Field-Based Evaluation of Local Scour in Complex Piers of a Bridge on the Dhaleswari River

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Dedication

My parents, without them I am nobody

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Abstract

Bangladesh is a land of rivers. Bridge, being an essential hydraulic structure for road and rail communications of the country, the Government of Bangladesh have already constructed a large number of bridges through its concern engineering departments. Many bridges would also need to be constructed in future. But this very essential and expensive bridge for road and rail communications could be at risk of failure due to inadequate consideration of local pier scour. Over-prediction of scour also has cost implication. After the historical flood of 1998, Bangladesh experienced a bridge-failure incident near Dhaka due to local scour. After the cyclone Sidr in 2007, a part of a bridge was collapsed in Patuakhali District. So, the bridge planners, designers and implementers require prediction of local scour as closely as possible. Practicing engineers in Bangladesh have chosen a wide range of empirical formulae for predicting local scour. Most of these formulae are experimental in nature. It is not yet known how good they are in the context of our alluvial river and seasonal hydrology. In this study, performance of some selected empirical formulae used in Bangladesh for scour estimation is assessed with respect to the field data under simple pier and complex pier considerations. The study also evaluates the suitability of the HEC-RAS model for scour prediction in the context of Bangladesh. For this purpose, local scour, water level, velocity, etc. were monitored throughout the pre-monsoon, monsoon and post-monsoon seasons of 2017 at a newly constructed bridge over the Dhaleswari River in Manikganj District. A total of five primary data set was collected. Other relevant secondary data required in the study was collected from relevant government and private organizations. The findings of the study reveal that the discharge in the river varied from 250 m^3/s on 20 May to 2267 m^3/s on 19 August to 772 m^3/s on 16 October, 2017. The 20-year flood discharge was 2040 m^3/s and the estimated peak discharge was about 2267 m^3/s. Thus, the 2017 flood was close to the design discharge condition of the bridge. The observed scour levels varied from -3.12 m PWD on 20 May to -4.98 m PWD on 19 August to -2.81 m PWD on 16 October, 2017. The maximum scour occurred at Pier No. 4 from the left bank of the river on 19 August during the peak flood. The maximum predicted scour level was -8.94 m PWD based on simple pier formulation using Lacey's equation. The maximum predicted scour level was -12.70 m PWD based on complex pier formulation using Jain and Fischer's equation. All the selected empirical equations as well as the HEC-RAS model gave higher values of pier scours than those measured in the field. The discrepancies between the measured and estimated values were more if complex pier formulation was used. The modified Lacey's equation estimated local scours better performed for simple pier consideration and over-estimated for complex pier consideration compared to original Lacey's equation. Estimated values of local scour followed the field observed trend of temporal changes for simple pier whereas estimated values had different trend while considering complex pier. Estimated values of local scour by using Melville's equation and considering simple pier provided the same value for all the data set for being considered as a narrow pier. On the other hand, the Melville's equation followed the pattern of field observed local scour except for peak flow when the pier was considered as a narrow pier. Among the equations, the FHWA method predicted the scour values (e.g., -5.03 m PWD on 19 August for simple pier) which were closer to the field observed values for both simple and complex pier considerations. The HEC-RAS model gave the closest scours (e.g., -6.94 m PWD on 19 August) in complex pier consideration. This indicates that the HEC-RAS model and the FHWA method are suitable for local scour prediction for alluvial rivers in Bangladesh and should be duly considered in pier scour estimation.

Table of Contents

List of Tables

List of Figures

List of Photos

List of Abbreviations

IWFM	Institute of Water and Flood Management
IWM	Institute of Water Modeling
BUET	Bangladesh University of Engineering and Technology
FHWA	Federal Highway Administration
RHD	Roads and Highways Department
HEC-RAS	Hydrologic Engineering Centers River Analysis System
BWDB	Bangladesh Water Development Board
MDWL	Mean Daily Water Level
WL	Water Level
EGL	Existing Ground Level
PWD	Public Works Department
LGED	Local Government Engineering Department
BM	Bench Mark
TBM	Temporary Bench Mark
FDOT	Florida Department of Transportation
GC	Growth Center
HFL	High Flood Level
SHWL	Standard High Water Level

List of Symbols

ρ	Fluid density
v	Kinematic viscosity
T	Temperature
l	Pier length
b_p	Pier width
D	Channel width
c	Cohesiveness of the sediment
ψ	Shape factor
α	Angle of repose
w	Fall velocity
S_0	Energy slope
g	Acceleration of gravity
V_c	Threshold velocity
V_a	Armour peak velocity
y	Flow Depth
d_{50}	Median grain size
b_e	Effective pier width
b^*	Pile cap width
b_{pc}	Effective diameter of the pile cap
b_{pg}	Effective diameter of the pile group
h	Hydraulic depth
Y	Distance in between initial bed level and top of the pile cap
d_s	Scour depth
K_s	Pier shape factor
K_1	Correction factor for pier nose shape
K_2	Correction factor for angle of attack
K_3	Correction factor for bed condition
K_4	Correction factor for armouring of bed material size
F_r	Froude number
V_1	Mean velocity of flow

σ_G	Sediment non-uniformity
V_{icD_X}	The approach flow velocity required to initiate scour at the pier for the grain size D_X
V_{cD_X}	The critical velocity for incipient motion for the grain size D_X
V_0	Approach flow velocity
V_{cD_X}	The critical velocity for incipient motion for the grain size D_X
D_x	Grain size for which x percent of the bed material is finer
K_u	Factor associated with the approach flow velocity required to initiate scour at the pier
Q	Discharge
f	Silt factor
M	M-factors for different river system

CHAPTER ONE

INTRODUCTION

1.5 Background and Present State of the Problem

Bangladesh is a land of rivers. The country is crisscrossed by an enormous number of rivers, most of them are alluvial in nature. Bangladesh, one of the biggest deltas in the world, is mainly composed of the sediments carried by the three major river systems, i.e., the Ganges-Padma, the Brahmaputra-Jamuna and the Meghna. These river systems from their huge catchment areas carry an immense amount of sediment to the sea.

Alluvial rivers carry the sediment of the same character as underlying in their beds. An alluvial river has a mobile bed, and is composed of non-cohesive granular materials. Such materials are generally unconsolidated silts and sands. The banks of such rivers are usually composed of clay, silt and/or sand. These rivers are self-formed; shaped by the magnitude and frequency of the floods that they experience; and the floods have the ability to erode, deposit and transport sediment. Many natural water courses and majority of the man-made canals are examples of alluvial channels.

The water and sediment discharges in natural alluvial streams, which have evolved over geologic times, are in equilibrium and produce no objectionable scour or deposition. Such alluvial rivers or natural streams reach a state of dynamic equilibrium when the governing factors such as water and sediment discharges, channel geometry, slope, water and sediment properties remain unchanged for a long enough period of time. The bed of such streams remains stable and is free of aggradation and degradation which means no scouring. However, if this delicate balance is disturbed by changing any of these properties through natural or man-made factors, like construction of bridges, weirs, barrages, dams, etc., a process of achieving a new equilibrium begins. The process is inevitably accompanied by aggradation and/or degradation along the river bed, which is called the long-term general scour.

Many of our road and rail communication systems require river crossings with bridges over them. These bridges are very expensive, but essential, hydraulic structures for a low/middle income and riverine country like ours. Local scour, which is defined as the change in elevation of the stream bed resulting from the erosive action of the flowing water over the mobile bed, is a consequence of sediment continuity and is generally aggravated by the presence of obstructions, such as waterway constrictions, piers, spurs, dikes, etc. Knowledge of maximum scour depth under various conditions, especially around bridge piers for different types of bed materials of the stream, is essential for the design of bridge foundations as well as for the proper maintenance of the bridge after its construction. The probable depth of scour mainly dictates the depth below which the bridge pier foundation must be taken. It is one of the major concerns in the hydraulic design of a bridge. Inadequate scour estimate can cause bridge failure, and hence estimation of probable scour depth is essential from the safety point of view of a hydraulic structure. A more precise scour depth estimation is required for an economical design as well. An under-estimation of scour depth may result in bridge failure, while its over-estimation may increase the construction cost. Thus, scour needs to be predicted as precisely as possible.

Many empirical formulae are used for local scour prediction, which are originated mostly based on laboratory data. They were derived based on experimental studies considering simple pier. However, among the wide variety of bridge piers available, complex piers are now widely used in our country. A complex bridge pier structure is composed of three components: Pier Column, Pile Cap and Pile Group (Figure 1.1). For complex pier scour estimation, the widely used empirical formulae which are based on simple pier consideration are used with an equivalent/effective pier diameter. However, it is not known how good such consideration is. Moreover, simple pier formulae are still in use in complex pier situation. So, there is an emerging need to evaluate the formulae, which are widely used in our country for local scour prediction, in relation to complex piers with field data. This study was undertaken to monitor the local scour at the Dhaleshwari bridge built with complex piers and to assess the suitability of different scour prediction formulae.

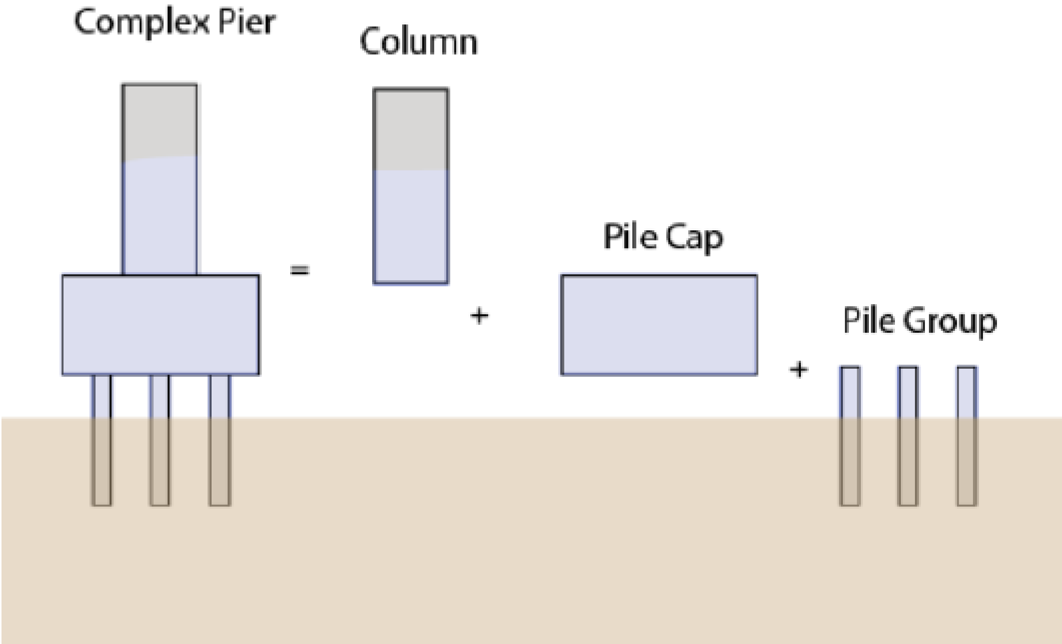

Figure 1.1: Components of a complex pier (Jones and Sheppard, 2004).

1.6 **Objectives of the Study**

This study was undertaken with the following specific objectives:

i. To measure the temporal variation of bridge pier scours;

ii. To estimate bridge pier scours by using different empirical formulae; and

iii. To compare the estimated scours with the measured scours to assess the performance of the empirical formulae in complex pier scour estimation.

1.7 **Importance of the Study**

Scour is the local lowering of stream bed around a hydraulic structure. Scour takes place around bridge piers, abutments, guide bunds, etc., due to the modification of flow pattern causing an increase in local shear stress which, in turn, leads to removal of material and hence scour. In the U.S.A. alone, over 500 bridges had failed since 1950 due to scour of foundation material (Huber 1991). A number of bridges in Bangladesh had also collapsed due to scouring. For example, the Turag-Bhakurta Bridge over the river Turag failed during the 1998 flood due to the inadequate estimation of design discharge and hence local scour (Bala *et al.* 2005). A part of a bridge in Kalapara upazila of Patuakhali district collapsed after the cyclone Sidr on 15 November, 2007 (Choudhury *et al.* 2015). Some piers of the Meghna Bridge have recently become vulnerable due to river bed erosion and scour (Hoque *et al.* 2015).

There are a large number of formulae for scour estimation. Most of these formulae were developed for simple piers. But the bridges now-a-days constructed have complex piers with pier stems, pile groups and pile caps. So, it is not yet known which formula or set of formulae would give better scour estimation for such piers in Bangladesh condition. Moreover, different studies use different formulae in estimation of local scour. For example, Sutradhar recommended Lacey's equation (Sutradhar, 1995); the Institute of Water and Flood Management (IWFM) uses Laursen, Breusers, Neill, Jain and Fischer, Chitale, and Melville formulae (IWFM, 2013); and the Institute of Water Modeling (IWM) estimates local scour by using FHWA, Laursen, and Melville equations (IWM, 2012). The bridge design manual prepared by BUET and IWM (2008) has suggested Lacey, Laursen, Melville and FHWA equations (BUET and IWM, 2008). The Roads and Highways Department (RHD) bridge designers' handbook has suggested the latter three formulae (RHD, 1999). There are many more formulae available which are not used in the above studies. For example, a different set of formulae are used in scour estimation in the HEC-RAS model. Therefore, it is necessary to evaluate the performance of the different formulae in predicting local scour in a complex pier in typical alluvial river condition of Bangladesh. The findings of the study will help in better understanding of pier scour in typical hydro-morphological context of Bangladesh and will be useful in bridge design and implementation.

1.8 **Organization of the Thesis**

This thesis contains six chapters. Chapter wise outlines of the contents are as follows:

The first chapter of the thesis gives a brief background of the study in the context of the river system of Bangladesh and the requirements of bridges for communication. A typical definition sketch of complex pier geometry is also given to ease the problem statement. It emphasizes the need for field based evaluation of scour formulae from the economical and safety perspectives of bridges. This chapter also provides specific objectives and importance of the present study.

The second chapter is on literature review. It includes the types of scour and subsequently bridge pier scour, mechanism of local scour, effect of different parameters on bridge pier scour, brief description of the formulae and model in use, and a short account of the previous studies on local scour both in Bangladesh and overseas.

Chapter three presents a brief history of the river over which the bridge was constructed; and details of the instruments used, the gage setup and data collected from primary and secondary sources. Development process of a stage-discharge relationship and complex pier scour estimation are also given in the last part of this chapter.

Analysis and interpretation of the scour estimates from the empirical formulae and the mathematical model and their further application to the bridge interventions are incorporated in chapter four. The results include the comparison of field observed values with formulae estimated and model simulated values, tempo-spatial variation of local scour and assessment of the suitability of the formula(e) and/or model.

Finally, the study ends in chapter five by drawing some conclusions based on the findings of the study and providing some recommendations for future study.

CHAPTER TWO

REVIEW OF LITERATURE ON SCOUR

2.6 Introduction

Local scour around any obstruction to the flow (bridge pier, for example) is the lowering of a portion by erosion of the channel bed below an assumed natural level or other appropriate datum, tending to expose or undermine foundations that would otherwise remain buried (Kabir, 1984). Generally, there are three major different ways of scour in an alluvial river bed. They are:

a) Changes in river regime resulting progressive degradation of the river bed. This process may be enhanced by channel improvements and constructions of upstream dams and reservoirs,
b) Temporary scour associated with the periodic rise in river stage during flood, or with the shifting of the thalweg of the stream, and
c) Local scour beyond the natural bed level, caused by an obstruction positioned in the stream.

The third type of scour, i.e. local scour, is the subject of this study.

Many studies have been conducted on local scouring, most of them are experimental in nature. In this study, a field based approach was selected for evaluating the applicability of different empirical formulae with the field measured data. For this purpose, proper understanding of scouring process was essential and review of related researches around the world was necessary.

This chapter presents an overview of relevant theory, scouring process, mechanism, etc. on local scour around bridge pier.

2.7 Mechanism of Local Scour

If any obstruction is placed across the flow, then there will be a great alteration in the flow pattern. This will happen due to the concentration of streamlines around it. The dominant feature of this flow, which develops near the obstruction, is the large-scale eddy structure or the system of vortices. These vortex systems are the basic mechanism of the development of local scour and form a fundamental part of the flow structure (Kabir, 1984). The vortices are formed at the base of the pier and down-flow occurs at the upstream face of the pier. At the face of the pier, there is stagnant flow which loses the acceleration while moving towards the pier. The pressure increases at the pier face as the approach flow velocity reduces itself to zero at the upstream side. The associated pressures are the highest near the surface, where the deceleration is the greatest, and decrease downwards. With the decrease in the velocity from surface to bed, the pressure accordingly decreases resulting in the formation of downward pressure gradient. The pressure gradient forces the flow down the face of the pier, resembling of a vertical jet. The flow resulting due to the pressure gradient impinges the streambed and creates a cavity/hole in the proximity of pier base. This flow impinging on the bed is the main scouring agent (Kothyari, 2007).

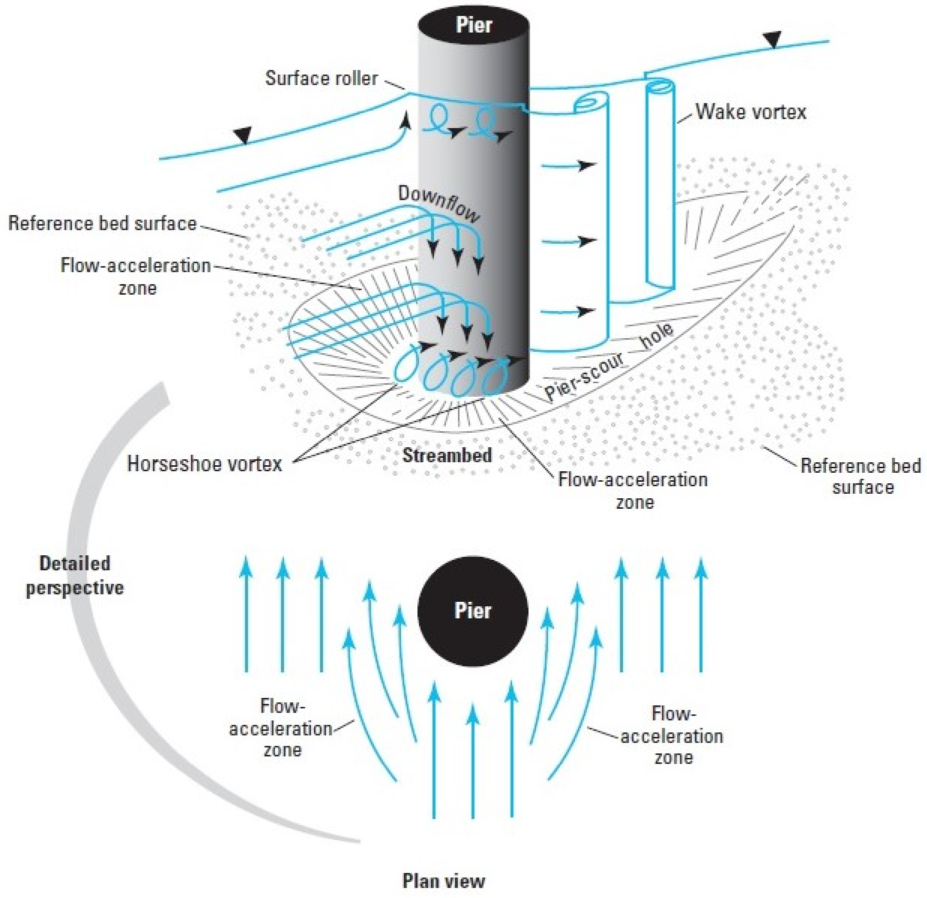

Figure 2.1: Flow and scour pattern at a circular pier (Kothyari, 2007).

Figure 2.1 shows the scour pattern at a circular pier under the action of currents. In the figure, the vortex motion induced by the existence of the pier carries along the bed sediments within the vicinity of the base of pier. The flow rolling up continues to create a hole and, due to interaction with the approaching flow, it develops into a complex vortex system. The vortex then extends itself downstream along the sides of the pier base. This vortex is mentioned as horseshoe vortex because of its great likeness to a horseshoe. The horseshoe vortex is effective in transporting the isolated particles away from the pier. In context with the development of the vortex, the scour depth increases and the strength of the horseshoe vortex tends to diminish, which leads to a reduction in the rate of sediment transport from the base of the pier. Wake vortices are the vertical vortices which are also formed in the vicinity of the pier base besides the horseshoe vortex, the reason for their development is the separation of flow at the sides of the pier. Although both the horseshoe and wake vortices erode the material from the base region of the pier, the intensity of the wake vortices reduces with the increase in the downstream distance of the pier, as a result it is seen that for a long pier immediately downstream there is a deposition of material (Jaiswal, 2016).

2.8 Types of Scour Occurring Around a Bridge

Two major types of scour can occur around bridges which make the total scour. They are: a) general scour and b) local scour.

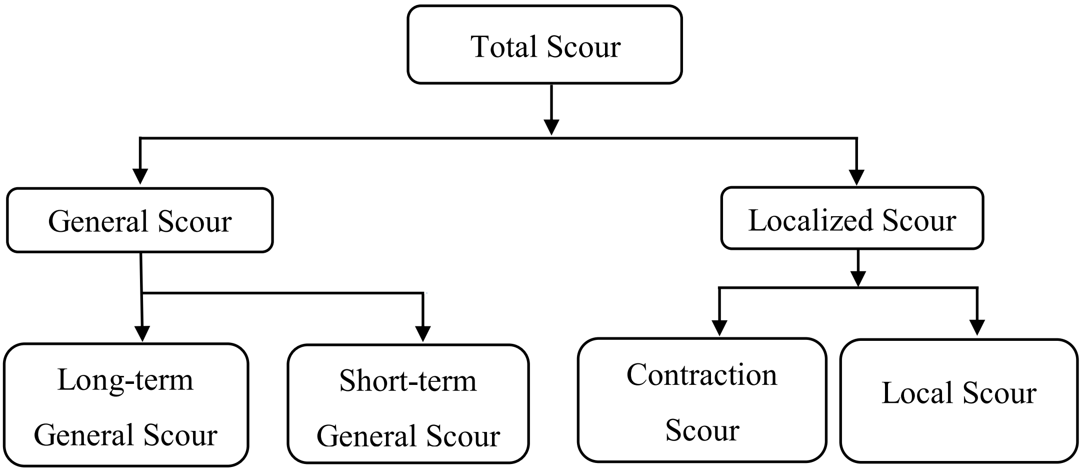

Figure 2.2: Types of scour associated with a bridge structure in a dynamic river (Melville and Coleman, 2000)

General scour occurs in two different ways: long-term general scour which includes progressive aggradation or degradation, channel widening and meander migration; and short-term general scour which refers to confluence scour, channel thalweg shifting and bed-form migration. Scouring can be either one or a combination of the distinct types of scour processes (Figure 2.2). However, the scour components outlined in the figure are generally applicable for the bridge structures constructed over a very dynamic river.

2.9 Factors Affecting Bridge Pier Scour

Local scour around bridge piers varies with many parameters, mostly inter-related to one another. The influence of one particular parameter on local scour depth may be obscured by that of others. In order to understand the complexity evolving with the inter-dependency of parameters, a systematic approach is required. In this regard, the variables affecting local scour depth is being categorized into five sub-groups of inter-dependent factors (Chiew, 1984):

- Fluid properties (density, ρ; kinematic viscosity, ν). Both of the parameters depend on temperature, T. In the cases of laboratory, temperature can be controlled and kept constant for most of the cases. On the other hand, it is almost impossible to maintain a controlled temperature on field. As a result, no data was found to ascertain the extent of the influence of temperature on the scour depth.

- Time is required to establish an equilibrium scour depth as scouring is a temporal process. Establishment of maximum scour depth depends on the duration of the flood.

- Pier size and others associated factors. The size (most of the cases width) of pier plays a vital role which affects the local scour. There are others factors like pier shape, angle of attack of the flow with respect to the pier axis, the ratio of the channel width to the pier width (aspect ratio, b_p/D), etc.

- Sediment properties (especially specific gravity and particle size). Others associated factors are: cohesiveness of the sediment (c), Shape factor (ψ), angle of repose (α) and fall velocity (w).

- Flow properties: water depth (h), energy slope (S_0) and the gravitational acceleration (g).

Later on, Melville (2008) published a paper showing the relation between the local scour depth at a bridge pier and its dependent parameters. This paper emphasizes on the underlying physics of the local scour processes. The outcome is restricted for unsubmerged bridges in straight channels (alluvial in types) with homogenous bed material. Tidal flows and waves were not incorporated in this study.

2.4.1 Effect of flow intensity

Under clear-water conditions, the local scour depth increases almost linearly with velocity to a maximum at the threshold velocity for uniform sediment and termed as the threshold peak. After exceeding the threshold velocity (V_c), the value of local scour decreases first and then the local scour starts to increase again to reach a second peak termed as live-bed peak. The live-bed peak provides a lesser value than the threshold peak (Figure 2.3). For the cases of sediment non-uniformity, the first peak is termed as armour peak (V_a), whereas the second peak is termed as live-bed peak. The live-bed peak requires the transition flatbed condition (all sediments particles are in motion).

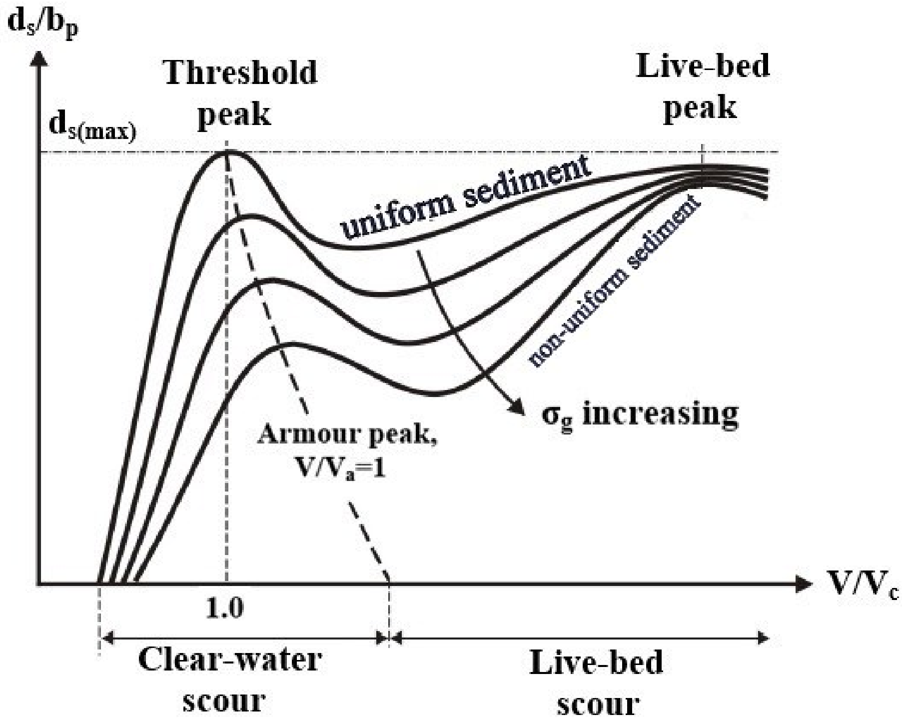

Figure 2.3: Local scour depth variation with flow intensity (Melville, 2008)

2.4.2 Effect of flow shallowness

Flow shallowness is the ratio of the depth of flow to the pier width ($\frac{y}{b_p}$). For narrow piers, scour depth increases proportionally with pier width and the flow depth has no affect upon local scour. For wide piers, scour depth increases proportionally with flow depth and the pier width has no affect upon local scour. For intermediate lengths (medium pier width and medium depth of flows), the scour depth depends on both pier width and depth of flow (Melville, 2008). These effects are represented schematically in Figure 2.4.

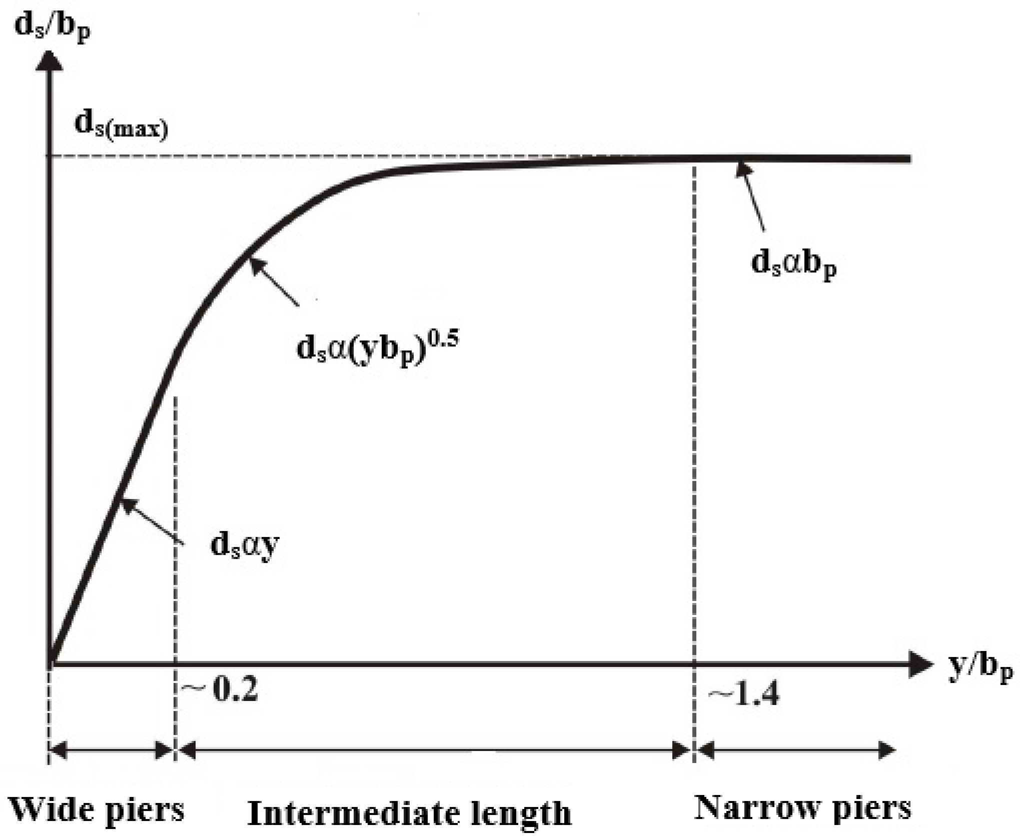

Figure 2.4: Local scour depth variation with flow shallowness (Melville, 2008)

2.4.3 Effect of sediment coarseness

Sediment coarseness refers to the ratio of the pier width (b_p) to the median grain size (d_{50}). For sediment uniformity, local scour depths remain unaffected by sediment coarseness except for the cases of relatively high sediment size. Ettema (1980) explained that for smaller value of sediment coarseness ($b/d_{50}<50$), the hole created by the down flow and hence the erosion are impeded because of the relatively larger value of individual grain sizes because the porous bed dissipates some of the downwards energy. When the particle size becomes so large, i.e. $b/d_{50}<8$, scour is caused mainly due to erosion and hence the scour is further reduced. This is because of the relatively larger individual grain sizes which is difficult to be carried over by the flow. Sheppard et al. (2004) stated that scour depth will reduce significantly for larger values of b/d_{50} for sandy materials (Figure 2.5).

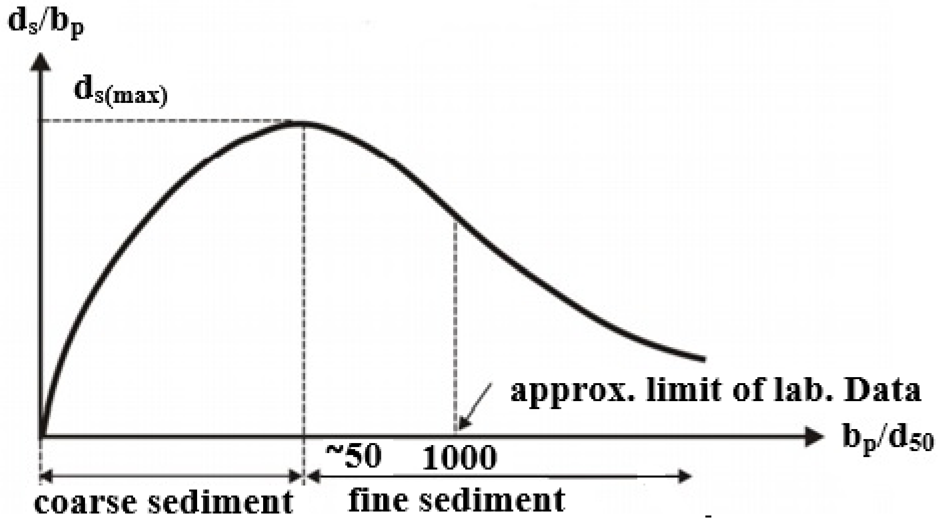

Figure 2.5: Local scour depth variation with sediment coarseness (Melville, 2008)

2.4.4 Effect of time

Live-bed equilibrium is reached more quickly than that of clear-water conditions (Chiew, 1984). Subsequently, fluctuations of the scour depth occur due to the passage of bed features past the pier (Melville, 2008). The temporal variation and the maximum depth of scour at bridge piers mainly depend on the characteristics of flow, pier and river-bed material and the development of equilibrium scour depth requires a long time of more than 3 to 4 days (Kothyari, 2007). The rate of local scour and the equilibrium local scour depth are different for clear-water and live-bed conditions (Ettema et al., 2011).

2.4.5 Effect of pier shape

Funde et al. (2018) carried out an experimental study by using three different pier shapes (elliptical, diamond and circular) and found that the elliptical shape produces the lowest amount of scour depth. Many researches are being conducted for pier shape (multiplying) factors around the world considering both simple piers and complex piers. Shape factors have to be considered for axial flow only because even a small angle of attack will eliminate the benefit of a streamlined shape (Melville, 2008). For bridge pier, any one condition from the four cases may occur for local scour (Figure 2.6).

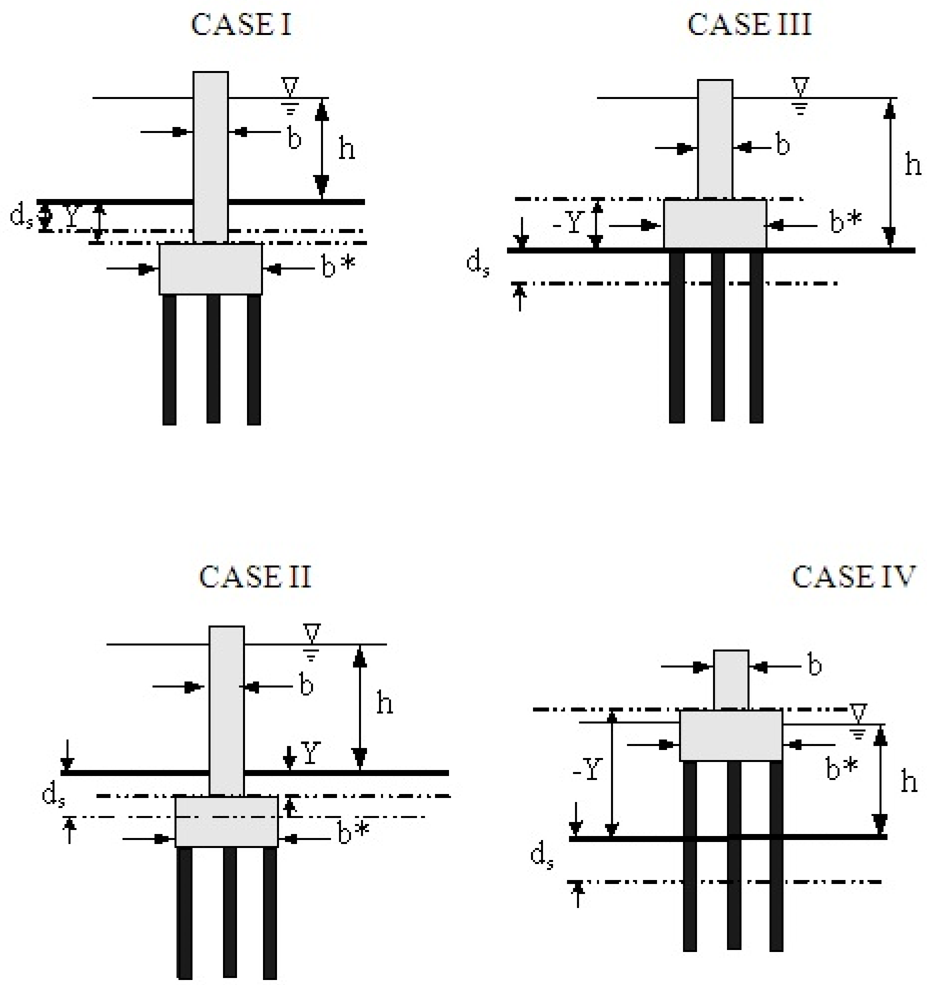

Figure 2.6: Potential local scouring with non-uniform pier shapes (IWFM, 2013)

For case I, local scour is unaffected by the presence of the pile cap and pile group because they remained buried below the base of the scour hole. For this situation, local scour can be estimated by considering pier diameter only (Melville and Coleman, 2000). For case II, where the pile cap level and the initial bed level is the same, the local scour depth is typically reduced from that of a simple pier due to interception of the downflow. For case III, the pile cap is exposed to flow (above the initial bed level) and the local scour depth can be increased or decreased compared to that at a simple pier. For case IV, pile cap top remains at or above the water surface level and provides the maximum level of local scour. For cases III and IV, pile cap and pile group are exposed to the flow (Melville, 2008). For the non-uniformity of pier shape for cases III and IV, the pier is considered as complex pier and an effective pier diameter is used in place of simple (uniform) pier diameter. The effective pier diameter can be estimated by using the following equation (Melville and Coleman, 2000):

$$b_e = b_p\left(\frac{h+Y}{h+b^*}\right) + b^*\left(\frac{b^*-Y}{b^*+h}\right) \tag{2.1}$$

where,

b_e = Effective pier width

b_p = Pier width

b^* = Pile cap width

h= Hydraulic depth of flow

Y = Distance between the initial bed level and the top of the pile cap

2.4.6 Effect of pier alignment

The depth of local scour for all shapes of pier is strongly dependent on the alignment to the flow. The depth of scour usually increases with the increase in angle of attack because increases in the angle of attack increase in the frontal pier width. However, for the cases of circular pier, there will be no affect because of no change in frontal pier width (Melville, 2008).

The maximum scour depth is closely related to the dimension of the pile group as a whole as seen from the upstream (for the case of exposed pile group where the pile cap is clear of the water surface) (Hannah, 1978). It is recommended to use a single line of piles for an angles of attack greater than 8 degrees.

2.10 Different Empirical Equations for Local Scour Estimation

Many empirical equations have been developed by different researchers across the globe using experimental observations and field data to estimate the value of scour depth around bridge piers. The suitability of using the formulae developed in laboratory experiment are not well known yet, especially under field condition for alluvial rivers of Bangladesh. During the literature review it was found that different institutions (IWM and IWFM, BUET) use different formulae for hydraulic design of bridges. Available manuals for the hydraulic design of bridges targeting bridge designers and practitioners in Bangladesh were also reviewed. It is found in reality that the manual was not followed even by the manual development authority. For example, BUET and IWM developed a manual on hydrologic and hydraulic designs of roads and bridges in 2008. But, the review of the study on a bridge conducted by IWM (2013) revealed that the organization itself is not following its suggested list of formulae. Moreover, different studies have used different empirical formulae (Table 2.1) which further suggests the need for a field-based study.

Table 2.1: List of empirical formulae used in different reports or manuals for estimation of local scour

IWFM	**IWM**	**BUET-IWM**	**RHD**
Breusers	FHWA	FHWA	Breusers
Laursen	Laursen	Laursen	Laursen
Melville	Melville	Melville	Melville
Jain and Fischer	-	Lacey	-
Chitale	-	-	-
Neill	-	-	-
Lacey	-	-	-

After reviewing these manuals and reports, no theoretical background or justification was found against the selection process of these formulae in the alluvial river context of Bangladesh. So, all the formulae used in these documents by IWFM, IWM, BUET-IWM and RHD are considered in this study. IWFM and IWM studies are hydraulic design reports, whereas BUET-IWM and RHD documents are design manuals. In spite of having some limitations, Lacey's formula is widely used in the Indian Subcontinent for local scour depth estimation. To overcome these limitations, a modified Lacey's formula was developed by Rahman and Haque (2003). The modified Lacey's formula was also included in this study for evaluation. Thus, this thesis used only the widely used formulae for local scour depth estimation in an alluvial river context of Bangladesh. It is obvious that there are many more formulae, which are kept out of this study. The purpose of this thesis is to see their suitability in the context of Bangladesh. Obviously, the lesser is the gap between the observed scour depth and the estimated scour depth, the better is the performance of an equation.

2.5.1 Breusers' equation

Breusers (1965) suggested an empirical equation, which is applicable for tidal flow, for scour depth calculation linking the depth of local scour, d_s and the pier width, b_p. The equation is expressed as:

$$\frac{d_s}{b_p} = 1.4 \tag{2.2}$$

where,

d_s = Scour depth

b_p = Pier width

2.5.2 Laursen's equation

Laursen (1963) provided an equation of local scour for threshold condition considering depth of flow and pier width. To develop this equation, he considered a clear-water condition, and hence it is applicable for this situation only. The mathematical expression of the equation is:

$$\frac{d_s}{b_p} = 1.34\left(\frac{h}{b_p}\right)^{0.5} \tag{2.3}$$

where,

d_s = Scour depth

b_p = Pier width

h = Hydraulic depth

2.5.3 Neill's equation

Neill (1987) expressed the relation between scour depth, d_s and pier width, b_p in terms of pier shape factor, K_s. Neill's equation for scour depth calculation for threshold condition is given below:

$$\frac{d_s}{b_p} = K_s \tag{2.4}$$

where,

$K_s = 1.5$ [For round nose and circular pier]

$K_s = 2.0$ [For rectangular pier]

2.5.4 Jain and Fischer's equation

Jain and Fischer (1980) modified Laursen's equation, which is applicable for a clear-water condition. According to them, the empirical equation is:

$$\frac{d_s}{b_p} = 1.86\left(\frac{h}{b_p}\right)^{0.5} \tag{2.5}$$

where,

d_s = Scour depth

b_p = Pier width

h = Hydraulic depth

2.5.5 Chitale's equation

Chitale (1988) stated that, the ratio of scour depth, d_s and bridge pier width, b_p is a constant value and always remain the same. The mathematical expression of the relation is given below:

$$\frac{d_s}{b_p} = 2.5 \tag{2.6}$$

where,

d_s = Scour depth

b_p = Pier width

2.5.6 Melville's equation

Melville (1997) provided a formula for scour depth calculation by considering three different conditions. They are:

$$\frac{d_s}{b_p} = 2.4; \quad \text{where, } \frac{b_p}{h} < 0.7$$

$$\frac{d_s}{b_p} = 2\left(\frac{h}{b_p}\right)^{1/2}; \text{ where, } 0.7 < \frac{b_p}{h} < 5$$

$$\frac{d_s}{b_p} = 4.5\left(\frac{h}{b_p}\right); \text{ where, } \frac{b_p}{h} > 5 \qquad (2.7)$$

where,

d_s = Scour depth

b_p = Pier width

h = Hydraulic depth

2.5.7 FHWA equation

A method for local scour depth calculation is provided in the sixth chapter of Hydraulic Engineering Circular (HEC) No. 18 of the United States Federal Highway Administration (FHWA) publication (FHWA, 2012). FHWA recommends the following Colorado State University (CSU) equation for calculation of both live-bed and clear-water local scour depth, d_s at bridge piers:

$$\frac{d_s}{h} = 2K_1K_2K_3K_4\left(\frac{b_p}{h}\right)^{0.65} F_r^{\,0.43} \qquad (2.8)$$

where,

h = Hydraulic depth, m

b_p = Pier width, m

K_1 = Correction factor for pier nose shape (K_1=1.0)

K_2 = Correction factor for angle of attack (K_2=1.0)

K_3 = Correction factor for bed condition (K_3=1.1)

K_4 = Correction factor for armouring of bed material size (K_4=1.0)

F_r = *Froude number* = $V_1/(gh)^{0.5}$

V_1 = Mean velocity of flow, m/s

g = Acceleration of gravity, m/s^2

The depth of local scour depends on flow velocity; for higher velocity there will be a deeper local scour and the local scour depth decreases with a decreasing trend in velocity. Moreover, there is a high probability that the scour depth is affected by the flow condition (subcritical or supercritical). Flow conditions can be easily delineated by Froude number. The flow is subcritical for F_r <1, the flow is critical for F_r =1, and the flow is supercritical for F_r >1.

K-factor values used in the FHWA equation (2.8) are given in Table 2.1 and can also be seen in related literatures.

Table 2.2: Correction factor, K_1 for pier nose shape (BUET-IWM, 2008)

Shape of pier nose	K_1
Square nose	1.1
Round nose	1.0
Circular cylinder	1.0
Group cylinder	1.0
Sharp nose	0.9

Correction factor for angle of attack, K_2 is 1.0 (considering θ= 0°). Correction factor for bed condition, K_3 is 1.1 unless there is a significant value of dune height. The correction factor K_4 decreases the scour depth for armouring of scour hole for bed material having D_{50} equal to or larger than 2.0 mm and D_{95} equal to or larger than 20 mm (FHWA, 2001). Correction factor for armouring of bed material size, K_4 is 1.0 (for d_{50}< 2 mm) (BUET-IWM, 2008). The value of K_4:

- $K_4 = 1$; for $D_{50} < 2$ mm or $D_{95} < 20$ mm and
- $K_4 = 0.4(V_R)^{0.15}$; for $D_{50} \geq 2$ mm or $D_{95} \geq 20$ mm.

where,

$V_R = \frac{V_0 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}}$

V_0 = approach flow velocity, m/s

V_{icD_X} = the approach flow velocity (m/s) required to initiate scour at the pier for the grain size D_X (m)

$V_{icD_X} = 0.645 \left(\frac{D_X}{a}\right)^{0.053} V_{cD_X}$

V_{cD_X} = the critical velocity (m/s) for incipient motion for the grain size D_X (m)

$V_{cD_X} = K_u y^{1/6} D_X^{1/3}$

y = depth of flow just upstream of the pier, excluding local scour, m

D_x = grain size for which x percent of the bed material is finer, m

K_u =6.19

2.5.8 Lacey's equation

Lacey's regime formula (1930) was developed on the basis of limited field data from irrigation canals in Punjab Province of India (Stevens and Nordin, 1987) having discharge ranges from 0.70 to 173 m^3/s. Lacey's regime formula was then modified for the estimation of local scour at the bridge site using some amplification factors. Even after almost 90 years of its development, the hydraulic engineers in the Indian Subcontinent still use it for the estimation of design scour depth around various hydraulic structures like bridge piers (Rakshit, 2009). The scour depth can be calculated from the equation below:

$$d_s = 0.473 \left(\frac{Q}{f}\right)^{\frac{1}{3}} \tag{2.9}$$

where,

d_s = Scour depth, m

Q = Discharge, m^3/s

f = Silt factor =$1.76\sqrt{d_{50}}, d_{50}\ is\ in\ mm.$

2.5.9 Modified Lacey's equation

Lacey's original formula always over-predict the scour depth around pier-like structures in alluvial rivers and thus limits its applicability (Kabir, Faisal and Khatun, 2000). In this regard, Rahman and Haque (2003) modified the Lacey's original equation by incorporating hydraulic depth and pier width. Afterward, applicability of the modified Lacey's equation was compared for the Jamuna, Ganges and Meghna rivers in Bangladesh. Comparison was made with the field observed local scour data to that of predicted local scour values by original Lacey's and modified Lacey's equations. Original Lacey's equation always over-predicted the scour than that observed in the field. Conversely, the modified Lacey's equation estimated values reasonably close to the observed values. The modified Lacey's equation (Rahman and Haque, 2003) is:

$$\frac{d_s}{b_p} = \left[0.47\, M^{\frac{1}{3}} \left(1 + 4.5\, \frac{b_p}{h}\right)^{\frac{1}{3}} - 1\right] \times \left(\frac{h}{b_p}\right) \qquad (2.10)$$

where,

d_s = Local scour depth, m

b_p = Pier width, m

M = M-factors for different river system (Table 2.2)

h = Hydraulic depth, m

Table 2.3: Suggested values of M for different river system (Rahman and Haque, 2003)

River name	M-factor
The Jamuna	30
The Ganges	25
The Meghna	15

2.11 HEC-RAS Model Simulation for Scour Depth Prediction

HEC-RAS (Hydraulic Engineering Center's River Analysis System) is a hydraulic model, which includes bridge scour prediction, developed by the Hydraulic Engineering Center (HEC) of the U.S. Army Corps of Engineers (USACE). It has been widely used by the State Department of Transportation and the private design practitioners in the United States as well as by the practicing engineers in Bangladesh for scour depth estimation (for example, IWFM has recently used this in a number of studies in relation to bridge scour estimation). The first version of HEC-RAS (version 1.0) was released in July, 1995. The most recent version of HEC-RAS is 5.0.5, released in June, 2018.

The HEC-RAS model allows its user to choose between the CSU equation and the Froehlich equation for the computation of local scour at bridge piers. The model sets the CSU equation for all the simulation by default, if otherwise not changed to Froehlich equation. The HEC-RAS model is easy to use and a open-source software. Being an open source software developed by the United States Army Corps of Engineers, the software is accepted by the most government and private agencies around the globe.

2.12 Previous Studies on Pier Scour

Many researches had been done so far with local scour around hydraulic structures like bridge, culvert, spur, dike, abutment, guide bundh, etc. around the world including Bangladesh. The researches include development of local scour depth prediction formula, understanding physics of local scour, identifying causes of local scour, etc. The studies conducted in Bangladesh is reviewed first and then the studies done elsewhere are reviewed.

2.7.1 Previous studies on local scour in Bangladesh

A laboratory-based experimental investigation by Kabir (1984) found that, scour depth depends on sediment size, pier shape and depth of flow. The scour depth is more for finer sediment material and is relatively less for coarser material. The maximum scour depth was found for rectangular pier. The circular, round nose and sharp nose piers had increasing scour depths, but less than the rectangular pier. The study also found that the maximum scour depth may occur upstream or downstream of the pier depending on the pier shape. Relative scour depth (d_s/b_p) was found to increase if there is a rise in relative approach depth (y/b_p).

Another experimental study was conducted by Khatun (2001) to investigate the effect of bed materials commonly found in the rivers of Bangladesh on local scour using the commonly used pier shape of our country. Another objective of the study was to investigate the effect of cohesive sediment on local scour. The maximum scour depth was found to the upstream face of the pier in variable water conditions. The slope of scour hole is steepest at the front side of the pier, gradually decreases side wise and flatter at the rear side. Again, increase in the intensity of scour lines with increasing discharge revealed that the slope of scour hole converted from flatter to steeper. The slope at the front side of the pier is more or less uniform for higher discharges (from 10 lps to 40 lps), whereas less uniform for lower flow conditions (for 4 lps and 6 lps). The extent of scour hole gradually increases with increasing discharge. As the experimentations are carried out in live bed regime, the equilibrium scour depth is reached relatively quickly than that in the clear water regime and at higher velocities, the equilibrium is attained very rapidly. No constant scour depth is found in equilibrium condition; it oscillates periodically with time. Again at the equilibrium state, the flow field is approximately symmetrical which is evident with the help of velocity vectors around the pier.

An experimental study on local scour around pier-like-structure on floodplain of a compound channel was conducted by Roy (2008). The study found that, in general, maximum scour depth occurred for finer bed material at a higher discharge. There was a decrease in scour depth with an increase in pier length-width ratio. Steeper, deeper and uniform scour was found for finer bed material in comparison with coarser bed material. Circular and round nose shapes of pier structure were used in the experiments similar to commonly used shapes in Bangladesh. The experiments were conducted for four pier length to pier width ratio (l/b_p) in which $l/b_p=1$ for circular structure and $l/b_p=2$, $l/b_p=3$, $l/b_p=4$ for round nose structure. A constant pier width has been maintained. Circular pier gave the higher scour because of its lower pier length comfortable for wake vortex to reach the bed level and make a deeper scour hole. Velocity is found higher in main channel than that of floodplain for the same discharge, as the depth of flow is more in the channel than that on the floodplain. It may also affect scour because, scour depth is found higher in main channel than that on floodplain for the same discharge in case of the same bed material.

A study on local scour of selected road culverts by Hossain (2011) found a higher rate of scour occurring in the downstream of a culvert with increased rate of discharge. Scour depth also increased with material fineness and discharge, and vice-versa. Scour at culvert outlet occurred in water recession time and was absent in water rising time. Scour hole occurred downstream of the culvert was greater in extent in both lateral and longitudinal directions. Culverts having apron and cut-off walls protected the structure from scouring. Lacey's formula was found to be the best predictor of local scour when compared with the filed data.

A study to evaluate the impact of the 1998 flood on major bridges of Bangladesh was undertaken. In this regard, a total of eleven bridges were studied (Hoque et.al. 1999 and Hoque et. al. 2002). The Turag-Bhakurta bridge is one of them which is 67 m long. An attempt to find out the causes of the Turag-Bhakurta bridge failures was made. Inflow channel (a branch from the river Dhaleswari) through Chira at village Bhakurta was an active dominant river during the rainy season with high velocity. Before construction of the bridge, the 1 km Turag-Bhakurta road was not developed and spill water passed through this section naturally towards the river Turag. The main flow was directed through the damaged section of the bridge. After construction of the bridge, the 1 km road was upgraded and raised which obstructed the outflow (Bala et. al., 2005). Scour depth at the bridge site at pier-1 reached its pile length (6.60 m) (settled down with adjacent deck slab and girder) and reached its pile length (13.40 m) (the bridge was washed away except the rehabilitated pier of 30 m length) during the flood of 1995 and 1998, respectively. Inflow discharges through Chira during 1995 and 1998 floods were about 1994 m^3/s and 2775 m^3/s, respectively. According to the Lacey's regime theory, safety exit lengths should be 212 m and 250 m for the flood of 1995 and 1998, respectively. The total existing bridge opening length was not adequate and affected the bridge during the floods. After analyzing the out flow discharge it was found that, opening should be 154 m and 176 m considering the flood of 1995 and 1998, respectively. But the bridge had an inadequate length of only 67 m. As a result, deep scour occurred at bridge pier during the 1995 and 1998 floods. During the flood of 1998, the observed flood level at bridge site was 9.70 m and submerged the superstructure of the bridge due to inadequacy of freeboard. Enormous hydraulic pressure was applied on the substructure of the bridge.

An evaluation of the impact of the 1998 flood on the morphological changes of the rivers around bridges was conducted (Hoque et. al., 2002). A detailed field survey around the Meghna bridge was carried out. Bathymetric data was collected by using the electronic distance meter (EDM) and echo sounder. To evaluate the morphological changes at the vicinity of the bridge, a river reach of about 1 km downstream and 5 km upstream was considered. After analyzing the bathymetry of May 1998 (pre-flood) and October 1998 (post-flood), a significant change was found to have occurred in the bed level of the river Meghna upstream of the bridge during the flood of 1998. Findings showed that, deeper scour depth was observed during the pre-flood time (May 1998) in comparison with the post-flood time (October 1998). But, in comparison with the results of February 1997, several scour holes were found in October 1998.

Though Lacey's equation is widely used in the Indian Subcontinent including Bangladesh for prediction of local scour depth around bridge pier, the formula is independent of any changes in b_p (pier width) and y (approach flow depth). But, it is established that, b_p/h is the dominant factor for the estimation of local scour depth around piers and abutments. Considering its limitation, original Lacey formula was modified introducing the parameters b_p and h, and a modified Lacey's formula was developed by Rahman and Hoque (2003). After its development, the usability of the formula was tested for the major rivers of Bangladesh. For test purpose, the observed scour depth data around different pier-like structures along the Jamuna, the Ganges and the Meghna rivers were compared with the predicted values by Lacey's original formula and modified formula. After comparison, it was found that the original Lacey's equation predicts constant scour depth for a specific type of structure in a particular river. But, the modified Lacey's formula predicted scour depth is a function of the pier width and closely comparable with Laursen's formula and Melville's formula up to the limit of $h/b_p < 1.5$. After analyzing, it was found that the original formula always over predicted the observed values. The modified Lacey's formula predicted values were closely comparable with the observed values and other available formula. Therefore, it is better to use the modified Lacey formula instead of the original one. One of the limitations of the Lacey's modified formula is that, it cannot cope with variable sloped-wall abutment like structures. The findings of the study (Rahman and Haque, 2003) suggest that the modified Lacey formula is applicable for pier-like-structures adopted in the large scale rivers in Bangladesh within $h/ b_p<1.50$.

A study showing the present status of three major bridges in Bangladesh was conducted where the presence of local scour was identified as a safety issue for the bridges. The riverbed measured along the center line of the existing Meghna Bridge showed a tremendous extent of riverbed scour leaving the bridge piers in a critical condition. The deepest riverbed was measured at -6.80 m PWD (in 1991), -21.95 m PWD (in 1997), -21.75 m PWD (in 2005), -19.26 m PWD (in 2010) and -21.55 m PWD (in 2012) in the bathymetric surveys of the Meghna River. Therefore, the Meghna riverbed scouring was becoming a critical issue day by day and undoubtedly necessitated an appropriate countermeasure for bridge construction (Hoque et al., 2015)

After the attack of the severe Category V Cyclone Sidr on November 15, 2007, while people were waiting on a bridge in Kalapara, a village in the Patuakhali district, for relief materials to arrive, the bridge suddenly collapsed and four people were killed and hundreds were injured.

Anwaruzzaman (1998) conducted a study to test the performance of some selected empirical formulae used to predict the scour depth by comparing with field and model values. For circular pier, Inglis' equation over predicted for both field and experimental data. Ahmed's equation also over predicted, with some exceptions. Blench's equation under predicted most of the experimental data, but there is both under and over estimation for field data. By comparing the data, it was concluded that Blench's equation is the best for predicting local scour in the case of circular pier. For round nose pier, both Inglis' and Ahmed's equations under predicted the field data and over predicted the experimental data. Blench's equation predicted both under and over the perfect line and also matched the line for some cases as well. So, Blench's equation came out as the best among the equations for round nose pier.

2.7.2 Previous studies on local scour around the world

Shukri (2017) conducted an experimental study on local scour depth around cylindrical piers under live bed scour condition. The author found that local scour depth increased with increasing discharge. Equilibrium scour depth was proportional to the approaching discharge to the pier. Kinetic energy of flow increased with added discharge and thus destructive effect of the flow increased, which was responsible for bridge collapse during the flood season. Depth of scour was proportional to the pier diameter for the same sediment size and discharge. Maximum scour depth was reciprocal to the sediment mean size.

Though Lacey's equation is widely used by the practicing engineers in Asia for design of hydraulic structures, especially for local scour prediction, there are a number of limitations of the equation. In a study by Mazumder (2007), it came out that the predicted local scour from Lacey's equation is excessive. Predicted local scour by using Lacey's equation varied for two water level considerations; they are:

a) Scour depth is 2.4 to 90 percentile excessive in comparison with Melville and Coleman, Richardson and Davis (HEC-18), Breussers, Raudkivi and Kothyari, Garde and Raju (for below HFL)

b) Scour depth is 10 to 275 percentile excessive in comparison with Melville and Coleman, Richardson and Davis (HEC-18), Breussers, Raudkivi and Kothyari, Garde and Ranga Raju (for HFL)

For both the cases, it was assumed that, the bed level did not change as it was found for low water profile.

Deshmukh and Raikar (2014) conducted an experimental study on pier scour under steady flow condition for the same discharge with the same depth of flow for different pier diameters. It was found that the scour depth goes on increasing with time up to certain limit and it attains a constant depth of scour which is considered as equilibrium scour depth. The scour depth increases with an increase in pier diameter by keeping all parameters, i.e. flow characteristics and sediment characteristics, the same. Maximum depth is observed on the upstream side of pier. Lateral extent is observed to be more on the upstream side of the pier. Non-dimensional scour depth gradient is different up to opening ratio of 0.8 having less slope and its slope is steep above 0.8 opening ratio.

Khassaf (2016) addressed three cases in a study of local scour depth around bridge piers using artificial neural network: the effect of pier size, flow velocity and flow depth. The findings of the study revealed that a larger pier diameter produces a deeper local scour upstream of the pier because the strength of horseshoe vortex is proportional to the pier diameter. An increment in flow velocity causes a higher flow intensity for constant flow depth and pier diameter, and thus will lead to more scour depth because of the velocity incensement. Local scour depth is also found to be proportional with the flow depth.

Choudhury and Hasnat (2015) analyzed the causes of 503 reported bridge collapses during the period of 1989 to 2000 in the United States and showed that, local scour is the second highest causes of the bridge collapse.

CHAPTER THREE

METHODOLGY AND FIELD DATA COLLECTION

3.1 The River System

The Dhaleshwari River is a distributary of the Jamuna River in central Bangladesh. It starts off the Jamuna near the northwestern tip of Tangail District. After that, it divides into two branches: the north branch retains the name Dhaleshwari and merges with the other branch, the Kaliganga, at the southern part of Manikganj District. Finally, the merged river meets the Shitalakshya near Narayanganj District. This combined river then flows southwards and falls into the Meghna River (Wikipedia, 2018). Figure 3.1 shows the adjoining river system of the Dhaleswari River.

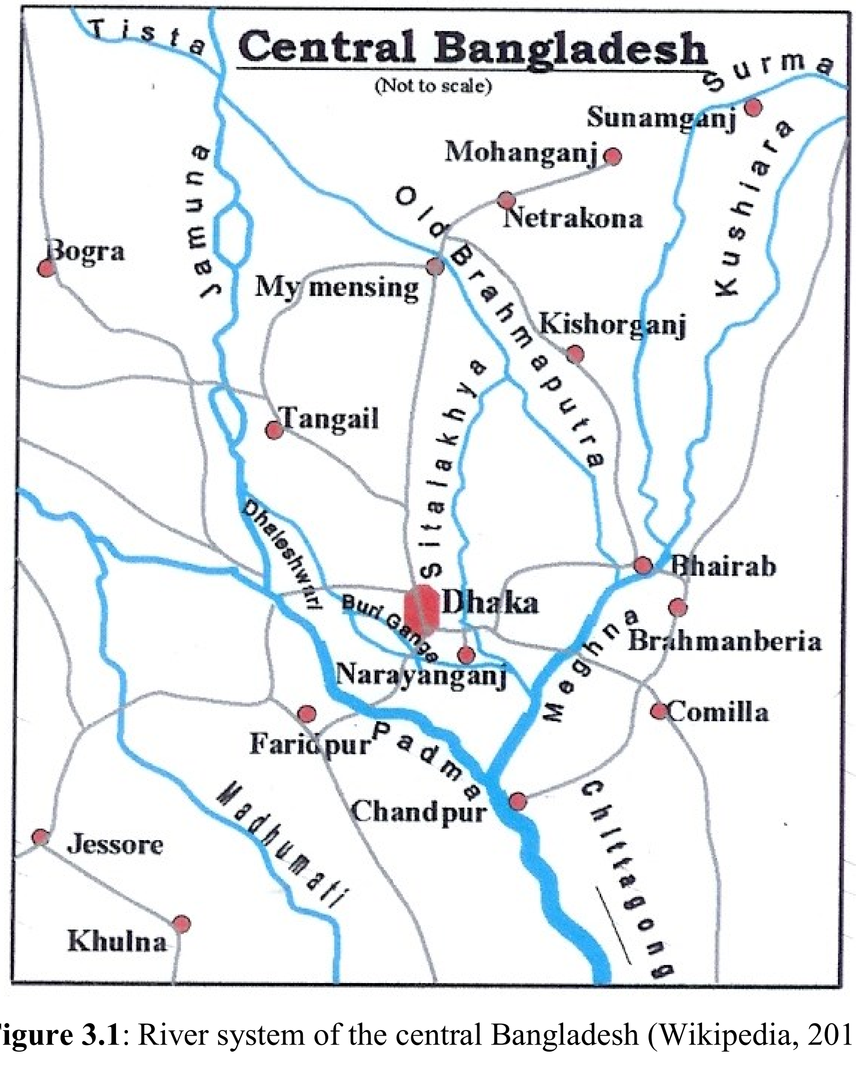

Figure 3.1: River system of the central Bangladesh (Wikipedia, 2018)

The river Kaliganga, originated from the Dhaleshwari River at Shaturia Upazila of Manikganj District, flows downstream and finally meets the Dhaleshwari River in Singair Upazila. On its way downstream from the origin to the end point, the Kaliganga River flows through Ghior, Sadar and Harirampur Upazilas of Manikganj District (Figure 3.2).

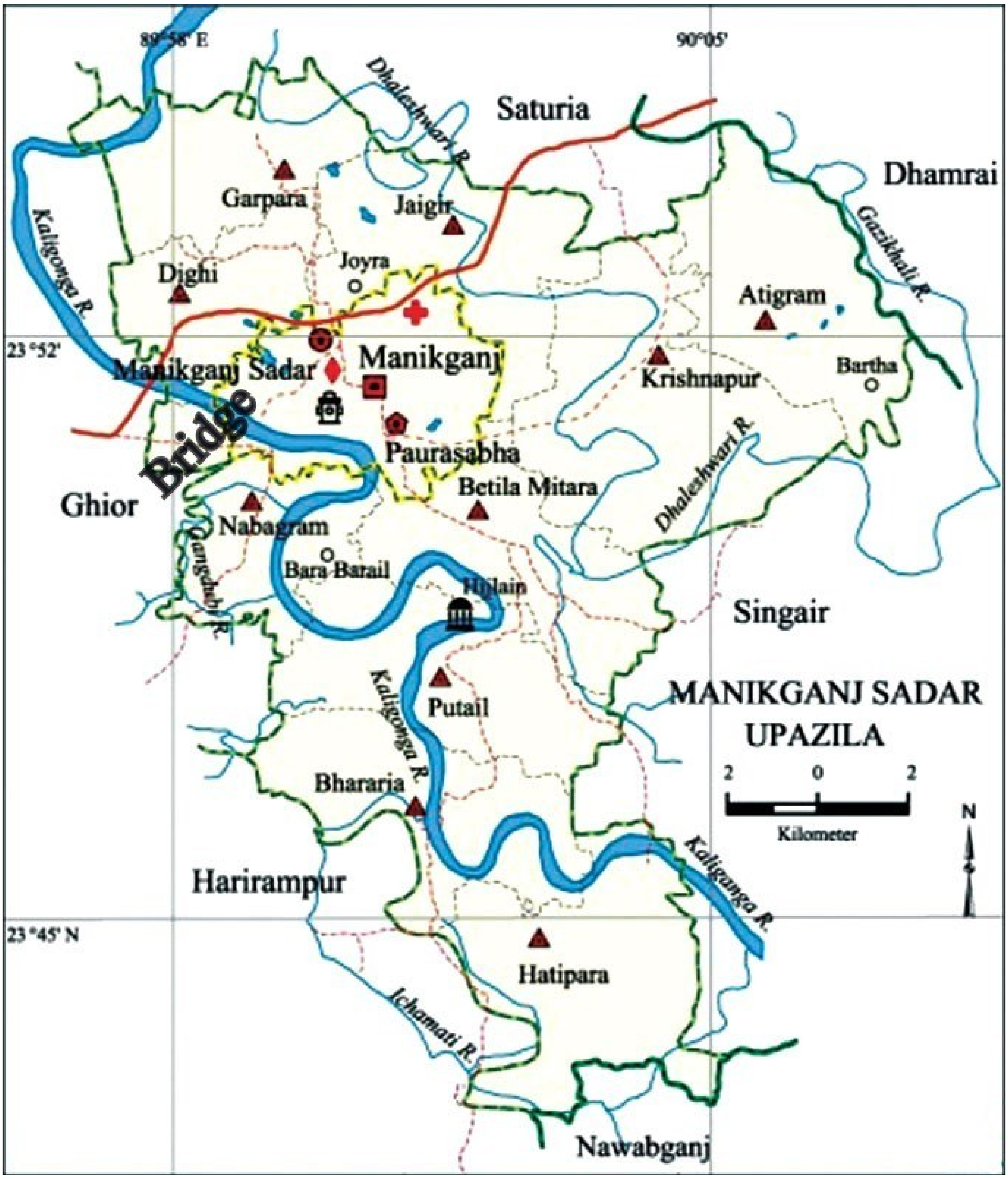

Figure 3.2: River system of Manikganj Sadar Upazila with bridge location (Banglapedia, 2010)

The Kaliganga River is one of the major branches of the Dhaleshwari River. Originally, it was a narrow khal, but with time the river got wider and deeper. The Manikganj District headquarter and Manikganj town are located on the left bank of the river. In the nineteenth century, the Manikganj town and the Sadar Thana were eroded by the river Kaliganga and the town was shifted to its present location and the Sadar Thana was located to Beutha on the bank of Kaliganga but was eroded away again. The Thana head quarter then was moved to its present location in Manikganj town. The total length of river is about only 160 km and the average depth is 10 m. The river is meandering in nature. Erosion is evident in some places in the course of the river. At both upstream (Pacbaroil Bot Tola) and downstream (Andarmanik graveyard) of the bridge location, bank erosion is taking place, but the constructed bridge location is relatively stable.

Hydro-morphological assessment of the Dhaleshwari reveals that, main channel and bank of the river was shifted significantly from the year 2003 to the year 2013 (Ahsan, 2018). The width of the channel has lessened, while the depth has increased. Figures 3.3 and 3.4 demonstrate the comparison of superimposed cross sections relatively nearest and farthest from the offtake.

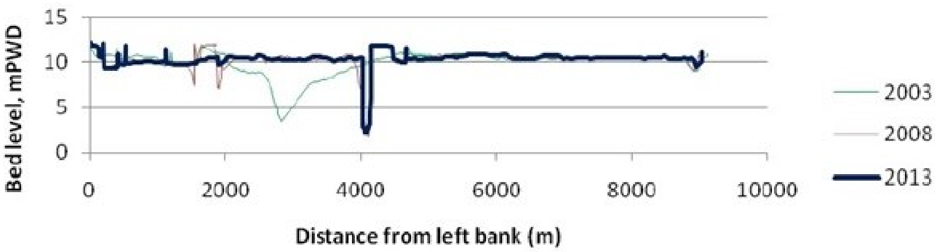

Figure 3.3: Superimposition of cross sections near offtake in different years (Ahsan, 2018)

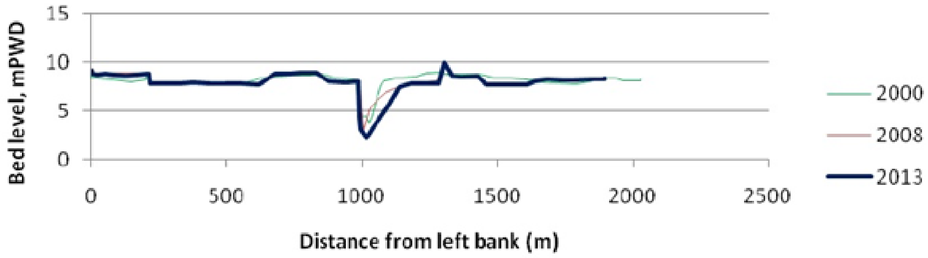

Figure 3.4: Superimposition of cross sections relatively far from offtake in different years (Ahsan, 2018)

3.2 The Bridge

The Dhaleshwari bridge is constructed over the Kaliganga River connecting the western part of the Sadar Upazila and growth centers in Ghior with the Upazila headquarter and is located at 23°50'48.00"N latitude and 89°59'56.07"E longitude. The catchment area of the Kaliganga River is 876 square kilometers. The river is non-tidal and perennial, that is, it flows year round. The length of the Dhaleshwari bridge is 297 m. The bridge has 8 numbers of pier, 9 spans with a length of 33 m each (IWFM, 2013). The piers of the Dhaleshwari bridge are of complex types. They are comprised of pier, pile cap and pile group. Pier width is 1.5 m. Two piers (at a distance of 6 m, center to center) are being constructed in a straight line over a pile cap. The length and breadth of the pile cap are 11 m and 7 m, respectively. There is a pile group below the pile cap with some smaller dimension than the pile cap. Total eight numbers of pile of 1.0 m diameter are being constructed in two rows (four number of piles in each row) (Appendix-D). The complex pier of the Dhaleswari bridge along with its field measured dimensions are given in Photo 3.1.

Photo 3.1: Photo showing bridge pier, pile cap, etc. (Pier no. 3).

The river flows into the main channel during the dry season because of its perennial nature and overflows the floodplains during the monsoon season. In this study, the local pier scouring was focused on the piers bearing numbers 3, 4, 5 and 6 from the left bank side because these piers are constructed in the main river (Photo 3.2). The left abutment of the bridge connects Manikganj District at the Beutha Ghat (7.03 m high from the existing ground level, EGL) with the Char Kushar by the right abutment having an elevation of 4.93 m from the EGL. The height of the bridge is 14.60 m PWD. The deck level of the bridge is 2 m above the bridge height, i.e., 16.60 m PWD (IWFM, 2013). The bridge has been constructed by the Local Government Engineering Department (LGED) under the "Three Important Large Bridges in Naogaon and Manikganj Districts Project". In this regard, the hydrological and morphological study (IWFM, 2013) had been carried out by the Institute of Water and Flood Management (IWFM) of Bangladesh University of Engineering and Technology (BUET). The bridge construction was completed in the month of October, 2016.

Photo 3.2: Photo taken from the downstream of the bridge displaying its piers, deck and left abutment.

3.3 Methodology of the Study

The Beutha bridge in Manikganj Sadar Upazila over the Dhaleswari/Kaliganga River is selected in this study for investigation of local scour around bridge pier. The reasons behind selecting this bridge are:

i. The hydro-morphological study report, structural designs and drawings, etc. were readily available as the author was associated with the hydro-morphological study of the bridge as a research assistant for IWFM, BUET;
ii. LGED has already implemented this bridge on this river; and
iii. The 2017 flood was the first flood for the bridge which provided an opportunity to study the temporal variation of local scouring from the very beginning of the bridge life.

The thesis was completed by a step by step methodology. To understand the methodology of the thesis, a flowchart of the methodology followed is given below.

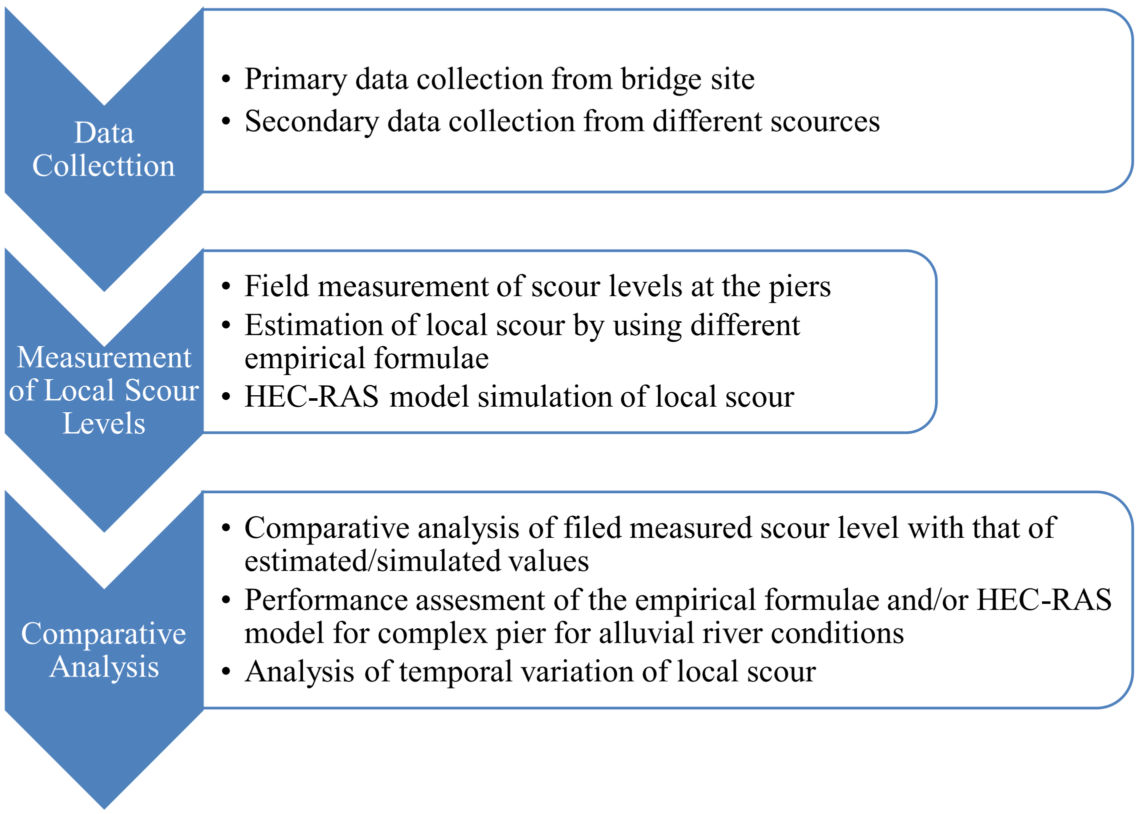

Figure 3.5: Flow chart showing the overall methodology of the study

3.4 Data Collection

Both primary and secondary data were required for meeting the objectives of this study. Brief descriptions of the various data collected and their collection processes are given below:

3.4.1 Primary data collection

A variety of primary data were collected from the field and other sources. The following data are collected from the field:

a) Bathymetric data of the river reach, both upstream and downstream of the bridge during the pre-monsoon, monsoon and post-monsoon conditions;
b) Bed material of the river for determination of grain sizes;
c) Water level of the river at the bridge site; and
d) Flow velocity of the river at the bridge site.

3.4.1.1 Bathymetric data

The local scour data around bridge piers were taken during the months of May to October, 2017 on a monthly basis through bathymetric survey. To conduct the bathymetric survey of the river, five field visits were made during the pre-monsoon, monsoon and post-monsoon seasons. The survey dates were selected so that the changes in bathymetry by the flood event can be taken. The event dates are 20 May (pre-monsoon), 24 June (monsoon), 4 August (monsoon), 19 August (monsoon) and 16 October (post-monsoon). As the objectives of the field survey were fixed earlier and the field trips were made to fulfil the fixed target, in every event date the same activities were performed. The bathymetric survey was conducted with the help of some associated devices: a hydrographic echo sounder (Echotrac CVM), a hydro-pro software installed in a portable computer, a differential global positioning system (DGPS), a transducer and two 12-volt batteries. All the above mentioned devices were set up in a locally available motor boat. Before starting, the workability of the full setup was checked. After that, the author and the surveyor guided the boatman to move in the selected river reach (some distances up of the bridge site, at the bridge site and some distances down of the bridge site). Special focus was put on the bridge site, especially around the piers in the main channel. The measurement set up automatically recorded the river bathymetric data for further use (Photos 3.3, 3.4, 3.5 and 3.6). The collected bathymetric data of the river are shown in Tables A.1 to A.5 in Appendix-A. Table A.1 shows the bed levels recorded in the month of May. Tables A.2, A.3, A.4 and A.5 refer to the observed bed levels in the months of June, August, August and October, respectively.

The observed bed levels around the bridge piers and both upstream and downstream of the bridge are shown in Figures B.1 to B.51 of Appendix-B. In the figures, distances from the left bank to the right bank are shown in the X-axis and bed levels in the Y-axis. Figures B.1 to B.6 are for May 20, 2017; Figures B.7 to B.12 are for June 24, 2017; Figures B.13 to B.25 are for August 4, 2017; Figures B.26 to B.38 are for August 19, 2017; and Figures B.39 to B.51 are for October 16, 2017.

Photo 3.3: A portable computer with the hydro-pro software used for bathymetric survey.

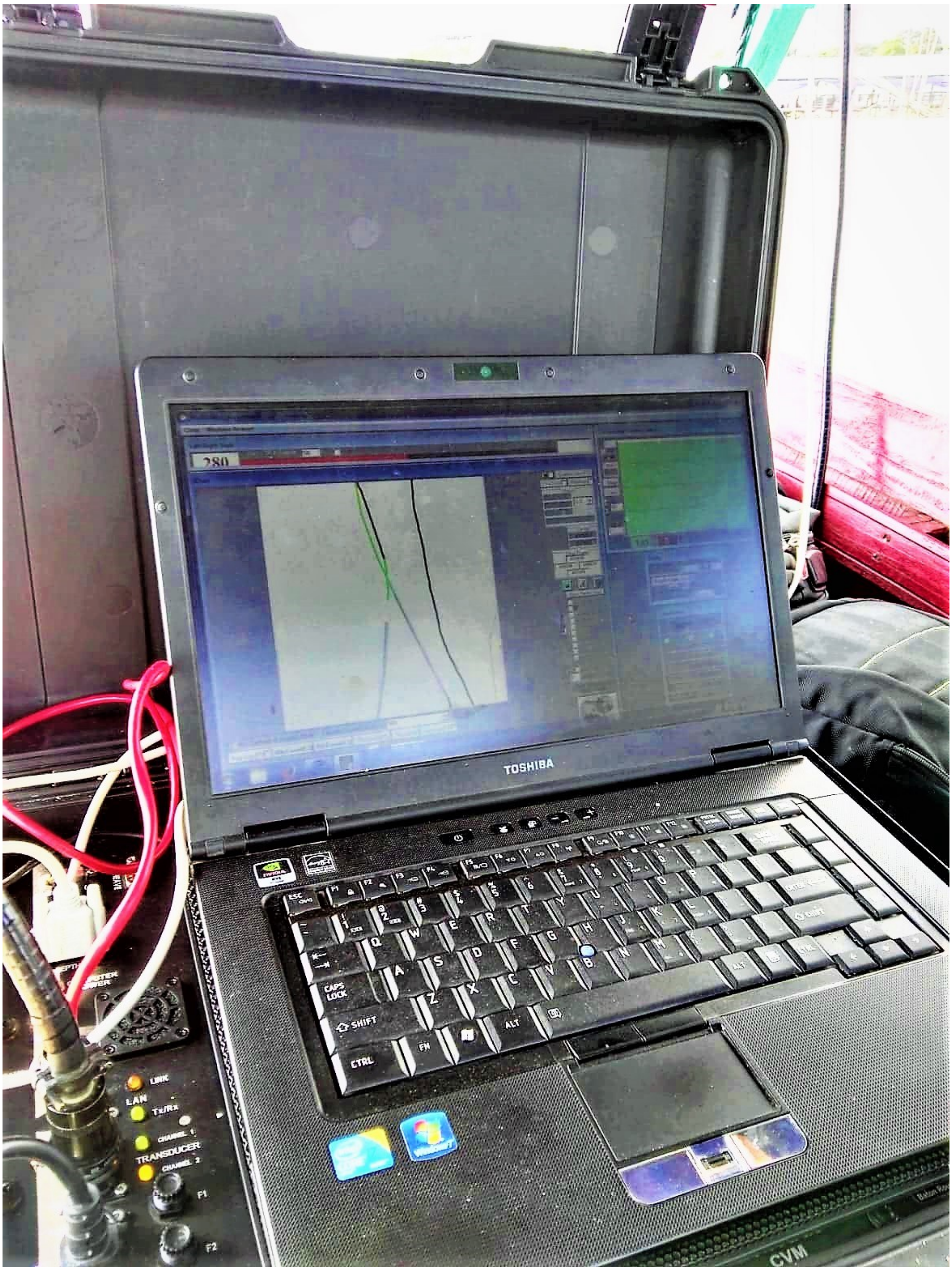

Photo 3.4: Portable computer along with echo-sounder on board for obtaining local scour bathymetry.

Photo 3.5: Transducer and DGPS antenna set up used for bathymetric data collection.

Photo 3.6: A view of the Dhaleswari Bridge taken from the boat used for bathymetric survey

3.4.1.2 Bed material sampling

Sand-bed streams (alluvial in type) have relatively homogeneous bed material gradation. Vertical and temporal variability are normally insignificant in stable sand-bed streams. Longitudinal variability typically occurs over distances of many kilometers. However, lateral variability, especially in bends, can be significant. In sand-bed rivers, sampling of bed material is most frequently done in the low-flow channel. The sampling equipment and methodology used depend on the river depth and velocity. Vertical variations in the bed material are usually insignificant in flowing water, and samples are collected from the surface. However, in standing water or on dry beds, a layer of fine material is sometimes found deposited on the bed surface during the recessional part of a flood hydrograph. It is standard practice to remove this fine surface layer before collecting a bed-material sample in this location. Einstein (1950) recommended using only the coarsest 90 percent of the sampled bed gradation for computation of bed-material load. He reasoned that the finest 10 percent of sediment on the bed was either material trapped in the interstices of the deposit or a lag deposit from the recession of the hydrograph and should not be included in bed-material load computations (USDA, 2007).

Four soil samples from the river bed were collected in August 2017 to estimate the grain size of the sediment particle. The four places are two along the bridge axis, one upstream of the bridge and the other downstream of the bridge. Both sieve and hydrometer analyses of the samples were carried out at the IWFM laboratory. Four grain size distribution curves drawn based on the four soil samples are given in Appendix C. The median diameter of the sediment from the curves was found to vary from 0.16 mm to 0.27 mm. The value (0.16 mm) reported in FAP 24 (1996) is at the lower end of the values found in this study. Hence, to be conservative, the smaller value (0.16 mm) obtained from the field samples is used in scour estimation in this study.

3.4.1.3 Water level

The daily water levels were required as basic data for this study. To ease the data collection process, some temporary bench marks (TBMs) were established on the left abutment and piers. The TBMs were set from the earlier bench mark (BM) kept in the premise of the Beutha Miftahul Hafizia Madrasa by IWFM (2013) (Photo 3.7). The TBM on the left abutment was set first from the BM by using the standard leveling instrument. After that, an intermediate TBM was established on pier no. 2, and a final TBM was marked on the pier no. 3 which is located in the main river (Photos 3.8, 3.9 and 3.10).

Photo 3.7: Bench Mark (BM) placed on the plinth level of the Madrasa compound by IWFM (2013).

Photo 3.8: Establishment of TBM on the left abutment

Photo 3.9: Setting up of the TBMs on a pier by the standard leveling instrument.

Photo 3.10: Photo showing the TBM established on the left abutment.

The daily water level was measured with respect to the above TBMs during 1 May – 31 October, 2017 by engaging a local gage reader. The water level was measured daily in the afternoon around 5:00 pm as per convenience of the gage reader.

3.4.1.4 Flow velocity

The river flow velocity was taken in every field trip. The velocity of flow in the river was measured by float method. Though the original plan was to use the ADCP available with IWFM for velocity measurement, it was not available for this study. A float (long rooted water hyacinth or partially filled plastic water container) was released from a fixed point upstream of the pier and the time was recorded with a stop watch. When the float passed another fixed point in the downstream of the pier, the time was recorded again. From the distance between the two points and the time elapsed during the travel of the float, the velocity of flow was estimated. Thus, this technique provided approximate velocity of flow in the river.

The maximum observed velocity is found to be 1.37 m/s on 19 August and the minimum observed velocity is found to be 0.35 m/s on 20 May, 2017. The average measured velocity is found to be 0.87 m/s. The observed velocity at the bridge site is given in the Table 3.1.

Table 3.1: Measured flow velocity of the Dhaleswari River at the bridge site

Date	Measured velocity (m/s)
20 May 2017	0.35
24 June 2017	0.55
4 August 2017	1.02
19 August 2017	1.37
16 October 2017	1.07

3.4.2 Secondary data collection

The Dhaleshwari bridge had been built by the Local Government Engineering Department (LGED). The planning of bridge construction had been started in the year of 2012. While planning for the bridge, LGED had assigned IWFM, BUET for conducting a ‘Hydrological and Morphological Study’ of the proposed bridge. For this purpose, IWFM had collected some data of the river. These data are:

a) Bathymetric data of the river at the bridge site including both its upstream and downstream;
b) Predicted maximum scour depth at the bridge site; and
c) Median size (d_{50}) value of the bed material.

These data were gathered from the final report prepared by IWFM, BUET (IWFM, 2013). The bathymetric data was collected in a river stretch starting from 2 km upstream of the bridge site and ending in 2 km downstream of the bridge site. Total 18 numbers of bathymetric data were collected in the study.

In the process of hydro-morphological study, maximum scour depth was estimated by IWFM (2013) using different empirical formulae. The formulae used are: Breusers (1965), Laursen (1963), Neill (1987), Jain and Fischer (1980), Chitale (1988) and Melville (1997). The maximum estimated scour depth for pier was found to be -4.46 m PWD by using Chitale (1988) formula.

Apart from the above data, data on water level and discharge of the river at Taraghat for the year of 2017 were collected from the Hydrology Directorate at Green Road of Bangladesh Water Development Board (BWDB). After analyzing the water level and discharge data collected from the Hydrology Directorate of BWDB, it was found that there is a lack of discharge data (on a daily basis) collected from BWDB. In fact, no daily discharge data was found from BWDB for the specific dates (20 May, 24 June, 04 August, 19 August, 16 October) of the five field trips. But the water level data was available on a daily basis. Since, the discharges of the specific dates are necessary to put as input value in estimation/simulation of local scour by using different formulae and mathematical model used in this study, a stage-discharge relationship was developed to estimate the discharges of the required dates and to put them in use when required.

3.4.2.1 Methodology for development of stage-discharge relationship (rating curves)

Barring a few exceptional cases, continuous measurement of stream discharge is difficult to obtain. Further, direct measurement of discharge is a time consuming and costly procedure. Besides, stage can be observed continuously or at regular short time intervals with comparative ease and economy. Hence, a two-step procedure is followed in practice:

- Step 1: A relationship between the discharge to the corresponding elevation of the water surface (gage) is established through a series of careful measurements.
- Step 2: The daily discharges are then estimated by using the previously determined stage-discharge relationship with the help of the routinely observed gage of the river. This stage-discharge relationship is alternatively known as the rating curve.

If G represents the stage for discharge Q, then the relationship between G and Q can possibly be approximated with an equation:

$$Q = C_r * (G-a)^{\beta} \tag{3.1}$$

Where, C_r and β are rating curve constants, 'a' is a constant which represents the gage reading corresponding to zero discharge. The constant 'a' can be measured when a stream is flowing under "section control" as the surveyed gage height of the lowest point of the section control feature.

3.4.2.2 Development of stage-discharge relationship for the Dhaleswari River

In Bangladesh, discharge measuring authority (BWDB) provided only 10 to 15 discharge data during the monsoon and post-monsoon periods of 2017 for Taraghat (SW137A) station of the Dhaleswari/Kaliganga River (Table 3.2).

The stage-discharge relationship (rating curve) from the available data of Taraghat station was developed by following the two steps given below:

Step 1: Discharge vs. stage (Q vs G) is plotted on an arithmetic graph paper and drawn a best fit curve. By extrapolating the curve by eye judgment, the value of G corresponding to Q = 0 is determined. It is the preliminary value of 'a'. Then, by using the value of 'a', log(Q) vs. log(G-a) is plotted on an arithmetic graph paper and verified whether the data plots as a straight line. If not, then another of value of 'a' is selected in the neighborhood of the previously assumed value. By a trial-and-error process, the acceptable value of ‘a’ is finally determined that gives a nearly straight line plot of log(Q) vs. log(G-a).

Table 3.2: Discharge data available for Taraghat station for the year 2017 (BWDB, 2017)

Date	Water Level	Discharge (m^3/s)
23-May-17	2.98	251.43
6-Jun-17	3.99	363.23
20-Jun-17	5.30	444.59
4-Jul-17	5.96	621.06
18-Jul-17	7.71	1668.23
2-Aug-17	6.66	1492.61
14-Aug-17	7.23	1758.92
5-Sep-17	7.68	2068.43
19-Sep-17	7.03	1858.93
3-Oct-17	6.15	1459.98
17-Oct-17	5.11	542.02

Step 2: The values of C_r, β and the coefficient of correlation, r are calculated by using simple formulae. Once the stage-discharge relationship is established for discharge station of Taraghat, discharge value can easily be calculated from the daily corresponding stage by using the stage-discharge equation. The equation used in this study is:

$$Q = 154.29*(G-1.9)^{1.34} \tag{3.2}$$

By using equation (3.2), discharges of the specific dates were estimated by previously developed stage-discharge relationship. The process of discharge estimation involved many graphical analyses. The estimated discharges and observed discharges (by BWDB) are plotted against the dates of occurrence in Figure 3.6.

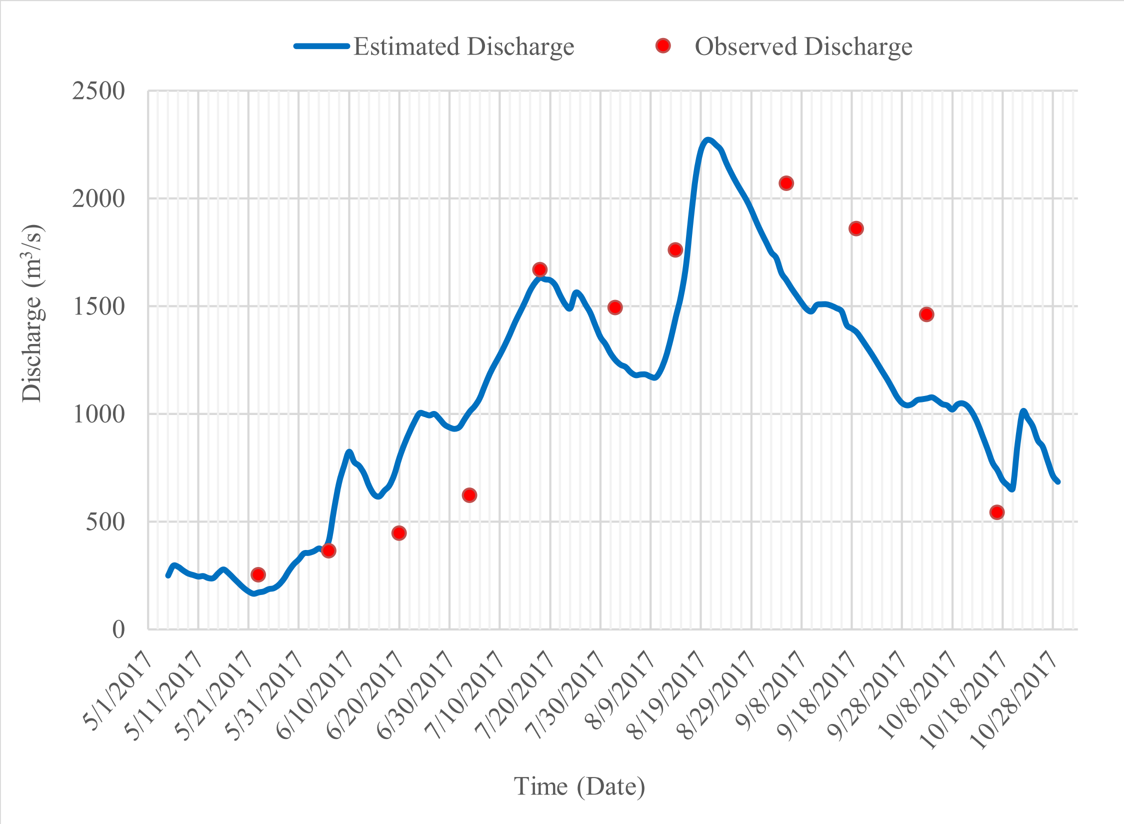

Figure 3.6: Estimated discharge hydrograph of the Kaliganga River at Taraghat for the year of 2017 (solid line) along with observed discharges (red dots)

Bridge geometries like pier size, shape, pile cap size, etc. are also needed in scour estimation. These were collected from the Executive Engineer's Office of LGED, Manikganj, as well as from the Navana Construction Company Ltd., Manikganj (Appendix D).

3.5 Formulae and Model Used for Scour Depth Prediction

Estimation of scour depth involved a number of empirical formulae. The formulae by Breusers, Laursen, Neill, Jain and Fischer, Chitale, Melville, Lacey and FHWA had been considered. Though Lacey's formula has some limitations, the formula is still in wide use in India (Jagadeesh, 2013). The equation of Melville and Coleman was used for their validation over a wide range of flow conditions, sediment sizes and pier dimensions (Coleman, 2005). Sheppard introduced the concept of effective pier by converting pier width, pile cap width and pile group width into an effective pier width. This latter equation also incorporates the ratios of scour depth to effective pier width and the effective pier width to median sediment size (FDOT, 2005).

Apart from these formulae, the HEC-RAS model was also used to estimate the scour depth. The model allows its users to compute contraction and local scours. Both live-bed and clear-water scours can be estimated with the model. Various hydraulic parameters, such as the percentage of flow, flow area, wetted perimeter, conveyance, hydraulic depth and flow velocity, inside and upstream of the bridge as well as at the approach section are used as inputs in the scour computation. Another parameter, discharge is also used as an input into the model. For this purpose, a rating curve based on the BWDB data was developed.

3.6 Consideration of Pier Geometry: Complex Pier

Local scour depths were estimated by using the different empirical formulae. Also, the HEC-RAS model simulated local scour depths were also obtained. Estimation of local scour depths by using different empirical formulae considered both simple and complex geometries of the bridge pier. In model simulation, only an obstruction of complex type was considered. Simple piers consider a simple structure of uniform shape from substructure (foundation) to superstructure. Complex piers consider a complex pier type with up to three components: pier, pile cap and piles/pile group (Figure 3.7).

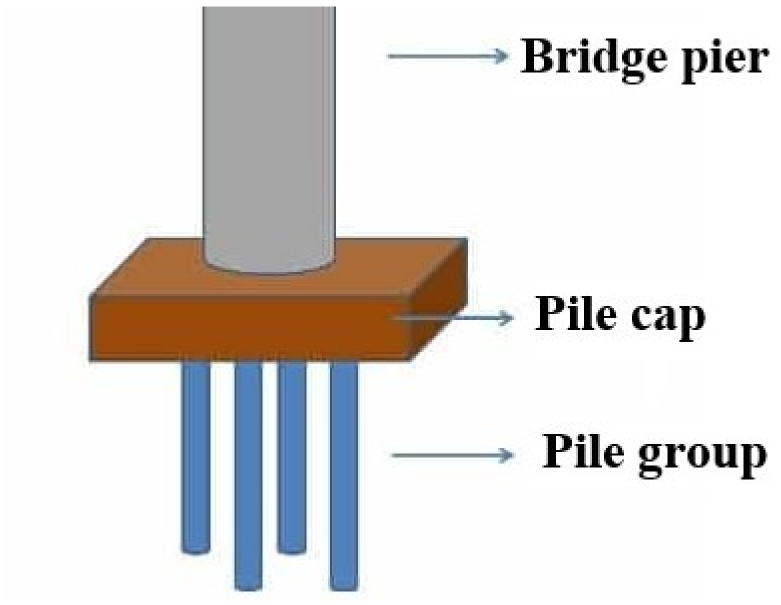

Figure 3.7: Components of a complex pier

There are some methods for predicting the equilibrium (maximum) local scour depth at complex piers. Melville and Coleman (2000) predicted local scour depth by using different effective pier diameter considering the position of pier, pile cap and pile group. This study used the method of Melville and Coleman (2000) which was discussed earlier in Chapter two. Richardson and Davis (2001) estimated the local scour depth by considering each component of the complex pier (pier, pile cap and piles/pile group). This method involved the superposition of the scour components. Coleman (2005) combined the effect of all components of the complex pier. He introduced four cases considering the position of pier, pile cap and pile group. Coleman (2005) and Melville and Coleman (2000) used the same formula for effective pier diameter (see equation 2.1). Sheppard (2005) introduced the representation of a complex pier which can be replaced by a single circular pile (penetrating the surface i.e. river bed) with an "effective diameter" denoted by b_e. The effective diameter (b_e) depends on the shape, size and location of the component and its orientation relative to the flow. The magnitude of the effective diameter (b_e) creates the same scour holes as with a complex pier for the same sediment and flow conditions. The total b_e for the structure can be approximated by the sum of the effective diameters of the components making up the structure (Figure 3.8). That is,

$$b_e \equiv b_p + b_{pc} + b_{pg}$$

where,

b_e = effective diameter of the complex pier

b_p = effective diameter of the bridge pier

b_{pc} = effective diameter of the pile cap

b_{pg} = effective diameter of the pile group

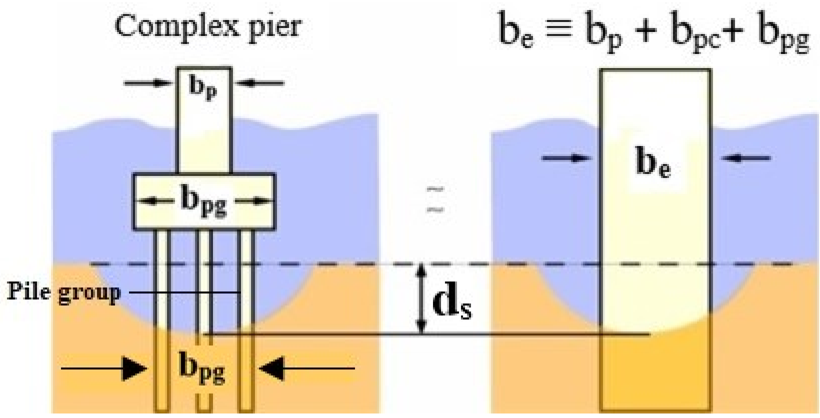

Figure 3.8: Definition sketch for total effective diameter for complex pier.

Arneson et al. (2012) developed a conceptual procedure of local scour prediction for complex pier. In this method, scour depth is determined for each components of the complex pier, separately. Scour depth found from the complex pier components (pier, pile cap, pile group) is then combined to estimate the total scour depth. This method is called “superposition of the scour components”.

Sheppard and Renna (2010) developed a method of superposition of scour components. In this method, separated local scour depth for each component making complex pier is estimated which ends up with their combination to get the total scour.

CHAPTER FOUR

FINDINGS ON PIER SCOURS AND DISCUSSIONS

4.1 Scour Depths from Field Measurements

The maximum pier scour levels obtained from field measurements during different times of the pre-monsoon, monsoon and post-monsoon seasons in the year of 2017 are given in Table 4.1. These maximum scours were obtained at pier no. 4 from the left bank. The thalweg of the river was close to this pier. It is seen from the table that the observed pier scour levels varied from -3.12 m PWD in May to -4.98 m PWD in August to -2.89 m PWD in October, 2017. Thus, the highest scour level was found in August when the flood was in its peak. It is also seen from the table that there were some scours throughout the observation period. This could be due to the fact that the velocity of the river in the observation period (May – October) was higher than the critical velocity required for sediment movement. This also indicates that the live-bed scour generally occurred during the 2017 flood in the Dhaleshwari River.

Table 4.1: Scour depths obtained from field measurements in 2017

Date	Scour level (m PWD)
20 May, 2017	-3.12
24 June, 2017	-3.76
04 August, 2017	-4.23
19 August, 2017	-4.98
16 October, 2017	-2.89

4.2 Field Observed Temporal Variation of Local Scour

From the primary and secondary bathymetric data, temporal and spatial variation of local scour was found. Figure 4.1 represents the temporal variation at bridge site (around piers) from the pre-construction period to the first flooding year of the bridge. Pier 1, pier 2, pier 3, pier 4, pier 5, pier 6, pier 7 and pier 8 are located at a distance of 33, 66, 99, 132, 165, 198, 231 and 264 m, respectively, from the left abutment. The left abutment and the right abutment are located at 0 and 297 m, respectively. From the figure it is seen that the occurrence of maximum local scour was in the main channel rather than on the flood plain. The maximum scour was found around pier 4 (at 132 m). Though piers 3, 4, 5 and 6 are constructed in the main channel, pier 4 experienced the maximum thalweg shifting. Initial water level was maximum around pier 3, but the maximum local scour could not happen around this pier because of the presence of flood plain in the left side. The temporal variation shows that the local scour depth changes with change in water level, discharge, etc. Thus, it is established that only pier width could not cause local scour, rather it required other associated parameters like water level, discharge, velocity, etc. Before construction of the bridge, there was an equilibrium condition in bed level. After construction of the bridge, a natural process of achieving a new equilibrium was started. In the course of achieving the new equilibrium, there was an increasing rate of local scour first. The amount of local scour reached a maximum value (threshold peak) for almost maximum (peak) discharge.

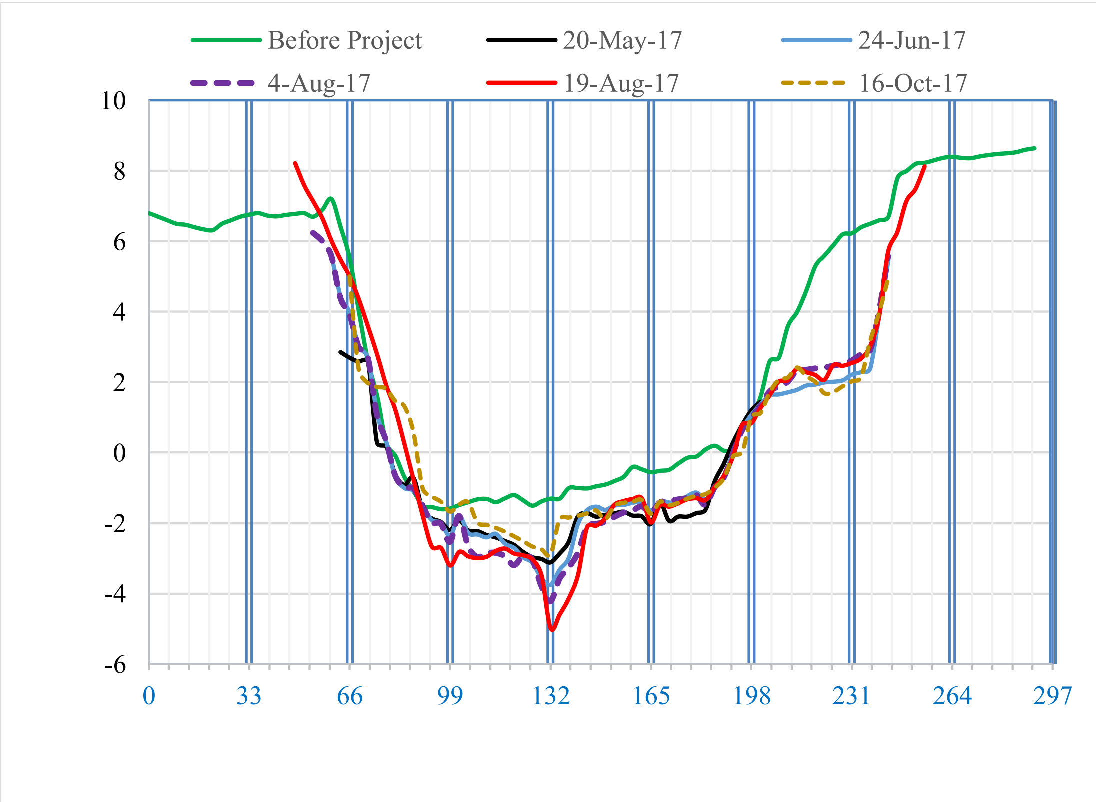

Figure 4.1: Scour holes around bridge piers (piers are located at distances of 33, 66, 99, 132, 165, 198, 231 and 264 m from the left abutment)

4.3 Spatial Variation of Local Scour Around Bridge Piers

From the field observed local scour values, it was seen that maximum local scour was found for the highest discharge. Thus the spatial variation of observed local scour around bridge piers was analyzed for the event date (19th of August) of the highest flood only. However, the local scour distribution is given for pier numbers 3, 4, 5 and 6 because they were constructed in the main river (Figure 4.2).

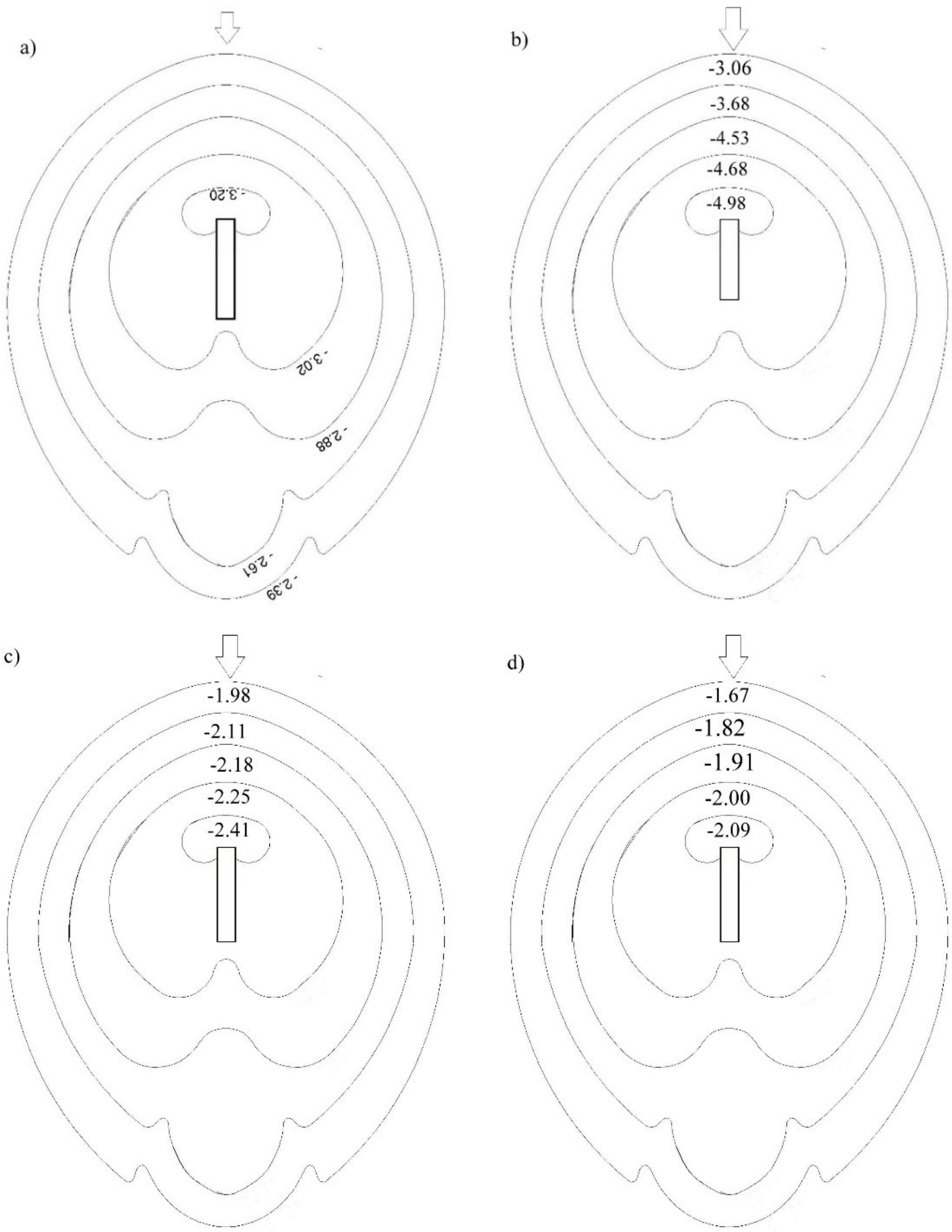

Figure 4.2: Spatial variation of local scour around selected piers of the Dhaleswari Bridge(a) Pier 3, b) Pier 4, c) Pier 5 and d) Pier 6).

From the figure it is seen that the amount of local scour in the upstream of the piers is greater than that in the downstream. The extent of the scour hole created downstream is wider than that in the upstream. In fact, the distance from the edge of the pier in the upstream to the river bed level is almost half of the same in the downstream.

4.4 Scour Depths from Empirical Formulae

Though there are a large number of empirical formulae for scour depth estimation, this study selected the formulae widely used in Bangladesh and estimated the scour depths by using these formulae. The selection is based on existing engineering practices and guidelines, and some theoretical contexts discussed earlier in the literature review part of this thesis (see Chapter two). The local scour depths estimated by using different empirical formulae during the feasibility study by IWFM (2013) is given in Table 4.2. The study of IWFM (2013) considered a pier diameter of 1.0 m.

Table 4.2: Estimated local scour depth in the Dhaleshwari River near the bridge by IWFM (2013) (considering pier width, b = 1.0 m)

Equation Name	Scour level (m PWD)
Breusers (1965)	-3.36
Laursen (1963)	-6.00
Neill (1987)	-3.46
Jain and Fischer (1980)	-7.57
Chitale (1988)	-4.46
Melville (1997)	-4.12

However, the actual pier diameter was found to be 1.5 m. So, a modified table replacing the original table was prepared considering a pier width of 1.5 m (Table 4.3).

Table 4.3: Estimated local scour depth in the Dhaleshwari River near the bridge according to IWFM (2013) (considering pier width, b = 1.5 m)

Equation Name	Scour level (m PWD)
Breusers (1965)	-4.06
Laursen (1963)	-6.92
Neill (1987)	-4.21
Jain and Fischer (1980)	-8.84
Chitale (1988)	-5.71
Melville (1997)	-5.56

It is seen from the modified Table 4.3 that the estimated scour levels by IWFM varied from -4.06 m PWD based on Breusers' equation to -8.84 m PWD based on Jain and Fischer's equation. Among the equations, Breusers' equation is applicable for tidal rivers and hence it is out of context for the Dhaleshwari River which is non-tidal in nature at the bridge site. The formulations by Neill, Chitale and Melville are similar and the Chitale's equation uses the highest factor. Hence, this gives the highest scour among the three equations. The equations of Laursen and Jain and Fischer are applicable for clear water scours and these give higher values compared to other equations in the table.

The bridge was inaugurated in the month of October, 2016 and local pier scour depths were estimated by using the selected empirical formulae for the flood of the year 2017 which was the first flooding year for this bridge. Local pier scour depths were estimated considering two pier types: simple and complex. For the cases of simple pier, the actual width of the pier was considered. For complex pier conditions, an effective diameter (b_e), equivalent to a single pier diameter, was used as pier width (Melville and Coleman, 2000). The effective diameter was found by using a formula (see Chapter 2, equation 2.1) referred by Melville and Coleman (2000). The estimation of local scour depths was carried out by using the field data of each field visit. Data of five field visits reinforced the estimation of local scour depths using field data of the concerned dates. Local scour levels were estimated considering both simple pier (hereafter "SP" in the Table 4.4) and complex pier (hereafter "CP" in the Table 4.4). The simple pier consideration used actual pier width on field, b_p=1.50 m and complex pier consideration used effective pier width, b_e=5.41 m, 4.55 m, 4.41 m, 4.02 m and 4.71 m for 20 May, 24 June, 4 August, 19 August and 16 October, respectively. The details of scour computation by using different empirical formulae are furnished in Appendix E. The estimated values of local scour depths are summarized in Table 4.4. From the table it is seen that:

- the estimated scour levels for 20 May, 2017 varied from -3.48 m PWD based on FHWA method to -5.81 m PWD based on Jain and Fischer's equation (for simple pier) and from -5.31 m PWD based on Lacey to -15.49 m PWD based on Chitale's equation (for complex pier);

- the estimated scour levels for 24 June, 2017 varied from -3.98 m PWD based on FHWA method to -7.36 m PWD based on Jain and Fischer's equation (for simple pier) and from -6.12 m PWD based on FHWA method to -13.34 m PWD based on Chitale's equation (for complex pier);

- the estimated scour levels for 04 August, 2017 varied from -4.21 m PWD based on Neill's equation to -7.65 m PWD based on Jain and Fischer's equation (for simple pier) and from -7.46 m PWD based on FHWA method to -12.99 m PWD based on Chitale's equation (for complex pier);

- the estimated scour levels for 19 August, 2017 varied from -4.21 m PWD based on Neill's equation to -8.94 m PWD based on Lacey's equation (for simple pier) and from -7.80 m PWD based on FHWA method to -12.70 m PWD based on Jain and Fischer's equation (for complex pier);

- the estimated scour levels for 16 October, 2017 varied from -4.06 m PWD based on FHWA method to -7.06 m PWD based on Jain and Fischer's equation (for simple pier) and from -6.37 m PWD based on FHWA method to -13.74 m PWD based on Chitale's equation (for complex pier).

Among the equations, Breusers' equation is applicable for tidal rivers and hence it is out of context for the Dhaleshwari River which is non-tidal in nature in the study reach. The formulations by Neill, Chitale and Melville (Melville's equation consider three different conditions but d_s/b_p=2.4 consideration was applicable for simple pier and complex pier (for peak flow condition only) consideration of this thesis) are similar, and the Chitale's equation uses the highest factor. Hence, this gives the highest scour among the three equations. The equations of Laursen and Jain and Fischer are applicable for clear water scours. The estimated scour levels by using Laursen's equation varied from -4.73 m PWD for 20 May, 2017 to -6.69 m PWD for 19 August, 2017 (for simple pier) and from -7.22 m PWD for 20 May, 2017 to -9.70 m PWD for 19 August, 2017 (for complex pier). The estimated scour levels by using Jain and Fischer's equation varied from -5.81 m PWD for 20 May, 2017 to -8.52 m PWD for 19 August, 2017 (for simple pier) and from -9.26 m PWD for 20 May, 2017 to -12.70 m PWD for 19 August, 2017 (for complex pier).

From the estimated values of local scour depths (summarized in Table 4.4), it can also be seen that Breusers' equation (though not applicable for this study because of its applicability for tidal rivers), Neill's equation, Chitale's equation, and Melville's equation predicted values were constant for all the collected data. That means the rise and fall in the stage of the river near the bridge site did not affect the estimated local scour values by using these formulae throughout the data collection period (pre-monsoon, monsoon and post-monsoon) of the year 2017. This happened because the formulations of these formulae were exclusive of the consideration of hydraulic depth, h. The only exception of this phenomena was the estimated values by using Melville's equation. Formulation of Melville's equation (equation 2.8) consider three classes of bridge piers (Melville, 1997). They are: narrow piers ($b_p/h < 0.7$), intermediate width piers ($0.7 < b_p/h < 5$) and wide piers ($b_p/h > 5$). Since the study bridge was with narrow piers for simple pier, and also for complex pier during peak flow condition, and the formulation of Melville for narrow piers considered the relationship of scour depth to bridge pier width to a constant value (i.e. 2.4) and thus gave the same values for all the data sets.

Table 4.4: Estimated local scour level in the Dhaleshwari River near the bridge site on 20th of May, 24th of June, 4th of August, 19th of August and 16th of October of the year 2017.

Equation Name	Scour level (m PWD)									
	20 May		24 June		4 August		19 August		16 October	
	SP	CP	SP	CP	SP	CP	SP	CP	SP	CP
Breusers (1965)	-4.06	-9.53	-4.06	-8.33	-4.06	-8.13	-4.06	-7.59	-4.06	-8.55
Laursen (1963)	-4.73	-7.22	-5.85	-8.73	-6.06	-8.99	-6.69	-9.70	-5.63	-8.47
Neill (1987)	-4.21	-10.08	-4.21	-8.79	-4.21	-8.58	-4.21	-7.99	-4.21	-9.03
Jain and Fischer (1980)	-5.81	-9.26	-7.36	-11.36	-7.65	-11.72	-8.52	-12.70	-7.06	-11.00
Chitale (1988)	-5.71	-15.49	-5.71	-13.34	-5.71	-12.99	-5.71	-12.01	-5.71	-13.74
Melville (1997)	-5.56	-9.81	-5.56	-12.06	-5.56	-12.45	-5.56	-11.61	-5.56	-11.68
Lacey (1939)	-5.31	-5.31	-7.28	-7.28	-7.64	-7.64	-8.94	-8.94	-6.84	-6.84
Modified Lacey (2003)	-5.35	-7.94	-7.01	-10.02	-7.36	-10.40	-8.44	-11.48	-6.67	-9.65
FHWA Method (2012)	-3.48	-5.46	-3.98	-6.12	-4.69	-7.46	-5.03	-7.80	-4.06	-6.37

**Note: In the table "SP" refers to simple pier and "CP" refers to complex pier.

4.5 Scour Depths from the HEC-RAS Model

Scours at piers were also computed using the HEC-RAS model. The computation within HEC-RAS is based upon the methods outlined in Hydraulic Engineering Circular No. 18 (FHWA, 2001). The model allows its users to compute contraction and local scours. Both live-bed and clear-water contraction scours can be estimated with the model. The live-bed contraction scour is estimated based on a modified Laursen's (1960) equation, and the clear-water contraction scour is estimated based on an equation of Laursen (1963). The computation of local scour at piers under both live-bed and clear-water conditions is based on either the Colorado State University equation (Richardson et al., 1990) or the Froehlich (1991) equation. Various hydraulic parameters, such as the percentage of flow, flow area, wetted perimeter, conveyance, hydraulic depth and flow velocity, inside and upstream of the bridge as well as at the approach section are used as inputs in the scour computation. In addition, the mean size fraction of the bed material (d_{50}) is required in estimating contraction scour. Pier shapes, bed condition (clear-water, plane bed and anti-dunes, and small, medium and large dunes), the angle of attack of flow hitting the piers, etc., are also the required inputs. In most cases, the model automatically selects the critical condition (live-bed or clear-water) and hence the appropriate equation for scour computation.

In the final report of IWFM, main analysis was conducted considering pier width, b_p = 1.0 m. Both the field measurements and the structural design drawings of the bridge established that, the actual pier width (constructed pier width) is 1.5 m. The structural drawings and related field evidence are included in Appendix E of this thesis. So, to conduct a feasible study, equivalency in between "without project" and "with project" values are required. Field data (water level, hydraulic gradient, discharge, etc.) collected during the pre-monsoon, monsoon and post-monsoon periods were used in the HEC-RAS model. For February, 2013, the maximum local scours are found to be 4.95 m and 5.21 m below the initial bed level for pier diameter of 1.0 m and 1.5 m, respectively (Figures 4.3 and 4.4).

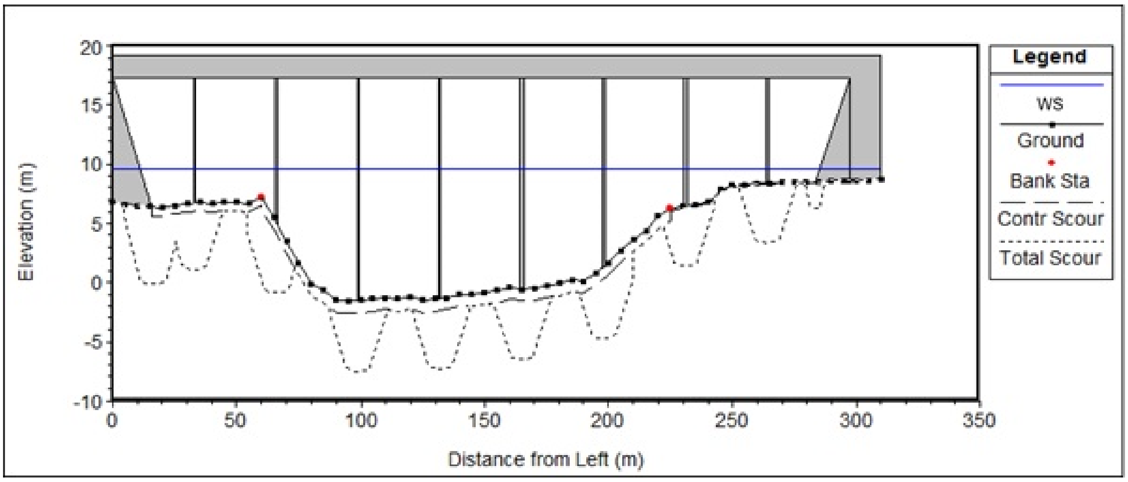

Figure 4.3: Variation of pier scour at the Manikganj Sadar Bridge under design discharge condition with the HEC-RAS model [the maximum local scour is found to be 4.95 m, with 1.0 m pier width]

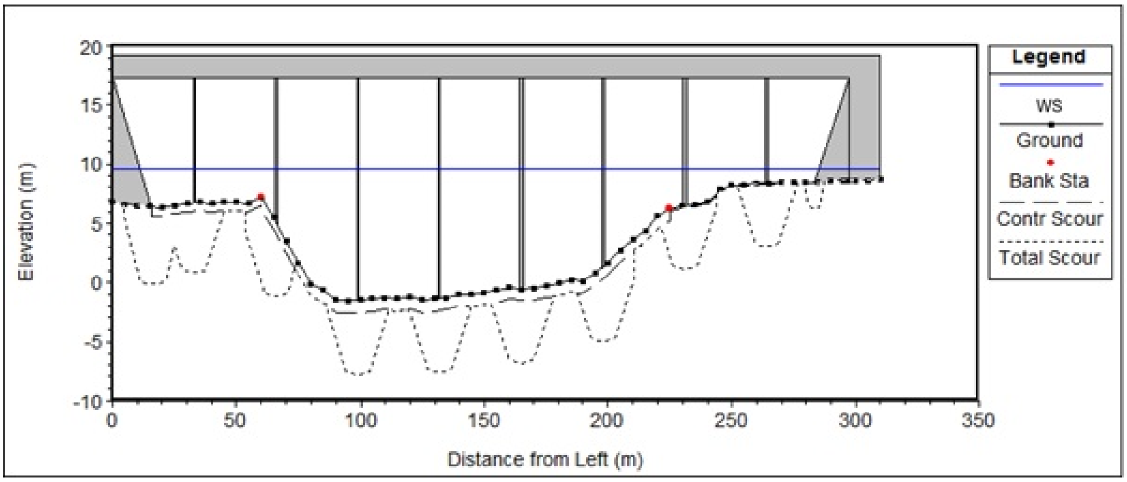

Figure 4.4: Variation of pier scour at the Manikganj Sadar Bridge under design discharge condition with the HEC-RAS model [the maximum local scour is found to be 5.21 m, with 1.5 m pier width]

For the estimation of pier scours for 2017, the actual observed discharges on the particular days (Table 4.5) were used for both 1.0 m and 1.5 m pier diameters. Again, for May 20, June 24, August 4, August 19 and October 16, 2017, the values of maximum local scour below the initial bed level are found to be 1.46 m (considering Q=250 m^3/s) (Figure 4.5), 3.63 m (considering Q=1000 m^3/s) (Figure 4.6), 4.47 m (considering Q=1216 m^3/s) (Figure 4.7), 5.44 m (considering Q=2267 m^3/s) (Figure 4.8) and 3.38 m (considering Q=772 m^3/s) (Figure 4.9), respectively.

Table 4.5: Estimated discharge at bridge site for each bathymetric survey data

Date	Water Level at Taraghat (m PWD)	Water Level at Bridge Site (m PWD)	Slope of Water Surface near Bridge (cm/km)	Estimated Discharge at Bridge (m^3/s)
May 20, 2017	3.08	2.85	4.6	250
June 24, 2017	5.94	5.61	6.87	1000
August 04, 2017	6.57	6.24	6.87	1216
August 19, 2017	9.22	8.29	20	2267
October 16, 2017	5.23	5.01	4.6	1610

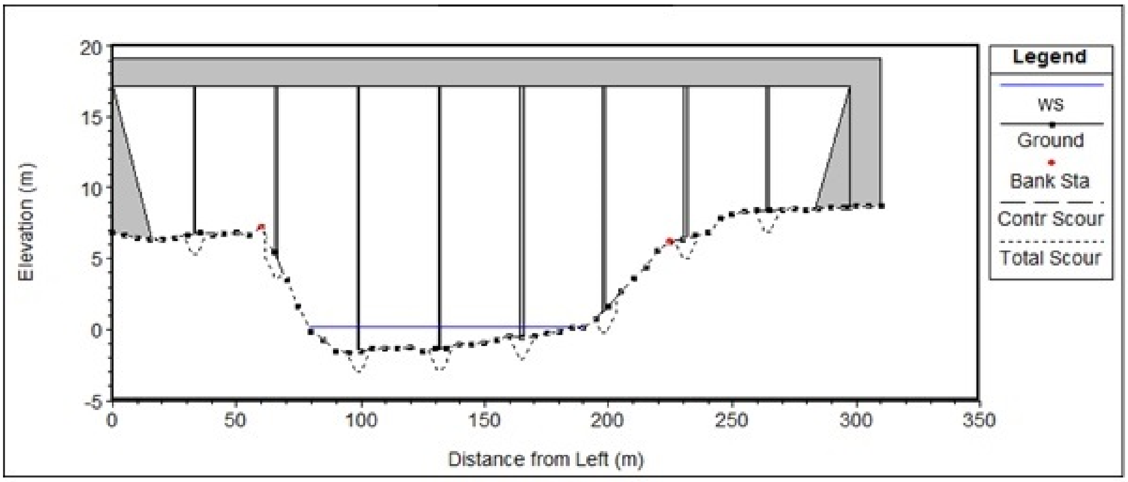

Figure 4.5: Variation of pier scour at the Manikganj Sadar Bridge under design condition with HEC-RAS model [the maximum local scour is found to be 1.46 m, with 1.5 m pier width, 20 May]

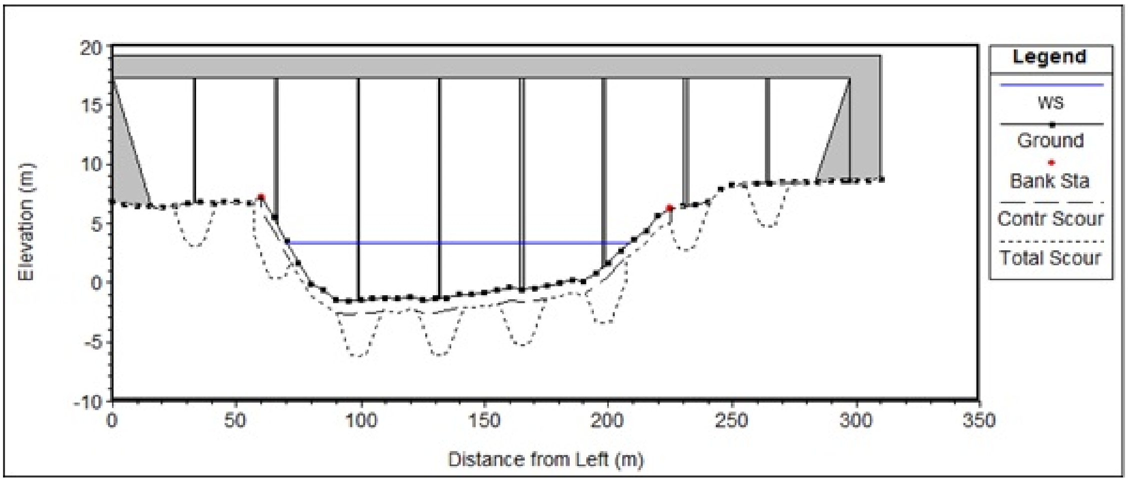

Figure 4.6: Variation of pier scour at the Manikganj Sadar Bridge under design condition with HEC-RAS model [the maximum local scour is found to be 3.63 m, with 1.5 m pier width, 24 June]

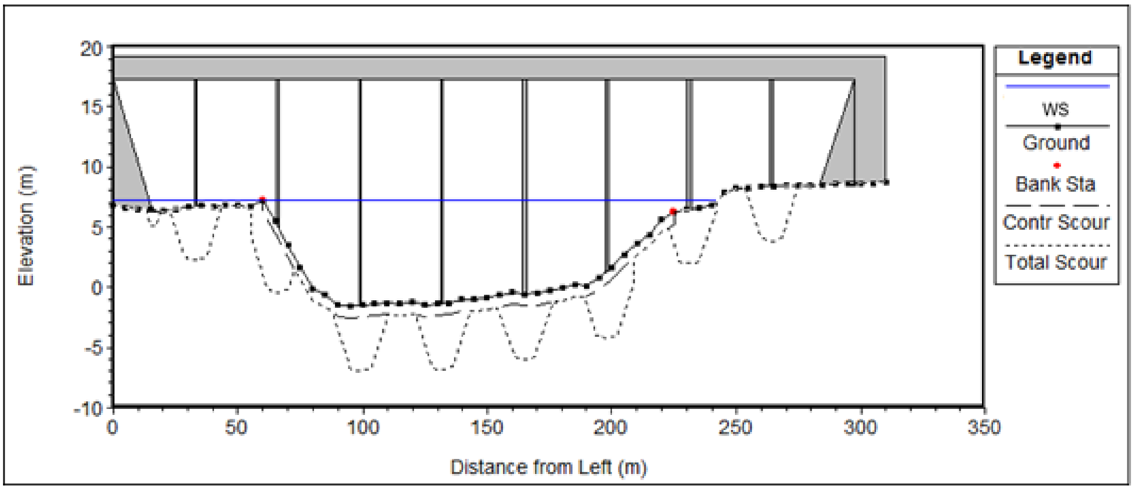

Figure 4.7: Variation of pier scour at the Manikganj Sadar Bridge under design condition with HEC-RAS model [the maximum local scour is found to be 4.47 m, with 1.5 m pier width, 4 August]

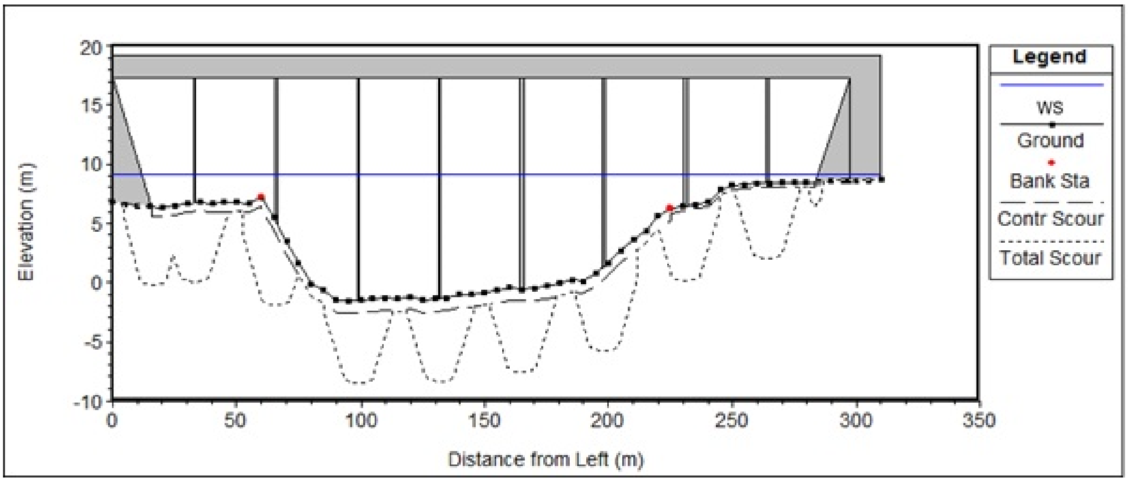

Figure 4.8: Variation of pier scour at the Manikganj Sadar Bridge under design condition with HEC-RAS model [the maximum local scour is found to be 5.44 m, with 1.5 m pier width, 19 August]

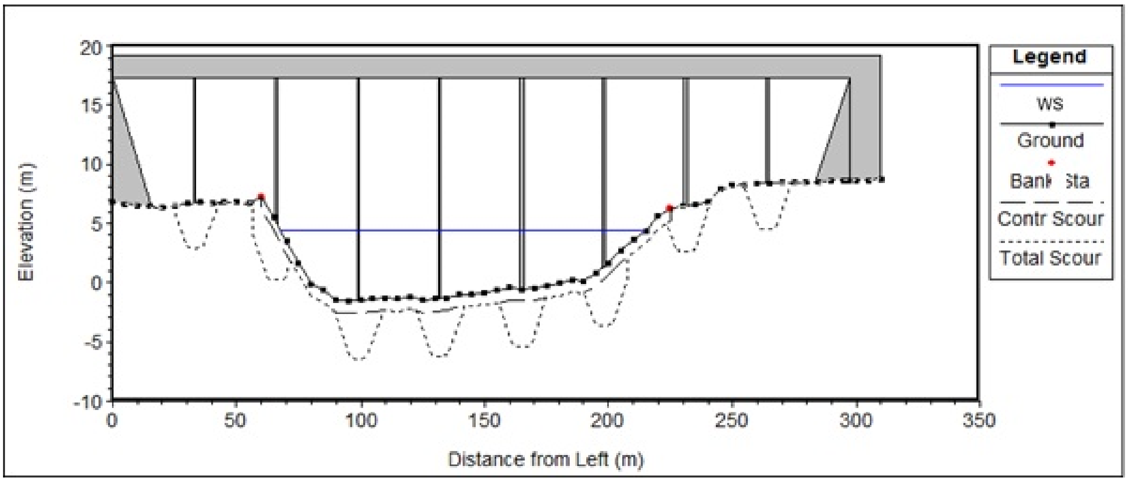

Figure 4.9: Variation of pier scour at the Manikganj Sadar Bridge under design condition with HEC-RAS model [the maximum local scour is found to be 3.38 m, with 1.5 m pier width, 16 October]

Comparative analysis of observed and model simulated values are shown in later part of this thesis. In this regard, HEC-RAS model simulated (maximum) values of local scours are summarized in Table 4.6.

Table 4.6: Local scour estimated (maximum) by using the HEC-RAS model

Date	Pier width (m)	Maximum local scour from initial bed level (m)	Estimated Discharge at Bridge site (m^3/s)
February 2013	1.0	4.95	2040
February 2013	1.5	5.21	2040
May 20, 2017	1.5	1.46	250
June 24, 2017	1.5	3.63	1000
August 04, 2017	1.5	4.47	1216
August 19, 2017	1.5	5.44	2267
October 16, 2017	1.5	3.88	772

4.6 Comparison of Field Observed Local Scours with Estimated Local Scours for Different Discharges

Field observed local scour depths along with estimated and model simulated values of local scours are plotted against time (in the primary axis). The values of estimated discharges (from the rating curve) are put in the secondary axis of the same graph (Figures 4.10 and 4.11). Figure 4.10 and Figure 4.11 represent the comparative analysis for simple pier and complex pier, respectively.

All the predicted values of local scour are higher than the observed values. There is an overlapping of values of Jain and Fischer's equation and Lacey's equation. For the pre-monsoon data (May 20), the local scour by using Jain and Fischer's equation is more than the Lacey's equation. But after that, the predicted values from Jain and Fischer are smaller compared with Lacey's and closer to the field observed values. After the peak flood, Lacey's equation given value is smaller than Jain and Fischer's equation. From the formulation of the two formulae, we know that Lacey's equation estimates the value of local scour considering discharge. The estimated value by using this formula will be lower for smaller value of discharge and higher for bigger value of discharge which is already seen from the previous analysis. On the other hand, Jain and Fischer's equation incorporates the dominant factor b_p/h in local scour pier estimation and thus does not too much overestimate the scour value for peak flow. Modified Lacey's equation better performs than that of Lacey's equation except for May 20. Here again, the incorporation of discharge and dominant factor b_p/h is the key. May 20 involves low discharge and gives lower values (Figure 4.10). The predicted values considering complex pier are much higher than that of simple pier (Figure 4.11). Though all the estimated values of local scour were changed following the same sequence of field observed local scour (for simple pier consideration), the estimated values of some equations (Breusers, Neill, Melville, Chitale) were not followed by the same sequence. Local scour estimation by using Breusers', Neill's and Chitale's equations depends only on the pier width. The estimated values were thus increased or decreased for increase or decrease in pier width. Estimated values of effective pier width were 5.41 m, 4.55 m, 4.41 m, 4.02 m and 4.71 m for May 20, June 24, August 4, August 19 and October 16, respectively. As a result, estimated maximum local scours by using these formulae were maximum for May 20; in this date the discharge was minimum. Since the minimum pier width was found on August 19, the minimum estimated value was also found on that day. Melville's equation was following the field observed trend except August 19 (peak flood). For this date, the value of dominant factor $b_p/h = 0.48$; the value of $b_p/h<0.7$ means narrow pier. The local scour estimation by Melville's equation provides minimum local scour. So, in the case of Melville's equation both minimum value of effective pier width and lowest factor multiplication caused this abrupt change in local scour. Original Lacey's equation estimated values were closer than that of modified Lacey's equation which justified the applicability of modified Lacey's equation for simple pier. It can be noted that the modified Lacey's equation was developed considering simple pier. HEC-RAS model was used only in complex pier consideration because bridge pier was input as whole in the simulation process. For May 20, HEC-RAS model simulated value was lower than the field observed local scour. This is because the HEC-RAS model was basically set for simulating peak hydraulic condition. Hence, the lower velocity and water depth were not adequately captured in the model.

From the analysis of figures, it can be easily understood that local scour changes with different hydraulic parameters like water level, pier width, discharge, velocity, etc.

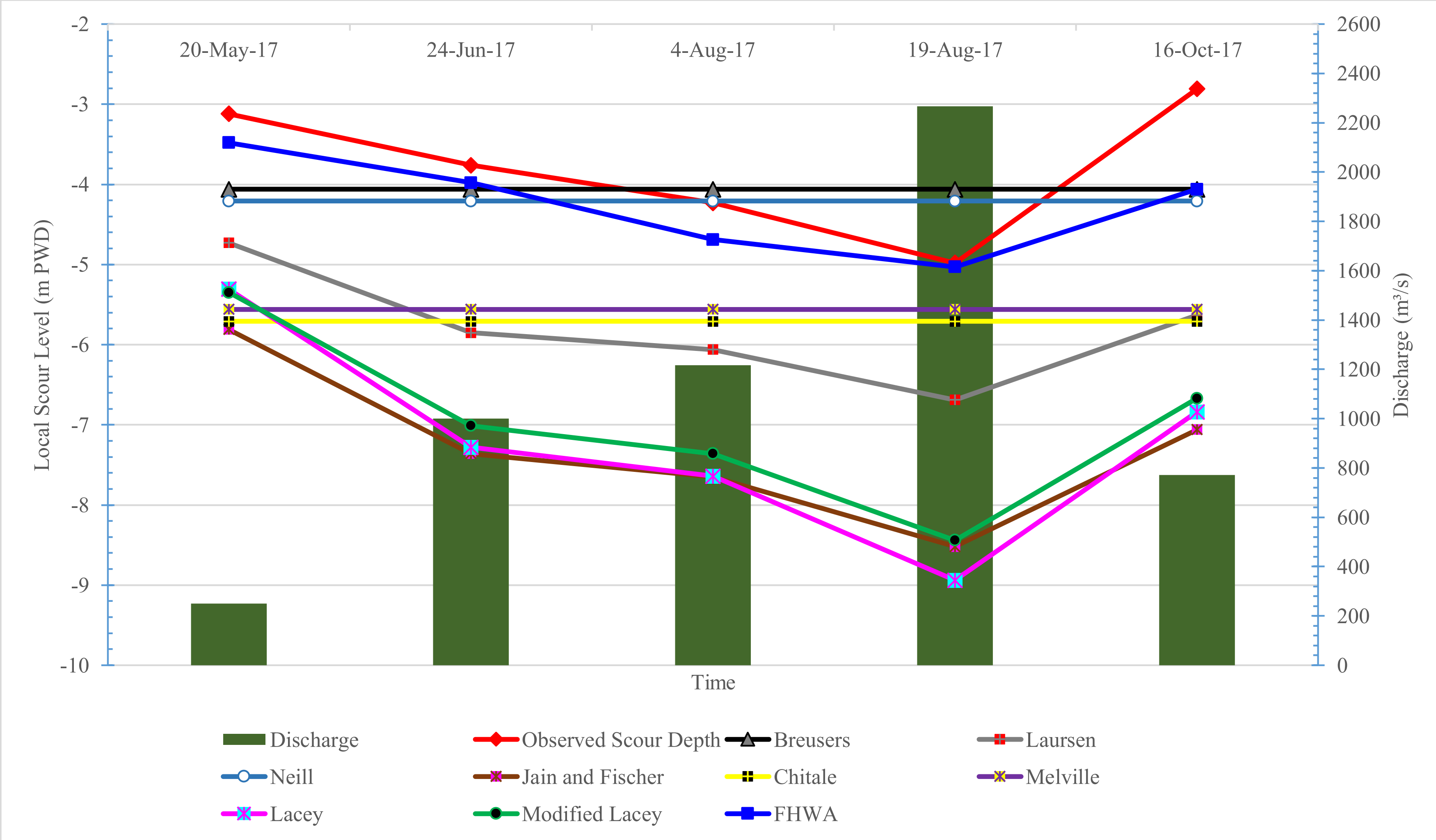

Figure 4.10: Comparison of field observed and estimated local scours considering simple pier.

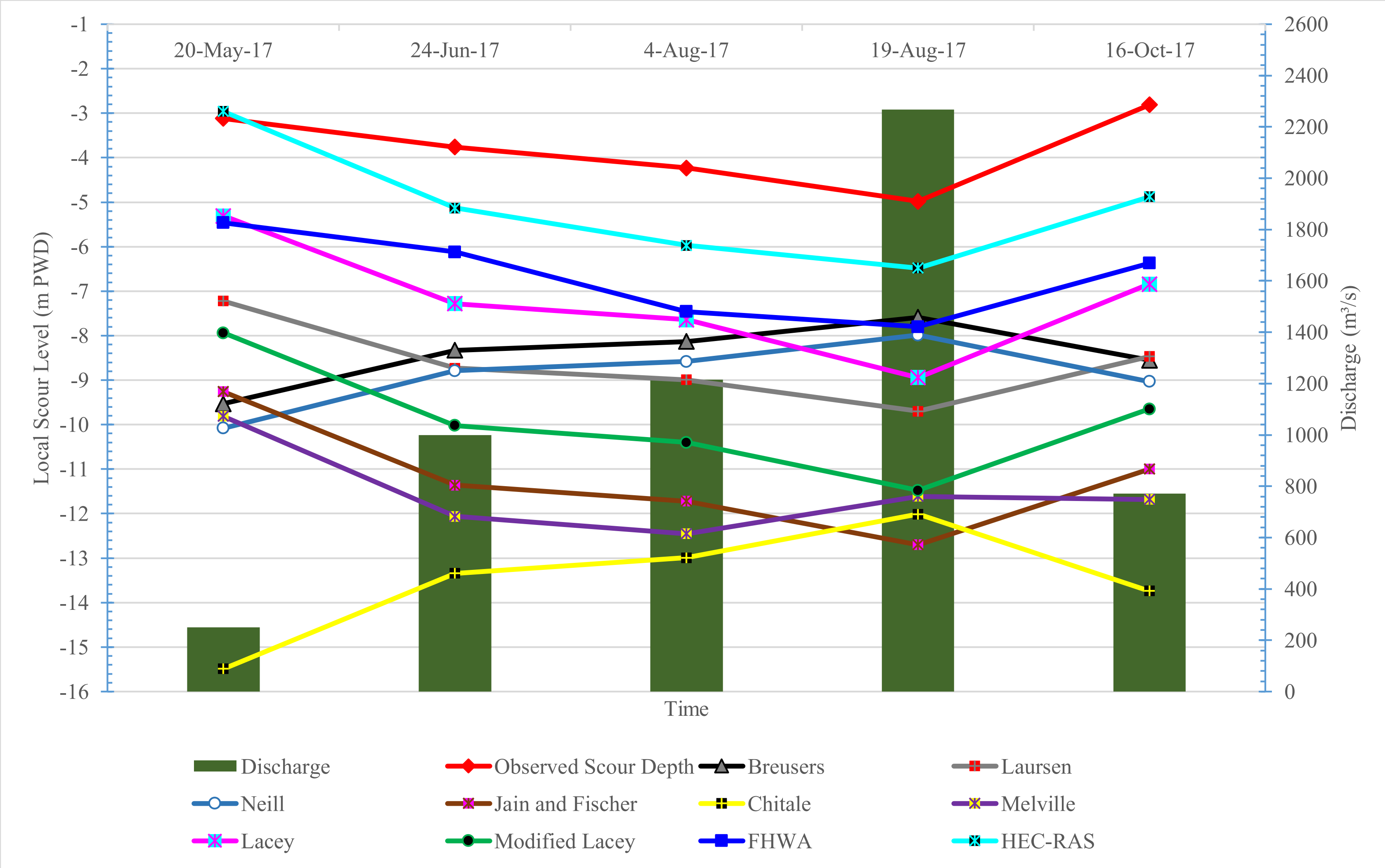

Figure 4.11: Comparison of field observed and estimated/simulated local scours considering complex pier.

4.7 Comparison of Empirical and Model Estimated Scours with Field Data

A comparison of the field observed scour depths with that of the estimated scour depths from the selected empirical formulae is made in Tables 4.7 and 4.8. This comparison of the field observed scour depth with the estimated scour depth for every set of scour data (the Breusers' equation given value is not considered for the comparison because of its inapplicability for the non-tidal river in the study bridge) is discussed below:

- For 20 May, 2017, the field observed scour level was -3.12 m PWD, whereas the estimated closest value for simple pier was -3.48 m PWD from the FHWA method and the next closest value was -4.21 m PWD from Neill's equation. Using complex pier, the closest value was -5.31 m PWD from Lacey's equation and the next closest value was -5.46 m PWD by the FHWA method. The simple pier consideration gave scour value which is closer than that of the complex pier consideration.

- For 24 June, 2017, the field observed scour level was -3.76 m PWD, whereas the estimated closest value for simple pier was -3.98 m PWD by the FHWA method and the next closest value was -4.21 m PWD by Neill's equation. Using complex pier, the closest value was -6.12 m PWD by the FHWA method and the next closest value was -7.28 m PWD by Lacey's equation. Here again, the simple pier consideration gave scour value which is closer than that of complex pier consideration.

- For 4 August, 2017, the field observed scour level was -4.23 m PWD, whereas the estimated closest value for simple pier was -4.21 m PWD by Neill's equation and the next closest estimated value was -4.69 m PWD by FHWA method. However, the Neill's equation underestimated the potential scour. Using complex pier, the closest value was -7.46 m PWD by the FHWA method and the next closest value was -7.64 m PWD by Lacey's equation. Here again, the simple pier consideration gave scour value closer than that of complex pier consideration.

- For 19 August, 2017, the field observed value was -4.98 m PWD, whereas the estimated closest value for simple pier was -5.03 m PWD by the FHWA method and the next closest estimated value was -5.56 m PWD by Melville's equation. However, the Neill's equation underestimated the potential scour. Using complex pier, the closest value was -7.80 m PWD by FHWA method and the next closest estimated value was -7.99 m PWD by Neill's equation. Here again, the simple pier consideration gave scour value closer than that of complex pier consideration.

- For 16 October, 2017, the field observed value was -2.89 m PWD, whereas the estimated closest value for simple pier was -4.06 m PWD by the FHWA method and the next closest estimated value was -4.21 m PWD by Neill's equation. Using complex pier, the closest value was -6.37 m PWD by the FHWA method and the next closest value was -6.84 m PWD by Lacey's equation. Here also, the simple pier consideration gave scour value which is closer than that of complex pier consideration.

The comparison further reveals:

- One empirical formula (FHWA method) gave closer values of scour to the field observed values. The FHWA method gave closer values consistently for all the observed values. The formulation of FHWA method incorporated hydraulic depth, more specially, b_p/h, the most dominant factor for local scour estimation. This formula also incorporated other parameters like angle of attack, bed conditions, armouring of bed material size, etc. and thus predicted closer values.
- In addition, the estimated value of local scour by modified Lacey was higher than that of the original Lacey for May 20. In this date, the discharge was minimum (250 m^3/s) of all the event dates and original Lacey's estimated value was subsequently minimum. This discharge is almost one-fourth of the design and observed peak discharges. In other dates, the estimated scour value by modified Lacey was lower than the estimated scour value by original Lacey. The latter result shows the acceptability of modified Lacey's equation to estimate local scour for simple pier.

- Two empirical formulae (FHWA method and Lacey's equation) gave closer value of scour to the field observed value. Here again, FHWA method gave closer values of local scour to that of field observed values. Since, this method considered many hydraulic parameters including the most dominant b_p/h so it gave better result for both simple pier and complex pier considerations. Lacey's formula also gave closer values to that of the observed values. Lacey's formula estimated local scour is proportional to the discharge of the river reach. That is why, the estimated values were larger than that of field observed values, because only the discharge is not responsible for causing local scour. Neill's equation estimated values is closer to that of the field observed values for peak flow, but not compatible for the simple pier consideration because it under-estimated the values of local scour for simple pier in some cases.

The above comparison of estimated scours with the observed scours indicates that the FHWA equation provides scour depths which are more or less close to the values measured in the field.

Table 4.7: Comparison of observed scour with estimated simple pier scour from empirical formulae

Date	Observed Scour Level	Breusers	Laursen	Neill	Jain and Fischer	Chitale	Melville	Lacey	Modified Lacey	FHWA
20 May	-3.12	-4.06	-4.73	-4.21	-5.81	-5.71	-5.56	-5.31	-5.35	-3.48
24 Jun	-3.76	-4.06	-5.85	-4.21	-7.36	-5.71	-5.56	-7.28	-7.01	-3.98
4 Aug	-4.23	-4.06	-6.06	-4.21	-7.65	-5.71	-5.56	-7.64	-7.36	-4.69
19 Aug	-4.98	-4.06	-6.69	-4.21	-8.52	-5.71	-5.56	-8.94	-8.44	-5.03
16 Oct	-2.89	-4.06	-5.63	-4.21	-7.06	-5.71	-5.56	-6.84	-6.67	-4.06

Table 4.8: Comparison of observed scour with estimated complex pier scour from empirical formulae

Date	Observed Scour Level	Breusers	Laursen	Neill	Jain and Fischer	Chitale	Melville	Lacey	Modified Lacey	FHWA
20 May	-3.12	-9.53	-7.22	-10.08	-9.26	-15.49	-9.81	-5.31	-7.94	-5.46
24 Jun	-3.76	-8.33	-8.73	-8.79	-11.36	-13.34	-12.06	-7.28	-10.02	-6.12
4 Aug	-4.23	-8.13	-8.99	-8.58	-11.72	-12.99	-12.45	-7.64	-10.40	-7.46
19 Aug	-4.98	-7.59	-9.70	-7.99	-12.70	-12.01	-11.61	-8.94	-11.48	-7.80
16 Oct	-2.89	-8.55	-8.47	-9.03	-11.00	-13.74	-11.68	-6.84	-9.65	-6.37

*Breusers' equation is not applicable for non-tidal river. So, the values by Breusers' equation is excluded from the comparison in the main text.

A comparison of the HEC-RAS model simulated scour with the observed scour is given in Table 4.9. It is seen from the table that the HEC-RAS model simulated pier scour depth is more or less close to the observed value. Since the model uses the bathymetric data in conjunction with the other related hydraulic parameters, its scour estimate appears to be reasonable.

Table 4.9: Comparison of observed scour with estimated complex pier scour from HEC-RAS model

Date	Observed maximum scour (m PWD)	Model simulated maximum scour (m PWD)
20 May	-3.12	-2.96
24 June	-3.76	-5.13
04 August	-4.23	-5.97
19 August	-4.98	-6.48
16 October	-2.89	-4.88

Field observations show that bridge scour predicted by HEC-RAS generally overestimated the actual scour depth. One of the reasons is that scour prediction equations used in HEC-RAS was developed based on scaling up the laboratory results, which are difficult to satisfy both the hydraulic and hydrodynamic similitudes. For the data of pre-monsoon period, HEC-RAS model simulated value is lower than that of the field observed value. It could be due to fact that the model was not set up targeting low flow.

After analyzing the data, it could be ended up by saying that, both FHWA method and HEC-RAS model are applicable for the alluvial rivers of Bangladesh. In addition, FHWA method creates the opportunity of considering simple pier condition. The method is also suitable for the complex pier condition, as it bears the second close position. So, FHWA method could be used for both simple pier and complex pier conditions. Also, it gives an opportunity to design the bridge pier considering simple pier as the analysis with the FHWA method considering simple pier gave closer value to the field observed value for all the observed values. These findings are given in Table 4.10.

Table 4.10: Comparison of observed scour with estimated complex pier scour from FHWA method and HEC-RAS model

Date	Observed maximum scour (m PWD)	Scour from FHWA method (simple pier, m PWD)	Scour from FHWA method (complex pier, m PWD)	Model simulated maximum scour (m PWD)
20 May	-3.12	-3.46	-4.77	-2.96
24 June	-3.76	-3.97	-5.70	-5.13
04 August	-4.23	-4.70	-7.02	-5.97
19 August	-4.98	-5.09	-7.65	-6.48
16 October	-2.89	-4.19	-5.84	-4.88

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

A variety of empirical formulae is used to estimate local scour, one of the major causes of bridge failures. Likewise, practicing engineers in Bangladesh use some empirical formulae for estimation of local scour. Study of bridge reports also reveals the use of mathematical model in conjunction with empirical formulae for local scour prediction. In this study, a comparison of field-observed local scours with formulae-estimated and model-simulated scours is made to identify which formula/formulae/model provide closer value to the field-observed value. The specific conclusions drawn from the study are as follows:

- The maximum depth of local scour (-4.98 m PWD) observed during the 2017 flood at the Dhaleswari bridge was around 19 August when the flow was at its peak.

- The measured pier scours were generally found to be lower than the estimated scours from the empirical formulae and HEC-RAS model. Thus, the use of equations and model may result in an over-estimation of pier scour.

- The complex pier formulation gave higher scours than that of the simple pier formulation. In most cases, the over-prediction using complex pier was too much from the observed value.

- Lacey’s equation was found to give much higher value of local scour than that observed in the field.

- The modified Lacey’s equation was found to perform better than the original Lacey’s equation for simple piers. However, the original Lacey’s equation performed better than that of the modified Lacey’s equation for complex piers.

- The formulations by Neill, Chitale and Melville are similar and the Chitale's equation uses the highest scour factor. Thus, Chitale's equation always gave a greater scour than the field-observed scour.

- Neill's equation provided lower values of scour for two simple pier cases, out of five. For the both cases, the equation-estimated values were lower than the field-observed values. The local scour by Neill's equation was found by multiplying the pier width with pier shape factor and is independent of water level and discharge. Thus, under-estimation of local scours occurred by Neill's equation for neglecting the hydraulic condition.

- The FHWA method provided local scour which was closer to and consistent with the observed value (for both simple and complex piers). The FHWA method incorporates almost all the parameters which affect local scours. Since the FHWA method considers many factors in estimating local scour, the results are the best among the selected empirical formulae.

- Comparison of HEC-RAS model simulated bridge scour with field-observed scour reveals that the model generally performs well with complex piers. Thus the model can be applied in typical alluvial river setting in Bangladesh.

5.2 Recommendations

Based on the findings of the present study and the experiences gained throughout the course of the study, the following recommendations are made:

- The FHWA method was found to provide closer and consistent scour estimates of bridge piers. Since this method incorporates many parameters which affect local scour, it is more rationale to use it in local scour prediction. So, this method is recommended to be incorporated in bridge scour estimation study. Preference can also be given to the scour values estimated from this method over other methods.

- The HEC-RAS model also performed well in local scour simulation of complex bridge piers. So, this method can also be incorporated in bridge pier scour estimation and preference can also be given to its values. In addition, further study can be undertaken to compare its performance with that of other hydraulic models like Delft-3D, FLOW-3D, etc.

- The local scour estimation by some empirical formulae (Lacey's and modified Lacey's equations) and HEC-RAS model required discharge at the bridge point. The required discharge was found from the rating curve generated from the secondary data of BWDB at Taraghat in the upstream. If the actual discharge at the bridge site could be used as an input, then the result could be more reliable. So, further study can be done with actual discharge data.

- This study involved complex pier formulations by Melville and Coleman (2000). Incorporating complex pier formulations by Richardson and Davis (2001), Coleman (2005), FDOT (2005), HEC-18 (2012), etc., a further study of similar kind can be made to assess the suitability of the different complex pier formulations.

- This study was conducted for a bridge over the Dhaleswari River. Similar studies can be conducted for other rivers to further verify the conclusions of this study. Such studies may strengthen the conclusions of this study.

- The study was conducted for only one flood season. Only five sets of bathymetric data were collected and analyzed in this study. To draw firm conclusions, more field data should be collected and analyzed including other years and rivers.

- Sand mining activities were seen in the river during the collection of field data. The impact of sand mining upon local scour around bridge piers was not considered in this study.

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APPENDIX A
PRIMARY BATHYMETRIC DATA

Table A.1: Observed bed levels in the Dhaleshwari River near the bridge on 20 May, 2017

Chainage	Bed Level (m PWD)					
	100 m u/s	50 m u/s	Bridge Site	100 m d/s	200 m d/s	300 m d/s
0	2.58	2.58	2.85	2.58	2.58	2.58
3	1.38	1.08	2.69	1.68	1.38	1.38
6	1.08	-0.82	2.58	1.28	1.08	1.08
9	0.78	-1.12	2.58	0.78	0.78	0.78
12	0.28	-1.32	0.28	0.28	0.28	0.28
15	0.08	-1.52	0.18	-0.32	0.08	0.08
18	-0.32	-1.42	-0.62	-0.92	-0.32	-0.32
21	-0.82	-1.12	-0.92	-1.42	-0.82	-0.82
24	-1.02	-1.12	-0.72	-2.52	-1.02	-1.02
27	-1.12	-1.42	-1.62	-2.22	-1.12	-1.12
30	-1.02	-1.62	-1.87	-1.82	-1.02	-1.02
33	-1.32	-1.82	-1.97	-2.12	-1.32	-1.32
36	-1.32	-1.92	-2.20	-2.02	-1.32	-1.32
39	-1.42	-1.92	-1.92	-1.82	-1.42	-1.42
42	-1.12	-2.02	-2.21	-1.62	-1.22	-1.22
45	-1.32	-2.12	-2.23	-1.52	-1.12	-1.12
48	-1.32	-2.22	-2.34	-1.42	-1.32	-1.32
51	-1.52	-2.32	-2.41	-1.32	-1.52	-1.52
54	-1.62	-2.32	-2.51	-1.22	-1.62	-1.62
57	-2.04	-1.72	-2.62	-1.22	-2.02	-2.04
60	-2.12	-2.22	-2.82	-1.52	-2.12	-2.12
63	-2.02	-2.02	-2.97	-1.82	-2.02	-2.02
66	-1.92	-1.92	-3.02	-2.22	-1.92	-1.92
69	-1.85	-1.92	-3.12	-2.12	-1.92	-1.85
72	-1.82	-1.72	-2.87	-2.02	-1.82	-1.82
75	-1.82	-1.72	-2.54	-1.82	-1.82	-1.82
78	-1.8	-1.62	-1.82	-1.12	-1.62	-1.8
81	-1.52	-1.42	-1.72	-1.62	-1.52	-1.52

84	-1.42	-1.32	-1.82	-1.42	-1.42	-1.42
87	-1.42	-1.32	-1.77	-1.52	-1.42	-1.42
90	-1.32	-1.22	-1.73	-1.42	-1.32	-1.32
93	-1.58	-1.32	-1.68	-1.32	-1.42	-1.58
96	-1.52	-1.62	-1.79	-1.32	-1.52	-1.52
99	-1.52	-1.72	-1.81	-1.22	-1.52	-1.52
102	-1.42	-1.82	-2.03	-1.12	-1.42	-1.42
105	-2.22	-1.72	-1.42	-1.32	-2.22	-2.22
108	-3.22	-1.92	-1.94	-1.12	-3.22	-3.22
111	-3.22	-1.82	-1.82	-1.22	-3.32	-3.22
114		-1.72	-1.82	-1.12	-2.92	-2.92
117		-1.02	-1.72	-1.02	-3.92	-3.98
120		0.08	-1.62	-1.12	-3.32	-3.32
123		0.48	-0.82	-0.52	-2.72	-2.72
126		0.78	-0.32	-0.32	-2.12	-2.12
129		1.78	0.28	-0.32	-1.92	-1.92
132		2.23	0.78	-0.22	-0.02	-0.02
135			1.19	0.08	0.78	0.78
138			1.44	1.18	1.18	1.18
141				2.01	1.89	2.43

Table A.2: Observed bed levels in the Dhaleshwari River near the bridge on 24 June, 2017

Chainage	Bed Level (m PWD)					
	200 m u/s	100 m u/s	Bridge Site	100 m d/s	200 m d/s	300 m d/s
0	5.46	5.46	5.61	5.46	5.46	5.46
3	4.76	3.76	4.86	3.76	3.16	3.46
6	4.66	3.16	4.56	3.56	2.36	3.16
9	4.56	1.96	3.96	2.96	1.86	3.26
12	4.46	0.96	3.76	2.16	1.56	3.16
15	4.66	0.06	3.26	1.56	0.96	2.86
18	2.56	-0.04	2.56	0.76	0.36	2.26
21	2.36	-0.04	1.86	-0.34	-0.24	1.76
24	2.26	-0.34	1.26	-1.04	-0.44	1.26
27	3.76	-0.74	0.66	-1.34	-0.64	1.06
30	3.46	-1.04	0.06	-1.44	-0.74	0.76
33	3.76	-1.34	-0.54	-1.44	-0.94	0.46
36	3.56	-1.54	-0.94	-1.34	-0.74	-0.04
39	3.46	-1.74	-1.04	-1.24	-0.64	-0.34
42	3.96	-1.94	-1.24	-1.04	-0.54	-0.44
45	3.56	-2.14	-1.44	-0.84	-0.64	-0.54
48	3.86	-2.24	-1.64	-0.94	-0.74	-0.64
51	3.76	-2.34	-1.74	-1.24	-1.34	-0.64
54	3.96	-2.34	-1.84	-1.64	-1.74	-0.64
57	3.96	-2.44	-1.94	-1.84	-1.84	-0.64
60	3.86	-2.44	-2.04	-1.94	-2.04	-0.94
63	3.96	-2.54	-1.94	-1.84	-2.04	-1.14
66	3.16	-2.54	-1.84	-1.54	-2.04	-1.44
69	3.26	-2.64	-1.84	-1.34	-2.14	-1.84
72	2.36	-2.64	-1.74	-1.24	-2.14	-1.94
75	2.76	-2.74	-1.64	-1.04	-2.04	-1.84
78	2.56	-2.74	-1.54	-0.94	-2.04	-1.84
81	2.86	-2.84	-1.74	-0.74	-1.94	-1.74

84	2.96	-2.84	-1.84	-0.74	-1.84	-1.64
87	3.16	-2.94	-1.74	-0.64	-1.54	-1.54
90	3.06	-3.44	-1.84	-0.74	-1.84	-1.04
93	2.86	-2.94	-1.74	-0.74	-1.54	-0.94
96	2.96	-2.84	-1.54	-0.84	-1.54	-0.94
99	3.96	-2.84	-1.44	-1.04	-1.54	-0.94
102	3.46	-2.94	-1.34	-0.94	-2.54	-0.84
105	4.46	-2.84	-1.24	-0.74	-1.64	-0.84
108	4.36	-2.64	-1.24	-0.54	-1.54	-0.94
111	4.06	-1.84	-1.14	-0.34	-1.34	-0.64
114	3.56	-1.34	-1.34	0.16	-1.24	-0.54
117	3.26	0.06	-1.54	0.76	-1.14	-0.54
120	2.96	1.26	-1.34	1.36	-1.04	-0.44
123	2.96	3.76	-1.54	1.76	-0.54	-0.44
126	2.86	5.16	-1.74	2.56	-0.24	-0.34
129	2.56	5.46	-1.84	5.46	0.16	-0.14
132	2.06		-1.94		0.56	-0.04
135	1.46		-1.84		1.46	0.06
138	0.06		-0.94		2.56	0.26
141	-0.64		-0.34		3.26	0.26
144	-1.14		0.16		1.90	0.86
147	-1.64		0.36		5.46	1.76
150	-2.24		0.26			3.86
153	-2.94		5.46			3.96
156	-3.34					5.46
159	-3.64					
162	-3.64					
165	-3.94					
168	-4.24					
171	-4.04					
174	-4.84					
177	-5.34					

180	-6.24					
183	-6.54					
186	-5.84					
189	-5.44					
192	-3.54					
195	-2.44					
198	-0.34					
201	0.96					
204	1.46					
207	2.36					

Table A.3: Observed bed levels in the Dhaleshwari River near the bridge on 4 August, 2017

Chainage	Bed Level (m PWD)					
	70 m u/s	60 m u/s	Bridge Site	30 m d/s	50 m d/s	60 m d/s
0	0.33	1.78	6.24	4.99	5.48	3.57
3	-0.05	0.71	6.01	4.01	4.70	2.83
6	-0.57	0.37	5.59	3.03	3.80	2.10
9	-2.22	0.23	4.39	2.14	2.92	1.36
12	-3.17	0.21	3.92	1.68	1.97	0.50
15	-3.50	-0.79	3.00	1.35	1.77	-0.39
18	-3.64	-2.09	2.70	0.02	1.44	-1.28
21	-3.69	-2.47	1.07	-1.31	0.24	-2.05
24	-3.70	-3.30	0.34	-1.92	-0.82	-2.05
27	-3.71	-3.46	-0.65	-2.23	-0.92	-2.03
30	-3.71	-3.76	-1.00	-2.25	-1.04	-2.16
33	-3.67	-3.75	-1.02	-2.37	-1.82	-2.18
36	-3.60	-3.81	-1.51	-2.62	-1.97	-2.14
39	-3.61	-3.55	-1.99	-2.63	-2.39	-2.01
42	-3.65	-3.55	-2.02	-2.69	-2.49	-1.90
45	-3.50	-3.61	-2.54	-2.57	-2.96	-1.82
48	-3.38	-3.54	-1.80	-2.84	-3.38	-1.83
51	-3.30	-3.50	-2.67	-2.81	-3.18	-1.69
54	-3.21	-3.31	-2.97	-2.74	-2.79	-1.48
57	-3.10	-3.26	-2.85	-2.65	-2.06	-1.86
60	-3.10	-3.11	-2.85	-2.47	-1.91	-1.76
63	-3.13	-2.96	-2.94	-2.29	-1.90	-1.94
66	-3.11	-2.97	-3.20	-2.11	-1.65	-1.88
69	-3.10	-3.23	-2.95	-2.02	-1.78	-1.71
72	-3.07	-3.40	-3.03	-2.04	-1.63	-2.08
75	-2.95	-3.29	-3.78	-2.31	-1.49	-2.08
78	-2.71	-3.05	-4.23	-2.50	-1.38	-2.28
81	-2.56	-3.03	-3.59	-2.54	-1.25	-2.28

84	-3.00	-3.02	-3.27	-2.28	-1.45	-2.50
87	-2.89	-3.09	-2.86	-2.48	-1.24	-2.50
90	-3.25	-3.01	-2.11	-2.39	-1.11	-2.42
93	-2.98	-2.85	-2.03	-2.30	-0.20	-2.19
96	-2.25	-2.64	-1.97	-1.99	0.23	-2.07
99	-1.16	-2.53	-1.83	-1.80	0.33	-1.97
102	-0.50	-2.47	-1.71	-0.84	0.52	-1.86
105	0.33	-2.36	-1.65	-0.52	1.03	-1.90
108	2.36	-2.16	-1.53	-0.43	1.39	-1.76
111		-2.07	-1.71	-0.52	1.73	-1.19
114		-1.73	-1.42	-0.12	2.31	-0.97
117		-1.11	-1.38	-0.12	2.97	-0.87
120		0.36	-1.33	0.36	3.66	-0.11
123		1.33	-1.29	1.06	3.86	1.67
126		1.44	-1.21	3.19	3.93	2.83
129			-1.49	4.24	3.83	4.10
132			-0.99		3.77	5.35
135			-0.71		3.81	0.06
138					3.91	0.26
141					3.80	0.26
144					3.77	0.86
147					3.83	1.76
150						3.86
153						3.96
156						5.46

Table A.4: Observed bed levels in the Dhaleshwari River near the bridge on 19 August, 2017

Chainage	Bed Level (m PWD)					
	70 m u/s	30 m u/s	Bridge Site	30 m d/s	50 m d/s	60 m d/s
0	0.33	4.35	8.22	5.48	3.57	2.24
3	-0.05	3.34	7.59	4.70	2.83	1.50
6	-0.57	2.24	7.13	3.80	2.10	0.76
9	-2.22	0.92	6.65	2.92	1.36	0.03
12	-3.17	0.41	6.01	1.97	0.50	-0.71
15	-3.50	-0.08	5.47	1.77	-0.39	-1.45
18	-3.64	-0.76	4.99	1.44	-1.28	-1.83
21	-3.69	-0.82	4.34	0.24	-2.05	-1.73
24	-3.70	-1.04	3.58	-0.82	-2.05	-1.74
27	-3.71	-1.22	2.78	-0.92	-2.03	-1.87
30	-3.71	-1.18	1.90	-1.04	-2.16	-1.98
33	-3.67	-1.17	1.21	-1.82	-2.18	-2.03
36	-3.60	-1.28	0.23	-1.97	-2.14	-2.17
39	-3.61	-1.87	-0.78	-2.39	-2.01	-1.96
42	-3.65	-2.59	-1.80	-2.49	-1.90	-2.06
45	-3.50	-3.79	-2.67	-2.96	-1.82	-2.07
48	-3.38	-3.77	-2.70	-3.38	-1.83	-1.91
51	-3.30	-3.79	-3.20	-3.18	-1.69	-1.83
54	-3.21	-3.69	-2.81	-2.79	-1.48	-1.78
57	-3.10	-3.50	-2.94	-2.06	-1.86	-1.72
60	-3.10	-3.40	-2.99	-1.91	-1.76	-1.60
63	-3.13	-2.86	-2.95	-1.90	-1.94	-1.66
66	-3.11	-2.93	-2.79	-1.65	-1.88	-1.47
69	-3.10	-2.70	-2.72	-1.78	-1.71	-1.33
72	-3.07	-2.44	-2.86	-1.63	-2.08	-1.27
75	-2.95	-2.47	-2.91	-1.49	-2.08	-1.75
78	-2.71	-2.47	-3.02	-1.38	-2.28	-1.71
81	-2.56	-2.62	-3.47	-1.25	-2.28	-1.56

84	-3.00	-2.66	-4.98	-1.45	-2.50	-1.44
87	-2.89	-2.73	-4.59	-1.24	-2.50	-1.35
90	-3.25	-2.56	-4.13	-1.11	-2.42	-1.26
93	-2.98	-2.53	-3.47	-0.20	-2.19	-1.17
96	-2.25	-2.53	-2.11	0.23	-2.07	-1.08
99	-1.16	-2.55	-2.07	0.33	-1.97	-1.17
102	-0.50	-2.43	-1.86	0.52	-1.86	-1.24
105	0.33	-2.35	-1.48	1.03	-1.90	-0.90
108	2.36	-2.26	-1.37	1.39	-1.76	-0.73
111		-1.80	-1.31	1.73	-1.19	-0.62
114		-1.91	-1.29	2.31	-0.97	-0.45
117		-1.74	-1.98	2.97	-0.87	1.15
120		-1.65	4.99	3.66	-0.11	

Table A.5: Observed bed levels in the Dhaleshwari River near the bridge on 16 October, 2017

Chainage	Bed Level (m PWD)					
	70 m u/s	30 m u/s	Bridge Site	30 m d/s	50 m d/s	60 m d/s
0	0.33	4.35	5.01	6.13	5.57	4.24
3	-0.05	3.36	2.37	4.97	3.28	1.59
6	-0.57	2.26	1.98	3.80	2.10	0.76
9	-2.22	0.92	1.87	2.92	1.36	0.03
12	-3.17	0.49	1.82	1.97	0.50	-0.71
15	-3.50	-0.08	1.47	1.77	-0.39	-1.45
18	-3.64	-0.81	1.32	1.44	-1.28	-1.83
21	-3.69	-0.82	0.58	0.24	-2.05	-1.73
24	-3.70	-1.04	-1.01	-0.82	-2.10	-1.74
27	-3.71	-1.22	-1.24	-0.92	-2.03	-1.99
30	-3.71	-1.91	-1.39	-1.04	-2.20	-1.98
33	-3.67	-1.17	-1.67	-1.82	-2.34	-2.20
36	-3.60	-1.32	-1.47	-1.97	-2.14	-2.17
39	-3.61	-1.87	-1.41	-2.41	-2.01	-1.96
42	-3.65	-2.59	-1.98	-2.49	-1.90	-2.06
45	-3.50	-3.74	-2.05	-2.96	-1.82	-2.07
48	-3.38	-3.77	-2.13	-3.34	-1.83	-1.91
51	-3.30	-3.79	-2.24	-3.18	-1.69	-1.83
54	-3.21	-3.89	-2.37	-2.79	-1.48	-1.78
57	-3.10	-3.50	-2.51	-2.06	-1.86	-1.72
60	-3.10	-3.40	-2.66	-1.91	-1.76	-1.60
63	-3.13	-2.86	-2.74	-1.90	-1.94	-1.66
66	-3.11	-2.93	-2.89	-1.65	-1.88	-1.47
69	-3.10	-2.70	-1.89	-1.78	-1.71	-1.33
72	-3.07	-2.44	-1.84	-1.63	-2.08	-1.27
75	-2.95	-2.47	-1.79	-1.49	-2.08	-1.75
78	-2.71	-2.47	-1.72	-1.38	-2.28	-1.71
81	-2.56	-2.62	-1.64	-1.25	-2.28	-1.56

84	-3.00	-2.66	-1.89	-1.45	-2.50	-1.44
87	-2.89	-2.73	-1.51	-1.24	-2.50	-1.35
90	-3.25	-2.56	-1.42	-1.11	-2.42	-1.26
93	-2.98	-2.53	-1.39	-0.20	-2.19	-1.17
96	-2.25	-2.53	-1.34	0.23	-2.07	-1.08
99	-1.16	-2.55	-1.73	0.33	-1.97	-1.17
102	-0.50	-2.43	-1.39	0.52	-1.86	-1.24
105	0.33	-2.35	-1.49	1.03	-1.90	-0.90
108	2.36	-2.26	-1.42	1.39	-1.76	-0.73
111		-1.80	-1.30	1.73	-1.19	-0.62
114		-1.91	-1.22	2.31	-0.97	-0.45
117		-1.74	-1.17	2.97	-0.87	1.15
120		-1.65	-0.99	3.66	-0.11	
123		-1.57	-0.72	3.82	1.67	
126		-1.41	-0.10	3.90	2.83	
129		-1.04	0.03	3.83	4.10	
132		-0.32	1.02	3.77	5.35	
135		0.85	1.13	3.81		
138		1.80	1.71	3.91		
141		4.47	2.07	3.80		
144			2.14	3.77		
147			2.42	3.83		

APPENDIX B
PRIMARY CROSS-SECTIONAL PROFILES

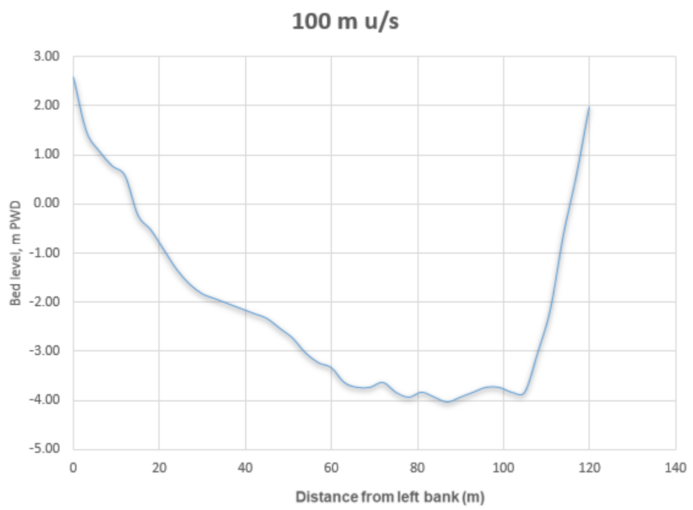


Figure B1: Observed bed level on May 20 at 100 m u/s

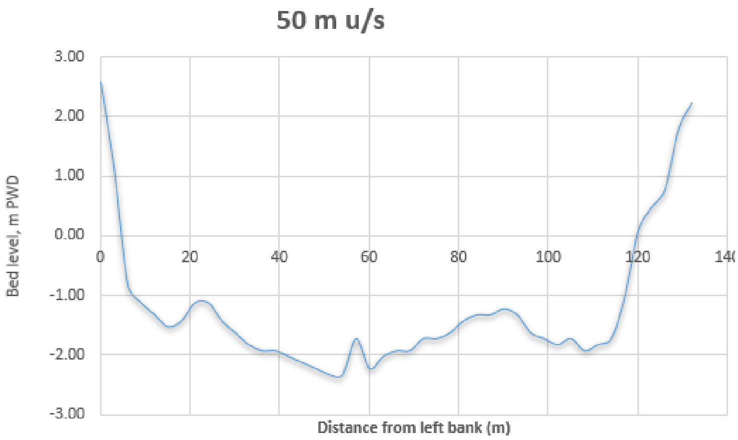


Figure B.2: Observed bed level on May 20 at 50 m u/s

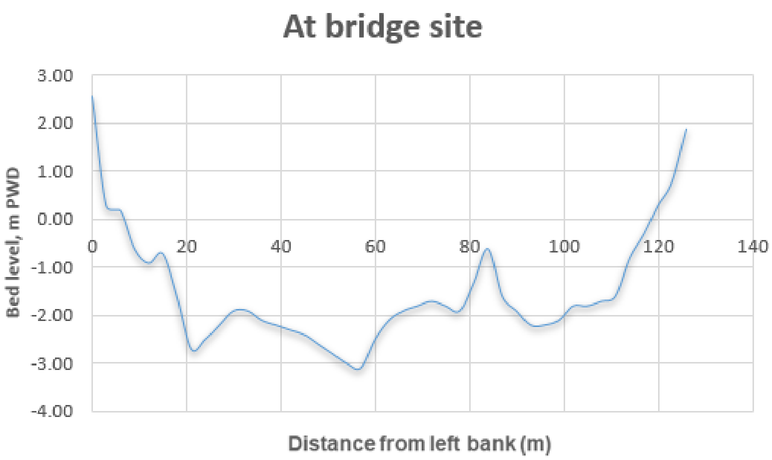


Figure B.3: Observed bed level on May 20 at bridge site

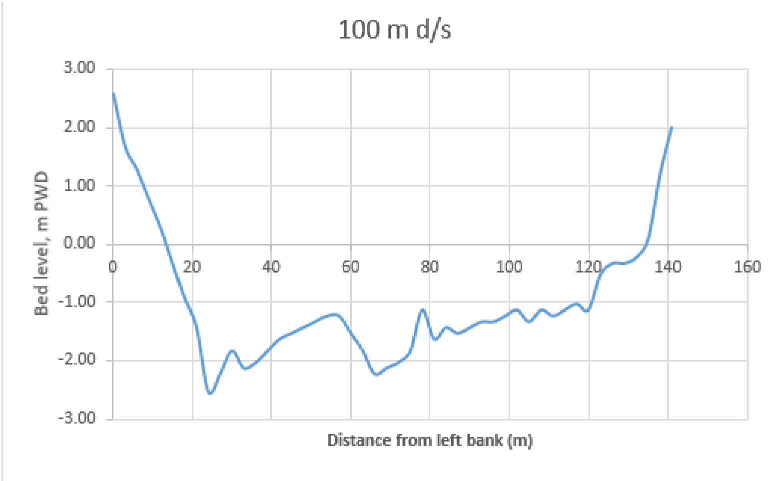


Figure B.4: Observed bed level on May 20 at 100 m d/s

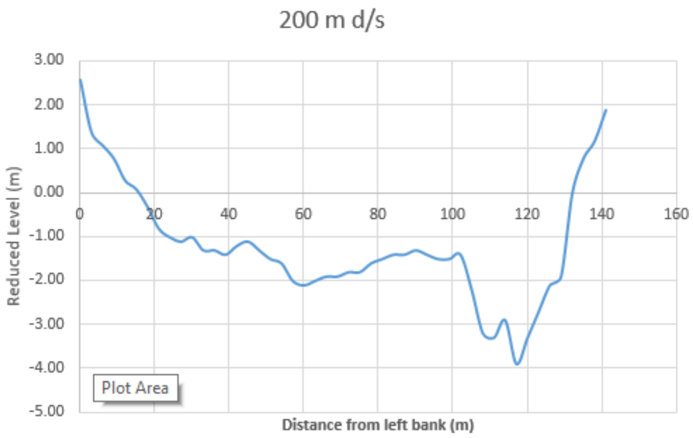


Figure B.5: Observed bed level on May 20 at 200 m d/s

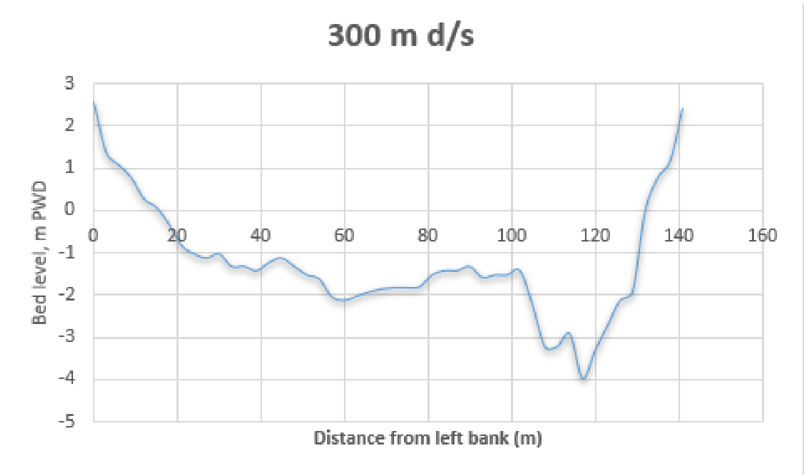


Figure B.6: Observed bed level on May 20 at 200 m d/s

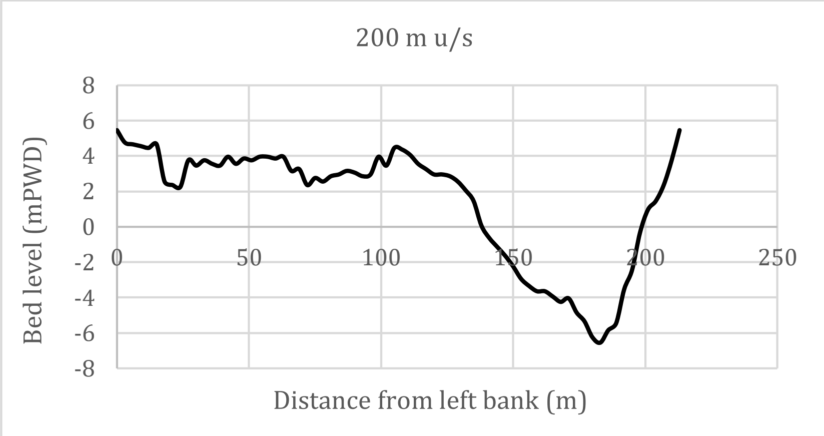


Figure B.7: Observed bed level on June 24 at 200 m u/s.

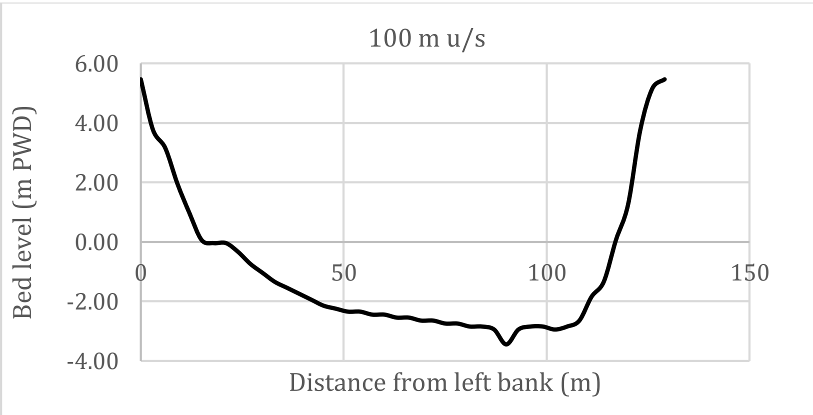


Figure B.8: Observed bed level on June 24 at 100 m u/s.

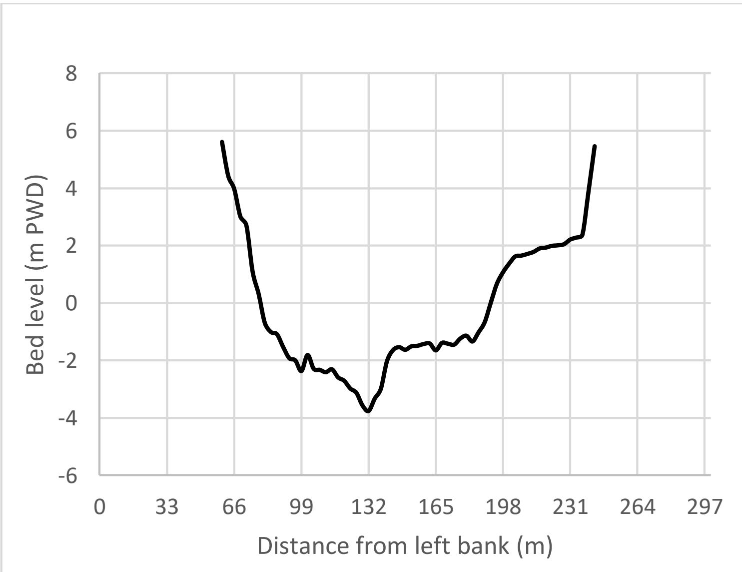


Figure B.9: Observed bed level on June 24 at bridge site.

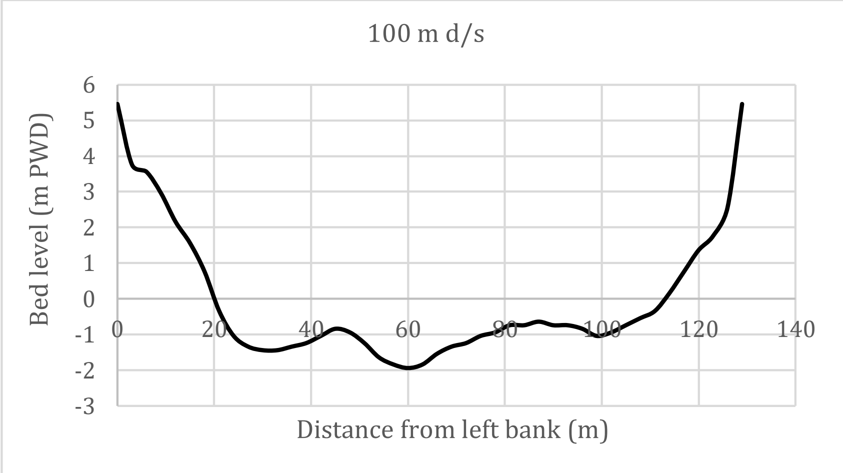


Figure B.10: Observed bed level on June 24 at 100 m d/s.

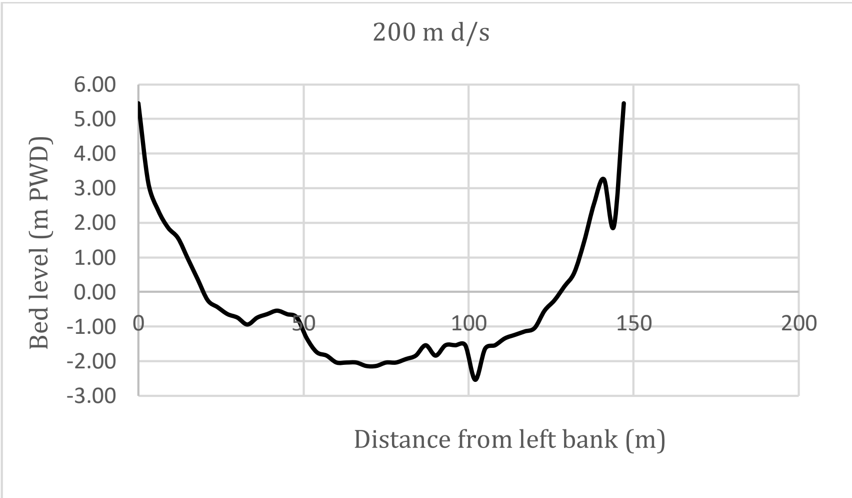


Figure B.11: Observed bed level on June 24 at 200 m d/s.

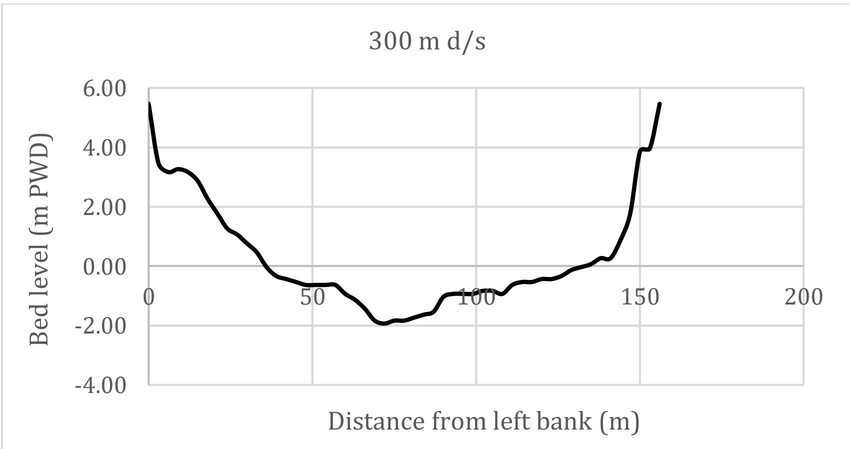


Figure B.12: Observed bed level on June 24 at 300 m d/s.

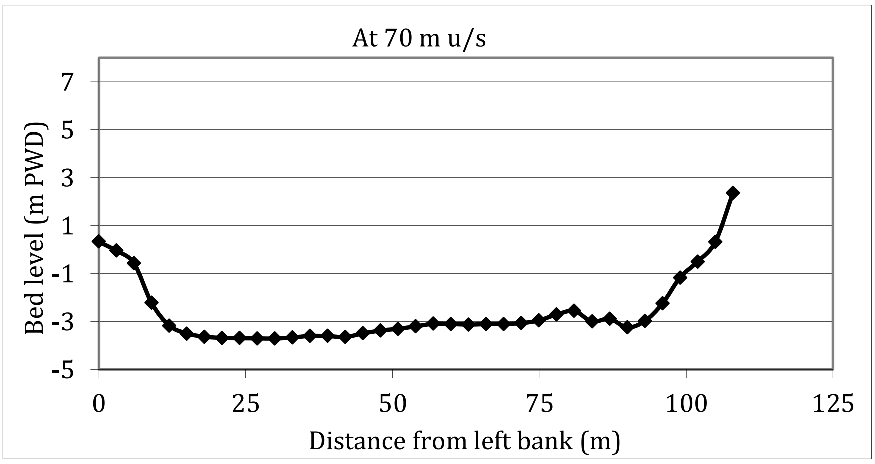


Figure B.13: Observed bed level on August 4 at 70 m u/s.

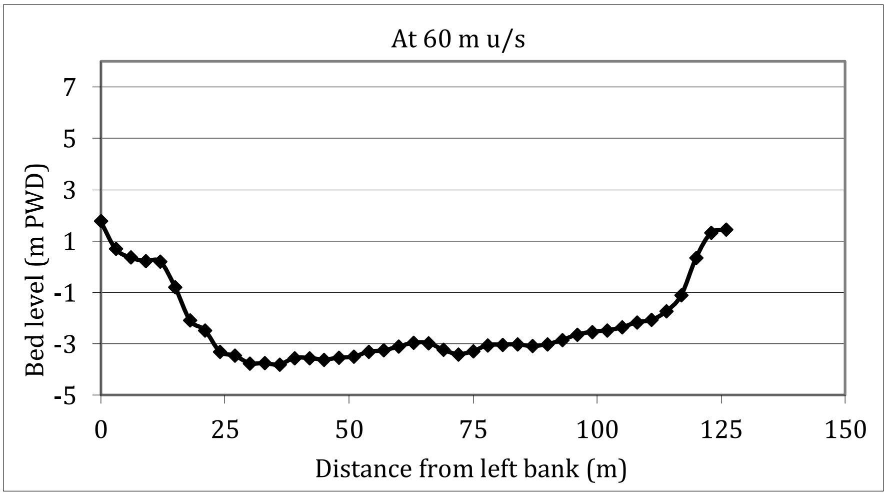


Figure B.14: Observed bed level on August 4 at 60 m u/s.

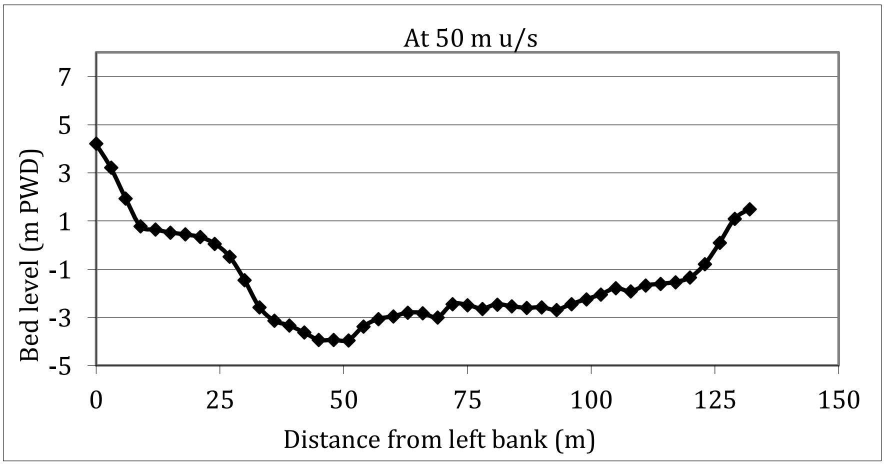


Figure B.15: Observed bed level on August 4 at 50 m u/s.

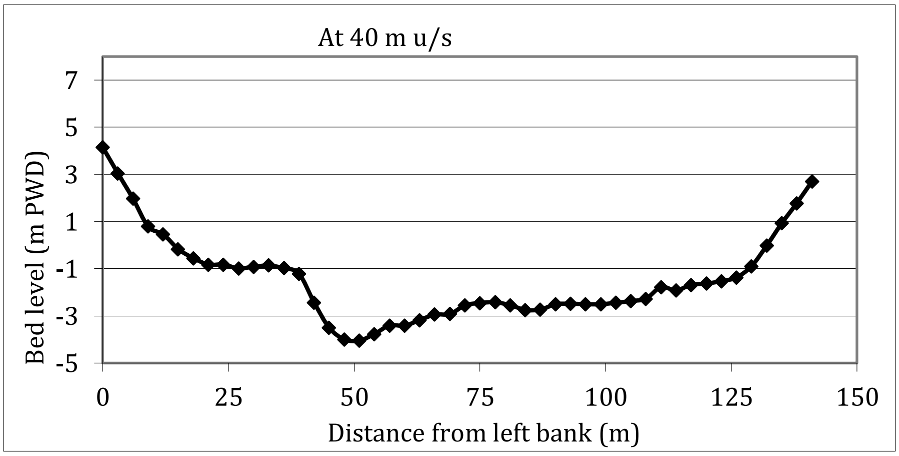


Figure B.16: Observed bed level on August 4 at 40 m u/s.

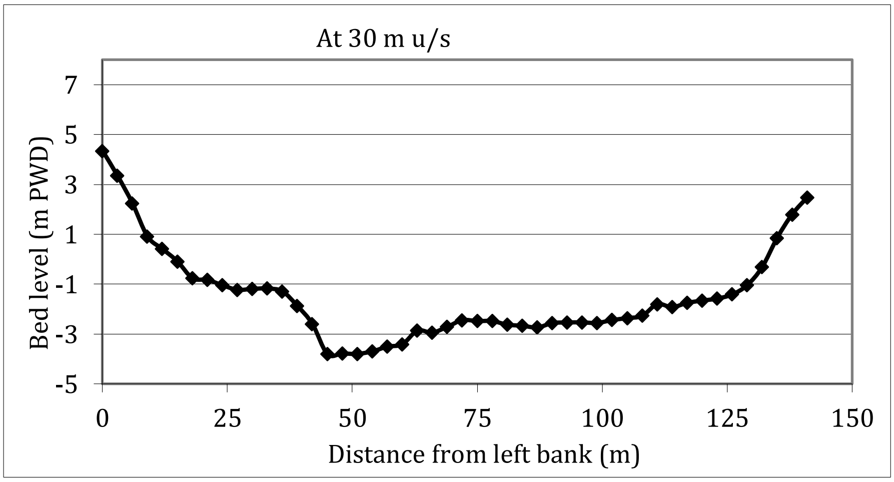


Figure B.17: Observed bed level on August 4 at 30 m u/s.

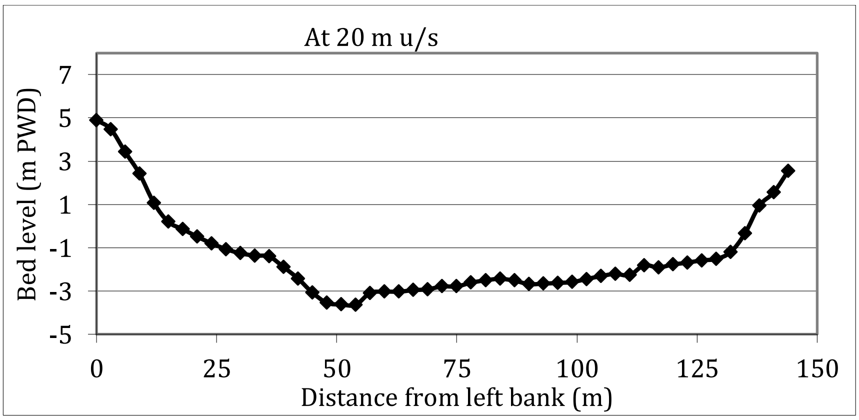


Figure B.18: Observed bed level on August 4 at 20 m u/s.

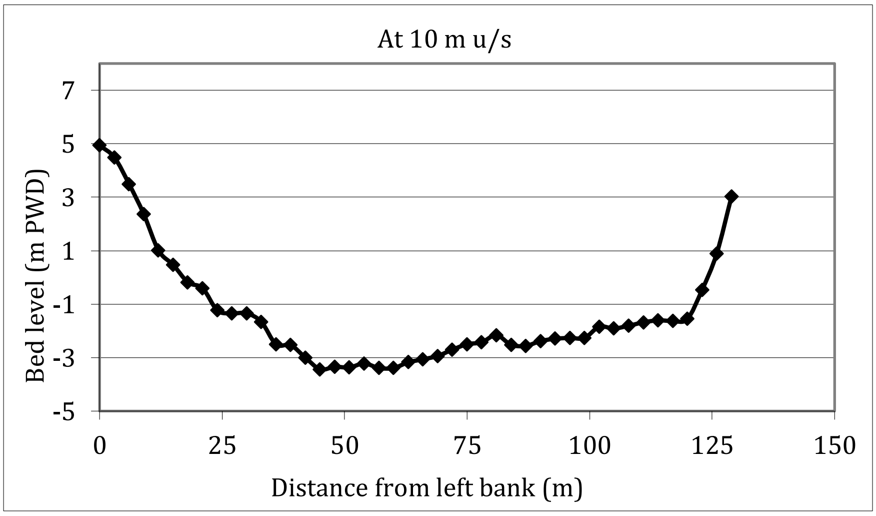


Figure B.19: Observed bed level on August 4 at 10 m u/s.

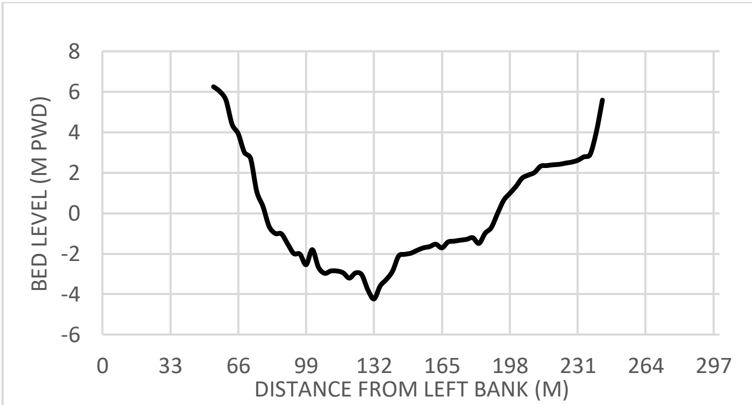


Figure B.20: Observed bed level on August 4 at bridge site.

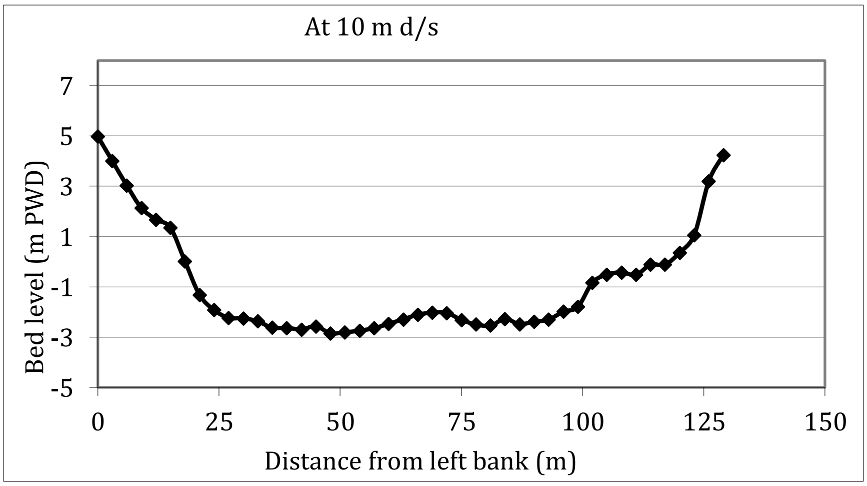


Figure B.21: Observed bed level on August 4 at 10 m d/s.

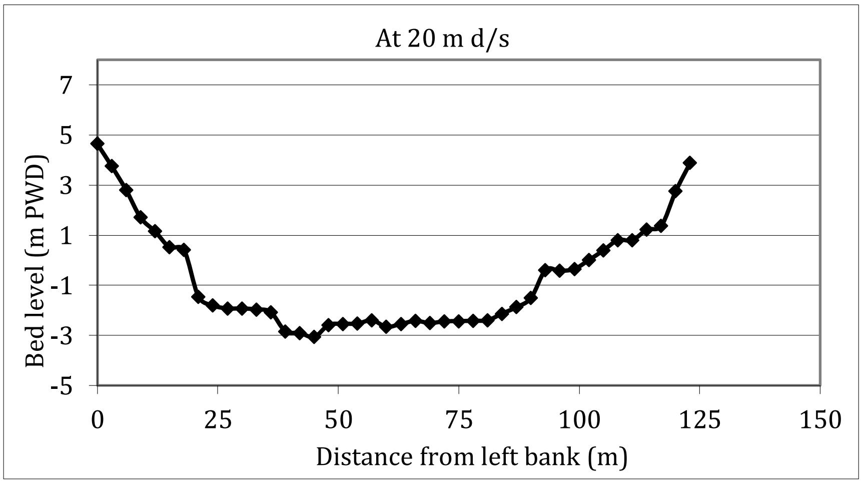


Figure B.22: Observed bed level on August 4 at 20 m d/s.

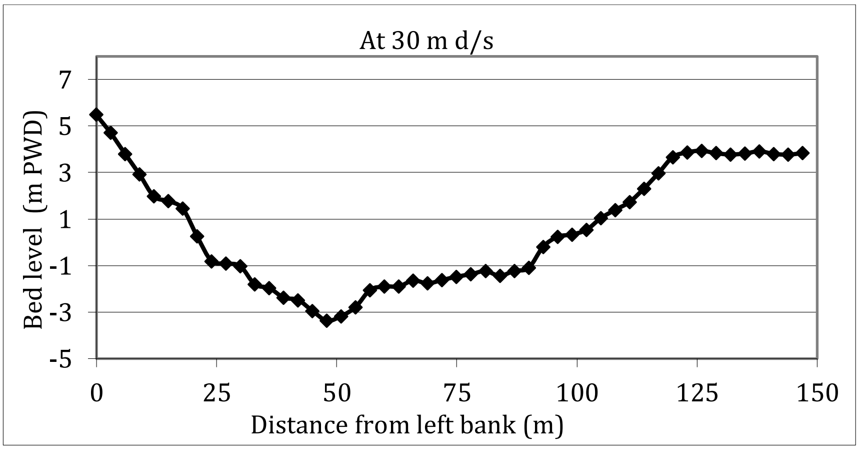


Figure B.23: Observed bed level on August 4 at 30 m d/s.

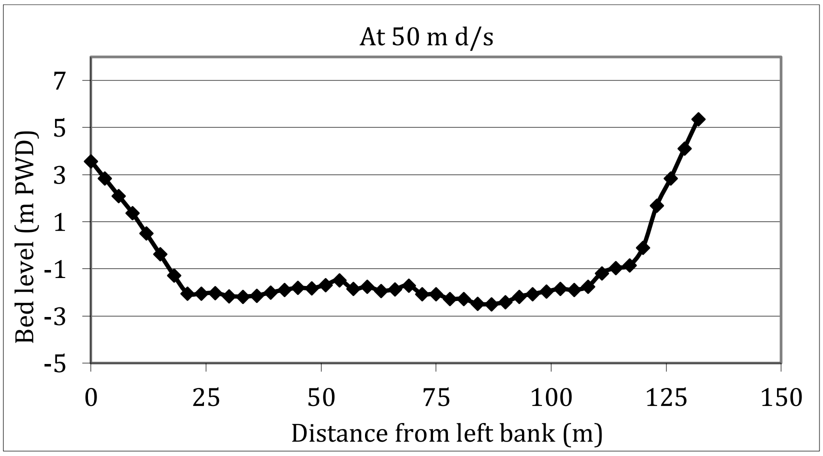


Figure B.24: Observed bed level on August 4 at 50 m d/s.

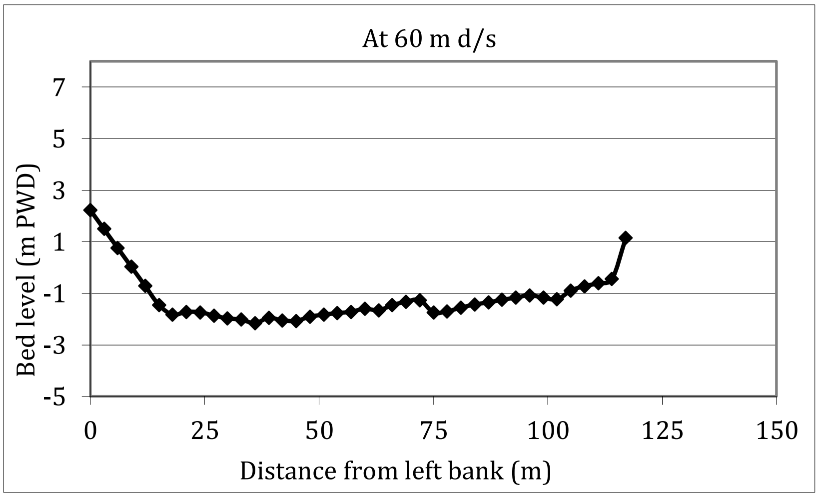


Figure B.25: Observed bed level on August 4 at 60 m d/s.

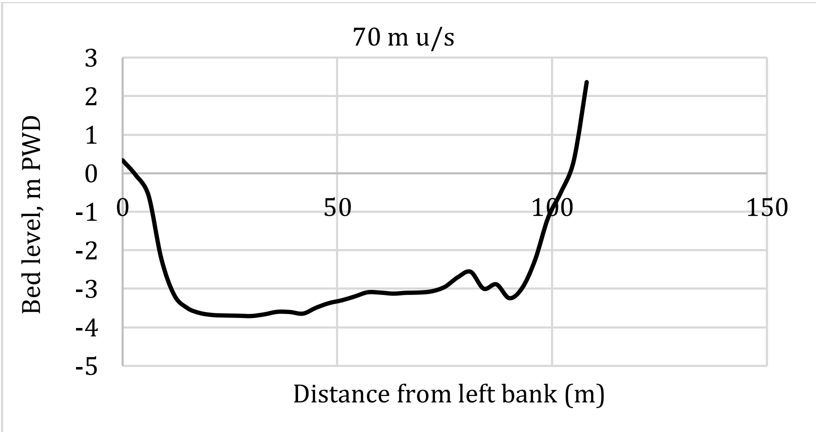


Figure B.26: Observed bed level on August 19 at 70 m u/s.

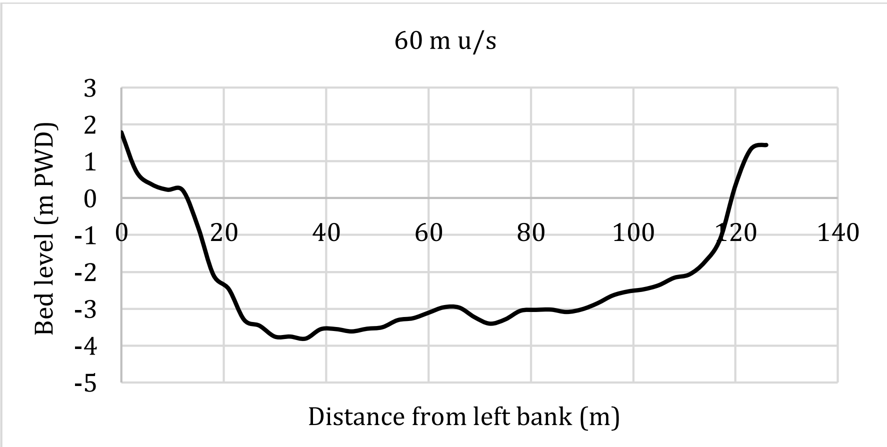


Figure B.27: Observed bed level on August 19 at 60 m u/s.

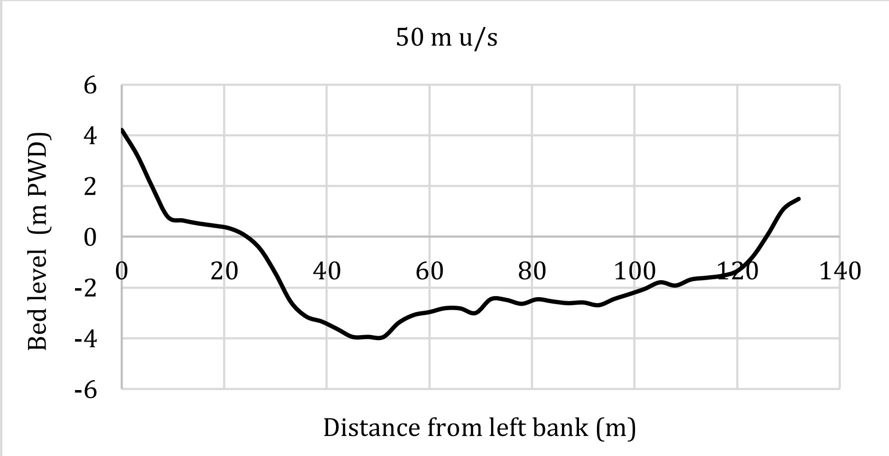


Figure B.28: Observed bed level on August 19 at 50 m u/s.

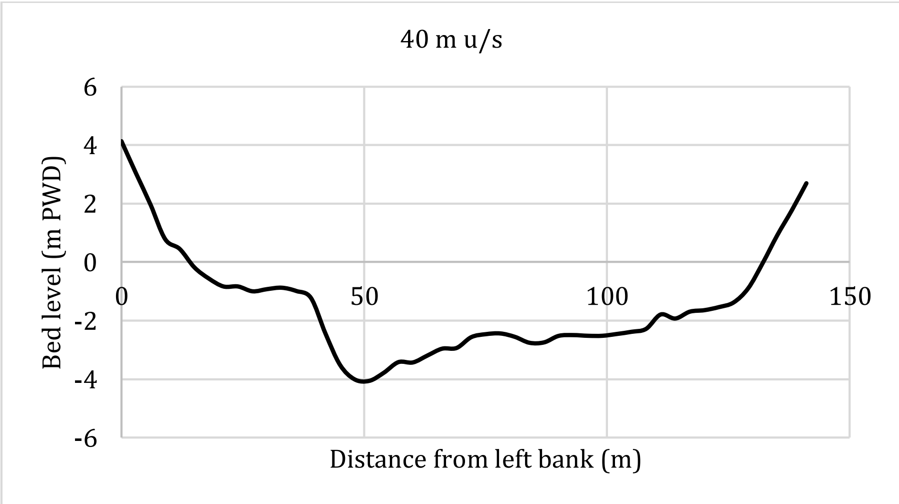


Figure B.29: Observed bed level on August 19 at 40 m u/s.

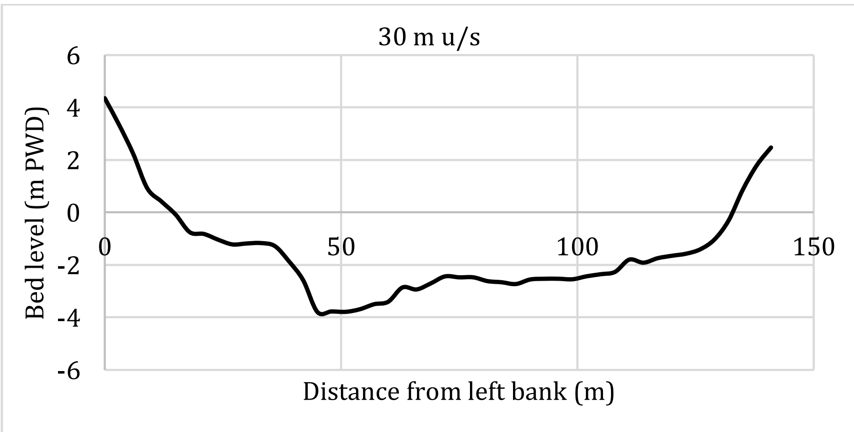


Figure B.30: Observed bed level on August 19 at 30 m u/s.

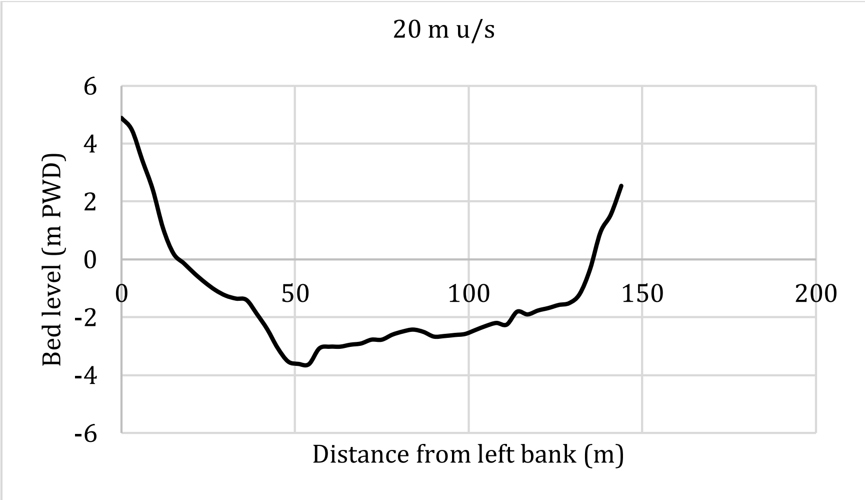


Figure B.31: Observed bed level on August 19 at 20 m u/s.

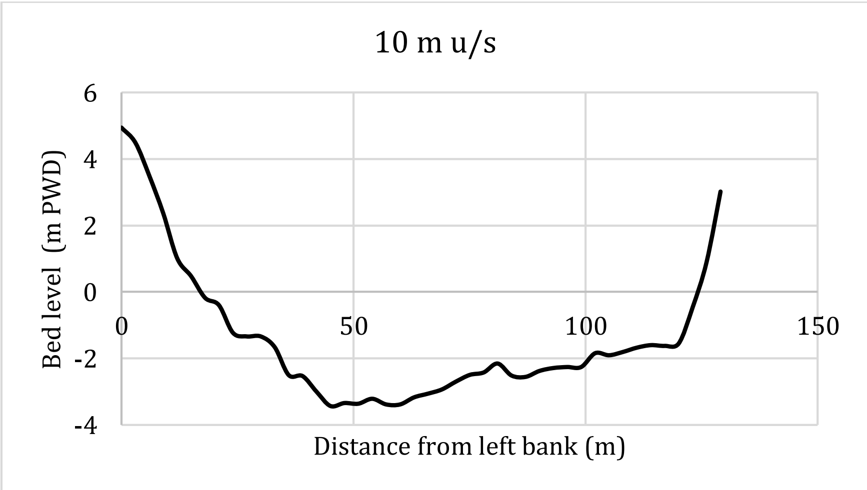


Figure B.32: Observed bed level on August 19 at 10 m u/s.

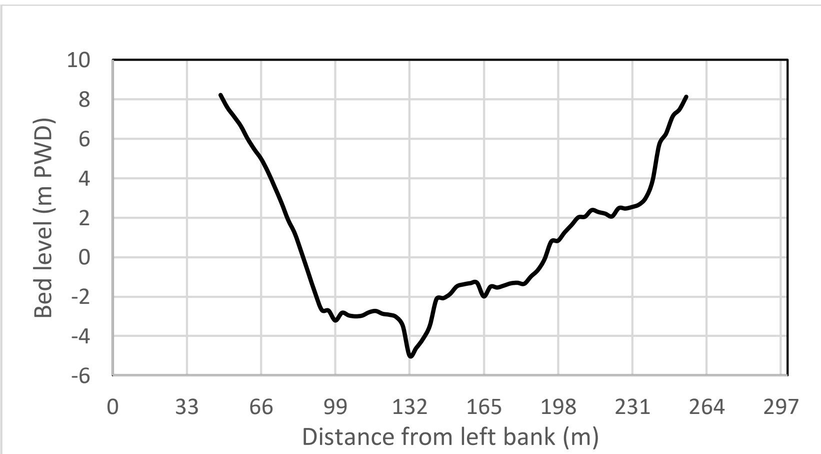


Figure B.33: Observed bed level on August 19 at bridge site.

Figure B.34: Observed bed level on August 19 at 10 m d/s.

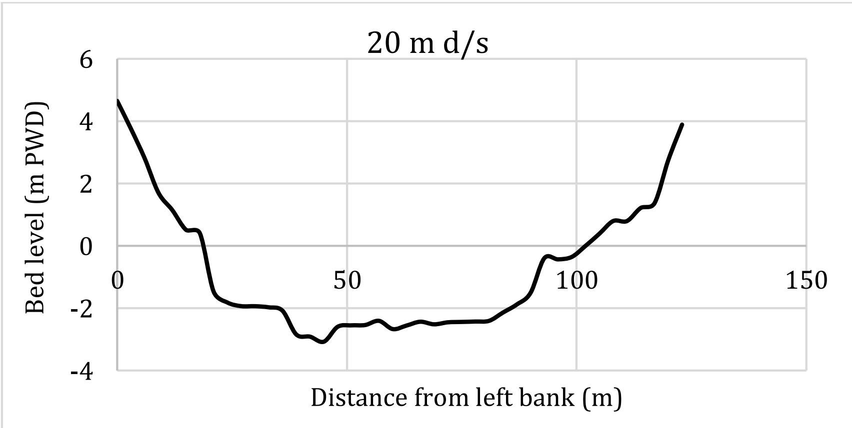


Figure B.35: Observed bed level on August 19 at 20 m d/s.

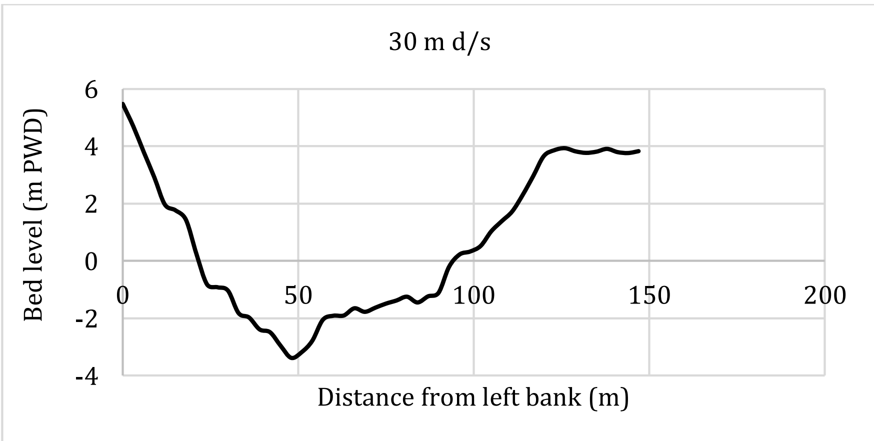


Figure B.36: Observed bed level on August 19 at 30 m d/s.

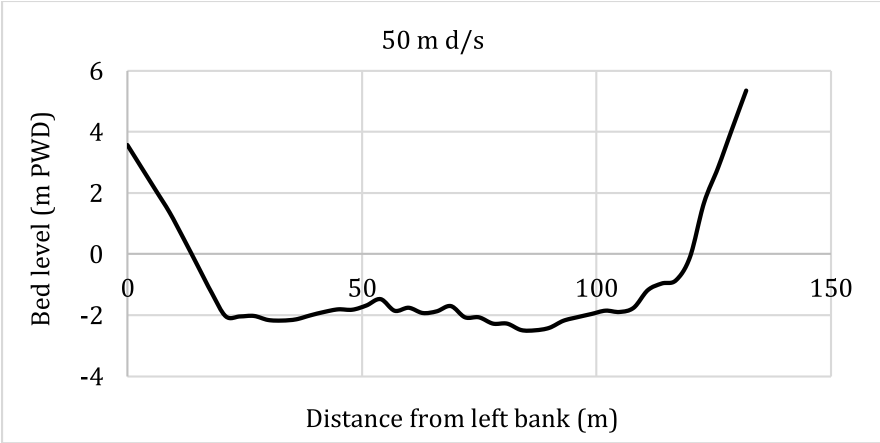


Figure B.37: Observed bed level on August 19 at 50 m d/s.

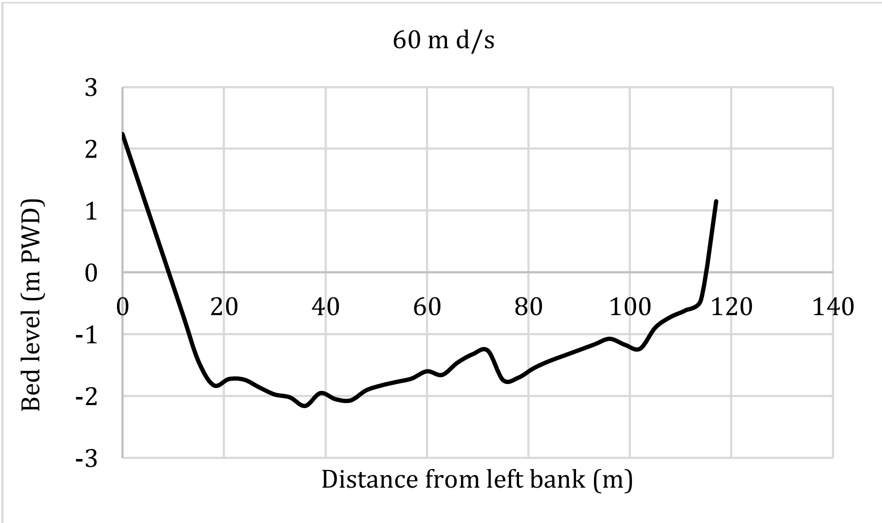


Figure B.38: Observed bed level on August 19 at 60 m d/s.

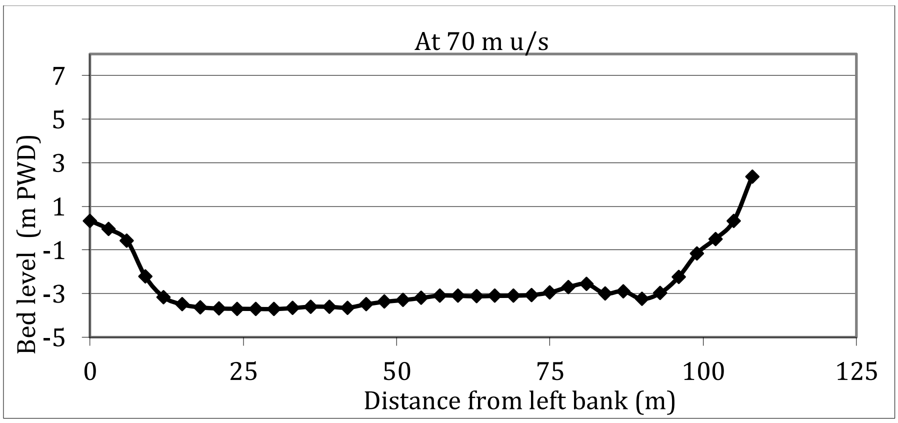


Figure B.39: Observed bed level on October 16 at 70 m u/s.

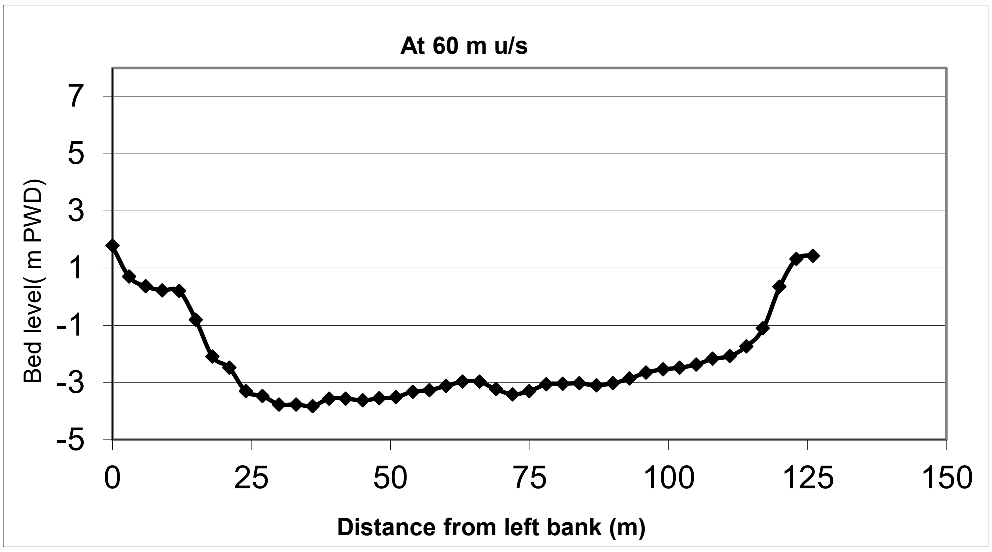


Figure B.40: Observed bed level on October 16 at 60 m u/s.

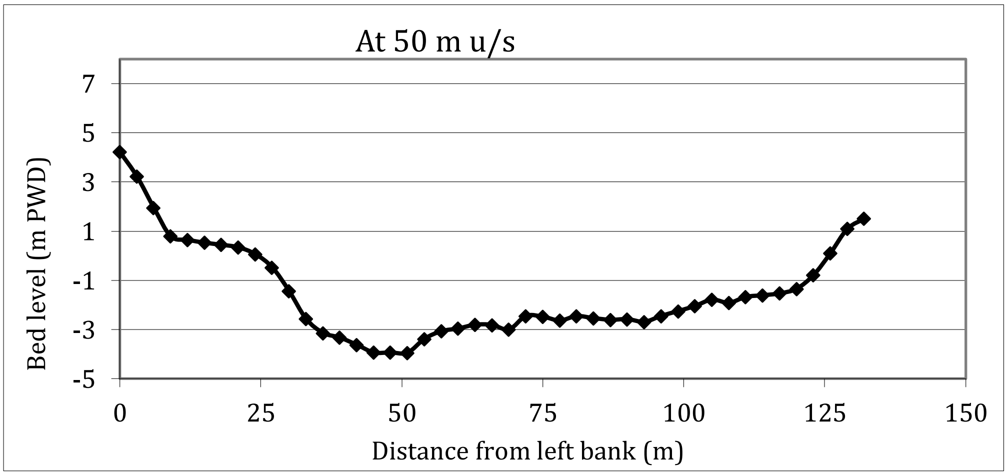


Figure B.41: Observed bed level on October 16 at 50 m u/s.

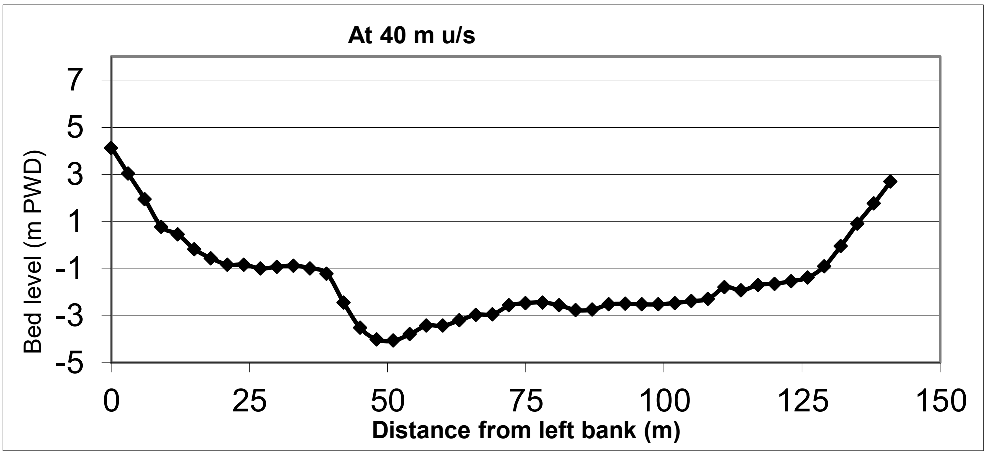


Figure B.42: Observed bed level on October 16 at 40 m u/s.

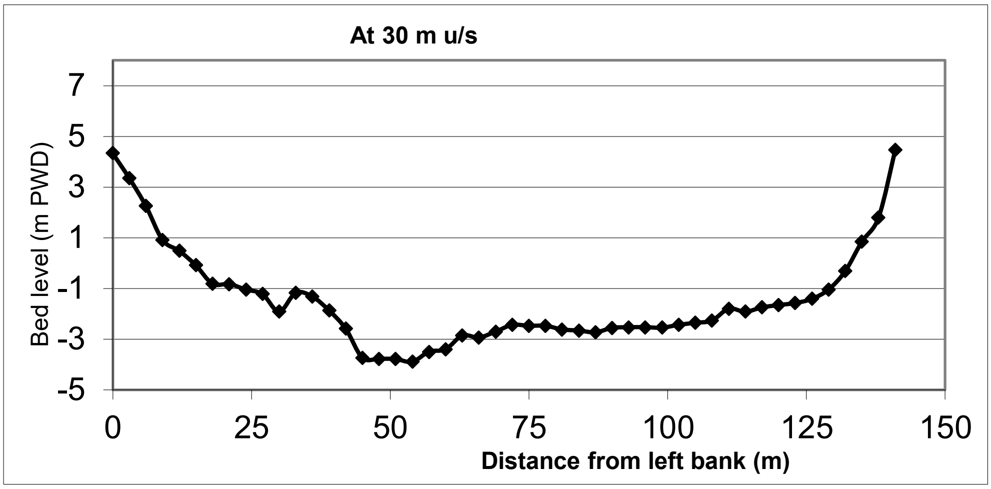


Figure B.43: Observed bed level on October 16 at 30 m u/s.

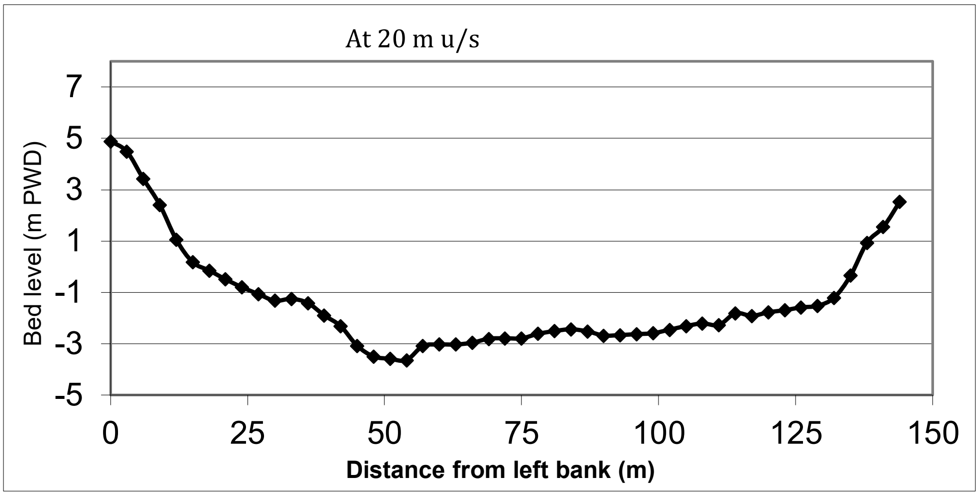


Figure B.44: Observed bed level on October 16 at 20 m u/s.

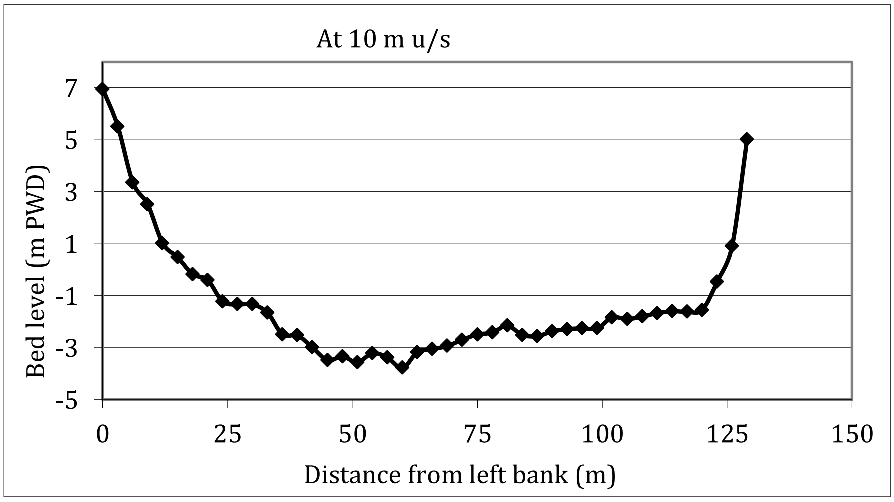


Figure B.45: Observed bed level on October 16 at 10 m u/s.

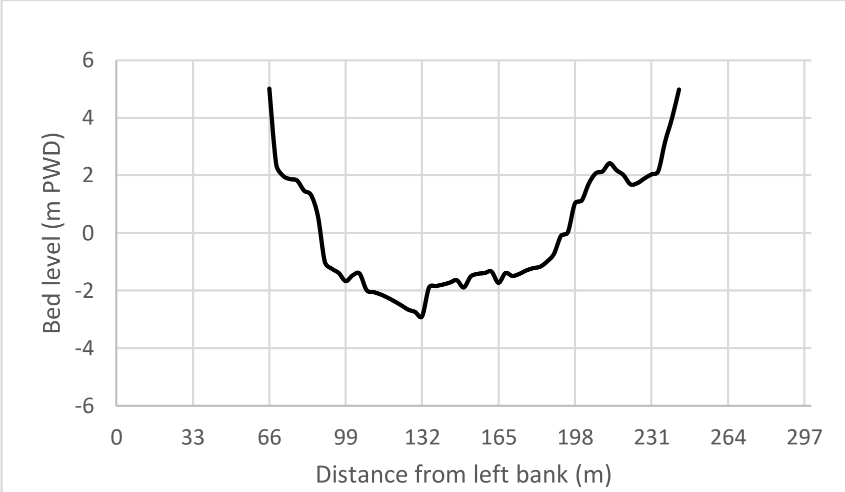


Figure B.46: Observed bed level on October 16 at bridge site.

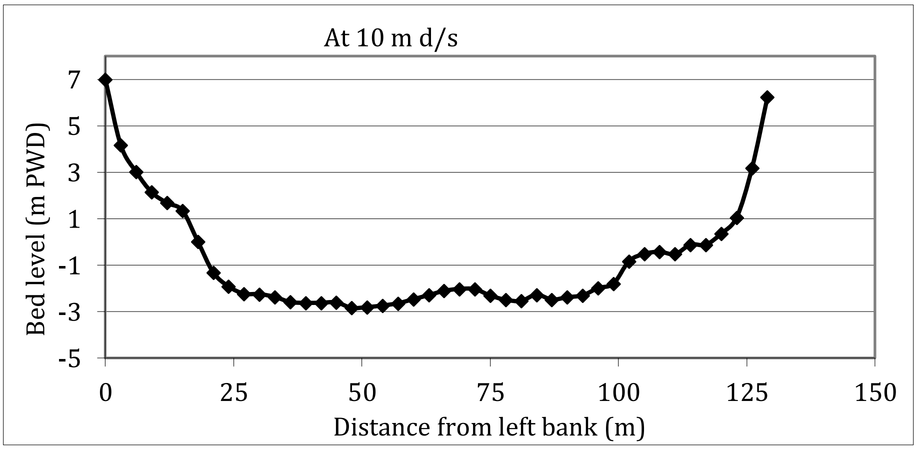


Figure B.47: Observed bed level on October 16 at 10 m d/s.

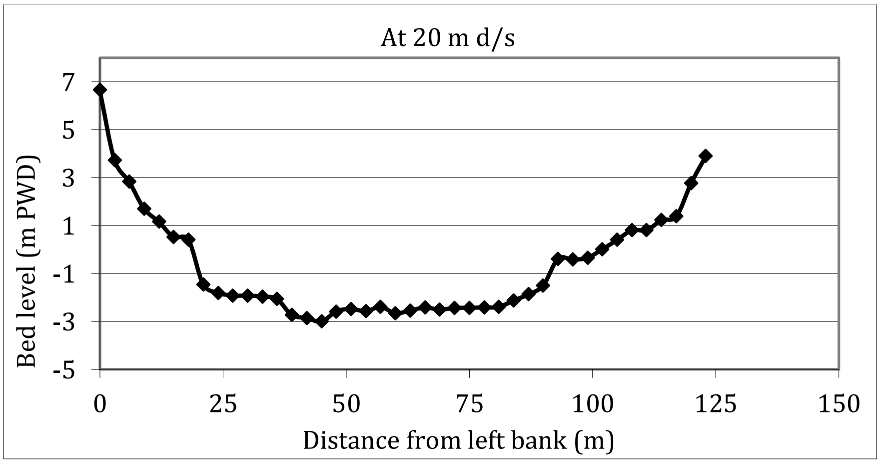


Figure B.48: Observed bed level on October 16 at 20 m d/s.

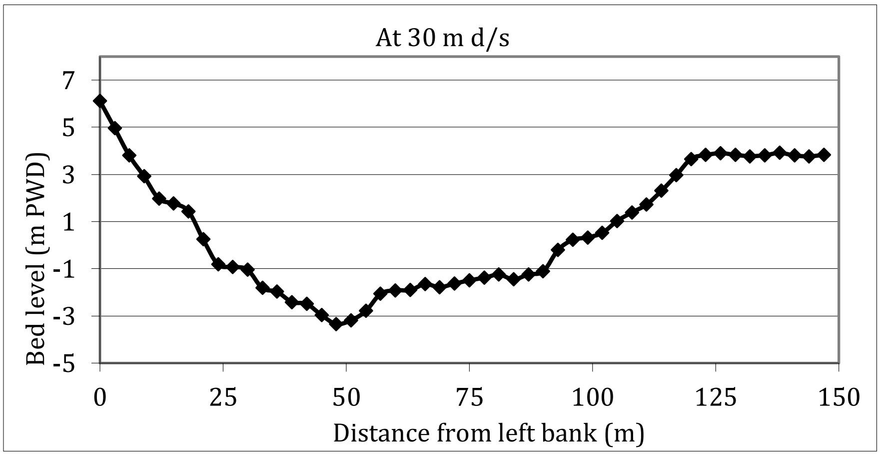


Figure B.49: Observed bed level on October 16 at 30 m d/s.

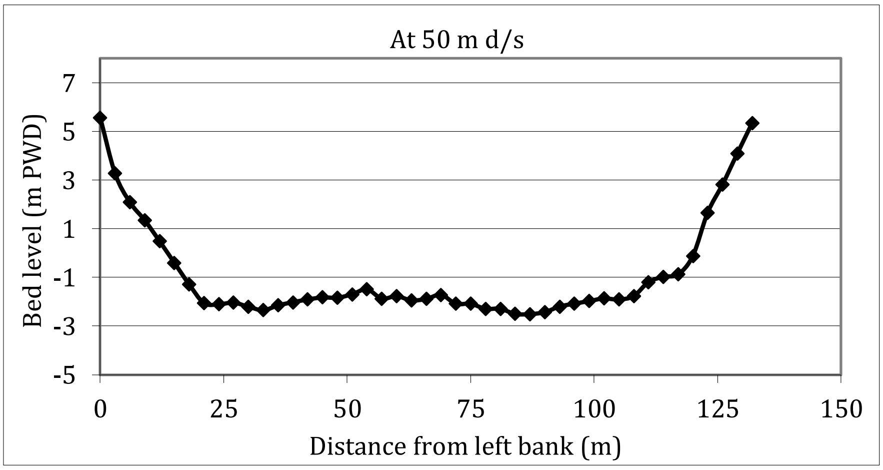


Figure B.50: Observed bed level on October 16 at 50 m d/s.

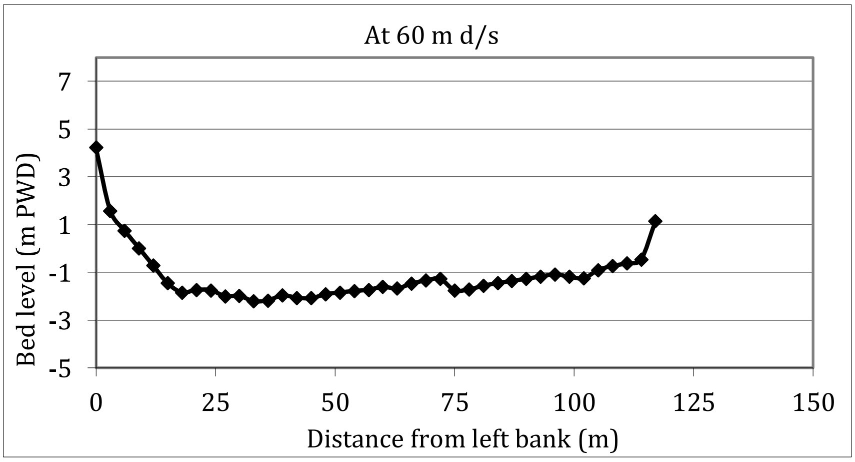


Figure B.51: Observed bed level on October 16 at 60 m d/s.

APPENDIX C
GRAIN SIZE ANALYSIS

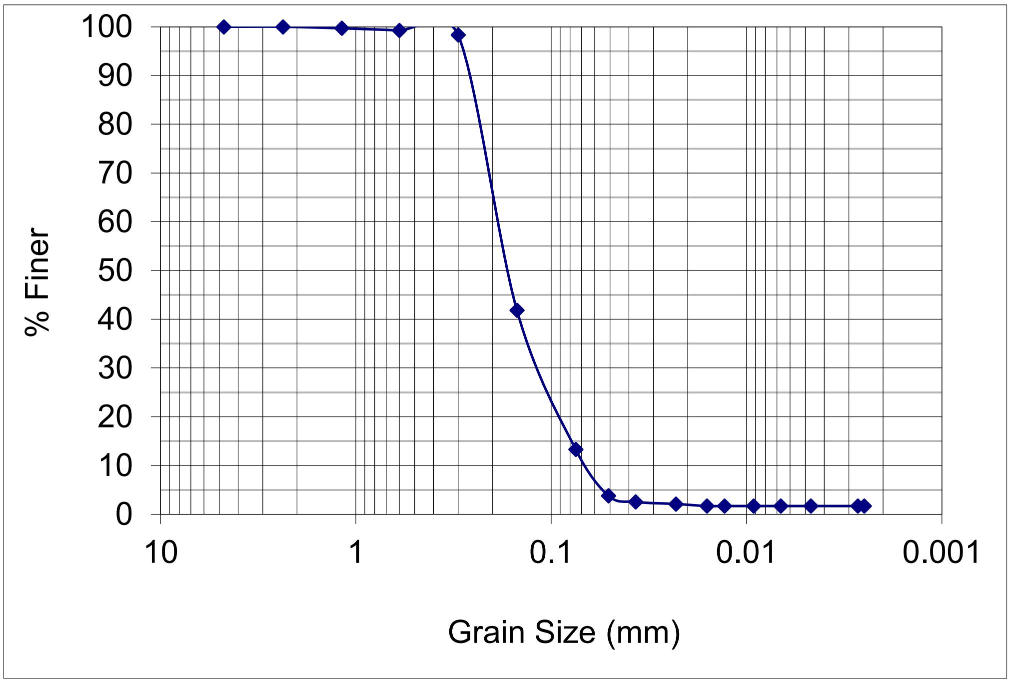


Figure C1: Grain size distribution curve of the bed sediment of the Dhaleswari River near the Dhaleswari bridge site.

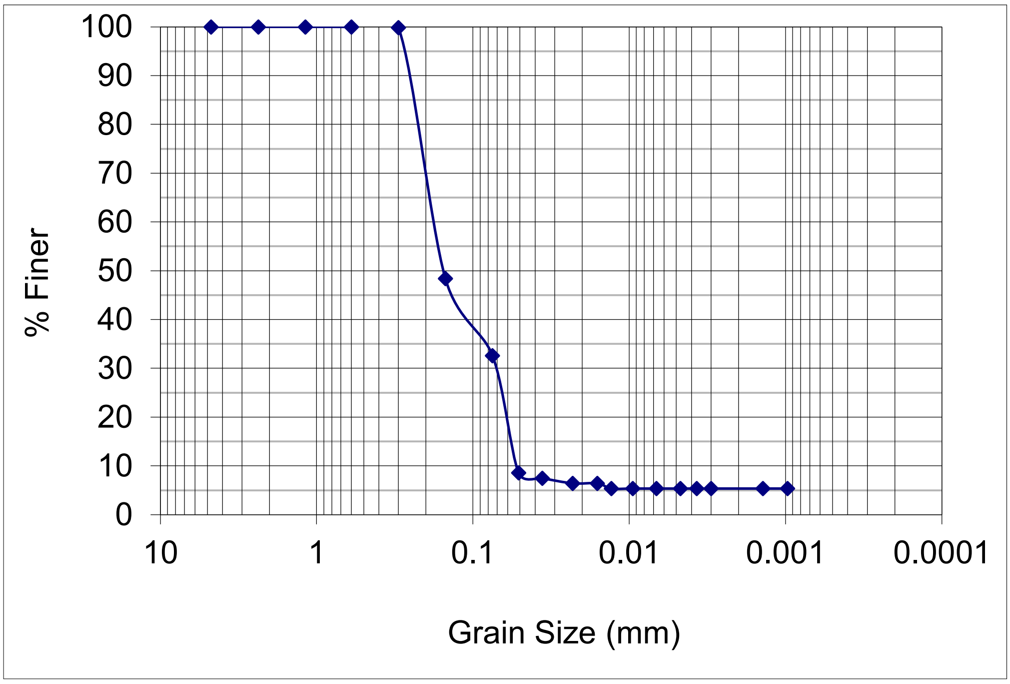


Figure C2: Grain size distribution curve of the bed sediment of the Dhaleshwari River near the Dhaleswari bridge site.

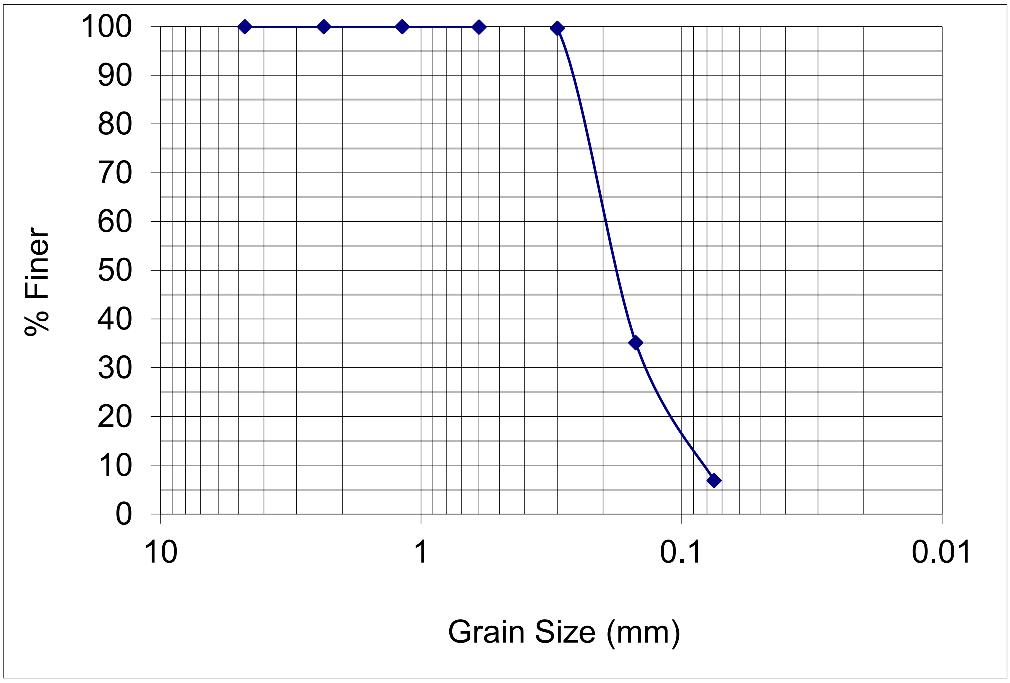


Figure C3: Grain size distribution curve of the bed sediment of the Dhaleshwari River near the Dhaleswari bridge site.

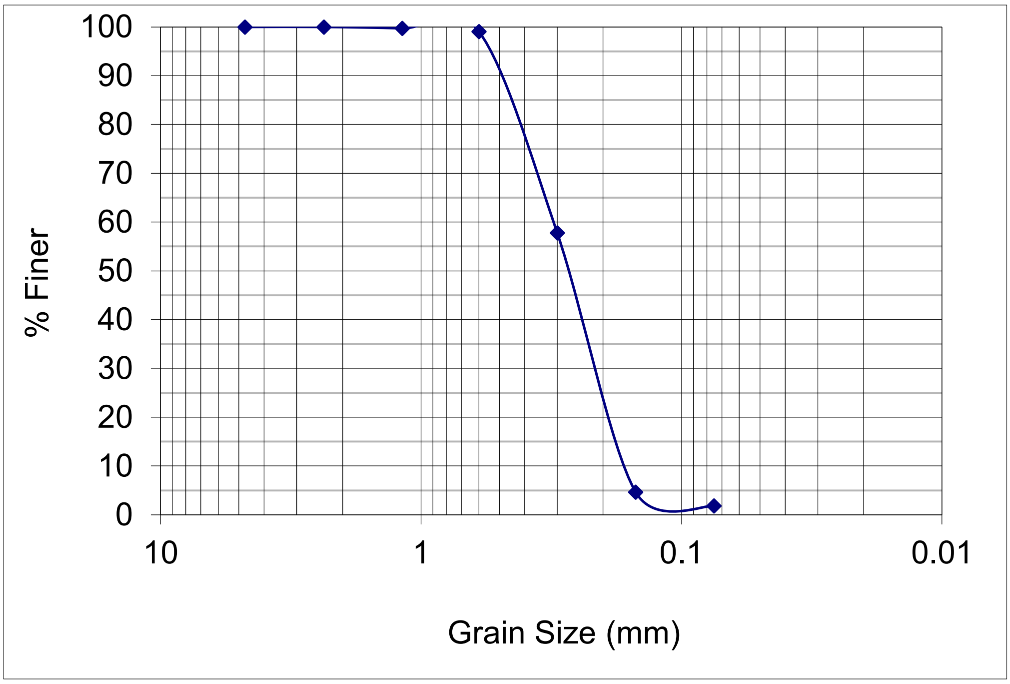
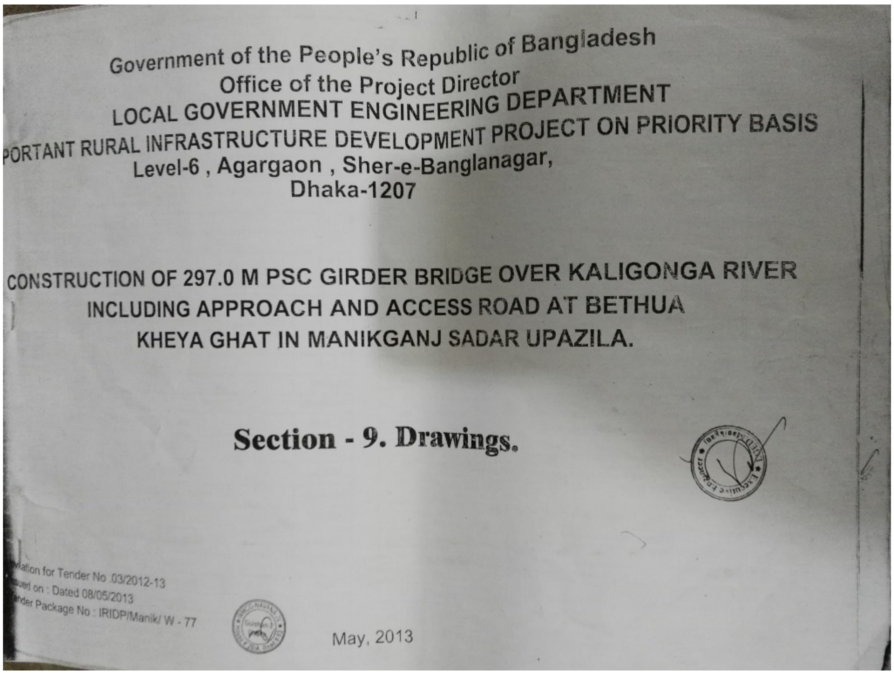


Figure C4: Grain size distribution curve of the bed sediment of the Dhaleshwari River near the Dhaleswari bridge site.

APPENDIX D
STRUCTURAL DRAWINGS OF THE BRIDGE PIER



Government of the People's Republic of Bangladesh
Office of the Project Director
LOCAL GOVERNMENT ENGINEERING DEPARTMENT
PORTANT RURAL INFRASTRUCTURE DEVELOPMENT PROJECT ON PRIORITY BASIS
Level-6 , Agargaon , Sher-e-Banglanagar,
Dhaka-1207

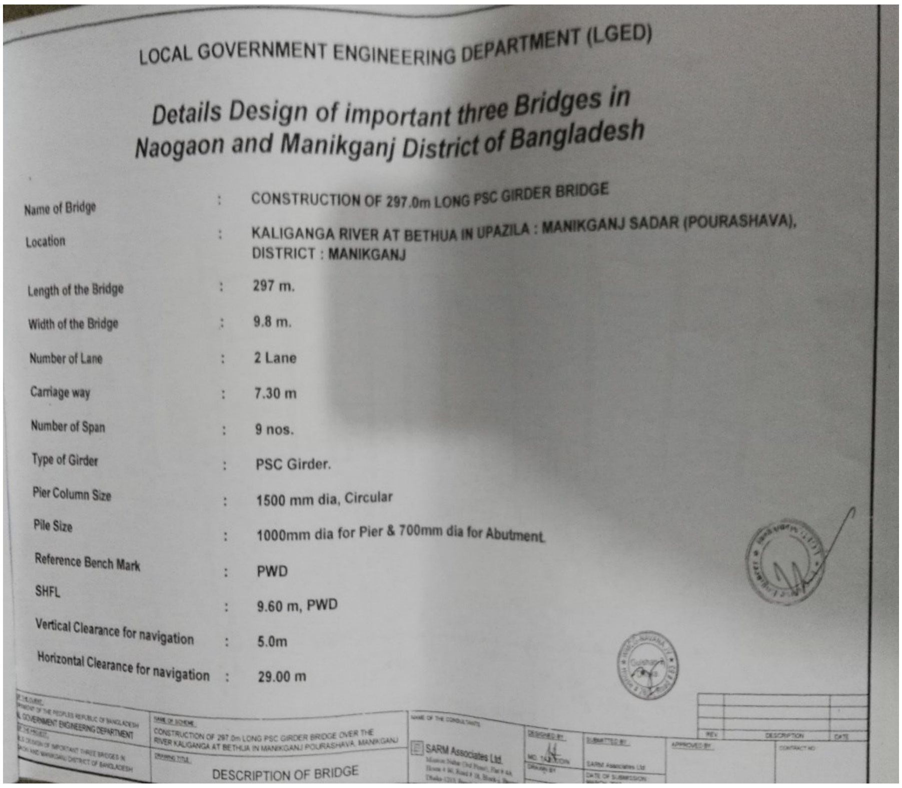
CONSTRUCTION OF 297.0 M PSC GIRDER BRIDGE OVER KALIGONGA RIVER
INCLUDING APPROACH AND ACCESS ROAD AT BETHUA
KHEYA GHAT IN MANIKGANJ SADAR UPAZILA.

Section - 9. Drawings.

...ation for Tender No .03/2012-13
...ued on : Dated 08/05/2013
...der Package No : IRIDP/Manik/ W - 77

May, 2013

Figure D1: Structural Drawing (Top page)



LOCAL GOVERNMENT ENGINEERING DEPARTMENT (LGED)

Details Design of important three Bridges in Naogaon and Manikganj District of Bangladesh

Name of Bridge	:	CONSTRUCTION OF 297.0m LONG PSC GIRDER BRIDGE
Location	:	KALIGANGA RIVER AT BETHUA IN UPAZILA : MANIKGANJ SADAR (POURASHAVA), DISTRICT : MANIKGANJ
Length of the Bridge	:	297 m.
Width of the Bridge	:	9.8 m.
Number of Lane	:	2 Lane
Carriage way	:	7.30 m
Number of Span	:	9 nos.
Type of Girder	:	PSC Girder.
Pier Column Size	:	1500 mm dia, Circular
Pile Size	:	1000mm dia for Pier & 700mm dia for Abutment.
Reference Bench Mark	:	PWD
SHFL	:	9.60 m, PWD
Vertical Clearance for navigation	:	5.0m
Horizontal Clearance for navigation	:	29.00 m

CONSTRUCTION OF 297.0m LONG PSC GIRDER BRIDGE OVER THE RIVER KALIGANGA AT BETHUA IN MANIKGANJ POURASHAVA, MANIKGANJ

DESCRIPTION OF BRIDGE

SARM Associates Ltd.

Figure D2: Structural Drawing (Description)

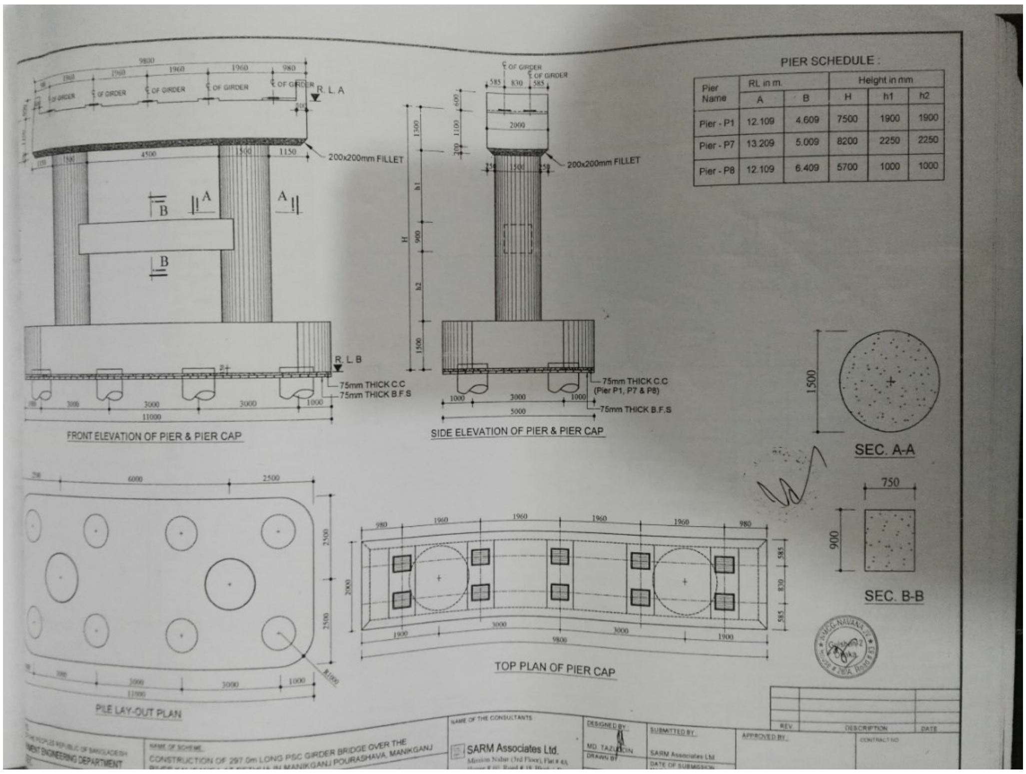


Figure D3: Structural Drawing (Piers 1, 7 and 8)

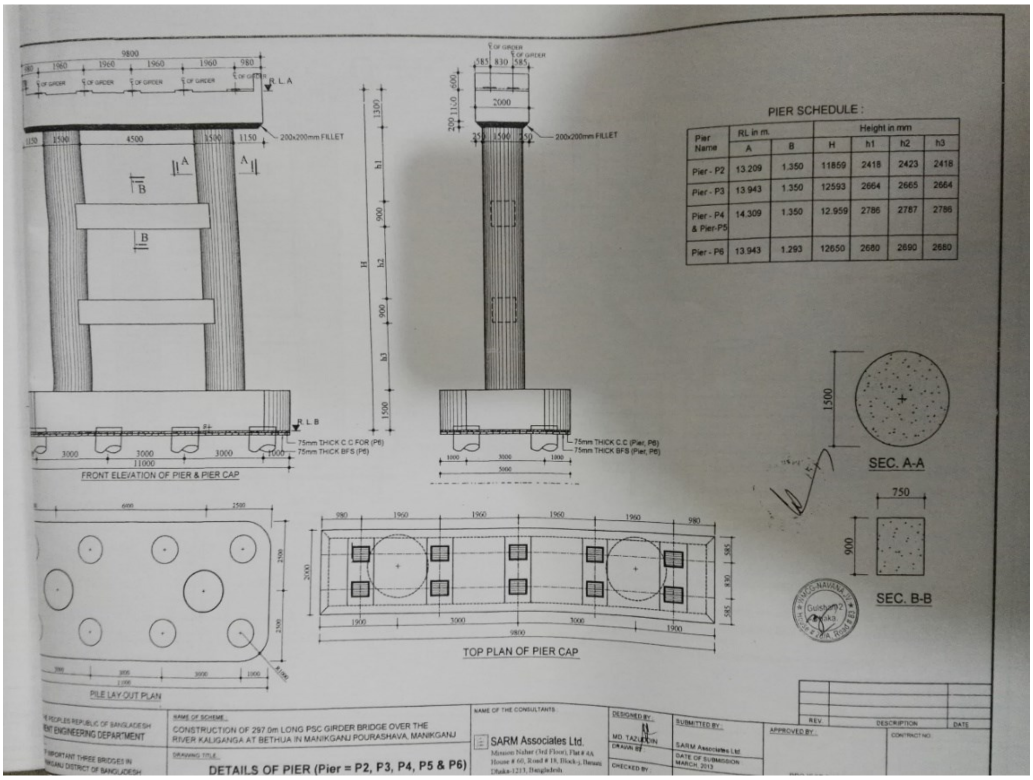


Figure D4: Structural Drawing (Piers 2, 3, 4, 5 and 6)

APPENDIX E
DETAILED CALCULATION OF LOCAL SCOUR

The estimation procedures of local pier scour are given only for the peak flow condition. In the peak flow condition, h = 8.29 m.

E1. Estimation of Local Scour Level by Using Breusers' Equation:

(1) We know, Breusers' empirical equation is $\frac{d_s}{b_p} = 1.4$

Using simple pier, that is, b_p = 1.50 m; we get d_s =2.1 m.
Using complex pier, that is, b_p = 4.02 m; we get d_s = 5.628 m.

(2) Height of sand dune (Δ) was estimated according to Julien et al. (1995),

$$\Delta = 2.5h^{0.7}{d_{50}}^{0.3}$$

Using hydraulic depth, h = 8.29 m and median grain size, d_{50} =0.16 mm; we get Δ = 0.46 m.

(3) Since, initial bed level was -1.5 m PWD and the estimated value was below the initial bed level.

(4) Estimated scour level = (1) - (2) - (3)
For simple pier, d_s = -2.56-1.5 = -4.06 m PWD
For complex pier, d_s = -5.88-1.5 = -7.59 m PWD

E2. Estimation of Local Scour Level by Using Laursen's Equation:

(1) We know that the mathematical expression of the Laursen's equation is,

$$\frac{d_s}{b_p} = 1.34\left(\frac{h}{b_p}\right)^{0.5}$$

Using simple pier, b_p = 1.50 m; we get d_s =4.73 m
Using complex pier, b_p = 4.02 m; we get d_s =7.74 m

(2) Sand dune height = 0.46 m
(3) Initial bed level = -1.5 m PWD
(4) Estimated scour level:
For simple pier, d_s = -6.69 m PWD and for complex pier, d_s = -9.70 m PWD.

E3. Estimation of Local Scour Level by Using Neill's Equation:

(1) Neill's equation, $\frac{d_s}{b_p}$ = K_s (For round pier, K_s = 1.5)

Using simple pier, b_p = 1.50 m; we get d_s =2.25 m

Using complex pier, b_p = 4.02 m; we get d_s =6.03 m

(2) Sand dune height = 0.46 m

(3) Initial bed level = -1.5 m PWD

(4) Estimated scour level:

For simple pier, d_s = -4.21 m PWD and for complex pier, d_s = -7.99 m PWD.

E4. Estimation of Local Scour Level by Using Jain and Fischer's Equation:

(1) Jain and Fischer's equation is, $\frac{d_s}{b_p} = 1.86\left(\frac{h}{b_p}\right)^{0.5}$

Using simple pier, that is, b_p = 1.50 m; we get d_s =6.56 m

Using complex pier, that is, b_p = 4.02 m; we get d_s =10.74 m

(2) Sand dune height = 0.46 m

(3) Initial bed level = -1.5 m PWD

(4) Estimated scour level:

For simple pier, d_s = -8.52 m PWD and for complex pier, d_s = -12.70 m PWD.

E5. Estimation of Local Scour Level by Using Chitale's Equation:

(1) Chitale's equation for local scour estimation, $\frac{d_s}{b_p} = 2.5$

Using simple pier, that is, b_p = 1.50 m; we get, d_s = 3.75 m

Using complex pier, that is, b_p = 4.02 m; we get, d_s =10.05 m

(2) Sand dune height = 0.46 m

(3) Initial bed level = -1.5 m PWD

(4) Estimated scour level:

For simple pier, d_s = -5.71 m PWD and for complex pier, d_s = -12.01 m PWD.

E6. Estimation of Local Scour Level by Using Melville's Equation:

(1) Melville's formula for narrow pier, $\frac{d_s}{b_p} = 2.4$

Using simple pier, that is, b_p = 1.50 m; we get, d_s = 3.75 m

Using complex pier, that is, b_p = 4.02 m; we get, d_s =10.05 m

(2) Sand dune height = 0.46 m

(3) Initial bed level = -1.5 m PWD

(4) Estimated scour level:

For simple pier, d_s = -5.71 m PWD and for complex pier, d_s = -12.01 m PWD.

E7. Estimation of Local Scour Level by Using FHWA Equation:

(1) FHWA recommended Colorado State University (CSU) equation,

$$\frac{d_s}{h} = 2K_1K_2K_3K_4\left(\frac{b_p}{h}\right)^{0.65} F_r^{\,0.43}$$

Using simple pier, that is, b_p = 1.50 m; we get, d_s = 3.07 m

Using complex pier, that is, b_p = 4.02 m; we get, d_s =5.84 m

(2) Sand dune height = 0.46 m

(3) Initial bed level = -1.5 m PWD

(4) Estimated scour level:

For simple pier, d_s = -5.03 m PWD and for complex pier, d_s = -7.80 m PWD.

E8. Estimation of Local Scour Level by Using Lacey's equation

(1) Lacey's regime formula, $d_s = 0.473\ \left(\frac{Q}{f}\right)^{\frac{1}{3}}$

Lacey's equation does not consider pier width and thus provides the same local scour for both simple and complex piers. We get, d_s = 6.98 m

(2) Sand dune height = 0.46 m

(3) Initial bed level = -1.5 m PWD

(4) Estimated scour level, d_s = -8.94 m PWD.

E9. Estimation of Local Scour Level by Using Modified Lacey's equation

(1) Modified Lacey's equation, $\frac{d_s}{b_p} = \left[0.47\, M^{\frac{1}{3}}\left(1 + 4.5\, \frac{b_p}{h}\right)^{\frac{1}{3}} - 1\right] \times \left(\frac{h}{b_p}\right)$

Using simple pier, b_p = 1.50 m; we get, d_s = 6.48 m

Using complex pier, b_p = 4.02 m; we get, d_s =9.52 m

(2) Sand dune height = 0.46 m

(3) Initial bed level = -1.5 m PWD

(4) Estimated scour level:

For simple pier, d_s = -8.44 m PWD and for complex pier, d_s = -11.48 m PWD.