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Local scour at abutments: A review

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Abstract. Failure of bridges due to local scour has motivated many investigators to explore the causes of scouring and to predict the maximum scour depth at abutments. In this paper, a detailed review of the up-to-date work on scour at abutments is presented including all possible aspects, such as flow field, scouring process, parameters affecting scour depth, time-variation of scour and scour depth estimation formulae.

Keywords. Abutments; open channel flow; erosion; scour; sediment transport; hydraulics.

1. Introduction

Scour is a natural phenomenon caused by the erosive action of flowing stream on alluvial beds. Failure of bridges due to scour at their foundations, which consist of abutments and piers, is a common occurrence. A study of the US Federal Highway Administration in 1973 concluded that of 383 bridge failures, 25% involved pier damage and 72% involved abutment damage (Richardson *et al* 1993). In a report submitted to the National Roads Board of New Zealand, Sutherland (1986) pointed out that of 108 bridge failures recorded, 29 were attributed to abutment scour during 1960–1984. According to Kandasamy & Melville (1998), 6 of 10 bridge failures that occurred in New Zealand during Cyclone Bola were related to abutment scour. In another report of the Department of Scientific and Industrial Research (DSIR) of New Zealand, Macky (1990) mentioned that about 50% of total expenditure was made towards bridge damage repairing and maintenance, out of which 70% was spent towards repairing abutment scour. Numerous equations have been developed to predict the magnitude of scour depth at abutments; and a large database of flume measurements is now available in the published literature. This paper presents a review of investigations conducted on local scour at bridge abutments. The flow field, scouring process, parameters affecting scour depth and

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A list of symbols is given at the end of the paper

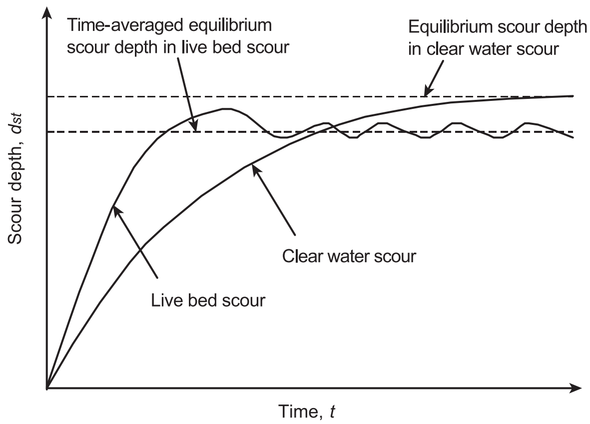


Figure 1. Time development of clear-water and live-bed scour (after Chabert & Engeldinger 1956).

time-variation of scour are discussed in detail. Equations for predicting maximum scour depth at abutments are outlined. As scour at groynes or spur-dikes are similar to that at long and thin abutments, investigations on scour at groynes or spur-dikes are also included.

2. Classification of local scour at abutments

Based on the mode of sediment transport by the approaching flow, Chabert & Engeldinger (1956) classified local scour into two categories, namely, clear-water scour and live-bed scour.

Clear-water scour takes place in the absence of sediment transport by the approaching flow into the scour hole. On the other hand, live-bed scour occurs when the scour hole is continuously fed with sediment by the approaching flow. The time-variation of clear-water and live-bed scours are shown schematically in figure 1. It was observed that maximum clear-water scour depth is approximately 10% greater than the live-bed scour depth.

3. Flow field and bed shear stress at abutments

The flow field at an abutment embedded vertically in a loose sediment bed of an open channel is complex in nature; and the complexity increases with the development of a scour hole involving separation of flow to form a three-dimensional vortex flow system at the base of the abutment. Though the flow field at piers has been well documented (Hjorth 1975; Melville 1975; Dey 1995; Dey *et al* 1995; Graf & Istiarto 2002), research on the flow fields at groynes or spur-dikes and abutments has been limited. A brief review of investigations on flow fields at groynes or spur-dikes and abutments is given below:

Rajaratnam & Nwachukwu (1983) measured the flow field and the shear stress at groynes placed on a planar bed. They found that the shear stress at the upstream corner of the groyne was amplified up to 5 times the approaching bed shear stress. Kwan (1988) and Kwan & Melville (1994) identified a primary vortex, which is similar to the horseshoe vortex at piers, along with the downflow as being the principal cause of scouring at abutments. At the upstream face of an abutment, a vertical pressure gradient is developed due to stagnation of

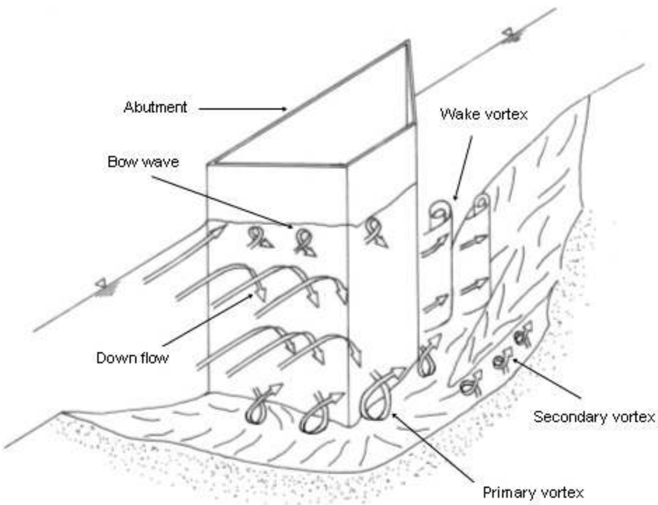


Figure 2. Schematic diagram of flow field at an abutment (after Kwan 1988).

the approaching flow. The pressure gradient drives the fluid downward and this rolls up to become the primary vortex, which enlarges its size with the development of the scour hole. Kwan & Melville (1994) also reported that the primary vortex and the downflow are confined mainly to the scour hole beneath the original bed level. The vortex flow and the downflow are relatively unaffected with change of approaching flow depth. The inner core of the primary vortex occupies 17% of the area of the scour hole and contains up to 78% of the total circulation in the flow. The primary vortex is elliptical in shape, with an inner core region being a forced vortex and an outer core region a free vortex. The maximum velocity and downflow component in the vicinity of the abutment are 1·35 and 0·75 times of the approaching flow velocity respectively. They identified a secondary vortex, with counter-rotational direction to that of the primary vortex, occurring next to the primary vortex. The secondary vortex is believed to have the effect of restricting the scouring capacity of the primary vortex. Downstream of the abutment, wake vortices are created due to the separation of flow upstream and downstream of the abutment corners. Unstable shear layers created due to the flow separation roll up to form eddy structures, termed wake vortices. The wake vortices that drift downstream owing to the mean flow act like small tornadoes and lift up sediments from the bed. These wake vortices are rather weak compared to the primary vortex. Major flow components at a wing-wall abutment, identified by Kwan (1988), are shown schematically in figure 2.

Molinas *et al* (1998) conducted experiments on vertical-wall abutments for flow Froude numbers ranging from 0·3 to 0·9 and for protrusion ratios (ratio of abutment length perpendicular to direction of flow to total channel width) of 0·1, 0·2 and 0·3. They report that shear stresses at abutments are amplified up to 10 times the bed shear stress of approaching flow and velocities are increased up to 1·5 times the approaching flow velocity, depending on the flow conditions and abutment protrusion ratios. They express the total bed shear stress as being the sum of the bed shear stresses due to the contraction and the abutment structure alone. The shear stress at the contraction is a function of the opening ratio, flow Froude number and protrusion length. On the other hand, the shear stress at the nose of the abutments is a function of the opening ratio and turning angle. Accordingly, they proposed the equations of bed shear stresses as

$$\hat{\tau}_t = \hat{\tau}_{cont} + \hat{\tau}_{nose}, \tag{1a}$$

$$\hat{\tau}_{cont} = (1/\alpha)\left[1 + 5{\cdot}46(1/\alpha)^{3{\cdot}89}\,\mathrm{F}_\mathrm{r}^{1{\cdot}74}(1/h)^{2{\cdot}5}\right], \tag{1b}$$

$$\hat{\tau}_{nose} = (1/\alpha^2)(1+\tan^2\theta_t)^{1/2} - 1. \tag{1c}$$

Using a two-dimensional depth-averaged k–ε model, Biglari & Sturm (1998) developed a numerical model of flow at vertical-wall abutments located on the floodplain of a compound channel. The numerical results were applied in a correlation of measured equilibrium clear-water scour depth with numerical values of local velocity near the upstream corner of the abutment. According to Ahmed & Rajaratnam (2000), the bed shear stress near the nose of the wing-wall abutment is amplified by nearly 3·63 times the bed shear stress of approaching flow. In comparison to the flow near a bridge pier, a greater skewness of velocity distributions is observed near an abutment, especially downstream. It shows the limitation of treating the abutment as a half-pier.

Flow fields at a wing-wall, semicircular and vertical-wall abutments on the plain bed were detected by Barbhuiya & Dey (2003, 2004a), using the acoustic doppler velocimeter (ADV). Also, Barbhuiya (2003) and Barbhuiya & Dey (2003b) measured the flow fields in scour holes at different vertical or azimuthal sections (figure 3) of vertical-wall, wing-wall and semicircular abutments. The time-averaged velocity components (u, v, w) were represented

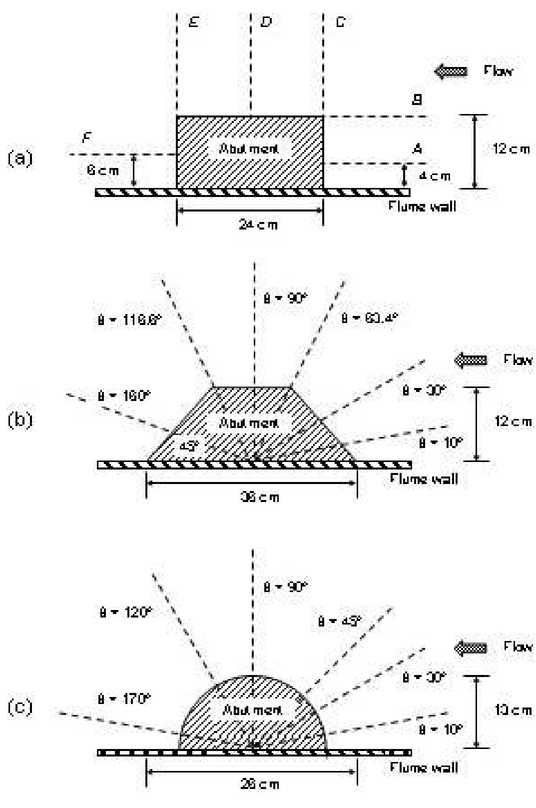


Figure 3. Sections of flow measurements (after Barbhuiya & Dey 2003b): **(a)** Vertical-wall abutment, **(b)** 45° wing-wall abutment and **(c)** semicircular abutment.

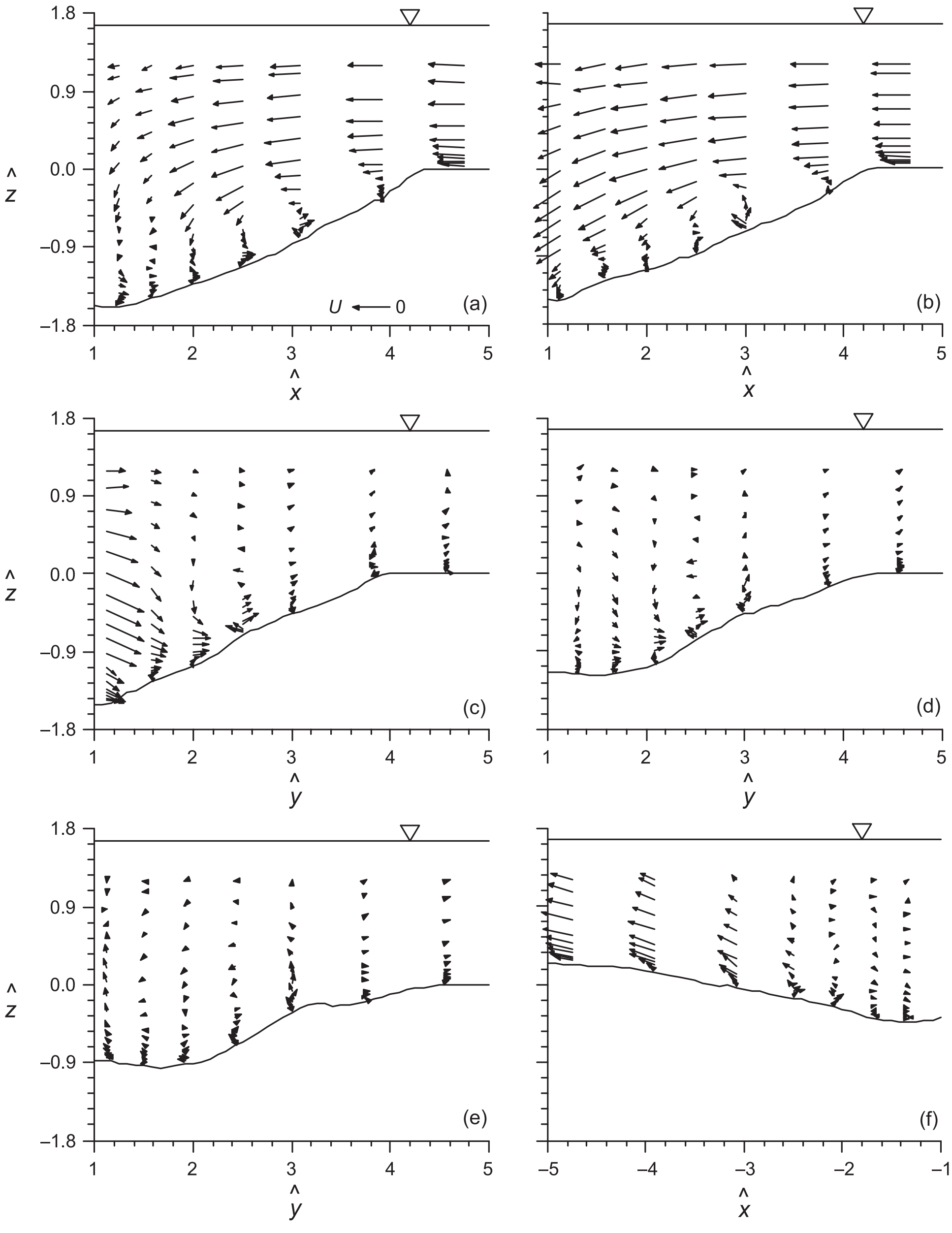


Figure 4. Normalized velocity vectors at vertical sections of a vertical-wall abutment (after Barbhuiya & Dey 2003b): **(a)** A, **(b)** B, **(c)** C, **(d)** D, **(e)** E and **(f)** F.

in Cartesian coordinates (x, y, z) for vertical-wall abutments. On the other hand, cylindrical polar coordinates (θ, r, z) were used to represent velocity components (u, v, w) for wing-wall and semicircular abutments. All linear dimensions and velocity components were normalized by the abutment length l and average approaching flow velocity U respectively. For vertical-wall abutment, the flow fields are shown in $\hat{x}\hat{z}$- and $\hat{y}\hat{z}$-planes at different vertical sections (figure 4); where $\hat{x}$ is x/l, x is the streamwise distance, $\hat{y}$ is y/l, y is the transverse distance,

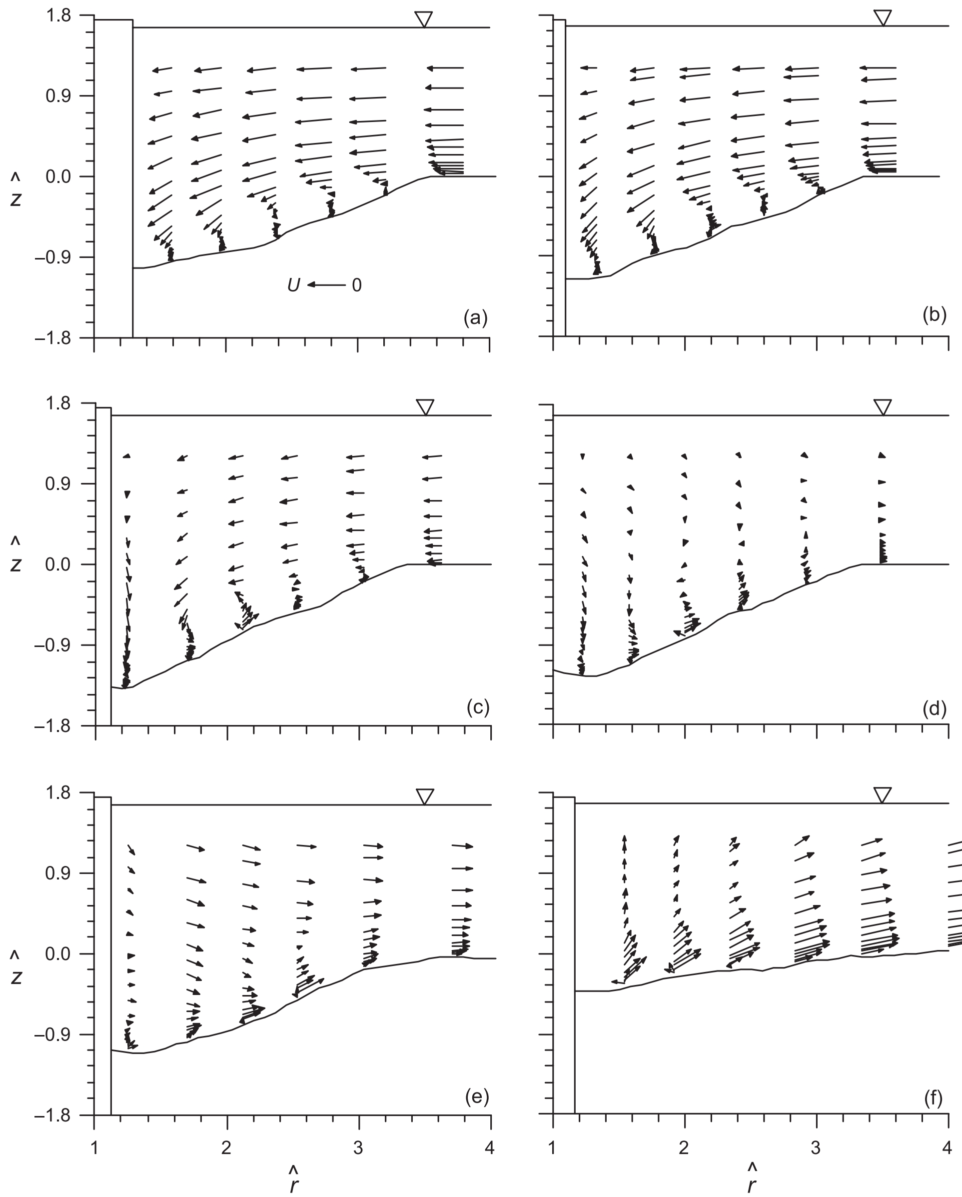


Figure 5. Normalized velocity vectors at azimuthal sections of a 45° wing-wall abutment (after Barbhuiya & Dey 2003b): $\theta = 10°$ **(a)**, 30° **(b)**, 63·4° **(c)**, 90° **(d)**, 116·6° **(e)** and 160° **(f)**.

z is the vertical distance, and $\hat{z}$ is z/l. On the other hand, the flow fields are depicted in the $\hat{r}\hat{z}$-plane at different azimuthal angles θ for wing-wall and semicircular abutments (figures 5 and 6); where $\hat{r}$ is r/l, and r is the radial distance. Observations on flow fields at vertical-wall, wing-wall and semicircular abutments are as follows (Barbhuiya & Dey 2003b).

For vertical-wall abutment, figure 4 exhibits the normalized time-averaged velocity vectors, whose magnitudes and directions, at sections A, B and F, are $(\hat{u}^2 + \hat{w}^2)^{1/2}$ and $\arctan(\hat{w}/\hat{u})$, respectively; and at sections D–E, $(\hat{v}^2 + \hat{w}^2)^{1/2}$ and $\arctan(\hat{w}/\hat{v})$; where $\hat{u}$ is u/U, $\hat{v}$ is v/U,

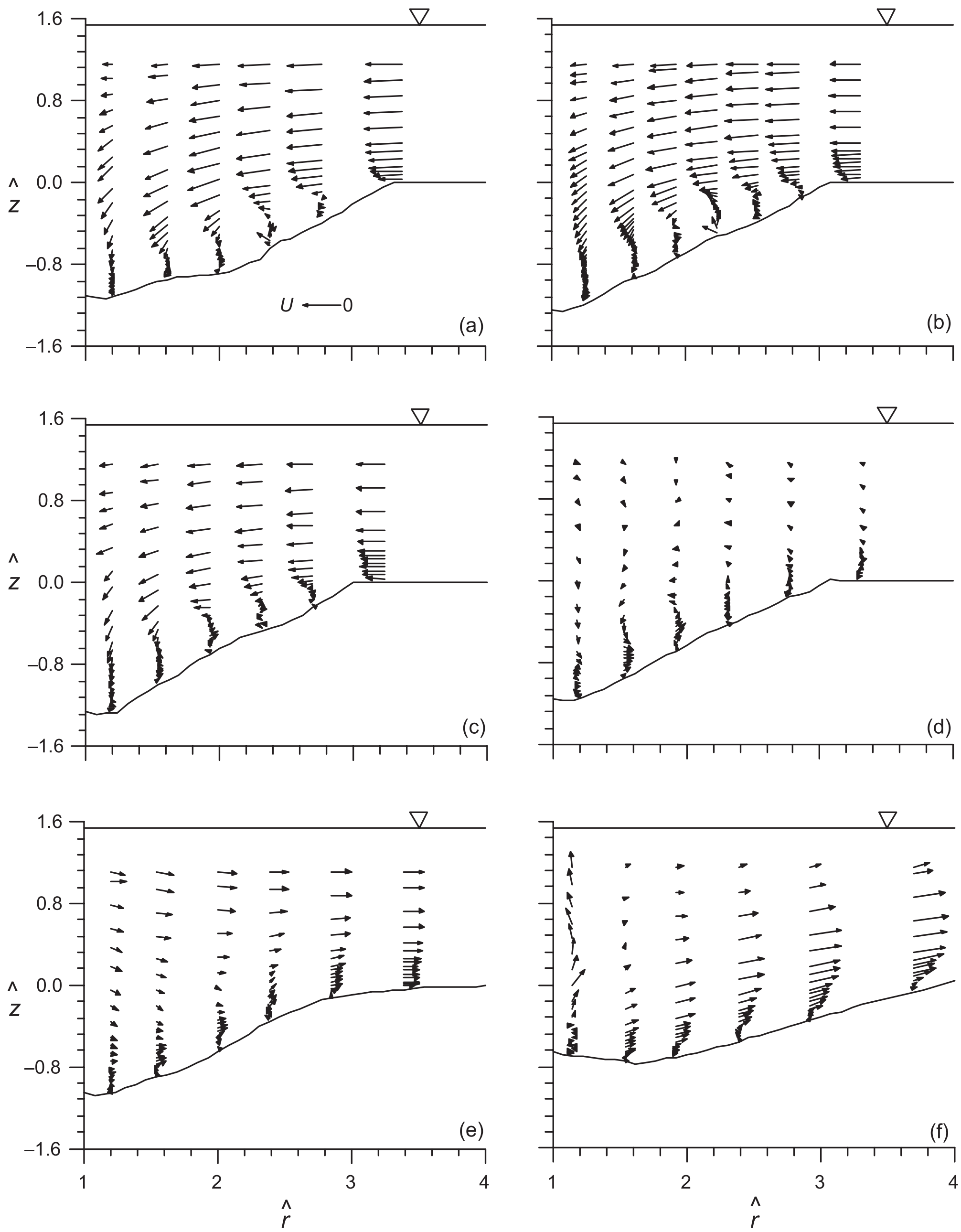


Figure 6. Normalized velocity vectors at azimuthal sections of a semicircular abutment (after Barbhuiya & Dey 2003b): $\theta = 10°$ **(a)**, $30°$ **(b)**, $45°$ **(c)**, $90°$ **(d)**, $120°$ **(e)** and $170°$ **(f)**.

and $\hat{w}$ is w/U. It is evident from the flow pattern upstream of the abutment that the flow separation at the edge of the scour hole produces a reversal of the flow field inside the scour hole. This confirms a strong helicoidal flow (primary vortex) inside the scour hole upstream of the abutment. The primary vortex is strong upstream of the abutment (see sections A–C) and decreases by the side of the abutment (see sections D and E). In addition, the downflow at the upstream face of the abutment adds fluid to make the primary vortex stronger. To be more explicit, at the upstream face of the abutment, an adverse pressure gradient creates a downflow being returned at the base of the scour hole, showing a strong counter-clockwise vortex, which is already referred to as the primary vortex. However, at section C, the deflection of flow at the outer edge of the abutment is prominent. At sections D and E, vortex motion is very feeble, confirming its attenuation. In the downstream (at section F), the flow field shows an upward flow near the abutment, causing suction, and further downstream, the flow directed outward becomes part of the main flow. Figures 5 and 6 display the normalized time-averaged velocity vectors at different azimuthal planes, having magnitude $(\hat{v}^2 + \hat{w}^2)^{1/2}$ and direction $\arctan(\hat{w}/\hat{v})$, for 45° wing-wall and semicircular abutments respectively. The nature of the flow field at wing-wall and semicircular abutments are almost similar, where the characteristics of the vortex flow inside the scour hole together with the downflow along the upstream face of the abutments are depicted. Circulation is strong upstream of the abutment and decreases with increase in θ. At 90°, vortex motion is very feeble. In the downstream, the flow is directed outwards with further increase in θ beyond 90°. At 160° (for 45° wing-wall abutment) and 170° (semicircular abutment), it shows an upward flow near the abutment, causing suction, and further downstream, the flow becomes part of the main flow.

4. Parameters related to scour at abutments

4.1 *Classification of parameters*

Parameters involved in the scour phenomenon at abutments can be grouped as follows.

(1) Parameters relating to the geometry of channel: width, cross-sectional shape and slope.
(2) Parameters relating to the abutment: size, shape, orientation with respect to main flow and surface condition.
(3) Parameters relating to the bed sediment: median size, grain size distribution, mass density, angle of repose and cohesiveness.
(4) Parameters relating to the fluid: density, viscosity, gravitational acceleration and temperature.
(5) Parameters relating to the approaching flow condition: mean flow velocity, flow depth, shear velocity and roughness.
(6) Time of scouring can be taken as an additional parameter for an evolving scour hole.

4.2 *Dimensional analyses*

Using the Buckingham p-theorem and employing physical reasoning, various investigators combined the appropriate parameters affecting the scour depth at an abutment in different nondimensional forms, which are as follows.

Garde *et al* (1961) gave the nondimensional parameters for scour at spur-dikes as

$$(d_s + h)/h = f(\alpha, \theta_a, \mathrm{F_r}, C_D), \tag{2}$$

where θ_a = angle of inclination of the spur-dike with respect to the main flow, termed angle of attack, $C_D = 4\Delta gd/(3\rho w_s^2)$, $\Delta = s - 1$, $s = \rho_s/\rho$.

Neglecting the effect of viscosity, Melville (1992) expressed the nondimensional parameters as

$$d_s/l = f\left(U^2/gd, h/l, d/l, \sigma_g, K_s, K_\theta, K_G\right), \tag{3}$$

where $\sigma_g = (d_{84}/d_{16})^{0\cdot5}$.

Sturm & Janjua (1994) used discharge contraction ratio instead of channel contraction ratio to analyze scour at vertical-wall abutments. They proposed a nondimentional relationship as

$$d_s/h = f(\mathrm{F_r}, \mathrm{F_{rc}}, M), \tag{4}$$

where $\mathrm{F_{rc}} = U_c/(gh)^{0\cdot5}$, and M = discharge contraction ratio, defined as the ratio of the discharge at approaching section through the opening width to the total discharge.

Lim (1997) obtained the following nondimensional parameters for scour at vertical-wall abutments neglecting the channel contraction effects:

$$d_s/h = f\left(\mathrm{F_d}, h/d, l/h, \sigma_g, K_s, \theta_a, K_G\right). \tag{5}$$

5. Influence of parameters on scour depth

5.1 *Approaching flow velocity*

The effect of approaching flow velocity U is incorporated in the scour predicting formulae in the form of flow Froude number $\mathrm{F_r}$ or shear velocity u_*. Garde *et al* (1961), Zaghloul & McCorquodale (1975), Zaghloul (1983), Rajaratnam & Nwachkwu (1983) and Froehlich (1989) included the flow Froude number in their analyses. Garde *et al* (1961) concluded that the flow Froude number for the normal channel flow adequately represents the effect of approaching flow velocity on the maximum scour depth. Kandasamy (1989) showed that the scour depth increases with increase in flow depth due to incorporation of the flow Froude number.

It is generally recognized that the shear velocity u_* is an important parameter not only in distinguishing clear-water scour from live-bed scour but also in representing the erosive power of the flowing stream for a given sediment size. Clear-water scour occurs for approaching flow velocity up to the critical velocity U_c for bed sediments, that is $U/U_c \leq 1$; while live-bed scour occurs when $U/U_c > 1$. For nonuniform sediments, Melville & Sutherland (1988) defined an armor velocity U_a, which marks the transition from clear-water to live-bed conditions for sediment-transporting flow and is equivalent to U_c for uniform sediments. Thus, for nonuniform sediments, live-bed conditions prevail when $U/U_a > 1$. However, if $U/U_a < 1$, armoring of the bed occurs as scouring proceeds and clear-water conditions exist. Laursen (1958, 1960, 1963), Gill (1972), Wong (1982), Tey (1984), Kwan (1984), Kandasamy (1989) and Melville (1992, 1995, 1997) considered shear velocity in their approach. Dongol (1994) conducted an extensive series of experiments to study the effect of approaching flow velocity on scour depth at vertical-wall, wing-wall and spill-through abutments under live-bed conditions in uniform and nonuniform sediments. His results are complimentary to the studies of Chiew (1984) and Baker (1986) for live-bed scour at bridge piers in uniform and nonuniform sediments, respectively.

It is recognized that under clear-water conditions, the maximum scour depth occurs when $U = U_c$. This scour depth is called the threshold peak. For $U/U_c > 1$, that is under live-bed conditions, scour depth initially decreases with increase in approaching flow velocity

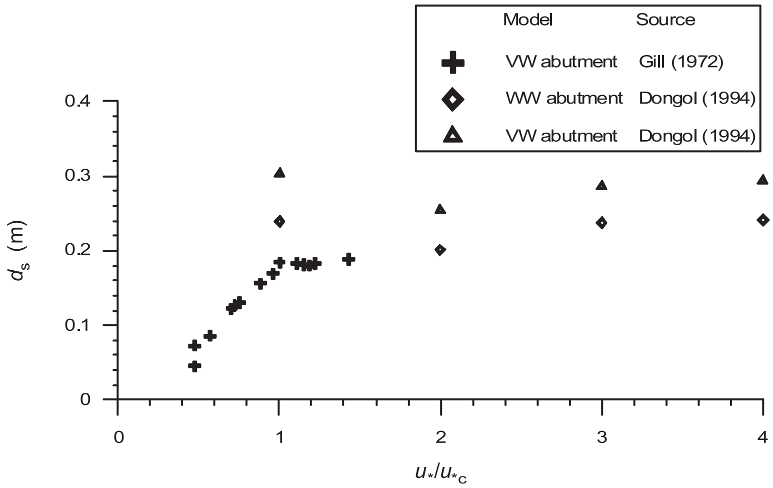


Figure 7. Variation of scour depth d_s with shear velocity ratio u_*/u_{*c}.

reaching a minimum value and then increases again toward a second maximum. The second maximum occurs at about the transitional flatbed stage of sediment transport on the channel bed and is termed live-bed peak. Data collected from different sources are plotted in figure 7 showing the effect of shear velocity ratio u_*/u_{*c} on scour depth.

5.2 *Approaching flow depth*

According to Laursen (1952), the approaching flow depth h is an important factor to determine scour depth. Experimental results of Gill (1972), Wong (1982), Tey (1984) and Kandasamy (1989) indicate that for constant value of the shear velocity ratio u_*/u_{*c}, the maximum scour depth increases with increase in approaching flow depth. It is also observed that the maximum scour depth increases at a decreasing rate with increase in approaching flow depth. After Kandasamy (1989), for shallow flow depths, the scour depth increases proportionally with h, but is independent of l. On the other hand, for intermediate flow depths, the scour depth depends on both h and l. However, Melville (1992) distinguished short and long abutments. He concluded that for short abutments ($l/h \geq 1$), the scour depth is independent of flow depth; and for long abutments ($l/h \geq 25$), the scour depth is dependent on flow depth. However, most abutments are neither long nor short, as a result of which the scour depth is influenced by both h and l. Dey & Barbhuiya (2004a) reported that for smaller flow depths, the equilibrium scour depth increases significantly with increase in h; where as for higher flow depths, equilibrium scour depth is independent of flow depth.

There is consensus that the maximum scour depth increases at a decreasing rate with increase in approaching flow depth and there exists a limiting depth corresponding to which the maximum scour depth is independent of the flow depth. Data collected from various sources are plotted in figure 8 to represent the variation of scour depth with flow depth.

5.3 *Abutment length, contraction ratio and opening ratio*

Abutment length and contraction ratio have extensively been used in formulating the maximum scour depth at abutments. The inverse of the opening ratio is termed contraction ratio.

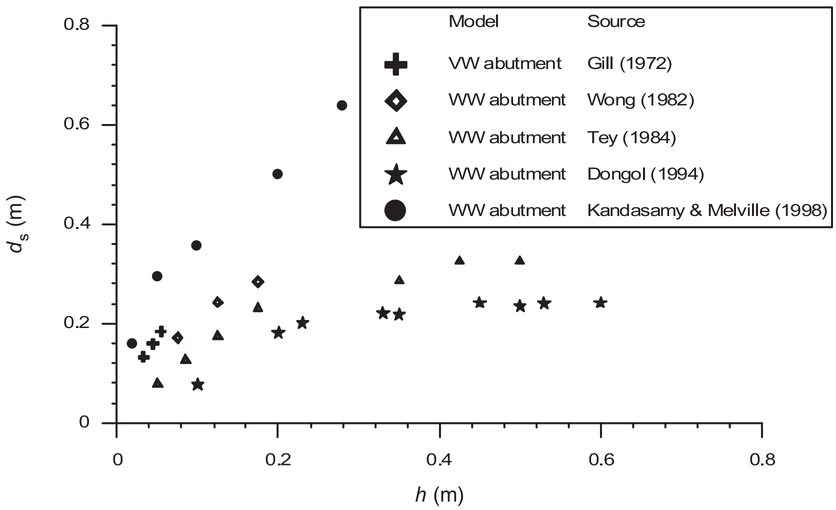


Figure 8. Variation of scour depth d_s with flow depth h.

Kandasamy (1989) pointed out that if the length of the abutment is increased, the opening ratio decreases, and the effect on scour depth of such a change can be ascribed to both a decrease in contraction ratio and an increase in abutment length. Garde *et al* (1961), Gill (1972), Zaghloul & McCorquodale (1975), and Rajaratnam & Nwachukwu (1983) used contraction ratio in their analyses. Neill (1973) argued that the use of contraction ratio as a scaling parameter cannot be justified in the case of a short abutment projecting into a very wide channel, and that contraction might be regarded as a secondary influence. Extending the same argument, Neill (1973) concluded that it is logically fallacious to express the results primarily in terms of contraction ratio. Cunha (1975) found that for flow without continuous sediment motion, scour depth does not depend on the contraction ratio and is only affected by local phenomena. A similar conclusion was also drawn by Liu *et al* (1961) for abutments and Laursen (1963) for bridge piers and abutments.

Laursen (1963), Neill (1973), Cunha (1975), Wong (1982), Tey (1984), Kandasamy (1989), Melville (1992) and Cardoso & Bettess (1999) advocated that scour is a local phenomenon and is independent of contraction ratio. Accordingly, abutment length has been proposed as scaling parameter. The convincing arguments are that as long as the scour hole does not extend to the opposite bank of the stream or the flume-wall, with other conditions being the same, maximum scour depth at an abutment of a fixed length is the same irrespective of the flume width or stream. A conciliatory approach would be to use contraction ratio and abutment length when the extent of the scour hole is affected and unaffected by the opposite bank or the flume-wall respectively. Figure 9 shows the dependency of equilibrium scour depth on abutment length.

5.4 *Size and gradation of sediments*

Characteristics of the bed sediments are derived from particle size distribution curves. The two most commonly used parameters are median sediment diameter d_{50} (or d) and geometric standard deviation $\sigma_g[= (d_{84}/d_{16})^{0\cdot5}]$ of particle size distribution, which is a measure of uniformity of the bed sediments. Laursen & Toch (1956), Ahmad (1953) and Izzard

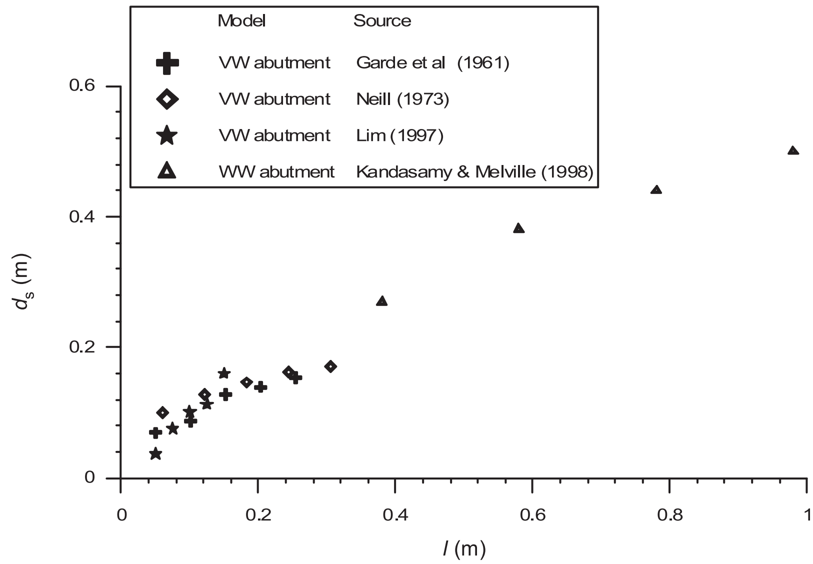


Figure 9. Variation of scour depth d_s with abutment length l.

& Bradley (1958) stated that the maximum scour depth is independent of the sediment size. Blench (1957), Garde *et al* (1961) and Gill (1972) reported that sediment size has an influence on the maximum scour depth. Laursen (1960) found that maximum scour depth is affected by sediment size under clear-water scour but not under live-bed scour. Results of Gill (1972) for two sediment sizes ($d = 1{\cdot}52$ mm and $0{\cdot}914$ mm) indicate that for the same value of $\tau_o/\tau_c < 1$, scour depth is greater with coarse sediments than with fine sediments, where $\tau_c =$ critical shear stress for sediment particles. However, for the same value of absolute approaching bed shear stress, fine sediments produce greater scour depth. For wing-wall, spill-through and semicircular abutments, Wong (1982) found that scour depth increases with increase in bed sediment size for a constant value of τ_o/τ_c, which is close to unity.

For live-bed scour in uniform sediments, the amount of sediment being transported by the approaching flow as bed load into the scour hole and that being picked-up from the scour hole at an abutment are the same at equilibrium conditions. Since sediment size does not have any effect on the existing balance of sediment continuity, the equilibrium scour depth is unaffected by change in sediment size.

Ahmad (1953), Izzard & Bradley (1958), Garde *et al* (1961) and Gill (1972) found that the rate of scour is different for different bed sediments. According to them, fine sediments are scoured at a faster rate than course sediments. Ramu (1964) observed that for the same sediment size, a change in sediment gradation σ_g affects the equilibrium scour depth. Ahmad (1953) asserted that equilibrium scour depth depends on the sediment gradation σ_g. Ettema (1980) and Raudkivi & Ettema (1983) found that the maximum clear-water equilibrium scour depth d_s at a bridge pier depends on the sediment grading σ_g. Their proposed relationship for estimating equilibrium scour depth in nonuniform sediments in terms of the geometric standard deviation σ_g is given by

$$d_s(\sigma_g)/D = K_\sigma d_s/D \tag{6}$$

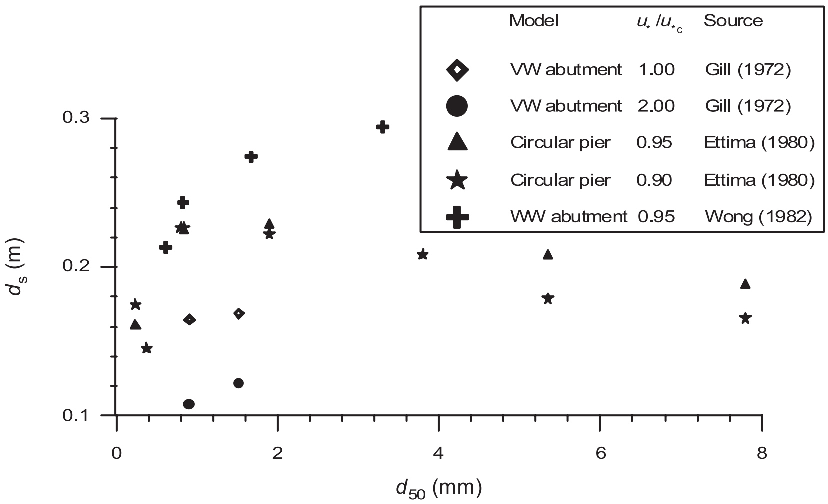


Figure 10. Variation of scour depth d_s with sediment size d_{50}.

Effects of sediment sizes and gradations are shown in figures 10 and 11 respectively. According to Dey & Barbhuiya (2004b), the effect of sediment gradation on scour depth is pronounced for nonuniform sediments, which reduces scour depth significantly due to the formation of armour-layers in scour holes.

Dey & Barbhuiya (2004b) conducted experiments to study the effects of thinly armour-layer on scour depth at abutments. They concluded that the scour depth at an abutment with an armour-layer in clear-water scour condition under limiting stability of surface particles (approaching flow velocity nearly equaling critical velocity for threshold motion of surface particles) is always greater than that without an armour-layer for the same bed sediments.

5.5 *Abutment shape*

The shape of the abutment plays an important role on equilibrium scour depth. Streamlined bodies, such as semicircular (SC), spill-through (ST) and wing-wall (WW) abutments,

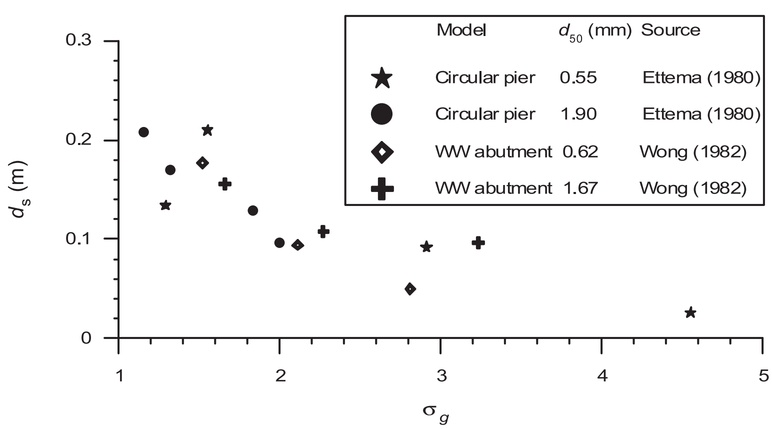
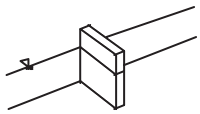
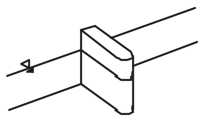
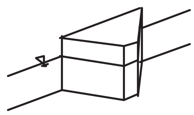
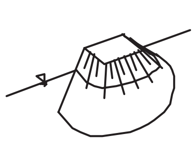


Figure 11. Variation of scour depth d_s with sediment gradation σ_g.

Table 1. Abutment shape factors.

Abutment model	Abutment shape	Shape factor, K_s
	Vertical-wall	1·00
	Semicircular ended	0·75
	45° wing-wall	0·75
	Spill-through with slope horizontal : vertical	
	0·5 : 1	0·60
	1 : 1	0·50
	1·5 : 1	0·45

produce vortice of feeble strength; while blunt obstructions, for example vertical-wall abutments, are capable of producing strong turbulent vortices. Consequently, a relatively large scour depth is observed at a blunt obstruction. From laboratory experimental data, Laursen & Toch (1956), Liu *et al* (1961), Garde *et al* (1961) and Wong (1982) concluded that vertical-wall abutments produce greater scour depth in comparison with spill-through and wing-wall abutments.

Melville (1992, 1995, 1997) used shape factor K_s to account for the effect of the shape of abutments on equilibrium scour depth. Commonly used abutment shapes and corresponding values of the shape factors are furnished in table 1. The vertical plate is the simplest shape of an abutment and is, therefore, used as reference. For spill-through abutments, the abutment length is taken as the length at mid-depth in the flow. The shape factors given in table 1 are derived from laboratory experimental data of Gill (1972), Wong (1982), Tey (1984), Kwan (1984, 1988), Kandasamy (1989) and Dongol (1994). However, Melville (1992) asserted that the importance of abutment shape diminishes when the abutment becomes longer. Thus, for $l/h \geq 10$, an adjusted shape factor K_s^* has been recommended. The adjusted shape factor K_s^* varies linearly within the value of K_s at $l/h = 10$ and unity at $l/h = 25$.

5.6 *Abutment alignment*

The angle of approaching flow with respect to the abutment alignment, termed angle of attack, significantly influences scour depth. It was experimentally studied by Laursen & Toch (1956), Garde *et al* (1961), Zaghloul (1983) and Kwan (1984). Garde *et al* (1961) reported that for the same flow, sediment and abutment conditions, the maximum scour depth is greatest for a spur-dike with an inclination of 90°. For all other inclinations the scour depth is smaller. Similar observations were also made by Kwan (1984). Zaghloul (1983) reported that the greatest equilibrium scour depth is seen for an upstream spur-dike inclination, and the smallest when

Table 2. Flow alignment factor for different angles of attack.

θ_a (deg)	30	60	90	120	150
K_θ	0·90	0·97	1·00	1·06	1·08

the spur-dike is inclined downstream. The magnitude of the equilibrium scour depth at a spur-dike placed normal to the flow is in-between the magnitude of scour depths at spur-dikes inclined upstream and downstream. Melville (1992) included the effect of abutment alignment incorporating the alignment factor K_θ in the design equations. The values of K_θ given in table 2 were derived by the envelope curve method from the data of Ahmad (1953), Laursen (1958), Sastry (1962), Zaghloul (1983), Kwan (1984) and Kandasamy (1985). Melville (1992) recommended that the alignment factor can be applied only to longer abutments ($l/h \geq 3$). Alignment effects are negligible for short abutments ($l/h \leq 1$) having $K_\theta = 1$. For abutment lengths between these two limits, a value of K_θ obtained from the linear interpolation has been recommended.

5.7 *Channel geometry*

Cross-sections of rivers may have different shapes depending on the geographical location, the characteristics of sediments through which they pass and the characteristics of its catchment area. Normally, in hilly regions, the cross-section of a river is parabolic with steep side slope, whereas in the plains, its cross-section is compound, with floodplains and a main channel. Thus, for bridges in the hilly region, abutments are founded in the main channel. On the other hand, for bridges in the plains, all abutments including the approach embankment may terminate in the floodplain or may extend into the main channel. Richardson & Richardson (1998) argued that experimental results in rectangular flumes do not accurately reflect the abutment scour process in compound channels.

Froehlich (1989) considered the channel geometry effect in calculating approaching flow Froude number. He calculated the number, based on the average velocity and the depth in the area obstructed by the embankment and abutment at the approaching flow cross-section. A systematic investigation of the effect of channel geometry on scour depth at an abutment located in a compound channel, comprising floodplains and main channel, was done by Melville & Ettema (1993) and Melville (1995). The study is limited to the case of an abutment spanning the floodplains and extending into the main channel. The effect of the channel geometry on scour depth is represented by a multiplying factor K_G, which is defined as the ratio of the scour depth at an abutment sited in a compound channel to a scour depth at an abutment sited in the corresponding rectangular channel of the same overall width as that of the compound channel and the same depth as that of the main channel of the compound section. In general, K_G depends on the size, shape, roughness of the main channel and floodplains, and the abutment length with respect to the floodplain width. Sturm & Janjua (1994) included discharge contraction ratio M in the equation of scour depth to consider the effect of channel geometry. They demonstrated that M represents the redistribution of flow between the main channel and the floodplains, as the flow passes through the bridge contraction. Cardoso & Bettess (1999) studied the effect of channel geometry by extending the abutment length up to the edge of the main channel. Results obtained by them were in conformity with the recommendations of Melville (1995) that scour at abutments on floodplains can be approximated by the scour in rectangular channels, if an imaginary boundary is assumed separating the flow in the main channel and the floodplain. However, they found that the time to reach equilibrium scour, when the scour hole extends into the main channel, is shorter than that to reach equilibrium scour, when the scour hole is confined to the floodplain.

5.8 *Time-variation of scour*

Figure 1 shows a schematic diagram of the time-variation of scour depth at a cylindrical pier (after Chabert & Engeldinger 1956). Time to reach equilibrium scour depth varies widely, ranging from a day to a fortnight. Anderson (1963) states "By virtue of the logarithmic character of the development of the scour region with time, a *practical equilibrium* is reached after a relatively short time, after which the increase in the depth and extent of scour becomes virtually imperceptible". Rouse (1965), however, states that scour is an ever-increasing phenomenon and there is no real equilibrium scour depth. Bresuers (1963, 1967) and Kohli & Hager (2001) agree with this view. However, Laursen (1952), Carstens (1966), Gill (1972) and Zaghloul (1983) believe that an equilibrium scour depth does exist.

Rouse (1965), Gill (1972), Rajaratnam & Nwachukwu (1983), Dargahi (1990), Ettema (1980), Kohli & Hager (2001), Oliveto & Hager (2002) and Coleman *et al* (2003) think that the variation of scour depth with time is logarithmic. Ahmad (1953), Franzetti *et al* (1982), Kandasamy (1989), Whitehouse (1997), Cardoso & Bettess (1999) and Ballio & Orsi (2000) propose an exponential time-variation of scour; while Bresuers (1967) and Cunha (1975) give a power law distribution. As per Dey & Barbhuiya (2004b), time variation of scour depth for uniform sediments is a family of parallel lines for different abutment lengths and sediment sizes. For nonuniform sediments, time variation of scour depth reduces with increase in nonuniformity of the particle size distribution of sediments. General consensus is that equilibrium scour depth at an abutment is attained asymptotically. Data collected from different sources are plotted on a logarithmic time-scale, as shown in figure 12. Equations relating the scour depth to time for abutments are limited. The equations available in the literature are given below.

Ahmad (1953) gives an equation of time-variation of scour depth at spur-dikes as

$$d_{st} = d_s(1 - me^{-nt}). \tag{7}$$

Laursen (1963) proposes the following differential equation for the temporal development of scour depth at vertical-wall abutments:

$$\frac{x_d^{3\cdot5}}{1 - x_d^{1\cdot5}}\mathrm{d}x_d = 0\cdot48\frac{(d/h)^{1\cdot5}\,(g/h)^{0\cdot5}}{(1/h)^2\,(\tau_o/\tau_c)^{1\cdot5}}\mathrm{d}t, \tag{8}$$

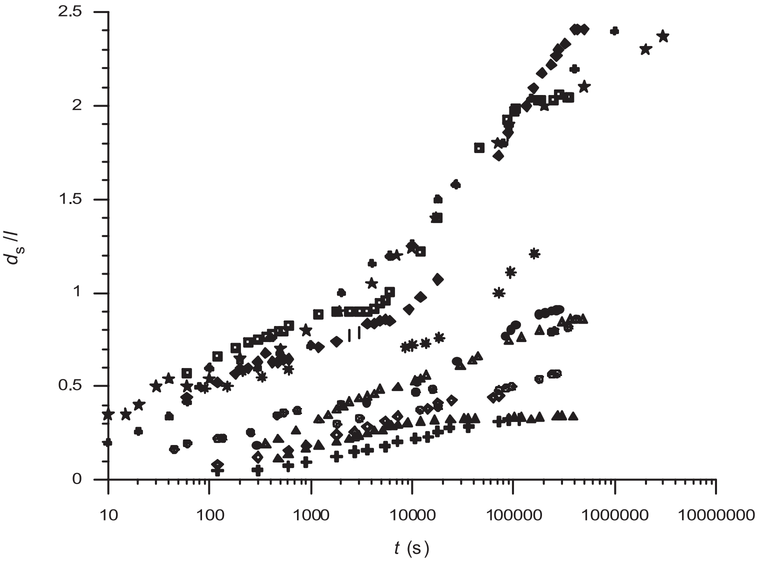
where $x_d = d_{st}/d_s$.

Cardoso & Bettess (1999) investigated the time evaluation of scour depth at vertical-wall abutments in compound channels under steady uniform flows being close to the critical velocity for bed sediment particles. The proposed equation is given below:

$$d_{st}/d_s = 1 - \exp\left[-1\cdot025\left(t/T^*\right)^{0\cdot35}\right]. \tag{9}$$

Kohli & Hager (2001) conducted laboratory experiments to study the influence of test duration on the scour depth at vertical-wall abutments placed in floodplain. They found that the densimetric particle Froude number has a significant effect on the scour depth. They suggested a logarithmic function of time-variation of scour depth as

$$d_{st} = (\mathrm{F}_\mathrm{d}^2/10)\,(hl/\cos\theta_a)^{0\cdot5}\log\left[t(\Delta gd)^{0\cdot5}/10h\right]. \tag{10}$$



Source	Model	d_{50} (mm)	σ_g	l (cm)	h (cm)	Symbol
Wong (1982)	WW abutment	0.62	1.30	60	12.5	✚
		0.83	1.30	60	12.5	◇
Kwan (1984)	SC abutment	0.80	1.28	63.5	10	◆
		0.85	1.30	66.5	5	▣
Tey (1984)	WW abutment	0.82	1.26	31	12.5	⊙
		0.82	1.26	40	50	●
Cardoso & Bettess (1999)	VW abutment	0.835	1.26	27	2.8	△
		0.835	1.26	80	5.5	▲
Balio & Orsi (2000)	VW abutment	0.82	1.26	10	9.2	★
		0.82	1.26	5	18.3	♣
Kohli & Hager (2001)	VW abutment	0.82	1.26	10	5.1	✳
		0.82	1.26	20	1	⊕

Figure 12. Variation of scour depth d_s with time t.

The effect of approaching flow velocity, abutment length and sediment size on time-variation of scour depth at thin abutments was studied by Santos & Cardoso (2001). Oliveto & Hager (2002) conducted experiments to study the temporal development of scour depth at circular piers and vertical-wall abutments. They used six different sizes of uniform and nonuniform sediments including one plastic sediment. Their proposed equation of clear-water scour depth is as follows:

$$d_{st}/L_R = 0{\cdot}068 N_s \sigma_g^{-0{\cdot}5} \mathrm{F}_\mathrm{d}^{1{\cdot}5} \log(T_R). \tag{11}$$

$N_s = 1{\cdot}25$ for a vertical-wall abutment.

Coleman *et al* (2003) analysed the time-variation of scour depth at vertical-wall abutment under clear-water condition. They put forward the following equation:

$$d_{st}/d_s = \exp\left[-0{\cdot}07\,(U_c/U)\,|\ln(t/T)|^{1{\cdot}5}\right]. \tag{12}$$

6. Estimation of scour depth

Equations for estimating scour depth at abutments are classified into the three following categories.

(1) Regime approach relating the scour depth to the increased discharge intensity;
(2) Empirical approach using dimensional analysis of the main parameters causing scour;
(3) Analytical or semi-empirical approach.

6.1 *Regime approach*

In the regime approach, scour depth is related to the discharge at the section under consideration.

Using Lacey's regime formula, the CBI (1949) proposed the following relationship to estimate the maximum scour depth at obstructions like spur-dikes or abutments:

$$d_s + h = 0{\cdot}47k_1\,(Q/f_1)^{1/3} \text{ in FPS units,} \tag{13}$$

(d is in mm), and k_1 depends on the type of obstructions (= 1 to 3·5).

Based on the laboratory experimental model studies, Ahmad (1953) suggested a relationship to estimate the maximum scour depth at spur-dikes as

$$d_s + h = k_2 q^{2/3}, \tag{14}$$

where k_2 is a constant depending on the flow intensity and angle of inclination of spur-dike.

Izzard & Bradley (1958) proposed an equation of scour depth at spur-dikes and abutments, based on a plot of data of laboratory experimental models and prototypes as

$$d_s + h = 1{\cdot}4q^{2/3}. \tag{15}$$

The above equations have an inherent drawback for estimation of scour depth. The regime concept, originating from the analysis of general scour in live-bed conditions, was extended to local scour at spur-dikes and abutments on the basis of observations. Whereas the local scour caused by the change of flow pattern at the obstructions is fundamentally different from that of general scour, the equations indicate that the scour depth is a function of flow intensity only. However, the sediment characteristics, rate of scour and mode of sediment transport are not considered.

6.2 *Empirical approach*

In the empirical approach, parameters involved in the abutment scour are correlated through dimensional analyses. Using regression analyses of the experimental data, the equations of scour depth at abutments are developed.

Liu *et al* (1961) investigated the scour at spill-through and vertical-wall abutments and suggested the following equations under clear-water scour:

$$d_s/h = 1{\cdot}1\left(\frac{l}{h}\right)^{0\cdot4} \mathrm{F}_{\mathrm{r}}^{0\cdot33} + 0{\cdot}3, \text{ for spill-through abutments,} \tag{16a}$$

$$d_s/h = 2{\cdot}15\left(\frac{l}{h}\right)^{0\cdot4} \mathrm{F}_{\mathrm{r}}^{0\cdot33} + 0{\cdot}3, \text{ for vertical-wall abutments.} \tag{16b}$$

Garde *et al* (1961) developed a relationship for the scour depth at spur-dikes under live-bed scour as

$$(d_s + h)/h = (K_1/\alpha)\, \mathrm{F}_{\mathrm{r}}^{\mathrm{n}} \tag{17a}$$

In continuation to their work, Garde *et al* (1963) modified the above equation due to inherent difficulties in determining the drag coefficient C_D and relating K_1 to C_D. The modified equation is given by

$$(d_s + h)/h = 4\eta_1\eta_2\eta_3(\mathrm{F}_{\mathrm{r}}^{\mathrm{n}}/\alpha). \tag{17b}$$

Starting from the equation of long contraction and using experimental data, Gill (1972) derived the following generalized equation of scour depth at spur-dikes:

$$\frac{d_s + h}{h} = 8{\cdot}375\left(\frac{d}{h}\right)^{0\cdot25}\left(\frac{B}{B-l}\right)^{6/7}\left[\left(\frac{B}{B-l}\right)^{1/n}\left(1-\frac{\tau_c}{\tau_o}\right)+\frac{\tau_c}{\tau_o}\right]^{-3/7}. \tag{18a}$$

In the above equation, τ_o is assumed to be equal to τ_c in live-bed scour. Gill (1972) further extended the above equation for three different flow conditions as follows:

$$\frac{d_s + h}{h} = 8{\cdot}375\left(\frac{d}{h}\right)^{0\cdot25}\left(\frac{B}{B-l}\right)^{6/7}\left(\frac{\tau_o}{\tau_c}\right)^{3/7}, \quad \text{for clear-water scour condition } (\tau_c/\tau_o > 0), \tag{18b}$$

$$\frac{d_s + h}{h} = 8{\cdot}375\left(\frac{d}{h}\right)^{0\cdot25}\left(\frac{B}{B-l}\right)^{(6/7)-(3/7n)}, \quad \text{for high sediment transport condition } (\tau_c/\tau_o > 0), \tag{18c}$$

$$\frac{d_s + h}{h} = 8{\cdot}375\left(\frac{d}{h}\right)^{0\cdot25}\left(\frac{B}{B-l}\right)^{6/7}, \quad \text{for maximum scour depth } d_s \text{ condition } (\tau_c/\tau_o \approx 1). \tag{18d}$$

In the above, n varies from 1·5 to 3.

Zaghloul & McCorquodale (1975) presented the following equation that includes the effect of angle of inclination of spur-dike with respect to the main flow:

$$d_s/h = 2{\cdot}62\mathrm{F}_{\mathrm{r}}^{2/3}(\alpha\theta_a)^{-0\cdot043}. \tag{19}$$

Froehlich (1989) analysed the scour data of different researchers using statistical method and developed the following equations of clear-water and live-bed scour depths:

$$\frac{d_s}{h} = 0{\cdot}78 K_s K_\theta \left(\frac{l}{h}\right)^{0\cdot63} \mathrm{F}_\mathrm{r}^{1\cdot16} \left(\frac{h}{d}\right)^{0\cdot43} \sigma_g^{-1\cdot87} + 1, \text{ for clear-water scour,} \tag{20a}$$

$$\frac{d_s}{h} = 2{\cdot}27 K_s K_\theta \left(\frac{l}{h}\right)^{0\cdot43} \mathrm{F}_\mathrm{r}^{0\cdot61} + 1, \text{ for live bed scour.} \tag{20b}$$

The coefficient of abutment alignment K_θ is $(\theta_a/90)^{0\cdot13}$. The addition of 1 at the right hand side of the equations results in large overestimation of the scour depths especially for large approaching flow depths.

Strum & Janjua (1994) conducted the experiments in a flume with a fixed-bed main channel and a movable-bed floodplain, where the abutment terminated. Using dimensional analysis and least-square regression analysis, they derived the following equation of clear-water scour depth at abutments in floodplains:

$$d_s/h = 7{\cdot}7\,(\mathrm{F_r}/M\mathrm{F_{rc}} - 0{\cdot}35) \tag{21}$$

Melville (1992, 1995, 1997) proposed a design method to estimate the scour depth at abutments based on empirical relationships containing different factors or coefficients. Each factor or coefficient represents the effect of flow depth, abutment size, flow intensity, sediment characteristics, abutment shape, abutment alignment and channel geometry on scour depth. The proposed equation is

$$d_s = K_{hl} K_I K_d K_s K_\theta K_G. \tag{22}$$

He argued that for short abutments ($l/h \le 1$), the scour depth scales with the abutment length; whereas for long abutments ($l/h \ge 25$), the scour depth scales with the flow depth. For all other abutments ($1 < l/h < 25$), the scour depth is proportional to $(hl)^{0\cdot5}$. Thus, according to Melville (1992), the coefficient accounting for the flow depth and abutment size is given by

$$K_{hl} = 2l, \text{ for } l/h \le 1, \tag{23a}$$

$$K_{hl} = 2(hl)^{0\cdot5}, \text{ for } 1 < l/h < 25, \tag{23b}$$

$$K_{hl} = 10h, \text{ for } l/h \ge 25. \tag{23c}$$

Melville & Sutherland (1988) presented a method for accounting sediment gradation effects including armor velocity U_a. The value of U_a is calculated to be $0{\cdot}8U_{cn}$. The flow intensity factor, K_I for uniform and nonuniform sediments is given by

$$K_I = [U - (U_a - U_c)]/U_c \text{ for } [U - (U_a - U_c)]/U_c < 1, \tag{24a}$$

$$K_I = 1 \text{ for } [U - (U_a - U_c)]/U_c \ge 1. \tag{24b}$$

The critical velocities U_c and U_{cn} can be determined from the logarithmic velocity distribution as

$$U_c/u_{*c} = 5{\cdot}75 \log(5{\cdot}53h/d), \tag{25a}$$

$$U_{cn}/u_{*cn} = 5{\cdot}75 \log(5{\cdot}53h/d_{50a}). \tag{25b}$$

The sediment size effect depends on the value of l/d as given by

$$K_d = 0{\cdot}57 \log(2{\cdot}24 l/d), \qquad \text{for } l/d \leq 25, \tag{26a}$$

$$K_d = 1, \qquad \text{for } l/d > 25. \tag{26b}$$

The abutment shape factor is assumed to be 1 for vertical-wall abutments and 0·75 for wing-wall abutments. Spill-through abutments are assigned values of $K_s = 0{\cdot}6$, 0·5 and 0·45 for 0·5 : 1 (horizontal : vertical), 1 : 1 and 1·5 : 1 side slopes, respectively. These values of shape factor apply only to shorter abutments ($l/h \leq 10$). Shape effects were found to be unimportant for long abutments, and hence, $K_s = 1$ for $l/h \geq 25$. For abutment lengths $10 < l/h < 25$, a linear interpolation has been proposed. Thus, adjusted shape factor K_s^* for intermediate abutments is

$$K_s^* = K_s + 0{\cdot}667(1 - K_s)[0{\cdot}1(l/h) - 1]. \tag{27}$$

In the above, shape factor K_s is referred to the case $l/h \leq 10$.

The value of abutment alignment factor $K_\theta = 1$ for an abutment aligned across the flow, that is an angle of alignment $\theta_a = 90°$. For $\theta_a < 90°$, the abutment is pointed downstream and vice versa. For alignment angles $\theta_a = 30°$, 60°, 120° and 150°, the values of $K_\theta = 0{\cdot}9$, 0·97, 1·06 and 1·08, respectively. Melville (1992) recommended that the alignment factor can be applied only to longer abutments ($l/h \geq 3$). Alignment effects are negligible for short abutments, and hence, $K_\theta = 1$ for $l/h \leq 1$. For abutment lengths $1 < l/h < 3$, a linear interpolation has been recommended. Thus, adjusted alignment factor K_θ^* for intermediate abutments is

$$K_\theta^* = K_\theta + (1 - K_\theta)[1{\cdot}5 - 0{\cdot}5(l/h)]. \tag{28}$$

Channel geometry factor K_G is defined as the ratio of the scour depth at a given abutment sited in the compound channel to that at the same abutment sited in a corresponding rectangular channel of the same overall width as that of the compound channel and the same depth as that of the main channel of the compound section. Melville & Ettema (1993) put forward the following equation for K_G:

$$K_G = \left\{1 - (l^*/l)\left[1 - N/N^*(h^*/h)^{5/3}\right]\right\}^{1/2}. \tag{29}$$

Kothyari & Ranga Raju (2001) defined an analogous pier, with size such that scour depth at the pier is same as that at the given spur-dike or abutment under similar hydraulic conditions. Size of the analogous pier is related to the parameters that influence the drag due to the flow past spur-dike or abutment and pier, and given by Kothyari & Ranga Raju (2001) is

$$b_s/K_s b_d = 0{\cdot}074(l/b_d)^{2{\cdot}7}(U/\sqrt{gd\Delta\rho_s/\rho}) + 0{\cdot}46. \tag{30}$$

Scour depth at abutments can then be calculated using the relationships for estimation of pier scour given by Kothyari *et al* (1992a,b) with size of the analogous pier taken as the pier width.

Based on the field data of scour at the end of spurs in the Mississippi river, Richardson *et al* (2001) proposed the equation below to estimate scour depth at an abutment for live-bed scour,

$$d_s/h = 7{\cdot}27 K_s K_\theta\, \mathrm{F}_\mathrm{r}^{0{\cdot}33}. \tag{31}$$

Richardson & Davis (2001) recommended that (20b) and (31) are adequate for the estimation of scour depth at abutments for clear-water scour and live-bed scour conditions. Equation (11) given by Oliveto & Hager (2002) may be applicable for the estimation of equilibrium scour depth, using a large magnitude of $T_R (\sim 10^6)$.

Recently, Dey & Barbhuiya (2004b) put forward an equation of clear-water scour depth at short abutments as follows:

$$d_s/l = 5{\cdot}16 K_s\,(h/l)^{0{\cdot}18} \left(U_c/\sqrt{\Delta g l}\right)^{0{\cdot}26}. \tag{32}$$

6.3 *Analytical or semi-empirical approach*

Laursen (1960, 1963) developed semi-empirical scour depth relationships for bridge abutments treating the abutment scour as a limiting case of scour through a long flow constriction. His proposed relationships for clear-water and live-bed scour depths at vertical-wall abutments as

$$l/h = 2{\cdot}75(d_s/h)\left[\left((d_s/\hat{d}_l h)+1\right)^{1{\cdot}7} - 1\right], \text{ for live-bed scour,} \tag{33a}$$

$$l/h = 2{\cdot}75(d_s/h)\left\{\left[\left[d_s/(\hat{d}_l h)+1\right]^{7/6}/(\tau_o/\tau_c)^{0{\cdot}5}\right] - 1\right\} \text{ for clear-water scour.} \tag{33b}$$

Laursen (1960, 1963) assumed $\hat{d}_1 = 12$ and $11{\cdot}5$ for clear-water and live-bed scour conditions, respectively.

Based on the flow continuity equation, scour geometry and a generalized power-law formula for flow resistance in alluvial channels, Lim (1997) presented a semi-empirical formula of equilibrium clear-water scour depth at an abutment as

$$d_s/h = K_s(0{\cdot}9X - 2) \tag{34}$$

where $X = \theta_c^{-0{\cdot}375}\mathrm{F}_\mathrm{d}^{0{\cdot}75}(d/h)^{0{\cdot}25}[0{\cdot}9(l/h)^{0{\cdot}5}+1]$, and $K_s = 1$ for vertical-wall abutment and for other shapes, the values of K_s are as given by Melville (1992). The above equation is valid up to $X = 2{\cdot}22$.

Following the approach of Lim (1997), Lim & Cheng (1998) introduced a semi-empirical equation for the time-averaged equilibrium live-bed scour at vertical-wall abutments as

$$\left(1+\frac{d_s}{2h}\right)^{4/3} = [1+1{\cdot}2\sqrt{l/h}]\Bigg/\left[\frac{u_{*c}^2}{u_*^2}+\left(\frac{l\tan\phi_s}{d_s}+1\right)^{2/3}\left(1-\frac{u_{*c}^2}{u_*^2}\right)\right]^{1/2}. \tag{35}$$

Kandasamy & Melville (1998) developed a relationship for maximum scour depth at piers and abutments aligned perpendicular to the flow,

$$d_s = K_s K_2 h^n l^{1-n}. \tag{36}$$

In the above, $K_2 = 5$ and $n = 1$ for $h/l \le 0{\cdot}04$, $K_2 = 1$ and $n = 0{\cdot}5$ for $0{\cdot}04 < h/l < 1$, and $K_2 = 1$ and $n = 0$ for $h/l > 1$.

Table 3. Scour depths estimated using equations of different investigators.

Investigator	d_s (m)
Melville (1992)	4·00
Lim (1997)	4·03
Froehlich (1989)	3·88
Richardson *et al* (2001)	8·90
Oliveto & Hager (2002)	9·90
Dey & Barbhuiya (2004b)	5·90

6.4 *Numerical example for estimation of design scour depth*

It is rather difficult to assess the adequacy of the proposed equations for the estimation of scour depth from the point of view of design. However, from close examination of the aforementioned equations, those given by Melville (1992), Lim (1997), Richardson *et al* (2001), Froehlich (1989), Oliveto & Hager (2002) and Dey & Barbhuiya (2004b) seem to be complete. In order to compare the estimated design scour depth at a short vertical-wall abutment (aligned along the approaching flow in a rectangular channel with uniform bed sediment) using the equations of the above investigators, the following example is presented.

- Abutment length, $l = 2$ m
- Approaching flow depth, $h = 2{\cdot}5$ m
- Uniform sediment size, $d_{50} = 1$ mm
- Geometric standard deviation of particle size distribution, $\sigma_g = 1{\cdot}1$
- Relative density of sediment particles, $s = 2{\cdot}65$
- Mean approaching flow velocity during flood peak, $U = 0{\cdot}5$ m/s
- Critical approaching flow velocity, $U_c = 0{\cdot}57$ m/s

Using the given data, a comparison of the estimated values of scour depth d_s obtained from the proposed equations of Melville (1992), Lim (1997), Richardson *et al* (2001), Froehlich (1989), Oliveto & Hager (2002) and Dey & Barbhuiya (2004b) is given in table 3. It is observed that the equations of Richardson *et al* (2001) and Oliveto & Hager (2002) provide conservative estimations of scour depth.

7. Closure

Though a large number of studies have been carried out on local scour at different abutments, and all the experimental and theoretical investigations bring us nearer to better understanding of the problem, it remains unexplored in many cases. In laboratory model studies, internal flow characteristics do not truly represent prototype bridge abutment scouring in rivers in view of large-scale distortion of the models. Also, from the available literature, it is revealed that the exact scour mechanism and effects of different parameters on scour depth are yet to be fully understood or explored. Moreover, most of the studies are restricted to uniform sediments. As natural riverbed sediments are nonuniform and at the upper reaches of hilly rivers, armouring of beds occurs due to natural sorting of bed sediments by the high flow velocity. Thus investigations on scour at abutments in nonuniform sediments and with armour-layered beds are important, and more of such studies are required along with field investigations on at least large-scale models.

List of symbols

B	channel or flume width;
b_d	width of cylindrical pier experiencing the same drag as that on abutment;
b_s	width of analogous pier;
C_D	drag coefficient of sediment particles;
D	pier diameter;
d, d_{50}	median diameter of sediment particles;
d_{16}	16% finer particle diameter;
d_{50a}	$d_{\max}/1{\cdot}8$;
d_{84}	84% finer particle diameter;
$\hat{d}_l$	ratio of scour depth at abutment to scour depth in equivalent long contraction;
$d_{\max}$	maximum particle size of a nonuniform sediment;
d_s	equilibrium scour depth in uniform sediment;
d_{st}	scour depth at time t;
$\mathrm{F_d}$	$U/(\Delta gd)^{0\cdot5}$, densimetric Froude number;
$\mathrm{F_r}$	$U/(gh)^{0\cdot5}$, approaching flow Froude number;
$\mathrm{F_{rc}}$	$U_c/(gh)^{0\cdot5}$, approaching flow Froude number corresponding to critical velocity;
f_1	Lacey's slit factor, $1{\cdot}76d^{0\cdot5}$;
g	gravitational acceleration;
h	approaching flow depth;
h^*	flow depth in floodplain;
$K_{1,2}$, $k_{1,2}$	coefficients;
K_d	particle size factor;
K_G	channel geometry factor;
K_{hl}	flow depth – abutment length factor;
K_I	flow intensity factor;
K_s, K_s^*	abutment shape factor and adjusted abutment shape factor respectively;
K_θ, K_θ^*	abutment alignment factor and adjusted abutment alignment factor respectively;
K_σ	function depending on σ_g;
L_R	reference length $l^{2/3}h^{1/3}$;
l	transverse length or protrusion length of abutment;
l^*	width of floodplain;
M	discharge ratio;
m	coefficients depending on bed sediment size;
N, N^*	Manning roughness coefficients for main channel and for floodplains respectively;
N_s	shape number;
n	exponents depending on bed sediment size;
Q	design discharge;
q	discharge intensity;
$\hat{r}$	r/l;
s	relative density of sediment particles;

T	time to equilibrium scour depth;
T_R	dimensionless time, $t(\Delta gd)^{0\cdot5}/LR$;
T^*	time when $d_t = 0\cdot632 d_s$;
t	time;
U	average approaching flow velocity;
U_a	$0\cdot8U_{cn}$;
U_c	critical velocity for sediment particles;
U_{cn}	critical velocity for armour particle size d_{50a};
u, v, w	time-averaged velocity components in (x, y, z) or (θ, r, z);
$\hat{u}$	u/U;
u_*	shear velocity of approaching flow;
u_{*c}	critical shear velocity for sediment particles;
u_{*cn}	critical shear velocity for armour particle size d_{50a};
$\hat{v}$	v/U;
$\hat{w}$	w/U;
w_s	settling velocity of sediment particles;
X	$\theta_c^{-0\cdot375}\,\mathrm{F}_\mathrm{d}^{0\cdot75}(d/h)^{0\cdot25}[0\cdot9(l/h)^{0\cdot5}+1]$;
$\hat{x}$	x/l;
x_d	d_{st}/d_s;
x, y, z	Cartesian coordinates;
$\hat{y}$	y/l;
$\hat{z}$	z/l;
α	$1 - l/B$, opening ratio;
Δ	$s - 1$;
ϕ_s	side slope angle of scour hole;
η_{1-3}	coefficients;
θ, r, z	cylindrical polar coordinates;
θ_a	angle of attack;
θ_c	$u_{*c}^2/(\Delta gd)$, shield's entrainment function;
θ_t	turning angle between bottom streamline and main flow direction, $1\cdot485\,\mathrm{F}_\mathrm{r}^{0\cdot13}(l/h)^{0\cdot06}$;
ρ, ρ_s	mass densities of water and of sediment particles respectively;
σ_g	geometric standard deviation;
τ_o	bed shear stress of approaching flow;
τ_c	critical shear stress of sediment particles;
τ_{cont}	shear stress due to contraction;
$\hat{\tau}_{cont}$	τ_{cont}/τ_o, bed shear stress due to contraction;
τ_{nose}	bed shear stress at nose region of abutment;
$\hat{\tau}_{nose}$	τ'_{nose}/τ_o, bed shear stress amplification only due to abutment;
τ'_{nose}	shear stress only due to abutment;
$\hat{\tau}_t$	τ_{nose}/τ_o, total shear stress amplification at vertical wall abutment.

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