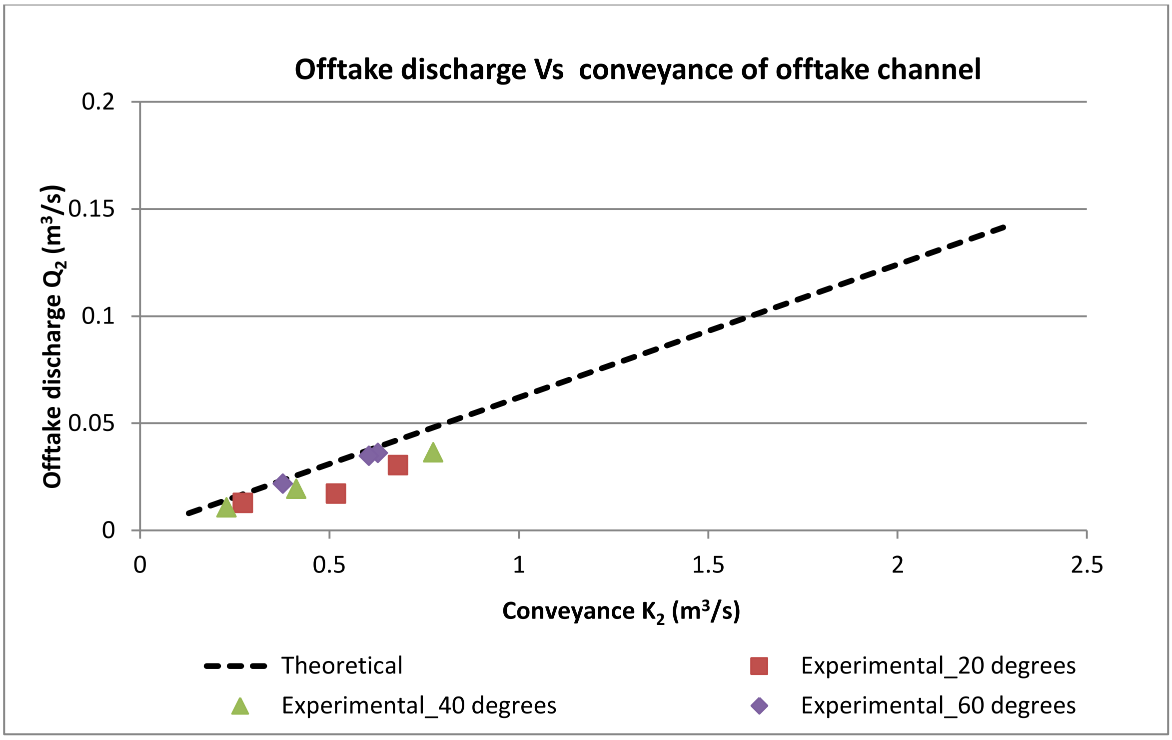


Figure 6.12: Plot of offtake discharge Vs. offtake channel conveyance

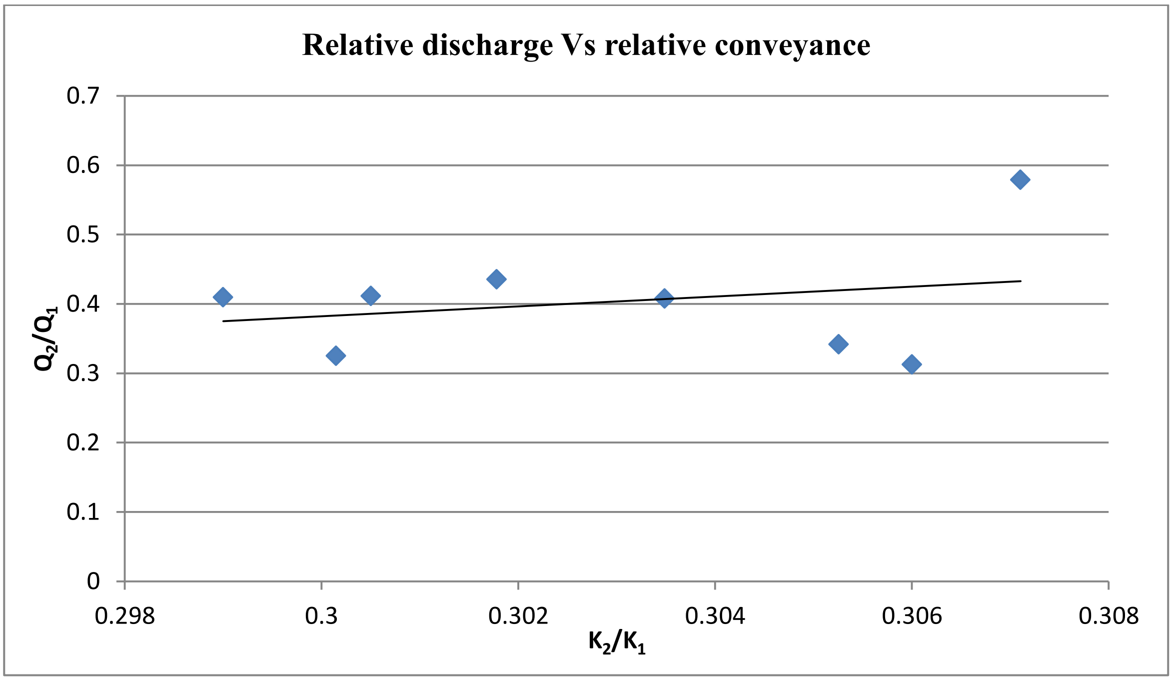


Figure 6.13: Relative discharge against relative conveyance

6.4 Sediment Transport

Sediment transport in rivers is associated with a wide variety of environmental issues such as velocity of flow, input sediment load, sediment size, channel geometry, bed and bank material etc. The interrelated characteristics of water that govern its ability to entrain and move sedimentary particles are density, viscosity, and acidity.

The lack of certainty in solving specific sediment transport problems is in part a result of the extremely limited number of situations in which predictive techniques, such as bed load or bed material transport formulas, have been substantiated by field measurement. Even for techniques that have been substantiated, little information is available about the specific hydraulic characteristics for comparison with conditions for the problem to be solved (Cooper et a1. 1972). In this thesis sediment data has been compared with three sediment transport formulas Ackers-White (1973), Engelund-Hansen (1967) and Van Rijn (1984).

Table 6.2: Comparison table for sediment discharge calculation

Angle (degrees)	Sediment discharge (ton/day)				
	Channel	**Experimental values**	**Ackers-White**	**Engelund-Hansen**	**Van Rijn**
No	main	7.3181	3.1090	26.3999	0.6477
	..	..	..	..	..
20	upstream main	7.1340	3.1768	20.8349	0.9008
	offtake	1.5523	3.2268	8.8177	1.1715
40	upstream main	6.3262	3.3884	23.2860	0.9189
	offtake	1.5150	1.5698	6.2530	0.4186
60	upstream main	5.3894	4.3077	24.9874	1.4594
	offtake	2.0505	4.1808	10.7311	1.5150

Table 6.1 and Table 6.2 shows higher diverting angle attracts more suspended sediment load. Sediment discharge is proportional to the water discharge. It has also been observed from **Table 6.2** that Ackers-White formula predicts sediment discharge reasonably where, Engelund-Hansen formula over estimate and Van Rijn formula under

estimate the results. As sediment transport formulas are greatly influenced by experimental condition and sediment size, it can be said that Acker's-White experimentation condition is better suited for representing laboratory setup established in the study.

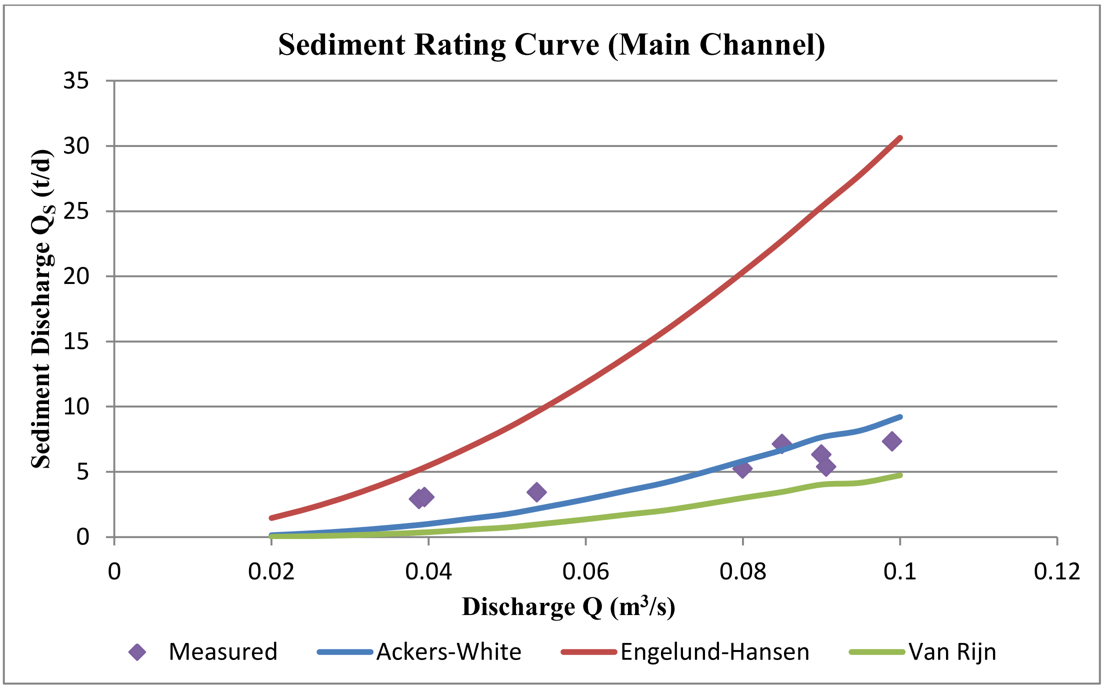


Figure 6.14: Sediment rating curve for main channel

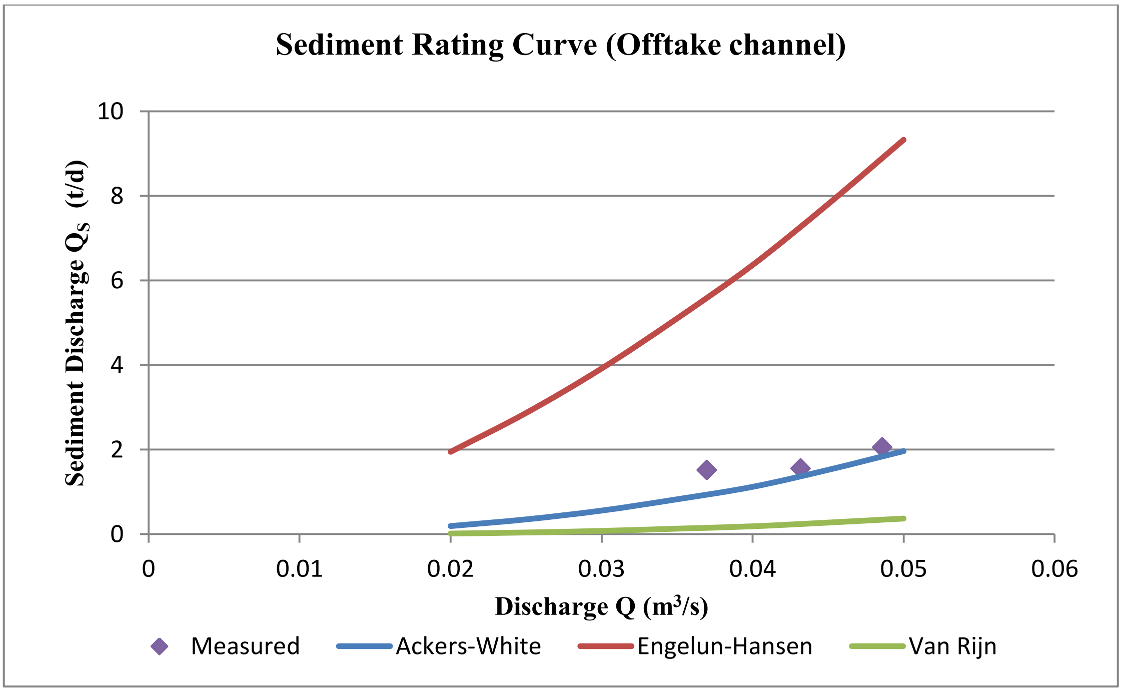


Figure 6.15: Sediment rating curve for offtake channel

Sediment rating curve has been prepared for the existing experimental setup using these three formulas. For preparation of sediment rating curve water discharge has been varied within the capacity of pump and experimental channel. Normal depths for the discharges have been calculated. Parameters required for the equations have been calculated to obtain sediment rating curves. Separate curves for main channel and offtake of constructed experimental set up have been shown in **Figure 6.14 and 6.15**.

For all cases, sediment transport rate increases with the increase of water discharge. Sediment transport behavior is closely matched with the prediction derived from Ackers-White formula. The sediment transport rating curve would help in predicting sediment discharge as a function of flow discharge for similar type of offtake configuration.

6.5 Development of Dimensionless Equation

Dimensional analysis represents a functional relationship among non-dimensional parameters involving in the phenomena. Dimensional analysis helps in determining a systematic arrangement of the variables in the physical relationship, combining dimensional variables to form non-dimensional parameters. Critical task during the analysis is to decide the governing parameters which would have significant effect on the developed functional relationship.

6.5.1 Flow Distribution at Offtake Bifurcation

Flow determination through a channel is dependent on fluid properties, channel geometry and properties of bed and bank material. At first governing parameters are to be decide for each channel to determine the equation for discharge on main and offtake channel. For determining flow distribution due to presence of offtake following parameters were taken into consideration for dimensional analysis

$$Q_1, Q_2, B_1, B_2, Y_1, Y_2, \rho, \mu, \theta, S \qquad (6.2)$$

Where, **Q** is the discharge, **B** is the width, **Y** is the flow depth and subscript 1 and 2 indicate upstream of main channel and offtake channel respectively. **ρ** is the density of fluid, **μ** is viscosity of fluid, **θ** is the angle of offtake and ***S*** is the slope of the channel.

Several methods for dimensional analysis had been performed to obtain best possible dimensionless equation to fit with the data obtained from the experiment. Among them equation obtained from Rayleigh method of dimensional analysis provides best possible solution to this problem. This form of dimensional analysis expresses a functional relationship of some variables in the form of an exponential equation. Equation obtained from dimensional analysis of considered parameters is as followed

$$\frac{Q_2}{Q_1} = \mathrm{a}\,.\,\theta.S\,(\frac{\nu}{q_1})^b(\frac{B_2}{B_1})^c(\frac{Fr_2}{Fr_1})^d \tag{6.3}$$

Here, q_1 is the discharge per unit width for upstream of main channel, $Fr_1$is Froude number for upstream of main channel, Fr_2 is the Froude number for offtake channel and **a, b, c, d** are powers of dimensionless terms. Powers of dimensionless terms are obtained by conducting experimentation for different angles of offtake. Thus for determining discharge ratio of main channel to offtake channel following set of equations have been obtained. Values obtained from the experiment and obtained from the Equation 6.3 have been plotted to determine the co-relation between them.

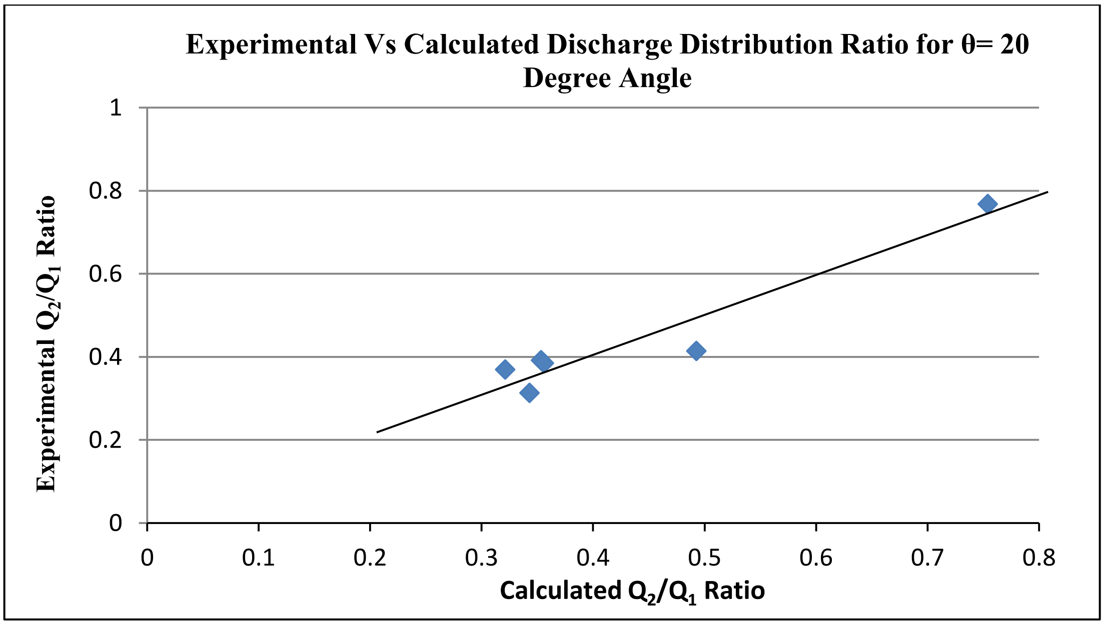


Figure 6.16: Experimental Vs. calculated discharge distribution ratio for 20 degree angle

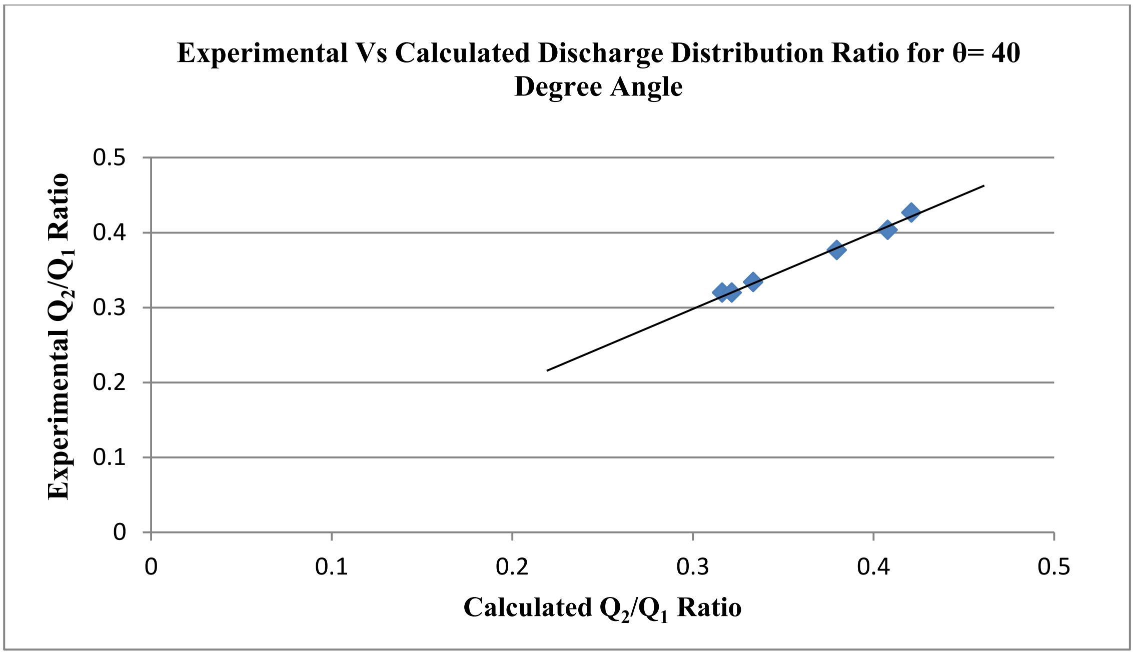


Figure 6.17: Experimental Vs. calculated discharge distribution ratio for 40 degree angle

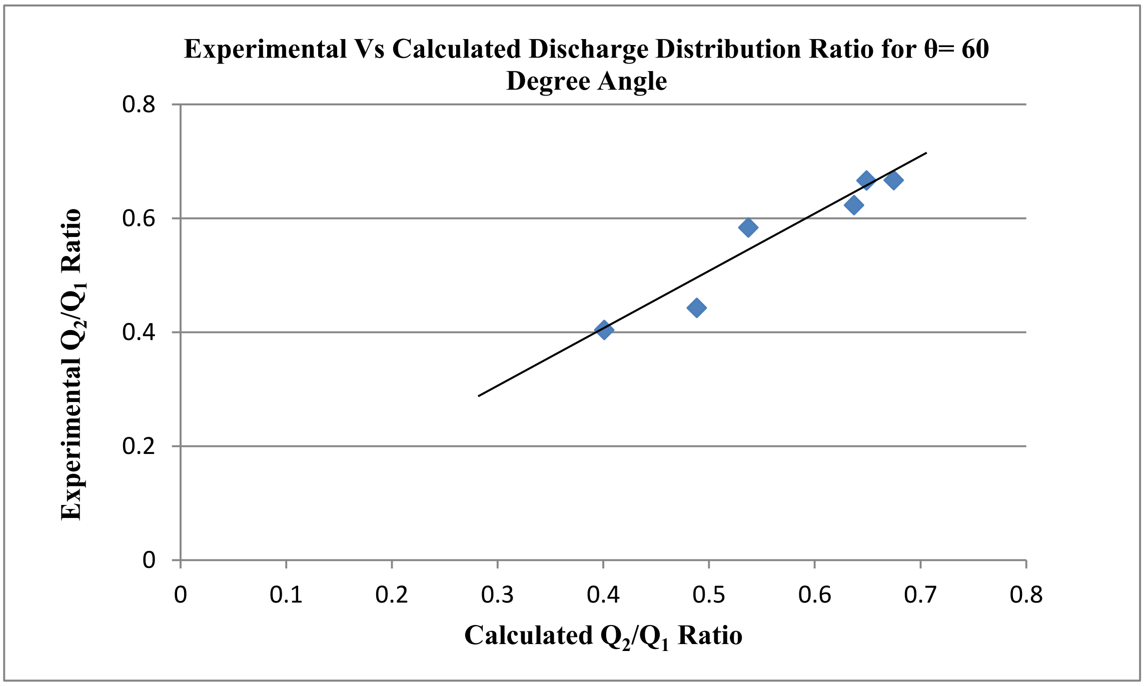


Figure 6.18 Experimental Vs. calculated discharge distribution ratio for 60 degree angle

Figure 6.16 to 6.18 show more than 90% co-relation for all the three cases and the discrepancy ratios vary between 0.87 - 1.2. As discrepancy ratio (DR) between 0.5 to 1.5 is considered as good prediction, thus predictability of the proposed equation is considered very good.

Table 6.3: Proposed equation and their co-relation with experimental values

Angle of offtake, θ	Calculated equation	R^2
20^0	$Q_2/Q_1 = 9.78\ \theta S\ (\nu/q_1)^{-0.279}\ (B_2/B_1)^{3.223}\ (Fr_2/Fr_1)^{0.647}$	0.916
40^0	$Q_2/Q_1 = 9.99\ \theta S\ (\nu/q_1)^{-0.002}\ (B_2/B_1)^{1.538}\ (Fr_2/Fr_1)^{0.837}$	0.993
60^0	$Q_2/Q_1 = 10.12\ \theta S\ (\nu/q_1)^{0.216}\ (B_2/B_1)^{-0.147}\ (Fr_2/Fr_1)$	0.926

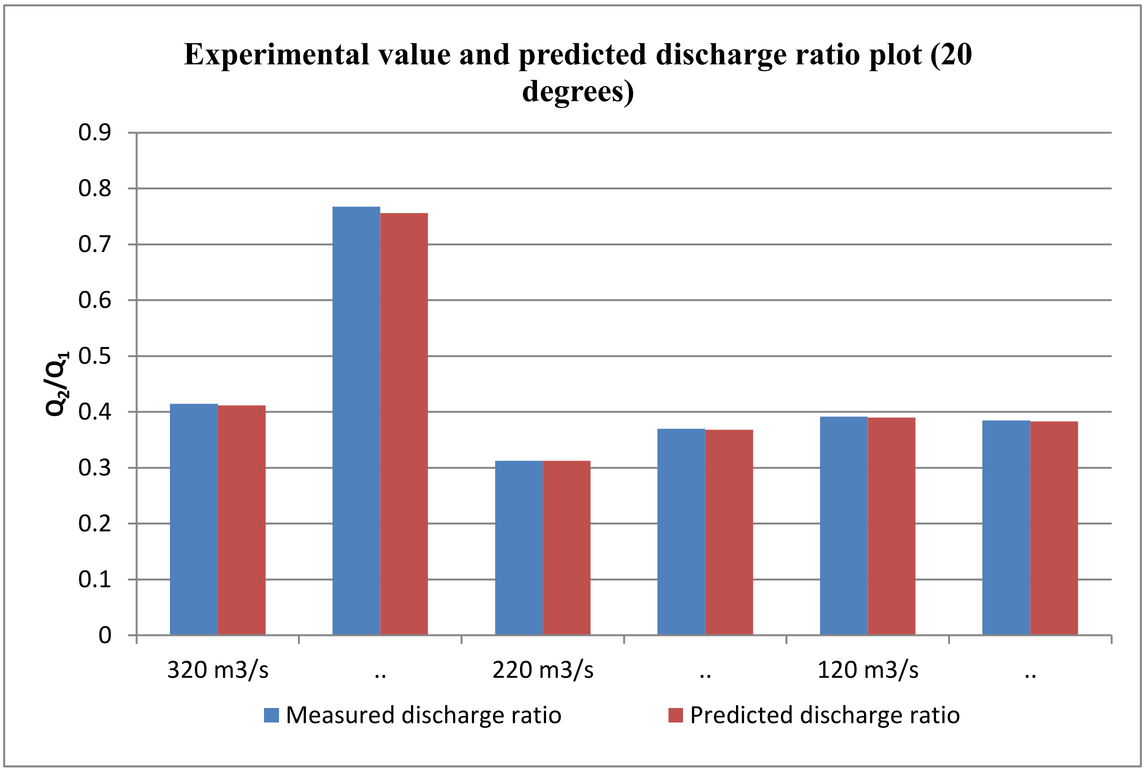


Figure 6.19: Column chart showing experimental vs. predicted Q_2/Q_1 values from equations for 20 degree offtake angle

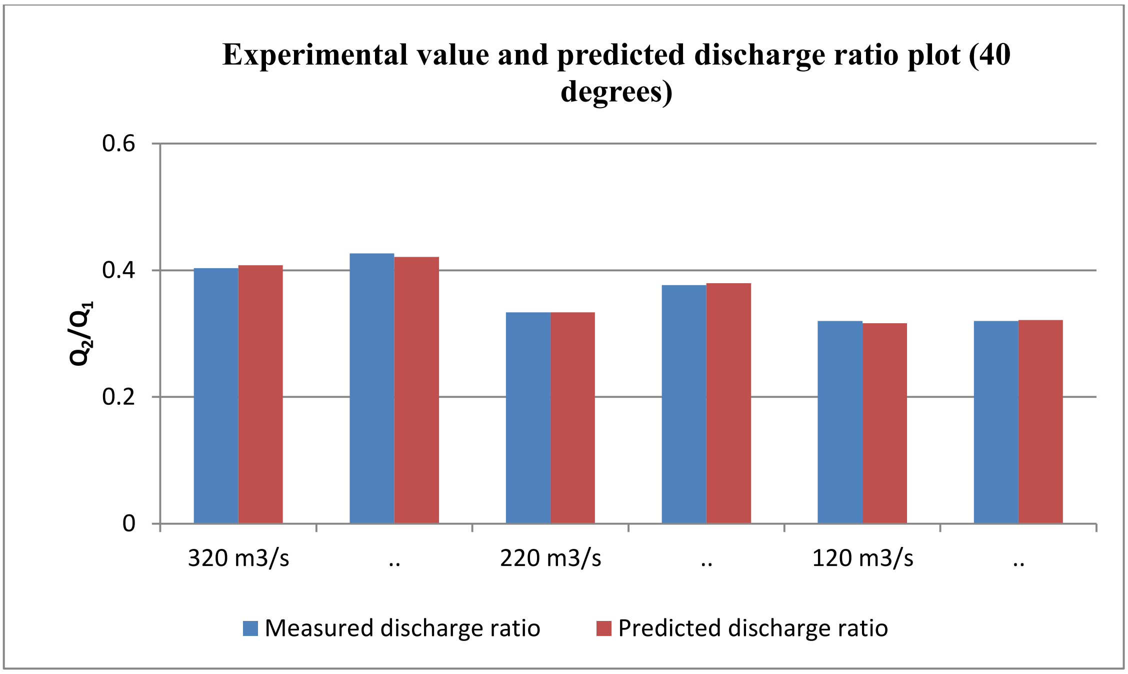


Figure 6.20: Column chart showing experimental vs. predicted Q_2/Q_1 values from equations for 40 degree offtake angle

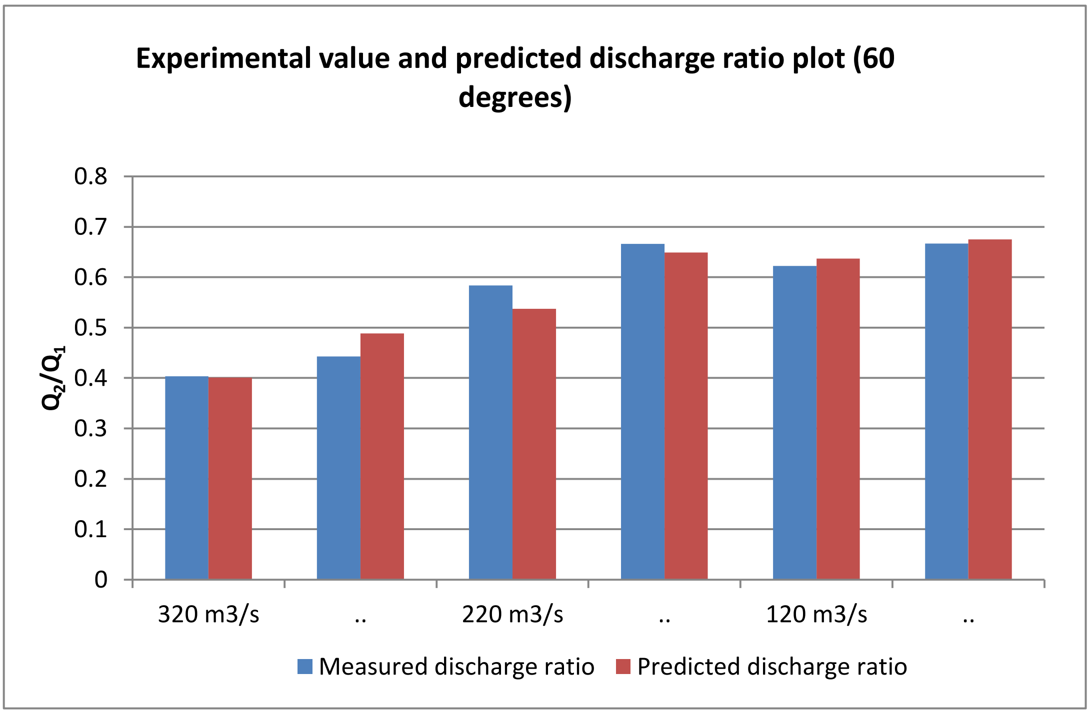


Figure 6.21: Column chart showing experimental vs. predicted Q_2/Q_1 values from equations for 60 degree offtake angle

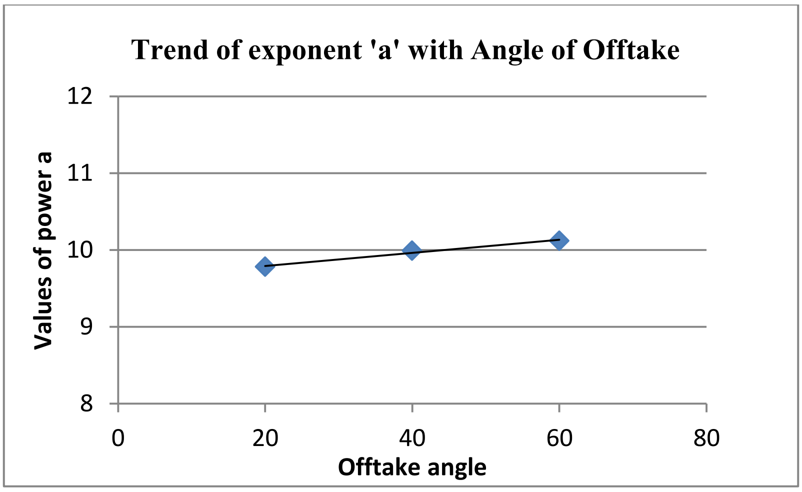


Figure 6.22: Trend of exponent of dimensionless term (for a)

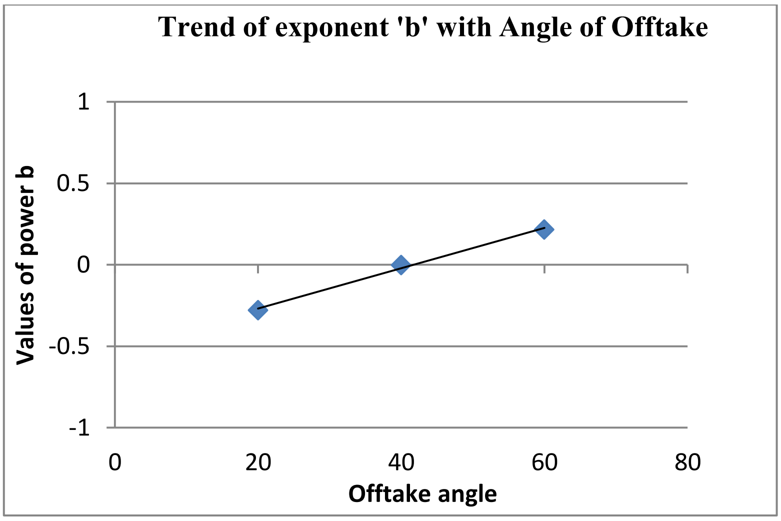


Figure 6.23: Trend of exponent of dimensionless term (for b)

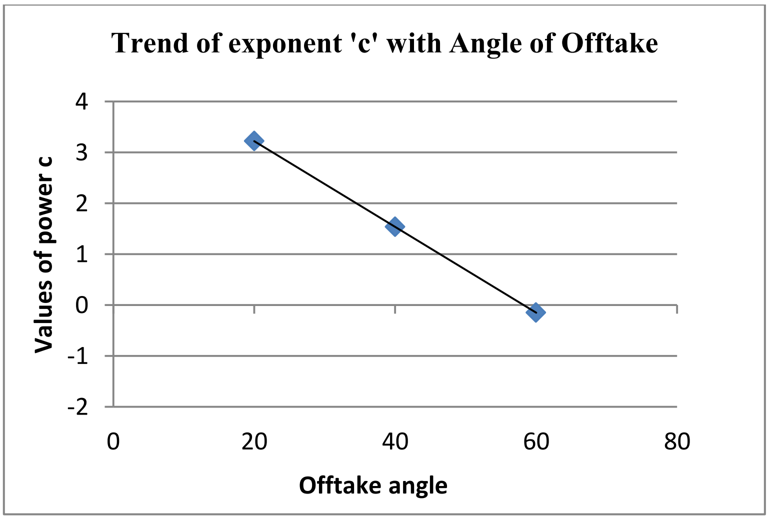


Figure 6.24: Trend of exponent of dimensionless term (for c)

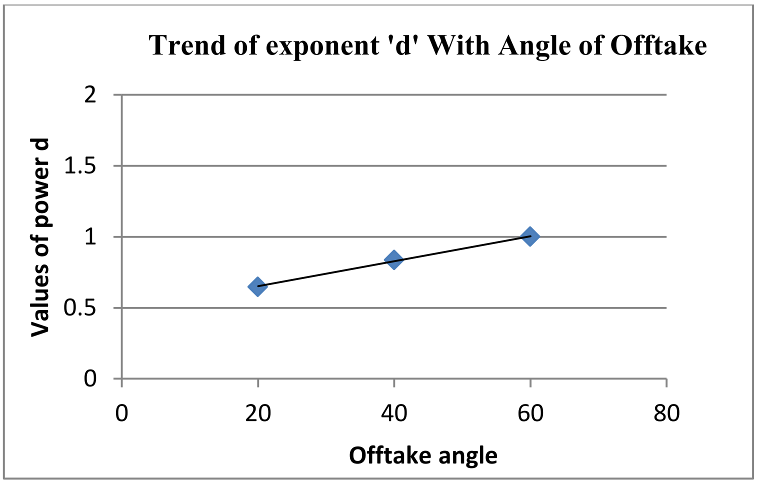


Figure 6.25: Trend of exponent of dimensionless term (for d)

Table 6.4: Trend of exponent of dimensionless term

Power of dimensionless term	**Equation in terms of offtake angle**	**Equation No.**
a	$\mathbf{a_\theta} = 0.008\boldsymbol{\theta} + 9.624$	6.4
b	$\mathbf{b_\theta} = 0.012\boldsymbol{\theta} - 0.517$	6.5
c	$\mathbf{c_\theta} = -0.084\boldsymbol{\theta} + 4.908$	6.6
d	$\mathbf{d_\theta} = 0.008\boldsymbol{\theta} + 0.475$	6.7

Table 6.4 shows dimensionless terms and their relation with offtake angle, θ. Correlation between exponents and θ varies from 1 to 0.98. Thus for moderate slope with variable offtake angle following equation can be proposed to predict discharge of offtake channel.

$$\frac{Q_2}{Q_1} = a_\theta . \left(\frac{\nu}{q_1}\right)^{b_\theta} \left(\frac{B_2}{B_1}\right)^{c_\theta} \left(\frac{Fr_2}{Fr_1}\right)^{d_\theta} . \theta . S \qquad (6.8)$$

Where, values of $\mathbf{a_\theta}$, $\mathbf{b_\theta}$, $\mathbf{c_\theta}$ and $\mathbf{d_\theta}$ are obtained from Equation 6.4 to 6.7 respectively.

6.5.2 Sediment Discharge Distribution at Offtake

The distribution of sediment over the downstream branches is governed by local geometry of the channel system and sediment properties. The dependent variables govern for sediment transport rate in the offtake are flow velocity (V), sediment discharge (Q_S), water depth (Y), and the bed level (Z). The independent variables are the longitudinal distance (x) and time (t).

According to Wang (1995) it is reasonable to assume that the nodal-point relation at the bifurcation depends on following dimensionless quantities

$$\frac{Q_{S2}}{Q_{S1}} = f\left(\frac{B_2}{B_1}, \frac{Q_2}{Q_1}, \frac{C_2}{C_1}, \frac{Y_2}{Y_1}\right) \tag{6.9}$$

Where, $\mathbf{Q_S}$ is the sediment discharge **Q** is the water discharge, **B** is the width, **Y** is the flow depth, **C** is the value of Chezy's coefficient and subscript 1 and 2 indicate upstream of main channel and offtake channel respectively.

For ease of calculation ratios of Chezy's co-efficient and flow depth are neglected. It is assumed in the equilibrium conditions stable equilibria remain stable. As same sediment material was issued throughout the experimental run ratio of Chezy's co-efficient would have insignificant effect on the proposed relation. Again as water discharge is a function of water depth, ratio of upstream and downstream water depth can be neglected.

Combining general equation of 1D nodal point proposed by Islam (1996) and modified equation for concave offtake proposed by Obasi (2012) following relationship has been given

$$\frac{Q_{S2}}{Q_{S1}} = x\left(\frac{q_2}{q_1}\right)^y \theta^z \tag{6.10}$$

Here, $\mathbf{q_1}$ is the specific discharge for upstream of main channel, $\mathbf{q_2}$ is the specific discharge for offtake channel and **x,y**, **z** are powers of dimensionless terms respectively. Analysis has been performed to determine the value of x, y and z of the equation. Proposed equation for sediment discharge ratio of offtake channel to upstream main channel is as follows

$$\frac{Q_{S2}}{Q_{S1}} = .043\left(\frac{q_2}{q_1}\right)^{1.039} \theta^{0.458} \tag{6.11}$$

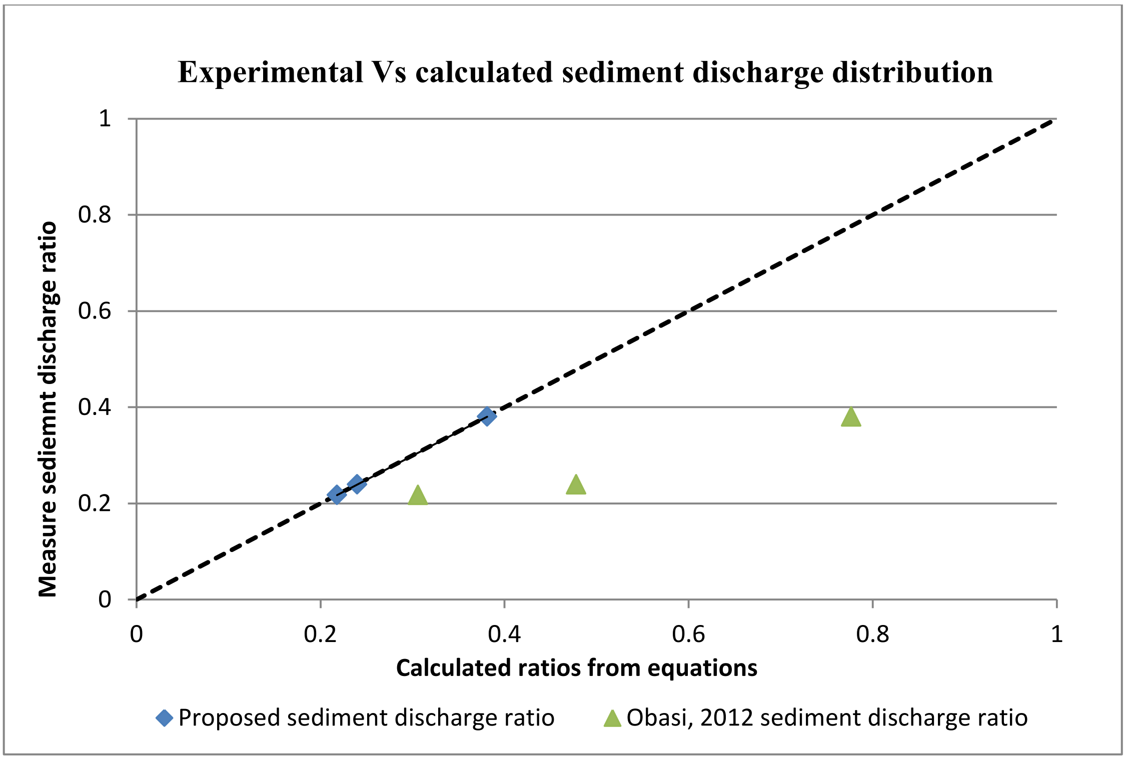


Figure 6.26: Experimental Vs. calculated sediment discharge distribution

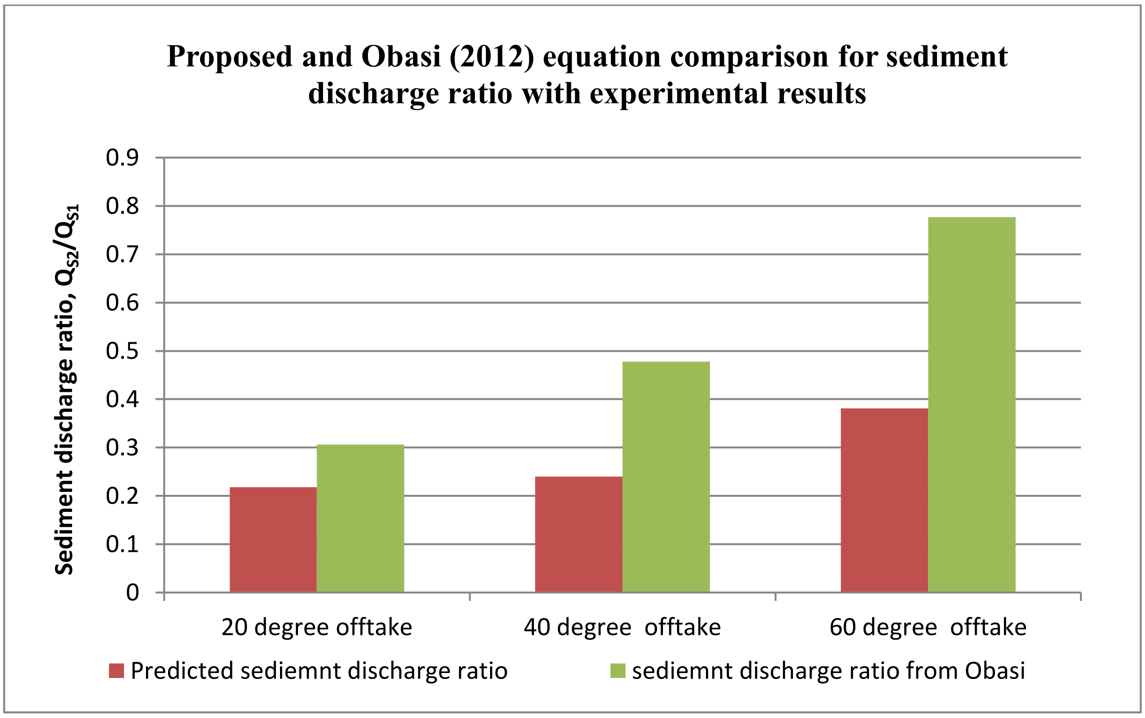


Figure 6.27: Proposed and Obasi's equation comparison for sediment discharge ratio

Proposed formula has been compared to measured data during experimental run, which shows that predicted sediment discharge ratio fits the discharge ratio obtained from the experimental. In **Figure 6.26 and 6.27,** Obasi's (2012) concave channel bifurcation equation has also been compared with proposed equation. Discrepancy ratio from proposed to measured value is almost equal to unity has been observed for our case.

6.6 Comparison of Proposed Prediction Equations with Field Observation

Comparison of proposed equation has been made to determine its applicability to practical field condition. Though successful comparison with actual field data involves lot of uncertainty and limitation, an attempt has been made in this study to compare the predicted equation as closely as possible.

In order to do that, reconnaissance study was conducted on river offtakes of Bangladesh to identify nearest matched offtake bifurcation system to the experimental configuration. Observation has been made for availability of measuring stations of Bangladesh Water Development Board (BWDB) for data comparison. Thus, four offtake systems of Bangladesh, Upper Korotoya- Dhepa, Surma- Botor Khal (offtake of Surma), Old Brahmaputra- Jhenai offtake and Ganges- Gorai were taken for relative comparison of proposed flow discharge distribution equation. Unattainable data of the selected rivers have been estimated or calculated with numerical methods. But for sediment discharge comparison, Ganges –Gorai offtake has been taken into consideration as there is lack of sufficient sediment data for other offtakes of Bangladesh.

6.6.1 Field Data Collection

Data for river offtake channels have been collected from BWDB (2011). These data have been used to verify the developed equations. The data include water level, cross sections and sediment size etc. Summary of the field data used is listed in **Table 6.5**.

Table 6.5: List of available data used for comparison

River Offtake System Name	Station Name	Station ID	Type of Data
Upper Korotoya-Dhepa	Khansama	SW 142	Water Level, Cross Section
	Kantanagar	SW 78	Water Level, Cross Section
Surma- Botor Khal	Chhatak	SW 268	Water Level, Cross Section
	Gobindaganj	SW 33	Water Level, Cross Section

River Offtake System Name	Station Name	Station ID	Type of Data
Old Brahmaputra-Jhenai offtake	Offtake of Jhenai	SW 134B	Water Level, Cross Section
Ganges –Gorai	Hardinge Bridge	SW 90	Water Level, Discharge, Velocity,
			Sediment Discharge
	Gorai Railway Bridge	SW 99	Water Level, Discharge, Velocity,
			Sediment Discharge

6.6.2 Comparison of Discharge Distribution

For comparison four river offtakes of Bangladesh have been selected, i.e. Korotoya-Dhepa, Old Brahmaputra- Jhenai Offtake, Surma- Botor Khal (Offtake of Surma) and Ganges- Gorai Offtake. During selection of offtakes, width and offtake angle were

Table 6.6: River parameters used for comparison

Name	Offtake angle (degrees)	Water level (m, RL)	Average cross-sectional area (m^2)	Avg. width (m)	Bank full discharge (m^3/s)	Slope
Korotoya	30	37	4500	375	3700	$4.5X10^{-5}$
Dhepa		42	359	120	170	..
Surma	40	7.5	1267	100	950	$6.5X10^{-5}$
Botor Khal		19	232	60	140	..
Old Brahmaputra	40	15	1611	200	550	$6.4X10^{-5}$
Jhenai Offtake		16	80	40	30	..
Ganges	65	10	..	5000	48000	$2.6X10^{-5}$
Gorai		7	..	1200	1300	..

considered so that river is comparable to the experimental channel. Cross section used for calculation of bankful discharge has been shown in **Figure A1** to **Figure A7**. For Ganges-Gorai River Offtake comparison average data and

Table 6.6 shows calculated value for cross sectional area, average width, bankful discharge and regime slope using Lacey's Regime Equation. Values obtained from the calculation were used in Equation 6.8 to obtain discharge distribution ratio.

As the experimental reach is of constant width and bed slope Equation 6.8 has been corrected for variation of width of the channel. Exponent of width ratio (B_2/B_1) has been multiplied by a factor C'.

$$C' = 0.1802 - 0.0041\theta \tag{6.12}$$

So the modified equation for field condition is as follows

$$\frac{Q_2}{Q_1} = a_\theta . \left(\frac{\nu}{q_1}\right)^{b_\theta} \left(\frac{B_2}{B_1}\right)^{c'_\theta} \left(\frac{Fr_2}{Fr_1}\right)^{d_\theta} . \theta . S \tag{6.13}$$

$$\text{Where, } c'_\theta = C' * c_\theta = 3.44X10^{-4}\,\theta^2 - 0.035\theta + 0.8844 \tag{6.14}$$

Value obtained from Equation 6.13 discharge ratio values are tabulated in **Table 6.7**.

Table 6.7: Observed and predicted value

River Offtake System Name	Field Observed Q_2/Q_1	Predicted Q_2/Q_1	Discrepancy Ratio
Upper Korotoya- Dhepa	0.046	0.080	1.58
Surma- Botor Khal	0.147	0.135	0.87
Old Brahmaputra- Jhenai offtake	0.055	0.042	0.77
Ganges –Gorai	0.05	0.0042	0.10

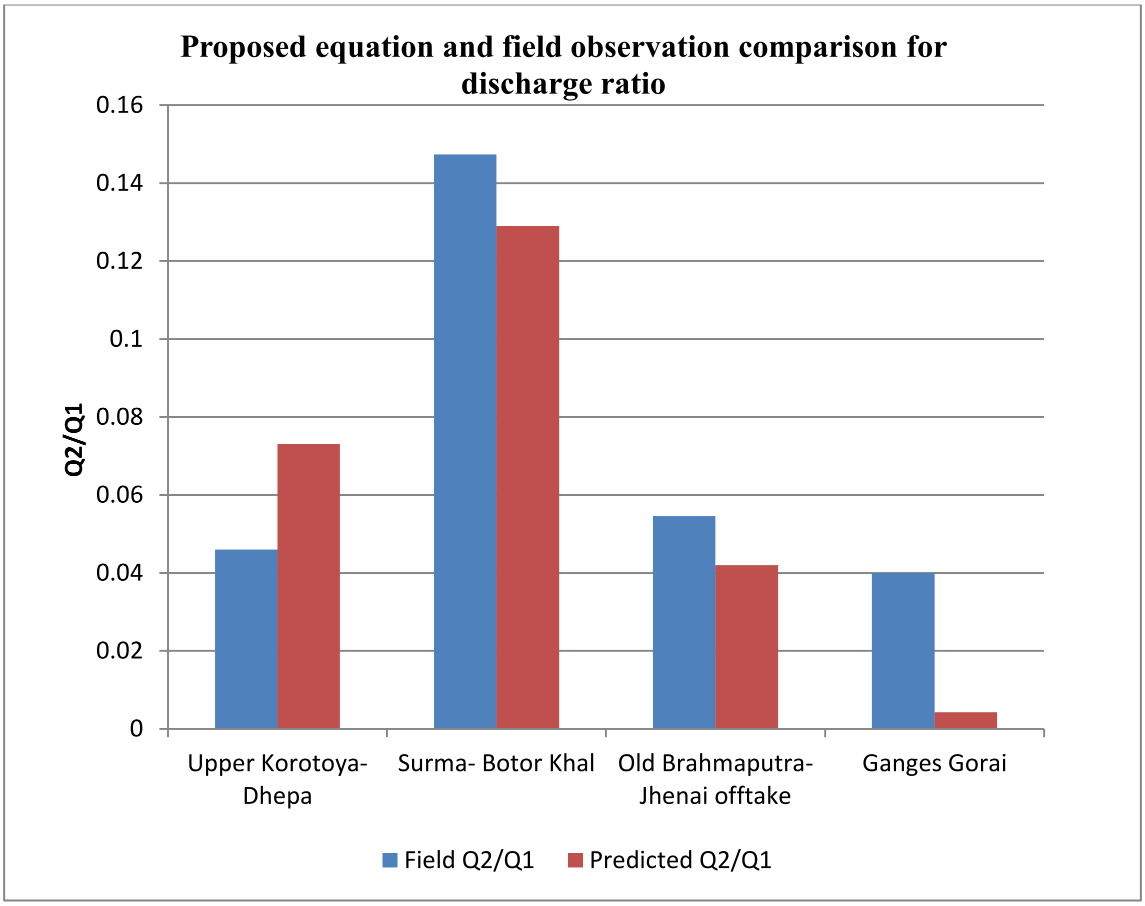


Figure 6.28: Field observation and calculated Q_2/Q_1 comparison

It has been observed from **Figure 6.28** that values calculated using proposed equation (Equation 6.13) predicts satisfactory. Discrepancy ratio of Korotoya- Dhepa, Surma-Botor Khal and Old Brahmaputra- Jhenai offtake system varies between 0.77 to 1.58.

Field comparison with the Gorai offtake has been done separately with the available time series discharge for 2015-16. It has been observed that discharge ratio (Q_2/Q_1) varies from 0.003 to 0.1 respectively from dry flow condition to high flow condition. The predicted value using Equation 6.13, the discharge ratio 0.004 falls within the observed range of field condition and it is adjacent to flow distribution ratio during dry flow.

In **Figure 6.29** degree of compliance of proposed equation can be seen. Dashed line shows discrepancies of predicted and observed value for both experimental and field condition.

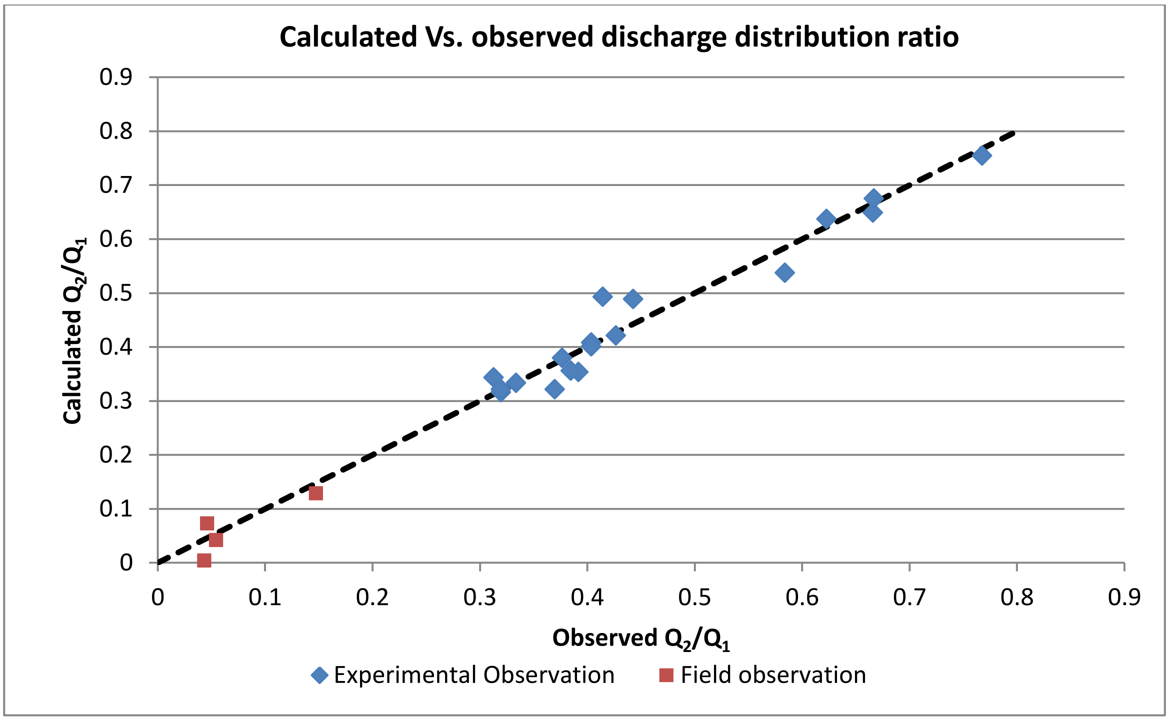


Figure 6.29: Calculated Vs. observed discharge distribution ratio

6.6.3 Comparison of Sediment Discharge Distribution

For sediment discharge comparison, field data of Ganges- Gorai River offtake has been used. Sediment concentration and water discharge data at specific stations mentioned in **Table 6.5** was obtained for Ganges-Gorai River offtake system and Equation 3.21 has been used to calculate sediment discharge (Q_S). Values obtained from field sediment concentration has been considered as field observed sediment discharge value and values predicted using Equation 6.11 has been considered as predicted sediment discharge value.

Both values are plotted in **Figure 6.30** which shows a good match between predicted and field observed values. During low flow discrepancy ratio varies from 0.64 to 1.18 while for high flow during monsoon and post monsoon season discrepancy ratio varies from 0.08 to 2.87.

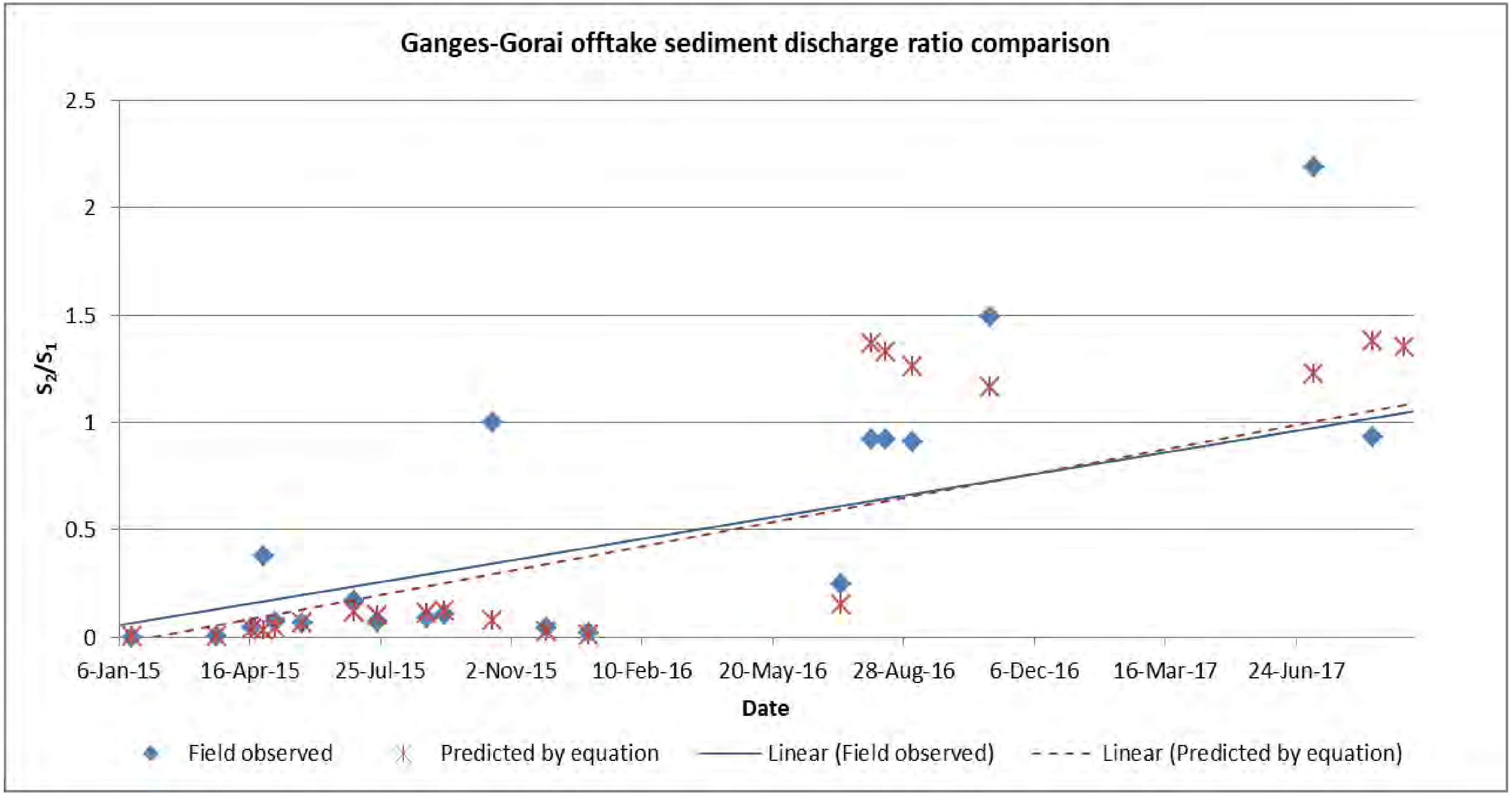


Figure 6.30: Predicted and observed sediment discharge distribution ratio comparison for Ganges-Gorai offtake

6.7 Discussions

Experimental setup was designed and constructed in which an offtake channel bifurcates at 20^0, 40^0 and 60^0 offtake angles. Discharge of 320 m^3/h (0.09 m^3/s), 220 m^3/h (0.06 m^3/s) and 120 m^3/h (0.03 m^3/s) was set for experimentation. Sediment run was made for maximum discharge. The discharge range and sediment size are dependant to the experimental setup which is limited to pump capacity and available space in the laboratory. Observation was also made for mobile bed condition. During the construction of the setup difficulties occur during the preparation of certain offtake angles such as 60^0. In low flow condition velocity measurement was found not suitable due to the limitation of velocity measuring device in the laboratory. Two exponential equations have been proposed to determine relative water and sediment discharge. It is found that the water and sediment discharge ratios of offtake to main channel, both increases with the increase of offtake angle. However, for measuring velocities along the flow direction, degree of increase of velocity for 40^0 and 60^0 offtake angle is comparatively lower than those measured for other offtake angles. It may be due to constructional difficulties as the space required for adaptation of the full length of angle has not been available in the laboratory. Similar types of study were conducted as reported in chapter 4 (Obasi, 2008 and 2012). Obasi (2008) proposed an equation which includes upstream Reynolds number and depth ratio only. However, difference in comparison signifies that present relationship for relative water discharge determination in offtake system includes Froude number ratio instead of Reynolds number which was not been included in available past studies on offtake analysis. As the flow in experiment is turbulent in nature due to dominate by gravity flow, inclusion of Froude number criteria in the relationship seems more relevant. For prediction of sediment discharge he also proposed similar type of relationship from experimentation of concave channel bifurcation. For the purpose of comparative analysis Obasi's (2012) has been compared. It has been found that present analysis under estimates more than 40% compared to Obasi's prediction. Such discrepancies may be due to the difference in channel configuration.

For comparison field data has been obtained for selected offtake system of Bangladesh. However the field data were not readily available for all the river cases selected for comparison. For some cases flow data were generated based on channel geometry. Comparison with the field data shows reasonable accuracy of the proposed equation.

CHAPTER SEVEN
CONCLUSIONS AND RECOMMENDATIONS

7.1 Introduction

In this study an experimental work has been conducted for understanding and predicting the flow and sediment distribution at river offtake. An experimental observation was made to understand flow and sediment transport behavior at the vicinity of offtake channels. Two relationships have been derived and proposed for the prediction of flow and sediment distribution of the main and offtake channel. Proposed equations have been applied for selected four River offtake systems of Bangladesh.

7.2 Conclusion

The study is mostly based on experimental observations and field application. The conclusions of the study can be set as follows:

i. Flow distribution behavior has been observed in a laboratory for three different discharge conditions these are 0.09 m^3/s, 0.06 m^3/s and 0.03 m^3/s. Test runs were conducted with three different offtake angles i.e 20^0, 40^0 and 60^0 for each discharge conditions. From the experiment it was found that discharge ratios of offtake to main channel increases with the increase of offtake angle as shown in **Table 6.1**. This means for greater offtake angle more water would flow through the offtake.

ii. Sediment distribution has also been observed for discharge i.e. 0.09 m^3/s for all the test scenarios. Experimental measurement shows that sediment discharge ratios of offtake to main channel increases with the increase of offtake angle.

iii. The ratio of sediment discharge to ratio of water discharge (σ- values), varies from 0.49 to 0.71 with the increase of offtake angles. It indicates that the higher value of σ relatively larger the amount of sediment enters into the offtake channel compared to the incoming flow rate.

iv. From flow observation it was found that, the average velocities at the offtake channel increase with the increase of offtake angles and decrease at the downstream end of the main channel as shown in **Figure 6.3 to Figure 6.4**. Flow observation at junction section shows that angle of offtake does not have significant effect on velocity profile of left segment **(Figure 6.7**). But for the middle and right segments an increase in offtake angle results increase in velocity as shown in **Figure 6.8 to Figure 6.9**.

v. Sediment discharge data obtained from experiment were compared with three well known sediment transport formulas. It has been found that Ackers-White formula predicts better sediment discharge compared to other two. However, sediment transport rates have been over estimated using Engelund-Hansen formula and underestimated using Van Rijn formula as shown in **Table 5.2**.

vi. Two equations i.e. **Equation 6.8** and **Equation 6.11** have been proposed for the estimation of water and sediment discharge ratios for both main offtake channels. These equations are dominant as a function of angle of offtake channels.

vii. Field data collected from selected river offtake system of Bangladesh has been used to verify the predictive performance of the developed equations. The degree of compliance has been assessed by calculating the discrepancy ratio. The discrepancy ratio for offtakes named Korotoya- Dhepa, Surma- Botor Khal and Old Brahmaputra- Jhenai offtake are found to be 1.58, 0.87 to 0.77 respectively.

viii. Similarly, performance of proposed equation for sediment discharge ratio calculation has been made for the Ganges- Gorai river offtake system and it has been observed that equation predicts reasonable good as shown in **Figure 6.30**. During low flow discrepancy ratio varies from 0.64 to 1.18 while for high flow during monsoon and post monsoon season discrepancy ratio varies from 0.08 to 2.87.

ix. In addition, flow and sediment transport behavior has been observed in mobile bed condition. Flow behavior has been visualized during the test runs. It has been observed that there are four different zones of flow variation in the vicinity of the offtake. These are stagnation, turbulence, siltation and velocity drop zones as shown in **Figure 5.35**.

x. Overall it can be said that the process of sediment erosion at inner side of offtake and deposition at the mouth of outer side of the offtake as observed in the experiment can be useful insight for understanding of flow and sediment behavior of the offtake channel.

7.3 Recommendations for Future Study

Following recommendations can be suggested for further study, these are as follows:

i. Large sand bed channel with wide range of discharge and offtake angle conditions can be used for similar experimental study.
ii. Improved measurement techniques with good quality equipment can be used for similar type of studies.
iii. The proposed equations for determination of flow and sediment discharge distribution can be verified with wide range of discharge and sediment data.
iv. Mathematical modeling can be applied along with the experimental study for offtake analysis.
v. Similar study may be undertaken in physical modeling facility considering a prototype condition. Scale model study for bifurcations along the char areas of braided Rivers of can be fruitful.
vi. Similar study may be undertaken in physical modeling facility for tributary channel.

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Appendix A

Tables and Figures

Table A1: Vertical velocity distribution for 20^0 offtake angle at Q= 0.09 m^3/s

Cross Section No.	Segment in cross section	Velocity (m/s)		
		0.2 depth	0.6 depth	0.8 depth
M4	LS	0.355	0.34	0.368
	M	0.36	0.26	0.29
	RS	0.245	0.147	0.17
M5	LS	0.387	0.24	0.31
	M	0.318	0.12	0.53
	RS	0.212	0.07	0.122
M6	LS	0.28	0.195	0.22
	M	0.252	0.095	0.076
	RS	0.174	0.105	0.042
Nose	LS	0.25	0.24	0.197
	M	0.199	0.154	0.125
	RS	0.212	0.155	0.172
O1	LS	0.41	0.347	0.312
	M	0.32	0.298	0.267
	RS	0.26	0.22	0.211
O2	LS	0.315	0.247	0.23
	M	0.296	0.272	0.277
	RS	0.266	0.25	0.279
O3	LS	0.288	0.262	0.25
	M	0.284	0.294	0.29
	RS	0.23	0.245	0.259
O4	LS	0.282	0.282	0.26
	M	0.28	0.274	0.285
	RS	0.24	0.246	0.24

Table A2: Vertical velocity distribution for 20^0 offtake angle at Q= 0.06 m^3/s

Cross Section No.	Segment in cross section	Velocity (m/s)		
		0.2 depth	0.6 depth	0.8 depth
M4	LS	0.282	0.257	0.212
	M	0.217	0.255	0.187
	RS	0.119	0.223	0.138
M5	LS	0.246	0.225	0.181
	M	0.169	0.222	0.149
	RS	0.065	0.185	0.102
M6	LS	0.182	0.152	0.072
	M	0.073	0.136	0.082
	RS	0.04	0.112	0.0357
Nose	LS	0.155	0.166	0.156
	M	0.079	0.166	0.11
	RS	0.1	0.212	0.15
O1	LS	0.23	0.243	0.192
	M	0.189	0.22	0.166
	RS	0.115	0.148	0.126
O2	LS	0.14	0.215	0.158
	M	0.179	0.21	0.154
	RS	0.148	0.149	0.142
O3	LS	0.17	0.225	0.153
	M	0.175	0.2	0.172
	RS	0.152	0.162	0.142
O4	LS	0.168	0.186	0.15
	M	0.166	0.185	0.148
	RS	0.151	0.14	0.12

Table A3: Vertical velocity distribution for 40^0 offtake angle at Q= 0.09 m^3/s

Cross Section No.	Segment in cross section	Velocity (m/s)		
		0.2 depth	0.6 depth	0.8 depth
M4	LS	0.34	0.43	0.309
	M	0.304	0.4	0.276
	RS	0.279	0.36	0.26
M5	LS	0.29	0.34	0.56
	M	0.296	0.33	0.25
	RS	0.262	0.26	0.25
5.15	LS	0.271	0.326	0.227
	ML	0.257	0.321	0.219
	MR	0.24	0.294	0.211
	RS	0.26	0.263	0.226
M6	LS	0.28	0.195	0.22
	M	0.203	0.267	0.189
	RS	0.198	0.224	0.164
Nose	LS	0.182	0.194	0.144
	M	0.24	0.19	0.12
	RS	0.29	0.18	0.14
O1	LS	0.351	0.4	0.336
	M	0.308	0.34	0.285
	RS	0.275	0.266	0.254
O2	LS	0.33	0.35	0.218
	M	0.325	0.307	0.308
	RS	0.288	0.5	0.29
O3	LS	0.327	0.357	0.3
	M	0.326	0.33	0.31
	RS	0.289	0.29	0.29
O4	LS	0.329	0.34	0.34
	M	0.328	0.32	0.31
	RS	0.299	0.25	0.26

Table A4: Vertical velocity distribution for 40^0 offtake angle at Q= 0.06 m^3/s

Cross Section No.	Segment in cross section	Velocity (m/s)		
		0.2 depth	0.6 depth	0.8 depth
M4	LS	0.207	0.209	0.23
	M	0.22	0.22	0.22
	RS	0.192	0.21	0.21
M5	LS	0.172	0.187	0.195
	M	0.192	0.2	0.19
	RS	0.18	0.179	0.185
5.15	LS	0.169	0.178	0.176
	ML	0.175	0.19	0.205
	MR	0.159	0.162	0.171
	RS	0.075	0.176	0.173
M6	LS	0.132	0.142	0.141
	M	0.139	0.163	0.126
	RS	0.129	0.139	0.102
Nose	LS	0.149	0.158	0.18
	M	0.159	0.172	0.165
	RS	0.2	0.186	0.195
O1	LS	0.199	0.208	0.23
	M	0.182	0.185	0.195
	RS	0.14	0.152	0.175
O2	LS	0.185	0.203	0.201
	M	0.185	0.189	0.212
	RS	0.156	0.149	0.202
O3	LS	0.182	0.195	0.222
	M	0.187	0.186	0.22
	RS	0.159	0.154	0.202
O4	LS	0.184	0.188	0.22
	M	0.182	0.189	0.22
	RS	0.162	0.153	0.1978

Table A5: Vertical velocity distribution for 60^0 offtake angle at Q= 0.09 m^3/s

Cross Section No.	Segment in cross section	Velocity (m/s)		
		0.2 depth	0.6 depth	0.8 depth
M4	LS	0.334	0.388	0.063
	M	0.316	0.386	0.117
	RS	0.301	0.336	0.154
M5	LS	0.29	0.309	0.244
	M	0.296	0.325	0.227
	RS	0.265	0.314	0.188
5.15	LS	0.26	0.307	0.171
	ML	0.255	0.317	0.121
	MR	0.245	0.292	0.145
	RS	0.269	0.3	0.168
M6	LS	0.148	0.23	0.119
	M	0.164	0.238	0.135
	RS	0.152	0.2	0.118
M7	LS	0.138	0.174	0.08
	M	0.135	0.23	0.085
	RS	0.188	0.21	0.0183
Nose	LS	0.2	0.293	0.3
	M	0.25	0.255	0.15
	RS	0.33	0.311	0.24
O1	LS	0.293	0.282	0.115
	M	0.26	0.327	0.227
	RS	0.27	0.324	0.32
O2	LS	0.448	0.435	0.429
	M	0.35	0.395	0.365
	RS	0.328	0.352	0.33
O3	LS	0.448	0.39	0.403
	M	0.388	0.39	0.374
	RS	0.36	0.301	0.295
O4	LS	0.316	0.396	0.428
	M	0.345	0.36	0.399
	RS	0.455	0.31	0.345

Table A6: Vertical velocity distribution for 60^0 offtake angle at Q= 0.06 m^3/s

Cross Section No.	Segment in cross section	Velocity (m/s)		
		0.2 depth	0.6 depth	0.8 depth
M4	LS	0.236	0.279	0.24
	M	0.236	0.285	0.246
	RS	0.22	0.253	0.222
M5	LS	0.193	0.244	0.185
	M	0.203	0.231	0.198
	RS	0.205	0.228	0.204
5.15	LS	0.167	0.218	0.155
	ML	0.168	0.248	0.181
	MR	0.1995	0.209	0.197
	RS	0.203	0.231	0.195
M6	LS	0.122	0.191	0.069
	M	0.1	0.17	0.102
	RS	0.096	0.131	0.095
M7	LS	0.092	0.081	0.072
	M	0.099	0.142	0.122
	RS	0.129	0.137	0.15
Nose	LS	0.253	0.25	0.19
	M	0.19	0.214	0.19
	RS	0.2	0.215	0.19
O1	LS	0.25	0.283	0.266
	M	0.27	0.289	0.253
	RS	0.284	0.292	0.259
O2	LS	0.37	0.319	0.367
	M	0.322	0.356	0.322
	RS	0.299	0.374	0.302
O3	LS	0.383	0.355	0.372
	M	0.328	0.339	0.332
	RS	0.259	0.253	0.276
O4	LS	0.365	0.342	0.367
	M	0.336	0.325	0.343
	RS	0.282	0.277	0.299

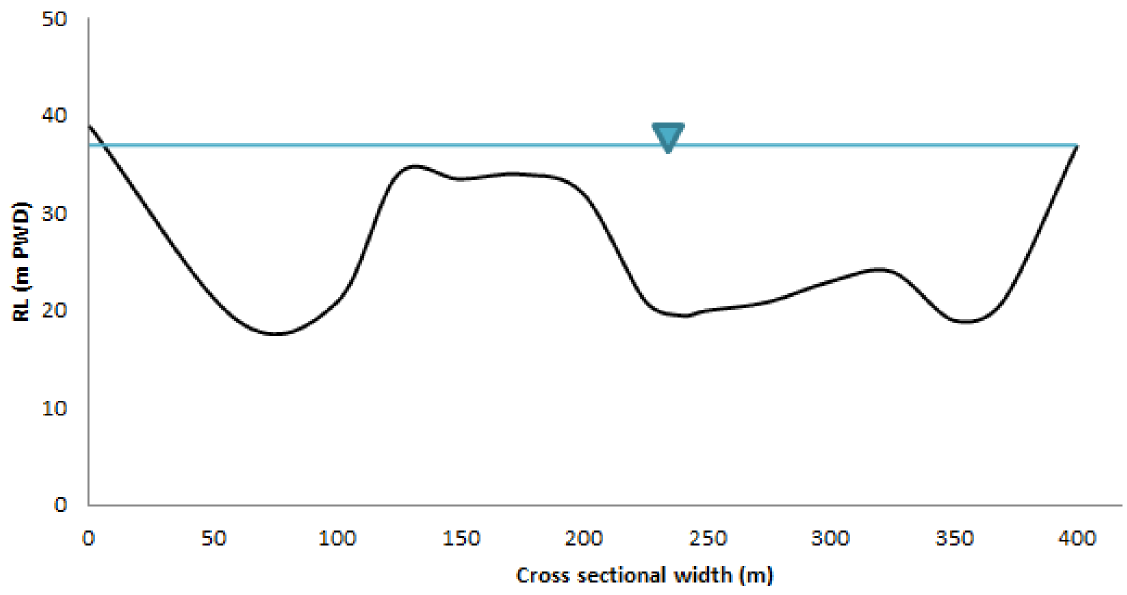

Figure A1: Schematic cross-section of Korotoya River (SW 142) (BWDB 2011)

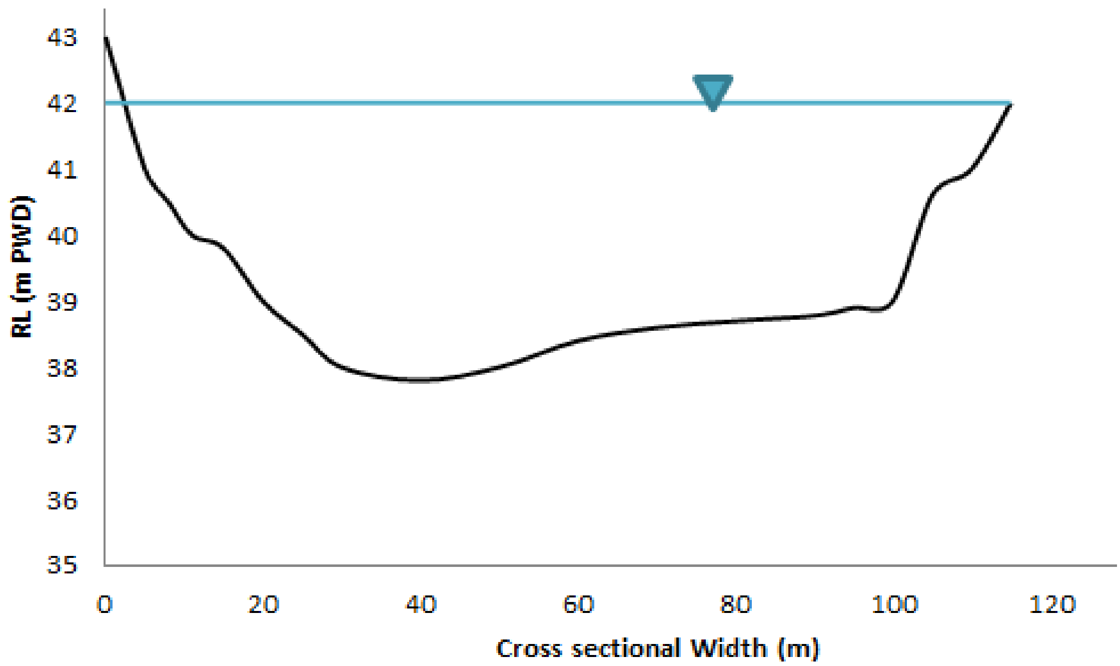

Figure A2: Schematic cross-section of Dhepa River (SW 78) (BWDB 2011)

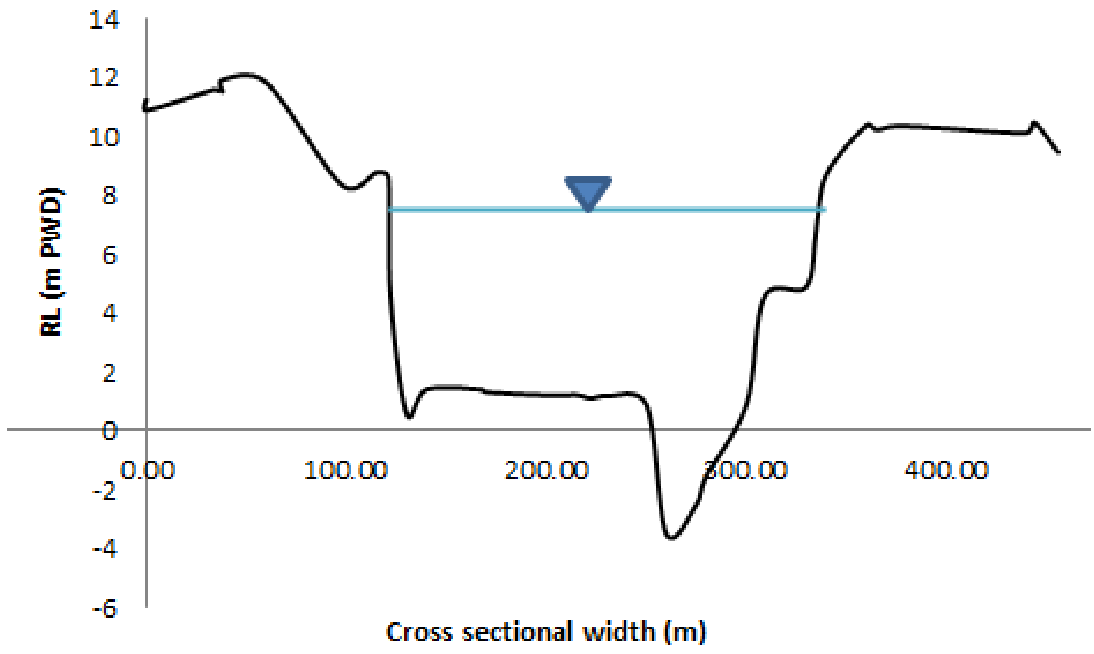

Figure A3: Schematic cross-section of Shurma River (SW 268) (BWDB 2011)

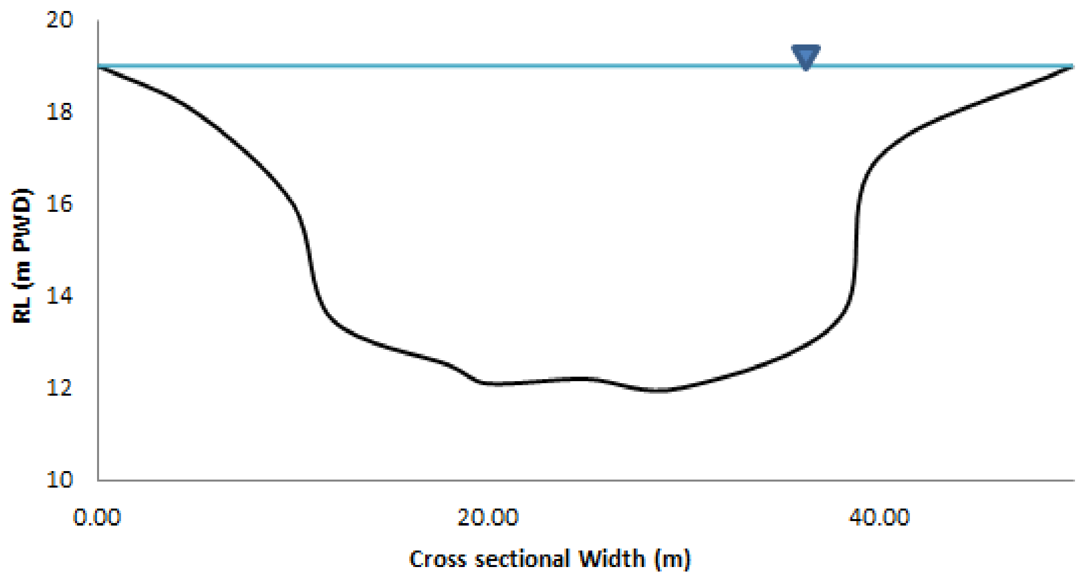

Figure A4: Schematic cross-section of Botor Khal (offtake of Shurma River, SW 33) (BWDB 2011)

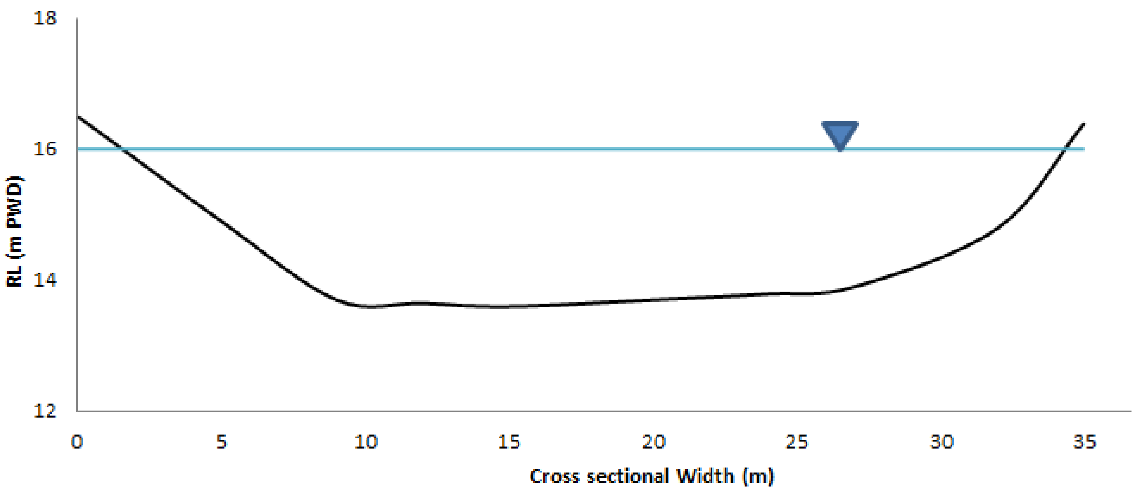

Figure A5: Schematic cross-section of Jhenai offtake (SW 34B) (BWDB 2011)

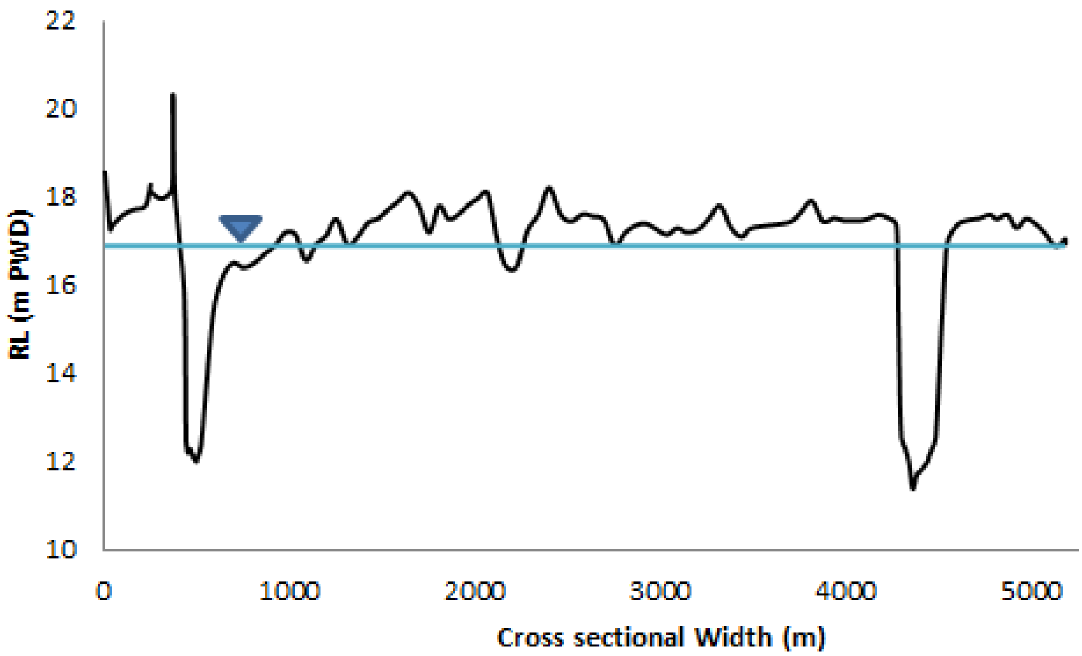

Figure A6: Schematic cross-section of Old Brahmaputra (RM OB5) (BWDB 2011)

Appendix B

Layout and Drawings

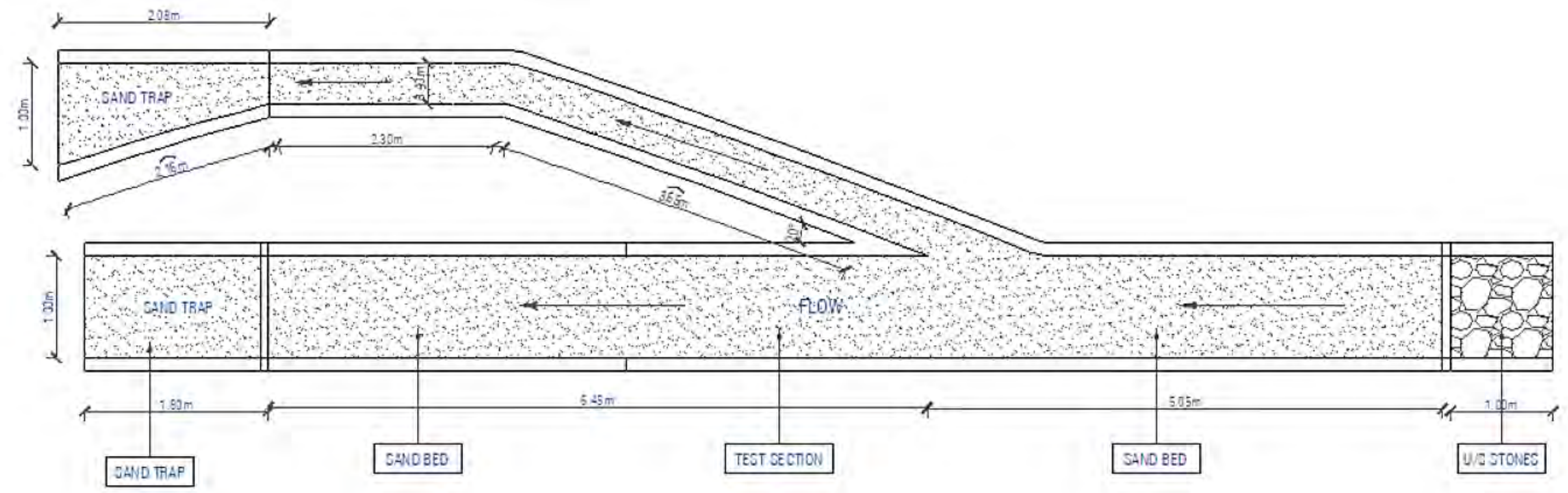

Figure B1: Layout of experimental channel for 20^0 offtake angle

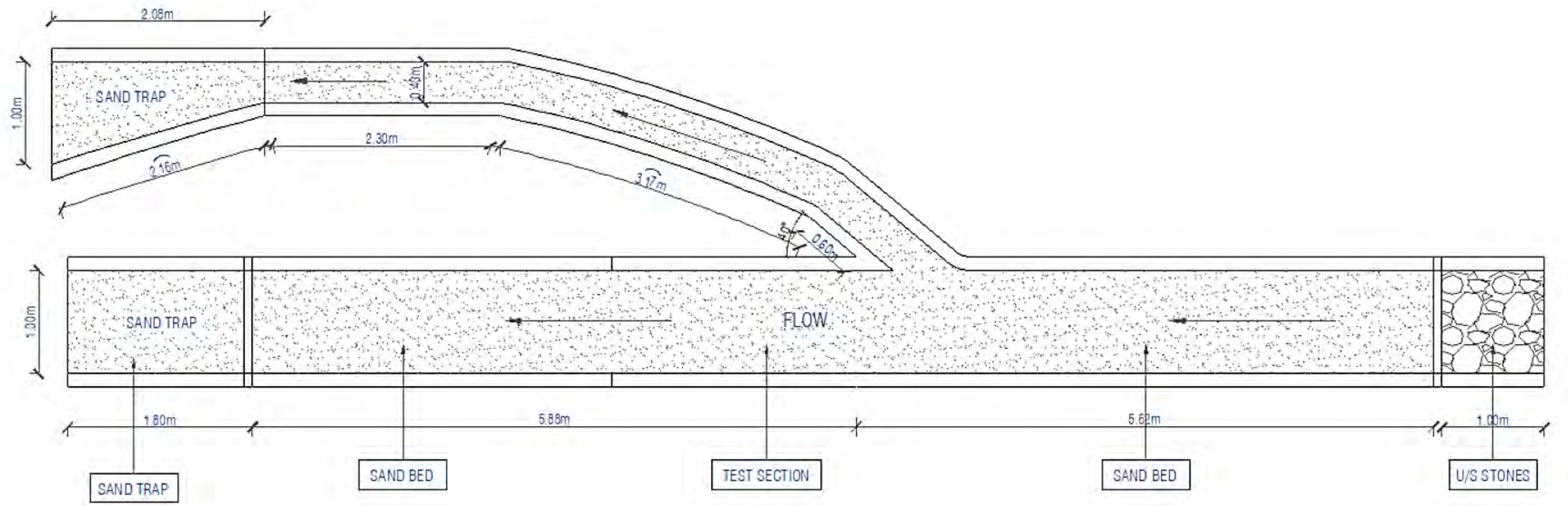

Figure B2: Layout of experimental channel for 40^0 offtake angle

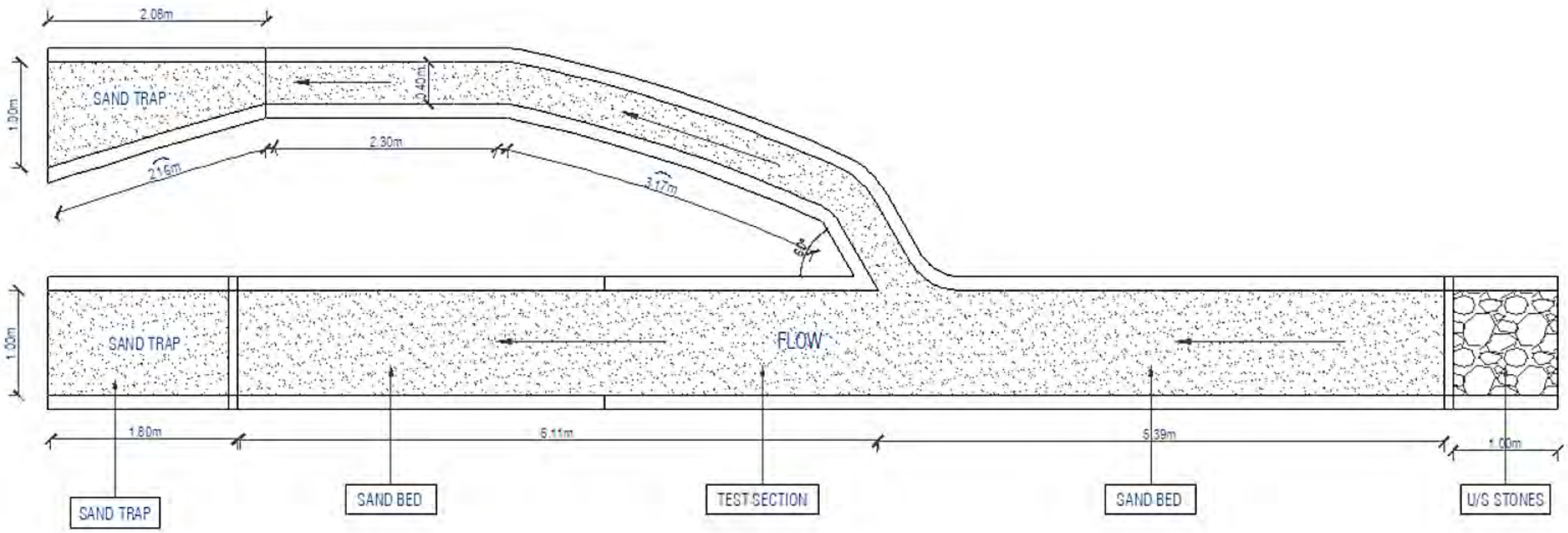

Figure B3: Layout of experimental channel for 60^0 offtake angle

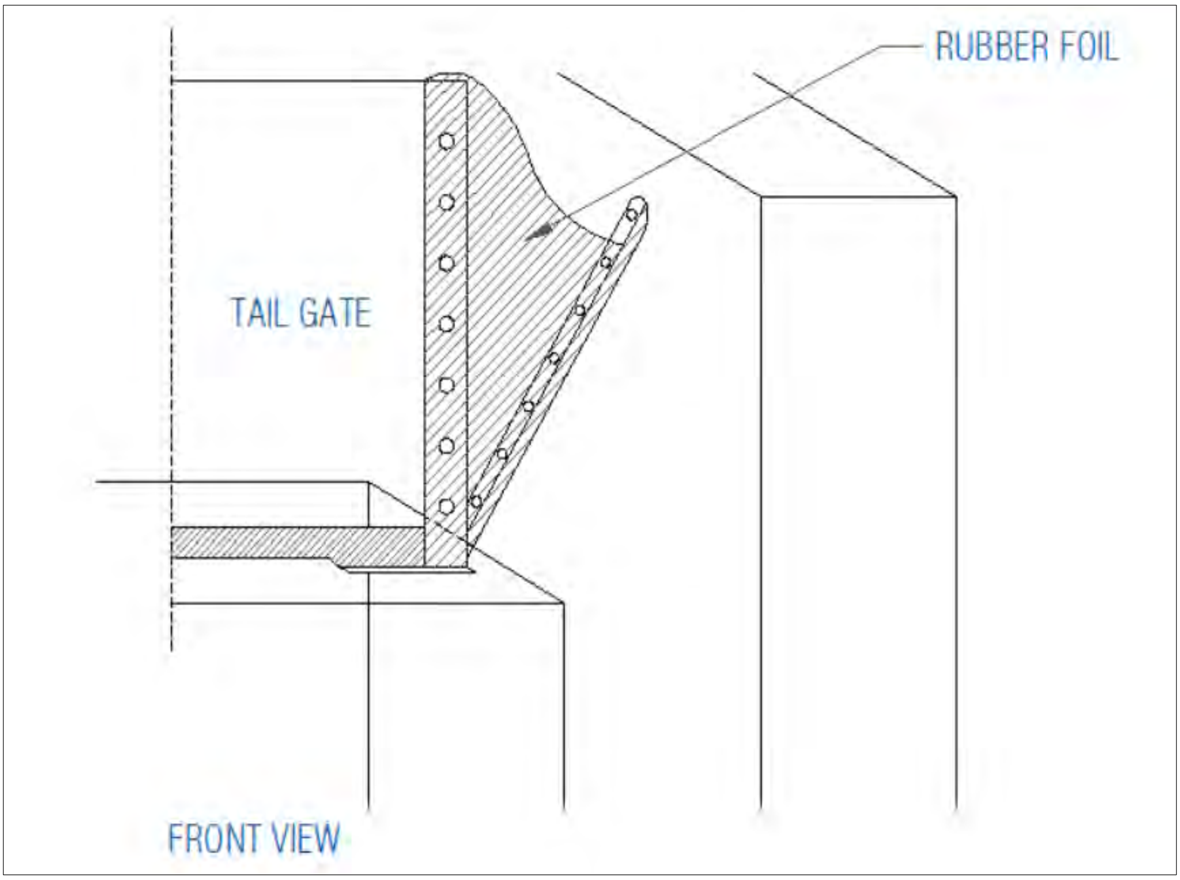

Figure B4: Schematic drawing of tail gate

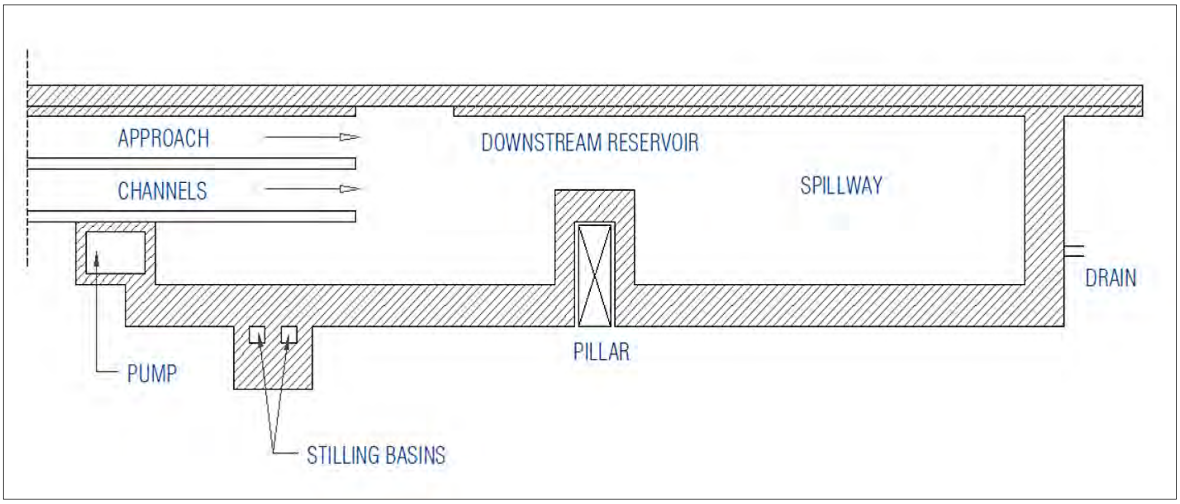

Figure B5: Schematic layout of downstream reservoir

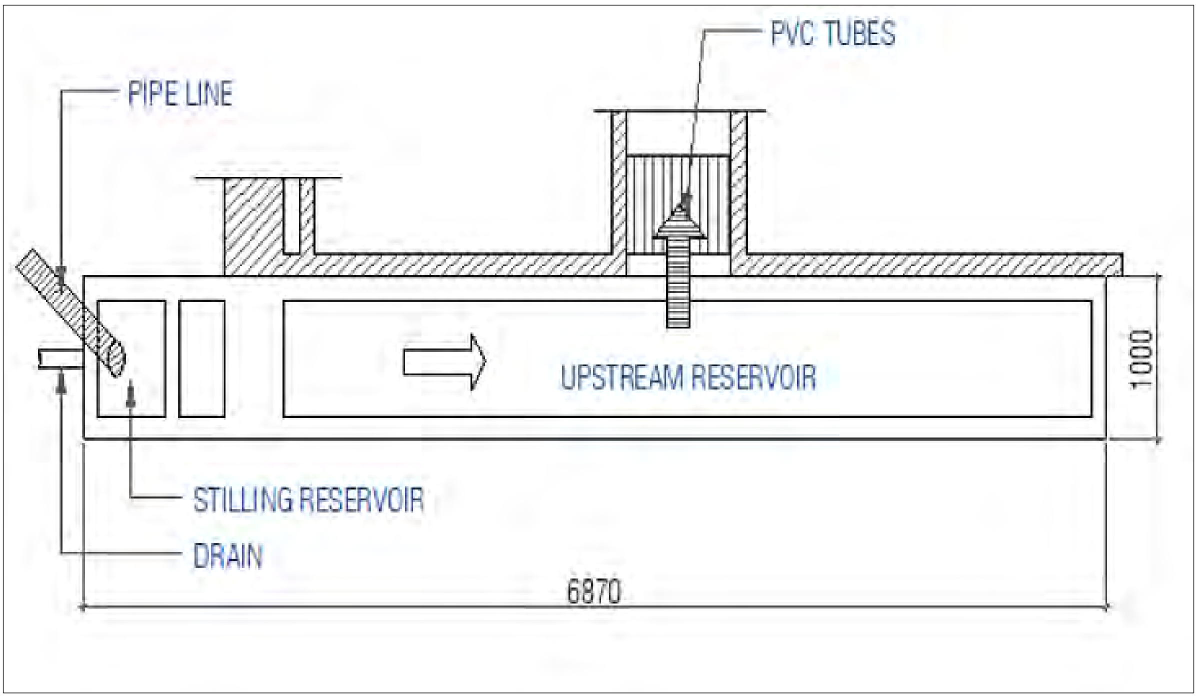

Figure B6: Schematic layout of upstream reservoir

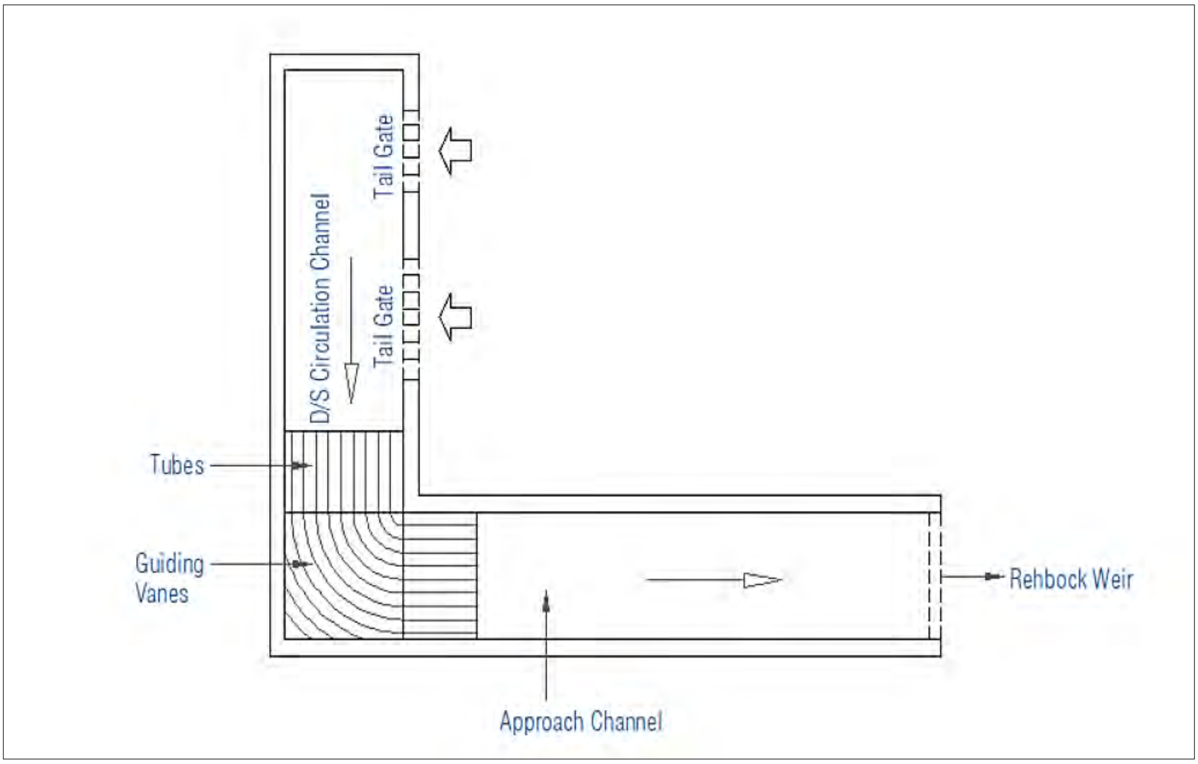

Figure B7: Detail of regulating system

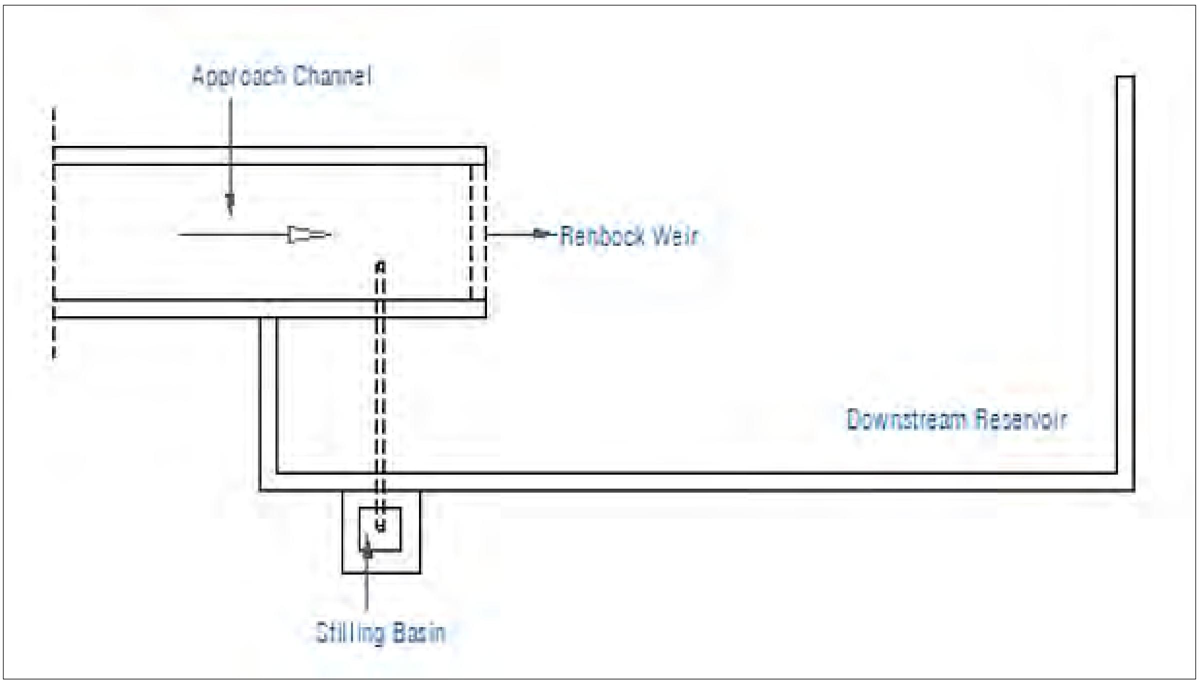

Figure B8: Approach channel and Rehbock weir

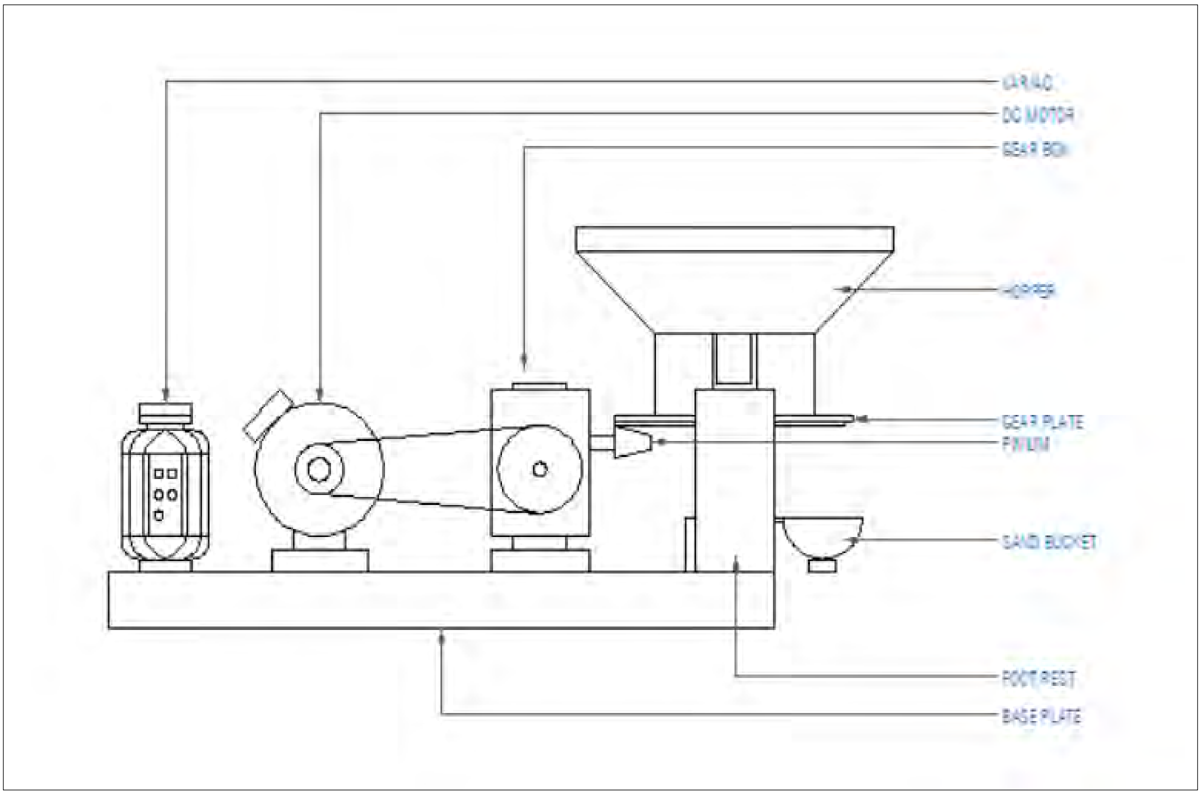

Figure B9: Diagrammatic view of sand feeder