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Combined abutment and contraction scour in compound channels for extreme flood events

by

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A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy

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August, 2016
Abstract

In this PhD project, the combination of abutment and contraction scour is investigated to better understand the scour mechanisms and scour patterns for extreme floods. Close-to-reality scour events were physically simulated using models built at 1:45 and 1:30 geometric scales of two-lane bridge prototypes. Scour and flow-measurement experiments under submerged orifice and overtopping flows were carried out. To better understand the effect of vertical contraction on abutment scour, free surface flows were also investigated for similar experimental conditions. The majority of the experiments were carried out in compound channels, simulating abutments set back from the main channel. Spill-through abutments were used in the live-bed scour regime, and both spill-through and wing-wall abutments were used in the clear-water regime. In addition, to better understand the effect of contraction length on abutment scour, and also to verify the long contraction theory for apron-protected, abrupt abutments, a series of long contraction experiments were carried out with vertical-wall abutments.

Several major conclusions can be drawn from the results of this project. For the investigated conditions, results show that vertical contraction significantly affects the flow pattern, the temporal development of scour and the final scour bathymetry. Comparing with submerged orifice flows, flow relief of the overtopping flows has a small effect on the near-bottom turbulence and the scour. Flow patterns at the initial state are found to correlate with scour patterns at the equilibrium state. In the bridge section, a “retreating” behaviour of the main channel bank is observed; at the equilibrium state, the side slope of the “retreated” main channel bank is observed to be invariant, presenting a simple geometric relationship between the depth of the scour hole and its location. For unprotected abutments, scour is centred at the
upstream corner of the abutment, regardless of contraction length; and for apron-protected abutments, scour differs significantly with contraction length.

Numerical and physical modelling work is required in the future to broaden the knowledge of abutment and contraction scour. Also, further research is required to improve the scour countermeasure design for abutments under pressure flows.
Acknowledgement

This PhD project was accomplished with help from many people. I am grateful to my main supervisor, Professor Bruce W. Melville, for his professional guidance, academic support, friendly encouragement and most importantly, for allowing me to explore freely. I would like to express my appreciation to my co-supervisor, Dr. Heide Friedrich, for her invaluable advice, support and inspiration. Also, I would like to express my appreciation to Dr. Keith Adams, for his valuable advice in English writing, and his useful assistance in proof reading of this thesis.

I gratefully acknowledge the financial support from China Scholarship Council. Also, the financial support provided by the National Cooperative Highway Research Program, Transportation Research Board, Project NCHRP 24-37 is gratefully acknowledged.

In addition, I wish to extend my sincere gratitude to the following people:

- Yingjie Luo, Geoff Kirby and Trevor Patrick for their technical support.
- Professor Terry Sturm, Professor Roger Nokes, Dr. Seungho Hong and Mr. Irfan Abid for their academic support and valuable advice.
- Grosdidier-Boyer Guillaume for his help in part of the experimental work as his Master of Engineering research project.
- Bill Jin, Henry Yang, Rouwei Wang and Shuyao Liu for their help in part of the experimental work as undergraduate summer research projects.
- Yinan Guo and Luning Li for their help in part of the experimental work as their Bachelor or Engineering final year projects.
• Dr. Dawei Guan, Wei Li, Cheng Chen, Lu Wang, Chuanguang Yuan, Yifan Yang, Dr. Stephane Bertin, Dr. Benjamin Levy, Dr. Reza Shafiei, Dr. Arash Farjood, for their friendship and valuable advice.

My deepest gratitude goes to my wife and my parents, for supporting and encouraging me during the study.
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List of Symbols and Abbreviations

\( a \) = an exponent parameter

\( A \) = flow area

\( A_e \) = “active” flow area obstructed by a roadway embankment at the approach section

\( Al \) = a parameter accounting for the effect of the alignment of the abutment

\( b \) = width of a pier

\( b_l \) = a coefficient

\( b_o \) = diameter of a prototype-size pier

\( B \) = total channel width

\( B_c \) = contracted channel width

\( B_f \) = floodplain width

\( B_m \) = main channel width

\( B_o \) = width of the bridge opening

\( C \) = coefficient of discharge

\( C' \) = standard value of coefficient of discharge

\( C_o \) = an empirical coefficient

\( C_r \) = amplification factor for macro turbulence

\( c \) = mean sediment concentration

\( c_n \) = a Strickler constant

\( c_s \) = speed of sound in water

\( d \) = distance

\( d_n \) = particle size for which \( n \)% of the sediment is finer

\( d_{50} \) = median grain size diameter (\( d_{16} \) and \( d_{84} \) similarly)

\( d_s \) = maximum scour depth measured from the original bed elevation
$d_{sc}$ = contraction scour depth measured from the original bed elevation

$d_{se}$ = equilibrium scour depth

$d_{s-FP}$ = maximum scour depth measured from the original elevation of the floodplain

$d_{s-MC}$ = maximum scour depth measured from the original elevation of the main channel

$D$ = diameter

$D_n$ = rock size for which $n\%$ of the rock is finer

$D_r$ = representative diameter of riprap

$E = \frac{1}{2} \rho \left( \overline{u^2} + \overline{v^2} + \overline{w^2} \right)$ = turbulent kinetic energy

$F_d = \sqrt{g \cdot d_{50}} = $ densimetric particle Froude number

$F_{dm}$ = densimetric mixture Froude number

$F_d^- = $ drag force

$F_l = f_a L / U = $ dimensionless frequency

$F_l = $ lift force

$F_n = $ gravitation force

$Fr = $ Froude number

$F_s = $ a parameter accounting for the effects of abutment length and, the position and the dimension of the scour hole on scour depth

$F_r = $ transverse component of gravitation force

$f = $ function

$f_r = $ recording frequency

$f_s = $ safety factor

$g = $ acceleration of gravity

$G = $ a parameter accounting for the effect of channel geometry
\( H \) = bed form height

\( H_b \) = distance from the bottom of the girder to the original channel bed

\( h_f \) = head loss due to friction

\( h_p = y_m - y_j \) = elevation difference between the floodplain and the main channel

\( k = \sqrt{u'^2 + v'^2 + w'^2} \) = overall turbulence intensity

\( k_F \) = a coefficient that adjusts for the Froude number

\( k_s \) = equivalent sand-grain roughness height

\( k_T \) = coefficient for influence of deck submergence

\( k_w = \frac{2\pi}{\lambda} \) = dimensionless dune wavenumber

\( \langle k \rangle \) = width-averaged overall turbulence intensity

\( K_{JL}, K_G, K_f, K_d, K_x, K_{x^*}, K_{\theta}, K_{\theta^*}, K_f, K_{\sigma} \) = parameters accounting for abutment length and flow depth, channel geometry, flow intensity, sediment gradation factor, abutment shape factor, adjusted abutment shape factor, bridge orientation factor, adjusted bridge orientation factor, scour time, and sediment nonuniformity, respectively

\( K_f, K_{\theta}, K_w \) = adjustment factors for spiral flow at the abutment toe, for velocity and for inadequate similitude of large-scale turbulence structures

\( K_n \) = the constant in Manning’s Equation

\( \kappa_G \) = geometric nonuniformity factor

\( L \) = energy containing eddy length-scale

\( L_a \) = abutment length

\( L_c \) = contraction length

\( L_R \) = reference length
\( L' = \frac{L_c}{B} = \text{relative contraction length} \)

\( L' = \text{length of active flow obstructed by the approach embankment} \)

\( L_s = \text{distance between the edge of the residual apron } (W_s \geq 0) \text{ to the nearest maximum scour depth} \)

\( l_s = \text{length of the riprap armour along the scour hole slope} \)

\( M = \frac{(Q - Q_{\text{obs}})}{Q} = \text{discharge contraction ratio} \)

\( m = \text{geometric contraction ratio} \)

\( N = \text{shape coefficient of abutments and piers} \)

\( n = \text{Manning roughness coefficient} \)

\( n_{\text{apron}} = \text{nominal number of riprap layers in the apron} \)

\( Q = \text{discharge} \)

\( Q = \text{relatively large discharge intensity} \)

\( Q_{bd} = \text{discharge obstructed by the bridge deck at the approach section} \)

\( Q_e = \text{“active” discharge obstructed by an approach embankment} \)

\( Q_{\text{total}} = \text{total discharge} \)

\( Q_f = \text{discharge that could pass through the bridge opening freely} \)

\( Q_f = \text{discharge on the floodplain} \)

\( Q_{ftr} = \text{unobstructed discharge on the floodplain of the approach section} \)

\( Q_m = \text{discharge in the main channel} \)

\( Q_{\text{submerged}} = \text{discharge beneath the bridge deck} \)

\( Q_{\text{obs}} = \text{discharge obstructed by an equivalent length of the embankment at the approach section} \)
\(Q_{OT}\) = overtopping discharge

\(Q_{ori}\) = discharge above the bridge deck at the approach section

\(q\) = width-averaged unit discharge

\(q\) = relatively small discharge intensity

\(q_f\) = width-averaged unit discharge on the floodplain

\(q_m\) = width-averaged unit discharge in the main channel (including main channel bank)

\(q_{\text{2-submerged}}\) = unit discharge for the flow area under the bridge deck

\(R\) = hydraulic radius

\(r_T\) = depth-amplification factor

\(r_{TA}\) = a coefficient intended to account for the additional scour for Scour Condition A

\(r_{TB}\) = a coefficient intended to account for the additional scour for Scour Condition B;

\(R_{av} = -u'v'\) = Reynolds shear stress component on the horizontal plane

\(S\) = channel slope

\(Sh\) = a parameter accounting for the effect of the shape of the abutment

\(S_s = \left(\rho_s - \rho\right) / \rho\) = submerged specific gravity for sediment

\(Sk_{uu}, Sk_{ww}\) = streamwise and vertical components of skewness

\(T = t \sqrt{g'd_{50}/L_R}\) = relative time

\(t\) = scour time

\(t_d\) = deck submergence

\(t_e\) = time required to achieve equilibrium scour depth

\(t^* = t_e \frac{V}{L_o}\) = dimensionless time parameter

\(u', v', w'\) = fluctuating longitudinal, transverse and vertical velocity components
\( u^* \) = shear velocity

\( u_c^* \) = critical shear velocity

\( u_{cr}^* \) = critical shear velocity for riprap

\( V \) = velocity

\( V_a \) = the velocity at which sediment particles become mobile in the armour layer of non-uniform sediments

\( V_x, V_y, V_z \) = mean velocity components in the longitudinal, transverse and vertical directions

\( V_c \) = critical velocity

\( V_{2\text{-submerged}} \) = width-averaged stream-wise velocity for the flow area under the bridge deck

\( V_{OT} \) = flow velocity over the bridge deck

\( V_{sr}, V_{yr} \) = reference streamwise and transverse surface velocity components, respectively

\( W_a = L_c \) = width of the abutment

\( W_{apron} \) = transverse extent of the apron

\( W_0 \) = minimum width of the apron

\( W_s \) = residual apron width after scour

\( w \) = fall velocity of the sediment

\( w_b \) = flow depth above the bridge deck

\( X, Y, Z \) = longitudinal, transverse and vertical distances

\( y \) = water depth

\( y_c \) = long contraction scour depth

\( y_{max} \) = maximum water depth after scour
\( y_{f0}, y_{m0} = \) flow depth far downstream of obstructions, on the floodplain and in the main channel, respectively

\( y_{f1}, y_{m1} = \) flow depth at the approach section, on the floodplain and in the main channel, respectively

\( y_{f2}, y_{m2} = \) flow depth at the bridge section, on the floodplain and in the main channel, respectively

\( y_{hd} = \) hydraulic depth

\( z_b = \) the distance from the measurement point to the bottom

\( \tau = \) time-averaged shear stress

\( \tau_c = \) critical shear stress for initiation of motion

\( \tau_{c,\theta} = \) critical shear stress for initiation of motion on a side slope of angle \( \theta \)

\( \tau_{uv}, \tau_{uw}, \tau_{vw} = \) normalized Reynolds shear stress

\( \sigma_g = \) geometric standard deviation

\( \rho = \) density

\( \alpha = \) the distance from the channel wall (on the floodplain side) to the outer edge of the scour hole

\( \alpha_i = \) a coefficient which takes into account the variation in velocity distribution

\( \alpha_k = \) a constant

\( \beta = \) side slope of the scour hole

\( \theta = \) slope angle

\( \theta' = \) Shields parameter

\( \nu = \) kinematic viscosity of water
\( \nu \) = dynamic viscosity of fluid

\( \varphi \) = an exponent parameter

\( \eta \) = an exponent parameter

\( \gamma, \gamma_s \) = specific weight of water and sediment, respectively

\( \lambda \) = dune length

\( \lambda_2 \) = a coefficient

\( \zeta \) = an exponent

\( \Delta \) = dune heights

\( \Delta y \) = difference in elevation of the water surface between the approach section and the bridge section

\( \psi \) = obstruction ratio

\( \phi \) = angle of repose

\( \mu \) = coefficient of Coulomb resistive force

\( \xi = \frac{q_{f1}}{MV_{c0}y_{f0}} \) = a composite parameter

**Subscripts**

\( c \) denotes the critical state

\( C \) denotes contraction

\( f \) denotes floodplain

\( m \) denotes main channel

\( i \) denotes incipience

\( \text{max} \) denotes the maximum value

\( s \) denotes sediment
$w$ denotes vertical component

0 denotes the cross-section far downstream the abutment

1 denotes cross-section 1 (C.S.1)

2 denotes cross-section 4 (C.S.4)

**Abbreviations**

$ADV =$ acoustic Doppler velocimeter

$COR =$ correlation of ADV measurements

$CW =$ clear-water

$FS =$ free surface

$LB =$ live-bed

$LSA =$ long setback abutment

$MSA =$ medium setback abutment

$NA =$ no abutment

$OT =$ overtopping

$SBR =$ set-back distance/average channel flow depth

$SG =$ specific gravity

$SNR =$ signal-to-noise ratio of ADV measurements

$SO =$ submerged orifice

$SSA =$ short setback abutment

$TKE =$ turbulence kinetic energy

$US =$ the United States
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Chapter 7 contains materials (section 7.2) that was published as the following: Xiong, X., Melville, W. B., Friedrich, H., (2013). "The effect of contraction length on abutment scour." 35th IAHR proceeding, Chengdu, China

| Nature of contribution by PhD candidate | Designing and performing experiments, data analysis and writing the manuscript |
| Extent of contribution by PhD candidate (%) | 75 |

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Chapter 1. MOTIVATIONS AND OBJECTIVES

As essential parts of a bridge, bridge abutments vertically support the superstructure and laterally resist the pressure of soil. During a flood, an abutment commonly narrows the channel and presents an abrupt obstruction to the flow, and subsequently generates scour to the adjacent waterway. Geotechnical failures of both the soil comprising the embankment and the soil underneath the abutment may occur during the abutment scour (Barkdoll et al. 2007). Sturm et al. (2011: 3) define bridge abutment scour as “scour at the bridge-opening end of an abutment, and directly attributable to the flow field developed by flow passing around an abutment”. Knowledge of scour depth and scour geometry around the bridge abutment during flood events is vital for optimal bridge foundation design (e.g., NCHRP 697).

Abutment scour has been attracting attention because of its large ‘contribution’ to bridge failures. The FHWA (Federal Highway Administration) reported there are approximately 590,000 highway bridges in the United States (US) National Bridge Inventory, and more than 20,094 bridges were declared scour critical (Gee 2008). According to Lagasse et al. (2007), scour causes 60% of bridge failures in the US. According to statistics compiled by the Structures Division of the New York State Department of Transportation, from 1966 to 2005, there had been at least 871 documented bridge failures related to scour in US (Hunt, 2009).

Extreme floods have caused catastrophic damages to bridges. For example, the Great Mississippi and Missouri Rivers Flood of 1993 caused 23 bridge failures, including 14 bridge failures because of abutment scour, and three bridge failures because of pier and abutment scour. During the 1994 tropical storm Alberto (Wikipedia, 2016), scour destroyed more than 150 bridges in Georgia. Chang (1973) reported a statistical summary of the cause and cost of bridge failures, showing that of 383 US bridge failures caused by catastrophic floods, 71.8%
of the bridge failures involved abutment damage. Detailed descriptions of scour leading to major bridge failures are presented in Transportation Research Record 950 (Davis, 1984).

Sutherland (1986) reported that, for all major bridge failures that occurred in New Zealand during the period 1960-1984, 29 out of 108 bridge failures were caused by abutment scour. Kandasamy and Melville (1989) found that 60% of bridge failures that occurred in New Zealand during Cyclone Bola were related to abutment or approach scour. In a survey conducted by Macky (1990), scour by rivers causes road expenditure in New Zealand of $36,000,000 per year. In addition, the survey showed that more than 35% of total expenditure was for bridge abutments and approaches.

There are four main reasons why it is difficult to fully understand the scour around abutments. Firstly, a long-standing problem of abutment scour is that it manifests itself as a case-specific interaction of complex processes. Given the complexity of scour processes, and the difficulty of fully comprehending the hydrological and the geotechnical conditions, it is understandable that current abutment scour formulas provide a wide range of scour depth estimates. After decades of efforts, a clear delineation of the dominant variables governing abutment scour has been obtained (Sturm et al., 2011). Among these efforts, physical modelling is well-acknowledged as an effective approach to abutment scour investigation, and modelling results have been reported by Laursen and Toch (1956), Laursen (1960, 1963), Kwan (1988), Kandasamy (1989), Dongol (1994), Froehlich (1989), Melville (1992, 1995, 1997), Melville and Coleman (2000), Melville et al. (2006), van Ballegooij (2005), Sturm and Janjua (1994), Sturm (1999a, 1999b, 2006), Ettema et al. (2008), and Ettema et al. (2010). However, the innate complexity of abutment scour in natural systems makes it difficult to fully simulate scour processes in laboratories. Therefore, different levels of simplification of either bridge
abutment models, flow conditions or channel geometries have been employed. The majority of previous studies were conducted in rectangular channels with clear-water (CW) free surface (hereinafter FS) flows, and more than half of the simplified physical models did not have scour countermeasures (Melville and Coleman, 2000). However, rivers usually evolve into compound cross-sections with a narrow and deep main channel, and two wide and shallow floodplains. Also, destructive abutment scour normally occurs in floods, during which live-bed (LB) scour commonly prevails in the main channel, and clear-water conditions prevail on the vegetated floodplain. Additionally, in practice, abutments are generally protected by suitable forms of scour countermeasures. The application of scour countermeasures for bridge foundations is included in engineering guidelines by Lagasse et al. (2009) and by Garcia (2008). However, these guidelines are mainly applicable to abutment scour under FS flows. Besides, though scour countermeasures are generally used in practice, the suggested scour depth prediction formulas by Sturm et al. (2011) are mainly based on studies without using scour countermeasures. In summary, it is necessary to conduct physical modelling that reveals close-to-reality scour processes, and to evaluate the maximum scour depth with consideration of scour countermeasures.

Secondly, in most situations, local scour and contraction scour occur simultaneously and interactively around an abutment. Local scour is mainly caused by a principal vortex system or large-scale turbulence structures generated by flow separation, whereas contraction scour is caused by flow acceleration due to physical contraction of the channel width. A former version of abutment scour guidance (the Federal Highway Administration’s Hydraulic Engineering Circular No. 18 (HEC-18), Richardson and Davis, 2001) recommended that the local scour component and the contraction scour component should be added to represent the total scour around an abutment. Many studies, for example those of Laursen (1963),
Holnbeck et al. (1993), Schreider et al. (2001), Hong (2005), and Ettema et al. (2010), suggested that local scour and contraction scour are not independent, additive components. The current version of HEC-18 (Arneson et al. 2012) recommends the following, as suggested by Ettema et al. (2010): the total scour around an abutment should be treated as an amplified long contraction scour; the scour amplification occurs near the abutment; and the scour is attributable to non-uniform distribution of flow around an abutment and to large-scale turbulence structures generated by the flow passing around the abutment. In summary, the combination of local scour and contraction scour components still requires extensive research.

Thirdly, the occurrence of vertical contraction (when the flow area is vertically reduced) and abutment scour further complicates abutment scour. During extreme floods, pressure flows may occur in the form of submerged orifice (hereinafter SO) flow (when the bridge deck is partially inundated) or overtopping (hereinafter OT) flow (when fully inundated). Extreme high flows have occurred several times in the US in the last 30 years. In the Great Mississippi and Missouri Rivers flood of 1993, new record flood levels were experienced in at least 17 cities from Rock Island to Hardin along the Mississippi River and Missouri River (Parrett et al, 1993). Inundated bridges were reported during the tropical storm Alberto in July 1994 (Stamey, 1997). During this storm, the Flint River and Ocmulgee River basins in Georgia experienced floods that exceeded the 100-year annual exceedance probability discharge along almost their entire lengths. The floods experienced in the Atlanta area in September 2009 were extremely large. Eighteen stream-gauges in the Metropolitan Atlanta area recorded flood magnitudes much greater than the estimated 500-year annual exceedance probability (Gotvald and McCallum 2010). During this flood, numerous bridge abutments and embankments were damaged during overtopping flows (although it is possible that, for such
events, damages occurred because of structural failures). From the available information (Wikipedia, 2016), during the catastrophic flooding in West Virginia in 24 June 2016, the amount of rain that fell on parts of West Virginia and southern Virginia exceeded the 100-year annual exceedance probability, and vertical contraction probably occurred during the flood. Other catastrophic floods that have occurred in the last 20 years include the following (Wikipedia, 2016): July 1997, Central European flood; September 1998, Yangtze River floods in China; June 2001, Houston floods in Texas, US; June 2005, floods in Alberta, Canada; June 2007, Hunter River floods in Australia; September 2007, Africa floods; January 2011, Brazil floods; April - May 2011, Mississippi River floods; June 2013, North India floods; April 2016, Houston floods in Texas, US. The above extreme flood cases highlight the need for studies of the scour situation where high water level occurs, and vertical contraction scour occurs concurrently with local scour and lateral contraction scour. Limited research regarding this scour combination has been conducted by Hong (2012) and Hong et al. (2015).

Finally, for the scour estimation method developed by Ettema et al. (2010), the long contraction scour depth estimate is based on results from simplified models: unprotected abutment models with smooth transitions between the approach section and the contracted section; and abutment models that either extend to the junction of the floodplain and the main channel in a compound channel, or are in a rectangular channel. In reality, the following situations commonly occur: a bridge abutment presents a short and abrupt contraction rather than a long and smooth contraction to the flow; for a medium setback abutment, scour holes developing on the floodplain and in the main channel merge or interact; and scour countermeasures provide dynamic protection for the abutment. The above realistic situations are the main differences to the experimental conditions for which the widely used long
contraction theory by Laursen (1960, 1963) was derived. Topics regarding the effect of contraction length on abutment scour, and the effect of scour countermeasures on long contraction scour are worthy of investigation.

The aim of this study was not to obtain a scour-depth estimation formula, because a widely applicable abutment scour formula may not even exist. The more realistic goals of this PhD project are summarized in the following:

- Design experiments to better simulate realistic scour processes in a compound channel; better understand the effects of SO and OT flows on abutment scour depth in a live-bed scour regime; better understand the patterns of bed forms around the bridge abutment and in the scour hole; and investigate live-bed scour both at the bridge section and in the scour hole.

- Investigate the general features of the flow field in the vicinity of an abutment for three flow conditions, i.e., FS, SO and OT flows, and better understand the effect of vertical contraction on near-bottom flow patterns.

- Better understand the effects of SO and OT flows on abutment scour depth in clear-water scour regime; investigate the temporal development of scour at the bridge section and in the scour hole of clear-water scour; determine the common geometric features of the scour hole; and provide practical results and suggestions, which can be readily used for scour countermeasure design.

- Better understand the effect of contraction length on abutment scour and better understand the effect of scour countermeasures on long contraction scour.
Chapter 2. LITERATURE REVIEW

2.1 Introduction

The interest in abutment scour shown by researchers stems from the disruptive consequences of abutment failures, and from the difficulty of providing useful abutment scour estimates. To understand the scour characteristics around abutments, and to develop scour-depth estimation methods, numerous studies have been carried out in the last 60 years, and many encouraging achievements have been made. This chapter presents a critical review of current knowledge of bridge abutment scour, and of the leading formulas currently employed for estimating scour depth.

The next three sections contain material pertaining to the mechanism of scour, classification of abutment scour, and topics of different scour components and their interactions. The following five sections review literature on flow characteristics around the abutment, long contraction scour, scour countermeasures, the temporal development of scour, and scale effect. Finally, the relevant experimental studies of Umbrell et al. (1998), Chang and Davis (1998a, 1998b), Melville (1992, 1995, 1997), Melville and Coleman (2000), Melville et al. (2006), Sturm and Janjua (1994), Sturm (2004, 2006), and Ettema et al. (2010), are reviewed.

2.2 Scour mechanism

Incipience of motion

Scour occurs when the channel material is removed by the flow; this removal of material is generally called sediment transport. There are two main modes of sediment transport in rivers: bed load and suspended load. For both modes of transport, hydrodynamic forces acting on the sediment particle exceed its resistant forces, which are mainly due to gravity. Conventionally,
the Shields parameter is used as the incipience criterion of a sediment particle, based on the time-averaged boundary shear stress and the physical properties (size and the specific gravity) of the sediment. With the development of hot-film anemometers and the hydrogen-bubble technique, the role of turbulence (that is bursting events: ‘ejections’, ‘sweeps’, ‘inward interactions’ and ‘outwards interactions’) on the incipient motion of sediment has been recognized. Diplas et al. (2008) proposed that impulse (instantaneous turbulent forces), rather than general hydrodynamic forces, is the reason for sediment motion under the two limiting conditions of pure drag and pure lift. In addition, Diplas et al. (2008) demonstrated that both the duration of turbulent forces and the magnitude of the instantaneous turbulent forces significantly influenced a sediment grain’s incipience of motion. Valyrakis et al. (2010) also proposed that impulse is a more appropriate and universal criterion for identifying incipience of motion.

However, the Shields parameter is more frequently used, particularly in practice, than the impulse considerations, to identify a sediment grain’s incipience of motion. Shields (1936) conducted a set of experiments, and introduced a parameter ($\theta^*$), now known as the Shields parameter that is a function of the shear Reynolds number ($\frac{u^* d}{v}$); from dimensional analysis and fluid mechanics theory:

$$\theta^* = \frac{\tau_c}{\rho g S_s} = f\left(\frac{u^* d}{v}\right)$$

Equation 2.1

in which $\tau_c$ is the critical bed shear stress for initiation of motion, $\rho$ is water density, $g$ is the acceleration of gravity, $S_s = (\rho_s - \rho)/\rho$ is the submerged specific gravity for the sediment, $d$ is the diameter of sediment particle, $f$ denotes function, $u^*$ is the critical
shear velocity \( u_* = \sqrt{\frac{\tau_v}{\rho}} \), \( \nu \) is the kinematic viscosity of water and the subscript \( s \) denotes sediment.

The Shields diagram is shown in Figure 2.1, in which \( \gamma_s \) and \( \gamma \) denote the specific weight of the sediment and water, respectively. Further, many other researchers, including Neil and Yalin (1969), Gessler (1970), Kennedy (1995) and Buffington (1999), contributed to the modern Shields diagram, as shown in Figure 2.2.

\[
\frac{d}{\nu} \sqrt{\frac{0.1(\frac{\gamma}{\gamma} - 1) \rho d}{\gamma}}
\]

**Figure 2.1** Shields diagram for initiation of motion (adapted from Vanoni (1964), reproduced from Sedimentation Engineering, American Society of Civil Engineers (ASCE), Manuals and Reports on Engineering Practice No.110, (Garcia, 2008))
According to the Strickler equation (Strickler, 1923), the relationship between the critical mean channel velocity \( V_c \) and the critical shear velocity \( u_c^* \) is as follows:

\[
\frac{V_c}{u_c^*} = 5.75 \log \left( \frac{5.53 \frac{y}{d_{50}}} \right)
\]

Equation 2.2a

in which \( y \) is flow depth, and \( d_{50} \) is the sediment size for which 50% of the sediment is finer by weight.

Also, \( V_c \) can be determined from Keulegan’s equation (Keulegan, 1938):

\[
V_c = 5.75 \sqrt{\rho \left( \frac{y}{\gamma} \right) g d_{50} \log \left( \frac{12.2R}{k_s} \right)}
\]

Equation 2.2b

in which \( R \) is hydraulic radius, and \( k_s \) is equivalent sand-grain roughness height. For non-uniform sediments, \( d_{50} \) alone is inadequate to determine \( V_c \) because the degree of resistance to incipient motion varies with sediment particle size and the interactions between sediment particles of different sizes are complex (Melville and Coleman, 2000). Sediment non-uniformity can be described by the geometric standard deviation \( \sigma_g \), which is defined as

---

**Figure 2.2** Modified Shields diagram (reproduced from Melville and Coleman, 2000)
\[ \sigma_g = \sqrt[\frac{d_{84}}{d_{16}}} \]

in which \(d_{16}\) and \(d_{84}\) are the sediment sizes for which 16% and 84% of the sediment is finer by weight. Sediment is considered uniform if \(\sigma_g < 1.5\).

**Bed forms**

Ripples, dunes and anti-dunes are the three typical types of bed forms in alluvial channels, and different types of bed forms may occur simultaneously. Schematic diagrams of all possible bed forms are shown in Figure 2.3.

Simons and Richardson (1966) classified different types of bed forms and described the flow conditions for them:

1. Ripples and dunes - occur in the lower flow regime with plane bed
2. Washed-out dunes - occur in the transitional flow regime
3. Anti-dunes - occur in the upper flow regime with plane bed.

The initiation of ripples is probably due to the turbulent bursting process (Raudkivi, 1997), and the grain Reynolds number \(\frac{du*}{v}\) was found to be critical in determining the existence of ripples for an erodible bed (Sumer and Bakioglu, 1984). Ripples commonly develop in fine grained sediments and occur mostly in the presence of a viscous sublayer (Garcia, 2008). Richards (1980) presented a two-dimensional linear stability analysis of turbulent flow over an erodible bed. By relating flow over an initial small perturbation to sediment transport, Richards showed that the wavelength of ripple is dependent on surface roughness of the bed, whereas the wavelength of dune is strongly dependent on flow depth. Dunes tend to develop when the flow intensity exceeds the threshold value of the sediment. For dunes to occur, the
following criterion (Kennedy, 1963) must be satisfied for a potential flow (irrotational velocity field) over a wavy bed:

\[ Fr^2 < \frac{1}{k_w} \tanh(k_w) \]  

Equation 2.4

in which \( Fr = \frac{V}{\sqrt{gy}} \) is the Froude number, \( k_w = \frac{2\pi}{\lambda} \) is the dimensionless dune wavenumber and \( \lambda \) is dune length.

According to Garcia (2008), well-developed dunes have dune heights (\( \Delta \)) up to one-sixth of the flow depth:

\[ \frac{\Delta}{y} \leq \frac{1}{6} \]  

Equation 2.5

Dune length (\( \lambda \)) can vary over a wide range:

\[ \lambda = \frac{2\pi y}{k_w} \]  

Equation 2.6

in which the typical range is given by \( 0.25 < k_w < 4.0 \).

Dunes always migrate downstream, whereas antidunes may migrate upstream or downstream. Dunes are commonly associated with subcritical flows (\( Fr < 1 \)), whereas antidunes are associated with supercritical flows (\( Fr > 1 \)).
2.3 Classifications of abutment scour

Scour at a bridge crossing can be classified into three different types, depending on the dominant scour agent: general scour, contraction scour, and local scour, defined as follows (Melville and Coleman, 2000):

**General scour:** general scour occurs irrespective of the presence of any human-imposed structure. Scour agents consist of sediment transport due to fluvial, geomorphic or hydrometeorological conditions over a wide range of space and time scales. Short-term general scour occurs in a single flood or several consecutive floods, and long-term general scour occurs because of progressive degradation of the riverbed due to natural or human-caused changes.
**Contraction Scour:** contraction scour occurs because of physical narrowing of the channel, caused by human-imposed foundations or the road approach embankments of a bridge, or caused by a natural contraction in the width of the river. At the contracted section, the flow accelerates and exerts greater entraining forces on the sediment, resulting in contraction scour throughout the contracted area.

**Local scour:** local scour is caused by interference of flow by a bridge pier or abutment (collectively termed obstructions here), and occurs locally at the obstruction. Scour agents mainly include large scale vortex systems at the base of the obstruction, downward flow at the upstream face of the obstruction, and the wake vortices downstream of the obstruction. Contraction and local scour only occur when the corresponding flow field has enough energy to transport the sediment. For both types of scour, either clear-water or live-bed scour may occur, depending on the flow and sediment properties.

In addition, scour can be classified by flow intensity:

**Clear-water scour:** clear-water scour occurs when the flow intensity is below the threshold of general sediment movement in the approach section. Under clear-water scour conditions, there is no sediment supply to the scour hole from upstream. Equilibrium scour depth is attained when the exerted shear stress in the scour hole is equal to its critical value.

**Live-bed scour:** live-bed scour occurs when the flow intensity is above the threshold of general sediment movement in the approach section. Under live-bed scour conditions, there is continuous sediment supply to the scour hole from upstream. Equilibrium scour depth is attained when the sediment supply into the scour hole is equal to the sediment transported out of the scour hole.
According to an abutment scour methodology called ABSCOUR (Maryland State Highway Administration (MSHA), 2010), abutments can be classified into four categories, based on the location of the abutment on the floodplain of a compound channel:

1. **Bankline abutment**
   Setback distance = 0

2. **Short setback abutment**
   Setback distance \( \leq 5 y_{hd} \)

3. **Intermediate setback abutment**
   \( 5 y_{hd} < \text{Setback distance} \leq 0.75 B_f \)

4. **Long setback abutment**
   Setback distance > 0.75 \( B_f \)

in which \( y_{hd} \) is the hydraulic depth in the channel, \( B_f \) is the width of the floodplain and the setback distance is from the abutment toe and the edge of the main channel.

For abutments with earth-filled embankments, four possible conditions of abutment scour may occur (Ettema et al., 2010; Sturm et al., 2011):

**Scour Condition A**: scour mainly occurs in the main channel of a compound channel. This scour condition may occur when the bed material of the main channel is much more erodible than that on the floodplain. Particularly for a short setback abutment, or even for a bankline abutment, scour may lead to potential geotechnical collapse of the main channel bank, and subsequently to undercutting of the abutment and embankment, as illustrated in Figure 2.4a.

**Scour Condition B**: scour mainly occurs on the floodplain of a compound channel. This scour condition may occur for a long setback or a medium setback abutment, and clear-water scour usually occurs. The scour hole may locally destabilize the embankment, as illustrated in Figure 2.4b.

**Scour Condition AB**: scour occurs in the main channel as well as on the floodplain. This scour condition may occur for an erodible embankment on an erodible floodplain.
**Scour Condition C**: Scour Condition A, B and AB may eventually breach the embankment and fully expose the abutment stub column, and scour may progress like pier scour, as illustrated in Figure 2.4c.

![Scour Condition A, B, and C Diagrams](image)

**Figure 2.4** Possible abutment scour conditions in a compound channel: a. Scour Condition A - scour of the main channel causes geotechnical failure of the main channel bank, resulting in
undercutting of a short setback abutment or a bankline abutment; b. Scour Condition B – scour of the floodplain causes geotechnical failure of a long setback abutment; and, c. Scour Condition C - breaching of the embankment exposes the abutment columns, and scour progresses like pier scour (reproduced from Ettema et al., 2010)

Alternatively, Melville and Coleman (2000) identified four cases of abutment scour, as schematically shown in Figure 2.5. These four types of scour are mainly classified in terms of the transverse extent of the abutment. In terms of the location of the scour hole, Cases A and C (long setback abutments) are similar to Scour Condition A; Case B and Case D (bankline abutment) are similar to Scour Condition B.

Figure 2.5 Four cases of local scour at abutment (reproduced from Melville and Coleman, 2000)

2.4 Interactions of scour components

Contraction and local scour are the two scour components usually occurring around abutments. Because studies of each scour component have mostly been conducted independently, proposed interrelation of the two scour components has been unclear and misleading, to some extent.
The contraction scour component has mainly been investigated using a long contraction abutment model. Pioneering work on long contraction was carried out by Straub (1934). His work was later developed by Laursen (1960, 1963), Komura (1966), Gill (1981), Lim and Cheng (1998) and Dey and Raikar (2005). In these long contraction studies, the local scour component was insignificant because the long contraction models used had a smooth transition between the original channel width and the contracted channel width.

The mechanism of local scour has been extensively studied in the last 30 years. Before the year 2000, abutment scour experiments were mainly conducted using rigid models without any forms of scour countermeasures (Kwan (1988), Kandasamy (1989), Dongol (1994) and Melville (1992, 1995, 1997)). After the year 2000, along with the extensive application of acoustic Doppler velocimetry (ADV), flow measurements were made in equilibrium scour holes, which were also caused by rigid abutments without scour countermeasures (Barbhuiya (2003), Barbhuiya and Dey (2003), Dey and Barbhuiya (2006a, 2006b) and Bressan et al. (2011)). In the scour and flow-measurement studies cited in this paragraph, equilibrium scour holes usually resembled frustums of inverted circular cones, centred on the upstream corner of the abutments, and scour agents were identified as a principle vortex system, downflow, and wake vortices. It is noteworthy that the contribution of contraction scour to the maximum scour depth was rarely discussed in these scour and flow measurement studies.

An overly conservative scour depth estimation method was presented in the former version of Hydraulic Engineering Circular No. 18 (HEC-18, Richardson and Davis 2001), which recommended that the local scour component and the contraction scour component should be added as independent contributions to the total scour. Examples of scour depth discrepancies between the predicted results and the field data (overprediction) can be found in Holnbeck et
al. (1993) and in Niezgoda and Johnson (1999). Results imply that the overprediction of HEC-18 can be attributed to the assumption of independent local and contraction scour components.

The interaction of local and contraction scour around abutments was investigated by Schreider et al. (2001). In their study, an abutment was set on the left and the right sides of a large scale rectangular channel (22-m long and 10-m wide). Three different experiments were designed:

- Using two smooth transition guide banks (Figure 2.6), the local scour components were excluded for both abutments, and only the contraction scour component occurred.
- Using one guide bank, the local scour component was excluded for one abutment.
- Using no guide banks, the local scour components were included for both abutments

Apart from the guidebanks, each experiment had the same experimental conditions, and the same duration of 5-hours. The final cross-sectional profiles of the scour holes for these three experiments are presented in Figure 2.7. The following scour depths were measured on the centreline of the flume (this scour depth was not in any of the local scour holes that formed):

- Using two guide banks, \( d_{sc} = 110\text{-mm} \)
- Using one guide bank, \( d_{sc} = 55\text{-mm} \)
- Using no guide banks, \( d_{sc} = 25\text{-mm} \)

in which \( d_{sc} \) is the measured contraction scour depth from the original bed elevation.

It is apparent that the occurrence of local scour at both abutments significantly reduced the contraction scour depth (by 77% in this study). Also, comparing results for one guide bank and two guide banks shows that local scour depth at one abutment was reduced by the
occurrence of the neighbouring local scour at the other abutment (red and pink profiles in Figure 2.7). Combining the depths results with temporal development of scour (Figure 2.8) shows that the two scour components occur simultaneously and interactively, but with different development rates; the occurrence of local scour reduces the magnitude of other scour components, including its own kind at a neighbouring location in the contracted section. Also, though not shown in the study by Schreider et al., it is reasonable to infer that the occurrence of contraction scour also reduces the local scour depth, because the development of contraction scour enlarges the flow area, and thereby reduces the sediment entraining capacity in the scour hole.

**Figure 2.6** Test with two smooth transition guide banks (reproduced from Schreider et al., 2001)
Research results presented by Laursen and Toch (1956) indicate a similar interaction between pier scour and contraction scour. In their research, multiple bridge piers (cylindrical models)
were evenly spaced across a model rectangular channel. Laursen and Toch defined an obstruction ratio, $\psi$, as the sum of the pier diameters divided by the channel width, $B$; they normalized scour depth, $d_s$, by dividing by the flow depth, $y$, or by dividing by pier width, $b$; and they normalized flow depth, $y$, by dividing by $b$. Figure 2.9 shows the relationship between $d_s/y$ and $\psi$, for various $y/b$. Also, Figure 2.9 includes the formula proposed by Straub (1940) for long contraction scour:

$$\frac{d_{sc}}{y} = \frac{1}{(1-\psi)^{7/4}} - 1$$  \text{Equation 2.7}

in which $d_{sc}$ is the long contraction scour depth in Straub’s formula.

Figure 2.9 shows that for a relatively deep flow, for example for $y/b = 9$, $d_s/y \approx 0.17$ when $\psi \approx 0.04$. In this situation scour was mostly attributable to local scour, and the channel was effectively un-contracted. The local scour depth was about $d_s/b \approx 1.5$.

When $\psi$ increased to 0.5 (many piers), the value of $d_s/y$ was more than three times of that for $\psi \approx 0.04$, and $d_s/b \approx 5.4$. In this case, $d_{sc}/b \approx 5$ ($d_s > d_{sc}$), and the measured scour comprised local scour and contraction scour.

Interactions of contraction scour and pier scour were also investigated by Hong (2005). Results of his experiments showed a reduction in contraction scour of about 25% when pier scour occurred concurrently with contraction scour. He explained that the reduction of contraction scour depth was probably because the rapid temporal development of pier scour caused a rapidly developing conveyance in the pier scour region, and thereby caused a relatively small conveyance in the contraction scour region.
In reality, abutments are short and abrupt contractions. Sturm et al. (2011) re-examined abutment scour, and defined it as “scour at the bridge-opening end of an abutment, and directly attributable to the flow field developed by flow passing around an abutment”. This definition includes the effects of flow acceleration due to physical contraction of the channel, as well as the effects of local turbulence structures induced by the abutment. They recommended that “abutment scour should be taken as some multiple of contraction scour rather than additive to it”, and this recommendation is included in the current version of HEC-18 (Arneson et al., 2012).

### 2.5 Flow patterns around a bridge abutment

In fluvial environments, flow causes and determines scour, and the development of scour in turn alters the flow field. In general, research concerning flow characteristics around a bridge...
abutment was conducted at two stages of the scour process: at the initial state of scour (the plane-bed condition) and at the equilibrium state of scour.

2.5.1 Flow pattern at the initial state of scour

At the initial state of scour, all abutments (including abutments with erodible embankments) can be treated as rigid abutments, and have a flow pattern similar to that of a short contraction. Three similar flow visualization techniques have been employed to investigate the flow structure around the abutment: 1. Chrisohoides et al. (2003) captured the coherent structures on the flow surface by using a digital camera to trace small paper punch outs. 2. van Ballegoooy (2005) investigated the surface two-dimensional flow fields for spill-through abutments of varying abutment lengths \( L_a \) and varying ratios of floodplain width to total channel width \( B_f/B \), by using a particle tracking velocimetry (PTV) technique developed by Nokes (2003). 3. Ettema et al. (2010) investigated the surface two-dimensional flow fields of spill-through and wing-wall abutments, by using a large-scale particle image velocimeter (LSPIV) technique developed at IIHR (Hydroscience & Engineering, University of Iowa). In addition, Hong (2012) investigated the three-dimensional flow features around spill-through abutments over an immobilized channel bed, by using both down-looking and side-looking ADVs. Measurements from the above four studies show that the flow fields around an abutment situated in a compound channel have the following features:

- Flow acceleration occurs both on the floodplain and in the main channel. The magnitude of acceleration increases with \( L_a \) and \( B_f/B \). Severe contraction occurs in the vicinity of the abutment, producing closely spaced stream lines which separate on the face of the abutment, as shown in Figure 2.10. The magnitude of total turbulence kinetic energy (TKE) peaks downstream of the abutment.
• Upstream of the abutment, two or more continuously deforming and interacting eddies exist in the centre of the recirculating region. The location of the flow reattachment point at the abutment face exhibits large-scale temporal unsteadiness, as shown in Figure 2.11.

• Downstream of the abutment, a large anticlockwise rotation occurs, and it extends out transversely past the end of the abutment, as shown in Figure 2.10. Noticeable reverse flow is initiated downstream of the abutment toe.

• For relatively long-extending (transverse direction) abutments, a small clockwise rotation occurs at the downstream corner of the abutment, next to the large anticlockwise rotation.

• The flow fields around wing-wall abutments have similar overall features of flow contraction, turbulence generation and dispersion, to those around spill-through abutments.

Figure 2.10 Surface flow patterns determined by LSPIV for a spill-through abutment; flow field at the initial state of scour for a, a long setback abutment over a fixed floodplain and b, a short setback abutment over a mobile floodplain (reproduced from Ettema et al., 2010)
Figure 2.11 A sequence of time-averaged digital images (time interval between each presented image is 10s) for the upstream corner region, over an immobilized bed (reproduced from Chrisohoides et al., 2003)

Besides free surface flow, Hong (2012) investigated SO and OT flow conditions (collectively termed pressure flows). For pressure flows, transversely-moving flow associated with downward and upward flows were observed at the upstream and the downstream sides of the abutment, as shown in Figure 2.12a and Figure 2.12b, respectively. The vertical components were induced by the bridge deck, indicating the existence of vertical contraction.
Figure 2.12 Exemplary flow patterns at a. a cross section at the upstream side the abutment and b. a cross section at the downstream side of the abutment. (reproduced from Hong, 2012, unit: ft.)

2.5.2 Flow pattern at the equilibrium state of scour

At the equilibrium state of scour, flow patterns vary with the erodibility of the material (including scour countermeasure materials) around abutments. Generally, there are two different flow patterns: the flow pattern for a scour hole centred at the upstream corner of a rigid abutment, and the flow pattern for a scour hole located downstream of an erodible abutment.

For an unprotected, rigid abutment sitting on an erodible channel bed, the following flow features were observed by Kwan (1984), Kwan (1988), and Dey and Barbhuiya (2005):

- For relatively short (transverse direction) abutments, the principal flow features included a surface roller, downflow at the upstream face of the abutment, a principal vortex at the base of the abutment, wake vortices at the downstream of the abutment and a secondary vortex next to the principal vortex, as shown in Figure 2.13 and Figure 2.14. The flow structures around a short abutment were similar to those around a pier. The downflow was caused by a decrease in the stagnation pressure down the
upstream face of the abutment. The principal vortex and the downflow were mainly responsible for scour: the downflow impinged on the bed, producing a groove immediately upstream of the abutment, and the principal vortex carried the dislodged sediments downstream. The magnitude of the downflow and the principal vortex developed with scour, and a balance was achieved at the equilibrium state. Downstream of the abutment, the flow field was very complex, with irregularities due to the vortex shedding. It was found that, in relatively deep flows, the flow patterns and the maximum downflow were insensitive to changes in the approach flow depth.

- For relatively long abutments (transverse direction), the flow structures are similar to those for shorter abutments (Figure 2.15). A strong principal vortex and a relatively weak downflow were identified. A large slowly circulating anticlockwise eddy was generated upstream of the abutment near the bank.

![Figure 2.13 Illustration of flow patterns in a scour hole for a short abutment. (reproduced from Melville and Coleman, 2000)](image)
Figure 2.14 Development of vortex systems around the scour hole (reproduced from Kwan, 1988)

Figure 2.15 Illustration of flow patterns in a scour hole for a long abutment. (reproduced from Melville and Coleman, 2000)

Ettema et al. (2010) reported LSPIV results for erodible abutments with rock riprap protecting the embankment side slopes. Results showed that the flow pattern varied substantially with scour bathymetries, erodibility of the floodplain, abutment types, and
different geometries of the channel (for example, different $B_f/B$ values). For example, compared with conditions for an immobilized floodplain, flow was drawn towards the abutment and the transverse extent of the wake region downstream of the abutment was much smaller for erodible floodplain conditions. A breach of the embankment during the experiment further complicated the flow pattern (Figure 2.16). Ettema et al. (2010) found that the general flow features for erodible abutments are:

a. General features are similar to typical flows through a short contraction (Figure 2.17).

b. A large wake region appeared downstream of the abutment. The location of the wake region was case-specific.

c. Flow distribution changed markedly as scour progressed and flow concentrated in the scour region.

Figure 2.16 Surface flow patterns determined by LSPIV for a spill-through abutment; flow field at the equilibrium state of scour with embankment breached a. a spill-through abutment and b. a wing-wall abutment. (reproduced from Ettema et al., 2010)
2.6 Long contraction scour

Solutions to the idealized long contraction theory proposed by Straub (1934) have been obtained analytically and through physical modelling (Laursen, 1960; 1963; Komura, 1966; Gill, 1981; Lim and Cheng, 1998; and Dey and Raikar, 2005). The well-accepted long contraction scour depth prediction methods for live-bed scour and clear-water scour were derived by Laursen (1960; 1963). The following derivation follows closely those of Laursen (1960; 1963).

For bankline abutments (live-bed scour of the main channel) the definition sketch is shown in Figure 2.18. The total discharge at the approach section \( Q_1 \) must be equal to that at the contracted section \( Q_2 \). For the simplified channel configuration shown in Figure 2.18, \( Q_1 = Q_{f1} + Q_{m1} \) and \( Q_2 = Q_{m2} \). Here the subscript ‘1’ denotes the approach section, ‘2’ denotes the contracted section, ‘ \( f \) ’ denotes floodplain, and ‘ \( m \) ’ denotes main channel.
Figure 2.18 Definition sketch of a long contraction for the live-bed scour condition; a. plan view and b. longitudinal view

According to the Manning equation,

\[ Q_{m1} = \frac{B_{m1} y_{m1}^{7/6} \sqrt{y_{m1} S_{m1}}}{n_{m1}} = q_{m1} B_{m1} \]  

Equation 2.8

\[ Q_{m2} = \frac{B_{m2} y_{m2}^{7/6} \sqrt{y_{m2} S_{m2}}}{n_{m2}} = q_{m2} B_{m2} \]  

Equation 2.9

in which \( S \) is the channel slope, \( n \) is the Manning roughness coefficient and \( q \) is the width-average unit discharge.

At the dynamic equilibrium state, equilibrium flow depth is governed by mass conservation of sediment for the approach section and the contracted section.

\[ c_{m1} Q_{m1} = c_{m2} Q_{m2} \]  

Equation 2.10

in which \( c \) is the mean sediment concentration, and
\[
c = \left( \frac{d}{y} \right)^{7/6} \left( \frac{Q^2}{120y^{7/3}d^{2/3}B^2} \right) b_1 \left( \frac{\sqrt{gyS}}{w} \right)^a \quad \text{Equation 2.11}
\]

in which \( d \) is the representative sediment diameter, the exponent \( a \) and the coefficient \( b_1 \) depend on the value of \( \frac{\sqrt{gyS}}{w} \), \( \sqrt{gyS} \) is the total shear velocity, and \( w \) is the fall velocity of the sediment.

From Manning’s equation,
\[
\sqrt{gyS} = \frac{QmB}{By^{7/6}} \quad \text{Equation 2.12}
\]

Assuming sediment geometrical characteristics are the same for the approach section and the contracted section, and neglecting variation of the exponent \( a \) and the coefficient \( b \) with \( \frac{\sqrt{gyS}}{w} \), the following is obtained from Equations 2.10-2.12:
\[
y_{m2}^2 \quad y_{m1}^2 \quad \left( \frac{q_{m2}}{q_{m1}} \right)^{6/7} \left( \frac{B_{m1}}{B_{m2}} \right)^{6 \left( \frac{s+2a}{3s} \right)} \left( \frac{n_{m2}}{n_{m1}} \right)^{6 \left( \frac{a}{3s} \right)} \quad \text{Equation 2.13}
\]

Because \( n_{m2} \approx n_{m1} \), the roughness effect can be safely neglected. For a long contraction by a bankline abutment (only the floodplain area is obstructed), \( B_{m2} = B_{m1} \), and Equation 2.13 leads to the following:
\[
y_{m2}^2 \quad y_{m1}^2 \quad \left( \frac{q_{m2}}{q_{m1}} \right)^{6/7} \quad \text{Equation 2.14a}
\]

Ettema et al. (2010) adapted the above derivation for a contracting stream tube, and proposed the following formula for a short contraction in the live-bed condition:
\[
y_{m_{\text{max}}}^2 \quad y_{m1}^2 \quad r_{TA} \left( \frac{q_{m2}}{q_{m1}} \right)^{6/7} \quad \text{Equation 2.14b}
\]
in which $y_{\text{max}}$ is the maximum scour depth in the main channel, and $r_{Ta}$ is a term that accounts for additional scour in Scour Condition A (including short setback abutments and bankline abutments); the additional scour is mainly attributable to turbulence. The value of $r_{Ta}$ varies with abutment shape, abutment set back distance from the main channel, channel geometry and flow conditions.

For Scour Condition B (clear-water flow on a wide floodplain) scour is commonly confined to the floodplain, and flow interference between the floodplain and the main channel is assumed to be negligible. Therefore, a simplified rectangular channel is used for analysis. The supply of sediment into the scour hole is considered to be negligible. The definition sketch is shown in Figure 2.19. At the equilibrium state, it is assumed that the shear stress at the contracted section ($\tau_{f2}$) is equal to the critical shear stress ($\tau_{fc}$).

**Figure 2.19** Definition sketch of a long contraction for the clear-water scour condition; *a.* plan view and *b.* longitudinal view
From Manning’s and Strickler’s equations (metric units),

\[
\tau_{f1} = \frac{V_{f1}^2 d^{1/3}}{30 y_{f1}^{1/3}} = \frac{Q_{f1}^2 d^{1/3}}{30 B_{f1}^2 y_{f1}^{7/3}} \quad \text{Equation 2.15}
\]

\[
\tau_{f2} = \tau_{f2e} = \frac{V_{f2}^2 d^{1/3}}{30 y_{f2}^{1/3}} = \frac{Q_{f2}^2 d^{1/3}}{30 B_{f2}^2 y_{f2}^{7/3}} \quad \text{Equation 2.16}
\]

in which \(\tau\) is the shear stress and \(V\) is the depth-average velocity.

Equations 2.15 – 2.16 lead to the following

\[
\frac{\tau_{f1}}{\tau_{f2e}} = \frac{Q_{f1}^2 B_{f1}^2 y_{f1}^{7/3}}{Q_{f2}^2 B_{f2}^2 y_{f2}^{7/3}} \quad \text{Equation 2.17}
\]

From discharge continuity, \(Q_{f2} = q_{f2} B_{f2} = Q_{f1} = q_{f1} B_{f1}\), and Equation 2.17 is transformed into the following:

\[
\frac{y_{f2}}{y_{f1}} = \left(\frac{\tau_{f1}}{\tau_{f2e}}\right)^{3/7} \left(\frac{q_{f2}}{q_{f1}}\right)^{6/7} \quad \text{Equation 2.18}
\]

In a manner similar to the above for live-bed scour (Scour Condition A above), Ettema et al. (2010) proposed the following:

\[
\frac{y_{f_{\text{max}}}}{y_{f1}} = r_{TB} \left(\frac{\tau_{f1}}{\tau_{f2e}}\right)^{3/7} \left(\frac{q_{f2}}{q_{f1}}\right)^{6/7} \quad \text{Equation 2.19}
\]

in which \(y_{f_{\text{max}}}\) is the maximum scour depth on the floodplain, and \(r_{TB}\) is a term that accounts for the additional scour in Scour Condition B.

Alternatively, following Sturm (2006) and expressing the critical velocity in terms of Manning’s and Strickler’s equations and expressing channel slope in terms of shear stress and water depth lead to the following:

\[
\frac{V_{fc}}{u^*} = \frac{K_n}{c_s \sqrt{g}} \left(\frac{y_{fc}}{d_{s0}}\right)^{1/6} \quad \text{Equation 2.20}
\]
in which \( u^* \) is the critical shear velocity, \( K_n \) is the constant in Manning’s equation and \( c_n \) is the constant in Strickler’s equation. From continuity,

\[
y_{f2c} = \frac{q_{f2}}{V_{f2c}} \tag{Equation 2.21}
\]

From Equation 2.20,

\[
\frac{V_{f1c}}{V_{f2c}} = \left( \frac{y_{f1c}}{y_{f2c}} \right)^{1/6} \tag{Equation 2.22}
\]

yielding the following equation for clear-water scour (Hong 2012):

\[
\frac{y_{f2c}}{y_{f1}} = \left( \frac{V_{f1}}{V_{f1c}} \right)^{6/7} \left( \frac{q_{f2}}{q_{f1}} \right)^{6/7} \tag{Equation 2.23}
\]

### 2.7 Scour countermeasures – rock riprap apron

Generally, there are two types of scour countermeasures for protecting bridge abutments: mechanical armouring (stabilizing) scour countermeasures, and flow realigning scour countermeasures. Mechanical armouring countermeasures consist of placing riprap, gabions mattresses, geo-bags, cable-tied blocks, grout-filled bags, grout mats, or abutment collars, on abutment slopes or on the adjacent channel bed. The purpose of mechanical armour is to prevent direct contact of the flow with the erosion-vulnerable materials, and to thereby enhance the erosion resistance of the abutment. Flow realigning scour countermeasures consist of guidebanks, dikes, or spurs upstream or downstream of the abutment, to direct the flow away from the abutment or to weaken the flow immediately around the abutment. Also, in-channel devices such as vanes and bendway weirs can be used to regulate the flow. Guidelines and selection criteria for countermeasures are provided by Parker et al. (1998), Barkdoll et al. (2007) and Lagasse et al. (2009).
A rock riprap apron is one of the most common forms of scour countermeasure. The protection mechanism of the apron does not prevent scour from occurring. On the contrary, experimental results presented by Melville et al. (2006) showed that the scour hole increased in size as the riprap apron width increased. For an abutment protected by a rock riprap apron, scour initiates around the outside edge of the apron. During the scour process, rocks in the apron roll into the eroded area, dynamically armouring part of the scour hole, and gradually deflecting the scour hole away from the abutment.

2.7.1 Failure of rock riprap aprons

Blodgett and McConaughy (1985) identified three principal failure modes for rock riprap on sloping embankments: particle erosion failure, translational slide failure, and slump failure. Chiew (1995) identified three modes of failure of rock riprap under clear-water conditions around bridge piers: riprap shear failure, winnowing failure, and edge failure. Under live-bed conditions, Chiew and Lim (2000) found two modes of failure of rock riprap around bridge piers: total disintegration and embedment. Though both of these studies investigated rock riprap around bridge piers, these failure mechanisms can also occur at abutments.

Different failure mechanisms are described below:

**Particle erosion failure** occurs when the flow shear stresses or velocities are excessive.

**Translational slide failure** occurs when the side slope is too steep or the toe of the riprap is undermined.

**Slump failure** occurs when excessive hydrostatic pressure exists in the base material.

**Riprap shear failure** occurs when hydrodynamic flow forces destabilize and dislodge rocks in the apron, because those rocks are not large and heavy enough; the associated failure mechanism is similar to particle erosion failure.
Winnowing failure occurs when the underlying finer bed material is sucked out through the voids of the rock riprap, leading to settlement of the apron.

Edge failure occurs when local scour occurs at the periphery of the apron, resulting in unstable riprap layers.

Total disintegration occurs when the hydrodynamic forces break up the riprap layer and wash way the rocks. Total disintegration is similar to riprap shear failure.

Embedment occurs when the trough of a sand dune arrives at the apron, causing rock riprap to subside to the trough level of the dune, with subsequent embedment by the next crest. Bed feature destabilization (caused by propagation of bed forms) and differential mobility (caused by stability differences between bed material and rock riprap) may cause embedment.

The relationship between flow conditions and failure mechanisms was schematically summarized by Melville and Coleman (2000), as shown in Figure 2.20, in which \( u^* \) is the shear velocity, and \( u_c^* \) and \( u_{cr}^* \) are the critical shear velocities for sediment particles (representative diameter \( d_r \)) and rock riprap (representative diameter \( D_r \)), respectively.
Figure 2.20 Failure mechanisms of rock riprap apron (reproduced from Melville and Coleman, 2000)

2.7.2 Design of the apron

Riprap size and the extent of the apron are critical for protecting the abutment. For a specific flow condition, an appropriate riprap size can reduce the probability of shear failure (normally occurs when riprap size is too small) and winnowing failure (normally occurs when riprap size is too large or the riprap is poorly graded); an appropriate extent of the apron can achieve optimal balance between the cost and the stability of the abutment.

Rock riprap size

Richardson and Davis (1995) proposed the following relationship between the rock riprap size and flow conditions for an abutment sitting on the floodplain of a compound channel:

$$\frac{D_{50}}{y_f^2} = \frac{K_s}{SG - 1} F_r^\phi$$  \hspace{1cm} \text{Equation 2.24}
in which \( D_{50} \) is the rock riprap size for which 50% of the riprap is finer by weight, \( K_s \) is the abutment shape factor (for a spill-through abutment, \( K_s = 0.89 \) when \( Fr \leq 0.8 \), and \( K_s = 0.61 \) when \( Fr > 0.8 \)), \( Fr \) is the Froude number in the contracted section \( (Fr = \sqrt{\frac{2y_f}{gy_{f2}}}) \), \( y_{f} \) is the flow depth on the floodplain of the contracted section, \( SG \) is the specific gravity, and the exponent \( \varphi \) varies with \( Fr \) (\( \varphi = 2 \) when \( Fr \leq 0.8 \), and \( \varphi = 0.28 \) when \( Fr > 0.8 \).)

Following a comparison with other rock riprap sizing proposals, including those by Simons and Lewis (1971), Brown and Clyde (1989), Croad (1989) and Pagan-Ortiz (1991), Melville and Coleman (2000) recommended the sizing proposal by Richardson and Davis (1995). This sizing proposal was later used and recommended by Lagasse et al. (2001), van Ballegoooy (2005), Lagasse et al. (2006), Barkdoll et al. (2007) and Lagasse et al. (2009).

**Apron extent**

Two types of rock riprap apron protection are suggested by researchers: 1. extending the apron down to the estimated scour depth, or 2. laying a launching apron at the periphery of the abutment toe. The latter one has been more commonly used because it is cost-effective and easier to construct.

Richardson and Davis (1995) recommended the following for the extent of the apron:

a. The launching apron should extend along the entire periphery of the abutment toe, as shown in Figure 2.21.

b. The width of the apron should be twice the flow depth on the floodplain near the embankment or 7.5-m, whichever is less.
For the width of the apron, Melville et al. (2006) suggested that “twice the flow depth” may be inadequate, and proposed the following equation for the minimum width of the apron ($W_0$) to prevent scour from undermining of the toe of a spill-through abutment:

$$\frac{W_0}{y_f} = 0.5 \left( \frac{d_{s-FP}}{y_f} \right)^{1.35}$$

Equation 2.25

in which $d_{s-FP}$ is the local scour depth measured from the un-scoured floodplain level, and $W_0$ is defined as the minimum apron width for which the residual apron width after scour ($W_s$) equals to zero. Equation 2.25 provides an envelope to the equilibrium scour data obtained by van Ballegooij (2005).
Lagasse et al. (2006) accepted the apron width recommendations of Richardson and Davis (1995), but suggested increasing the apron extent. Because scour is commonly initiated near the downstream toe of the abutment (Figure 2.22), they suggested that the apron should extend from the abutment back along the downstream side of the embankment, and that this extent should be twice the flow depth on the floodplain near the embankment or 7.5-m, whichever is greater, as shown in Figure 2.22.

![Figure 2.22 Plan view of the apron around a spill-through abutment on the floodplain of a compound channel (after Lagasse et al. 2006)](image)

For the thickness of the apron, the gradation and the density of riprap, and the filter underlying the abutment, Lagasse et al. (2006) provided the following guidelines:

1. The rock riprap apron thickness should not be less than the larger of 1.5 times $D_{50}$ and 1.0 times $D_{100}$. The rock riprap thickness should be increased by 50% when it is installed under water.
2. The target uniformity ratio of the riprap, $D_{85}/D_{15}$, is 2.0 and the allowable range of $D_{85}/D_{15}$ is 1.5 to 2.5.

3. The minimum allowable specific gravity of the riprap is 2.5.

4. Geotextile filters or granular filters should be used. The filter must retain the coarser particles of the bed material while remaining permeable enough to allow infiltration and exfiltration to occur freely.

5. In cases where the abutment extends into the main channel and dune-type bed forms are present, only a geotextile filter should be used for the protection.

### 2.8 The temporal development of scour

In laboratory studies, a clear-water scour experiment usually takes several days or even several weeks to reach equilibrium depth; live-bed scour takes less time, ranging from several hours to several days. The temporal development of scour is therefore important in assessing the scour depth that would be reached during the period of a flood event, particularly for clear-water scour.

Hoffmans and Verheij (1997) identified four phases of local scour around abutments: an initial phase, a development phase, a stabilization phase and an equilibrium phase. The time history of scour is asymptotic to the equilibrium depth for probably all scour conditions, that is, for rigid or erodible, protected or unprotected abutments, in rectangular or compound channels, and for clear-water or live-bed flows. The mathematical expressions for the asymptotic scour development with time mainly comprise exponential relationships (e.g., Cardoso and Bettess, 1999 and Coleman et al., 2003) or logarithmic relationships (e.g., Oliveto and Hager, 2002).
Cardoso and Bettess (1999) conducted 14 abutment scour experiments in a 2.44-m wide compound channel. The abutments were rigid and unprotected, and short setback, long setback and bankline abutments were used. Their results suggest that flows in the main channel and the floodplain may interact, causing a rapidly developing scour hole extending from the floodplain to the main channel. Based on data from 6 long setback abutment experiments, Cardoso and Bettess (1999) proposed the following equation:

\[
\frac{d_s(t)}{d_s(t_c)} = 1 - \exp \left[ -1.025 \ln \left( \frac{t}{t_c} \right)^{0.35} \right]
\]

Equation 2.26

in which \(d_s(t)\) is the scour depth after scour time, \(t\), and \(t_c\) is the time required to achieve equilibrium. The applicability of Equation 2.26 is restricted by the limited range of the data from which it is derived.

Based on 110 clear-water scour experiments on vertical-wall bridge abutments of various lengths (rigid and unprotected), and for a wide range of combinations of flow shallowness (\(y/L_a\)) and flow intensity (\(V_1/V_c\)), Coleman et al. (2003) proposed the following expression:

\[
\frac{d_s(t)}{d_s(t_c)} = \exp \left[ -0.07 \left( \frac{V_1}{V_c} \right)^{-1} \ln \left( \frac{t}{t_c} \right)^{1.5} \right]
\]

Equation 2.27

They proposed the following expressions to estimate the equilibrium time for abutment scour:

\[
t^* = 10^6 \left( \frac{V_1}{V_c} \right)^3 \left( \frac{y}{L_a} \right)^3 \left[ 3 - 1.2 \left( \frac{y}{L_a} \right) \right]
\] \quad \text{for } y/L_a < 1 \text{ and } \frac{L_u}{d_{50}} > 60 \quad \text{Equation 2.28}

\[
t^* = 1.8 \times 10^6 \left( \frac{V_1}{V_c} \right)^3
\] \quad \text{for } y/L_a \geq 1 \text{ and } \frac{L_u}{d_{50}} > 60 \quad \text{Equation 2.29}

in which \(t^* = t_c \frac{V_1}{L_a}\).
Results implied that $y/L_a$ does not affect the relative scour depth $\left( \frac{d_y(t)}{d_y(t_i)} \right)$, but $y/L_a$ does affect the time to achieve equilibrium.

Oliveto and Hager (2002) investigated the temporal development of both pier and abutment scour. They used 3 uniform and 3 non-uniform sediments, and conducted about 200 pier and abutment scour experiments (rigid and unprotected abutments). They found that the temporal development of scour was influenced mainly by three composite parameters:

1. The reference length ($L_R$), which is a composite parameter of abutment length (or pier width) and the approaching flow depth. Here, $L_R = b^{2/3} y_1^{1/3}$ for piers, $L_R = L_a^{2/3} y_1^{1/3}$ for abutments, where $b$ denotes the width (diameter) of the pier and $L_a$ denotes abutment length.

2. The densimetric mixture Froude number ($F_{dm}$). Here, $F_{dm} = \sigma_g^{-1/3} V_i \sqrt{g' d_{50}}$, where $\sigma_g = \sqrt{d_{44}/d_{04}}$ is the geometric standard deviation, $V_i$ is the depth-averaged velocity in the approach section, $g' = \left[ \left( \rho_s - \rho \right)/\rho \right] g$, and subscript $s$ denotes sediment.

3. The relative time $T$. Here, $T = t \sqrt{g' d_{50}}/L_R$.

Based on similarity arguments and an analogy to flow resistance, they proposed the following relationship:

$$\frac{d_y(t)}{L_R} = 0.068 N \sigma_g^{-1/2} F_d^{-1.5} \log(T) \quad F_d > F_{di}$$

Equation 2.30

in which $N$ is the shape coefficient ($N = 1$ for a circular pier, and $N = 1.25$ for a rectangular abutment or pier), $F_d$ and $F_{di}$ are the densimetric and the inception densimetric particle Froude numbers ($F_d = V/\sqrt{g' d_{50}}$, $F_{di} = V_i/\sqrt{g' d_{50}}$), $V$ is the cross-sectional average velocity and the subscript $i$ denotes incipience.
It should be noted that Equations 2.27 to 2.30 imply that scour development is subject to flow intensity.

Under live-bed scour conditions, Ballio et al. (2010) experimentally investigated the temporal scales of scour around a vertical-wall abutment (rigid and unprotected). From the results of their study, they found that variation of live-bed scour depth with time was logarithmic. For the investigated flow intensity range ($1 < V/V_c \leq 2$), the equilibrium time rapidly decreased with increase in flow intensity.

The majority of experiments regarding scour development were carried out with rigid, unprotected abutments, as indicated above. The practical applications of these relationships are limited, because in reality, protected abutments with erodible embankments are commonly used.

Hong (2012) conducted 18 clear-water experiments, using erodible embankments and protection by launching aprons. Because he found that the scour hole geometry and location varied with time, for each experiment he measured the entire moveable bed section at six intermediate times before equilibrium. He found that the normalized maximum flow depths ($y_{\text{max}}(t)/y_1$) had linear relationships with the logarithm of the normalized time ($\log(V/V_c)/y_1$), though the gradients of the linear relationships varied with experiments. Here, $y_{\text{max}}(t)$ is the maximum flow depth for each intermediate measurement, and the values of $V_1$ and $y_1$ were different for different abutment setback distances ($V_1 = V_{f1}$ and $y_1 = y_{f1}$ for long setback abutments, and $V_1 = V_{m1}$ and $y_1 = y_{m1}$ for short setback and bankline abutments). He observed that the values of the normalized time to equilibrium ($V_1 t_c/y_1$) greatly depended on flow types (i.e., FS, SO or OT) and the setback distance of the abutment.
2.9 Scale effect

Because the dimensions of laboratory flumes are limited by practical considerations, investigations using full-size models are not possible and scaled models are used. The undesirable side effects of using scaled models are the inherent scale effects. For abutment scour in cohesionless sediments, scale effects result from size limitations of cohesionless sediments (because small sediment grains, e.g., \(d<0.2\)-mm, have cohesive properties), and from the contradictions between geometric similitude requirements and dynamic similitude requirements. Ettema et al. (1998) proposed a set of dimensionless parameters that could be used to describe scour processes at a pier:

\[
\frac{d_s}{b} = f\left(\frac{V}{V_c}, \frac{V^2}{gb}, \frac{y}{b}, \frac{b}{d_e}, \frac{\rho Vb}{\nu}\right)
\]

Equation 2.31

in which \(d_s\) is scour depth, \(b\) is pier width or diameter, \(V\) is the mean velocity, \(V_c\) is the critical value of \(V\) in terms of incipient motion of sediment grains, \(y\) is flow depth, \(d_e\) is representative sediment particle size, and \(\rho\) and \(\nu\) are the density and dynamic viscosity of fluid, respectively. These dimensionless parameters are characterized by properties of flow, structure (pier), and sediment.

Ettema et al. (1998) realized that satisfying all similitude requirements of all the dimensionless parameters in Equation 2.31 was almost impossible, unless using undistorted models. Mainly because of failures in obtaining realistic \(b/d_e\) values, similitudes of particle mobility \((V/V_c)\), \((V^2/gb)\) or Froude number \((V^2/gy)\), and flow shallowness \((y/b)\) could not be simultaneously achieved in laboratory flumes.

Sheppard et al. (2004) conducted a series of large scale experiments in a 6.1-m wide channel.
Their results showed a continuous decrease in \( d_s/b \) as \( b/d_s \) increased from 25 to 4155, implying overly conservative estimations of pier scour in small-scale studies in laboratories.

Ettema et al. (2006) experimentally investigated the frequency and the intensity of shed vortices and wake vortices (collectively termed large-scale turbulence structures) for 6 cylindrical pier models with different pier diameters. Their results revealed a relationship between the scour depth and the magnitude of large-scale turbulence structures. To account for inadequate similitude of large-scale turbulence structures, they tentatively suggested the following adjustment factor \( (K_w) \):

\[
K_w = 0.95 \left( \frac{b_0}{b} \right)^{-0.26}
\]

Equation 2.32

in which \( b_0 \) represents the diameter of a prototype pier \( (b_0 = 0.4\text{-m in their study}) \) and \( b \) represents the diameter of a laboratory-used pier, and \( b \ll b_0 \).

In addition, Lee and Sturm (2009) found that the value of \( b/d_s \) was responsible for different magnitudes of large-scale unsteadiness of the horseshoe vortex. They also stated that the size constraint of cohesionless sediments was probably a source of overestimation of scour depth around hydraulic structures. They applied a least-squares regression analysis to their data, which yielded the following empirical relationships between \( d_s/b \) and \( b/d_{50} \):

\[
\frac{d_s}{b} = 5.0 \log \left( \frac{b}{d_{50}} \right) - 4.0, \quad \text{for} \quad 6 \leq b/d_{50} \leq 25 \quad \text{Equation 2.33 (a)}
\]

\[
\frac{d_s}{b} = \frac{1.8}{(0.02b/d_{50} - 0.2)^2 + 1} + 1.3, \quad \text{for} \quad 25 < b/d_{50} \leq 10000 \quad \text{Equation 2.33 (b)}
\]

in which \( d_{50} \) is the median sediment grain size.
Scale effects are also inherent in physical simulations of abutment scour, though Ettema et al. (2010) stated that a 1:30 scale for model abutments was large enough and the estimation of scour depth was relatively reliable.

2.10 Leading abutment scour estimation formulas

_Umbrell et al. (1998)_

Umbrell et al. (1998) experimentally investigated the clear-water scour caused by vertical contraction. They used a 1:15 scale model of a prototype two-lane bridge deck, in the test section of a 1.8-m wide rectangular channel. The main variables were submergence levels of the bridge deck, sediment size, and flow intensities \( V/V_c \). Flow depth was set at 0.305-m for all experiments. Most of the experiments were run for only 3.5-hours, and equilibrium scour depths were estimated by extrapolations.

They assumed that, at the equilibrium state, the flow velocity over the bridge deck \( V_{OT} \) was approximately equal to the approach velocity \( V_1 \) (Figure 2.23), and that the flow velocity under the bridge deck was equal to the threshold velocity \( V_c \). From continuity,

\[
Q_{total} = V_1 y_1 B = Q_{OT} + Q_{submerged} = V_1 w_b B + V_c (H_b + d_s) B
\]

Equation 2.34

in which \( Q_{total} \) and \( y_1 \) are the total discharge and the flow depth in the approach section, respectively, \( B \) is the channel width, \( Q_{OT} \) is the overtopping discharge over the bridge deck, \( Q_{submerged} \) is the discharge beneath the bridge deck, \( w_b \) is the flow depth above the bridge deck, \( H_b \) is the distance from the bottom of the deck girder to the original bed and \( d_s \) is the equilibrium scour depth. The definition sketch is shown in Figure 2.23.

Simplifying and rearranging Equation 2.34 yields:

\[
\frac{(H_b + d_s)}{y_1} = \frac{V_1}{V_c} \left(1 - \frac{w_b}{y_1}\right)
\]

Equation 2.35
Through regression analysis, Umbrell developed the following best-fit relationship for all their experimental data:

\[ d_s = y_i \left[ 1.102 \left( 1 - \frac{w_b}{y_i} \right) \frac{V_1}{V_c}^{0.603} \right] - H_b \]  

Equation 2.36

for which \( R^2 = 0.81 \).

For SO flows, \( w_b = 0 \), and the best-fit relationship becomes the following:

\[ d_s = 1.102 y_i \left( \frac{V_1}{V_c} \right)^{0.603} - H_b \]  

Equation 2.37

for which \( R^2 = 0.79 \).

![Figure 2.23 The definition sketch of vertical contraction, longitudinal view (after Umbrell et al. 1998)](image)

**Froehlich (1989)**

Froehlich (1989) assembled data from several physical modelling studies for both clear-water and live-bed abutment scour. The sources of data are shown in Table 2.1. Through multiple linear regression analysis, he developed the following quantitative relationships between normalized scour depth and the relevant dimensionless parameters:

For clear-water scour,
\[
\frac{d_s}{y_2} = 0.78K_sK_\theta \left( \frac{L'}{y_2} \right)^{0.65} Fr^{1.16} \left( \frac{y_2}{d_{50}} \right)^{1.87} \sigma_g^{-1.87} \quad \text{Equation 2.38}
\]

For live-bed scour,
\[
\frac{d}{y_2} = 2.27K_sK_\theta \left( \frac{L'}{y_2} \right)^{0.43} Fr^{0.61} \quad \text{Equation 2.39}
\]

The terms in Equation 2.39 have the following meanings:

- \(d_s\) is the local scour depth measured from the ambient original channel bed;
- \(y_2\) is the flow depth in the bridge section;
- \(K_s\) is the abutment and embankment shape factor and has the following values:
  - 1.00 for a vertical abutment that has square or rounded corners, and a vertical embankment;
  - 0.82 for a vertical abutment that has wing walls and a sloped approach embankment;
  - 0.55 for a spill-through abutment and a sloped approach embankment;
- \(K_\theta\) \(=\left(\frac{\theta}{90}\right)^{0.13}\) is the abutment alignment factor (\(\theta < 90^\circ\) if the embankment points downstream, \(\theta = 90^\circ\) if the embankment is perpendicular to the flow and \(\theta > 90^\circ\) if the embankment points upstream);
- \(L' (= A_e/y_2)\) is the length of active flow obstructed by the approach embankment;
- \(A_e\) is the flow area obstructed by a roadway embankment in the approach section;
- \(Fr (= \frac{V_e}{\sqrt{g y_2}} = \frac{Q_e}{A_e \sqrt{g y_2}})\) is the Froude number;
- \(Q_e\) is the active discharge obstructed by the abutment and the approach embankment;
- \(d_{50}\) is the median sediment grain size;
- \(\sigma_g\) is the geometric standard deviation.

For design purposes, Froehlich added safety factors \((f_s)\) to Equations 2.38 and 2.39.
\[
\left( \frac{d_s}{y_1} \right)_{\text{design}} = \left( \frac{d_s}{y_1} \right)_{\text{expected}} + f_s
\]

Equation 2.40

in which \( \left( \frac{d_s}{y_1} \right)_{\text{expected}} \) is given by Equation 2.38 for clear-water scour and by Equation 2.39 for live-bed scour. Here, 98% of the source data could be enveloped by Equation 2.40 when \( f_s = 1.0 \).

Table 2.1 Sources of data used by Froehlich (1989)

<table>
<thead>
<tr>
<th>Source of Measurements</th>
<th>Number of Measurements</th>
<th>Clear-water</th>
<th>Live-bed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ahmad (1953)</td>
<td>11</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Karaki (1959)</td>
<td>5</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Liu and others (1961)</td>
<td>15</td>
<td>79</td>
<td></td>
</tr>
<tr>
<td>Garde and others (1961,1963)</td>
<td>25</td>
<td>64</td>
<td></td>
</tr>
<tr>
<td>Tison (1962)</td>
<td>3</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Gill (1972)</td>
<td>60</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>Wong (1982)</td>
<td>6</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Rajaratnam and Nwachukwu (1983)</td>
<td>6</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Kwan (1984)</td>
<td>17</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Tey (1984)</td>
<td>10</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Kandasamy (1985)</td>
<td>6</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>164</td>
<td>170</td>
<td></td>
</tr>
</tbody>
</table>


Melville (1992) proposed the following equation for estimating the maximum abutment scour depth \( (d_s) \):
\[ d_s = f \left( \frac{L_a}{y}, \frac{V^2}{g d_{so}}, \frac{d_{so}}{y}, \sigma_g, Sh, Al, G \right) \]  

Equation 2.41

in which \( L_a \) is the abutment length, \( y \) is the flow depth, \( V \) is the mean velocity, \( d_{so} \) and \( \sigma_g \) is the median size and the geometric standard deviation of the sediment, respectively, \( Sh \) and \( Al \) are parameters accounting for the effects of the shape and the alignment of the abutment, respectively, and \( G \) is a parameter accounting for the effect of channel geometry. Combining abutment scour data collected by Gill (1972), Wong (1982), Tey (1984), Kwan (1984, 1988), Kandasamy (1989) and Dongol (1994) yielded the following (Melville 1992):

\[ d_s = K_{yl}K_yK_dK_sK_\theta K_G \]  

Equation 2.42

in which the \( K \) factors represent the effect of the parameters on scour depth in Equation 2.41: flow depth: abutment length factor (\( K_{yl} \)), flow intensity factor (\( K_y \)), sediment size factor (\( K_d \)), sediment nonuniformity factor (\( K_\sigma \)), abutment shape factor (\( K_s \)), abutment alignment factor (\( K_\theta \)) and channel geometry factor (\( K_G \)). Each \( K \) factor is explained below.

Experimental results (Melville, 1992) show that values of \( K_{yl} \) depend on the values of \( L_a/y \), and \( K_{yl} \) is defined in the following:

\[ K_{yl} = 2L_a \quad \text{for } L_a/y < 1 \quad \text{(short abutment)} \]  

Equation 2.43 (a)

\[ K_{yl} = 2\sqrt{L_a}y \quad \text{for } 1 < L_a/y < 25 \quad \text{(intermediate abutment)} \]  

Equation 2.43 (b)

\[ K_{yl} = 10y \quad \text{for } L_a/y > 25 \quad \text{(long abutment)} \]  

Equation 2.43 (c)

Each of the above three equations envelopes the \( K_{yl} \) data for the corresponding range of \( L_a/y \). For small values of \( L_a/y \) (Equation 2.43(a)), the magnitudes of the dominant scour agents (primary vortex and downflow) vary with abutment length, and therefore scour depth varies with abutment length; for large values of \( L_a/y \) (Equation 2.43(c)), scour holes occur...
around the end of the abutment, and the dimensions of scour holes are dependent on flow depth and are independent of abutment length. For intermediate abutments (Equation 2.43(b)), both flow depth and abutment length affect the final scour depth.

\( K_I \) is defined in the following:

\[
K_I = \frac{V - (V_a - V_c)}{V_c} \quad \text{for} \quad \frac{V - (V_a - V_c)}{V_c} < 1
\]

\[ \text{Equation 2.44 (a)} \]

\[
K_I = 1.0 \quad \text{for} \quad \frac{V - (V_a - V_c)}{V_c} \geq 1
\]

\[ \text{Equation 2.44 (b)} \]

in which \( V_c \) is the critical velocity for incipience of motion for uniform sediments, \( V_a \) is the velocity at which sediment particles become mobile in the armour layer of non-uniform sediments (\( V_a = V_c \) for uniform sediments). It should be noted that Equation 2.44(b) may provide conservative estimations of \( K_I \) in live-bed flow conditions, particularly for non-uniform gravel beds, as shown in Figure 2.24.

**Figure 2.24** Relationship between flow intensity and local scour depth (reproduced from Melville and Coleman, 2000)
$K_d$ is defined in the following:

$$K_d = 0.57 \log \left( \frac{2.24 L_a}{d_{50}} \right) \quad \text{for } \frac{L_a}{d_{50}} \leq 25$$

Equation 2.45 (a)

$$K_d = 1.0 \quad \text{for } \frac{L_a}{d_{50}} > 25$$

Equation 2.45 (b)

In physical modelling, it is common to have $\frac{L_a}{d_{50}} > 25$, and $K_d = 1.0$ is commonly used. For non-uniform sediments, $d_{50}$ should be replaced with the armour layer median size, $d_{50a}$, in Equation 2.45.

The effect of sediment nonuniformity is reflected in the expressions of $K_i$ (see Figure 2.24) and $K_d$, so $K_d$ is conservatively assumed to be unity; and this assumption may lead to overestimation of the scour depth for non-uniform sediments.

$sK^*$ is defined in the following:

$$sK^* = K_s \quad \text{for } \frac{L_a}{y} \leq 10$$

Equation 2.46(a)

$$sK^* = K_s + \frac{2}{3} \left( 1 - K_s \right) \left( 0.1 \frac{L_a}{y} - 1 \right) \quad \text{for } 10 < \frac{L_a}{y} < 25$$

Equation 2.46(b)

$$sK^* = 1.0 \quad \text{for } \frac{L_a}{y} \geq 25$$

Equation 2.46(c)

in which $K_s$ is given in Table 2.2 for commonly used abutment shapes. It can be seen that $sK^*$ has a large effect on abutment scour depth, and the effect of $sK^*$ weakens as $L_a/y$ increases (Melville, 1992), because the effect of $sK^*$ on scour occurs at the end of the abutment.
Table 2.2 Abutment shape factor (Melville and Coleman, 2000)

<table>
<thead>
<tr>
<th>Abutment shape</th>
<th>$K_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical-wall</td>
<td>1.0</td>
</tr>
<tr>
<td>Wing-wall</td>
<td>0.75</td>
</tr>
<tr>
<td>Spill-through (H:V=0.5:1)</td>
<td>0.6</td>
</tr>
<tr>
<td>Spill-through (H:V=1:1)</td>
<td>0.5</td>
</tr>
<tr>
<td>Spill-through (H:V=1.5:1)</td>
<td>0.45</td>
</tr>
</tbody>
</table>

$K_\theta^*$ is defined in the following:

$$K_\theta^* = K_\theta$$  \hspace{1cm} \text{for } \frac{L_a}{y} \geq 3 \hspace{1cm} \text{Equation 2.47 (a)}

$$K_\theta^* = K_\theta + (1 - K_\theta) \left(1.5 - 0.5 \frac{L_a}{y}\right)$$  \hspace{1cm} \text{for } 1 < \frac{L_a}{y} < 3 \hspace{1cm} \text{Equation 2.47 (b)}

$$K_\theta^* = 1.0$$  \hspace{1cm} \text{for } \frac{L_a}{y} \leq 1 \hspace{1cm} \text{Equation 2.47(c)}

in which $K_\theta$ is given in Table 2.3, and $\theta$ is the angle between the axis of the abutment and the flow direction.

Table 2.3 Abutment alignment factor (Melville and Coleman, 2000)

<table>
<thead>
<tr>
<th>$\theta$ (°)</th>
<th>Abutment pointing downstream</th>
<th>Abutment pointing upstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>0.9</td>
<td>120</td>
</tr>
<tr>
<td>45</td>
<td>0.95</td>
<td>135</td>
</tr>
<tr>
<td>60</td>
<td>0.98</td>
<td>150</td>
</tr>
</tbody>
</table>

The channel geometry factor, $K_G$, is defined as the ratio of abutment scour depth at a compound channel to that for the same abutment sitting in an equivalent rectangular channel.

For abutments in rectangular channels, or in compound channels with the abutment set well back from the main channel (Case A and Case C in Figure 2.5), $K_G=1.0$.

For $B_f/L_a \leq 1.0$, including Cases B and D abutments (Figure 2.5), $K_G$ was defined as the following:
in which $B_f$ is the width of the floodplain, $n$ is Manning roughness coefficient, and subscripts “$f$” and “$m$” denote floodplain and main channel, respectively. Equation 2.48 was derived by Melville (1995) by assuming that 

$$d_s \propto \sqrt{\sum A_j (V_j/V)}$$

for intermediate length abutments, in which $A_j$ is the subarea of the flow area obstructed by abutments, $V_j$ is the mean velocity in subarea $A_j$ and $V$ is the mean velocity in the whole flow area $A$.

For $B_f/L_a > 1.0$ in compound channels, Melville and Coleman (2000) suggested that $G$ could be conservatively estimated by interpolation between Case C and Case D. Alternatively, Melville et al. (2006) proposed Equations 2.49 and 2.50, to predict local scour depth for intermediate and short setback abutments:

$$d_{s-FP} = d_s + F_s (y_m - y_f)$$

Equation 2.49

$$F_s = 1 - \left(1 - \frac{L_a}{B_f} \right)^{2(a/B_f - 1)}, \quad \frac{a}{B_f} > 1$$

Equation 2.50

in which $d_{s-FP}$ is the scour depth measured from the un-scoured floodplain bed level, $d_s$ is the scour depth measured from the un-scoured bed level, $F_s$ accounts for the effects of abutment length and, the position and the dimension of the scour hole on scour depth; the term “$y_m - y_f$” denotes the elevation difference between the main channel and the floodplain (this term is associated with the behaviour of the main channel bank); and $a$ is the distance from the channel side wall (the floodplain side) to the outer edge of the scour hole. It should be noted that Equations 2.49 and 2.50 were derived for spill-through abutments.

Sturm and Janjua (1994) investigated clear-water abutment scour in a compound channel. Through dimensional analysis, they proposed the following relationship between the normalized equilibrium scour depth, \( \frac{d_s}{y_{f1}} \), and the relevant dimensionless variables:

\[
\frac{d_s}{y_{f1}} = f \left[ \frac{V_{f1}}{\sqrt{g y_{f1}}}, \frac{d_{50}}{y_{f1}}, m, \frac{B_m}{B_f}, \frac{B_f}{y_{f1}}, \frac{y_{ml}}{y_{f1}} \right]
\]

Equation 2.51

in which \( V_{f1} \) and \( y_{f1} \) are the mean velocity and the flow depth in the approach section of the floodplain, respectively, \( d_{50} \) is the median sediment grain size, \( m \) is the geometric contraction ratio (depending on \( L_a \) and channel width, \( B \)), \( B_m \) and \( B_f \) are the widths of the main channel and the floodplain, respectively, and \( y_{ml} \) is the flow depth in the approach section of the main channel. It should be noted that Equation 2.51 does not include the effects of abutment alignment, abutment shape, or nonuniformity of sediment on scour depth. They argued that \( m \) is inappropriate for a compound channel, because the flow distribution is non-uniform across the channel. They suggested replacing \( m \) with the discharge contraction ratio \( M = (Q - Q_{obst})/Q \), in which \( Q_{obst} \) is the portion of discharge obstructed by an equivalent length of the embankment in the approach section (Figure 2.25), and \( Q \) is the total discharge. They observed that \( M \) is influenced by \( B_m/B_f \), \( B_f/y_{f1} \), \( y_{ml}/y_{f1} \) and \( m \), and that \( d_{50}/y_{f1} \) could be replaced with a critical Froude number for initiation of motion (Jain 1981). They simplified Equation 2.51 to the following:

\[
\frac{d_s}{y_{f1}} = f_2[F_{r1}, F_{rc}, M]
\]

Equation 2.52

in which \( F_{r1} \) is the Froude number in the approach section of the floodplain, and \( F_{rc} \) is the critical value of \( F_{r1} \). Assuming uniform flow occurs on the floodplain, the estimation of \( M \) is defined in the following:
\[ M = \frac{Q_{m2} + Q_{f2}}{Q_{m1} + Q_{f1r}} = \frac{q_{f1}}{q_{f2}} \left( 1 + \frac{Q_{m1}}{Q_{f1r}} \right) \left( 1 + \frac{Q_{m2}}{Q_{f2}} \right) \]  

Equation 2.53

in which \( q_{f1} \) and \( q_{f2} \) are the width-averaged unit discharges on the floodplain (in the approach section and in the bridge section, respectively), and \( Q_{f1r} \) is the unobstructed discharge on the floodplain in the approach section. Considering that a very small depth change occurs in the bridge section, and that the ratio of main channel to floodplain discharge is mainly dependent on flow depth, values of \( \frac{Q_{m1}}{Q_{f1r}} \) are assumed to be approximately equal to \( \frac{Q_{m2}}{Q_{f2}} \). Therefore,

\[ M \approx \frac{q_{f1}}{q_{f2}} \]  

Equation 2.54

In their study, the calculated values of \( M \) are slightly larger than the measured \( \frac{q_{f1}}{q_{f2}} \) values.

The best-fit relationship for their experimental data is expressed in the following:

\[ \frac{d_s}{y_{f1}} = 7.70 \left( \frac{Fr}{MFr_c} - 0.35 \right) \]  

Equation 2.55

It should be noted that Equation 2.55 was obtained for an erodible floodplain and a fixed main channel. Scour developing in an erodible main channel, and the interaction of scour holes in the floodplain and in the main channel, are both expected to affect the maximum scour depth.
Sturm (2006) extensively investigated scour cases C and D (see Figure 2.5). Experimental conditions in his study comprised a matrix of 2 channel geometries, 2 abutment types, 3 uniform sediments and varying values of flow intensity, flow depth, and abutment length. Through dimensional analysis, they proposed the following relationship (similar to Equation 2.52):

$$
\frac{d_s}{y_{f0}} = f \left[ \frac{y_{f1}}{y_{f0}}, Fr_t, Fr_c, M, \frac{V_{e0}t}{y_{f0}} \right]
$$

Equation 2.56

in which $y_{f0}$ is the unobstructed flow depth on the floodplain, $V_{e0}$ is the critical velocity for $y_{f0}$, $Fr_c = V_{e0}/\sqrt{gy_{f0}}$ (following Pagan-Ortiz,1991), and $t$ is the scour time. Equation 2.56 considers the effects of backwater ($y_{f1}/y_{f0}$) and time ($V_{e0}t/y_{f0}$) on abutment scour. It
should be noted that the definition of $Fr_c$ is slightly different between Equations 2.52 and 2.56, and the relationship between them is given by the following:

$$\frac{Fr_f}{Fr_c} = \frac{V_{f_1}}{V_{c_0}} \sqrt{\frac{y_{f_0}}{y_{f_1}}}$$  \hspace{1cm} \text{Equation 2.57}$$

Combining $\frac{y_{f_1}}{y_{f_0}}, Fr_i, Fr_c, M$ into one parameter, $\frac{q_{f_1}}{Mq_{f_0c}}$, yields:

$$\frac{d_s}{y_{f_0}} = f_2 \left[ \frac{q_{f_1}}{Mq_{f_0c}}, \frac{V_{c_0}t}{y_{f_0}} \right]$$  \hspace{1cm} \text{Equation 2.58}$$

in which $q_{f_1} = V_{f_1}y_{f_1}$ and $q_{f_0c} = V_{c_0}y_{f_0}$. Here, it is assumed that $\frac{y_{f_0}}{y_{f_1}} \approx 1.0$.

At the equilibrium state, modifying the long contraction theory of Laursen (1963), and expressing the critical velocity in terms of Manning’s and Strickler’s equations yield the following equation:

$$\frac{y_{f_2}}{y_{f_0}} = \left( \frac{q_{f_2}}{V_{c_0}y_{f_0}} \right)^{6/7}$$  \hspace{1cm} \text{Equation 2.59}$$

Combining Equation 2.59 with Equation 2.54 (Sturm and Janjua, 1994) yields:

$$\frac{d_s}{y_{f_0}} + 1 = (\xi)^{6/7}$$  \hspace{1cm} \text{Equation 2.60}$$

in which $\xi = \frac{q_{f_1}}{MV_{c_0}y_{f_0}}$.

Sturm found that the best-fit relationship for his experimental data is the following:

$$\frac{d_s}{y_{f_0}} = 8.1K_s (\xi - 0.4)$$  \hspace{1cm} \text{Equation 2.61}$$

in which $K_s$ is an abutment shape factor, and

$$K_s = 1.52 \frac{\xi - 0.67}{\xi - 0.40} \text{ for } 0.67 \leq \xi \leq 1.2$$
Different values of $K_s$ may be used for different studies. It should be noted that in moving from Equation 2.60 to Equation 2.61, the exponent increased from 6/7 to 1. This modification is made by postulating that local scour is a constant amplification of the theoretical contraction scour. This constant amplification concept was previously proposed by Chang and Davis (1998, 1999), and was later developed by Ettema et al. (2010).

**ABSCOUR-Chang and Davis (1998, 1999)**

The scour estimation method, ABSCOUR, was first incorporated in a Microsoft Excel spreadsheet for both clear-water and live-bed flow conditions, and ABSCOUR was further developed by MSHA (2010). ABSCOUR modified Laursen’s long contraction theories (1960, 1963), with considerations of the effects of non-uniform flow distributions and the associated higher velocities and turbulences in the bridge section. In effect, ABSCOUR treats scour around an abutment as an amplification of long contraction scour.

For live-bed flows

\[ y_2 = K_\theta K_S K_f K_v \eta y_c = K_\theta K_S K_f \left( K_v \frac{q_2}{q_1} \right)^\eta \]  

Equation 2.62

in which $y_2$ and $y_c$ are the flow depths at the dynamic equilibrium state and in a long contraction, respectively, $K_\theta$ is the abutment skewness factor, $K_S$ is the abutment shape factor ($K_S = 1.0$ for vertical-wall abutment, $K_S = 0.55$ for spill-through abutment, $K_S = 0.82$ for wing-wall abutment), $K_f$ is an adjustment factor for spiral flow at the abutment toe ($K_f = 0.35 + 3.2 Fr_i$, where $Fr_i$ is the Froude number in the approach section and $Fr_i$ depends on the transverse location of the abutment), $K_v$ is an adjustment factor for velocity ($K_v = 0.8 (q_1/q_2)^{1/5} + 1; K_v = 1.8$ if the calculated value of $K_v > 1.8$), $q_1$ and $q_2$ are the unit
discharges in the approach section and in the bridge section, respectively, and the exponent $\eta$ is an experimentally derived constant related to sediment transport rate

$$\eta = 0.11\left(\tau_c/\tau_t + 0.4\right)^{2.2} + 0.623,$$

in which $\tau_c$ is the critical shear stress ($N/m^2$) and $\tau_t$ is the boundary shear stress in the approach section ($N/m^2$).

For clear-water flows

$$y_2 = K_f K_s K_p \eta y_c$$

Equation 2.63

in which $K_p = 0.8 \left( B_2/B_1 \right)^{1.5} + 1$ ($B_1$ and $B_2$ are the flow width in the approach section and in the bridge section, respectively), $K_f = 0.1 + 4.5 Fr_t$, the exponent $\eta = 0.857$, and $y_c$ is given by:

$$y_c = \left[ q_2/6.35(d_{50})^{1/3} \right]^{0.36}$$

for $d_{50} \geq 0.03$-m

$$y_c = \left[ q_2/4.16(d_{50})^{1/4} \right]^{0.5}$$

and $\zeta = \frac{1}{1 + \frac{1}{8d_{50}^{0.18}}}$

for $0.0003$-m $\leq d_{50} \leq 0.03$-m

$$y_c = 1.49q_2^{0.67}$$

for $d_{50} \leq 0.0003$-m

**Ettema et al. (2010)**

The major contributions of Ettema et al. (2010) to the knowledge of abutment scour are presented in the following:

1. **Practical considerations of the construction features of spill-through and wing-wall abutments in physical modelling.** In practice, a spill-through abutment usually comprises a concrete stub supported by a pile cap on two rows of circular piles (Figure 2.26a); a wing-wall abutment usually has similar foundations as a spill-through abutment, except that it has wing-walls extending from the central stub (Figure 2.26b); the abutment structure usually adjoins an earthfill approach
embankment, and the embankment is commonly constructed with H:V=2:1 side slopes.

2. *Effects of the erodibility of the floodplain and the embankment on abutment scour.*

Geotechnical failures occur because of the collapse of the main channel bank in Scour Condition A, and because of undermining of the earthfill embankment in Scour Condition B.

3. *Simulating Scour Condition C (when the abutment column is fully exposed).* A breached embankment is found to have similar scour processes to those at a pier.

4. *Scour depth design curves for different scour conditions (Figures 2.27 and 2.28).* In these figures, $L_a$ is the abutment length, $B$ is the channel width, $y_{\text{max}}$ is the maximum water depth at the equilibrium state, $y_c$ is the contraction scour depth, $r_{\text{A}}$ is the amplification factor for Scour Condition A, $H$ is the bed form height, $q_1$ and $q_2$ are the width-averaged unit discharges in the approach section and the bridge section, respectively, $y_f$ is the flow depth on the floodplain and $B_f$ is the width of the floodplain.

5. *Consideration of scale effects on frequency and vortices of large-scale turbulence structures, and subsequently on pier and abutment scour.* Results show that the normalized scour depth increases as the pier diameter decreases.

And most importantly,

6. *Viewing abutment scour as an amplification of long contraction scour, rather than a linear combination of contraction and local scour depths.* They proposed the following:
Equation 2.64 has the desirable attribute of reflecting the mechanisms of both scour due to physical contraction and scour due to large-scale turbulent structures.

Figure 2.26 Abutment models with practical considerations; a. a spill-through abutment and b. a wing-wall abutment (reproduced from Ettema et al. 2010)
Figure 2.27 Design curves for abutments subject to Scour Condition A, with fixed floodplains; a. for spill-through abutments and b. for wing-wall abutments (reproduced from Ettema et al. 2010)
Figure 2.28 Design curves for abutments subject to Scour Condition B; a. for spill-through abutments and b. for wing-wall abutments (reproduced from Ettema et al. 2010)
2.11 Summary

Scour around abutments has been extensively investigated since Straub presented his long contraction theory in 1934. Besides an increase in knowledge of abutment scour depth, there has been an expansion in knowledge of abutment-ambient flow features, scour countermeasures, and scour features for different flow and geotechnical environments. The following are well-accepted:

1. Local scour and contraction scour develop simultaneously and interactively around abutments.
2. An abutment imposes a short contraction on the flow, and the associated turbulence structures significantly affect scour development.
3. Abutment scour should be treated as an amplification of long contraction scour depth.
4. Use of scour countermeasures can significantly alter the scour development and the final scour depth. A principal vortex system and the associated downflow dominate the scour process for unprotected abutments; whereas large-scale turbulence structures dominate the scour process for protected abutments.
5. Compared with the available field data, estimation of scour depth from the results of physical modelling may be overly conservative, particularly for estimates from models of unprotected and rigid abutments situated in simplified rectangular channels.

The following outlines the current research gaps, which are also the research focuses of this PhD project:

1. Abutment scour under submerged orifice and overtopping flows.
2. Live-bed scour in compound channels, particularly for compound channels with equivalent erodibility of the main channel and the floodplain, and the corresponding characteristics of bed forms around the abutment.
3. Three-dimensional flow and turbulence features with and without vertical contractions.

4. Understanding of scour features for clear-water and live-bed scour conditions, and practical applications of this understanding.

5. The temporal development of scour for protected abutments, under clear-water and live-bed conditions.

6. The effect of contraction length on abutment scour.

7. The effect of scour countermeasures on short and long contraction scour.
Chapter 3. METHODOLOGY

3.1 Introduction

All the abutment scour experiments in this project were carried out in the Hydraulic Engineering Laboratory of the Department of Civil and Environmental Engineering, University of Auckland, Auckland, New Zealand. The following experiments were carried out:

1. A series of live-bed (including critical flows) scour experiments was carried out in a 1.54-m wide model compound channel. The test models were based on a prototype two-lane bridge with spill-through abutments, and were built at a 1:45 geometric scale (sediments not scaled in this PhD project). Flow measurements were carried out in detail at the initial state of scour (the state when scour is about to commence under the designed flow) for all the corresponding scour experiments. FS, SO and OT flows were investigated.

2. A series of clear-water scour experiments was carried out in a 2.40-m wide model compound channel. The test models were based on a prototype two-lane bridge with spill-through and wing-wall abutments, and were built at a 1:30 geometric scale. Flow measurements were taken at the initial state of scour for long setback spill-through abutments. FS, SO and OT flows were investigated.

3. A series of live-bed scour experiments was carried out in a 2.40-m wide model compound channel. The test models were based on a prototype two-lane bridge with spill-through abutments and were built at a 1:30 geometric scale. FS, SO and OT flows were investigated.

4. A series of clear-water long-contraction scour experiments was carried out in a 1.54-m wide flume: eight scour experiments (without countermeasures) in a 1.54-m wide
model rectangular channel, and ten scour experiments (with and without countermeasures) in a 1.54-m wide model compound channel. Flow measurements were taken at the initial state of scour for two experiments in the 1.54-m wide model compound channel. Only vertical-wall abutments were used. Only FS flow was investigated.

The following were the aims of the experiments described in the items above:

- Experiments described in items 2 and 3 were designed to better understand the effect of scour regime on abutment scour.
- For experiments described in item 1, flow measurements, particularly the near-bottom measurements, were designed to provide insight into the flow interaction at the contracted section of a compound channel, and insight into the development of scour at different sub-sections of the channel.
- Experiments described in item 4 were designed to better understand the effect of contraction length and the effect of countermeasures on long contraction scour.

The temporal development of scour was investigated. For the live-bed scour experiments, the temporal development of scour at several suitable positions was measured during the experiment; for the clear-water scour experiments, the bed bathymetry of the whole test section was measured at selected time intervals throughout the experiment.

### 3.2 Live-bed Experiments in the 1.54-m wide channel

Including the preliminary experiments, 26 live-bed scour experiments and 22 flow-measurement experiments (at the initial state of scour) were carried out. For these experiments, live-bed or critical flow conditions prevailed in the main channel, and clear-water flow conditions prevailed on the floodplain.
3.2.1 Flume

A 1.54-m wide, 1.2-m deep and 45-m long sediment recirculating flume was used. This flume has a rectangular cross section, and consists of an 8-m long inlet section (Figure 3.1a), a 30-m long channel and a 7-m long outlet section (Figure 3.1b). The flume is supported on two castellated-beams, and centrally pivoted, allowing ready adjustment of the flume slope. For the purpose of scour experiments, a 4.6-m long test section (recess section) was located at about 26-m downstream from the inlet section. Except for the test section area, a false floor 0.4-m in height was paved along the length of the channel (Figure 3.2). The flume is equipped with a 30-KW sand pump (100-mm diameter pipeline), a 22-KW water pump (250-mm diameter pipeline) and a 45-KW water pump (300-mm diameter pipeline). In this study, the sand pump was used to recirculate all the slurry, and the 22-KW water pump was used to recirculate the rest of the water. Each pump is controlled by a variable speed controller, providing the designed flow rate in the flume. It should be noted that the inlet is divided into three sub-inlets (Figure 3.1a), and the slurry enters into the flume from the top sub-inlet. Clear water for filling the flume enters the flume at the downstream end of the flume (Figure 3.2), and the whole flume can be drained via a pipe at the bottom of the flume. Both sides of the flume have glass walls, allowing clear observation of the experiment. The sediment sump at the downstream end of the flume is deep enough to trap most of the moving sediment.

Figure 3.1 Initial layout for the 1.54-m wide flume a. inlet section b. outlet section
Figure 3.2 Longitudinal section view of the 1.54-m wide flume (compound channel included)
3.2.2 The model compound channel

A 1.08-m wide floodplain, a 0.26-m wide main channel bank (H:V=2:1) and a 0.2-m wide main channel were built along the flume (Figures 3.3, 3.4a and 3.4b). Outside the test section, the floodplain was constructed 300-mm above the false floor using concrete blocks. The main channel bank was constructed from bent galvanized steel sheets, nailed onto the capping blocks in the floodplain. The steel sheets were bolted together, and were coated with the same sediment as in the main channel to simulate the roughness of a sandy bed. Heavy lead blocks and cement grout were used to stabilize the junction wall between the main channel and the floodplain, and cement grout was used to prevent sediment from moving from the sandy main channel into the semi-hollow floodplain base. For the scour experiments, the test section and the main channel were filled with uniform sand. The pre-scour bottom of the main channel was filled with a 170-mm depth of sand. This 170-mm depth of sand allowed full development of bed forms in the main channel for the flow depths in this live-bed scour study.

On the floodplain, both upstream and downstream of the test section, uniform rocks ($D_{50} \approx 16$-mm, Figure 3.5) were used to simulate vegetation and to adjust the flow distribution across the channel. The distribution density of the roughness rocks was selected by trial and error. The inlet section was modified for the compound channel geometry (Figure 3.6), to ensure all sediment slurry was guided into the main channel. Also, wave skimmers and PVC-pipe flow straighteners were used to smooth the flow and to reduce the turbulence generated at the entrance of the flume.

To measure the flow fields at the initial state of scour, the sand areas in the flume (that is, the whole length of the main channel and the whole test section) were immobilized using galvanized steel sheets, coated with the same sediment as above. All the gaps between the
steel sheets and between the steel sheets and glass walls were filled with polyurethane filler, which dried out in about a week. After immobilizing the sand area, flow measurements were taken along the flume. Results showed that uniform flows were achievable providing that the distribution density of roughness rocks and flow-specific channel slopes were well chosen. For the subsequent experiments, suitable channel slopes were used and uniform flows were obtained.

**Figure 3.3** Cross section view of the LB 1.54-m wide channel; unit: mm.

**Figure 3.4** The model compound channel in the LB 1.54-m wide flume a. model building in progress b. the completed model compound channel with an immobilized test section
Figure 3.5 Uniform roughness rocks outside the test section in the LB 1.54-m wide flume

Figure 3.6 The modified inlet for the LB 1.54-m wide compound channel; a. without flow and b. with flow
3.2.3 The model spill-through abutment and the model bridge deck

In practice, a spill-through abutment usually comprises a stub supported by a pile cap on circular piles; the approach embankment is usually earth-filled; the side-slopes of earthfill approach embankments and the spill-through slope commonly are H:V=2:1. In this project, the model abutment was constructed similarly to an abutment in practice, except for the abutment dimensions. Figure 3.7 shows the model abutment stub used in this flume. The criterion of abutment failure in this project is defined as when erosion reaches the toe of the abutment. It was unnecessary to use an erodible abutment because embankment failure was not allowed, and therefore rigid embankments were used. Three abutment lengths, termed long setback abutment (LSA, Figure 3.8a), medium setback abutment (MSA, Figure 3.8b) and short setback abutment (SSA, Figure 3.8c), were investigated.

Figure 3.7 Abutment stub model for spill-through abutments in the LB 1.54-m wide flume
Figure 3.8 Spill-through abutment models in the LB 1.54-m wide flume  

- **a.** long setback abutment \( \left( \frac{L_u}{B_f} = 0.5 \right) \)
- **b.** medium setback abutment \( \left( \frac{L_u}{B_f} = 0.65 \right) \)
- **c.** short setback abutment \( \left( \frac{L_u}{B_f} = 0.8 \right) \);

\( L_u \) is the length of the abutment and \( B_f \) is the width of the floodplain.

Two-lane bridges are widely used in relatively remote areas. A typical two-lane bridge has the following dimensions (model dimensions in brackets): the overall width including the road shoulders is 12.04-m (270-mm (= 12.04-m/45)); the bridge deck slab thickness is 0.46-m (10-mm); the bridge barrier width is 0.46-m (10-mm), and its height is 0.61-m (15-mm); five girders are evenly placed, with width and height of each girder being 0.46-m (10-mm). The above prototype bridge dimensions are commonly used by the Illinois, Iowa, New York and Georgia DOTs (Department of Transportation) in the U.S. The model bridge deck is shown in Figures 3.9 and 3.10.
3.2.4 Sediment and rock riprap apron

3.2.4.1 Sediment

Using a 2-mm mesh size sieve, all the sediment was filtered to remove contaminants (small gravels, etc.) prior to the experiments. Two batches of sediment were used and the gradation curves for both sieve analyses are shown in Figure 3.11. Properties for the sediment are as following: $d_{16} = 0.65$-mm, $d_{30} = 0.73$-mm, $d_{50} = 0.84$-mm, $d_{84} = 1.10$-mm, $\sigma = \sqrt{d_{84}/d_{16}} = 1.35$. Sediment used in this study is considered to be uniform in size because the geometric
standard deviation of sediment $\sigma_g < 1.50$. It should be noted that these two batches of sediment were used for all the experiments in this project.

The value of flow velocity for incipient motion of sediment particles in fully turbulent flow is calculated using the following equation from Melville and Coleman (2000):

\[
    u_c^* = 0.0115 + 0.0125d^{1.4}_{50} \quad 0.1\text{-mm}<d_{50}<1.0\text{-mm} \quad \text{Equation 3.1}
\]

\[
    \frac{V_c}{u_c} = 5.75 \log(5.53 \frac{y}{d_{50}}) \quad 0.1\text{-mm}<d_{50}<1.0\text{-mm} \quad \text{Equation 3.2}
\]

in which $u_c^*$ is the critical shear velocity, and $V_c$ is the critical velocity.

**Figure 3.11** The gradation curves for the sand used in this project
3.2.4.2 Rock riprap apron

A rock riprap apron was used as a scour countermeasure in this project.

Size of the rock riprap

For different scour experiments, different sizes of rock riprap were used for scour protection. The following equation, from Hydraulic Engineering Circular No. 23 (HEC-23) (Lagasse et al. 2009), was used as the size selection criterion:

\[ \frac{D_{50}}{y} = \frac{K_s}{(SG - 1)} Fr^2 \quad Fr \leq 0.8 \quad \text{Equation 3.3a} \]

\[ \frac{D_{50}}{y} = \frac{K_s}{(SG - 1)} Fr^{0.28} \quad Fr > 0.8 \quad \text{Equation 3.3b} \]

Definition of each parameter can be found in Chapter 2. For OT flows, the overtopping discharge was not included for the determination of Fr. Based on the flow features in this project, five size groups of rock riprap were used: 8-13mm; 13-16mm; 16-19mm; 19-25mm; 25-31mm. For some experiments, mixing different groups of rock riprap was required. A few examples of rock riprap sizes are shown in Table 3.1.

<table>
<thead>
<tr>
<th>Exp.</th>
<th>Q_{submerged} (cm^3/s)</th>
<th>A_2 (cm^2)</th>
<th>V_2 (cm/s)</th>
<th>SBR</th>
<th>Fr</th>
<th>K_s</th>
<th>Calculated D_{50} (mm)</th>
<th>Used rock riprap size</th>
</tr>
</thead>
<tbody>
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<td>867</td>
<td>41.6</td>
<td>1.68</td>
<td>0.37</td>
<td>0.89</td>
<td>9.5</td>
<td>8-13mm</td>
</tr>
<tr>
<td>2</td>
<td>56752</td>
<td>911</td>
<td>62.3</td>
<td>1.68</td>
<td>0.69</td>
<td>0.89</td>
<td>15.5</td>
<td>13-16mm</td>
</tr>
<tr>
<td>3</td>
<td>52679</td>
<td>911</td>
<td>57.8</td>
<td>1.68</td>
<td>0.72</td>
<td>0.89</td>
<td>18.1</td>
<td>16-19mm</td>
</tr>
<tr>
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<td>55820</td>
<td>847</td>
<td>65.9</td>
<td>1.72</td>
<td>0.88</td>
<td>0.61</td>
<td>20.3</td>
<td>19-25mm</td>
</tr>
<tr>
<td>5</td>
<td>77530</td>
<td>911</td>
<td>85.1</td>
<td>1.68</td>
<td>1.16</td>
<td>0.61</td>
<td>23.1</td>
<td>19-25mm, 25-31mm (1:1)</td>
</tr>
<tr>
<td>6</td>
<td>71470</td>
<td>911</td>
<td>78.5</td>
<td>1.68</td>
<td>1.07</td>
<td>0.61</td>
<td>22.6</td>
<td>19-25mm, 25-31mm (1:1)</td>
</tr>
</tbody>
</table>

Note: Q_{submerged} is the discharge under the bridge deck, and definitions of other parameters are the same as those in Chapter 2.
Layout of the apron

The layout of the rock riprap apron was decided following the suggestion of Lagasse et al. (2006), as shown in Figures 3.12 and 3.13. Because the geometric ratio is 1:45, the required apron extent from the toe towards the bridge waterway is:

Minimum {twice the flow depth on the floodplain near the embankment, 170-mm}.

At the downstream side of the abutment, the required apron extent from the abutment back along the embankment is:

Maximum {twice the flow depth on the floodplain near the embankment, 170-mm}

It should be noted that, geotextile was not used in this flume because, in live-bed flow, sand dunes could expose the geotextile and cause geotextile failure of the apron.

Figure 3.12 Plan view of the extent of rock riprap apron (reproduced from Lagasse et al. 2006)
3.2.5 Measurement equipment

Scour depth, scour history and bed bathymetry measurements were carried out using ultrasonic depth sounders. Velocity and turbulence measurements were carried out using a side-looking and a down-looking ADV (Nortek ADV). Water surface profile measurements were carried out using a point gauge.

3.2.5.1 Ultrasonic depth sounder

An ultrasonic depth sounder comprises an electric pulse generator, an ultrasonic transducer and a signal processor. The electric pulse generator generates a high-frequency electric current, and transmits the current to the transducer. The ultrasonic transducer converts the electric current into ultrasound, and vice versa. The basic working mechanism is: piezoelectric crystals change size and shape when a voltage is applied; AC voltage makes them oscillate and produce ultrasonic sound.

The time \( t \) for the sound waves to travel through the water (sound wave generation and return) is recorded, and the distance \( d \) from the sensor to the reflective object is given by:
\[ d = \frac{c_s t}{2} \]

in which \( c_s \) is the speed of sound in water, which is nearly constant because the water temperature in the laboratory was always about 20°C. The measuring precision of the probe is \( \pm 0.5\text{-mm} \).

Ultrasonic depth sounders can be used in two ways:

1. For scour history measurement, a depth sounder was positioned vertically (verified by a spirit level) at a fixed location, and the scour depth measurement was made at adjustable frequencies. Multiple data-acquisition channels were available, thus multiple locations could be measured simultaneously using multiple sounder probes. The depth data were recorded using software \textit{DS2006 v21}.

2. For bed bathymetry measurements, transverse bed profiles were measured every 50 – 100-mm along the length of the test section. A depth sounder was mounted on a trolley, which could be moved transversally across the flume. A digital potentiometer (attached to wheel on the trolley) tracked the transverse position of the depth sounder. Measurements were made every 1.96-mm along each transverse profile.

### 3.2.5.2 Vectrino+ acoustic Doppler velocimeter

The applicability of ADV measurements was evaluated by Voulgaris and Trowbridge (1998). Two ADVs were used in this study, a side-looking Vectrino+ and a down-looking Vectrino+. Both ADVs are high-resolution and three-dimensional (four acoustic receivers) Doppler current meters. The side-looking Vectrino+ (Figure 3.14a) was mainly used for velocity measurements in shallow water, the top layer of water and the overtopping flow. Because the sampling volume is located at 50-mm away from the transmitter, the down-looking Vectrino+ (Figure 3.14b) was mainly used for velocity and near-bottom turbulence measurements in relatively deep water.
The underlying principles of these two ADVs are the same: the central transmitter sends out high frequency (10MHz) short pairs of sound pulses, and these sound pulses are partly reflected by the suspended moving particles in the sampling volume. The reflected sound pulses have Doppler frequency shifts, which are caused by the moving particles in the flow. These suspended moving particles are very small and move with the flow, so it is reasonable to assume that the particle velocity that is measured is the velocity of the surrounding flow. For each receiver, the frequency shifts are correlated with the beam velocity. After a matrix transformation, the point velocity (three dimensional in an orthogonal coordinate system) in the sampling volume is obtained.

The raw ADV data are normally contaminated by spikes and Doppler noises, and post-processing is essential to obtain acceptable data (Chanson 2008). The quality of the measurement is dependent on the presence of scatters and their behaviours within the sampling volume. The signal-to-noise ratio (SNR) and the correlation (COR) are two important data-quality parameters. Filtering the data using one or both of these parameters can improve the quality of the measurement (Wahl 2000). The raw data were post-processed using WinADV (Wahl, 2000) to remove spikes (using the phase-space threshold method of Goring and Nikora (2002)), and to remove the data with less than 70% signal COR and less than 15-dB SNR. It should be noted that three-dimensional flow information only requires three receivers. The extra information that is provided by a four-receiver probe can be used to reduce signal noises using the noise-reduction method proposed by Hurther and Lemmin (2001). More details of ADV data treatment are presented in Chapter 5.

During the experiments, the ADV was mounted on a screw-driven moving structure (Figure 3.15), which precisely controlled the vertical and horizontal positions of the ADV: each
screw rotation produces 1.5-mm of movement. In addition to the distance measuring ability of the ADV probe, the accurate three-dimensional location of the sampling volume (the measurement point) is attainable. Before the measurement, the following matters were considered:

1. No obstructions between the sensor and the sampling volume
2. As a measurement quality assurance, Probe Check (Vectrino user guide 2009) was carried out and the results (including the transmit pulse, the receive volume and the bottom echo) were compared with the standard graph in the manual.
3. The flow was well seeded, using spherical fused borosilicate glass (8-13 micron, 1.1-g/cc).
4. The transmit axis was perpendicular to the main flow direction; the probe stem was vertical to the horizon (checked by a spirit level).
5. The mounting structure was heavy enough (about 25kg) to reduce the vibration-caused accelerations in the data.

![Figure 3.14 Vectrino+ ADV](image)

**Figure 3.14** Vectrino+ ADV **a.** a side-looking probe **b.** a down-looking probe; during the experiment, the receiver with red band points to the flow
3.2.6 Experimental procedures

For each flow condition, flow-measurements were carried out over an immobilized test section and scour experiments were carried out over a mobile test section.

3.2.6.1 Preliminary experiments

Four preliminary experiments (LB_P1 to LB_P4) were carried out to find a suitable experimental methodology for the subsequent experiments. For these preliminary experiments, a medium setback abutment and FS flow condition were investigated. The purposes of these preliminary experiments included:

- Understanding the influences of main channel bank protection upstream of the bridge section
- Understanding the influences of main channel bank protection at the bridge section
- Selection of the size of the riprap and the dimensions of the apron
- Understanding the influences of erodibility of the embankment.

Figure 3.15 The mounting structure for ADV; the ADV was mounted on the vertical alloy beam
The experimental setup for LB_P1 is shown in Figure 3.16, and included the following: 1. one layer of riprap \( (D_{50} = 16\text{-mm}) \) was used around the abutment spill-through slope and the embankment slope; 2. one layer of capping blocks \( (390\text{-mm} \times 140\text{-mm} \times 40\text{-mm}) \) was used on the main channel bank and the edge of the floodplain, both upstream of and well downstream of the abutment; 3. one layer of truncated rectangular pyramid concrete blocks was used on both the floodplain and the main channel bank in the vicinity of the abutment. The truncated rectangular pyramid concrete blocks in the vicinity of the abutment were used to prevent erosion occurring on the main channel bank and at the edge of the floodplain. This erosion would have occurred in the absence of the abutment structure under the flow condition in this study. All the bank protection surfaces were at the levels of the pre-scoured channel cross section to avoid flow obstruction. It was observed that, though the concrete blocks were manually removed on a continuous basis when they became dislodged, they adversely affected the flow pattern and hence the scouring process, particularly when the bed forms migrated into the abutment section. Figure 3.17 shows that the spill-through slope was hydraulically eroded and the abutment stub was fully exposed because of embankment breaching, indicating that better protection around the abutment was required. In addition, the capping blocks downstream of the abutment were severely undermined, rendering the scour depth measurement in the contracted section unreliable.

LB_P2 was set with capping blocks upstream of the abutment section but without concrete blocks in the vicinity of and immediately downstream of the abutment, as shown in Figure - 3.18.
Figure 3.16 Experimental setup of test LB_P1, with capping blocks both upstream and downstream of the bridge section, and truncated rectangular pyramid concrete blocks at the bridge section; before scour, viewed from downstream.

Figure 3.17 Scour results of test LB_P1, with capping blocks both upstream and downstream of the bridge section, and truncated rectangular pyramid concrete blocks at the bridge section; after 72 hours, viewed from downstream.

The time sequence of scour photos in Figure 3.18 show that the embankment lost its integrity soon after the commencement of the experiment. After only a few minutes, initiation of
failure was obvious at the upstream corner of the abutment. Initiation of failure was caused by sediment being continuously winnowed out from the voids of the riprap at the upstream corner of the abutment. A similar failure procedure was detailed by Ettema et al. (2015). After 72 hours, the embankment was breached and the abutment stub was again fully exposed. Scour reached upstream to the capping blocks, and severe undermining occurred beneath them. The latter phenomenon was probably because the capping blocks were too large and not flexible enough.

Figure 3.18 Time sequences of scour in test LB_P2, with large capping blocks on the upstream side of the bridge section

LB_P3 was set up without any form of bank protection. The scour result for LB_P3 is shown in Figure 3.19. After 72 hours, though the spill-through slope failed again, the extent of the damage and the maximum scour depth were both much less than those for LB_P2 (see final bed bathymetries in Figure 3.20). It was observed that, in the absence of bank protection, erosion occurred rapidly on the main channel bank upstream of the abutment, leading to
decreased scour around the abutment. Results imply that the approach flow distribution altered rapidly as the main channel collapsed at the upstream side of the abutment, directly changing the flow pattern around the abutment, and eventually affecting the scour depth and the scour hole geometry.

Figure 3.19 Scour result of test LB_P3, without any forms of bank protection; a. view from above and b. view from downstream

Figure 3.20 Comparison of scour bathymetry of a. LB_P2 and b. LB_P3 (dark areas (negative values) denote scour, colour scale unit: m)

LB_P4 was set up with two layers of riprap on the spill-through slope of the abutment and on the embankment slope, and with a rock riprap apron around the abutment toe (Figure 3.21), all as recommended in HEC-23 (Lagasse et al. 2009). At the upstream side of the abutment, the capping blocks were replaced with mats of cable-tied small tiles, which allowed flexible deformation when scour developed around them. Results after 72 hours showed that the
abutment remained intact during the experiment and the cable-tied tiles deformed smoothly around the periphery of the scour hole (Figure 3.22).

Figure 3.21 Set-up of test LB_P4, with two layers of riprap rocks on the spill-through slope, and a riprap rock apron around the abutment toe; cable-tied tiles were used as the upstream bank protection. left figure: view from above; right figure: view from downstream

Figure 3.22 Scour result of test LB_P4, with intact abutment after 72 hours; left figure: view from above; right figure: view from downstream

The results of the preliminary experiments suggest that the main channel bank protection is necessary, to ensure the channel configuration immediately upstream of the abutment retains its shape in the developing phase of scour. Cable-tied tiles were used in all subsequent live-bed experiments to protect the main channel bank on the upstream side of the abutment. The results also suggest that embankment breaching occurred when incorrectly designed riprap
protection was used. In all subsequent experiments, the model size of riprap rocks and the dimension of the apron (both varying with flow conditions) were determined by the method recommended in HEC-23 (Lagasse et al. 2009). In addition, to avoid sediment particles being carried away at the interface of the bridge deck and the erodible embankment (particularly by the plunging jet of the OT flow), non-erodible embankments were used in all subsequent experiments.

3.2.6.2 The general procedure for scour experiments

For each formal experiment, sediment was levelled using a channel-shaped plywood former mounted on the carriage over the flume. After that, the abutment, the selected countermeasures, the bridge deck, the main channel bank protection (see Section 3.2.6.1) and ultrasonic depth sounders were positioned, as shown in Figure 3.23. The flume was then slowly filled with water, without disturbing the levelled bed. When the water level reached the desired elevation, the wave skimmer was adjusted for the flow depth and the channel slope was adjusted for the flow. Afterwards, the pump controller was activated and the discharge was slowly increased to the desired value over about a minute. After the water depth stabilised, the overflow pipe was adjusted as required to ensure that the flow depth in the channel did not change because of vaporization or leakage. The scour experiments typically last for 20-192 hours. Some experiments were run with a few more hours of post-equilibrium time to better observe the propagation of sand dunes. During the experiment, five ultrasonic depth sounders were positioned in the vicinity of the bridge section to record the scour history at different sub-sections of the channel. At the end of each scour experiment, the bed bathymetry was measured using one ultrasonic depth sounder. A camera (Nikon D90 with 8 - 12-mm wide angle lens) was mounted at the main-channel side of the flume, taking time lapse photos of the experiment through the glass wall of the flume. For the first two
hours, the photo recording frequency was 60-360 frames/hour, depending on the flow intensity; for the rest of the time, the recording frequency was 12 frames/hour.

**Figure 3.23** The experimental preparations in the test section of the LB 1.54-m wide compound channel at four stages of preparations (for FS flows, the bridge deck model was not used)

### 3.2.7 Flow measurements

Flow measurements were taken over an immobilized channel bed. Immobilizing the mobile section can be achieved by spraying polyurethane (Hong, 2012). Preliminary trials suggest that neither the dried-out polyurethane nor paving with steel sheets on top of the sediment bed provided a reliable measurement environment, because of unequal settlement over the 0.8-m thick sand bed in the test section. To overcome this unequal settlement problem, all the sand in the test section was removed. Concrete blocks were placed in the recess and galvanized steel sheets, bent to the compound channel shape, were placed on top (Figure-
3.24). Grooves for riprap were cut into the sheets for different abutment lengths. The steel sheet surface was coated with the experiment sand. The gaps between the steel sheets and between the steel sheet and the glass wall were filled with polyurethane filler, which dried out in about a week.

![Groove for riprap](image)

**Figure 3.24** Reconstruction of the test section in the LB 1.54-m wide channel for flow measurements: a. model construction in progress, view from downstream; b. completed model, view from top

After the completion of the channel bed, the construction quality was inspected, and the morphology of the immobilized bed was measured using an ultrasonic depth sounder. The measurements were taken with a spacing of 1.96-mm in the transverse direction and 100-mm in the streamwise direction. Results showed that the immobilized test section had about ± 5-mm unevenness, as shown in Figure 3.25. The bed condition around the abutment was further improved, after which it had about ± 2-mm unevenness.
Figure 3.25 Construction quality inspection for the test section of the LB 1.54-m wide channel. (a) three-dimensional side view (b) two-dimensional cross-sectional view: the narrow band shows the maximum elevation differences along the test section.

Flow measurements were carried out in detail at the approach section, at the bridge section (C.S.4 and C.S.5), and along the flume immediately downstream of the abutment (Figure 3.26). An example photo of flow measurement is shown in Figure 3.27.

C.S.1: For the first six experiments, at C.S.1, measurements were taken down multiple vertical profiles, which were laterally spaced by 50-100 mm in the main channel, and 100-200 mm on the floodplain. For each vertical profile in the main channel, typically 15-20
measurements were taken down the water depth; on the floodplain, 5-15 measurements were taken down the water depth. Each measurement took 2-minutes; the measurement duration was decided after a time sensitivity analysis (see Figure A8 in the Appendix). Depth-averaged velocity results suggest that, at the approach section, the point velocity at $Z = 0.4 \, y$ (measured from the bottom; the coordinate system is shown in Figure 3.26) well represents the depth-averaged velocity of the vertical profile. This is because a logarithmic vertical velocity distribution existed at the approach section. Therefore, for the rest of the experiments, only the measurement at $Z = 0.4 \, y$ was taken for each vertical profile. For each experiment, measurements at $Y = 100$-mm were taken every 1-mm down the vertical profile to determine the magnitude of the shear velocity in the main channel, $u^*_{\kappa}$. 

**C.S.4:** The vertical velocity distribution at the bridge section was not logarithmic. Therefore, at each vertical profile, multiple measurements were taken to obtain the depth-averaged velocity. Measurements were taken down multiple vertical profiles, which were laterally spaced by 50-mm in the main channel and on the floodplain. For the near-bottom measurements (5 and 10-mm above the bottom), each measurement took 5-minutes; for the rest of the water depths, each measurement took 2-minutes;

**C.S.5:** Only near-bottom measurements were taken, at every 50-mm laterally. Each measurement took 5-minutes.

*Along the flume at the downstream side of the abutment:* Only near-bottom measurements were taken, at 100-mm spacing along the flume, for five different lateral positions. Each measurement took 5-minutes.
Data quality analysis and flow-measurement results are presented in Chapter 5.

**Figure 3.26** Flow measurements for the LB 1.54-m wide channel. **a.** measurements for the short setback spill-through abutment and **b.** measurements for the long setback spill-through abutment (unit: mm)

**Figure 3.27** Flow measurements over an immobilized bed
3.3 Experiments in 2.4-m wide channels

It should be noted that two 2.4-m wide flumes, and accordingly two 2.4-m wide compound channels, were used. To distinguish between these two flumes, the following nomenclatures are used: the LB 2.4-m wide compound channel (used for live-bed experiments), and the CW 2.4-m wide compound channel (used for clear-water experiments). The main differences between the 1.54-m wide channel and the 2.4-m wide channels are the geometric ratio of the model and the width ratio of the main channel and the floodplain. For the experiments in the 2.4-m wide channel, the geometric ratio is 1:30, and $B_m/B_f = 0.5$. ($B_m$ is the width of the main channel, including the main channel bank).

The same compound channel configuration was used in the two 2.4-m wide flumes: a 1.6-m wide floodplain, a 0.2-m wide main channel bank (2:1) and a 0.6-m wide main channel (Figure 3.28). Outside the test section, the floodplain was constructed 270-mm above the false floor (using concrete blocks). To prevent sediment leaking into the semi-hollow floodplain area, L-shaped metal was installed both upstream and downstream of the flume (Figure 3.28). Along the full length of the flume, the main channel was filled with uniform sand to a depth of 170-mm above the false floor. As for the 1.54-m wide compound channel, the main channel banks upstream and downstream of the test section were constructed from galvanized steel sheets, coated with the same sand used in this study, to simulate the roughness of the sediment bed. Both upstream and downstream of the floodplain, uniform rocks ($D_{50} \approx 22$-mm) were used to simulate the vegetation, and to adjust the flow distribution across the channel.
All the experiments in the 2.4-m wide channels had a two-lane bridge deck model and one of several abutment models. The dimensions of the two-lane bridge deck model used in both 2.4-m wide channels (Figure 3.29) are:

- The overall width is 400-mm
- The slab depth of the bridge deck is 15-mm
- The bridge barrier width is 15-mm, and its height is 22-mm
- Five girders were evenly placed; the width and the height of each girder are 15-mm.

**Figure 3.28** Cross-sectional view of 2.4-m wide compound channels; unit: mm.

**Figure 3.29** The two-lane bridge deck model for both 2.4-m wide compound channels
The required apron extent from the toe towards the bridge waterway is:

Minimum \{twice the flow depth on the floodplain near the embankment, 255-mm\}

At the downstream side of the abutment, the required apron extent from the abutment back along the embankment is:

Maximum \{twice the flow depth on the floodplain near the embankment, 255-mm\}

Other than these dimension differences, the bed materials, the selection criterion of the size of the rock riprap, and experimental procedures were the same as described in Section 3.2.

The main differences between the two 2.4-m wide flumes are the following:

- Length of the flume
- Water recirculation system
- Sediment recirculation system.

These differences are explained below.

**3.3.1 Experiments in the LB 2.4-m wide channel**

Altogether, 9 scour experiments were carried out. For each experiment, a clear-water scour regime prevailed on the floodplain, and a live-bed scour regime prevailed in the main channel.

**3.3.1.1 Flume and the model compound channel**

A 2.4-m wide, 0.6-m deep and 25-m long sediment recirculating flume was used to conduct the scour experiments (Figures 3.30a and 3.30b). The flume is supported on two I-shape beams. The beams are supported upon two screw-jacks, and two pivot supports, allowing adjustment of the flume slope. The flume consists of a short inlet section (Figure 3.30c), 17-m long channel and a 7.6-m long outlet section (Figure 3.30d). A 4.0-m long, 0.6-m deep test section is located at about 11-m downstream from the inlet section, for the purpose of scour experiments. The flume is equipped with a 15-KW sand pump (80-mm diameter pipeline),
and two 55-KW water pumps (400-mm diameter pipeline), allowing a maximum of 1000L/s discharge in the flume. These pumps are controlled by variable speed controllers, providing the designed flow rate in the flume. The required modification of the inlet (Figure 3.30c) and the operating system of this flume are similar to those for the 1.54-m wide flume.

Figure 3.30 Photos for the LB 2.4-m wide flume: a. viewed from upstream; b. viewed from the true right side; c. view of the inlet (with PVC-pipe flow straightener); the top sub-inlet was modified for the compound channel; d. view of the outlet
3.3.1.2 The model spill-through abutment and model bridge deck

Two 1:30 scale model spill-through abutments (Figure 3.31) and a 1:30 scale model two-lane bridge deck were used in this live-bed study. Except for the dimension differences, the abutment was constructed similarly to that in the 1.54-m wide channel study.

![Diagram showing spill-through abutment dimensions for the LB 2.4-m wide flume](image)

**Figure 3.31** Spill-through abutment dimensions for the LB 2.4-m wide flume; a. \( L_a/B_f = 0.65 \); b. \( L_a/B_f = 0.80 \)
3.3.1.3 The general procedure for scour experiments

The general experimental procedures (shown in Figure 3.32.) are similar to those described in Section 3.2.

Figure 3.32 The general experimental procedures for the LB 2.4-m wide compound channel:

a. the test section was levelled by a channel-shaped plywood former; b. spill-through abutment and rock riprap aprons were placed on the floodplain; c. bridge deck was put on the top of the embankment, and then water slowly entered the flume from the downstream end; d. after all the preparation, the pumps were activated

3.3.1.4 Flow measurements

Approach section (C.S.1)

Flow measurements at \( Z = 0.4y \) (measured from the bottom, used as the depth-averaged velocity) were taken down multiple vertical profiles, which were laterally separated by 100-mm across the approach section. The approach section was located 3.0-m upstream of the
abutment. These measurements were taken about 1-hour after the commencement of each experiment, when the flow depth stabilized (in the beginning of a scour experiment, the flow depth at the approach section would drop to some extent because of the erosion at the bridge section).

**Bridge section**

The abutment was moved upstream to the location of C.S.1, to measure the flow distribution around the abutment over an immobilized channel bed, as shown in Figure 3.33. Around the abutment, the sand in the main channel was replaced by flow resistant gravels. Without this replacement, bed forms in the main channel could significantly affect flow measurements. These gravels were levelled to the initial main channel elevation, to ensure the measurement environment (mainly with respect to geometry) was similar to an un-scoured environment. During the measurement, flow-entrained sediments filled the voids of gravels, and the top layer of gravels developed into an armour layer. Roughness difference between a sandy bed and an armoured bed was assumed to be negligible for a short distance along the main channel. Moving the abutment upstream was only for time-averaged velocity measurements, because these measurements were not sensitive to slightly different bottom conditions.

![Image](image_url)

**Figure 3.33** The abutment was moved upstream into the immobilized area for flow measurements around the abutment in the LB 2.4-m wide channel

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3.3.2 Experiment in the CW 2.4-m wide channel

Altogether, 25 scour experiments and 7 flow-measurement experiments were carried out. For each experiment, a clear-water scour regime prevailed on the floodplain and in the main channel.

3.3.2.1 Flume and compound channel

A 2.4-m wide, 0.6-m deep and 16.5-m long flume was used to conduct the clear-water experiments. The flume is supported by two universal beams which pivot about a central support. Each universal beam is supported by a screw jack at each end, allowing adjustment of the flume slope. The flume consists of an inlet tank (with a baffle system and a wire mesh inside), a 13-m long channel, a sediment collection sump and a tailgate outlet. A PVC-pipe flow straightener was used at the upstream end to smooth the flow. Two indoor reservoirs (top and main) in the laboratory were used to recirculate the water for this flume. The top reservoir has a fixed water level that functioned as a constant head tank; the main reservoir is below the laboratory floor. The discharge into the flume is regulated using butterfly valves, allowing discharges from 0 to about 255-L/s. The flow rate in the flume was calculated using ADV measurements across the approach section. At the downstream end of the flume, a plywood weir was employed to control the water depth in the flume.

The flow path in this recirculating system is the following: the water flowed down from the constant head tank (top reservoir), into the inlet tank, through the baffle zone and the flow straightener, over the downstream weir and into the outlet of the flume; from the outlet, the water entered the main reservoir, and then was pumped up into the top reservoir.

To achieve uniform flow in the main channel and on the floodplain, pipe plugs and concrete blocks were used (by trial and error) at the upstream side of the flow straightener to regulate
the flow. Both upstream and downstream of the test section, uniform rocks ($D_{50} \approx 22$-mm) were used on the floodplain to further regulate the flow distribution across the flume.

As described previously, a compound channel was built along this 2.4-wide flume. Based on the experimental observations in the longer 1.54-m wide flume, the inertia of the flow keeps the majority of the discharge in the main channel zone for a distance of about 5-m downstream of the abutment. Therefore, a suitable size of opening was made in the plywood weir on the main channel side (Figure 3.34). This opening regulated the flow distribution downstream of the abutment, causing the majority of the flow to stay in the main channel after passing the abutment.

![Figure 3.34](image)

**Figure 3.34** Compound channel in the CW 2.4-m wide flume, and the opening in the plywood weir (marked by a red circle)

### 3.3.2.2 Bridge and abutment structure models

Two 1:30 scale model spill-through abutments, two 1:30 scale model wing-wall abutments (45° side wing angle) and a 1:30 scale model two-lane bridge deck were used in this clear-
water study (Figure 3.35). Two abutment lengths were investigated: $L_a/B_f = 0.80$ and $L_a/B_f = 0.50$. The detailed dimensions of the abutment models are shown in Figure 3.35.

![Diagram of abutment models](image)

**Figure 3.35** The dimensions for the abutment models in the CW 2.4-m wide compound channel; **a.** long setback spill-through abutments, $L_a/B_f = 0.50$; **b.** short setback spill-through abutments, $L_a/B_f = 0.80$; **c.** long setback wing-wall abutments, $L_a/B_f = 0.50$; **d.** short setback wing-wall abutments, $L_a/B_f = 0.80$

### 3.3.2.3 Experimental preparations

Because this flume is relatively short, moving the abutment upstream (as was done in the LB 2.4-m wide flume, see Section 3.3.1.3) would reduce the upstream length to 4.2-m, and this upstream length may have been too short to achieve uniform flow for a 2.4-m wide channel. For the flow measurements, the test section was immobilized using galvanized steel sheets as was done in the 1.54-m wide channel. To avoid unequal settlement over thick sand in the recess, PVC pipes of appropriate lengths were embedded into the sandy bed down to the
recess bottom, as shown in Figure 3.36. The general preparations for flow measurements are shown in Figure 3.37.

**Figure 3.36** PVC pipes were embedded into the sand to better support the structure, and to avoid unequal settlement of the structure over the sandy bed (CW 2.4-m wide compound channel)
Figure 3.37 Preparation for flow measurement in the CW 2.4-m wide compound channel: a. galvanized steel sheets, bent to the compound channel shape, were placed on top of the recess section; b. the steel sheets were coated with sand; c. a long setback abutment was positioned on the immobilized floodplain

3.3.2.4 Experimental procedures

During an experiment, the dimensions and the location of the scour hole changes over time because of the apron. To better understand the developing history of the whole scour hole, an overhead camera was used for FS flows. Moreover, for most of the experiments, the bed bathymetry of the whole test section was measured at selected time intervals using an ultrasonic depth sounder. Typically, seven sets of measurements (1-hour, 5-hour, 20-hour, 40-hour, 70-hour, 100-hour and 120-hour) were carried out for each experiment. In order to ensure that the sandy bed was not disturbed during the intermediate measurements, the discharge was lowered well below the required value, and a large aluminium sheet was placed immediately upstream of the plywood weir to impede the flow. The results of the scour history are presented in Chapter 6.
3.3.2.5 Flow measurements

Velocity measurements were taken at the approach section (C.S.1) and the bridge section (C.S.4), as shown in Figure 3.38. Each measurement took 2-minutes. For each vertical profile at C.S.1, the measurement at $Z = 0.4 \, y$ was taken to be depth-averaged velocity. For the long setback abutments (Figure 3.38a), multiple measurements were taken down each vertical profile at C.S.4 to obtain the unit-width discharge. Flow measurements in the 1.54-m wide channel suggested that nearly uniform velocity distribution occurred at the bridge section for experiments with short setback abutments. Therefore, for the short setback abutments ($L_s/B_f = 0.80$) in this clear-water study, only overtopping flow measurements were taken at C.S.4; the unit-width discharge at C.S.4 was evaluated using the measured discharge at C.S1. It was assumed that, at the bridge section, unit-width discharge values for the wing-wall abutment were the same as those for the spill-through abutment.
Figure 3.38 Flow measurements for the CW 2.4-m wide compound channel a. measurements for the long setback spill-through abutment and b. measurements for the short setback spill-through abutment

3.4 Long contraction experiments

The long contraction scour theory was firstly proposed by Straub (1934), and was further developed by Laursen (1963), based on a vertical-wall abutment model (wide and unprotected) extending across the floodplain of a compound channel. However, in reality, a bridge abutment generally imposes a short and abrupt contraction on the flow.

Different contraction lengths (short and long) can be found in reality in the form of abutments for narrow (one-lane) and wide (multiple-lane) bridges. To further investigate the effect of contraction length on abutment scour, experiments with varying abutment widths were carried out. The test model used in this long contraction study was designed to be representative of vertical-wall abutments set back from the main channel of a compound channel. For an abutment set well back from the main channel, a simplified channel configuration - a rectangular channel - was used (abutment scour Case A in Chapter 2). For
an abutment set back from the main channel but the abutment causing scour in the main channel, a compound channel configuration was used. For the experiments conducted with a compound channel, the effect of countermeasures was also briefly investigated. In addition, to better understand the effect of contraction length on the flow field in the vicinity of the abutment, a pair of contrasting flow-measurement experiments was carried out.

3.4.1 Flume

The 1.54-m wide flume was used for all the scour and flow-measurement experiments. Details of the flume are presented in Section 3.2.1.

3.4.2 Flow measurements

For the rectangular channel study, point velocity was measured using a Flow Tracker Handheld-ADV, which can measure velocities as low as 0.001-m/s. Each measurement took 2-5 minutes. The operating principle of the Flow Tracker Handheld-ADV is the same as that of the Vectrino+ ADV.

Particle tracking velocimetry (PTV) (Smits and Lim 2000) was used to obtain the surface flow fields. PTV is a flow visualisation technique designed for two-dimensional surface-flow-field measurements. The travel paths of the tracked particles are digitally recorded, allowing velocity estimates to be obtained. The coordinate value of each particle in frame P is compared with the coordinate value in frame P-1 and P+1, and the two-dimensional flow field is estimated from the position shift over time. The following PTV set-up was similarly used by Ballegooy (2005) and Khan (2012), and the measurement accuracy was verified by a side-looking ADV. The particles used in this study were uniform plastic balls \(D = 6.5\text{-mm}, \rho=500\text{-kg/m}^3\) and were painted fluorescent-yellow. The particles were illuminated under ultra-violet lights, to obtain strong light intensity for the camera (Figure 3.39). The seeding
density (by trial and error) was about 40–50 particles/m²; neither a too dense (particles may stick together and cluster along the flow) nor a too sparse (not enough information can be collected) seeding density can produce acceptable results. The camera (Nikon D90 with 8-12-mm wide angle lens) was mounted vertically over the abutment. Each video recording (frame rate 24-Hz) was of 2-5 minutes duration. Post-processing was carried out using the software Streams (version 2.03) by Nokes (2012). The general post-processing procedure is:

- Image capture
- Image processing – particle identification
- Particle tracking
- Velocity field generation.

![Image here]

**Figure 3.39** The fluorescent-yellow particles for the PTV measurements, a. under normal LED lights; b. under ultra-violet lights

### 3.4.3 Long contraction experiments in a rectangular channel

Experiments in this study were used to simulate a condition where the abutment is set well back from the main channel, and abutment-caused scour occurs on the floodplain only. A rectangular channel configuration (channel width \( B = 1.54\)-m) was used, as shown in Figure 3.40.
3.4.3.1 Abutment model

Vertical-wall abutments with two lengths ($L_a = 400$-mm and 800-mm) and two widths ($L_c = 50$-mm and 1100-mm) were used in this study (Figure 3.41). The abutment models were box shaped and were made from acrylic sheet. The models were attached to the true left side of the flume, near the middle of the sand recess length, ensuring enough space for scour development in the test section. Each abutment model sat on the bottom of the recess, and no structure vibrations were observed during the experiment. Countermeasures were not used in this study.

Figure 3.40 The rectangular channel configuration used for long contraction experiments
Figure 3.41 Dimensions of the vertical-wall abutment models in the rectangular model channel

3.4.3.2 Experimental procedure

For each experiment, the sand in the test section was levelled to the elevation of the false floor, and then a 100-mm or 150-mm depth of water was slowly introduced into the flume. Prior to the experiment, a suitable channel slope was determined to ensure uniform flow occurred. An ultrasonic depth sounder was positioned at the upstream corner of the abutment, where scour typically developed the most rapidly and the deepest. Scour history was recorded for the duration of the experiment, until equilibrium scour depth was reached. Equilibrium was defined as when the scour depth increment was less than or equal to 5% of water depth in 24-hours, e.g., 5-mm for 100-mm depth (Coleman et al. 2003). Each experiment normally took 150 – 240-hours.

The surface flow field was obtained for the initial state of scour, and the equilibrium state, using the PTV technique. The pre-scour surface flow field was measured with the sandy bed
immobilized using an aluminium plate. At the end of each experiment, the detailed bed bathymetry was measured.

3.4.4 Long contraction experiments in a compound channel

Experiments in this part of the long contraction study were used to simulate the condition where the abutment is set back from the main channel, and the abutment-caused scour occurs on the floodplain and in the main channel. A compound channel with a 1.08-m wide floodplain, a 0.26-m wide main channel bank (2:1) and a 0.2-m wide main channel was used for this study.

3.4.4.1 Abutment model

Vertical-wall abutments with varying widths (12-mm, 45-mm, 135-mm, 810-mm, 1600-mm and 2445-mm, Figure 3.42) and a constant length (540-mm, $L_a/B_j=0.50$) were used in this study. The acrylic box-shaped models were positioned on the floodplain, near the middle of the sand recess length. Each abutment model sat on the bottom of the recess, and no structure vibrations were observed during the experiments. Experiments with and without rock riprap apron were carried out in this study, to investigate the effect of countermeasures on contraction scour.

3.4.4.2 Scour experiment

For each experiment, the channel bed in the test section was shaped to the compound channel configuration. The water depth on the floodplain was set at 180-mm for all the experiments in this study. Prior to each experiment, the channel slope was adjusted to ensure uniform flow occurred. Multiple ultrasonic depth sounders were positioned at the expected location of the scour hole. The expected location of the scour hole was at the upstream corner of the abutment if countermeasures were not used; the expected location was about 300-mm to 800-
mm downstream of the upstream face if countermeasures were used. Scour history was recorded for the duration of the experiments, until dynamic equilibrium scour depth was reached. Each experiment took 22 - 48-hours. At the end of each experiment, the detailed bed bathymetry was measured.

Figure 3.42 Vertical-wall abutment models used in the compound channel (for short and long contraction scour)
### 3.4.4.3 Flow experiment

The purpose of this flow-measurement study was to understand the flow pattern around short and long contractions, and to investigate the scour differences between a short and a long contraction in a compound channel. At a 1:45 geometric scale, flow features were investigated around a one-lane vertical-wall bridge abutment model ($L_C = 135$-mm) and a six-lane vertical-wall bridge abutment model ($L_C = 810$-mm). These two model abutments were set back from the main channel ($L_n/B_f = 0.50$). The test section was immobilized to allow pre-scour flow fields around the abutment to be measured, using a down-looking Vectrino+. The plan view of the measurement positions (black dots) is shown in Figure 3.34.

![Figure 3.34 Plan view of the flow measurements for (a) short contraction (model one-lane bridge abutment); (b) long contraction (model six-lane bridge abutment); unit: mm](image)
For each vertical profile, measurements were made at 10-mm, 25-mm, 40-mm, 55-mm, 70-mm, 85-mm, 105-mm and 115-mm from the bottom. Each near-bottom (10-mm) measurements took 5-minutes, and each of the rest took 2-minutes. The results are presented in Chapter 7.
Chapter 4.  LIVE-BED SCOUR STUDY

4.1 Introduction

The aim of this live-bed scour study was to investigate scour characteristics of an abutment situated on the floodplain of a compound channel, in which clear-water flows prevail on the floodplain and live-bed or critical flows prevail in the main channel. Flow conditions, varying from FS, SO to OT flows, were investigated. For abutment scour in the live-bed scour regime, the following are the major differences between a compound channel and a rectangular channel are: behaviours of the main channel bank; flow distribution at the approach section, flow redistribution at the bridge section, and the corresponding scour in the main channel and on the floodplain. In this study, a specific type of scour countermeasure - rock riprap apron - was used for each flow condition. The protection and failure mechanisms of riprap in the apron in the scouring process are discussed in Sections 4.2.3 and 4.2.4.

This chapter presents the experimental results collected using two recirculating flumes, as described in Chapter 3. The main body of the live-bed data, results of the LB 1.54-m wide channel are presented in Sections 4.2 to 4.4; supplementary data, results of the LB 2.4-m wide channel, are mainly presented in Section 4.5.

For all the live-bed scour experiments, experimental setups comprise a matrix of flow condition, abutment length and flow intensity, as summarised below and detailed in Table 4.1:

- Three flow conditions: FS, SO and OT conditions.
- Four abutment lengths: short setback abutment (SSA, $L_a/B_f = 0.8$), medium setback abutment (MSA, $L_a/B_f = 0.65$), long setback abutment (LSA, $L_a/B_f = 0.5$) and no abutment (NA, $L_a/B_f = 0.0$).
• Flow intensity in the main channel: \(0.99 \leq V_{ml}/V_{mic} \leq 1.71\); two flow intensity groups are defined as the following: \(0.99 \leq V_{ml}/V_{mic} \leq 1.15\) (denoted by \(q\) in Tables 4.1 and 4.2; hereinafter termed the relatively small flow intensity group) and \(1.34 \leq V_{ml}/V_{mic} \leq 1.71\) (denoted by \(Q\) in Tables 4.1 and 4.2; hereinafter termed the relatively large flow intensity group).

• Flow intensity on the floodplain: \(V_{f1}/V_{f1c} \leq 1.00\).

Presented results consist of final bed bathymetry, temporal development of scour, the maximum scour depth, water surface profiles and the Reynolds shear stress distributions prior to scour.

Table 4.1 Experimental setups for the LB 1.54-m wide channel

<table>
<thead>
<tr>
<th>Experiment</th>
<th>(L_d/B_f)</th>
<th>(Q) (L/s)</th>
<th>Flume slope (%)</th>
<th>(y_f/y_{ml})</th>
<th>(V_f/V_{f1c})</th>
<th>(V_{ml}/V_{mic})</th>
<th>(q_f/q_{f1})</th>
<th>(q_{ml}/q_{mlc})</th>
<th>(u^*_{ml}) (cm/s)</th>
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<td>1.00</td>
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122
Note: channel width is not included in the experiment names for the 1.54-m wide channel; \( q \) and \( Q \) denote smaller discharge and larger discharge, respectively; subscript \( f \) denotes floodplain; \( m \) denotes main channel; \( c \) denotes the critical state; 1 denotes C.S.1; 2 denotes C.S.4; \( u^* \) is the shear velocity, which is determined from the measured mean velocity distribution in the near-boundary region. The value of each parameter can be found in Tables A1 and A2 in the appendix.

### Table 4.2 Experimental setups for the LB 2.4-m wide channel

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<th>Experiment</th>
<th>( L_d/B_f )</th>
<th>( Q ) (L/s)</th>
<th>Flume slope (%)</th>
<th>( y_f/y_{ml} )</th>
<th>( V_f/V_{fc} )</th>
<th>( V_m/V_{mc} )</th>
<th>( q_f/q_f )</th>
<th>( q_m/q_{ml} )</th>
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</table>

Note: channel width is included in the experiment names to distinguish between experiments in the two flumes; Experiment LB_2.4m_SSA_SO_q_r was a repetition of LB_2.4m_SSA_SO_q, but with a longer scour duration. The value of each parameter can be found in Tables A1 and A2 in the appendix.

### 4.2 Experimental observation in the LB 1.54-m wide channel

#### 4.2.1 Behaviour of the main channel bank

An important consideration in this study, and in engineering applications, is that sediment particles on a side slope have lower critical shear stresses compared with those on a horizontal plane (Simons and Senturk, 1992). The well-known Lane (1955) relation defines this comparison,

\[
\frac{\tau_{c\theta}}{\tau_c} = \cos \theta \sqrt{1 - \left(\tan \theta \tan \phi\right)^2} \tag{4.1}
\]
in which, $\tau_{c,\theta}$ is the critical shear stress on a side slope of angle $\theta$ ($0^\circ \leq \theta < 90^\circ$), $\tau_c$ is the critical shear stress on a horizontal plane, and $\phi$ is the angle of repose.

Under the flow circumstances in this study, the main channel bank would still be eroded to some extent in the absence of the abutment structure and the bridge deck model. This portion of erosion is difficult to distinguish from the scour caused by the abutment and other hydraulic structures. In the case of a channel in coarse alluvium with no hydraulic structures, a self-formed stable main channel bank may deform observably during an extreme hydrological event; in addition, other components of scour would simultaneously occur if hydraulic structures intruded into the flow. Therefore in a flood event, total scour at a structure site is a combination of scour components concurrently induced by all possible factors, including the side slope factor.

For the situation with no model hydraulic structures, possible effects on the main channel bank during the passage of a flood are schematically shown in Figure 4.1. In this study, the scour pattern sketched in Figure 4.1b dominated.

According to (ASCE 2006: Chapter 2), a force balance would be achieved for a simplified model of a spherical particle on a side slope (see Figure 4.2),

$$\mu^2 \left| \vec{F}_n + \vec{F}_l \right|^2 = \left| \vec{F}_d \right|^2 + \left| \vec{F}_l \right|^2$$

Equation 4.2

in which $\mu$ is the coefficient of Coulomb resistive force; $\vec{F}_n$ is the component of the gravitational force acting downward normal to the side slope; $\vec{F}_l$ is the lift force, acting upward normal to the side slope; $\vec{F}_d$ is the drag force, acting in the streamwise direction and
$F_t$ is the transverse component of the gravitational force, acting downward along the side slope.

Both $F_d$ and $F_t$ increase when the flow intensity increases. To achieve the new balance, either the side slope angle would decrease to increase the value of $F_u$ and to decrease the value of $F_t$ (see Figure 4.1a), or, as found to be the dominant behaviour in this study, the side slope would retreat (see Figure 4.1b).

![Figure 4.1](image1.png)

**Figure 4.1** Responses of the main channel bank during the passage of a flood; **a.** the side slope decreases; **b.** the side slope retreats and the original angle is maintained.

![Figure 4.2](image2.png)

**Figure 4.2** Definition sketch for a spherical particle on a side slope

Contaminant erosion of the main channel bank complicates the scour process and the scour-estimation at bridge abutments. The overall erosion process is affected by the erosion-resistant properties of the main channel bank material, water depth differences between the main channel and the floodplain, and the initial main channel bank slope before a flood event.
In this study, the floodplain was as erodible as the main channel bed, while in reality, more complicated erodibility combinations occur. Further research is required, regarding the effect of the relative erodibility of the floodplain and the main channel. In this respect, preliminary findings about soil cohesive strength can be found in Ettema et al. (2010) and Ettema et al. (2015).

4.2.2 Behaviour of sand dunes in the bridge section

In this study, bed forms propagated mainly in the form of sand dunes. The propagation of bed forms inevitably altered the scour at the bridge section, and was found to endanger the geo-stability of the abutment.

Previous studies indicate that under live-bed scour conditions, dune troughs were the main cause of the destabilisation of riprap, during the embedment process (Chiew and Lim, 2000), as reviewed in Chapter 2. However, observations of the scour process in this study showed that dune crests can also exacerbate the erosion at the bridge section. It was found that when a large-size sand crest avalanched into the bridge section, the flow area was accordingly reduced, increasing the averaged flow velocity in the contracted section, and thus further promoting the erosion around the abutment toe. This was particularly apparent for a short setback abutment, and it could be worse for a bankline abutment. In this situation, sediment particles close to the abutment toe are prone to winnowing, leading to disintegration of the riprap apron. In other words, large-size sand crests cause deposition at the lower part of the bridge section, reduce the flow area, increase the flow velocity, and then exacerbate the winnowing process at the upper part of the bridge section (close to the abutment toe). When the rocks in the apron close by the abutment toe were removed, scour undercut the spill slope material and the abutment was threatened by a geotechnical failure. In summary, dune
troughs lead to embedment of the peripheral riprap in the main channel, while dune crests tended to intensify sediment winnowing failure close to the abutment toe.

Upstream of the bridge, bed forms migrated mainly in the deeper main channel, and in the main channel bank area when the bed forms were well-developed. However, the migration pattern was unpredictable at the bridge section. Based on laboratory observations, bed forms propagated through the bridge section in four different ways:

(a) Bed forms did not directly interact with the abutment or the riprap (Figure 4.3). These bed forms typically occurred with long setback abutments – in these cases, sand dunes have enough space to pass through without interfering with the abutment.

(b) Bed forms shifted transversally towards the abutment at the bridge section (Figure 4.4). These bed forms typically occurred with medium setback abutments and with relatively small flow intensities – in these cases, large volume dunes were found to pass through the scour hole.

(c) Bed forms shifted transversally towards the scour hole on the downstream side of the abutment (Figure 4.5). These bed forms typically occurred with short setback abutments and relatively small flow intensities - in these cases, deep and large scour holes formed downstream of the abutment, and dunes were found to avalanche into the scour hole).

(d) Bed form amplitudes reduced while passing by the abutment (Figure 4.6). These bed forms typically occurred with short setback abutments and with relatively large flow intensities – in these cases, the flow intensity increased significantly at the bridge section, and the flow rapidly carried away the sediment and reduced the height of the sand dunes.

It was found that dune height could reach 180-mm (more than the 130-mm level difference between the floodplain and the main channel), causing a total flow-area reduction of 8%,
which directly increased the drag and lift forces on the boundary materials. In addition, vortices at the leeside of the dune aggravated the embedment of the riprap.

Figure 4.3 Sand dunes that stayed in the main channel while passing the abutment: a. schematic diagram; b. exemplary photo of large-amplitude sand dunes that stayed in the main channel while passing the abutment; and c. exemplary photo of small-amplitude sand dunes that stayed in the main channel while passing the abutment
Figure 4.4 Sand dunes that shifted towards the abutment while passing the abutment: a. schematic diagram; b. exemplary photo of large-amplitude sand dunes that shifted towards the abutment while passing the bridge section; and c. exemplary photo of small-amplitude sand dunes that shifted towards the abutment while passing the bridge section.
Figure 4.5 Sand dunes that shifted into the scour hole after passing the abutment. a. schematic diagram; b. exemplary photo

Figure 4.6 Sand dunes whose amplitudes reduced while passing the abutment: a. schematic diagram; b. exemplary photo
The development and migration pattern of the bed forms varied with the flow condition and the abutment configuration. Figure 4.7 illustrates graphically the temporal development of scour at three representative locations along C.S.5. Data were obtained with a long setback abutment ($L_u/B_f = 0.5$). Results show that:

1. For the relatively small flow intensity group, periodicity of bed forms was similar among different flow conditions in the bed form development phase (except for LB_LSA_FS_q). When the bed forms were well-developed, FS flows obviously had the smallest frequencies.

2. Well-developed bed forms oscillated around different equilibrium elevations for different flow conditions. For example, in the main channel ($Y = 100$-mm), for the relatively large flow intensity group and for OT flow, scour oscillated around $+50$-mm (50-mm beneath the original bed elevation), while for SO flow it oscillated around $+30$-mm, and for FS flow, around 0-mm. These oscillating elevations in the main channel were related to the magnitude of contraction scour for different flow conditions.

3. For the relatively large flow intensity group, bed forms transversally migrated into the floodplain. As shown in Figures 4.7a, 4.7c and 4.7e, for the large flow intensity group, small-amplitude bed forms were observed at the edge of the floodplain at $Y = 460$-mm. For the low flow intensity group in Figures 4.7b, 4.7d and 4.7f, bed forms typically would not cross the centre of the main channel bank at $Y = 330$-mm.

Figures 4.7 and 4.8 show the temporal developments of scour at C.S.5 with a short and a medium setback abutment, respectively. Data were obtained for the OT flow condition. Comparing Figures 4.7 and 4.8 with Figure 4.6b shows that the bed forms for the short
setback abutments were negligible, which is probably because, for a severe contraction, the bed forms changed from dunes to flat-bed at the bridge section.

It should be noted that the scoured sand from the test section entered the sand recirculation system during the experiment, and as a result, the periodicity and the amplitudes of the bed forms may have altered because the averaged sand level in the main channel increased. To reduce the effects of this extra sand entering the recirculation system, an estimated volume of sand, approximately equal to the sand carried out from the test section, was taken out of the system, from both ends of the main channel, before the experiment.

In addition, experimental observation suggests that, though the main channel was only set 200-mm wide, rapid scour development within the main channel bank resulted in a relatively wide cross section for dune propagations, and as such, the side wall effect on scour development was found to be insignificant.
Figure 4.7 The temporal development of scour in the main channel of long setback abutments; a. for LB_LSA_OT_Q, b. for LB_LSA_OT_q, c. for LB_LSA_SO_Q, d. for LB_LSA_SO_q, e. for LB_LSA_FS_Q and f. for LB_LSA_FS_q

Figure 4.8 The temporal development of scour in the main channel a. for LB_SSA_OT_q, b. for LB_MSA_OT_q
4.2.3 Behaviour of rock riprap in the apron

In this study, both clear-water and live-bed scour regimes occurred simultaneously but in different subsections of the channel. Though the apron was initially placed in the clear-water zone, bed forms and rapid flows in the main channel significantly affected the behaviour of the rocks in the riprap apron.

The dimensions of the apron and the selection of the size of riprap were based on HEC-23 (Lagasse et al. 2009). Neither Riprap shear erosion nor total disintegration occurred during the experiments. However, because the riprap rocks used were uniform, and a geotextile filter was not employed, winnowing erosion was commonly observed and was the major mode of destabilization in this study. Edge erosion occurred occasionally, and was observed only for LB_SSA_OT_Q and LB_SSA_SO_Q. Due to the propagation of bed forms, embedment of riprap was frequently observed in the main channel and in the lower part of the main channel bank.

Though the flow and geometry conditions were varied systematically in this study, there were two patterns of apron rock behaviour: sliding down and embedment. These two processes are closely related to the bedform-induced degradation discussed by Chiew (2004). The sliding down process is demonstrated in the photo series in Figure 4.9. Soon after the commencement of each experiment, rapid scour occurred on the main channel bank and at the edge of the floodplain (an extreme case is shown in Figure 4.10). This induced a differential mobility between riprap and sediment particles, similar to that found around a bridge pier by Chiew and Lim (2000). Riprap in the apron became mobile once the sediment particles beneath or close to them were carried away. The rate of sliding decreased as the sediment erosion proceeded. This process generally lasted more than 24-hours for the small
flow-intensity experiments, and was much faster for the high flow-intensity ones. An example of a long-duration sliding process is shown in Figure 4.11. The bright zone in Figure 4.11c shows the movement of the riprap during the 20-minute period after 60-hours of scour.

Embedment mainly occurred to the riprap that slid into the main channel. Chiew and Lim (2000) explained that large values of $D_{50}/d_{50}$ and the propagation of bed forms are the main causes of this behaviour. The process of embedment by the passage of bed forms is shown in Figure 4.12. Similar to embedment failure at bridge piers, the entire riprap eventually subsided on top of the re-built main channel bank. Subsidence was mainly attributable to winnowing.

The behaviour pattern of the riprap layer is similar when it is subjected to the FS, SO and OT flow conditions. This similarity is because the sliding and embedment processes are governed by similar sediment erosion processes. The observed difference is the rate of sliding and embedment, which are dictated by flow intensity at the bridge section.
**Figure 4.9** Photo series of riprap in the apron sliding down, LB_MSA_OT_q

**Figure 4.10** Exemplary photo of scour soon after the commencement of experiment, LB_SSA_OT_Q
**Figure 4.11** Apron layout for LB_SSA_OT_Q a. \( t = 60 \)-hours; b. \( t = 60 \)-hours + 20-minutes; c. subtraction of Figure 4.11a and Figure 4.11b (Figure 4.11b – Figure 4.11a)

**Figure 4.12** Exemplary photos of bed forms embedding riprap in the main channel
4.2.4 Failure mechanisms of non-erodible embankment

In this study, erosion reached the toe of the abutment spill-through slope for short setback abutments under pressure flows, as shown in Figures 4.13(a - d). As illustrated previously (see Section 4.2.3), riprap rocks slipped down and were embedded as scour developed. When the most vulnerable apron rock adjacent to the abutment toe lost its stability, flow opened up a small gap into the floodplain materials (sediment) beneath the embankment. Sediment particles were then washed out, intermittently, till equilibrium scour was reached. In this study, scour extended deep into the embankment, exposing the piles and rendering the whole abutment toe unsupported.

Two contrasting experiments were conducted to investigate the effect of a geotextile filter on abutment failure (Figure 4.13). A piece of light, flexible and permeable clothing material was initially used as a geotextile filter beneath the apron. Without the geotextile filter, failure was initiated at the abutment toe 70-minutes after the commencement of the experiment. The gap enlarged progressively, concurrently with the embedding process of the rocks at the abutment toe. After 24-hours, a destructive opening was observed and the whole embankment foundation was undermined (Figure 4.14a). By contrast, with the geotextile filter in place, winnowing and rock embedment were effectively mitigated. Failure around the abutment toe started after 20-hours. After 24-hours, erosion around the abutment toe was insignificant and the abutment retained its integrity. However, the filter was somewhat exposed; exposure of the filter edge can be detrimentally because the flow may lift up the whole filter.
Figure 4.13 Photos of abutment failures with non-erodible short setback abutments: a. LB_SSA_SO_q; b. LB_SSA_OT_q; c. LB_SSA_SO_Q and d. LB_SSA_OT_Q

Figure 4.14 Scour results for LB_SSA_OT_Q a. without geotextile filter and b. with the geotextile filter in place
(Eve 1999) reported that the geotextile filter may induce edge failure of the riprap. Another concern for this study is that the scale effect of the filter is difficult to evaluate. Therefore, geotextile was not used in any of the subsequent experiments. This method of testing without geotextile filters for the spill-through abutment was used in previous research, e.g., van Ballegooy (2005), Ettema et al. (2010) and Hong (2012). Experiments were terminated when incipient erosion and collapse of the embankment occurred, unless continuation was desirable for the purpose of laboratory observation. Altogether, 4 out of 22 experiments conducted in the 1.54-m wide flume exhibited embankment failure.

4.3 Water surface profiles in the LB 1.54-m wide channel

The degree of backwater was found to affect $y/f_1$, and also to some extent the discharge obstructed by the hydraulic models (Sturm, 2006), and hence affect the flow redistribution at the bridge section. Water depth profiles were measured prior to scour (over an immobilized bed), to investigate the head loss at the bridge section, and to quantify the degree of backwater in each experiment.

Figures 4.15a and 4.15b show the water depth profiles along the flume for the long and the short setback abutments, respectively. The presented water-surface elevations are the mean of measurements from more than five lateral positions (normally at $Y = 100$-mm, 460-mm, 800-mm, 1100-mm and 1400-mm). The nearly constant flow depth on the upstream side of the bridge indicates that uniform flow was achieved for each experiment. Typically, when the flow approached the bridge section, the flow surface rapidly dropped after passing through the contracted section, followed by a mild rise downstream of the bridge. Head loss $\Delta y$ for the clear-water scour regime has been found to be negligible in previous research (e.g. Hong 2012). However, it needs to be considered in the live-bed scour regime, as shown in Figure.
4.15. Head loss was found to vary with the flow intensity \((V/V_c)\), water depth \((y)\), and the geometric contraction ratio \((m)\). The latter two parameters can be represented by the discharge contraction ratio \(Q_{\text{obst}}/Q\), where \(Q_{\text{obst}}\) is the portion of the total discharge obstructed by the area of the embankment and the bridge deck at the bridge section, and is equal to sum of these areas multiplied by the corresponding flow velocity at the approach section. Thus,

\[ Q_{\text{obst}} = Q - Q_f \quad \text{Equation 4.3} \]

in which \(Q\) is the total discharge, and \(Q_f\) is the discharge that could pass through the bridge opening freely (i.e., without disturbance or contraction by a bridge deck or embankment). The definition sketch of \(Q_{\text{obst}}\) can be found in Figure 2.25 in Chapter 2. The definition of \(Q_{\text{obst}}\) is adapted from Sturm (2006).

Water surface profile data in this study were evaluated using the procedure proposed by Kindsvater et al. (1953), using the following equations:

\[
C = \frac{Q}{A_2 \sqrt{2g \left( \Delta y + \alpha_i \frac{V^2}{2g} - h_f \right)}} \quad \text{Equation 4.4}
\]

\[
C = k_f k_r C' \quad \text{Equation 4.5}
\]

in which \(C\) is the coefficient of discharge, \(A_2\) is the gross area of the bridge section, \(\Delta y\) \((= y_1 - y_2)\) is the difference in elevation of the water surface between the approach section and the bridge section, \(\alpha_i\) is a coefficient which takes into account the variation in velocity distribution, \(h_f\) is the head loss due to friction, \(C'\) is the standard value of the coefficient of discharge corresponding to the given values of \(m\) and \(W_a/B_o\), \(W_a\) is the width of the abutment, \(B_o\) is the width of the bridge opening, \(k_f\) is a coefficient that adjusts for the influence of deck
submergence \((t_s)\) and \(k_f\) is a coefficient that adjusts for the Froude number \((Fr)\). The values of \(C'\), \(k_r\) and \(k_f\) can be found in Kindsvater et al. (1953).

The evaluation involves comparing the calculated discharge coefficient \((C)\) from Equation 4.4 (\(\Delta y\) is the key parameter) and Equation 4.5 (\(Q_{\text{obs}}/Q\) is the key parameter). Results are shown in Table 4.3 and Figure 4.16. It can be seen that the procedure proposed by Kindsvater et al. (1953) could well predict the discharge coefficient for most of the FS and SO experiments (except for LB_SSA_SO_Q). Because Equations 4.4 and 4.5 were derived without regard to OT flows, the proposed procedure could not well predict most of the OT experiments in this study. Because \(\Delta y\) and \(Q_{\text{obs}}/Q\) are the key parameters in Equation 4.4 and Equation 4.5, respectively, comparison of the results in Table 4.3 and Figure 4.16 implies that \(\Delta y\) is closely related to \(Q_{\text{obs}}/Q\).
Figure 4.15 Width-averaged water surface profile along the flume (measured from the un-scoured floodplain level) a. short setback abutments, b. long setback abutments

Table 4.3 Parameters for discharge coefficient calculation

<table>
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<th>$y_{m2}$</th>
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<th>$C'$</th>
<th>$h_f$</th>
<th>$k_T$</th>
<th>$Fr$</th>
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<td>0.51</td>
<td>76%</td>
</tr>
</tbody>
</table>
Based on the Bernoulli energy equation and the equation of continuity, for a horizontal bottom:

\[ y_1 + \alpha_1 \frac{V_1^2}{2g} = y_2 + \alpha_2 \frac{V_2^2}{2g} + h_f + \lambda_2 \frac{V_2^2}{2g} \]  

Equation 4.6

Here subscript 1 denotes the approach section, and subscript 2 denotes the bridge section. Because the Bernoulli energy equation is applied over a short distance, \( h_f \approx 0 \). Also, the contraction head loss is assumed to be negligible.

Kindsvater et al. (1953) found that \( \frac{Q_{obs}}{Q} \) and \( \frac{W}{B_y} \) are the two key parameters determining the discharge coefficient. From Equations 4.4 and 4.6,
\[
\frac{1}{C^2} = \alpha_1 + \lambda_2 = f_1 \left( \frac{Q_{\text{obst}}}{Q} \right) \left( \frac{W_a}{B_o} \right) = \frac{\Delta y}{V_z^2/2g} + \frac{\alpha_i V_1^2/2g}{V_z^2/2g}
\]
Equation 4.7

Because \( \frac{W}{B_o} \) was constant for each abutment length in this study,

\[
f_i \left( \frac{Q_{\text{obst}}}{Q} \right) = \frac{\Delta y}{V_z^2/2g} + \frac{\alpha_i V_1^2/2g}{V_z^2/2g}
\]
Equation 4.8

\[
\frac{\alpha_1}{2g} = \left[ f_i \left( \frac{Q_{\text{obst}}}{Q} \right) \frac{V_z^2}{V_1^2} - \alpha_i \right] V_1^2
\]
Equation 4.9

According to the definition of \( \frac{Q_{\text{obst}}}{Q} \), for a rectangular channel geometry, \( \frac{V_1}{V_2} \approx \frac{Q - Q_{\text{obst}}}{Q} \). For the compound geometry in this study, \( \frac{V_1}{V_2} \) has a complicated relationship with \( \frac{Q - Q_{\text{obst}}}{Q} \), but \( \frac{V_1}{V_2} \) is mainly determined by \( \frac{Q - Q_{\text{obst}}}{Q} \). Therefore,

\[
f_2 \left( \frac{Q_{\text{obst}}}{Q} \right) V_1^2
\]
Equation 4.10

Because the flow parameters in the main channel dominate (the final scour holes for all the experiments were located in the main channel), the main channel flow intensities are used herein. The best semi-empirical relationship for the data in this study was obtained through multiple linear regression analysis as,

\[
\Delta y = 18.6 \frac{V_1^2}{2g} \left( \frac{Q_{\text{obst}}}{Q} \right)^{1.5}
\]
Equation 4.11
and as shown in Figure 4.17. It should be noted that Equation 4.11 was derived with very limited data and with three assumptions ($h_j \approx 0$, contraction loss $\approx 0$ and only accounting for flow intensities in the main channel). Therefore, the applicability of this equation is limited.

**Figure 4.17** Head loss as a function of flow intensity and discharge re-distribution

4.4 Scour results for the 1.54-m wide channel

4.4.1 The effect of flow condition on scour

This section discusses the differences in scour development and scour depth caused by the three flow conditions.
To minimize the effects of bed forms, results obtained from the small flow intensity group and the medium setback abutments (LB_MSA_FS_q, LB_MSA_SO_q and LB_MSA_OT_q) are used herein for most of the discussion.

The differences between SO and FS flows are in:

- vertical contraction
- flow depth
- flow distribution at the approach section

*Vertical contraction* essentially increases the average velocity at the bridge section (and within a finite distance downstream of the bridge section); the lift and drag forces over the streambed increase accordingly. The effect of vertical contraction can be represented by $Q_{ad} / Q$, where $Q_{ad}$ is the portion of discharge “obstructed” by the bridge deck at the approach section (Figure 4.18). This is similar to the definition of $Q_{obs}$. Results from this study show that more than 20% of the approaching discharge was forced underneath the bridge deck ($Q_{ad}$) for SO flows (Table 4.4).

*Flow depth* influences the magnitude of turbulence structures, and hence the scour. For a given bridge site, SO flows are deeper than FS flows. In this compound channel study, the width-average flow depth, $\bar{y}$, is used in the following analysis, and $\bar{y}$ (SO) $\approx 1.33 \bar{y}$ (FS). Melville (1992) found that the scour depths around intermediate-length abutments (intermediate-length abutment is defined as $(1 < L_a/\bar{y} < 25)$) have the relationship $d_s \propto \sqrt{L_a/\bar{y}}$. Because the abutments used in this study had intermediate lengths with size-depth ratio, $L_a/\bar{y}$, ranging from 3.0 to 8.6, the relatively deep SO flows were expected to cause deeper scours than the FS flows.
In addition, flow distribution at the approach section also has a large influence on scour. Compared with FS flows, SO flows had a larger proportion of discharge flowing on the floodplain at the approach section, therefore, they had a larger proportion of discharge being forced into the contracted section (Table 4.5). The combined effect of the above differences caused the differences, between the two flow conditions, in scouring development and the maximum scour depth.

Figures 4.19 a– d show the scouring development differences between LB_MSA_FS_q and LB_MSA_SO_q at C.S.5. Results show that, for the selected transverse positions, more than 50% of the total scour occurred in the first 5-hours, and during this period the scour rate in the SO flow appreciably exceeded that in the FS flow. After 10-hours, the scour rates between these two flow conditions were nearly equal. This scour development indicates that the majority of the scour differences between the two flow conditions occurred in a relatively short period of time after the commencement of the experiment, and the flow features prior to scour (see Chapter 5) are important for evaluating the subsequent scour features. Figure 4.19a shows an abrupt rise in scour depth at \( t \approx 37 \) hours and drop at \( t \approx 47 - 58 \) hours for LB_MSA_SO_q. This was probably because the scoured sediment in the test section re-entering the sediment-recirculating system. It is difficult to maintain the same sediment volume in the main channel, given that a large amount of sediment was removed from the test section. For these live-bed experiments, a certain amount of sediment was removed prior to the experiments, to alleviate the above issue.

Figures 4.20 and 4.21 show bed morphologies at the dynamic equilibrium state for LB_MSA_FS_q and LB_MSA_SO_q, respectively. The scour hole for LB_MSA_SO_q (slightly deepened by a dune trough) was significantly larger and deeper than that for
LB_MSA_FS_q. Both scour holes were centralized on the lower edge of the main channel bank, but the scour hole in LB_MSA_SO_q was elongated by a stronger contracted flow. Influences of the bed forms were more prominent for the SO flows, because they had relatively deeper water depths (Raudkivi, 1998).

**Figure 4.18** Schematic diagram of vertical discharge distribution: a. OT flow; b. SO flow

**Table 4.4** Discharge obstructed by the bridge deck at the initial state, for the LB 1.54-m wide compound channel

<table>
<thead>
<tr>
<th>Experiment</th>
<th>$Q_{bd}/Q$</th>
<th>Experiment</th>
<th>$Q_{bd}/Q$</th>
<th>Experiment</th>
<th>$Q_{bd}/Q$</th>
</tr>
</thead>
<tbody>
<tr>
<td>LB_SSA_FS_q</td>
<td>0.0%</td>
<td>LB_MSA_FS_q</td>
<td>0.0%</td>
<td>LB_LSA_FS_q</td>
<td>0.0%</td>
</tr>
<tr>
<td>LB_SSA_SO_q</td>
<td>21.1%</td>
<td>LB_MSA_SO_q</td>
<td>21.6%</td>
<td>LB_LSA_SO_q</td>
<td>23.6%</td>
</tr>
<tr>
<td>LB_SSA_OT_q</td>
<td>17.3%</td>
<td>LB_MSA_OT_q</td>
<td>16.3%</td>
<td>LB_LSA_OT_q</td>
<td>16.1%</td>
</tr>
<tr>
<td>LB_SSA_FS_Q</td>
<td>0.0%</td>
<td>LB_MSA_FS_Q</td>
<td>0.0%</td>
<td>LB_LSA_FS_Q</td>
<td>0.0%</td>
</tr>
<tr>
<td>LB_SSA_SO_Q</td>
<td>22.1%</td>
<td>LB_MSA_SO_Q</td>
<td>23.8%</td>
<td>LB_LSA_SO_Q</td>
<td>23.7%</td>
</tr>
<tr>
<td>LB_SSA_OT_Q</td>
<td>16.5%</td>
<td>LB_MSA_OT_Q</td>
<td>15.5%</td>
<td>LB_LSA_OT_Q</td>
<td>16.9%</td>
</tr>
</tbody>
</table>
Table 4.5 Discharge measurements at C.S.1 and C.S.4 at the initial state, for the LB 1.54-m wide compound channel

<table>
<thead>
<tr>
<th>Experiment</th>
<th>$Q_{m1}$</th>
<th>$Q_f$</th>
<th>$Q_f/Q$</th>
<th>$Q_{m2}$-submerged</th>
<th>$Q_{2}$-submerged</th>
<th>$Q_{2}$-submerged</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L/s</td>
<td>L/s</td>
<td></td>
<td>L/s</td>
<td>L/s</td>
<td></td>
</tr>
<tr>
<td>LB_SSA_FS_q</td>
<td>26.3</td>
<td>9.8</td>
<td>27.2%</td>
<td>32.3</td>
<td>3.0</td>
<td>8.5%</td>
</tr>
<tr>
<td>LB_SSA_SO_q</td>
<td>33.2</td>
<td>23.6</td>
<td>41.6%</td>
<td>49.1</td>
<td>6.0</td>
<td>10.9%</td>
</tr>
<tr>
<td>LB_SSA_OT_q</td>
<td>41.8</td>
<td>42.6</td>
<td>50.5%</td>
<td>45.7</td>
<td>6.4</td>
<td>12.2%</td>
</tr>
<tr>
<td>LB_SSA_FS_Q</td>
<td>37.3</td>
<td>18.5</td>
<td>33.2%</td>
<td>46.9</td>
<td>5.3</td>
<td>10.2%</td>
</tr>
<tr>
<td>LB_SSA_SO_Q</td>
<td>44.0</td>
<td>33.5</td>
<td>43.2%</td>
<td>64.4</td>
<td>8.6</td>
<td>11.8%</td>
</tr>
<tr>
<td>LB_SSA_OT_Q</td>
<td>59.1</td>
<td>63.9</td>
<td>51.9%</td>
<td>65.0</td>
<td>6.5</td>
<td>9.1%</td>
</tr>
<tr>
<td>LB_MSA_FS_q</td>
<td>26.1</td>
<td>11.2</td>
<td>30.1%</td>
<td>28.9</td>
<td>6.5</td>
<td>18.3%</td>
</tr>
<tr>
<td>LB_MSA_SO_q</td>
<td>34.6</td>
<td>22.4</td>
<td>39.3%</td>
<td>43.8</td>
<td>11.0</td>
<td>20.1%</td>
</tr>
<tr>
<td>LB_MSA_OT_q</td>
<td>46.5</td>
<td>44.5</td>
<td>48.9%</td>
<td>41.3</td>
<td>9.3</td>
<td>18.3%</td>
</tr>
<tr>
<td>LB_MSA_FS_Q</td>
<td>38.3</td>
<td>17.3</td>
<td>31.1%</td>
<td>42.7</td>
<td>9.5</td>
<td>18.1%</td>
</tr>
<tr>
<td>LB_MSA_SO_Q</td>
<td>47.2</td>
<td>31.6</td>
<td>40.1%</td>
<td>59.7</td>
<td>13.0</td>
<td>17.9%</td>
</tr>
<tr>
<td>LB_MSA_OT_Q</td>
<td>63.9</td>
<td>66.0</td>
<td>50.8%</td>
<td>56.3</td>
<td>12.9</td>
<td>18.7%</td>
</tr>
<tr>
<td>LB_LSA_FS_q</td>
<td>26.5</td>
<td>13.8</td>
<td>34.3%</td>
<td>28.4</td>
<td>11.9</td>
<td>29.6%</td>
</tr>
<tr>
<td>LB_LSA_SO_q</td>
<td>36.0</td>
<td>23.4</td>
<td>39.4%</td>
<td>41.5</td>
<td>16.0</td>
<td>27.9%</td>
</tr>
<tr>
<td>LB_LSA_OT_q</td>
<td>47.1</td>
<td>45.0</td>
<td>48.9%</td>
<td>35.5</td>
<td>12.3</td>
<td>25.8%</td>
</tr>
<tr>
<td>LB_LSA_FS_Q</td>
<td>40.6</td>
<td>19.3</td>
<td>32.2%</td>
<td>41.2</td>
<td>18.7</td>
<td>31.2%</td>
</tr>
<tr>
<td>LB_LSA_SO_Q</td>
<td>50.5</td>
<td>32.6</td>
<td>39.2%</td>
<td>57.9</td>
<td>19.4</td>
<td>25.1%</td>
</tr>
<tr>
<td>LB_LSA_OT_Q</td>
<td>64.5</td>
<td>65.5</td>
<td>50.4%</td>
<td>52.9</td>
<td>18.4</td>
<td>25.9%</td>
</tr>
</tbody>
</table>

Note: $Q_{m2}$-submerged denotes the main channel discharge under the bridge deck at C.S.4; $Q_{2}$-submerged denotes the floodplain discharge under the bridge deck at C.S.4; $Q_{2}$-submerged denotes the total discharge under the bridge deck at C.S.4;
Figure 4.19 Scour development at C.S.5 for LB_MSA_FS_q and LB_MSA_SO_q for selected transverse positions. a. Y =330-mm; b. Y =460-mm; c. Y =600-mm; d. Y =740-mm

Figure 4.20 Scour bathymetry at the dynamic equilibrium state for LB_MSA_FS_q
The differences between SO flows and OT flows are in:

- flow relief
- flow distribution at the approach section
- water depth
- possible turbulence generated by the bridge deck

Flow relief is the main difference between SO and OT flows. In this research, OT flows had a range of 37.6% - 48.0% of the total discharge overflowing the bridge deck at the bridge section. The flow regime over the bridge deck changed from a surface-flow regime (Figure 4.22a) to a surface-wave regime (Figure 4.22b) as the flow rate increased at the bridge section (flow regime is defined in Guan et al. 2014). The influence of the above two flow regimes can be reflected by $y_{m1}/y_{m2}$ or $y_{m1}/y_{m0}$: if $y_{m1}/y_{m2}$ or $y_{m1}/y_{m0}$ are large, the head loss is correspondingly large. Flow patterns are described in detail in Chapter 5. $Q_{OT}$ is defined as the portion of discharge, at the approach section, above the horizontal plane of the bridge deck and $Q_{or}$ is the portion of discharge overtopping the bridge deck at the bridge section (see Figure 4.18). Results in Table 4.6 show that $Q_{OR}/Q > Q_{OT}/Q$, which means part of the flow originally below the bridge deck was discharged over the bridge. At the bridge section, discharge beneath the bridge deck, which is shown in Table 4.7, was directly linked
to the scouring process and the final scour depth. Given that the flow area under the bridge deck is the same for SO and OT flows, the average velocity under the bridge deck in SO flows was slightly larger than that in the OT flows. However, it should be noted that data in Table 4.6 and Table 4.7 were obtained with a fixed channel bed, and flow measurements conducted at the equilibrium state show that $Q_{or}/Q$ decreased by about 30% during the experiment. Overtopping discharge was found to decrease rapidly in the beginning of the OT experiments and then the decreasing rate reduced as the flow opening enlarged. Figure 4.23 shows schematically the differences in the discharges under the bridge deck for these two types of flows.

It should be noted that the discharge distributions (lateral and vertical) at the bridge section would be case specific. The configuration of the streambed, the layout of the hydraulic structures, and the approaching flow distribution are key factors determining the flow pattern and subsequently the scour process around the abutment. These key factors require extensive future research, especially numerical simulations with systematic variation of the key factors.

![Figure 4.22](image)

**Figure 4.22** Side view showing surface flow regime at the beginning of the experiment for a. LB_LSA_OT_q and b. LB_SSA_OT_Q
Table 4.6 Discharge over the bridge deck at C.S.1 and C.S.4 at the initial state, for the LB 1.54-m wide compound channel

<table>
<thead>
<tr>
<th>Experiment</th>
<th>( Q_{OT}/Q )</th>
<th>( Q_{OT}/Q )</th>
</tr>
</thead>
<tbody>
<tr>
<td>LB_SSA_OT_q</td>
<td>28.9%</td>
<td>37.6%</td>
</tr>
<tr>
<td>LB_SSA_OT_Q</td>
<td>31.5%</td>
<td>41.9%</td>
</tr>
<tr>
<td>LB_MSA_OT_q</td>
<td>25.5%</td>
<td>44.4%</td>
</tr>
<tr>
<td>LB_MSA_OT_Q</td>
<td>29.2%</td>
<td>46.7%</td>
</tr>
<tr>
<td>LB_LSA_OT_q</td>
<td>26.4%</td>
<td>48.0%</td>
</tr>
<tr>
<td>LB_LSA_OT_Q</td>
<td>29.1%</td>
<td>45.0%</td>
</tr>
</tbody>
</table>

Table 4.7 Discharge under the bridge deck at the initial state, for the LB 1.54-m wide compound channel

<table>
<thead>
<tr>
<th>Experiment</th>
<th>( Q_{2\text{-submerged}}(OT) )</th>
<th>Experiment</th>
<th>( Q_{2\text{-submerged}}(SO) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>LB_SSA_OT_q</td>
<td>52.7</td>
<td>LB_SSA_SO_q</td>
<td>55.1</td>
</tr>
<tr>
<td>LB_SSA_OT_Q</td>
<td>71.5</td>
<td>LB_SSA_SO_Q</td>
<td>73.0</td>
</tr>
<tr>
<td>LB_MSA_OT_q</td>
<td>50.6</td>
<td>LB_MSA_SO_q</td>
<td>54.8</td>
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<tr>
<td>LB_MSA_OT_Q</td>
<td>69.2</td>
<td>LB_MSA_SO_Q</td>
<td>72.7</td>
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<tr>
<td>LB_LSA_OT_q</td>
<td>47.8</td>
<td>LB_LSA_SO_q</td>
<td>57.5</td>
</tr>
<tr>
<td>LB_LSA_OT_Q</td>
<td>71.5</td>
<td>LB_LSA_SO_Q</td>
<td>77.3</td>
</tr>
</tbody>
</table>

Figure 4.23 Schematic diagram of the differences in the discharges under the bridge deck for SO and OT flows; y values increase over time, because overtopping discharges gradually decrease for OT flows.
Figures 4.24a – d depict the scouring development differences between LB_MSA_OT_q and LB_MSA_SO_q at C.S.5. Consistent with the discharge distributions at the bridge section (see Figure 4.23), these two pressure flows had overlapping scour histories at the beginning of the experiments.

Figure 4.24 Scour development at C.S.5 for LB_MSA_OT_q and LB_MSA_SO_q. a. Y =330-mm; b. Y =460-mm; c. Y =600-mm; d. Y =740-mm

At point D (Figure 4.24), the temporal development of scour diverged after about 1-hour for the unarmoured (by riprap) bed areas (Y = 330 and 460-mm), and after more than 40-hours for the armoured bed areas (Y = 600 and 740-mm). Temporal differences of scour between these two flows corresponded with the flow differences as sketched in Figure 4.23.

As discussed previously, water depth influences scour. At the bridge section, for OT flows, the bridge deck separated the flow and the overtopping discharge could not influence the scour at the bridge section. However, downstream of the bridge, the overtopping flow and
the flow under the bridge deck merged to form one whole flow and the flow depth could affect scour.

The effect of flow distribution at the approach section has been discussed previously. Figures 4.25a and 4.25b show the bed bathymetries at the dynamic equilibrium state for LB_MSA_OT_q and LB_MSA_SO_q. Bed bathymetry measurements at the end of the experiment show that bed forms did not migrate into the scour hole for LB_MSA_OT_q, while for LB_MSA_SO_q, a dune trough deepened the scour by about 30-mm at the bridge section. Obviously, the OT flow produced a deeper and more elongated scour hole than the SO flow. The maximum scour point for the OT flow moved further downstream than for the SO flow, which is similar to the findings by Hong et al. (2015). This scour phenomenon (with constant approach flow intensity, OT scour depth > SO scour depth) occurred in all the six live-bed pressure flow comparison groups. The scour hole for the OT flow was about 50% deeper than that for the SO flow. Three possible causes are listed below:

1. As shown in Table 4.8, at the equilibrium state, OT flow had about 15% more of the total discharge flowing beneath the bridge deck than in the SO flow. This could be the primary reason for the scour differences.

2. For the OT flow, the overtopping discharge merged with the discharge below the bridge to form one whole flow at the downstream side of the bridge. Flow energy integrated along flow depth in the OT flow exceeds that in the SO flow. As discussed before, scour depth increases with the flow depth for abutments in the intermediate-length range.

3. OT flow had larger amplitude bed forms than SO flow, and larger sand dunes intensify the scour (see Section 4.2.3).
**Figure 4.25** Scour bathymetry at the dynamic equilibrium state for a. LB_MSA_OT_q b. LB_MSA_SO_q

**Table 4.8** Discharge under the bridge deck at the equilibrium state

<table>
<thead>
<tr>
<th>Experiment</th>
<th>$Q_{\text{submerged}}$ (OT)</th>
<th>Experiment</th>
<th>$Q_{\text{submerged}}$ (SO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LB_SSA_OT_q</td>
<td>61.6</td>
<td>LB_SSA_SO_q</td>
<td>55.1</td>
</tr>
<tr>
<td>LB_SSA_OT_Q</td>
<td>86.9</td>
<td>LB_SSA_SO_Q</td>
<td>73.0</td>
</tr>
<tr>
<td>LB_MSA_OT_q</td>
<td>62.7</td>
<td>LB_MSA_SO_q</td>
<td>54.8</td>
</tr>
<tr>
<td>LB_MSA_OT_Q</td>
<td>87.4</td>
<td>LB_MSA_SO_Q</td>
<td>72.7</td>
</tr>
<tr>
<td>LB_LSA_OT_q</td>
<td>61.1</td>
<td>LB_LSA_SO_q</td>
<td>57.7</td>
</tr>
<tr>
<td>LB_LSA_OT_Q</td>
<td>88.9</td>
<td>LB_LSA_SO_Q</td>
<td>77.3</td>
</tr>
</tbody>
</table>

**Note:** $Q_{\text{OT}}$ is assumed to reduce by 30% during every OT experiment.
As shown in Figure 4.26, with the same flow intensity in the main channel, the magnitude of the maximum scour depth increased as the flow type changed from FS to SO and to OT flows. Detailed flow patterns, which can also be used to explain this trend, are in Chapter 5.

**Figure 4.26** Variation of $d_s$ with $L_a/B_f$ , a. relatively small flow intensity group; b. relatively large flow intensity group (see Table 4.1)
4.4.2 The effect of abutment length on scour

Melville (1992) found that for unprotected abutments situated in a rectangular channel, local scour depth increased with increasing abutment length, but at a decreasing rate as scour depth approached a limit. Melville et al. (2006) found similar results for protected abutments situated in compound channels. Both of these studies featured a clear-water scour regime. In this study, because a live-bed scour regime existed in the main channel, bed-forms caused some slight deviations, as shown in Figure 4.26.

Another approach for understanding the effect of abutment length is the dependence of scour depth on a discharge contraction ratio. This parameter is fundamentally determined by the abutment length (if abutment length is the only variable) and it has extensive applicability, especially in compound channels. Since the majority of the scour holes were located in the region of the main channel and main channel bank (even for the long setback abutment), unit discharge contraction ratio in the main channel, \( q_{m2}/q_{m1} \) is used herein. Nearly linear relationships exist between \( q_{m2}/q_{m1} \) and \( y_{\text{max}}/y_{m0} \) (Figures 4.27a and 4.27b), where \( y_{m0} \) is the mean flow depth in the whole main channel far downstream of the bridge. It should be noted that \( q_{m2} \) includes the overtopping discharge and \( y_{m0} \) is used to account for the geometrical nonuniformity in the main channel and on main channel bank.

The effect of abutment length on scour development is distinct. A short-setback abutment generally has more uniform flow at the contracted section. The disturbed riprap rapidly re-armoured a newly formed main channel bank, leading to a uniform erosion process at all measurement positions along C.S.5 (Figure 4.28a). This uniform erosion process resulted in an overall undercutting in the contracted section.
Figure 4.27 Normalized scour depth, $\frac{y_{max}}{y_{m0}}$, as a function of $\frac{q_m^2}{q_m^1}$: a. for the low flow intensity group; b. for the high flow intensity group

For the long setback abutment, or if the abutment did not extend into the floodplain ($L_a=0$), the erosion rate at different transverse positions (mostly unarmoured by riprap) varied greatly,
depending on the erosive forces induced by the flow and the erosion resistance determined by the boundary condition. The ratio of bed shear stress and the threshold shear stress, $\tau/\tau_c$, is used to estimate the erosion rate. Here,

$$\tau = C_e E = \frac{1}{2} C_e \rho \left( u'^2 + v'^2 + w'^2 \right)$$  \hspace{1cm} \text{Equation 4.12}

in which $\tau$ is the time-averaged shear stress, $C_e (=0.19)$ is an empirical coefficient and $E$ is the turbulent kinematic energy. The velocity measurements for the estimated bed shear stress were all conducted at 10-mm above the bed, following Guan et al. (2014). The threshold shear stress on a plane boundary is estimated from the modified Shields formula proposed by Brownlie (1981), and the estimation of shear stress on the sloping bank is based on the Lane (1955) relation.

For the long setback abutment, the distributions of $\tau/\tau_c$ for each experiment are shown in Figure 4.29a. Results show that sediments near the intersection of the floodplain and the main channel bank are more prone to entrainment. Estimated values of $\tau/\tau_c$ for LB_LSA_OT_q are supported by the temporal development of scour in the first hour (Figure 4.28b). In general, locations with larger $\tau/\tau_c$ feature higher scour development rates, except for the scour development trend at $Y =600$-mm, where the scour was greatly affected by the scour at $Y =460$-mm. The relationship between $\tau/\tau_c$ and scour development indicates that the scour patterns are related to the flow patterns in the un-scoured state.

When there is no abutment in the channel, scour development rates at different transverse locations vary even more than those for a long setback abutment, as shown in Figures 4.29a and 4.29b. $\tau/\tau_c$ values were relatively large on the upper half of the main channel bank, and so were the scour developments at these two positions.
Figure 4.28 Scour development for the experiment a. LB_SSA_OT_q; b. LB_LSA_OT_q; and c. LB_NA_OT_q
4.4.3 The effect of flow intensity on scour

Flow intensity significantly affects the amplitudes and the periodicity of the bed forms, the embedment of riprap, the equilibrium time and the scour depth around the abutment.

The maximum scour depths (including the effect of bed forms) at the dynamic equilibrium state are shown in Table 4.9. Because different channel geometry and flow regimes were
investigated in this study, of interest is a comparison of scour trends. Results (Table 4.9) in this study show that:

- For the FS flow condition, experiments with larger flow intensity \( V_{ml}/V_{mlc} \approx 1.4 \) produced deeper scour depths than smaller flow intensity \( V_{ml}/V_{mlc} \approx 1.0 \).
- For the SO and OT flow conditions, experiments with smaller flow intensities \( V_{ml}/V_{mlc} \approx 1.0 \) produced deeper maximum scour depths (half dune height considered) than larger flow intensities \( V_{ml}/V_{mlc} \approx 1.4 \).
- Scour depth difference between these two flow intensities was about 10% in each flow condition.

A clear relationship between flow intensity and the maximum scour depth could not be obtained from the limited data in this study. Previous research (Melville, 1992, Melville and Coleman, 2000) showed that, for varying flow intensities, the maximum scour depth normally peaks at \( V/V_c = 1 \); for the lower end of the live-bed regime, e.g., \( 1 < V/V_c \leq 2 \), scour depth normally decreases with flow intensity, due to sediment supply into the scour hole. In this study, similar decreasing trends were found for SO and OT flows.

A possible explanation for FS flow reacting differently to pressure flows in terms of flow intensity is as follows: The dominant scouring feature in FS_q experiments was the geotechnical failure of the main channel bank, which was exacerbated by increasing flow intensity; all LB_FS_q experiments did not reach a live-bed condition across the whole main channel during the whole experiment, which can be corroborated by the propagation of bed forms over time at \( Y = 100 \) and 330-mm. As an example, Figure 4.30e shows that for LB_MSA_FS_q, very small-amplitude bed forms were observed at \( Y = 100 \)-mm, and no bed forms were observed at \( Y = 330 \)-mm. The effect of flow intensity on scour development is
shown in Figure 4.30. Bed forms passed by the abutment more frequently when $V_{ml}/V_{m1c} \approx 1.4$; this significantly reduced the time required to achieve dynamic equilibrium. For similar $V_{ml}/V_{m1c}$, scour depth at the bridge section was similar for pressure flows; for FS flow scour depth was less. Though the scour depths altered little (about 10%) when the flow intensity increased by 40%, for the higher flow intensity, abutments were subjected to more severe local scour around the abutment toe. In addition, sediment winnowing and the embedment of riprap increased the rate of undermining. These observations indicate that scour depth should not be used as the sole criterion for evaluating the stability of the abutment, especially in situations when the scour hole is located downstream of the abutment. Therefore, in this study, scour developments were recorded at C.S.5, where turbulence intensity downstream of the bridge was found to be the highest (See Chapter 5).

**Table 4.9** The maximum scour depth for experiments in the LB 1.54-m wide channel

<table>
<thead>
<tr>
<th>Flow condition</th>
<th>$L_d/B_f$</th>
<th>$d_s$ (mm)</th>
<th>$d_s$ (mm)</th>
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<tr>
<td></td>
<td>$V_{ml}/V_{m1}\approx1.0$</td>
<td>$V_{ml}/V_{m1}\approx1.4$</td>
<td>$V_{ml}/V_{m1}\approx1.4$</td>
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<tr>
<td>FS</td>
<td>0.8</td>
<td>214.0</td>
<td>&lt; 234.0</td>
</tr>
<tr>
<td></td>
<td>0.65</td>
<td>157.8</td>
<td>&lt; 175.3</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>166.2</td>
<td>&lt; 183.7</td>
</tr>
<tr>
<td>SO</td>
<td>0.8</td>
<td>300.0</td>
<td>&gt; 270.0</td>
</tr>
<tr>
<td></td>
<td>0.65</td>
<td>262.3</td>
<td>&gt; 226.7</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>204.0</td>
<td>&gt; 180.4</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>172.7</td>
<td>&gt; 165</td>
</tr>
<tr>
<td>OT</td>
<td>0.8</td>
<td>378.0</td>
<td>&gt; 353.0</td>
</tr>
<tr>
<td></td>
<td>0.65</td>
<td>352.5</td>
<td>&gt; 317.0</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
<td>277.8</td>
<td>&gt; 260.0</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>188</td>
<td>$\approx$ 189</td>
</tr>
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</table>
Figure 4.30 Scour development for each experiment with medium setback abutment, at five transverse positions on C.S.5: a. LB_MSA_OT_q, b. LB_MSA_OT_Q, c. LB_MSA_SO_q, d. LB_MSA_SO_Q, e. LB_MSA_FS_q, and f. LB_MSA_FS_Q
It is generally recognized that the scour hole is located downstream of the abutment toe when scour countermeasures are used (van Ballegooy 2005, Ettema et al. 2010, Hong et al. 2015). Melville et al. (2006) pointed out that a fundamental design requirement is the provision of toe protection to mitigate the effect of scour hole development near the base of the embankment batter. More importantly, it is the scour at the bridge section that directly threatens the safety and the geotechnical stability of the abutment. Scour data at C.S.4 are selected for discussion of scour at the bridge section.

Figures 4.31a, 4.32a and 4.33a present the bed bathymetries at the end of each experiment, for a long setback, a medium setback and a short setback abutment, respectively. Figure 4.33a includes two additional experiments: LB_SSA_SO_q was repeated with smaller riprap, and LB_SSA_OT_Q was repeated with a geotextile filter beneath the apron. Though the flow condition, abutment length, the size of the riprap, and the dimension of the apron varied greatly among experiments, the main channel sloping bank behaved always in the same manner: retreat towards the abutment toe, and maintenance of a slope of approximately H:V = 2:1. This finding may provide an alternative method of assessing the geotechnical slope-stability, particularly for possible failures in pressure flows.

Figures 4.31b, 4.32b and 4.33b show the conceptual scour bathymetries, for the three lengths of abutments. Bed forms in the main channel are removed and simplified channel geometries are presented. The stability threshold is when $W_s = 0$, where $W_s$ is defined as the residual apron width in the equilibrium state. Results suggest that, when the abutment toe is protected by rock riprap aprons, the final bed bathymetry at the bridge section is strongly related to the extent of the abutment. Results also suggest that the bed bathymetry at the bridge section may
be predictable, if the angle of the re-armoured slope, $\beta$ (Figures 4.31b, 4.32b and 4.33b and 4.34), and the scour depth in the main channel, $d_s$, are available. Here, $\beta \approx 27^\circ$.

![Diagram a](image1)

![Diagram b](image2)

**Figure 4.31** Bathymetry at C.S.4 at the equilibrium state for long setback abutments **a.** measured bed bathymetry and **b.** schematic bed bathymetry
Figure 4.32 Bathymetry at C.S.4 at the equilibrium state for medium setback abutments. 

- Figure a: Measured bed bathymetry.
- Figure b: Schematic bed bathymetry.

**Figure 4.32** Bathymetry at C.S.4 at the equilibrium state for medium setback abutments. 

- Figure a: Measured bed bathymetry.
- Figure b: Schematic bed bathymetry.
Figure 4.33  Bathymetry at C.S.4 at the equilibrium state for short setback abutments a. measured bed bathymetry and b. schematic bed bathymetry
Figure 4.34 Schematic diagram of the layout of the riprap apron around the bridge abutment after scour; a. before scour and b. after scour

Figure 4.34a shows the schematic layout of the riprap apron at the abutment toe. Here, $W_{apron}$ is the width of the apron at the start of the experiment and $n_{apron}$ is the nominal number of riprap layers in the apron. HEC-23 (Lagasse et al. 2009) recommends that $n_{apron}$ should not be less than 1.5 and $n_{apron} = 2$ was used in this study. Figure 4.34b shows the idealized redistribution of riprap rocks (one layer) at the threshold of abutment stability. Here, $l_s$ is the slope length of the riprap armour in the equilibrium state; $h_D$ is the pre-scour elevation.
difference between the floodplain and the main channel and; $d_{s-MC}$ is the scour depth measured from the original elevation of the main channel. The following equations can be obtained for the threshold condition.

\[ l_s = n_{apron} W_{apron} \quad \text{Equation 4.13} \]

\[ \sin \beta = \frac{h_D + d_{s-MC}}{l_s} \quad \text{Equation 4.14} \]

Therefore,

\[ n_{apron} \geq \frac{h_D + d_{s-MC}}{W_{apron} \sin \beta} \approx 2.2 \frac{h_D + d_{s-MC}}{W_{apron}} \beta \approx 27^\circ \quad \text{Equation 4.15} \]

is suggested as a riprap apron design relationship.

### 4.4.5 Scour at the downstream side of the bridge

#### 4.4.5.1 Temporal development of scour in the scour hole

For an abutment with countermeasures, the position of the maximum scour depth moves downstream in the developing stage and the scour hole will eventually be situated some distance downstream of the abutment. This distance is strongly dependent on the experimental environment (van Ballegooy 2005, Ettema et al. 2010, Hong 2012). For experiments with a short setback abutment, one ultrasonic depth sounder was tentatively placed at the expected distance downstream of the abutment of the maximum scour position, based on laboratory experience. The ultrasonic depth sounder was purposely placed at lateral position $Y = 460$-mm, where scour was not significantly affected by the migrating bed forms. Records of temporal development of scour in the scour hole were thereby obtained for four experiments: LB_SSA_SO_q, LB_SSA_OT_q, LB_SSA_SO_Q and LB_SSA_OT_Q. In Figure 4.35, the exact position of the ultrasonic depth sounder is marked in red in the bed bathymetry of each experiment for the dynamic equilibrium state. The corresponding
temporal development of scour is shown in Figure 4.36, and is re-plotted in Figure 4.37 with logarithmic X and Y axis scales.

Results show that, for the scour hole located at the downstream side of the abutment, the temporal developments of scour were similar to those around an unprotected circular pier (Melville and Chiew, 1999); later, they will also be shown to be similar to those at the upstream corner of an unprotected vertical abutment (see Chapter 7). Because a small range of flow intensities was used in this study, the effect of flow intensity was not considered. The results show that at the probable maximum scour position, about 60% of the equilibrium scour depth was reached in just 10% of the time required to reach equilibrium.

**Figure 4.35** Position of the ultrasonic depth sounder (marked in red) in the equilibrium bathymetry for each of the experiments
Figure 4.36 Scour development in the scour hole located downstream of the abutment for the four short setback pressure flow experiments.

Figure 4.37 Scour development in the scour hole of the four short setback pressure flow experiments.
The time and scour depth data are well described by the following equation, which is shown in Figure 4.37 as a dashed line:

\[
\frac{d}{d_{se}} = 1.56 \exp \left\{ -0.45 \left( \frac{t}{t_c} \right)^{-0.325} \right\}
\]

Equation 4.16

in which \( R^2 = 0.90–0.97 \). Clearly, further research is required to confirm this temporal development finding, in particular, special attention should be paid to the effect of flow intensity.

### 4.4.5.2 Scour depth in the scour hole

Ettema et al. (2010) viewed abutment scour as essentially inseparable from contraction scour, and proposed that abutment scour depth should be an amplification of long-contraction scour:

\[
\text{abutment scour depth} = (\text{coefficient}) \times (\text{long – contraction scour depth})
\]

Equation 4.17

The “coefficient” term accounts for the large-scale turbulence structures and the non-uniformity of flow. The “long-contraction scour depth” term can be estimated based on the live-bed contraction scour relationship proposed by Laursen (1960),

\[
y_{\text{max}} = y_i \left( \frac{q_2}{q_1} \right)^{6/7} = y_c
\]

Equation 4.18

Equation 4.18 was derived for an unprotected abutment model extending into the main channel, which is different from the configuration in this study. To better understand the long contraction scour in a configuration similar to that used in this study, several long-contraction experiments were conducted, and the results are reported in Chapter 7. One example of long contraction with an apron-protected abutment is shown in Figure 4.38. Long-contraction results show that riprap slid down and covered the eroded main channel bank, as described previously for a short contraction (see Section 4.2.3). The simplified cross-sectional channel geometries of long contractions are illustrated in Figures 4.39a, 4.39b and 4.39c, for the long, medium and short setback abutments, respectively. Here, \( y_{ml} \) is defined as the mean flow
depth in the main channel at the approach section, and $\bar{y}_{mc}$ is defined as the mean flow depth in the contracted section of the main channel of a long contraction scour.

Assuming that no abutment failure occurred in long contraction experiments, and using Laursen’s long contraction theory for live-bed scour, the following equations can be written:

$$\bar{y}_{mc} = \frac{\kappa_{m} y_{m1}}{\left(\frac{q_{m2}}{q_{m1}}\right)^{6/7}}$$  \hspace{1cm} \text{Equation 4.19}

$$y_{max} = \frac{\kappa_{G} C_{T} y_{m1}}{\left(\frac{q_{m2}}{q_{m1}}\right)^{6/7}} = r_{T} y_{m1}\left(\frac{q_{m2}}{q_{m1}}\right)^{6/7}$$  \hspace{1cm} \text{Equation 4.20}

in which $\kappa_{G}$ accounts for the geometry nonuniformity in the main channel, and $C_{T}$ accounts for the influences of macro turbulence generated by flow passing into the bridge waterway. $r_{T} = \kappa_{G} / C_{T}$, is the depth-amplification factor. Since the same main channel geometry was used in this study, $\kappa_{G}$ cannot be discussed here and is assumed to be a constant. The dimensionless flow data are shown in Table 4.10.

**Figure 4.38** Equilibrium bed bathymetry for a long-contraction experiment with the abutment situated on the floodplain and the abutment protected by an apron
Figure 4.39 The simplified cross-sectional channel geometries of long contractions for a. long setback abutment; b. medium setback abutment; c. short setback abutment. Note: the maximum scour depths were located in the main channel area; therefore $d_{MC}$ is measured from the average main channel bottom level to the deepest scour depth. Note: the bottom horizontal reference line is arbitrarily selected, and $y_{mc}$ is calculated using Equation 4.20.
Table 4.10 Key dimensionless flow parameters for the live-bed study

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<th>Experiment</th>
<th>$y_m$</th>
<th>$q_{m2}/q_m$</th>
<th>$y_mC$</th>
<th>$y_{max}$</th>
<th>$y_{max}/y_mC$</th>
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Note: $y_{max}$ represents measured scour depth, which is not shown in Figure 4.39.

The scour and flow data are discussed in terms of the normalized maximum flow depth in the main channel, $y_{max}/y_m$ and $y_{max}/y_mC$; the normalized long-contraction scour depth in the main channel, $y_mC/y_m$; and the unit discharge ratio in the main channel, $q_{m2}/q_m$. Figure 4.40 shows the relationships between $y_{max}/y_m$ and $q_{m2}/q_m$, and between $y_mC/y_m$ and $q_{m2}/q_m$. Results show that for all the experiments, $y_{max}>y_mC$ as expected. When the abutment does not intrude into the floodplain at all, $q_{m2}/q_m \approx 1.0$. Four pressure flow experiments were conducted without an abutment on the floodplain: LB_NA_SO_q, LB_NA_OT_q, LB_NA_SO_Q and LB_NA_OT_Q. For these experiments, results were...
found to consistently give $y_{max}/y_{m1} \approx 1.4$. At slightly larger values of $q_{m2}/q_{m1}$ ($q_{m2}/q_{m1} \approx 1.1$, mainly occurring for LSA experiments), $y_{mC}/y_{m1}$ values are slightly larger than 1.0 and $y_{max}/y_{m1}$ values fluctuate around 1.5. Scour in the range of $q_{m2}/q_{m1} \approx 1.1$ is mainly attributable to macro turbulence. As the abutment extends further towards the main channel, both $q_{m2}/q_{m1}$ and $y_{mC}/y_{m1}$ increase to about 1.5, and $y_{max}/y_{m1}$ values peak at about 2.2. Scour in the range of $q_{m2}/q_{m1} \approx 1.5$ is attributable to both the contraction (laterally and vertically) and macro turbulence. The values of $y_{max}/y_{mC}$ reach the maximum of 1.88 for $q_{m2}/q_{m1} \approx 1.15$.

The data are re-arranged and plotted in Figure 4.41 and Figure 4.42, to highlight the effect of abutment length and flow condition on the relationship between $q_{m2}/q_{m1}$ and $y_{max}/y_{mC}$, respectively. Figure 4.42 shows that the upper-bound envelope is delineated mainly by OT flow data, suggesting that the depth-amplification factor for the OT flow is larger than that for the other two flow conditions. It should be noted that scour mainly occurred around the main channel bank area in this study, and flow data in the main channel are used for all the three abutment lengths. Previous researchers usually developed a relationship between flow data and scour data on the floodplain for the long setback abutment (e.g., Hong et al. 2015). Scatter prevails in the relationships between $y_{max}/y_{mC}$ and $q_{m2}/q_{m1}$, which shows that the relationship between the amplification factor $r_i (= y_{max}/y_{mC})$ and $q_{m2}/q_{m1}$ is imprecise. This imprecise relationship is because $r_i$ not only accounts for the effect of channel geometry, turbulence structures, abutment shape, erodibility of the embankment and the channel, but also the effect of vertical contraction; whereas $q_{m2}/q_{m1}$ is mainly related to the channel geometry and the lateral and vertical contraction.
**Figure 4.40** Variations of $y_{\text{max}}/y_{m1}$ and $\bar{y}_{mC}/\bar{y}_{m1}$ with unit-discharge ratio $q_{m2}/q_{m1}$

**Figure 4.41** Variation of $y_{\text{max}}/\bar{y}_{mC}$ with $q_{m2}/q_{m1}$, showing the differences between different abutment lengths
Figure 4.42 Variation of $\frac{y_{\text{max}}}{y_{mC}}$ with $\frac{q_{m2}}{q_{m1}}$, showing the differences between flow conditions

4.5 Experiments in the LB 2.4-m wide channel

For experiments in the LB 2.4-m wide channel, behaviours of the main channel bank, propagation of bed forms, and behaviours of riprap in the apron were observed to be similar to those presented in Section 4.2, and therefore they are not discussed here. Scour results at the bridge section and in the scour hole are presented in the following.

4.5.1 Scour at the bridge section

Scour results at C.S.4 of the LB 2.4-m wide channel are similar to those presented in Section 4.4.4 for the LB 1.54-m wide channel, as shown in Figures 4.43 and 4.44. For the investigated flow conditions (see Table 4.2), dynamic equilibrium bed profiles at C.S.4 were featured with H:V = 2:1 side slopes adjoining the abutment toe. These experiments were terminated because scour reached the abutment toe, and further erosion and undermining of the abutment were expected to occur for longer scour durations. Therefore, it is suggested to use Equation 4.15 as one of the rock riprap apron design criteria. In addition, in reality, it is suggested to monitor the residual apron width (if rock riprap apron is used as scour
countermeasure) on a regular basis. This geometrical feature at the bridge section (side slope H:V=2:1) was consistently observed in this live-bed study (and the subsequent clear-water studies), and is discussed in detail in Chapter 6.

**Figure 4.43** Bed bathymetry at C.S.4 at the equilibrium state for short setback abutments (LB 2.4-m wide channel); **a.** measured bed bathymetry profiles; **b.** a schematic bed bathymetry profile
Figure 4.44 Bed bathymetry at C.S.4 at the equilibrium state for medium setback abutments (LB 2.4-m wide channel); a. measured bed bathymetry profiles; b. a schematic bed bathymetry profile

4.5.2 Scour at the downstream side of the abutment

At the end of each experiment, the bed morphology was obtained, as exemplarily shown in Figure 4.45. Scour history was tentatively measured at the expected maximum scour position. The locations of measurements were marked by white circles, as shown in Figures 4.45(a-c). Records of temporal development of scour in the scour hole were obtained for three experiments: LB_2.4m_MSA_FS_q, LB_2.4m_MSA_SO_q and LB_2.4m_MSA_OT_q. General scour features in the LB 2.4-m wide channel are the following:
• For FS flows, equilibrium scour depth is quickly achieved, and no obvious scour holes occur; instead, the scour pattern features a relatively uniform scour depth in the contracted area, and is significantly influenced by closely-spaced bed forms, as exemplarily shown in Figures 4.45a and 4.46.

• For SO and OT flows, relatively deep scour holes occur downstream of the abutment, as exemplarily shown in Figures 4.45b and 4.45c. Small-amplitude sand dunes propagate in the main channel, causing limited influence on the maximum scour depth (Figure 4.46). With similar flow intensities at the approach section, SO flows in this 1:30 scale study have similar scour developments and similar (or slightly larger) scour depths to those of OT flows (Figures 4.45b, 4.45c and 4.46). In terms of the maximum scour depth, the scour trend in this 1:30 scale study, SO ≥ OT, is different from that for the 1:45 scale study, in which results show an OT ≥ SO trend. Hong (2012) found a trend of SO > OT in his 1:30 scale scour experiments, in which flow intensities were also similar at the approach section; he attributed the scour difference between different pressure flows to flow relief of OT flows. However, magnitude differences of $Q_{\text{submerged}}$ in this live-bed study suggest that, with similar flow intensities at the approach section, the OT flow may cause equivalent scour depth of, or slightly deeper scour depth than, the SO flow (Table 4.8 and Table 4.11). Results suggest that flow relief of OT flows is not the decisive factor of the scour difference between SO and OT flows. Scour difference between SO and OT flows may be determined by: the geometric factor, the channel geometry factor, the flow regime factor and the bridge deck submergence level. It is difficult to determine the contribution of each factor from limited experiments in this study. Combining results from this chapter and from Hong (2012) shows that OT flows unnecessarily cause more scour depth than SO flows; for similar flow intensities at the approach section, it is possible to have either
SO ≥ OT or SO < OT, in terms of the maximum scour depth. In addition, varying differences of scour depths between SO and OT flows highlight the need to investigate the effect of different levels of $Q_{OT}/Q$ on scour depth; the investigation could consider the vertical and the lateral flow distributions beneath the bridge deck, for both the initial and the equilibrium states of scour.
Figure 4.45 Bed morphology at the end of the experiment; a. LB_2.4m_MSA_FS_q; b. LB_2.4m_MSA_SO_q and c. LB_2.4m_MSA_OT_q. Note: colour scale unit: mm; a relatively small range of colour scale is used for the FS flow to highlight the bed forms; locations of the maximum scour depth on the floodplain and in the main channel are marked with a red star and a red cross, respectively; locations of the scour history measurements are marked with white circles.

Figure 4.46 The temporal development of scour at the expected positions of maximum scour in the LB 2.4-m wide compound channel.
Table 4.11 Submerged discharge at the initial and the equilibrium states, for LB_2.4m_MSA_SO_q and LB_2.4m_MSA_OT_q

<table>
<thead>
<tr>
<th></th>
<th>LB_2.4m_MSA_SO_q</th>
<th>LB_2.4m_MSA_OT_q</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_{\text{submerged}}$ (L/s)</td>
<td>122.6</td>
<td>114.6</td>
</tr>
<tr>
<td>Equilibrium</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$Q_{\text{submerged}}$ (L/s)</td>
<td>122.6</td>
<td>139.1</td>
</tr>
</tbody>
</table>

4.6 Scour depth results for all the live-bed experiments

The combined maximum scour depth results in this live-bed study are shown in Figures 4.47, 4.48 and 4.49.

Figure 4.47 shows that the amplification factor, $r_T$, varies differently with $q_{m2}/q_{m1}$ for different flow conditions:

- For FS flows, results show an increasing trend of $r_T$ with $q_{m2}/q_{m1}$ increasing from 1.07 to 1.26, and then a decreasing trend of $r_T$ with $q_{m2}/q_{m1}$ increasing from 1.26 to 1.46; the maximum value of $r_T$ is 1.58 for the investigated flow conditions.

- For SO flows, results show an increasing trend of $r_T$ with $q_{m2}/q_{m1}$ increasing from 1.15 to 1.52; the maximum value of $r_T$ is 2.06 for the investigated flow conditions.

- For OT flows, the trend is similar to that of FS flows. Results show an increasing trend of $r_T$ with $q_{m2}/q_{m1}$ increasing from 1.00 to 1.15, and then a decreasing trend of $r_T$ with $q_{m2}/q_{m1}$ increasing from 1.15 to 1.36; the maximum value of $r_T$ is 1.87 for the investigated flow conditions.

It should be noted that scour trends for FS and OT flows in this study are similar to FS trends obtained by Ettema et al. (2010).
Figure 4.48 show that the amplification factor, $r_T$, is weakly related to $L_u/B_f$. This weak relationship is expected, because $r_T$ is significantly affected by flow conditions and channel geometries. Ranges of $r_T$ values widen for $L_u/B_f$ increasing from 0 to 0.8. The upper limit of $r_T$ values are attained with pressure flows, presenting a linear relationship with $L_u/B_f$:

$$r_T = 0.75 L_u/B_f + 1.4 \quad 0 \leq L_u/B_f \leq 0.8$$  \hspace{1cm} \text{Equation 4.21}

Through regression analysis (Figure 4.49), an approximately linear relationship is obtained between $\frac{y_{\text{max}}}{y_{\text{m0}}}$ and $(q_{m2}/q_{m1})^{6/7}$, suggesting the applicability of Laursen’s long contraction theory on experimental conditions in this live-bed study. All the data lie in a narrow band, with upper and lower envelopes expressed by the following linear equations:

$$\frac{y_{\text{max}}}{y_{\text{m0}}} = 3.2 \left( \frac{q_{m2}}{q_{m1}} \right)^{6/7} - 1.5 \quad \text{Equation 4.22a}$$

$$\frac{y_{\text{max}}}{y_{\text{m0}}} = 1.5 \left( \frac{q_{m2}}{q_{m1}} \right)^{6/7} - 0.45 \quad \text{Equation 4.22b}$$

in which $1.02 \leq q_{m2}/q_{m1} \leq 1.52$. 
Figure 4.47 Variations of $y_{\text{max}} / \bar{y}_C$ and $q_{m2}/q_{m1}$ for a. live-bed FS flows, b. live-bed SO flows and c. live-bed OT flows

Figure 4.48 Variations of $y_{\text{max}} / \bar{y}_C$ and $L_d/B_f$ for all the live-bed scour experiments
Figure 4.49 Variations of $\frac{y_{max}}{y_{m0}}$ and $(q_{m2}/q_{m1})^{6/7}$ for all the live-bed scour experiments

4.7 Comparison with other research results

The maximum scour depths from this study are compared with results from five previous studies: Froehlich (1989), Melville et al. (2006), Sturm (2006), Ettema et al. (2010) and Hong et al. (2015).

Froehlich (1989)

HEC-18 (2012) recommends predicting the abutment scour depth using a regression equation developed by Froehlich (1989). This equation was determined by multiple linear regression analysis based on a total of 170 live-bed experiments under free-surface flow conditions, with 98% of the data enveloped by the following equation:
\[
\frac{d_{s-MC}}{y_2} = 2.27K_sK_\theta \left( \frac{L'}{y_2} \right)^{0.43} Fr^{0.61} + 1
\]  
Equation 4.23

in which the definition of each parameter can be found in Chapter 2. Although there are multiple differences between the Froehlich study and this study, considering that they both reveal live-bed scour patterns around hydraulic structures, there is a value in conducting the following comparison.

In this study, the flow area obstructed by the bridge deck was taken into consideration when calculating \( Fr \) for pressure flows. Because the maximum scour depths were located in the main channel area, \( d_{s-MC} \) is measured from the average main channel bottom level to the deepest scour depth (see Figure 4.39). The length of embankment blocking “active” flow can be determined from the flow distributions at C.S.1. Similar magnitudes of flow intensity were achieved across the floodplain for each experiment (as exemplarily shown in Figure 4.50), and therefore \( L' = L_a \). The comparison between the results from this study and Equation 4.23 is shown in Figure 4.51.

Equation 4.23 was derived from FS flows only, but it can be seen from Figure 4.51 that Equation 4.23 envelopes almost all the results (except for SO flows in the LB 2.4-m wide channel) from this live-bed study when \( L' \) and \( Fr \) are defined as noted above. Over-prediction by Equation 4.23 is mainly because data assembled by Froehlich (1989) were all from experiments conducted with unprotected abutments in rectangular channels. Scour depth in compound channels may be less than scour in rectangular channels (Melville, 1995). It is difficult to determine the contribution of vertical contraction in the results from this study, because multiple variables, i.e., flow conditions, channel configurations and the status
of countermeasures, are involved at any site. The safety factor (value “1” in Equation 4.23) needs to be increased to “2” to envelope all the data in this live-bed study.

**Figure 4.50** Flow distribution examples at C.S.1; **a.** for the 1:45 scale flume and **b.** for the 1:30 scale flume
Based on scour results under clear water conditions, Melville et al. (2006) proposed Equations 4.24 - 4.25, to predict local scour depth at spill-through abutments situated on the floodplain of a compound channel:

\[ d_{sf} = d_s + F_s (y_m - y_f) \]  
\[ F_s = 1 - \left(1 - \frac{L_a}{B_f}\right)^{2(a/B_f - 1)} \]

\[ , \quad a/B_f > 1 \]

in which the definition of each parameter can be found in Chapter 2.

In this study, \( \alpha = B \) because all scour holes extended to the channel wall on the main channel side.

**Figure 4.51** Comparison with the equation developed by Froehlich (1989)
In Equation 4.24, the values of $d_s$ were based on the scour prediction method proposed by Melville (1997) and Melville and Coleman (2000):

$$d_s = K_x K_f K_d K_r K_y L$$

Equation 4.26

These $K$ factors are described in detail in Chapter 2.

In this study, spill-through abutments were used and they were aligned perpendicular to the approach flow ($K_x = 0.5$, $K_d = 1.0$), uniform sediment was used ($K_r = 1.0$), and dynamic equilibrium was achieved ($K_y = 1.0$). Melville (1992, 1995) suggested that $K_f$ should be taken equal to unity because live-bed conditions exist in the main channel, hence $K_f = 1.0$.

The function $F_r$ in Equation 4.25 already accounts for the geometry effect and the model abutments had “intermediate” lengths ($K_g = 1.0$, $K_{yl} = 2\sqrt{L_a y_f}$). Therefore,

$$d_s = \sqrt{L_a y_f}$$

Equation 4.27

Experiments with scour holes located in the main channel and in the main channel bank in Melville et al. (2006) are selected for comparison, as shown in Figure 4.52.

Results show that Equations 4.24 to 4.25 well predict the scour depths in this live-bed study. It is possible, however, that this agreement is achieved because two scour components offset each other: the “reductive” component caused by bed forms (this component is not considered because $K_f = 1.0$) and the “incremental” component caused by vertical contraction (this component is not considered in Equation 4.26). Further investigation of the scour component attributable to vertical contraction, is needed. An additional factor, $K_v$, should be introduced to Equation 4.26 to account for the vertical scour component.
Figure 4.52 Comparison with the formula developed by Melville et al. (2006)

Ettema et al. (2010)

As discussed previously, Ettema et al. (2010) provided an approximate estimation of scour depth based on the long-contraction theory. Scour trends are discussed here in terms of the relationship between the amplification factor \( r_T = \frac{y_{max}}{y_{m0}} \) and the discharge contraction ratio in the main channel \( q_{m2}/q_{m1} \). Comparisons are made for Scour Condition A (scour only occurs in the main channel, where live-bed scour prevailed) and Scour Condition AB (scour extends from the floodplain into the main channel under clear-water scour condition), as shown in Figure 4.53 and Figure 4.54, respectively.

Results in Figure 4.53 and Figure 4.54 show that, except for three outlying points (SO flows in the LB 2.4-m wide channel), pressure flows are associated with a new relationship between
\(\frac{y_{\text{max}}}{y_{mC}}\) and \(\frac{q_{m2}}{q_{m1}}\), which envelopes the existing FS data in both studies. The new envelope is mainly defined by the data from OT flows; a peak value of \(\frac{y_{\text{max}}}{y_{C}}\) of about 1.9 occurs when \(\frac{q_{m2}}{q_{m1}} \approx 1.2-1.3\), where the peak value is also found for the exclusively FS flows from Ettema et al. (2010). Figure 4.53 and Figure 4.54 show that FS data in this study are within or close to the envelope defined by Ettema et al. (2010). Scour condition AB occurring in this live-bed study further relieved the contraction, which in turn resulted in relatively smaller values of \(\frac{y_{\text{max}}}{y_{C}}\). For the limiting condition of “no abutment” in a finite-width channel (\(\frac{q_{m2}}{q_{m1}} \approx 1.0\)), \(\frac{y_{\text{max}}}{y_{C}}\) is approximately equal to 1.4, based on the results from four pressure flow experiments in this study. The three outlying points highlight the need for further studies on large scale simulations of SO flows.

![Figure 4.53](image)

**Figure 4.53** Comparison with the results obtained by Ettema et al. (2010) for a compound channel with fixed floodplain
Figure 4.54 Comparison with the results obtained by Ettema et al. (2010) for a compound channel with erodible floodplain

Hong et al. (2015)

Hong et al. (2015) combined the effect of vertical contraction and lateral contraction through the contraction ratio \((q_2/q_1)\), and extended the scour prediction method for FS flows (proposed by Laursen, 1963) to pressure flows. The extended method is based on the modified long-contraction theory under the clear-water scour regime:

\[
\frac{y_{\text{max}}}{y_0} = r_f \left( \frac{y_1}{y_0} \right) \left[ \frac{V_1}{V_c} \left( \frac{q_2}{q_1} \right) \right]^{6/7}
\]

Equation 4.28

Because live-bed scour mainly occurred in the main channel in this study, flow parameters in the main channel are used, and the flow intensity term “\(V_m/V_{mc}\)” is taken as unity.
Figure 4.55 includes clear-water data obtained by Hong et al. (2015) and by Sturm (2006) for bankline and short setback abutments. Results show that Equation 4.29 is applicable to the research results in this study, particularly for the results obtained with the low flow intensity group. Because of the scour-hole refilling mechanism, scour depths in the live-bed regime are typically less than those in the critical condition (Melville and Coleman, 2000). As discussed previously (Section 4.2.2), this is mostly true for this study. Therefore it is conservative to take \( V_{m1}/V_{mic} = 1 \) for the live-bed regime, and LB_Q data points (marked in red) in Figure 4.55 would move leftward and better align with the best fit line if the influence of the flow intensity is properly taken into account. Results in this study and in Hong et al. (2015) imply that \( q_{m2}/q_{m1} \) can adequately represent total contraction (lateral contraction + vertical contraction) for short setback and bankline abutments.

For all results from this study and for results from Hong et al. (2015), variation of the amplification factor \( r_f = y_{max}/y_C \) with the discharge contraction ratio in the main channel \( (q_{m2}/q_{m1}) \) is shown in Figure 4.56. As discussed previously, \( r_f \) is a function of abutment shape, abutment length, channel configuration and ad-hoc floodplain features including countermeasure status and vegetation status (Ettema et al. 2010). The scatter of the data points in Figure 4.56 is expected, given the reason provided in Section 4.4.5.2. The data points tend to be grouped in different parts of the graph (Figure 4.56), mainly because experimental conditions differ significantly between this study and that by Hong et al. (2015). It can be seen that, for all results from this study and for results from Hong et al. (2015),
$y_{max}/\bar{y}_C$ has an upper bound at $y_{max}/\bar{y}_C \approx 2.0$, and $y_{max}/\bar{y}_C \geq 1.1$ for the majority of the data.

![Graph](image)

**Figure 4.55** Comparison with the formula developed by Hong et al. (2015)
Figure 4.56 Variation of $\frac{y_{\text{max}}}{\bar{y}_C}$ with $q_{m2}/q_{m1}$; comparison with the data obtained by Hong et al. (2015)
Chapter 5. FLOW PATTERNS AROUND ABUTMENTS

5.1 Introduction

Rivers often have compound cross-sections with wide and vegetated floodplains adjoining narrow but deep main channels. During flood seasons, flows around bridges exhibit flow redistribution, local turbulence in the vicinity of bridge piers and abutments, lateral contraction, and possibly vertical contraction. Scour induced by these flow features can lead to bridge failures (Parola et al. 1997; Morris and Pagan-Ortiz 1999; Richardson and Davis 2001). Vertical contraction scour occurs either with submerged orifice (SO) flow (when the upstream flow surface is between the bridge deck girders and bridge superstructure) or overtopping (OT) flow (when the upstream flow surface is above the bridge superstructure) at the bridge section. Compared with local scour and lateral contraction scour, vertical pressure scour has been insufficiently studied, especially for compound channel configurations. Extreme hydrologic events can result in pressure flows, causing bridge damage and loss of life. In 2009, floods with more than 500-year recurrence magnitudes occurred in the Atlanta area, Georgia, U.S.A. The peak stage of the floods reached up to nearly 6.1-m higher than the estimated peak stage of the 500-year flood, causing widespread damage to bridge embankments and abutments with overtopping flows, and resulting in more than $193 million damage and ten deaths (Gotvald and McCallum 2010).

Abutment scour has received close attention in the last 20 years, and a clear delineation of scour classifications, identification of dominant parameter groups, and evaluations of scour formulas have been obtained (Sturm et al. 2011). Given the innate complexity of abutment scour under natural conditions, many of the previous studies focused on idealized channel configurations (e.g. rectangular channel), simple flows (e.g. free surface (FS) flow with either
clear-water or live-bed scour regime) and simplified abutment layouts (e.g. abutment without protection). However in nature, rivers normally have compound channel configurations, and the flow interactions between the floodplain and the main channel has a geometric influence on the scour development. During floods, the clear-water scour regime generally prevails on the floodplain, while at the same time the live-bed scour regime prevails in the main channel. The different sediment entraining capacities affect the scour development in different parts of the channel, with subsequent effect on the maximum scour depth and on the geometry of the scour hole. Also, the foregoing devastating flood example highlights the need for further research of abutment scour, especially close-to-reality investigation on abutment scour under pressure flows. Additionally, scour countermeasures are generally utilized in bridge construction, thus including scour countermeasures in physical modelling should produce more realistic results for use in engineering practice.

Flow characteristics around the abutment are inherently related to the corresponding scour. Over the last 20 years, particular attention has been given to flow characteristics in the equilibrium state of clear-water scour (Kwan and Melville 1994; Melville and Coleman 2000; Dey and Barbhuiya 2005; Dey and Barbhuiya 2006(a); Dey and Barbhuiya 2006(b); Guan et al. 2014). Relationships between the initial flow fields and the equilibrium scour features are relatively obvious for non-protected rigid abutment models because the scour hole tends to have a stable location at the upstream abutment corner during the whole scour development. For erodible abutment models with protection, however, scour development features gradual displacement of the scour hole to the downstream side of the bridge, dynamic armouring when launching apron rocks are used, and even wash-out of embankment fill material. Previous research results have shown that, for an earth-filled embankment with launching apron as protection, the near-bottom turbulence intensities prior to scour could be used as a
scour depth predictor in the clear-water scour regime (Hong et al. 2015). For live-bed scour regimes, relationships between the initial flow field and the final scour bathymetry are more complicated because of the propagation of sand dunes, and even more so for a combination of clear-water and live-bed regimes in a compound channel.

Both bursting phenomena and large-scale vertical vortices play important roles in sediment entrainment (Nezu and Nakagawa, 1993), and hence scour around hydraulic structures. Over the past two decades, developments in velocity measuring instruments, e.g., acoustic Doppler velocimeters (ADVs), allowed improvements in turbulence measurements (Nikora and Goring 1998; Hurther and Lemmin 2001; Garcia et al. 2005; Garcia et al. 2007; Chanson 2008; Hurther and Lemmin 2008; Doroudian et al. 2010; Khorsandi et al. 2012). However, ADV measurements are affected by the measurement environment, and by Doppler noise - a positive bias in the high-frequency range of the power spectrum (Dombroski and Crimaldi 2007). Near the boundary, ADV data should be used cautiously especially when there is a strong velocity gradient within the measuring volume (Dombroski and Crimaldi 2007). The Doppler noise is inherent in the measuring principle (Garbini et al. 1982) and it occurs irrespective of flow conditions, boundary closeness, and the spatiotemporal resolutions.

The purpose of this chapter is to present flow characteristics around setback spill-through abutments situated on the floodplain of a compound channel under three different flow conditions: FS, SO and OT flows. The results will provide a basis for the further evaluations of scour and for optimization of countermeasures.
5.2 Methodology

The experiments (Table 4.1) were conducted in a 1.54-m wide, 1.2-m deep and 45-m long flume, which is supported on two castellated-beams and centrally pivoted, allowing ready adjustment of flume slope. The flume is equipped with a 30-kw sand pump, a 22-kw water pump and a 45-kw water pump. Both of the two water pumps have variable speed controllers to regulate the flow rate. All sediment slurry was guided into the main channel to ensure no sediment transport on the floodplain. A 1.08-m wide floodplain, a 0.26-m wide main channel bank (2:1) and a 0.2-m wide main channel were built along the flume (Figures 5.1 and 5.2). Starting at 30-m downstream from the inlet section, a 4.6-m long test section was located as shown in Figure 5.1. A Cartesian coordinate system is implemented as shown in Figures 5.1 and 5.2. To measure the flow fields prior to scour, the test section was immobilized using metal sheets and coated with near uniform sand ($d_{16} = 0.65$-mm $d_{50} = 0.73$-mm $d_{84} = 0.84$-mm $d_{94} = 1.10$-mm, $\sigma_g = 1.35$). Outside the test section, the floodplain was roughened using uniform rocks, and the main channel was immobilized and coated using the sand as above. The test models were designed to represent a prototype, two-lane bridge at a 1:45 geometric scale (Figure 5.2). A launching apron was used as a scour counter-measure around the abutment. The apron dimensions and the apron-rock size were determined based on the recommendations in HEC-23 (Lagasse et al. 2009). Six different streamwise flume slopes (Table 4.1) were used to achieve uniform flows in this study, and the flow depth along the flume was measured for each experiment. After adjusting the flume slope for each case, preliminary velocity measurements were conducted along the flume to check that uniform flow was generated. To evaluate the quality of construction (maximum deviation is 5-mm), the fixed bed morphology was obtained using ultrasonic depth sounders, with measurement spacing of 1.96-mm in the transverse direction and 50-mm in the streamwise direction. A
four-receiver Vectrino+ down-looking ADV (Nortek, AS) was used to measure the flow field around the abutment. Measurements were conducted at three cross sections (C.S.1, C.S.4 and C.S.5) and along the flume on the downstream side of the bridge (Figure 5.1). The ADV was attached to a very rigid carriage and no noticeable vibration occurred during the measurements; the probe alignment was checked after each movement. For selection of a sampling rate, Nezu and Nakagawa (1993) suggested a dimensionless frequency, \( F_r = f_R L / V > 20 \), as the sampling frequency criterion (\( f_R \) is the recording frequency, \( L \) is the energy-containing eddy length-scale, which can be assumed to be equal to the water depth, and \( V \) is the convective flow velocity). In this study the sampling rate was chosen as 200-Hz (the upper limit of temporal resolution) for all the measurements. Increasing temporal resolution increases Doppler noise energy level. To reconcile this contradiction, the noise-reduction method proposed by Hurther and Lemmin (2001) (hereafter called HLP) was applied. In addition, the parasitic noise was estimated by averaging the two vertical noise levels, to reduce the deviation between the two independent vertical measurements (Blanckaert and Lemmin 2006).

The ADV configurations, measurements and post-process procedures followed the general guidelines of Garcia et al. (2007). In the near-bed flow zone, the sampling height was set at 1 to 2.5-mm, as suggested by Dey et al. (2011) and Guan et al. (2014). Above this zone, the sampling height was set at 2.5 to 7-mm. Since the optimum sampling time for a given turbulence level is case-dependent, a sensitivity analysis to determine sampling numbers was conducted prior to actual measurement, to ensure that at least the second order moment did not vary by more than 2% of that from the longest sampling times. Sensitivity analysis examples of each setup argument can be found in Figures A6 – A8 in the appendix. For each turbulence measurement, typically 24,000 samples were recorded; for highly turbulent zones,
e.g. the near-bed flow zone at the bridge section, at least 60,000 samples were recorded, as suggested by Krogstad et al. (2005). Velocity range selection depended on the measurement environment, and ranged from 10-cm/s (e.g. C.S.1 in LB_SSA_FS_Q, Table 4.1) to 250-cm/s (e.g. C.S.4 and C.S.5 in LB_SSA_SO_Q and LB_SSA_OT_Q), to avoid signal wrapping and to achieve higher accuracy. Flow was well seeded using spherical fused borosilicate glass (8-13 micron, 1.1-g/cc).

Generally, the raw data were first post-processed using WinADV (Wahl, 2000) to remove spikes (using the phase-space threshold method of Goring and Nikora (2002)), and to remove the data with less than 70% signal correlation (COR) and less than 15-dB signal noise ratio (SNR). Afterwards, the HLP method was applied to reduce the noise contained in the measurements. For highly turbulent flows in the near-bed zones, COR was set at average 60%, as applied by Chanson (2008) in his first stage of signal processing in steady flows. Other researchers, including Martin et al. (2002), Cea et al. (2007), Lacey and Rennie (2012), Romagnoli et al. (2012) and Baki et al. (2015), also published results featuring low correlation values. Wahl (2000) noted that samples with COR much less than 70% can still provide good data, especially when both SNR and flow turbulence are high. In general, more than 80% of the data remained after the above post-processing. After following the above procedures, the uncorrected and corrected power spectral density of a representative measurement is shown in Figure 5.3. This representative measurement is from LB_SSA_OT_Q (C.S.4, Y =510-mm, Z = 5-mm above the bottom), in which position the flow is quite turbulent (as shown later in Figures 5.12 and 5.14). The Kolmogorov “-5/3” slope of the spectrum in the inertial subrange super-imposed on Figure 5.3b indicates an effective correction. It should be noted that due to the limitations of down-looking ADVs, flow information for the top 50-mm of the flow is not available.
Figure 5.1 Plan view of the test section for a. a short setback abutment and b. a long setback abutment (unit: mm)

Figure 5.2 Sectional view along the centreline of the bridge (unit: mm)
5.3 Results

5.3.1 Approach section (C.S.1)

Figure 5.4 shows the vertical distributions of the three normalized turbulence intensity components at C.S.1 from LB_SSA_OT_Q, where \( u' \), \( v' \) and \( w' \) are the streamwise, transverse and vertical components, respectively. Relative water depth, \( z_b/y \), ( \( z_b \) is the distance from the measurement point to the bottom) is used here to integrate the floodplain and main channel measurements. At C.S.1, \( u'/V > v'/V > w'/V \) is found, agreeing with predictions by Nezu and Nakagawa (1993) and measurements by Babaeyan-Koopaei et al. (2002) and Koziol (2013) for turbulent open-channel flows. Moving from the floodplain to
the main channel, the distributions of $u'/V$, $v'/V$ and $w'/V$ tend to merge, presenting near-isotropic turbulence. This can be attributed to the three-dimensional nature of the flow around the floodplain-main channel interface (Babaeyan-Koopaei et al. 2002; Koziol 2013). On the floodplain, $w'/V$ increases from the bottom upwards to $z_b/y = 0.2$ and then slightly decreases towards the water surface. This $w'/V$ distribution is different from the corresponding exponential decay equations of Nezu and Rodi (1986) and Nikora and Goring (2000), which were derived for smooth and gravel boundaries, respectively. One reason for this is the presence of roughness elements upstream of C.S.1 in this study. A similar vertical turbulence intensity distribution was found by Koziol (2013) for a vegetated compound channel. The increasing trend of the three normalized turbulence intensity components from $z_b/y = 0.6$ upwards in the main channel agrees with the measurements conducted by Koziol (2013). This increasing trend is due to suppression of the vertical movement of eddies, and to surface waves (Nezu and Nakagawa 1993).

Figure 5.5 shows the vertical distributions of $u'/V$, $v'/V$ and $w'/V$ at $Y = 100$-mm (centre of the main channel) for short setback abutment experiments. From the bottom to $z_b/y = 0.2$, all experiments have decreasing trends of $u'/V$ and $v'/V$. In contrast, the distribution of $w'/V$ is much more uniform. For the depth range $0.3 < z_b/y < 0.5$, the distributions of $u'/V$ and $v'/V$ exhibit a decreasing trend for shallower water experiments (LB_SSA_FS_q and LB_SSA_FS_Q), and are approximately constant for deeper water experiments (LB_SSA_OT_q and LB_SSA_OT_Q). This difference is probably caused by roughness elements. According to Koziol (2013), a more uniform distribution of normalized turbulence intensities (as observed for OT flows) occurs because flow in this depth range in the main channel adjoins roughness-affected flow on the floodplain and features a considerable
momentum exchange between the main channel and the floodplain. Above $z_b/y = 0.5$, mildly increasing trends of $u'/V$, $v'/V$ and $w'/V$ are observed.

![Figure 5.4 Distributions of normalized turbulence intensity components for LB_SSA_OT_Q](image)

![Figure 5.5 Distributions of normalized turbulence intensity components at Y =100-mm for short setback abutment experiments; simplified experiment names are used in the legend](image)

Figures 5.6a and 5.6b show the cross-sectional distributions of normalized overall turbulence intensity $k'/V$ at C.S.1 for LB_SSA_OT_q and LB_SSA_OT_Q, respectively. $k' = \sqrt{u'^2 + v'^2 + w'^2}$, is a different form of turbulent kinetic energy (TKE) representation. The dashed line represents the width-averaged water surface level. It is apparent that $k'/V$ gradually decreases upwards over the whole measurement range. A relatively high magnitude of $k'/V$ was found on the floodplain and at the top of the main channel bank, and a lower magnitude of $k'/V$ occurs in the main channel. Similar results were obtained by Koziol (2013). For the two flow rates, the general distribution pattern of $k'/V$ remained the same while the near bottom (within 20-mm above the bed) turbulence intensity changes significantly. Overall, the $k'/V$ level for the higher flow rate case (LB_SSA_OT_Q) is only 212
5.5% less than that for LB_SSA_OT_q, but in the near bottom region, this difference reaches 23.4%.

Figure 5.6 Normalized overall turbulence intensity distribution at C.S.1 for a. LB_SSA_OT_q and b. LB_SSA_OT_Q

5.3.2 Downstream of the bridge

The near-bed (5-mm above the bed) normalized turbulence intensities along four longitudinal lines at the downstream side of the bridge (see Figures 5.1 and 5.2) are shown in Figures 5.7a, 5.7b and 5.7c, for LB_SSA_FS_Q, LB_SSA_OT_Q and LB_LSA_OT_Q, respectively. The measurements were made at different subsections, i.e., main channel (Y =100-mm), main channel bank (Y =300-mm) and floodplain (Y =510-mm, 560-mm, 610-mm and 860-mm, see Figure 5.2). For the three experiments, the normalized turbulence intensities (\(k/\bar{u}^*, u'/\bar{u}^*, v'/\bar{u}^*\) and \(w'/\bar{u}^*\)) along each longitudinal line all gradually converge to a constant level downstream of the bridge. For the main channel and the main channel bank areas, \(u'/\bar{u}^*, v'/\bar{u}^*\) and \(w'/\bar{u}^*\) increase mildly along the flume for the three experiments. By contrast, on the floodplain, these turbulence intensities experience peaks at C.S.5 for LB_SSA_FS_Q and
LB_SSA_OT_Q, but eventually all decrease to magnitudes similar to those in the main channel and main channel bank. The above trends at different subsections of the channel are probably because of turbulence energy transfer from the floodplain to the main channel (due to inertia, net mass transfer is from the floodplain into the main channel in the discussion area). At C.S.1, these normalized turbulence intensities show consistently similar but smaller magnitudes on the above longitudinal lines. This similarity suggests that the flow may achieve a quasi-equilibrium state within a certain distance downstream from the bridge.
Figure 5.7 Normalized turbulence intensity components along the flume downstream of the bridge for a. LB_SSA_FS_Q, b. LB_SSA_OT_Q and c. LB_LSA_OT_Q
5.3.3 Bridge section (C.S.4 and C.S.5)

It should be noted that the location definition of C.S.4 (downstream side of the bridge deck) and C.S.5 (downstream side of the embankment) in this study follows Hong et al. (2015), who found that, in clear-water experiments, the most turbulent flow occurs at C.S.5. As shown in Figures 5.7 (a-c), the strongest turbulence intensity occurs close to the abutment toe at C.S.5 (for LB_LSA_OT_Q, C.S.4 has the same turbulence intensity as C.S.5), which is consistent with results of Hong et al. (2015). Near-bottom (5-mm above the bed) measurements were conducted at C.S.4 and C.S.5 and are used in all subsequent near-bottom flow analyses. More comprehensive flow measurements were conducted at C.S.4, and the corresponding flow fields are presented.

Figures 5.8 and 5.9 show the distribution of the normalized velocity components at C.S.5 for the short setback and long setback abutments, where $V_x$, $V_y$ and $V_z$ are the streamwise, transverse and vertical velocity components, respectively. In this study, $u^* = u_{nl}^*$ for consistency in comparison of different embankment lengths for all three flow types. Figures 5.8 and 5.9 show that the bridge deck intensifies the velocity components across the bridge section, but does not change the general distributions of the velocity components. As shown in Figures 5.8a and 5.9a, vertical contraction causes $V_x/u^*$ for the SO and OT flows to be larger than that for the FS flows. In addition, because the overtopped discharge produces some flow relief, $V_x/u^*$ in the SO condition is slightly larger than that in the OT condition. The positive $V_y/u^*$ peaks in Figure 5.8b indicate the existence of clock-wise vortices on the top of the main channel bank, and the negative $V_y/u^*$ values for the rest of the main channel bank show that downslope flow prevails on the slope.
Compared with the short setback abutment, the long setback abutment (see Figure 5.9) has smaller $V_x/u^*$, $V_y/u^*$ and $V_z/u^*$, and smaller magnitude variations across the section. For each flow condition, relatively stable $V_y/u^*$ occur on the floodplain, which weaken to zero at the top of the main channel bank. For the rest of the main channel bank, $V_y/u^*$ values remain small. This suggests that a discernable downslope flow demands a relatively strong transverse flow.

**Figure 5.8** Normalized velocity components at 5-mm above the bed of C.S.5 for short setback abutments; simplified experiment names are used in the legend.
Figure 5.9 Normalized velocity components at 5-mm above the bed of C.S.5 for long setback abutments; simplified experiment names are used in the legend.

Figures 5.10 (a-d) show cross-sectional velocity vectors, which are determined from the time-averaged values of $V_y$ and $V_z$, superimposed on the $V_x$ distribution at C.S.4 for LB_SSA_OT_q, LB_LSA_OT_q, LB_LSA_OT_Q and LB_MSA_OT_Q, respectively. Results show that for each abutment length, $V_x$ increases from zero to very high magnitudes over a similar distance (approximately 120-mm) from the abutment toe, and then decreases in a relatively slow zone. This shows how the compressed streamlines behave close to the abutment toe, and that the flow behaviour follows a regular pattern irrespective of flow rate and geometric contraction ratio. It is suggested that the design of scour countermeasures should take the above flow behaviour into consideration when determining the minimum protection width. For the rest of the floodplain, $V_y$ beneath the bridge deck appears to be uniform over the water depth. In the main channel, $V_y$ weakens significantly, and weak upward flow prevails.
Figure 5.10 Velocity distributions at C.S.4 for a. LB_SSA_OT_q, b. LB_LSA_OT_q, c. LB_LSA_OT_Q and d. LB_MSA_OT_Q; colour scale unit ($V_x$): cm/s
The time-averaged Y-Z plane flow patterns in the main channel of C.S.4 are presented in Figures 5.11 (a-l). Results show that both the abutment length and the flow condition significantly affect the Y-Z plane flow pattern at the bridge section. Under the FS flow, enclosed circular flows in the centre of the main channel can be clearly seen with long and medium setback abutments (Figures 5.11 (b-d)). The circular flow stems from the diverged flow entering into the main channel along the main channel bank, and the loop is completed by weak mass transfer towards the abutment near the surface. This circular flow is induced by a transverse flow shooting into the main channel, but can be overwhelmed by a stronger transverse flow. The example is shown in Figure 5.11a, in which case the short setback abutment induces stronger transverse flow, and the weak circular flow disappears. Under the SO flow, both the $V_y$ and $V_z$ components grow stronger due to the vertical contraction, and the $V_z$ component dominates close to the bridge deck. Open-ended circular flows can be clearly seen with the long and medium setback abutments (Figures 5.11 (f-h)). The circle is discontinuous near the surface, where the $V_y$ component (for SO flow) has a magnitude similar to that of the FS flow, but the $V_z$ component has increased appreciably due to the flow relief. OT flows have weak near-bottom flow in the main channel, but induce the strongest upward flow among the three flow conditions at C.S.4 in the upper flow layer (see Figures 5.11 (i-l)). This is possibly because the OT flow generates a wake vortex region featuring relatively lower pressure, which further draws up the flow beneath the deck.

The Y-Z plane flow pattern could be used to understand the transverse mass transfer process at the bridge section of a compound channel. For all the experiments conducted in this study, the near-bottom flow on the main channel bank area has a downslope component, generating an extra sweeping force on the sediment particles. For all the three flow conditions, the short setback abutment induces the strongest downslope flow, which could exacerbate the erosion
on the main channel bank. In addition, for the same abutment length, the SO flow generates the strongest downslope flow. Therefore, it is suggested that the combination of short setback abutment and SO flow requires extra attention in terms of the stability problem of the main channel bank.
Figure 5.11 Y-Z plane flow patterns in the main channel at C.S.4

The near-bottom turbulence quantities are important variables for understanding the impact of local coherent flow structures on scour development around the abutment (Chrisohoides et al. 2003, Hong et al 2015). Figures 5.12 and 5.13 show the distributions of the three normalized turbulence intensity components at 5-mm above the bed of C.S.5, for the
experiment with a short setback abutment and a long setback abutment, respectively. The general spatial distributions of these turbulence quantities are the same for experiments of the same abutment length, regardless of the flow rates and the flow conditions in this study. Transverse distributions of turbulence intensities for SO and OT flows are nearly superimposed, suggesting that the overtopping discharge relief of OT flows has little effect on the near bottom turbulence.

For the short setback abutment (Figure 5.12), $u'/u^*$, $v'/u^*$ and $w'/u^*$ peak around the same area (110-mm to 160-mm transversally away from the abutment toe), where the streamlines are the most compressed (Figure 5.10a). On the floodplain, higher magnitudes of $u'/u^*$, $v'/u^*$ and $w'/u^*$ were found for pressure flows than for FS flows, indicating that the bridge deck intensifies turbulence in all three directions, and therefore, intensifies the overall turbulence. On the main channel bank and in the main channel, the magnitude differences among the three flow conditions is small for $v'/u^*$ and $w'/u^*$, but remains distinguishable for $u'/u^*$. For the long setback abutment (see Figure 5.13), each normalized turbulence intensity component falls within a narrow band with mild peaks close to the abutment. For the same flow condition, the spatial distributions of $u'/u^*$, $v'/u^*$ and $w'/u^*$ are nearly superimposed, suggesting that the flow rate differences have negligible effect. In contrast, for the short setback abutment, normalized turbulence intensity increases along with the flow rate (see Figure 5.12).

In terms of the cross-sectional distribution of turbulence intensities, results in this study correspond well with those obtained by Hong (2012). The magnitudes differences of the normalized turbulence intensities between this study and those of Hong (2012) are probably because of different ADV signal noise treatments and different flow regimes.
Figure 5.12 Normalized turbulence intensity components at 5-mm above the bed of C.S.5 for short setback abutments; simplified experiment names are used in the legend.
Figure 5.13 Normalized turbulence intensity components at 5-mm above the bed of C.S.5 for long setback abutments; simplified experiment names are used in the legend.

As shown in Figures 5.14 and 5.15, the normalized Reynolds shear stress distributions, 
\[ \tau_{uv} = -\overline{u'v'}/u^* , \tau_{uw} = -\overline{u'w'}/u^* , \tau_{vw} = -\overline{v'w'}/u^* , \]
also have similar spatial trends at C.S.5 for experiments with the same abutment length. For the short setback abutment (Figure 5.14), the general distributions of \( \tau_{uw} \) and \( \tau_{uv} \) for each experiment feature troughs around 160-mm away from the abutment toe, at which position the normalized velocity and the normalized turbulence intensities feature peaks (see Figures 5.8 and 5.12). This correspondence is as expected. \( \tau_{vw} \) is uniformly distributed across the whole section, and maintains a near zero magnitude. For the long setback abutment (Figure 5.15), the variations and the magnitudes of \( \tau_{uw} , \tau_{uv} \) and \( \tau_{vw} \) are small, in accordance with the spatial distributions of turbulence intensities (see Figure 5.13).
Figure 5.14 Normalized Reynolds shear stress components at 5-mm above the bed of C.S.5 for short setback abutments; simplified experiment names are used in the legend.
Figure 5.15 Normalized Reynolds shear stress components at 5-mm above the bed of C.S.5 for long setback abutments; simplified experiment names are used in the legend.

Figures 5.16a and 5.16b show the cross-sectional distributions of $k/u^*$ at C.S.4 for LB_SSA_OT_q and LB_LSA_OT_q, respectively. For LB_LSA_OT_q, the most turbulent zones are centralized beneath the bridge deck. This zone migrates laterally into the main channel for LB_SSA_OT_q, due to a stronger transverse flow from the floodplain. The flow pattern near the abutment toe presents zones of alternate high and low turbulence, corresponding with the behaviour of compressed streamlines (see Figures 5.10a and 5.10b).

Figures 5.16c and 5.16d show the cross-sectional distributions of $u'/u^*$ at C.S.4 for LB_SSA_OT_q and LB_LSA_OT_q, respectively. It is apparent that, for both the experiments, the spatial distribution of $u'/u^*$ is very similar to that of $k/u^*$. On the
floodplain, although the flow is significantly disturbed laterally by the abutment and vertically by the bridge deck, the streamwise turbulence component contributes the most to the overall TKE (on average, $u'^{2}/k^{2} = 0.50$ for LB_SSA_OT_q, and $u'^{2}/k^{2} = 0.53$ for LB_LSA_OT_q). Figures 5.17 and 5.18 show the distributions of subtractions of $k/u^*$ and $\nu_{s}/u^*$ at C.S.4 (LB_SSA_OT_q – LB_LSA_OT_q) for the overlapping area, respectively. Results show that the cross-sectional distributions of $k/u^*$ are highly negatively correlated with those of $\nu_{s}/u^*$: the high $k/u^*$ zone is accompanied by a low $\nu_{s}/u^*$ zone, and vice versa. A comparison of the turbulence distribution with the velocity distribution in Figures 5.10a and 5.10b suggest that, at the bridge section, the main source of the overall TKE ($\frac{1}{2}\rho k^{2}$) comes from the streamwise TKE component ($\frac{1}{2}\rho u'^{2}$).
Figure 5.16 Normalized turbulence intensity distributions at C.S.4 for a. $k/u^*$ for LB_SSA_OT_q, b. $k/u^*$ for LB_LSA_OT_q, c. $u'/u^*$ for LB_SSA_OT_q and d. $u'/u^*$ for LB_LSA_OT_q

Figure 5.17 Subtraction of normalized overall turbulence intensity ($k/u^*$) at C.S.4 for LB_LSA_OT_q and LB_SSA_OT_q, (LB_SSA_OT_q – LB_LSA_OT_q)
Figure 5.18 Subtraction of normalized streamwise velocity ($V_x/u^*$) at C.S.4 for LB_LSA_OT_q and LB_SSA_OT_q. (LB_SSA_OT_q – LB_LSA_OT_q)

5.4 Discussion and analysis

Figures 5.19a and 5.19b compare the estimated bed shear stress and the threshold bed shear stress at C.S.4. Four methods are commonly used to estimate the bed shear stress (see Soulsby 1981; Galperin et al. 1988; Biron et al. 2004; Pope et al. 2006); of these, the TKE method, formulated by Soulsby (1981), is used here:

$$\tau = C_o E = \frac{1}{2} C_o \rho \left( u'^2 + v'^2 + w'^2 \right)$$

Equation 5.1

in which $\tau$ is the time-averaged shear stress, $C_o$ is an empirical coefficient and $E$ is the turbulent kinematic energy. The measurements for the estimated bed shear stress were all conducted at 10-mm above the bed, following Guan et al. (2014). The threshold shear stress on the plane boundary is obtained from the modified Shields formula proposed by Brownlie (1981), and the estimation of shear stress on the main channel bank is based on the Lane (1955) relation.

Figure 5.19a shows that live-bed regime prevails over the whole cross-section of LB_SSA_OT_Q. The shear stress on the floodplain considerably exceeds the threshold values, especially at around $Y =510$-mm, where $\tau/\tau_e$ reaches 17.6. Therefore, it is reasonable to infer that sediment erosion in this region develops most rapidly during the initial stage of
the scour experiment. Based on HEC-23 (Lagasse et al. 2009), the launching apron for LB_SSA_OT_Q extends 170-mm from the abutment toe to Y =505-mm, where the sediment is the most vulnerable to erosion by the flow.

Figure 5.19b shows the results for the long setback abutment. Results show that around the abutment toe, the launching apron designed based on HEC-23 (Lagasse et al. 2009) affords sufficient protection for the FS flows, but insufficient protection for the pressure flows. It can be seen that, for the pressure flows, the most vulnerable region is near the abutment toe, where $1.99 \leq \tau/\tau_c \leq 6.35$. On the main channel bank and in the main channel, $\tau$ exceeds $\tau_c$ for most of the measurements ($0.81 \leq \tau/\tau_c \leq 2.39$). For the free surface flows, in contrast, the most vulnerable region is in the main channel and on the main channel bank, where on average $\tau/\tau_c = 1.20$ for LB_LSA_FS_q and $\tau/\tau_c = 2.25$ for LB_LSA_FS_Q. Near the abutment toe, the maximum value of $\tau/\tau_c$ is 0.94 for LB_LSA_FS_q, and 1.92 for LB_LSA_FS_Q. These results imply that, for the long setback abutment, vertical contraction significantly affects the location and strength of the erosive regions, and thereby the vulnerability of the embankment toe to erosion. The above results highlight the necessity of further investigating the required dimensions of the launching apron for pressure flows, for both long setback and short setback abutments.
Figure 5.19 A comparison of estimated shear stress and threshold shear stress at C.S.4 for a. LB_SSA_OT_Q and b. long setback abutments

Figure 5.20a and 5.20b show the variations of turbulence intensity terms, i.e., $k_{\text{max}}/u^*$ and $\langle k \rangle_f/u^*$ with $L_a/B_f$ for all the experiments. $k_{\text{max}}$ is the maximum turbulence intensity at 5-mm above the bed across C.S.5, $\langle k \rangle_f$ is the width-averaged turbulence intensity across the floodplain section of C.S.5. Results show that $k_{\text{max}}/u^*$ and $\langle k \rangle_f/u^*$ exhibit approximately linear relationships with $L_a/B_f$ for all the three flow conditions. The trends for SO flows and OT flows nearly coincide, which can also be seen in Figures 5.13 and 5.14. The pressure
flows produce higher turbulence intensity than the free surface flow, but both trends have similar gradients. For each flow condition in this study, the formula proposed by Melville (1995) in a compound channel is adapted into:

\[
\frac{d_s}{\sqrt{y_{m2}L_a}} = \alpha_k K_I K_d K_s^* K_p^* K_G
\]

Equation 5.2

in which \(d_s\) is the maximum scour depth; \(K_I\), \(K_d\), \(K_s^*\), \(K_p^*\) are parameters accounting for flow intensity, sediment gradation factor, adjusted abutment shape factor and adjusted bridge orientation factor, respectively (\(K_I\) could be replaced by \(u^*\) when the water depth is kept the same); \(\alpha_k\) is a constant for each flow condition. Because the same channel geometry is used, for the investigated abutment lengths, the channel geometry factor \(K_G\) is assumed to be a constant; the values of \(y_{m2}, K_d, K_s^*, K_p^*\) are constant for each flow condition, with \(L_a\) the only variable.

Equation 5.2 suggests that, for each flow condition, \(d_s\) has an approximate linear relationship with \(\sqrt{L_a u^*}\). Since \(k_{max}/u^*\) and \((k)_f/u^*\) exhibit approximately linear relationship with \(L_a\) (\(B_f\) is constant) in this study (Figure 5.20), results suggest that \(k_{max}/u^*\) and \((k)_f/u^*\) may be closely related to the maximum scour depth around the abutment.
Figure 5.20 a. Variation of the maximum normalized turbulence intensity ($k_{\text{max}}/u^*$) with $L_a/B_f$; b. Variation of the width-averaged normalized turbulence intensity ($\langle k \rangle_f/u^*$) with $L_a/B_f$;

Figures 5.21a and 5.21c show the relationship between $k_{\text{max}}/u^*$ and $q_{f2}/q_{f1}$ at C.S.5, and Figures 5.21b and 5.21d show the relationship between $\langle k \rangle_f/u^*$ and $q_{f2}/q_{f1}$ at C.S.5. Prior to scour, flow parameters on the floodplain are more representative of turbulence conditions than those in the main channel, and hence $q_{f2}/q_{f1}$ is discussed here. The parameter $q_{f2}/q_{f1}$ is a combined expression of channel configuration, lateral contraction and vertical contraction. Parameters $\langle k \rangle_f/u^*$ and $k_{\text{max}}/u^*$ usefully express the magnitude of normalized TKE.

Figure 5.21 shows that the smallest values of $q_{f2}/q_{f1}$ are mainly obtained with long setback abutments, in which situations local turbulence intensities are relatively weak. As $q_{f2}/q_{f1}$ increases, the values of $\langle k \rangle_f/u^*$ and $k_{\text{max}}/u^*$ attain peaks when $q_{f2}/q_{f1} \approx 1.3-1.4$. The peak values occur for a short setback abutment with pressure flows. For the higher values of $q_{f2}/q_{f1}$ (1.5 ~ 2.0), $q_{f2}/q_{f1}$ increases as the abutment intrudes less, and this range of
The values $q_{f2}/q_{f1}$ only occur with FS flow in this study. Eventually, $\langle k \rangle /u^*$ and $k_{\text{max}}/u^*$ reduce asymptotically towards a constant level. It should be noted that trends in Figure 5.21 are very similar to the variation of $y_{\text{max}}/y_c$ with $q_2/q_1$, which was obtained for spill-through abutments by Ettema et al. (2010). $y_{\text{max}}$ is the maximum flow depth at the location of maximum scour, and $y_c$ is the estimated hypothetical long contraction scour depth. $y_{\text{max}}/y_c$ is the depth-amplification factor, which mainly accounts for the additional scour attributable to macro-turbulence generated by flow passing the abutment. This similarity with the results of Ettema et al. (2010) indicates that pressure flows could be treated the same way as FS flow. In addition, this agreement suggests that $\langle k \rangle /u^*$ and $k_{\text{max}}/u^*$ could be alternative depth-amplification factors for evaluation of the contribution of macro-turbulence to final scour, as verified by Hong et al. (2015) for clear-water flow scour.

**Figure 5.21** a. and c. Variation of $k_{\text{max}}/u^*$ with the unit discharge ratio on the floodplain $q_{f2}/q_{f1}$; b. and d. Variation of ($\langle k \rangle /u^*$) with $q_{f2}/q_{f1}$.
5.5 Summary

Flow characteristics around three setback spill-through abutments situated in a compound channel under three flow conditions are presented. Results are presented for three locations along the channel: the approach section, the bridge section and downstream of the bridge.

Moving from the floodplain to the main channel, the distributions of $u'V$, $v'V$ and $w'V$ at the approach section tend to merge, presenting near-isotropic turbulence. Water depth differences correlate with the normalized turbulence distribution differences, mainly because of the compound channel configuration and the effect of modelled vegetation on the floodplain. Flow rate differences do not result in significant changes in the general turbulence pattern, but do result in magnitude differences both in the main channel and on the floodplain, particularly for the near-bottom region.

Downstream of the bridge, the near-bottom normalized turbulence intensities along different longitudinal lines within the compound channel gradually converge to a constant level. This occurs because of an increasing trend along longitudinal lines in the main channel and on the main channel bank, and a decreasing trend along longitudinal lines on the floodplain.

At the bridge section, results from the near-bottom measurements show that the flow condition and the flow rate slightly influence the general pattern of normalized flow velocities, turbulence intensities and Reynolds shear stresses in all three coordinate directions, but cause non-ignorable differences to the magnitudes of the above flow quantities. Close to the abutment, compressed streamlines behave regularly, irrespective of flow rate and geometric contraction ratio. The transverse-vertical flow pattern in the main channel of the
bridge section varies with flow conditions and abutment lengths. Enclosed circular flows in the centre of the main channel are found only with a combination of long or medium setback abutments and a free surface flow condition. The near-bottom flow on the main channel bank has a downslope component for all the flow conditions in this study, and the combination of submerged orifice with a short setback abutment produces the strongest downslope flow. The near bottom turbulence intensity has a linear relationship with the length of abutment. For each abutment length, the near bottom turbulence intensity exhibits alternate high and low zones, answering to the behaviour of compressed streamlines. A strong interdependence is also found between turbulence intensity and convective velocity at the rest of the bridge section. The estimated shear stress results show that around the abutment toe, the launching apron design based on HEC-23 provides adequate protection for free surface flows, but inadequate protection for the pressure flows, especially with a short setback abutment.

The relationship between the normalized turbulence intensity, \( \langle k \rangle / u^* \) and \( k_{\text{max}}/u^* \), and the unit discharge ratio, \( q_{f2}/q_{f1} \), is similar to the relationship between the normalized maximum flow depth \( y_{\text{max}}/y \) and \( q_{f2}/q_{f1} \) revealed by Ettema et al. (2010). This similarity suggests that pressure flows could be treated the same way as free surface flow using the long-contraction theory. In addition, this agreement suggests that the normalized turbulence intensity parameters, \( \langle k \rangle / u^* \) and \( k_{\text{max}}/u^* \), could be used to evaluate the contribution of macro-turbulence to final scour.
Chapter 6.  CLEAR-WATER SCOUR STUDY

6.1 Introduction

The aim of this clear-water study was to investigate scour characteristics of an abutment situated on the floodplain of a compound channel, in which a clear-water flow regime prevailed in both the floodplain and the main channel. This chapter presents the experimental results obtained using the CW 2.4-m wide flume, and a constant channel slope of 0.05%. Results obtained using this flume and the LB 2.4-m wide flume can be used to understand the effect of scour regimes. Experimental setups (Table 6.1) in this study comprise a matrix of flow condition, abutment length, flow intensity and abutment type as below:

- Three flow conditions: FS, SO and OT flow conditions
- Two abutment lengths: short setback abutment ($L_a / B_f = 0.8$) and long setback abutment ($L_a / B_f = 0.5$)
- Flow intensity on the floodplain: $0.45 \leq V_{f1} / V_{fc1} \leq 0.89$; Flow intensity in the main channel: $0.50 \leq V_{m1} / V_{mc1} \leq 1.02$;
- Two abutment shapes: spill-through (H:V = 2:1) and wing-wall (side wing angle 45°) abutments

Presented results consist of final bed bathymetry, temporal development of scour and the maximum scour depth.
Table 6.1 Experimental setups for the clear-water study

<table>
<thead>
<tr>
<th>Experiment</th>
<th>$L_o/B_f$</th>
<th>$Q$ (L/s)</th>
<th>$y_{f1}/y_{m1}$</th>
<th>$y_{f0}/y_{f1}$</th>
<th>$V_{f1}/V_{f1c}$</th>
<th>$V_{m1}/V_{m1c}$</th>
<th>$q_{m2}/q_{m1}$</th>
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<td>0.80</td>
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<th>Experiment</th>
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<th>$y_{f0}/y_{f1}$</th>
<th>$V_{f1}/V_{f1c}$</th>
<th>$V_{m1}/V_{m1c}$</th>
<th>$q_{m2}/q_{m1}$</th>
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<td>0.56</td>
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<td>CW_LSA_OT_WW_3</td>
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<td>0.79</td>
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**Note:** CW denotes clear-water scour; WW denotes wing-wall abutment. The value of each parameter can be found in Tables A3 and A4 in the appendix.
6.2 Experimental observation

6.2.1 Behaviour of the main channel bank

The behaviour of the main channel bank under live-bed condition is described in Section 4.2.1. In this clear-water study, as shown in Figure 6.1, both behaviours illustrated in Figures 4.1a and 4.1b occurred at the bridge section:

- for short setback abutments, the main channel bank “retreated” towards the abutment and retained the initial slope angle ($\beta_1 \approx \beta_2$, spill-through abutments in Figure 6.1a and wing-wall abutments in Figure 6.1b);
- for long setback abutments, the main channel bank was eroded to some extent and the slope decreased ($\beta_1 \geq \beta_2$, spill-through abutments in Figure 6.1c and wing-wall abutments in Figure 6.1d).

For long setback abutments, the behaviour of the main channel bank in this study is different from that reported for the live-bed study, in which the main channel bank “retreated” towards the abutment. The main reason for this difference is probably the different magnitudes of $V_m$. Flow measurements (see Chapter 5) show that, at the bridge section, flow intensity for the upper half of the main channel bank is approximately equal to that for the lower half. Besides, according to the logarithmic velocity distribution, the threshold velocity for the upper half of the main channel bank is slightly smaller than for the lower half ($\frac{\left\langle V_{uv} \right\rangle_{\text{upper-half}}}{\left\langle V_{uv} \right\rangle_{\text{lower-half}}} \approx 0.67$, $\frac{\left\langle V_{uv} \right\rangle_{\text{upper-half}}}{\left\langle V_{uv} \right\rangle_{\text{lower-half}}} \approx 0.93$). Therefore, sand particles on the upper half of the main channel bank are slightly more inclined to become mobile. When $V_{m2}$ is only slightly larger than $V_{m2c}$, e.g., $V_{m2}/V_{m2c} \approx 1.10$, scour occurs mainly on the upper half of the main channel bank, resulting in
\( \beta_1 > \beta_2 \). In contrast, when \( V_{m2} \) increases, e.g., \( V_{m2}/V_{m2c} \geq 1.50 \), intensive erosion occurs on the whole of the main channel bank, resulting in a “retreating” behaviour and \( \beta_1 \approx \beta_2 \). It can be seen in Figures 6.1c and 6.1d and Table 6.1 that, \( \beta_1 > \beta_2 \) occurred for the experiments with relatively small \( V_{m1} \) (hence small \( V_{m2} \)); and \( \beta_1 \approx \beta_2 \) occurred for the experiments with relatively large \( V_{m1} \) (hence large \( V_{m2} \)).

For the long setback abutments in this clear-water study, typically two scour holes appeared: one in the main channel, and one on the floodplain. The geometry of the re-formed main channel bank was jointly shaped by these two scour holes, which are determined by \( q_{m2}/q_{m1} \) and \( q_{f2}/q_{f1} \).
Figure 6.1 Measured bed bathymetry at C.S.4 at the equilibrium state for a. short setback spill-through abutments; b. short setback wing-wall abutments; c. long setback spill-through abutments; d. long setback wing-wall abutments

6.2.2 Scour development around the abutment

For an apron-protected abutment on an erodible channel bed, the development pattern of the scour hole is distinguished by its varying geometry, varying location of the maximum scour depth and increasing scour depth. These scour development features are mainly attributable to the flow around the abutment. In reality, a flood event normally lasts for less than the time to equilibrium scour, and it may take several successive flood events to achieve clear-water scour equilibrium. Understanding the scour development around the abutment is important in practice.
For FS flows in this study, the temporal development of scour was recorded by an overhead camera. For pressure flows, the bed condition could not be clearly recognized on the photo recordings because of the surface waves generated by the bridge deck; hence the photo recordings are not available for pressure flows.

For FS flows, selected photo recordings, overlain with representative pathlines of the water flow around the abutment, are shown in Figure 6.2 and Figure 6.3 for a short setback spill-through abutment (CW_SSA_FS_ST_3) and a long setback spill-through abutment (CW_LSA_FS_ST_2), respectively. Figure 6.2 shows that, for the short setback spill-through abutment, when t = 0.5-hour, the streamlines were deflected laterally towards the main channel, and an elongated scour hole was initiated at the upstream corner of the apron. During the first 0.5-h of scour, strongly-contracted streamlines adjacent to the abutment and the “retreating” behaviour of the main channel bank dominated the scour development. As time passed, riprap slid down on account of scour and the resulting armouring mechanism in turn affected the development of the scour hole. As a result, the partially armoured scour hole gradually deepened and expanded. Figure 6.2 shows that the representative water flow around the abutment had varying pathlines over time: pathlines varied from being deflected towards the main channel in the early stage of scour to being slightly curved back towards the floodplain in the final stage of scour. Accordingly, the thalweg of the scour hole curved slightly towards the floodplain. At the equilibrium state, the scour hole developed into a curved form, with the maximum scour depth on the downstream side of the abutment. Because the abutment was positioned close to the main channel, the development of scour was inevitably influenced by the geometry of the main channel and the flow in it; for example, the majority of the scour hole was on the pre-scour main channel bank, and the scour depth was greatly affected by the flow in the main channel.
Figure 6.2 Temporal development of scour for experiment CW_SSA_FS_ST_3

For the long setback spill-through abutment (Figure 6.3), the scour hole on the floodplain developed in a similar way to that for the short setback spill-through abutment. Because the abutment was set well back from the main channel, flow features on the floodplain dominated the scour process. In the early stage of scour, the scour hole developed laterally towards the main channel; at the equilibrium state, the scour hole developed into the form of a circular arc, extending laterally towards the floodplain. This distinct change of scour development was due to the following:

- Initially, the obstructed discharge was deflected towards the main channel, and this deflected discharge and the associated turbulence structures caused the scour;
As scour developed, conveyance around the abutment increased and the associated velocity and flow depth increased. Therefore, the inertia of the obstructed discharge gradually reduced, and the flow started to return to the floodplain at the downstream side of the abutment. This “returning” flow resulted in the scour hole being curved towards the floodplain.

The “returning” feature for the short setback abutment was less obvious than that for the long setback abutment (see photos at t = 70-hours in Figures 6.2 and 6.3). This is because relatively intensive flow in the main channel affected the scour development for the short setback abutment.

Figure 6.3 Temporal development of scour for experiment CW_LSA_FS_ST_2
6.3 Temporal development of scour

To further investigate the temporal development of scour, particularly for the pressure flows, the bed bathymetry of the whole test section was measured at selected time intervals throughout the experiment using an ultrasonic depth sounder. Results are shown in Table 6.2 and in Figures 6.4-6.20. To highlight the scour hole, for each experiment the final scour bathymetry is re-plotted with a different colour scale. The locations of the maximum scour depth at each measurement time are shown by a white cross and a white star on the floodplain and in the main channel, respectively (Figures 6.4-6.20). For each experiment, at least six intermediate measurements were taken, and one more measurement was taken to ensure the equilibrium state had been achieved. The location of the scour hole, the pattern of scour development and the time to reach equilibrium scour depth were case-specific, mainly depending on the following parameters: $q_z/q_1$, $V_i/V_{ic}$, $y_i$ and $K_y$, where $K_y$ is the abutment shape factor.

Table 6.2 Scour depth (mm) at several intermediate times for the clear-water study

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<th>Experiment NO.</th>
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<th>5</th>
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**Note:** All the scour depths, $d_{s,FP}$, were measured from the pre-scour elevation of the floodplain. N/A: not available. For some experiments, two scour holes developed: in the floodplain for long setback abutments, or in the main channel for short setback abutments. For these cases, both scour holes are presented and are distinguished using superscripts.

Superscript 1: scour depth measured at $t = 160$-hours; 2: scour depth measured at $t = 6$-hours; 3: scour depth measured at $t = 50$-hours; 4: scour depth measured at $t = 94$-hours; 5: scour depth measured at $t = 100$-hours; 6: scour depth measured at $t = 110$ hours; 7: FP values are the maximum scour depths on the floodplain for experiment CW_LSA_OT_WW_1; 8: FP values are the scour depths in the scour hole near the abutment for experiment CW_LSA_OT_WW_1; 9: MC values are the maximum scour depths in the scour hole at the bridge section for experiment CW_SSA_OT_WW_1; 10: MC values are the maximum scour depths in the scour hole downstream of the abutment for experiment CW_SSA_OT_WW_1; 11: MC values are the maximum scour depths in the scour hole at the bridge
6.3.1 Scour development for long setback spill-through abutments

Figures 6.4-6.9 show the scour development over time for six long setback spill-through abutment experiments (one FS, one SO, and four OT flow experiments). Results show the following:

- Scour development interacted with the sliding process of apron rocks. The relocated riprap partially covered the abutment side of the scour hole, causing the scour hole to expand, either transversally or longitudinally. The winnowing process occurred significantly more slowly under the clear-water flow than under the live-bed flow, because the flow rate is less, and also because small apron rocks were used and they effectively prevented the sediment particles from being removed from the voids of the rocks.

- For the long setback spill-through abutment, scour mainly occurred on the floodplain. However, the development of scour on the floodplain was inevitably affected by the scour development in the main channel. For example, Figure 6.5 shows that relatively rapid scour development in the main channel quickly enlarged the flow area at the bridge section, with the result that no further scour development occurred on the floodplain from 1-hour after the commencement of the experiment.

- For each experiment, one scour hole gradually developed around the abutment toe on the floodplain. The locations of these scour holes are found to be mainly dependent on $V_{f1}/V_{c1}$. For experiments with relatively large $V_{f1}/V_{c1}$ (e.g., $V_{f1}/V_{c1} > 0.57$ in this study), the scour hole initially appeared on the extension of the centreline of the abutment, and then expanded downstream over time. The maximum scour depths
were located downstream of the abutment at the equilibrium state (Figures 6.4, 6.8 and 6.9). For experiments with relatively small $V_{f1}/V_{fc1}$ (e.g., $V_{f1}/V_{fc1} < 0.57$ in this study), the scour hole initially appeared on the extension of the centreline of the abutment, and then stayed there or moved slightly downstream of its initial position at the equilibrium state (Figures 6.5, 6.6 and 6.7).

- Scour in the main channel is mainly dependent on $V_{f1}/V_{fc1}$, $y_{f1}$, $V_{m1}/V_{mc1}$ and $y_{m1}$. The scour depth in the main channel was due to the values of $V_{m1}/V_{mc1}$ and $y_{m1}$, while the geometry of the scour hole in the main channel was mainly due to the values of $V_{f1}/V_{fc1}$ and $y_{f1}$. Because a long setback abutment was used, the presence of the abutment had limited influence on the flow in the main channel. Therefore, the contracted flow was relatively uniform across the main channel, and the corresponding scour was approximately proportional to $V_{m1}/V_{mc1}$ (Figure 6.10, except for one FS flow outlier). This FS flow outlier was possibly the result of sand dunes occurring in the main channel ($V_{m1}/V_{mc1} = 1.03$). Obvious main-channel scour holes were observed when large values of $V_{f1}/V_{fc1}$ occurred on the floodplain. Examples can be seen in Figure 6.8 ($V_{f1}/V_{fc1} = 0.81$) and in Figure 6.9 ($V_{f1}/V_{fc1} = 0.79$). This is because, for large values of $V_{f1}/V_{fc1}$, a relatively large portion of the discharge on the floodplain was forced into the main channel at the bridge section, helping the development of the scour hole in the main channel.

- For a narrow range of $V_{f1}/V_{fc1}$ (0.48 $< V_{f1}/V_{fc1} < 0.55$), the development of the non-dimensionalized scour depth ($d_{s,FP}/\sqrt{L_{x}y_{f0}}$) on the floodplain varies significantly with flow conditions, as shown in Figure 6.11. Here, $\sqrt{L_{x}y_{f0}}$ is used because
intermediate length abutments were used (Melville, 1997). The temporal development of scour is described in detail in Section 6.3.4.
Figure 6.4 The temporal development of scour for CW_LSA_FS_ST_2; colour scale unit: mm, the same below.
Figure 6.5 The temporal development of scour for CW_LSA_SO_ST_1
Figure 6.6 The temporal development of scour for CW_LSA_OT_ST_1
Figure 6.7 The temporal development of scour for CW_LSA_OT_ST_2
Figure 6.8 The temporal development of scour for CW_LSA_OT_ST_3
Figure 6.9 The temporal development of scour for CW_LSA_OT_ST_4
Figure 6.10 Variation of $V_m/V_{mc}$ and $y_{max}/y_{m0}$ for the long setback spill-through abutments

Figure 6.11 Scour development for experiments with similar $V_{f1}/V_{fc1}$ values ($0.48 < V_{f1}/V_{fc1} < 0.55$)

6.3.2 Scour development for long setback wing-wall abutments

Experiments CW_LSA_SO_ST_1 (see Figure 6.5), CW_LSA_OT_ST_2 (see Figure 6.7), CW_LSA_OT_ST_3 (see Figure 6.8) and CW_LSA_OT_ST_4 (see Figure 6.9) were repeated with a long setback wing-wall abutment, as experiments CW_LSA_SO_WW_1, CW_LSA_OT_WW_1, CW_LSA_OT_WW_2 and CW_LSA_OT_WW_3, respectively. Plots of the temporal development of scour for these wing-wall experiments can be found in
the appendix. A comparison of results for the long setback spill-through and the wing-wall abutments (Figure 6.12) shows the following:

- The presence of an apron greatly affects the difference between scour development for a wing-wall abutment and a spill-through abutment. Previous research in a rectangular channel showed that, without an apron, a 45° wing-wall abutment causes about 40% deeper scour than a spill-through abutment (H:V = 2:1) for the same flow conditions (e.g., Melville and Coleman 2000). In this compound-channel study, with an apron, compared with the spill-through abutment (H:V = 2:1) with the same flow condition, the 45° wing-wall abutment caused equivalent or slightly smaller scour depth (and a slightly smaller scour hole) in the vicinity of the abutment on the floodplain, and equivalent or slightly larger scour depth (and a slightly larger scour hole) in the main channel (Figures 6.13-6.16). The scour depth difference between the two abutment types in each contrasting group, both in the main channel and on the floodplain, is less than 10%. The difference between the scour observed in this study and that reported by Melville and Coleman (2000) is because the high turbulent areas in the vicinity of the abutment are protected by the erosion-resistant apron rocks, and as a result the difference in turbulence magnitudes caused by the two abutment types is not reflected in differences in erosion on the floodplain. The explanation for the slightly larger main-channel scour for the wing-wall abutment is that, for the same abutment length, a wing-wall abutment has a slightly larger contraction ratio than a spill-through abutment.

- The temporal development of scour for a wing-wall abutment varies significantly with change in the flow features, particularly for change in flow intensity. For example, for overtopping flow with $V/f_1/V_1 = 0.49$, more than 50% of the equilibrium scour depth occurred in the first hour, and the time to equilibrium was less than 20 hours (Figure
6.12b). In contrast, the time to equilibrium increased to 120 hours when \( V_{f1}/V_{fc1} \approx 0.8 \), as shown in Figures 6.12c and 6.12d. In addition, the temporal development of scour on the floodplain may be different from that in the main channel, and an example is shown in Figure 6.12a. The interacting features of scour in different subsections of the channel result in complex scour development.

![Graph a](image1)

![Graph b](image2)
Figure 6.12 Temporal development of scour for spill-through and wing-wall abutments:

**a.** CW_LSA_SO_ST_1 and CW_LSA_SO_WW_1, $V_{f1}/V_{f1} = 0.56$, $V_{m1}/V_{m1} = 0.87$;

**b.** CW_LSA_OT_ST_2 and CW_LSA_OT_WW_1, $V_{f1}/V_{f1} = 0.49$, $V_{m1}/V_{m1} = 0.56$;

**c.** CW_LSA_OT_ST_3 and CW_LSA_OT_WW_2, $V_{f1}/V_{f1} = 0.81$, $V_{m1}/V_{m1} = 0.91$;

**d.** CW_LSA_OT_ST_4 and CW_LSA_OT_WW_3, $V_{f1}/V_{f1} = 0.79$, $V_{m1}/V_{m1} = 0.79$;
Figure 6.13 Bed bathymetry: a. CW_LSA_SO_ST_1; b. CW_LSA_SO_WW_1

Figure 6.14 Bed bathymetry: a. CW_LSA_OT_ST_2; b. CW_LSA_OT_WW_1
Figure 6.15 Bed bathymetry: a. CW_LSA_OT_ST_3; b. CW_LSA_OT_WW_2

Figure 6.16 Bed bathymetry: a. CW_LSA_OT_ST_4; b. CW_LSA_OT_WW_3

6.3.3 Scour development for short setback wing-wall abutments

Figures 6.17-6.20 show the scour development over time for four short setback wing-wall experiments (one FS, and three OT experiments). Results show the following:
During each of the four experiments, two scour holes appeared; for some experiments, they later merged as one. One scour hole was centred on the extension of the centreline of the abutment in the main channel, and the other one was usually located downstream of the abutment on the main channel bank (except for experiment CW_SSA_OT_WW_2). Because of the strong lateral contraction, the scour hole on the abutment centreline developed faster and deeper than the one located downstream of the abutment.

Scour development on the floodplain was dominated by retreat of the main channel bank and the scour development in the main channel. Figures 6.17-6.20 show the following: (a) scour on the floodplain was initiated at the upstream corner of the abutment (except for CW_SSA_FS_WW_1, in which scour was initiated on the extension of the centreline of the abutment); and (b) the deepest scour locations on the floodplain gradually moved downstream during scour development. At the equilibrium state, the maximum scour depth for each of the four experiments was consistently located on the edge of the pre-scour main channel bank, downstream of the abutment.

The locations of the scour holes are greatly dependent on $\frac{V_{f1}}{V_{fc1}}$ and $\frac{V_{m1}}{V_{mc1}}$, and this dependency is similar to that for the long setback abutments, as discussed previously.

Similar to previous discussion, the time needed for equilibrium is greatly affected by the flow intensity. For example, for two overtopping experiments, it took about 20-hours for experiment CW_SSA_OT_WW_2 ($\frac{V_{f1}}{V_{fc1}}=0.48$, $\frac{V_{m1}}{V_{mc1}}=0.50$) to reach equilibrium; whereas it took about 120-hours for experiment CW_SSA_OT_WW_4 ($\frac{V_{f1}}{V_{fc1}}=0.80$, $\frac{V_{m1}}{V_{mc1}}=0.76$) to reach equilibrium.
Figure 6.17 The temporal development of scour for CW_SSA_FS_WW_1
Figure 6.18 The temporal development of scour for CW_SSA_OT_WW_1
Figure 6.19 The temporal development of scour for CW_SSA_OT_WW_2
Figure 6.20 The temporal development of scour for CW_SSA_OT_WW_3
6.3.4 Summary of the temporal development of scour

Previous discussion (Sections 6.3.1 - 6.3.3) shows that the equilibrium time for clear-water scour is strongly related to the flow intensity. This finding is compared with the relationship by Coleman et al. (2003):

\[ t^\ast = 10^6 \left( \frac{V_1}{V_c} \right)^3 \left( \frac{y}{L_c} \right) \left[ 3 - \left( \frac{y}{L_c} \right) \right] \quad \text{for} \quad y/L < 1 \quad \text{and} \quad L/d_{so} > 60 \]  

Equation 6.1

\[ t^\ast = 1.8 \times 10^6 \left( \frac{V_1}{V_c} \right)^3 \quad \text{for} \quad y/L \geq 1 \quad \text{and} \quad L/d_{so} > 60 \]  

Equation 6.2

in which \( t^\ast = t_e V_1/L_a \), \( t_e \) is the time to achieve equilibrium and \( y \) is the flow depth in the rectangular channel. Equations 6.1 and 6.2 were derived for clear-water scour, and for vertical-wall abutments in a rectangular channel.

Equilibrium time data for the long setback abutment (\( L_a / B_f = 0.5 \), scour occurred on the floodplain) and for the short setback abutment (\( L_a / 0.5B_f = 0.53 \), scour occurred in the main channel) are combined and the results are shown in Figure 6.21 and 6.22. Results show that in this study, \( t_e \propto \frac{V_1}{V_c} \), and \( t^\ast \propto \left( \frac{V_1}{V_c} \right)^3 \). The proportional relationship between \( \left( \frac{V_1}{V_c} \right)^3 \) and the non-dimensionalized time \( t^\ast \) is consistent with the finding by Coleman et al. (2003).

Based on the modified-long contraction theory (Equation 4.16), Hong (2012) proposed a third dimensionless parameter, \( (V_1/V_{ic})(q_z/q_i) \), to account for the effect of flow redistribution at the abutment section and the effect of compound channel geometry. Results in Figure 6.23 show that a generally increasing trend exists between \( t^\ast \) and \( (V_1/V_{ic})(q_z/q_i) \), but the relationship is not clear for the available data.
It should be noted that in this study, $t_e$ may be larger than the actual equilibrium time, because of the relatively large time gap between consecutive measurements. Therefore, it may be slightly conservative to use $t_e$ for the estimation of equilibrium time.

**Figure 6.21** Relationship between the equilibrium time $t_e$ and flow intensity, $V_i/V_c$

**Figure 6.22** Relationship between the non-dimensionalized time $t^*$ and $(V_i/V_c)^3$
Figure 6.23 Relationship between the non-dimensionalized time $t^*$ and $(V_f/V_{f_1})(q_2/q_1)$

Figure 6.24 shows the relationship between $d_{v,FP}$ and $t_f$, with both parameters normalized by their equilibrium values. The distribution of the data points indicates that the variation of $d_{v,FP}/d_{w,FP}$ with $t_f/t_v$ is relatively insensitive to $V_f/V_{f_1}$; the upper envelope is defined by both the relatively small flow intensity experiments (e.g., CW_LSA_SO_WW_1) and the relatively large flow intensity experiments (e.g., CW_LSA_OT_WW_2). This insensitivity may be attributable to the interactions of flow and scour between the floodplain and the main channel. Results show that the pressure flow data are located between an upper envelope, $d_{v,FP}/d_{w,FP} = (t_f/t_v)^{0.1}$, and a lower envelope, $d_{v,FP}/d_{w,FP} = (t_f/t_v)^{0.2}$; whereas the free surface flow data are located between an upper envelope, $d_{v,FP}/d_{w,FP} = (t_f/t_v)^{0.2}$, and a lower envelope, $d_{v,FP}/d_{w,FP} = (t_f/t_v)^{0.3}$. 
Note: D/S in the legend denotes the scour hole downstream of the abutment (see Table 6.2)

**Figure 6.24** Relationship between the normalized scour depth \( \frac{d_{v-FP}}{d_{w-FP}} \) and the normalized time \( \frac{t}{t_e} \)

Vertical contraction is the main reason for the temporal development differences between the pressure flows and the FS flows. It is apparent that scour develops faster for the pressure flows. Equation 6.1 indicates that flow depth may affect the temporal development of scour. Figure 6.25 shows the relationship between the maximum water depth \( y_{max}(t) \) and \( t \), with both parameters normalized by their equilibrium values. It can be seen that the pressure flow data are located in a slightly narrower band than the corresponding data in Figure 6.24, with upper and lower envelopes described by \( y_{max}(t)/y_{max}(t_e) = \left( \frac{t}{t_e} \right)^{0.35} \) and \( y_{max}(t)/y_{max}(t_e) = \left( \frac{t}{t_e} \right)^{0.1} \), respectively. Similar to the results shown in Figure 6.24, FS flows generally have smaller values of \( y_{max}(t)/y_{max}(t_e) \), enveloped by \( y_{max}(t)/y_{max}(t_e) = \left( \frac{t}{t_e} \right)^{0.1} \) and \( y_{max}(t)/y_{max}(t_e) = \left( \frac{t}{t_e} \right)^{0.18} \).
Figure 6.25 Relationship between the normalized water depth \( (y_{\text{max}}(t)/y_{\text{max}}(t_c)) \) in the scour hole and the normalized time \( (t/t_c) \).

Results in Figures 6.24 and 6.25 suggest that water depth has an insignificant influence on the temporal development of scour in this study.

Coleman et al. (2003) proposed the following time coefficient \( K_i \) in the scour prediction formula:

\[
K_i = \frac{d_{y_{\text{FP}}}}{d_{w_{\text{FP}}}} = \exp \left[ -0.07 \left( \frac{V_c}{V_{cl}} \right)^{-1} \ln \left( \frac{t}{t_c} \right) \right]
\]

Equation 6.3

Figure 6.26 shows the comparison between the temporal development of scour in this study and \( K_i \) in Equation 6.3. Results show that Equation 6.3 over-estimates the scour development in this study, particularly for the free surface flows. The over-estimation is mainly because
the rock riprap used in this study significantly slowed the scour development. However, Equation 6.3 envelops all the data, suggesting that it can be used as a conservative approach for evaluating the temporal development of scour.

Figure 6.26 Comparison of $K_t$ proposed by Coleman et al. (2003) with the measured temporal development of scour in this study

### 6.4 Scour around the abutment

As discussed in Chapter 4, for an apron protected abutment, the scour hole usually occurs at the downstream side of the abutment. However, results in this clear-water study show that the scour hole tends to occur at the bridge section for relatively small-flow-intensity experiments. In addition, scour at the bridge section directly threatens the safety of the abutment.
In this section, the scour at the bridge section (C.S.4) and the maximum scour depth in the scour hole are discussed.

### 6.4.1 Scour at the bridge section

Figures 6.27(a-d) show the measured bed bathymetries at the equilibrium state, for a short setback spill-through abutment, a short setback wing-wall abutment, a long setback spill-through abutment, and a long setback wing-wall abutment, respectively. For each experiment, the residual apron width, $W_S$, was found to be larger than zero at the equilibrium state. Similar to the equilibrium bathymetry at C.S.4 for live-bed experiments, for all clear-water experiments in this study, the slope ($\beta$) adjacent to the abutment is approximately H:V=2:1 ($\beta \approx 27^\circ$). When the clear water results are combined with the results in Section 4.4.4, it is apparent that none of the following factors by itself determines the value of $\beta$: flow intensity, the size of the riprap, the dimensions of the apron, channel geometry, or flow conditions. For all the reported live-bed and clear-water experiments, the invariants are the properties of the sand and the water. For the sand used in this study the submerged angle of repose, $\phi$, is $36^\circ$ in still water (Guan et al 2013). At the equilibrium state, $\tau = \tau_c$ for the side slope at the bridge section and in the scour hole, and $\phi$ decreases to $27^\circ$ in the moving water. Without the protection of the riprap rocks, the angle of the slope in the scour hole may be smaller than $27^\circ$, and examples can be found in Figures 6.27c and 6.27d. Results show that, when the riprap rocks are dynamically interacting with the erosion process around the abutment, a threshold value of $\beta$ is generally attained around the abutment. Combining all the results for the slope in the scour hole and at the bridge section, it is justifiable to conclude that $27^\circ$ is the threshold value of $\beta$ for the flows in this study. Therefore, according to the well-known Lane (1955) relation, the ratio of the normalised critical Shields stress for
initiation of motion on the side slope of the equilibrium scour hole ($\tau_{c\theta}$, $\theta = \beta = 27^\circ$) and that on a horizontal bed ($\tau_{c0}, \theta = 0^\circ$) is 0.64.

$$\frac{\tau_{c\theta}}{\tau_{c0}} = \cos \theta \sqrt{1 - \left( \frac{\tan \theta}{\tan \phi} \right)^2} = \cos 27^\circ \sqrt{1 - \left( \frac{\tan 27^\circ}{\tan 36^\circ} \right)^2} = 0.64$$

In addition, it is suggested that the side slope of the embankment, the side slope of the spill-through slope, and the slope of the main channel bank are set with $H:V=2:1$ or less steeper.

![Diagram a](image_a)

![Diagram b](image_b)
Figure 6.27 Measured bed bathymetry at C.S.4 at the equilibrium state for a. short setback spill-through abutments, b. short setback wing-wall abutments, c. long setback spill-through abutments, d. long setback wing-wall abutments

Figure 6.28 shows the schematic bottom elevations at C.S.4 for relatively small and large flow intensities. Figures 6.29(a-d) show the conceptual bed bathymetries for the corresponding measured bed bathymetries in Figures 6.27(a-d). Sand ridges between the scour holes in the main channel and on the floodplain (marked in circles, see Figures 6.27c and 6.27d) are removed and the simplified channel geometries are presented. These simplified channel geometries mainly present the common geometry of the scour hole (the H:V=2:1 slope adjoining the abutment), but do not show the actual scour depth. Figure 6.29 suggests a simple geometric relationship between the maximum scour depth and the location of the maximum scour at the bridge section:
\[ \tan \beta = \frac{d_{s-FP}}{L_s} \]  

Equation 6.4

in which \( L_s \) is the distance between the edge of the residual apron (\( W_S \geq 0 \)) and the nearest maximum scour depth.

Because \( \beta \) depends mainly on the sand property, this geometric relationship may only apply for soil conditions similar to those in this study. Though \( W_S > 0 \) was found in this clear-water study, \( n_{apron} \geq \frac{d_{s-FP}}{W_{apron} \sin \beta} \) (as previously suggested in Chapter 4) is still suggested as a riprap apron design relationship.

Figure 6.28 Schematic bottom elevation at C.S.4 at the equilibrium state for long setback abutments, illustrating the effect of flow intensity on the residual apron (\( W_S \)) and the final geometry
Figure 6.29 Schematic bed bathymetry at C.S.4 at the equilibrium state for a. short setback spill-through abutments, b. short setback wing-wall abutments, c. long setback spill-through abutments, d. long setback wing-wall abutments
6.4.2 Scour at the downstream side of the bridge

As discussed in Chapter 2 and Chapter 4, the following expression is widely accepted as a useful scour-estimation method, and this expression is the analysis basis for this study:

Abutment scour depth = (coefficient) x (long-contraction scour depth)

The “long-contraction scour depth” term can be estimated based on the clear-water contraction scour relationship proposed by Laursen (1963):

\[ y_c = y_1 \left( \frac{V_1}{V_{lc}} \right) \left( \frac{q_2}{q_1} \right)^{6/7} \]  \hspace{1cm} \text{Equation 6.5}

which was derived with an unprotected abutment model extending across the floodplain of a compound channel. For experimental conditions similar to those in this study, Hong et al. (2015) confirmed the validity and applicability of Equation 6.5. Similarly to the “apron-protected long-contraction” analysis in Section 4.4.5.2,

\[ y_{\text{max}} = r_T y_1 \left( \frac{V_1}{V_{lc}} \right) \left( \frac{q_2}{q_1} \right)^{6/7} = r_T y_c \]  \hspace{1cm} \text{Equation 6.6}

in which \( r_T \) is the depth-amplification factor, accounting for the influences of channel geometry, countermeasures status, abutment shape, and the contribution of macro turbulences. Results in Section 6.3.2 show that, because of the apron, the scour differences between the wing-wall abutment and the spill-through abutment are minimal. In the following analysis, the effect of the abutment type is therefore not discussed. The complicacy of \( r_T \) is discussed later.

For the short setback abutment experiments, Equation 6.6 can be written as the following:

\[ y_{\text{max}} = r_T y_{m1} \left[ \frac{(V_{m1}/V_{m1c})(q_{m2}/q_{m1})}{(V_{m1}/V_{m1c})(q_{m2}/q_{m1})} \right]^{6/7} = r_T y_{mC} \]  \hspace{1cm} \text{Equation 6.7}

in which \( r_T = \kappa_G C_T \), \( \kappa_G \) accounts for the geometric nonuniformity in the main channel, and \( C_T \) accounts for the influences of macro turbulence. Since the same main channel geometry was used for all experiments in this study, \( \kappa_G \) is assumed to be a constant.

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For the long setback abutment experiments, Equation 6.6 can be written as the following:

\[
y_{\text{max}} = r_T y_{fC} \left[ \left( \frac{V_{f1}}{V_{f1e}} \right) \left( \frac{q_{f2}}{q_{f1}} \right) \right]^{\frac{6}{7}} = r_T y_{fC}
\]

Equation 6.8

in which \( r_T = \kappa_G C_T \). For the long setback abutment, because the scour hole of interest is located on the floodplain, and this scour hole generally did not expand laterally into the main channel, \( \kappa_G = 1 \), and \( r_T = C_T \).

For the short setback abutments, the following normalized scour and flow parameters in the main channel are discussed: the normalized maximum flow depth, \( \frac{y_{\text{max}}}{\bar{y}_{m1}} \) and \( \frac{y_{\text{max}}}{\bar{y}_{mc}} \); the normalized long-contraction scour depth, \( \frac{\bar{y}_{mc}}{\bar{y}_{m1}} \); the unit discharge ratio, \( \frac{q_{m2}}{q_{m1}} \), and the normalized flow intensity, \( \frac{V_{m2}}{V_{m1}} \). The values of these dimensionless flow parameters are shown in Table 6.3.

Figure 6.30a shows the relationships between \( \frac{y_{\text{max}}}{\bar{y}_{m1}} \) and \( \frac{q_{m2}}{q_{m1}} \), and between \( \frac{\bar{y}_{mc}}{\bar{y}_{m1}} \) and \( \frac{q_{m2}}{q_{m1}} \). Because different flow intensities and flow conditions were used, it is expected that the values of \( \frac{y_{\text{max}}}{\bar{y}_{m1}} \) and \( \frac{\bar{y}_{mc}}{\bar{y}_{m1}} \) vary for the same value of \( \frac{q_{m2}}{q_{m1}} \), as shown in Figure 6.30a. Figure 6.30b shows the relationships between \( \frac{y_{\text{max}}}{\bar{y}_{m1}} \) and \( \left( \frac{q_{m2}}{q_{m1}} \right)^{\frac{6}{7}} \left( \frac{V_{m2}}{V_{m1}} \right)^{\frac{6}{7}} \), and between \( \frac{\bar{y}_{mc}}{\bar{y}_{m1}} \) and \( \left( \frac{q_{m2}}{q_{m1}} \right)^{\frac{6}{7}} \left( \frac{V_{m2}}{V_{m1}} \right)^{\frac{6}{7}} \), illustrating Equation 6.7 graphically. For the flow condition \( 0 < \left( \frac{q_{m2}}{q_{m1}} \right)^{\frac{6}{7}} \left( \frac{V_{m2}}{V_{m1}} \right)^{\frac{6}{7}} < 1 \), Equation 6.5 indicates that contraction scour does not occur and the measured scour depth is attributable to macro turbulence generated by the abutment (given that the long contraction theory applies to the flow conditions in this study). Figure 6.30b shows that for the OT flow, \( \frac{y_{\text{max}}}{\bar{y}_{m1}} \) has nearly a linear relationship with \( \left( \frac{q_{m2}}{q_{m1}} \right)^{\frac{6}{7}} \left( \frac{V_{m2}}{V_{m1}} \right)^{\frac{6}{7}} \); for the FS and SO
flows, the limited data present scattered points, enveloped by two dashed lines in Figure 6.30b.

For the short setback experiments, the maximum value of \( \frac{y_{\text{max}}}{y_{m1}} \) is 2.25.

The data are re-arranged and plotted in Figure 6.31a, Figure 6.31b and Figure 6.31c, to show the relationships between the amplification factor \( \frac{y_{\text{max}}}{y_{mC}} (= r_y) \) and \( \frac{q_{m2}}{q_{m1}} \), \( \left( \frac{V_{m2}}{V_{m1}} \right) \), and \( \left( \frac{q_{m2}}{q_{m1}} \right) \left( \frac{V_{m2}}{V_{m1}} \right) \), respectively. Results, as plotted in Figures 6.31(a-c), show that, for each flow condition, results are scattered, suggesting weak or no relationships between each pair of plotted normalized parameters. This lack of clarity in the relationships are somewhat expected, because \( r_y \) mainly varies with macro turbulence. Hong (2012) and Hong et al. (2015) proposed that the macro turbulence causes an increase in bed-level TKE, which causes scour; they proposed using the maximum width-averaged TKE near the bed at C.S.5 to evaluate \( r_y \). Also, Hong (2012) hypothesised that the width-averaged TKE varies with \( q_2/q_1 \), and that therefore \( r_y \) varies with \( q_2/q_1 \); through regression analysis, Hong found that

\[
r_y \propto \left( \frac{q_{m2}}{q_{m1}} \right)^{-0.12}
\]

for the short setback abutment. In this clear-water study, \( \left( \frac{q_{m2}}{q_{m1}} \right)^{-0.12} \) has a very narrow range of 0.94 to 0.96, but \( r_y \) ranges from 1.50 to 2.20 (0.96/0.94 \neq 2.20/1.50).

Therefore, the relationship between \( r_y \) and \( \left( \frac{q_{m2}}{q_{m1}} \right)^{-0.12} \) hypothesised by Hong (2012) does not apply to the results in this study. The TKE data are not available for this clear-water study. However, the positive relationship between the near bottom TKE and \( r_y \) for the live-bed flows in Chapter 4 does support the TKE concept of Hong (2012) and Hong et al. (2015).
Table 6.3 Key dimensionless flow parameters for the clear-water study

<table>
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<tr>
<th>Experiment NO.</th>
<th>$y_{m1}$ (mm)</th>
<th>$y_{m0}$ (mm)</th>
<th>$y_{m1}/y_{m0}$</th>
<th>$V_{m1}/V_{m1c}$</th>
<th>$q_{m2}/q_{m1}$</th>
<th>$y_{mc}/y_{m1}$</th>
<th>$y_{max}/y_{m1}$</th>
<th>$y_{max}/y_{C}$</th>
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<td>1.04</td>
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<td>1.75</td>
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<thead>
<tr>
<th>Experiment NO.</th>
<th>$y_{f1}$ (mm)</th>
<th>$y_{f0}$ (mm)</th>
<th>$y_{f1}/y_{f0}$</th>
<th>$V_{f1}/V_{f1c}$</th>
<th>$q_{f2}/q_{f1}$</th>
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Note: the over-bar on the main channel flow parameter denotes the mean value of the whole main channel.
Figure 6.30 Variations of $\frac{y_{\text{max}}}{y_{m1}}$ (the first dependent variable) or $\frac{y_{mC}}{y_{m1}}$ (the second dependent variable) for the short setback abutment experiments, with a. $q_{m2}/q_{m1}$; b. $(q_{m2}/q_{m1})^{6/7}(V_{m2}/V_{m1})^{6/7}$
Figure 6.31 Variations of $\frac{y_{\text{max}}}{y_{mc}}$ for the short setback abutment experiments, with a. $q_{m2}/q_{m1}$, b. $V_{m1}/V_{m1c}$, c. $(q_{m2}/q_{m1})(V_{m1}/V_{m1c})$
The results for the long setback abutment are plotted in Figures 6.32 and 6.33. Figure 6.32 shows an approximate linear relationship between \( \frac{y_{\text{max}}}{y_{f_1}} \) and \( \left( \frac{q_{f_2}}{q_{f_1}} \right)^{6/7} \left( \frac{V_{f_1}}{V_{f_{1c}}} \right)^{6/7} \), and a linear relationship between \( \frac{y_{fC}}{y_{f_1}} \) and \( \left( \frac{q_{f_2}}{q_{f_1}} \right)^{6/7} \left( \frac{V_{f_1}}{V_{f_{1c}}} \right)^{6/7} \). For all these long setback experiments, \( \frac{y_{fC}}{y_{f_1}} \leq 1.0 \) and \( \frac{y_{\text{max}}}{y_{f_1}} > 1.0 \), suggesting that scour is all attributable to macro turbulence (see Equation 6.8). However, flow measurements at the bridge section suggest that contraction scour occurred; for example, below the bridge deck of experiment CW_LSA_OT_ST_3, flow measurements show that width-averaged flow intensity \( \overline{V_{f_2}}/V_{f_{2c}} = 1.1 \), suggesting the existence of contraction scour on the floodplain.

Therefore, for the pressure flows, particularly for the overtopping flows, using Equation 6.5 as the analysis basis requires \( r_t \) to account for the effect of vertical contraction for the pressure flows, and to account for the effect of discharge relief for the overtopping flow.

As expected, the relationship between \( \frac{y_{\text{max}}}{y_{fC}} \) and \( \left( \frac{q_{f_2}}{q_{f_1}} \right) \left( \frac{V_{f_1}}{V_{f_{1c}}} \right) \) is unclear, as shown in Figure 6.33. For FS flows, \( \frac{y_{\text{max}}}{y_{fC}} \approx 2.8 \) and the values of \( \frac{y_{\text{max}}}{y_{fC}} \) are slightly larger than those reported by Ettema et al. (2010). For pressure flows, \( \frac{y_{\text{max}}}{y_{fC}} \geq 3.0 \) and \( \frac{y_{\text{max}}}{y_{fC}} \) reaches a maximum of 5.1. The significant increases of the values of \( \frac{y_{\text{max}}}{y_{fC}} \) for pressure flows are mainly because of the vertical contraction and the associated enhanced macro turbulences. Also, for the short setback abutment in either clear-water or live-bed flows (reported in Chapter 4), \( \frac{y_{\text{max}}}{y_{fC}} \) is larger for pressure flows than for free surface flows.

To unify the three flow conditions in the form of \( y_{\text{max}} = r_t y_C \), flow features under the bridge deck (submerged flows), including the near bottom TKE distributions, the width-averaged values of \( \frac{V_{2-\text{submerged}}}{V_{2c}} \), and \( \frac{q_{2-\text{submerged}}}{q_1} \), need to be extensively studied in the future.
Figure 6.32 Variations of $\frac{y_{\text{max}}}{y_{f1}}$ (the first dependent variable) or $\frac{y_{fC}}{y_{f1}}$ (the second dependent variable) with $(\frac{q_{f2}}{q_{f1}})^{6/7}(\frac{V_{f1}}{V_{f1c}})^{6/7}$ for the long setback abutment experiments.

Figure 6.33 Variations of $\frac{y_{\text{max}}}{y_{fC}}$ with $(\frac{q_{f2}}{q_{f1}})(\frac{V_{f1}}{V_{f1c}})$ for the long setback abutment experiments.
The above results suggest that \( r_t \) is a very complex parameter. It is suggested different ranges of \( r_t \) values for different flow conditions should be used for predicating abutment scour depth.

### 6.5 Comparison with other research results

The maximum scour depths from this clear-water study are compared with results from six previous studies, by Melville (1995, 1997), Melville et al. (2006), Sturm (2006), Ettema et al. (2010) and Hong et al. (2015).


The National Cooperative Highway Research Program 24-27 (NCHRP 24-27) recommends the scour estimation method proposed by Melville (1997) as the most applicable method for short, solid wall abutments. For the long setback abutment (short abutment), at the equilibrium state, the local scour depth at a bridge abutment is closely related to the abutment length:

\[
d_s = K_s K_I K_d K_o K_c K_y K_{yl}
\]

Equation 6.9

The \( K \) values can be determined from their definitions by Melville (1997) and Coleman et al. (2003). In this study, spill-through and wing-wall abutments are found to cause similar scour depths, therefore \( K_s = 0.5 \), as proposed by Melville (1997) for all the experiments. Abutments were aligned perpendicular to the approach flow (\( K_o = 1.0 \)), uniform sediment was used (\( K_d = 1.0 \)), and equilibrium was achieved (\( K_t = 1.0 \)). Because the abutment was positioned well back from the main channel, an imaginary boundary between the floodplain and the main channel is justifiable (\( K_{yl} = 1.0 \)). The model abutments had “intermediate” lengths (\( K_{yl} = 2 \sqrt{L_y y'/f'} \)). Therefore,
\[ d_s = \sqrt{L_a y_f} \]  

Equation 6.10

Figure 6.34 compares the predicted scour depths according to Equation 6.10 and the measured scour depths. Results show that, Equation 6.9 can predict the scour depths in this study. However, this agreement in Figure 6.34 may be achieved because two scour components offset each other: the “reductive” component caused by the apron (not considered in Equation 6.10) and the “incremental” component caused by vertical contraction (not considered in Equation 6.10).

For the short setback abutment, the parameter \( K_G \) in Equation 6.9 (Melville, 1995) accounts for the influence of the channel geometry:

\[ K_G = \sqrt{1 - \frac{L_a}{L}} \left[ 1 - \left( \frac{y_f}{y_m} \right)^{5/3} \right] \quad \text{for } L_a > L \]  

Equation 6.11

Here, because short setback abutments were used, an approximation, \( L_a/L = 1 \), is introduced to obtain \( K_G \) values. Figure 6.35 compares the measured scour depths with the scour depths predicted by Equation 6.9 with \( K_G \) obtained from Equation 6.11. There is a large amount of scatter, but Equation 6.9 conservatively envelopes the measured data. The over-predication of scour depth by Equation 6.9 with \( K_G \) from Equation 6.11 is possibly because the apron effectively mitigates the scour development.

Melville et al. (2006) proposed another scour-prediction method for a compound channel:

\[ d_{s-pp} = d_s + F_s (y_m - y_f) \]  

Equation 6.12

\[ F_s = 1 - \left( 1 - \frac{L_a}{B_f} \right)^{2(\alpha/B_f-1)} , \quad \frac{\alpha}{B_f} > 1 \]  

Equation 6.13

This method was derived for protection by an apron, and it considered the elevation difference between the main channel and the floodplain. The method is described in detail in Section 4.5. Figure 6.36 compares the measured scour depths with the scour depths predicted
by Equations 6.12 and 6.13. Results show that Equations 6.12 and 6.13 well predict the scour depths in this clear-water study. Comparison of these clear-water results with the live bed scour results in Chapter 4 shows that the method proposed by Melville et al. (2006) is suitable for scour depth estimation for short setback abutments in both clear-water and live-bed conditions.

**Figure 6.34** Comparison with the equation developed by Melville (1997), for a long setback abutment

**Figure 6.35** Comparison with the equation developed by Melville (1995) for a short setback abutments in a compound channel
Figure 6.36 Comparison with the equation developed by Melville et al. (2006) for short setback abutments in a compound channel

Ettema et al. (2010)

NCHRP 24-27 also recommends the scour estimation method proposed by Ettema et al. (2010) because it has the desirable attributes of reflecting the physics of the abutment scour process both in terms of flow constriction and turbulence structures. The clear-water scour results from this study are compared with the results from the study by Ettema et al. (2010), in terms of the relationship between the amplification factor \( r_f \) and the discharge contraction ratio. The results for the short setback abutments are not discussed here, because in the study by Ettema et al. (2010) a live-bed regime existed in the main channel, but a clear-water regime existed in this study.

For the long setback abutments, the comparative results are shown in Figure 6.37. It is obvious that \( r_f \) from this study is larger than that obtained by Ettema et al. (2010),
particularly for the OT flows. The $r_T$ values from this study are at least twice those obtained by Ettema et al., probably resulting from the following:

1. Ettema et al. (2010) had erodible embankments and an apron was not used; in contrast, abutments in this study remained intact because of protection by the apron;
2. Ettema et al. (2010) only considered FS flows. For the pressure flows in this study, the final scour depth is greatly affected by the vertical contraction.

Regarding the magnitude of $r_T$, the effect of vertical contraction on scour is found to be more significant for the floodplain than for the main channel. It should be noted that Ettema et al. (2010) also found that the scour results for spill-through and wing-wall abutments were similar.

![Figure 6.37](image)

**Figure 6.37** Comparison with the results obtained by Ettema et al. (2010) for long setback abutments. *Note:* WW = wing-wall abutment

**Sturm (2006) & Hong et al. (2015)**

Sturm (2006) reported a series of free-surface flow experiments conducted in a compound channel, with un-protected solid abutments of different lengths. Hong et al. (2015) reported a
series of experiments conducted in a compound channel, with two abutments at the same time, one on each side of the channel; the spill-through abutments were erodible and were protected by a rock riprap apron; free surface and pressure flows were investigated.

The results from long setback abutment experiments in this clear-water study are compared with results from Sturm (2006) and Hong et al. (2015), in terms of the non-dimensional variables: \( \left( y_{f1}/y_{f0} \right) \left[ \left( V_{f1}/V_{f1c} \right) \left( q_{f2}/q_{f1} \right) \right]^{\frac{6}{7}} \) and \( y_{\text{max}}/y_{f0} \), as shown in Figure 6.38. It is noted that, \( y_{f1}/y_{f0} \) accounts for the backwater effect, and \( y_{f1} \left[ \left( V_{f1}/V_{f1c} \right) \left( q_{f2}/q_{f1} \right) \right]^{\frac{6}{7}} \) is the idealized long contraction scour depth. Results show that, except for three OT flows (arrowed in Figure 6.38), results from this study agree well with those reported by Hong et al. (2015). This agreement is achieved with concurrent differences in two variables: different model scale (1:45 for Hong et al. (2015) and 1:30 for this study), and different erodibility of the abutment (erodible abutments for Hong et al. (2015) and non-erodible abutments for this study). Because the abutment toes were only mildly eroded in Hong et al. (2015), the effect of the erodibility is negligible. Therefore, the data agreement suggests that the effect of model scale (between 1:30 and 1:45) is insignificant. The three outlying points (arrowed in Figure 6.38) of OT flows are possibly because of differences in the magnitude of \( Q_{\text{ot}}/Q \): for the three outlying points of OT flows, the magnitude of the overtopping discharge \( Q_{\text{ot}}/Q \) ranges from 6.8% to 22.8%; whereas for the rest of OT flows in Figure 6.38, \( Q_{\text{ot}}/Q \) ranges from 31.1% to 46.7%. For the smaller values of \( Q_{\text{ot}}/Q \) there would be a smaller magnitude of flow relief, and possibly a resulting stronger magnitude of vertical contraction. Further research, regarding the effect of different magnitudes of flow relief on abutment scour, is needed. Because Sturm (2006) used an unprotected, solid abutment, the maximum scour occurred around the upstream toe of the abutment, where the horse-shoe vortex dominates the
scour. In contrast, for this clear-water study, the scour process is greatly affected by the mobile riprap, and the dominant scouring power is the macro turbulence shedding from the abutment toe and flow contraction. Previous studies (e.g., van Ballegooy, 2005) show that the riprap apron significantly mitigates the scour, and, therefore, it is expected that the values of normalized scour depth for the unprotected abutment would be larger than those for the protected abutment, as shown in Figure 6.38.

Figure 6.38: Comparison with results of Sturm (2006) and Hong et al. (2015) for LSA experiments

The results from short setback abutment experiments in this clear-water study are compared with those from Sturm (2006) and Hong et al. (2015) in terms of the non-dimensional variables: \( \left( \frac{y_{m1}}{y_{m0}} \right) \left( \frac{V_{m1}}{V_{m0}} \right) \left( \frac{q_{m2}}{q_{m1}} \right)^{6/7} \) and \( y_{max}^{\prime} / y_{m0} \), as shown in Figure 6.39. Hong et al. (2015) found that the bankline and the short setback abutments usually have similar scour patterns; therefore results of bankline abutments are included. Results show that, though the abutment erodibility and the status of countermeasures vary greatly among the three comparative studies, the normalized scour depth values lie in a narrow band. By
comparing with the long setback abutment results in Figure 6.38, the narrow band in Figure 6.39 suggests that the normalized scour depths in the main channel are less sensitive to changes in abutment erodibility and the status of countermeasures. This insensitivity is because flow contraction plays a greater role in the resulting total scour depth than the macro turbulences around the abutment; as a result, the erosional strength of the abutment and the contribution of the riprap become less important. Results in Figures 6.38 and 6.39 suggest the applicability of Equation 6.6.

Figure 6.39 Comparison with results of Sturm (2006) and Hong et al. (2015) for the SSA experiments
Chapter 7. LONG CONTRACTION SCOUR STUDY

7.1 Introduction

The aim of this long contraction scour study was to investigate scour characteristics of setback abutments with varying contraction lengths. The well-known long contraction theory (Laursen, 1963) used simplified abutment models, as described in Chapter 2. However, in reality, bridge abutments have the following features:

- Bridge abutments impose short and abrupt contractions, rather than smooth and long contractions. Bridge abutments are commonly set back from the main channel, rather than extended to the main channel;
- Different contraction lengths (short and long) can be found in the form of abutments for narrow (one-lane) and wide (multiple-lane) bridges.
- Scour countermeasures are commonly employed around abutments, and this factor is worth considering when investigating the long contraction scour depth.

Experimental setups in this study comprise a matrix of channel configuration, abutment length, contraction length, flow intensity and the status of scour countermeasures:

- Two channel configurations were used: a rectangular channel \((B = 1.54\text{-m})\) was used for a condition with the abutment set well back from the main channel, and abutment-caused scour occurred on the floodplain only (abutment scour Case A in Chapter 2); a compound channel \((B_f =1.08\text{-m}, \ B_m = 0.46\text{-m})\) was used for a condition with the abutment set back from the main channel, but the abutment-caused scour occurred both on the floodplain and in the main channel.
• For the rectangular channel study, two abutment lengths and two contraction lengths were used. Flow intensities \( V_i/V_i \)c varied from 0.50 to 0.88. Four groups of experiments were carried out (Groups 1 - 4, Table 7.1).

• For the compound channel study, one abutment length and five contraction lengths were used. One flow intensity was used: \( V_f/V_f \)c \( \approx 0.78 \) and \( V_m/V_m \)c \( \approx 0.99 \). Five groups of experiments were carried out (Groups 5 - 9, Table 7.1): six experiments with rock riprap aprons protecting the abutments, and four experiments without any form of scour countermeasures.

In this study, only FS flows were investigated, and only vertical wall abutments were used.

The presented results consist of final bed bathymetry, temporal development of scour, two-dimensional surface flow fields and three-dimensional flow fields around the abutment.

Table 7.1 Experimental parameters for the long contraction study

<table>
<thead>
<tr>
<th>Exp.</th>
<th>( y_f ) (mm)</th>
<th>( L_a ) (mm)</th>
<th>( L_C ) (mm)</th>
<th>( V_i/V_i )c</th>
<th>( L_d/B )</th>
<th>( L_c/B )</th>
<th>( L_c/L_a )</th>
<th>( t ) (hours)</th>
<th>Countermeasure</th>
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<tbody>
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<td>Group 1</td>
<td>RC1</td>
<td>100</td>
<td>400</td>
<td>1100</td>
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<td>0.03</td>
<td>0.13</td>
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<td>400</td>
<td>1100</td>
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<td>0.71</td>
<td>2.75</td>
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<td>0.71</td>
<td>1.38</td>
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<td>0.03</td>
<td>0.06</td>
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<td>0.71</td>
<td>1.38</td>
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<tr>
<td>Group 4</td>
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<td>0.52</td>
<td>0.03</td>
<td>0.06</td>
<td>&gt;240</td>
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<table>
<thead>
<tr>
<th>Exp.</th>
<th>( y_f ) (mm)</th>
<th>( L_a ) (mm)</th>
<th>( L_C ) (mm)</th>
<th>( V_i/V_i )f</th>
<th>( L_d/B_f )</th>
<th>( L_c/B_f )</th>
<th>( L_c/L_a )</th>
<th>( t ) (hours)</th>
<th>Countermeasure</th>
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<td>0.25</td>
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<tr>
<td>Group 7</td>
<td>CC6</td>
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<td>540</td>
<td>810</td>
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<td>0.75</td>
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<tr>
<td>Group 8</td>
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<td>540</td>
<td>2445</td>
<td>0.78</td>
<td>0.50</td>
<td>2.26</td>
<td>4.53</td>
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</tr>
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<td>45</td>
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<td>0.50</td>
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<td>2.96</td>
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Note: For the name of the experiment, RC indicates rectangular channel, and CC indicates compound channel. \( L_c \) is the contraction length, and also the abutment width.
7.2 Long contraction scour in a rectangular channel

This part of the study investigated the effect of contraction length on local scour around solid vertical abutments in a rectangular sand-bed channel. The effects of contraction length on two-dimensional surface velocity fields for both the plane-bed and scoured bed condition, and on equilibrium scour bathymetries are presented and discussed. Four groups of experiments were performed (two experiments per group, Table 7.1), covering eight possible combinations of the abutment length (400 or 800-mm), contraction length (i.e., abutment width, 50 or 1100-mm) and flow intensity (relatively small or large). Within each group, the velocities in the two experiments were approximately equal (flow intensity difference occurs in Group 3).

7.2.1 Introduction

After decades of bridge scour research, estimates of scour depth still vary substantially. This variability is due to the innate complexity of the flow, combined with site-specific approach geometries and boundary materials. Sturm et al. (2011) presented a clear delineation of the dominant variables governing abutment scour. However, the full influences of some variables are still unclear and the innate complexity could result in some secondary variables being poorly defined.

As river flow passes around a bridge abutment, the streamlines converge due to channel contraction and then diverge after passing through it. Over more than half a century, the contracting process has been theoretically explained and physically modelled, based on the idealized long contraction theory of Straub (1934) and Laursen (1960, 1963). Komura (1966) and Webby (1984) defined a long contraction as having at least the same length as the width of the un-contracted upstream channel. In reality, however, abutments are often only as wide
as bridge decks. Bridge decks can range from single lane to multi-lane; nevertheless, for many bridge designs, abutment widths are narrow compared with upstream channel width.

Contraction scour and local scour have been widely accepted as interdependent processes, and usually occur concurrently, although displaying different magnitudes (Chang and Davis 1998, Chang and Davis 1999). Schreider et al. (2001) investigated the interaction between abutment scour and contraction scour at relief bridges in a flood plain. Their results revealed that contraction scour depth could be drastically reduced by the co-existence of abutment scour, and that the abutment scour develops faster than the contraction scour. However, the opposite effect, that of contraction scour on abutment scour, has rarely been studied and the effect is still not clear. Briaud et al. (2005) extended the cohesive soils-erosion function apparatus method to the case of contraction scour in fine-grained soils. Their results from flume experiments showed that the maximum scour depth was not influenced by the contraction length $L_c$, when $L_c$ was between 0.5 and 6.8 times the contracted channel width $B_c$. The results of numerical simulations by Briaud et al. (2005) showed that the maximum initial shear stress along the centreline of the channel is insensitive to the contraction length, except for very small contraction lengths, in which case the maximum initial shear stress decreases. Their results, from experiments that all had the same hydrodynamic and geometric conditions, also showed that when $L_c/B_c$ is smaller than 0.25, the maximum scour depth is much larger than when $L_c/B_c$ has larger values. Hager and Dupraz (1985) and Wu and Molinas (2001) investigated the effects of the relative contraction length both theoretically and experimentally. The relative contraction length, $L^* = L_c/B$, where $B$ is the approach channel width. The analysis of Wu and Molinas showed that the discharge coefficient is affected by up to 6% by $L^*$, because contraction length causes an increase in energy losses.
within the contracted reach. Kohli and Hager (2001) demonstrated that for contraction lengths $L_c$ between 0.02 and 0.30-m (channel width 1-m), the effect of contraction length is insignificant for the maximum scour depth.

This part of the long contraction study presents the results of a comparison of the effects of long and short contraction on local scour around solid vertical abutments in a rectangular sand-bed channel. The objective of this part of the study is to determine the effect of contraction length on flow features around the abutment and on the associated abutment scour depth. Two-dimensional velocity fields, for both the plane-bed and scoured bed condition, and equilibrium scour contours are presented and discussed.

### 7.2.2 Bed morphologies at the equilibrium state

Figures 7.1(a-f) show the bed morphologies at the equilibrium state for experiments RC1 - RC8, respectively. All the scour holes resemble frustums of inverted circular or elliptical cones, centred on the upstream corner of the abutments, with the shapes partially interrupted by the abutments themselves. Eroded bed material was transported mainly to the downstream side of the abutment, forming an elongated bank close to the flume wall.

For Group 2 experiments (RC3 and RC4, Figures 7.1c and 7.1d) and Group 4 (RC7 and RC8, Figures 7.1g and 7.1h) experiments, with near threshold velocity, the deepest scour holes progressively moved from the upstream corner of the abutment towards the true left side channel wall before equilibrium was reached. An elongated elliptical scour hole, about 50-mm wide and 150-mm long, formed, extending transversally from the upstream abutment tip towards the true left bank. The suggested explanation is that, for these experiments, the scour agents changed from locally contracted flow and turbulence to downflow and the principal
vortex. As was similarly observed by Kwan (1984), sand ridges were formed at the interface between the principal vortex and the secondary vortex (Figures 7.1c, 7.1d, 7.1g and 7.1h). It should be noted that the sand ridges have similar shapes for long and short contraction lengths.

For Group 1 experiments, with smaller velocity and smaller lateral contraction (Figures 7.1a and 7.1b), the sand bed at the true right side of the flume remained intact throughout the experiments, because flow was highly contracted locally around the abutment. For Group 2 experiments with larger velocity (Figures 7.1c and 7.1d), but the same lateral contraction as Group 1, erosion outside the hole was initially absent, but gradually spread across the whole section for both short and long contraction lengths. In contrast, for Group 4 experiments (Figures 7.1g and 7.1h), which had both larger velocity and more severe lateral contraction, contraction scour was observed during the initial phase of the experiments.
Figure 7.1 Bed bathymetry at the equilibrium state of scour: a. Experiment RC1; b. Experiment RC2; c. Experiment RC3; d. Experiment RC4; e. Experiment RC5; f. Experiment RC6; g. Experiment RC7; h. Experiment RC8;

There are two other interesting findings:

1. A comparison of Group 1 and Group 3 results shows that the scour holes for the smaller and larger contractions ($L_a = 400$-mm and 800-mm, respectively) have similar shape and extent. The more severe contraction (Group 3) generated deeper scour holes, but the true left extent of these scour holes (from the centre of the scour hole to the true left edge of the scour hole) was 400-mm, which is similar to the true left extent of the Group 1 scour holes.

2. For the long contraction experiments in Groups 1 and 3 (smaller velocities), Figures 7.1a and 7.1e show that at the downstream corner of the wide abutment, relatively shallow scour holes were generated, attributed to wake vortices and flow divergence.
Above all, the most noteworthy finding is that, for all four combinations of lateral contraction and flow velocity, the final morphologies of the scour holes were insensitive to contraction length. For similar velocities within each group of experiments, the shape and extent of the scour holes are similar to each other.

### 7.2.3 Surface velocity field

#### 7.2.3.1 Surface velocity field for the plane-bed condition

Figures 7.2 and 7.3 show the plane-bed two-dimensional surface velocity fields obtained from PTV software, Streams 2.00 (Nokes 2012), for relatively large velocity, large contraction ($L_a = 800$-mm), and long and short contraction, respectively (Group 4 in Table 7.1). These diagrams show the typical acceleration patterns resulting from physical contraction in width. The principal features of the surface flow for a narrow abutment, in plane-bed condition, and $V_i \approx 0.6V_{ic}$ (partially based on Figure 7.3 and partially on laboratory observation), are the following:

1. A small slowly circulating anticlockwise eddy upstream of the abutment, which is like a stagnant water region
2. Laterally converging streamlines that do not diverge until about 3-m downstream from the abutment
3. Intensive flow turbulence just downstream of the abutment nose
4. A small clockwise circulating eddy just downstream of the abutment
5. A much larger-scale eddy downstream near the bank.

For higher velocity, $V_i \approx 0.9V_{ic}$, transverse contraction and turbulence were more significant but the two small eddies immediately upstream and downstream of the abutment (items 1 and 4, above) were too feeble to be noticed. In addition, the large eddy downstream of the
abutment moved further downstream and became weaker because of the inertia effect. The flow pattern of the wider abutment had features similar to those of the narrow abutment (as above); the only noticeable difference was a large scale reverse eddy by the abutment side face (abutment side face here is the plane parallel to the flume wall, Figure 7.2). Flow in the immediate vicinity of the abutment side face is characterized by a reverse direction flow. It is worth noting that in the plane-bed condition, the backwater effect was pronounced. For $V_t \approx 0.9V_{le}$, the water level just upstream of the abutment was 120-mm and the water level just downstream was 80-mm. The water level difference diminished after about 24 hours, at which time the scour hole had developed to about 70% of its equilibrium depth.

Figures 7.2 and 7.3 show that, in general, the surface flow field is indistinguishable for the long and short contraction experiments. Regardless of contraction length, transverse mass transfer after the contracted reach is very small in consequence of the inertia phenomenon. The small amount of mass transfer could be attributed to pressure difference between stagnant subsections and high velocity subsections.

Figures 7.4 and 7.5 show the streamwise velocity $V_x$ and transverse velocity $V_y$ distribution curves at different longitudinal transects for the Group 4 experiments in the plane-bed condition, for which surface velocity vectors are shown in Figures 7.2 and 7.3. The reference velocities $V_{xr}$ and $V_{yr}$ are the mean streamwise and transverse surface velocity components, respectively, at $X = 750$-mm. Here, because of the large backwater effects and intensive contraction at the initial state, the surface velocity components upstream ($X = 750$-mm) are more representative than the mean velocity as the reference values. Except for streamlines close to the abutment ($690$-mm < $Y$ < $740$-mm, around Transect D), the streamwise velocity from $X = 750$-mm to $X = -500$-mm at Transects A ($Y$= 140-mm), B ($Y$= 340-mm) and C
(Y= 540-mm) reached up to 2.75 times the reference velocity, and then gradually reduced after X = −500-mm. The surface velocities after X = −1375-mm could not be covered by the camera, but their values do not affect the findings reported herein. It is obvious from Figure 7.4 that this acceleration pattern is almost the same for both contraction lengths in this study. For the narrow abutment, at Transect D (Y = 710-mm), streamwise velocity fell to almost zero at X = −50-mm (at the downstream corner of the abutment) and then started to increase. In contrast, for the wider abutment at Transect D, velocity close to the abutment side face changed direction at X = −50-mm and maintained a small value (about 58-mm/s) until the downstream corner of the abutment.

Figure 7.5 shows that transverse velocities on Transect D diverted the most and velocities on Transect A the least. Regardless of contraction length, transverse velocities at different transects decreased to zero around X = −500-mm and leveled off with small negative values; i.e., with or without lateral restraint, flow started to diverge 500-mm downstream of the abutment's upstream corner and maintained a stable but small transverse velocity. For the narrow abutment, it seems that there was a water wall acting just like a wide abutment, preventing lateral mass and energy transport.

From the discussions above, for the un-scoured state, surface velocity fields for the long and the short contraction experiments have very high similarity. The accelerated water body did not diverge significantly until about 3-m (this value would be dependent on contraction ratio and approaching velocity) after the obstruction, rendering the contraction length effect secondary or even non-contributing.

Lim and Nugroho (2004) proposed that the near-bottom shear stress distribution is similar to the surface velocity field, because the velocity profile in their work was reasonably well
represented by a logarithmic relationship. The validity of this assumption was afterwards verified by van Ballegoooy (2005). Thus, for both the wide and narrow abutments, the driving force of erosion, i.e., shear stress acting on sediment, is similar, resulting in similar equilibrium bed morphologies, and also similar maximum scour depths (Section 7.2.2.2).

For small flow velocities, inertia phenomena were less significant than for near threshold velocities. However, laboratory observation and the equilibrium bed morphologies in Figures 7.1a, 7.1b, 7.1e and 7.1f still demonstrated that contraction length has little effect on abutment scour for small velocity experiments.

![Two-dimensional surface flow field around a 1100 mm-wide abutment for the plane-bed condition, Experiment RC7](image)

**Figure 7.2** Two-dimensional surface flow field around a 1100 mm-wide abutment for the plane-bed condition, Experiment RC7
Figure 7.3 Two-dimensional surface flow field around a 50 mm-wide abutment for the plane-bed condition, Experiment RC8
**Figure 7.4** Streamwise velocity distributions of four longitudinal transects for the plane-bed condition. Red lines correspond to 1100-mm wide abutment in Figure 7.2; black lines correspond to 50-mm wide abutment in Figure 7.3.

**Figure 7.5** Transverse velocity distributions of four longitudinal transects for the plane-bed condition. Red lines correspond to 1100-mm wide abutment in Figure 7.2; black lines correspond to 50-mm wide abutment in Figure 7.3.
7.2.3.2 Surface velocity field at the equilibrium state

Figures 7.6 and 7.7 show surface velocity fields superimposed on the bed bathymetries at the equilibrium state, for the same long and short contraction experiments (Group 4) as in the previous discussion.

For the long contraction experiment (Figure 7.6), velocity vectors from both sides of the approach section converged to the centre of the contracted channel. In the contracted section of the channel, centred at the point $X = -800$-mm, $Y = 640$-mm, the predominant feature in the flow field is a rectangular shaped area where the transverse velocity is so significant that flow lines are almost perpendicular to the centre line of the flume. In addition, laboratory observation shows that upward flow exists along the side face of the abutment, indicating an open-ended lee eddy in this area. The above described stream-wise vortex (with a horizontal axis pointing downstream) might be attributed to subsurface flux from the other side of the flume; alternatively, or in addition, it might be attributed to the existence of the sand ridge at around $Y = 200$-mm. After passing the bluff body, because of shallower water depth resulting from sand deposition, the flow turns towards the abutment side of the flume, and its velocity increases to larger than that of the approach flow.

For the short contraction experiment (Figure 7.7), the velocity field is different to that for the long contraction experiment, and it features weaker transverse velocity near the true right side of the channel, weaker diverging flow once past the abutment downstream corner and little evidence of a stream-wise vortex. However, the velocity field around the upstream corner of the abutment maintains high similarity with the long contraction velocity field, and at this position scour holes originated and developed for both the long and the short contraction experiments. The combination of scour bathymetries and surface velocity fields shows that the scour holes in Figures 7.6 and 7.7 experienced similar scouring processes.
Figure 7.6 Two-dimensional surface flow field around a 1100-mm-wide abutment at the equilibrium state, superimposed on the equilibrium scour hole bathymetry; Experiment RC7

Figure 7.7 Two-dimensional surface flow field around a 50-mm-wide abutment at the equilibrium state, superimposed on the equilibrium scour hole bathymetry; Experiment RC8
7.2.4 Equilibrium scour depth

As discussed above, for relatively small flow intensity, the location of the maximum scour depth \(d_s\) always occurred at the upstream corner of the abutment; for relatively large flow intensity, the elliptically-shaped deepest area moved toward the true left bank along the upstream face of the abutment. For the smaller velocity experiments, the scouring process was dominated by the principal vortex and the locally contracted flow, and the downflow was a less significant scour agent. In contrast, for the higher velocity experiments, the deepest scouring area stretched 150-mm from the abutment toe towards the left bank, where downflow still contributed the most. For \(1 < L_o/y < 25\), Melville (1992) suggests that \(d_s \propto \sqrt{L_o/y}\). To remove the effect of velocity differences within each group on the maximum scour depth, \(K_j = \frac{V}{V_1}\) is introduced following Equation 6.9, and therefore the maximum non-dimensional scour depth, \(\frac{d_s V_1}{\sqrt{L_o/y V_1}}\), is discussed. As shown in Table 7.2, when the relative contraction length \(L' = L_c/B\) changes from 0.03 (= 50/1540) to 0.71 (= 1100/1540), the maximum non-dimensional scour depth due to the contraction length effect changes by less than 10% (Group 3), which is in accordance with the results of Wu and Molinas (2001) and Kohli and Hager (2001). The effect is so small that the suggested value of the correction factor for scour depth, \(K_w\), for the effect of contraction length is 1.1 for \(L_c/B \leq 0.03\) (or \(L_c/B_c \leq 0.04\), in which \(B_c\) is the contracted channel width). Compared with the fine-grained soil experimental results of Briaud et.al (2005), the results indicate that even when \(L_c/B_c\) is much smaller than 0.25, increasing the predicted maximum scouring values is not necessary for a sand river-bed.
Table 7.2 Maximum scour depth for the long contraction study (rectangular channel)

<table>
<thead>
<tr>
<th>Experiment</th>
<th>RC1</th>
<th>RC2</th>
<th>RC3</th>
<th>RC4</th>
<th>RC5</th>
<th>RC6</th>
<th>RC7</th>
<th>RC8</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V$ (mm/s)</td>
<td>214</td>
<td>210</td>
<td>304</td>
<td>290</td>
<td>214</td>
<td>172</td>
<td>304</td>
<td>281</td>
</tr>
<tr>
<td>$d_s$ (mm)</td>
<td>194</td>
<td>195</td>
<td>378</td>
<td>377</td>
<td>238</td>
<td>212</td>
<td>411$^e$</td>
<td>394</td>
</tr>
<tr>
<td>$L^*$</td>
<td>0.71</td>
<td>0.03</td>
<td>0.71</td>
<td>0.03</td>
<td>0.71</td>
<td>0.03</td>
<td>0.71</td>
<td>0.03</td>
</tr>
<tr>
<td>$\frac{d_s V}{\sqrt{L_s y V}}$</td>
<td>1.56</td>
<td>1.60</td>
<td>2.15</td>
<td>2.24</td>
<td>1.36</td>
<td>1.50</td>
<td>1.65</td>
<td>1.71</td>
</tr>
</tbody>
</table>

Note: superscript $^e$: the scour hole reached to the bottom and the scour depth was extrapolated from the surrounding slope using Matlab.

7.3 Long contraction scour in a compound channel

The effect of contraction length on abutment scour was investigated around solid vertical abutments in a compound channel. The objective of this study was to better understand the effect of contraction length on flow features around the abutment and on the associated abutment scour depth, and to investigate the effect of scour countermeasures on abutment scour with varying contraction lengths. Three-dimensional velocity fields for the plane-bed condition, and equilibrium scour bathymetries are presented and discussed.

Five groups of experiments were performed, with two experiments per group (Table 7.1), covering the eight possible combinations of four contraction lengths and with or without rock riprap apron protection. Two additional experiments with apron-protected abutments (Group 9, CC9 and CC10) were conducted to fill data gaps resulting from large changes in $L_c$. All six apron-protected experiments had aprons with the same transverse extent, the same riprap size, and all had the same scour duration of 48-hours. It should be noted that the streamwise
extent of the apron varied with contraction length. All four experiments without scour countermeasures had a duration of 22-hours.

7.3.1 Experimental observation

To show the scour development differences between a short and a long contraction, the development of scour (for experiments CC9 and CC10) was recorded by an overhead camera. In the absence of rock riprap apron, the scour hole was centred at the upstream corner of the abutment, and scour differences could not be distinguished from photo recordings of different experiments. Therefore, the two discussed experiments had rock riprap aprons protecting the abutments. The scour development features of these two experiments (Experiment CC9 in Figure 7.8 and Experiment CC10 in Figure 7.9) are the following:

1. In the initial state of scour (e.g., in the first hour), the scour differences between these two contraction length experiments were found to be indistinguishable: scour initiated at the downstream side of C.S.3 (the cross section of the abutment upstream face, see Chapter 3), developing transversely towards the main channel because of the strongly diverged flow (arrowed direction in Figures 7.8 and 7.9, t = 1-min); as scour developed (t = 10-min and t = 1-hour), the conveyance around the abutment increased, and the diverged streamlines gradually became parallel to the approaching flow; and elongated scour holes appeared, with the scour thalweg approximately parallel to the approaching flow.

2. After the initial state, for a short contraction (Figure 7.8), the scour development was similar to that described in Section 6.2.2: the rock riprap dynamically armoured part of the scour hole, with the scour thalweg curving towards the floodplain; and the scour hole developed into a curved form, with the maximum scour depth at the downstream side of the abutment. For a long contraction (Figure 7.9), the transverse
mass transfer was impeded by the abutment. Moreover, the rock riprap in the long-extending apron (streamwise direction) slid down and mitigated scour along the width of the abutment. Therefore, the scour thalweg slightly curved towards the main channel, and a narrow and elongated scour hole occurred; the maximum scour depth was located downstream of C.S.3.

To summarise, scour development differences were found to be noticeable between a narrow and a wide apron-protected abutment (between a short and a long contraction). The development differences were mainly caused by different streamwise extents of rock riprap apron, and also by different widths of impermeable abutments.

**Figure 7.8** The development of scour for Experiment CC9 ($L_c = 45$-mm)
Figure 7.9 The development of scour for Experiment CC10 ($L_c = 1600$-mm)

7.3.2 The temporal development of scour

For the unprotected abutment, the scour hole typically was centred at the upstream corner. To obtain the temporal development of scour, six ultrasonic depth sounders were positioned on C.S.3. Measurements were taken at $Y = 100$-mm, $330$-mm, $460$-mm, $600$-mm, $860$-mm and $1000$-mm (the upstream corner), and the temporal development of scour for Experiment CC1 is exemplarily shown in Figure 7.10. It is apparent that scour developed most rapidly at the upstream corner. For each of the above transverse positions, temporal developments of scour were compared for different contraction length abutments (Figures 7.11(a-f)). Results show that, for each selected transverse position in this part of the study, scour histories for different contraction lengths lay in a narrow band for $L_c$ varying significantly from $12$-mm to $2445$-
mm. These narrow bands in Figure 7.11 indicate that the scour history of the whole scour hole may be unrelated to contraction length. The indication is supported by similar bed morphologies after the same scour duration (see Figure 7.14). Also, temporal developments of scour shown in Figure 7.11 indicate that equilibrium scour depths were achieved.

**Figure 7.10** The temporal development of scour at six selected transverse locations for an unprotected abutment (Experiment CC1) at C.S.3
Figure 7.11 Temporal developments of scour around unprotected abutments for different contraction lengths at C.S.3  

a. Y = 1000-mm (the upstream corner);  
b. Y = 860-mm;  
c. Y = 600-mm;  
d. Y = 460-mm;  
e. Y = 330-mm; and  
f. Y = 100-mm

For the apron-protected abutment, the location and the geometry of the scour hole were found to vary with contraction length. To better understand the features of the temporal development of scour for different contraction lengths, the following setups were made:
• For experiments CC2 \((L_c = 12\text{-mm})\) and CC8 \((L_c = 2445\text{-mm})\), five ultrasonic depth sounders were positioned on C.S.3, and the results are shown in Figure 7.12.

• For Experiment CC9 \((L_c = 45\text{-mm})\) and CC10 \((L_c = 1600\text{-mm})\), five ultrasonic depth sounders were positioned on cross sections where the maximum scour depths were expected to occur: for Experiment CC9 \((L_c = 45\text{-mm})\), five ultrasonic depth sounders were positioned on the cross section 835-mm downstream of C.S.3; for Experiment CC10 \((L_c = 1600\text{-mm})\), five ultrasonic depth sounders were positioned on the cross section 350-mm downstream of C.S.3. The results are shown in Figure 7.13. As shown in the equilibrium bed bathymetry plots in Figures 7.15b and 7.15e, the positioning of these two expected cross sections was found to be precise enough to represent the cross sections at the maximum scour holes.

Figures 7.12(a – e) show that, for each selected transverse position on C.S.3, scour histories for experiments CC2 and CC8 lay in a narrow band for a large variation of \(L_c\) from 12-mm to 2445-mm. Combining the temporal development results for protected and unprotected abutments reasonably leads to the conclusion that, for the experimental condition in this study, the effect of contraction length is insignificant in the temporal development of scour at C.S.3, regardless of the presence or absence of the rock riprap apron.
Figure 7.12 Temporal developments of scour around apron-protected abutments for two different contraction lengths at C.S.3 a. Y = 860-mm; b. Y = 600-mm; c. Y = 460-mm; d. Y = 330-mm; and e. Y = 100-mm

Figures 7.13(a-f) show that, for transverse positions close to the centre of the scour hole (e.g., Y = 460-mm and 600-mm), scour histories varied significantly with contraction length when rock riprap aprons were employed. The transverse positions of the maximum scour depth were found to be different for different contraction lengths: for the short contraction experiment \( (L_c = 45\text{-mm}) \), the equilibrium scour hole centred at \( Y = 600\text{-mm} \), on the floodplain; for the long contraction experiment \( (L_c = 1600\text{-mm}) \), the equilibrium scour hole centred at \( Y = 400\text{-mm} \), on the main channel bank. This location difference was explained in Section 7.3.1. The development differences between the short and the long contractions are probably due to:

1. The near-bottom spatial distributions of turbulence around the short and long contraction abutments are different; these differences are discussed in Section 7.4.
2. The streamwise extent of the apron affects the temporal development of scour. For relatively narrow abutments (e.g., $L_c \leq 135$-mm), the maximum scour depth was located about 800-mm downstream of C.S.3, and the equilibrium scour hole was only partly armoured by the rock riprap, as shown in Figure 7.8. For relatively wide abutments (e.g., $L_c \geq 810$-mm), the maximum scour depth was located about 350-800-mm downstream of C.S.3, and most of the equilibrium scour hole was armoured.

![Figure 7.13](image.png)

**Figure 7.13** The temporal development of scour with different contraction lengths, at different transverse distance across the expected scour hole; a. $Y = 860$-mm; b. $Y = 600$-mm; c. $Y = 460$-mm; d. $Y = 400$-mm; e. $Y = 330$-mm; and f. $Y = 100$-mm
7.3.3 Bed morphologies at the equilibrium state

Results of the aforementioned rectangular channel study show that the effect of contraction length on equilibrium bed morphology is insignificant for unprotected abutments. Results in this compound channel study show that:

- The effect of contraction length on equilibrium bed morphology is insignificant when the abutment is not protected by any form of scour countermeasures.
- The effect of contraction length on equilibrium bed morphology is significant when the abutment is protected by a rock riprap apron.

To highlight the location of the maximum scour depth in each experiment, a red cross and a read star are used on the floodplain and in the main channel, respectively (Figures 7.14 and 7.15).

For each unprotected abutment, two scour holes occurred, one on the floodplain and the other in the main channel. The maximum scour hole was centred at the upstream corner of the abutment, with $L_t/L_o \approx 0$, as shown in Figures 7.14(a - d). Here, $L_t$ is the transverse distance between the abutment and the centre of the scour hole. These scour holes resemble frustums of inverted circular cones, similar to those in the rectangular channel study; the shape and the dimensions of the scour holes are similar, regardless of contraction length. The scour agent mainly consists of locally contracted flow, local turbulence, the principal vortex, and the associated downflow. A relatively shallow scour hole occurred in the main channel, approximately 500-mm downstream of C.S.3. Except for the longest-contraction experiment ($L_c = 2445$-mm), the shape and the dimensions of the scour holes in the main channel were similar; the longest-contraction abutment caused a further elongation of the scour hole. In the main channel, the scour agent mainly comprises contracted flow, shedding turbulences and the secondary vortex. It should be noted that, at the downstream corner of the longest-
contraction abutment, a relatively shallow scour hole was generated (Figure 7.14d), mainly due to the wake vortices.

Considering the combined rectangular (Section 7.2.1) and compound channel results, it is reasonable to conclude the following: for all the reported unprotected-abutment experiments in this study, the contraction length does not affect the shape, the dimension, or the location of the maximum scour hole; and the channel configuration does not affect the shape or the location of the maximum scour hole.

Additional experiments would be necessary to verify the presented results for a wider range of contraction lengths, flow depths, channel geometries, flow intensities and abutment setback distances.
Figure 7.14 Bed bathymetry at the equilibrium state of scour: a. Experiment CC1, unprotected abutment, $L_c = 12$-mm; b. Experiment CC3, unprotected abutment, $L_c = 135$-mm; c. Experiment CC5, unprotected abutment, $L_c = 810$-mm; and d. Experiment CC7, unprotected abutment, $L_c = 2445$-mm
For apron-protected abutments, the equilibrium scour holes were located downstream of C.S.3, as shown in Figures 7.15(a-f). The shape, the dimension and the location of the scour holes were experiment-specific, depending greatly on the contraction length. Because the apron effectively mitigated the scour in the vicinity of the abutment, the observed scour holes in Figures 7.15(a-f) were mainly caused by contracted flows and turbulence structures. Most notably, short contraction abutments caused larger and deeper scour holes than long contraction abutments. For relatively narrow abutments ($12\text{-mm} \leq L_c \leq 810\text{-mm}$), the maximum scour holes were located $800 - 835\text{-mm}$ downstream of C.S.3, and they were centred on the floodplain, with $L_s \approx 400\text{-mm}$. By contrast, for relatively wide abutments ($1600\text{-mm} \leq L_c \leq 2445\text{-mm}$), two relatively shallow scour holes occurred: one was located about $470\text{-mm}$ downstream of C.S.3 and centred on the main channel bank, with $L_s \approx 600\text{-mm}$; the other was located about $750\text{-mm}$ downstream of C.S.3, and was centred in the main channel. The data in this study are insufficient to obtain a clear relationship between $L_c$ and the location of the scour hole. However, Figure 7.15 suggests that the scour hole may “move” upstream, and “move” towards the main channel when a narrow abutment is replaced with a wide one.
Figure 7.15 Bed bathymetry at the equilibrium state of scour: a. Experiment CC2, apron-protected abutment, \( L_c = 12 \text{-mm} \); b. Experiment CC9, apron-protected abutment, \( L_c = 45 \text{-mm} \); c. Experiment CC4, apron-protected abutment, \( L_c = 135 \text{-mm} \); d. Experiment CC6, apron-protected abutment, \( L_c = 810 \text{-mm} \); e. Experiment CC10, apron-protected abutment, \( L_c = 1600 \text{-mm} \); and f. Experiment CC8, apron-protected abutment, \( L_c = 2445 \text{-mm} \); colour unit: mm. Note: the colour scale used in this figure is different from that in Figure 7.14

### 7.3.4 Equilibrium scour depth

Figure 7.16a shows the relationship between the normalized scour depth on the floodplain and the normalized contraction length.
For unprotected abutments, the values of $d_{s-FP}/y_{f1}$ varied slightly from 1.57 to 1.62 when $L_c/B_j$ increased from 0.01 to 2.46. Here, $d_{s-FP}$ is the scour depth measured from the initial floodplain bed. Considering the possible uncertainties of physical modelling, it is reasonable to conclude that $d_{s-FP}/y_{f1}$ does not change with $L_c/B_j$ in this study. This lack of change is because the scour agents (including locally contracted flow, local turbulence, the principal vortex and the associated downward flow at the upstream face of the abutment) depend on the approaching flow and the contraction ratio, but do not depend on contraction length.

For apron-protected abutments, the values of $d_{s-FP}/y_{f1}$ decreased from 1.72 to 1.13 when $L_c/B_j$ increased from 0.01 to 2.46. This decrease is because the scour agents (including contracted flow, shedding turbulence, the secondary vortex) depend on the approaching flow, the extent of the apron and the flow pattern downstream of C.S.3. Apparently, a wide impervious abutment wall and a long-extending apron significantly altered the flow and scour patterns during the experiment. For the data in this study, the best-fit relationship is the following (the red dashed line in Figure 7.16a):

$$d_{s-FP}/y_{f1} = 1.2 \left( \frac{L_c}{B_j} \right)^{-0.082} \quad \text{for} \quad 0.01 \leq \frac{L_c}{B_j} \leq 2.26$$

Equation 7.1

in which $R^2 = 0.99$.

Figure 7.16b shows the relationships between the normalized scour depth in the main channel and the normalized contraction length. Results suggest that, the effect of the contraction length on the scour hole in the main channel is significant.
For unprotected abutments, the values of $d_{s-MC}/y_m$ more than doubled, from 0.09 to 0.20, when $L_c/B_f$ increased from 0.01 to 2.46. Here, $d_{s-MC}$ is the scour depth measured from the initial main channel bed. The increasing trend of $d_{s-MC}/y_m$ is because of the following: for a long contraction abutment, the flow was laterally restricted for a relatively long distance, and contraction scour was fully developed in the contracted reach; for a short contraction abutment, the inertia of flow gradually reduced as scour developed, the contracted flow was partly relieved and returned to the floodplain, and relatively shallow scour occurred in the main channel. For the available data, the best-fit relationship is the following (the black dashed line in Figure 7.16b):

$$d_{s-MC}/y_m = 0.045 L_c/B_f + 0.093 \quad \text{for} \quad 0.01 \leq L_c/B_f \leq 2.26 \quad \text{Equation 7.2}$$

in which $R^2 = 0.95$.

For protected abutments, the values of $d_{s-MC}/y_m$ decreased from 0.46 to 0.27 when $L_c/B_f$ increased from 0.01 to 2.46. This decreasing trend of $d_{s-MC}/y_m$ is probably because, for the short contraction abutment, scour on the floodplain developed rapidly and a relatively large scour hole extended into the main channel; hence a relatively deep scour depth occurred in the main channel. For the long contraction abutment, the long-extending rock riprap apron effectively mitigated the scour; hence relatively shallow scour depth occurred. For the data in this study, the best-fit relationship is the following (the red dashed line in Figure 7.16b):

$$d_{s-MC}/y_m = 0.30 \left( L_c/B_f \right)^{-0.10} \quad \text{for} \quad 0.01 \leq L_c/B_f \leq 2.26 \quad \text{Equation 7.3}$$

in which $R^2 = 0.91$.

One interesting finding is that, on the floodplain, the equilibrium scour depths are similar for any unprotected abutment and the apron-protected short-contraction abutment; in the main
channel, the equilibrium scour depths are similar for the unprotected long-contraction abutment and the apron-protected long-contraction abutment.

**Figure 7.16** Relationships between scour depth and the contraction length: **a.** variations of \( d_{s,FP} / y_{f1} \), the normalized scour depth on the floodplain with \( L_c / B_f \), and **b.** variations of \( d_{s,MC} / y_{m1} \), the normalized scour depth in the main channel with \( L_c / B_f \).
Figure 7.17 shows six cross-sectional profiles of the maximum scour holes at the equilibrium state for the six apron-protected abutments. It is apparent that there are two groups of profiles:

- The long-contraction group ($810\text{-mm} \leq L_c \leq 2445\text{-mm}$, in which the scour holes were located between the downstream and the upstream faces of the abutment), for which the residual apron width $W_s > 0$; and the side slope of the scour hole $\beta_3 \approx 27^\circ$. The geometrical relationship between the location of the scour hole and the maximum scour depth is: $\tan \beta_3 = \frac{d_{s,FP}}{L_{s1}}$. Here, $L_{s1}$ is the transverse distance from the edge of the residual apron to the location of the maximum scour depth.

- The short-contraction group ($12\text{-mm} \leq L_c \leq 135\text{-mm}$, in which the scour holes were located downstream of the abutment), for which scour extended towards the true left flume wall (to about $Y = 1280\text{-mm}$ in this study). The residual apron width $W_s < 0$. The side slope of the scour hole $\beta_4 \approx 27^\circ$, as for the long-contraction group. The geometrical relationship between the location of the scour hole and the maximum scour depth is: $\tan \beta_4 = \frac{d_{s,FP}}{L_{s2}}$. Here, $L_{s2}$ is the transverse distance from the location of the maximum scour depth to the edge of the un-scoured floodplain bed (i.e., the top of the slope at $Y = 1200\text{-mm}$).

For apron-protected abutments, results show that contraction lengths significantly affect the transverse and the longitudinal locations of the scour hole. For a particular situation (channel configuration, flow intensity, and flow depth) there may be a critical contraction length ($L_{ct}$), for which the corresponding cross section sketch is shown in Figure 7.18. For experiments with $L_{ct}$, the residual width of the apron $W_s = 0$, and the scour hole is centred at the cross
section at the downstream face of the abutment (C.S.4). Assuming that the critical contraction length $L_{ct}$ exists, for apron-protected abutment and the flow condition in this part of the study, results suggest that $135 \text{mm} < L_{ct} < 810 \text{mm}$ (see Figure 7.17). However, the scour hole was located close to the downstream face of the abutment when $L_c = 810 \text{mm}$ (see Figure 7.15d). Considering the available data, $L_{ct} \approx 810 \text{mm}$ (a six-lane bridge model) in this part of the study. Short contraction is defined as $L_c < L_{ct}$, and long contraction is defined as $L_c > L_{ct}$. The value of $L_{ct}$ is expected to be case-specific. Komura (1966) and Webby (1984) defined long contraction as $L_{ct}/B \geq 1.0$; whereas in this study, $L_{ct}/B \approx 0.5$.

This part of the study was not designed to quantify $L_{ct}$, instead it was designed to prove that contraction length may affect total scour around apron-protected abutments. In prototype conditions, $L_{ct}$ may exist for apron-protected abutments. However, quantifying $L_{ct}$ requires further studies.

**Figure 7.17** Cross section profiles of the maximum scour holes for different apron-protected abutments. *Note:* the black dashed line denotes the cross section profile before scour
Figure 7.18 The cross section profile of the maximum scour hole for a critical width \( L_{cr} \) abutment. *Note:* the black dashed line denotes the cross section profile before scour; the blue line denotes a threshold scour condition and the residual apron width \( W_3 = 0 \).

For un-protected abutments, \( L_{cr} \) may not exist, and is not discussed here. It should be noted that for the four un-protected abutments in this study, the side slopes of the scour hole \( \beta_5 \approx \beta_6 \approx 27^\circ \), as shown in Figure 7.19 (see Chapter 6 for the discussion of \( \beta \)).

Figure 7.19 Cross section profiles of the maximum scour holes for different width un-protected abutments. *Note:* the black dashed line denotes the cross section profile before scour.
The contribution of local turbulence is significant for general abutments. To evaluate the contribution of the turbulence on scour, the measured scour depths in this study are compared with scour depths predicted by the long contraction theory (Laursen, 1963).

The theoretical long contraction scour depth in this study is:

\[ y_{fc} = y_{f1} \left[ (V_{f1} / V_{f1c})(q_{f2} / q_{f1}) \right]^{6/7} = 192 \text{-mm} \]

The comparison is shown in Figure 7.20. The amplification factor, \( r_T \), mainly reflects the contribution of turbulence structures in the flow.

![Figure 7.20 Variation of \( L_c/B_f \) with the amplification factor \( r_T \)](image)

**Figure 7.20** Variation of \( L_c/B_f \) with the amplification factor \( r_T \)

Results show that, the amplification factor \( r_T \) ranges from 2.0 to 2.6 for the apron-protected, long setback, vertical-wall abutments in this study. The reasons for \( r_T \) varying with \( L_c/B_f \) are the following:

1. For a long contraction, the long extent of the apron can greatly armour the scour hole, leading to relatively small values of \( r_T \); for a short contraction, the short extent of the apron can only partly armour the scour hole, leading to relatively large values of \( r_T \).
2. The near-bottom turbulence structures vary with contraction lengths, therefore the values of $r_T$ vary with contraction lengths. The near-bottom flow features are discussed in the following section.

For apron-protected abutments in a compound channel, the correction factor, $K_w$ (see Section 7.2.3), is tentatively suggested to be 1.3 ($2.6/2.0$) when $L_c/B_f \leq 0.125$. The value of $K_w$ is expected to be case-specific and change of experimental conditions.

Results in Chapter 6 suggest that, for apron-protected abutments, the abutment shape insignificantly affects $r_T$. Combining results from this study with those calculated by Hong et al. (2015) shows that $r_T \approx 2.6$ for long setback, short contraction abutments in FS flows.

### 7.3.5 Flow fields around a short and a long contraction abutments

As the initiator of scour, the near-bottom flow is critical to scour development. To better understand the scour differences between the short and the long contraction apron-protected abutments, flow fields around a one-lane bridge abutment model ($L_c = 135$-mm, short contraction) and a six-lane bridge abutment model ($L_c = 810$-mm, long contraction) were measured for the plane-bed condition before scour. Flow field measurements were mainly conducted downstream of the abutment. The prototypes of these two models can be found in real life. The flow information, particularly the near-bottom flow (10-mm above the plane-bed) on the floodplain, is discussed.

Figures 7.21a and 7.21b show the time-averaged X-Y velocity vectors at 10-mm above the plane-bed for short and the long contraction models, respectively. Results show that, at the
initial state of scour, the X-Y plane time-averaged velocity field was insignificantly affected by different contraction lengths. For both models, flow intensities by the abutment side face (Figure 7.21) were relatively small. Weak anti-clockwise vortices were observed for the measured range of the long contraction (see the rectangular-marked area, Figure 7.21b).
Figure 7.21 Two-dimensional velocity vectors at 10-mm above the plane-bed for a. the short contraction model; b. the long contraction model; part of the flow area beside the abutment side face is magnified.

Figures 7.22a and 7.22b show the spatial distribution of the total turbulence kinetic energy \( \text{TKE} = 0.5 \left( \langle u'^2 \rangle + \langle v'^2 \rangle + \langle w'^2 \rangle \right) \), cm\(^2\)/s\(^2\) at 10-mm above the plane-bed for these two abutment models. The area marked by black dashed lines indicates the rock riprap apron in the scour experiments. The layouts of the apron followed the design guidelines of Lagasse et al. (2006). Results show that the distribution of TKE was significantly affected by contraction length in this study.
For the short contraction model (Figure 7.22a), TKE values peaked at about 165-cm²/s², and a relatively small area of high-intensity turbulence (red zone) was located at about 260-mm downstream of C.S.3. The trajectory of the shedding turbulence is marked by a green dashed line; scour was initiated along this line (see Figure 7.8). It can be seen that the protection by the apron was inadequate for the corresponding scour experiment: most of the high turbulence intensity area was not protected at the initial state of the experiment.

For the long contraction model (Figure 7.22b), peak TKE values (∼ 200-cm²/s²) were higher than those for the short contraction model. A relatively large area of high-intensity turbulence (red zone) was observed, and its centre was located at about 480-mm downstream of C.S.3. It can be seen that the protection by the apron was adequate in the corresponding scour experiment. The trajectory of the shedding turbulence is marked by a blue dashed line. The trajectory of the shedding turbulence for the short contraction model is superimposed on Figure 7.22b. It can be seen that for the long contraction, the shedding turbulence trajectory moved slightly towards the abutment.
Figure 7.22 The distribution of total turbulence kinetic energy (TKE) at 10-mm above the plane-bed for **a.** the short contraction (the one-lane bridge abutment) model; **b.** the long contraction (the six-lane bridge abutment) model, unit: cm²/s²

To better illustrate the difference in the TKE distribution for these two models, the spatial distribution of the difference of the TKE values at 10-mm above the plane-bed (long contraction – short contraction) is presented in Figure 7.23. It can be seen that the long contraction model had larger TKE values along the abutment side face (the red zone), while the short contraction model had larger TKE values (the blue zone) diagonally downstream of the abutment. In scour experiments, the more turbulent zone for the long contraction model (red zone in Figure 7.23) was protected by a long-extending apron, while the more turbulent zone for the short contraction model (blue zone in Figure 7.23) was not well protected by the corresponding apron.
Figure 7.23 Magnitude differences of the total turbulence kinetic energy (TKE) at 10-mm above the plane-bed between the one-lane and the six-lane bridge abutment (the magnitude differences represent the reductions between the TKE values for the one-lane bridge abutment in comparison to those for the six-lane bridge abutment), unit: cm²/s²

Similarly, the spatial distribution of the magnitude difference of the Reynolds shear stress at 10-mm above the plane-bed is shown in Figure 7.24. Here, the result of the principal Reynolds shear stress component, $R_{\text{w}} (= -u'v')$, is presented. Results in Figure 7.24 concur with those in Figure 7.23, and show that the long contraction abutment model had relatively strong turbulence close to the side face of the abutment (red zone), yet this area was well protected by the apron in the corresponding scour experiment; the short contraction model caused relatively strong turbulence (blue zone) at the downstream side of the abutment, beyond the protection of the apron. It should be noted that, the blue zone was close to the maximum equilibrium scour hole for the short contraction (marked by a red star).
Figure 7.24 Magnitude differences of the principal Reynolds shear stress component ($R_{uv}$) at 10-mm above the plane-bed between the short and the long contraction models, (the magnitude differences represent the reductions between the TKE values for the one-lane bridge abutment in comparison to those for the six-lane bridge abutment) unit: cm²/s²

The skewness coefficient measures the asymmetry of the velocity distributions and it reveals the presence of high-magnitude events within the velocity signal (Buffin-Belanger and Roy, 1998). A positive skewness suggests relatively rare, large positive values are more frequent than large negative values and vice versa (Lacey and Roy 2008, Lacey and Rennie, 2012). The streamwise and vertical components of skewness are defined as $Sk_{uu} = \frac{u'u''}{\sigma_u^3}$, and $Sk_{ww} = \frac{w'w''}{\sigma_w^3}$, respectively. For describing turbulent events, the terms ‘ejection’ ($Sk_{uu} < 0$, $Sk_{ww} > 0$), ‘sweep’ ($Sk_{uu} > 0$, $Sk_{ww} < 0$), ‘inward interaction’ ($Sk_{uu} < 0$, $Sk_{ww} < 0$), and ‘outward interaction’ ($Sk_{uu} > 0$, $Sk_{ww} > 0$) are used, following Nakagawa and Nezu.
(1977). Figure 7.25 shows the spatial distributions of $Sk_{uuu}$ and $Sk_{www}$ at 10-mm above the plane-bed for the two abutment models. Results in Figure 7.25 suggest that:

- Sweeping and ejection events mainly occurred in the vicinity of the abutment
- Inward interaction mainly occurred beside the abutment side face
- Outward interaction dominated the rest of the measurement area, particularly where flow diverged towards the main channel.

It should be noted that there is a close resemblance between the distributions of $Sk_{uuu}$ for the two contraction models. The short contraction model had more frequent sweeping and ejection turbulent events around the toe of the abutment than the long contraction model; whereas the long contraction model had more frequent inward interaction events along the side face of the abutment than the short contraction model.
Figure 7.25 Spatial distributions of skewness coefficients at 10-mm above the plane-bed for
a. $S_k_{uu}$ for the long contraction model; b. $S_k_{uu}$ for short contraction model; c. $S_k_{ww}$ for the
long contraction model; and d. $S_k_{ww}$ for short contraction model

Velocity measurements were taken adjacent to the side face of the abutment at $Y = 960$-mm
(40-mm away from the abutment side face), and the results are shown in Figures 7.26 and
7.27.

Figure 7.26 shows the velocity vectors in the X-Z plane. For these two models, the velocity
vectors at the upstream side of the abutment ($X \leq 0$) were similar, as expected; downstream
of C.S.3, weak reverse flows occurred. The reverse direction flow was probably caused by
pressure differences between the fast transversely moving flow and the slow moving flow
along the abutment side face. Observable reverse direction flows (marked by blue rectangles
in Figure 7.26) occurred within the longitudinal extent of contraction length, as shown in
Figures 7.26a and 7.26b. These weak reverse direction flows occurred over the apron zone, and probably were too feeble to contribute to scour. Downstream of these weak reverse direction flows, the X-Z plane flow decreased to near-zero magnitude.

![Figure 7.26](image)

**Figure 7.26** The X-Z velocity vectors at Y = 960-mm for a. the short contraction model; b. the long contraction model; reverse flows are marked by blue rectangles

Figure 7.27a and Figure 7.27b show the spatial distribution of TKE at Y = 960-mm for these two abutment models, and Figure 7.27c shows the magnitude differences of TKE (long contraction – short contraction). Results suggest that the contraction length affects the spatial distribution of TKE on the longitudinal transect close to the abutment side face; relatively turbulent flows occurred along the measured depth for the extent of the contraction length, as shown in Figure 7.27a and Figure 7.27b. Combining these results with the results in Figure 7.26 shows that the turbulence presented in Figure 7.27a and Figure 7.27b represents the fluctuating properties of the reverse direction flow.
Flow features for the unprotected areas significantly affect the scour development. The results for the longitudinal transect at $Y = 760$-mm (70-mm away from the edge of the apron) are exemplarily presented in Figure 7.28. Results show the spatial distributions of TKE were similar for the two models within a 400-mm distance downstream of C.S.3. Noticeable magnitude differences appeared after 400-mm downstream of C.S.3; flows were more turbulent for the short contraction model. From experimental observations (also see Figure 7.8 for scour development around a short contraction abutment), the relatively intense turbulence zone (between $X = 400$-mm and 800-mm) correlates with the temporal
development of scour for the short contraction abutment. Combining these turbulence results in conjunction with previous discussions about scour and the corresponding near-bottom flow differences suggests that scour can be predicted by flow features (e.g., TKE distributions) before scour. In this study, the deeper scour depth for the short contraction model is reflected in its stronger turbulence in the unprotected scour area.

Figure 7.28 The spatial distribution of TKE at Y = 760-mm for a. the short contraction model; b. the long contraction model; c. magnitude differences between the short and the long contraction models (long contraction – short contraction), unit: cm²/s²
Chapter 8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

A close-to-reality abutment scour project, consisting of several separate but closely related studies, is conducted to better understand the scour mechanisms and the scour patterns under free surface, submerged orifice and overtopping flows. Also, short and long contraction scour are investigated, with and without rock riprap apron. The following sections present the conclusions for each study in turn.

8.2 Live-bed scour study

The conclusions from the live-bed scour study are:

1. Four behaviours of sand dunes are observed, depending on the dunes’ characteristics and the interaction between the sand dunes and the abutment scour (see Figures 4.3 - 4.6). Also, it is found that both dune crests and dune troughs are detrimental to abutment scour: rock riprap may embed in the sand when a dune trough arrives at the bridge section; or the sediment winnowing process, in areas close to the abutment, may become more rapid when a large-amplitude dune crest arrives at the bridge section.

2. Scour results show that a rock riprap apron built to the guidelines of Lagasse et al. (2006) may not provide adequate protection for pressure flow conditions (Section 4.2.4).

3. Head loss at the bridge section is found to be significant. Results confirm the procedure proposed by Kindsvater et al. (1953) for FS and SO flows (see Table 4.3 and Figure 4.16). Equation 4.11 semi-empirically depicts the relationship between the head loss, and the flow intensity and the flow distribution.
4. The effect of flow condition on the temporal development of scour and the scour depth is discussed in terms of differences in vertical contraction, flow depth, flow distribution and flow relief. Results show that for similar flow intensities at the approach section, SO flows cause significantly more rapid and deeper scour than FS flows; comparing with SO flows, OT flows cause equivalent or slightly different scour depths. The scour depth differences between SO and OT flows vary with experimental conditions: scour depth (OT) ≥ scour depth (SO) for the 1:45 scale study (Figure 4.24) and scour depth (SO) ≥ scour depth (OT) for the 1:30 scale study (Section 4.5.2, Table A2). It is noteworthy that, for OT flows, flow relief may slow down the scour development for a limited time, but as scour develops, part of the overtopping discharge enters beneath the bridge deck and contributes to scour.

5. The effect of abutment length on scour development is distinct. For FS, SO and OT flows, scour depths increase with increase in abutment length (see Figure 4.26). For short setback abutments, nearly uniform scour occurs across the contracted section; for long setback abutments or no abutments, scour is deepest at the intersection of the floodplain and the main channel, where the bed shear stress is the highest across the contracted section. For all the live-bed data, the upper envelope is given by Equation 4.21.

6. The side slope of the scour hole at the bridge section maintains an H:V = 2:1 slope, representing a “retreating” behaviour of the main channel bank (Figures 4.31-4.33). To prevent undermining of the abutment toe, Equation 4.15 is proposed as a rock riprap apron design relationship, which is schematically shown in Figure 4.34.

7. For the apron-protected abutments, the available data suggest that the scour depth at the maximum scour location is asymptotic to the equilibrium state depth (Figures 4.36
and 4.37). Equation 4.16 provides the best-fit relationship for the available scour development data.

8. A nearly linear relationship between the normalized scour depth and the unit discharge ratio is obtained by adapting the long contraction theory of Laursen (1960). All the live-bed data lie in a narrow band, with the upper and the lower envelopes expressed by Equation 4.22a and Equation 4.22b, respectively.

9. The relationships between the scour amplification factor and the unit discharge ratio are shown in Figures 4.47a, 4.47b and 4.47c for FS, SO and OT flows, respectively.

10. Live-bed scour results are compared with formulas by Froehlich (1989), Melville et al. (2006), Ettema et al. (2010) and Hong et al. (2015). Results show that the Froehlich (1989) formula envelops the majority of the data in this live-bed study (see Figure 4.51). The Melville et al. (2006) formula predicts the live-bed scour depth (see Figure 4.52); but it is possible that the “reductive” component caused by bed forms and the “incremental” component caused by vertical contraction offset each other. OT flow results in this live-bed scour study provide a new envelope curve to the design curves by Ettema et al. (2010) (Figures 4.53 and 4.54). The formula proposed by Hong et al. (2015) can be applied to live-bed results by taking the flow intensity factor to be unity (see Figure 4.55). Results in this study and from the study by Hong et al. (2015) imply that the unit discharge ratio can adequately represent the total contraction for the investigated experimental conditions.
8.3 Flow patterns around abutments

The conclusions from the flow pattern measurements study are:

1. At the approach section, moving from the floodplain to the main channel, the distributions of the normalized turbulence intensities tend to merge, presenting near-isotropic turbulence, as shown in Figure 5.4.

2. Flow achieves a quasi-equilibrium state within a distance of about 650-mm downstream from the bridge. Within this distance, individual and overall turbulence intensities along different longitudinal lines all gradually converge to a constant level, as shown in Figure 5.7. Results show that the strongest turbulence intensity occurs close to the abutment toe at about 150-mm downstream of the abutment (C.S.5), as shown in Figure 5.7.

3. Close to the abutment toe, the behaviour of the compressed streamlines is the same, irrespective of flow rate and geometric contraction ratio differences. The magnitudes of the streamwise flow component and the turbulence intensities peak in the same region, at 110-mm to 160-mm transversally away from the abutment toe, as shown in Figures 5.8 and 5.10.

4. Results show that both the abutment length and the flow condition significantly affect the Y-Z plane flow pattern at C.S.4, as shown in Figures 5.11 (a-l): under FS flows, enclosed circular flows in the centre of the main channel can be clearly seen with long and medium setback abutments, but for short setback abutments this circular flow is found to be overwhelmed by a strong transverse flow; under SO flows, incomplete circular flows can be clearly seen with the long and medium setback abutments, with the discontinuity near the surface; under OT flows, weak near-bottom flow in the main channel and strong upward flow are observed; and downslope flow is generally
observed for each flow condition, and the combination of SO flow with a short setback abutment causes the strongest downslope flow.

5. The general spatial distributions of normalized turbulence intensities and normalized Reynolds shear stress are the same for experiments of the same abutment length, regardless of the flow rates and the flow conditions, as shown in Figures 5.12 to 5.15. For each abutment length, pressure flows have higher magnitudes of normalized turbulence intensities and normalized Reynolds shear stress in all three directions; the flow relief of OT flows is found to have insignificant effect on the near bottom turbulence. Abutment length obviously affects the spatial distribution and the magnitude of flow and turbulence intensities: short setback abutments have obvious peaks or troughs in the spatial distributions, whereas long setback abutments have relatively flat spatial distributions. Results show that at C.S.4, the normalized turbulence intensity is negatively correlated with the normalized streamwise flow intensity: the high turbulence intensity zone is accompanied by low streamwise flow intensity zone, and vice versa, as shown in Figures 5.17 and 5.18.

6. The distribution of normalized bed shear stress (see Figure 5.19) suggests that a launching apron, following the guidelines of Lagasse et al. 2006, may provide sufficient protection for the FS flows, but insufficient protection for the pressure flows, particularly for a combination of short setback abutments and pressure flows. Results imply that vertical contraction significantly affects the vulnerability and the location of the erodible regions for a long setback abutment.

7. Results suggest that the maximum scour depth around the abutment is related to both the maximum value of the normalized turbulence intensity \( \left( \frac{k_{\text{max}}}{u^*} \right) \) and the width averaged value of the normalized turbulence intensity on the floodplain \( \left( \frac{\langle k \rangle}{u^*} \right) \). These two normalized parameters may be used to evaluate the contribution of macro-
turbulence to final scour (It should be noted that, prior to scour, flow parameters on the floodplain are more representative of turbulence conditions than those in the main channel, and hence flow parameters on the floodplain are discussed in Chapter 5).

8.4 Clear-water scour study

The conclusions from the clear-water scour study are:

1. Two types of main channel bank behaviour occur: for the short setback abutment and the combination of long setback abutment and relatively large flow intensity, the “retreating” behaviour dominates (see Figures 6.1a and 6.1b); for the combination of long setback abutment and relatively small flow intensity, the main channel bank slope decreases (see Figures 6.1c and 6.1d).

2. The influence of abutment shapes becomes secondary with the presence of aprons around the abutment toes. For the same flow conditions, spill-through abutments and wing-wall abutments have similar scour developments and less than 10% scour depth differences (Sections 6.3.1 and 6.3.2).

3. For FS flows, flow pathlines vary from “deflected towards the main channel” in the early stage of scour to “slightly curved back towards the floodplain” in the final stage of scour. At the equilibrium state, the scour hole developed into a curved form, with the maximum scour depth on the downstream side of the abutment (see Figures 6.2 and 6.3).

4. Scour occurs concurrently and interactively on the floodplain and in the main channel. For both long and short setback abutments, two scour holes occur in the developing phase of scour; for some experiments, these two scour holes merge into one at the equilibrium state (see Figures 6.4 - 6.9, 6.13 - 6.16 and 6.17 - 6.20). For both long and
short setback abutments, the time needed for equilibrium and the locations of the
scour holes are both dependent on flow intensities.

5. A proportional relationship exists between the normalized equilibrium time, $t^*$, and
the cube of the normalized flow intensity, $(V_1/V_{c1})^3$, and this relationship is consistent
with the finding by Coleman et al. (2003). In terms of the temporal development of
scour, the effect of flow intensity is secondary, and the effect of flow depth is
significant for the investigated conditions. Also, results show that pressure flows have
more rapid scour development than FS flows (see Figures 6.23 and 6.24).

6. For all the scour experiments in this project, the slope ($\beta$) adjacent to the abutment at
the bridge section is approximately H: V=2:1. Combining all slope results in the scour
hole and at the bridge section suggests that 27° is the threshold value of $\beta$ for the
sediment used in this project. In addition, a simplified geometric relationship is
proposed between the maximum scour depth and the location of the maximum scour
at the bridge section (see Equation 6.4).

7. By adapting the long contraction theory of Laursen (1963), for both long and short
setback abutments, approximate linear relationships are found between $y_{\text{max}}/y$ and
$(q_z/q_1)^{6/7} (V_1/V_{c1})^{6/7}$, as shown in Figures 6.30b and 6.32; imprecise relationships are
found between $y_{\text{max}}/y_C$ and $(q_z/q_1)(V_1/V_{c1})$, as shown in Figures 6.31c and 6.33.

8. Clear-water scour results are compared with formulas by Melville (1995, 1997),
Results show that the Melville (1997) formula predicts the scour depth for long
setback abutments (Figure 6.34), and the Melville (1995) formula envelops the scour
depth data for long setback abutments (Figure 6.35). It is noteworthy that the Melville
et al. (2006) formula is found to be suitable for scour depth estimation for short
setback abutment scour in both clear-water and live-bed conditions (Figures 4.52 and 6.36) in this project. The Ettema et al. (2010) formula underestimates the amplification factors for pressure flows (for both clear-water and live-bed experiments), mainly because the amplification factor also accounts for vertical contraction in this project (Figure 6.37). Results show that the majority of the scour results from this clear-water study agree well with those reported by Hong et al. (2015) (Figures 6.38 and 6.39). The scour depths recorded for the long setback abutment experiments (Figure 6.38) are less than predictions from the formula by Sturm (2006), mainly because apron-protected abutments are used in this study, but unprotected abutments were used by Sturm (2006).

8.5 Long contraction scour study

The conclusions from the long contraction scour study are:

8.5.1 Long contraction scour in a rectangular channel

1. At the initial stage of the scouring process, the following is observed: the velocity field is only slightly affected by contraction length; regardless of contraction length, velocity components are similar in the proximity of the abutment; there is a change of velocity direction beside the abutment; and velocity in the transverse direction is weak along and after the contracted section, indicating a low level of lateral water flux (see Figures 7.2-7.5).

2. At the equilibrium scour state, the following is observed: for the long contraction experiment there is a large scale vortex (with a downstream-pointing horizontal axis) near the middle of the contracted reach; surface velocity is relatively uniformly distributed because of discharge redistribution after erosion; and the velocity field
around the scour hole is similar for the long and short contraction experiments (see Figures 7.6 and 7.7). The results suggest that abutment scour is insensitive to contraction length and the values of the proposed correction factor $K_\nu$ (accounting for the effect of contraction length) are tentatively suggested to be 1.1 for $L_c/B_c \leq 0.04$, and 1.0 otherwise.

### 8.5.2 Long contraction scour in a compound channel

1. For unprotected vertical-wall abutments, the temporal development of scour, the location of the scour hole, and the equilibrium scour depth are insensitive to contraction length (see Figures 7.11 and 7.14).

2. For apron-protected vertical-wall abutments, scour features vary with contraction length and the corresponding streamwise extent of the apron: the equilibrium scour depth decreases with increase of contraction length (see Figures 7.13 and 7.15); longitudinally, the equilibrium scour holes are located further downstream of C.S.3 for short contraction abutments than for long contraction abutments (see Figure 7.15); transversally, the equilibrium scour holes are located on the floodplain for short contraction abutments, whereas the equilibrium scour holes are located on the main channel bank for long contraction abutments (see Figure 7.15); relationships between the normalized scour depth and the normalized contraction length are obtained for the investigated conditions (see Figure 7.16 and Equations 7.1-7.3); and the amplification factor, $r_T$, decreases with increase in contraction length (see Figure 7.20). For apron-protected abutments in a compound channel, the correction factor, $K_\nu$, is tentatively suggested to be 1.3 for $L_c/B_f \leq 0.125$. 
3. Flow features for the plane-bed condition are found to correlate with the corresponding scour results (see Figures 7.22-7.24, and 7.28). For the six-lane bridge abutment model (long contraction), the most turbulent zone is located beside the abutment side face. For the one-lane bridge abutment model (short contraction), the most turbulent zone is located diagonally downstream of the abutment (see Figures 7.22-7.24). The extent of the apron (as in design guidelines by Lagasse et al. 2006) is found to be inadequate for the short contraction abutment, but is adequate for the long contraction abutment (see Figures 22-24). In addition, observable reverse direction flows occur within the contraction length, beside the abutment side face (see Figures 7.21 and 7.26).

8.6 Recommendations

Further research is recommended to improve the robustness of the research results in this project. The recommendations for future studies are:

1. Varying levels of submergence of the bridge deck should be extensively investigated under close-to-reality experimental conditions. It would be helpful to study the scour patterns for a wide range of the ratio of overtopping discharge to total discharge.

2. Wide ranges of flow intensity for the live-bed scour regime should be extensively investigated. Particularly, it would be helpful to quantify the characteristics of bed forms at the approach section and the bridge section.

3. In this project, the floodplain material and the main channel material have the same erodibility. A more challenging situation would be the use of cohesive material in the channel, or the use of cohesive material on the floodplain and non-cohesive material in the main channel, the latter being more representative of prototype conditions.
4. Experiments with different extents and thicknesses of rock riprap apron should be systematically carried out for pressure flow conditions. The efficiency of other types of scour countermeasures, for example, cable-tied blocks apron, should be investigated.

5. In this project, uniform rock riprap was used in the apron. The performance of non-uniform rock riprap is worthy of investigation.

6. Exclusively investigating the scale effect on abutment scour could be helpful for understanding the scour differences between submerged orifice and overtopping flows.

7. Flow patterns at the equilibrium state should be investigated for abutment scour under pressure flows and free surface flows.

8. Flow patterns at the equilibrium state should be investigated for long contraction scour with apron-protected abutments.

9. Long contraction scour with apron-protected abutments should be investigated for varying abutment shapes, channel geometries, flow depths and flow distributions. In particular, live-bed scour regime is worthy of investigation for long contraction scour.
REFERENCES


364


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## APPENDIX

### Table A1 Flow parameters for the live-bed study

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**Note:** N/A denotes not available; all the flow measurements were conducted at the initial state of scour
Table A2 Other flow and scour results for the live-bed study

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**Note:** N/A denotes not available; all the flow measurements were conducted at the initial state of scour.
Table A3 Flow parameters in the CW 2.4-m wide channel

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<td>150.4</td>
<td>112.6</td>
</tr>
<tr>
<td>CW_SSA_OT_ST_4</td>
<td>182.5</td>
<td>190.5</td>
<td>183.0</td>
<td>21.1</td>
<td>24.3</td>
<td>66.8</td>
<td>94.3</td>
<td>69.6</td>
</tr>
<tr>
<td>CW_SSA_FS_ST_3</td>
<td>81.1</td>
<td>89.1</td>
<td>88.6</td>
<td>19.0</td>
<td>28.3</td>
<td>45.1</td>
<td>65.4</td>
<td>65.4</td>
</tr>
<tr>
<td>CW_SSA_OT_ST_5</td>
<td>128.5</td>
<td>141.5</td>
<td>142.0</td>
<td>21.4</td>
<td>26.3</td>
<td>56.5</td>
<td>89.7</td>
<td>86.9</td>
</tr>
<tr>
<td>CW_SSA_SO_WW_1</td>
<td>135.0</td>
<td>137.0</td>
<td>137.0</td>
<td>21.8</td>
<td>28.0</td>
<td>58.7</td>
<td>87.6</td>
<td>84.8</td>
</tr>
<tr>
<td>CW_SSA_OT_WW_1</td>
<td>82.3</td>
<td>85.3</td>
<td>81.4</td>
<td>18.4</td>
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<td>46.3</td>
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<td>65.1</td>
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<td>CW_SSA_OT_WW_2</td>
<td>168.0</td>
<td>170.0</td>
<td>168.8</td>
<td>18.0</td>
<td>19.7</td>
<td>50.2</td>
<td>74.8</td>
<td>61.0</td>
</tr>
<tr>
<td>CW_SSA_OT_WW_3</td>
<td>185.1</td>
<td>191.1</td>
<td>184.1</td>
<td>30.3</td>
<td>30.4</td>
<td>84.5</td>
<td>120.2</td>
<td>84.6</td>
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Note: “Equi” denotes equilibrium state; “initial” denotes initial state
Table A4 Other flow and scour results in the CW 2.4-m wide channel

<table>
<thead>
<tr>
<th>Experiment No.</th>
<th>Initial $Q_{OT}/Q$</th>
<th>Equi $Q_{OT}/Q$</th>
<th>Measured $Q_1$</th>
<th>Measured $Q_2$</th>
<th>Equi $d_{s,FP}$</th>
<th>t</th>
</tr>
</thead>
<tbody>
<tr>
<td>CW_SSA_FS_ST_1</td>
<td>0.0%</td>
<td>0.0%</td>
<td>54.3</td>
<td>N/A</td>
<td>166.0</td>
<td>102</td>
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<tr>
<td>CW_SSA_SO_ST_1</td>
<td>0.0%</td>
<td>0.0%</td>
<td>80.0</td>
<td>N/A</td>
<td>292.0</td>
<td>112</td>
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<tr>
<td>CW_SSA_OT_ST_1</td>
<td>36.0%</td>
<td>23.4%</td>
<td>119.7</td>
<td>N/A</td>
<td>319.0</td>
<td>240</td>
</tr>
<tr>
<td>CW_SSA_FS_ST_2</td>
<td>0.0%</td>
<td>0.0%</td>
<td>74.7</td>
<td>N/A</td>
<td>256.0</td>
<td>172</td>
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<tr>
<td>CW_SSA_SO_ST_2</td>
<td>0.0%</td>
<td>0.0%</td>
<td>122.0</td>
<td>N/A</td>
<td>360.0</td>
<td>62</td>
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<tr>
<td>CW_SSA_OT_ST_2</td>
<td>25.2%</td>
<td>15.0%</td>
<td>150.4</td>
<td>N/A</td>
<td>422.0</td>
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<td>CW_SSA_OT_ST_3</td>
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<td>27.2%</td>
<td>189.0</td>
<td>N/A</td>
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<td>CW_SSA_OT_ST_4</td>
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<td>28.4%</td>
<td>119.6</td>
<td>N/A</td>
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<tr>
<td>CW_SSA_FS_ST_3</td>
<td>0.0%</td>
<td>0.0%</td>
<td>65.4</td>
<td>N/A</td>
<td>255.0</td>
<td>240</td>
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<td>CW_SSA_OT_ST_5</td>
<td>6.2%</td>
<td>N/A</td>
<td>92.6</td>
<td>N/A</td>
<td>394.7</td>
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<tr>
<td>CW_SSA_OT_WW_1</td>
<td>6.2%</td>
<td>N/A</td>
<td>90.4</td>
<td>N/A</td>
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<tr>
<td>CW_SSA_OT_WW_2</td>
<td>31.2%</td>
<td>N/A</td>
<td>88.6</td>
<td>N/A</td>
<td>173.7</td>
<td>120</td>
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<td>CW_SSA_OT_WW_3</td>
<td>45.7%</td>
<td>N/A</td>
<td>155.7</td>
<td>N/A</td>
<td>336.1</td>
<td>144</td>
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<table>
<thead>
<tr>
<th>Experiment No.</th>
<th>Initial $Q_{OT}/Q$</th>
<th>Equi $Q_{OT}/Q$</th>
<th>Measured $Q_1$</th>
<th>Measured $Q_2$</th>
<th>Equi $d_{s,FP}$</th>
<th>t</th>
</tr>
</thead>
<tbody>
<tr>
<td>CW_LSA_FS_ST_1</td>
<td>0.0%</td>
<td>N/A</td>
<td>58.4</td>
<td>59.8</td>
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<td>160</td>
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<tr>
<td>CW_LSA_OT_ST_1</td>
<td>6.2%</td>
<td>N/A</td>
<td>88.6</td>
<td>89.7</td>
<td>181.2</td>
<td>120</td>
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<tr>
<td>CW_LSA_OT_ST_2</td>
<td>31.2%</td>
<td>N/A</td>
<td>89.2</td>
<td>87.7</td>
<td>101.1</td>
<td>120</td>
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<tr>
<td>CW_LSA_FS_ST_2</td>
<td>0.0%</td>
<td>N/A</td>
<td>88.6</td>
<td>89.3</td>
<td>160.9</td>
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<tr>
<td>CW_LSA_OT_ST_3</td>
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<td>139.7</td>
<td>138.5</td>
<td>311.7</td>
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<tr>
<td>CW_LSA_OT_ST_4</td>
<td>45.7%</td>
<td>N/A</td>
<td>158.3</td>
<td>151.0</td>
<td>266.0</td>
<td>144</td>
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<tr>
<td>CW_LSA_SO_ST_1</td>
<td>0.0%</td>
<td>N/A</td>
<td>84.8</td>
<td>86.9</td>
<td>121.9</td>
<td>120</td>
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<tr>
<td>CW_LSA_SO_WW_1</td>
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<td>N/A</td>
<td>84.8</td>
<td>86.9</td>
<td>117.4</td>
<td>120</td>
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<tr>
<td>CW_LSA_OT_WW_1</td>
<td>0.0%</td>
<td>N/A</td>
<td>89.2</td>
<td>87.7</td>
<td>88.7</td>
<td>120</td>
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<tr>
<td>CW_LSA_OT_WW_2</td>
<td>22.8%</td>
<td>N/A</td>
<td>139.7</td>
<td>138.5</td>
<td>256.8</td>
<td>144</td>
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<tr>
<td>CW_LSA_OT_WW_3</td>
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<td>N/A</td>
<td>158.3</td>
<td>151.0</td>
<td>246.8</td>
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</table>
a4. LB_SSA_FS_Q

a5. LB_SSA_SO_Q

a6. LB_SSA_OT_Q
b1. LB_MSA_FS_q

b2. LB_MSA_SO_q

b3. LB_MSA_OT_q
c1. LB_LSA_FS_q

c2. LB_LSA_SO_q

c3. LB_LSA_OT_q
c4. LB_LSA_FS_Q

c5. LB_LSA_SO_Q

c6. LB_LSA_OT_Q
d1. LB NA SO_q

d2. LB NA OT_q

b3. LB NA SO_Q
Figure A1 Bed bathymetry for each experiment in the LB 1.54-m wide compound channel; 

- $\mathbf{a1 – a6.}$ with short setback abutments, $L_a/B_f=0.8$; 
- $\mathbf{b1 – b6.}$ with medium setback abutments, $L_a/B_f=0.65$; 
- $\mathbf{c1 – c6.}$ with long setback abutments, $L_a/B_f=0.65$ and 
- $\mathbf{d1 – d4.}$ with no abutments
Figure A2 The temporal development of scour for CW_LSA_SO_WW_1
Figure A3 The temporal development of scour for CW_LSA_OT_WW_1
Figure A4 The temporal development of scour for CW_LSA_OT_WW_2
Figure A5 The temporal development of scour for CW_LSA_OT_WW_3
a. Values of $V_x$ (near-bottom measurements) for two sample volumes

b. Values of $k$ (near-bottom measurements) for two sample volumes

Figure A6 Sensitivity analysis for sample volumes; measurements were made at $Y = 510$ or $560$-mm, 100 to 500-mm downstream of C.S.4, experiment LB_SSA_SO_Q
a. Values of $V_x$ (near-bottom measurements) for three velocity ranges

b. Values of $k$ (near-bottom measurements) for three velocity ranges

**Figure A7** Sensitively analysis for velocity ranges; measurements were made at $Y = 10$ to 560-mm, 100 to 500-mm downstream of C.S.4, experiment LB_SSA_SO_Q
Figure A8 Sensitively analysis for recording durations; measurements were made at $Y = 510$ to 660-mm, 0 to 300-mm downstream of C.S.4, experiment LB_SSA_SO_Q