Flow distributions and scouring around abutments in two-stage channels

Joko Nugroho

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**FLOW DISTRIBUTIONS AND SCOURING
AROUND ABUTMENTS
IN TWO-STAGE CHANNELS**

JOKO NUGROHO
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
2006

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AROUND ABUTMENTS
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JOKO NUGROHO

School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University
in fulfilment of the requirements for the degree of
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2006

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ABSTRACT

Flow and scouring around bridge support structures in an alluvial channel are important and interesting research topics in hydraulics, as they are related to the safety of the bridges and waterways. Two parts of bridge structures which usually occupy some portion of the flow area in channels are abutments and piers. An abutment is commonly placed in the floodplain of a two-stage channel. In a two-stage channel, the interaction between floodplain and main channel flow is an essential feature that causes a momentum exchange between them. The momentum exchange affects the flow distribution in the floodplain and main channel, and the scouring around the abutment.

The objectives of the present research are to study the flow distribution and scouring around abutment terminated at the floodplain of a two-stage channel under clear water scour condition. Flowfield observations and scouring experiments were done to achieve the objectives. The experiments were conducted in a 19 m long asymmetrical two-stage channel. Three different lengths of abutment were used in the experiments. Flowfield observations around an abutment were done for a flat bed and scoured bed condition to study the flow distribution in the initial condition and after scouring process had reached its equilibrium. In the scouring experiments, the flow discharges as well as the flow depth ratios were varied for different abutment lengths, and each experiment was done for a relatively long time to ensure that the equilibrium scour depth has been reached. Two experiments to study the effect of separating the interaction of the floodplain and main channel flow on the equilibrium scour depth were also done. The measured parameters in these experiments were the inflow component discharge, flow depth, velocity distribution around abutment and scour development with time.

Three types of scour hole geometry were observed in this study, i.e. the inverted cone type, edge-truncated inverted cone type and the extended upstream edge-truncated inverted cone type. For low flow depth ratios, a shallow secondary scour was formed and it propagate upstream near to the side wall after a maximum scour depth attained. This is due to the change of flow distribution of the approach flow in the floodplain after the scour depth reaches its maximum.

The presence of the abutment caused an increase of turbulence at the region close to and at the downstream of the abutment. Near the abutment tip, in the flat bed condition, the velocities at the lower depth were higher than those of the upper part due to the effect of the flow diversion and the associated downflow. The bed shear stress amplification in the flat bed condition for a point close to the abutment tip was 3.8 times larger than that of the approach section or 1.8 times larger than the critical shear stress of the sediment used. After the scour hole was formed, the velocity vectors around the abutment showed less deflection compared to that of the flat bed condition and an anticlockwise vortex, looking downstream, was formed at the section adjacent to the abutment.

A relationship linking the flow depth ratio to the approach flow on the floodplain is proposed. The corrected floodplain approach flow velocity is used in the formulation of a semi-empirical equation for the equilibrium scour depth, which involves bulk parameters related to the flow, sediment and abutment properties. The predictive model is able to give good estimate of the maximum scour depth for abutment terminating in the floodplain. An empirical relationship in the form of multivariate monomial for the non-dimensional average scouring rate, $d_{se}/(t_e \cdot U_f)$, where d_{se} = equilibrium scour depth, t_e = equilibrium scour time, and U_f = floodplain flow velocity, was proposed as a function of non-dimensional parameters of flow, sediment properties and abutment length.

Experiments on the effect of two separators of different length placed in the junction of the floodplain and the main channel were also done. The results on the effect of separating the flow interaction between floodplain and main channel showed that the flow velocity in the floodplain was lower in the non-interacting case compared to that of interacting condition. The former is due to the absence of the momentum exchange between floodplain and the main channel. The equilibrium scour depths in the non-interacting condition experiments were lower and the time to reach the equilibrium condition was shorter than that in interacting condition. In these experiments, the effect of the length of the separated reach at the upstream of the abutment was insignificant to the flow distribution and scouring process.

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LIST OF SYMBOLS

A	= approach flow area in the corresponding rectangular channel
A_i	= wetted area of subsection i
B	= un-contracted channel width
B_2	= contracted width at abutment site section
B_f	= floodplain width
b_{se}	= width of the scoured area
C	= speed of sound
C_1	= coefficient
C_2, C_3, C_4	= exponent
COH	= channel coherence
D	= sediment size
D^*	= dimensionless particle diameter
$DISDAF$	= discharge adjustment factor
d	= flow depth
d_f	= floodplain flow depth
d_m	= main channel flow depth
d_n	= sand diameter in which $n\%$ is finer
d_s	= scour depth
d_{se}	= equilibrium scour depth
d_{sc}	= scour depth at the two-stage channel
F_c	= Froude number at critical flow for particle movement
$F_{doppler}$	= change in received frequency (Doppler shift)
F_f	= Froude number of the approach flow in floodplain
F_o	= densimetric Froude number
F_{oe}	= equivalent densimetric Froude number
F_{source}	= frequency of transmitted sound
g	= gravitational acceleration
K_G	= geometrical factor
K_s	= abutment shape factor
K_s^*	= abutment shape factor for abutment of intermediate length

K_θ	= abutment alignment factor
K_θ^*	= abutment alignment factor for abutment of intermediate length
k^+	$= 1/2 \cdot (u_{rms}^2 + v_{rms}^2 + w_{rms}^2) / u_*^2$
L	= abutment length
L_e	= equivalent abutment length
M	= contraction ratio
n_M	= Manning roughness coefficient
n_{Mf}	= Manning roughness coefficient of the floodplain
n_{Mm}	= Manning roughness coefficient of the main channel
Q	= the total discharge of the approach flow
Q_2'	= discharge within the width of b_{se}
Q_2''	= discharge within the un-scoured width at abutment site section
Q_a	= discharge component intercepted by abutment
Q_c	= discharge at unobstructed width
Q_f	= discharge at the main channel of a two-stage channel
Q_i	= discharge in the area upstream of the abutment
Q_m	= discharge at the floodplain of a two-stage channel
Q_o	= discharge along unobstructed width at the approach section
Q_r	= discharge at corresponding rectangular channel
Q_w	= discharge over an arbitrary width w_*
Q_w'	= discharge component related to specific width of channel at end of abutment
R	= hydraulic radius
r	= constant for a fraction of scour depth
S	= specific gravity
S_e	= energy slope
t_e	= equilibrium time
U	= depth-averaged streamwise velocity component
U_1	= average velocity at approach section
U_{2c}	= average velocity within b_{se} at equilibrium condition or $t = t_e$
U_{2m}	= average velocity within b_{se} at initial time or $t = 0$
U_c	= critical mean velocity for sediment motion

U_e	= width-averaged equivalent velocity
U_f	= floodplain flow velocity
U_i	= mean velocity of subsection i
U_{mc}	= improved mean flow velocity at main channel
$U_{mc-DCM-V}$	= mean flow velocity at main channel, obtained by Vertical Divided Channel Method
$U_{mc-DCM-H}$	= mean flow velocity at main channel, obtained from Horizontal Divided Channel Method
u	= time-mean streamwise velocity
u'	= streamwise velocity fluctuation
u_{rms}	= rms value of streamwise velocity
u^+	= u_{rms} / u_*
u_*	= shear velocity
u_{*l}	= shear velocity at approach section
u_{*m}	= maximum shear velocity
u_{*c}	= critical shear velocity
uv^+	= $-\overline{u'v'} / u_*^2$
uw^+	= $-\overline{u'w'} / u_*^2$
V	= velocity of source relative to receiver
v	= time-mean spanwise velocity
v'	= spanwise velocity fluctuation
v_{rms}	= rms value of the spanwise velocity component
v^+	= v_{rms} / u_*
w	= time-mean vertical velocity
w'	= vertical velocity fluctuation
w_{rms}	= rms value of the vertical velocity component
w^+	= w_{rms} / u_*
w_*	= an arbitrary width
y	= spanwise position
z	= vertical position
z_a	= local flow depth

z_o	= roughness height parameter
κ	= von Karman coefficient
ν	= kinematic viscosity of fluid
θ	= angle of the abutment alignment to the flow direction
θ_c	= Shields' entrainment function
Δ	= $(\rho_s - \rho)/\rho$
ρ	= density of fluid
ρ_s	= density of sediment
σ_g	= geometrical standard deviation of sediment
τ_a	= shear stress at approach section
τ_b	= bed shear stress
τ_c	= critical shear stress for related sediment size
τ_{*c}	= dimensionless critical bed shear stress or Shields number
τ_{yx}	= Reynolds shear stress on a horizontal plane
τ_{zx}	= Reynolds shear stress on a vertical plane
ξ	= weighting factor for weighted divided channel method

CHAPTER 1. INTRODUCTION

1.1. Background

The riverbed around structures built in waterways is often susceptible to an intense, local bed level degradation or scour. These structures include the bridge-support structures, i.e. abutments and piers of bridges. The scouring process is a result of an increasing bed shear stress and transport capacity along a certain part of a cross section in a particular reach. An intensive scouring process, usually in high flow season, can undermine the foundation of a support structure and may cause a partial or total failure of the bridge system, thereby interrupting the land and waterway transportation activities. This description shows the significance of the research on bridge hydraulics in providing knowledge on the scouring process to minimize the risk of bridge failure.

Bridge scour can be divided into three types; i.e., local scour, contraction scour, and general scour. Local scour is caused by the presence of a structure which occupies a relatively short reach of the channel. Contraction scour is caused by the decrease of the width of an open channel in a considerable reach of the channel. General scour is caused by the change of fluvial systems components that influence the sediment transport along the influenced reach in the longer order than local and contraction scour. Based on these criteria, scouring around river-crossing support-structures is essentially a localized phenomenon.

Study of localized scour can be grouped as pier-focused, abutment-focused, and the whole system of bridge hydraulics. The earlier research studies started with the structures sited in rectangular open channels. Recent research considers real conditions by accommodating the case of river-crossing support-structures, especially for abutment, in two-stage channels. However, the parameters used to quantify the two-stage channel flow effect are rather difficult to be implemented. Hence, an approach based on the basic parameters of the flow will be helpful to address the two-stage channel effect in the scouring around abutment in floodplains.

The flow in two-stage channels and around abutment are complex-phenomena that have been studied extensively. However few studies investigate the flow distribution related to local scour around abutments sited in a floodplain.

1.2. Objectives and Scope

The objectives of the present research are to determine how the flow field around an abutment in a two-stage channel influences abutment scour. The abutment is placed in the floodplain region, and flow on the floodplain is clear-water flow, i.e., there is no sediment transport from upstream except in the area influenced by the presence of the abutment.

To achieve its objectives the study is divided into three parts. The first part is a literature study on the hydraulics of two-stage channels, and flow distribution and scour around abutment in rectangular and two-stage channels. The review on two-stage channels hydraulics is very important to understand the flow characteristics of two-stage channels. The important characteristic of two-stage channels is the momentum exchange due to the interaction between the floodplain and main channel flow along the junction of the floodplain and main channel. The momentum exchange affects the flow and shear stress distribution in the floodplain and main channel. The review of flow and scour at abutments provides explanations of the local flow and shear stress distribution around abutments, and their relationship with the scouring process. The review of scour around abutment placed in two-stage channels shows how the two-stage channel geometry influences the scouring process for different place of abutment termination, i.e. in the floodplain, junction or main channel.

The second part is the experimental study. This part aims to elucidate the scouring process and flow behavior around an abutment terminating in the floodplain. Observations of flow distributions around an abutment were done for both flat bed and scoured bed conditions. In the scouring experiments, the effect of the two-stage channel geometry in terms of the flow depth ratio, i.e. ratio of floodplain flow depth to the main channel flow depth, to the scouring process were observed for different discharges and abutment lengths.

The third part of the study discusses and analyses the experimental results. The experiments on the flow characteristics around abutment are discussed in terms of the mean velocities, turbulent intensities, bed shear stresses and Reynolds shear stresses. The experiments on scouring around abutment are analyzed based on a semi-empirical analysis of the flow and scouring process taking into account the significant parameters, i.e. abutment dimension, approach flow characteristics and sediment properties.

1.3. Thesis Organization

This thesis is organized into six chapters. Chapter One introduces the background, objectives and scope of the research. Chapter Two presents the review of the published literatures related to hydraulics of two-stage channels, and flow and scour around abutments. Chapter Three provides the details of the experiments, materials and measurement techniques of the experimental works carried out in the present study. Chapter Four presents the results and discussion of the flow distributions around abutment placed in the floodplain during the initial flat bed condition and in the scoured condition at the equilibrium scour condition. Chapter Five presents a discussion on the scouring process, and proposes a predictive model to estimate the equilibrium scour depth and average scouring rate based on the published and present study data. Finally, the conclusions and recommendations for further researches, drawn from the results of the present study, are presented in Chapter Six.

CHAPTER 2. LITERATURE REVIEW

2.1 Introduction

Scour of alluvial channels is dependent on the force or stress exerted by the flow on the exposed bed sediments. In general, this force is related to flow distribution. The characteristics of flow distribution in two-stage (compound) channels are different from those in single channels. This difference will affect the channel scour around an abutment terminating at the floodplain of a two-stage channel. Hence, the study of scour around an abutment on a floodplain is closely related to two specific fields of research in hydraulics: flow in two-stage channels, and scour around abutments. To give background to these research fields, this chapter reviews previous researches on flow around abutments, flow in two-stage open channels, and localized scour around abutment.

2.2 Scouring Process around Abutments

Scouring in rivers is the erosive action of water that causes bed and bank materials to be eroded and transported downstream. It is a natural phenomenon of rivers (Breusers and Raudkivi, 1991). Scour types can be divided into:

- General scour, which is usually the result of natural processes, irrespective of the presence of structures. Sometimes this is also due to man-made structures such as dams.
- Constriction scour, which occurs if there is a narrowing of a water course which in turn causes an increase in the sediment transport capacity along the narrowed part of the channel.
- Local scour, which is the direct result of placement of man made structures that partially obstruct the flow in the waterways.

The state of a local scouring process around a structure, based on sediment transport conditions, can be further divided into:

- Clear-water scour, a state when the ambient flow is not capable to transport the bed material from the approach section and the scour is localized at the abutment location.
- Live-bed scour, a state when the ambient flow is capable to transport the bed material, such that there is sediment transport in the channel.

Scour around an abutment or a pier are considered as a localized scour phenomenon. The similarity of scouring processes at abutments to that of piers is well recognized by some researchers (Melville and Coleman, 2000). Kwan (1984) observed that the depth of scour around a semi-circular abutment was slightly lower than that of a cylindrical pier of the same radius. He suggested that this difference was possibly caused by development of the boundary layer along the wall where the abutment is attached.

The behaviour of flow around an abutment, and local scour at an abutment is well described in some published articles such as Breusers and Raudkivi (1991), Kwan and Melville (1994) and Melville and Coleman (2000). Figure 2.1 depicts the main features of the flow past an abutment i.e. surface roller, down flow, principal vortex and wake vortex. The mechanism of scouring around an abutment is strongly associated with these features. The entrainment zone is located at the principal vortex area near to the upstream corner of the protruded part of the abutment. The role of the principal vortex is similar to the horseshoe vortex in the pier case. A significant finding by Kwan (1988) was that for deeper flows the vortex system is not affected by flow depth. This implies that there should be a limit of flow depth to influence the scour development (Melville and Coleman, 2000). For a long abutment, Kwan (1984) observed that there is a large secondary horizontal circulating reversed eddy at the upstream of the abutment, and the presence of principal vortex induced the growth of a weak secondary vortex of opposite rotation. As a result, a ridge is formed along the line between the principal and secondary vortex (see Figure 2.2).

Figure 2.1. Flow structure at a wing-wall abutment (after Breuser and Raudkivi, 1991).

Figure 2.2. Formation of a ridge by primary and secondary vortex (after Kwan and Melville, 1994).

2.3 Flow around Abutments

Rajaratnam and Nwachukwu (1983a) conducted experiments on the structure of turbulent flow near groin-like structures at a planar bed. They used Prandtl-type pitot-static tubes to measure the velocity at the undisturbed flow region, and three-tube yaw probe for measurement in the skewed flow region. The bed shear stress was measured using the Prandtl-tube as a Preston tube. It was noticed that amplification in bed shear stresses near the nose of abutment and at the area close to the groin ranged from 3 to 5 times the bed shear stresses at the approach section (see Figure 2.3). It was observed that the velocity along a vertical axis near the nose of the groin and at the backward flow region was almost uniformly distributed. The formation of a jet-like flow occurred close to the bed at the downstream of the groin.

Kwan and Melville (1994) reported the results of a detail observation on the flow-field in a local scour hole around a wing-wall bridge abutment by means of the hydrogen bubble technique. Their experiment was conducted under clear-water scour condition. At the approach section, the flow was seen as one-dimensional flow. When it was approaching the abutment, the flow started to change its velocity distribution in the horizontal, lateral and vertical directions, and formed a three-dimensional flow pattern. The flow at the abutment nose was nearly uniform, a fact that was also found in Rajaratnam and Nwachukwu's (1983a) work. The approach flow will separate into three main directions upon reaching the abutment. In general the approach flow will dive downward and then deflected outward from the abutment. The upper layer will move towards the abutment, while the lower layer near the bed will move away from the abutment. These movements generate primary vortex that rotates in a counter-clockwise direction (looking downstream). Beside the primary vortex, there was a secondary vortex of small intensity, which rotates in a clock-wise direction.

x: longitudinal coordinate; z: lateral coordinate; b: length of groin

Figure 2.3. An example of bed shear stress contour around a vertical plate (after Rajaratnam and Nwachukwu, 1983a).

Molinas, Kheireldin, and Wu (1998) investigated the shear stress amplification around a vertical wall abutment. They analyzed the shear stress at the nose of the abutment as the sum of shear stress amplification caused by contraction and shear stress amplification due to the abutment structure. The shear stress amplification caused by contraction was calculated by means of the continuity and energy equations, and formulated as a function of the channel opening ratio, approach Froude number, and protrusion length. The flow conditions are limited to moderate contractions ($0.7 < M < 1$, where M is a ratio of the unobstructed width at the abutment site cross section to the width of the approach cross section) and sub-critical approach flows. They obtained a total amplification shear stress factor as high as 10 at the abutment site compared to that at the approach section. Compared to Rajaratnam and Nwachukwu's (1983a) work, Molinas, Kheireldin and Wu (1998) applied a wide range of Froude number (0.3 to 0.9).

Ahmed and Rajaratnam (2000) reported that there was still a lack of understanding of the flow around abutments. In their single experiment they focused on the approach flow, the deflected and skewed flow near the abutment. They found that there was a complex 3D skewed flow in the upstream and surrounding regions of the abutment. The decrease of boundary layer thickness was found at the plane of

symmetry, where the vertical distribution velocity was found to be nearly uniform at the upper portion. It was found that the accelerated flow concentrated mostly at the lower level near the abutment nose where the deepest scour holes are generally found. However, there is no specific conclusion related to the involved parameters on the observed phenomena as they presented results of single experiment.

The above review shows that there are two main facts which are significant for scour analysis: i.e., three dimensional flow, and the shear stress amplification caused by the abutment. Pertaining to the case of an abutment in a floodplain, this shear stress is the one that will be the main contributor to scour around the structure. The high shear stress will dislodge the sediments from the bed, and transport them downstream by the vortex system generated at the abutment.

2.4 Flow in Two-stage Channels

2.4.1 Flow characteristic in two-stage channels

The effect of the interaction mechanism on shear distribution in channel of complex section had been described by Myers and Elsayy (1975). At the time of their study, vortices with vertical axes along the junction between main channel and floodplain had been known as a momentum transfer mechanism from the main channel to the floodplain. The results of their study show that there was a decrease (up to 22%) in main channel shear and an increase (up to 260%) in floodplain shear (see Figure 2.4) compared to separated floodplain and main channel condition. Figure 2.4 shows that the increase of floodplain shear is higher for lower flow depth. Although significant findings were presented, they did not proposed any model which reflect the effect of two-stage channel geometry on shear distribution.

Rajaratnam and Ahmadi (1979) presented typical distributions of velocity profile and bed shear stress of an asymmetric two-stage channel obtained from their experiments. Typically, the velocity in the main channel reaches a maximum value at a certain distance from the bed, and decreases as the distance increases. They derived a non-dimensional relationship between the velocity parameters and physical length scales of the channel using the logarithmic profile of uniform flow

and found a similarity of flow profile at the main channel and the floodplain for each horizontal plane for every vertical position which they had measured. They found that the flow interaction, between the main channel and the floodplain, influenced the flow at the centre line of the main channel as the main channel width was not sufficiently large.

Figure 2.4. Increase of floodplain shear stress due to interaction between floodplain and main channel (after Myers and Elsayy, 1975).

Bhowmik and Demissie (1982) analyzed flow in several natural two-stage channels. They used field data of five rivers in the USA (Salt Creek, Minnesota River, Sangamon River, Tuxachanie Creek, and Big Black River) that had different types of main channel and floodplain. Their analysis showed that these rivers tend to behave as a single channel as the return period of flow increases. It was found that for a flood return period about 40 years or more, the rivers would have the

characteristics of a single channel (see Figure 2.5). This is because the flow depth will increase with return period. As the flow depth increases, the velocity difference between floodplain and main channel flow decreases and the two-stage channel will act almost as a single channel for further flow depth increase. However, it should be noted that the return period of flow may be changed due to the change in land use characteristic of the drainage area.

Figure 2.5. Relationship among the ratio of floodplain over total cross sectional area, the ratio of floodplain over total discharge and the return period of the flow of five rivers observed by Bhowmik and Demisie (1982).

Knight and Demetriou (1983) analyzed the apparent shear force acting in the vertical interface between floodplain and main channel, and the horizontal interface dividing the lower main channel at the floodplain level from the region above. It was shown that those apparent shear forces depends on the width ratio and flow depths. The apparent shear force values on the vertical interface were always positive which means the floodplain retarded the main channel flow. The apparent shear force values on the horizontal interface were positive at low relative depths and negative at high relative depths. The ratio of the mean velocity in the

floodplain and main channel over the two-stage channel mean velocity was found to be closed to unity as the stage increases (see Figure 2.6). In Figure 2.6, d_f , d_m , B and b refer to floodplain flow depth, main channel flow depth, half width of cross section and half width of the main channel, respectively. Similar behaviour was observed in natural channels by Bhowmik and Demisie (1982) (see Figure 2.7).

Figure 2.6. Relationships of flow-depth ratio to the normalized mean velocity in the floodplain and the main channel (after Knight and Demetriou, 1983), where $\bar{u}$ is mean velocity in the floodplain or main channel and U is cross-sectional mean velocity.

Figure 2.7. Relationships of mean velocities with flow depths of (a) Sangamon river and (b) Salt Creek river in the USA (after Bhowmik and Demisie, 1982).

Tominaga and Nezu (1991) investigated the effect of channel roughness and relative flow depth ratio of floodplain to the main channel upon the turbulent structure. The turbulent structures of two-stage channels were observed by means of a fibre-optic laser Doppler anemometer system in an asymmetric two-stage channel. They observed a pair of longitudinal-vortex (secondary current) along the junction of main channel and floodplain. It was observed that the strength of the vortex at floodplain and at main channel was strongly influenced by the flow depth ratio of floodplain to main channel depth. At higher flow depth ratio, the floodplain longitudinal vortex was dominant, but for the lower ratio the main channel longitudinal vortex was stronger. The turbulence intensities showed an increasing magnitude at the junction. The bed roughness did not have significant effect on the span-wise bed shear stress distribution. The momentum transfer from the main channel towards the floodplain was analyzed by the Reynolds stress distribution at the junction. Their evaluation showed that the secondary current near the junction has an important role on the span-wise momentum transport.

2.4.2 Computation of component discharge in two-stage channel

The conventional method of discharge computation in a two-stage channel based on the uniform flow formula is by dividing the channel into many subsections and then analysing them separately as single channel, and finally sum-up all of those component discharges. As there is an interaction between the main channel and the floodplain, as reviewed from many research reports in the previous sections, this condition will influence the conveyance in two-stage channels.

Keller and Rodi (1988) developed a two-dimensional κ - ϵ turbulence model to calculate the turbulent shear stress and the bed shear stress terms using quadratic friction law to obtain the flow characteristics parameter of a two-stage channel. The difference between model and experimental results on velocity and shear stress distribution occurred mainly near the wall and the main channel banks. The transverse mixing in the model was found to be lower than that in the experiment.

Four methods of computation of two-stage channel flow conveyance, i.e. separate channel method, inclined interface method, area method and K-method, were discussed by Stephenson and Kolovopoulos (1990) to investigate the effects of the momentum transfer. The model proposed by Prinos and Townsend (1984), was used as the parameter to compute the apparent shear stress needed in some of the conveyance computation methods. According to Stephenson and Kolovopoulos (1990), area method was found to be the most reliable in the discharge computation. In this method, there is an additional area to be included to the floodplain area or subtracted from the main channel. This area is obtained by employing equilibrium of force equation obtained by the vertical interface and inclined interface. The momentum transfer caused the shift of rating curve, a delay of water level falling, an increase in flood plain flow and a decrease in the main channel carrying capacity. It should be noted that the range of applicability of the chosen method is limited to the applicability of Prinos and Townsend's (1984) apparent shear stress model.

It can be concluded that interaction between floodplain and main channel is the main character which affect two-stage channel conveyance. Ackers (1993)

proposed a discharge calculation method based on this interaction. Two factors, i.e. discharge adjustment factor (DISADF) and channel coherence (COH), were introduced. The DISADF is the ratio of actual discharge to the sum of the discharge estimation of main channel and floodplains computed using a resistance formula, while the COH is defined as the ratio between the channel conveyance obtained from a calculation of two-stage channels as a single channel cross-section to that obtained from the sum of separate channel. These conveyance values are computed by a standard friction formula. Overall resistance for the computation was obtained using perimeter weighted friction factor.

According to the experimental results, especially on DISADF, Ackers (1993) defined four regions of flow behaviour as shown in Figure 2.8. Region 1 is for shallow depths, where the interference effect increases progressively with depth until the flow depth ratio is equal to 0.2, and the loss of conveyance is over 10 percent. Region 2 is for the flow depth ratio between 0.2 and 0.4 and the interference is decreasing and reaching its minimum interference at 0.4. At that flow depth ratio, the loss of conveyance is about 4 percent. At region 3, where the flow depth ratio is between 0.4 and 0.5, the interference increases again and the minimum loss of conveyance is around 6.5 percent. For flow depth ratio greater than 0.5, the two-stage channel can be considered as a single channel.

Ackers (1993) provide a practical method in two-stage channel discharge computation based on the regional depth characteristic. This shows nonlinearity of relationship between the characteristic of two-stage channel with flow depth.

Figure 2.8. Discharge Adjustment Factor (DISADF) versus Flow Depth Ratio and Channel Coherence (COH).

Lambert and Myers (1998) proposed a weighted divided channel method (WDCM) for prediction on stage and discharge relationship. The development of WDCM was initiated by conceptualising the interaction between main channel and floodplain as a certain extent of influence of floodplain wetted perimeter to the main channel. The mean flow velocity at the main channel is obtained from the formula:

$$U_{mc} = \xi U_{mc-DCM-V} + (1 - \xi) U_{mc-DCM-H} \quad (2.1)$$

where U_{mc} = improved main channel velocity, ξ = weighting factor, $U_{mc-DCM-V}$ = main channel velocity obtained by vertical divided channel method, and $U_{mc-DCM-H}$ = main channel velocity obtained by horizontal divided channel method.

The mean flow velocity at the floodplain is also obtained using the same formula, in which the velocity terms in Equation (2.1) are replaced by the floodplain velocity. It was mentioned that the weighting factor was not necessarily the same. The weighting factor of 0.5 was suggested for smooth floodplain while 0.2 for rough floodplain.

Lambert and Myers' (1998) method is simple, but justification on the weighting factor for wide range of application is needed.

Bousmar and Zech (1998) proposed a one-dimensional approach on flow computation method for flow computation in two-stage channels based on model of momentum transfers between main channel and floodplain. Geometrical changes in cross sectional are also modelled as a process of momentum transfer. The effect of both momentum transfers was included in one-dimensional flow equation as an additional head loss. The advantage of this method is that the energy slope can also be computed directly.

2.5 Studies on Scouring around Abutments

2.5.1 Abutment scour in rectangular channels

Laursen (1960) extended a solution of scouring at a channel contraction to compute depth of scour around an abutment in rectangular channel. The idea was based on observation that the degree of contraction will not affect local depth of scour as long as there is no overlapping of the scour holes around neighboring constructions. He assumed that the scour depth for long contraction is a fraction $1/r$ of the abutment scour depth and the scour width for normal sand is about 2.75 times the depth of the scour as shown in Figure 2.9. His experiments were conducted in live-bed condition. The solution proposed is as follows:

$$\frac{Q_a}{Q_w} \frac{w_*}{d} = 2.75 \frac{d_{se}}{d} \left[\left(\frac{1}{r} \frac{d_{se}}{d} + 1 \right)^{7/6} - 1 \right] \quad (2.2)$$

and

$$\frac{Q_w}{w_*} = \frac{Q_c}{2.75 d_{se}} \quad (2.3)$$

where Q_a = discharge at the upstream of the abutment, Q_w = discharge over an arbitrary width w_* , Q_c = discharge at the upstream of the unobstructed width, d_{se} =

equilibrium scour depth, d = approach flow depth and r = a factor for a fraction of the scour depth.

Figure 2.9. Definition sketch of variables on Laursen's scour depth formula.

Gill (1972) found that the effect of approach flow depth and sediment size on the equilibrium depth of scour holes around abutments was found to be significant. Even though the effect of abutment length was not clearly discussed in his work, it was included in his formulation of equilibrium scour depth. The empirical equation was based on Du Boys bed load equation and scour in long constriction, and valid for the range of his experimental data.

$$\frac{d + d_{se}}{d} = 8.375 \left(\frac{D}{d} \right)^{0.25} \left(\frac{B}{B-L} \right)^{6/7} \left[\frac{1}{\left(\frac{B}{B-L} \right)^{1/n} \left(1 - \frac{\tau_c}{\tau_a} \right) + \frac{\tau_c}{\tau_a}} \right]^{3/7} \quad (2.4)$$

in which $1 - \tau_c/\tau_a = 0$ for $\tau_c/\tau_a \geq 1$, d = approach flow depth, d_{se} = equilibrium scour depth, D = sediment size, B = uncontracted channel width, L = abutment length, n

= a numerical exponent, τ_a = shear stress at approach section, and τ_c = critical shear stress for related sediment size. The equation for equilibrium condition in clear water scour is obtained for $\tau_c/\tau_a > 1$ and maximum scour depth is when $\tau_c/\tau_a = 1$.

Melville (1992) developed empirical scour formulations for two cases i.e. short abutment and long abutment based on the extensive laboratory study data on abutments scour at Auckland University, New Zealand. Various factors of flow intensity, flow depth, sediment gradation, abutment length, shape and alignment were determined and included in the formulation. In his analysis, the sediment gradation factor was assumed as unity, which is a condition where sediment size has no effect and the approach was based on the maximum envelope curve of scour depth data relevant to each factors. The approach is similar to a method for local scour around bridge pier developed by Melville and Sutherland (1988). Melville (1992) suggested further study, especially on the impact of river geometry and lateral flow distribution.

Melville's (1992) equations for the equilibrium scour depth are as follows.

$$d_{se} = 2K_s L, \text{ for } \frac{L}{d} < 1, \text{ short abutment} \quad (2.5 \text{ a})$$

$$d_{se} = 10K_\theta d, \text{ for } \frac{L}{d} > 25, \text{ long abutment} \quad (2.5 \text{ b})$$

$$d_{se} = 2K_\theta^* K_s^* \sqrt{Ld}, \text{ for } 1 \leq \frac{L}{d} \leq 25, \text{ intermediate abutment} \quad (2.5 \text{ c})$$

where d_{se} = equilibrium scour depth, L = length of abutment, d = approach flow depth, K_θ = alignment factor, K_s = shape factor, K_θ^* = alignment factor for intermediate abutment, varies linearly from unity at $L/d = 1$ to K_θ for $L/d \geq 3$, and K_s^* = shape factor for intermediate abutment, varies linearly from unity at $L/d = 25$ to K_s for $L/d \leq 10$.

The method proposed by Melville (1992) is likely to have a conservative result due to the envelop approach.

Lim (1997) proposed a formula for clear water scour depth around abutment, based on the continuity equation and logarithmic velocity distribution for fully rough flow. The two equations were applied at the scour-hole cross section. The proposed equation is a function of the approach flow depth (d), Shields' entrainment function (θ_c), densimetric Froude number (F_o), abutment length (L) and representative sand diameter (d_{50}). Lim's equation is

$$\frac{d_{se}}{d} = K_s (0.9X - 2) \quad (2.6)$$

where

$$X = \left\{ \theta_c^{-0.375} F_o^{0.75} \left(\frac{d_{50}}{d} \right)^{0.25} \left[0.9 \left(\frac{L}{d} \right)^{0.5} + 1 \right] \right\} \quad (2.7)$$

The equation is only valid for $X > 2.22$, and it gives $\frac{d_{se}}{d} = 0$ when $X = 2.22$.

The densimetric Froude number (F_o) in Equation (2.6) is defined as:

$$F_o = \frac{U}{\sqrt{g(S-1)d_{50}}} \quad (2.8)$$

where U = approach flow velocity, g = gravitational acceleration, S = specific gravity of the sediment.

Kothyari and Ranga Raju (2000) proposed an analogous pier-scour method to assess abutment scour. They define first the pier that experienced the same drag force as the given abutment under given flow conditions. Then they derived the analogous pier that will give the same scour depth caused by the abutment. This method does not seem to be practical compared to the other proposed method.

2.5.2 Abutment scour in two-stage channels

The cross section of natural rivers usually consists of a main channel and one or two floodplains, hence the shape of cross section cannot be simplified by a rectangular one. The following review presents some of the works in this area. A summary of two-stage channel geometrical parameters of the works by Sturm and Janjua (1994), Dongol (1994), Kouchakzadeh and Townsend (1997), Cardoso and Bettess (1999), Lim and Yu (2001) and the present study are shown in Table 2.1.

Sturm and Janjua (1994) investigated the role of two-stage channel hydraulics in the clear-water scour process around an abutment which terminates in the floodplain of a horizontal two-stage channel. The scouring duration in their experiments lasted for 10 to 12 hours. Visual observations were used to determine the state of scouring process. According to the present author's experience, the rate of scour depth development at near equilibrium phase usually takes a long time, hence it is possible that the scour depths measured in Sturm and Janjua's (1994) work were less than the scour depth at equilibrium state.

The parameters which they considered important in the scouring process were Froude number of the approach flow in the floodplain ($F_f = U_f / \sqrt{gd_f}$) and the Froude number of critical flow for particle movement ($F_c = U_c / \sqrt{gd_f}$) and the discharge contraction ratio ($M = Q_o / Q$); where u_f = floodplain flow velocity, u_c = critical mean velocity for sediment motion in the floodplain, Q_o = discharge along unobstructed width at the approach section, and Q = the total discharge of the approach flow. They proposed a formula in terms of these parameters based on regression analysis and best-fit linear equation of the experimental data, i.e.

$$\frac{d_{se}}{d_f} = 7.70 \left[\frac{F_f}{MF_c} - 0.35 \right] \quad (2.9)$$

They emphasized the contraction ratio, M , as the significant parameter which reflects the two-stage channel geometry on scour around abutment.

Table 2.1. Range of geometrical properties of experiments on scour around abutment in two-stage channels.

No.	Researchers	Width			Flow depth			Abutment Length L (m)	L/Bf	Channel		
		Floodplain B _f (m)	Main channel B _m (m)	Ratio B _f /B _m	Floodplain d _f (m)	Main channel d _m (m)	Ratio d _f /d _m			Length (m)	Slope S	Cross section
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
1	Sturm and Janjua (1994)	2.490	0.310	8.03	0.11	0.23	0.48	0.58 1.17 1.55	0.23 0.47 0.62	5.18	Horz.
		2.590	0.200	12.95	0.08	0.29	0.28	1.17 1.55	0.45 0.60	5.18	Horz.	
2	Dongol (1994)	0.000	2.400	0.00	0.05 - 0.22	0.10 - 0.27	0.50 - 0.81	1.39	-	15.1	n.a.
		0.490	1.910	0.26	0.05 - 0.22	0.10 - 0.27	0.50 - 0.81	1.39	2.84	15.1		
		0.835	1.565	0.53	0.05 - 0.22	0.10 - 0.27	0.50 - 0.81	1.39	1.66	15.1		
		1.190	1.210	0.98	0.05 - 0.22	0.10 - 0.27	0.50 - 0.81	1.39	1.17	15.1		
3	Kouchakzadeh and Townsend (1997)	0.381	0.203 (half width)	1.88	0.29 - 0.42	0.39 - 0.52	0.74 - 0.81	0.09 0.15 0.21	0.24 0.39 0.55	12.2	0.0006 - 0.0009 Adjustable
4	Cardoso and Bettess (1999)	0.800	1.640	0.49	0.03 - 0.09	0.11 - 0.17	0.27 - 0.53	0.15 0.27 0.40 0.53 0.67 0.80	0.19 0.34 0.50 0.66 0.84 1.00	22	Adjustable
5	Lim and Yu (2001)	0.280	1.007	0.28	0.054 - 0.095	0.100 - 0.150	0.54 - 0.63	0.585	2.09	7	Horz.
		0.104	0.388	0.27	0.017 - 0.067	0.100 - 0.150	0.17 - 0.45	0.155	1.49	20	0.002	
6	Present study	1.000	0.6	1.67	0.0375 - 0.10	0.1875 - 0.25	0.2 - 0.4	0.20 0.35 0.50	0.20 0.35 0.50	19	0.001053 Fixed

Table 2.1.xls

Melville (1995) found that in the case of two-stage channels, the scour depth prediction should indicate how to quantify the velocity and abutment length. He identified four types of abutment positions in two-stage channels, as shown in Figure 2.10.

Figure 2.10. Four types of bridge abutment scour (Melville, 1995)

Melville (1995) proposed a method, based on his earlier formulation (Melville, 1992), for scour around abutment in a two-stage channel. The equilibrium scour depth is scaled to the square root of the flow area blocked by the abutment in a rectangular channel.

$$d_{se} \propto \sqrt{Ld} \propto \sqrt{A} \quad (2.10)$$

The equivalent expression for the scour in a corresponding two-stage channel is

$$d_{sc} \propto \sqrt{\sum \left(A_i \frac{U_i}{U} \right)} \equiv \sqrt{\frac{\sum Q_i}{Q_r/A}} \quad (2.11)$$

where A_i = subsection area, U = mean approach flow velocity in the main channel under incipient bed sediment motion, U_i = velocity at subsection area A_i , Q_i = discharge in the area upstream of the abutment, Q_r = discharge in the corresponding rectangular channel, A = approach flow area in corresponding rectangular channel.

In this method a reduction factor K_G is introduced to take account of the reduced scour in a two-stage channel compared to that obtained from calculation using rectangular channel flow formula.

$$K_G = \frac{d_{sc}}{d_{se}} \quad (2.12)$$

From Equations (2.10), (2.11) and (2.12),

$$K_G = \sqrt{\frac{\sum Q_i}{Q_r}} \quad (2.13)$$

Melville (1995) applied the method for a two stage channel of case B (see Figure 2.4) as follow:

Figure 2.11. Abutment Terminating in the Main Channel of Two-stage Channel.

Eq. (2.13) can be expressed as:

$$K_G = \sqrt{1 - \frac{B_f}{L} \left(1 - \frac{d_f U_f}{d_m U_m} \right)} \quad (2.14)$$

Evaluation on the velocity terms using Manning's equation gives the following expression of K_G ,

$$K_G = \sqrt{1 - \frac{B_f}{L} \left[1 - \left(\frac{d_f}{d_m} \right)^{5/3} \frac{n_{Mm}}{n_{Mf}} \right]} \quad (2.15)$$

where n_{Mm} and n_{Mf} is Maning's roughness coefficient of the main channel and the floodplain, respectively.

Melville (1995) gave an alternative approach by determining an equivalent length L_e of the actual abutment (of length L). This L_e is used in the calculation of scour depth by using the flow and geometrical parameters of the corresponding rectangular channel.

$$L_e = L \left\{ 1 - \frac{B_f}{L} \left[1 - \left(\frac{d_f}{d_m} \right)^{5/3} \frac{n_{Mm}}{n_{Mf}} \right] \right\} \quad (2.16)$$

Melville (1995) analyzed Sturm and Janjua's (1994) data based on the assumption that Case C can be considered as a special condition of Case A, by supposing there is an imaginary flow boundary at the edge of the main channel which separates the floodplain and main channel. His analysis showed that the measured scour depths were less than the estimated ones. He mentioned that the difference is because Melville's (1992) method was based on the equilibrium condition while the Sturm and Janjua's (1994) scour depth data was recorded before the equilibrium condition had attained. However, he stated that the concept of imaginary flow boundary for Case C needs to be checked.

Kouchakzadeh and Townsend (1997) showed that the discharge ratio Q_a/Q_w' , which is the ratio of flow intercepted by abutment (Q_a) and flow related to a specific channel width at the abutment end (Q_w'), is a significant parameter to account for the lateral momentum transfer and should be incorporated in the scour prediction formulation. A scour depth prediction formula obtained from multiple-regression on their data was proposed as follows:

$$\frac{d_{se}}{d_f} = 13.5 \left(\frac{Q'_w}{Q_a} \right)^{3.9} F_f^{1.17} F_c^{-0.25} \quad (2.17)$$

where F_f = Froude number of the approach flow in the floodplain, and F_c = Froude number of the flow at the floodplain in which the sediment will start to move. They mentioned that the discharge ratio Q_a/Q'_w will accommodate the influence of lateral momentum transfer, abutment length and relative depth on the scouring process. In regard to the significance of Q'_w , it would be a great advantage to present Q'_w in terms of the known parameters of flow and two-stage channel geometry.

In Kouchakzadeh and Townsend's (1997) experiments, the scour depth was measured after 5 hours of test duration. These results were then extrapolated upwards to the equilibrium scour depth based on their interpretation that the 5-hours test was equivalent to 60% (for $d_{50} = 0.5$ mm) and 65% (for $d_{50} = 0.7$ mm) of equilibrium scour depth. This extrapolation is questionable and rather simplistic as the length of abutment and the shape of the abutment tip would also affect the rate of scour during the scouring process (see Chapter 5). The process will also be affected by the interaction between the flow in floodplain and main channel especially for the case of longer abutment which is terminated near to the edge of the main channel.

Cardoso and Bettess (1999) studied the effects of scouring time and channel geometry on scour at abutments in a two-stage channel. Their study focused on abutments terminating in the floodplain and at junction of the floodplain and main channel (Case C and D, in Figure 2.10). According to their scour depth data, they confirmed Melville's (1995) hypothesis which mentioned that scour of abutment for Case C can be considered as a special case of Case A. But they did not give sufficient explanation on the specific effects of two-stage channel hydraulics, for example, the effect of the ratio of floodplain flow depth over the main channel flow depth.

Lim and Yu (2001) proposed a formulation for scour around abutment in two-stage channels with the abutment terminating in the main channel, based on the

formulation for abutment in rectangular channel by Lim (1997). They improved the Lim's formulation by introducing an equivalent abutment length (L_e) and a width-averaged equivalent velocity (U_e), to account for the flow and geometry of two-stage channels. Their formula, given below, agrees well to within the range of $\pm 10\%$ when compared to the experimental data.

$$\frac{d_{se}}{d_m} = 0.9K_s \left\{ (\theta_c)^{-0.375} (F_{oe})^{0.75} \left(\frac{d_{50}}{d_m} \right)^{0.25} \left[0.9 \left(\frac{L_e}{d_m} \right)^{0.5} + 1 \right] - 2 \right\} \quad (2.18)$$

The F_{oe} , L_e and U_e are defined as follows:

$$F_{oe} = U_e / \sqrt{(S-1)g d_{50}} \quad (2.19)$$

$$L_e = L \left\{ 1 - \frac{B_f}{L} \left[1 - \left(\frac{d_f}{d_m} \right)^{5/3} \right] \right\} \quad (2.20)$$

$$U_e = \frac{B_f U_f + (L - B_f) U_m}{L} \quad (2.21)$$

where d_m = flow depth in the main channel, B_f = floodplain width.

From the above review on the research of scour around abutments in two-stage channels some important information on its development and scour processes can be extracted as follows. There are very few studies in this area so far and the first attempt appears to be the work by Sturm and Janjua (1994). As the ratio of main channel to floodplain width in their experiments was very small (see Table 2.1), the effect of the two stage channel may not be so apparent. Melville (1995) postulated an imaginary flow boundary which separates the floodplain flow and main channel flow in analysing scour around abutment for Case C. He also proposed a scour prediction method for Case B scour by introducing the concept of a corresponding rectangular channel using a channel geometry factor (K_G) and an equivalent abutment length (L_e). The imaginary flow boundary was confirmed by Cardoso and Bettess' (1999) experiments but the effect of two-stage hydraulics on the scour

around abutment was not explained. A study by Kouchakzadeh and Townsend (1997) also proposed a scour prediction formula, which is closely related to that of Sturm and Janjua (1994) but their emphasis was on the clear-water scour near threshold flow condition for sediment motion. Lim and Yu's (2001) study can be considered as a new approach on scour around abutment in two-stage channels, when it is terminated in the main channel. In general, the above review shows a general lack of clear relationship between the abutment scour and research on the hydraulics of two-stage channels.

2.6 Temporal Scour Development

The temporal scour development is an important factor to be considered in the abutment design process. This is because the flow will be dependent on the particular hydrograph of the river which eventually will affect the scour development. Hence, design for scour at an abutment on a floodplain can use the fraction of the equilibrium scour depth by considering the hydrograph characteristic of the river.

Cardoso and Bettess (1999) studied the scour-time development around abutment terminating on floodplain. Three phases of scour-time development, i.e. initial, principal and equilibrium phase, were identified in their study. The principal and equilibrium phase were identified in most of their experiments, while the initial phase were reported to be identified in some cases. Equilibrium scour was obtained on the average after the experiment was run for 68 hours. The equilibrium phase was reached in a shorter time for the scour in the main channel compared to that on the floodplain. This was due to the live-bed sediment transport condition in the main channel. The equilibrium time was calculated as the time when the regression line of the principal phase intercepts the regression line of the equilibrium phase. Temporal scour development on scour around piers by Ettema (1980), Franzetti et al. (1982) and Whitehouse (1997) were used in Cardoso and Bettess (1999) to analyze the case of scour around abutment terminating on a floodplain. All of the methods gave a good performance in representing the temporal scour development for the principal phase of scour.

Dey and Barbhuiya (2004) studied the scour development around short abutment (abutment length/flow depth ≤ 1) in a rectangular channel in uniform and nonuniform sediments. They proposed a semiempirical model of the time variation of scour depth around a vertical wall, a 45° wing wall and a semicircular abutment. An empirical model of maximum scour depth was also proposed based on their data. The model has a tendency to underestimate the scour depth data at the initial period. This model's characteristic should be considered when it is applied for prediction of short duration based on flood hydrograph. The maximum duration of the test was 48 to 50 hours which seems to be inadequate to reach equilibrium, according to the data trend in Lim's (1997) and Melville's (1995) works.

Coleman, Lauchlan and Melville (2003) studied the scour-time development around abutment in a rectangular channel under clear-water scour condition. Their work focused establishing an equivalent formulation similar to that of Melville and Chiew (1999) for pier scour. They found that the time factor was only dependence to the flow intensity for $L/d_{50} > 100$. The remaining factor related to sediment size, abutment length and flow-depth are the same as given in Melville (1997). Coleman, Lauchlan and Melville (2003) defined the equilibrium condition as “the time at which the rate of scour reduces to 5% of the smaller of the foundation length (pier diameter or abutment length) or the flow depth in the succeeding 24-hour period”.

CHAPTER 3. EXPERIMENTAL PROGRAMME

3.1 Introduction

Laboratory experiments were done to obtain data of flow and scour depth to support the analysis of flow and scouring around abutments terminated in a floodplain of two-stage channels. The experiments are divided into two parts, i.e. detail measurement of flow distribution and scouring experiments. This chapter describes the apparatus and materials, measurement techniques and procedures used in the experiments. The data obtained in this study will also contribute to the existing published data on scour around abutment terminating at a floodplain of two-stage channel.

3.2 Experimental Apparatus and Materials

3.2.1 Two-stage channel

The two-stage channel dimensions were 19 m long, 1.6 m wide and had sidewall of 1 m high measured from the bed of the main channel. The channel had a longitudinal bed slope of 0.00116. There was a 2.5 m long sand recess section, at 11 m downstream from the flume entrance. The sand-recess of the floodplain and main channel part was separated by a steel plate at the interface between the floodplain and the main channel. The main channel bank height was 0.15 m. At the downstream end of the channel there was a tail-gate which can be moved vertically using two electric-motors. A sump below the floor provided water for the test. A 22 kW submersible pump was used to pump water from the sump to provide a continuous flow to the channel. The pump was able to deliver a maximum discharge of $0.120 \text{ m}^3/\text{s}$. The water passed through a header tank before it flowed into the flume. The header tank was equipped with perforated steel plates, sponge layer and cloth to reduce surface water undulation. The arrangement of the water recirculation system of the two-stage channel is depicted in Figure 3.1. The photograph of the system is presented in Figure 3.2.

Figure 3.1. Schematic diagram of the two-stage channel and water recirculation system.

Figure 3.2. The 19-m two-stage flume.

The widths of the sand recess are 1 m and 0.6 m for the floodplain and main channel, respectively. They were 2.5 m long and the depths are 0.40 m and 0.25 m for the floodplain and main channel, respectively. The sand recess in the main channel was covered by a piece of aluminum plate to keep it in the flat bed condition. The cross section of the sand recess is depicted in Figure 3.3 and the close up view of the flume is shown in Figure 3.4.

Figure 3.3. Cross-sectional view of the flume at sand recess.

Figure 3.4. Close up view of the 19-m two-stage flume.

3.2.2 Bed material

The sand used in the experiment had a median size, d_{50} of 0.9 mm and a geometric standard deviation, $\sigma_g = (d_{84}/d_{50})$ of 1.05. The sand characteristics was of the quartz type, and was a non-ripple forming size since its size was greater than 0.6 - 0.7 mm or 0.5 - 0.9 (Chien and Wan, 1999) and considered uniform as $\sigma_g < 1.5$. The sieve analysis of the sediment size distribution is shown in Figure 3.5.

Immediately downstream of the sand recess was a sediment trap to catch the sediment which was scoured from the sand bed. For two meters length upstream of the sand recess, the floodplain bed was coated with the same kind of sand. This coating was expected to make the flow fully developed before it reached the sand recess.

Figure 3.5. Distribution of the sediment size.

3.2.3 Abutment

The three vertical-wall abutments used in the experiments were 0.20 m, 0.35 m and 0.50 m long (protruding across the flume) and 0.07 m wide (see Figure 3.6). The abutments were made from 10 mm thick perspex. It was placed 1 m after the upstream edge of the sand recess. The abutment was firmly attached to the sidewall using silicon glue. The available space within the abutment model made the

measurement of scour depth possible using a periscope-like apparatus (see Figure 3.17). A picture of an installed abutment on the sand recess can be seen in Figure 3.7.

Figure 3.6. Abutments of 20 cm, 35 cm and 50 cm length.

Figure 3.7. A 20 cm long abutment is attached to the side wall at the sand recess section.

3.3 Measurement Techniques

3.3.1 Measurement of velocity and flow distribution

Velocity measurements were done to obtain the cross sectional flow distribution and vertical velocity distribution at certain locations. Two propeller current meters (8-mm diameter and 3-mm diameter) and a 3-D ADV were used for velocity measurements. The 8-mm and 3-mm propeller current meters were used generally to measure the depth-averaged mean velocity in shallow water condition and when only a streamwise velocity distribution profile was needed. For the 8-mm propeller meter the velocity was obtained from the frequency reading and calibration chart given by the manufacturer, while for the 3-mm propeller meter the velocity can be obtained from file record saved in a computer system connected to the display module through a data acquisition card. The 3-mm propeller meter was only used for several experiments, because the propeller was very small and could be disturbed or damaged easily by suspended material. The 8-mm and 3-mm propeller systems are presented in Figure 3.8 and Figure 3.9, respectively.

Figure 3.8. The 8-mm propeller meter.

Figure 3.9. The 3-mm propeller meter.

A comparison between the measurement results of using the 8-mm and 3-mm propeller meter is presented in Figure 3.10. Even though the comparison presented here only covered a small range of velocity, the comparisons between measured discharge by flowmeter and velocity integration showed good agreement within ± 10 % range. Figure 3.11 shows the comparison between the discharge obtained from the flow meter and from integration of the measured velocity distributions.

Figure 3.10. Velocity profile measured by 8-mm and 3-mm propeller current meter.

Figure 3.11. Comparison between the discharge measured by flowmeter and the discharge obtained from the integration of measured velocity distributions (d_f = flow depth in the floodplain, d_m = flow depth in the main channel, d_f/d_m = flow depth ratio).

The 3-D ADV was used to measure the velocity and its fluctuation. In order to have a record of the fluctuations and a representative mean velocity, a 25 Hz sampling rate was applied. Each reading was taken for a period of 30 to 60 seconds. In operating the ADV, some considerations must be taken into account as explained in the following sections.

The principle behind the operation of the ADV is based on the Doppler effects. The Doppler effect states that if a sound source is moving relative to the receiver, the frequency of the sound received by the receiver is shifted from the transmitted-frequency by the amount:

$$F_{\text{doppler}} = -F_{\text{source}} \frac{V}{C} \quad (3.1)$$

where F_{doppler} = change in received frequency (Doppler shift), F_{source} = frequency of transmitted sound, V = velocity of source relative to receiver and C = speed of sound. The velocity V represents the relative speed between source and receiver (i.e. motion that changes the distance between the two). Motion perpendicular to the line connecting source and receiver does not introduce a Doppler shift. If the

distance between the two objects is decreasing, the frequency would increase; and vice versa.

A 3-D ADV has one transmitter and three receivers. The intersection of the transmitted beam pattern and received beam pattern form a sampling volume. During its operation, the transmitter generates a short pulse of sound wave of a known frequency. This pulse will intercept any suspended solid particles along its path as well as in the sampling volume. The suspended particles then reflect a portion of the sound wave along the receiver axis towards the receiver. The velocity of the particle is then analyzed from the shift of the received pulse frequency. The sequence of this basic operation is depicted in Figure 3.12.

note: $F_s = F_{\text{source}}$; $F_d = F_{\text{doppler}}$

Figure 3.12. Basic operation of an ADV.

The parameters that can be observed for obtaining good data are the signal to noise ratio (SNR) and correlation (COR). A SNR value above 15 is considered good for a 25 Hz sampling rate to capture the fluctuation of the flow velocity, and SNR value of 5 is acceptable for mean velocity measurement (Sontek, 1997). The COR is an indicator of the relative consistent behavior of scatter in a sampling volume during the sampling period. The recommended value of COR is above 70 (Sontek, 1997). Wahl (2000) stated that a COR value of less than 70 could sometimes provide good data especially when the SNR is high and the flow is relatively turbulent.

There are three types of 3-D ADV based on the direction of measurement, i.e. up-looking, down-looking, and side-looking. Figure 3.13 shows these three types of probe. The different types are basically to accommodate different position of measurement and to minimize disturbance on flow during measurement. The minimum depth in which the velocity measurement can be done depends on the geometry of 3-D ADV. For example, when using down-looking probe, the position from the water surface to 6.5 cm below water surface cannot be covered by the 3-D ADV. The up-looking 3-D ADV cannot cover the area between the bed and 8 cm above the bed. Therefore the minimum depth that can be measured by the up-looking and down-looking 3-D ADV is around 14.5 cm.

Down-looking probe

Up-looking probe

Side-looking probe

Figure 3.13. Three types of ADV probe.

Figure 3.14. The computer system connected to the ADV module.

Discrepancies were observed between the measured velocities obtained from the propeller current meter and the 3-D ADV. It was believed that the measured velocities by the 3-D ADV was accurate as the checking on probe alignment by a program provided by the manufacturer was done and showed no physical defect on it and also during the measurement the SNR was maintained to be around 15. Hence a correction on the current meter results was done based on the 3-D ADV measurement results. A new relationship between frequency and velocity was obtained by appropriate regression on the data of frequency obtained by propeller and velocity data obtained by 3-D ADV. An example of measured velocity comparison after correction applied to the 8-mm propeller results is shown in Figure 3.15.

Figure 3.15. Velocity profile measured by propeller and ADV, after correction applied to the propeller's result.

3.3.2 Measurement of inflow discharge

The inflow discharge was measured using a magnetic flow meter which can measure up to an accuracy of 1 m³/hr. The inflow discharge was controlled by a pump and a valve connected at a section of the supply pipe line. The speed of the pump was controlled using a frequency inverter controller.

3.3.3 Measurement of flow-depth and water surface slope

The water level was measured at the approach flow section, and also along the channel to obtain the overall water surface elevation. The error of the reading was in the order of ± 2 mm due to surface wave. The water surface slope was obtained from the observed water levels using a panel of water manometers (see Figure 3.16). The manometers were fitted to a board equipped with scale and the water level was read from the manometers. The manometers were connected to plastic hose which were inserted to the flume at locations of 6 m, 8 m, 10 m, 12 m and 14 m from the flume entrance.

Figure 3.16. Water manometers.

3.3.4 Scour depth

The scour depth was measured using a periscope equipped with a scale attached to its side as shown in Figure 3.17. The periscope can be inserted into the box-like abutment. The position of the bed surface was observed from the periscope and the respective vertical position was observed from the scale. The change of bed level in an interval of time was obtained from the difference between two subsequent scale readings, i.e. at the beginning and the end of the interval. The error in reading was ± 1 mm.

Figure 3.17. The periscope.

3.3.5 Visual flow observation

In order to provide information on how the flow passes the region near the abutment, detailed flow visualization was done for one flat-bed experiment and one scoured-bed experiment. The visualization was performed by releasing dye at the upstream part of the abutment. Some pictures and video records were taken at various dye releasing locations around the abutment, in flat bed and scoured bed experiments, to observe the flow movement. An example of the visual flow observation is presented in Figure 3.18. The results were used in the analysis of the flowfield in Chapter 4.

Figure 3.18. An example of the visual flow observation.

3.4 Experiments

In general, the experiments can be divided into two types: fixed bed and scouring experiments.

3.4.1 Fixed bed experiment

The purpose of the fixed bed experiment is to perform detailed velocity measurements. The measurements were taken at several cross sections in the vicinity of abutment site. The details of the point of measurement are given in Chapter 4. The channel bed in the sand recess must be fixed to prevent changes of the bed geometry during the measurement. In the flat bed condition the sand recess was covered with sand-coated aluminum plate (see Figure 3.19). To carry out the measurement after the equilibrium scour depth was attained, the scour hole surface must be preserved. The scour hole surface was dried first and then was fixed by spraying solution of sodium bicarbonate and then sodium silicate. This method was proposed by Benson, et.al. (2001). The bed elevation was checked before the experiment to ensure that the scour hole profile was unchanged. The fixed bed experiment was only done for flow depth ratio (d_f/d_m) of 0.4, where the floodplain flow depth (d_f) = 0.105 m, main channel flow depth (d_m) = 0.255 m, discharge (Q) = 70 l/s and abutment length L = 0.20 m. The results and analysis of the fixed bed experiments will be presented in Chapter 4.

Figure 3.19. The sand recess covered with aluminum plate coated with sand.

3.4.2 Scouring experiments

In the scouring experiments, the flow discharges as well as the flow depth ratios were varied for different abutment lengths. For a typical test, the discharge was controlled using the frequency inverter controller and valve connected to the pump and the flow depth was controlled by tail gate adjustment. In the beginning of the experiment the tail gate was set at a sufficiently high level while the flume was slowly filled with water. Then the discharge was gradually increased up to the designated value while the tail gate was gradually adjusted to the position which gave the desired flow depth for the given discharge. This tail gate position was known, by trial test, prior to the start of each experiment. The scouring process was allowed to proceed until the equilibrium scour depth was attained. The scour depth development was monitored and recorded for different time interval according to the scouring rate. The recording time interval in the beginning of the scour could be in the range of 2 to 10 minutes, and after the scouring rate decreased the interval could be extended to 2 to 6 hours. The flow velocity, flow depth and discharge at the approach section were also measured to provide approach flow data related to the scour experiment. Measurement of cross sectional flow distributions at the abutment's site cross section after the equilibrium scour had been formed were also done for three tests: Runs 20-60-04, 20-80-04 and 20-100-05.

The code for every test was given in the form of XX-YY-ZZ, where XX denotes the length of abutment in cm, YY denotes the discharge in l/s and ZZ denotes the flow depth ratio, i.e. the ratio of the flow depth in the floodplain to the flow depth in the main channel written without decimal point. As an example, Run 20-60-02 is the code for the test with abutment length of 20 cm, flow discharge of 60 l/s and flow depth ratio of 0.2. The scouring experiments which had been done are tabulated in Table 3.1 and the details of the hydraulic conditions of the test are presented in Table 3.2. The results and analysis of the scouring experiments will be presented in Chapter 5.

Table 3.1. Scouring experiments.

Abutment Length	Q	d_f/d_m			
cm	l/s	0.2	0.3	0.4	0.5
20	50	V	V	V	
	60	V	V	V	
	70	V	V	V	
	80			V	
	100				V
35	50	V	V	V	
	60	V	V	V	
50	50	V	V	V	
	60	V			

Table 3.2. Hydraulic conditions of the tests.

Run	L Cm	Q_{fm} l/s	d_f cm	U_f cm/s	Fr_f	d_m cm	U_m cm/s	Fr_m	U_c cm/s	U_f/U_c	Q_{act} l/s	% Diff.
20-50-02	20	50	3.75	20.73	0.342	18.75	36.71	0.271	31.5	0.658	49.1	-1.89
20-60-02	20	60	3.75	25.77	0.425	18.75	42.30	0.312	31.5	0.818	57.3	-4.80
20-70-02	20	70	3.75	28.8	0.475	18.75	49.39	0.364	31.5	0.914	66.4	-5.48
20-50-03	20	50	6.40	20.18	0.255	21.40	27.64	0.191	34.6	0.583	48.4	-3.30
20-60-03	20	60	6.40	24.06	0.304	21.40	30.73	0.212	34.6	0.695	54.9	-9.38
20-70-03	20	70	6.40	28.26	0.357	21.40	37.67	0.260	34.6	0.817	66.5	-5.33
20-50-04	20	50	10.00	17.68	0.179	25.00	20.63	0.132	37.4	0.473	48.6	-2.83
20-60-04	20	60	10.00	21.47	0.217	25.00	24.80	0.158	37.4	0.574	58.7	-2.27
20-70-04	20	70	10.50	22.99	0.227	25.50	27.12	0.171	37.4	0.615	65.6	-6.65
20-80-04	20	80	10.00	29.86	0.301	25.00	34.57	0.221	37.4	0.798	81.7	2.10
20-100-05	20	100	15.00	30.69	0.253	30.00	33.34	0.194	39.5	0.777	106.0	5.70
20-70-04A	20	70	10.50	21.38	0.211	25.50	29.06	0.184	37.4	0.572	66.9	-4.62
20-70-04B	20	70	10.50	24.42	0.241	25.50	n.a.	n.a.	37.4	0.653	66.9	-4.62
35-50-02	35	50	3.75	20.73	0.342	18.75	36.71	0.271	31.5	0.658	49.1	-1.89
35-60-02	35	60	3.75	25.77	0.425	18.75	42.30	0.312	31.5	0.818	57.3	-4.80
35-50-03	35	50	6.40	20.18	0.255	21.40	27.64	0.191	34.6	0.583	48.4	-3.30
35-60-03	35	60	6.40	24.06	0.304	21.40	30.73	0.212	34.6	0.695	54.9	-9.38
35-50-04	35	50	10.00	17.68	0.179	25.00	20.63	0.132	37.4	0.473	48.6	-2.83
35-60-04	35	60	10.00	21.47	0.217	25.00	24.80	0.158	37.4	0.574	58.7	-2.27
50-50-02	50	50	3.75	20.73	0.342	18.75	36.71	0.271	31.5	0.658	49.1	-1.89
50-60-02	50	60	3.75	25.77	0.425	18.75	42.30	0.312	31.5	0.818	57.3	-4.80
50-50-03	50	50	6.40	20.18	0.255	21.40	27.64	0.191	34.6	0.583	48.4	-3.30
50-50-04	50	50	10.00	17.68	0.179	25.00	20.63	0.132	37.4	0.473	48.6	-2.83

Note:

Fr_f = Froude number of the floodplain flow; Fr_{int} = Froude number of the main channel flow; Q_{fm} = Discharge measured by flow meter

Q_{act} = Discharge obtained from integration of the measured velocity distribution; % diff = $(Q_{act} - Q_{fm}) / Q_{fm} * 100$

3.4.3 Additional experiments

In order to study the effect of the flow interaction between the floodplain and the main channel, two experiments (Run 20-70-04A and 20-70-04B, in Table 3.2) were specifically done where the floodplain flow is prevented from interacting with the main channel. The purpose of these experiments is to study what effect the interaction of the floodplain and main channel flow had on the equilibrium scour depth. For Run 20-70-04A a metal plate of 1 mm thick, 7.2 m long and 0.4 m high, was attached to the main channel bank and this serve as a separator to prevent floodplain and main channel flow from interacting between a section 5.2 m upstream of the abutment site to a section 2.0 m downstream of the abutment site. The second experiment was done with a 2.4 m long metal plate as the separator. The separator prevented the floodplain and main channel flow from interacting between a section 0.4 m upstream of the abutment site to a section 2.0 m downstream of the abutment site. In Table 3.2, these two additional experiments were referred as Run 20-70-04A and 20-70-04B, respectively. Figure 3.21 schematically shows the placement of the separator between the floodplain and main channel.

Figure 3. 20. Schematic description of separator placement.

Figure 3.21. Placement of the 2.4 m separator, view towards upstream direction.

CHAPTER 4. FLOWFIELD OBSERVATION

4.1 Introduction

The presence of the abutment in a channel cross section will reduce the flow area at the abutment site cross section and affect the local flow distribution along the affected reach. The change on the local flow distribution will be followed by local scour development around the tip of the abutment due to the increase of shear stress and local sediment transport mechanism. The scouring process will reach an equilibrium state when there is no more change of the scour hole. In this study, measurements and observations on the flow characteristics in a two-stage channel were done for two cases; i.e. in the flat bed condition and when the scoured bed had reached its equilibrium condition. The flow conditions are as follows: discharge $Q = 70$ l/s, abutment length $L = 20$ cm and flow depth ratio $d_f/d_m = 0.4$, where $d_f = 10.5$ cm and $d_m = 25.5$ cm (see Table 5.1, Run 20-70-04). The scouring process was of the clear water scour type. The observations and results of the flow characteristics are presented and discussed in this chapter in terms of the mean velocities, turbulent intensities, bed shear stresses and Reynolds shear stresses. The results for flat bed and scoured bed condition are given in separate sections.

4.2 Flat Bed Conditions

For the flat bed conditions, the flow characteristics were measured at six cross sections as shown in Figure 4.1. The black dots in the figure indicate locations where measurements at the floodplain and some part of the main channel were done for cross sections A, B, D and F. In order to give additional detail information on the local flow field around the abutment, measurements were also done at sections C and E for a region close to the abutment and side wall of the floodplain. The x- and y-distance of these six sections are indicated in Figure 4.1 with respect to the abutment site. The measured velocity at cross section A represents the undisturbed approach velocity. This section is 2 m upstream of the abutment and the velocity and flow depth at this section were assumed to be unaffected by the presence of the

abutment. There are altogether 34 black dots in Figure 4.1 and at each location, the vertical velocity distribution was measured using either the ADV or the propeller meters (see Chapter 3).

Figure 4.1. Measurement points for flat bed experiment.

4.2.1 Velocity distributions

4.2.1.1 Velocity vectors in horizontal and vertical planes

Figures 4.2 to 4.4 show the velocity vectors at three horizontal x-y planes at $z = 0.5$, 3 and 5 cm, respectively, where z is the vertical distance from the floodplain bed level. In these figures, the vectors represent the resultant of the streamwise (u) and spanwise (v) velocity components. At the approach section A, the velocity at the floodplain is lower than that of the main channel at $z = 0.5$, whereas for higher elevation the velocity at the floodplain is only slightly less than that of the main channel. At the abutment section D, the velocity vectors in the floodplain can be seen to be diverted due to the presence of the abutment. The velocity here is also higher compared to that at sections B and C. The increase of velocity and the concentrated flow around the abutment tip will cause a local scouring process if the local velocity is higher than the critical velocity of sediment motion. Near the

abutment tip, it can be seen that the deflection angle near the bed is larger compared to that at higher level. Figure 4.5 shows that the shifting of the streak line in the lateral direction at the lower part (dy_a) is wider than at the upper part of the flow depth (dy_b). This is because the blocked flow is released close to the bed due to the down-flow at the abutment causing the magnitude of the spanwise velocity component near the bed to be higher than at other level. Ahmed and Rajaratnam (2000) observed similar flow deflection behavior for a wing-wall abutment in a rectangular channel. Because of this diverted flows, the velocity in the main channel at sections D and F increases considerably compared to that at section A.

Figure 4.2. Velocity vectors in x-y plane at an elevation 0.5 cm above the floodplain.

Figure 4.3. Velocity vectors in x-y plane at an elevation 3 cm above the floodplain.

Figure 4.4. Velocity vectors in x-y plane at an elevation 5 cm above the floodplain.

Figure 4.5. Streak line of injected dye at (a) lower and (b) upper part of the flow depth.

Figures 4.6 to 4.9 show the velocity vectors in the vertical y - z planes at cross sections A, B, D and F, respectively. The velocity vectors in these figures are the resultant of the spanwise (v) and vertical (w) velocity components. At section A the velocity vectors show a net flow from the main channel towards the floodplain. This is because the flow velocity in the main channel is higher than the velocity in the floodplain; this difference in velocity causes the transfer of momentum from the main channel towards the floodplain part. Closer to the abutment, at section B the net flow direction is towards the main channel and the velocity is much higher compared to A due the converging flow near the abutment site. At section D the velocity vector has the same pattern with that of section B, but its magnitude is higher and a significant downward vertical velocity component can be seen at the abutment. Near the abutment tip, the velocities at the lower depth were higher than those of the upper part. This is due to the effect of the flow diversion and the associated downflow at the abutment causing the flow velocity to be higher at the lower depth. This is the main cause of the scouring action at the abutment. At section F, which is downstream of the abutment location, the flow pattern is irregular and the separation of flow is observed.

In the main channel, close to the junction of the floodplain and main channel, the secondary flow was observed to change its direction and its magnitude. The magnitude of the secondary flow in the region close to the junction was increasing as the flow passed by the abutment site's cross section, due to the presence of the

abutment. The change of secondary flow pattern and magnitude at other region in the main channel is not significant.

Figure 4.6. Velocity vectors at cross section A.

Figure 4.7. Velocity vectors at cross section B.

Figure 4.8. Velocity vectors at cross section D. (Note: the velocity scale is different because of the higher velocity at this section)

Figure 4.9. Velocity vectors at cross section F.

4.2.1.2. Vertical distributions of streamwise (u), spanwise (v) and vertical (w) velocity components.

Figures 4.10, 4.11 and 4.12 show the vertical distribution of the streamwise (u), spanwise (v) and vertical (w) velocity components, respectively. The individual graph in each figure shows the vertical distribution of a velocity component at the same transverse position (y) but different in its longitudinal location (sections A, B, D and F). Hence it shows the change of each velocity component along the streamwise direction. Figure 4.13 shows a three dimensional plot of these measured velocities. The legend in Figures 4.10 to 4.12 with “Prop” and “ADV” indicates the equipment used in the velocity measurements; “Prop” refers to propeller current meter and “ADV” refers to 3D-ADV.

Distribution of streamwise velocity components (u) along x

Figure 4.10 shows graphs of dimensionless u/U versus z/z_a for various transverse (y) locations, where z_a = flow depth at the measurement point and U = mean velocity of two-stage channel. The figure shows two typical changes of streamwise velocity distributions in the floodplain. For $y = 80$ to 140 cm in the floodplain (see Figure 4.10), it was observed that the streamwise velocity decreases first and then increases as it passes section D. The decrease of the streamwise velocity was caused by the change of the flow direction due to the presence of the abutment. As it flows passed section D, the flow was deflected less and the increase in velocity was due to the flow area reduction. The decrease of the streamwise velocity was compensated by the increase of the spanwise velocity (see Figure 4.10). At the other part of the floodplain the velocity generally increased gradually in the x-direction. For $y = 120$ cm at section F, however, there is a decrease for the upper half part due to the vortices generated at the abutment.

In the main channel the velocity distribution also changed as the flow approached section D. At $y = 50$ cm, the velocity was higher at the lower and upper part but no change was observed for the mid level. At $y = 30$ cm the velocity profile generally increased in the streamwise direction as it flows passed the abutment.

Distribution of spanwise velocity components (v) along x

Figure 4.11 shows graphs of v/U versus z/z_a for various y -locations. At cross section A it can be seen that the direction of v/U (negative value) is from the main channel towards the floodplain. The direction of v/U (positive value) changed from floodplain towards the main channel as the flow is closer to cross section D. This shows the flow diversion towards the main channel due to the abutment. The largest magnitude of v/U was observed at cross section D.

In the floodplain, the maximum magnitude of v can be up to 0.82 times the approach streamwise cross sectional mean velocity, U , especially near the tip of the abutment (see also Figure 4.2). At other part it is in the range of 0.15 to 0.50 U . In the floodplain, v/U gradually increases at the lower part near the bed.

In the main channel, for $y = 50$ cm, the velocity distribution resembled an “S” shape with positive v/U in the lower part and negative v/U in the upper part. The peak magnitudes of the negative and positive values of v/U increased as the flow passed the abutment site. This shows that the presence of the abutment affected the v/U distributions in the main channel. At $y = 30$ cm, all v/U are positive, and the maximum was around 0.10, which was lower than that in the floodplain.

Distribution of vertical velocity component (w) along x

Figure 4.12 shows graph of w/U versus z/z_a for various y -locations. Generally, the magnitude of w/U ranges from 0 to 0.20. At $y = 140$ cm, it can be seen that the highest w/U is at the tip of abutment. This indicates the presence of downward flow near the tip of abutment as the flow accelerates due to the reduction of flow area. At other transverse positions in the floodplain, the magnitude of w/U is in the range of 0 to 0.05. In the main channel, a gradual increase of w/U was observed for $y = 50$ cm, while for $y = 30$ cm the changes were less notable.

Figure 4.10. Vertical distributions of u/U (note: FP = floodplain, MC=main channel).

Figure 4.11. Vertical distributions of v/U (note: FP = floodplain, MC=main channel).

Figure 4.12. Vertical distributions of w/U (note: FP = floodplain, MC=main channel).

Figure 4.13. Three dimensional plot of the measured velocity.

4.2.2 Bed shear stresses

The bed shear stress can be determined from the near bed velocity gradient where the velocity distribution can be assumed to follow the logarithmic law or by extrapolating the value of the measured Reynolds shear stress distribution ($-\overline{\rho u'w'}$) towards the bed. The Reynolds shear distribution will be discussed in the next section. As the value of the Reynolds shear stresses were measured (using ADV) only for the lower half of the floodplain flow depth, the vertical distribution of streamwise velocity is used to determine the level where the Reynolds shear stress is zero. The streamwise velocity profile was measured using propeller-type current meter. Comparisons of these two methods along sections A and D are presented in Figures 4.14 and 4.15. There are differences in the results, but overall they are comparable except for the points at the junction and in the main channel. The bed shear stress is generally lower using the Reynolds shear data. The bed shear stress close to the tip of the abutment is difficult to be extrapolated from the Reynolds stress as it has a non-linear distribution.

As the bed shear stress near to the abutment tip is the crucial data for this discussion, it is decided to use only the bed shear stress obtained from near bed velocity gradient. Figure 4.16 shows a comparison of the bed shear stress distribution at sections A and D. It can be seen that the bed shear stress at the measured point close to the tip of the abutment is considerably higher than the approach section. The bed shear stress obtained for a point close to the abutment tip was 3.8 times higher than that of the approach section or 1.8 times larger than the critical shear stress of the sediment used. The bed shear stress amplification is known to be the main cause of intense local scouring process around the abutment in conjunction with the transport due to the vortices system. For comparison, Rajaratnam and Nwachukwu (1983a) found that the amplification of bed shear stress at the area close to the groin ranged from 3 to 5 times the bed shear stresses at the approach section, while Molinas, Kheireldin, and Wu (1998) reported the total amplification, due to the contraction and the presence of abutment, at the nose of the abutment was 10 times the bed shear stress at the approach section.

Figure 4.14. Bed shear stress at cross section A.

Figure 4.15. Bed shear stress at cross section D.

Figure 4.16. Bed shear stress distribution at cross sections A and D, obtained from velocity distribution.

4.2.3 Reynolds shear stresses

In turbulent flow, the fluctuation of u , v and w , expressed as u' , v' and w' , respectively, will cause momentum exchanges between fluid layers and result in a turbulent shear stress namely Reynolds shear stress. This stress is proportional to the average value of the product of the fluctuation of the velocity components, i.e. $\overline{u'v'}$ and $\overline{u'w'}$. In this study, the Reynolds shear stresses were computed from the measured velocity components (u , v and w) obtained from the 3D-ADV records. A down-looking 3D-ADV probe was used in the measurement. Hence, the upper 6.5 cm portion of the flow depth was not covered due to the space requirement of this type of probe (see Section 3.3.1).

The Reynolds shear stresses on the horizontal (τ_{yx}) and vertical (τ_{zx}) planes are defined as follows

$$\tau_{yx} = -\rho \overline{u'v'} \quad (4.1)$$

$$\tau_{zx} = -\rho \overline{u'w'} \quad (4.2)$$

The terms $\overline{u'v'}$ and $\overline{u'w'}$ are computed as the covariance of u and v , and u and w , respectively.

$$\overline{u'v'} = \frac{\sum uv}{n-1} - \frac{\sum u \sum v}{n(n-1)} \quad (4.3)$$

$$\overline{u'w'} = \frac{\sum uw}{n-1} - \frac{\sum u \sum w}{n(n-1)} \quad (4.4)$$

where n = the number of data.

The shear velocity in the form of u_*^2 is used to normalize the Reynolds shear stresses. The shear velocity was obtained from the streamwise velocity profile in the middle of the floodplain at the approach section. By assuming that the velocity profile is logarithmic, the velocity distribution, at the lower half of the velocity profile (Nikora and Smart, 1997), can be expressed as:

$$\frac{u}{u_*} = \frac{1}{\kappa} \ln \left(\frac{z}{z_o} \right) \quad (4.5)$$

where u is velocity at depth z , κ is von Karman coefficient, z_o is roughness parameter. The following expression can be obtained by rearranging κu as a function of $\ln(z)$,

$$\kappa u = u_* \ln z - u_* \ln z_o \quad (4.6)$$

Using Equation (4.6), u_* can be found from the slope of the regression line by plotting κu versus $\ln(z)$. The u_* in the middle of the floodplain at section A (approach section) was found to be 0.017 m/s.

The distributions of the dimensionless Reynolds shear stresses for various transverse locations in the floodplain and main channel, plotted as uw^+ and uv^+ versus z/z_a are shown in Figures 4.17 and 4.18, respectively, where $uw^+ = \overline{u'w'}/u_*^2$, $uv^+ = \overline{u'v'}/u_*^2$. Most of the measured uw^+ has positive values.

The uw^+ close to the abutment tip, downstream of the abutment and close to the main channel bank show negative values for some points. These negative uw^+ correspond to the negative value of $\partial U/\partial z$ of the vertical distributions of streamwise velocity profile due to the effect of secondary flow. Similarly, the negative values of uv^+ in Figure 4.18 correspond to negative value of $\partial U/\partial y$. This can be seen mostly for the region close to the abutment tip and main channel. The Reynolds shear stress usually corresponds to the gradient of the primary mean velocity (Tominaga and Nezu, 1991) and the strength of secondary flow.

Figure 4.18 shows that the values of uv^+ are negative in the main channel close to the junction at the approach section (i.e. at $y = 50$ cm), while positive values were observed in the floodplain at the junction (i.e. at $y = 60$ cm). This indicates that the momentum is transferred from the main channel towards the floodplain at the approach section. Close to the abutment (section D, $y = 120$ and 140 cm) the negative values of uv^+ are much higher, compared to the approach section, indicating the transfer of momentum and secondary flow due to the presence of the abutment.

Figure 4.17. Vertical distributions of uw^+ (note: FP = floodplain, MC=main channel).

Figure 4.18. Vertical distributions of uv^+ (note: FP = floodplain, MC = main channel).

4.2.4 Turbulent intensities and turbulent kinetic energy

The turbulent intensities and turbulent kinetic energy for the flat bed condition are normalized by the shear velocity obtained for the middle of the floodplain at the approach section. Figures 4.19 to 4.21 show the turbulent intensities distribution, plotted as u^+ , v^+ and w^+ versus z/z_a , where $u^+ = u_{rms}/u_*$, $v^+ = v_{rms}/u_*$ and $w^+ = w_{rms}/u_*$. Figure 4.22 shows the distribution of the turbulent kinetic energy, k^+ , against z/z_a , where $k^+ = 1/2 \cdot (u_{rms}^2 + v_{rms}^2 + w_{rms}^2)/u_*^2$.

In the floodplain, Figure 4.19 shows that increase of u^+ is much higher for the region close to the abutment tip (at location D, for $y = 140$ cm) compared to the rest of the region. The maximum u^+ was observed at section F, where the wake generated at the abutment maintained its strength before its gradual decay into smaller vortices further downstream. Close to the junction between the floodplain and main channel, the increase in u^+ was observed at sections B, D and F compared to the approach section A. In the main channel, the increase of u^+ was observed between $z/z_a = 0.35$ to 0.65 . The distribution of v^+ in Figure 4.20 shows similar trend to u^+ . The same trend was also observed for w^+ (Figure 4.21) except that there is no apparent increase of w^+ at the junction. Figures 4.18 to 4.20 show there is no apparent difference in the distributions of turbulent intensities in the middle of the main channel for all the cross sections.

In Figure 4.22, the normalized turbulent kinetic energy (TKE) distribution shows similar distribution pattern as that of u^+ and v^+ . The increase of the TKE shows the generation of turbulence energy from the mean velocity due to the presence of the abutment. The increase of turbulence, generated by the presence of the abutment, contributes in scouring and transporting mechanism of the scoured sediment farther downstream.

Figure 4.19. Vertical distributions of u^+ (note: FP = floodplain, MC=main channel).

Figure 4.20. Vertical distributions of v^+ (note: FP = floodplain, MC=main channel).

Figure 4.21. Vertical distributions of w^+ (note: FP = floodplain, MC=main channel).

Figure 4.22. Vertical distributions of k^+ (note: FP = floodplain, MC=main channel).

4.3 Scoured Bed Condition

Under the same flow conditions as used in the fixed bed experiment, the bed was allowed to scour until the equilibrium scour depth was achieved. The flow measurements in the scoured bed were done for cross sections A', B', C' and D', as shown in Figure 4.23, where A' is 10 cm from the floodplain sidewall, B' is 20 cm upstream of the abutment, D' is 50 cm downstream of the abutment and section C' is adjacent to the abutment. A 3D-ADV with down-looking probe was used for all sections. For sections A' and D', an up-looking 3D-ADV probe was also used. The black dots in Figure 4.23 represent the vertical where the velocity profiles were measured. The many measurements for the scoured bed condition was necessary as the geometry of the scoured bed was more varied compared to the flat bed condition.

Figure 4.23. Measurement points for scoured bed experiment (note: FP = floodplain, MC=main channel).

4.3.1 Scour hole geometry

For this experiment, the scouring process took 21 days to reach the equilibrium state and the dimensions of the scour hole were 0.235 m deep and 0.60 m wide at the abutment site. The picture of the scour hole is presented in Figure 4.24. Figure 4.25 shows the contour of the equilibrium scour hole and its three dimensional

surface plot is shown in Figure 4.26. The deepest scour depth was found adjacent to the upstream corner of the abutment. In the beginning of the scour process the shape of the scour hole looks like an inverted cone. As the scour hole increased in depth, the edge of the scour widened and finally reached the side wall. The final shape of the equilibrium scour hole resembles one half of an inverted cone.

Figure 4.24. The equilibrium scour hole.

Figure 4.25. Contours of equilibrium scoured bed (note: contour line in cm).

Figure 4.26. Three dimensional surface plot of scour hole.

4.3.2 Velocity distributions

4.3.2.1 Flowfield in section A'

Figure 4.27 shows the flowfield in the scour region at the longitudinal section A'. The velocity vectors in this figure show clearly the formations of two clockwise vortices in the scour hole immediately upstream of the abutment. The first vortex is formed at the bottom corner of the scour hole and the second one is on the upstream slope of the scour hole. The former is caused by the down-flow along the upstream face of the abutment, while the latter is caused by the gradual change of water depth as the flow comes into the scour hole area. This type of circulation is quite similar to flow over backward facing step. Some depositions were observed at the upstream slope of the scour hole, probably due to the up-sloping flow which is able to transport sediments up and deposit them at certain place on the slope. The deposition area is indicated by small humps on the upstream slope (see Figure 4.27).

Figure 4.28 shows contours of dimensionless spanwise velocity, v/U , inside the scour hole for the region upstream of the abutment. It can be seen that the flow close to the upstream face of the abutment is in the direction towards the side wall. The flow above the zero bed level also has the same direction as that immediately upstream of the abutment. The direction of the spanwise velocity in between these two regions is moving away from the side wall. This shows that most of the blocked flow upstream of the abutment was “released” along the circumference of the upstream scour hole to join the unobstructed flow. Figure 4.29 (a) shows the streak line of injected dye at the upstream of the abutment and Figure 4.29 (b) shows the dye is dispersed at the bottom of the scour hole. The area of dispersion is indicated by the dashed line in Figure 4.29 (b). This dispersion is related to the downflow and up-sloping flow at the bottom of the scour hole (see Figures 4.27 and 4.33). The maximum spanwise velocity in this section is around 0.3 times the mean approach flow velocity and it is located somewhere above the middle of the slope.

Figure 4.27. Velocity vectors at cross section A'.

Figure 4.28. Contours of transverse velocity component, v/U at cross section A'.

Figure 4.29. (a) Streak line of the injected dye, at a point upstream of the abutment and (b) dispersion of the dye at the bottom of the scour hole in scoured bed condition.

4.3.2.2 Flowfield in sections B', C', and D'

Figure 4.30 and Figure 4.31 shows the velocity vectors in x-y planes at an elevations of $z = 2.5$ cm and 5.0 cm, respectively in the equilibrium scoured bed condition. The deflection of the velocity vector close to the tip of the abutment in

these figures is less than that of flat bed condition (see Figures 4.3, 4.4 and 4.32). This is because the flow area increased considerably after the scour hole formed.

Figures 4.30 and 4.31 also show that the direction of the velocity vectors at the scour hole region are straight unlike the deflected vectors for the flat bed condition. The velocity vectors at 50 cm downstream (section D') of the abutment were also different from the flat bed case due to the scour hole geometry. The velocity is non-uniform and the highest velocity is recorded in the centerline of the scour hole as most of the blocked flow is concentrated in this path.

Figure 4.30. Velocity vectors in x-y plane at an elevation 2.5 cm above the floodplain in scoured bed condition.

Figure 4.31. Velocity vectors in x-y plane at an elevation 5 cm above the floodplain in scoured bed condition.

Figure 4.32. Streak line of the injected dye, at upper part of the flow depth, in (a) scoured bed and (b) flat bed condition.

The velocity vectors of the secondary flow and the streamwise velocity contours at sections B', C' and D' are shown in Figures 4.33 to 4.38. The secondary flow in these sections will be discussed first, followed by the streamwise velocity contours.

Figure 4.33 shows the transverse flow pattern at section B'. It can be seen that the secondary flow close to the sidewall is in the downward direction and it is in the up slope direction along the slope of the scour hole. As a result a counter clockwise vortex is formed inside the scour hole. The magnitude of the flows in the floodplain

adjacent to the hole and inside the main channel is lower than those inside the scour hole. The secondary flow inside the main channel is in the direction towards the floodplain.

At section C' (Figure 4.35), it can be seen that the counter clockwise vortex is stronger than that at section B'. A similar vortex was also observed by Kwan and Melville (1994) in their experiment for a wing-wall abutment placed in a rectangular channel. At section D' (Figure 4.37) a clockwise circulation is formed behind the abutment. The secondary flow above the scour hole bed is in the up slope direction. At this section, beside the streamwise sediment transport, the strong up-sloping flow transported the scoured sediment to the left and right side of the scour hole. The scoured sediments were deposited further downstream and also along the right and left edge of the scour hole. The deposition along the sidewall at the area behind the abutment was higher compared to the opposite side. In the flatter part of the floodplain, the net flow is towards the main channel while the net flow direction in the main channel is towards the floodplain. As a result, there is a small region near the junction where a weak circulatory flow is observed. Generally, the secondary flow at sections B', C' and D' has a net direction towards the floodplain.

The contour lines of the streamwise velocity component are shown in Figures 4.34, 4.36 and 4.38. In section B', negative streamwise velocity contours are observed inside the scour hole, probably due to the blockage of the abutment. This is clearly shown in section A' (Figure 4.27), as this portion of section B' is also part of the measurement for section A'. Inside the scour hole, close to the side wall, the velocity contours dip downward. The dip in contours at section C' is more pronounced due to the effect of secondary flow. At this section no negative streamwise velocity is observed. At section D' the contour lines are quite different to that in sections B' and C'. The shape of the contours follows the bed contour in section D'. In general, the shapes of the contour lines correspond to the secondary flow velocity vectors. In the main channel, the streamwise velocity contours shows that there is no significant effect due to the presence of the abutment. This is probably because the abutment length is relatively short.

Figure 4.33. Velocity vectors at cross section B' (main flow into paper).

Figure 4.34. Contours of streamwise velocity component, u/U at cross section B'.

Figure 4.35. Velocity vectors at cross section C'.

Figure 4.36. Contours of streamwise velocity component, u/U at cross section C'.

Figure 4.37. Velocity vectors at cross section D'.

Figure 4.38. Contours of streamwise velocity component, u/U at cross section D'.

4.3.3 Reynolds shear stresses

Figures 4.39 to 4.46 show the contour of the normalized Reynolds shear stresses, $uw^+ = -\overline{u'w'}/u_*^2$ and $uv^+ = -\overline{u'v'}/u_*^2$, for cross sections A', B', C' and D'. The shear velocity at the centerline of the floodplain at the approach section is used to normalize the Reynolds shear stresses.

The contour plot of the Reynolds shear stresses at cross section A' are presented in Figures 4.39 and 4.40. Figure 4.39 shows that uw^+ has positive sign and the higher stresses are observed in front of the abutment. At the corner of the scour hole, negative uw^+ is observed (dashed line in Figure 4.39). This is related to the direction of the near bed velocity and its gradient. The magnitude of uv^+ is less than uw^+ and has negative value at some locations i.e. at the upstream face of the abutment.

The contour of the normalized Reynolds shear stress, uw^+ , for sections B', C' and D' are shown in Figures 4.41, 4.43 and 4.45, respectively. In the main channel below $z = 0$, there is not much difference in the isoline pattern. The Reynolds shear stresses are generally below 0.4. The pattern in this region is rather symmetrical and the maximum stress occurs at about 5 cm above the main channel bed level. In the region above $z = 0$, the symmetrical pattern can be seen at section D'.

The changes of uw^+ distribution inside the scour hole can be observed in sections B', C' and D'. In section B' the maximum shear stress occurs at $z = 0$, and $y = 20$ cm from the sidewall, and only positive stresses are recorded in this section. Negative stresses were observed at sections C' and D'. In section C', negative values were observed at $z = -10$ to -15 cm, and below $z = -15$ to the bed the stresses are positive. In section D', which is downstream of the abutment, the Reynolds shear stresses immediately at the back portion of the abutment, where some of the scoured sediment was deposited, were negative. But at the other side of the scoured profile it has positive value. This shows the contribution of the Reynolds shear stress on the sediment transport process, i.e., in the deposition and transportation of sediments, which can be observed clearly at the region downstream of the abutment.

The maximum value of uw^+ inside scour hole region is about 4.0, which is ten times larger than that in the main channel. There was not much difference in the stress distributions at the floodplain outside the scoured bed region, as the 3D flow feature is less at this part.

Figures 4.42, 4.44 and 4.46 show the normalized Reynolds shear stress uv^+ of sections B', C', and D'. Almost all the stress values in the main channel and the non-scoured floodplain region were negative. The only region where positive stress is observed is in the main channel at the region close to the main channel sidewall. Different contour pattern of uv^+ were observed in the scour hole region of sections B', C' and D'.

At section B', negative value was observed for the region close to the sidewall and above the upper part of the scour hole. In the middle part of the scour hole, the stress was positive. At cross section C', the stress in most part of the scour hole is positive. Negative stress value was observed close to the upper part of the abutment tip near the surface and on the upper part at the edge of the scour hole. At cross section D' most of the regions in the scour hole have negative stress value except those above its edge. The difference in the sign of uv^+ indicates the momentum transfer in the lateral direction between two regions of different sign.

In the main channel, the magnitudes of uw^+ are of the same order compared to those in the flat bed experiment. In the floodplain, the magnitudes of uw^+ are slightly lower compared to those in the flat bed experiment. At the downstream of the abutment, at the scour hole region, the magnitudes of uw^+ are slightly higher compare to those in the flat bed experiment. This is because flow concentration in this part, where at the flat bed condition the flow is deflected towards main channel.

The measured uv^+ in the flat bed experiment was of the same order to that in the scoured bed experiment, except in the scour hole region at the downstream of the abutment. In this region the magnitudes of uv^+ are significantly higher, up to three times, compared to those in the flat bed experiment. The differences in the magnitudes of uw^+ and uv^+ were caused by the formation of the scour hole around the abutment which affects the lateral shear and secondary flow strength.

Figure 4.39. Normalized Reynolds shear stress, uw^+ at cross section A' ($x = 0$ cm indicates up stream face of abutment).

Figure 4.40. Normalized Reynolds shear stress, uv^+ at cross section A' ($x = 0$ cm indicates up stream face of abutment).

Figure 4.41. Normalized Reynolds shear stress, uw^+ at cross section B'.

Figure 4.42. Normalized Reynolds shear stress, uv^+ at cross section B'.

Figure 4.43. Normalized Reynolds shear stress, uw^+ at cross section C'.

Figure 4.44. Normalized Reynolds shear stress, uv^+ at cross section C'.

Figure 4.45. Normalized Reynolds shear stress, uw^+ at cross section D'.

Figure 4.46. Normalized Reynolds shear stress, uv^+ at cross section D'.

4.3.4 Turbulent intensities and turbulent kinetic energy

Figures 4.47 to 4.49 show the contour lines of normalized turbulent intensities of streamwise, spanwise and vertical velocity components, $u^+ = u_{rms}/u_*$, $v^+ = v_{rms}/u_*$ and $w^+ = w_{rms}/u_*$, respectively, at cross section A'. The turbulent intensities were normalized by the shear velocity u_* obtained at the centerline of the floodplain. It can be seen that u^+ increases as it flows into the scour hole. The values of u^+ was higher in front of the abutment face and at the upstream edge of the scour hole. The higher value at the upstream abutment face is related to the downflow due to the presence of the abutment, while the higher value at the upstream end of the scour hole is due to the difference in the primary mean velocity above the bed level and inside the scour hole. The highest v^+ was only observed at the upstream part of the scour hole.

Figure 4.50 shows contour lines of normalized turbulent kinetic energy (TKE), $k^+ = 1/2 \cdot (u_{rms}^2 + v_{rms}^2 + w_{rms}^2)/u_*^2$. The regions with high TKE were found in front of the upstream abutment face, where the scouring process took place due to the downflow, and immediately below the upstream edge of the scour hole as the flow dives into the scoured region. The magnitudes of u^+ , v^+ , w^+ and k^+ above the floodplain bed level are in the same order compared to that of the flat bed condition.

Figures 4.51 to 4.62 show the turbulent intensities and turbulent kinetic energy of sections B', C' and D'. In general, the turbulent intensities in the scour hole for all the sections are in the same range of magnitude between 0 to 4.

At cross section B', the contours of u^+ , v^+ and w^+ exhibit similar patterns. In the scour hole there are two regions with high turbulent intensities. The first is located at 20 cm from the sidewall slightly below the initial bed level and the second is about 10 cm from the sidewall, and 12 cm below the initial bed level. These two regions correspond to the vortex formed inside the scour hole (see also Figures 4.27 and 4.33). In the main channel, the u^+ , v^+ and w^+ are symmetrical in pattern, and the order of magnitude is $u^+ > v^+ > w^+$.

Different contour pattern of u^+ , v^+ and w^+ at cross section C' can be seen in Figures 4.55, 4.56 and 4.57. The maximum u^+ was observed inside the scour hole at around initial bed level. The maximum v^+ was observed at about 10 cm below the initial bed level, close to the abutment tip, and the maximum w^+ was observed at the initial bed level and close to the bottom corner of the scour hole. In the main channel, u^+ increased at the region close to the main channel bank, while for v^+ and w^+ there are no significant change.

At cross section D', downstream of the abutment, the maximum value of u^+ and v^+ was found close to the water surface, and the maximum w^+ value was found to be below the water surface level. In the main channel, all the turbulent intensity components exhibit the same pattern as that of section C'.

Figures 4.54, 4.58 and 4.62 show the turbulent kinetic energy at sections B', C' and D'. In the scour hole, TKE at cross section C' was less than those at sections B' and D'. This is probably due to the depth of the scour hole at section C' which is deeper than that of sections B' and D'. At section D', where the flow depth in the scour hole is shallower than that of section B', the largest magnitude of TKE is significantly higher compared to that of section B'. In general, an increase in mean velocity will be followed by an increase in turbulent intensity as the mean velocity is the source of turbulence generation. Figure 4.54 shows that inside the scour hole at section B' there are two regions of peak magnitude of TKE. This feature is not found at sections C' and D'.

For the main channel and at the region outside of the scour hole in the floodplain, the magnitude of u^+ , v^+ , w^+ and k^+ in the scoured bed experiments are in the same order compared to that of the flat bed experiments. However, at the downstream of the abutment the k^+ for the scoured bed condition is higher compared to that of the flat bed experiment. This is because the flow is more concentrated along the centerline of the scour hole after the formation of the scour hole.

Figure 4.47. Normalized turbulent intensity of streamwise velocity component, u^+ at cross section A' (x = 0 cm indicates up stream face of abutment).

Figure 4.48. Normalized turbulent intensity of spanwise velocity component, v^+ at cross section A' (x = 0 cm indicates up stream face of abutment).

Figure 4.49. Normalized turbulent intensity of vertical velocity component, w^+ at cross section A' (x = 0 cm indicates up stream face of abutment).

Figure 4.50. Normalized turbulent kinetic energy, k^+ at cross section A' (x = 0 cm indicates up stream face of abutment).

Figure 4.51. Normalized turbulent intensity of streamwise velocity component, u^+ at cross section B'.

Figure 4.52. Normalized turbulent intensity of spanwise velocity component, v^+ at cross section B'.

Figure 4.53. Normalized turbulent intensity of vertical velocity component, w^+ at cross section B'.

Figure 4.54. Normalized turbulent kinetic energy, k^+ at cross section B'.

Figure 4.55. Normalized turbulent intensity of streamwise velocity component, u^+ at cross section C'.

Figure 4.56. Normalized turbulent intensity of spanwise velocity component, v^+ at cross section C'.

Figure 4.57. Normalized turbulent intensity of vertical velocity component, w^+ at cross section C'.

Figure 4.58. Normalized turbulent kinetic energy, k^+ at cross section C'.

Figure 4.59. Normalized turbulent intensity of streamwise velocity component, u^+ at cross section D'.

Figure 4.60. Normalized turbulent intensity of spanwise velocity component, v^+ at cross section D'.

Figure 4.61. Normalized turbulent intensity of vertical velocity component, w^+ at cross section D'.

Figure 4.62. Normalized turbulent kinetic energy, k^+ at cross section D'.

4.4 Summary

Observations on the flow around an abutment sited on the floodplain of a two-stage channel showed that the relatively short abutment affects significantly the velocity vectors for region close to the abutment. The streamwise velocity accelerates as it flows pass the abutment site cross section. A small increase of the secondary flow magnitude was observed at the main channel, especially at cross section just downstream of the abutment. The spanwise velocity has a net flow direction from the floodplain towards the main channel for sections near the abutment, while the reverse is true for the approach cross-section.

For the flat bed experiment, the largest magnitude of the spanwise and vertical velocities close to the abutment tip were around 0.8 and 0.2 times that of the mean streamwise velocity at the approach section (section A), respectively. Near the abutment tip where the deepest scour depth usually occurred, the velocities at the lower depth were higher than those of the upper part due to the effect of the flow diversion and the associated downflow. The bed shear stress amplification for a point close to the abutment tip was 3.8 times larger than that of the approach section or 1.8 times larger than the critical shear stress of the sediment used.

Most of the measured uw^+ has positive values, except for some points close to the abutment tip, downstream of the abutment and close to the main channel bank. The negative uw^+ corresponded to the negative gradient of the vertical distributions of streamwise velocity profile. Similarly, the negative values of uv^+ corresponded to negative gradient of the vertical distributions of spanwise velocity profile. This can be seen mostly for the region close to the abutment tip and main channel. The different sign of uv^+ which indicates the momentum transfers and secondary flow formation and strength were observed at the region near to the junction. An increase of negative uv^+ magnitude was observed at the region downstream of the abutment.

The presence of the abutment increases the turbulent kinetic energy especially at the region close to the abutment due to the vortices generated by the flow blockage and diversion.

After the scour hole was established, the overall spanwise velocity in the floodplain decreased and an anticlockwise vortex was formed in the scour hole adjacent to the abutment tip. A similar vortex was also observed by Kwan and Melville (1994) in their experiment for a wing-wall abutment placed in a rectangular channel. In the present study, it was observed that at 50 cm downstream of the abutment this vortex was weak and there was another vortex with clockwise direction of rotation.

The values of u^+ and v^+ were higher in front of the abutment face and at the upstream end of the scour hole. In general, the turbulent intensities in the scour hole for all the sections are in the same range of magnitude between 0 to 4. In the scour hole, TKE at abutment site section was less than those at the upstream and downstream section. This is probably due to the depth of the scour hole at the abutment site section which is deeper than that of upstream and downstream sections.

Figures 4.63 and 4.64 schematically present the summary of the observed flow features (main flow and secondary flows) at initial condition and at equilibrium condition, respectively.

Figure 4.63. Schematic of flow features at initial (flat bed) condition.

Figure 4.64. Schematic of flow features at equilibrium condition.

CHAPTER 5. SCOUR AROUND AN ABUTMENT PLACED IN A FLOODPLAIN

5.1 Introduction

This chapter presents the result of experiments on scouring process and a proposed model to estimate the equilibrium scour depth for an abutment terminated in the floodplain of a two-stage channel. The study was conducted under clear water scour condition. The scouring process was observed in terms of its geometry and the scour depth development with time towards the equilibrium state. The results showed three types of scour hole geometries after the equilibrium state was attained. A predictive model to estimate the equilibrium scour depth was developed based on the flow continuity equation, scour geometry, flow resistance in alluvial channel and the experimental results. The time development of scour hole was also analyzed to provide an understanding of the time required for the scouring process to reach its equilibrium.

5.2 Scour Hole Geometry

There were three types of the equilibrium scour hole geometry observed in the experiments, i.e. the inverted cone type, edge-truncated inverted cone type and the upstream edge-truncated inverted cone type. The graphical presentation and sample pictures of these geometries are given in Figures 5.1 to 5.6 and their details are described in the following paragraphs.

Inverted cone shape

Generally, it was observed that the scour began at the side of the abutment tip which then grew deeper and wider and finally touched the upstream corner of the abutment's tip. Figures 5.1 and 5.2 show the inverted cone shape type scour hole where the deepest scour was found at the upstream corner of the abutment. The sediments were mainly scoured and transported from the deepest part, and this in turn caused sediments from the surrounding region to slide into the deepest part.

This way the scour hole increased its depth and width as time progresses. The down flow at the upstream face of the abutment also caused the sediments to slide towards the deepest part of the scour hole.

Edge-truncated inverted cone shape

Figures 5.3 and 5.4 show typical examples of the edge-truncated inverted cone shape type scour hole. For a relatively deep scour hole, the edge of the scour hole can reach the sidewall where the abutment is attached. In this case the shape of the cross section of the scour hole shape resembles a half inverted cone, but the surface of the sediment adjacent to the upstream face of the abutment has a mild slope and the deepest scour hole is at the upstream corner of the tip of the abutment.

Extended upstream edge-truncated inverted cone shape

For shallow floodplain flow depth condition and after the equilibrium scour depth has been attained, it was observed that the transport of sediment continues and this propagates towards the upstream direction and created another shallow scoured zone, as shown in Figures 5.5 and 5.6. After the main scour hole has been established, a portion of the blocked flow goes straight into the main scour hole. However a portion of the flow near the sidewall is in the stagnant region and causes the flow to divert and concentrate along its course. This concentrated flow led to scouring of the secondary scour hole towards the upstream direction (see Figure 5.6).

Scour hole development

The development of the scour hole geometry always starts at the side of the abutment nose. In a short time, the scour will reach the upstream corner of the tip of the abutment and the scour hole grew deeper. This stage was also reported by Oliveto and Hager (2005) for the case in rectangular channels. The deepest part of the scour hole was observed at the upstream corner of the tip of the abutment, especially for the first and second types of scour hole geometry shown in Figures 5.1 and 5.3, respectively. As scouring progresses and at the equilibrium state, the

scour hole shape may take the form shown in Figures 5.1 and 5.3, or the scour hole shape will continue to evolve and has a final shape similar to that shown in Figure 5.5. During this continuing evolution of the scour shape formation, there is not much increase of the scour depth at the area close to the abutment tip. Most of the observed scour holes were found to be of the first and second types. The third type (Figure 5.5) was observed for the case of shallow floodplain flow depth. The extended upstream edge scoured zone had a mild slope and shallow depth, and from observations it was propagating upstream after the near equilibrium scour depth at the area close to the abutment tip was attained. For the experiments conducted in this study, the scouring duration up to the equilibrium stage ranges from 2 to 522 hours (see Table 5.1).

Figure 5.1. Typical cross sectional and plan view of an inverted cone scour hole.

Figure 5.2. Example of an inverted cone scour hole (note: the arrow indicates the flow direction).

Figure 5.3. Typical cross sectional and plan view of edge-truncated inverted cone scour hole.

Figure 5.4. Example of an edge-truncated inverted cone scour hole.

Figure 5.5. Typical cross sectional and plan view of an edge-truncated inverted cone scour hole with a secondary upstream edge scoured zone.

Figure 5.6. Example of an edge-truncated inverted cone scour hole with a secondary upstream edge scoured zone.

Table 5.1. Experimental conditions and results.

No.	Expts.	Q_{fm} l/s	L cm	d_f cm	d_m cm	d_f/d_m	t_e hr	t_{actual} hr	d_{se} cm	U_t cm/s	U_m cm/s	U_c cm/s	U_t/U_c	Q vel l/s	Remarks
1	20-50-02	50	20	3.75	18.75	0.20	2.0	3	3.0	20.73	36.71	31.5	0.66	49	
2	20-60-02	60	20	3.75	18.75	0.20	27.6	46	10.9	25.77	42.30	31.5	0.82	57	
3	20-70-02	70	20	3.75	18.75	0.20	45.0	45	10.8	28.80	49.39	31.5	0.91	66	
4	20-50-03	50	20	6.40	21.40	0.30	96.0	120	10.0	20.18	27.64	34.6	0.58	48	
5	20-60-03	60	20	6.40	21.40	0.30	350.0	503	15.3	24.06	30.73	34.6	0.70	55	
6	20-70-03	70	20	6.40	21.40	0.30	53.0	71	18.6	28.26	37.67	34.6	0.82	66	
7	20-50-04	50	20	10.00	25.00	0.40	48.0	48	5.5	17.68	20.63	37.4	0.47	49	
8	20-60-04	60	20	10.00	25.00	0.40	168.0	192	13.2	21.47	24.80	37.4	0.57	59	
9	20-70-04	70	20	10.50	25.50	0.41	523.0	546	23.5	22.99	27.12	37.4	0.61	66	
10	20-80-04	80	20	10.00	25.00	0.40	121.0	150	23.7	29.86	34.57	37.4	0.80	82	
11	20-100-05	100	20	15.00	30.00	0.50	435.1	486	34.1	30.69	33.34	39.5	0.78	106	
12	20-70-04A	70	20	10.50	25.50	0.41	222.0	246	17.3	21.38	29.06	37.4	0.57	67	7.2 m divider
13	20-70-04B	70	20	10.50	25.50	0.41	264.0	264	17.3	24.42	n.a.	37.4	0.65	67	2.4 m divider
14	35-50-02	50	35	3.75	18.75	0.20	180.0	192	9.9	20.73	36.71	31.5	0.66	49	
15	35-60-02	60	35	3.75	18.75	0.20	183.9	198	14.8	25.77	42.30	31.5	0.82	57	
16	35-50-03	50	35	6.40	21.40	0.30	115.9	180	15.8	20.18	27.64	34.6	0.58	48	
17	35-60-03	60	35	6.40	21.40	0.30	322.0	333	24.1	24.06	30.73	34.6	0.70	55	
18	35-50-04	50	35	10.00	25.00	0.40	168.0	174	11.2	17.68	20.63	37.4	0.47	49	
19	35-60-04	60	35	10.00	25.00	0.40	174.0	240	19.8	21.47	24.80	37.4	0.57	59	
20	50-50-02	50	50	3.75	18.75	0.20	72.3	78	12.7	20.73	36.71	31.5	0.66	49	
21	50-60-02	60	50	3.75	18.75	0.20	43.0	138	18.5	25.77	42.30	31.5	0.82	57	
22	50-50-03	50	50	6.40	21.40	0.30	44.0	78.7	15.4	20.18	27.64	34.6	0.58	48	
23	50-50-04	50	50	10.00	25.00	0.40	150.0	168	15.9	17.68	20.63	37.4	0.47	49	

Note:

Q_{fm} : Discharge obtained from flow meter reading

Q_{vel} : Discharge obtained from integration of measured point velocities.

5.3 Prediction Model of Equilibrium Scour Depth

Two-stage channels have different flow characteristics compared to channels with a simple cross sectional geometry such as prismatic trapezoidal or rectangular channels. The difference in flow characteristics leads to the question of its effect on the local scouring process around structures sited in such channels. There were some studies which investigated the effect of a two-stage channel geometry on scour around an abutment terminated on a floodplain (see Chapter Two). Sturm and Janjua (1994) mentioned that the discharge ratio at the approach and abutment site will influence the abutment scour depth. Melville (1995) suggested an imaginary boundary concept for abutment terminating on the floodplain of two-stage channels. Cardoso and Bettess (1999) compared their measurement result on scour depth around abutment terminating at a floodplain to Melville's (1992) maximum limit (envelope) of scour depths around abutment placed in rectangular channel. Cardoso and Bettess (1999) scour depths were less than the limit defined by Melville (1992) and they supported the idea of an imaginary boundary concept. However, Cardoso and Bettess did not provide analysis on the equilibrium scour depth prediction. Kouchakzadeh and Townsend (1997) showed that the ratio of the flow intercepted by the abutment and that related to a specific channel width at the abutment end is a significant parameter to account for the lateral momentum transfer and the scouring at the abutment. Their scour prediction model gave a good agreement with the equilibrium scour depth in the range of $\pm 10\%$. The equilibrium scour depths in their experiments were obtained by extrapolation from their results of 5 hours scouring experiments. Based on their comparison study, the scour depths after 5 hours were approximately 60% and 65% of the equilibrium scour depth for the two sands of $d_{50} = 0.5 \text{ mm}$ and $d_{50} = 0.7 \text{ mm}$ used in their study, respectively.

The present study shows that there is considerable flow interaction between the floodplain and the main channel which is thought to be a major factor affecting the scouring process. Comparison of the flow distribution at the approach section and the cross section at the abutment site for the flat bed conditions in Experiments 20-60-03 and 20-80-04, presented in Figures 5.7 and 5.8, respectively, showed an increase in the main channel discharge and a decrease in the floodplain discharge.

This change may affect the scouring process and can be included as being the geometry effect of two-stage channels on equilibrium scour depth.

Figure 5.7. Unit discharge at approach section and abutment-site cross-section, Experiment 20-60-03.

Figure 5.8. Unit discharge at approach section and abutment-site cross-section, Experiment 20-80-04.

At the initial (flat bed) condition the flow in the floodplain will be diverted towards the main channel. The strength of diversion will be dependent on the abutment length (L), floodplain flow depth (d_f), floodplain width (B_f) and the flow velocity in the floodplain (U_f) and main channel (U_m). In the beginning, the amount of the flow diverted to the main channel ranges from zero (for a case where $L = 0$) up to the amount of flow blocked by the abutment (for a case where $L = B_f$). However, as the scour hole develops, the flow distribution will not be the same as its initial condition (see Figures 5.7 and 5.8). In regards to the flow and scouring relationship, it could be expected that the changes in flow distribution affect the scouring process, while the changes in the scour hole geometry also affect the flow distribution.

The partial flow diversion due to the abutment reduces the flow in the floodplain and this affects the scour processes around abutment terminated in this region. As a result of the diversion, the scour depth around an abutment in the floodplain of a two-stage channel would be expected to be less than that in a rectangular channel given the same flow characteristics in the approach section. However, due to the interaction between the floodplain and main channel flows (see Figure 4.30 in Chapter Four), it is possible that the flow concentrates through the scour hole and this may cause an increase in the equilibrium scour depth. Hence, in the analysis of scouring process towards the equilibrium state, the effect of the interaction between floodplain and main channel flow needs to be taken into account.

Once a scour hole is formed it can be assumed that the mechanism of scouring inside the scour hole for abutment terminated in a floodplain is similar to that in a rectangular channel. The difference in the scour depth attained is believed to be mainly caused by the portion of the flow which is diverted in the process due to the effect of the two-stage channel geometry. If the channel properties can be accounted for, then it is possible to develop a scour depth prediction model based on the formulation of scour around abutment sited in a simple rectangular channel. This is attempted in the following sections.

In general, scour prediction formulas can be grouped into three types. They are the regime approach (Ahmed, 1953) which formulate the scour depth as a function of discharge; the dimensional approach (Garde et al. 1961, Liu et al. 1961, Froeichlich 1989 and Melville 1992); and analytical or semi-empirical approach (Laursen 1963, Gill 1972, Lim 1997, Rahman 1998) based on the increase in sediment transport due to the increase of shear stress caused by the abutment. Lim's (1997) model is based on the premise that the obstruction in a channel would increase the shear stress at the area adjacent to the structure. The increase of the shear stress will increase the sediment transport capacity in the locality. A semi-empirical approach as a function of the flow-continuity, scour geometry and a generalized form of the power-law formula for flow resistance in an alluvial channel is developed according to that premise.

Lim's (1997) formulation had been tested on 252 data of clear water scour case and employs general properties of sediment and flow. These points serve to show the generality of the formulation and the approach is improved for the case of abutment terminating in a floodplain. The definition sketch for abutment scour in a rectangular channel (Lim, 1997) is shown in Figure 5.9. The flow continuity equation is applied at the approach cross section and at the abutment site cross section:

$$Q_1 = Q_2' + Q_2'' \quad (5.1)$$

where Q_1 = total discharge of approach flow; Q_2' = discharge in the scoured cross section within the width b_{se} ; and Q_2'' = discharge within the width $(B_2 - b_{se})$.

At the initial condition, with the time $t = 0$:

$$U_1 d_1 B_1 = U_{2m} d_1 b_2 + Q_2'' \quad (5.2)$$

where U_1 = average velocity at approach section; d_1 = flow depth at approach section; B_1 = channel width; U_{2m} = average velocity within b_{se} at $t = 0$. After the equilibrium scour depth condition had been established, the average velocity in the

scour hole is a minimum U_{2c} and it is assumed that there is no more sediment transported from the scour hole. This situation gives:

$$U_1 d_1 B_1 = U_{2c} \left(d_1 + \frac{1}{2} d_{se} \right) b_2 + Q_2 \quad (5.3)$$

where d_{se} = equilibrium scour depth.

Figure 5.9. Definition sketch of scour hole around abutment in a rectangular channel.

From Equations (5.2) and (5.3):

$$\frac{U_{2m}}{U_{2c}} = \left(1 + \frac{d_{se}}{2d_1}\right) \quad (5.4)$$

A logarithmic velocity distribution for fully rough flow is approximated by power-law form (Chen 1991):

$$\frac{U}{u_*} = m \left(\frac{d}{k_s} \right)^n \quad (5.5)$$

where U = flow velocity; u_* = shear velocity; d = normal flow depth; k_s = equivalent sand roughness ($=2d_{50}$); m and n = coefficient and exponent, respectively, which depend on the types of the bed forms (Chitale 1967) or the applicable range of U/u_* used to approximate the logarithmic velocity distribution (Chen 1991).

At $t = 0$, U and u_* would be at their maximum, i.e. U_{2m} and u_{*m} . At equilibrium condition they would be at their critical values, i.e. U_{2c} and u_{*c} . Applying these values into Equation (5.5), the velocity distribution at initial and equilibrium condition are expressed in Equations 5.6 and 5.7, respectively.

$$\frac{U_{2m}}{u_{*m}} \approx m \left(\frac{d_1}{2d_{50}} \right)^n \quad (5.6)$$

$$\frac{U_{2c}}{u_{*c}} \approx m \left[\frac{\left(d_1 + \frac{1}{2} d_{se} \right)}{2d_{50}} \right]^n \quad (5.7)$$

From Equations (5.4), (5.6) and (5.7) the following relationship can be obtained:

$$\frac{u_{*m}}{u_{*c}} = \left(1 + \frac{d_{se}}{2d_1}\right)^{n+1} \quad (5.8)$$

Lim (1997) proposed an empirical relationship of u_{*m}/u_{*1} , where u_{*m} = maximum shear velocity due to abutment and u_{*1} = shear velocity at the approach section, with abutment length and approach flow depth based on the experimental results of Rajaratnam and Nwachukwu (1983a).

$$\frac{u_{*m}}{u_{*1}} = \left[1.2 \left(\frac{L}{d_1} \right)^{0.5} + 1 \right] \quad (5.9)$$

If the floodplain is assumed to be wide compared to the main channel width, then as a first approximation the two-stage geometry can be ignored and the approach is based on a rectangular channel geometry. Based on this approximation, the value of n and m can be assigned as $1/3$ and 3.9 , respectively (Lim, 1997). For the assigned values of n and m , and using Equations (5.8) and (5.9), and applied to the flow in the floodplain of a two-stage channel, the following expression is obtained:

$$\frac{d_{se}}{d_f} = K_s \left\{ 2 \left(\frac{u_{*1}}{u_{*c}} \right)^{0.75} \left[0.9 \left(\frac{L}{d_f} \right)^{0.5} + 1 \right] - 2 \right\} \quad (5.10)$$

where K_s = the shape factor of the abutment, is added to Equation (5.10) to make the formula more general and d_f = floodplains flow depth. To simplify the equation further, the shear velocity can be expressed as a function of mean velocity, Manning roughness coefficient, n_M , gravitational acceleration and hydraulic radius, R :

$$u_* = \sqrt{\frac{\tau}{\rho}} = \sqrt{gRS_e} = \sqrt{\frac{g U^2 n_M^2}{R^{1/3}}} = U n_M \sqrt{\frac{g}{R^{1/3}}} \quad (5.11)$$

As the shear velocity is proportional to the mean velocity and assuming that the rest of the parameters are of the same order of magnitude, the ratio of the approach and critical value of shear velocity in Equation (5.10) can be substituted by their corresponding mean velocity.

$$\frac{d_{se}}{d_f} = K_s \left\{ 2 \left(\frac{U_f}{U_c} \right)^{0.75} \left[0.9 \left(\frac{L}{d_f} \right)^{0.5} + 1 \right] - 2 \right\} \quad (5.12)$$

Equation (5.12) is preferred as it uses bulk parameters of the flow and abutment which are more advantageous in the practical sense. The application of Equation (5.12) on the present experimental data of scour around abutment terminating at the floodplain is presented in Figure 5.10. The shape factor value was taken as 1.0 in this case since vertical-wall abutment is used in the test. Equation 5.12 is valid only

for $\left(\frac{U_f}{U_c}\right)^{0.75} \left[0.9 \left(\frac{L}{d_f}\right)^{0.5} + 1 \right] \geq 1$, so that $d_{se}/d_f = 0$ when

$$\left(\frac{U_f}{U_c}\right)^{0.75} \left[0.9 \left(\frac{L}{d_f}\right)^{0.5} + 1 \right] = 1.$$

Figure 5.10. The plot of computed d_f/d_m using Equation (5.12) compared to the measured data.

Figure 5.10 shows that the present data generally give a fair agreement between the predicted scour depths and the measured ones, except for the data of Experiment 20-50-02, which is significantly over predicted. This data was excluded from further analysis. Further examination suggests the data points can be grouped into

three clusters. The first, second and third clusters are assigned for flow depth ratios, d_f/d_m , of 0.2, 0.3 and 0.4, respectively. For data with flow depth ratio of 0.2, Equation (5.12) tends to over-predict the equilibrium scour depth, while for the flow depth ratio of 0.4, the equation tends to under-predict the equilibrium scour depth. The second cluster for $d_f/d_m = 0.3$ shows better agreement between the predicted and the measured equilibrium scour depth compared to the other two clusters.

As mentioned earlier, it is thought that the two-stage geometry would affect the scouring process and the difference in predicted and measured scour depth shown in Figure 5.12 can be improved if the two-stage channel geometry is taken into account. The degree of interaction between floodplain and main channel flow is closely related to the flow depth at the floodplain and the main channel (Bhowmik and Demissie, 1982). However, the role of this interaction in the scouring process is not clearly known. Considering the parameters in the right hand side of Equation (5.12), the effect of the two-stage channel geometry could affect the scouring process through the approach flow velocity, U_f , and the floodplain flow depth, d_f . Generally, during the scouring process the change in flow depth was observed to be insignificant. Hence, to account for the effect of two-stage channel geometry, it is proposed that an adjustment to the approach mean flow velocity at the floodplain be introduced, i.e., an adjustment factor, α , for the velocity term is introduced into Equation 5.12., as follows:

$$\frac{d_{se}}{d_f} = K_s \left\{ 2 \left(\frac{\alpha U_f}{U_c} \right)^{0.75} \left[0.9 \left(\frac{L}{d_f} \right)^{0.5} + 1 \right] - 2 \right\} \quad (5.13)$$

As the degree of interaction in two stage channel is related to the flow depth ratio (Ackers, 1993), it is logical to assume that α is a function of the flow depth ratio, d_f/d_m . Regarding to the expected relationship of α and d_f/d_m , it would be beneficial to use a broader range of d_f/d_m from the point of view of application. In this respect, the data for d_f/d_m of 0.5 and 0.6 from the works by Teo (1999) and Lee (2000), respectively, are used in the analysis. These two works were conducted earlier in the Hydraulic Laboratory of NTU. The plot of measured data and

computed value of equilibrium scour depth of the present data, and those by Teo and Lee is presented in Figure 5.11. The figure shows that Teo's and Lee's data of d_f/d_m 0.5 and 0.6 forms a different cluster where the prediction is much lower than the measured values. The trend clearly shows the effect of d_f/d_m on the equilibrium scour depth.

A least square analysis for each data cluster was done and the adjustment factor α as a function of the flow depth ratio is shown in Table 5.2 and also plotted in Figure 5.12.

Figure 5.11. The plot of computed d_f/d_m using equation 5.12 compared to the measured data, including Teo's and Lee's data.

Table 5. 2. Adjustment factor for approach flow velocity

d_f/d_m	α
0.2	0.85
0.3	1.07
0.4	1.11
0.5	1.28
0.6	1.21

Figure 5.12. Adjustment factor of approach flow velocity as the function of flow depth ratio.

For d_f/d_m more than 0.5 the difference between flow velocity in the floodplain and in the main channel is small. Hence, the effect of two-stage channel geometry is minor and the channel can be treated as a single channel. Knight and Demetriou (1983) mentioned that the interaction between floodplain and the main channel occurs mainly for the low d_f/d_m , and they used $d_f/d_m = 0.5$ as the upper limit of their experiments on interaction between floodplain and main channel flow. The proposed predictive model is suggested for d_f/d_m lower than 0.5. For d_f/d_m more than 0.5, the predictive model of single rectangular channel, e.g. Equation (5.12), can be used to compute the equilibrium scour depth.

The adjustment factor can be expressed as a function of d_f/d_m as follows.

$$\alpha = 1.54 \left(\frac{d_f}{d_m} \right)^{0.35} \quad (5.14)$$

Equation (5.14) is limited for d_f/d_m ranges from 0.2 to 0.6 and the function is biased more to d_f/d_m less than 0.5. Substitution of this relationship into Equation 5.13 gives:

$$\frac{d_{se}}{d_f} = K_s \left\{ 2.76 \left(\frac{d_f}{d_m} \right)^{0.26} \left(\frac{U_f}{U_c} \right)^{0.75} \left[0.9 \left(\frac{L}{d_f} \right)^{0.5} + 1 \right] - 2 \right\} \quad (5.15)$$

Figure 5.13 shows that Equation (5.15) gives a good agreement between the measured and the predicted equilibrium scour depth for the present data. It should be mentioned that the scour holes for the present data set do not encroached into the bank of the main channel.

Figure 5.13. The plot of computed df/dm against measured df/dm of present data with vertical-wall abutment.

In Equation (5.15) the critical mean velocity for sediment motion, U_c , was computed using an equation obtained from logarithmic distribution of velocity:

$$\frac{U_c}{u_{*,c}} = 5.75 \log \left(5.53 \frac{d}{d_{50}} \right) \quad (5.16)$$

where $u_{*,c}$ = critical shear velocity based on the d_{50} size. The critical shear velocity can be obtained by computing the dimensionless sediment diameter, D_* , and dimensionless critical shear stress, $\tau_{*,c}$, using Equations (5.17) and (5.18),

respectively. Using these two dimensionless parameter the u_{*c} can be computed using (5.19).

$$D_* = d_{50} \left(\frac{\Delta g}{\nu^2} \right)^{1/3} \quad (5.17)$$

$$\tau_{*c} = 0.13 D_*^{-0.392} e^{-0.015 D_*^2} + 0.045 [1 - e^{-0.068 D_*}] \quad (5.18)$$

$$u_{*c} = \sqrt{\tau_{*c} \Delta g d_{50}} \quad (5.19)$$

where $\Delta = (\rho_s - \rho)/\rho$, ρ_s = sediment density and ρ = water density. Equation (5.18) was proposed by Yalin and Silva (2001) based on experimental data with the shear Reynolds number ($u_* D / \nu$, with D = characteristic diameter of the particle) ranging from 0.03 to 2000.

Comparison with other published data

Equation (5.15) is applied to three published data on the study of clear water scour around abutment terminating on a floodplain of two-stage channels. The works of Sturm and Janjua (1994), Kouchakzadeh and Townsend (1997) and Cardoso and Bettess (1999) will be used to assess the generality of the formulation. In terms of time duration, the data of Cardoso and Bettess (1997) is superior to the others as their tests were conducted for very long time up to the equilibrium scour depth stage. On the other hand, Sturm and Janjua (1994) run their scouring tests for only 10 to 12 hours, and it is unlikely that the scour has reached the equilibrium condition based on the experience of the present study. Although Kouchakzadeh and Townsend (1997) performed each scouring test for 5 hours, they obtained the equilibrium scour depth indirectly by extrapolation, based on the preliminary tests which showed that the scour depth at 5 hours was approximately 60% and 65% of the equilibrium scour depth for the sand size of $d_{50} = 0.5$ mm and 0.7 mm that they used, respectively.

Beside the difference in the test duration, there is also difference in the shape of the abutment used by the above mentioned researchers. Sturm and Janjua (1994) used vertical-wall abutments for all of their tests. Kouchakzadeh and Townsend (1997)

used four types of abutment shape, i.e.: vertical-wall, wing-wall, semi-circular tip, and spill-through. Cardoso and Bettess (1999) used thin plate, placed vertically in the floodplain, to simulate the abutment structure. In order to have an equivalent expression of scour depth from different type of abutment, the shape factors for abutment suggested by Melville (1992), as presented in Table 5.3, is used. In the analysis, the scour depth caused by an abutment of other type than a vertical-wall abutment was converted into an equivalent scour depth caused by vertical-wall using the appropriate shape factor, K_s .

Table 5.3. Shape factors proposed by Melville (1992)

Shape of the abutment	Shape factor, K_s
Vertical-wall or vertical-plate	1.00
Vertical-wall with semicircular end	0.75
Wing-wall	0.75
Spill-through, V:H = 1:0.5	0.60
Spill-through, V:H = 1:1.0	0.50
Spill-through, V:H = 1:1.5	0.45

Comparisons between the computed and the measured data of these published data are presented in Figures 5.14 to 5.18. The computed d_{se}/d_f is obtained using Equation (5.15). The published data of Sturm and Janjua (1994), Kouchakzadeh and Townsend (1997) and Cardoso and Bettess (1999) are presented in Table 5.6, 5.7 and 5.8, respectively.

Figure 5.14 shows that most of the plot of the measured data of Sturm and Janjua are over predicted. Only a few data are predicted within $\pm 30\%$ limit. The computed data of $U_f/U_c \geq 0.50$ are shown to be closer to the measured data than those of $U_f/U_c < 0.50$. This is because the development of the scour depth will be faster at the higher flow intensity than at the lower flow intensity. Hence the scour depth at the same time duration of test for the higher intensity flow will be closer to that of equilibrium scour depth than those of lower flow intensity.

The discrepancy between the measured and computed scour depth may be attributed to the short duration of the tests. Based on the experience in the present study, for 10 and 12 hours scouring duration, the average scour depth is 68% and 70% of the

equilibrium scour depth (see Table 5.4), respectively. A plot based on an assumption that the scour depth achieved in between 10 and 12 hours test duration is 69% of the equilibrium scour depth is shown in Figure 5.15. The figure shows a much better agreement compared to the original non-extrapolated data.

Table 5.4. Measured scour depths at 5, 10 and 12 hours, as a percentage of equilibrium scour depth based on present data.

Expts.	% depth		
	5 hrs	10 hrs	12 hrs
20-50-02	n.a.	n.a.	n.a.
20-60-02	95	95	96
20-70-02	94	97	97
20-50-03	50	58	61
20-60-03	49	56	58
20-70-03	71	76	78
20-50-04	n.a.	n.a.	n.a.
20-60-04	47	57	60
20-70-04	44	46	47
20-80-04	68	77	79
20-100-05	41	48	50
35-50-02	49	51	54
35-60-02	87	94	95
35-50-03	51	63	65
35-60-03	54	60	62
35-50-04	47	55	59
35-60-04	52	62	64
50-50-02	65	78	80
50-60-02	75	85	88
50-50-03	67	72	74
50-50-04	45	57	61
average	61	68	70
min	41	46	47
max	95	97	97

In the Kouchakzadeh and Townsend's (1997) study, the U_f/U_c for all the test were close to the threshold condition of clear water scour. Comparison between the extrapolated equilibrium scour depth and the computed equilibrium scour depth for $d_{50} = 0.5$ mm and 0.7 mm are presented in Figures 5.16 and 5.17, respectively. The assumption on the extrapolation that the scour depth at 5 hours duration is about 60 to 65 of the equilibrium is acceptable (see Table 5.4). The effect of the abutment shape has been included in the computed equilibrium scour depth. Figure 5.17 shows a better predicted result compared to Figure 5.16. The discrepancy between

the computed and extrapolated equilibrium scour depth, especially in Figure 5.16, may be due to the possibility of unnoticeable sediment transport into the scour hole. This argument is based on the approach velocities which are nearly the same and some of them are slightly higher than the critical value of sediment motion (see Table 5.6). Another possible reason is due to the size of the sediment. The sediment with $d_{50} = 0.5$ mm can be grouped into the ripple forming sediment (Chien and Wan, 1999). Hence if there is ripple formation, the transported sand may have reached the scour hole and reduce the scour depth.

The plot of measured and computed data from Cardoso and Bettess (1999) is shown in Figure 5.18. The data used here are those with abutment lengths of 0.147, 0.270, 0.400 and 0.530 m, where the scour hole does not encroached into the main channel (experiments number 1 to 8, in Table 5.7). Cardoso and Bettess' (1999) data showed that the U_f/U_c values in their experiments are close to the critical value of sediment motion (see Table 5.7). However, the computed values of U_c based on Equation (5.16) show a lower value of U_f/U_c . Comparison between computed and measured scour depth are shown in Figure 5.18. The open circle data points are based on U_f/U_c provided by Cardoso and Bettess. The computed equilibrium scour depths (black circles in Figure 5.18) using U_c obtained from Equation (5.16) show reasonably good agreement to the measured ones.

The above comparisons show that Equation (5.15) gives a good agreement, within a band of $\pm 30\%$, to the measured equilibrium scour depth data available in the published literatures. Compared to the available methods in computing the scour around abutment terminating in two-stage channel, the parameter which is used to account for the two-stage channel geometry effect in the proposed equation is clearly defined and is easily obtained from the bulk property of the flow. Hence, in terms of reliability and applicability, the proposed equation can be used to predict the equilibrium scour depth around abutment terminating in a floodplain of two-stage channels within the range of parameters used in its development and verification (see Table 5.5).

Table 5.5. Range of important parameters of the data used in the development and verification of Equation (5.15).

	Value	
	min	max
d_{50} (cm)	0.05	0.30
d_f/d_{50}	16.17	166.67
d_f/d_m	0.19	0.61
L/b_f	0.18	0.66
L/d_f	0.56	29.60
u_f/u_c	0.39	1.19

b_f = floodplain width.

Figure 5.14. Measured and computed equilibrium scour depth of Sturm and Janjua's (1994) data.

Figure 5.15. Comparison between extrapolated equilibrium scour depth of Sturm and Janjua's (1994) data with computed equilibrium scour depth.

Figure 5.16. Measured and computed equilibrium scour depth of Kouchakzadeh and Townsend's (1997) data, for $d_{50} = 0.5$ mm. (VW=vertical wall, WW=wing-wall, SC=semi-circular, ST= spill-through)

Figure 5.17. Measured and computed equilibrium scour depth of Kouchakzadeh and Townsend's (1997) data, for $d_{50} = 0.7$ mm.

Figure 5.18. Measured and computed equilibrium scour depth of Cardoso and Bettess' (1999) data.

Table 5. 6. Data of Sturm and Janjua (1994)

Expts.	Channel slope	d _f cm	d _m cm	d _f /d _m	U _f cm/s	L cm	d ₅₀ cm	t _e hr	d _{se} cm	Shape	d _{se} /d _f	U _c cm/s	U _f /U _c
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	0.00	4.9	25.7	0.19	29.9	58.4	0.3	n.a.	8.7	VW	1.77	52.6	0.57
2	0.00	5.0	25.8	0.19	33.8	58.4	0.3	n.a.	15.6	VW	3.13	52.7	0.64
3	0.00	5.2	25.9	0.20	43.0	58.4	0.3	n.a.	18.8	VW	3.66	53.1	0.81
4	0.00	4.9	25.5	0.19	24.7	58.4	0.3	n.a.	6.3	VW	1.31	52.4	0.47
5	0.00	7.4	28.2	0.26	25.9	58.4	0.3	n.a.	7.9	VW	1.07	57.3	0.45
6	0.00	7.3	28.0	0.26	32.0	58.4	0.3	n.a.	17.9	VW	2.46	57.0	0.56
7	0.00	7.0	27.7	0.25	29.9	58.4	0.3	n.a.	15.6	VW	2.23	56.7	0.53
8	0.00	7.7	28.5	0.27	24.4	58.4	0.3	n.a.	10.9	VW	1.42	57.7	0.42
9	0.00	10.6	31.3	0.34	29.0	58.4	0.3	n.a.	13.0	VW	1.23	61.4	0.47
10	0.00	10.6	31.3	0.34	28.7	58.4	0.3	n.a.	10.4	VW	0.99	61.4	0.47
11	0.00	10.6	31.3	0.34	25.6	58.4	0.3	n.a.	10.1	VW	0.95	61.5	0.42
12	0.00	10.3	30.5	0.34	30.2	58.4	0.3	n.a.	15.4	VW	1.50	61.1	0.49
13	0.00	5.1	25.8	0.20	29.6	116.8	0.3	n.a.	16.2	VW	3.18	52.9	0.56
14	0.00	5.5	26.3	0.21	24.1	116.8	0.3	n.a.	9.9	VW	1.79	53.9	0.45
15	0.00	5.4	26.0	0.21	32.0	116.8	0.3	n.a.	19.5	VW	3.63	53.5	0.60
16	0.00	5.4	26.1	0.21	28.7	116.8	0.3	n.a.	15.8	VW	2.92	53.6	0.54
17	0.00	7.5	28.2	0.27	22.6	116.8	0.3	n.a.	11.6	VW	1.55	57.4	0.39
18	0.00	7.3	28.0	0.26	29.6	116.8	0.3	n.a.	20.4	VW	2.82	57.0	0.52
19	0.00	7.0	27.7	0.25	30.8	116.8	0.3	n.a.	19.9	VW	2.82	56.7	0.54
20	0.00	7.5	28.2	0.27	26.2	116.8	0.3	n.a.	13.5	VW	1.80	57.4	0.46

Table 5.6. Data of Sturm and Janjua (1994), continued.

Expts.	Channel slope	d_f <i>cm</i>	d_m <i>cm</i>	d_f/d_m	U_f <i>cm/s</i>	L <i>cm</i>	d_{50} <i>cm</i>	t_e <i>hr</i>	d_{se} <i>cm</i>	Shape	d_{se}/d_f	U_c <i>cm/s</i>	U_f/U_c
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
21	0.00	10.5	31.2	0.33	27.7	116.8	0.3	n.a.	19.4	VW	1.86	61.3	0.45
22	0.00	10.0	30.7	0.33	24.1	116.8	0.3	n.a.	12.3	VW	1.23	60.8	0.40
23	0.00	5.4	26.2	0.21	35.7	116.8	0.3	n.a.	20.1	VW	3.70	53.7	0.67
24	0.00	5.3	26.0	0.20	26.8	154.9	0.3	n.a.	13.7	VW	2.59	53.4	0.50
25	0.00	5.2	26.0	0.20	25.3	154.9	0.3	n.a.	10.5	VW	2.00	53.3	0.47
26	0.00	5.3	26.0	0.20	34.4	154.9	0.3	n.a.	20.5	VW	3.88	53.4	0.64
27	0.00	7.0	27.7	0.25	25.0	154.9	0.3	n.a.	11.4	VW	1.63	56.7	0.44
28	0.00	7.6	28.3	0.27	23.5	154.9	0.3	n.a.	10.6	VW	1.39	57.6	0.41
29	0.00	7.6	28.3	0.27	24.1	154.9	0.3	n.a.	12.4	VW	1.62	57.6	0.42
30	0.00	7.6	28.3	0.27	27.1	154.9	0.3	n.a.	16.0	VW	2.10	57.6	0.47
31	0.00	7.6	28.3	0.27	32.0	154.9	0.3	n.a.	20.4	VW	2.68	57.6	0.56
32	0.00	7.6	28.3	0.27	30.2	116.8	0.3	n.a.	19.5	VW	2.56	57.6	0.52
33	0.00	7.6	28.3	0.27	27.1	116.8	0.3	n.a.	15.1	VW	1.99	57.6	0.47
34	0.00	7.6	28.3	0.27	25.0	116.8	0.3	n.a.	9.6	VW	1.26	57.6	0.43
35	0.00	6.8	27.5	0.25	24.7	116.8	0.3	n.a.	11.4	VW	1.68	56.3	0.44

Note:

VW = vertical wall abutment.

The U_c values in column (13) and used in column (14) are obtained from Equation (5.16).

Table 5. 7. Data of Kouchakzadeh and Townsend (1997).

Expts.	Channel slope	d _f cm	d _m cm	d _f /d _m	U _f cm/s	L cm	d ₅₀ cm	t _e hr	d _{se} cm	Shape	d _{se} /d _f	U _c cm/s	U _f /U _c
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	0.00060	4.2	14.6	0.29	28.5	21.0	0.05	n.a.	10.9	VW	2.595	23.9	1.19
2	0.00075	3.3	13.7	0.24	24.5	15.0	0.05	n.a.	8.0	VW	2.424	23.0	1.07
3	0.00090	2.9	13.3	0.22	24.2	9.0	0.05	n.a.	5.4	VW	1.862	22.5	1.08
4	0.00060	4.2	14.6	0.29	28.5	21.0	0.05	n.a.	8.4	WW	2.000	23.9	1.19
5	0.00075	3.3	13.7	0.24	24.5	15.0	0.05	n.a.	6.6	WW	1.985	23.0	1.07
6	0.00090	2.9	13.3	0.22	24.2	9.0	0.05	n.a.	5.2	WW	1.793	22.5	1.08
7	0.00060	4.2	14.6	0.29	28.5	21.0	0.05	n.a.	7.9	SC	1.881	23.9	1.19
8	0.00075	3.3	13.7	0.24	24.5	15.0	0.05	n.a.	6.3	SC	1.909	23.0	1.07
9	0.00090	2.9	13.3	0.22	24.2	9.0	0.05	n.a.	5.0	SC	1.724	22.5	1.08
10	0.00060	4.2	14.6	0.29	28.5	21.0	0.05	n.a.	7.9	ST	1.881	23.9	1.19
11	0.00075	3.3	13.7	0.24	24.5	15.0	0.05	n.a.	6.3	ST	1.909	23.0	1.07
12	0.00090	2.9	13.3	0.22	24.2	9.0	0.05	n.a.	4.2	ST	1.448	22.5	1.08

Note:

VW = vertical wall abutment

WW = wingwall abutment

SC = semicircular abutment

ST = spillthrough abutment

The U_c values in column (13) and used in column (14) are obtained from Equation (5.16).

Table 5.7. Data of Kouchakzadeh and Townsend (1997), continued.

Expts.	Channel <i>slope</i>	d_f <i>Cm</i>	d_m <i>cm</i>	d_f/d_m	U_f <i>cm/s</i>	L <i>cm</i>	d_{50} <i>cm</i>	t_e <i>hr</i>	d_{se} <i>cm</i>	Shape	d_{se}/d_f	U_c <i>cm/s</i>	U_f/U_c
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
13	0.00090	3.8	14.2	0.27	29.5	21.0	0.07	n.a.	11.7	VW	3.079	27.6	1.07
14	0.00090	3.8	14.2	0.27	28.1	15.0	0.07	n.a.	9.9	VW	2.605	27.6	1.02
15	0.00090	3.8	14.2	0.27	26.2	9.0	0.07	n.a.	7.7	VW	2.013	27.6	0.95
16	0.00090	3.8	14.2	0.27	29.5	21.0	0.07	n.a.	10.0	WW	2.632	27.6	1.07
17	0.00090	3.8	14.2	0.27	28.1	15.0	0.07	n.a.	8.1	WW	2.132	27.6	1.02
18	0.00090	3.8	14.2	0.27	26.2	9.0	0.07	n.a.	6.8	WW	1.789	27.6	0.95
19	0.00090	3.8	14.2	0.27	29.5	21.0	0.07	n.a.	9.6	SC	2.526	27.6	1.07
20	0.00090	3.8	14.2	0.27	28.1	15.0	0.07	n.a.	8.3	SC	2.184	27.6	1.02
21	0.00090	3.8	14.2	0.27	26.2	9.0	0.07	n.a.	6.5	SC	1.711	27.6	0.95
22	0.00090	3.8	14.2	0.27	29.5	21.0	0.07	n.a.	9.9	ST	2.605	27.6	1.07
23	0.00090	3.8	14.2	0.27	28.1	15.0	0.07	n.a.	8.2	ST	2.158	27.6	1.02
24	0.00090	3.8	14.2	0.27	26.2	9.0	0.07	n.a.	6.0	ST	1.579	27.6	0.95

Note:

VW = vertical wall abutment

WW = wingwall abutment

SC = semicircular abutment

ST = spillthrough abutment

The U_c values in column (13) and used in column (14) are obtained from Equation (5.16).

Table 5.8. Data of Cardoso and Bettess (1999)

Expts.	Channel <i>slope</i>	d_f <i>cm</i>	d_m <i>cm</i>	d_f/d_m	U_f <i>cm/s</i>	L <i>cm</i>	d_{50} <i>cm</i>	t_e <i>hr</i>	d_{se} <i>cm</i>	Shape	d_{se}/d_f	U_c <i>cm/s</i>	U_f/U_c	U_f/U_c^*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
1	n.a.	3.1	11.1	0.28	23.9	14.70	0.0835	37.0	12.2	VP	3.94	25.2	0.95	0.82
2	n.a.	7.3	15.3	0.48	26.2	14.70	0.0835	120.0	22.0	VP	3.01	28.2	0.93	0.77
3	n.a.	2.8	10.8	0.26	23.4	27.00	0.0835	101.0	12.7	VP	4.54	24.8	0.94	0.82
4	n.a.	7.8	15.8	0.49	26.1	27.00	0.0835	72.0	24.2	VP	3.10	28.4	0.92	0.76
5	n.a.	2.8	10.8	0.26	23.7	40.00	0.0835	27.0	12.3	VP	4.39	24.8	0.96	0.83
6	n.a.	7.9	15.9	0.50	26.2	40.00	0.0835	45.0	24.6	VP	3.11	28.5	0.92	0.76
7	n.a.	3.7	11.7	0.32	23.5	53.00	0.0835	32.0	20.1	VP	5.43	25.9	0.91	0.78
8	n.a.	8.8	16.8	0.52	29.0	53.00	0.0835	25.0	30.1	VP	3.42	28.9	1.00	0.83
9	n.a.	3.3	11.3	0.29	25.8	66.50	0.0835	7.0	21.9	VP	6.64	25.3	1.02	0.87
10	n.a.	6.0	14.0	0.43	29.2	66.50	0.0835	12.0	25.1	VP	4.18	27.4	1.07	0.89
11	n.a.	8.4	16.4	0.51	28.2	66.50	0.0835	21.0	33.0	VP	3.93	28.7	0.98	0.81
12	n.a.	3.4	11.4	0.30	26.2	80.00	0.0835	7.0	22.0	VP	6.47	25.4	1.03	0.88
13	n.a.	5.5	13.5	0.41	29.3	80.00	0.0835	8.0	27.0	VP	4.91	27.1	1.08	0.90
14	n.a.	8.4	16.4	0.51	26.7	80.00	0.0835	6.0	31.1	VP	3.70	28.7	0.93	0.77

Note:

VP : vertical plate abutment

Column (13) : U_c provided in Cardoso and Bettess (1999)Column (15) : U_f/U_c^* , computed using u_c obtained from Equation (5.16).

5.4 Time Development of Scour Hole

Flow in a natural channel is unsteady due to the continuous change of the inflow hydrograph. In this situation, the scour depth around a structure placed in the channel may not always attain the equilibrium especially if the flow does not last as long as the time to reach the equilibrium state. In this respect, the information on the time development towards the equilibrium scour depth is needed to estimate the scour depth for a time shorter than equilibrium time, besides the information on equilibrium scour depth. An estimation of scour depth that consider the time duration of flow will give more advantages, e.g. it could give a design which takes into account the time duration effect and could reduce the construction cost in case the estimated scour depth related with the flow hydrograph is less than the equilibrium scour depth.

In general, scour depth development for any structure placed in a channel has a typical curve of development of the scour depth with time. In the beginning of the scour, the scouring rate is very high which then decreases as the scour hole gets deeper and finally reaches its equilibrium scour depth. In this study, the equilibrium time, t_e , is defined as the time when the increase of scour depth is less than or equal to a minimum reference length. The reference length was defined as the smallest unit length difference that can be observed from the meter scale attached to the “periscope” (see Figure 3.17), i.e. 1 mm for the interval of 6 to 12 hours. In some tests, fluctuation of scour depth was observed when the equilibrium condition was about to be achieved. In this case, the equilibrium time is defined as the time when the maximum scour depth was recorded at the first time in the scour-time curve. Hence, examination on the record of scour-time development is important to determine the equilibrium time.

As an example, Figure 5.19 shows the scour-time development of three different abutment lengths, i.e. Runs 20-50-03, 35-50-03 and 50-50-03. The equilibrium time, t_e , for each run is obtained from the examination of the respective scour development graph. The graph of dimensionless scour depth versus dimensionless time can be produced based on the d_{se} and t_e obtained from the observation. Figure

5.20 shows the dimensionless scour development using the data shown in Figure 5.19.

Figure 5.19. Time development of scour for three different abutment lengths under the same flow condition.

Figure 5.20. Dimensionless scour-time development of the data used in Figure 5.19.

For the present data, the plot of temporal scour development for different discharges, Q , and flow depth ratios, d_f/d_m , are presented in Figure 5.21. Only experiments with $Q = 50$ l/s and 60 l/s are presented, so there are at least two data plots of different abutment lengths for each Q . Most of the temporal scour development data is available except for Runs 20-50-04 and 20-50-02, where the scour depth changes were relatively fast towards its equilibrium state. At the beginning of the scouring process the scour depth increases very fast and it could reach 50% of the equilibrium scour depth, d_{se} , in only 5% of the time needed to reach the equilibrium state, t_e . In general, about 80% of the equilibrium scour depth is reached when the time, t , is about 20% of t_e . As mentioned earlier in this section, some fluctuations on the scour depth were observed in some tests. The fluctuations in depth were caused by the difference between sediment transport capacity at the base of the scour hole and the amount of the sediments which slide into the base from the side slope of the scour hole. As the scour depth increases, the side slope also increases and more sediments will slide into the base of the scour hole compared to a case of a relatively shallow scour hole. Example of this case can be seen in Figure 5.21 for Run 35-60-04 (second graph in the third row, data plot for $L = 35$ cm).

The plots of dimensionless scour development with different flow depth ratios, d_f/d_m , are presented in Figures 5.21 and 5.22. The discharge and abutment length are kept constant for each graph. The graphs show that all the data points form a unique curve shape, except for those experiments where the equilibrium was reached at relatively short duration. For these experiments, the scour could reach 90% of the equilibrium scour depth in 10% of the time to reach equilibrium condition, t_e . This case occurred for low flow depth, especially for $L = 20$ cm. This is probably due to the relatively low flow depth, low velocity and relatively short abutment which form a relatively weak vortex system. Hence, the scouring process in the beginning was very fast, but the time to reach equilibrium state was relatively long due to the weak vortex strength.

Figure 5.21. Temporal development of scour depth of different abutment lengths for a certain Q and flow depth ratio.

Figure 5.22. Temporal development of scour depth of different flow depth ratio for a certain Q and abutment length.

5.5 Equilibrium Scouring Time, t_e

Based on physical reasonings, the time development of a local scour hole towards its equilibrium state will depend on the characteristics of the flow, sediment and geometry of the abutment. Generally, the relationship of the scour equilibrium time as a function of the flow, sediment and abutment variables for a two-stage channel can be written as:

$$t_e = f_1(U_f, U_c, \mu, \rho_w, g, d_f, d_{50}, \rho_s, \sigma_g, L, K_s, \theta) \quad (5.20)$$

where t_e = time to reach equilibrium state, U_f = approach mean flow velocity, U_c = critical velocity for sediment motion, μ = viscosity of water, ρ_w = density of water, g = gravitational acceleration, d_f = flow depth in floodplain, d_{50} = median size of sediment, ρ_s = density of sediment, σ_g = sediment uniformity, L = abutment length, K_s = shape factor, and θ = abutment angle to the flow direction.

In Equation (5.20), the parameter U_c is included so that it can be used to define the state of scour process, i.e. whether clear scour or live bed scour. The critical velocity for sediment motion, U_c , is also a function of flow depth, channel bed roughness and sediment properties (Lauchlan and Melville, 2001). For a vertical wall abutment placed perpendicular to the flow direction, the function can be written as:

$$t_e = f_2(U_f, U_c, \nu, \rho_w, g, d_f, d_{50}, \rho_s, \sigma_g, L) \quad (5.21)$$

Assuming for a case of uniform sediment gradation of a sediment size and a condition where the viscous effect can be neglected, the function can be represented as:

$$t_e = f_3(U_f, U_c, d_f, d_{50}, L) \quad (5.22)$$

Equation (5.22) shows that the time to reach equilibrium is a function of almost the same parameters of equilibrium scour depth (see Equation (5.15)). Hence, any form of expression involving t_{se} and d_{se} can be arranged as a function of the parameters in

the right hand side of Equation (5.22). An expression, d_{se}/t_e , which reflects the average scouring rate is proposed as a function of the parameters on the right hand side of Equation (5.22).

In the recent time, equilibrium scour depth data are available and adequate to confirm empirical or semi-empirical model to estimate the equilibrium scour depth. Hence, it is possible to find such an average scouring rate d_{se}/t_e for certain sediment, flow and abutment characteristics. However, it should be noted that there are many suggestion in the determination of the time when an equilibrium scour depth has been reached. A relationship of the significant dimensionless variables in the process is proposed as:

$$\frac{d_{se}}{t_e} \frac{1}{U_f} = f_4 \left(\frac{U_f}{U_c}, \frac{L}{d_f}, \frac{d_{50}}{d_f} \right) \quad (5.23)$$

This means that the average scouring rate depends on the flow intensity, ratio of abutment length to the flow depth and relative sediment size to the flow depth.

A multivariate monomial function is assumed to be valid for this relationship and it may be written as:

$$\frac{d_{se}}{t_e} \frac{1}{U_f} = C_1 \left(\frac{U_f}{U_c} \right)^{C_2} \left(\frac{L}{d_f} \right)^{C_3} \left(\frac{d_{50}}{d_f} \right)^{C_4} \quad (5.24)$$

A least square multiple regression is then used to find the coefficient and the exponents of Equation (5.24) using the present data and available published data of equilibrium scour depth and its time development. The data of Rajaratnam and Nwachukwu (1983b), Lim (1997), Cardoso and Bettess (1999) and the present data were used to find the empirical relationship in the form of Equation (5.24). These data were used as they provided the equilibrium scour depth and its time development. The equilibrium time data was obtained based on the definition of t_e used this study (see Section 5.4) and also from the examination of the scour-time development curve.

By the examination on the scour time development curve, only 16 data of the present study were used. The data of Runs 20-50-02, 20-50-04, 20-60-02, 20-70-02 and 35-60-02 were not used in the analysis because of the difficulties in the determination of the time of an equilibrium state from the examination of time-scour development. This is due to the fluctuation of the scour depth caused by the secondary scour (see Section 5.2) and the unrecorded scour depth for a relatively long interval (the equilibrium condition had been attained, but the time when it was attained can not be identified).

For the data of rectangular channel used in the regression, the approach mean velocity and approach flow depth were used for U_f and d_f , respectively. The coefficient and exponents obtained from the regression of the available data, including from the present study, are found as: $C_1 = 0.023 \times 10^{-6}$, $C_2 = 2.163$, $C_3 = 0.864$, and $C_4 = -0.949$. Comparison between the measured and computed values is shown in Figure 5.23. The square of correlation coefficient (R^2) between the computed and measured value of $d_{se}/(t_e \cdot U_f)$ is 0.80. This shows that the statistical performance of the proposed equation is good, and in general, Figure 5.23 also shows that most of the computed values of $d_{se}/(t_e \cdot U_f)$ follow the measured data and supports the proposed relationship in the form of Equation (5.24). Table 5.9 shows the range of data used in the development of Equation (5.24).

Table 5.9. Range of important parameters of the data used in the development of Equation (5.24).

	Value	
	min	max
d_{50} (cm)	0.084	0.14
d_f/d_{50}	33.53	166.67
L/d_f	0.33	14.29
U_f/U_c	0.41	0.83
t_e (hrs)	27.5	561

Figure 5.23. Comparisons of measured and computed $d_{se}/(t_e.U_f)$ using Equation (5.24).

The equilibrium time, t_e , can be estimated using the computed value of d_{se}/t_e from Equation (5.24) and d_{se} from Equation (5.15). Figure 5.24 shows a result obtained from calculation of the t_e based on the measured d_{se} and Equation (5.24). The majority of the computed t_e are in the $\pm 30\%$ bands, this shows that Equation (5.24) provides a good relationship among the parameters within the range of the data used in its development. However, some of the scattered points are observed to be out of the $\pm 30\%$ bands, and some of the present data show significant difference compared to the measured equilibrium time, especially for t_e more than 300 hours.

It should be noted that the relatively slow scouring rate near the equilibrium condition causes a small difference in the determination of the equilibrium scour depth will result in a significant difference in term of time.

Figure 5.24. Comparisons of measured and computed t_e using average scouring rate obtained from Equation (5.24).

The scour depth at certain time under the steady approach flow velocity could be obtained from the relation between d_s/d_{se} and t/t_e . Based on the typical shape of the curve, the nature of relationship of d_s/d_{se} and t/t_e can be expressed as:

$$\frac{d_s}{d_{se}} = \exp \left[a \left| \ln \left(\frac{t}{t_e} \right) \right|^b \right] \quad (5.25)$$

Coleman, Lauchlan and Melville (2003) proposed an empirical relationship based on the two non-dimensional parameters in Equation (5.25) and their equation also involves an additional flow intensity factor, U/U_c , as follows

$$\frac{d_s}{d_{se}} = \exp \left[-0.07 \left(\frac{U}{U_c} \right)^{-1} \left| \ln \left(\frac{t}{t_e} \right) \right|^{1.5} \right] \quad (5.26)$$

This equation is general and may be used for the present experiments to compute the d_s versus t relationship, using d_{se}/t_e from Equation (5.24) and d_{se} from Equation (5.15).

Comparisons of the dimensionless scour-time development obtained from Equation (5.26) by using U_f and the present experimental results are presented in Figures 5.25 to 5.28. Figure 5.29 shows the comparisons between Cardoso and Bettess' (1999) data and the one obtained of Equation (5.26). In general the computed scour-time development curves match well with the experimental results and the data of Cardoso and Bettess' (1999). Again, noticeable differences are observed for those experiment conducted with small flow depth ratio (Runs 20-60-02 and 20-70-02), where the equilibrium scour depth was attained in a relatively short time. For Run 20-100-05, the difference was probably due to the large flow depth ratio ($d_f/d_m = 0.5$), and the size of the scour hole which, at equilibrium condition, occupied about 80% of the floodplain width and the scour depth of 34.1 cm.

Figure 5.25. Measured and computed dimensionless scour depth development in experiment with abutment length = 20 cm.

Figure 5.26. Measured and computed dimensionless scour depth development in experiments with abutment length = 20 cm. (continued)

Figure 5.27. Measured and computed dimensionless scour depth development in experiments with abutment length = 35 cm.

Figure 5.28. Measured and computed dimensionless scour depth development in experiments with abutment length = 50 cm.

Figure 5.29. Measured and computed dimensionless scour depth development for Cardoso and Bettess' (1999) data.

5.7 Tests with Separation of Floodplain and Main Channel Flow

In order to study the effect of the absence of interaction between the main channel and the floodplain flow on the flow distribution and the scouring process, a vertical metal sheet was placed along the junction of the floodplain and the main channel, for Runs 20-70-04A and 20-70-04B. The metal sheet was placed to omit interactions between the flow in the floodplain and that in the main channel. The length of the separated section of Runs 20-70-04A and 20-70-04B was 7.2 m and 2.4 m, respectively. The downstream ends of both separators were at 2 m downstream of the abutment site, hence the upstream ends were at 5.2 m and 2.4 m upstream of the abutment site, respectively. The separators provided a condition of non interaction between the floodplain and the main channel flow along the isolated reach.

The mean velocity profile in the floodplain of the isolated and non-isolated floodplain is presented in Figure 5.32. In this figure, the junction of the floodplain and the main channel is at $y = 60$ cm, and the floodplain sidewall is at $y = 160$ cm. For Runs 20-70-04 and 20-70-04A, the flow measurement was done at a cross section 2 m upstream of the abutment, while for Run 20-70-04B, it was done at a cross section 0.38 m upstream of the abutment (see Figure 5.30). Figure 5.32 shows that the effect of the length of separator is insignificant to the velocity distribution in the blocked portion of the floodplain. This shows that the separator successfully cut off the interaction between the floodplain and the main channel in the separated reach, and the result is that the velocity in the floodplain is lower than that in the non-separated case.

Figure 5.33 shows the velocity distribution in the main channel at 2 m upstream of the abutment site for Runs 20-70-04A and 20-70-04 (see Figure 5.31 for location of measurement). The figure shows that the mean velocity in the interacting condition is lower compared to that in the non-interacting condition, because of the absence of momentum exchange between floodplain and main channel flow.

Figure 5.34 shows the measured scour depth development for the three tests. The equilibrium scour depth in the separated condition is about the same and is

generally shallower than the non-separated case. This is because the presence of the separator at the abutment site prevented the exchange of momentum between the main channel and floodplain flow and the lower flow velocity in turn affected the scouring process. Figure 5.34 also shows that the time to reach the equilibrium scour condition is shorter for the separated condition. The pictures of the scour hole formed in Runs 20-70-04A, 20-70-04B and 20-70-04 are presented in Figures 5.35 to 5.37, respectively.

Figure 5.30. Location of measurement on the floodplain.

Figure 5.31. Location of measurement in the main channel.

Figure 5.32. Depth-averaged velocity across the floodplain of Runs 20-70-04A (with divider), 20-70-04B (with divider) and 20-70-04 (without divider).

Figure 5.33. Depth-averaged velocity across the main channel of Runs 20-70-04A (with divider) and 20-70-04 (without divider).

Figure 5.34. Scour depth development of Runs 20-70-04A, 20-70-04B and 20-70-04.

Figure 5.35. Equilibrium scour hole formed in Run 20-70-04A after 222 hours.

Figure 5.36. Equilibrium scour hole formed in Run 20-70-04B after 264 hours.

Figure 5.37. Equilibrium scour hole formed in Run 20-70-04 after 522 hours.

5.8 Summary

The scouring process around abutment terminating on a floodplain was observed in this chapter. The scouring process always starts at the upstream corner of the abutment tip. The final shape of the scour hole generally resembled an inverted cone with the deepest point at the upstream corner of the abutment tip. Three types of scour hole geometry were observed from the experiments, i.e. the inverted cone type, edge-truncated inverted cone type and the extended upstream edge-truncated

inverted cone type. If the flow depth ratio is low, a shallow secondary scour will be formed and it will propagate upstream near to the side wall after the equilibrium scour depth is attained at the abutment. This scouring process is due to the change of flow distribution of the approach flow after the scour depth had reached its equilibrium at the abutment.

The flow interaction between the floodplain and main channel is related to the interface between them. A relationship linking the flow depth ratio to the approach flow on the floodplain is proposed. The corrected floodplain approach flow velocity is used in the formulation of a semi-empirical equation for the equilibrium scour depth. The proposed equilibrium scour depth model involves bulk parameters related to the flow, sediment and abutment properties.

Based on physical reasonings and dimensional analysis, it is assumed that the dimensionless average scouring rate, $d_{se}/(t_e U_f)$, is a function of the flow, sediment properties and abutment length. An empirical relationship for $d_{se}/(t_e U_f)$ is proposed as a function of these parameters. Using the proposed equilibrium scour depth and average scouring rate formulas, the time required to reach equilibrium scour depth, t_e , can be estimated. Once t_e is obtained, the scour depth at any time t , for $0 \leq t \leq t_e$, can be estimated using a natural relationship between d_s/d_{se} and t/t_e , e.g. the one proposed by Coleman, Lauchlan and Melville (2003).

At the downstream end of the abutment, beside the streamwise sediment transport, the strong up-sloping flow transported the scoured sediment to the left and right side of the scour hole. The scoured sediments were deposited further downstream and also along the right and left edge of the scour hole. The deposition along the sidewall at the area behind the abutment was higher compared to the opposite side. The sediment transport conditions at this part were observed to be related to the Reynolds shear stresses distributions.

Two special tests were conducted to study the effect of isolating the flow interaction between the floodplain and main channel. The results show that the flow velocity in the floodplain was lower in the non-interacting case compared to that of

interacting condition. This is due to the absence of the momentum exchange between the floodplain and main channel. The equilibrium scour depths in the non-interacting condition were observed to be lower and the time to reach equilibrium condition was shorter than that of the interacting condition. This test shows the importance of the two-stage channel geometry on the flow distribution and its associated scouring process at the abutment site.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This thesis presents a study of the flow distribution and scouring around an abutment placed in the floodplain of a two-stage channel. To achieve the objectives, the study was divided into two parts: study on the flow distributions, and of scouring process around a vertical-wall abutment terminating in the floodplain of a two-stage channel. Experimental works were done for both parts to provide sufficient data for analysis. There are three important results of the study, they are: flow distribution, a formulation of equilibrium scour depth and an empirical relationship for average scouring rate. The novel of the proposed equilibrium scour depth formula is on the involvement of bulk parameters related to the flow in the floodplain and the main channel. The conclusions drawn from the study are given in the ensuing sections.

6.2 Flowfield Observations

Observations of the flow around an abutment sited on the floodplain of a two-stage channel showed that the relatively short abutment affects significantly the velocity vectors for flow region close to the abutment. Near the abutment tip, where the deepest scour depth usually occurred, the velocities at the lower depth in flat bed condition were higher than those of the upper part due to the effect of the flow diversion and the associated downflow. The largest magnitudes of the spanwise and vertical velocities close to the abutment tip were around 0.82 and 0.2 times that of the mean streamwise velocity at the approach section, respectively.

After the scour hole was established, the overall spanwise velocity in the floodplain decreased. The velocity vectors around the abutment showed less deflection compared to that of the flat bed condition. In the scour hole, an anticlockwise vortex, looking downstream, similar to that observed by Kwan and Melville (1994) for a wing-wall abutment placed in a rectangular channel, was formed at the section

adjacent to the abutment. Downstream of the abutment, this vortex was weakened and a clockwise vortex was formed.

For the flat bed condition, the bed shear stress amplification for a point close to the abutment tip was observed. In this study, it was 3.8 times larger than that of the approach section or 1.8 times larger than the critical shear stress of the sediment used.

For the flat bed condition, the turbulent kinetic energy increases at the region close to the abutment and at the downstream of the abutment due to the vortices generated by the flow blockage and diversion. There is no apparent difference in the distributions of turbulent intensities in the middle of the main channel for all the cross sections. For the scoured bed experiments, the magnitude of turbulent intensities and turbulent kinetic energy in the main channel and at the region outside of the scour hole in the floodplain are in the same order compared to that of the flat bed experiments.

For the flat bed and for equilibrium conditions, Reynolds shear stress distributions were observed to be nonlinear due to the lateral shear and secondary flow, especially near the abutment tip, inside the scour hole and close to the junction of the floodplain and the main channel.

6.3 Scour around Vertical-Wall Abutment Placed in a Floodplain

The scouring process always starts at the upstream corner of the abutment tip. Three types of scour hole geometry were observed in this study, i.e. the inverted cone type, edge-truncated inverted cone type and the extended upstream edge-truncated inverted cone type. In low flow depth ratio, a shallow secondary scour was formed and propagated upstream near to the side wall after equilibrium scour depth attained. This is due to the change of flow distribution of the approach flow in the floodplain after the scour depth reached its equilibrium. Most of the scour holes observed in this study resembled an inverted cone with the deepest point at the upstream corner of the abutment tip.

At the downstream of the abutment, besides the streamwise sediment transport, the strong up-sloping flow transported the scoured sediment to the left and right side of the scour hole. The scoured sediments were deposited farther downstream and also along the right and left edge of the scour hole. The deposition along the sidewall at the area behind the abutment was higher compared to the opposite side.

A relationship linking the flow depth ratio to the approach flow on the floodplain was proposed. The corrected approach flow velocity in the floodplain was then used to develop a semi-empirical formula for the equilibrium scour depth. The proposed equilibrium scour depth model involves bulk parameters related to the flow, sediment and abutment properties. The range of the important parameters used in the development and verification of this model is given in Table 5.5.

$$\frac{d_{se}}{d_f} = K_s \left\{ 2.76 \left(\frac{d_f}{d_m} \right)^{0.26} \left(\frac{U_f}{U_c} \right)^{0.75} \left[0.9 \left(\frac{L}{d_f} \right)^{0.5} + 1 \right] - 2 \right\}$$

Based on physical reasonings and dimensional analysis, it was assumed that the dimensionless average scouring rate, $d_{se}/(t_e U_f)$, is a function of the flow, sediment properties and abutment length. An empirical relationship for average scouring rate has been proposed as a function of these non-dimensional parameters. A prediction on the time to reach equilibrium scour depth can be obtained using the proposed equilibrium scour depth and average scour rate formulas.

$$\frac{d_{se}}{t_e} \frac{1}{U_f} = C_1 \left(\frac{U_f}{U_c} \right)^{C_2} \left(\frac{L}{d_f} \right)^{C_3} \left(\frac{d_{50}}{d_f} \right)^{C_4}$$

where: $C_1 = 0.023 \times 10^{-6}$, $C_2 = 2.163$, $C_3 = 0.864$, and $C_4 = -0.949$.

The scour depth at a certain time before the equilibrium scour depth is attained can be estimated using a general scour-time relationship between d_s/d_{se} and t/t_e . The range of the important parameters used in the development and verification of this scouring rate model is given in Table 5.9.

The effect of separating the flow interactions between floodplain and main channel on the scouring at the abutment was investigated. The experiments were conducted using two separators of different lengths placed in the junction of the floodplain and the main channel. The results showed that in the non-interacting condition, the flow

velocity in the floodplain was lower compared to that when the flow is allowed to interact. This is due to the absence of the momentum exchange between the floodplain and main channel flow. The equilibrium scour depths in the non-interacting condition were lower and the time to reach equilibrium condition was shorter than that in the interacting condition. The length of the separator on the flow condition at the upstream reach of the abutment site does not show any significant effect on the flow distribution and scouring process.

6.4 Recommendations

Study on flow and scouring process in an alluvial channel, in general, has been well developed. However, in the case of an abutment placed in two-stage channels, there are still space for further research due to the complexity of the flow interaction and the scouring process.

The local features of flow around abutment terminating in floodplain can be further investigated in terms of different two-stage channel geometry parameters (floodplain width, main channel width, slope of the main channel bank), abutment shape and abutment alignment. Study on the simultaneous effect due to the presence of other structures, e.g. piers, in a same cross section where an abutment is placed may also be considered for future research.

In real condition, the floodplain flow usually occurs in high-flow seasons. Hence, there are two transition periods, i.e. when the flow depth is increasing and receding. The mass and momentum transfers through the interface of floodplain and main channel will be significant in the transition period and may have an implication to the scouring process around abutment terminating in floodplains.

In the effort to combat the abutment scouring problem, knowledge on the flow structure around an abutment and its associated scouring process is important. The above recommendations will be useful as a part of a study on appropriate method of scour protection for abutment in floodplains.

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APPENDICES

Figure 1. Scour depth development of Run 20-60-02.

Figure 2. Equilibrium scour hole of Run 20-60-02 after 46 hours (note: contour line in cm).

Figure 3. Scour depth development of Run 20-70-02.

Figure 4. Equilibrium scour hole of Run 20-70-02 after 45 hours.

Figure 5. Scour depth development of Run 20-50-03.

Figure 6. Equilibrium scour hole of Run 20-50-03 after 120 hours (note: contour line in cm).

Figure 7. Scour depth development of Run 20-60-03.

Figure 8. Equilibrium scour hole of Run 20-60-03 after 503 hours (note: contour line in cm).

Figure 9. Scour depth development of Run 20-70-03.

Figure 10. Equilibrium scour hole of Run 20-70-03 after 71 hours.

Figure 11. Scour depth development of Run 20-60-04.

Figure 12. Equilibrium scour hole of Run 20-60-04 after 192 hours.

Figure 13. Scour depth development of Run 20-70-04.

Figure 14. Equilibrium scour hole of Run 20-70-04 after 546 hours.

Figure 15. Scour depth development of Run 20-80-04.

Figure 16. Scour depth development of Run 20-100-05.

Figure 17. Scour depth development of Run 35-50-02.

Figure 18. Equilibrium scour hole of Run 35-50-02 after 192 hours.

Figure 19. Scour depth development of Run 35-60-02.

Figure 20. Equilibrium scour hole of Run 35-60-02 after 198 hours.

Figure 21. Scour depth development of Run 35-50-03.

Figure 22. Equilibrium scour hole of Run 35-50-03 after 180 hours.

Figure 23. Scour depth development of Run 35-60-03.

Figure 24. Equilibrium scour hole of Run 35-60-03 after 333 hours.

Figure 25. Scour depth development of Run 35-50-04.

Figure 26. Equilibrium scour hole of Run 35-50-04 after 174 hours.

Figure 27. Scour depth development of Run 35-60-04.

Figure 28. Equilibrium scour hole of Run 35-60-04 after 240 hours.

Figure 29. Scour depth development of Run 50-50-02.

Figure 30. Equilibrium scour hole of Run 50-50-02 after 78 hours.

Figure 31. Scour depth development of Run 50-60-02.

Figure 32. Equilibrium scour hole of Run 50-60-02 after 138 hours.

Figure 33. Scour depth development of Run 50-50-03.

Figure 34. Equilibrium scour hole of Run 50-50-03 after 79 hours.

Figure 35. Scour depth development of Run 50-50-04.

Figure 36. Equilibrium scour hole of Run 50-50-04 after 168 hours.

Figure 37. Scour depth development of Run 20-70-04 B.

Figure 38. Equilibrium scour hole of Run 20-70-04 B after 264 hours.

Figure 39. Scour depth development of Run 20-70-04A.

Figure 40. Equilibrium scour hole of Run 20-70-04A after 246 hours.

Table 1. Shear velocities and bed shear stresses at section A, Run 20-70-04, flat bed condition.

y	u*			τ_b		
	a	b	c	a	B	c
(m)	(m/s)			(N/m²)		
1.50	0.015	0.012	0.013	0.225	0.135	0.170
1.40	0.018	0.014	0.014	0.320	0.202	0.194
1.20	0.014	0.010	0.014	0.204	0.100	0.190
1.00	0.016	0.011	0.015	0.243	0.110	0.216
0.80	0.015	0.012	0.014	0.237	0.137	0.187
0.60	0.013	0.007	0.014	0.172	0.044	0.186
0.50	0.019	0.017	0.012	0.353	0.276	0.148
0.30	0.021	0.015	0.013	0.428	0.228	0.174

Table 2. Shear velocities and bed shear stresses at section D, Run 20-70-04, flat bed condition.

y	u*			T		
	a	b	c	a	B	c
(m)	(m/s)			(N/m²)		
1.400		0.004			0.016	
1.350	0.031	0.017		0.967	0.296	
1.200	0.016	0.014	0.014	0.243	0.185	0.198
1.000	0.016	0.016	0.015	0.259	0.256	0.218
0.800	0.018	0.015	0.014	0.335	0.228	0.199
0.600	0.024	0.017	0.015	0.581	0.286	0.238
0.500	0.019	0.016	0.013	0.372	0.240	0.175
0.300	0.019	0.010	0.015	0.346	0.104	0.221

Note, for Tables 1 and 2:

a : obtained from velocity profile measured using 3D-ADV

b : obtained from velocity profile measured using 8-mm propeller current meter

c : obtained from Reynolds stress profile measured using 3D-ADV