Design Method for Local Scour at Bridge Piers

By B. W. Melville[1] and A. J. Sutherland[2]

Abstract: A design method for the estimation of equilibrium depths of local scour at bridge piers is presented. The method is based upon envelope curves drawn to experimental data derived mostly from laboratory experiments. The laboratory data include wide variations in flow velocity and depth, sediment size and gradation, and pier size, shape, and alignment. Local scour depth estimation is based upon the largest possible scour depth that can occur at a cylindrical pier, which is $2.4D$, where D = the pier diameter. According to the method, this depth is reduced using multiplying factors where clear-water scour conditions exist, the flow depth is relatively shallow, and the sediment size is relatively coarse. In the case of nonrectangular piers, additional multiplying factors to account for pier shape and alignment are applied. The method of estimation of local scour depth is summarized in a flow chart.

Introduction

Selection of an appropriate design method to estimate equilibrium depths of local scour at bridge piers has perplexed designers for many years. There is a wide variety of formulas available, and no obvious similarity in either their appearance or their predictions. A review of the more popular formulas has been given by Raudkivi and Sutherland (1981). The same publication presents a design method based on many laboratory studies. The method considered sediment gradation and size and flow depth effects but did not consider the possibility of live-bed scour.

Studies involving live-bed scour with uniform and nonuniform sediments have now been completed (Chee 1982; Chiew 1984; Baker 1986). These have explained the apparent conflicts arising from earlier studies, some of which showed live-bed scour exceeding clear-water scour (Garde 1961; Blench 1969; Jain and Fischer 1979) and others of which showed the reverse (Chabert and Engeldinger 1956; Laursen 1958, 1962; Hancu 1971; Breusers et al. 1977). These new data have made it possible to place all data within one framework. Clear-water scour becomes a special case, as does scour of uniform material, a rare event in practice.

Live-bed scour introduces two further processes. Threshold or critical flow conditions become of significance and are described herein by the mean flow velocity U_c appropriate for the stated sediment and channel slope. Of equal importance are the flow conditions beyond which armoring of the channel bed is impossible. There are also characterized by the mean flow velocity U_{ca}, which again depends on the sediment characteristics and channel slope.

Threshold and limiting armor conditions are discussed first. The general framework for scour estimation is then presented, and the effect of the

[1]Sr. Lect. in Civ. Engrg., Univ. of Auckland, Auckland, New Zealand.
[2]Reader in Civ. Engrg., Univ. of Canterbury, Christchurch, New Zealand.

Note. Discussion open until March 1, 1989. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on October 13, 1987. This paper is part of the *Journal of Hydraulic Engineering*, Vol. 114, No. 10, October, 1988. ©ASCE, ISSN 0733-9420/88/0010-1210/$1.00 + $.15 per page. Paper No. 22830.

J. Hydraul. Eng., 1988, 114(10): 1210-1226

significant parameters described. This leads directly to a design method, which is used on some illustrative examples.

Threshold Conditions

The Shields diagram (Henderson 1966) remains the most effective way of determining threshold conditions for uniform sediments. For given fluid density and viscosity and sediment density, the Shields diagram can be used, as in Fig. 1, to obtain a plot of shear velocity u_{*c} against grain size, d_{50}. Water and sediment with densities of 1,000 kg/m^3 and 2,650 kg/m^3, respectively, have been assumed in the preparation of Fig. 1. A useful relation for the coarser grain sizes $d_{50} > 6$ mm is

$$u_{*c} = 0.03\ d_{50}^{1/2} \quad \text{(1)}$$

where u_{*c} is in m/s, and d_{50} in mm. Threshold shear velocity is converted to threshold mean flow velocity U_c using, as an approximation, the logarithmic form of the velocity profile

$$\frac{U_c}{u_{*c}} = 5.75 \log\left(5.53\ \frac{y}{d_{50}}\right) \quad \text{(2)}$$

where y = flow depth. For uniform sediments, U_c marks the transition from clear-water to live-bed scour conditions.

Determination of threshold conditions for nonuniform sediments is not so clear-cut. The effects of particle size distribution may be important, and thus U_c would depend upon both the median grain size d_{50} and the geometric standard deviation $\sigma_g = d_{84}/d_{50}$. With nonuniform sediments, a flow can disturb the grains, removing some but simply rearranging others into a stable pattern that develops into an armored bed and stabilizes. Such flows are considered herein to be above threshold. Values of U_c are found using a suggestion first made by Neill (1968) that many of the intermediate sizes will move when flow conditions based on d_{50} exceed the Shields criterion. Neill claimed this to work well at least for $\sigma_g < 2.5$. Fig. 1 and

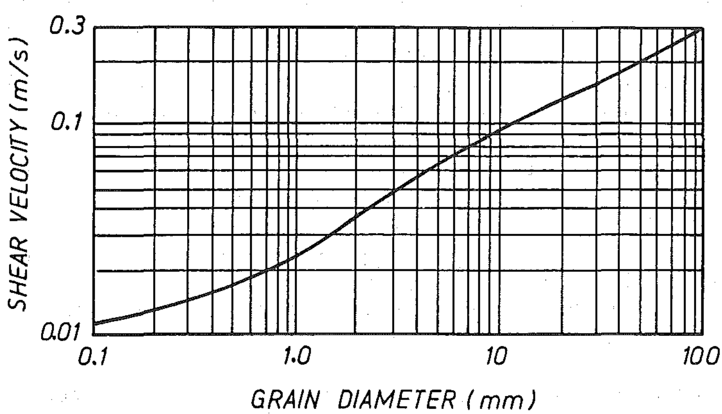


FIG. 1. Shields Chart for Threshold Condition of Uniform Sediments in Water

J. Hydraul. Eng., 1988, 114(10): 1210-1226

TABLE 1. Exponent m-Values for Eq. 3

Assumed value of d_{max} (1)	m (2)
d_{90}	1.28
d_{95}	1.65
d_{98}	2.06
d_{99}	2.34

Eq. 2 based on d_{50} are used herein to determine U_c for nonuniform material.

Limiting Armor Conditions

The flow condition beyond which armoring of a nonuniform channel bed is impossible is termed the limiting armor condition. This condition represents the coarsest or most stable armored bed for a given bed material and can be characterized by the mean velocity U_{ca}. At flows greater than U_{ca}, no armor layer can form, while at flows less than U_{ca}, the armored bed will be finer (d_{50a} smaller) or less stable than that formed at U_{ca}.

Each sediment has a unique value of U_{ca}, dependent on the sediment size and grading. For a given d_{50}, U_{ca} increases with increasing σ_g. Chin (1985) showed that the value of U_{ca} for a given sediment is dependent on the maximum d_{max} size and gives a method to determine U_{ca} using d_{max}, which can be found from d_{50} and σ_g if a logarithmic normal distribution is assumed:

$$d_{max} = \sigma_g^m d_{50} \quad (3)$$

where m depends upon the size chosen for d_{max} as shown in Table 1.

Chin (1985) showed that the d_{50} size of the coarsest possible armor, d_{50a}, is equal to $d_{max}/1.8$. This value of d_{50a} is used to determine the critical shear velocity of the armored bed, u_{*ca} from Shields (Fig. 1). Calculation of U_{ca} then follows from Eq. 2, using d_{50a} as the grain size. Chin's (1985) laboratory experiments were conducted with no sediment replenishment from upstream of the test section. The test beds were allowed to degrade until erosion had effectively stopped. The more common condition for scouring at bridge sites involves sediment supply from upstream. Baker (1986) showed, using a sediment-recirculating flume, that in the latter case, the appropriate flow velocity U_a to characterize the limiting armor condition for scour determination is $0.8U_{ca}$. Baker explained that the bed armor layer for the condition of sediment supply from upstream is finer than the critical armor layer as determined by Chin (1985). This is because the sediment input from upstream limits erosion on the approach bed to the bridge site, effectively preventing the exposure of the number of coarse grains necessary to form the critical armor layer. For nonuniform sediments, U_a is considered to mark the transition from clear-water to live-bed scour conditions.

In practice, the value of U_a, determined using $U_a = 0.8U_{ca}$, needs to be checked against the physical constraint that U_a cannot be less than U_c. In the case of $U_a < U_c$, the proposed remedy is to set $U_a = U_c$ and assume

J. Hydraul. Eng., 1988, 114(10): 1210-1226

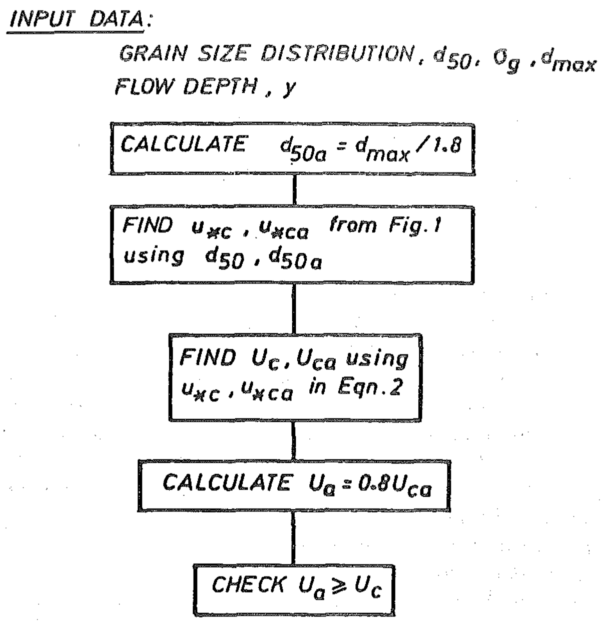


FIG. 2. Flow Chart to Calculate U_a, Limiting Armor Velocity

that the bed material is behaving as a uniform sediment. The method of evaluation of U_a is summarized in the flow chart (Fig. 2).

Framework for Analysis of Local Scour

Equilibrium depth of local scour d_s at a pier can be written

$$d_s = f(\rho, \nu, U, y, \rho_s, d_{50}, \sigma_g, g, D, Sh, Al) \quad (4)$$

where ρ and ν = fluid density and kinematic viscosity; U = mean flow velocity; y = flow depth; ρ_s = sediment density; g = gravitational acceleration; D = pier width normal to the flow; Sh and Al = parameters describing the shape and alignment of the pier; and f = "a function of." Eq. 4 can be written

$$\frac{d_s}{D} = f\left(\frac{UD}{\nu}, \frac{U^2}{gd_{50}}, \frac{y}{D}, \frac{\rho_s}{\rho}, \frac{d_{50}}{D}, \sigma_g, Sh, Al\right) \quad (5)$$

The density ratio is assumed constant, and the Reynolds number UD/ν influences are assumed negligible for the highly turbulent flows envisaged. Thus

$$\frac{d_s}{D} = f\left(\frac{U^2}{gd_{50}}, \frac{y}{D}, \frac{d_{50}}{D}, \sigma_g, Sh, Al\right) \quad (6)$$

The functional relation has been evaluated using laboratory data by writing it in the form

$$\frac{d_s}{D} = K_I K_y K_d K_\sigma K_s K_\alpha \qquad (7)$$

where the Ks are expressions, presented in the following, describing the influence of each parameter in Eq. 6: K_I = flow intensity; K_y = flow depth; K_d = sediment size; K_σ = sediment gradation; and K_s and K_α = pier shape and alignment, respectively. Each parameter is now considered individually.

Flow Intensity

The Shields diagram defines a u_{*c} for a given d_{50}. A corresponding U_c can be found for the given flow depth, and thus the parameter U^2/gd_{50} can be written, using the given data as U/U_c. As such, this ratio is a measure of flow intensity and determines whether or not grain motion occurs. For $U/U_c < 1$, clear-water scour conditions pertain for both uniform and nonuniform sediments. If $\sigma_g < 1.3$, the sediment acts as a uniform sediment, and for $U/U_c > 1$, live-bed scour occurs. If $\sigma_g > 1.3$ and $1 < U/U_c < U_a/U_c$, then armoring of the bed will occur as scouring proceeds, and it is considered that clear-water scour exists. For $1 < U/U_a$, there is no armoring, and live-bed scour occurs.

The flow intensity parameter thus determines the scouring processes that are important. It is shown later in the paper that a more appropriate form for this parameter is $[U - (U_a - U_c)]/U_c$, in which the difference $(U_a - U_c)$ reflects the influence of sediment grading.

Flow Depth Ratio

Ettema (1980), Chiew (1984), and others showed that scour depth increases with flow depth up to a limiting value of the flow depth ratio y/D, beyond which there is no influence of flow depth. Ettema explained that for shallow flows, the surface roller that forms ahead of the bridge pier interferes with the scour action of the horseshoe vortex because the two have opposite senses of rotation. With increasing flow depth, the interference reduces until there is no significant effect at $y/D \sim 3$. Data by Shen (1966), Ettema (1980), Chee (1982), and Chiew (1984) for uniform sediments under clear-water and live-bed conditions are shown in Fig. 3, in which K_y, the flow depth factor, is the ratio of d_s/D at a particular value of y/D to that at $y/D \geq 4$. These data all have $D/d_{50} \geq 50$. Ettema (1980) showed that the relationship between scour depth and flow depth is, in fact, a family of curves at different values of D/d_{50}. However, the effects of sediment size ratio will be considered separately in the following.

Also shown in Fig. 3 are data by Davoren (1985), obtained at a 1.5-m diameter pier in the Ohau River, New Zealand, during steady high flows. The bed material was nonuniform, with d_{50} = 20 mm and σ_g = 5.3, such that D/d_{50} = 75 and $D/d_{max} \sim 5$.

For design purposes, the expression recommended for K_y is

$$K_y = 1.0, \qquad \text{if } y/D > 2.6 \qquad (8a)$$

$$K_y = 0.78\,(y/D)^{0.255}, \qquad \text{if } y/D < 2.6 \qquad (8b)$$

This is shown in Fig. 3.

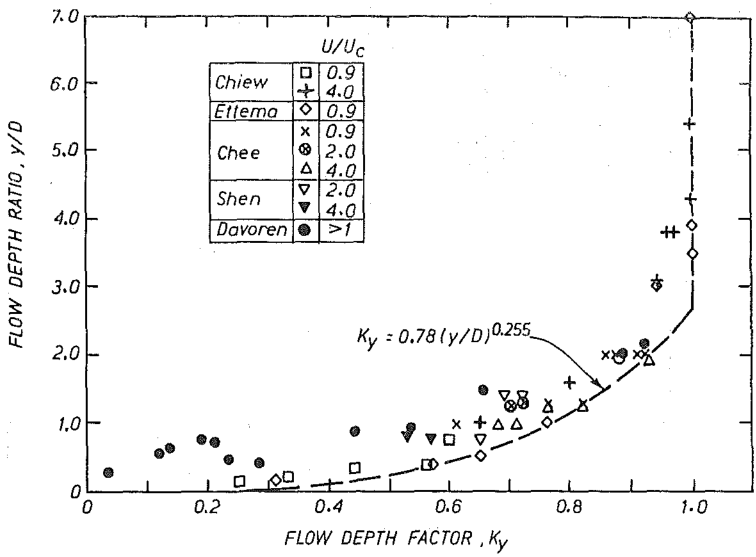


FIG. 3. Influence of Flow Depth on Scour Depth

Sediment Size Ratio

Chiew's (1984) live-bed data for uniform sediments and those of Ettema (1980) derived from clear-water flows delineate the effects of the sediment size ratio D/d_{50} on scour depth as shown in Fig. 4. K_d, the sediment size factor, is the ratio of d_s/D at a particular value of D/d_{50} to that at $D/d_{50} \geq 50$, beyond which there is no effect of sediment size. The data show that sediment that is large relative to the pier size effectively limits the scour depth. Ettema (1980) identified two main causes for the reduction in scour depth at low values of D/d_{50}:

1. The erosion process can be impeded by particles that are large relative to the size of the groove that forms at the base of the scour hole in front of the pier. The groove has been identified as the principle erosion zone for bridge pier scour (Ettema 1980; Melville 1975; Shen et al. 1966) and forms under the action of the downflow that occurs in front of the pier. The downflow is recognized as the main scouring agent (Ettema 1980; Melville 1975; Shen et al. 1966).
2. Coarse particles in the groove may allow the downflow to penetrate the bed, thus dissipating some of the flow energy.

For design purposes the expression recommended for K_d is

$$K_d = 1.0, \quad \text{if } D/d_{50} > 25 \quad (9a)$$

$$K_d = 0.57 \log (2.24\, D/d_{50}), \quad \text{if } D/d_{50} < 25 \quad (9b)$$

This is shown in Fig. 4.

Sediment Gradation Effects

Work by Ettema (1976) showed that scour depths are reduced dramatically as σ_g increases. The tests were all clear-water tests. Live-bed tests

J. Hydraul. Eng., 1988, 114(10): 1210-1226

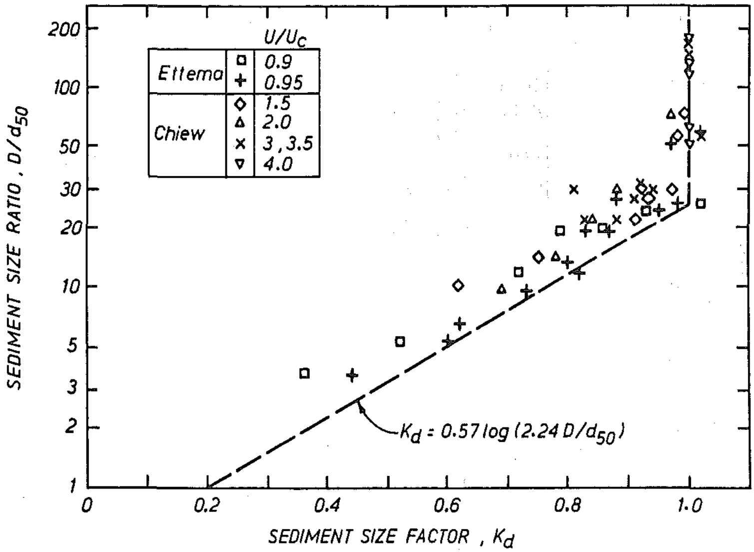


FIG. 4. Influence of Sediment Size on Scour Depth

by Baker (1986) have shown that under these conditions, the reductions are not so dramatic and that at very high flows ($U/U_c > 4.0$), scour depths become almost independent of σ_g.

The results of Baker's (1986) study are shown in Fig. 5, in which all sediments have d_{50} = 0.6 mm, and scour depths are temporal average values (i.e., scour depth due to the passage of bed features is excluded). In general, scour depths for a given U/U_c decrease as σ_g increases, and also the initial peak or armor peak occurs at higher values of U/U_c as σ_g increases. The reason for the reduction in scour depths is the presence of an armor layer on the approach bed and in the scour hole. At low flow velocities such that $U_c < U < U_a$, the approach bed is covered in an armor layer, and conditions resemble those of clear-water scour. With increasing flow velocity, the armor layer coarsens, and larger scour depths are recorded. The "coarsest possible" armored bed for a given sediment occurs at the velocity U_a, when conditions on the approach bed resemble the threshold condition for uniform bed material. The armor peak, or local maximum scour depth for a particular σ_g, occurs at $U = U_a$. As previously discussed, each sediment has a unique value of U_a (or U_c) dependent on the sediment size and grading or d_{max} size. A larger d_{max} leads to a coarser armored bed (at the "coarsest possible" condition) and a correspondingly higher U_a.

Beyond U_a, live-bed conditions pertain, with armoring diminishing with increasing flow velocity until all bed particles are mobile at velocities supporting a transition flat bed. Dunes begin to form just above the armor peak velocity U_a, and the scour depth decreases because, on average, the supply of sediment from incoming bed features is greater than that removed by the scour mechanism. In effect, there is insufficient time between the passage of successive bed features and the consequent input

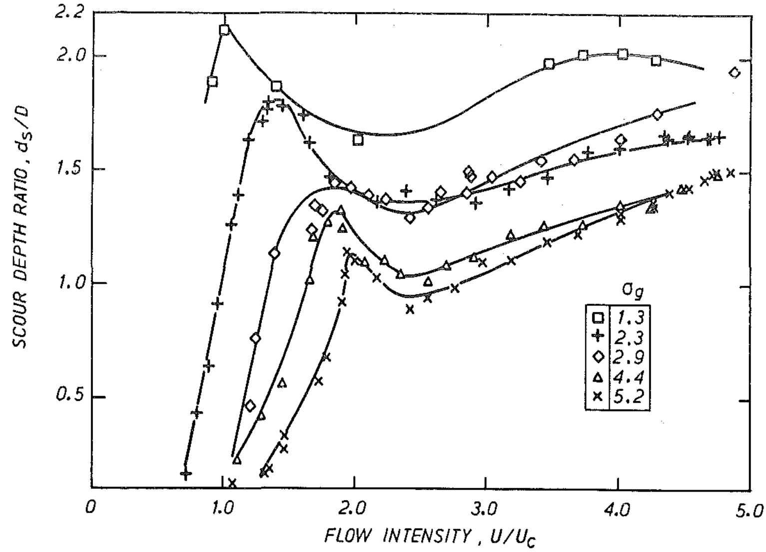


FIG. 5. Influence of σ_g on Scour Depth, Data by Baker (1986), d_{50} = 0.6 mm

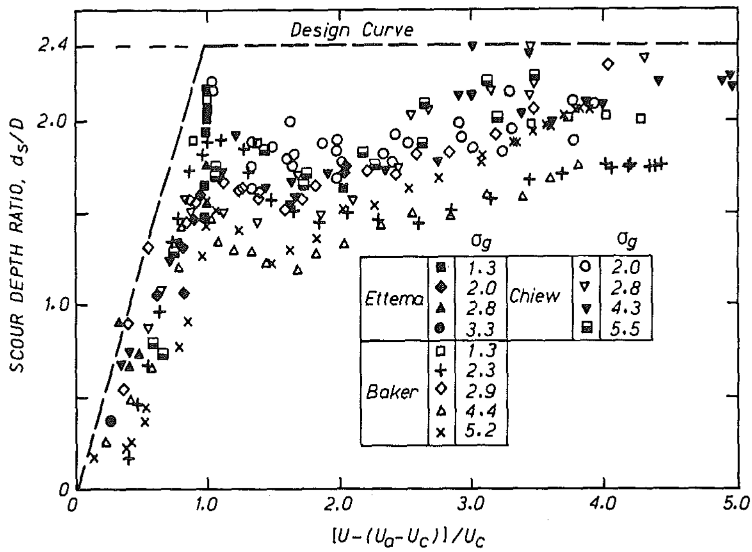
of sediment to the scour hole for the flow to remove the added bed material. The average scour depth attains a local minimum near the velocity producing the largest dunes, and then increases again, reaching a plateau at the transition flat bed condition.

The data in Fig. 5 include the effects of relative sediment size because the coarser particles do not satisfy the condition $D/d_{50} > 50$, as discussed previously. The sediment size factor K_d is used to adjust scour depths for the effects of relative sediment size and is based on the d_{50} size for uniform sediments. However, the d_{50} size does not adequately describe a nonuniform sediment. To adjust for the effect of sediment size, the d_{50} size of the coarsest possible armor layer d_{50a} is adopted for nonuniform sediments. There are two reasons for the choice of d_{50a} as the "effective size." First, d_{50a} is a quantity that can be determined using Chin's (1985) method as discussed previously, and second, the adjusted scour depths as shown in Fig. 6 are conservative (i.e., larger than if a smaller particle size had been used to estimate K_d).

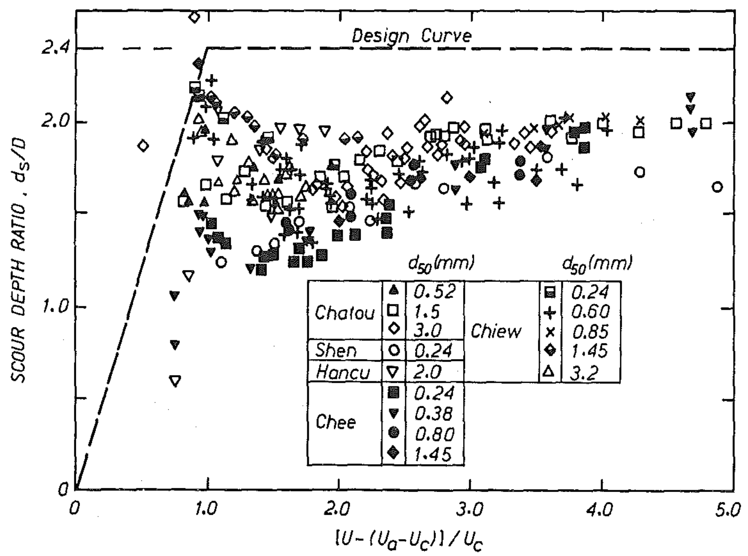
The velocity transformation in Fig. 6 has the effect of aligning the armor peaks for nonuniform sediments with the threshold peak for uniform sediment, i.e., $[U - (U_a - U_c)]/U_c = 1.0$ corresponds to $U = U_a$ for nonuniform sediment, and $U = U_a \equiv U_c$ for uniform sediment and is taken as the transition from clear-water to live-bed scour conditions for all sediments. Also shown in Fig. 6 are data by Ettema (1976), (d_{50} = 0.55–4.1 mm, σ_g = 1.3–4.6), Chiew (1984), (d_{50} = 0.24–3.2 mm, σ_g = 1.3–6.5), Chee (1982), (d_{50} = 0.24–1.45 mm, $\sigma_g \sim 1.3$), Chabert and Engeldinger (1956), (d_{50} = 0.52–3.0 mm), Hancu (1971), (d_{50} = 2.0 mm), and Shen et al. (1966), (d_{50} = 0.24 mm). Where necessary, the data have been adjusted for relative sediment size.

The effects of σ_g have now been largely accounted for by the introduc-

J. Hydraul. Eng., 1988, 114(10): 1210-1226



(a)



(b)

FIG. 6. Influence of Flow Intensity on Scour Depth: (*a*) Data by Baker (1986), Chiew (1984), and Ettema (1980); (*b*) Data for Uniform Sediments by Chabert and Engeldinger (1956) (Chatou Data), Chee (1982), Chiew (1984), Hancu (1971), and Shen et al. (1966)

tion of U_a into the abscissa. The remaining effects are the smaller values of d_s/D at $[U - (U_a - U_c)]/U_c = 1.0$ as σ_g increases, and a confused picture at higher values of $[U - (U_a - U_c)]/U_c$, where the σ_g value seems of little significance.

For design purposes, an envelope curve is recommended as shown in Fig. 6. It is unreasonable to reduce d_s/D below 2.4 for the larger flows in a design situation, as failure would then be possible at less than the design flood. Accordingly, it is recommended that, for all σ_g values

$$K_I = 2.4 \left|\frac{U - (U_a - U_c)}{U_c}\right|, \quad \text{if } \frac{U - (U_a - U_c)}{U_c} < 1 \quad \text{(10a)}$$

$$K_I = 2.4, \quad \text{if } \frac{U - (U_a - U_c)}{U_c} > 1 \quad \text{(10b)}$$

and, consequently, $K_\sigma = 1.0$.

Shape and Alignment Effects

Factors to account for shape and alignment have been published by many researchers (Chabert and Engeldinger 1956; Laursen 1958; Laursen and Toch 1956; Tison 1940; Venkatadri 1965). A selection of shape factors K_s is given in Table 2. These apply to piers aligned with the flow and are based on $K_s = 1.0$ for cylindrical piers. Streamlined shapes generally have $K_s < 1.0$, but for angles of attack greater than 10°–15°, any advantage of streamlining disappears. In such cases, it is recommended that K_s be not less than 1.0. Alignment factors K_α taken from Laursen (1958) are shown in Fig. 7. It is recommended that Table 2 and Fig. 7 be used with the design method proposed herein, and that K_s be not less than 1.0.

Design Method

The method is shown in flow chart form in Fig. 8. Required input data are the flow conditions y and U for the design flood, the sediment properties expressed either as d_{50} and σ_g or d_{50} and $d_{\max}$, pier size D, and pier shape and alignment.

Preliminary calculations are required to determine u_{*c} for d_{50} using the Shields diagram or Fig. 1, and U_c from the velocity profile of Eq. 2. Next, the median grain size of the coarsest possible armor d_{50a} and the limiting armor velocity U_a are determined using the method that has been outlined. The parameter $[U - (U_a - U_c)]/U_c$ is then found, and its magnitude indicates whether clear-water or live-bed scour is being considered, and thus the appropriate value of K_I can be found.

For clear-water scour, if $\sigma_g > 1.3$, the bed armors. In this case, the effective grain size is d_{50a}, and K_d must be found from Eq. 9, using d_{50a}. If $\sigma_g < 1.3$, armoring does not occur, and d_{50} is used in Eq. 9 to find K_d. For live-bed scour, it is not apparent which size fraction should be used to calculate K_d. Andrews and Parker (1985), drawing on both laboratory and field data, have suggested that with nonuniform sediments, a coarser surface layer is formed even during transport. To be conservative, it is recommended here that the d_{50} size, rather than something larger, be used to find K_d for live-bed scour.

TABLE 2. Pier Shape Factors, K_s

Shape in plan (1)	Length/ width (2)	Reference: Tison (1940) (3)	Laursen and Toch (1956) (4)	Chabert and Engeldinger (1956) (5)	Venkatadri (1965) (6)
Circular	1.0	1.0	1.0	1.0	1.0
Lenticular	2.0	—	0.97	—	—
	3.0	—	0.76	—	—
	4.0	0.67	—	0.73	—
	7.0	0.41	—	—	—
Parabolic nose	—	—	—	—	0.56
Triangular nose, 60°	—	—	—	—	0.75
Triangular nose, 90°	—	—	—	—	1.25
Elliptic	2.0	—	0.91	—	—
	3.0	—	0.83	—	—
Ogival	4.0	0.86	—	0.92	—
Joukowski	4.0	—	—	0.86	—
	4.1	0.76	—	—	—
Rectangular	2.0	—	1.11	—	—
	4.0	1.40	—	1.11	—
	6.0	—	1.11	—	—

The remaining factors K_y, K_α, and K_s are then found, and local scour depth calculated from

$$d_s = K_I K_d K_y K_\alpha K_s D \quad (11)$$

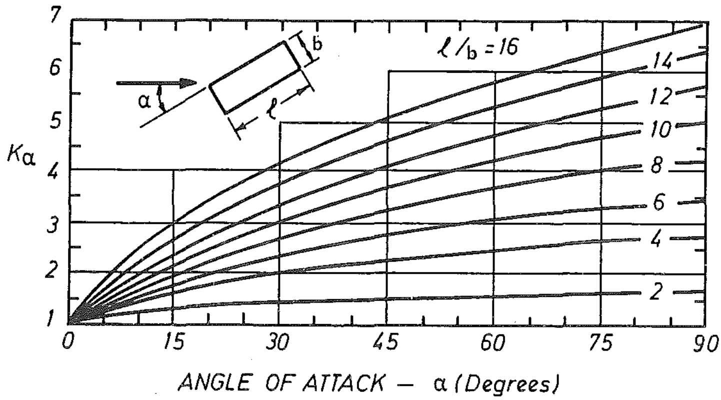


FIG. 7. Alignment Factor K_α for Piers Not Aligned with Flow

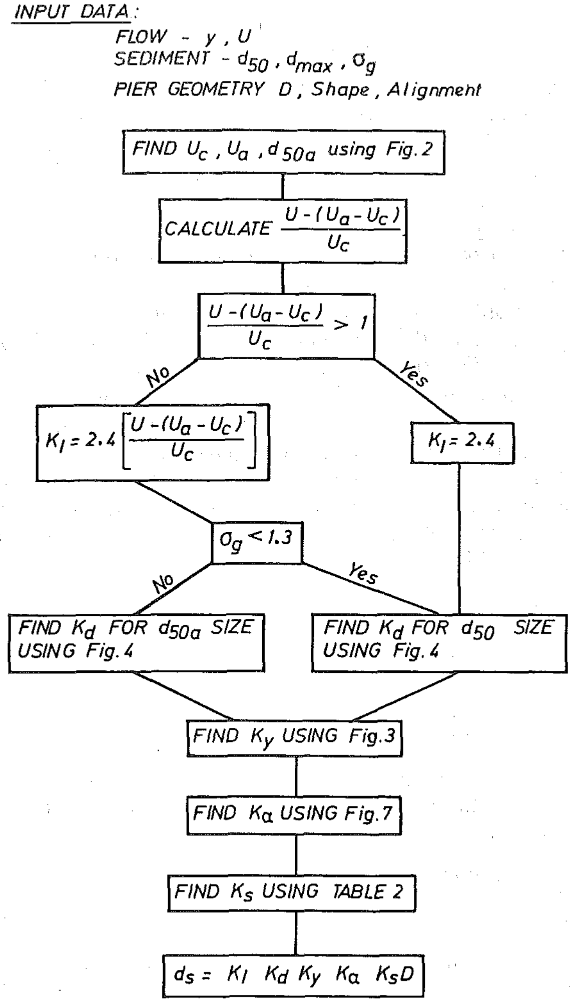


FIG. 8. Flow Chart for Determination of Design Local Scour Depth

For cylindrical piers, K_α and K_s are always unity.

To demonstrate the method, two examples are considered. Find the local scour depth at a 1.2-m diameter pier under the following conditions: first, $d_{50} = 0.5$ mm, $\sigma_g = 2.5$, $y = 1.5$ m, and $U = 1.5$ m/s, and second, $d_{50} = 100$ mm, $\sigma_g = 2.0$, $y = 1.8$ m, and $U = 2.0$ m/s.

For $d_{50} = 0.5$ mm: $u_{*c} = 0.017$ m/s (Fig. 1), and $U_c = 0.412$ m/s (Eq. 2). Equating d_{max} with d_{95}, we have $d_{max} = \sigma_g^{1.65} d_{50} = 2.27$ mm, and $d_{50a} = 1.26$ mm. Thus, $u_{*c} = 0.0265$ m/s, $U_{ca} = 0.582$ m/s, and $U_a = 0.8U_{ca} = 0.466$ m/s. Now $[U - (U_a - U_c)]/U_c = 3.51$, and live-bed scour pertains. Thus, $K_I = 2.4$. To find K_y, $y/D = 1.25$, and thus, $K_y = 0.83$. To find K_d, $D/d = 2{,}400$, and thus, $K_d = 1.0$. Therefore, $d_s = K_I K_d K_y D = 2.4 \times 1.0 \times 0.83 \times 1.2 = 2.38$ m.

For d_{50} = 100 mm: u_{*c} = 0.3 m/s, and U_c = 3.45 m/s. We find $d_{max} = d_{95} = \sigma_g^{1.65} d_{50}$ = 314 mm, and d_{50a} = 174 mm. Thus, $u_{*ca} = 0.03(174)^{1/2}$ = 0.396 m/s, giving U_{ca} = 4.00 m/s, and U_a = 3.20 m/s. Because $U_a < U_c$, take $U_a = U_c$ = 3.45 m/s. Therefore, $[U - (U_a - U_c)]/U_c = 0.580$, and clear-water scour occurs. For clear-water scour, K_d should be based on d_{50a}. Thus $K_I = 2.4 \times 0.580 = 1.39$. To find K_y, $y/D = 1.5$, and thus, K_y = 0.865. To find K_d, $D/d = 6.9$, and thus $K_d = 0.678$. Therefore, $d_s = K_I K_y K_d D = 0.98$ m.

In Figs. 9 and 10, several of the more widely recognized existing design methods are compared with the proposed method. Fig. 9 applies to uniform sediments and Fig. 10 to nonuniform sediments. The comparison is achieved by applying the various design methods to laboratory data for cylindrical piers selected from Chabert and Engeldinger (1956), Shen et al. (1966), Hancu (1971), Ettema (1976, 1980), Chiew (1984), and Baker (1986). The data were chosen to include a wide range of values of pier size, flow velocity, flow depth, sediment size, and sediment grading.

The design methods used for this comparison are those by Laursen and Toch (1956), Hancu (1971), Shen (1971), and Breusers et al. (1977). Each is quoted here in the correct form for application to cylindrical piers. Laursen and Toch (1956) proposed a design curve from their model studies, which Neill (1964) expressed as

$$d_s = 1.35\, D^{0.7} y^{0.3} \qquad (12)$$

Hancu (1971) proposed the equation

$$\frac{d_s}{D} = 2.42\left(2\,\frac{U}{U_c} - 1\right)\left(\frac{U_c^2}{gD}\right)^{1/3} \qquad (13)$$

in which $(2U/U_c - 1) = 1.0$ for live-bed scour. For clear-water scour, Shen (1971) suggested the use of the following equation which formed an envelope to all the known data:

$$d_s = 0.00022\left(\frac{UD}{\nu}\right)^{0.619} \qquad (14)$$

In the case of live-bed scour, Shen (1971) proposed that either the Larras (1963) equation

$$d_s = 1.05\, D^{0.75} \qquad (15)$$

or the Breusers (1965) equation

$$d_s = 1.4D \qquad (16)$$

should be used. Breusers et al. (1977) concluded that scour depth is described by the following function

$$\frac{d_s}{D} = \mathrm{f}\left(\frac{U}{U_c}\right)\left[2 \tanh\left(\frac{y}{D}\right)\right] \qquad (17)$$

in which

$$\mathrm{f}\left(\frac{U}{U_c}\right) = 0, \qquad \text{for } \frac{U}{U_c} < 0.5 \qquad (18a)$$

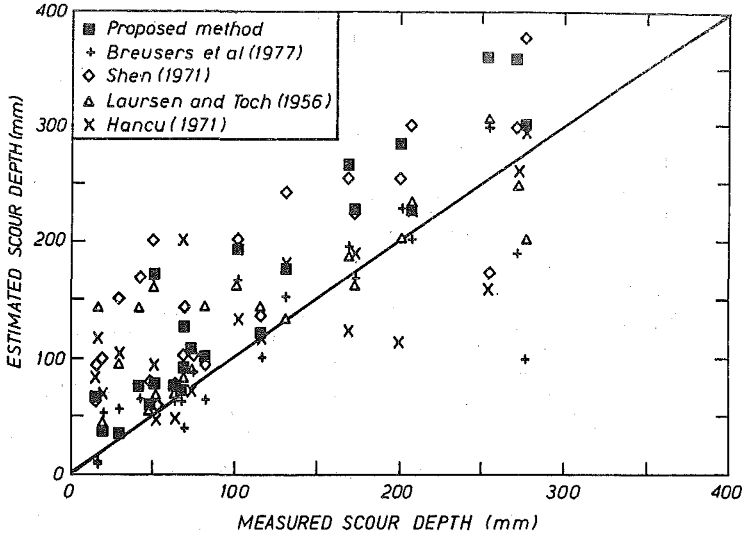


FIG. 9. Comparison for Uniform Sediments of Existing Design Methods with Proposed Method

$$f\left(\frac{U}{U_c}\right) = \left(2\frac{U}{U_c} - 1\right), \quad \text{for } 0.5 \leq \frac{U}{U_c} < 1.0 \quad \text{(18b)}$$

$$f\left(\frac{U}{U_c}\right) = 1, \quad \text{for } \frac{U}{U_c} \geq 1.0 \quad \text{(18c)}$$

In Figs. 9 and 10, the diagonal lines indicate perfect agreement. Points that plot above these lines represent conservative estimates of scour depth. The proposed method is everywhere conservative, as expected, because it is based on envelope curves to data. Being a design method, conservatism is a necessary feature. The existing methods all underestimate the scour depth in some instances, particularly for uniform sediments. For nonuniform sediments, the existing methods generally predict larger scour depths than the proposed method. This is also expected because the existing methods do not take into account the sediment grading.

Conclusion

A design method is presented that enables prediction of local scour depths at bridge piers. The method is based on laboratory data, derived mainly from experiments with cylindrical piers, for which the largest possible local scour depth is 2.4D. Multiplying factors, which have the effect of reducing this value, are applied where clear-water scour conditions exist (K_I), the flow depth is relatively shallow (K_y), and the sediment is relatively large (K_d). In the case of noncylindrical piers, two additional multiplying factors are applied. These are K_s, the shape factor, which can

J. Hydraul. Eng., 1988, 114(10): 1210-1226

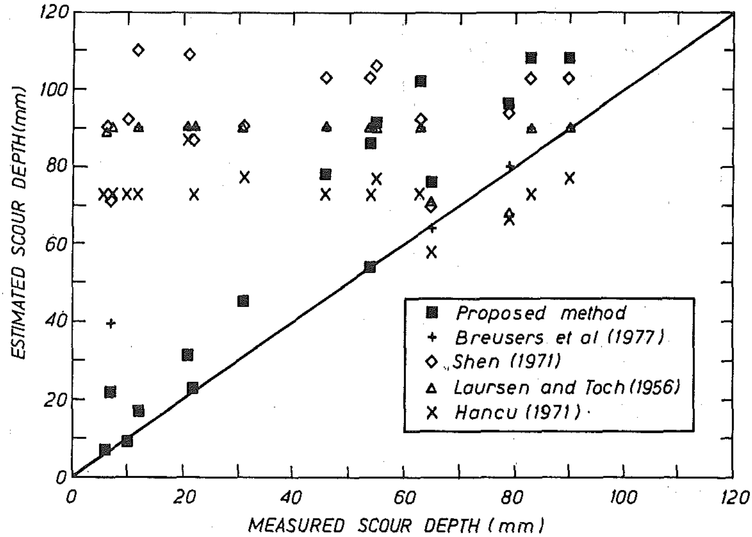


FIG. 10. Comparison for Nonuniform Sediments of Existing Design Methods with Proposed Method

have values up to 1.4, and K_α, the alignment factor, which can be as large as 7.0. The design method is summarized in the flow chart (Fig. 8).

Acknowledgments

Appreciation is recorded for the financial support provided by the Road Research Unit, National Roads Board (N.Z.) for many of the projects described herein. The work of many students involved in these projects is gratefully acknowledged.

Appendix I. References

Andrews, E. D., and Parker, G. (1986). "Formation of a coarse surface layer as the response to gravel mobility." *Sediment transport in gravel bed rivers*, C. R. Thorne, J. C. Bathurst, and R. D. Hey, eds., John Wiley and Sons, New York, N.Y., 260–300.

Baker, R. E. (1986). "Local scour at bridge piers in non-uniform sediment," thesis presented to the University of Auckland, at Auckland, New Zealand, in partial fulfillment of the requirements for the degree of Master of Engineering.

Blench, T. (1969). *Mobile-bed fluviology.* University of Alberta Press, Edmonton, Alberta.

Breusers, H. N. C. (1965). "Scouring around drilling platforms." *Hydraulic Research 1964/65, IAHR Bull.*, 19, 276.

Breusers, H. N. C., Nicollet, G., and Shen, H. W. (1977). "Local scour around cylindrical piers." *J. Hydr. Res.*, 15(3), 211–252.

Chabert, J., and Engeldinger, P. (1956). *Etude des affouillements autour des piles des ponts.* Laboratoire National d'Hydraulique, Chatou, France (in French).

Chee, R. K. W. (1982). "Live-bed scour at bridge piers," thesis presented to the University of Auckland, at Auckland, New Zealand, in partial fulfillment of the requirements for the degree of Master of Engineering.

Chiew, Y. M. (1984). "Local scour at bridge piers," thesis presented to the University of Auckland, at Auckland, New Zealand, in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Engineering.
Chin, C. O. (1985). "Stream bed armouring," thesis presented to the University of Auckland, at Auckland, New Zealand, in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Engineering.
Davoren, A. (1985). "Local scour around a cylindrical bridge pier." *Publication No. 3*, Hydrology Centre, Ministry of Works and Development. Christchurch, New Zealand.
Ettema, R. (1976). "Influence of material gradation on local scour," thesis presented to the University of Auckland, at Auckland, New Zealand, in partial fulfillment of the requirements for the degree of Master of Engineering.
Ettema, R. (1980). "Scour at bridge piers," thesis presented to the University of Auckland, at Auckland, New Zealand, in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Engineering.
Garde, R. J. (1961). "Local bed variations at bridge piers in Alluvial Channels." *Univ. of Roorke Res. J.*, IV(1).
Hancu, S. (1971). "Sur le calcul des affouillements locaux dams la zone des piles de ponts." *Proc., 14th I.A.H.R. Congress*, Paris, France, Vol. 3, 299–313.
Henderson, F. M. (1966). *Open channel flow*. MacMillan Publishing Co. Inc., New York, N.Y.
Jain, S. C., and Fischer, E. E. (1979). "Scour around bridge piers at high Froude numbers." *Report No. FH-WA-RD-79-104*, Federal Highway Administration, Washington, D.C.
Larras, J. (1963). "Profondeurs maximales d'erosion des fonds mobiles autour des piles en riviere." *Ann. Ponts et Chaussees*, 133(4), 411–424.
Laursen, E. M. (1958). "Scour at bridge crossings." *Bulletin No. 8*, Iowa Highway Research Board, Ames, Iowa.
Laursen, E. M. (1962). "Scour at bridge crossings." *Trans.*, ASCE, 127, Part I, 116–179.
Laursen, E. M., and Toch, A. (1956). "Scour around bridge piers and abutments." *Bulletin No. 4*, Iowa Highway Research Board, Ames, Iowa.
Melville, B. W. (1975). "Local scour at bridge sites," thesis presented to the University of Auckland, at Auckland, New Zealand, in partial fulfillment of the requirements for the degree of Doctor of Philosophy in Engineering.
Neill, C. R. (1964). "River bed scour, a review for bridge engineers." *Contr. No. 281*, Research Council of Alberta, Calgary, Alberta, Canada.
Neill, C. R. (1968). "Note on initial movement of coarse uniform material." *J. Hydr. Res.*, IAHR, 6(2).
Raudkivi, A. J., and Sutherland, A. J. (1981). "Scour at bridge crossings." *Report No. 51*, Road Research Unit, National Roads Board, Wellington, New Zealand.
Shen, H. W. (1971). "Scour near piers." *River mechanics*, Vol. II, Chap. 23, Ft. Collins, Colo.
Shen, H. W., et al. (1966). "Mechanics of local scour." National Bureau of Standards, Institute of Applied Technology, U.S. Department of Commerce, Washington, D.C.
Tison, L. J. (1940). "Erosion autour des piles de ponts en riviere." *Annales des Travaux Publics de Belgique*, 41(6), 813–817 (in French).
Venkatadri, C., et al. (1965). "Scour around bridge piers and abutments." *Irrig. Power*, Jan., 35–42.

Appendix II. Notation

The following symbols are used in this paper:

Al = effect of pier alignment;
D = pier width as shown in Fig. 7 (pier diameter for cylindrical piers);

d_{max} = maximum particle size for a nonuniform sediment;
d_s = equilibrium scour depth below mean bed level;
d_{50} = median particle size;
d_{84} = particle size for which 84% are finer;
d_{50a} = d_{50} size of the coarsest armor layer;
g = gravitational acceleration;
K_d = sediment size factor;
K_I = flow intensity factor;
K_s = pier shape factor;
K_y = flow depth factor;
K_α = pier alignment factor;
K_σ = sediment gradation factor;
m = exponent in Eq. 3;
Sh = effect of pier shape;
U = mean approach flow velocity;
U_a = mean approach flow velocity at the armor peak = 0.8 U_{ca};
U_c = mean approach flow velocity at threshold condition;
U_{ca} = mean approach flow velocity beyond which armoring of channel bed is impossible;
u_{*c} = critical shear velocity defined by Shields function;
u_{*ca} = critical shear velocity of armored bed;
y = flow depth;
ν = kinematic viscosity;
ρ = fluid density;
ρ_s = sediment density; and
σ_g = geometric standard deviation of particle size distribution = d_{84}/d_{50}.

J. Hydraul. Eng., 1988, 114(10): 1210-1226