Scientific Research Communications

Review Article

Sci. Res. Comm. (2021)
Volume 1, Issue 1
doi: 10.52460/src.2021.006

On Local Scouring at Single Piers

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Received: 29.06.2021 Accepted: 16.07.2021

Abstract

The paper mostly summarizes disperse contributions of the authors published during the last fifteen years on the scour depth at single piers. These contributions rely on unique experiments in the sense that they are systematically longer than most of those found in the literature. The characterization of the effects of flow intensity, relative sand size, flow shallowness, time and pier shape and alignment is significantly improved as compared with existing literature. Our contributions consist on refinements of the model suggested by the school of Auckland, initiated by Raudkivi and boosted by Melville and his students. A considerable number of empirical equations and charts expresses those contributions.

Keywords: Single piers; flow intensity; flow depth factor; sediment size factor; time factor; pier shape and alignment.

1. Introduction

Local scour around bridge piers and abutments is a frequent cause of partial failure or collapse of bridges. The reconstruction or rehabilitation of destroyed or damaged bridges frequently amounts to significant monetary costs. Above all, the casualties that occur in many of these disasters raise the societal claim for security. Enhanced security involves failure prevention, which in turn requires the accurate prediction of the scour depth or the adoption of proper mitigation scour countermeasures. In spite of the remarkable progresses registered since the mid-last century, scouring remains only a partly solved problem, due to the large number of variables involved in the processes and the inherent complexity of their phenomenological interactions.

The scope of this paper is scouring at single piers. Single piers are characterized by a unique cross-section along their main axes, assumed vertical. Local scouring at such single piers has been extensively studied. Research has been made mostly through experimentation. Early contributions of Chabert and Engeldinger (1956), Laursen and Toch (1956), Laursen (1963) or Shen et al. (1966) deserve to be mentioned, whereas one of the most comprehensive reviews on bridge scouring was offered by Melville and Coleman (2000).

For uniform flows in straight open channels, the maximum scour depth at a given moment, d_s, was shown to be described through the following parametric equation (see Fael (2007)):

$$\Pi_{d_s} = \phi\left(\Pi_d ; \Pi_U ; \Pi_{D_{50}} ; \sigma_D ; s; \Pi_\nu ; \Pi_f ; \Pi_\theta ; \Pi_W ; \Pi_G ; \Pi_t \right) \tag{1}$$

where Π stands for non-dimensional parameter, ϕ means "function of" and subscripts refer to the variables influencing scouring. These are, namely, d = flow depth; U = average approach flow velocity; D_{50} = median grain size of the bed sediment; ν = kinematic water viscosity; f = pier shape; θ = pier alignment; W = channel width; G = geometry of the channel cross-section. Non-dimensional parameters σ_D and s stand for, respectively, gradation coefficient and specific gravity of the bed sediment. The basic variables used to derive Eq. (1) were the characteristic length of the pier cross-section, D_p, the gravitational acceleration, g, and the water density, ρ.

$$\Pi_{d_s} = \phi\left(\Pi_d ; \Pi_U ; \Pi_{D_{50}} ; \sigma_D ; \Pi_\nu ; \Pi_f ; \Pi_\theta ; \Pi_t \right) \tag{2}$$

In this equation, it is assumed that the effect of flow contraction on scouring at single piers vanishes in wide channels and that the specific sediment gravity is practically invariant, notably for sand. It is also assumed that the rectangular cross-section is the reference shape of open channels.

According to Melville and Coleman (2000), the previous equation can be materialized for piers as follows:

$$\Pi_{d_s} = K_d K_U K_{D50} K_{\sigma_D} K_\nu K_f K_\theta K_t \quad \text{with} \quad \Pi_{d_s} = \frac{d_s}{D_p} \quad \text{or} \quad \Pi_{d_s} = \frac{d_s}{d} \tag{3}$$

It should be noted here that K_d refers to the effect of the relative flow depth or flow shallowness, $\Pi_d = d/D_p$; K_U accounts for the effect of flow intensity, $\Pi_U = U/U_c$ (U_c = critical velocity of beginning of sediment motion); K_{D50} reflects the effect of relative sediment size or sediment coarseness, $\Pi_{D50} = D_p/D_{50}$; $K\sigma_D$ refers to the armoring effect (which mostly depends on σ_D); K_ν accounts for the effect of fluid viscosity as captured through any form of the Reynolds number, *e.g.*, $\Pi_\nu = u_* D_{50}/\nu$ (u_* = friction velocity); K_f and K_θ attend, respectively, to the effects of shape and alignment of the pier; and K_t varies with the non-dimensional time, $\Pi_t = Ut/D_p$.

In the last fifteen years, we have revisited local scouring at single piers inserted in channel beds composed of practically uniform ($\sigma_D < 1.5$; $K\sigma_D = 1.0$) non-ripple forming sand ($D_{50} > 0.6$ mm). Under these conditions, we have indeed contributed to the enhanced characterization of Eq. (3). This paper reviews those contributions, synthetizing mostly the works of Lança et al. (2010), Simarro et al. (2011), Lança et al. (2013) and Fael et al. (2016). It does not cover the effects of water viscosity, bed armoring (due to the wide granular distribution of the bed material) or ripple forming bed material ($D_{50} \leq 0.6$ mm) since we performed only a very limited number of experiments on these effects. The effect of viscosity can indeed be anticipated to be negligible since the flow field around obstacles tends to be rough turbulent, irrespective of the approach flow regime (smooth, transitional or rough).

Prior to addressing those contributions, we include a short description of the experimental facilities and granular materials used in the studies, as reported by Fael et al. (2014), and assess the calculation of the equilibrium scour depth from scour depth time records.

2. Experimental Facilities and Granular Materials

Three horizontal-bed flumes were used in the studies. Each flume included a central reach consisting of a rectangular recess box in the bed (Figure 1), where the piers were installed. The main features of the flumes are shown in Table 1, where W = flume width, L = flume length, ℓ = distance from flume entrance to the recess box, ℓ_1 = length of bed recess box and d_r = its depth (Figure 1).

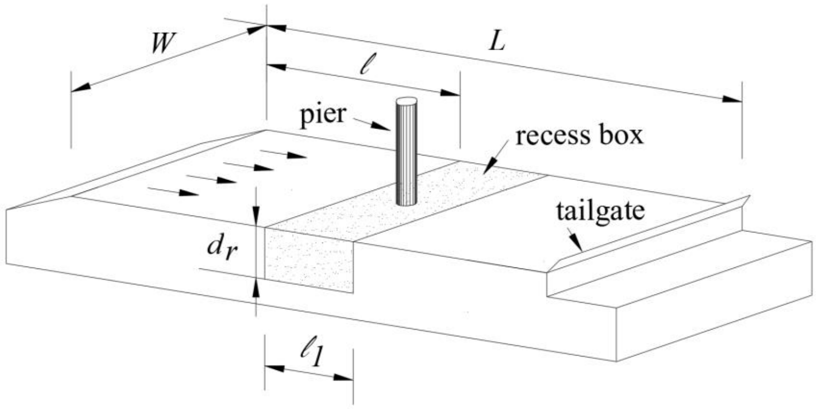

Figure 1. Sketch of the flumes

Table 1. Main features of the flumes

Flume	W (m)	L (m)	ℓ (m)	ℓ_1 (m)	d_r (m)
F_1	4.00	28.00	13.90	3.00	0.60
F_2	0.83	12.70	5.00	3.10	0.35
F_3	1.00	33.20	16.00	3.20	0.35

Two natural quartz sands were used: sand S_1, defined by D_{50} = 1.28 mm and σ_D = 1.46; sand S_2, defined by D_{50} = 0.86 mm and σ_D = 1.36). Herein $\sigma_D = (D_{84.1}/D_{50} + D_{50}/D_{15.9})/2$ and D_{50} = sand particle sieving diameter for which 50% are finer by weight. Both sands can be considered as uniform, since $\sigma_D < 1.5$. The specific gravity was verified to be $s \approx 2.65$ in all cases.

Distinctive characteristics of the experiments were 1) the absence of contraction scour and wall effects and 2) their long duration. They typically lasted longer than 7 days, this way allowing the proper assessment of equilibrium scour depth. Another remarkable feature is the large number of experiments covering uncommon ranges of the relative sediment size or coarseness, $\Pi_{D50} = D_p/D_{50}$.

3. Experimental Equilibrium Scour Depth

Local scouring occurs for two distinct sediment transport conditions: i) under *clear-water*, i.e., in the absence of sediment movement in the bed of the approach flow, which implies the bed shear stress to be smaller or, at most, equal to the critical bed shear stress of beginning of sediment motion; ii) under *live-bed*, i.e., when noticeable bed sediment movement occurs in the approach flow, meaning that the bed shear stress is larger than the critical bed shear stress.

Conceptually, under *clear-water*, a scour hole is in equilibrium when the scour depth practically does no increase anymore, while under *live-bed*, a scour hole is in equilibrium when the time-averaged amount of sediment leaving the scour hole equals the time-averaged amount of sediment that it captures from upstream. Starting from a flatbed, the time required to reach equilibrium scour depends on the state of sediment movement. For *live-bed* conditions, there is the simultaneous removal of grains originated from the scour hole and of those trapped by the same scour hole as they move downstream. The quantity of material that initially leaves the scour hole exceeds the quantity of material coming in and the scour depth increases. After a comparatively short time, both quantities tend to be equal. This state of equilibrium is known as dynamic equilibrium since the scour depth normally oscillates. The oscillations reflect the movement of bedforms – ripples, dunes, antidunes – that, in turn, induce periodical variations of the amount of sediments falling into the scour hole. In this context, it is pertinent to define the maximum scour depth, d_{sm}, as the sum of the average equilibrium scour depth, d_{se}, with a value that depends on the semi-amplitude of bedforms.

For *clear-water* flow conditions, the increase of scour depth induces the continuous reduction of bed shear stress inside the scour hole until it becomes conceptually insufficient to transport sediment particles downstream. As scouring progresses, the scour rate decreases. The equilibrium depth is reached asymptotically and equilibrium is usually said to be static, in contrast with the *live-bed* case. It should be noticed here that, though the most frequent scour disasters occur under live-bed conditions associated with floods, scour due to long-lasting clear-water flows can originate slightly deeper scour holes.

The scour depth has classically been assumed to evolve in three phases, particularly in the *clear -water* flow case. These phases are i) the *initial phase*, where scour depth increases very quickly; ii) the *principal phase*, where the scour hole increases in depth and plan extent at a progressively decreasing rate; iii) the *equilibrium phase*, where the scour depth is assumed to stop increasing.

Until to the beginning of the present millennium, most studies assumed the existence of a finite time to reach equilibrium, both for *clear-water* and *live-bed* conditions. Many research works, including those of Cardoso and Bettess (1999), Fael et al. (2006) or Cardoso and Fael (2010), adopted this concept. At the dawn of the XXI century, Oliveto and Hager (2002; 2005) stated that “end scour as the equilibrium state between the vortical agents and the resistance of sediments to be scoured does not normally exist”, in disagreement with previous research works. Although there is no evidence supporting the inexistence of end scour, we recognize that the probability of occurrence of a sufficiently strong turbulent event capable of entraining bed grains is never null. Thus, it can be assumed that the mentioned probability decreases as scouring progresses, the scour depth evolving to a finite equilibrium value.

In other words, time to equilibrium can be conceived as infinity (notably, under clear-water flow conditions) although the equilibrium scour depth is indeed finite. Under these assumptions, the equilibrium scour depth of each experiment was derived by adjusting the polynomial equation,

$$d_s = p_1\left(1-\frac{1}{1+p_1p_2t}\right)+p_3\left(1-\frac{1}{1+p_3p_4t}\right)+p_5\left(1-\frac{1}{1+p_5p_6t}\right) \tag{4}$$

to the recorded time evolution of the scour depth. Parameters p_1 to p_6 were obtained through regression analysis and the equilibrium scour depth was calculated as $d_{se} = p_1 + p_3 + p_5$ for $t = \infty$. The above polynomial was suggested by Lança et al. (2010) as an improvement of a similar proposal by Bertoldi and Jones (1998). Lança et al. (2010) have shown that the method leads to practically invariant results for d_{se} as soon as experiments last for at least 7 days, which applies to the present experimental data.

4. Flow Intensity Factor

Referring to Eq. (3), let us consider that scouring occurs in a uniform flow on a straight, wide and rectangular open channel. Assuming also that i) the bed material is uniform ($K\sigma_D = 1.0$); ii) the pier is a cylinder ($K_\theta = K_f = 1.0$); iii) viscosity does not play a significant role on scouring ($K_\nu = 1.0$); iv) the scouring process has sufficiently approached the equilibrium stage ($K_t = 1.0$); v) K_{D50} = const., Eq. (3) comes

$$\prod_{dse} = \varphi(\Pi_d;\Pi_U) \qquad \text{with} \qquad \prod_{dse} = \frac{d_{se}}{D_p} \qquad \text{or} \qquad \prod_{dse} = \frac{d_{se}}{d} \tag{5}$$

where d_{se} stands for equilibrium scour depth. For slender cylindrical piers, when the average approach velocity, U, reaches a value of the order of $0.5U_c$ (Chabert and Engeldinger, 1956; Hanco, 1971), the flow structure around the pier induces local bed shear stresses that promote the beginning of grains motion, i.e., the initiation of the scour process (see Figure 2) although there is no sediment transport in the approach channel. To a given value of the approach flow velocity in the range $\approx 0.5U_c < U < U_c$ corresponds one single value of the equilibrium scour depth, d_{se}, as soon as $\Pi_d = d/D_p$ remains constant. The scour depth, d_{se}, increases linearly with U/U_c until it reaches its maximum for $U = U_c$ ($\Pi_U = 1.0$). Increasing further the velocity, the bed grains of the approach flow will move and dunes will develop in the bed. For $U > U_c$, the movement of dunes downstream feeds the scour hole with successive waves of bed material that the vortical system (horseshow vortex plus wake vortices) keeps removing from the scour hole, this way reducing its capacity to erode the original bed. Through this mechanism, dunes tend to induce a small decrease of d_{se} as U increases in the range $U_c < U < \approx 2U_c$ ($1 < \Pi_U < \approx 2$). A minimum of the equilibrium scour depth occurs for $U \approx 2U_c$. For approach flow velocities in the range $\approx 2U_c < U < \approx 4U_c$, dunes become longer and their upstream slope become milder as flow velocity increases. The scouring process will be influenced by the arrival of other dunes with lower height and volume; the vortical system tends to be increasingly able to move the incoming sediments downstream. Consequently, the equilibrium scour depth increases again gradually with the approach flow velocity. A new maximum of d_{se} is reached for $U \approx 4U_c$ ($\Pi_U \approx 4$). This maximum is of the order of the one observed for $U = U_c$ but it occurs for the upper-regime flat bed. For velocities higher than $\approx 4U_c$, antidunes normally develop, and the scour depth tends to decrease again for the same reasons as those why the occurrence of dunes also decreases the equilibrium scour depth.

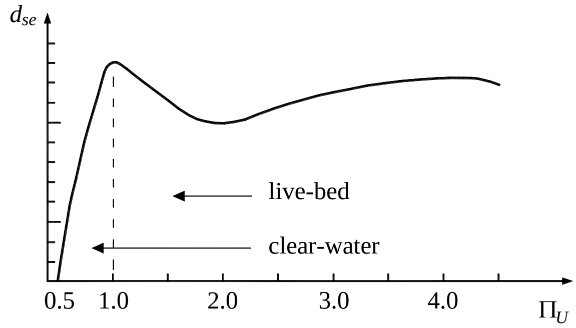

Figure 2. Typical variation of d_{se} with U for comparatively coarse uniform bed sediment (adapted from Breusers and Raudkivi (1991))

It should be emphasized here that the condition of scour inception – given as U/U_c = 0.5 and meaning that the critical value of the flow intensity for scour inception would be Π_{Ui} = 0.5 – has not been consensual. For instance, Chiew (1995) indicated Π_{Ui} = 0.3, Melville (1992, 1997) suggested charts according to which Π_{Ui} = 0 and Hager and Oliveto (2002) proposed an equation to calculate Π_{Ui} as a function of the flow blockage.

In view of the lack of consensus, Fael et al. (2006) investigated the scour inception at vertical wall abutments by performing 31 long lasting (more than 7 days) experiments with sand S1 in the flume F_1. Abutments' length, ℓ_a, varied in the range 0.64 m < ℓ_a < 1.86 m, the flow depth was kept practically constant ($\approx 0.06 \leq d \leq \approx 0.07$ m), the relative abutment length, $1/\Pi_d = \ell_a/d$, spanned from 8.9 to 30.1, the flow intensity covered the range $\approx 0.4 < \Pi_U < \approx 1.0$ and wall effects and contraction scour were absent. Fael et al. (2006) also selected 25 experiments from the literature, covering lower values of ℓ_a/d, between 2.0 and 17.4, and $\approx 0.6 < \Pi_U < \approx 1.0$. From the data collected, Fael et al. (2006) concluded that the scour inception at vertical wall abutments is defined by

$$\Pi_{Ui} = \frac{1}{1 + 0.402\left(\ell_a/d\right)^{0.648}} \tag{6}$$

Since abutments can be conceptually regarded as half-width piers, meaning that they mimic piers with $D_p = 2\ell_a$, the previous equation can be written as

$$\Pi_{Ui} = \frac{1}{1 + 0.257\left(D_p/d\right)^{0.648}} \tag{7}$$

Eq(s). (6) and (7) are plotted in Figure 3 together with the equation suggested by Hager and Oliveto (2002). In this figure, ℓ_O stands for pier width, D_p, or abutment length, ℓ_a, depending on the equation.

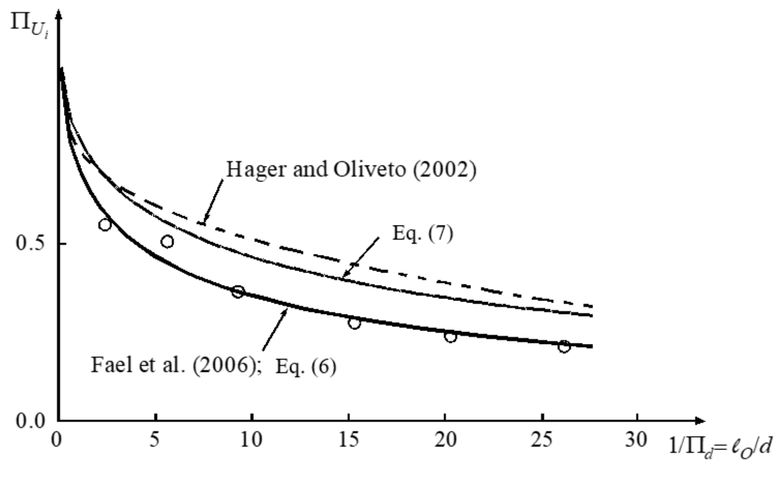

Figure 3. Variation of Π_{Ui} with $1/\Pi_d$

Further assuming that the scour reduction occurring for $\Pi_U \approx 2.0$ is negligible, it can be concluded that K_U, defined as

$$K_U = \frac{d_{se}\ (\text{for a given } \Pi_U)}{d_{se}\ (\text{for } \Pi_U = 1.0)} \tag{8}$$

can be obtained as follows:

$$K_U = \begin{cases} 0 & \Pi_U < \Pi_{Ui} \\ \dfrac{1}{1-\Pi_{Ui}}\left(\Pi_U - \Pi_{Ui}\right) & \Pi_{Ui} \le \Pi_U < 1 \\ 1 & \Pi_U \ge 1 \end{cases} \tag{9}$$

This equation reflects the influence of Π_U and Π_d – trough Π_{Ui} – on K_U. It compares with other contributions according to Figure 4.

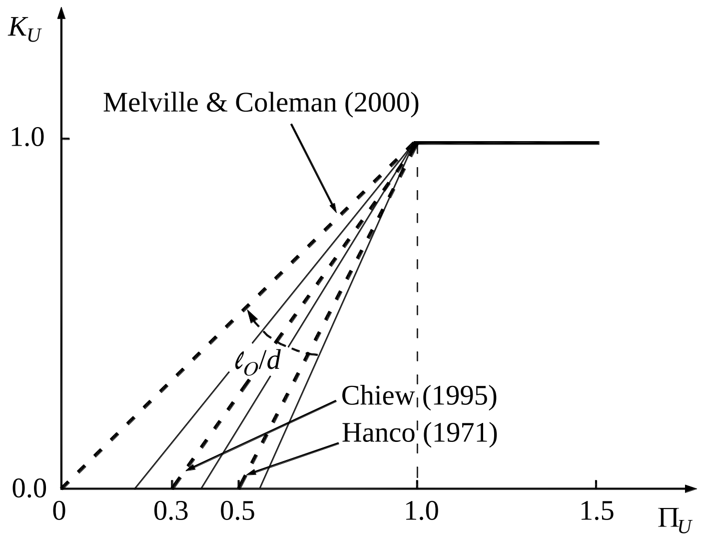

Figure 4. Variation of K_U with Π_U

5. Effects of Flow Depth and Sediment Size on the Equilibrium Scour Depth

Resuming the scenario of rough turbulent uniform flows on uniform non-ripple forming sand-beds in straight, very wide, rectangular open channels and also considering constant values of flow intensity, Π_U, Eq. (3) reads

$$\Pi_{d_s} = K_d K_{D_{50}} K_f K_\theta K_t \tag{10}$$

The equilibrium scour depth – where $\Pi_{ds} = \Pi_{dse}$ – at cylindrical piers ($K_f = K_\theta = 1.0$) can be described as

$$\Pi_{dse} = K_d K_{D_{50}} \tag{11}$$

It is unanimously recognized that the relative approach flow depth, $\Pi_d = d/D_p$, is a key parameter of the scour process. On the contrary, many authors (*e.g.* Ettema (1980), Melville and Chiew (1999)) have successively assumed that the normalized equilibrium scour depth, $\Pi_{dse} = d_{se}/D_p$ or $\Pi_{dse} = d_{se}/d$ does not depend on the relative sediment size, $\Pi_{D50} = D_p/D_{50}$, as soon as $\Pi_{D50} > \approx 25 – 50$. Under this assumption, the equilibrium scour depth would be given as

$$\frac{d_{se}}{D_p} = \varphi\left(\frac{D_p}{d}\right) \quad \text{or} \quad \frac{d_{se}}{d} = \varphi\left(\frac{D_p}{d}\right) \quad \text{or} \quad d_{se} = \varphi\left(d; D_p\right) \tag{12}$$

Kandasamy (1989) collected scour data for piers and abutments corresponding to comparatively low values of Π_{D50}. In the attempt to unify the analysis of both obstacle types, he used a common characteristic length, ℓ_O, to represent the abutment length (assuming $D_p = 2.3\ \ell_a$) and the pier diameter. He obtained the 3D representation of the surface

$$d_{se} = \varphi\left(d; \ell_O\right) \tag{13}$$

included in Figure 5 for $\Pi_U \approx 1.0$, *i.e.*, corresponding to the maxima scour depths. According to Kandasamy (1989), this surface can be divided into four zones. Zone 1 consists of the planar area defined by $d > b_1\ell_O$ (surface OAB). In this zone, covering the domain of slender piers, the scour depth does not depend on the flow depth, meaning that

$$d_{se} = C_1 \ell_O \tag{14}$$

Zone 4 (surface ODE) would be a planar surface too, obeying the condition $\ell_O > b_2 d$. According to Kandasamy (1989), it would apply to long abutments, for which the equilibrium scour depth is independent of the obstacle length and only depends on the flow depth ($d_{se} = C_2 d$).

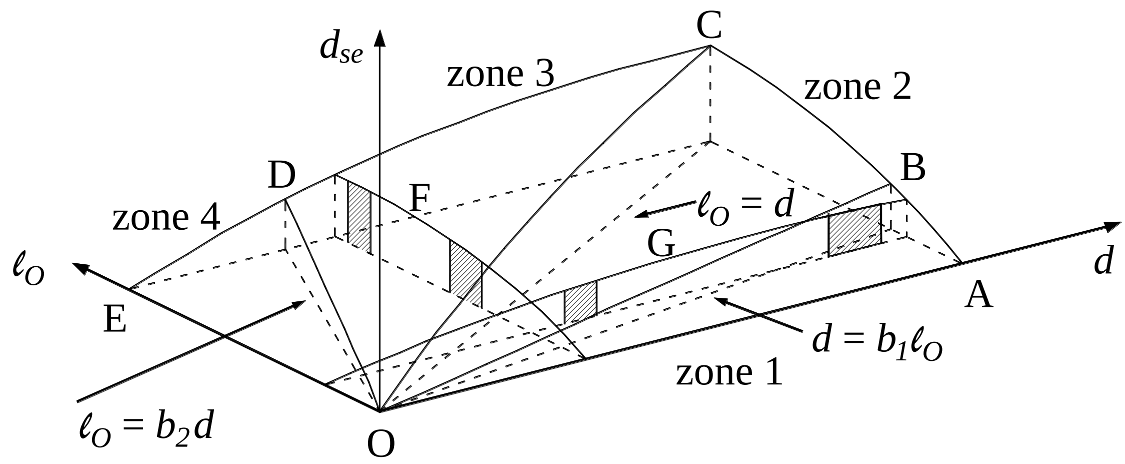

Figure 5. Shape of the function $d_{se} = d_{se}\left(d; L\right)$ as defined by Kandasamy (1989)

Figure 5 includes curves F and G, parallel to the axis of obstacle length and to the axis of flow depth, respectively. The curve F establishes the variation of d_{se} with ℓ_O for d = const.; curve G establishes the variation of d_{se} with d for ℓ_O = const. Along curve F, which intersects zones 1, 2 and 3, the scour depth increases strong and linearly with ℓ_O in zone 1; in zone 2 (where piers prevail), the increase slows down; this effect is even more pronounced in zone 3, corresponding to $\ell_O > d$. Had the curve crossed zone 4, d_{se} would become constant and proportional to d. Qualitatively, curve G presents the same type of variation as curve F; it is obvious that the rate of increase of scour depth diminishes from zone 4 to zone 1. To the right of the line OB, d_{se} is independent of d and becomes $C_1\ell_O$ (see Eq. (14)).

Several contributions can be found in the literature corresponding to specific forms of Eq. (13). One of the most widely spread was obtained by Melville (1992; 1997) who suggested the following envelop curves for the calculation of equilibrium scour depth at cylindrical piers:

$$\frac{d_{se}}{D_p} = 2.4 \qquad \text{for} \qquad D_p / d \le 0.7 \left(\Pi_d \ge 1.43\right) \tag{15}$$

$$\frac{d_{se}}{D_p} = 2\left(\frac{D_p}{d}\right)^{0.5} \qquad \text{for} \qquad 0.7 < D_p / d \le 5 \left(0.2 \le \Pi_d < 1.43\right) \tag{16}$$

$$\frac{d_{se}}{d} = 4.5 \qquad \text{for} \qquad D_p / d > 5 \left(\Pi_d < 0.2\right) \tag{17}$$

It should be noticed here that the contribution of Melville (1992; 1997) implicitly fixes b_1 = 0.7, b_2 = 5, C_1 = 2.4 and C_2 = 4.5.

The cancelation of the relative sediment coarseness effect – associated with Π_{D50} – assumed by Ettema (1980), Kandasamy (1989) and Melville (1992; 1997), among many others, was disputed by Sheppard et al. (2004) and Lee and Sturm (2009), according to whom Π_{dse} decreases with increasing relative sediment size, particularly, for $\Pi_{D50} > \approx 50$. In spite of the importance of this contribution, it did not sufficiently pervade the hydraulics community until to the present. Consequently, Lança et al. (2013) reassessed the influence of Π_{D50} on scouring. They performed thirty eight tests with sand S_2, lasting between 7 and 14 days on this effect, covering values of the relative sediment size in the range $58 \le \Pi_{D50} \le 465$ and relative flow depth, Π_d, in the range $0.5 \le \Pi_d \le 5.0$, for flow intensity close to the condition of initiation of motion ($0.93 \le \Pi_U \le 1.04$). In their tests, there were no significant wall as well as contraction scour effects.

The collected data allowed the characterization of Eq. (11). The values of $\Pi_{dse} = d_{se}/D_p$ are plotted against Π_{D50} in Figure 6. Data of six long duration clear-water experiments ($T_d \ge 6$ days) by Sheppard et al. (2004) for $\Pi_{D50} > 500$ and Π_U sufficiently close to 1.0 (0.85 to 1.21) are also included. Figure 6 renders it clear that Π_{D50} influences Π_{dse} by decreasing the normalized scour depths as Π_{D50} increases in the range of the study.

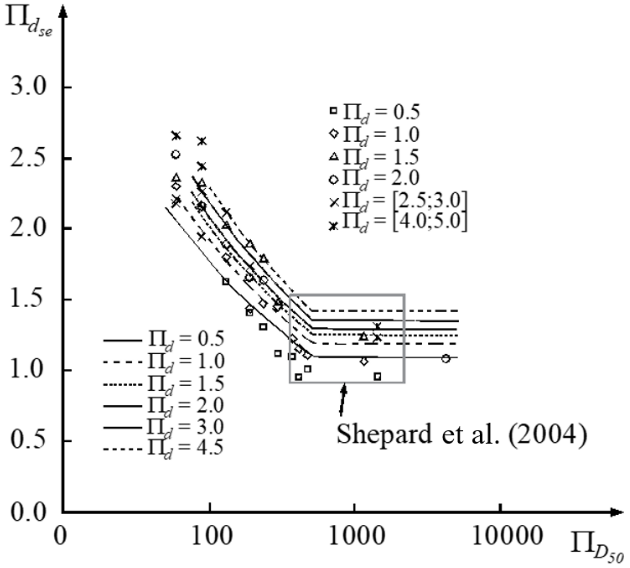

Figure 6. Effect of Π_{D50} and Π_d on Π_{dse}, Lança et al. (2013)

Lança et al. (2013) suggested the upper-bound predictor of the equilibrium scour depth given by Eq. (11), assuming that K_d is the predictor of Melville (1997) slightly modified to read:

$$K_d = \begin{cases} 2.3\left(\Pi_d\right)^{1/3} & 0.50 \le \Pi_d \le 1.45 \\ 2.6 & \Pi_d > 1.45 \end{cases} \tag{18}$$

and that K_{D50} is given by:

$$K_{D50} = \begin{cases} 1.0 & 60 < \Pi_{D50} \le 100 \\ 5.8\left(\Pi_{D50}\right)^{-0.38} & 100 < \Pi_{D50} \le 500 \\ 0.55 & \Pi_{D50} > 500 \end{cases} \tag{19}$$

In engineering practice, the use of Eq(s).(18) and (19) requires the application of appropriate multiplying factors – see Eq. (3) – to take into account the effects of flow intensity, viscosity, pier shape, pier alignment, gradation coefficient, flow contraction, channel cross-section shape and time. The effect of flow intensity was presented in section 4. Sections 6 and 7 assess the time factor and the pier shape and alignment factors, respectively.

6. Time Factor

The data of Lança et al. (2010), covering time durations between 24.9 and 45.6 days, complemented with data of one 58.2 days-long experiment found in the literature, were reassessed by Simarro et al. (2011) who have shown that the exponential function suggested by Franzetti et al. (1982),

$$K_t = \frac{d_s}{d_{se}} = 1 - \exp\left[-a_1 \Pi_t^{a_2}\right] \qquad \text{with} \qquad \Pi_t = \frac{Ut}{D_p} \tag{20}$$

is a good predictor of the scour depth time evolution for *clear-water* flow conditions. The use of this equation requires the proper knowledge of a_1 and a_2, assuming that d_{se} is known.

In the sequence of the assessment performed by Simarro et al. (2011), Lança et al. (2013) also revisited the proposal of Franzetti et al. (1982). Contrary to these authors, they have concluded that a_1 and a_2 are not constant. The multiplying coefficient a_1 varies in the range $0.005 \leq a_1 \leq 0.080$, with an average value of 0.031, whereas a_2 varies within the range $0.212 \leq a_2 \leq 0.458$, with an average value of 0.311. Lança et al. (2013) have shown that a_1 and a_2 depend on Π_{D50} as follows:

$$a_1 = 1.22\left(\Pi_{D50}\right)^{-0.764} \qquad a_2 = 0.09\left(\Pi_{D50}\right)^{0.244} \tag{21}$$

From the above, the model of Franzetti et al. (1982) for the prediction of scour depth time evolution can be applied to cylindrical piers in wide channels whose bed is composed of non-ripple forming uniform sand whenever the approach flow velocity is close to the critical velocity of beginning of motion, i.e., for $\Pi_U \approx 1.0$. In this case, the time factor, K_t, reads as follows:

$$K_t = 1 - \exp\left\{-1.22\left(\Pi_{D50}\right)^{-0.764}\left[\Pi_t\right]^{0.09\left(\Pi_{D50}\right)^{0.244}}\right\} \tag{22}$$

Further research is needed to include the effect of Π_U on K_t.

7. Pier Shape and Alignment

The pier shape multiplying factor K_f of Eq. (3) is defined as the ratio between the scour depth at a pier with a given cross-section shape and the scour depth at the standard section-shape pier (usually the circular pier), all the other parameters kept constant. Likewise, the pier alignment or orientation factor, K_θ, is defined as the ratio between the scour depth at a pier aligned with a given angle (angle of attack) towards the flow direction and the scour depth at an equal pier aligned with the flow direction (zero angle of attack), all the other parameters kept unchanged.

The effects of pier shape and alignment have received little attention since Laursen and Toch (1956). Yet, a number of pier shape factors, K_f, were suggested by different researchers. Richardson and Davis (2001) recommended the following expression that describes the values of K_θ obtained by Laursen and Toch (1956) for the particular case of rectangular piers:

$$K_\theta = \left(\cos\theta + \ell_p/D_p \sin\theta\right)^{0.65} \tag{23}$$

where ℓ_p stands for pier length and D_p reads as pier width.

According to Richardson and Davis (2001), K_θ replaces the product $K_f K_\theta$ if the angle of attack, θ, is larger than 5° and $2 < \ell_p/D_p < 16$. This suggestion corroborates Laursen and Toch (1956) who found that the shape effect becomes negligible ($K_f = 1.0$) in the above domain.

Fael et al. (2016) extended the existing experimental evidence on K_f and K_θ by performing fifty five tests with sand S_2 in Flume F_1 bisected longitudinally to render $W = 2.0$ m. They tested rectangular square-nosed, rectangular round-nosed and oblong piers as well as zero-spacing (packed) pile-groups (see Figure 7) for skew angles $\theta = \{0°; 30°; 45°; 60°$ and $90°\}$ and aspect ratios $\ell_p/D_p = \{1.33; 2.0; 4.0\}$, i.e., extending down the lower limit of the experiments by Laursen and Toch (1956).

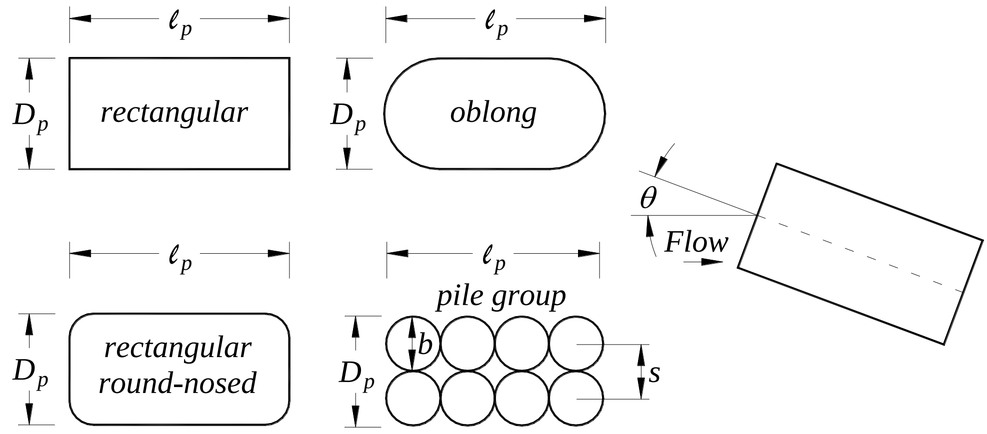

Figure 7. Tested pier shapes

Within their experimental domain, Fael et al. (2016) concluded that *i*) the shape factor is K_f = 1.0, for rectangular round-nosed and oblong cross-section piers, and K_f = 1.2, for rectangular square-nosed and pile-group cross-section piers; *ii*) the shape effect is non-negligible at skewed piers, contrary to what is usually accepted, although the associated shape coefficients remain in the narrow range of 1.0 to 1.2; *iii*) Eq. (23) constitutes a good predictor of K_θ for ℓ_p/D_p = 4.0, whereas it is better described by

$$K_\theta = \begin{cases} 1+\dfrac{4\theta}{1000} & \text{for} \quad \dfrac{\ell_p}{D_p}=1.33 \\ 1+\dfrac{8\theta}{1000} & \text{for} \quad \dfrac{\ell_p}{D_p}=2.00 \end{cases} \tag{24}$$

In short, the scour depth at skewed piers depends on both shape and alignment as soon as $\ell_p/D_p < 4.0$. This is relevant whenever equilibrium scour is to be calculated at piers whose cross-section is defined by small aspect ratios, ℓ_p/D_p.

8. Concluding Remarks

This paper aims at reviewing contributions of the authors towards a more precise prediction of the equilibrium scour depth at single vertical piers. It covers the effects of *i*) flow intensity under *clear-water* flow conditions, *ii*) flow depth and sediment size, *iii*) time evolution, and *iv*) alignment and cross-section pier shape. The paper also deals with the controversial concept and assessment of the equilibrium scour depth. Our contributions do not accrue from a new framework; they are rather a refinement of the model suggested by the school of Auckland, initiated by Raudkivi and boosted by Melville and his students. The enhanced predictive capacity associated to our works arises from precise equilibrium scour values properly estimated from unusually long experiments covering uncommonly high laboratory relative pier sizes.

Our most import contributions express *K*-factors of Eq. (3) and can be summarized as follows:

i) The critical value of flow intensity above which scouring is triggered is not a constant value but rather a function of the relative flow depth as given by Eq(s). (6) or (7).
ii) Flow intensity factor can be predicted by Eq. (9).

iii) The relative sediment size factor is not constant for values of Π_{D50} above 25 – 50, as it is frequently assumed, but it rather decreases in the range $100 < \Pi_{D50} < 500$. It is described by Eq. (19).
iv) The effect of the relative flow depth or flow shallowness can be described by Eq. (18) instead of Eq(s). (15), (16) and (17).
v) The time factor is given by Eq. (22) for flow intensities close to 1.0, reflecting the effect of the relative grain size in the range $60 < \Pi_{D50} < 500$.
vi) The shape effect is non-negligible at comparatively short skewed piers.
vii) Eq. (24) accounts for the alignment effect for small values of the pier cross-section aspect ratio.

It should be noted here that predictions of the improved Auckland's model (Eq. (3)) can be significantly smaller mostly due to the correction associated with the relative sediment size factor.

Another important contribution of the authors to the topic of local scouring is of instrumental nature and refers to the evaluation of the equilibrium scour depth in *clear-water* scour experiments. These should last for at least about one week and the issuing scour values be adjusted by the six parameters polynomial given by Eq. (4) (or a similar equation) to render the value of the scour depth at infinite time.

Nomenclature

a_1 coefficient of the model of Franzetti et al. (1982)[-]
a_2 coefficient of the model of Franzetti et al. (1982)[-]
b_1 experimental cst. [-]
b_2 experimental cst. [-]
b pier diameter in a pile group [m]
C_1 experimental cst. [-]
C_2 experimental cst. [-]
d flow depth [m]
$D_{15.9}$ sand particle sieving diameter for which 15.9% are finer by weight [m]
D_{50} sand particle sieving diameter for which 50% are finer by weight; median grain size of the bed sediment [m]
$D_{84.1}$ sand particle sieving diameter for which 84.1% are finer by weight [m]
D_p, characteristic length of the pier cross-section [m]
d_r depth of bed recess box [m]
d_s scour depth [m]
d_{se} equilibrium scour depth [m]
d_{sm} maximum scour depth [m]
f pier shape [-]
G channel cross-section geometry [-]
g gravitational acceleration [ms^{-2}]
K_d flow shallowness or relative flow depth factor [-]
K_{D50} relative sediment size factor or sediment coarseness factor [-]
K_f pier shape factor [-]
K_U flow intensity factor [-]
K_t time factor [-]
K_ν fluid viscosity factor [-]
K_θ pier alignment factor [-]
$K\sigma_D$ armoring factor [-]
ℓ distance from flume entrance to the recess box [m]
L flume length [m]
ℓ_1 length of bed recess box [m]
ℓ_a abutment length [m]
ℓ_O obstacle length [m]
ℓ_P pier length [m]
$p_1 - p_6$ experimental constants [-]
s specific gravity of the bed sediment [-]
t time [s]
U average approach flow velocity [ms^{-1}]
u_* friction velocity [ms^{-1}]
U_c critical velocity of beginning of sediment motion [ms^{-1}]
W channel width [-]
Π any non-dimensional parameter [-]

$\Pi_d = d/\ell_a$ and $\Pi_d = d/D_p$ relative flow depth or flow shallowness [-]
Π_{ds} non-dimensional scour depth [-]
$\Pi_{dse} = d_{se}/d$ and $\Pi_{dse} = d_{se}/D_p$ non-dimensional equilibrium scour depth [-]
$\Pi_{D50} = D_p/D_{50}$ relative sediment size or coarseness [-]
Π_f pier shape parameter [-]
Π_G channel cross-section parameter [-]
$\Pi_t = Ut/D_p$. non-dimensional time [-]
$\Pi_U = U/U_c$ flow intensity [-]
Π_{Ui} scour inception parameter [-]
Π_W cross-section aspect ratio [-]
$\Pi_\nu = u_* D_{50}/\nu$ particle Reynolds number [-]
Π_θ alignment parameter [-]
ϕ function of [-]
ν kinematic water viscosity [m^2s^{-1}]
θ pier alignment [°]
ρ water density [kgm^{-3}]
σ_D gradation coefficient of the bed sediment [-]

Author Statement

The authors confirm contribution to the paper as follows: all authors have contributed equally to the study conception and design, analysis and interpretation of results, draft manuscript preparation. All authors reviewed the results and approved the final version of the manuscript.

Conflict of Interest

The authors declare no conflict of interest.

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