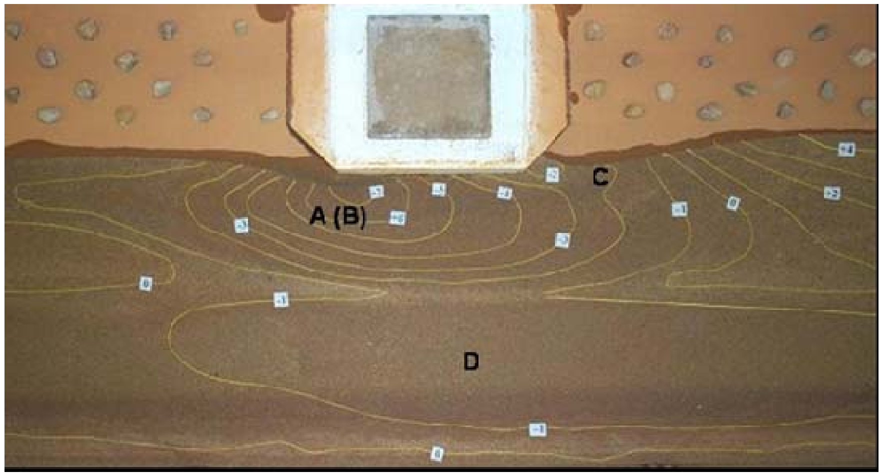
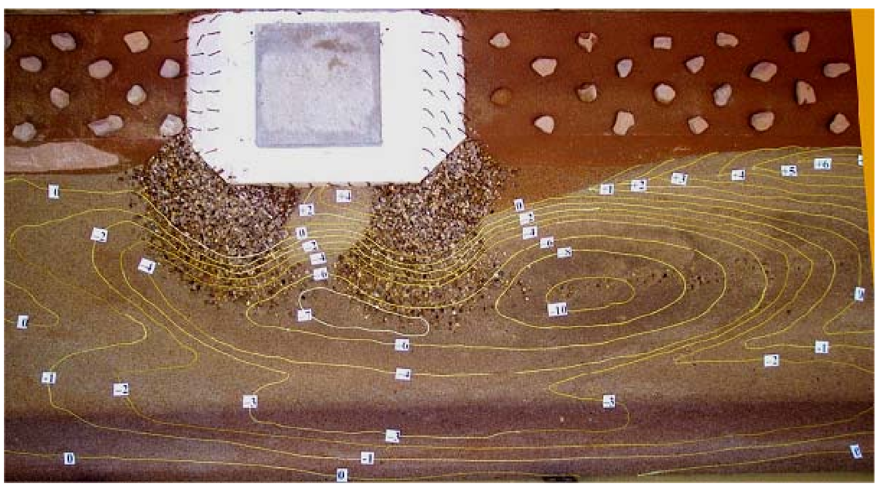
COUNTERMEASURES AGAINST SCOUR AT ABUTMENTS





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ABSTRACT

Scour at bridge abutments can cause damage or failure of bridges and result in excessive repairs, loss of accessibility, or even death. To mitigate abutment scour, both clear-water and live-bed laboratory experiments in a compound channel were performed using parallel walls and spur dikes as countermeasures, respectively. In addition, collars were tested under clear-water conditions only. It was found that both parallel walls and spur dikes can efficiently protect the abutment from scour provided that they are properly designed. Design guidelines were discussed for both countermeasures. Preliminary clear-water tests of collars as countermeasures against scour showed some promising results although further tests are needed to determine their efficiency and design parameters.

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A_e	flow area of an approach cross section obstructed by a roadway embankment
B_1	width of flume
B_2	constricted channel width
$\bar{b}$	dimensionless abutment length
c	cohesiveness
C_r, C_o	constant
C_D	drag coefficient
d_s	clear water scour depth measured from the bed surface
dc	clear water scour depth at parallel wall
dabut,avg	time averaged scour at abutment for live-bed scour
dmax,abut,inst	maximum instantaneous scour depth at abutment;
dcm,avg	time averaged scour depth at parallel wall
dmax,cm,inst	maximum instantaneous scour depth at parallel wall
d_{spl}	scour depth of the last dike far downstream from the first dike
d_{spn}	scour depth at the n^{th} dike
$d_{max,spn,avg}$	time averaged scour induced by n^{th} spur dike (first being at the most upstream)
$d_{max,spn,inst}$	maximum instant scour induced by n^{th} spur dike
$d_{max.up.abut}$	maximum scour depth at upstream corner of abutment
$d_{max.dn.abut}$	maximum scour depth at a short distance downstream of the downstream corner of the abutment
$d_{max.ch}$	maximum scour depth in the channel away from the abutment
$d_{max.col}$	maximum scour depth at collar

d_{50} median particle size

$d_{84.1}$ particle size for which 84.1 percent of the sediment mixture is finer

$d_{15.9}$ particle size for which 15.9 percent is of the sediment mixture finer

$\overline{d_s}$ dimensionless maximum scour depth

D distance or diameter of cylinder pier

D_r rock size for guide banks

D_s distance between two successive spur dikes or between spur dike and abutment

f Lacey silt factor

F_n Froude number

F_f approach flow Froude number on the floodplain

F_{fc} critical Froude number on the floodplain

F_{bo} Blench's zero bed factor which is a function of grain size

g gravitational constant

h_1 main channel bank height

k function of approach conditions

$\kappa - \varepsilon$ viscous and turbulent

K function of C_D which varies between 2.5 and 5.0

k_s Nikuradse sand roughness coefficient

K_G channel geometry factor

K_I flow intensity factor

K_y flow depth factor

K_d sediment size factor

K_σ	nonuniformity factor
K_s	shape factor
K_s^*	adjusted shape factor
k_θ	alignment factor
K_θ^*	adjusted alignment factor
L	protrusion length of abutment (spur dike)
L_p	protrusion length of rock wall bottom past abutment;
L_s	length of solid parallel wall or protrusion length of spur dike;
L_w	parallel wall width;
L_f	floodplain length
L'	A_e/y
l	Prandtl mixing length
M	discharge contraction ratio
n	function of C_D which varies between 0.65 and 0.90,
n_m	Manning's roughness n in main channel
n_f	Manning's roughness n on floodplain
Q	total stream discharge
Q_f	lateral or floodplain flow on one side measured at a certain section
Q_{100}	discharge in 100 feet of stream adjacent to abutment
Q_m	discharge in main channel
q	flow intensity
q_2	discharge per unit width at constricted section
q_{f1}	the unit discharge at the beginning of scour in the approaching floodplain
q_{f0c}	the critical unit discharge in the floodplain for the undisturbed floodplain
R	radius of curvature of the axis of the principal vortex
r	ratio of scour depth at abutment to that at long contraction used by Laursen
Re_*	particle Reynolds number
S_e	energy slope

So	Slope of channel
S_s	specific gravity
S_p	particle shape factor
S_b	side slope of rock walls
S_n	end slope of rock walls
t	time
T	temperature
U	mean approach velocity
U_c	critical velocity
U_a	mean approach flow velocity at the armor peak
u_*	shear velocity
u_{*c}	critical shear velocity
U_{ab}	the maximum resultant velocity at the upstream corner of the abutment face y_{ab}
	floodplain flow depth at the location of U_{ab} in the contracted section
Y_0	flow depth
y_s	depth of flow at abutment, i.e., $d_s + y_0$
y_f	mean flow depth on floodplain
y_{f0}	undisturbed average flow depth in the approach floodplain
y_m	flow depth in main channel
V_u	peak downflow velocity
V	transverse velocity
ν_t	kinematic viscosity due to turbulence
w	sediment fall velocity
W	effective bottom width of rock wall countermeasure
ϕ	angle of repose
ι	Prandtl mixing length
θ	angle of attack or abutment (spur dike) skewness
θ_1	Shields entrainment function
θ_c	critical Shields entrainment function
ρ	density of water

σ_g sediment gradation

τ total shear stress

τ_0 the approach shear stress

τ_c critical shear stress

τ_υ viscous shear stress

τ_t turbulent shear stress

μ dynamic viscosity

β constant for flow concentration

$\%_{max,abut, avg}$ percent reduction in time averaged scour at abutment

$\%_{max,abut,inst}$ percent reduction in maximum instantaneous scour at abutment

CHAPTER 1. INTRODUCTION

1.1 Introduction

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams. Different materials scour at different rates. Loose granular soils are rapidly eroded under water action while cohesive or cemented soils are more scour-resistant. However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand bed steams. Scour will reach its maximum depth in sand and gravel bed materials in hours; cohesive bed materials in days; glacial tills, poorly cemented sand stones and shales in months; hard dense and cemented sandstone or shales in years; and granites in centuries. Massive rock formations with few discontinuities can be highly resistant to scour and erosion during the lifetime of a typical bridge. Detailed discussion and equations for calculating all bridge scour components are presented in HEC-18 (Richardson and Davis, 1995).

Total scour at a highway crossing is comprised of the following three components: aggradation or degradation, contraction scour and local scour. Aggradation or degradation is long-term streambed elevation changes due to natural or human-induced causes within the reach of the river on which the bridge is located. Contraction scour involves the removal of material from the bed and banks across all or most of the width of the channel. This scour can result from a contraction of the flow by the approach embankments to the bridge encroaching onto the floodplain and/or into the main channel, from a change in downstream control of the water surface elevation, or from the location of the bridge in relation to a bend. In each case, scour is caused by an increase in transport of bed material in the bridge cross section. Local scour occurs around piers, abutments, spurs and embankments and is caused by the acceleration of the flow and the development of vortex systems induced by these obstructions to the flow.

Bridge failures due to total scour at bridge foundations (i.e., bridge abutments and piers) have prompted a heightened interest in scour prediction and scour countermeasures. Data showed that the problem of scouring at bridge abutments is quite significant.

Richardson et al. (1993) quoted a study produced in 1973 for the U.S. Federal Highway Administration that concluded of 383 bridge failures, 25% involved pier damage and 72% involved abutment damage. According to Melville (1992), of the 108 bridge failures surveyed in New Zealand during the period of 1960 –1984, 29 were attributed to abutment scour. Melville also mentioned that 70% of the expenditure on bridge failures in New Zealand was due to abutment scour.

There have been several studies on pier scour and pier scour countermeasures. Some of these are Ettema (1980), Johnson (1994), Jones (1989), Lagasse et al. (2001), Mueller et al. (1999), Richardson et al. (1975, 1999, 2001). For bridge pier countermeasures, the NCHRP 24-7 (1998) project final report named "Countermeasures to Protect Bridge Piers from Scour" has reviewed nearly all the literature in this aspect and also has given recommendations and design suggestions for a number of countermeasures. Also the Federal Highway Administration has developed several comprehensive technical manuals (HEC-18, HEC-20 and HEC-23) for dealing with the problem of bridge scour. Moreover, a field survey of pier countermeasures was carried out across the United States.

However, the scour at bridge abutments has received less attention, and countermeasures for abutment scour are greatly needed.

1.2 Scope of Study

After a review of the literature and a survey of practice by state department of transportation engineers by Barkdoll (unpublished data), some countermeasures emerged as worthy of further study. Selected countermeasures that reduce local scour at bridge abutments include armoring countermeasures, such as riprap and cable-tied blocks, river training countermeasures that prevent outflanking of the bridge opening, such as spur dikes and parallel walls, and some other innovative approaches including hinged slab/block systems, collars and vanes attached to the abutment.

The ultimate aim of this study of countermeasures against scour at bridge abutments is to determine design guidelines for those countermeasures that are efficient in preventing or reducing local scour at the abutments of small county bridges, and through which bridge abutments can be well protected. The prerequisite to such guidelines is obtaining normalizing parameters that collapse experimental data and adequately scale model study results to field situations. Countermeasures that will be investigated here include spur dikes, parallel walls, and collars. Since few countermeasures can totally eliminate scour for all of the complex situations that occur in natural rivers and for all of the various bridge configurations encountered, it becomes necessary that a goal of scour reduction be defined. For the purpose of this project, a primary goal is to identify countermeasures that reduce maximum local scour depth by 65 to 75 percent of the scour depth occurring at an unprotected abutment.

An extensive study is presented on the effect of countermeasure length, alignment, relative position and material on protection efficiencies. Previous literature, which will be reviewed in Chapter 2 and 3, has revealed the primary parameters of spur dikes are protrusion length, number of spur dikes, distance between spur dikes, and angle to flow. The primary parameters for parallel walls are length, plan shape, and construction material. And the primary parameters for collars that are attached around piers are streamwise length, protrusion width, and vertical position of collar relative to the riverbed. These parameters of different countermeasures will be investigated experimentally under clear-water and live-bed scour conditions.

CHAPTER 2. A LITERATURE REVIEW ON BRIDGE ABUTMENT SCOUR

2.1 Introduction

The problem of scour around bridge abutments has been identified as one with potentially catastrophic results. Excessive scour can cause abutment damage, which will result in the bridge failure and potential loss of life due to bridge collapse. Before considering any countermeasures, it is important to get as much information about the scour mechanism and parameters that affect local scour at bridge abutments as possible.

2.2 Bridge Abutment Scour Mechanism

The general scour mechanism at piers is well understood after several comprehensive studies including Melville (1975) and Ettema (1980), in which the principal downflow and the horse shoe vortices and the wake vortices induced by the presence of the pier are responsible for the scour around the structure. Recent studies of abutment scour by Wong (1982), Tey (1984), Kwan (1984, 1988), Kandasamy (1985, 1989), Dongol (1994) and many others have shown that the scour mechanism at abutments is very similar to the scour mechanism at piers. The downflow and the principal vortex at the upstream corner of the abutment, together with the secondary vortices and wake vortices at the middle part and the downstream corner of the abutment, cause complex interactions between the fluids and the bed material and are mainly responsible for the scour at abutments. Observations of flow patterns around abutments derived from flow visualization techniques using dye injection, dye crystals strategically placed on the sand bed, paper floats, smoke tunnel experiments by various researchers including Liu (1961), and Gill (1970) are summarized in Figure 2-1.

In addition to the vortex systems, other mechanisms may exist to cause local scour. Hagerty and Parola (1992) reported that seepage effects could be very important to interact with turbulent vortices to aggravate local scour. The fluctuating pressure differences induced by the flow separation at abutments cause seepage into and out of the abutment foundation; seepage forces cause ejection of sediment particles from the bed where seepage emerges beside the abutment.

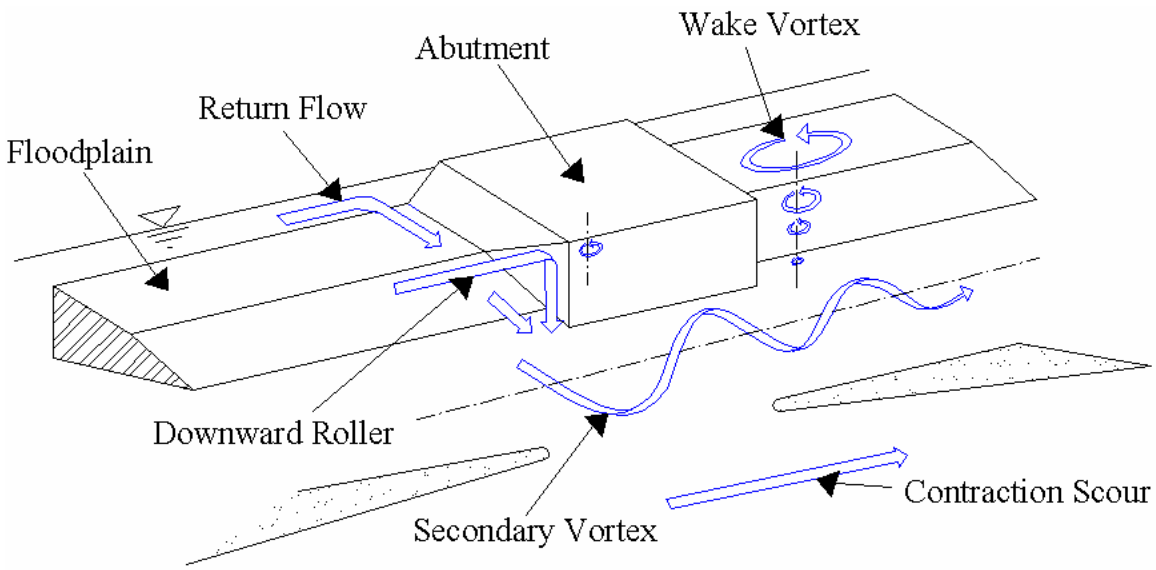


Figure 2-1: Flow patterns around a wing-wall abutment.

Molinas et al. (1998) studied experimentally the shear stress distribution around vertical wall abutments. For Froude numbers ranging from 0.30 to 0.90 and for protrusion ratios of 0.1, 0.2 and 0.3 it was found that the highest values of shear stresses occur at the upstream abutment corner. In the experimental study, shear stresses around vertical wall abutments were amplified up to a factor of 10 depending upon flow conditions and abutment protrusion ratios. It was also found that shear stress amplification due to local effects at the nose region of a vertical wall is a function of the opening ratio and turning angle. Shear stress amplification due to channel restrictions is a function of opening ratio, approach Froude number, and protrusion length. Both formulas for estimating shear stress amplification due to local effects at the nose region of vertical wall abutments and for estimating shear stress amplification due to channel restriction are proposed. The sum of the two shear stress amplifications equals the total nose shear stress amplification.

Ahmed and Rajaratnam (2000) investigated the flow around a 45^0 wing-wall bridge abutment. It was found that the approach flow turns into a complex 3D skewed flow in the upstream and surrounding regions of the abutment. The bed shear stress was found to increase substantially near the abutment, reaching a peak value of $\tau/\tau_0 \approx 3.63$ at the abutment nose, with τ and τ_0 being the shear stress and the approach shear stress at the bed, respectively. They also found that the skewing of the flow around the bridge abutment is greater than flow around bridge piers.

2.3 Parameters Influencing Local Scour at Abutments

The parameters affecting the local scour at abutments are listed in Table 2-1. Using these variables, the scour depth at the abutment may be represented functionally as:

$d_s = f$ [Channel (B_1, S_0, K_g), abutment (L, θ, K_s), flow (y_0, U, S_e, g), bed material ($d_{50}, S_s, \sigma_g, w, S_p, \phi, c, \theta_c, \mathrm{Re}_*$), fluid ($\rho, \mu$), temperature (T), time (t)]

in which f represents 'a function of '. Some of these variables can be neglected under certain circumstances. By doing so and non-dimensionalizing the remaining, the following variables are believed to be most important in influencing the scour at abutments: time t, velocity U/U_c, flow depth y_0/L, sediment size L/d_{50}, sediment gradation σ_g, abutment shape K_s, abutment length L/y_0, abutment skewness θ and channel geometry K_g. Knowledge of how these variables affect the abutment scour is required in designing scour countermeasures. Discussion of these variables is followed next.

2.3.1 Time Evolution

In local scour studies, maximum scour depth is very important because only by knowing the maximum scour depth are engineers able to know how much the bridge foundations need to be protected from scour. Therefore, to know the temporal evolution of the scour and the time needed for the scour to reach the equilibrium state is very helpful in obtaining the maximum scour depth. Also, by obtaining the temporal evolution of scour process, shorter scour experiments can be run without failing to obtain the maximum scour depth and time and funds can be saved. Also scour depth at a certain moment of a flood hydrograph could be predicted with the help of the scour time evolution curve.

It is established that the local scour depth increases progressively with time and reaches equilibrium. Three phases have been identified by various researchers including Gill (1972), Wong (1982), Tey (1984), Kwan (1984), Ettema (1980), Kandasamy (1985) and Dongol (1994) as the initial, the principal, and the equilibrium phases, which can be shown by the different slopes on the plots of the scour depth against the logarithm of time.

A number of approaches concerning the quantitative description of the time evolution of scour depth around abutments, spur dikes and cylindrical piers exist at present. Most of them apply to the principal phase and consist of formulas that include one or more coefficients established experimentally. Among those formulas are Ahmad (1953), Liu (1961), Cunha (1975), Laursen (1963), Ettema (1980), Franzetti et al. (1982), Whitehouse (1997) and Oliveto and Hager (2002). Cunha (1975) concluded that the time to reach the equilibrium scour depth under clear-water conditions is much longer than under live-bed conditions.

Table 2-1: Parameters influencing local scour at abutments.

Variable name	Symbols	Attribution
Length of abutment	L	Abutment
Angle of attack	θ	
Shape of abutment	K_s	
Normal flow depth	y_0	Flow
Mean approach velocity	U	
Gravitational acceleration	g	
Energy slope of flow	S_e	
Width of channel	B_1	Channel
Slope of channel	S_0	
Geometry of channel	Kg	
Median size	d_{50}	Bed material
Specific gravity	S_s	
Standard deviation	σ_g	
Fall velocity	W	
Particle shape factor	S_p	
Angle of repose	ϕ	
Cohesiveness	C	
Dimensionless critical shear stress	θ_c	
Particle Reynolds number	Re_*	
Density	ρ	Fluid
Dynamic viscosity	μ	
Temperature	T	
Time	T	Time

2.3.2 The Effect of Flow Velocity

It is well known that when the shear stress the flow exerts on the particles is greater than the critical shear stress that the particles can resist, the particles begin to move and scour starts. For a fully developed turbulent flow like the downflow that causes the scour at the abutment corner and that occurs in natural channels, the total shear stress of the flow can be expressed as (Dongol 1994):

$$\tau = \tau_v + \tau_t = \rho \cdot \iota^2 \cdot (du/dy)^2 = \rho \cdot u_*^{\ 2} \qquad \text{(2-1)}$$

where ι = Prandtl mixing length; ρ = density of water; τ_t = turbulent shear stress; τ_v = viscous shear stress; τ = total shear stress; du/dy = vertical gradient of velocity; u_* = shear velocity.

It can be seen from the equation above that the shear stress of the flow, the shear velocity and the velocity gradient of the flow are related to each other. Generally, the shear velocity ratio u_*/u_{*c} is a good measure of the strength of the downflow and therefore the scouring potential of the vortex structures at abutments. It is often used in the relations of scour prediction equations. But because of the difficulties of measuring the shear velocity under live-bed conditions, the velocity ratio U/U_C has been used in preference to the shear velocity ratio as a measure of the flow intensity.

From data of Dongol (1994) it was found that the scour depth increases almost linearly with flow velocity or shear velocity until the flow velocity reaches the threshold condition. After that, for live-bed conditions, the scour depth varies depending on the bed regime on the approach flow bed. For pier studies with non-rippling sediment ($d_{50} \geq 0.7$ mm), the equilibrium scour depth at first decreases with flow velocity beyond the threshold condition and then reaches a minimum value. Thereafter the scour increases again towards a second maximum with further increase of the flow velocity. Fluctuations of scour depth occur due to an imbalance of the sediment transport into and out of the scour hole. The scour depth at the second maximum is less than that at the threshold condition. But for rippling sediment ($d_{50} \leq 0.7$ mm), the scour depth at the threshold condition is less than that at the second maximum because of the ripple formation at velocities below the threshold that causes a steady supply of sediment into the scour hole, which induces less scour. Chiew (1984) indicated this trend for rippling and non-rippling sediment scour using his pier scour data and he also suggested that scour depths reduce for very high velocities beyond the transition flat bed condition where velocities are supercritical.

Experiments conducted at bridge abutments showed that the influence of flow velocity is very similar to that for piers according to Gill (1972), Kandasamy (1989) and Dongol (1994). All data indicated an increasing trend in the relationship between scour depth and flow velocity.

Data of Dongol showed the relationship between scour depths with respect to velocity ratio using many different abutment models at different flow depths with different sediment sizes over a wide velocity range. He concluded that the maximum scour depth is observed at the threshold condition and decreases thereafter with further increase in the velocity, reaching minimum at higher velocities. Then after that, the scour depth increases again attaining, at the transition flat bed condition, a value equivalent to that at the threshold condition. Thereafter the scour depth decreases with further increase in the velocity with the formation of anti-dunes.

Melville and Sutherland (1988) proposed a flow intensity factor K_I to account for the effect of velocity on the abutment scour. Based on a series of pier and abutment scour data available, Melville (1997) has been able to define the flow intensity factor as:

$$K_I = \frac{U - (U_a - U_c)}{U_c} \quad \text{when} \quad \frac{U - (U_a - U_c)}{U_c} \prec 1 \qquad (2\text{-}2)$$

$$K_I = 1 \quad \text{when} \quad \frac{U - (U_a - U_c)}{U_c} \geq 1 \qquad (2\text{-}3)$$

where U = mean approach flow velocity; U_a = mean approach flow velocity at the armor peak; U_c = mean approach velocity at threshold condition. K_I is formulated to include the sediment gradation effects as well.

2.3.3 <u>The Effect of Flow Depth</u>

Pier scour researchers found that for constant shear velocity ratio, u_* / u_{*c}, the relative scour depth, d_s / D, where D is the diameter of the cylindrical pier, increases at a decreasing rate with increasing normalized flow depth, Y_0 / D, towards a limiting value beyond which the effect of Y_0 / D is negligible. Ettema (1980) showed also that the limiting value of the Y_0 / D ratio for scour depth at piers depends upon sediment size as well. Abutment scour data by Wong (1982), Tey (1984), Kwan (1988), Kandasamy (1989), and Dongol (1994) also showed a similar trend, i.e., the scour depth at abutment increases with flow depth at a decreasing rate towards a limiting value beyond which there is no effect of flow depth. The limiting value of the ratio was found to be about 3 ~ 4. Data from Gill (1972) and Dongol (1994) showed that this limiting value increases as the sediment size decreases. This trend was observed both at clear-water conditions and live-bed scour conditions.

Melville (1992) recommended a flow depth factor K_y to account for the effects of flow depth on scour. This factor will be discussed later with the abutment length factor K_L.

2.3.4 <u>The Effect of Sediment Size</u>

It is important to distinguish between clear-water and live-bed scour when considering the effects of sediment size on scour. Under live-bed conditions, some early pier scour researchers argued that there is no significant effect of sediment size on local scour, but some others suggested that the scour depth decreases with an increase in the sediment size. For clear-water conditions, most studies have shown that sediment size has an effect on local scour.

The scour studies for piers by various researchers show that under clear-water conditions the scour depth increases with an increase in sediment size for finer sediments, while the reverse trend is true for coarser sediments. Dongol (1994) divided the sediment size relative to the abutment length into four groups on the basis of scour development for different values of L/d_{50} at the principal stage as follows: (1) $L/d_{50} > 100$: fine sediment; (2) $100 > L/d_{50} > 40$; intermediate sediment; (3) $40 > L/d_{50} > 10$: coarse sediment; (4) $10 > L/d_{50}$: very coarse sediment.

Ettema (1980) data for pier scour, together with abutment data obtained by Gill (1970), Wong (1982), Kwan (1987) and Kandasamy and Dongol are plotted by Kandasamy (1989). All data apply to threshold conditions and non-rippling sediments with $\sigma_g < 1.3$. The curves showed that the scour depth increases rapidly with sediment size to a peak at $L/d_{50} > 50$, for a constant pier diameter or abutment length, and then decreases. It needs to be noted that data by Gill and Wong were measured before equilibrium had been reached so that the scour depth may be smaller than otherwise it should be. But Dongol (1994) found that the relative equilibrium scour depth, d_s/L decreases as L/d_{50} decreases below 40. For large values of L/d_{50}, the effect of sediment size is insignificant.

Melville and Sutherland (1988) proposed a sediment size factor K_d to account for the effect of sediment size on abutment scour and defined it as:

$$K_d = 0.57\log\left(2.24\frac{L}{d_{50}}\right) \quad \text{for} \quad \frac{L}{d_{50}} \le 25 \qquad (2\text{-}4)$$

$$K_d = 1.0 \quad \text{for} \quad \frac{L}{d_{50}} \ge 25 \qquad (2\text{-}5)$$

2.3.5 <u>The Effect of Sediment Gradation</u>

River bed materials are generally nonuniform. A measure of the nonuniformity of the sediment is geometrical standard deviation σ_g. For the log-normally distributed sediment, σ_g is given by

$$\sigma_g = \sqrt{\frac{d_{84.1}}{d_{15.9}}} = \frac{d_{84.1}}{d_{50}} \tag{2-6}$$

The effect of sediment gradation on scour depth depends upon whether the scour occurs under clear-water or live-bed conditions. For sediment with the same d_{50} and under similar flow conditions, it is found that less scour is developed in nonuniform sediments than in uniform sediments. Scour depth is seen to decrease progressively with increasing σ_g (Ettema (1980), Wong (1982), Melville (1992) and Dongol (1994)).

Data by Ettema (1976) showing the effect of sediment gradation on scour depth at piers at the critical condition are plotted by Dongol (1994) together with data by Wong (1982) for wing-wall abutment. The diagram is plotted using K_σ versus sediment gradation, σ_g, where K_σ is a nonuniform factor defined as:

$$K_\sigma = \left(\frac{d_s}{L(D)}\right)_{\sigma_g} \bigg/ \left(\frac{d_s}{L(D)}\right)_{\sigma_g = 1} \tag{2-7}$$

where L(D) represents the abutment length L if it's abutment and the pier diameter D if it's a pier that is considered.

From that plot it was found that except for rippling sediment, the scour depths for both pier and abutment scour have peak values with $\sigma_g = 1$ and decrease with increasing σ_g. It is attributed to the armoring effects due to the formation of armour layers in the scour holes that protect the underlying finer fractions from erosion and inhibit the scour development. For the rippling sediment at $\sigma_g = 1$, due to formation of ripples and continuous transport of the sediments into the scour hole, a lesser scour depth is observed. At slightly high values of $\sigma_g = 1.5$, a weak armour layer forms stabilizing the bed and inhibiting ripple formation. As a consequence, the scour depth reaches its peak. With even greater values of $\sigma_g > 1.7$, the armoring becomes a prominent feature and a consequent reduction of scour depth is observed. At still higher values of $\sigma_g > 2$, rippling sediment behaves like nonrippling sediment.

The effect of sediment gradation on scour depth in live-bed conditions is more complex, due to the different mobility associated with the sediment (Bake, 1986).

Dongol (1994) studied the effects of sediment gradation on scour depth for different flow depths. It was found that there is an increase in the armour peak velocity, U_a and a decrease in the relative scour depth, d_s / L, with increasing σ_g. This is due to an increase in coarseness of the armor layer with σ_g, which protects the underlying sediment and inhibits the further development of scour for a wide range of flow conditions. Also the armoring on the bed inhibits erosion of the sediment, therefore restraining dune formation and reducing dune height. It is also seen that the effect of sediment gradation on scour depth is more prominent at lower flow velocities.

2.3.6 The Effect of Abutment Length

Abutment length is one of the most important parameters influencing the scour process and depth at an abutment. Various experimental data have shown that the scour depth increases with increasing abutment length. Various dimensionless parameters have been used to evaluate the effect of abutment length on scour depth including contraction ratio L / B_1, opening ratio $(B_1 - L) / B_1$, and ratio of abutment length to flow depth L / y_0, where B_1 is the channel width. Some researchers have used opening ratio and contraction ratio to evaluate the effect of abutment length on scour depth, for instance, Laursen (1962), Garde et al. (1961), Field (1971), Gill (1970), Zaghloul and McCorquodale (1975), Zaghloul (1983), and Rajaratnam and Nwachukwu (1983). Some other researchers, including Laursen (1962), Cunha (1975), Kwan (1984), Kandasamy (1989), Melville (1992), and Dongol (1994, have preferred using the L / y_0 ratio to represent the abutment length.

Kandasamy (1989) studied the scour at abutments under clear-water conditions comprehensively and based on his experimental data he presented a three dimensional graphical relationship to explain the interaction of abutment length, flow depth and scour depth, defining the functional relationship as $d_s = f(L, Y_0)$. He divided the surface defined by the relation $d_s = f(L, Y_0)$ into four different zones.

Zone 1: In this zone, which he called the scour situation as "scour at short abutments", he found that the scour depth is independent of flow depth. The scour depth increases with the increasing abutment length rapidly and almost linearly. A dead water zone ahead of the abutment is virtually nonexistent.

Zone 2 and 3: In these two zones, according to Kandasamy, scour occurs at medium and intermediate length abutments. The rate of scour depth increase with abutment or flow depth is less than that at a short abutment. A dead water zone develops as a consequence of the greater deflection of flow around the relatively longer abutment.

Zone 4: at very large values of L / y_0, the ratio reaches the limiting condition and maximum scour depth is observed. The scour situation is termed as "scour at a long abutment".

Melville (1992) plotted abutment scour data by Gill (1972), Wong (1982), Tey (1984), Kwan (1984, 1988), Kandasamy (1989) and Dongol (1990) as d_s / y versus L / y and d_s / L versus y / L and drew envelope curves to these data. Then he proposed a scour equation as:

$$d_s = 10y \quad \text{for} \quad \frac{L}{y} \geq 25$$

$$d_s = 2L \quad \text{for} \quad \frac{L}{y} \leq 1 \qquad \text{(2-8)}$$

$$d_s = 2\sqrt{yL} \quad \text{for} \quad 1 \leq \frac{L}{y} \leq 25$$

Dongol (1994) studied the effect of abutment length on scour. His data was plotted together with data by Kandasamy (1989) and Tey (1984) for wing wall abutments and Kwan (1984) for semi-circular end abutments. From his plot, it was found that when $L/y \geq 60$ the rate of increase of scour depth with abutment length is insignificant. And for $L/y \geq 100$, scour depth virtually remains constant.

Further data were plotted by Melville (1997) from scour data at wing-wall, vertical wall, and spill-through (with H:V slopes of 0.5:1, 1:1, 1.5:1, respectively) abutments, augmented by data from Dongol (1994) as d_s/y versus L/y and d_s/L versus y/L. These data were obtained under different flow intensity values of U/U_c and with uniform sediment. These data verified Melville's (1992) abutment scour formulation.

2.3.7 The Effect of Abutment Shape

The local scour magnitude at abutments differs depending on the shape of the abutment. The strength of downflow and secondary flow that is responsible for the scour can be greatly reduced at streamlined abutments than at blunt shaped abutments. Therefore, the scour at streamlined abutments is less than that at the blunt shaped abutments. The effect of abutment shape on the local scour at abutments has long been of interest to researchers.

Inglis' (1949) studies of spur dikes showed that the scour depth at a spur dike varied with the sharpness of curvature of the spur. Liu et al. (1961) found that the scour at a wing wall abutment is twice that at a spill-through abutment. Laursen and Toch (1956) found that the shape of the abutment affects the scour depth with a 15% reduction for streamlined abutments. Rajaratnam and Nwachukwu (1983) observed the shear stress to be 14% higher at a plate groyne compared to a circular groyne. Data from Dongol (1994) showed the maximum scour happened at the vertical wall abutment, and the smallest scour occurred at the most gentle sloped spill-through abutment.

To account for different shapes of abutment, Field (1971), Melville (1992) and some other researchers have proposed different coefficients and relationships for design purposes. The effect of shape is expressed using the shape factor K_s by Melville and the values of the shape factors are given in Table 2-2. The vertical wall abutment is used as a reference. For the spill-through abutment, the abutment length was taken as the length at mid-depth in the water. The shape factors given in Table 2-2 were derived by a procedure to produce the best collapse of data over a range of experimental conditions. These data included Gill (1972), Wong (1982), Tey (1984), Kwan (1984, 1988), Kandasamy (1989) and Dongol (1990) and were all obtained at the threshold condition with uniform sediments.

Table 2-2: Shape factors (Melville 1995)

Abutment shape	**Shape factor K_s**
Vertical plate or narrow vertical wall	1.0
Vertical wall abutment with semicircular end	0.75
45^0 wing wall	0.75
Spill-through (H:V):	
0.5:1	0.60
1:1	0.50
1.5:1	0.45

The effects of abutment shape are included by applying the shape factor given in Table 2-2. It is postulated that the importance of abutment shape should diminish as the abutment including the bridge approach embankment becomes longer, because the effect of shape is wrought only at the end of the abutment. An adjusted shape factor was recommended by Melville (1992) for different values of L/y, i.e.,

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

$$K_s^* = K_s + (1 - K_s)\left(0.1\frac{L}{y} - 1.5\right) \quad \text{for} \quad 10 < \frac{L}{y} < 25 \qquad (2\text{-}9)$$

$$K_s^* = 1 \quad \text{for} \quad \frac{L}{y} \geq 25$$

2.3.8 <u>The Effect of Abutment Skewness</u>

Ideally, bridges are constructed at a straight reach of the river channel and normal to the flow, giving the shortest span and eliminating skewness. However, skewness is likely, due to the existing road layout constraints and river channel geometry. Abutment skewness is defined as the inclination of the abutment to the mean flow direction and is denoted by θ as indicated in figure 2-2.

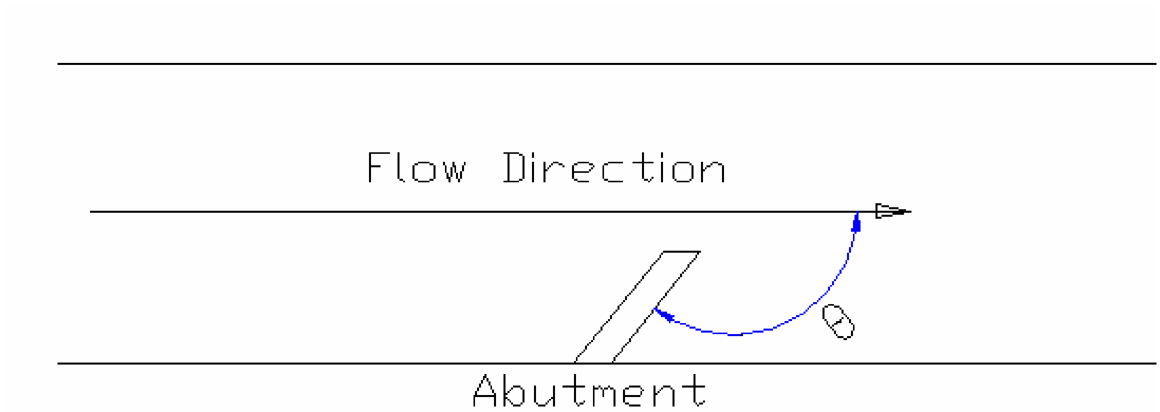


Figure 2-2: Abutment skewness angle definition.

The effect of abutment skewness on abutment scour has been studied by various researchers, among which are Ahmad (1953), Laursen (1958), Sastry (1962), Field (1971), Zaghloul (1983), Kwan (1984) and Kandasamy (1985). The results of these studies in terms of the alignment factor k_θ, defined as the ratio of the scour depth at a skewness other than 90^o to that at 90^o, apply to both live-bed and clear-water conditions and show that the scour depth increase with an increase in θ for $\theta \leq 90^o$. For $\theta > 90^o$, data by Kwan and Kandasamy show that the scour depth decreases with increasing skewness while data by other researchers show contradictory results. Dongol (1994) argued that the short running time of their experiments compared to Kwan and Kandasamy's casts doubt on the equilibrium scour depths. For design purpose, Melville (1992) proposed an abutment alignment factor K_θ and recommended an envelope curve drawn to these data. The effect of abutment alignment depends on the abutment length. It is postulated that the importance of skewness should diminish as the abutment becomes shorter. Melville recommended using an adjusted alignment K_θ^* factor for different limits of L/y, i.e.,

$$
\begin{aligned}
&K_\theta^* = K_\theta \quad \text{for} \quad \frac{L}{y} \geq 3 \\
&K_\theta^* = K_\theta + (1 - K_\theta)\left(1.5 - 0.5\frac{L}{y}\right) \quad \text{for} \quad 1 < \frac{L}{y} < 3 \qquad \text{(2-10)} \\
&K_\theta^* = 1 \quad \text{for} \quad \frac{L}{y} \leq 1
\end{aligned}
$$

2.3.9 The Effect of Approach Channel Geometry

The approach channel geometry can have a very significant influence on the local scour depth at bridge abutments, particularly for long abutments. Bridge abutments can be set back from the natural stream bank or can project out into the flow, and there can be varying amounts of over-bank flow that are intercepted by the approaches to the bridge and returned to the stream at the abutment. The scour at an abutment can be caused by the abutment projecting into the flow, by the bridge approaches intercepting overland flow and forcing it back into the channel at the abutments, or a combination of conditions, as discussed by Richardson et al. (Melville, 1992).

Melville (1995) has classified abutment scour as three basic cases. Case I applies to a bridge spanning a well-defined river channel with no floodplain. In Case II, the bridge approach spans the floodplain and protrudes into the main channel of a compound river channel, the abutment being sited in the main channel. In Case III, the bridge approach and abutment only span part of the floodplain. Case III can be further divided into two types. The first type is the case that the abutment is set back from the main channel bank; the second type is the case that the abutment terminates right on the edge of the main channel bank.

Melville (1992) proposed a factor K_G to include the approach channel geometrical effects. The K_G factor will depend on the relative size, shape, and roughness of the main channel and the floodplain channel, and on the abutment length in relation to the floodplain width.

Dongol (1994) studied the effect of cross-sectional shape of the approach channel on the scour depth at a wing wall abutment of fixed length under clear-water conditions in an idealized compound channel. Uniform sediment is used and the abutment is oriented perpendicular to the flow direction. His data are plotted as K_G versus y_m/y_f for each value of L_f/L, where y_f and L_f are the flow depth on the floodplain and the length of the floodplain, respectively. Here K_G, originally defined as:

$$K_G = \frac{\left(\frac{d_s}{L}\right)_{compound.channel}}{\left(\frac{d_s}{L}\right)_{rec\tan gular.channel}} \tag{2-11}$$

which was also theoretically derived by Dongol as:

$$K_G = \sqrt{1 - \frac{L_f}{L}\left[1 - \left(\frac{y_f}{y_m}\right)^{5/3}\left(\frac{S_f}{S}\right)^{1/2}\left(\frac{n}{n_f}\right)\right]} \tag{2-12}$$

By assuming equal slope in the main and flood channels the geometry factor K_G simplifies to:

$$K_G = \sqrt{1 - \frac{L_f}{L}\left[1 - \left(\frac{y_f}{y_m}\right)^{5/3}\left(\frac{n}{n_f}\right)\right]} \tag{2-13}$$

By assuming equal roughness in the main and flood channels the geometry factor K_G simplifies to:

$$K_G = \sqrt{1 - \frac{L_f}{L}\left[1 - \left(\frac{y_f}{y_m}\right)^{5/3}\right]} \tag{2-14}$$

It was found that both K_G and the scour depth decrease with both increasing L_f/L and y_m/y_f as expected. At high y_m/y_f values, that is, relative shallow flood channels, K_G tends to be independent of y_m/y_f. It was also found that the equation derived slightly overpredicts the experimental data in general.

By assuming equal slope only, equation (2-12) was plotted by Melville (1995). The resulting plot showed K_G varies as a function of y_m/y_f and L_f/L for n_m/n_f =1.0 and 0.2. It was shown that the effect of slightly higher roughness in the floodplain compared to that in the main channel is a reduction in K_G and hence in scour depth, especially at small values of y_m/y_f, that is, for computed channels having more uniform depth overall. For large y_m/y_f, $L_f/L \approx 1.0$ and small n_m/n_f, that is, for the case of an abutment sited at about the edge of a relatively shallow and rough floodplain, the scour depth can be as small as 10 percent of that in the corresponding rectangular channel. The K_G values predicted using this equation should be confirmed by experiment.

Based on all the scour factors that are discussed above, the following relation has been proposed by Melville to describe the local equilibrium scour at bridge abutments and to facilitate design. Designing examples have been given in Melville (1997).

$$d_s = K_{yL} K_I K_d K_S K_\theta K_G \qquad (2\text{-}15)$$

2.4 <u>Experimental Studies of Scour at Abutment in Compound Channels</u>

Sturm and Janjua (1993) performed an experimental study in a wide flume with a defined main channel adjoining a floodplain with movable bed. A proposed relationship for the abutment scour depth is suggested from dimensional analysis, and it is shown from a regression analysis that the scour data are well represented by the approach Froude number and the discharge contraction ratio in the proposed relationship. These results do not apply to an abutment encroaching onto the banks of the main channel.

Sturm and Janjua (1994) studied clear-water scour around a bridge abutment in a wide floodplain of a compound channel. First through dimensional analysis and contraction theory derivation, they came up with a preliminary relationship of the form $d_s / y_f = f[F_f, F_c, M]$, where d_s = scour depth at the abutment, y_f = approach flow depth on the floodplain, F_f = approach flow Froude number on the floodplain, F_{fc} = critical Froude number on the floodplain, M = discharge contraction ratio defined as the ratio of the discharge in the approach section with a width equal to the opening width and the total approach discharge. Then experiments were conducted in a flume with a fixed-bed main channel and a movable-bed floodplain in which the abutment terminated. Flow depth, discharge, compound-channel geometry, and abutment length were varied, and measurements of approach velocity distribution and scour depth were obtained. They found that the experimental data of dimensional scour depth displayed a considerable collapse when plotted against $F_f / (M * F_{fc})$, which showed that the ratio of the approach Froude number to the critical Froude number, and the discharge contraction ratio, can be used to explain much of the observed variation in scour hole depth. Through a least-squares regression analysis, they gave the best-fit linear equation of their data as:

$$\frac{d_s}{y_f} = 7.70[\frac{F_f}{M * F_{fc}} - 0.35] \qquad (2\text{-}16)$$

They argued that the discharge ratio M reflects the influence of the compound-channel geometry and roughness on the flow field, and ultimately on the scour hole depth. The proposed relationship is limited to the range of values covered by the variables in this study.

Sturm and Sadiq (1996) conducted an experimental study of the depth of clear-water scour around the end of a square-edged bridge abutment terminating in the floodplain of a compound channel. It is indicated that a discharge contraction ratio arising from a theoretical contraction scour analysis for equilibrium conditions can be used for explaining the effect of flow distribution on the local abutment scour depth in the case where significant backwater occurs from bridge contraction. The use of reference values of approach flow depth and velocity in the floodplain for undisturbed conditions without the bridge is shown to collapse experimental results for scour depth in both the case of a contraction with negligible backwater and the case of a contraction with significant backwater in the bridge approach section.

Sturm and Chrisochoides (1997) investigated experimentally clear-water scour around a bridge abutment for variable embankment lengths in a compound channel with the setback of the abutment face relative to the edge of the main channel varying from 0.17 to 0.66 for three different sediment sizes and two different channel cross-sections. They found that both measured and numerically predicted velocities U_a in the local scour region near the face of the upstream end of the abutment at initiation of scour explain, at least in part, the measured values of equilibrium scour depth.

Sturm and Chrisochoides (1998a) investigated experimentally clear-water scour around a bridge abutment for variable embankment lengths in a compound channel with the setback of the abutment face relative to the edge of the main channel varying from zero to a large value. Vertical-wall and spill-through abutments with three different sediment sizes were investigated. They found that the scour data can be represented in terms of the dimensionless ratio of $q_{f1}/(Mq_{f0c})$, in which M = discharge contraction ratio as defined above; q_{f1} = the unit discharge at the beginning of scour in the approaching floodplain; q_{f0c} = the critical unit discharge in the floodplain for the undisturbed floodplain. The limitation of their work may be that since their flume bed slope was fixed to 0.0022, for different discharges the scour data may be not comparable since they resulted from different shear stress ratios.

Sturm (1998) presented two different formulations for clear-water scour prediction for flow around bridge abutments on the floodplain of a compound channel. The first formulation is of the form of

$$\frac{d_s}{y_{f0}} = C_r\left(\frac{q_{f1}}{Mq_{f0c}} - C_0\right) \qquad (2\text{-}17)$$

where d_s = scour depth at the abutment; q_{f1} and q_{f0c} are defined above; y_{f0} = undisturbed average flow depth in the approach floodplain; C_r and C_0 are constants to be determined by experiments. In this formulation, the scour depth is related to hydraulic variables in the approach section to the bridge with emphasis on parameterizing the redistribution of flow between the main channel and floodplain due to the flow contraction caused by the bridge opening. The second formulation is of the form of

$$\frac{d_s}{y_{f0}} = f\left(\frac{U_{ab}}{U_c} - 1, \frac{y_{ab}}{y_{f0}}\right) \qquad (2\text{-}18)$$

where U_{ab} = the maximum resultant velocity at the upstream corner of the abutment face; y_{ab} = floodplain flow depth at the location of U_{ab} in the contracted section; U_c = critical velocity. In this second formulation, scour depth is related to local values of the hydraulic variables near the face of the abutment. The first formulation requires a one-dimensional numerical flow model to compute the scour prediction variables while the second formulation needs a two-dimensional numerical model to compute the required flow variables. Both formulations are tested with a series of experimental data collected in laboratory compound channels having three different geometries and three different sediment sizes and a similar degree of correlation for each of the formulations is shown by regression analysis.

Cardoso and Bettess (1999) conducted experiments in a compound channel using abutments that extended different distances onto the floodplain including right up to the edge of the main channel (Melville, Type III) to study the effect of time and channel geometry on scour at those abutments. The experiments were performed with flow on the floodplain being critical. They found that for clear-water scour the equilibrium was reached after 68 hours on average, and for live-bed conditions equilibrium was attained after 10 hours on average. The three phases were identified in the time development of the scour. Moreover, the scour development in the principal phase is well described by the theories of Ettema (1980), Franzetti (1982), and Whitehouse (1997). However, time needed for clear-water scour to reach the equilibrium state seems to vary with sediment size. Sturm and Janjua (1993) found that for sediment size of 3.3 mm it only took 12~16 hours to reach a state when there was no further movement of sediment out of the scour hole, which implies the equilibrium state is reached.

Cardoso and Bettess (1999) found in their experiments that unless the abutment approached to the edge of the main channel, there was little interaction between the flow in the main channel and flow on the floodplain. They also found that as the ratio of the abutment length to the floodplain width L/L_f tends from smaller than 1 to 1, increase of scour depth was not found at the tip of the abutment; on the contrary, their data indicate a flattening or decreasing trend for L/L_f >0.8. These findings contradict with Kouchakzadeh and Townsend (1997).

In Kouchakzadeh and Townsend (1997), they found that at small floodplain depths, the difference between main channel and floodplain velocities initiates a strong lateral momentum transfer in the form of banks of vortices having their vertical axes along the main channel and floodplain junction regions even without the presence of the abutment. They found that as the L/L_f ratio increases from 0.24 to 0.55, both the shear stresses and the scour depth increase at the tip of the abutment for both interacting flow and non-interacting flow (the flow on the floodplain being isolated from the flow in the main channel by a plate placed on the edge of floodplain along the flow direction). For the case of abutments terminating near a river's main channel and floodplain junction regions, under the condition of strong flow interaction, the lateral momentum transfer can produce a 15-30% increase in local scour at the abutments. Therefore, the conclusion was drawn that scour can be reduced by isolating the main channel flow from the floodplain flow.

These vortices act as mechanisms for transferring momentum between the main channel and floodplain zones by their continuous emergence and decay (Sellin, 1964). Myers and Elsawy's (1975) study quantified the impact of LMT on both the value and distribution of boundary shear stress in the main channel and floodplain of their asymmetric compound-shaped channel. For their shallowest floodplain depth, they observed 260 and 200 percent increases in maximum and average floodplain shear stresses, respectively, under conditions of interacting flows over isolated flows.

CHAPTER 3. A LITERATURE REVIEW ON BRIDGE ABUTMENT SCOUR COUNTERMEASURES

3.1 Introduction

A wide variety of countermeasures have been used to control channel instability and scour at bridge foundations. In HEC-23 (Lagasse et al., 1997) a countermeasure matrix is organized to highlight the various groups of countermeasures and to identify their individual characteristics. Countermeasures have been organized into groups based on their functionality with respect to scour and stream instability. The three main groups of countermeasures are: hydraulic countermeasures, structural countermeasures, and monitoring. Hydraulic countermeasures can be further classified as river training structures and armoring countermeasures.

Countermeasures for local scour at abutments consist of countermeasures that improve flow orientation at the bridge end and move local scour away from the abutment, as well as revetments and riprap placed on spill slopes to resist erosion.

Selected countermeasures including spur dikes, guide banks and collars will be reviewed and studied subsequently.

3.2 Spurs dikes

3.2.1 Introduction

Spur dikes have been studied intensively for many years mostly as river training or river rehabilitation structures instead of abutment scour countermeasures. Studies on spur dikes include Lacey (1936), Inglis (1949), Mushtaq Ahmad (1953), Laursen (1953, 1962b), Andru and Blench (1957), Iwagaki (1958), Garde (1961), Liu et al. (1961), Cunha (1973), Gill (1972), Garde (1961), Franco (1982), Copeland (1983), Rajaratnam and Nwachukwu (1983a), Zaghloul (1983), Brown (1985), Suzuki et al. (1987), Khan and Chaudry (1992), Wu and Lim 1993), Molis et al. (1995), Mayerle et al. (1995), Muneta and Shimizu (1994), Tominaga et al. (1997), Zhang and Du (1997), Soliman et al. (1997), Farsirotou et al. (1995), Kuhnle et al. (1997, 1998, and 1999), Shields et al. (1991, 1995, 1998), Lagasse et al. (2001). Spur dike length, alignment with flow, flow structure around spur dikes, construction materials and many other parameters have been investigated. Scour depth predictors around spur dikes and spur dike design guide lines are provided.

Spur dikes have been used extensively in all parts of the world as river training structures to enhance navigation, improve flood control, and protect erodible banks (Copeland, 1983). Spur dikes are structures that project from the bank into the channel. There are a variety of terms that refer to these transverse structures, including spur dikes, transverse dikes, cross dikes, spurs, wing dams, jetties, groins, deflectors. While there are some differences in the use of these terms they may be taken to be generally synonymous. Following usage of the U.S. Army Corps of Engineers (Franco, 1982; Copeland, 1983), spur dikes will be used here. Spur dikes may be permeable, allowing limited passage of water at a reduced velocity, or they may function to completely block the current (impermeable). They may be constructed out of a variety of materials including masonry, concrete, earth and stone, steel, timber sheet piling, gabions, timber fencing, or weighted brushwood fascines. They may be designed to be submerged regularly by the flow or to be submerged only by the largest flow events.

A spur dike serves one or more of the following functions: (1) Training of the stream flow; (2) Protection of the stream bank from erosion; (3) Improvement of depth for navigation (Garde, 1961). In recent years, porous and overflow-type spur dikes have been shown to have ecological effects providing appropriate habitat for fish and other aquatic life in highly degraded streams. Because of the deposition induced by spur dikes, they may protect a stream bank more effectively and at less cost than revetments (Lagasse et al., 2001). Spur dikes are usually built in a group of two or more, and may be at right angles to the bank, angled upstream, or angled downstream. The crest of the individual dikes might be level or sloping from the bank towards the channel. The crest of each succeeding dike in a system might be at the same, at a higher, or at a lower elevation than the one upstream, based on the low-water plane (Franco, 1982). Even with the widespread use of spur dikes, there has been no definitive hydraulic design criteria developed. Design guidance is based largely on experience and practices within specific geographical areas. This is due to the wide range of variables affecting the performance of the spur dikes and the differing importance of these variables with specific applications. The main site-specific parameters affecting the performance of the spur dikes include: channel width, depth, flow velocity, shape of flow hydrographs, sinuosity of the channel, bed material size distribution and transport rate, material characteristics of the bank (Copeland, 1983). Parameters that affect the performance of spur dikes include length, width, height, shape, orientation angle, permeability, construction materials, and longitudinal extent of the spur dike field (Melville and Coleman, 2000).

Spur dikes may be classified based on their permeability: high permeability - retarder spur dikes; impermeable - deflector spur dykes; and intermediate permeability - as retarder/deflector (Brown, 1985). Permeability of a spur dike may be defined as the percentage of the spur dike surface facing the flow that is open. A permeable spur dike of the hurdle or pile cluster type has the advantage of slowing down the current instead of deflecting it, thus hastening the deposition of sediment and building up of the high ground and the bank lines. This is especially effective on rivers carrying a considerable amount of suspended sediment. Permeable spur dikes have been used in India and Pakistan as a temporary measure to meet exigencies with success. But permeable spur dikes have their draw-backs. Because (1) they are not as strong as the solid types in resisting the forces exerted by floating debris or ice; (2) they are a nuisance to navigation during high water period if built low, and the cost of long piles may be prohibitive if built high; (3) the best type of permeable construction is obtained by using piles, but this would not be feasible where the river bed is of gravel and boulders; they will not resist the flow very well unless they be of the tetrahedral type such as those used in California and Japan, where slant posts connected to base beams are weighted down by concrete blocks or stones in wire crates (The United Nations 1953).

A qualitative guide as to the type of spur dike to use for a specific situation is given in a table in Lagasse et al.(2001). This table provides preliminary advice on the type of spur dike that may be most suitable for a given circumstance.

3.2.2 <u>Local Scour at Spur Dikes</u>

The local scour pattern at spur dikes is quite similar to the scour pattern at bridge abutments since they are essentially similar structures and have similar configurations in rivers. The flow adjacent to a spur dike is characterized by a system of vortices that is formed as the flow is diverted around the structure. Flow velocity is greatest at the edge of the structure where the protrusion into the channel is greatest. This flow velocity peak and high turbulence causes bed material to be suspended intermittently and transported by the flow. For simple-shaped spur dikes (i.e. flat plates without overtopping flow), the maximum depth of scour occurs at the tip of the structure. For more complicated spur dike shapes and overtopping flows the shape of the scour hole may become more complex. Predicting the vertical and lateral extent of the local scour is critical for determining the length of bank the spur dike will protect and for determining the depth of spur dike required to protect its base.

An established procedure for predicting the maximum scour depths associated with spur dikes is currently lacking. The many complicating parameters of the stream and the spur dike design are undoubtedly a factor in the lack of a unified prediction relation for prediction of scour in the vicinity of spur dikes. Equations to predict the maximum depth of scour have been developed by several researchers. These equations were derived from experiments in laboratory flumes in which the maximum depth of scour associated with spur dikes was measured. Even with modern equipment the maximum scour depth associated with spur dikes in the field is very difficult to collect. Unsteady flows, nonuniform and sometimes varying sediment sizes, and the difficulty of determining the actual location of the stream bottom, make collection of field data challenging and rare.

Lacey (1936) gives a formula for maximum scour depth at a spur-dike. Based mainly on the field observation, Inglis (1949) compared the maximum scour depths with those obtained from Lacey's equation. He recommended certain coefficients to be applied to Lacey's regime depth to find the maximum scour depth and also analyzed the influence of size of material on the maximum scour depth. Based upon experiments with bed material ranging in size from 0.06 mm to 0.37 mm, Inglis concluded that gradation is a factor affecting the maximum scour. His study also indicated that the maximum scour depth depends on local discharge (on the local mean velocity of flow).

Ahmad (1953) conducted investigations on the behavior of spur dikes using sand of 0.37 mm and 0.7 mm mean diameter and drew some valuable conclusions regarding the effect of various parameters on the maximum scour. He studied the effect of different discharge intensity, sand grade, flow concentration and angle of the spur dike to flow on the scour depth and scour pattern around a spur dike. He gave a formula for calculating the scour depth at a spur dike nose for different conditions of flow concentration and angles of approach based on the experiments. He also determined the shape of the falling apron in plans for different types of spur dikes and for various angles of approach to a T-headed spur. He was of the opinion that the maximum scour depth is unaffected by change in the sediment grade. But the rate of the development of the scour is much more rapid with finer sand.

Laursen (1953) has stated that scour around bridge piers is a function of depth of flow and is independent of mean velocity and size of bed material for a given opening ratio. Laursen (1962b) maintained that under conditions of no sediment supply, such as a relief bridge, the velocity and the sediment size are important in determining the depth of scour and that under conditions of sediment supply by the approach flow well above the critical tractive force, the velocity and sediment size have little effect except insofar as they determine the mode of sediment movement.

Andru and Blench (1957), after analyzing various laboratory and field data on the scour at obstructions, concluded that the depth of local scour depends on discharge intensity and the size of bed material. Izzard and Bradley (1957) feel that the sediment size and gradation affects only the rate of development of scour hole and not the maximum scour hole.

Also according to Iwagaki (1958), the size of the material is important in the study of scour, and the drag coefficient C_D is a significant parameter that takes into consideration the size and the specific weight of the sediment.

Garde (1961) conducted experiments with four sizes of spurs which gave the contraction ratio of 0.90, 0.835, 0.667, and 0.530 in a channel of 0.61-m width. They used four sizes of sand, namely, 0.3 mm, 0.46 mm, 1 mm, and 2.3 mm in diameter. According to Garde, Froude number is a significant parameter for scour around obstructions and also the size of sediment has an influence both on the rate of scour and the maximum scour depth.

Liu et al. (1961) did experiments in two flumes with sediment sizes of 0.56 mm and 0.65 mm, respectively, and found that (1) the effect of flow velocity on scour is appreciable; (2) when the flow carries appreciable bed load, the opening ratio has no appreciable effect on the depth of scour. But when the flow carries no bed load, the limiting maximum scour is found to be a function of the opening ratio; (3) the scour caused by flow carrying bed load is a local phenomenon, while the scour caused by flow carrying no bed load is related to the overall geometry of the constriction. Liu did not consider the effect of sediment size on scour in this study.

Liu et al. (1961) and Cunha (1973) also determined that the contraction ratio was not important once sediment movement was established. However, Liu et al. considered velocity to be an important parameter with or without sediment movement.

Tison (1962) did experiments in a 70 cm wide, 20 m long flume with a mean sand size of 0.45 mm. He disagrees with Garde (1961). Garde found that with other conditions remaining the same, the maximum scour depth has the greatest magnitude for the spur dike inclination of 90^0. While Tison shows by experiments and theoretical analysis that the spur dike of upstream inclination should have the maximum scour depth.

Gill (1972) conducted experiments using a flume 0.76 m wide, 0.46 m deep and 12.2 m long and two sizes of sand of 1.5 mm and 0.91 mm. He concluded that depth of equilibrium scour is affected both by size of bed material and depth of uniform flow upstream of the spur location. And the depth of maximum scour occurs when the sand bed upstream of the spur is at the threshold of movement. The combined effect of the bed material and the flow depth was empirically formulated.

Copeland (1983) carried out model tests in a 130- by 50ft sand bed flume with a channel top width of 8ft and an average depth of 0.24ft. The stream sinuosity was 1.6 and the slope was 0.0012. He suggested that the coarse fraction of the bed material is an important factor that affects the scour depth around the spur dike.

Rajaratnam and Nwachukwu (1983a) tried to study the flow structures near spur dikes by measuring the flow in a straight tilting rectangular flume, 120 ft long, 3 ft wide and 2.5 ft deep with smooth bed and sides. Most of the tests were done with a smooth bed. They found that when a groin is placed in a channel, it causes a significant disturbance to the flow for a short distance upstream and for a longer distance downstream. The disturbed flow was analyzed by splitting it into a deflected flow region and a shear layer. For the deflected flow region, the skewed turbulent boundary layer model of Johnston was found to be valid. In the shear layer, the velocity distributions for any horizontal plane were found to be similar. They also found that the maximum bed shear stress occurred near the groin nose. The shear stress amplification τ/τ_o varies with the ratio of the length of groin and the width of channel b/B. The τ/τ_o ratio was 5 and 3 for the 6-in. and 3-in. groin respectively.

Zaghloul (1983) conducted experiments in a 36 ft long, 1.5 ft wide and 2 ft deep aluminum flume with Plexiglas wall to study the effects of upstream flow conditions, sediment characteristics, and spur dikes' geometry on the maximum scour depth and scour pattern around a spur dike. Through dimensional analysis and experimental data verification, he found that the maximum scour depth is significantly affected by the opening ratio of the channel and the Froude number of the flow. He also came up with a maximum scour depth prediction equation which he found is identical in form to the regime equations by Lacey and Blench. His experiments were conducted under clear-water conditions but most runs were run for only 2.5 hours.

Suzuki et al. (1987) performed experiments to discuss the characteristics of the local bed form around a series of spur dikes in an alluvial bed with continuous sediment motion.

He used two flumes, one 0.5m wide and the other 0.4m wide and uniform sand with diameter of 0.6mm. He found that the bed form around a series of spur dikes which were constructed to form a low-water bed changes greatly with the ratio of the distance between the neighboring spur dikes L to the dikes length B. He discovered that when the L/B ratio is between 4 and 8, the general bed degradation by a series of spur dikes is almost the same as that of a long contraction for the same contraction ratio (Gill, 1972). When the L/B ratio is smaller than 4, the degradation caused by those spur dikes becomes larger than for a long contraction, while it becomes smaller when L/B greater than 8. He also found that the local scour around the first spur dike is similar to that of a single spur dike and its depth depends on the flow depth H when H/B is smaller than 1.5. This can be expressed by Laursen's equation (1963) fairly well. When H/B is greater than 1.5, the scour depth is supposed to be dependent mainly on B and Gill's equation is more favorable.

Suzuki also found that the scour around the second spur dike is strongly influenced by the first spur dike. The scour depth of the dike far downstream from the first dike is smaller than that of the first dike when L/B is less than 12.

Tominaga et al. (1997) studied experimentally the flow and turbulent structures around spur dikes with various permeability. The experiments were conducted in an 8m long, 0.3m wide and 0.4m deep tilting flume. Three types of spur dikes were set up. The first is a solid type made of wood, the second is a semi-porous type made of the gravel-filled box frame and the third is a porous type made of only the box frame. The spur dike model is 15cm long, 3cm deep and 10cm high in the non-submerged case, 5cm high in the submerged case. He found that the processes of momentum exchange are different from each other with respect to the permeability of the spur dike. The recovering process of the streamwise velocity distribution becomes slow with an increase of the permeability. When the permeability is relatively small, the secondary flow is generated and plays an important role on the mixing process. In the submerged spur dike, the recirculation zone is limited to a short distance but the lateral difference of the streamwise velocity is not reduced for a long reach. So the type of the spur dike should be determined considering each characteristic function.

Kuhnle et al. (1997, 1998, and 1999) performed a series of experiments in a flume with 30 m length, 1.2 m width, and 0.6 m height to investigate the volume of scour holes associated with spur dikes. By varying the spur dike orientation angles, overtopping flow height, and the contraction ratio, respectively, they found that: (1) the contraction ratio and flow depth are positively correlated with the volume of the scour for a given elapsed time under steady flow; (2) the geometry of the scour hole has been shown to be affected by the value of the overtopping ratio. The larger the overtopping ratio is, the greater is the erosion of the bed in the near bank region and the more the region of maximum scour shifts toward the channel bank. It also caused a secondary scour zone to form downstream of the spur dike; (3) for the three angles of spur dike and two contraction ratios being considered, 135^o spur dikes had the lowest, 90^o spur dikes had an intermediate value, and 45^o had the highest bed erosion in the vicinity of the channel bank. The volume of the scour hole was greatest for the 135^o spur dikes. They also presented a simple procedure to predict the area and volume of scour around spur dikes.

In addition, a series of studies of the local scour phenomenon around bridge abutments were also carried out by various investigators. These studies have been reviewed in a previous chapter.

3.2.3 Empirical Formulas

Some equations for predicting the scour depth at the nose of spur dikes are listed below and all of them are empirical equations. These equations were derived from tests in laboratory flumes with limited verification by prototype testing. There has been a general lack of agreement of the important variables needed to predict maximum scour depth. This disagreement has been possibly settled by Melville (1992, 1997) in which the ratio of the length of the structure to the flow depth determines the form of the equation. Melville's (1992) equations were technically derived for bridge abutments, however, in many cases, particularly in experimental studies, model bridge abutments are similar to spur dikes since they are structures with similar characteristics.

$$y_s = 0.90\left(\frac{Q^2}{f^{1/3}}\right)$$ Khosla (1936) (3-1)

$$\frac{y_s}{y} = KF_n^{2/3}\left(\frac{B_1}{B_2}\right)^{2/3} g^{1/3} - 1.0$$ (from Kheireldin1995) Inglis (1949) (3-2)

$$y_s = kq_2^{\;0.67}$$ Ahmad (1953) (3-3)

$$y_s = yK\left(\frac{B_1}{B_2}\right)F_n^n$$ Garde et al. (1961) (3-4)

$$y_s = 0.3y + 2.15y\left(\frac{L}{y}\right)^{0.4} F_n^{0.33}$$ Liu et al. (1961) (3-5)

$$y_s = k\left(\frac{q_2^{\;2}}{F_{bo}}\right)^{0.33}$$ Blench (1969) (3-6)

$$y_s = 8.375y\left(\frac{D_{50}}{y}\right)^{0.25}\left(\frac{B_1}{B_2}\right)^{0.83}$$ Gill (1972) (3-7)

$$\frac{L}{y} = 2.75\frac{y_s - y}{y}\left[\left(\frac{1}{r}\frac{(y_s - y)}{y} + 1\right)^{1.70} - 1\right]$$ Laursen (1962) (3-8)

$$\frac{y_s - y}{y} = 4.0F_r^{0.33}$$ Richardson (1976) (3-9)

$$\frac{y_s - y}{y} = 0.32 + K[\frac{L}{y}]^{1/2}$$ Kwan (1984) (3-10)

$$\frac{y_s - y}{y} = 0.78K_S K_\theta (\frac{L'}{y})^{0.63} F_a^{\;1.16} (\frac{y}{d_{50}})^{0.43} \sigma_g^{-1.87}$$ Froehlich (1989) (3-11)

$$y_s = 10K_\theta y + y \qquad \frac{L}{y} \geq 25$$

$$y_s = 2K_s L + y \qquad \frac{L}{y} \leq 1$$ Melville (1992) (3-12)

$$y_s = 2K_s^* K_\theta^* (yL)^{0.5} + y \qquad 1 < \frac{L}{y} < 25$$

$$\frac{y_s}{y} = 3.36 F_n^{\ 0.488} (U/U_c)^{0.788} (L/y)^{0.538} (B_1/y)^{-0.142} \quad \text{Zhang and Du (1997)} \qquad (3\text{-}13)$$

where B_1 = original channel width, B_2 = constricted channel width, C_D = drag coefficient, D_{50} = median grain size, σ_g = geometric standard deviation of the bed material, F_{bo} = Blench's zero bed factor which is a function of grain size, F_n = Froude number, f = Lacey silt factor, g = acceleration of gravity, k = function of approach conditions, K = function of C_D which varies between 2.5 and 5.0, L = effective length of spur dike, $L' = A_e / y$, A_e = flow area of an approach cross section obstructed by a roadway embankment, n = function of C_D which varies between 0.65 and 0.90, Q = total stream discharge, q_2 = discharge per unit width at constricted section, r = assumed multiple of scour at spur dike taken as 11.5 by Laursen, U = average velocity in unconstricted section, y = average depth in unconstricted section, y_s = equilibrium scour depth below water surface, ρ = mass density of water, K_s = abutment shape factor by Melville, K_θ = abutment alignment factor by Melville, K_θ^* = adjusted abutment alignment factor by Melville, K_s^* = adjusted abutment shape factor by Melville.

In Equation (3-2), k varies between 0.8 and 1.8. This equation is for spurs projecting straight or upstream with sloping head.

Equation (3-3) is derived from dimensional analysis based on a hypothetical experiment. Because the terminal velocity of sediment is taken as a factor, the equation should be for flow with sediment transport. The coefficient k is determined by experiments and for different positions and flow conditions. It has different values ranging from 1.2 to 2.25.

The author of Equation (3-4) first carried out dimensional analysis and concluded that the maximum scour depth is a function of opening ratio, spur inclination angle to the channel, Froude number of the flow, and the drag coefficient of the sediment. Then he plotted his data from his experiment and ended up with the expression of the form of the above equation. K and n are functions of C_D. For the condition of the experiment, which is for 0.29 mm sediment, 90^0 spur dike inclination, and four specific opening ratios, K is determined to vary between 5.00~2.75 and n equals 2/3. The author does not state if this is for clear-water scour or sediment transporting scour.

Equation (3-5) pertains to vertical-wall and vertical board models caused by flow carrying appreciable bed load and for spur normal to bed flow.

The author of Equation (3-6) refers to the work of Andru in which data from various sources are plotted as $y_s F_b^{1/3}$ against q. Equation (3-6) is the best fit line for those data. Here *k* varies between 2.0 and 2.75.

Gill started the derivation of Equation (3-7) from an alternative form of Straub's formula, which was obtained from a simple case of long contraction and supposed to be suitable for use in the case of local erosion around spur dikes. Since Equation (3-7) has been obtained from data having a narrow range, for example, only two sizes of sand were considered. It is only an empirical result and only valid for the results of this investigation. This equation is for flow without sediment transportation and its validity can be extended with more experimental data.

Laursen assumed that a bridge crossing be a long contraction foreshortened to such an extreme that it has only a beginning and an end so that solutions for long contraction can be modified to describe the scour at bridge piers and abutments with the use of experimentally-determined coefficients. Equation (3-8) is thus obtained with some qualitative flume test data and it is for an encroaching abutment normal to the flow with sediment transport. L is the effective length of the abutment and r should be determined by experiment.

Equation (3-9) is for spurs normal to flow in the Mississippi River with L/y>25.

In Equation (3-10), K is a coefficient depending on the lateral scour hole slope. K=0.68, 1.1 and 1.36 for lateral slopes 10^o, 25^o and 35^o, respectively.

Equation (3-11) is for clear-water scour. For live-bed scour condition, the regression model reduces to

$$\frac{y_s - y}{y} = 2.27 K_S K_\theta (\frac{L'}{y})^{0.43} F_a^{\ 0.61} \qquad \text{Froehlich (1989)} \qquad (3\text{-}14)$$

K_S=1.0 for a vertical wall abutment that has square or rounded corners, and a vertical embankment; 0.82 for a vertical abutment that has wingwalls and a sloped approach embankment; and 0.55 for a spillthrough abutment and a sloped approach embankment. $K_\theta = (\theta / 90)^{0.13}$. For design purpose, the author recommended that a factor of safety be added to the value of maximum relative depth of scour computed using Eq. (3-11) or Eq. (3-14).

Through the analysis of local scour, Melville (1992) established the relationship between d_s / L (or d_s / y) and the multiplication of a serious of K factors. The K factors are expressions describing the influence of each parameter including flow intensity, flow depth, sediment size, sediment gradation, abutment length, abutment shape and alignment and approach channel geometry. Using laboratory data available, the author tried to determine each of these K values and finally obtained Equation (3-12). Because of a lack of data, these three equations do not include the effects of sediment characteristics, lateral distribution of flow, or river geometric features including over bank flow and may lead to conservative estimates of scour depth in some instances. However, the design method has the advantage of simplicity.

Zhang and Du (1997) did experiments in a flume of 2.4 m wide and 26 m long to study the maximum scour depth around spur dike using sand bed of average sand diameter of 0.66 mm. The median size of the sand is 0.60 mm and the standard deviation of the sand size distribution is 1.90. For spur dikes perpendicular to the flow and maximum scour condition, they proposed Equation (3-13) using dimensional analysis and regression analysis techniques based on their experimental results. In Equation (3-13), U_c is the flow velocity of beginning of sediment motion.

Comparison of local scour depths at spur dikes (bridge abutments) computed using some of these equations listed above yields a large variation in predicted values. Kheireldin (1995) tested most of these predictor equations using experimental data from Liu (1961), Gill (1972) and Zaghloul (1974) and he found that there are large discrepancies between the different predictors and the measured scour data. Niehus (1996) compared field scour data measured by the USGS with predicted data by most of these equations. He found that almost all the abutment scour prediction equations substantially overestimated the scour depth.

3.2.4 Semi-analytical and Analytical Formulae

Based on the continuity equation, scour geometry, and a generalized form of the power-law formula for flow resistance in an alluvial channel, Lim (1997) proposed a semi-analytical equation (3-15) for the maximum clear-water scour condition around an abutment perpendicular to the flow direction.

$$y_s / y = 1.8(L / y)^{0.5} \tag{3-15}$$

Equation (15) is shown to be in close agreement with the formulas of Melville (1992).

Rahman and Muramoto (1998) considered some experimental features and developed a simplified model for the maximum scour depth prediction using the continuity relation between the inflow and outflow in the restricted flow concentration region of a scour hole. They analytically derived several formulae for vertical wall abutments and sloped wall abutments. For instance, the equation in the case of vertical wall abutments can be derived as

$$\overline{d_s} = \sqrt{\{\tan\phi(1-\beta)/\beta\}\bar{b}} \tag{3-16}$$

where

$\overline{d_s}$ -Dimensionless maximum scour depth, ϕ= rest angle, $\bar{b}$= dimensionless abutment length, β= constant for flow concentration.

The model above was compared with previously-proposed formulae by Laursen (1962), Melville (1992), and Lim (1997) for the prediction of maximum scour depth ($u_* = u_{*c}$). The prediction by the present model was found to be situated in the middle of the previous formulae. But the present model is not able to explain the experimental features of long abutments (L/y>25) where d_s/y takes a constant value (=10) according to Melville's empirical formula. Moreover, the effect of Fr on d_s/y in supercritical flows is uncertain.

3.2.5 Numerical Simulations around Spur Dikes

The most important aspects to be considered in the spur dike design are the layout, plan view, shape, length, spacing, crest longitudinal shape, crest elevation, orientation, permeability, construction materials and local scour (Alvarez, 1989). The majority of these aspects can nowadays be examined with numerical models. Numerical model simulations are usually more cost-effective and faster than physical model studies, and have no inherent limitation on spatial extent.

Zaghloul et al. (1973) solved the Helmholtz vorticity equation and Poisson type equation using constant eddy viscosity. The velocity and vorticity distributions were obtained but computed velocities were not compared with any experimental results.

Tingsanchali (1990) used a two-dimensional depth-averaged equation and a $\kappa-\varepsilon$ model to close the Reynolds' stresses. The velocities agreed with the experimental data when a correction factor for the streamline curvature was applied to the $\kappa-\varepsilon$ model.

Khan et al. (1992) tried to simulate the flow around spur-dikes by solving the depth-averaged Navier-Stokes equations using a two-dimensional mathematical model. MacCormack's explicit finite difference scheme was used and the equations were solved first by neglecting the effective stresses and then by including the turbulent stresses. In all his three trials, the model didn't accurately reproduce the recirculation length. The best recirculation was obtained when turbulent stresses were considered with $\nu_t = 0.001$ with free-slip boundary condition. He concluded that properly reproduced recirculation length is needed to get an accurate velocity prediction. In addition, he concluded that the constant eddy viscosity model does not give good results for flow problems with circulation. Therefore, a turbulence model, like a one- or two-equation model is needed which predicts the distribution of ν_t over the entire flow.

Wu et al. (1993) used multilayer feedforward neural networks to predict the maximum scour depth at spur dikes. What they did is to find a collection of the published laboratory flume data on local scour at spur dikes. Then they used some of those data to train the neural network and let it fully capture the relationship embedded in those data and test the model with the remaining data. For both uniform and non-uniform sediments, neural-network-based models for predicting the maximum local scour depth in clear-water and live-bed scour conditions were constructed. A composite model that uses the combined data on clear-water and live-bed scour was also established. The author concluded that the neural network technique can be successfully used in the correlation of laboratory flume data for prediction of maximum local scour depth in the clear-water and live-bed scour conditions. He also stated that, compared with traditional approaches, this technique is simple and flexible, requires no complicated mathematical modeling and makes no assumptions about the relationship between the local scour depth and the scour parameters.

Mayerle et al. (1995) used a three-dimensional numerical model. Six eddy viscosity turbulence approaches have been implemented to the model with the objective to verify the influence on the flow conditions in the vicinity of spur dikes with different boundary conditions. They found that definite dependence of the eddy viscosity field exists especially on the reattachment lengths behind the dike, and on the velocities and slopes of the free surface. But in the vicinity of the spur dike none of the eddy viscosity approaches tested were able to capture the flow patterns properly. Among the eddy viscosity approaches tested, the one based on the mixing length and the assumption of the local equilibrium for the turbulent kinetic-energy-produced wake length were in very good agreement with the measurements and gave reasonable flow patterns. However, it should not be applied indiscriminately since the mixing length is based on measurements in straight rectangular channels.

Molls et al. (1995) numerically simulated the flow near a spur-dike by solving the depth-averaged equations using both an implicit ADI scheme and the explicit MacCormack scheme. Both inviscid and viscous solutions were obtained. A constant eddy viscosity model was used to close the effective stresses with the viscosity near the spur obtained from the dimensionless, turbulent, eddy viscosity reported by Tingsanchali and Maheswaran. Also an assumption of constant eddy viscosity near the spur throughout the entire flow field was made. The ADI scheme was solved in conjunction with a packed grid and the velocity field and recirculation length closely matched the experimental data and computed results obtained by Nawachukwu and Tingsanchali and Maheswaran, respectively. It was demonstrated that, provided with a reasonable estimate of the eddy viscosity near the spur, the constant eddy viscosity model produces results similar to those obtained with more advanced turbulence models. Through a sensitivity analysis, the ADI scheme was found to be insensitive to Chezy C but very sensitive to turbulent viscosity.

Interestingly, Molls' ADI scheme, Tingsanchali and Maheswaran's (1990) TEACH model and the MacCormack scheme underpredict the experimental velocities at a certain location downstream of the spur.

Soliman et al. (1997) used a 2-D mathematical model (TRUSOLA) to simulate a Nile bend located about 3 km downstream of Naga Hammadi barrage. In this model, the shallow water assumption is used to model the flow, and the vertical momentum equation is reduced to a hydrostatic pressure relation. The vertical water motion is derived from the horizontal flow field using the continuity equation. The program followed the Boussinesq approximation. Different lengths and spacing were used to simulate the effect of spur dikes on water levels and velocity components. About fifty runs were carried out. The author concluded that: (1) spur dikes would cause appreciable increase in the head upstream if it constricted more than 6% of the channel width; (2) A pocket of deposited material occurs between the spurs for all tests with spacing less than 4L but disappeared for a spacing of 7L.

(3) Flow separation occurred at the tip of the first spur for the entire tests. Eddy formation only appeared for spur lengths greater than 18% of the channel width. (4) The longitudinal velocity U increased with the increase of the spur lengths that caused a decrease in the channel width. The transverse velocity V increased in the case of single spur, whereas for double and triple spurs no conclusive trend was found.

3.2.6 Design Considerations for Spur Dike Systems

3.2.6.1 Length and Spacing of Spur Dikes

Spacing and length of spur dikes has been related to the length of bank that is protected by each structure. This varies with local variables such as bank curvature, flow velocity, and whether the structures are designed for navigation or bank protection.

Ahmad (1951) found that a single spur can protect lengths between three to five times the spur projections. Copeland (1983) concluded that the effective length is a more important factor than the spur dike angle in providing bank protection. He also suggested that the spacing-length ratio should be determined by studying proposed bank protection with spur dikes on a site-specific basis using experiences in similar conditions or a model study. Brown (1985) stated that the spacing of spurs in a bank-protection scheme is a function of the spur's length, angle, and permeability, as well as the channel bend's degree of curvature. The direction and orientation of the channel's flow thalweg plays a major role in determining an acceptable spacing between individual spurs in a bank-stabilization scheme. He gave an example and recommended that a spacing criteria based on the projection of a tangent to the flow thalweg, projected off the spur tip should be used. For the spur length, he recommended that the projected length of impermeable spurs should be held to less than 15 percent of the channel width at bank-full stage. While for permeable spurs they should be held to less than 25 percent of the channel width.

Central Board of Irrigation and Power (1989) stated that the spacing between spurs depends on the length protected by each one in the series, the curvature of the reach, the location of spurs (whether on concave banks or convex banks), and the cost of construction etc. So the spacing of spurs could be varied according to different design conditions. A spacing of 2 to 2.5 times of the length of spurs is the general practice. For the length of spurs, no general rules can evidently be formulated. It depends entirely on the exigencies arising in a specific case. But length should not be shorter than that required to keep the scour hole formed at the nose away from the bank. Normally spurs longer than 1/5 the river width are not provided.

Alvarez (1989) recommended that the length of the spur dikes should be kept between the following limits $y \le L \le 0.25B_1$ when the river bank is almost parallel to the river axis, where y and B_1 are the mean flow depth and the mean width of the free flow surface. Also when spur dikes are built in straight stretches without anchoring in the bank, the spacing should be 4L to 6L, where L is the working spur dike length. If the curve is regular with only one curvature radius, 2.5~4L is optimum; if the curve is irregular, the spacing should be found graphically. Recommendations from several sources are shown in Table 3-1.

3.2.6.2 <u>Orientation of Spur Dikes</u>

The orientation of spur dikes has typically been determined by experience in specific geographical areas and by preference of engineers. There is considerable controversy as to whether spur dikes should be oriented with their axis in an upstream or downstream direction.

Thomas and Watt (1913) concluded that the various alignments were probably of slight importance. Franzius (1927) reported that spur dikes directed upstream are superior to normal and downstream-oriented spur dikes with respect to bank protection as well as sedimentation between the dikes.

Strom (1941) reported that the usual practice in New Zealand was to incline impermeable groins slightly upstream, but that downstream-oriented spur dikes had also been used successfully. Strom stated that a spur dike angled downstream tends to swing the current below it toward midstream; this has a reflex action above the dike which may induce the current to attack the bank there. Thus, downstream-oriented dikes should only be used in series so that the downstream protection afforded by each dike extends to the one below it.

Table 3-1. Spur dike spacing recommendations.

Reference	Spacing to length ratio	Comments
Acheson (1968)	3-4	depends on curvature and slope
Ahmad (1951)	4.3 5	straight channels curved channels
Copeland (1983)	2 - 3	concave banks
Grant (1948)	3	concave banks
Los Angeles District (1980)	1.5 2.0 2.5	concave banks straight banks convex banks
Maza Alvarez (1989)	5.1 - 6.3 2.5 - 4	straight channels curved channels
Neil (1973)	2 4	 if 2 or more dikes
Richardson et al. (1988)	2 - 6	depends on flow and dike characteristics
Strom (1962)	3 - 5	
Suzuki et al. (1987)	< 4	straight channels
United Nations (1953)	1 2 - 2.5	concave banks convex banks

The United Nations (1953) reported that the present practice was to construct spur dikes either perpendicular to the bank or to orient them upstream. This publication states that downstream-oriented dikes tend to bring the scour hole closer to the bank. An upstream dike angle varying between 100 and 120 degrees was recommended for bank protection. It also discussed how to build a system of spur dikes.

Mamak (1964) stated that dikes are usually set perpendicular to the flow or set upstream at angles between 100 and 110 degrees.

Lindner (1969), reporting on the state of knowledge for the U. S. Army Corps of Engineers, stated that there has not been a sufficiently comprehensive series of tests either in the field or by model to conclude that any acute or obtuse angle for the alignment at dikes is superior or even as good as perpendicular to flow. Thus, he recommended perpendicular dikes except in concave bendways where they should be angled sharply downstream. Neill (1973) recommended using upstream-oriented dikes. After reviewing much of the literature on spur dikes Richardson and Simons (1973) recommended perpendicular spur dikes, suggesting that spur dikes with angles between 100 and 110 degrees could be used to channelize or to guide flow.

In the United States, the U. S. Army Corps of Engineers (1978) has generally oriented its spur dikes perpendicular or slightly downstream. On the Missouri River, dikes are generally oriented downstream with an angle of 75 degrees. On the Red and Arkansas Rivers, dikes are placed normal to flow or at angles of 75 degree. The St. Louis District uses both perpendicular and downstream oriented dikes. The Los Angeles District (1980) used dikes with an angle of 75 degrees pointing downstream.

As late as 1979, Jansen (1979) concluded that there is no definite answer as to whether spur dikes should be oriented upstream or downstream, and recommended using the cheapest solution---that being the shortest connection between the end of the dike and the bank. This corresponds with Lindner (1969). Copeland (1983) found that the scour depth was more severe for spur dikes with an upstream orientation than for those with a downstream orientation. And he found there is no indication that the scour hole is closer to the bank for spur dikes pointed downstream. He concluded that the spur dike should be oriented perpendicular to the bank to obtain the most effective bank protection.

Brown (1985) recommended that retardance spurs be designed perpendicular to the primary flow direction and spurs within a retardance/diverter or diverter spur scheme be set with the upstream-most spur at approximately 30 degrees to the main flow current at the spur tip, and with subsequent spurs having incrementally larger angles approaching a maximum of 90 degrees at the downstream end of the scheme.

Central Board of Irrigation and Power (1989) has the opinion that spurs should be oriented perpendicular to the bank or upstream for bank protection purposes. It stated that a spur pointing downstream will invariably accentuate the existing conditions and may create undesirable results. It recommended that a repelling spur be angled 100-120 degrees upstream.

Alvarez (1989) recommended an angle of 70^0 facing downstream for a regular curve or a straight stretch. He suggested that level-crested spur dikes may be oriented perpendicular or upstream forming an angle of no more than 120^0. He also gives design guidelines on crest longitudinal slopes and elevation of spur dikes.

From studies of single spur dikes (45, 90, and 135^0) in a straight channel, Kuhnle et al. (1998) concluded that the volume of the scour hole was greatest for upstream (135^0) facing spur dikes, while the potential for bank erosion was greatest for the downstream (45^0) facing spur dikes.

There are proponents of both upstream-facing and downstream-facing spur dikes (Copeland, 1983). As in other design factors of spur dikes the best orientation is most likely a function of the local conditions and the purpose of the structures. There is a lack of agreement as to the most advantageous orientation to construct spur dikes. Permeable spur dikes are usually designed to decrease the flow near the bank. They are generally not strongly affected by the angle and are usually built at 90 degrees from the bank to have the maximum effect on near bank velocity and to use the least material (Lagasse et al., 2001).

3.2.6.3 Permeability

Permeability of spur dikes is a very important factor that affects the function of the spur dikes and also an important design consideration. Generally, it is believed that the greater the spur dike permeability, the less severe the scour pattern downstream of the spur tip, the lower the magnitude of flow concentration at the spur tip. The more permeable the spur, the shorter the length of channel bank protected downstream of the spur's riverward tip.

Brown (1985) gave some recommendations for application of different spur permeability for different purposes as follows: (1) where it is necessary to provide a significant reduction in flow velocity, a high level of flow control, or where the structure is being used on a sharp bend, the spur's permeability should not exceed 35 percent; (2) where it is necessary to provide a moderate reduction in flow velocity, a moderate level of flow control, or where the structure is being used on a mild to moderate channel bend, spurs with permeability up to 50 percent can be used; (3) spur permeability up to 80 percent can be used in some special cases but is not recommended; (4) jack and tetrahedron retardance spurs may be used only when it can be reasonably assumed that the structures will trap a sufficient volume of floating debris to produce an effective permeability of 60 percent or less; (5) Henson-type spurs should be designed to have an effective permeability of 50 percent. (6) if minimizing the magnitude of flow deflection and flow concentration at the spur tip is important to a particular spur design, a spur with a permeability greater than 35 percent should be used; (7) impermeable spurs should not be used along channel banks composed of highly erodible material unless measures are taken to protect the channel banks in this area.

Central Board of Irrigation and Power (1989) discussed the construction, types, and application of permeable and impermeable spurs. It stated that permeable spurs have the important advantage of being cheap and are effective only in rivers which carry heavy suspended load. But they are usually not strong enough to resist shocks and pressures from debris, floating ice and logs. Submerged permeable spurs are preferable to submerged solid spurs since the former do not create turbulent and eddy conditions as strong as with the latter in the case of deep and narrow rivers where depths are considerable. Impermeable spurs are unsuitable in boulder rivers, in streams where there is a clay layer beneath sand –as in deltaic rivers-at the nose of the spurs, and in flashy rivers in which floods rise and fall so quickly that desired silting does not take place.

Spur dikes with permeability up to about 35 percent do not affect the length of the channel bank protected. For permeability values above 35 percent, the length of bank protected decreases with increasing permeability. High permeability spur dikes generally are most suitable for mild bends where small reductions in flow are sought. Permeable spur dikes may be susceptible to damage from debris and ice (Lagasse et al., 2001).

3.2.6.4 Spur Height, Crest Profile, and Spur Head Form

The spur height should be sufficient to protect the regions of the channel bank impacted by the erosion processes active at the particular site. Although the general practice in design of spur dikes in the field has mostly been to place all dike crests at about the same height with respect to low water level, the crest or crown of a dike need not be horizontal depending on what effect of dike upon flow is sought (Klingeman et al. 1984). There are often situations where a variable-height crown is advantageous. The sloping-crown or stepped-down crown, in which the dike crown slopes downward or is stepped downward from bank to mid-channel, appears to have an advantage where acceleration of shoal erosion is needed over a wide range of stages but where a gradually-diminishing channel contraction with increasing stage will suffice. Such a crown design may be required where a spur dike with a level crown would produce objectionable velocities as the stage rises. It often will be less costly to build a stepped-down crown dike than a level crown one if high velocities are not a concern and if the sloping spur dike can produce the shoal erosion desired (Lindner 1969). There is also less tendency for flanking or scouring near the bank ends to occur with sloping dikes than with level dikes (Franco, 1967).

Furthermore, Franco (1967) concluded that stepped-down spur dike systems (a dike is low with respect to the next dike upstream) are more effective in reducing disturbances of channel flow and inducing more sediment deposition in the dike field than systems with all dikes level and that level dike fields are more effective than stepped-up fields. He also recommended that level-crested dikes be placed normal to the flow or oriented downstream and sloping crested dikes be normal to the flow or oriented upstream.

There are different spur head forms that function quite differently when placed in the river channel. Ahmad (1953) classified them in four types as straight, hockey, inverted hockey and T-headed. His tests showed that the maximum depth of scour and area of the apron needed at the T-headed spur are relatively small compared with straight and hockey spurs. He also concluded that the T-headed spur is economical in stone. The interesting thing to be noticed from his data is that the minimum scour depth occurred at the inverted hockey spur.

Franco (1967) performed experiments with the length of the L-head (similar to inverted hockey type spur by Ahmad) equal to half of the distance between the ends of adjacent dikes. He found that the L-head tended to prevent sediment-carrying bottom currents from moving into the areas between the dikes. Maximum scour depth at the ends of the dikes was reduced appreciably, as was the elevation of deposition between spur dikes.

In a series of tests by Linder et al. (1964) it was found that the L-head should close 45 to 65 percent of the gap between the dikes in a dike field, and that little benefit was derived from building the L-head above the water surface. The results indicated that the L-heads decreased the scour around the ends of the spur dikes compared with stubbed-off dikes, and improve navigation conditions and depths by reducing eddy disturbances and by causing the contraction to persist more continuously along the dike system, thus producing more uniform bed configuration and consistent depths. In addition, almost any reasonable degree of closure will give added protection to the concave bank over no closure at all. However, L-heads are expensive so that additional testing and experience are needed to show whether their merits are sufficient to recommend their general use in connection with channel contraction.

J-heads, which are similar to hockey-stick-shaped spur dikes, together with T-heads are also designed with the hope of improving their function. However, there has not been enough investigation of these various shapes to ascertain whether they have any advantage over the L-head. It is believed that the J-head as well as the T-head would possess disadvantages that would make their choice over the L-head injudicious, except possibly for certain specific purposes. In addition, under any condition it would appear that the upstream leg should not be as high as the remainder of the dike (Lindner 1969).

Brown (1985) recommended that: (1) if the design flow stage is lower than the channel bank height, spurs should be designed to a height no more than three feet lower than the design flow stage; if higher, to bank height; (2) permeable spurs should be designed to a height that will permit the passage of heavier debris over the spur crest and not cause structural damage; (3) when possible, impermeable spurs should be designed to be submerged by approximately three feet under their worst design flow condition, thus minimizing the impacts of local scour and flow concentration at the spur tip and the magnitude of flow deflection; (4) for spur crest file, permeable spurs should be designed with level crests unless bank height or other special conditions dictate the use of a sloping crest design; impermeable spurs should be designed with a slight fall towards the spur head, thus allowing different amounts of flow constriction with stage (particularly important in narrow-width channels), and the accommodation of changes in meander trace with stages; (5) a simple straight spur head form is recommended and the spur head or tip should be as smooth and rounded as possible.

3.2.6.5 Other Design Considerations

There are several other design considerations including the channel bed and channel bank contact, spur systems geometry, extent of channel bank protection, etc. Brown (1985) and FHWA (2001) gave recommendations and design examples of spur dikes including all these aspects.

3.2.7 Application of Spur dikes

Spur dikes are used to alter flow direction, induce deposition, or reduce flow velocity. Their main use is to protect banks that contain bridge abutments from eroding. Spur dikes are commonly used to realign streams as they approach a bridge abutment. A bridge abutment may be in danger of being severely eroded when it is subjected to high velocity flow from a channel that has changed course due to meandering of the channel. Spur dikes may also be used to establish and maintain the alignment of a channel. They have been used to decrease the length of the bridge required and reduce the cost and maintenance of the bridge in actively migrating braided channels (Lagasse et al., 2001).

A groin with a submerged dike at its toe was constructed on the coast of Washington to protect a stretch of state highway from destruction due to shore erosion. It was found that the structure performed well as expected and has helped relocate the deep tidal channel 500 to 2000 feet further away from the shore in spite of some expected scour development at the toe of the dike (Sultan et al. 2002).

According to Richardson et al. (1984), spurs may protect stream banks at less cost than riprap revetment, and by deflecting the current away from the bank and causing deposition, they may more effectively protect banks from erosion than revetment. Uses other than bank protection include the constriction of long reaches of wide, braided streams to establish a stable channel, constriction of short reaches to establish a desired flow path and to increase sediment transport capacity, and control of flow at a bend.

3.3 Hardpoints

Hardpoints consist of stone fills spaced along an eroding bank line, protruding only short distances into the channel. A root section extends landward to preclude flanking. Hardpoints are most effective along straight or relatively flat convex banks where the streamlines are parallel to the bank lines and velocities are not greater than 3 m/s within 15 m of the bank line. Hardpoints may be appropriate for use in long, straight reaches where bank erosion occurs mainly from a wandering thalweg at lower flow rate. They would not be effective in halting or reversing bank erosion in a meander bend unless they were closely spaced, in which case spurs, retarder structures, or bank revetment would probably cost less (HEC-23).

3.4 Guide banks

3.4.1 Introduction

Guide banks are earth or rock embankments placed at abutments to improve the flow alignment and move the local scour away from the embankment and bridge abutment. The major use of guide banks in the United States has been to prevent erosion by eddy action at bridge abutments or piers where concentrated flood flow traveling along the upstream side of an approach embankment enters the main flow at the bridge (HEC-23). There also have been various studies on guide banks. Among those studies are Spring (1903), Karaki (1959, 1961) Bradley (1978), Brice and Blodgett (1978), Chitale (1980), Smith (1984), Neill (1973), Richardson et al. (1984), and Lagasse et al. (2001). Guide bank orientation, length, crest height, shape and size, downstream extent and as many other concerns are investigated. Design guidelines for guide banks are given in Neil (1973), Bradley (1978), Ministry of Works and Development (1979), Central Board of Irrigation and Power (1989) and Lagasse et al. (1995, 2001). These studies are reviewed below.

Guide banks provide a smooth transition for flow on the floodplain to the main channel. The effectiveness of guide banks is a function of river geometry, quantity of flow on the floodplain, and size of bridge opening (Richardson et al. 1984). By establishing smooth parallel streamlines in the approaching flow, guide banks improve flow conditions in the bridge waterway. Scour, if it occurs, is near the upstream end of the guide bank away from the bridge.

Guide banks can protect not only bridge abutments from local scour, but also the approach embankment because of the still water area behind it. When embankments span wide floodplains, the flows from high waters must be aligned to flow smoothly through the bridge opening. Overbank flows on the floodplain can severely erode the approach embankment and could increase the depth of the scour at the bridge abutment. Guide banks can be used to redirect the flow from the embankment and to transfer the scour away from the abutment. Guide banks serve to reduce the separation of flow at the upstream abutment face and maximize the total bridge waterway area, and reduce the abutment scour by lessening the turbulence at the abutment face (Lagasse et al., 2001).

There are practically two kinds of guide banks. One is the American practice, which is to give guide banks an elliptical form convergent to the opening, whereas the other in Pakistan and India, guide banks are straight and parallel to the opening with a curved section at the upstream and downstream ends. Mahmood stated that parallel guide banks straighten the flow more effectively than convergent ones (Richardson et al., 1984).

Design guidelines for guide banks are given in Neil (1973), Bradley (1978), Ministry of Works and Development (1979), Central Board of Irrigation and Power (1989) and Lagasse et al. (1995).

3.4.2 Experimental Studies

Karaki (1959, 1961) carried out a laboratory study in a flume 16 ft wide and 84 ft long with an erodible bed for clear-water flow on the floodplain with a spill-through abutment, wing-wall abutment and skewed embankments models. He concluded: (1) guide banks are effective measures to reduce scour at bridge abutments and the effectiveness of guide banks is a function of the geometry of the roadway embankments, flow on the floodplain, and size of bridge opening. (2) The proper location for an earth embankment guide bank is at the abutment with the slope of the guide bank tangent to the slope of the abutment. (3) Curved guide banks are more efficient than straight ones because of the smoother streamlining of the flow. He graphically set forth tentative criteria for design derived from his qualitative experiment results. This criterion has limitations because: the results are qualitative and performed with uniform approach flow; an arbitrary time limit of 5 minutes was imposed for east test for the study.

Herbich (1967) studied the effect of guide banks on the flow pattern at the bridge crossing using a fixed-bed model and a movable-bed model. In the fixed-bed model, he found, in the case of a 90 degrees approach, that: (1) the guide banks produced a marked improvement in the uniformity of velocity across the constriction; (2) the length of the guide banks appear to be unimportant in reduction of velocities provided it is greater than a certain minimum length; (3) the contraction ratio is important and the average reduction in velocities to about nine tenths of the original (without guide banks) is evident for each of them; (4) the patterns of reduction are a function of the contraction ratios and it is more significant than the average reduction; (5) with the abutment skewed at 60 degrees to the flow, the addition of the guide banks decreases the velocities along the upstream abutment to as low as sixty percent of the original (without guide banks); on the downstream side the velocities increase from the twenty three percent contraction but decreases for the other contractions. In a few tests of movable bed studies, Herbich found that the scour with clear-water is greater than the scour with sediment-transporting flow.

Smith (1984) presented a paper covering the chronology of events at the east abutment of Outlook Bridge on the South Saskatchewan River from 1936 to 1981. Abutment scour on the prototype under flood conditions was observed to extend about 5 m below the footing, endangering the stability of the structure. After failure of an attempted remedial countermeasure using bags filled with sand-cement, a 1:100 scale, erodible, sand bed model was constructed and tested using guide wall and riprap as countermeasures. The model included an abutment and two bridge spans. Three steel sheet pile guide walls of different lengths (Wall A is 60 cm long, Wall B is 30 cm long and a semicircular cell C) were simulated with galvanized sheet metal. The alignment of wall A and B formed a quarter ellipse in plan. The tests were performed under clear-water conditions and each test was run for 1 hour. It was found that the longest guide wall completely eliminated the scour at the abutment. But there is a 3.8 cm scour hole at the upstream end of the guide wall. A less desirable feature of this design was that the train of secondary scour holes passed through the position of the first piers and caused an increase in pier scour. The 30-cm guide wall also produced ideal protection for the abutment and no increase in pier scour, but head end scour was increased about 1 cm to 4.8 cm. With the half-cell on the upstream face of the abutment, a conical scour hole of 6.0 cm deep developed. The scour extended so far around the half-cell that the approach grade was severely scoured on one side and the footing was exposed on the other. It was concluded that the half-cell was not a satisfactory alternative. The 30-cm wall was considered to be the most promising. However, a single wall cantilevered out of the river bed was considered vulnerable to damage by ice and also of questionable stability if there was scour on the flow side and no scour on the other side. Accordingly, a variation of a 30-cm wall was selected for further study, and consisted of a multi-cell guide wall of the type so often used for steel cofferdams. It was found that this wall not only can protect the abutment as well as the former 30-cm wall, but also shows more stability than the first one. It was concluded that abutment scour could be controlled with an earth guide bank, consisting of a 30-cm cellular sheet pile guide wall. Test results for the cellular steel guide wall were judged to be most reliable in terms of predicting prototype behavior, and this design appeared to give an ideal solution. A particularly desirable feature of the guide wall was that it eliminated all attack on the upstream face of the highway grade at the abutment. In 1970 it was decided to add an earth guide bank to the existing riprap protection. The most recent sounding in June 1981 showed no change in riprap position, and the abutment was well protected by the combination of the guide bank and the riprap.

3.4.3 Guide Bank Design Considerations

The important factors for guide bank design are orientation relative to the bridge opening, plan shape, length (upstream and downstream of the abutment), cross-sectional shape, crest elevation, and protection of the structure from sour. These aspects are discussed below.

3.4.3.1 Plan Shape

Guide banks with elliptical shape, straight, and straight with curved ends were found to have satisfactory performance in the field. The plan shape of the guide banks depends on the type of channel (meander or braided), direction of the streamlines of the flow approaching the opening and location of the crossing (Richardson et al., 1984). It can be divergent, convergent or parallel. Conflicting recommendations on the plan shape of guide banks occur in the literature, but majority opinion favors converging curved banks forming a bell-mouth entry to the waterway opening. Straight, parallel guide banks can be used successfully in certain situations, but in general they tend to cause formation of a bar alongside one bank, thereby concentrating the flow on the other side of the waterway opening. Divergent guide banks, elliptical guide banks and also straight guide banks with composite curve for upstream head are found in India.

Two guide banks are generally required when the waterway opening is located in the middle of a wide floodplain or braided stream where the direction of the main flow can shift from side to side. A single guide bank may be sufficient when the stream is confined to one side of a valley or where advantage can be taken of a natural inerodible bank on one side (Neill, 1973).

3.4.3.2 Length

There is no hard and fast rule regarding the length of the guide banks to be adopted normally. Views of different authorities are cited below. Guide banks at study sites tended to be longer than recommended by Bradley (1978) at most sites, except at five sites where they ranged from 5 m to 23 m. All guide banks appeared to perform satisfactorily. Not enough short guide banks were included in the study to reach conclusions regarding length (HEC-23).

Karaki (1959, 1961) concluded that as the length of guide banks increases, there is an increase of the width of spread of the concentrated flow, which leads to a reduction of local velocity and results in smaller depth of scour. However, Herbich (1967) suggested that length of guide banks are not important provided they are over a certain length and this is usually satisfied when a certain shape is developed.

Spring (1903) has the opinion that the length of guide bank is dependant on the following considerations: first, the distance necessary to secure a straight run for the river through the bridge; and secondly, the length necessary to prevent the formation of a bend of the river, above and behind the guide banks, circuitous enough to breach the main railway approach bank; thirdly, it depends to a certain extent on the length of the bridge for a given river, provided that the bridge is designed so that all its spans shall be acting fairly equally, instead of some having considerably more flow than others; fourthly, the upstream length of the guide banks is dependant also to some extent on the breadth of the unnarrowed river.

Spring suggested that (1) the length of the upstream part of the guide banks may be made equal to or up to a tenth longer than the bridge. But close attention should be paid to the possibility of the river eroding the main approach bank. In especially wide rivers this may involve the use of very long guide banks; (2) the length of the downstream part of the guide bank may be a tenth to a fifth of the length of the bridge so that the swirl downstream of the bridge can be kept far enough away not to endanger the approach bank.

Karaki (1959, 1961) gave a tentative method to determine the length of the guide banks based on the laboratory data obtained. Based on the information obtained from model studies performed at Colorado State University, field data collected by the U.S. Geological Survey during floods in the State of Mississippi, and field observations by Schneible during floods, Bradley (1978) developed a chart for determine the length of guide banks. In this chart, the discharge ratio Q_f / Q_{100} (Q_f is the lateral or floodplain flow on one side measured at a certain section, cfs; Q_{100} is the discharge in 100 feet of stream adjacent to abutment, measured at a designated section, cfs.) is shown as the ordinate, the length of dike L as the abscissa, and the family of curves are for different values of the average velocity through bridge opening. He also suggested that for skewed crossings, the length of guide banks be set using the monograph for the side of the bridge crossing which yields the longest guide bank length, if guide banks are required on both sides of the channel.

3.4.3.3 Radius of Curved Head

Karaki (1959, 1961) stated that there is a multiplicity of curved shapes that could be used: parabolic, hyperbolic, spiral, elliptical, and circular. But of these, the elliptical is probably the simplest geometrical shape and the one to be considered because of its adaptability to field layout. Among those elliptical, the one with a 2.5:1 major to minor axis ratio is shown to the most effective.

Herbich (1967) recommended that for a 90 degree approach, (1) a curved guide bank should be used and a spiral shape may fulfill the requirements; (2) the guide banks should be designed for maximum flood flow and they should be protected and maintained regularly. For a 60 degree approach, besides some of those recommendations above, he suggested that (1) guide banks at both abutments are necessary and for the upstream one, an elliptical shape with axis ratio of 2.5:1 is the most effective and for the downstream guide bank straight at 5-degree inclination toward the center of the opening. A tail end, curved in shape is necessary at the downstream corner of the downstream abutment. (2) For the upstream abutment, a comparatively long guide bank is needed in case the scour hole is attracted to the abutment.

Chitale (1980) discussed the radius of curved heads of upstream guide banks and found that there are big discrepancies between different methods for determining the radius proposed by different investigators and the field data are not sufficient to prove which is better or not.

3.4.3.4 Guide Bank Slope and Its Protection

The proper section of the guide bank is probably the most important thing to be considered. Cheapness of construction of guide banks is essential because the chief object of guide banks is that the cost of an unduly long bridge may be saved. Therefore, instead of building a high wall on deep foundations, the engineer makes his guide banks of sand and earth on a plan suitable for the proper guidance of the river currents and for the discouragement of swirls; then he lays such a covering of loose stone, boulders, riprap, or one-man-rock, over the exposed part of them, as will serve to protect them against every conceivable kind of attack to which they are liable (Spring, 1903). Spring stated that in India one-man-rocks weighing from 60 to 120 lbs with irregular angles are suitable for the armoring of the guide bank slopes. Although rocks with high specific gravity, say 3.0, are preferred, rocks with a specific gravity of 2.44 are usually used because of its cheapness and availability. Recommendations for the thickness of stone pitching and soling for permanent slopes required at the head, body, and tail of guide banks, for river flowing in alluvial plains can be found in Central Board of Irrigation and Power (1989).

In the Guidelines for Design and Construction of River Training and Control Works for Road Bridges published by the Indian Roads Congress (1985), the size of stone required on the sloping face of the guide banks on the river side is given as $D_r = 0.0282U^2$ for 2:1 side slope and $D_r = 0.0216U^2$ for 3:1 side slope.

Bradley (1978) suggested that if rock is used as a facing on a guide bank, it should be well graded and a filter blanket should be used if the relative gradations of the rock and of the guide bank material require it. If it is constructed of earth, it should be compacted to the same standards as the roadway embankment and should extend above expected high water.

Riprap can be used to inhibit erosion of the embankment materials. As an alternative, guide banks can be constructed entirely of riprap-size material, if such material is readily available. Rock riprap should be placed on the stream-side face as well as around the end of the guide bank. It is not necessary to riprap the side of the guide bank adjacent to the highway approach embankment. The designer is referred to standard references such as HEC-11 (FHWA, 1989), Richardson et al. (1990), or Pagan-Ortiz (1991) for design procedures for sizing riprap at abutments. Riprap should be extended below the bed elevation to a depth as recommended in HEC-11. Additional riprap should be placed around the upstream end (nose) of the guide bank to counter the effects of scour expected at this location.

3.4.3.5 Apron

To obviate damages due to scour at the toe of the guide banks with consequent undermining and collapse of the stone pitching, a stone cover, know as apron, should be laid beyond the toe on the horizontal river bed, so that scour undermines the apron first, starting at its farthest end and works backward towards the slope. Adequate quantity of stone for the apron has to be provided to ensure complete protection of the whole of the scoured face. This quantity will obviously depend on the apron thickness, depth of scour and the slope of the launched apron. These along with size of stone are considered by different investigators.

In the conventional practice in India following Spring, Gales, Inglis and Sethi, the size for stone should be the same as that used from pitching on the slope of guide banks. Each stone weighs about 35 to 55 kg.

According to Spring (1903), It is important to make sure that the apron is made broad enough and thick enough not only to cover up the erodible matter exposed by sub-surface erosion, but also to keep the deep part of the erosion so far off, that the automatically-pitched sub-surface slope may have an inclination of fully two to one. The apron stone should be laid, all along the foot of the slope stone, at a breadth equal to 1.5 times the maximum depth of scour, and of an average thickness equal to 1.88 times the thickness of the slope stone layer. He also recommended that instead of being given a uniform thickness all over, the apron be made of the same thickness as that of the slope at the place where slope and apron join, and about 2.75 times that thickness at its outer fringe.

Observations at guide banks on various rivers have shown that the actual slopes of launched aprons range from 1.5:1 to 3:1, and are even flatter. However, in most cases the average approximates to 2:1. The face slopes of the launched apron, should not, therefore, be assumed steeper than 2:1 or flatter than 3:1 (Central Board of Irrigation and Power). Bell, the originator of the guide bank system, recommended a breadth of apron equal to 1.25 times the scour depth below the bed level at which the apron is laid. Spring suggested from his considerable experience that it should not be less than 1.5 times the scour depth for the shank and 2 times for the head, as has been adopted also by Gales. In addition to 1.5 times the scour depth, Gales provides an extra 4.6 to 7.6 m width of apron at the foot of the slope, which he calls the berm.

Regarding the thickness of apron as laid, people found that according to Spring's recommendations, when the apron is completely launched, its thickness is less than that of the permanent slope near its toe. This deficiency in the distribution of stone was overcome differently by different engineers. Gales advocated the uniform apron and provision of a berm, which is only an addition length of the apron.

3.4.3.6 Construction and Maintenance

The construction process is important to help keep the cost of guide bank low and guarantee the completion of the guide bank in time. Several factors to be considered are the magnitude of construction arrangements for earthwork, the magnitude of construction arrangements for stone, time available for construction and labor availability.

No matter how well they are designed at the beginning, guide banks may find themselves exposed during some period of their existence, at some part of their perimeter, to scour so severe that there seems danger. Therefore, maintenance work is necessary. Service tracks should be made and maintained in case train loads of stones are required to be transported to a threatened place.

Because stone has to be thrown in from time to time as apron stone gets washed away, maintenance may be very expensive-as exemplified by the case of the Hardinge Bridge, where nearly a million pounds was spent on repairs in a period of two years(Inglis, 1949).

Bradley (1978) suggested that: (1) deep trees as close to the toe of the guide bank embankment as construction will permit; (2) do not allow the cutting of channels or the digging of borrow pits near guide banks or along the upstream side of embankments; (3) if drainage is important, put small pipe through guide bank or embankment to drain pockets left behind banks after floods recedes.

Brice and Blodgett (1978) compiled case histories and provided information on scour and countermeasures at 224 bridge sites. Guide bank performance was found to be generally satisfactory at all study sites evaluated. They found that most guide banks are constructed of earth with revetment to inhibit erosion. At two sites, guide banks of concrete rubble masonry performed well. Riprap revetment is most common, but concrete revetment with rock riprap toe protection, rock-and-wire mattress, gabions, grass sods have also performed satisfactorily. Since partial failure of a guide bank during a flood usually will not endanger the bridge, wider consideration should be given to the use of vegetative cover for protection. Partial failure of any countermeasure is usually of little significance so long as the purpose of protecting the highway stream crossing is accomplished.

3.4.3.7 Conclusions

Properly designed guide banks and abutment riprap provide an acceptable alternative to designing bridge abutments to withstand the development of the full scour prism predicted by equations currently available for estimating abutment scour. Since they have excellent performance in the field, guide banks should be given serious considerations as a countermeasure for abutment scour when developing plans for repair or replacement of those endangered bridges. They can eliminate many scour problems associated with bridge crossings and their use can result in a worthwhile savings to the highway program.

3.5 <u>Collars</u>

Another type of countermeasure that was found helpful for pier scour was a kind of appurtenance called collars. Basically it is a piece of circular or rectangular shaped steel plate attached around the pier sitting horizontally a short distance from the bed. Papers of collars on abutment scour protection were not found in the literature.

3.6 Summary

Although many studies have been carried out for each of these countermeasures, there are still needs for further studies of their use in protecting bridge abutments. For instance, spur dikes have not specifically been tested and used for abutment protection. Guide banks have been tested and used for spill-through abutments on rivers with wide floodplains or rectangular channel only. For small country bridges with wing-wall abutments terminating on the main channel bank, studies are still needed. In addition, there are many controversial design guidelines and small design issues that need to be clarified. Collars have been tested for pier scour protection rather for abutment protection.

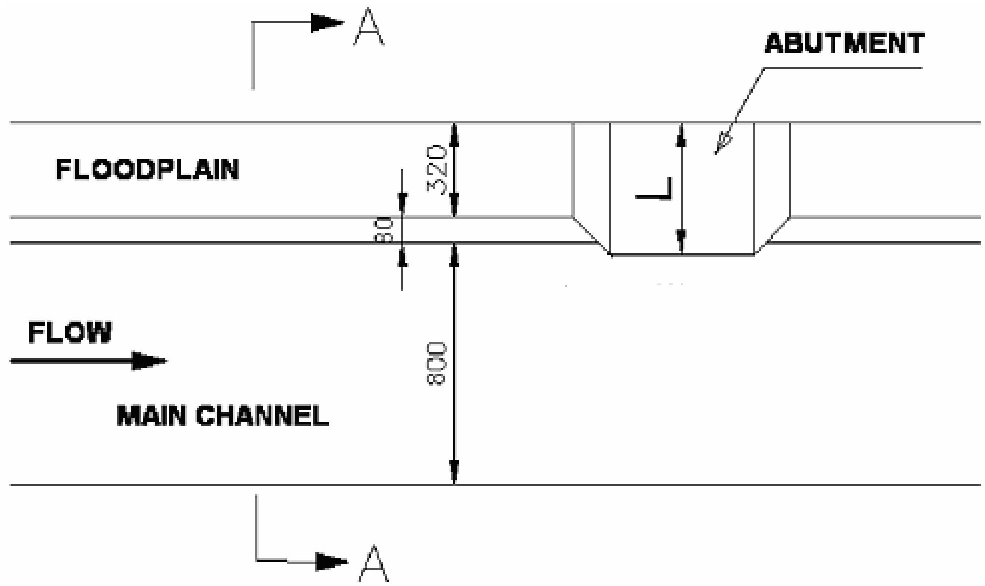
Countermeasures are often damaged or destroyed by the stream, and stream banks and beds often erode at locations where no countermeasure was installed. However, as long as the primary objectives are achieved in the short-term as a result of countermeasure installation, the countermeasure installation can be deemed a success (HEC-23). Therefore, to achieve long-term protection, maintenance, reconstruction, and installation of additional countermeasures as the responses of streams and rivers to natural and man-induced changes is needed.

Among armoring countermeasures, riprap and cable tied blocks are of the most interest by hydraulic engineers. These two countermeasures have been investigated by various researchers as erosion control devices and bank revetments. Since studies of riprap and cable tied blocks are not included here, the literature is not reviewed herein.

CHAPTER 4. EXPERIMENTAL APARATUS AND PROCEDURE

6.1 The Flume

All of the experiments were conducted in a flume located in the hydraulic laboratory at the USDA-ARS National Sedimentation Laboratory, Oxford, MS. The flume channel was 30 m long, 1.2 m wide and 0.6 m deep, and was supported in the center at two points and on the ends by four screw jacks that allow the channel slope to be adjusted. The wing-wall abutment model was located over a 3 m long, 1.2 m wide, by 1.2 m deep recessed section of the flume 22 m downstream from the inlet tank. The test section was 22 m downstream from the inlet and the channel was 1.2 m wide, thereby making the test section 18 times the width downstream from the inlet. This is enough to ensure fully-developed flow at the test section. Uniform flow was established for each experimental run by the adjustment of the flume slopes and pump speed until the water surface line, the bed surface and the flume slope were parallel to one another along a 12 m transect in the approach channel. The channel section of the experiments is illustrated in Fig. 4-1. A flume photo is shown in Figure 4-2 that indicates all of the elements in the flume, such as the abutment model, floodplain, main channel, instrument carriage and flume inlet and outlet etc. A compound channel was used in all experiments consisting of an asymmetric floodplain of width 320 mm next to a main channel, with a bank slope of 1:1. The elevation difference between the top of the floodplain and the main channel bed was 80 mm. The rigid floodplain was made of galvanized steel plate and glued down onto the flume bottom. A layer of sand was glued onto the floodplain to add roughness. In addition, since most floodplains are heavily vegetated and therefore have high roughness, gravel with a mean diameter of 4.5 cm was placed in a staggered arrangement on the floodplain in later runs of baseline cases (Fig. 4-3). By measuring both the velocity profiles in the main channel and on the floodplain and by using Manning's equation, the roughness of the floodplain and the main channel bed under clear-water condition (a velocity ratio of 0.9) were found to be 0.030 and 0.014, respectively.



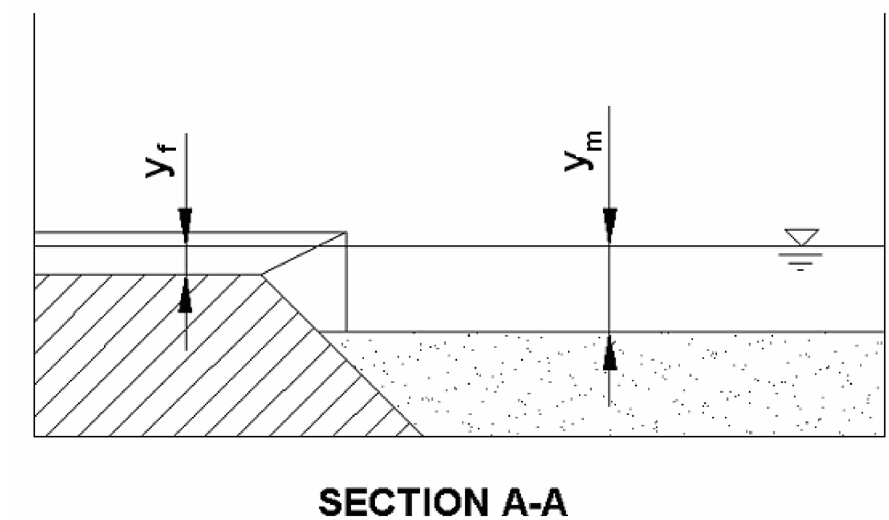


Figure 4-1: Dimension Sketch for Experimental Compound Channel (mm)

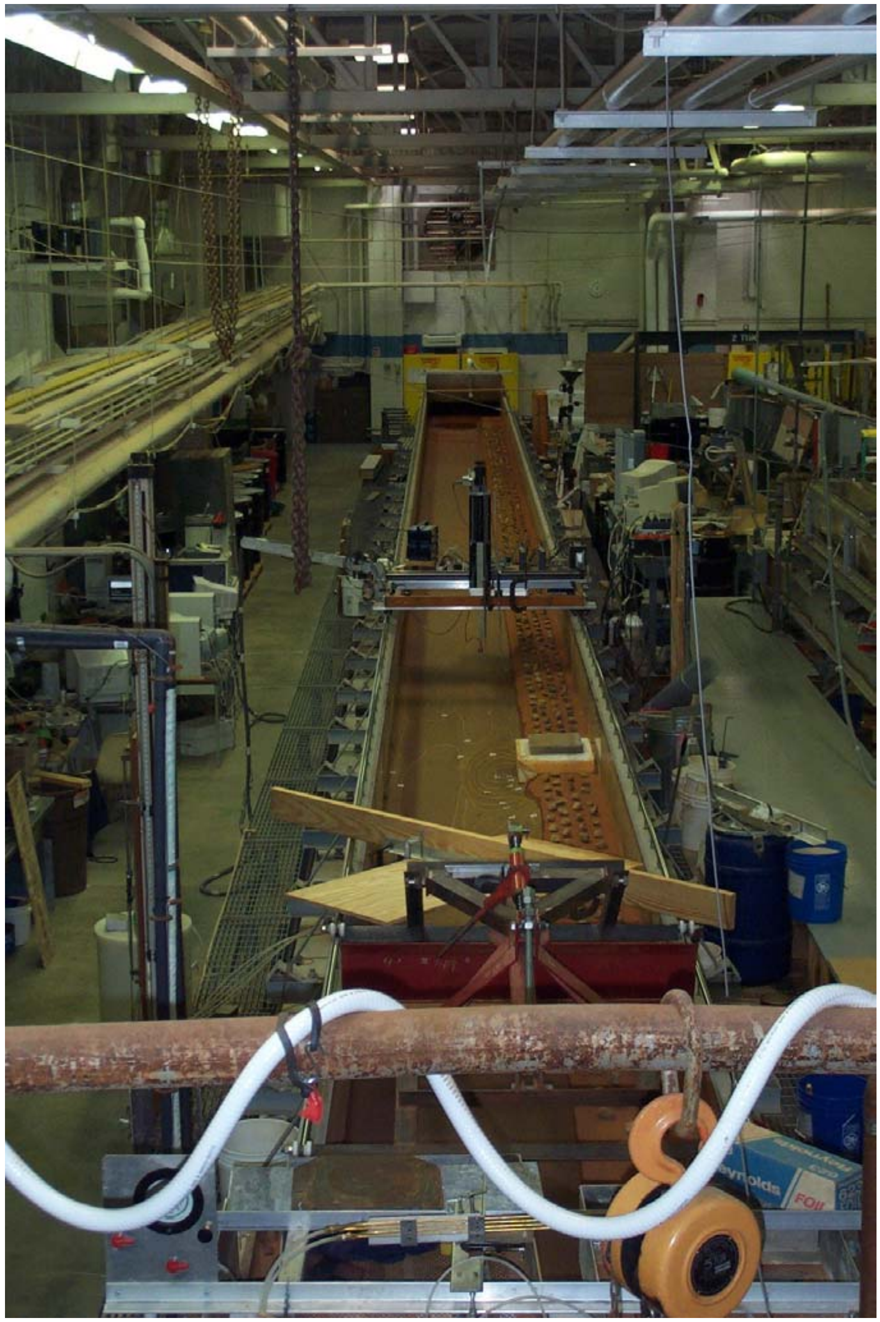


Figure 4-2: A photo of the flume. Looking towards upstream.

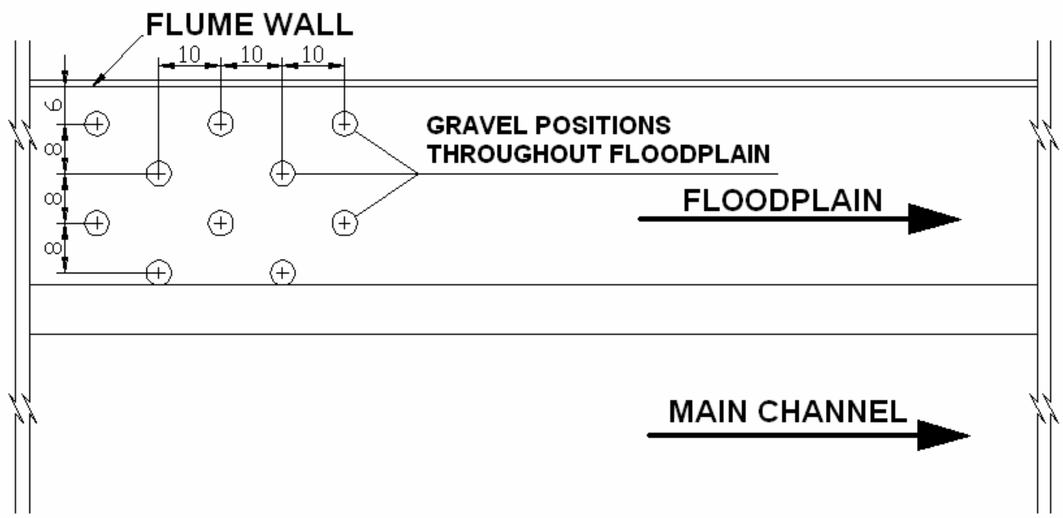


Figure 4-3: Scheme of staggered placement of gravel on floodplain to provide roughness. Gravel was placed throughout the floodplain (cm).

6.1 <u>The Abutment Model</u>

The wing-wall abutment model was made of sheet steel. The dimensions of the model are shown in Fig. 4-4. The abutment terminated on the bank slope of the main channel as illustrated in Fig. 4-1, which corresponds to the Type III abutment of Melville (1992). The height between the top of the floodplain and the top of the abutment was 60 mm. The abutment length was about 1/3 of the channel width and was observed not to alter the flow enough to interact with the far flume wall.

6.1 <u>Sediment Characteristics</u>

The bed material sediment used in the main channel had a diameter of 0.8 mm. The standard deviation of the sediment diameter, σ_g, $\left[\sigma_g = \left(D_{84}/D_{16}\right)^{1/2}\right]$, was equal to 1.37. According to a modified version of Shields diagram (Miller et al., 1977), the critical shear velocity of the bed sediment is 1.995 cm/s. In live-bed conditions, the sediments were circulated with the water by the pump. At the upstream inlet of the flume a gradual-transition contraction was built to guide the sediment into the main channel.

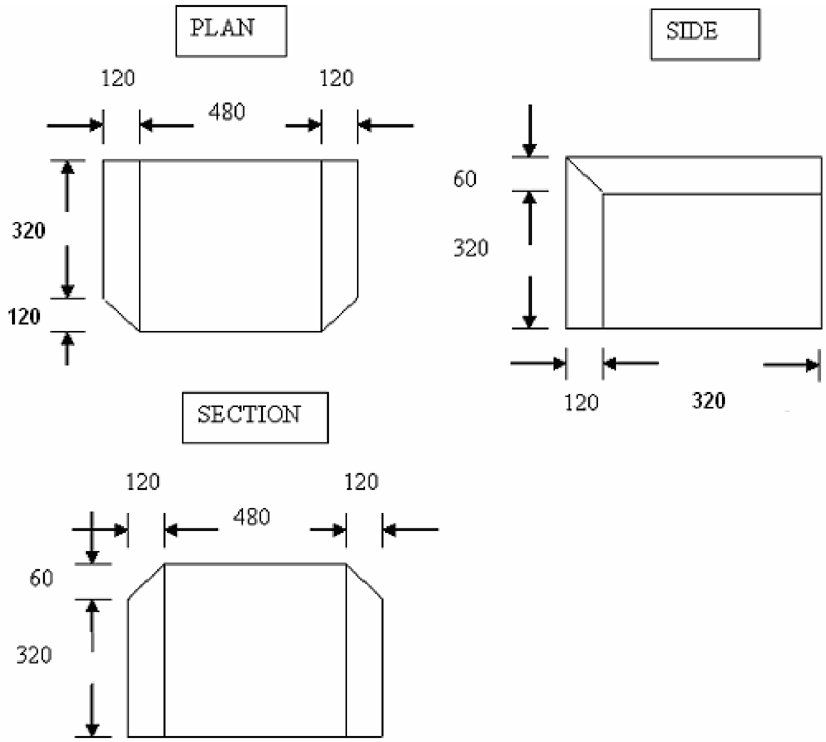


Figure 4-4: Dimensions of Abutment Model (mm)

6.1 U/U_c Ratio

For all clear-water scour experimental conditions herein, a U/U_c ratio of 0.9 was used with U being the overall average velocity in the whole cross-section of the compound channel and U_c being the critical velocity of the sediment.

The mean velocity of the flow is given by

$$\frac{U}{u_*} = 5.75\log(\frac{Y_o}{k_s}) + 6.0 \qquad (4\text{-}1)$$

where k_s is the roughness height of the bed.

At the threshold condition,

$$\frac{U_c}{u_{*c}} = 5.75\log(\frac{Y_o}{k_s}) + 6.0 \qquad (4\text{-}2)$$

So for clear-water conditions, where the bed is stable and k_s is constant,

$$\frac{U}{U_c} = \frac{u_*}{u_{*c}} \qquad (4\text{-}3)$$

For our experiments,

$$u_{*c} = 1.995 cm/s = 0.01995 \text{ m/s},$$

Setting $\frac{U}{U_c} = \frac{u_*}{u_{*c}}$ = 0.9 for clear-water scour, then

$$u_* = 0.017955 \text{ m/s}$$

Since

$$u_* = \sqrt{gRS} \qquad (4\text{-}4)$$

If flow depth is set, the slope of the flow should be able to be determined. U and U_c can also be determined by selecting $k_s=2D_{50}=1.6$ mm.

Three flows were used with velocity ratios (U/U_C, where U is the average velocity of the flow and U_C is the critical velocity for incipient motion for the bed material) of 0.9, 1.5, and 2.3. The critical velocity of the bed material was calculated using the velocity distribution relation for a rectangular cross-section, rough wall, and free surface as shown in Equation (4-2) above.

For clear-water conditions (U/U_C=0.9), the experiments were run for 4800 minutes so that the local scour had reached a near equilibrium value. For live-bed conditions (U/U_C=1.5 and 2.3), all experiments were run for 3000 minutes to assure that at least 125 bed forms migrated past the abutment.

6.1 Instrumentation

Velocity profiles were collected 15 m downstream of the inlet tank with a 2-mm outside diameter total head tube mounted on a point gage at the channel centerline. Flow rate in the flume was measured using a pressure transducer connected to a Venturi meter in the return pipe. Flow depth was controlled by the volume of water in the flume and measured by taking the difference in elevation between the bed and water surface over a 12-m long transect in the approach section. Water surface and bed surface profiles were collected using two acoustic distance measurement devices, the remote measurement unit (RMU) that operates in air and the BASIS (bed form and sediment information system) that operates underwater. These instruments were mounted on an instrument carriage, which traveled on rails over the channel. The instrument carriage was equipped with a computer-controlled 3-axis precision positioning system which allowed transects of the scour hole to be automatically collected using the BASIS. The bed elevation of the area in the vicinity of the abutment was measured at the completion of the clear-water experiments using the BASIS. For the live-bed experiments, the bed elevation of a 2.5-m long flow parallel transect from 13 to 50 mm from the abutment (depending on the size of the bed forms) was measured continuously for 125 min after the scour reached equilibrium state. The probe of the BASIS detects the bed elevation once every minute at a certain point along this transect. The distance between two successive points detected along the transect by the probe is about 1.5 cm. The time-averaged and instantaneous scour depth adjacent to the abutment were determined from this record. Flow depth was also measured and checked using the point gage.

6.1 Experimental Procedure

1. Place the abutment model (with or without countermeasure models) in the flume.
2. Level the sediment bed surface with the scraper blade mounted on the carriage that rode on the rails.
3. Wet and drain the flume completely.
4. Collect the profile of the bed surface using the RMU.
5. Fill the flume with water and obtain the desired depth.
6. Collect an initial set of transects of the scour region around the abutment using the BASIS program.

7. Set the predetermined flume slope and start the pump.
8. Adjust the pump speed to obtain uniform flow at the selected flow depth using the point gages by measuring the water surface elevation at both ends of the 12-m transect in the center of the channel. Check further the water surface slope along the 12-m transect using the RMU device to ensure the uniformity of the flow. Maintain the same rate of flow and approach depth for the entire experimental run.
9. Collect transects of the scour region at 30 min intervals. Increase this time interval to 60 min to 90 min or more as the experiment progresses and changes in the scour region become slower.
10. Continue the experiment until the changes in the scour hole become very slow (approximately 80 hours).
11. Stop the pump, dewater the flume carefully and contour the scour hole.
12. Take a photo of the scour hole.

CHAPTER 5. BASELINE EXPERIMENT

6.1 Clear-water Scour Baselines

6.3.1 Experimental Results

In order to study the efficiency of a certain type of countermeasure in preventing scour at the bridge abutment, the baseline scour, i.e., the scour depth at the bridge abutment without any countermeasures was determined first as a reference. Several runs of experiments were carried out under clear-water scour conditions with a velocity ratio of 0.9 to determine the worst scour scenario at the bridge abutment. These experiments were done with varying flow depths both in the floodplain and in the main channel. The floodplain was first roughened only with sand of the same size as the bed material and later was further roughened with staggered gravel. The gravel was used to simulate a rough floodplain and had a mean diameter of 4.5 cm. The placement of the gravel is shown in Figure 4-3. Table 5-1 is the experimental results of these baseline tests.

Table 5-1. Baseline clear-water experimental results with U/U_c=0.9.

Run No.	t (min)	Q (m^3/s)	y_f (cm)	y_m (cm)	h_1 (cm)	$d_{max.up.abut}$ (cm)	$d_{max.dn.abut}$ (cm)	$d_{max.ch}$ (cm)	Floodplain roughness
B1	4800	0.0366	4.5	13.2	8.7	3.60	3.83	4.00	Sand (0.8mm)
B2	2920	0.0335	1.2	9.9	8.7	5.00	--	--	Sand (0.8mm)
B3	4800	0.0387	5.2	13.2	8	5.30	3.91	7.00	Sand (0.8mm)
B4	4800	0.0353	3	11	8	4.70	3.44	4.00	Sand (0.8mm)
B5	4800	0.0387	5.2	13.2	8	7.77	2.90	1.50	Sand (0.8mm) plus staggered gravel (Fig. 5-2)

Note: t = Run time; Q = Total discharge; y_f = Flow depth on floodplain; y_m =Flow depth in main channel; h_1 = Main channel bank height; $d_{max.up.abut}$ = Scour depth at upstream corner of abutment; $d_{max.dn.abut}$ = Scour depth at a short distance downstream of the downstream corner of the abutment; $d_{max.ch}$ = Scour depth in the channel away from the abutment.

6.3.2 Discussion of Clear-water Baseline Experimental Results

5.1.2.1 Scour pattern

Two scour patterns were discovered depending upon the difference of the roughness on the floodplain. Without gravel on the floodplain the scour pattern of Fig. 5-1 occurred in which there were five scour locations. Tests B1 ~B4 (not roughened with gravel) had similar scour patterns. The first scour hole, Zone A, was at the upstream corner of the abutment and posed the greatest threat to the stability of the abutment. Scour in Zone B was located some distance away from the abutment face in the bridge crossing and was where the maximum scour hole was located. Since it was away from the abutment, it was considered to not threaten the abutment. Scour in Zone C was a short distance downstream of the abutment. This scour zone may pose a threat to the main channel bank immediately downstream of the abutment. Scour in Zone D was far out into the main channel and, therefore, posed no threat to the abutment. Scour in Zone E was located at a short distance upstream of the abutment corner and seemed to be the initial part of Scour Zone B. With gravel on floodplain Test B5 had a slightly different scour pattern as shown in Fig. 5-2, where scour Zones A, B and E merged to be the maximum scour location and was located at the upstream corner of the abutment.

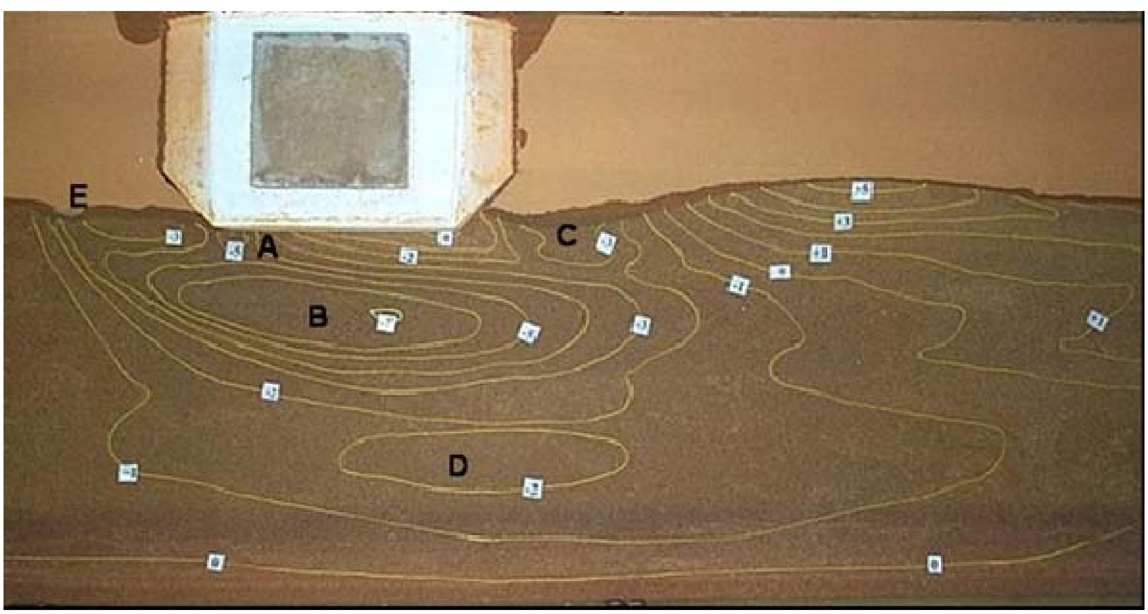


Figure 5-1: String contour of baseline Test B3. Flow is from left to right.

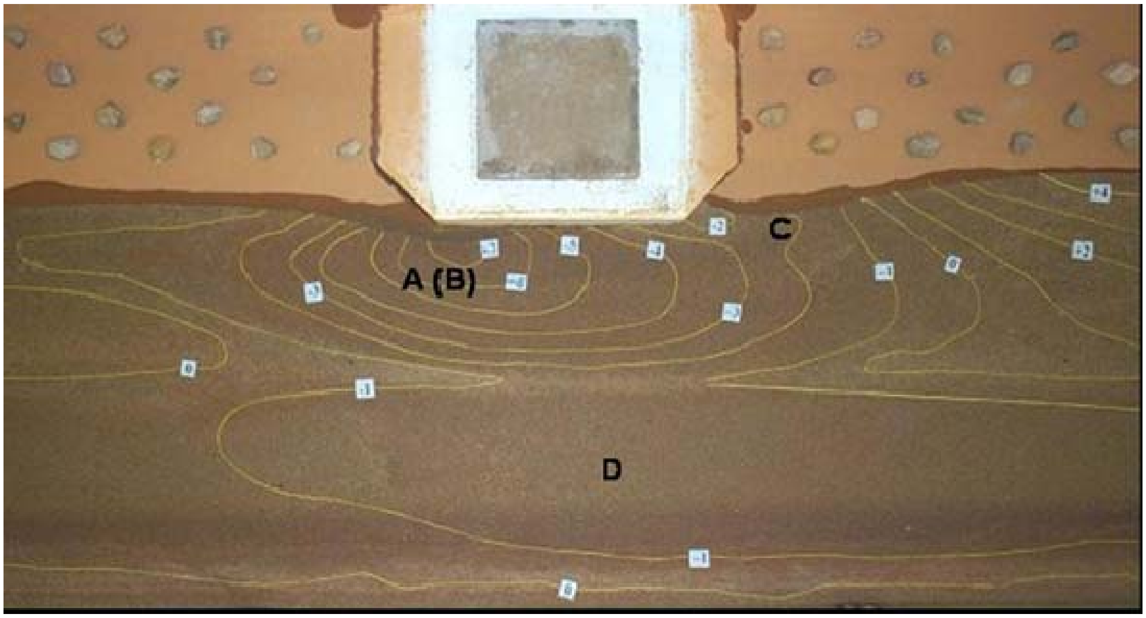


Figure 5-2: String contour of B5 with gravel on the floodplain. Flow is from left to right.

5.1.2.2 Scour Mechanism

In the base-line experiments, it was found that each of the scour zones identified in Tests B1~B5 was formed by different mechanisms. In Zone A scour was caused by a combination of a downward roller from the water striking the upstream abutment corner, return flow from the floodplain flowing down towards the main channel bed, and vortex shedding from the upstream abutment corner. In Zone B scour was caused by a secondary vortex oriented horizontally and parallel to the streamwise flow direction. Scour in Zone C was caused by the wake vortex induced by flow separation. Scour in Zone D was caused by an increase in main channel velocity above the critical value for sediment movement caused by abutment-induced contraction scour. Scour in Zone E was simply the initial part of scour Zone B and was caused by the flow coming down from the floodplain into the main channel. A general scour pattern was shown in Fig. 5-3.

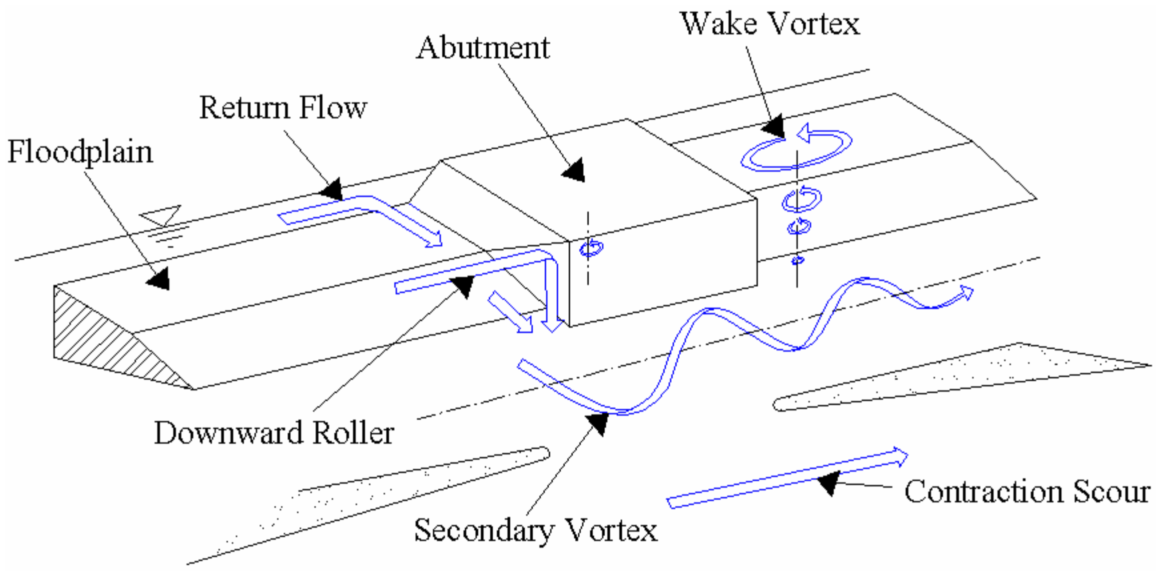


Figure 5-3: Flow patterns around a wing-wall abutment. Flow is from left to right.

5.1.2.3 <u>Effect of Floodplain Roughness on Scour Depth at Upstream Corner of the Abutment</u>

The mechanisms of the formation of the four scour holes in Test B1~B4 were explained above. Also, it was seen that the maximum scour depth was found in the bridge crossing a distance away from the abutment instead of being at the upstream corner of the abutment. It then was hypothesized that the reason the maximum scour depth was not located right at the upstream corner of the abutment was because the velocity ratio between the floodplain flow and the main channel flow was so high that the floodplain flow was able to jet into the main channel a distance away in front of the abutment instead of being fully confined around the abutment corner. To solve this problem with the hope of the maximum scour depth taking place right at the upstream corner of the abutment, the floodplain was further roughened with gravel of average diameter 4.5 cm as shown in Fig. 4-2. As was expected, the result in Fig. 5-2 showed that the scour in Zones A, B and E merged and the maximum scour hole was found right at the upstream corner of the abutment.

5.1.2.4 Effect of Main Channel Height on Scour

The shooting effect must also be caused by the main channel bank height together with a relatively high velocity ratio between floodplain and main channel. In another words, the main channel bank height is also an important parameter that affects the abutment scour pattern in a Type II (Melville, 995) abutment configuration. When the bank height approaches zero, the scour pattern turns to be Type I scour, which is abutment scour in a rectangular channel. In this case, no matter in what range the velocity ratio between the floodplain and the main channel is, the maximum scour will happen around the upstream corner of the abutment, because the approach flow obstructed by the protrusion of the abutment always makes contact with the bed around the abutment corner while it enters the bridge crossing. Therefore the downflow and secondary vortex will exert significant shear stress on the bed and cause scour. When the bank height increases, the flow coming from the floodplain must travel a distance before it hits the bed in the main channel after it enters the bridge crossing. In the process, the flow may avoid contacting the corner of the abutment or at least reduces the extent it otherwise does to the bed at the upstream corner of the abutment if the transverse horizontal distance the flow travels from the floodplain edge before it fully hits the bed in the main channel is greater than the horizontal distance from the floodplain edge to the corner of the abutment. In Fig. 5-1 it was shown that when the velocity ratio between the floodplain and the main channel flow was relatively high, the floodplain flow came off the floodplain edge from a distance away upstream of the upstream corner of the abutment (Point E) and made full contact with the main channel bed at point B where the maximum scour hole was found. However, In Fig. 5-2 it was shown the flow was fully constricted at the upstream corner of the abutment.

5.1.2.5 Evolvement of Scour Holes

Clear-water data of all runs indicated that the upstream scour holes (A and B in Figure 5-1 or A(B) in Figure 5-2) develop faster at the beginning of the experiment than the downstream scour holes (C in Figure 5-1 or Figure 5-2) because the strength of the vortex systems at the upstream corner of the abutment were generally stronger than those at the downstream end of the abutment. Therefore, the upstream scour hole reaches equilibrium quicker than the downstream scour hole. Depending on the height of the main channel bank and flow conditions, the depth of the downstream scour hole may or may not be greater than the scour depth at the upstream corner of the abutment. Figures 5-4 and 5-5 show the evolution of the scour depths of both the upstream and downstream scour holes with time for Test B1 and B4, respectively.

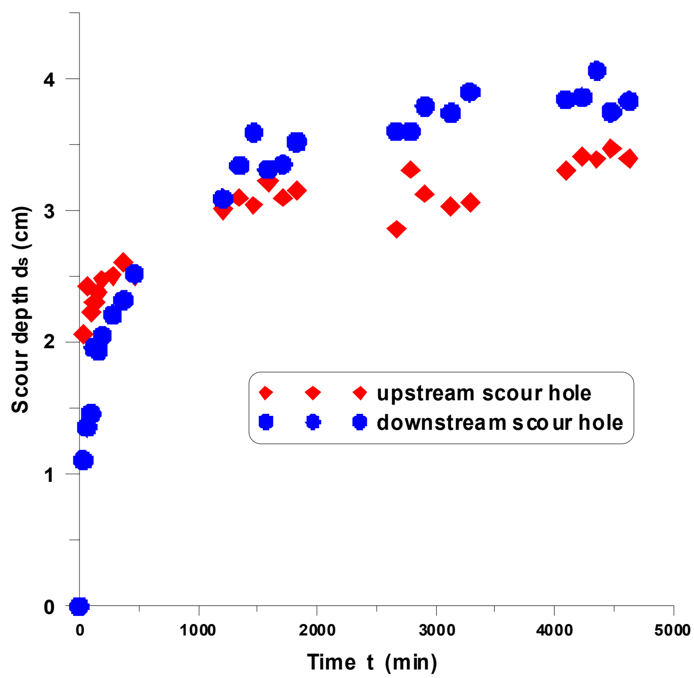


Figure 5-4: Plot of the time evolution of the scour depth of both the upstream (A) and downstream (C) scour hole at the abutment in Test B1.

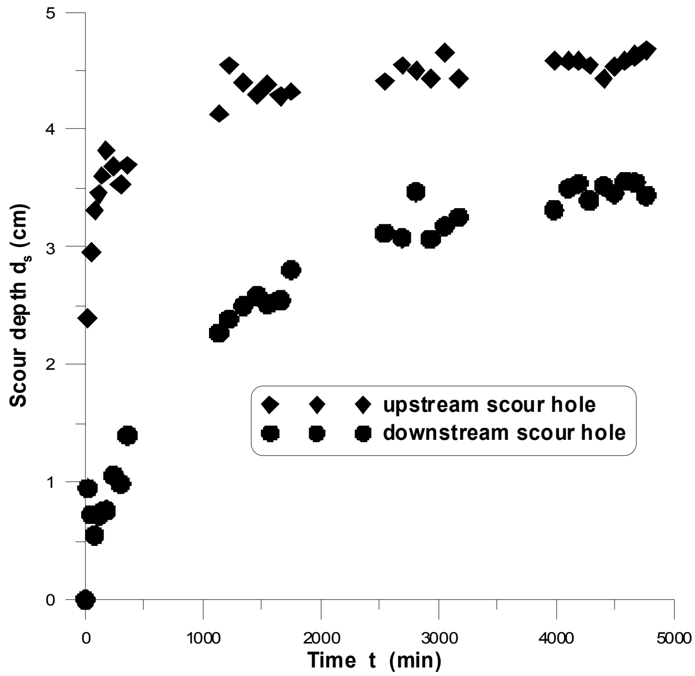


Figure 5-5: Plot of the time evolution of the scour depth of both the upstream (A) and downstream scour hole (C) at the abutment in Test B4.

5.2 Live-bed Scour Baseline Experiments

When the velocity of the flow gets higher and the velocity ratio or the shear stress ratio of the flow exceeds one, then the bed materials of the river begin to move and bed forms are initiated. Under live-bed conditions in a certain range of velocity ratios, although it has been concluded that the local scour at the bridge abutment is less than what it is under critical condition ($U/U_c = 1$) provided that all other parameters are the same, it is believed that the fluctuating bed forms and the higher shear stress may pose more threat and challenge to the stability and practicability of the countermeasures. Therefore, live-bed experiments should be implemented to test those successful countermeasures in clear-water scour to confirm their efficiency in other river scour conditions.

In order to study further the efficiency of those countermeasures tested in clear-water condition under live-bed conditions, the live-bed baseline, i.e., the scour depth at the bridge abutment without any countermeasures should be determined first as a reference. The live-bed baseline scour results were obtained under the flow conditions shown in Table 5-2.

Table 5-2. Experimental results for baseline scour depth for three velocity ratios.

Velocity ratio U/U_c	**Flow depth in main channel y_m (cm)**	**Flow depth in flood plain y_f (cm)**	**Total discharge Q (m^3/s)**	**Run time, t (min)**	**Time-averaged baseline scour depth $d_{abut,avg}$ (cm)**	**Instantaneous maximum scour at abutment $d_{max,abut,inst}$ (cm)**
0.9	13.2	5.2	0.0387	4800	7.77	7.77
1.5	13.2	5.2	0.0622	3000	7.23	15.00
2.3	13.2	5.2	0.0966	3000	7.52	17.28

In the live-bed condition, due to the wavy water surface and the fast moving and fluctuating bed forms, velocity profile measurements turned out to be difficult. Therefore, the flow was mainly controlled by the discharge and the average water surface profile and the average bed profile along a 12-m transect in the middle of the approach channel. First the discharge was set to be 1.5 times the discharge at the critical condition, and then the slope was adjusted until the average water surface slope and average bed surface slope are equal to the flume slope, i.e., the uniform live-bed flow condition through trial and error. The flow depth was also adjusted to be 13.2-cm deep as it was in the clear-water case.

Also, due to the fast change of the bed profiles at the bridge crossing, it is impractical to measure the bed profile as it was done in clear-water scour condition, which took 14 transects and more than 13 minutes to cover the whole scour region as well as each point once. Therefore, in this case, only one 2.5 m long transect at the main channel side of the abutment was chosen to monitor the time evolution of the scour at the edge of the abutment. This transect starts from a point 1.5 m upstream of the upstream abutment tip and travels parallel to the flow just to the right of the abutment. Each loop of the transect takes 54.90 seconds and a total of 133 loops is set to capture the local scour as well as the bed forms along this transect in a time range of 2 hours. The data obtained from this transect enables us to see where the maximum local scour takes place at the abutment and how it evolves with time.

The bed form shape across the channel is far from uniform. Close to the opposite wall where these is no floodplain, the bed form amplitude is relatively small; close to the floodplain the amplitude of the bed forms is relatively greater than it is at the opposite wall.

Figure 5-6 is a plot of the time-averaged local scour depth along the 2.5-m transect at the abutment versus distance starting from a point 1.5 m upstream of the upstream abutment corner for the 1.5 velocity ratio case. Figure 5-7 is a plot of the time evolution of baseline scour at the upstream abutment corner versus time after the scour reached equilibrium.

From Figure 5-6 it can be seen that the maximum time-averaged local scour still took place at the upstream corner of the abutment with a scour depth of 7.23 cm. This agreed with the baseline scour pattern in the clear-water scour condition. The scour depth dropped to 4.01 cm as it approached the downstream end of the abutment. Deposition began to occur at a point 45 cm downstream of the downstream abutment end.

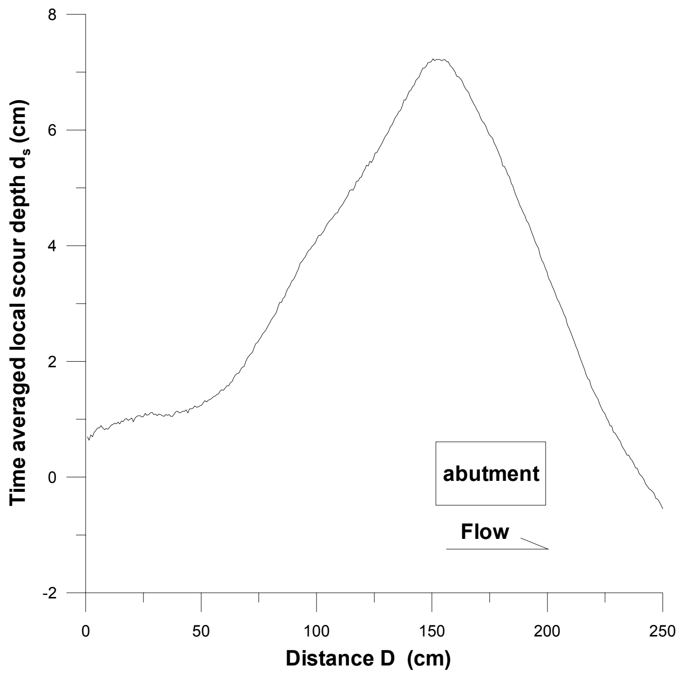


Figure 5-6: Time-averaged local scour depth along the 2.5-m transect at the abutment versus distance starting from a point 1.5 m upstream of the upstream abutment corner.

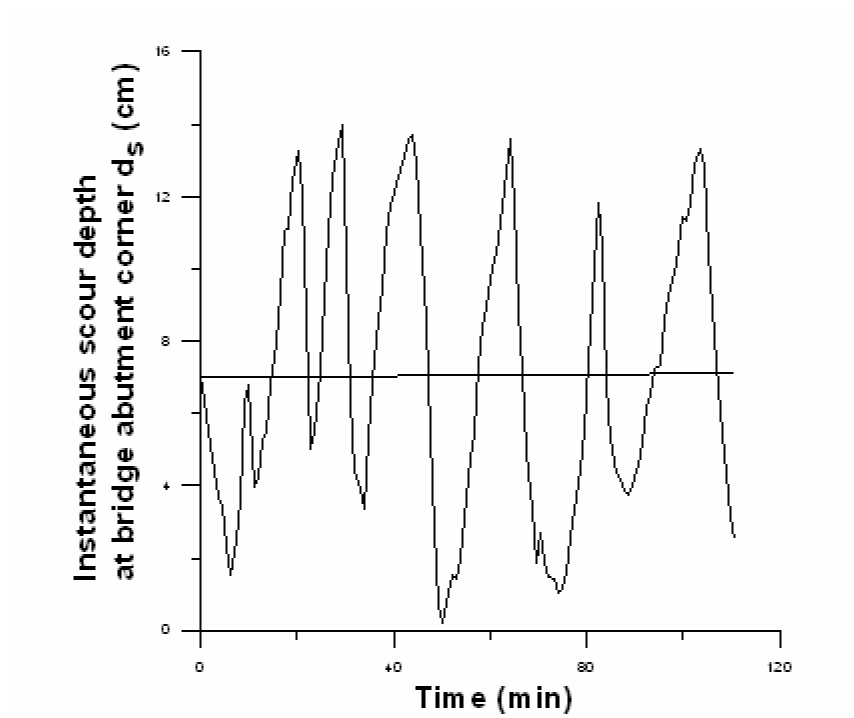


Figure 5-7: Time evolution of baseline scour at the upstream abutment corner after equilibrium was reached.

From Figure 5-7 it is seen that the scour depth at the upstream abutment corner varied from near 0 to 14 cm with time. However, it varied about a mean value of 7.23 cm and had fluctuation amplitude of about 7 cm. The fluctuation was mainly due to the bed forms. When the crest of bed forms came, the scour depth reached its minimum value, which is a few millimeters above zero; when trough of bed forms came, the scour depth reached its maximum value.

5.3 Conclusions

Through these baseline experiments in a compound channel with various flow depths on the floodplain and main channel, various roughnesses on floodplain, and various bank heights, it was found that:

Under clear-water conditions:

1. Five locus of scour were found as shown in Fig. 5-1 in the whole scour region provided the roughness on the floodplain was the same as it was in the main channel and the velocity ratio between the floodplain flow and the main channel flow was relatively high. The floodplain flow tended to shoot into the main channel at a distance away upstream from the upstream corner of the abutment instead of being fully constricted at the abutment corner. Under this condition, the maximum scour in the whole region was normally found in Zone B (Figure 5-1).
2. The more the floodplain flow was constricted at the abutment, the greater the scour hole would be at the upstream corner of the abutment. By increasing the roughness on the floodplain and decreasing the velocity ratio between the floodplain and the main channel the floodplain flow was able to be fully constricted around the corner of the abutment. As a consequence, the scour zones A, B and E were incorporated together and the maximum scour depth was found at the upstream corner of the abutment.

3. The principal vortex systems and secondary vortex systems at the upstream corner of the abutment were stronger than the wake vortex systems at the downstream corner of the abutment. Consequently, the scour hole induced by the principal vortex systems and the secondary vortex systems reaches equilibrium quicker than the scour hole induced by the downstream wake vortex systems.
4. Test B5 will be used as the clear-water baseline condition for following countermeasure experiments with a scour depth of 7.77 cm as a reference.

Under live-bed conditions:

1. Maximum scour took place at the upstream corner of the abutment. Time-averaged scour depth at the upstream corner is less than the scour depth under critical clear-water condition, while instantaneous scour depths were between values near zero to values nearly twice the maximum scour under clear water flows because of the superposition of the trough of bed forms (Figure 5-7);
2. Live-bed scour reaches equilibrium quicker than clear-water scour.

Results in Table 5-2 will be used as the references to evaluate the efficiency of countermeasures that will be tested later.

CHAPTER 6. PARALLEL WALL AS AN ABUTMENT SCOUR COUNTERMEASURE

6.1 Introduction

Scour at an abutment can cause damage or failure of bridges and result in excessive repairs, loss of accessibility, or even death. Scour mitigation at bridges has received much attention in the past few decades. Hydraulic countermeasures against bridge abutment scour can be classified as either river training structures or armoring countermeasures. Considerations in choosing the appropriate method of mitigation, other than design constraints, include maintenance and inspection requirements, enhancement of the physical environment, and constructability. Design specifications for many of these scour mitigation techniques can be found in Hydraulic Engineering Circular 23 (Lagasse et al. 2001).

Guide banks are earth or rock embankments placed at abutments to improve the flow alignment and move the local scour away from the embankment and bridge abutment. The guide bank provides a smooth transition for flow on the flood plain to the main channel. The major use of guide banks in the United States has been to prevent erosion by eddy action at bridge abutments or piers where concentrated flood flow traveling along the upstream side of an approach embankment enters the main flow at the bridge (Lagasse et al. 2001). There also have been various studies on guide banks. Among those studies are Spring (1903), Karaki (1959, 1961) Neill (1973), Bradley (1978), Chitale (1980), Smith (1984), Richardson et al. (1984), and Lagasse et al., (2001). Guide bank orientation, length, crest height, shape and size, downstream extent and many other concerns are investigated. Design guidelines for guide banks are given in Neil (1973), Bradley (1978), Ministry of Works and Development (1979), Central Board of Irrigation and Power (1989) and Lagasse et al. (1996, 1999, 2001).

However, despite the design guidelines and studies in the literature, there are still issues remaining to be dealt with for certain types of bridges in certain environments. For instance, for small county bridges where wing-wall abutments prevail and terminate on the riverbanks, specific design guidelines have not been developed. Guide banks have been specifically designed for spill-through abutments on rivers with wide floodplains where the guide bank can be designed such that the slope of the guide bank is tangent to the slope of the abutment and there is no protrusion of the abutment into the flow beyond the slope of the guide bank. However, in a wing-wall case, this point may not be achieved readily because of the vertical front faces of the abutments. In this situation either the slope of the guide bank protrudes out beyond the abutment face or the abutment face protrudes out beyond the guide bank slope. The impacts of these configurations on local scour at abutments need to be studied. Another issue is that a careful review of those guidelines (Bradley 1978) for determining the length of guide banks shows that they were designed for spill through abutments in wide floodplain rivers and they may not apply to smaller county bridges. There are many gray areas that were not addressed and may be important for small county bridges. For instance, first, it is recommended that if the length read from the design chart is less than 9.1m (30 ft), a guide bank is not needed. This might not be true for a small two lane bridge whose width is about 9 m and a 9 m long guide bank may make a great difference in protecting the bridge abutments; second, it is recommended that for chart lengths from 9 m to 30 m, a guide bank no less than 30 m long be constructed. However, according to Herbich (1967), the length of the guide banks appear to be unimportant in reduction of velocities provided it is greater than a certain minimum length. Therefore, an unnecessarily long guide bank may increase the cost of the structure and not improve its effectiveness. Yet another issue is that the parameters defined and used in determining the length of guide banks may not be easily available. For instance, the total stream discharge, Q, the lateral or floodplain flow discharge Q_f and the discharge in the 100 feet of stream adjacent to the abutment, Q_{100} (Bradley, 1978). In addition, in spite of the fact that an elliptical-shaped end seems to be favorable by all design recommendations because the curved head can direct the flow smoothly into the main channel and reduce scour at the guide bank end, for small county rivers and streams whose floodplains are relatively narrow and are mostly farmlands under cultivation, the floodplain flow velocity may be relatively low and a curved head may not be justified. Most importantly, for those abutments terminating on the riverbanks, a curved end stretching out from the bank into the farmland may be aesthetically and practically not acceptable. This chapter deals with design issues for parallel walls on small rivers with wingwall abutments. These parallel walls are essentially scaled-down and simplified versions of guide banks. This work fills a need for low-cost countermeasures for small bridges with wingwall abutments.

6.2 Conceptual Model

Figure 6-1 is a sketch of the conceptual model of parallel wall as countermeasure against abutment scour in a compound channel. A wall is attached at the upstream end of the abutment and is parallel to the flow direction. A wall thus installed is believed to have the following functions as conceptualized in the figure.

First, it can push the scour inducing downflow and secondary vortex upstream away from the abutment so that that scour won't be happening any more at the upstream corner of the abutment provided that the length of the wall is long enough; second, it can create a slow-moving or dead water zone behind itself on the floodplain. In the case where there is no dead water zone, the return flow from the floodplain would flow along the roadway and bridge abutment embankment towards the main channel causing embankment scour. The presence of this wall and thus the dead water zone helps slow down the scour and erosion of the embankment; third, the wall helps straighten and improve the flow through the bridge crossing.

Bearing this in mind, the wall length and different wall materials (solid, rock) will be tested as follows.

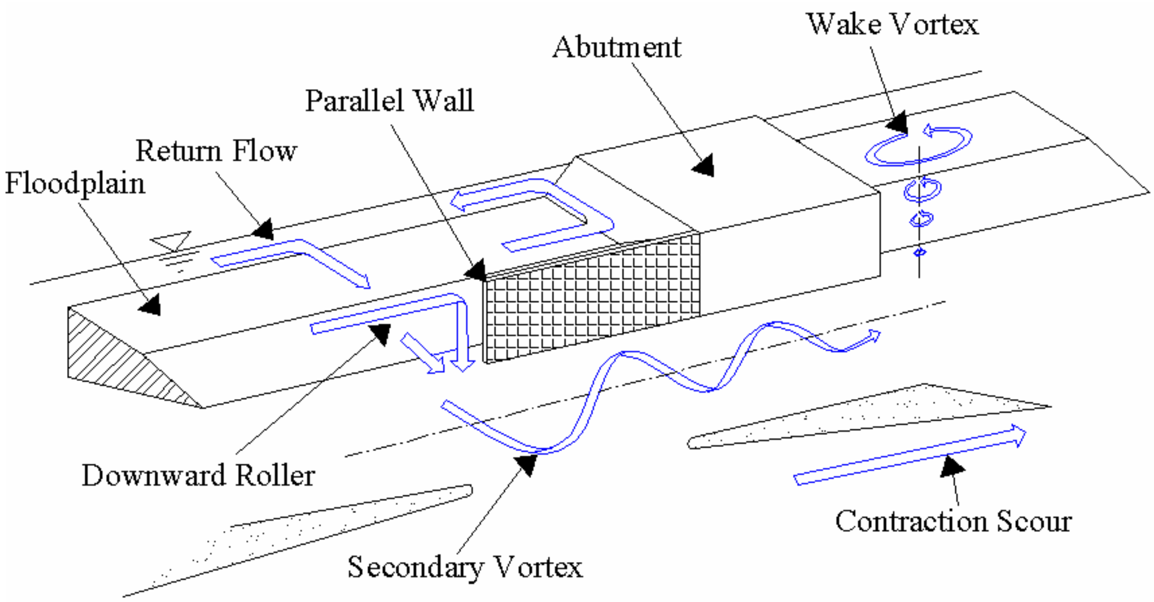


Figure 6-1: A sketch of the conceptual model of parallel wall as countermeasure against abutment scour in a compound channel.

6.3 <u>Results</u>

6.3.1 <u>Solid Parallel Wall</u>

A series of rectangular solid walls made from 13-mm thick plywood of different lengths, L_s, attached to the upstream end of the abutment and parallel to the flow direction were tested (Fig. 6-1). Solid walls are tested because in certain areas rocks may be not readily available and cost efficient. All the solid parallel walls were seated at the bottom of the bank slope and aligned with the abutment face parallel to the flume wall. The top of each wall was the same height as the top of the abutment except in one clear-water case, in which the wall height was 5.2 cm lower than the water surface. In these cases, the flow depth on the floodplain, y_f, was equal to 5.2 cm and the flow depth in the main channel, y_m, was 13.2 cm. The velocity ratio U/U_c was about 0.9, 1.5 and 2.3 in the center of the entire channel for each of the three flow conditions tested. Table 6-1 gives results of the solid wall experiments under clear-water conditions and Table 6-2 gives results under live-Bed conditions. Figure 6-2 is a string contour of the 1.2L solid wall run in

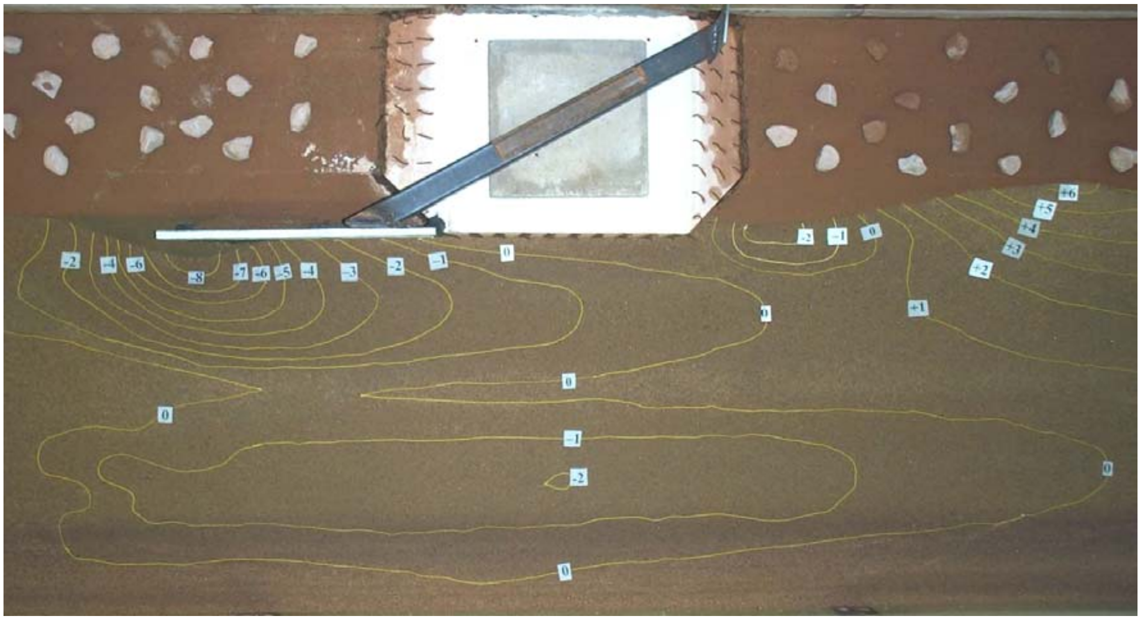


Fig. 6-2: A string contour of the 1.2L solid wall run in clear-water scour. y_m = 13.2 cm, y_f = 5.2 cm, Q = 0.0379 m^3/s, time = 4800 minutes. Flow is from left to right. Contour interval is 1 cm.

Table 6-1: Solid wall experimental results for clear-water scour (run time=4800 min., Q=0.0379 $\pm$ 0.003m^3/s, U/U_c=0.9).

Solid length, Ls	**Maximum scour depth at abutment, d_s (cm)**	**Scour reduction rate (%)**	**Maximum scour at the countermeasure, d_c (cm)**
0.3L, rectangular	6.25	19.6	8.65
0.5L, rectangular	4.01	48.4	8.10
0.6L, rectangular	2.95	62.0	7.71
0.7L, rectangular	2.15	72.3	7.83
0.8L, rectangular	1.43	81.6	7.62
1L, rectangular	0.33	95.8	7.71
1.2L, rectangular (Fig. 6-2)	-0.40	105.1	8.30
1L,submerged, top even with floodplain	4.00	48.5	5.50

Table 6-2: Solid wall experimental results for live-bed scour (run time=3000 min., Q=0.0619 ± 0.0015m^3/s for U/U_c=1.5, and Q=0.0619 ± 0.0015m^3/s for U/U_c= 2.3, all walls were rectangular shaped and emergent).

L_s	U/U_c	$d_{abut,avg}$ (cm)	$\%_{max,abut,\ avg}$	$d_{max,abut,inst}$ (cm)	$\%_{max,abut,inst}$	$d_{cm,avg}$ (cm)	$d_{max,cm,inst}$ (cm)
0.6L	1.5	5.05	30	12.98	13	7.57	14.36
0.9L	1.5	4.04	44	9.56	36	7.9	15.86
1.2L	1.5	2.57	63	10.12	33	7.74	13.81
1.5L	1.5	0.22	92	5.49	63	7.86	14.20
1.6L	2.3	2.23	70	8.84	49	9.21	16.43
1.9L	2.3	3.57	53	8.94	48	9.82	16.73

L_s = wall length, U = average velocity in main channel, U_c = critical velocity for sediment movement, $d_{abut,avg}$ = time-averaged scour depth at abutment, $\%_{max,abut,\ avg}$ = percent reduction in time-averaged scour depth at abutment, $d_{max,abut,inst}$ = maximum instantaneous scour depth at abutment, $\%_{max,abut,inst}$ = percent reduction in maximum instantaneous scour depth at abutment, $d_{cm,avg}$ = time-averaged scour depth at the countermeasure, $d_{max,cm,inst}$ = maximum instantaneous scour depth at the countermeasure

clear-water scour where y_m = 13.2 cm, y_f = 5.2 cm, Q = 0.0379 m^3/s, time = 4800 minutes. Flow is from left to right. Contour interval is 1 cm.

6.3.2 Discussion of Solid Wall Length

Fig. 6-3 is a plot of the maximum scour depths at the abutment and the maximum scour depth in the vicinity of the upstream end of the wall versus the length of the wall in terms of the abutment length, L, for both clear-water (U/U_c =0.9) and live-bed (U/U_c =1.5) experiments. It is seen that, as the length of the wall increases from 0.3L to 1.2L, the scour at the abutment decreases rapidly. There is no scour at the abutment corner in 4800 minutes of running when the wall reaches a length of 1.1L.

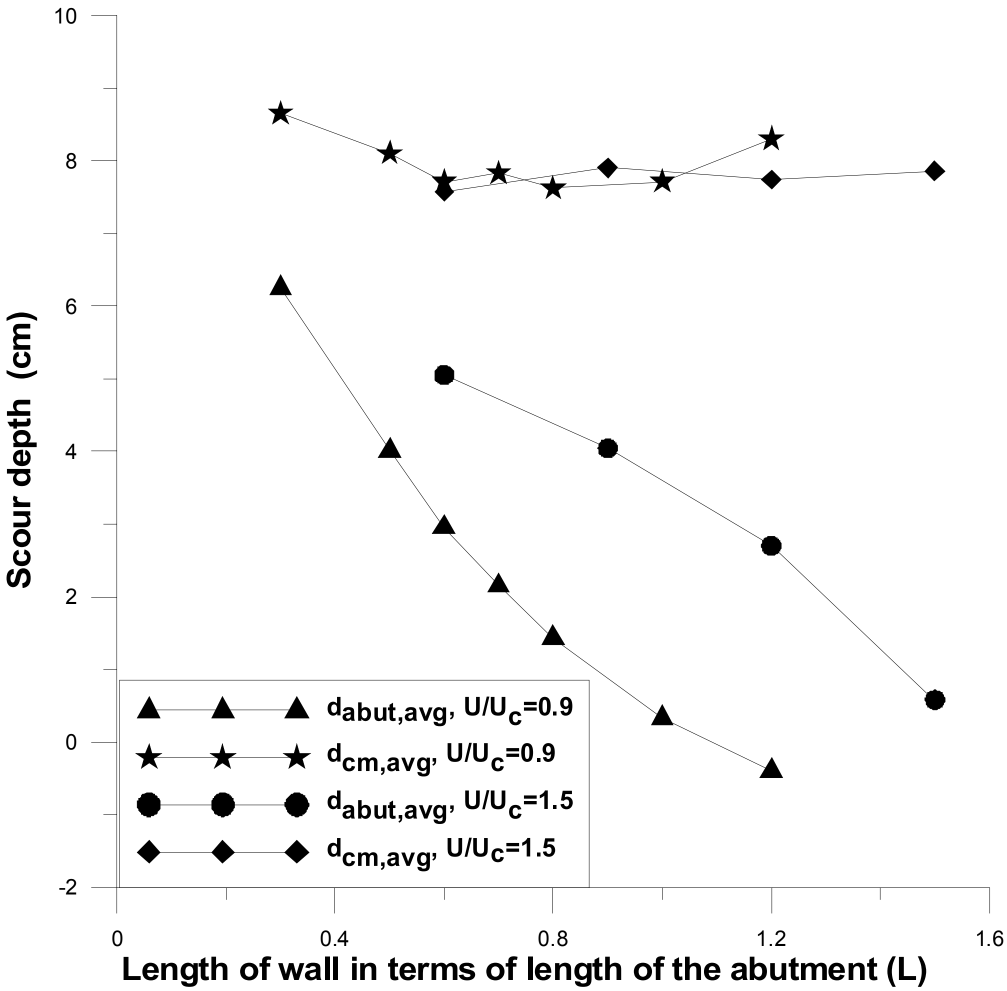


Fig. 6-3: Scour depth at both abutment and upstream end of solid walls Versus length of wall in terms of abutment length for U/U_c=0.9 and 1.5, respectively.

It is also seen from Fig. 6-3 that as the length of the wall increases from 0.6L to 1.5L, the scour at the abutment decreases rapidly for the live-bed case. The average scour depth at the abutment corner tends to zero if the wall length reaches a length of 1.6L. The amplitudes of the bed forms were significant, however. For instance the maximum trough depths of the bed forms in the approaching channel ranged from 9.32 to 11.50 cm. The presence of the solid wall did not affect the dune amplitudes as the bed forms migrated past the wall and abutment. This is why the instantaneous scour depth at the abutment fluctuates about its average value. From the live-bed experimental data it is seen that as the time-averaged scour depth at the bridge abutment changes from 5.05 cm (0.6L case) to 0.22 cm (1.5L case), the maximum contribution from the bed forms to the scour depth varies from 7.93 cm to 4.91 cm. It can be seen that although increases in solid wall length decreased the amplitude of the bed forms, the decreases are not significant, which implies that if the height of bed forms constitutes a large part of the local instantaneous scour depth, it can only be completely eliminated when the presence of the solid wall happens to be able to change the flow condition in the bridge crossing into a transition regime under which the dunes completely disappear and a flat bed with bed material transport is formed. This transition regime may or may not be readily achieved depending on the approach flow conditions and the constriction ratio of the channel.

It was also found from the live-bed experimental data with a velocity ratio of 2.3 that when the length of the wall was increased from 1.6L to 1.9L, the scour reduction rate at the abutment decreased from 70 percent to 53 percent instead of being increased. This may be due to imperfect construction of the floodplain or wall, or it may be that these two scour values are within the range of scatter of the scour data for this high-sediment-transport flow.

In summary, it was found that, in general, walls attached to the upstream end of the abutment were able to move the scour hole upstream from the abutment corner and therefore, were effective as a scour countermeasure. It was also found that, for clear-water scour conditions, as the length of the wall increased, the scour at the abutment declined. In live-bed experiments, however, when the length of the wall becomes longer than a certain length, the scour at the abutment begins to increase.

6.3.3 <u>Parallel Rock Wall Experimental Results</u>

A series of rock walls of different lengths, L_w, and different protrusion lengths, L_p, were tested (Fig. 6-4) both under clear-water and live-bed conditions. This design was thought to be easier to construct in the field and less expensive than a solid wall. In these experiments, the flow depth on the floodplain (y_f) was equal to 5.2 cm and the flow depth in the main channel (y_m) was 13.2 cm. The velocity ratio (U/U_c) was 0.9 in the centerline of the entire channel for clear-water experiments and 1.5 and 2.3 for live-bed experiments, respectively. The top of each wall was the same height as the top of the abutment so that they were not submerged by the flow. The experimental results are tabulated in Table 6-3 and Table 6-4.

Table 6-3: Experimental data of parallel rock walls in clear-water scour (Q = 0.0385 ± 0.003m³/s, t = 4800 min., U/U_c=0.9).

Test No.	Gravel diameter D (mm)	Wall length, L_w (L)	Side slope, S_b	End slope, S_n	Wall protrusion, L_p (W)	Maximum Scour at Abutment, d_s (cm)	Scour Reduction (%)
1 (Fig. 6-5)	6.7~9.5	1.5	18/13.2	30/13.2	0.5	0.2	97
2	6.7~9.5	0.5	18/13.2	30/13.2	0.5	5.21	29
3	6.7~9.5	1.0	18/13.2	30/13.2	0.5	5.06	35
4 (Fig. 6-6)	6.7~9.5	1.5	18/13.2	30/13.2	0.25	0.45	94
5	6.7~9.5	0.5	18/13.2	30/13.2	0.25	5.3	32
6	6.7~9.5	1.0	18/13.2	30/13.2	0.25	2.35	70
7 (Fig. 6-7)	6.7~9.5	1.5	18/13.2	30/13.2	0	1.9	76
8	6.7~9.5	1.0	18/13.2	30/13.2	0	1.95	75
9	6.7~9.5	0.5	18/13.2	30/13.2	0	2	74
10	6.7~9.5	0.25	18/13.2	30/13.2	0	2.8	64
11	6.7~9.5	2.0	18/13.2	30/13.2	0	2	74
12	19~50	1.5	18/13.2	30/13.2	0	0.3	96

From Tests 1(Fig. 6-5), 2, and 3 in Table 6-3 it was found that with a protrusion length of the wall base of 0.5W beyond the abutment face into the main channel, there tended to be a separation zone behind the downstream end of the wall, causing a significant local scour hole. When the length of the wall was 1.5L, the scour hole was relatively small (8.81 cm) and did not pose a direct threat to the abutment. However, when the length of the wall was 0.5L, the scour holes were 12.96 cm and the abutment was highly threatened. These scour holes could pose significant threat to a pier if a pier is located near the abutment.

Table 6-4: Rock wall experimental results in live-bed scour (run time=3000 min., Q=0.0619 ± 0.0015m³/s for U/U_c=1.5, and Q=0.0966 ± 0.003m³/s for U/U_c= 2.3, all walls were rectangular shaped and emergent).

Lw	U/U_c	$d_{abut,avg}$ (cm)	$d_{max,abut,inst}$ (cm)	$\%_{max,abut,\ avg}$	$\%_{max,abut,inst}$
0.5L	1.5	3.19	5.13	56	66
1.0L	1.5	2.59	5.14	64	66
1.5L	1.5	1.87	4.73	74	68
0.5L	2.3	2.23	4.07	70	76

L_w = wall length, U = average velocity in main channel, U_c = critical velocity for sediment movement, $d_{abut,avg}$ = time-averaged scour depth at abutment, $\%_{max,abut,\ avg}$ = percent reduction in time-averaged scour depth at abutment, $d_{max,abut,inst}$ = maximum instantaneous scour depth at abutment, $\%_{max,abut,inst}$ = percent reduction in maximum instantaneous scour depth at abutment, $d_{max,cn,avg}$ = time-averaged scour depth at the countermeasure, $d_{max,cm,inst}$ = maximum instantaneous scour depth at the countermeasure

From Tests 4 (Fig. 6-6), 5, and 6 (Table 6-3) it was found that with a protrusion of the wall base of 0.25W beyond the abutment face into the main channel, there was still a separation zone right behind the downstream end of each wall. However, the scour holes caused by the separation in the 0.25W protrusion cases were smaller than they were in the corresponding 0.5W tests. For instance, the scour hole depths were 4.30cm, 5.67cm and 8.04 cm for 1.5L, 1L and 0.5L, respectively. However, these holes were closer to the abutment due to the reduced protrusion of the wall into the main channel. These scour depths were at the downstream end of the rock wall, while the scour values presented in Table 6-3 were at the upstream abutment corner and, therefore, do not match.

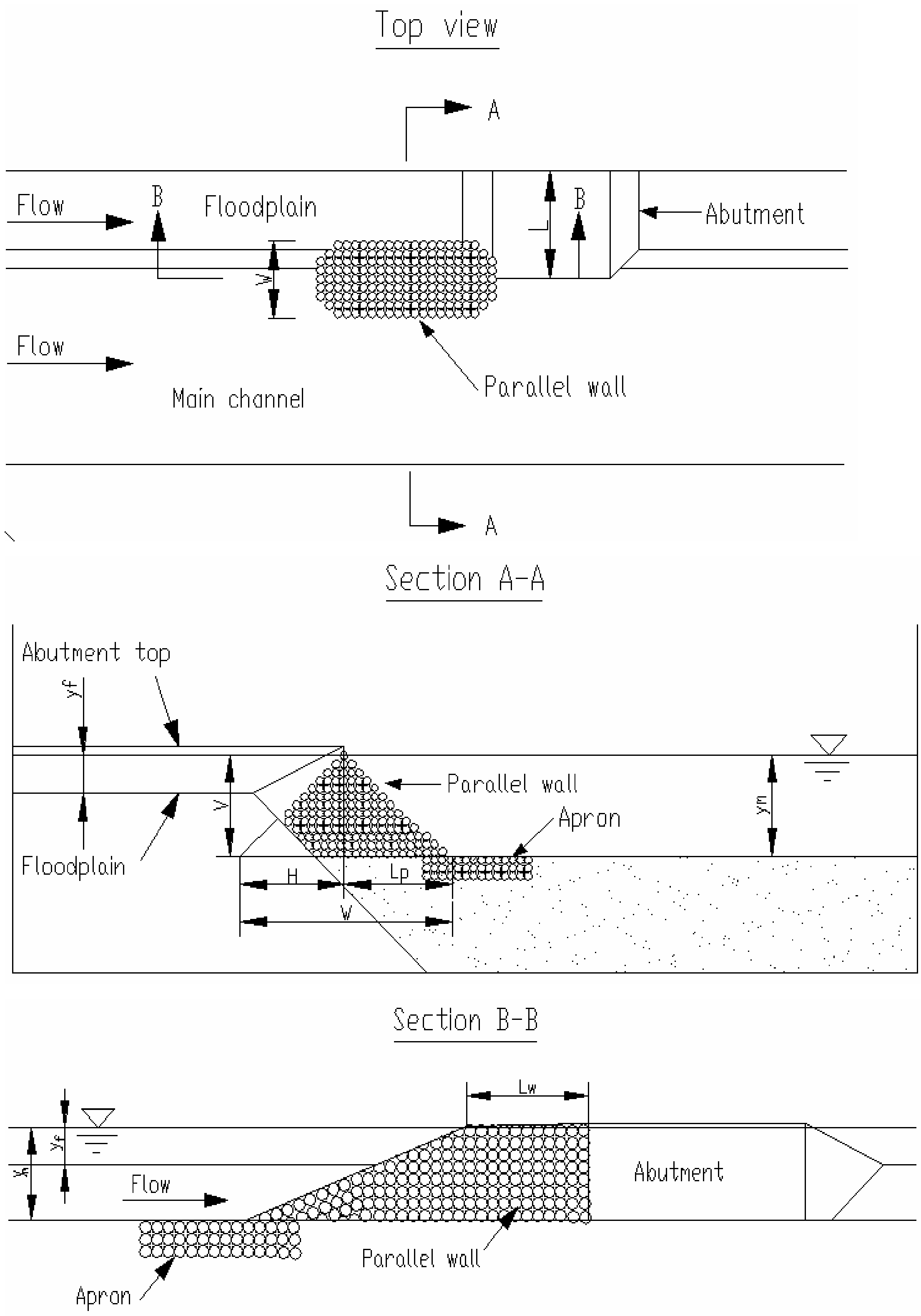


Fig. 6-4: Definition sketch of parallel wall (aprons were only present in live-bed experiments).

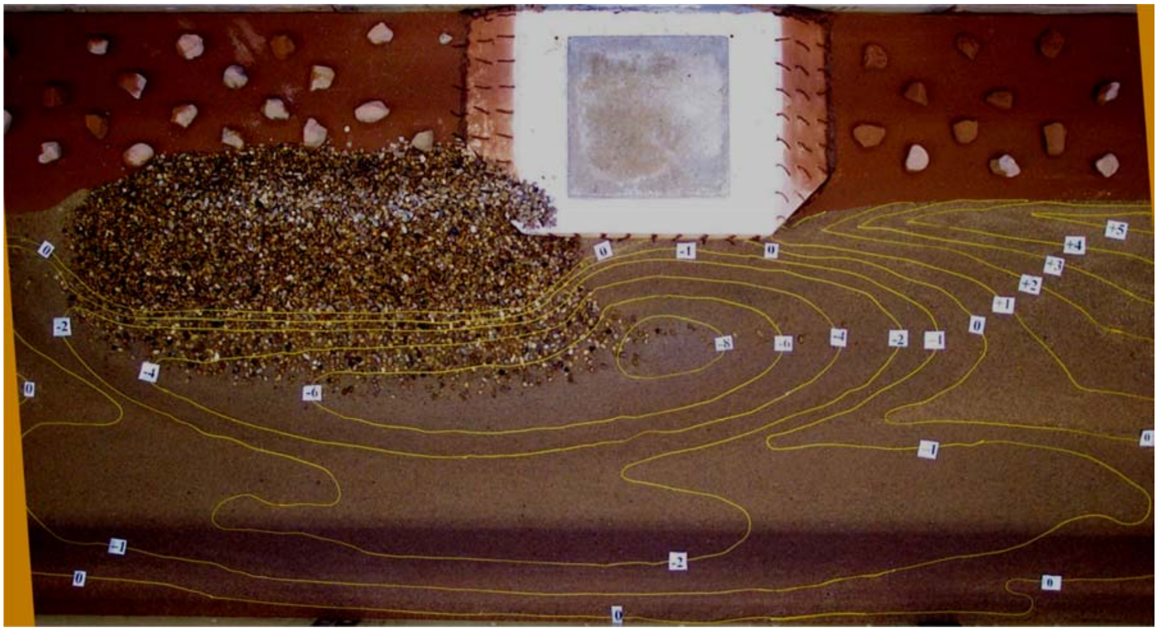


Fig. 6-5: Scour contours of test No 3. in Table 6-3 with gravel diameter 6.7~9.5 mm, 1.5L length, rock wall end slope H:V=30/13.2, side slope H:V=18/13.2, a small apron at the end . Wall base protruded out into main channel from abutment half wall width. y_m = 13.2 cm, y_f = 5.2 cm, Q = 0.0369 m^3/s, t = 4800 min. Flow from left to right.

From Tests 7 (Fig. 6-7), 8, 9, 10 and 11 in Table 6-3 it was found that when the wall base did not protrude beyond the abutment, separation that appeared in 0.25W and 0.5W protrusion cases disappeared. However, the abutment now was partly protruding out beyond the wall slopes, causing constriction of the flow coming from the wall slope. Fortunately, due to the high roughness of the wall slope, near-wall velocity of the flow being constricted was mitigated compared to the flow in the main channel and, therefore, the constriction of the flow did not cause significant scour at the abutment. It was found that the scour depth at the abutment ranged from 1.9 cm to 2.0 cm when the length of the wall was from 0.5L to 2L. The scour depth was about 2.8 cm when the wall was 0.25L long, which indicated that for zero wall base protrusion, the scour depth at the upstream corner was not significantly affected by the length of the wall.

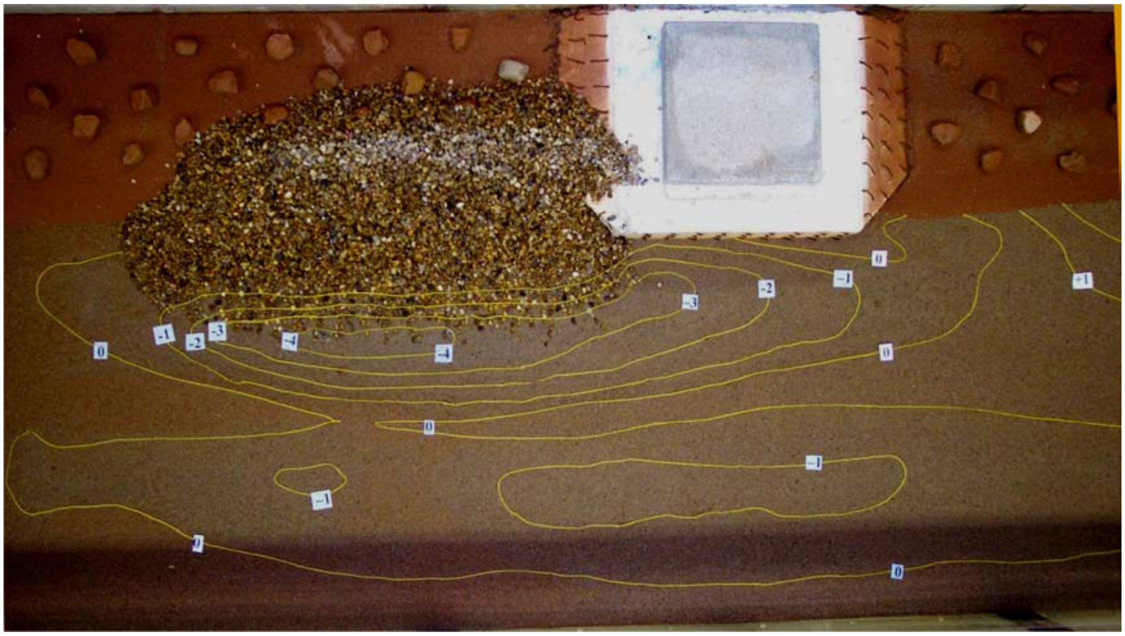


Fig. 6-6: Scour contour of test No 6. in Table 4 with gravel diameter 6.7~9.5 mm, 1.5L length, rock wall end slope H:V=30/13.2, side slope H:V=18/13.2, a small apron at the end . Wall base protruded out into main channel from abutment a quarter wall width. y_m = 13.2 cm, y_f = 5.2 cm, Q = 0.0373 m^3/s, t = 4800 min. Flow from left to right.

Fig. 6-7: Scour contour of Test 9. in Table 6-3 with gravel diameter 6.7~9.5 mm, 1.5L length, rock wall end slope H:V=30/13.2, side slope H:V=18/13.2, a small apron at the end . Wall base is even with abutment. ym = 13.2 cm, yf = 5.2 cm, Q = 0.0371 m3/s, t = 4800 min. Flow from left to right.

6.3.4 Discussion

Fig. 6-8 is a plot of scour depth at the bridge abutment versus rock wall length for different wall protrusion lengths for clear-water experiments. It can be seen from the plot that for protrusion lengths, L_p, of 0.25W and 0.5W, increases of wall lengths can reduce scour at the abutment significantly. However, for the case of no protrusion, increases of wall lengths do not show obvious effects in reducing scour at the abutment except when the wall is less than 0.5 W.

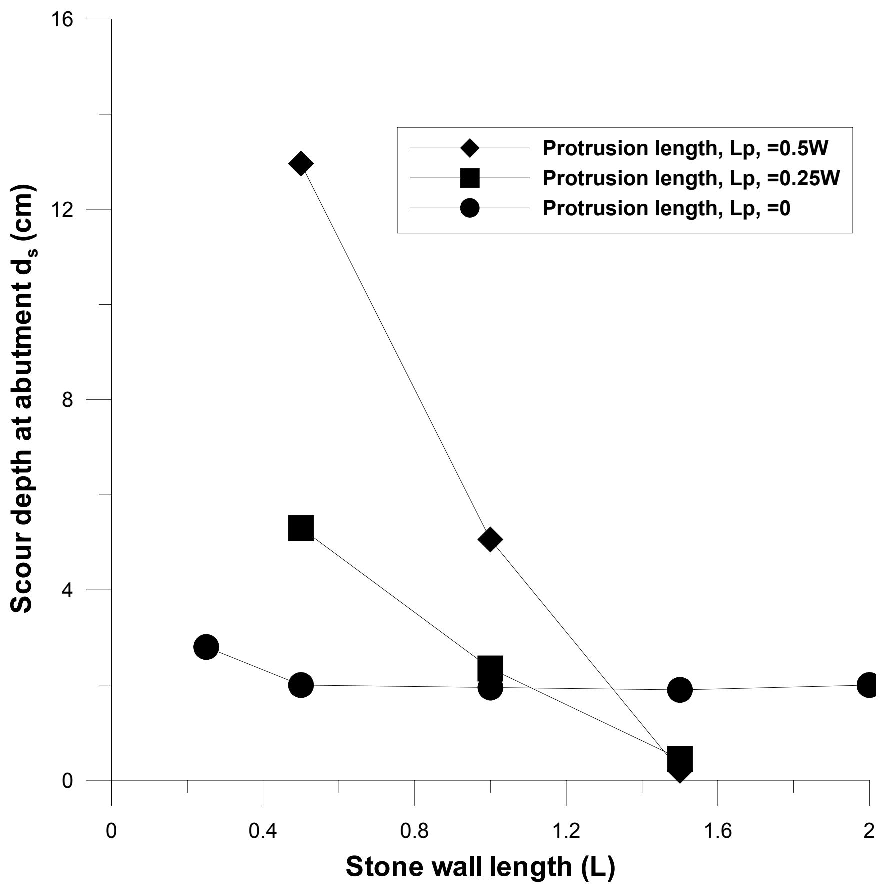


Fig. 6-8: Plot of scour depth at bridge abutment versus rock wall length for different wall protrusion lengths under clear-water conditions (U/U_c=0.9).

Fig. 6-9 is a plot of the maximum scour depth caused by the wall in the channel versus rock wall length for different wall protrusion lengths for clear-water experiments. It is seen from this plot that for the 0.25W and 0.5W protrusion lengths, increases in wall lengths can significantly reduce the maximum scour depth that is induced by the presence of the walls. While for walls with protrusion length of zero, increases in wall length result in essentially no reduction in scour depth (i.e. scour at abutment) when wall lengths are greater than 0.5L.

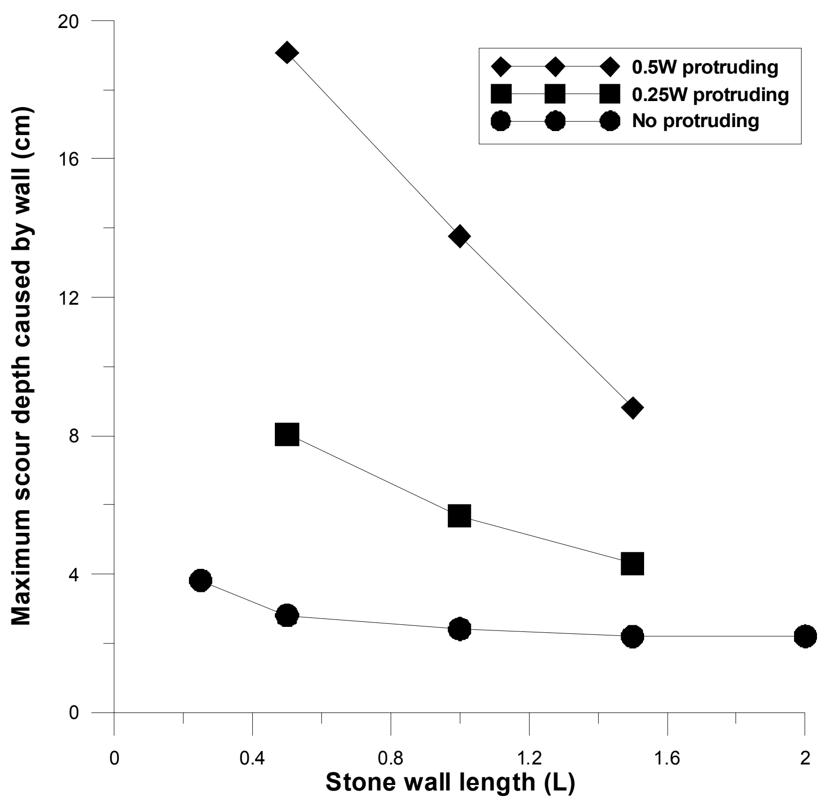


Fig. 6-9: Plot of the maximum scour depth caused by the wall in the entire channel versus rock wall length for different wall protrusion lengths under clear-water conditions (U/U_c=0.9).

Fig. 6-10 is a plot of both time-averaged and maximum instantaneous scour depth at the bridge abutment versus rock wall length for zero protrusion length under live-bed conditions. The time-averaged scour depth was calculated by measuring the scour depth at regular time intervals and then averaging over time. It gives a sense of the average depth of scour. The maximum instantaneous scour depth is the maximum scour measured at any time in the scour time series data collected. Even though this maximum scour value would not persist at the abutment, it could perhaps cause some brief undermining of the abutment structure and, therefore, is reported here as a parameter of interest. It is found that increases in wall lengths from 0.5L to 1.5L were able to reduce the time-averaged scour depth at the bridge abutment from 3.19 cm to 1.87 cm, which was about 41 percent. However, the increases in wall lengths only reduced the maximum instantaneous scour from 5.13 cm to 4.73 cm, which was only about 8 percent.

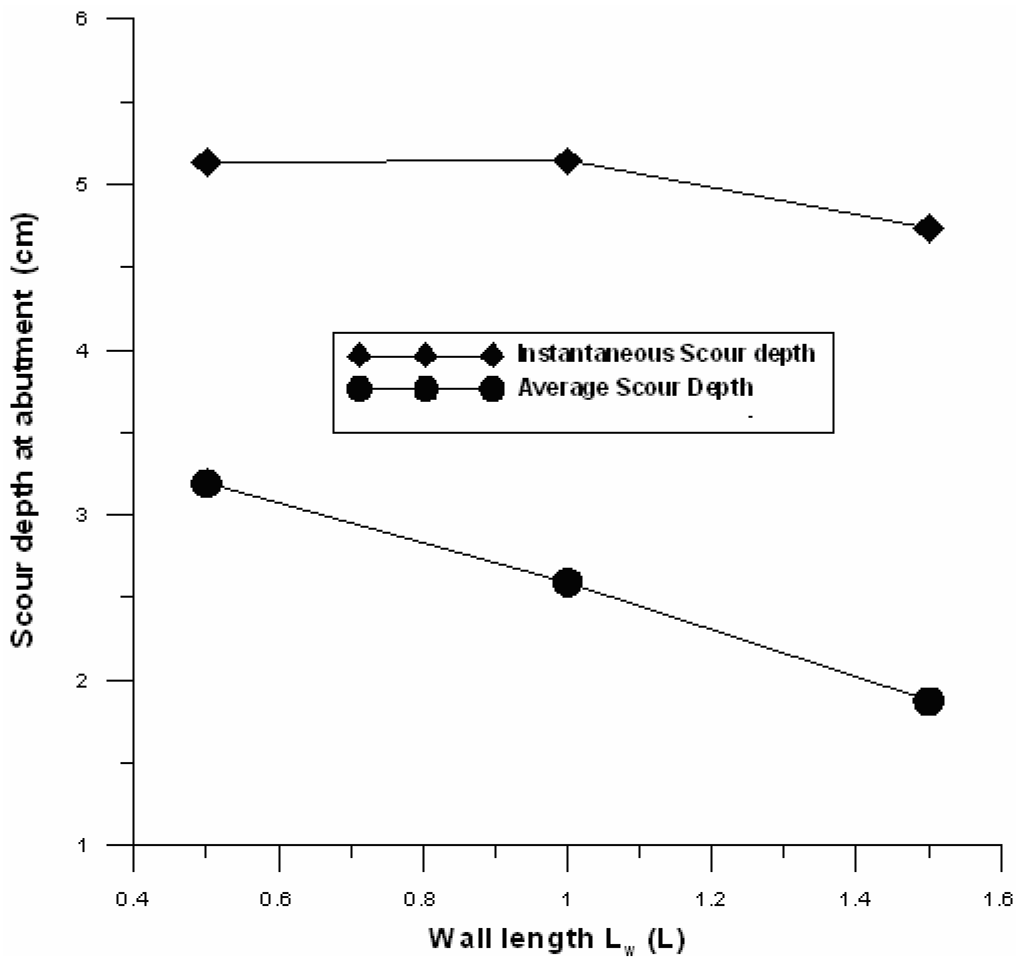


Fig. 6-10: Plot of both time-averaged and maximum instantaneous scour depth at bridge abutment versus rock wall length for zero protrusion under live-bed conditions, U/U_c = 1.5.

In general it was found that (1) walls which were set back onto the floodplain such that the base of the walls were even with the abutment (protrusion length, Lp=0) were very effective in protecting the abutment, (2) walls whose base protruded into the main channel beyond the abutment (protrusion length, Lp=0.25W or 0.5W) tended to produce significant scour in the bridge crossing and potentially threaten the middle and downstream abutment end while the length of these walls became shorter than a certain length, and (3) for zero protrusion length, Lp=0, the scour protection of the walls was not sensitive to the lengths of the wall unless it was less than 0.5L. Below this length the degree of scour protection decreased.

Regarding the amount of mass transfer or water flow through the rock wall itself, during the experiments, dye was injected in the floodplain side of the parallel walls. For all of the flow conditions investigated, no significant amount of dye was observed to flow through the rocks. This indicated that there is no significant flow transfer through the wall. In addition, an 0.8L long parallel solid wall with 85 circular holes with 8 mm diameter uniformly distributed on the wall was tried and compared with the 0.8L impermeable wall. The result was quite similar. This showed that when the permeability of the wall is smaller than a certain value, the solid wall acts like an impermeable one.

Cases of clear-water and live-bed scour were investigated. It could be that during flow conditions in which there was less downstream velocity, that there is more flow through the wall. This would most likely not be critical for scour, however, since clear-water and live-bed scour are the two worst cases for scour.

In all of the experiments reported on here, there was no gap between the countermeasure and the abutment. Care should be taken to ensure this, as a high-velocity jet may form if such a gap existed, that could exacerbate the scour depth in unknown ways.

6.4 Design of Parallel Wall for Scour Prevention at Wing-Wall Abutments

Despite these limitations, general preliminary design guidelines can be established for the use of parallel solid walls and parallel rock walls to reduce scour at typical county bridges with wing-wall abutments terminating on or protruding a distance out beyond the main channel banks.

6.4.1 Protrusion of Wall

The best position for solid walls would be such that the solid wall's face is aligned with the abutment's face so that there is no protrusion by either structure.

For parallel rock walls with lateral protrusions of 0.25W and 0.5W, a separation zone is formed behind the walls, causing scour holes and threatening the abutment foundation. Also the protrusion of the wall into the main channel further constricts the bridge crossing and reduces its conveyance capacity. Therefore, parallel rock walls of zero protrusion are recommended

6.4.2 Length of Wall

For a parallel solid wall countermeasure, the clear-water experimental results showed that a solid wall with a length of 1.1L will completely eliminate the local scour at the bridge abutment (Fig. 6); while with a velocity ratio of 1.5 a solid wall of length 1.6L will be able to reduce the time-averaged scour to zero; and for a velocity ratio of 2.3, a solid wall of length 1.6L will reduce the time-averaged scour up to 70 percent. Further increases in the length of the wall will result in decreasing of scour reduction rate. Therefore, it is recommended that the length of the parallel solid wall be 1.6L.

For the parallel rock walls of zero protrusion (Fig 6-9), a rock wall length of 0.5L is recommended.

6.4.3 Height and Width of Wall Crest

Heights for both solid wall and parallel rock wall should be at least as high as the bottom height of the lowest bridge deck so that flow can not enter the bridge crossing at the abutment even in the worst case scenario.

The width of a solid wall depends on the construction material used. It could be a several centimeters if a steel plate is used. The width of a parallel rock wall crest need not be larger than one stone diameter.

6.4.4 Slope of Wall and Apron

The side slope of the rock wall must be less than the rock's angle of repose to insure stability. However, the side slope should be as high as possible so that the protrusion of the abutment beyond the wall slope is the minimum. It is recommended that the side slope of the wall be about 5 degree less than the angle of repose of the rocks. The slope at the upstream end of the wall can be much flatter than the side slope to help stabilize the end of the wall and reduce local scour there. Therefore, an end slope of 2~2.5:1(H:V) is preferred.

Aprons are always needed at both the bottom of the side slope and the upstream end slope so that when scour takes place at those spots aprons can launch to prevent sliding of rocks from both slopes. Thickness, area limit and relative position with the parallel rock wall should be determined according to the scour depth and position of the scour hole along the wall.

6.4.5 Comparison of Solid and Rock Parallel Walls

If the best designs for both solid wall and rock wall are compared, it can be seen that the rock walls have advantages over the solid wall. Table 6-5 shows scour depths for both solid and rock walls for both clear-water and live-bed scour conditions. It can be seen that the rock wall allows less scour at the abutment than the solid wall. The solid wall, therefore, seems to be feasible only when the cost of rocks is prohibitively high.

Table 6-5: Comparison of rock and solid wall countermeasure performance (scour depth (cm)).

Wall Type / Scour Condition	Clear-water Scour	Live-bed Scour
Solid Wall	-0.40 at abutment 8.3 at solid wall	5.49 at abutment 7.86 at solid wall
Rock Wall	0.3 max. at abutment 4.33 max. at rock wall	4.73 max. at abutment 8.58 max. at rock wall

6.5 Conclusions

1. A parallel solid wall attached at the upstream corner of the abutment parallel with the flow can be used as a countermeasure against abutment scour. The length of the solid wall should be 1.6L to obtain acceptable scour reduction rate at the abutment for the conditions tried in this study.
2. A parallel solid wall attached at the upstream corner of the abutment parallel with the flow may or may not be able to reduce the amplitude of the bed forms that pass through the bridge opening depending on the changes of the flow parameters from the approaching channel after entering the bridge crossing.
3. There may be significant scour at the upstream solid wall end, so no other structures should be located in this region.
4. Parallel rock walls attached at the upstream of the abutment can also be used as countermeasures against scour at the abutment. The foot of the wall should not protrude into the main channel beyond the abutment and a top wall length of 0.5L will provide sufficient protection. The side slope of the rock wall should be on the order of 30^{o} but in no case should be steeper than about 70% of the rocks angle of repose.
5. Rock walls have more advantages than solid walls in terms of efficiency, stability and cost etc.

CHAPTER 7. SPUR DIKE AS AN ABUTMENT SCOUR COUNTERMEASURE

7.1 Introduction

Scour at bridge crossings can cause damage or failure of bridges and result in excessive repairs, loss of accessibility, or even death. Mitigation against scour at bridges has received much attention in the past few decades. Hydraulic countermeasures against bridge abutment scour can be classified as river training structures and armoring countermeasures. Considerations in choosing the appropriate method of mitigation include maintenance and inspection requirements, enhancement of the physical environment, and construction methods needed other than design constraints. Design specifications for many of these scour mitigation techniques can be found in Lagasse et al., 2001.

Spur dikes, the focus of this paper, have been used extensively in all parts of the world as river training structures to enhance navigation, improve flood control, and protect erodible banks (Copeland, 1983). Spur dikes are structures that project from the bank into the channel. They may be classified based on their permeability: high permeability - retarder spur dykes; impermeable - deflector spur dikes; and intermediate permeability - as retarder/deflector (Brown, 1985). They may be constructed out of a variety of materials including masonry, concrete, earth and rock, steel, timber sheet piling, gabions, timber fencing, or weighted brushwood fascines. They may be designed to be submerged regularly by the flow or to be submerged only by the largest flow events.

A spur dike serves one or more of the following functions.

(1) Training of the stream flow. For instance, spur dikes are commonly used to realign streams as they approach a bridge abutment. A bridge abutment may be in danger of being severely eroded when it is subjected to high velocity flow from a channel that has changed course due to meandering.

(2) Protection of the stream bank (may or may not contain bridge abutments) from erosion.

(3) Increasing of the flow depth for navigation (Garde et al., 1961) or improvement of aquatic habitat.

In recent years, porous and overflow-type spur dikes have been shown to provide improved pool habitats for fish and other aquatic life in severely degraded streams (Shields et al., 1995). Volumes of the scour hole in the vicinity of model spur dikes were measured in a laboratory flume under clear-water overtopping flows with varying angles and contraction ratios to maintain bank protection and enhance aquatic habitats (Kuhnle et al.,1997, 1998 ,1999). In addition, because of the deposition induced by spur dikes, they may protect a stream bank more effectively and at less cost than revetments (Lagasse et al., 2001).

Spur dikes constructed on or adjacent to an abutment to counter local scour have not been tried before to our knowledge. This technique would constitute a combination of bank-hardening and flow-altering to counter local scour. By the proper placement of spur dikes around the abutment the flow can be redirected and scour near the abutment can be reduced. Local scour can threaten the spur dike itself, however (Fig. 7-1).

The goals of this study was to determine the optimal length, spacing, number, and relative position of spur dikes using laboratory experiments with a wingwall abutment to manage scour in both clear-water and live-bed conditions. Design guidelines are presented.

Fig. 7-1: Photograph of excessive scour around a poorly-positioned spur dike. (Flow from left to right.)

7.2 <u>Conceptual Model</u>

Figure 7-2 is a sketch of the conceptual model of a spur dike as a countermeasure against abutment scour in a compound channel. A spur dike is placed a certain distance away upstream of the abutment and is perpendicular to the flow direction. Flow on the floodplain can only go around the main channel end of the spur dike. A spur dike thus installed is expected to be able to block the floodplain flow from hitting the abutment face and direct the flow into the main channel. It may create wake vortices behind itself. The effects of these wake vortices and scour hole at the spur dike structure on the abutment scour should be evaluated experimentally to determine the best configuration of spur dikes as countermeasures. In the following experimental studies, number of spur dikes, distance between spur dikes and distance between spur dike and abutment, protrusion length, and construction material of spur dikes will be tested as parameters.

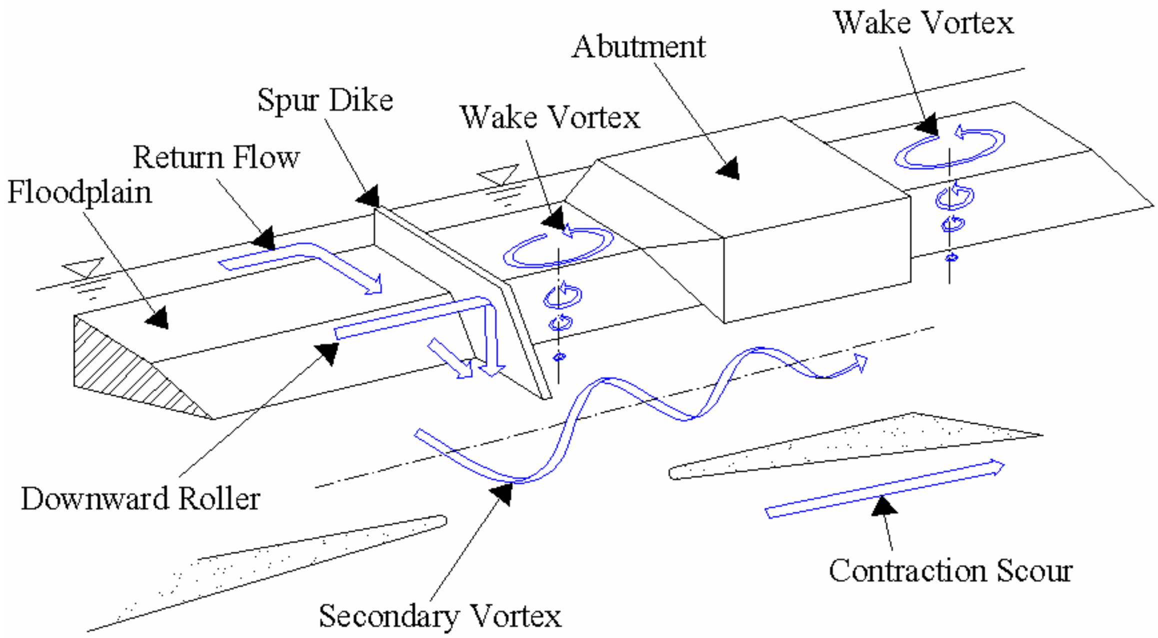


Figure 7-2: A sketch of the conceptual model of a spur dike as a countermeasure against abutment scour in a compound channel.

7.3 Results

8.4.1 Solid Spur Dikes

A series of solid spur dikes made from 13-mm thick plywood were tested under the same flow condition as in the clear-water baseline test (U/U_c =0.9). The flow depth in the main channel (y_m) was 13.2 cm, and the flow depth in the floodplain (y_f) was 5.2 cm. Only one spur dike located upstream of the abutment was tested in each experiment. The variables experimented with for the solid spur dikes were length (L_{sp}), distances upstream of the abutment (D_s), and orientation angle with respect to the flow (θ). In all cases the top of the spur dikes was higher than the water surface. The experimental results are listed below in Table 7-1.

Table 7-1: Preliminary Solid Spur Dike Experimental Results ($Q=0.0387 \pm 0.003 m^3/s$, $U/U_c=0.9$, y_m = 13.2 cm, y_f =5.2 cm).

Run No.	Spur dike description notes	L_s (L)	D_s (L)	θ (deg)	t (min)	$d_{.abut.avg}$ (cm)	$\%_{max,abut,avg}$ (%)	$d_{max.sp.avg}$ (cm)
Sp-1	Rectangular , protrusion length equal to the width of floodplain	0.7	2	90	1254	4.52	32.2	--
Sp-2		1	2	90	1540	4.60	34.8	10.52
Sp-3	Fig. 6	0.7	1	45	4800	4.69	39.6	14.40
Sp-4		0.7	1.5	45	1685	4.50	35.2	13.90
Sp-5	Fig. 7	0.7	1.5	45	1440	4.33	36.5	13.53
Sp-6	Fig. 8	1	0.58	90	2580	5.80	20.6	--

Note: L_s = Spur dike protrusion length; D_s = distance between the farthest spur dike tip at the main channel end and abutment tip; θ = Spur dike orientation angle with respect to the flow; t = Run time; $d_{abut.avg}$ = Time averaged scour depth at abutment; $\%_{max,abut,\ avg}$ = Percent of scour reduction; $d_{max.sp.avg}$ = Maximum scour depth at spur dike.

The flow-perpendicular length of spur dike was found to be an important variable in protecting the abutment. Flow-perpendicular lengths were restricted to the length of the abutment or less in this experimental series to prevent contraction of the flow in the main channel. The six cases tested show that spur dikes of the same flow perpendicular length as the abutment do not protect it from scour regardless of spacing or orientation angle (Figs. 7-3, 4 and 5). When the spacing (D_s) of the spur dikes was less than the flow perpendicular length of the abutment (L), the upstream corner of the abutment will fall into the scour hole induced by the spur dike (Fig. 7-5). When the spur dikes were far away ($D_s \geq$ 1.5L) from the abutment, a narrow channel formed between the deposited sand and the abutment corner (Fig. 7-4). This caused scour at the upstream corner of the abutment. In addition, the spur dikes caused the formation of huge scour holes which would undoubtedly threaten the stability of the stream bank and the stability of the spur dike itself. In summary, the effective reduction of local scour at bridge abutments using spur dikes requires that their flow-perpendicular length be greater than the flow perpendicular length of the abutment.

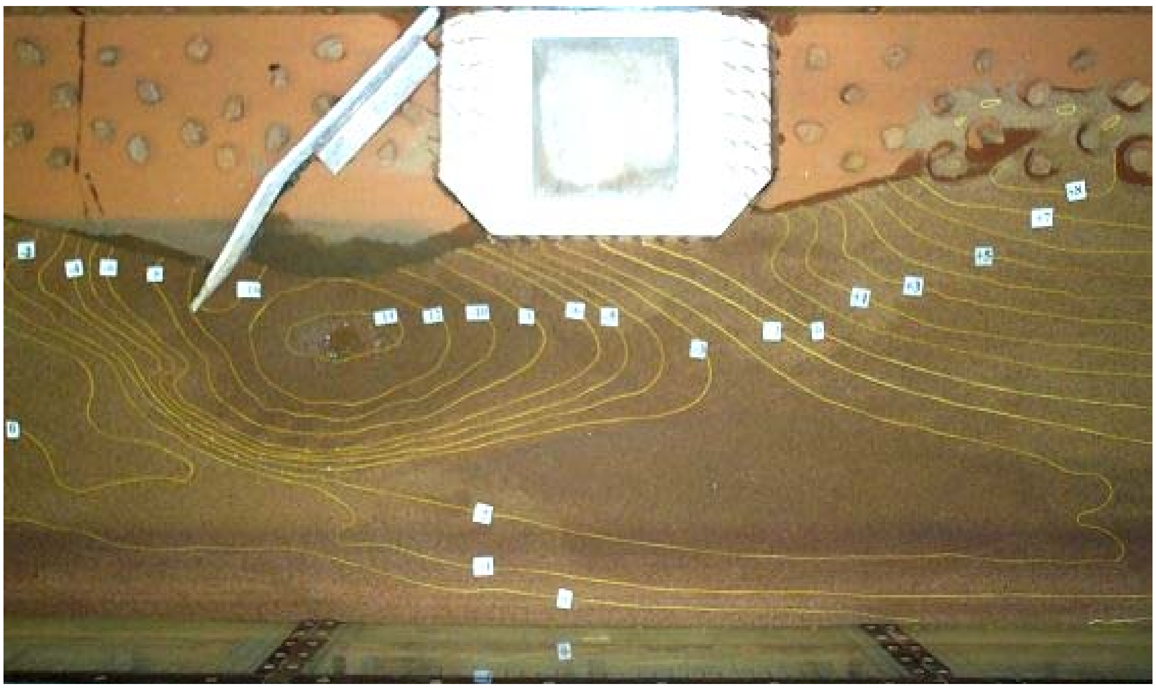


Fig. 7-3: Photograph of Sp-3. Flow from left to right.

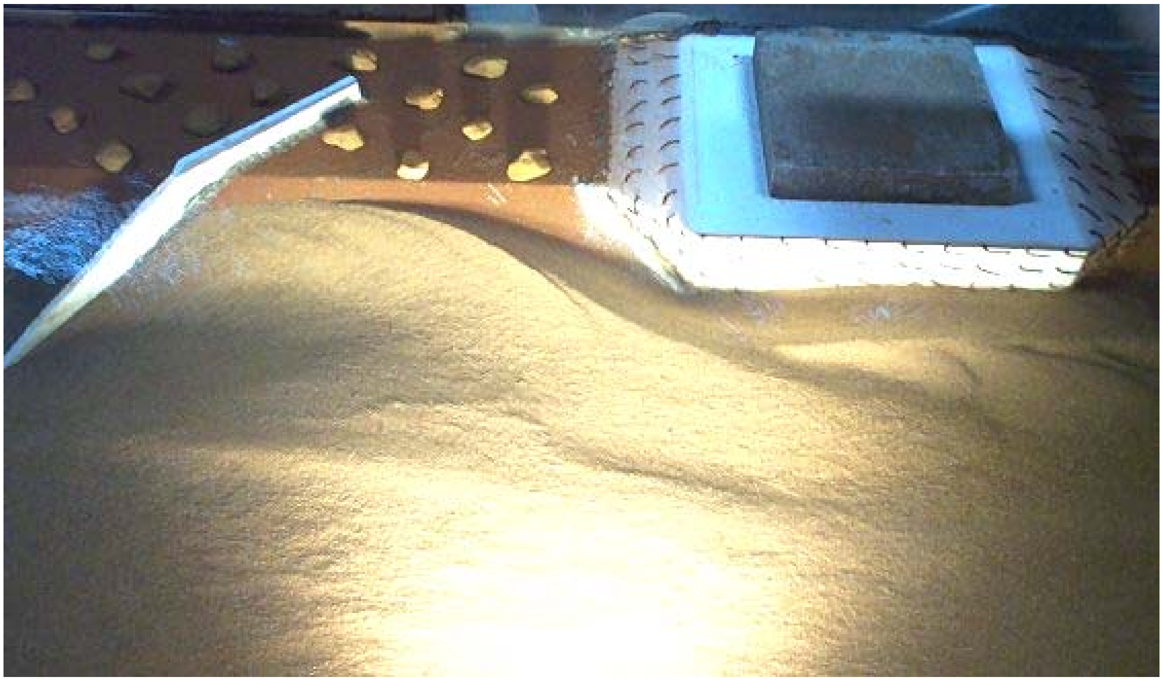


Fig. 7-4: Photograph of Sp-5. Flow from left to right.

Fig. 7-5: Photograph of Sp-6. Flow from left to right.

7.3.2 <u>Rock Spur Dike</u>

Four cases of rock spur dikes each with the same length perpendicular to the flow were tested under clear water flows. These rock spur dikes were constructed from the flume wall extending out into the main channel perpendicular to the flow direction. At the stream end of each spur dike there was a transverse slope of H:V=22/13.2. Therefore, the top protrusion length of the spur dike was L (44 cm) and the bottom protrusion length was 1.5L (66 cm). Each spur dike had a top width of 10 cm and a bottom width of 40 cm. The rocks were of 6.7 ~ 9.5 mm diameter. In these cases, the flow depth on the floodplain (y_f) was equal to 5.2 cm and the flow depth in the main channel (y_m) was 13.2 cm. The mean velocity ratio (U/U_c) was 0.9 in the middle of the main channel. The top of each spur dike was the same height as the top of the abutment. The experimental data for rock wall spur dikes in clear-water scour conditions are summarized in Table 7-2.

From the clear-water rock spur dike data (Figs. 7-6, 7, 8 and 9) it is seen that spur dikes with a top length of 1L and bottom length of 1.5L provided protection to the bridge abutment from scour. These lengths are believed to be the minimum length required to be sufficient to protect the abutment.

It was found from Tests Sp-8 (Fig. 7-7) and Sp-9 (Fig. 7-8) that as the distance between two successive spur dikes increased from 1L to 2L, the scour depth between the first two spur dikes increased from 7.62 cm to 13.14 cm, a more than 70 percent increase. Although this increase in scour depth did not pose a direct threat to the abutment, it did pose a threat to the two spur dikes and may cause these spur dikes to partially fail, therefore ultimately posing an indirect threat to the abutment. From the scour condition between the second and the third spur dikes in these two tests, it was found that in Test Sp-8, the scour depth was 6.61 cm and greater than the scour depth in Test Sp-9. However, the scour hole in Test Sp-8 was deflected out into the main channel by the spur dikes and had no effect on the abutment, while in Test Sp-9, the abutment was threatened. Therefore, the best spacing between spur dikes is concluded to be 1L.

Table 7-2: Clear-water experimental data of rock spur dikes ($Q = 0.0368 \pm 0.0016\ m^3/s$, $U/U_c = 0.9$, $y_m = 13.2$ cm, $y_f = 5.2$ cm, gravel diameter D = 6.7 ~ 9.5 mm, running time t = 4800 min, all spur dikes had a top protrusion length of 1.0L , a bottom protrusion length of 1.5L, and end slope of H:V=22/13.2).

Run No.	n	D_s, (L)	$d_{abut.avg}$, (cm)	$\%_{max,\ abut}$	$d_{max.sp1.avg}$, (cm)	$d_{max.sp2.avg}$, (cm)	$d_{max.sp3.avg}$, (cm)
Sp-7 (Fig. 7-6)	2	1	0	100	8.75	11.00	--
Sp-8 (Fig. 7-7)	3	1	0	100	7.62	6.61	7.71
Sp-9 (Fig. 7-8)	3	2	2.00	74.3	13.14	3.50	6.89
Sp-10 (Fig. 7-9)	2	1	0	100	7.56	10.30	--

Note: n = number of spur dikes; D_s = Spacing between spur dikes or spur dike and abutment; $d_{abut.avg}$ = Time averaged scour depth at abutment; $\%_{max,\ abut}$ = Percent of scour reduction; $d_{max.sp1.avg}$ = Maximum scour depth behind the first spur dike; $d_{max.sp2.avg}$ = Maximum scour depth behind the second spur dike; $d_{max.sp3.avg}$ = Maximum scour depth behind the third spur dike.

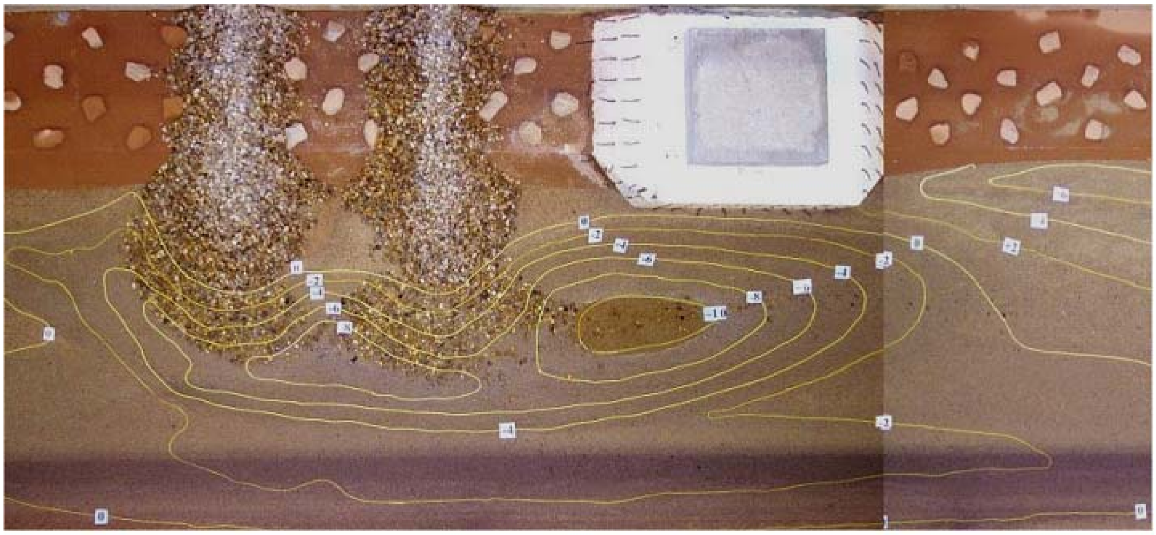


Fig. 7-6: Scour contour of Test Sp-7 with two spur dikes upstream of the abutment. Flow was from left to right.

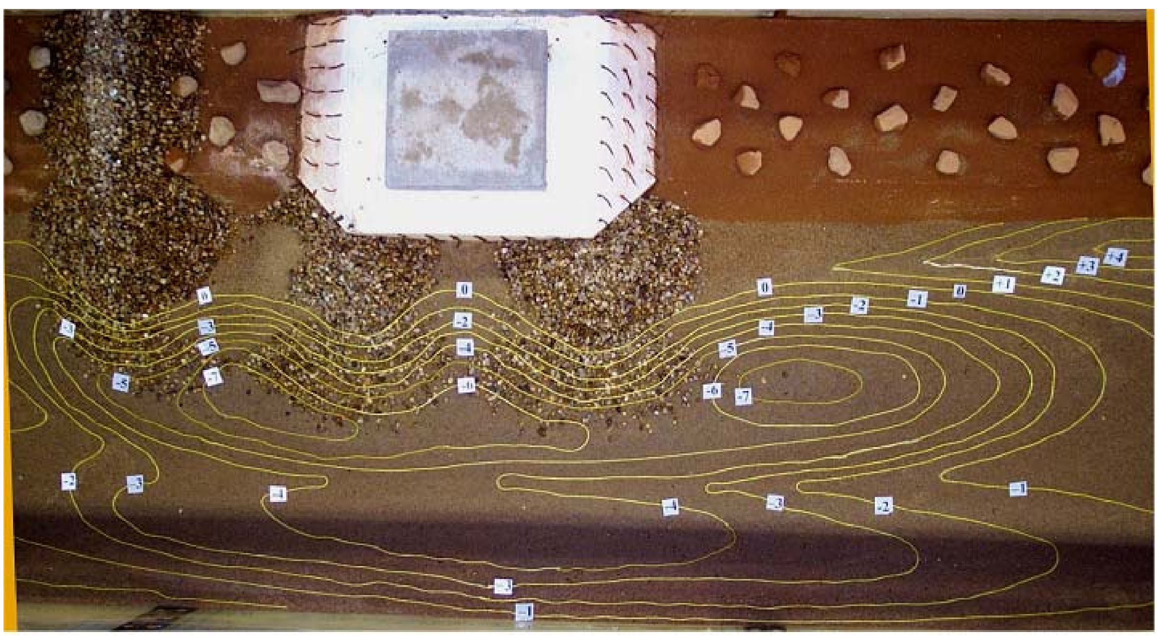


Fig. 7-7: Scour contour of Test Sp-8 with three spur dikes (including the two formed by the abutment). The flow was from left to right.

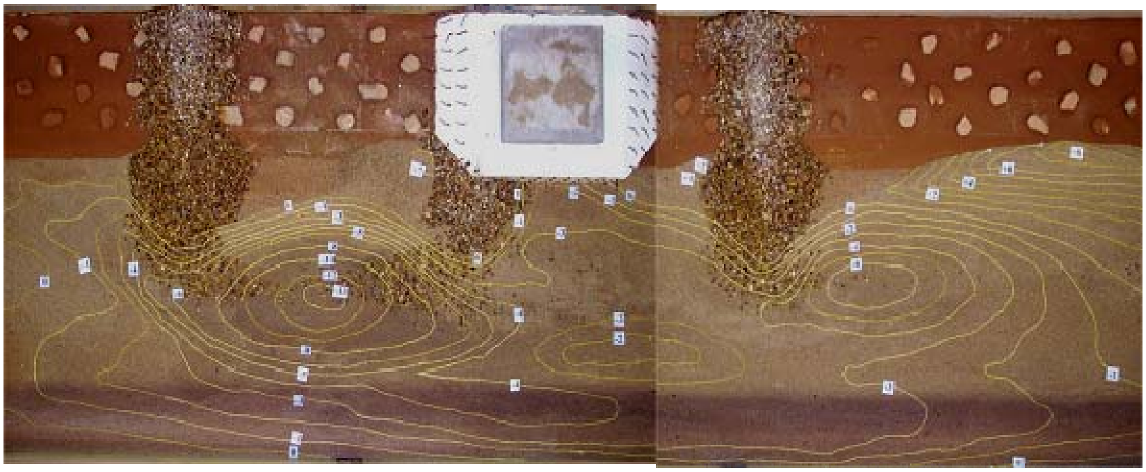


Fig. 7-8: Scour contour of Test Sp-9 with three spur dikes (including the one formed by the abutment). The flow was from left to right.

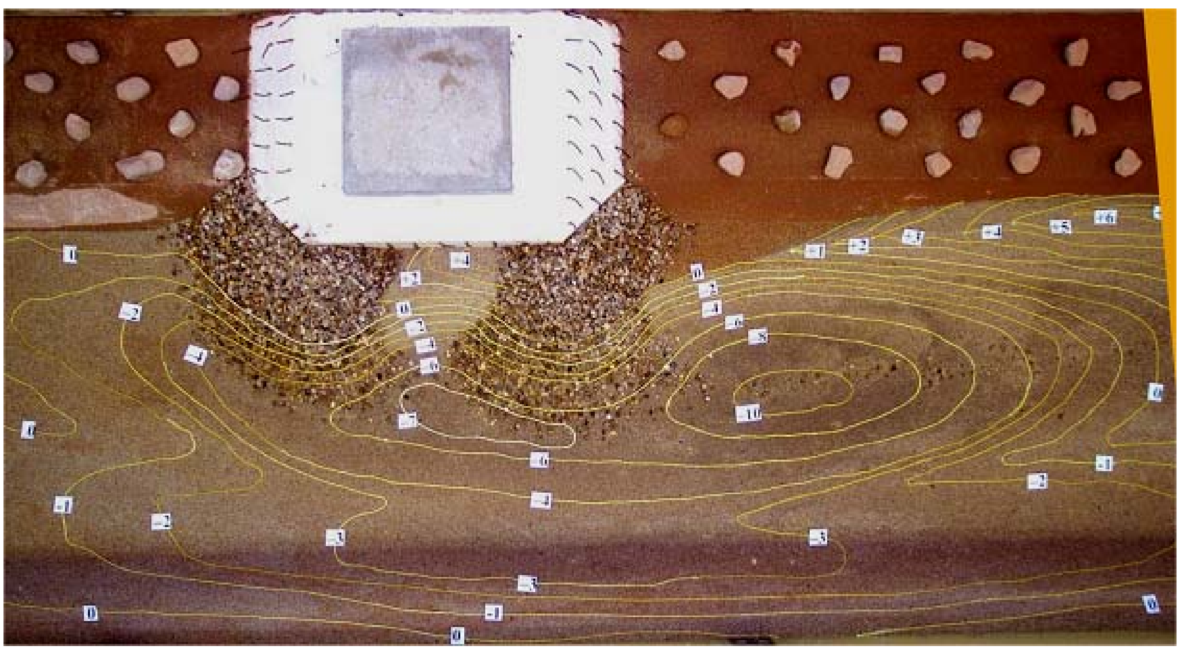


Fig. 7-9: Scour contour of Test Sp-10 with two spur dikes (both are located at the abutment). The flow was from left to right.

Generally the larger the number of spur dikes, the better the abutment will be protected, but at a higher cost. To minimize the cost, the number of spur dikes should be minimized. With this in mind, Sp-10 (Fig. 7-9) was performed with only two end-slope spur dikes attached at the upstream and downstream corners of the abutment. It was found that the two spur dikes thus located directed the flow away from the abutment thereby protecting the abutment and upstream and downstream banks.

The advantage of these two end slopes are (1) they can direct the flow away from the abutment and protect the abutment, (2) they can provide extra stabilization to the abutment, especially when some rocks fill in the existing scour holes at both corners of the abutment, (3) as the scour in the bridge crossing develops both spur dikes sink and collapse to the original bed level and the rock material is distributed by the flow such that they are still able to function as riprap to protect the abutment, and (4) two spur dikes attached at the abutment face use less rock than spur dikes attached at the floodplain bank. The disadvantage is that they may cause contraction scour. In relatively wide bridge crossings, this may not cause a significant problem, but may be a problem for narrower ones.

Positioning of the spur dikes is important to the protection of the abutment. As shown in the baseline case, the upstream corner of the abutment was the point that was most likely to scour. Another scour-prone location would be the downstream end of the abutment where the flow leaves the bridge crossing and separates. Therefore, spur dikes located at both ends of the abutment yielded the best results as shown in Test Sp-10. The advantage of this configuration is that by being attached to the face of the abutment instead of the bank of the flood channel, the amount of rock material is minimized and the cost of the construction of the spur dikes reduced.

From the clear-water experimental data, the configurations of Test Sp-8 and Sp-10 were considered to have the most potential for protecting the abutment and were tested further under live-bed flows. No further experiments were conducted with the configurations used in Sp-7 or Sp-9.

The same spur dike configurations used in Sp-8 and Sp-10 were tested under live bed conditions as countermeasures against scour at the abutment. Test Sp-11 had basically the same spur dike configuration as Case Sp-10. Test Sp-12 had a similar spur dike configuration as Test Sp-11 except that there was initially a semicircular ring-shaped apron around each of the spur dike ends. Each apron had a width of about 20 cm and was about 3 rock diameters thick. Test Sp-13 and Test Sp-14 had similar spur dike configurations as Test Sp-10 except the aprons and rock size varied. There were similar aprons around the first two spur dikes in the later cases. In these cases, the flow depths were the same as for the clear water experiments. The velocity ratios (U/U_c) used were 1.5 and 2.3. The rock sizes used for the velocity ratio of 1.5 were of 19 ~ 50 mm in diameter and for the 2.3 velocity ratio were 50 ~ 70 cm in diameter. The top of each spur dike was approximately the same height as the top of the abutment. The live-bed experimental data are listed in Table 7-3.

Table 7-3: Live-bed experimental data of rock spur dikes (Q = 0.0627 ± 0.003 m^3/s for velocity ratio of 1.5 and 0.0985 for velocity ratio of 2.3, y_m = 13.2 cm, y_f =5.2 cm, running time t = 3000 min).

Run No.	U/U_c	$d_{abut.avg}$, (cm)	$\%_{max.abut.avg}$	$d_{max.sp1.avg}$, (cm)	$d_{max.sp1.inst}$, (cm)	$d_{max.sp2.avg}$, (cm)	$d_{max.sp2.inst}$, (cm)
Sp-11 (Fig. 7-10)	1.5	-1.03*	114	5.11	10.35	4.92	9.93
Sp-12	1.5	-1.42	120	4.45	9.51	5.03	7.49
Sp-13 (Fig. 7-11)	1.5	-2.66	136	5.39	8.90	4.68	7.74
Sp-14 (Fig. 7-12)	2.3	-0.03	100	6.97	10.94	--	--

*** Negative scour depths indicate deposition. $d_{abut.avg}$ = Time averaged scour depth at abutment; $\%_{max.abut}$ = Percent of scour reduction; $d_{max.sp1.avg}$ = Time-averaged scour depth in front of the 1st spur dike; $d_{max.sp1.inst}$ = Maximum instantaneous scour depth in front of the 1st spur dike; $d_{max.sp2.avg}$ = Time-averaged scour depth at the 2nd spur dike; $d_{max.sp2.inst}$ = instantaneous scour depth at the 2nd spur dike.**

Data of Test Sp-11 (Fig. 7-10) showed that with a velocity ratio of 1.5, two dikes attached at both ends of the abutment were able to reduce scour over 100 percent (cause deposition). Deposition of 1.0 to 4.0 cm occurred between the two slopes at the abutment during 3000 minutes of running time. The maximum scour at these spur dikes happened at Point A where the foot of the upstream bank and the upstream side of the first spur dike meet. This was different from the scour pattern in clear-water scour conditions. Sediment moved along a scour zone starting from Point A and proceeded along the edge of the launched apron of the spur dikes towards Points B, C and D, covering part of these aprons and extending past the bridge crossing while keeping a distance from the abutment face. Therefore after equilibrium was reached, the deposition between the two slopes at the abutment was not affected any more. Local scour holes were found at points C and D where flow separated between the two spur dikes. The top of the two spur dikes was initially as high as the flow surface in Test Sp-11. As the scour developed, the rocks on the slope of the spur dikes kept sliding into the scour hole causing the top of the two spur dikes to subside until the equilibrium state of the scour process was reached. At the end of the test, the top of the first spur dike sank 7.50 cm and the top of the second spur dike sank 2.00 cm. To reduce this sinking, Test Sp-12 was tested with two semicircular aprons of 20 cm width and 3 rock diameter thicknesses placed around the edge of each spur dike, respectively. After 3000 minutes of running time, the presence of the aprons helped improve the deposition between the two spur dikes at the abutment and reduced the scour depth at the upstream side of the first spur dike (Table 7-3). However, the top of the first spur dike still sank 5.00 cm.

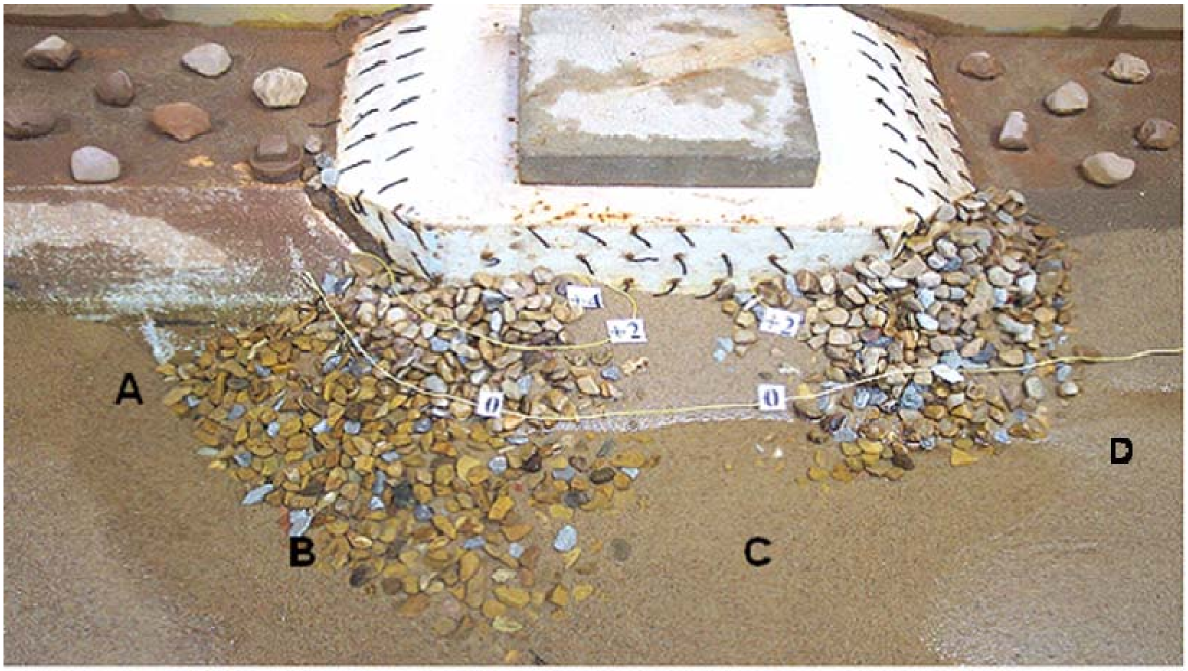


Fig. 7-10: Photograph of Test Sp-11 with two spurs attached at both ends of the abutment. Flow from left to right.

Although the configuration in Test Sp-11 and Sp-12 can protect the bed around the abutment, there is a concern about the portion of the floodplain at the upstream corner of the abutment. For erodible floodplains the spur dikes thus placed are not able to protect the floodplain. Compared to Test Sp-11 and Sp-12, data in both Table 7-3 and Fig. 7-11 showed that the configuration of Test Sp-13 can protect not only the channel bed around the abutment but also the floodplain and the abutment fill. The minimum deposition of sediment around the abutment was found to be 2.66 cm. Also because of the protection of the first spur dike, the spur dikes at both corners of the abutment experienced very little subsidence.

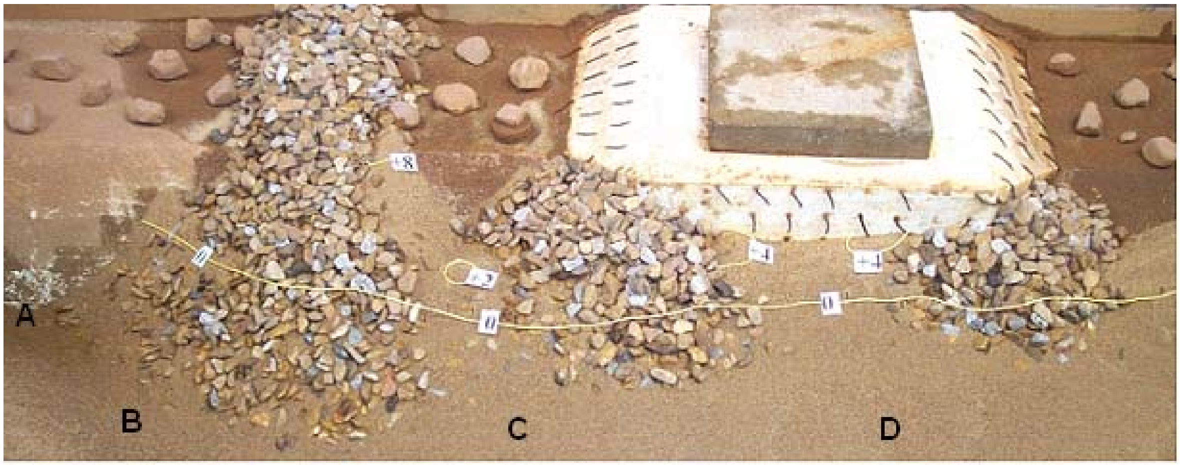


Fig. 7-11: Photograph of Sp-13 with three spur dikes. Flow from left to right.

The same spur dike configuration as in Test Sp-13 was tried further in Test Sp-14 with a velocity ratio of 2.3. The spur dikes were made of rocks with size of diameter of 50 ~ 70 mm in order to resist transport by the flow. The spur dikes still performed protected the abutment from scour successfully (Table 7-3, Fig. 7-12).

Fig. 7-12: Photograph of Sp-14 with three spur dikes. Flow from left to right.

Rock spur dikes have more advantages than solid spur dikes. First, unlike solid spur dikes that need foundations, rock spur dikes do not need a traditional foundation. Instead they can launch aprons around the structure edges as the scour holes develop and prevent them from failing. Second, a sloped end at the main channel end of a spur dike whose top protrusion length is L will provide extra protrusion length and more deflection. Third, rock spur dikes may make deposition at the upstream of the abutment possible. The reason that the sediment did not deposit at the upstream corner of the abutment can be attributed to the fact that the abutment structure had a very smooth surface, and the flow velocity near the abutment surface was relatively high. This unimpeded velocity prevents settling of sediment. To conquer this problem, a pile of gravel placed at the upstream increases the roughness of the abutment and decreases the flow velocity so that sediment can deposit at the upstream corner of the piled rocks. Fourth, upward sloping rock spur dikes with relatively high friction roughness slow and guide the flow to climb up the slope instead of producing the scour-inducing downflow.

7.4 Design of Spur Dikes for Scour Prevention at Wing-wall Abutments

From the experimental results, it was concluded that spur dikes with top protrusion length of 1L and bottom length of 1.5L were sufficiently long to provide protection to the bridge abutment. The amount of material in the spur dikes can be greatly reduced if they are attached at the face of the abutment. The top length 1L is believed to be the minimum length required to protect the abutment while the bottom length is designed as a function of the rock's angle of repose. It was concluded for straight channels that the best spacing between successive spur dikes was 1L. A spacing of 1L or less was able to restrict the flow from full separation behind each spur dike. As a consequence the scour depth behind each spur dike was less and the scour hole was pushed farther away from the spur dike end into the main channel.

It was concluded that 3 spur dikes, with the first one located 1L distance upstream of the upstream abutment corner, and the remaining two attached at the upstream and downstream corners of the abutment, respectively, would be the best configuration for preventing scour of the bed near the abutment for this experimental setup (stream-wise width of abutment is around L). For a bridge abutment whose stream-wise width is longer than 1L, a spur dike attached at the downstream end is recommended, with additional spur dikes located upstream at distances of 1L until the upstream corner of the abutment is met. One more spur dike is preferred upstream of the one at the upstream corner of the abutment. For a bridge abutment whose stream-wise width is less then the flow perpendicular length of the abutment (L), three spur dikes are recommended: one at the upstream corner of the abutment, one at a distance of 1L upstream of the abutment, and the other at 1L distance downstream of the abutment.

In addition to the variables directly tested in the laboratory experiments described above, the following design guidelines are offered. The top height of each spur dike should be high enough so that it is not overtopped by the flow during flooding, since all of the experiments described here are for emergent spur dikes and the flow patterns will be greatly altered if the spur dikes were overtopped. The top width of the spur dikes should be wide enough to allow vehicle traffic, if needed. The rock size of the spur dikes should be great enough to resist the flow stress in the worst flood situation. It was observed in the laboratory that end slopes of spur dikes can be constructed as steep as possible since they are able to adjust themselves to a stable state as scour holes develop around them. Semicircular aprons launch rocks into the scour hole during its development.

7.5 Conclusions

For the clear-water and live-bed scour and a Type III abutment configuration in a straight channel it can be concluded that:

1. A single spur dike made of a solid plate having the same or shorter protrusion length as the abutment and placed upstream of the abutment was not able to protect the abutment. The downflow and the principal vortex are very strong at the stream end of the structure. As a consequence, a huge scour hole was always found at the end of the structure which threatened both the structure and the channel bank.
2. Rock spur dikes show several advantages over rigid spur dikes and are preferred.
3. Three rock spur dikes as configured in Tests Sp-9 and Sp-13 were considered the best configuration for protecting the abutment. This configuration can provide 100 percent

protection to the abutment under the velocity ratios (U/U_c) of 0.9, 1.5, and 2.3. Two spur dikes at the upstream and downstream corners of the abutment were also successful at preventing scour in both clear-water and live-bed experiments.

CHAPTER 8. BRIDGE ABUTMENT COLLAR AS A SCOUR COUNTERMEASURE

9.1 Introduction

Parallel walls and spur dikes have been previously tested as successful countermeasures against abutment scour in this study. A flow-altering countermeasure that has not previously been tested for abutments in a compound channel is a horizontal collar. Collars attached to piers have been studied as either an armor layer of the bed or a downflow-halting device by Kapoor and Keana (1994) , Kumar et al. (1999), and Borghei et al. (2004). Collars block the downflow found at the leading edge of piers and abutments and eliminate scour-inducing secondary vortices.

This chapter describes laboratory experiments with collars at a vertical-face wing-wall abutment placed at the main channel edge, an abutment configuration typical of older bridges on smaller streams. The authors are unaware of other studies of this type.

8.2 Conceptual Model

Figure 8-1 is a sketch of the conceptual model of a collar as a countermeasure against abutment scour in a compound channel. A collar is attached around the bridge abutment and has a certain areal covering extent of the bed. A collar thus installed is expected to be able to prevent the bed materials from being entrained by the return flow from the floodplain, the downflow and the secondary vortex systems. In the following experimental studies, the areal extent in all directions and the vertical elevation of the collar will be studied under clear-water conditions to determine the best configuration for collar to be a successful countermeasure.

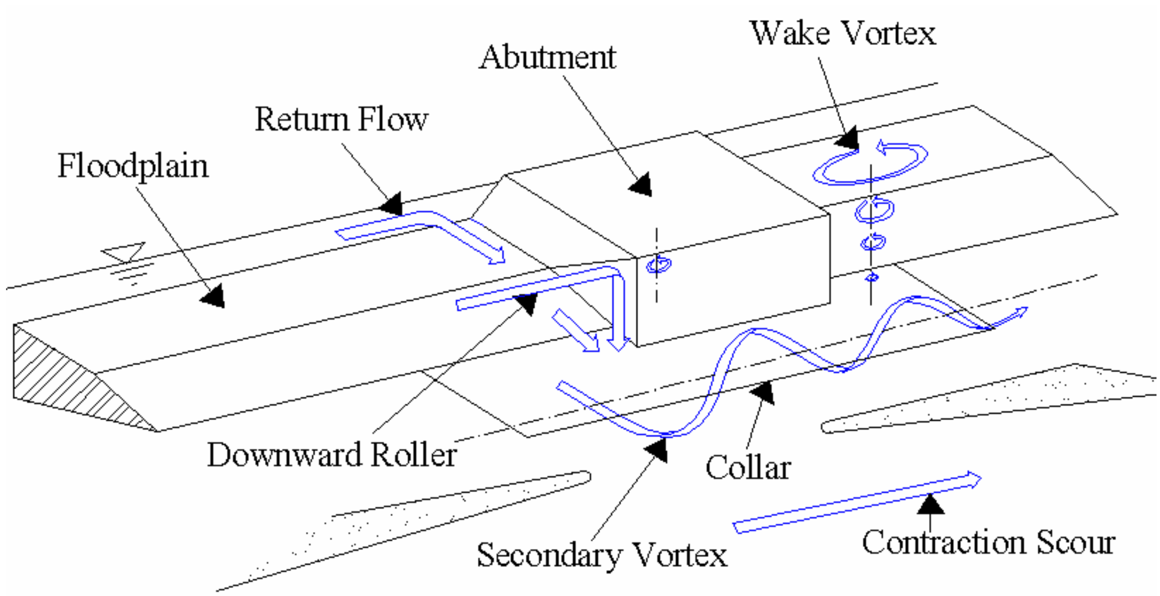


Figure 8-1: A sketch of the conceptual model of a collar as a countermeasure against abutment scour in a compound channel.

8.3 Collar Results

A series of collars of different lengths and widths were attached to the bridge abutment under clear-water conditions as countermeasures against scour at the abutment. These collars were made from steel and were seated horizontally at the desired elevation. The flow depth on the floodplain, y_f, was equal to 5.2 cm and the flow depth in the main channel, y_m, was 13.2 cm. The velocity ratio, U/U_c was 0.9 at the center of the entire channel as in the baseline tests. Table 2 gives the dimensions of each collar configuration tested.

Table 8-1 gives the results of the scour experiments with collars. Fig. 8-2 shows scour contours for equilibrium condition for Test T3. It was found that the collars were able to protect the bridge abutment efficiently by isolating the return flow and the secondary vortices from the bed around the abutment that ordinarily would cause local scour. The minimum collar dimensions that eliminated local scour were those with a width of 0.23L (L is the abutment length perpendicular to the flow direction) for elimination of local scour, a width of 0.8L for maximum reduction of scour at the edge of the collar, and having the collar located at a vertical location of $0.08y_m$ below the mean bed sediment elevation (y_m is the main channel flow depth). After removal of the collar, there was no scour observed under the collar around the abutment for any of the cases tested.

Table 8-1. Dimensions and positions of collars tested (run time=4800 min., y_m = 13.2 cm, y_f = 5.2, Q=0.0387 ± 0.001m^3/s, U/U_c=0.9).

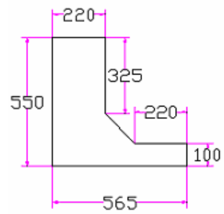
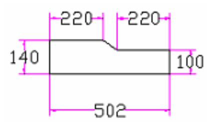
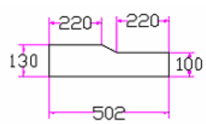
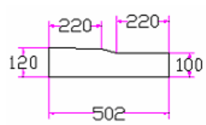
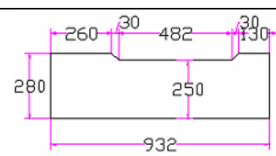
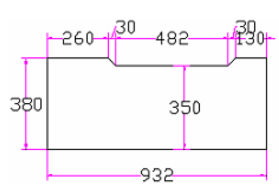
No.	Dimensions (mm)	Elevation	$d_{max.abut}$ (cm)	$\%_{max,abut,avg}$	$d_{max.col}$ (cm)
T1		Floodplain level.	4.48	42.3	--
T2		Bed level.	3.51	54.8	8.14
T3		1 cm below bed. (Fig. 6)	1.92	75.3	7.10
T4		2 cm below bed.	2.00	74.3	7.89
T5		1 cm below bed.	1.00	87.1	4.54
T6		1 cm below bed. (Fig. 10)	1.00	87.1	1.00

Figure 8-2: Elevation contours of Test T3 with collar at 1 cm below bed elevation. Flow from left to right. (See Table 8-1).

8.4 Discussion

8.4.1 Protrusion Width

Figure 8-3 is a plot of the maximum scour depth at both the bridge abutment and at the main channel edge of the collar versus the transverse collar width for all collar cases when the collar elevation was 1 cm below the initial bed level. It can be seen from Figure 8-2 that the maximum local scour depth under the main channel edge of the collar decreased from 7.10 cm to 1.00 cm as the width of the collar beyond the abutment increased from 10 cm to 35 cm.

Further examination of the experimental results shows that the maximum local scour depths at the main channel edge of each of these collars had a similar magnitude as the scour depth at the same location in the baseline case with no countermeasures. Figure 8-4 is a plot of the transverse bed profile in the bridge crossing of the baseline case and the scour profile formed by the maximum local scour depth values under the edge of the various collars of different widths. Fig. 8-4 suggests that the presence of the collar did

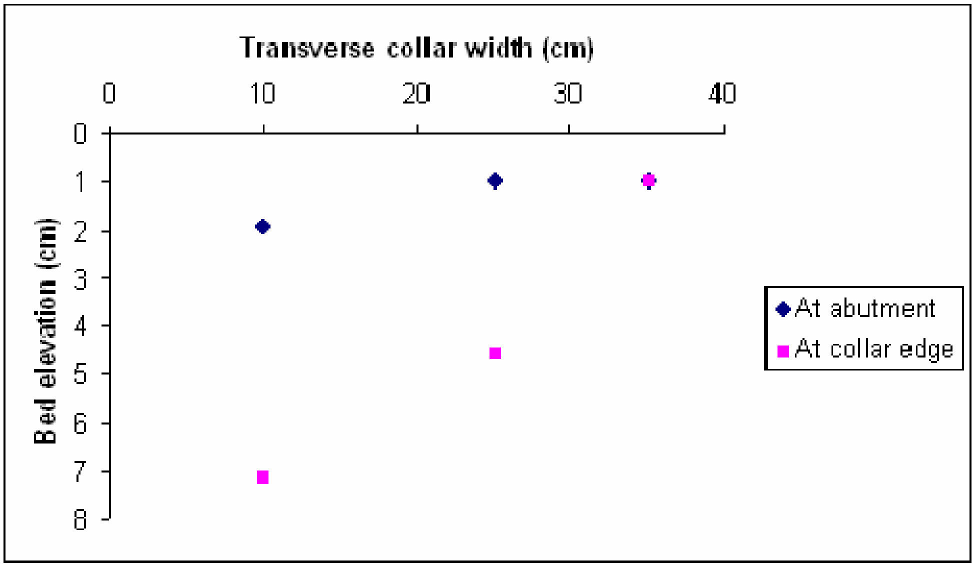


Figure 8-3: Plot of the scour at both bridge abutment and the main channel edge of the collar versus the transverse collar width for a collar elevation of 1 cm below the bed.

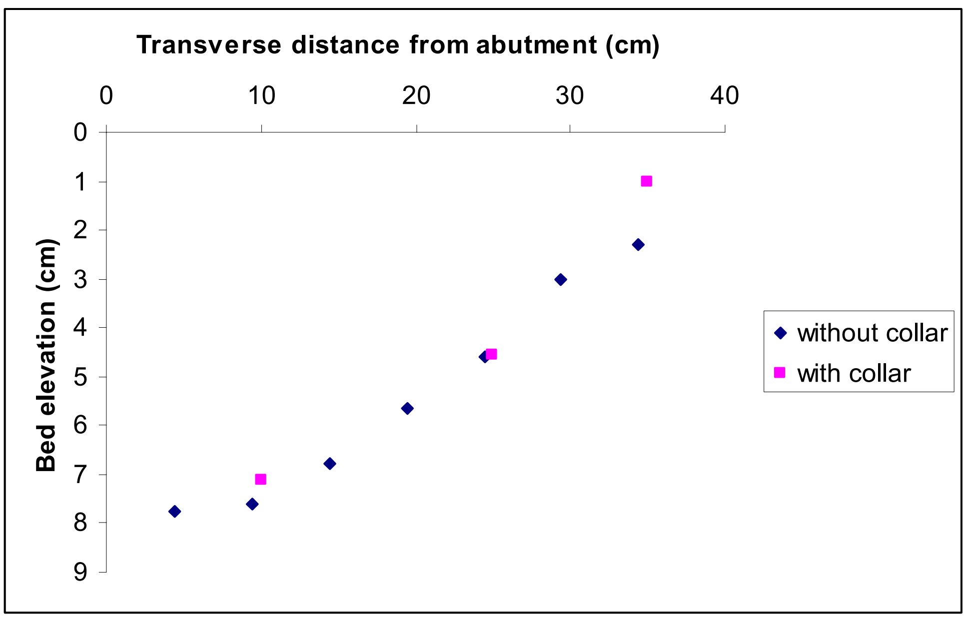


Figure 8-4: Plot of the transverse scour profile in the bridge crossing of the baseline case and the scour profile formed by the maximum local scour depths under the main channel edge of the various collars of different widths at the end of 4800 min. View is looking downstream.

not change the strength of the vortex, but protected the abutment from scour by not allowing the scour-inducing secondary vortex to interact with the bed sediment.

8.4.2 Collar Elevation

To determine the optimal collar elevation, three different elevations of the collars were used in the experiments. Figure 8-5 shows the scour depth at the abutment and at the edge of the collars versus collar elevation for collars with a width of 10 cm. It is evident that an elevation of 1 cm below the original bed level had the least scour. This corresponds to an elevation of $1/13.2=0.08y_m$, where y_m is the flow depth in the main channel. The collar should be lower than the bed in order to keep the secondary vortex above it and not interacting with the bed sediment.

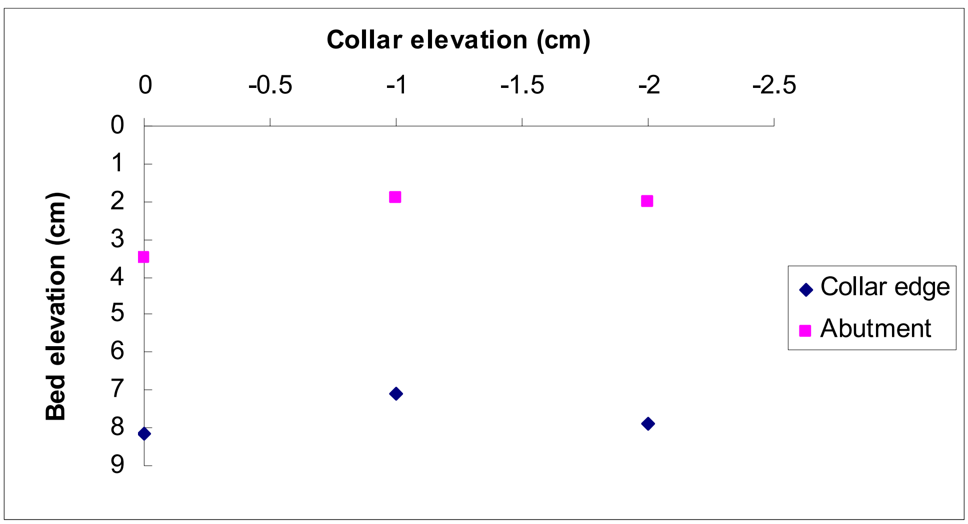


Figure 8-5: Plot of the bed elevation at the abutment and at the edge of the collars versus collar elevation. (All collars had a transverse width of 10 cm from the abutment face).

8.4.3 Streamwise Collar Length

At the upstream edge of the collar a shallow scour hole perpendicular to the flow was found in Tests T2 ~T4. This scour hole started from the main channel bank and went transversely towards the opposite channel wall and was connected to the scour hole at the main channel edge of the collar. This scour hole remained at the collar leading edge and, therefore, did not threaten the abutment. In Tests T5 and T6, the upstream end of the collars was still buried in the sand at the end of the experiments and, therefore, no scour was found. The upstream end of the collar should be long enough such that the scour hole won't threaten the abutment.

There was always scour downstream of the trailing edge of the collar. For Test T3 and T4, the trailing edge of the collar ended in the middle of the bridge crossing and scour holes of more than 1.92 cm at the abutment were found in both cases. These scour holes posed a threat to the middle of the abutment structure and may be eliminated simply by extending the trailing edge of the collar to a location that is downstream of the abutment structure (Figure 8-6). The extension of the downstream collar length may increase scour magnitude. This was observed in Tests T5 and T6 where the scour hole was more than 5 cm in Test T5 and more than 6 cm in Test T6. The scour location is not in the bridge crossing, however, and, therefore, should not correspond with a pier location.

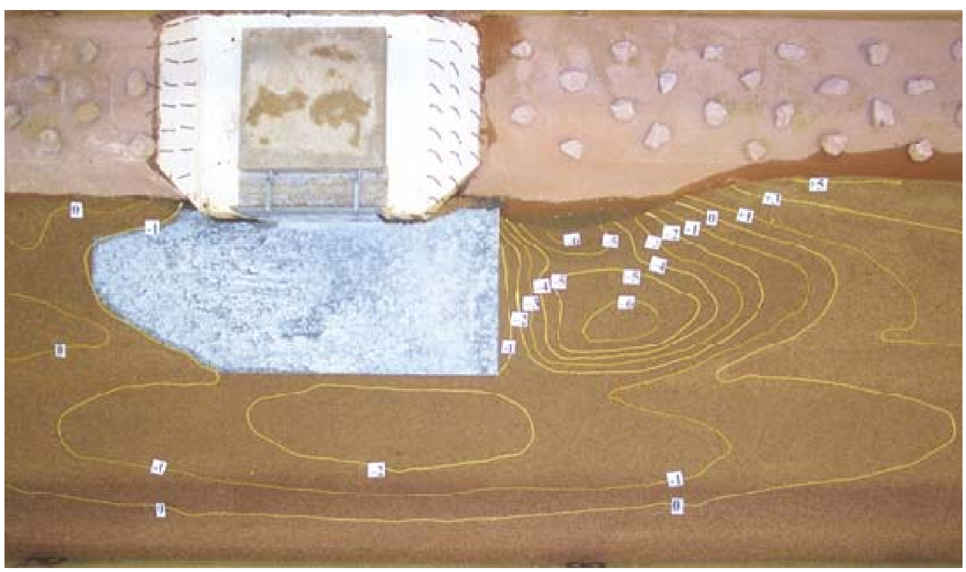


Figure 8-6: Scour contour of T6 with a collar attached along the abutment. The collar width is 35 cm. Collar elevation is 1 cm below the original bed. Flow from left to right.

8.4.4 Temporal Scour Variation

It was observed that, unlike the rapid scour at the upstream and downstream abutment corners in the baseline case, the scour in the first ten hours under the main channel edge of the collar was very slow in all collar cases. Figure 8-7 is a plot of the temporal evolution of scour under the edge of the plate in Test T3. This delayed scour constitutes another advantage of using the abutment collars.

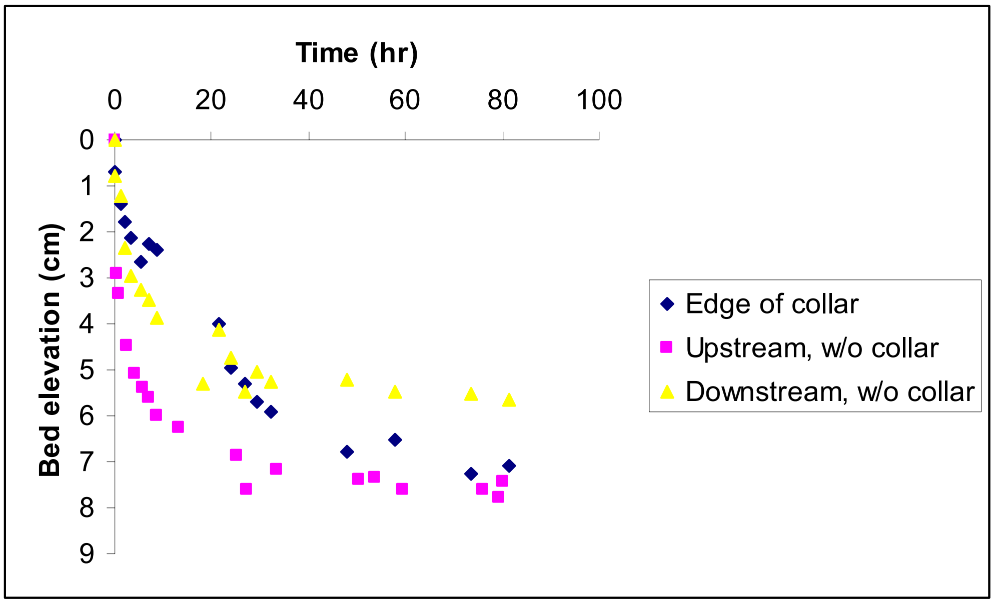


Figure 8-7: Plot of the scour depth variation under the main channel edge of the collar and for both the upstream and downstream scour holes versus time in Test T3. Note delayed scour in first 10 hours by collars.

8.5 Conclusions

From these clear-water experimental data, it can be concluded that:

1. Collars were found to be effective at preventing local scour at vertical wall bridge abutments. The collars isolated the turbulent flow and vortex systems from the bed material and therefore prevented the bed underneath the collar from scouring.

2. The further the collar extended downstream of the abutment, the farther downstream the scour hole was located. As the transverse width of the collars increased, the

depth of the scour hole at the edge of the collar decreased. The scour became insignificant as the main channel edge of the collar was extended beyond the local scour hole area measured in the baseline case without countermeasures. The trailing edge of the collar should extend to a location downstream of the abutment.

3. Based on these experiments the collar elevation should be $0.08y_m$ below the original bed level and the collar width should be at least 0.23L, where L is the abutment length perpendicular to the flow direction.

CHAPTER 9. SUMMARY

9.1 Summary

Scour at bridge abutments can cause damage or failure of bridges and result in excessive repairs, loss of accessibility, or even death. To mitigate abutment scour, both clear-water and live-bed laboratory experiments in a compound channel were performed using parallel walls and spur dikes. In addition, collars were also tested under clear-water conditions only.

Two types of parallel walls were tested: the first type was made of a wood plate and the second was made of piled rocks. For solid parallel walls, a series of rectangular straight plates of different length attached to the upstream end of a wing wall abutment parallel to the flow direction were employed. The velocity of the flow for the three cases was either 0.9, 1.5 or 2.3 times the incipient motion velocity for bed sediment movement. The bed material was sand with a mean diameter of 0.8 mm and a standard deviation of 1.37. All the plates were seated at the bottom of the compound channel bank slope and were even with the abutment face.

It was found that straight plates thus situated caused the scour hole to be shifted away from the upstream corner of the abutment and to be effective as a countermeasure to prevent scour there. As the length of the plate increased, the scour at the abutment declined. It was found that a length of 1.6L, with L being the length of the abutment perpendicular to the flow, caused the scour to be eliminated at the abutment for a velocity ratio (U/U_c) of 0.9 (clear-water scour). Similarly, a 1.6L long wall can eliminate the time-averaged scour depth at the abutment 100 percent for a velocity ratio of 1.5, and 70 percent for a velocity ratio of 2.3. If the upstream end of the wall is anchored below the scour depth, this countermeasure can be feasible for situations where rock is expensive.

For parallel rock walls, various values of wall length and protrusion length into the main channel were tested. It was found that a wall that does not protrude into the main channel with a length of 0.5L minimizes scour at the abutment for all three different flow velocity ratios (0.9, 1.5, and 2.3).

Experiments using spur dikes as a countermeasure against local scour at wingwall abutments were made in a laboratory flume channel. Flow velocity ratios of 0.9, 1.5 and 2.3 times the incipient motion of the bed material sediment were used in a compound channel with a model abutment. The bed material was sand with a median diameter of 0.8 mm and a standard deviation of 1.37. A series of configurations of spur dikes with varying lengths, spacings, number, and positions with respect to the abutment were tested. The most effective configuration to prevent local scour at the abutments consisted of three spur dikes composed of rock located upstream of the abutment and at the two corners.

To mitigate abutment scour, flat, horizontal, steel collars were attached around a wing-wall abutment ending at the main channel edge under clear-water flow conditions in a laboratory flume channel. It was found that these collars were able to protect the bridge abutment efficiently by eliminating secondary vortices that ordinarily would cause local scour. The minimum collar dimensions that eliminated local scour were a flow perpendicular width of 0.23 L (L is the abutment length perpendicular to the flow direction) and a flow parallel length of 0.7 times the flow parallel abutment width. It was determined that a vertical location of 0.08 y_m (where y_m is the main channel flow depth) below the mean bed sediment elevation gave the best results of scour reduction. In addition, the collar not only reduced scour magnitude near the abutment, but also retarded the development of the scour hole.

9.2 Limitations of Study

There are some limitations about this study. For instance, the length of the abutment was fixed, so were the floodplain width and the contraction ratio of the channel at the bridge crossing. If the floodplain width increases, there could be more returning floodplain flow and cause more scour at the parallel wall and the abutment. Also, the highest velocity ratio tried herein is 2.3. This ratio could be higher in the real field during storm events. Solid walls with length of equal or more than 1.9L could be tested further. The rigid floodplain could also be a limitation because in the real situations floodplains are generally erodible. The results here assume that the channel embankment does not erode. This corresponds to the case of cohesive channel banks, which is not uncommon. However, this should not have a major effect since the safety of the walls is not dependent on the floodplain. Since these results were obtained from the experiments at a greatly reduced scale, extrapolating to full-scale conditions could cause distortions. The velocity ratio (U/U_c), a commonly used similarity control variable in all hydraulic experiments balancing the hydraulic forces with the sediment resisting forces, was used to control the similarity of the experiments. In addition, bed forms typical of natural rivers were observed throughout the experiments. This does not guarantee, however, that there will not be differences between the laboratory and field results due to differences in scaling of the turbulent eddies and vortices formed. Scour produced at a small scale is usually more severe than that in the filed, however, making the results of the lab study conservative.

9.3 Future Work

The design guidelines offered here for each countermeasure are developed under limited laboratory conditions. More work can follow after this study. For instance, it would be necessary to test the effect of roughness on the floodplain on the scour depth at the abutment. Channel setup with erodible floodplain, non-uniform sediments will make the experiments more realistic. Different abutment orientations, length, shapes, and configurations with higher flow velocity ratios can be tested.

Effect of permeability of rock structures on their own stability and protection efficiencies can be studied. Collars as countermeasures against abutment scour under live-bed conditions are difficult to test since it is hard to monitor and measure the scour depth under the collar around the abutment. However, it should be tested for the integrity of the study. Also, methods of how to attach a collar to a bridge abutment should also be studied to make this collar as practical as other countermeasures.

A 3-D numerical model can be employed to simulate the flow patterns around these countermeasures to have a better understanding of how these structures work and to facilitate design.

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