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H. N. C. Breusers [a], G. Nicollet [b] & H.W. Shen [c]
[a] Delft Hydraulics Laboratory, Delft, The Netherlands
[b] Laboratoire National d'Hydraulique, Chatou, France
[c] Colorado State University, Fort Collins, U.S.A.
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LOCAL SCOUR AROUND CYLINDRICAL PIERS

EROSION LOCALE AUTOUR DES PILES CYLINDRIQUES

by/par

H. N. C. BREUSERS

Delft Hydraulics Laboratory, Delft, The Netherlands

G. NICOLLET

Laboratoire National d'Hydraulique, Chatou, France

H. W. SHEN

Colorado State University, Fort Collins, U.S.A.

(IAHR Task Force on Local Scour around Piers)

Summary A "state of the art" report on the subject of local scour around cylindrical piers is given here. After a description of the scouring process, a critical review of literature on model and field data is presented, and the empirical data are compared with theoretical considerations. The final result is a set of design suggestions together with possibilities for protection against scour.

Sommaire Ce rapport donne le point des connaissances dans le domaine de l'érosion locale en autour des piles cylindriques: analyse du processus d'érosion, étude critique des résultats expérimentaux modèle et nature disponibles dans la littérature, confrontation des résultats aux schémas theoriques. En conclusion est proposée une loi pour la prévision de la profondeur d'affouillement, ainsi que des dispositifs de protection.

1 Introduction

At the request of the IAHR Section on Fluvial Hydraulics a task force was formed to prepare a state of the art report on local scour near piers. The present Report is the result of individual contributions but has been critically reviewed by all members.

The Report is principally restricted to the following conditions:

- cylindrical piers (all shapes),
- non-cohesive granular bed material, and
- one-way current (no tidal influence and waves).

The following aspects are presented:

- the description of the scouring process and an analysis of relevant parameters;
- a description of model and field data;
- a comparison of data with theoretical work and a discussion on the influence of various parameters; and
- the protection against scour and the development of suggestions for design relations.

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It will be clear that, as in many other fields of sediment transport, upto now no entirely satisfactory theoretical and experimental results have been obtained, because the processes involved of water and sediment movement are too complicated and experimental data are incomplete and sometimes conflicting. It has seemed possible however, to give a reasonable description of the scouring process and to make suggestions for design relations on local scour near piers.

2 Description of the flow field around a pier and the scour process

2.1 *Flow field*

The dominant feature of the flow near a pier is the large-scale eddy structure, or the system of vortices which develop about the pier. These vortex systems are the basic mechanism of local scour, which has long been recognized by investigators (see TISON (1940), KEUTNER (1932), POSEY (1949), LAURSEN and TOCH (1956), NEILL (1964), BATA (1960), ROPER, SCHNEIDER and SHEN (1967), Highway Research Board (1970) and MELVILLE (1975)).

It has been described by ROPER, SCHNEIDER and SHEN that, depending on the type of pier and free-stream conditions, the eddy structure can be composed of any, all, or none of three basic systems: the horseshoe-vortex system, the wake-vortex system, and/or the trailing-vortex system. The vortex systems are an integral part of the flow structure and strongly affect the vertical component of the velocity in the neighbourhood of the pier.

The vortex filaments, transverse to the flow in a two-dimensional undisturbed velocity field, are concentrated by the presence of a blunt-nosed pier to form the horseshoe-vortex system. The mechanism by which the concentration is accomplished is the pressure field induced by the pier. If the pressure field is sufficiently strong, it causes a three-dimensional separation of the boundary layers which, in turn, rolls up ahead of the pier to form the horseshoe-vortex system.

A blunt-nosed pier is one which induces a sufficiently large pressure gradient to initiate the process just described. All other piers are referred to as sharp nosed, and it is important to know that, at least conceptually, no vorticity is created at the nose of such piers, although actually some vortex systems always evolve around any bridge piers. The blunt-nosed pier serves as a focusing or concentrating device for the vorticity already present in the undisturbed stream. For a three-dimensional pier, as shown in Fig. 1, the ends of the vortex filaments, composing the horse-shoe-vortex, stretch downstream toward infinity, increasing the rotational velocities in the vortex core in accordance with the kinematic laws of vortex behaviour. Clearly, the geometry of the pier

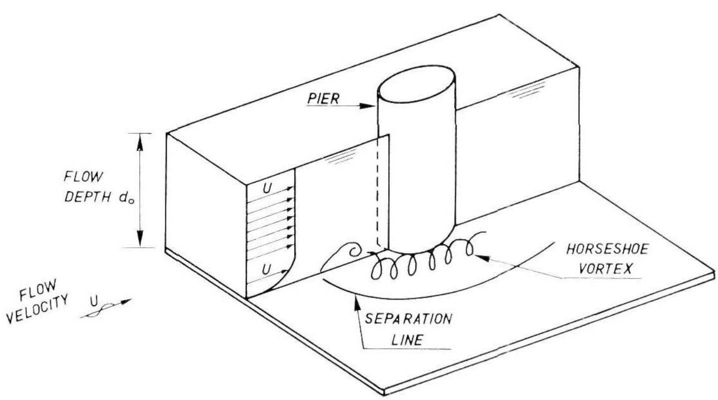
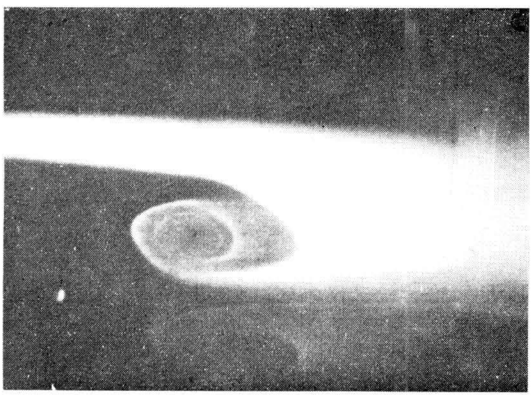


Fig. 1.

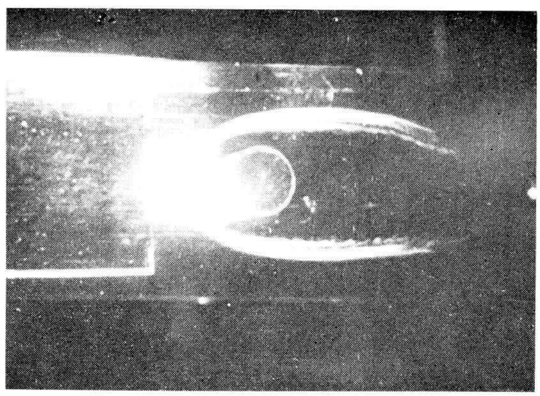
is important in determining the strength of the horseshoe-vortex, although this system is not steady for all flow conditions studied. SCHWIND (1962) noted that for some Reynolds numbers the horseshoe-vortex is shed periodically, while SHEN, SCHNEIDER and KARAKI (1969) and MELVILLE (1975) noticed that the shedding is observable during scour as slugs of sediment being pulsed around the pier.

Some pier shapes, such as wedge or lenticular, may be either blunt-nosed or sharp-nosed, depending on the wedge angle and the angle of attack of the undisturbed flow. SHEN and SCHNEIDER (1970) found in the limited number of experiments they conducted that a wedge-shaped pier with a wedge angle of 30° in a plane bed may be considered to be sharp-nosed. However, an asymmetrical dune moving past this pier can change the local angle of attack so that the pier acts as a blunt-nosed pier. In this case a large scour hole develops at the nose of the pier.

MELVILLE (1975) measured mean flow directions, mean flow magnitude, turbulent flow fluctuations, turbulent power spectra and shear stresses around a circular pier (5.08 cm in diameter) for flat-bed, intermediate and equilibrium scour holes, in a 45.6 cm wide laboratory flume. He found that a strong vertically downward flow developed ahead of the cylinder as the scour hole enlarged. The size and the circulation of the horseshoe-vortex increased rapidly, and the velocity near the bottom of the hole decreased as the scour hole was enlarged. The magnitude of the downflow appeared to be directly associated with the rate of scour. The rate of increase of circulation fell off as the scour hole developed and reached a constant value at the equilibrium stage. Spectra of turbulent velocity fluctuations near the bed of the scour hole indicated a greater energy content in the 1 to 10 Hz range than that of the approached flow and a corresponding lesser energy content at higher frequencies. The combination of temporal mean bed shear and turbulent agitation at the bed tended to decrease as the scour hole enlarged until equilibrium was reached.



(a) upstream elevation view



(b) top view

Fig. 2. Horse shoe-vortex after Taylor (1965).

The vorticity concentrated in the wake-vortex system is generated by the pier itself, contrary to the case of the horseshoe-vortex. The wake-vortex system is formed by the rolling up of the unstable shear layers generated at the surface of the pier, and which are detached from either side of the pier at the separation line. At low Reynolds numbers (3 to 5 $< \overline{R} <$ to 50), these vortices are stable and form a standing system downstream close to the pier. For Reynolds numbers of practical interest, however, the system is unstable, and the vortices are shed alternately from the pier and are convected downstream. The strength of the vortices in the wake system varies greatly according to the pier shape and fluid velocity. A streamlined pier will create a relatively weak wake, but a blunt body produces a very strong one. The regularity of shedding ranges from the very stable VON KÁRMÁN vortex state (80 to 90 $< \bar{R} <$ 150 to 300) to a practically chaotic state in the transcritical range [$3.5 \times 10^6 < \bar{R}$), ROSHKO (1961)].

The wake-vortex system is related to the so-called upflow which has been observed by POSEY (1949), MOORE and MASCH (1963), and others. Large scour holes may develop downstream from piers when the horseshoe-vortex system does not form or is adequately controlled, as demonstrated by the experiments of SHEN and others (1966). The wake-vortex system acts somewhat like a vacuum cleaner in removing the bed material which is then carried downstream by the eddies shedding from the pier.

MELVILLE (1975) found that:

"Under equilibrium conditions for the scour hole, vortex shedding occurs at a value of the Strouhal Number, based upon cylinder diameter and mean approach flow velocity varying from 0.229 tot 0.238, that is, an increase of about 15% from that for the two-dimensional case. The vortex pattern generated is consistent with the occurrence of span-wise cells of constant shedding frequency separated at the discontinuities by longitudinal vortices. The shedding frequency between successive cells decreases with depth. The lower limit for consistent shedding appears to be at about the level of the undisturbed bed. Vortex convection speeds and separation distances downstream from the cylinder decrease with depth. Individual vortices are convected downstream at a speed initially less than that of the approach flow but becoming nearly constant and equal to the approach flow velocity at 8 cylinder diameters downstream. The vortices which are initially shed with their axes vertical are progressively bent by the mean flow as they are convected away from the cylinder. The cast-off vortices aid the erosion process at the cylinder. Each of the concentrated vortices acts with its low pressure centre as a vacuum cleaner. During the initial period of scour activity bursts of sediment transport away from the bed are evident with the generation of each vortex. A ripple is formed on the downstream mound coinciding with the path followed by the cast-off vortices. Based on observations of dye traces introduced into the flow, it is postulated that the arms of the horseshoe-vortex, extending around the circumference of the cylinder, oscillate laterally and vertically at the same frequency as the shedding of wake vortices. Consider the sequence involved in the shedding of two vortices, one from each side of the cylinder, that is, one period of wake-vortex generation: the decreased pressure within an individual cast-off vortex draws up fluid from the horeshoe vortex region, pulling the vortex arm with it. As this first wake-vortex passes downstream, the arm of the horseshoe-vortex recedes back into the scour hole, while the other arm of the vortex is similarly affected by the second wake vortex shed from the other side of the cylinder".

The trailing-vortex system usually occurs only on completely submerged piers and is similar to that which occurs at the tips of finite lifting surfaces in finite wing theory. It is composed of one or more discrete vortices attached to the top of the pier and extending downstream. These vortices form when finite pressure differences exist between two surfaces meeting at a corner, such as at the top of the pier.

ROPER (1965 and 1967) gave a more detailed description of these vortex systems and many of the remarks made in the few preceding paragraphs were his.

HUNG (1968) made detail velocity and pressure distribution measurements near a circular cylinder in an open channel. The width of the channel was 1.2 m, the depth of flow was 0.195 m, the cylinder was 4.3 cm in diameter, and the average flow velocity was 0.39 m/s in the upstream approach section.

The pressure coefficient C_p is defined as follows:

$$C_p = \frac{p - p_y}{\frac{1}{2}\varrho U_y^2} \tag{1}$$

where p is the local measured pressure, p_y is the upstream undisturbed flow static pressure at y, ϱ is the fluid density, U_y is the upstream undisturbed flow velocity at level y, and y is the reference elevation above channel bottom.

The C_p measurements as a function of the elevation and relative cylinder location are shown in Fig. 3.

PETRYK (1969) observed under the same flow conditions as HUNG (1968) that the secondary flow along the front and back of the cylinder is downward, and at the back of the cylinder the pressure is higher near the surface than near the bottom.

The downward secondary flow along the front of the cylinder is attributed to the non-uniform approach velocity. The downward circulation pattern at the back of the cylinder disagrees with previous investigations where a two-dimensional object was placed in a non-uniform flow field.

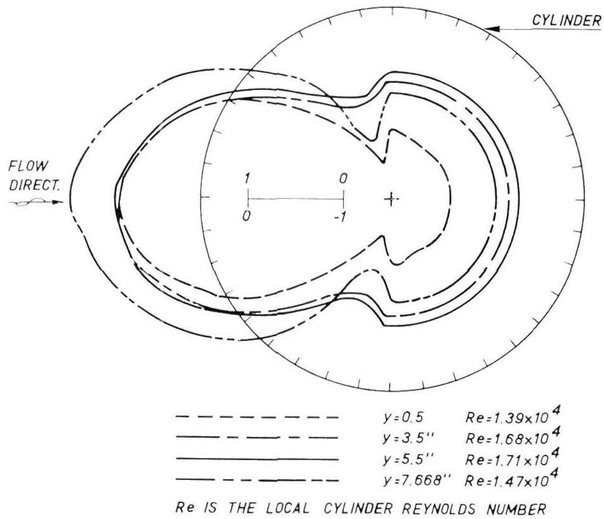


Fig. 3. Pressure coefficient C_p.

In shear flows it has generally been reported that the secondary flow in the near wake region of a cylinder is in the direction of increasing velocity head. This phenomenon has been deduced from the fact that generally as the approach velocity to the cylinder increases, the pressure at the back of the cylinder decreases. It follows that the secondary flow should be in the direction of decreasing pressure, or in the direction of increasing velocity head. All wind tunnel investigations report this circulation pattern [see BAINES (1965) and ROPER (1967)]. DALTON and MASCH (1968) also found that the secondary flow was in the direction of increasing velocity head. They placed a cylinder in a water tunnel with a linear velocity profile, and demonstrated that this secondary flow pattern was applicable to flow without free surface effects.

MOORE and MASCH (1963) and ROPER (1965) reported the same secondary flow pattern downstream of a cylinder in an open channel flow with a non-uniform velocity profile. The downward circulation observed at the back of the cylinder under the flow conditions given in the beginning of this Section have been explained by PETRYK (1969): (i) the free surface effect, and (ii) the vortex shedding pattern at the back of the cylinder. The vortices are shed irregularly and their strength is relatively low. The flow in the separated region circulates quiescently.

The pressure throughout the separated region is expected to be approximately hydrostatic because of the relatively low flow velocities in that region. The re-entrainment velocity is expected to be higher near the surface than near the bottom because of the higher approach velocity near the surface. This higher re-entrainment velocity, impinging on the rear protion of the cylinder, appears to be enough to cause a pressure gradient downward. It follows that, with a downward pressure gradient, the secondary flow is also downward.

At lower velocities the vortex shedding pattern changes and the secondary flow is directed upward. The separated region swings from side to side as the strong vortices are shed alternately from the cylinder, causing separation points on the cylinder and the rear stagnation point to vibrate with the vortex shedding frequency. A very good description of this separation phenomenon is given by MATTINGLY (1962). A sketch showing a strong vortex in the upper half of the separated region is shown in Fig. 4. The upper vortex is shedded and then a strong vortex in the lower half is formed, it is shed, and so on.

Under these latter flow conditions, the higher velocity near the surface forms stronger vortices which are produced immediately behind the cylinder. Therefore it follows that the pressure behind the cylinder will decrease with increasing distance from the floor in a fully-developed channel flow. In this case, the free surface appears to have little effect and the secondary flow is upward.

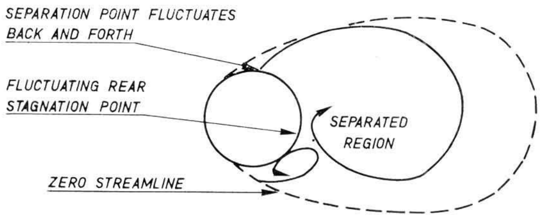


Fig. 4. Separation and oscillation behind cylinder (top view) (after Petryk, 1969).

VAUTIER (1972) measured flow characteristics around two-inch diameter vertical cylindrical piers in two separate flumes (0.45 meter and 2.4 meters wide) with the same approached flow conditions – flow depth 0.15 m, mean flow velocity range from 0.15 to 0.3 meter per second, and a fairly uniform sand of 0.4 mm in size. After a scour hole had reached its maximum size, the entire bed was stabilized, using PVA glue and shellac varnish. His measurements showed that (1) pier wake-vortex shedding frequencies were between 0.75 to 1.09 cycles per second; (2) pier nose-vortex shedding frequencies were in the range of 0.25 to 0.50 cycles per second; and (3) there was no significant difference in both the autocorrelation function and the spectral density for flow velocity measurements at correspondingly the same locations in the two flumes.

HJORTH (1975) studied the flow field around cylinders with circular and square cross sections. The theoretical part consisted of an analytical approach, using potential flow disturbed by a simple shear field, and the experimental part comprised measurements of wall shear stress and pressure field around the cylinders. For a circular cylinder it was found that the maximum average wall shear stress was 12 times that in the undisturbed approach flow. However, this is not in accordance with observations that scour near the pier starts at about 50% of the critical velocity for material transport in the undisturbed part of the bed.

2.2 *Scour process*

The dominant feature of scour process around a blunt-nosed pier is the horseshoe-vortex system. Since the horseshoe-vortex is being stretched the most at point A (about 70 degrees from the main flow direction, see Fig. 1) of a circular pier and near the corners of a square pier, the rotational velocity in the vortex core is the greatest in that neighbourhood. If the scouring potential created by this velocity is strong enough to overcome the particles' resistance to motion, scour will be initiated there. Sediment particles will be dislodged free along the front portion of the pier and carried out of the scour hole either by the horseshoe-vortex system and/or by the wake-vortex system like a vacuum cleaner.

Melville (1975) noted that:

"The horseshoe-vortex is initially small in cross-section and comparatively weak. With the formation of the scour hole, however, the vortex rapidly grows in size and strength as additional fluid attains a downwards component and the strength of the down flow increases. The down flow acts somewhat like a vertical jet in eroding the bed . . . Contours of [measured] bed shear stress, mean flow magnitudes and directions, and turbulent intensities on the bed of the scour hole remain remarkably similar throughout the development of the scour hole after its initial formation. This is a direct consequence of the similarity of shape of the scour hole which is apparent during its growth. As the scour hole enlarges, the circulation associated with the horseshoe-vortex increases, due to its expanding cross-sectional area, but at a decreasing rate, with the rate of increase being controlled by the quantity of fluid supplied to the vortex via the down flow ahead of the cylinder. This in turn is determined by the discharge of the approach flow; or, for a particular flow depth and width, by the magnitude of the velocity of the approach flow. The magnitude of the down flow near the bottom of the scour hole decreases as the depth of the hole increases. Hence the rate of erosion decreases. The armour coat, if present, helps to limit erosion. At a certain stage equilibrium is reached. The combination of the temporal mean bed shear and the turbulent agitation near the bed becomes incapable of removing further bed material from the scour area ahead of the cylinder and in the lower portion of the scour hole. Hence equilibrium is a condition at which the depth of scour ahead of the cylinder is just sufficient so that the magnitude of the vertically downwards flow ahead of the cylinder can no longer dislodge surface grains at the bed. This suggests that the equilibrium depth of scour for a particular bed material and under clear-water scour conditions should be a function of the magnitude of the downwards flow ahead of the cylinder, which in turn is primarily a function of the diameter of the cylinder and the magnitude of the approach flow velocity. Following this reasoning, the flow depth has only an indirect effect on the magnitude of the down flow and hence on the depth of scour. Although equilibrium is obtained for the depth of scour ahead of the cylinder, erosion continues in the downstream dune region. The mound immediately behind the cylinder is progressively flattened and extended downstream by the flow out of the scour hole. This flow is directed up and out of the scour hole parallel to the downstream bed, and curves slightly inwards behind the cylinder. At equilibium the flow near the bed of the scour hole has a greater concentration of energy in the low frequency range than the approach flow".

For a sharp nosed pier, in the absence of a strong horseshoe-vortex system large scour holes may develop downstream from piers by the wake-vortex system, as was demonstrated by the experiments of SHEN, SCHNEIDER and KARAKI (1966).

3 Analyses of scouring parameters

The magnitude which interests the designer for determining the pier foundation depth is the maximum depth to be reached by the scouring process. For this reason, the quantitative study will be limited to the maximum depth d_s reached by the scour hole around the pier after sufficient time has elapsed to reach the equilibrium. d_s is measured below ambient bed level.

Limiting the study to the case of the isolated bridge pier in a river whose flow is assumed to be steady and uniform, there are many parameters which may influence the scouring phenomenon:

Variables characterizing the fluid:
- g acceleration due to gravity,
- ϱ density of fluid, and
- ν kinematic viscosity of fluid.

Variables characterizing the bed material:
- ϱ_s density of the sediment,
- size distribution,
- grain form, and
- cohesion of material.

Variables characterizing the flow:
- d_0 depth of approach flow,
- $\overline{U}$ mean velocity of undisturbed flow, and
- k the roughess of the approach flow.

Variables characterizing the bridge pier:
- its shape,
- its dimensions,
- its surface condition, and
- any protection systems.

The list of parameters is very long and some of them are, moreover, difficult to quantify, such as the particle size distribution, the grain form, or the cohesion of the bed materials.

For this reason, the analysis has been made mainly for the following restrictive conditions:

Bed material: the sediment is non-cohesive and has a uniform size D.
Flow: – channel sufficiently wide so that the bridge pier does not cause a significant contraction;
– flat bed, without dunes or ripples, so that the roughness k depends only on the diameter

of the sediment D and the flow follows some resistance law relating mean velocity to hydraulic gradient I; and
- only ultimate steady-state scour is considered.

Bridge pier: cylindrical, circular, perfectly smooth.

The parameters which remain are:
- for the fluid: ϱ density, ν kinematic viscosity, and g acceleration due to gravity;
- for the bed material: D diameter of sediment and ϱ_s its density;
- for the flow: d_0 the depth and $\overline{U}$ the mean velocity of the undisturbed flow; and
- for the pier: its diameter b.

Therefore the scouring depth d_s depends on eight parameters:

$$d_s = f_1(\varrho, \nu, g, D, \varrho_s, d_0, \overline{U}, b) \tag{2}$$

These parameters may be replaced by the following ones:

$$d_s = f_2(\varrho, \nu, g, D, \Delta, d_0, U_*, b) \tag{3}$$

with $\Delta = (\varrho_s - \varrho)/\varrho$, the relative submerged density and $U_* = (gd_0 I)^{\frac{1}{2}}$
It has been assumed therefore that only the relative density is of importance.

The theorem of Vaschy-Buckingham allows us to write:

$$\underset{1}{\frac{d_s}{b}} = f_3\left(\underset{2}{\frac{U_* D}{\nu}}, \underset{3}{\frac{U_*^2}{\Delta g D}}, \underset{4}{\Delta}, \underset{5}{\frac{d_0}{b}}, \underset{6}{\frac{D}{b}}\right) \tag{4}$$

The justification for the choice of the dimensionless groups is the following:

1 Experiments have clearly demonstrated that it was possible to relate the scour depth to the diameter of the pier. This may be explained physically by the fact that scouring is due to the horseshoe-vortex system whose dimension is a function of the diameter of the pier.

2 and 3 These are classical parameters in the study of bed load.

5 and 6 These ratios relate the size of the pier to that of the flow and of the sediment.

The Equation (4) can be considerably simplified by the following considerations:
- The experimental studies conducted by CHABERT and ENGELDINGER (1956) and by RAMETTE and NICOLLET (1971) have shown that, for a pier with a given diameter b and a sediment of a given diameter D, the limiting scour depth d_s goes through a maximum d_{sm} for flow conditions corresponding to incipient movement in the absence of obstacles ($\tau = \tau_c$). Above τ_c, the scouring depth varies as a function of the inflow of particles and fluctuates owing to progression of bed forms. It is, therefore, very difficult to define the limit depth d_s. It is, however, possible to state that d_s is equal to or slightly lower than d_{sm} (about 10% according to SHEN). This important result has been confirmed by the study of HANCO (1971).
- The influence of the deformation of the free surface on the flow field is negligible if the Froude number of the flow is sufficiently low.

– There is an empirical relation for initiation of motion, relating

$$\frac{U_{*c}D}{\nu} \quad \text{and} \quad \frac{U_{*c}^2}{\Delta g D}$$

– The term Δ is constant by considering only natural sediments (pebbles, gravel or sand, $\Delta \simeq 1.65$).

Under these assumptions Equation (4) may be simplified to:

$$\frac{d_s}{b} = f\left(\frac{U_*}{U_{*c}}, \frac{d_0}{b}, \frac{D}{b}\right) \quad \text{or} \quad \frac{d_{sm}}{b} = f\left(\frac{d_0}{b}, \frac{D}{b}\right) \tag{5}$$

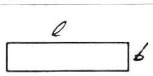
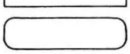
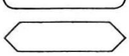
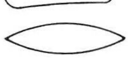
This means that the scour depth d_s will depend mainly on the ratio of mean velocity to mean critical velocity and the relative values of grain size, flow depth and pier diameter.

4 Description of model data

Numerous references on local scour experiments on piers can be found in literature. Few of them, however, are of a general nature with independent and sufficient variation of parameters. In most cases velocities were below or at the critical velocity for initiation of motion. Increasing the pier diameter was often done at constant water depth, thereby decreasing their ratio. Also scouring time will not have been sufficient in many cases to obtain the equilibrium scour depth. Some of the most interesting references are summarised below.

4.1 One of the first references on local scour is the article by DURAND-CLAYE (1873) (see also FLAMANT (1900) p. 281/282). He compared the scour for a square-nosed, a round-nosed and a triangular-nosed rectangular pier. The first one gave a maximum scour depth, whereas the triangular one gave the smallest scour depth.

4.2 TISON (1940 and summary in 1961). He has given much attention to the influence of shape, velocity profile and other parameters. The curvature of the flow at the upstream side of the pier is mentioned as the main cause of secondary vertical currents and local scour.
Most tests were done in a flume with a width of 0.7 m, a discharge of 0.03 m^3/s, a water depth of 0.105 m, a mean velocity of 0.41 m/s, and a medium-size sand, $D = 0.48$ mm.

shape	b (cm)	l (cm)	d_s (cm)
	6	24	11.4
	6	24	8.17
	6	24	7.0
	5.2	21.5	6.2
	6.0	24	5.45
	3.4	24	3.3

In a special test the bed upstream of the pier was roughened with gravel with $D = 1$ to 2 cm, thereby increasing the velocity gradient near the bed. The lenticular shape gave a maximum scour depth of 7.1 cm instead of 5.45 cm, showing the influence of the velocity profile. A gradual increase of the thickness of a lenticular pier from 5.3 cm at the water surface to 8.1 cm near the bed gave a decrease in scour depth from 7.1 to 4.6 cm, whereas a flared pier with a wide base gave very little scour under the same conditions. With a round-nosed circular pier a positive rake decreased the scour, whereas a negative rake increased the scour. The influence of the angle of attack was studied with the lenticular pier (6×24 cm).

$\alpha = 0°$	6°	14.5°
$d_s = 5.45$ cm	6.95 cm	>10.0 cm

Maximum scour depth occurred at the upstream nose for the rectangular pier and at the sides for the streamlined shapes. The length of a rectangular pier was not important at zero angle of attack. Tests with rectangular piers of 2.7×12 cm in a water depth of 6.0 cm at $\bar{v} = 0.32$ m/s showed no mutual influence on maximum scour depth for spacings equal to or larger than 11.6 cm (spacing/width radio $\geqslant 4.3$).

4.3 INGLIS (1948). Tests were performed on a rectangular round-nosed pier with $l = 19.2$ m and $b = 11.3$ m ($l/b = 1.7$) on lengths scales of 1:40, 1:65, 1:105 and 1:210 under zero angle of attack. The results are difficult to interpret because both $\bar{U}$ and d_0 were varied simultaneously.

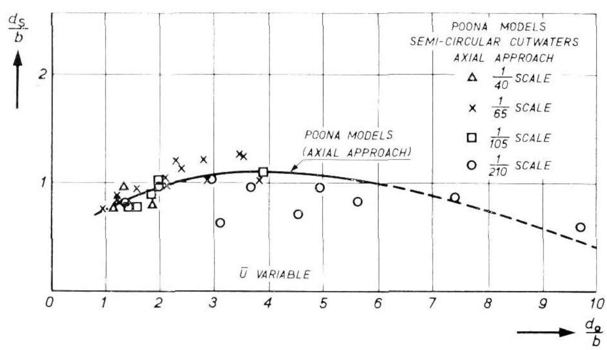


Fig. 5. Scour at bridge piers (Thomas 1962).

Maximum values of d_s/b were in the order of 1.3 (see Fig. 5, THOMAS 1962). Tests were carried out with sand with median grain sizes of 0.3 and 1.3 mm and were run until a zero net transport was obtained (no sand-feeding). From the experimental data the following relation was derived:

$$\frac{d_0 + d_s}{b} = 1.7 \left(\frac{q^{\frac{2}{3}}}{b} \right)^{0.78} \quad \text{(ft-units, coefficient = 2.32 for m-units)} \tag{6}$$

The relation has limited applicability for $b \to 0$ and for increasing $\bar{U}$ at constant d_0, as has been shown by NEILL (1960, 1965). THOMAS (1967) stated that the formula should not be used outside the experimental range: $q^{\frac{2}{3}}/b = 2$ to 10. A major disadvantage of the relation is the combination of undisturbed water depth and scour depth.

Several authors have converted the original relation thus:
BLENCH (1962):

$$\frac{d_r + d_s}{d_r} = 1.8\left(\frac{b}{d_r}\right)^{\frac{1}{4}} \qquad d_r = \text{regime depth} \tag{7}$$

ARUNACHALAM (1965, 1967) with the aid of the Kennedy-relation:

$$\overline{U} = 0.84 d_r^{0.34} \qquad \text{(ft-units)} \tag{8}$$

gave:

$$\frac{d_s}{b} = \frac{d_r}{b}\left[1.95\left(\frac{d_r}{b}\right)^{-\frac{1}{6}} - 1\right] \quad \text{in which } d_r = 0.9q^{\frac{2}{3}} \text{ (ft-units) or } d_r = 1.334q^{\frac{2}{3}} \text{ (m-units)} \tag{9}$$

or:

$$\frac{d_r + d_s}{d_r} = 1.95\left(\frac{b}{d_r}\right)^{\frac{1}{6}} \tag{10}$$

4.4 CHABERT and ENGELDINGER (1956) performed an extensive programme of measurements on the various aspects of local scour around piers. The main variables were velocity, pier diameter (2.5 to 30 cm), water depth (0.1 to 0.35 m), grain size (0.26, 0.52, 1.5 and 3.0 m) and pier shape. Also many devices to reduce the scour were tested. The study on the influence of flow velocity showed that two regimes should be distinguished: for velocities at or below the threshold velocity of movement of the bed material scour depth approaches a limit asymptotically (see Fig. 6) whereas for a larger velocity scour depth fluctuates due to the periodic dumping of material in the scour hole by moving dunes (Fig. 7). Maximum scour depth was obtained at velocities near the threshold velocity, whereas scour started at about half the threshold velocity (see Fig. 8).

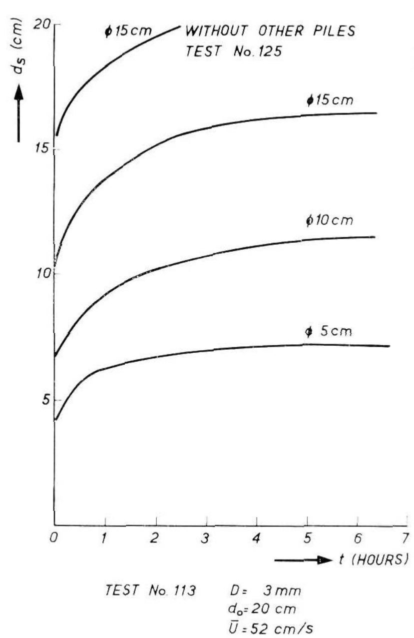


Fig. 6. Scour as a function of time $U < U_c$.

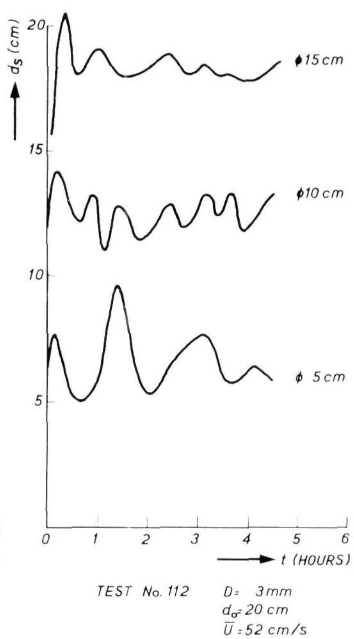


Fig. 7. Scour as a function of time $U > U_c$.

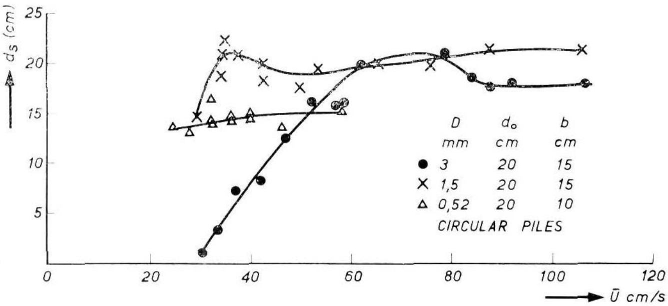


Fig. 8.

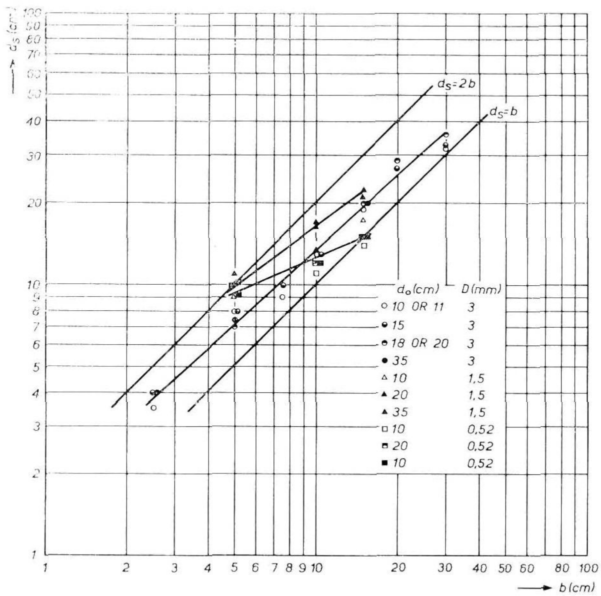


Fig. 9.

The influence of grain size, pier diameter and water depth can be seen from Fig. 9, which shows a small influence of grain size, a negligible influence of water depth for water depth/pier diameter ratios larger than one, and an increase of scour with b^{α} in which $\alpha \leqslant 1$. The latter influence may have been sligthly obscured by the fact that the d_0/b ratio decreased with increasing pier diameter b for these tests.

The influence of pier shape and angle of attack can be seen from Fig. 10 which shows that at a zero angle of attack the scour depth may be minimised by streamlining the pier, but that this ad-

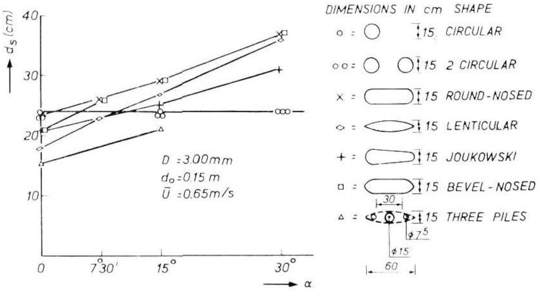


Fig. 10.

vantage disappears for angles of attack above 10°. An exception is formed by the system of 2 circular piers at a spacing of 3 pier diameters, which shows only a minor influence of the angle of attack.

Some care should be given to the interpretation of the results, because three piers were simultaneously tested in the flume at a separation of 6 m. Fig. 6 shows that some influence of the upstream piers was present.

4.5 LAURSEN and TOCH (1956, 1953) investigated the influence of pier shape, angle of attack, water depth, velocity and sediment size. The effects of pier shape and angle of attack were studied at the standard test condition: $b = 0.06$ m, $d_0 = 0.092$ m, $\overline{U} = 0.38$ m/s and $D = 0.58$ mm (see following Table):

angle of attack	l/b	relative scour depth *		
		round-nosed	elliptic	lenticular
0°	1:1	1.00		
	3:2	1.00		
	2:1	1.00	0.91	0.91
	3:1	1.00	0.83	0.76
10°	3:1	1.02	0.98	0.98
20°	3:1	1.13	1.06	1.02
30°	2:1	1.17	1.13	1.13
30°	3:1	1.24	1.24	1.24

* relative to scour for a circular pier with $b = 0.06$ m.

The influence of water depth, mean flow velocity and sediment size was studied with a dumb-bell pier under an angle of attack of 30°. The results are given in Fig. 11, from which it was concluded that there was no systematic influence of grain size and velocity in the range studied. There is an influence of water depth as might be expected in view of the large projected width of the pier (dimensions 0.06×0.4 m, $b = 0.06$ m, $b_{eff} = 0.25$ m). Scour depth varied with time due to the passage of dunes; the values given are averages.

The authors presented also a graphic design relation for rectangular piers under zero angle of attack, which was expressed by NEILL (1964b) as:

$$\frac{d_s}{b} = 1.5\left(\frac{d_0}{b}\right)^{0.3} \tag{11}$$

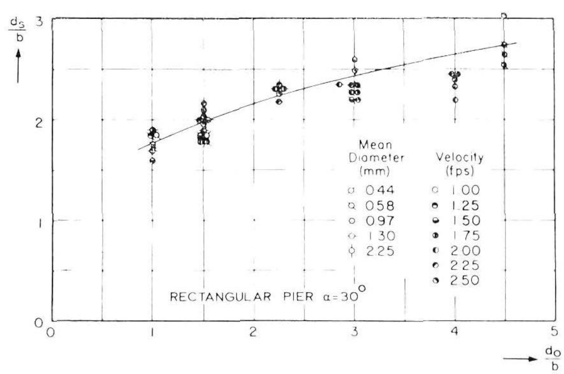


Fig. 11.

4.6 VARZELIOTIS 1960 (quoted from Neill 1964a). Varzeliotis did experiments with 1.7 mm sand with pier shape, angle of attack, velocity and water depth as variables. Standard test conditions were flow depth 0.107 m, mean velocity 0.48 m/s and $b = 0.025$ m. Here are some of his results: Influence of shape:

square-nosed	0.067 m
round-nosed	0.038 m
bevel-nosed	0.041 m
lenticular	0.030 m

The length of a round-nosed pier had no influence for zero angle of attack and length/width ratios of 1 to 20. The influence of pier width was studied with constant depth with the results:

$b =$	0.025	0.05	0.075	0.1	m
$d_s =$	0.038	0.079	0.114	0.127	m

Variation of the angle of attack for a round-nosed pier with $l/b = 6$ gave the following result:

$\alpha =$	0°	7.5°	15°	30°	45°	
$d_s =$	0.035	0.041	0.048	0.083	0.132	m

Water depth and velocity were increased simultaneously during tests with increasing discharge intensity and a round-nosed pier of dimensions 0.05 × 0.15 m under zero angle of attack. Assuming that mean velocity has no great effect in the range used (0.4 to 0.58 m/s, see also Fig. 8), it may be concluded that d_s increases slowly with d_0 upto d_0/b equal 2 to 3:

$\bar{U} =$	0.40	0.45	0.48	0.51	0.53	0.56	0.58	m/s
$d_0 =$	0.073	0.091	0.107	0.122	0.134	0.146	0.159	m
$d_s =$	0.076	0.084	0.088	0.093	0.096	0.094	0.101	m

4.7 TARAPORE (1962) reported some experiments with circular piers ($b = 0.05$ m, $D = 0.15$ and 0.5 mm), from from which it may be concluded that d_s increases with d_0/b upto d_0/b equal to about one and remains constant thereafter ($d_s/b \simeq 1.4$). The development of scour depth with time may be represented with a logarithmic relation. TARAPORE showed that this corresponds to an exponential decrease of velocity near the bed *in* the scour hole, assuming a standard type of bed-load transport relation to be valid in the scour hole.

4.8 LARRAS (1963, 1960) analysed the data given by CHABERT and ENGELDINGER (1956). He concentrated on the maximum scour depth near the threshold velocity of the undisturbed bed material and gave a relation expressing scour depth as a function of pier diameter, with water depth and grain size neglected:

$$d_{sm} = 1.05 b^{0.75} \qquad \text{(m-units)} \tag{12}$$

Tables were given for the influence of pier shape and angle of attack, with the circular pier as a basis. Lenticular shapes gave a relative scour depth of 0.75, elliptical shapes 0.85, rounded piers 1.0 and rectangular ones 1.1 to 1.4. The advantage of the first two shapes disappears for angles of attack of 10° or more.

4.9 NEILL (1964a) gave an excellent review of the work of TISON, INGLIS, LAURSEN and TOCH, CHABERT and ENGELDINGER and VARZELIOTIS. He concluded in favour of relations expressing scour depth as measured from the original bed surface. Suggestions for design were given. For extrapolation to prototype conditions, NEILL suggested a linear increase of scour depth with pier diameter with a relative value of 1.5 to 2.5 for a round-nosed pier. The effects of grain size distribution, local conditions (contraction, embankments) should be investigated in more detail, preferably on the basis of field data.

4.10 NEILL (1964b). In this report detailed attention was given to the influence of the actual river on the scour phenomena, of which the local scour near the pier is only one aspect. A review of literature on model and field data as well as recommendations for design were presented.

4.11 ARUNACHALAM (1965). For the modification of the Inglis-relation see 4.3 The influence of an angle of attack can be taken into account by substituting the projected width of the pier in the relation given.

4.12 NEILL (1965) described some field data on local scour and gave a critical comparison of existing relations for local scour. This gave rise to an extensive discussion by people involved in the development of regime formulas (see Chapter 5).

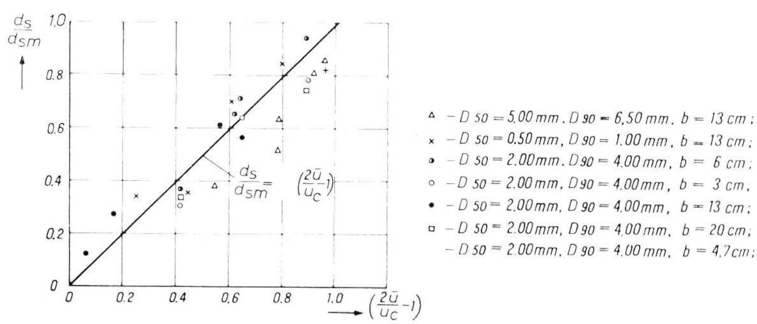


Fig. 12.

4.13 HÎNCU (1965, for a French translation see HANCO (1971)) gave experimental results for circular piers ($b = 3$, 4.7, 6, 13 and 20 cm) in coarse material ($D_{50} = 0.5$, 2 and 5 mm). The scour depth was constant ($d_s = d_{sm}$) above a certain velocity ($\overline{U}_c$). At lower velocities a linear relation with velocity was obtained:

$$\frac{d_s(\overline{U})}{d_{sm}} = \left(\frac{2\overline{U}}{\overline{U}_c} - 1\right) \qquad \text{(see Fig. 12)} \tag{13}$$

The influence of water depth was negligible for $d_0/b > 1$, and d_{sm} increased with grain size. The results were correlated with the expression:

$$\frac{d_{sm}}{b} = 2.42\left(\frac{\overline{U}_c^2}{gb}\right)^{\frac{1}{3}} \qquad \left(\frac{\overline{U}_c^2}{gb} = 0.05 \text{ to } 0.6\right) \tag{14}$$

With a relation given for $\overline{U}_c$:

$$\overline{U}_c = 1.2\sqrt{gD\frac{\varrho_s-\varrho}{\varrho}\left(\frac{d_0}{D}\right)^{0.2}} \approx 1.54D^{0.3}d_0^{0.2}g^{0.5} \tag{15}$$

for natural sands, the relation may be converted into:

$$\frac{d_{sm}}{b} = 3.3\left(\frac{D}{b}\right)^{0.2}\left(\frac{d_0}{b}\right)^{0.13} \tag{16}$$

4.14 SHEN, SCHNEIDER, KARAKI (1966a, b, 1969), ROPER, SCHNEIDER, SHEN (1967), SHEN (1971). In the first reference (1966a) a review of existing literature is given. An analysis of the flow field and the horseshoe-vortex system near a circular pier gave the conclusion that the circulation of the vortex is proportional to $\overline{U}\cdot a$ ($a = b/2$). The next conclusion that the local scour depth will be a function of this factor divided by the kinematic viscosity, being a Reynolds number, is not so obvious.

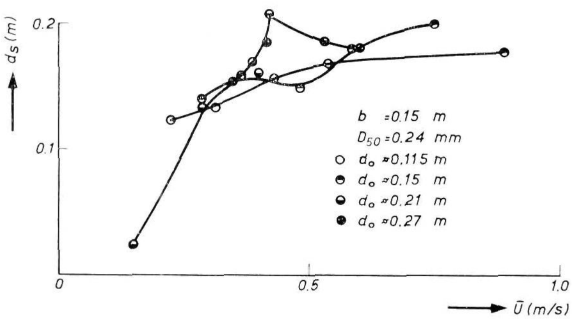


Fig. 13.

Experimental results (21 tests) were given for a circular pier with $b = 0.15$ m in 0.24 mm sand. One test was done with $b = 0.15$ m and 0.46 mm sand and two tests with $D = 0.9$ m in 0.46 mm sand. Results for the 0.24 mm sand are shown in Fig. 13. The scour depths for the 0.9 m pier were 0.67 m and 0.55 m respectively for $d_0 = 0.67$ m, $\overline{U} = 0.66$ m/s and $d_0 = 0.61$ m, $\overline{U} = 0.50$ m/s.

From these data and other results from literature a relation was derived of the form:

$$d_s = 0.000059\ \mathrm{Re}^{0.512}\ \text{(m-units) (SHEN 1966a)} \tag{17}$$

$$d_s = 0.00022\ \ \mathrm{Re}^{0.619}\ \text{(m-units) (SHEN 1969) see Fig. 14} \tag{18}$$

This relation must be considered as an upper envelope because scour depth does *not* increase with $\overline{U}$ for $\overline{U} > \overline{U}_c$ (CHABERT and ENGELDINGER).

For d_{sm} another relation is given:

$$\frac{d_{sm}}{d_0} = 2\{F^2(b/d_0)^3\}^{0.215} \qquad F = \overline{U}/\sqrt{gd_0} \tag{19}$$

or

$$\frac{d_{sm}}{b} = 2F^{0.43}\left(\frac{d_0}{b}\right)^{0.355} \tag{20}$$

which is similar to the design relation given by LAURSEN and TOCH. The latter may be approximated by:

$$\frac{d_{sm}}{b} = 1.35\left(\frac{d_0}{b}\right)^{0.3} \qquad \text{for a circular pier} \tag{21}$$

d_{sm} fluctuates with time for $\overline{U} > \overline{U}_c$. The authors advised to take $d_{sm}+0.5$ dune height for design purposes.

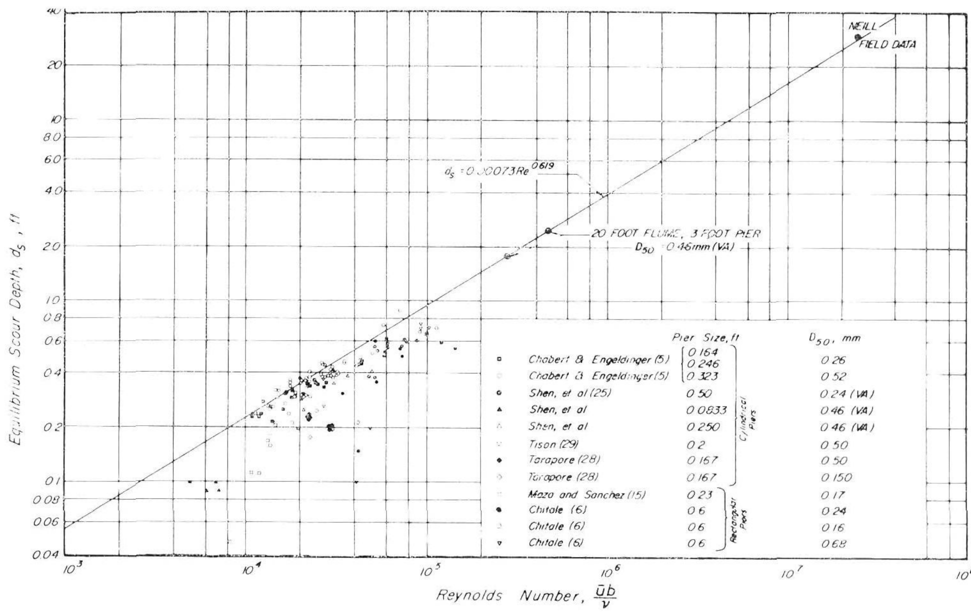


Fig. 14.

The influence of grain size was considered to be negligible for $D < 0.5$ mm (SHEN et al 1966a). The influence of pier shape was studied by SHEN et al (1966b). Adding a sharp nose (top angle 15° or 30°) to a blunt-nosed pier gave a reduction in maximum scour depth. Roughening the upstream face of the pier to decrease the vertical velocities or the strength of the vortex had no effect. SHEN et al (1969) gave a summary of SHEN (1966a, b) and new experiments. The data were also compared with other design relations such as given by LARRAS (1963): $d_s = 1.05\, kb^{0.75}$ (m-units) in which $k = 1.0$ for a circular pier and 1.4 for a rectangular pier, and by BREUSERS (1965): $d_s = 1.4b$ for circular piers. These relations were considered as an upper limit for scour with continuous transport.

In the discussion on SHEN et al (1969), BREUSERS (1970) stressed the empirical knowledge that in general a linear scaling-up of scour depth with pier dimensions may be expected. Comparisons of model and prototype data were given which pointed to this linear relationship (Fig. 15). VEIGA DA CUNHA (1970) stated that the relation given by SHEN et al (1966a) can be valid for clear-water scour only because scour is independent of velocity for velocities above the threshold velocity (Fig. 16), as was shown by CHABERT and ENGELDINGER (1956). The Reynolds number is apparently an unsuitable parameter to characterise the scour depth. According to VEIGA DA CUNHA, also the ratio of water depth to pier diameter should be considered (Fig. 17).

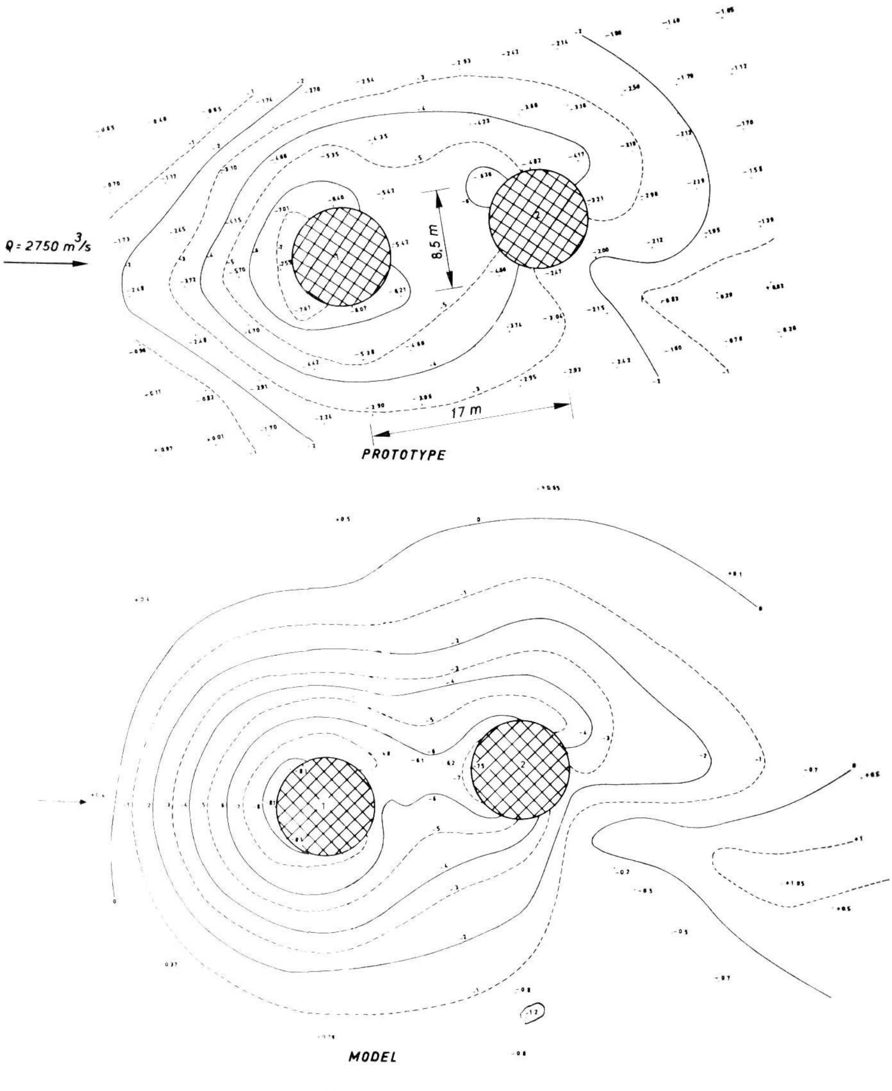


Fig. 15.

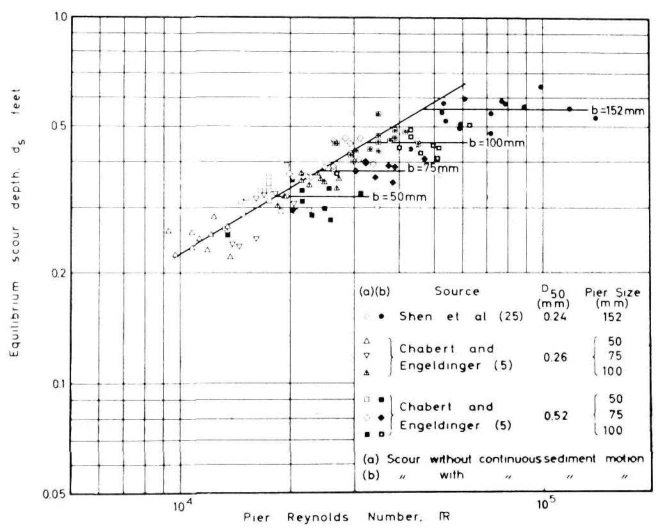


Fig. 16. Scour with and without continuous sediment motion.

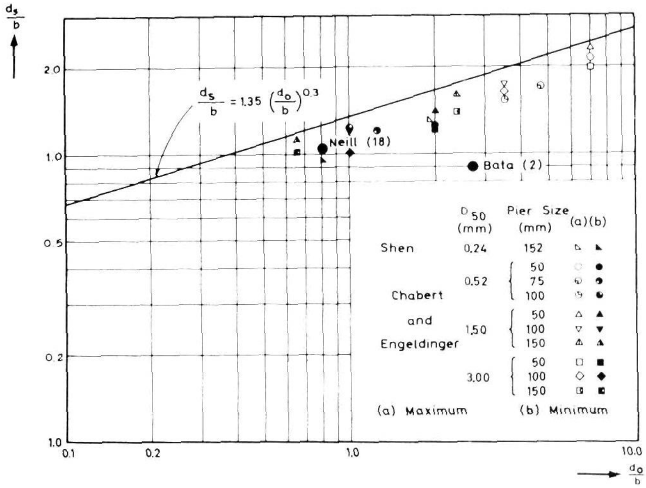


Fig. 17. Influence of d_0/b.

4.15 Maza Alvarez and Sanchez Bribiesca (1966, 1967, 1968) presented a general discussion on the various types of scour in a river and gave results of flume tests on circular, rounded and rectangular piers in sand with diameters of 0.17, 0.56 and 1.3 mm. Some results for a circular pier ∅ 13.3 cm are shown in Fig. 18. Maximum scour depth is in the order of 1.5 times the diameter for a circular cylinder and 2.0 for a rectangular pier under zero angle of attack. The influence of water depth seems insignificant, whereas a linear increase of scour depth with velocity is observed for velocities below the threshold value.

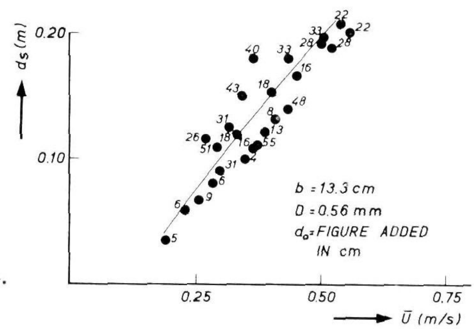


Fig. 18.

4.16 Coleman (1971) analysed data from Shen et al (1969) and results from experiments on circular piers with $b = 0.045$ and 0.076 m in sand with $D = 0.1$ mm under conditions of continuous sediment transport. The correlation obtained was:

$$\frac{d_s}{b} = 1.49 \left(\frac{\bar{U}^2}{g d_0}\right)^{1/10} \tag{22}$$

which can be transformed into $d_s = 1.4b$ (Breusers 1965) with a minor change of coefficients.

4.17 Nicollet (1971a, b) extended the experiments by Chabert and Engeldinger (1956) with respect to the following variables:

– Grain size and gradiation,
– the velocity at which the scouring process starts,
– the influence of bed material density, and
– the influence of aspect ratio for a round-nosed pier.

The first aspect was studied by performing tests with grain sizes of 0.94, 1.93 and 3 mm in the flume used by CHABERT and ENGELDINGER and with gravel (7, 15 and 25 mm) in a large channel (4 m wide) with a water depth of 1.5 m and pier sizes of 0.5 and 1.0 m. The results for d_{sm}, the maximum value of d_s are presented in Fig. 19. Scour depth increases with grain size upto $D = 2$ mm for constant water depth.

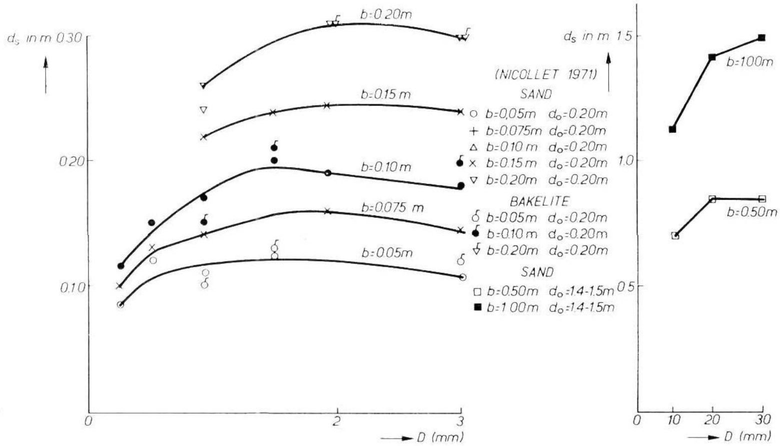


Fig. 19.

Tests with a widely graded material ($D_5 = 0.24$ mm, $D_{50} = 0.7$ mm, $D_{90} = 4$ mm) gave a much lower value of d_s/D (in the order of 0.5 instead of 1.5) than with uniform sand under simular conditions (see also Para. 6.6).

Results for bakelite ($\varrho_s = 1320$ kg/m^3) are also plotted in Fig. 19 from which it may be concluded that ϱ_s is not a significant parameter as far as d_{sm} is concerned. The influence of aspect ratio was studied with $b = 0.1$ m and $l/b = 1$, 2 and 3 (see Fig. 20), which shows that the aspect ratio only has a slight influence on d_{sm}.

Special attention was given to the velocity for initiation of scour. The ratio of this velocity to $\overline{U}_c$, the velocity for initiation of movement of the undisturbed bed material, was 0.42 to 0.53 for a

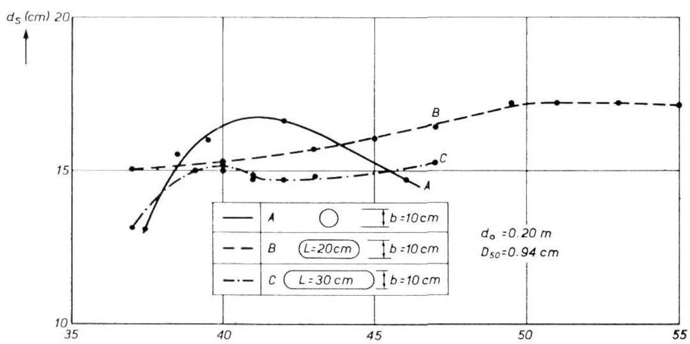


Fig. 20.

circular pier and 0.5 to 0.65 for round-nosed piers. For design of scour protection by rip-rap, the ratio given by HANCO (0.5) is suggested.

4.18 From the tests by DIETZ (1972) on circular piers with various bed materials it may be concluded that scour depth increases with d_0/b upto $d_0/b = 3$ (see Fig. 21). Scour increased linearly with b for $b = 0.043$ to 0.135 m. Several shapes were investigated. When the cylindrical pier was taken as a reference, the following ratios were measured:

shape	round-nosed	elliptical			rectangular		
aspect ratio	–	1:2	1:3	1:5	1:1	1:3	1:5
ratio	0.95	0.9	0.85	0.72	1.4	1.2	1.1

4.19 Systematic tests were performed under geometrically similar conditions by BONASOUNDAS (1973) on circular piers ($b = 0.05$, 0.10, 0.125 and 0.15 m, grain size 0.63, 1.15 and 3.3 mm). The results are summarised in Fig. 22 for $\overline{U}/\overline{U}_c \geqslant 1.0$. The scour depth given is that measured after 2 hours and is not the equilibrium value. The figure shows that d_s increases with d_0/b upto $d_0/b = 2$. The influence of grain size is relatively unimportant for constant D and d_0. Scour depth increases roughly with b, keeping d_0/b and grain size constant.

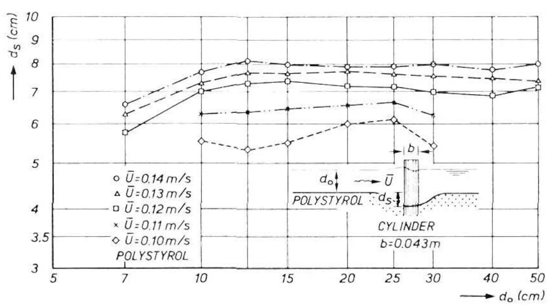


Fig. 21.

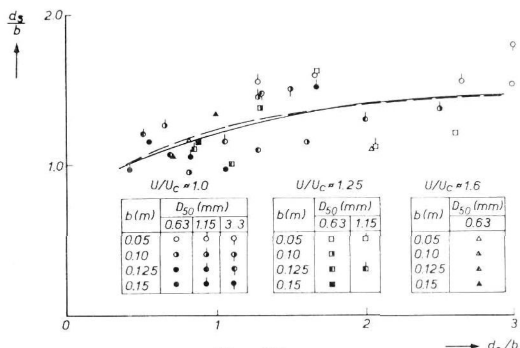


Fig. 22.

4.20 WHITE (1975a, b) presented experimental data for various pier shapes in a coarse sand ($D_{50} = 0.9$ mm, $D_{90} = 3.4$ mm) for high Froude numbers (0.8 to 1.2). The influence of the Froude number was only small in this range. The influence of the pier width decreased with decreasing water depth. The results are difficult to interpret because of interdependence of the variables. The tests were done for scour prediction in steep mountain streams.

4.21 CARSTENS and SHARMA (1975) argued that for large values of b (offshore oil storage tanks) the scour depth will not increase linearly with b for several reasons: the special velocity distribution (Ekman spiral), the large ratio of b/d_0, and the absence of thick layers of sand. They also stated that protections against scour should not increase linearly with b as far as dimensions are concerned.

4.22 NICOLLET (1975) gave results for a test in cohesive material ($D_{50} = 2.2$ µm). Initiation of scour occurred at 60% of the critical velocity without the presence of the pier. The scour depth was in the order of 0.045 to 0.065 m for a circular pier, with $D = 0.05$ m at velocities of 0.7 to 0.8 m/s. The scour hole was more elongated in the downstream direction and more irregular than with sand as bed material.

4.23 Tests with a large diameter pier (up to 0.75 m) were described by TORSETHAUGEN (1975). Polystyrene was used as bed material and special attention was given to the time-history of the scour:

$$d_s/d_{se} = \exp[-(t_0/t)^{0.5}] \tag{23}$$

The correlation obtained for d_{se} was:

$$d_{se}/b = 1.8(\overline{U}/\overline{U}_c - 0.54)d_0/b \qquad d_0/b < 1.0 \tag{24}$$

where most experiments were for $\overline{U}/\overline{U}_c = 0.8$ and $d_0/b = 0.2$ tot 0.65. The scouring depths given are below those found by other investigators for similar conditions, but no explanation is given.

4.24 BASAK et al (1975) performed tests with square piers in coarse sand ($D_{50} = 0.65$ mm, $D_{90} = 1$ mm). Pier width ranged from 0.04 to 0.5 m but the water depths were small (up to 0.14 m). For most of the tests $\overline{U} > \overline{U}_c$, but as both depth and velocity were varied simultaneously, no independent variation of parameters was obtained. The results for square piers were correlated with the equation:

$$d_s = 0.558b^{0.586} \qquad \text{(m-units)} \tag{25}$$

for varying d_0, which can be interpreted only as a decrease of d_s/b with increasing b/d_0. The results are interpreted in a better way by plotting d_s/b versus d_0/b, which shows that for constant d_0/b, d_s increases linearly with b (see Fig. 23). Increasing the length width ratio of rectangular piers gave no increase in scour depth for $l/b = 1$ to 6. For rectangular piers under an angle of attack $0 \leqslant \alpha \leqslant 90$,

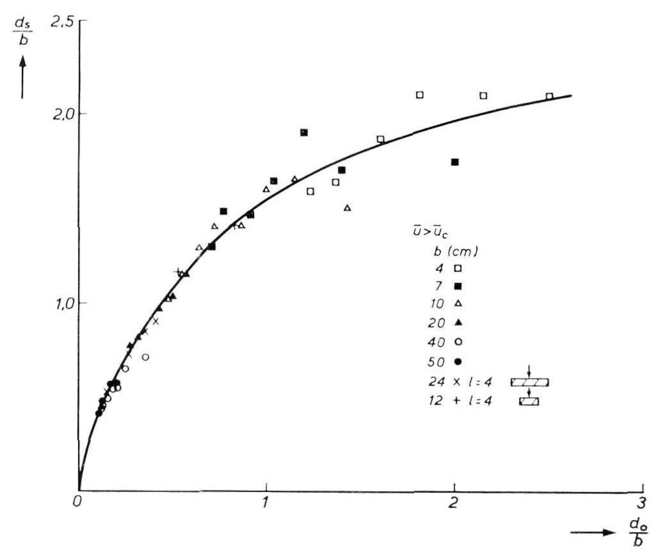


Fig. 23.

the relation given above was also valid if the projected width was taken for b. Interesting results were obtained with rows of square piers. For a row aligned with the flow, maximum scour always occurred at the upstream face of the first pier, so that no influence of spacing and number of piers on d_s was found. For the other piers minimum scour was observed for a centre spacing /width ratio of 4.

For rows perpendicular to the flow, scour depth decreased with increasing spacing upto a spacing/width ratio of 5.

4.25 MELVILLE (1975) made very detailed measurements of the turbulent velocity field around a circular pier and in the scouring hole. For a brief summary see Chapter 2. Also field data are given, (see Section 5.9).

4.26 ETTEMA (1976) studied the influence of median grain size ($D_{50} = 0.55$ to 6.0 mm) and gradation (σ/D_{50} upto 1.6) on local scour near a circular pier with $b = 0.1$ m and $d_0 = 0.6$ m. The experiments were carried out at or below the critical flow velocity. For a discussion of the results, see Sections 6.2 and 6.6. The development of the scour depth with time was described satisfactorily with a log(t)-relation.

5 Description of field data

Field data should give a final proof of the relations established on the basis of small-scale experiments. It is unfortunate, therefore, that the availability of well-documentated field data is limited. Use of the data is also hampered by complicated geometrical shapes, variability of bed material, and inaccuracy of measured quantities. It is, however, possible to present a few cases, mainly taken from NEILL (1964a, b) and MELVILLE (1975).

5.1 INGLIS (1949), Indian Rivers. Observations in the period 1924–1942 on 17 bridges in rivers with discharges from 850 tot 63,000 m^3/s were recorded. Data were presented as a table of total scoured depth, measured from the water surface to the bottom of the scour. This total depth is the sum of general scoured depth, scour due to contraction and local scour due to the piers. The depths were compared with the Lacey regime depth:

$$d_{\text{Lacey}} = 0.473(Q/f)^{\frac{1}{3}} \qquad \text{(m or ft-units)} \tag{26}$$

in which f = silt factor = 1.76 $(D)^{\frac{1}{2}}$, D = median grain size in mm. The average value was 2.09 with a r.m.s. value of 12.9% (see ARUNACHALAM, 1965). Individual values of the ratio varied between 1.73 and 2.62. For design purposes generally a value of 2.0 is used, therefore:

$$d_0 + d_s = 0.95(Q/f)^{\frac{1}{3}} \tag{27}$$

ARUNACHALAM (1965) re-analysed the data and found that the correlation could be improved by leaving f out of the correlation. The result was:

$$d_0 + d_s = (2.09)\,0.473Q^{\frac{1}{3}} \simeq 0.95Q^{\frac{1}{3}} \tag{28}$$

The r.m.s. value reduced to 8.45% with individual ratios varying from 1.72 to 2.59. ARUNACHALAM stated that it may not be concluded that the grain size is not important as the grain size only varied from 0.17 to 0.39 mm (f from 0.71 to 1.10).

5.2 Laursen and Toch (1956), Skunk River. Field observations at the nose of a single rounded pier in the middle of a straight reach of a sand-bed river were obtained with an electric resistivity device. The width of the pier is not exactly defined but was approximately 1.2 m. The maximum value of d_s was 2.0 m at a flow depth of 3.7 m. Neither velocities nor bed material were given. Scale models (1:12 and 1:24) with 0.58 mm sand and a velocity of 0.53 m/s gave a good correspondence with the field observations (see Fig. 24).

5.3 Larras (1960) gave two tables with scoured depths around bridge piers. The first table contained depths observed *after* a flood had passed and must, therefore, as is clearly stated in the paper, have been smaller than the maximum depths during the flood. Pier widths ranged from 0.5 to 6.5 m and scoured depths from 0.6 to 4.3 with d_s/b ratios from 0.4 tot 1.2.

The second table presented *estimated* scoured depths, *including* general scour, based on accidents or incidents with bridge piers. Values of pier widths ranged from 0.7 to 4.2 m, estimated d_s/b values from 1.3 to 2.0.

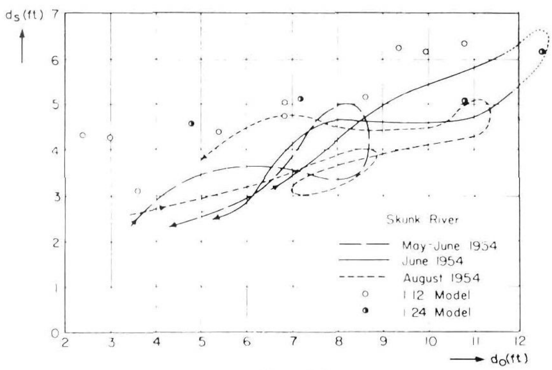


Fig. 24.

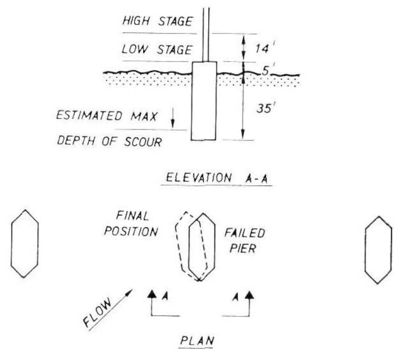


Fig. 25.

5.4 Neill (1964a, b) reported several cases of scour near bridges which in some cases led to bridge failure. The following cases are of interest for the aspect of local scour:
Bridge C. Figure 25 shows the principal features of this case. Slender piers were supported by a concrete caisson from 10.6 m below the bed to 1.5 m above it. The bed was composed of sand and the pier was skewed at least 40° to the current. During a flood one caisson settled and tipped sideways towards the flow, as shown. The maximum scoured depth was estimated at 9.4 m below low-water bed on the exposed face of the caisson. General scour was not known, but using Laursen's design data a scour depth almost equal to the observed depths was predicted.
Bridge F. Scour in a stable gravel-bed river. In this case a total scoured depth ($d_0 + d_s$) of about 15.9 m at the nose of a 6 m wide pier at zero angle of attack was observed. Using Blench's method to determine a "zero flood depth" and a multiplication factor of 2.0 (Inglis) resulted in a depth of 15.3 m.

5.5 Neill 1965, Beaver River. Scour data on two bridges (la Corey and Beaver Crossing) were given. Scouring was mainly due to constriction of the flow; only limited local scour near the piers was observed. The scoured depth was compared with the two-dimensional Lacey depth:

$$d_{2,\text{Lacey}} = 0.9(q^2/f)^{\frac{1}{3}} \qquad \text{(ft-units, 1.34 for metric units)} \tag{29}$$

f was taken equal to 1.0 ($D = 0.5$ mm).

To compute q, the clear water-surface width beneath the bridge at a low-water stage was taken. The ratios of total scoured depth and $d_{2,\text{Lacey}}$ were 1.22 and 1.3 respectively. The article about this gave rise to interesting discussions by several authors involved in the development of regime relations (LACEY, INGLIS, THOMAS, BLENCH).

5.6 ARUNACHALAM (1965). ARUNACHALAM also re-analysed the data given by INGLIS with a relation developed from the Inglis/Poona relation (developed from model tests on the Hardinge bridge). The resulting equation was:

$$d_s/b = d_r/b[1.95(d_r/b)^{-\frac{1}{6}} - 1] \tag{30}$$

in which some influence of pier width is present. For $d_r/b \simeq 1$, this relation reduces to $d_s + d_r \simeq 1.95 d_r$.

Correlation with 11 out the 17 bridges where b was known, resulted in a r.m.s. value of 10% instead of 12.7% with Equation (27). The relation also gave good results for some other cases, but overestimated the scour for the data given by NEILL (1965).

5.7 *Ministry of Railways, India* (1967, 1968, 1972).

5.7.1 1967 Report. At the request of the Indian Railway Board a measuring campaign on railway bridges in all parts of India was started. After a careful selection, only 8 bridges out of 48 were used, because many bridges were protected with stone pitchings or because observations were incomplete. The bridges had spans from 9 to 23 m, the bed material was coarse sand to gravel with silt factors from 1.83 to 2.9 ($D = 1$ to 3 mm). Discharges varied from 35 to 600 m^3/s. The computed LACEY depths varied from 1.35 to 3 m and the observed total scour depths ($d_0 + d_s$) from 2.3 to 5.5 m. The dimensions of the piers were not given (in one case $b = 2.44$ m). For flow parallel to the pier, maximum scour occurs at the nose and an average ratio of ($d_0 + d_s$) to LACEY depth of 1.71 was found (r.m.s. value 19%). For currents inclined to the pier upto 35°, maximum scour occurred on the side of the pier under attack and averaged $1.99 d_{\text{Lacey}}$ (r.m.s. value 15%). Combination of all data gave $(d_0 + d_s)/d_{\text{Lacey}} = 1.93$.

The correlation with Q was better than with the discharge intensity computed from an estimated stream width near the piers. In this case the depth was computed from the "two-dimensional" Lacey relation.

$$\begin{aligned} d_{2,\text{Lacey}} &= 1.34(q^2/f)^{\frac{1}{3}} \qquad \text{(m-units)} \\ &= 0.9\ (q^2/f)^{\frac{1}{3}} \qquad \text{(ft-units)} \end{aligned} \tag{31}$$

5.7.2 1968 Report. Scour around the piers of Ganga Pul at Makameh. This bridge has 14 spans of 123 m, and the width of the wells was 9.75 m. 18 observations are given for various monsoon floods (5,000 to 34,000 m^3/s) in the period 1958–1967. Corresponding LACEY depths were 7.5 to 14.5 m using a silt factor of 1.15 (sediment size not given). Average scoured depth at the nose of the piers gave a ratio $(d_0 + d_s)/d_{\text{Lacey}} = 1.75$, whereas for an inclined attack scour along the sides averaged $2.15 d_{\text{Lacey}}$.

It was observed that during non-monsoon floods the value of $(d_0 + d_s)/d_{\text{Lacey}}$ was *larger*, possibly due to a lower silt content, according to the author. Corresponding values from 13 observations were $2.94 d_{\text{Lacey}}$ at the nose and $2.7 d_{\text{Lacey}}$ for scour along the sides of the piers. Measured scour depths during non-monsoon floods were slightly smaller, however, than during the largest monsoon flood.

5.7.3 1972 Report. Further observations (50) on 4 out of the 8 bridges mentioned in the 1967 Report are given with discharges from 60 to 500 m^3/s, d_{Lacey} 1.5 to 3 m and (d_0+d_s) values from 2.1 to 6.8 m. Maximum scoured depth was generally found along the sides of the piers.

The final correlation for all (93) observations was (see Fig. 26) $(d_0+d_s) = 1.92\ d_{Lacey}$ (r.m.s. value 15%, correlation coefficient 0.79). Correlation with the two-dimentional LACEY depth gave the result (see Fig. 27): $(d_0+d_s) = 1.46 d_{2\ Lacey} = 1.46[1.34(q^2(f)^{\frac{1}{3}}]$ (r.m.s. value 15%, correlation coefficient 0.80; q is the local discharge intensity near the piers). In 36 out of the 50 observations also the depth between the piers was measured and correlated with Q and q. The best correlation was obtained with the Lacey expression (correlation coefficients of 0.74 and 0.76 respectively for the three and two-dimensional cases).

In this series of observations pier width was also measured and a Laursen type of plot (d_s/b vs d_0/b) was given after reduction for effects of shape and angle of attack. In 14 cases d_0 was estimated from Lacey's formula. The result was discouraging. No correlation with Reynolds or Froude number was obtained. The authors also tried correlations of the form

$$d_s+d_0 = kQ^af^b \quad \text{or} \quad kq^af^b$$

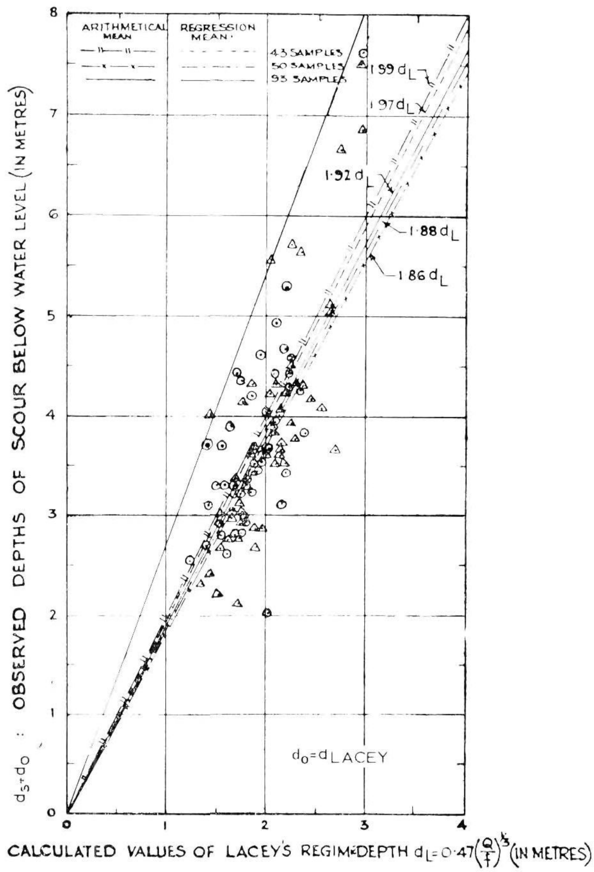


Fig. 26. Scour depth as a function of total discharge.

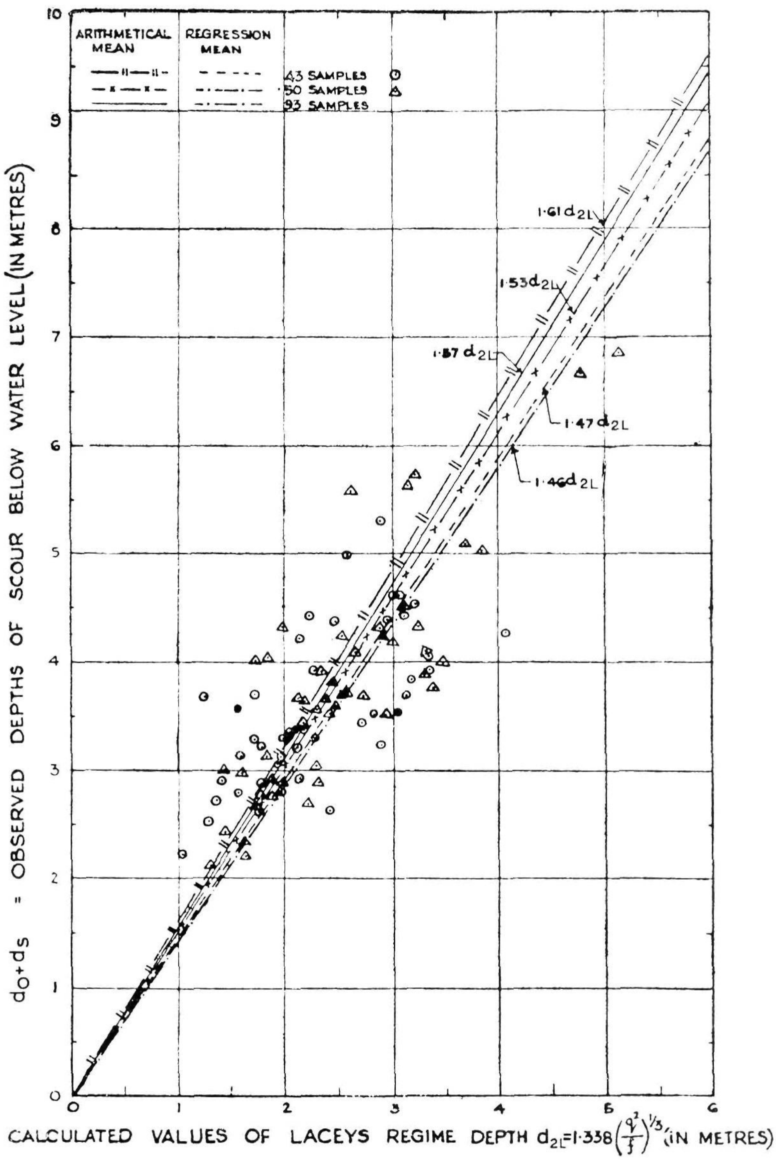


Fig. 27. Scour depth as a function of local discharge intensity.

It appeared that the effect of the silt factor was insignificant, so that the following relations were given:

$$(d_s + d_0) = 0.72Q^{0.33} \tag{32}$$

and

$$(d_s + d_0) = 2.31q^{0.37} \tag{33}$$

(m-units, r.m.s. value 16%, correlation coefficient 0.74 in both cases).

5.8 Breusers (1971) gave a comparison between the local scour measured in nature and in a model for the Onitsha Bridge on the Niger River. The piers consisted of two circular piles with a diameter of 8.5 m and a centre-line separation of 17 m. At a low stage of the river ($\overline{U} = 0.7$ m/s

$d_0 = 9$ m) scour was measured and reproduced in model tests (scale 1:53) under the condition that the scale of mean velocity and critical velocity for all fractions was equal. For the result see Fig. 15.

5.9 MELVILLE (1975) gave a literature review and compared the predictions with field data from New Zealand. The cases in which local scour depth was clearly defined are:

5.9.1 Tuakau Bridge, lower Waikato River. Data: mean depth 3.0 m, mean velocity 0.87 m/s, bed material $D_{15} = 0.38$ mm, $D_{50} = 0.78$ mm, $D_{85} = 2.09$ mm. The pier shape was rectangular with dimensions 8.85 by 2.44 m with chamfered corners and an angle of attack of 10% (estimated LAURSEN and TOCH correction factor 1.3). Maximum scour depth was estimated at 2.75 m but can have been greater in view of the limited number of depth observations.

5.9.2 Big Wanganui River Bridge. Mean water depth 3.8 m, mean velocity 4.27 m/s, bed material $D_{50} = 0.23$ m with $\bar{U}_c = 5.9$ m/s. The scour for a pier measuring 8.5 by 1.63 m at a supposed angle of attack of 10° (LAURSEN-TOCH factor 1.5) was estimated at 4.88 m.

5.9.3 Matawhera Railway Bridge. Water depth 3.0 m, mean velocity 2.25 m/s, bed material $D_{50} = 7$ mm, $\bar{U}_c = 1.12$ m/s. Estimated scour depth 4 m for a pier measuring 6.8 by 1.5 m (see Fig. 28). Maximum angle of attack was 45°, LAURSEN-TOCH correction factor 1.75.

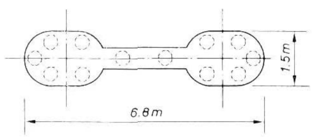


Fig. 28.

5.9.4 Bull's Bridge, Rangitikei River. The river bed was composed of a shingle surface stratum ($D_{50} = 11$ mm) overlying a thin layer of silty clay and a thick layer of fine river sand ($D_{50} = 0.15$ mm). This combination proved extremely dangerous with respect to scour. Once the upper layers were eroded, scour proceeded in the fine sand without an upstream supply. A scoured depth in the order of 12 m developed which caused the complete failure of the bridge. Water depth before scour was 2.7 m and mean velocity 2.8 m/s. Fine sediments underlying coarse sediments or less erodible layers are, therefore, a potential danger.

MELVILLE (1975) concluded from his study at the above four bridge sites that for *Clear Water Scour* Shen's Equation (18) for the equilibrium depth of scour forms an envelope to all the available experimental data and should be used for clear water scour at bridge piers. For *Sediment-transporting Scour* Laursen's relationship appears to be reliable. For flows in which the Froude number is greater than 0.5, the larger of the scour depths given by Laursen and Shen's Equation (19) should be used. MELVILLE apparently misquoted Shen's Equation (19). However, using the corrected equation, it was found that the results were in even better agreement with Melville's field data.

5.10 NORMAN (1975) collected and analyzed depths of scour around piers at seven bridge sites in Alaska, U.S.A., plus four other bridge sites, and found that all scour depths were below the curve $d_s = 3b^{0.8}$. The two largest scour depths were about 7 meters.

6 Comparison of data with dimensional analysis and theoretical work, and influence of parameters

A theory in the sense of a complete model for computing the velocity field and the related local sediment-transport rate in the scouring hole has not yet been developed, mainly because the flow field is too complicated. Some attempts have, indeed, been made (TARAPORE (1962), GRADOWCZYK (1968), ZAGHLOUL (1975) and others), but they can only be considered as explanatory in view of the underlying assumptions.

An analysis of experimental data, with dimensional analysis as some kind of framework, seems, therefore, the only possibility to derive general relationships. The dimensional analysis resulted in a relation of the form:

$$\frac{d_s}{b} = f\left(\frac{\overline{U}}{\overline{U}_c}, \frac{D}{b}, \frac{d_0}{b}\right) \tag{5}$$

neglecting the influence of shape, Froude number, bed-material density and gradation. In this chapter the influence of various parameters and other factors is discussed.

6.1 *Influence of $\overline{U}/\overline{U}_c$*

From the results of various investigations a reasonable picture of the influence of this parameter can be obtained. The following regimes may be distinguished:

a. $\overline{U}/\overline{U}_c \leqslant 0.5$ – no scour (see HANCO 1967, NICOLLET 1971a, b)

b. $0.5 \leqslant \overline{U}/\overline{U}_c \leqslant 1.0$ – clear water scour. In this interval some investigators found that the scour depth increases almost linearly with $\overline{U}$. (See Fig. 8 for $D = 3$ mm (CHABERT and ENGELDINGER 1956), Fig. 17 (HANCO 1967), and Fig. 18 (MAZA 1968)). The function:
$$d_s/d_{sm} = (2\overline{U}/\overline{U}_c - 1) \tag{13}$$
given by HANCO is a good approximation in this interval. The limiting scour depth is approached slowly (Fig. 6).

c. $\overline{U}/\overline{U}_c \geqslant 1.0$ – scour with sediment motion. Here scour depth does not increase further with velocity, apparently because the dynamic equilibrium between transport out of the scouring hole and the supply is not influenced by the magnitude of the transport rate. Sometimes a slight decrease of d_s with $\overline{U}$ is observed. Scour depth fluctuates with time due to the influence of moving bed forms (Fig. 7). The limiting scour depth is defined here as the time-averaged value (NEILL 1964a). The maximum d_{sm} is defined as the maximum with respect to velocity.
For most practical problems an estimate of d_{sm} is sufficient because in a natural river the condition $\overline{U}/\overline{U}_c \geqslant 1.0$ will almost certainly be met during floods. Therefore further discussions will be concentrated on d_{sm}.

6.2 *Influence of D/b*

NICOLLET (1971a, b) reported systematic tests with a large variation in grain size, and the results were interpreted as giving an influence of D/b. If the results are replotted as d_s against D for constant b and d_0, it follows that scour depth is a function of D (Fig. 19). The influence of b is mainly due to a simultaneous variation of d_0/b (d_0 was constant and equal to 0.2 m in most tests). Maximum scour depth as a function of grain size occurred at $D = 2$ mm.

LAURSEN and TOCH (1956) did not observe an influence of D for $D = 0.5$ to 5 mm.

The results by BONASOUNDAS also show some increase of scour depth with increasing grain size in the range 0.6 to 3.3 mm.

The results of ETTEMA (1976) for $\overline{U}/\overline{U}_c \simeq 1.0$ show some increase of d with D (uniformly graded) upto $D \simeq 4$ mm, as can be seen from the following table:

D_{50} (mm)	0.55	0.70	0.85	1.9	4.1	6.0
d_s/b	1.45	1.75	2.0	2.05	2.2	2.1

In conclusion, it may be stated that the influence of grain size is limited for single particle size sediment. The main effect of D/d_0 is the influence on the velocity profile which is a function of this parameter. An increase in velocity gradient (with increasing D/d_0) will increase the strength of the vortex system, as has been shown by TISON (1940).

6.3 *Influence of d_0/b*

This factor gives the most conflicting statements. The following schools may be distinguished:

a. The regime theory which gives the scour depth as a function of the regime water depth. As an example the Inglis relation is taken:

$$d_s \approx d_{\text{Lacey}} = 0.473(Q/f)^{\frac{1}{3}} \tag{27}$$

b. Modification of this type of relation to introduce some effect of d_0/b, for example, ARUNUCHALAM (1965):

$$d_s/d_r = 1.95(b/d_r)^{\frac{1}{6}} - 1 \qquad (d_0 = d_r = \text{regime depth}) \tag{30}$$

or BLENCH:

$$d_s/d_r = 1.8(b/d_r)^{\frac{1}{4}} - 1 \tag{7}$$

c. Relationships expressing d_s/b as a function of b/d_0:
LAURSEN and TOCH (1956), NEILL (1965):

$$d_s/b = 1.5(d_0/b)^{0.3} \qquad \text{for rectangular piers} \tag{11}$$

This relation was also given by VEIGA DA CUNHA (1970):

$$d_s/b = 1.35(d_0/b)^{0.3} \qquad \text{for circular piers} \tag{34}$$

HANCO (1971):

$$d_s/b = 3.3(D/b)^{0.2}(d_0/b)^{0.13} \tag{16}$$

d. Relations giving scour depth as a function of pier diameter:

LARRAS (1963)	$d_s = 1.05b^{0.75}$	(m-units)	(12)
SHEN (1969a)	$d_s \sim b^{0.619}$	(for constant $\overline{U}$)	(18)
BREUSERS (1965)	$d_s = 1.4b$	(circular piers)	(35)
BARAK (1975)	$d_s = 0.558b^{0.586}$	(rectangular piers)	(25)

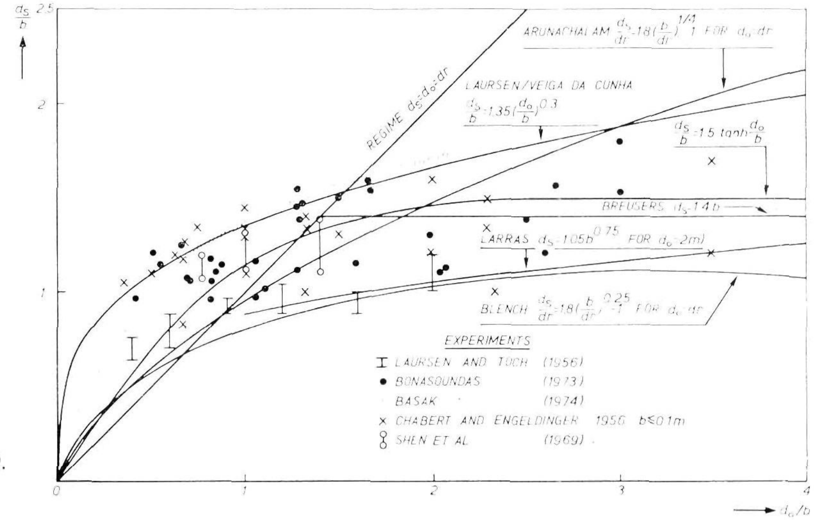


Fig. 29.

The experimental evidence is compared with some of these expressions in Fig. 29. The results of LAURSEN were scaled with the effective width (0.6 ft) instead of the real width (0.2 ft) because a pier under an angle of 30° was used ($k_\alpha = 2.5$), and they were also corrected for pier shape (1.1). The results of BASAK were also reduced with a factor 1.2 because rectangular piers were used in the experiments.

Many authors state that the influence of water depth can be neglected for $d_0/b > 1$ or 2. It is not clear whether this is due to the experimental set-up (not fully developed velocity profiles or inlet conditions). Also the influence of simultaneously testing more piers (CHABERT and ENGELDINGER) is not clear. From the experimental results it may be concluded, however, that for $d_0/b > 3$ the influence of this parameter can be neglected. For smaller values some empirical relation is necessary. The following relation gives a good description for the full range of d_0/b:

$$d_s/b = 1.5 \tanh \frac{d_0}{b} \tag{36}$$

For $d_0/b \to 0$ the relation over-estimates the scour depth if compared to the regime theory, but experiments also point to higher d_s/b values for $d_0/b = 0.4$ to 1.

Relations of the type b and c and BREUSERS (1965) satisfy the basic linear relationship between scour depth and pier dimension for geometric similarity. This linear relation is considered essential for model studies and is stressed by LAURSEN and TOCH (1956) in these words:

"On the basis of all the accumulated evidence, both laboratory and field, it appears that the depth of scour can be regarded as a function of the geometry alone, and that the scour depth can be treated like any other length in the comparison of model and prototype" (under the condition d_0/b is constant).

NEILL (1964a) puts it this way:

"The available evidence suggests that if the pier dimension and depth of flow are scaled up uniformly, then the scour depth may be scaled up by approximately the same factor" (also for constant d_0/b).

These statements are only valid, of course, under the condition that $\overline{U}/\overline{U}_c \geq 1.0$.

6.4 *Shape of the pier*

TISON (1964) showed qualitatively that scour around piers can be affected by the curvature of streamlines. SHEN, SCHNEIDER and KARAKI (1969) classified pier shapes in two categories:
(i) Blunt-nosed pier where a strong horseshoe-vortex system and thus the maximum scour depth occur at the pier nose. The upstream pier shape should have a strong influence on the scour depth, and the length of the pier and downstream pier shape should have a minimum effect if the blunt-nosed pier is aligned with flow.
(ii) Sharp-nosed pier, where the horseshoe-vortex system is very weak and the maximum scour depth occurs near the downstream end. For long piers under an angle of attack the point of maximum scour depths shifts towards the downstream end of the pier (LAURSEN and TOCH, 1956, MAZA ALVAREZ, 1968).

The effect of pier shape on scour depth is significant, and some important studies have been conducted by FLAMANT (1900), REHBOCK (1921), YARNELL and NAGLER (1931), KEUTNER (1932), TISON (1940), ISHIHARA (1942), SCHNEIBLE (1951), LAURSEN and TOCH (1956), CHABERT and ENGELDINGER (1956), ROMITA (1960), KNEZIVIC (1960), VARZELIOTIS (1960), LARRAS (1962), MAZA and SANCHEZ (1964), PAINTAL and GARDE (1965) and SHEN and SCHNEIDER (1970). CHABERT and ENGELDINGER (1956) tested scour around six pier shapes (shown in Fig. 30). See also Fig. 10.

Their results indicate that (i) Group 1 for shapes 1, 2, 4 and 6 have approximately the same maximum scour depths for correspondingly the same flow conditions; (ii) the maximum scour depths for shape 3 were between 33% to 86% of that for the same corresponding flow velocities of Group 1 (with greater scour depths ratios for greater corresponding flow velocities); and (iii) the maximum scour depths for shape 5 varied between 50% to 100% of that for the same corresponding flow velocities for Group 1 (with greater scour depths ratios for greater flow velocities).

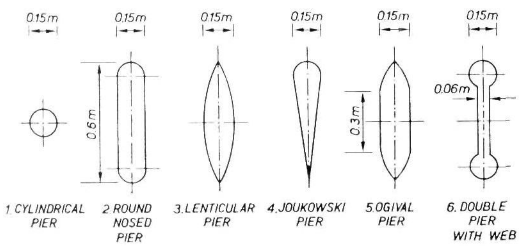


Fig. 30.

NOSE FORM	LENGTH-WIDTH	SHAPE	SHAPE COËFFICIËNT
RECTANGULAR			1.00
SEMICURCULAR			0.90
ELLIPTIC	2:1		0.80
	3:1		0.75
LENTICULAR	2:1		0.80
	3:1		0.70

Fig. 31.

PAINTAL and GARDE (1956) observed that the upstream nose of the pier plays an important part in the phenomenon of scour and that the rear of the pier had no effect. Some of their tests were conducted on piers with upstream triangular noses having different apex angles (15° tot 180°), and their results indicated that maximum scour depth increased with increasing apex angles. They also found that the length of pier has a neglibible effect on maximum scour depths for their pier shapes with a sediment size of 2.5 mm (the pier was aligned with flow direction).

LAURSEN (1960) found that the shape coefficient (defined as the ratio between scour depth of a particular shape to that of the rectangular shape) varies with the shape of pier as shown in Fig. 31.

SHEN and SCHNEIDER (1970) tested nine pier shapes and found that (i) maximum scour depth occurred at the downstream end of a sharp-nosed pier with a wedge nose angle of 30°, (ii) a rectangular pier with roughened upstream face and roughened horizontal apron seemed to have no effect on scour, and (iii) a rectangular pier on a flat footing supported on piers with a vertical lip around the edge of the footing is an effective device for reducing scour (40–50%) if the top of the flat footing with vertical lip is placed at the proper elevation (see also Para. 7).

A pier consisting of two or more circular piers seems to be an attractive one where there is an appreciable angle of attack (CHABERT and ENGLEDINGER 1956). According to DIETZ, who made systematic tests with a system of 2 circular piers, d_{sm} is not influenced by the angle of attack for centre-line spacings larger than 3b.

Taking together all evidence, it is concluded that if the circular or the round-nosed pier is taken as a reference, a reduction in the order of 25% in scour depth can be obtained by streamlining the pier, although his positive effect disappears for angles of attack larger than 10 to 15°. On the other hand, a rectangular pier gives 20 to 40% more scour than the reference pier.

6.5 *Influence of angle of attack*

The influence of an angle of attack has been studied by LAURSEN and TOCH (1956). See Fig. 32 for an empirical relation for k_α, which is the ratio of scour depth at an angle of attack α to that a zero angle of attack. The results of CHABERT and ENGELDINGER (1956) for $l/b = 4$ (see Fig. 10) and VARZELIOTIS (1960) for $l/b = 6$ (both rounded piers) are also given in this figure. It may be concluded that the LAURSEN and TOCH relation gives a good estimate for k_α. Some authors have proposed the use of the projected width in their formulas (ARUNACHALAM (1965) and BARAK (1974), but this gives an overestimate in most cases.

For piers consisting of circular piers with a spacing of more than 3b (DIETZ 1973) and, of course, for a single circular pier, no influence of an angle of attack has to be considered.

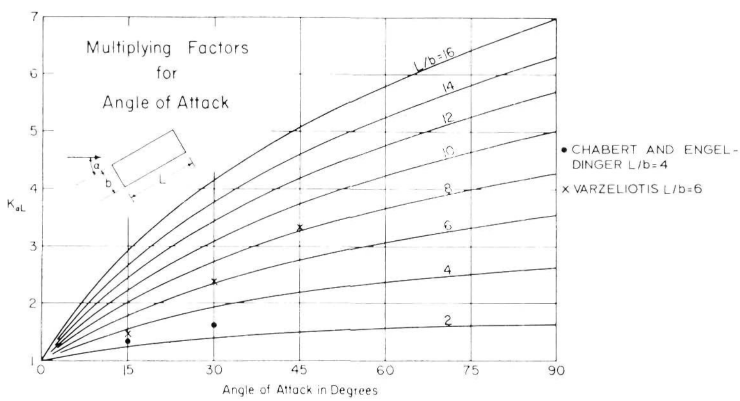


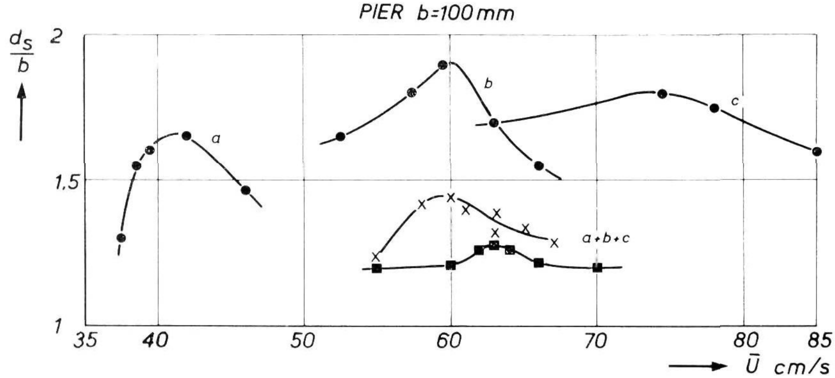
Fig. 32.

6.6 *Influence of sediment particle distribution*

To determine the influence of sediment particle size distribution, a series of tests were conducted by RAMETTE and NICOLLET (1971) with the same hydraulic conditions and the same circular piers as those used for tests with uniform bed materials. The sand for these tests was a 33% mixture of the sands a, b and c previously used, giving a rather straight grading curve between 0.8 and 3.2 mm.

Experimental data on scouring depth limits are presented as a function of flow velocity in Fig. 33, for a pier with a 10 cm diameter. Tests with materials of uniform size carried out for velocities

Fig. 33.



			SAND	SIEVE SIZES mm	mm
SAND OF UNI-FORM TEXTURE	●—●	TEST WITHOUT NOURISHMENT	a	0,77-1.12	0,94
			b	1,62-2.24	1,93
			c	2,77-3,17	2,97
SAND MIXED TEXTURE	x—x	WITHOUT NOURISHMENT	SAND MIXTURE 33% a + 33% b + 33% c		
	■—■	WITH NOURISHMENT			

near incipient movement did not show any significant change in the bed upstream of the pier, and it was therefore not necessary to feed bed load in the channel. For the mixture, the flow velocities corresponded to an intense movement of the fine elements with the formation of dunes in certain cases. Two series of tests were conducted: the first one with additional sediment ensuring the stability of the bed upstream, and the other without any feeding. The scouring depth was measured in all cases with respect to the mean level of the bed upstream.

In Fig. 33, it can be seen that:

- For the mixture, the maximum scouring is obtained for a flow velocity in the vicinity of the velocity giving the maximum scouring for the component *b* alone;
- the maximum scouring depth limit for the mixture is about 25% smaller than those obtained with each of the components taken separately;
- in the case of the mixture, the variation in scouring depth for the flow velocities giving a significant bed load transport is always far below the limit values of d_s obtained with the components of the mixture; and
- the scouring depths are greater in the absence of solid addition upstream, i.e., when the bed load is provided only by the materials in place.

These results tend to demonstrate that the values of d_s around the beginning of bed load transport for a material of uniform gradation cannot be exceeded if a certain particle size gradation is considered and that this conclusion is independent of the flow conditions.

ETTEMA (1976) also studied the influence of bed material gradation on local scour, and found that for σ/D_{50} ratios above 0.3 scour depth decreases dramatically with σ/D_{50} (σ = standard deviation of grain size distribution). These experiments were done at or slightly below the critical velocity, so that no general conclusions can be drawn. The reduction in scour depth is due to armouring effects in the scouring hole.

6.7 *Influence of bed material density*

Several authors have carried out experiments with various bed material densities (NICOLLET (1971a) and DIETZ (1972)) under identical conditions. The conclusion of these experiments is that the density only has an influence on the maximum scouring depth. There is some tendency for scour depth to increase with decreasing bed material density for identical $\overline{U}/\overline{U}_c$, but the use of low material densities in model tests seems to be possible in cases where reproduction of a Froude number is necessary.

6.8 *Influence of flow duration*

The limiting scour depth, on which this Report concentrates, will be reached only during floods of a sufficiently long duration. For very short floods time may be important, because maximum scour will occur on the receding flood. At this stage the river bed has been lowered to its lowest level and with decreasing flow the general sediment transport is already greatly reduced so that clear-water scour conditions prevail. Here the rate of scour development can have an important influence on the maximum scour depth (RAUDKIVI, 1976, personal communication).

7 Scour protection

After making the best choice of the pier shape for minimizing scour, additional arrangements can be considered to prevent the formation of the scour hole, thereby permitting smaller foundation depths. In the final analysis, it is the cost criterion which will allow a decision as to the suitability of these arrangements.

The main scour protection systems which have yielded valid results are a caisson placed around the pier and whose top is under the average level of the river bed, additional structures such as small piles placed above the main pier, and mats of riprap (see Fig. 34).

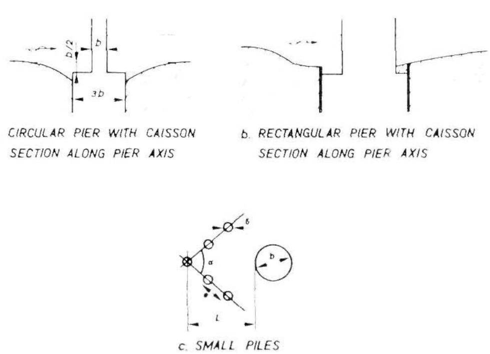


Fig. 34.

7.1 *Foundation caisson*

Engineering requirements may call for the founding of the pier on a caisson of larger dimensions. In this field, CHABERT and ENGELDINGER (1956) investigated a circular pier founded on a circular caisson. Tests in an experimental channel enabled them to conclude that the best system appeared to be a caisson having a diameter three times the diameter of the pier, and a top elevation about half the diameter of the pier below the natural bed. This would reduce scour to only one-third of that reached with the pier alone.

SHEN and SCHNEIDER (1970) investigated a variant of the caisson system in which the caisson is surrounded by a vertical lip (cut-off sheet-pile). The main idea was to contain the horseshoe-vortex inside an enclosure allowing it to escape downstream. These tests were conducted on rectangular piers, and under the optimum conditions of overall dimensions, it was possible to obtain a bed level corresponding to the lip, above and on the sides of the pier, and even to get an accretion downstream.

This system made it possible to reduce scour by half. However, the dimensions of the platform and of the lip in relation to the pier and to the other parameters (flow and sediment) were not examined in a sufficiently systematic manner to enable general laws to be formulated on the valid dimensions in the general case. The lip certainly allows a reduction in the dimensions of the caisson. Reduction of scour depth upto 50% can be obtained by placing a horizontal flat plate with a diameter of at least 3 times the pier diameter some distance (0.3 to $0.4D$) below the undisturbed bed level (see CHABERT and ENGELDINGER 1956, TANAKA 1969, THOMAS 1967).

7.2 *Additional structures placed upstream: piles*

Additional structures are sometimes provided to protect the bridge piers from collision with floating bodies: example of dolphins on navigable waterways.

CHABERT and ENGELDINGER (1956) investigated the installation of small piles above the pier itself, the main purpose being to break the incident current and in this way weaken the vortex generating the erosion.

A large number of parameters are necessary for the definition of such a structure: n piles of diameter δ, spaced e from each other, open according to the angle α and a distance L from the pier (five parameters). No general law could be formulated concerning such a system, but laboratory tests have made it possible to observe scour reductions as high as 50%. Similar reductions were mentioned by LEVI and LUNA (1961) for a vertical strip placed at 2b upstream of an rectangular pier. The optimum width of the strip was equal to the pier width b.

It must be mentioned, however, that all these constructions were not tested under general conditions, so for practical applications special tests are advised.

7.3 *Riprap mats*

The most usual method for remedying erosion is the dumping of stones into the scour hole. Many authors have examined this problem (see list in H. W. SHEN "River Mechanics", 1972) and have made recommendations regarding the choice of materials. Experience has shown that this type of protection is the only one allowing scour to be totally prevented.

CARSTENS (1966) used the fact that the maximum velocity around the cylinder in two-dimensional flow is approximately twice the velocity in undisturbed flow in order to state that the flow velocity giving the initial scour at the base of the pier must be half that corresponding to general bed load movement. The tests of S. HANCU (1971), confirmed by RAMETTE and NICOLLET (1971), have in fact shown that for a given sediment scour begins to appear at the foot of a circular pier at a flow velocity equal to half the critical velocity, irrespective of the diameter of the pier. These results make it possible to determine the weight of riprap capable of preventing any scour. In fact, in a river where the extreme flood velocity is $\overline{U}_{\max}$, it is enough to place boulders whose critical velocity is $\overline{U}_c = 2\overline{U}_{\max}$. Velocities are defined here as mean-on-vertical values.

The diameter of the boulders D as a function of $\overline{U}_c$ can be determined, for example, by means of the ISBASH (1935) formula:

$$\overline{U}_c = 0.85\sqrt{2g\frac{\varrho_s - \varrho}{\varrho}\cdot D} \tag{37}$$

where ϱ_s and ϱ denote the specific weight of the boulders and of the water. For $\varrho_s = 2650$ kg/m^3 this reduces to $\overline{U}_c \simeq 5\sqrt{D}$ (m-units), which covers also the data given by MAZA and SANCHEZ (1964) and NEILL (1973).

The horizontal dimensions of the protection to prevent any scour should be at least 2 times the width of the pier measured from the face of the pier. For the thickness it is suggested to take at least three times the diameter of the stone. A good inverted filter is necessary to prevent leaching of the bed material (POSEY, 1974). The top of the protection should be at some distance below the normal bed level to prevent excessive exposure.

7.4 *Conclusion*

Although valid results can be obtained by means of foundation caissons below the bed level or by means of piles placed upstream of the main pier, no general law as yet enables the dimensions of these protective structures to be determined.

On the other hand, an effective method of preventing any scour consists of providing a riprap protection in which the stone dimensions, stone gradation and the location for protection can be evaluated by estimating the river flow velocity.

8 Practical aspects for design

From the material presented it is concluded that the scour depth may be described by a function of the form:

$$\frac{d_s}{b} = f\left\{\frac{\overline{U}}{\overline{U}_c}, \frac{d_0}{b}, \text{shape, angle of attack}\right\} \tag{38}$$

For practical applications the following relation is suggested:

$$\frac{d_s}{b} = f_1\left(\frac{\overline{U}}{\overline{U}_c}\right)\cdot\left[2.0\tanh\left(\frac{d_0}{b}\right)\right]\cdot f_2(\text{shape})\cdot f_3\left(\alpha, \frac{l}{b}\right) \tag{39}$$

(the constant has been taken as 2.0 instead of 1.5 to be on the safe side).
in which

$$\begin{aligned} f_1\left(\frac{\overline{U}}{\overline{U}_c}\right) &= 0 \quad \text{for} \quad \frac{\overline{U}}{\overline{U}_c} \leqq 0.5 \\ &= \left(2\frac{\overline{U}}{\overline{U}_c} - 1\right) \quad \text{for} \quad 0.5 \leqq \frac{\overline{U}}{\overline{U}_c} \leqq 1.0 \\ &= 1 \quad \text{for} \quad \frac{\overline{U}}{\overline{U}_c} \geqq 1.0 \end{aligned} \tag{40}$$

f_2 (shape) $= 1.0$ for circular and rounded piers
$= 0.75$ for stream-lined shapes
$= 1.3$ for rectangular piers

f_3 (α, l/b) – see Fig. 32

Suggestions for bottom protections to decrease the scour depth have been given in Chapter 7. Generally speaking a flexible protection at some depth below the normal river bed will give the best results. The necessary stone size for a given maximum mean velocity $\overline{U}_{\max}$ can be obtained from:

$$\overline{U}_{\max} = 0.5\overline{U}_c = 0.42\sqrt{2g\frac{\varrho_s - \varrho}{\varrho}\cdot D} \tag{41}$$

A good filter construction is necessary.

Rigid footings should be designed carefully and be placed at some depth below the level of general scour.

If the footing is exposed to the flow, scour depth will increase due to the greater effective width. Attention had to be given to the following special effects:

- *Debris and ice* can increase the effective size of the piers and therefore the local scour.
- *Flash floods* can give a greater scour depth because of unsteady transport conditions. Also non-monsoon floods can give relatively large scour depths (Min. of Railways India, 1968). Laursen and Toch (1956) suggest a 50% increase in design scour depth.
- *Dunes and sand waves* can change the angle of attack and increase the local depth near the pier. Shen, Schneider and Karaki (1969) suggest adding 50% of the dune height to scour depth.
- *A cohesive upper layer* can be disturbed near the pile and cause an increase in scour depth because upstream supply is not present (Melville 1975). The same effect is caused by vegetation in a dry period.
- *General scour* due to degradation, contraction, shifting channels, or bed level variations during floods has to be added to the local scour near the piers (see for example, Neill 1973).
- Intensive *suspension* of sediment in large fine-bed rivers may invalidate the emperical relations.
- *Bad placement of riprap* can provoke scour.

9 Discussion, research needs

Although it has been possible here to present a wealth of experimental data and useful design relations have been developed, it cannot be said that all aspects of local scour near bridge piers have been cleared up. Theoretical developments are limited, and there is not much hope for a rapid success in this complicated interaction of flow field and sediment transport. More experimental data for large diameters piers ($b > 0.5$ m) in the full range of water depths ($d_0/b = 0.5$ to 4) and sufficiently large flow velocity would be helpful to test the relations given.

Also prototype data are needed to improve the relations developed on the basis of model experiments. The data would have to give all relevant information on geometry, bed material and flow field, and are therefore difficult to obtain because maximum scour depth will occur during floods.

It is hoped that the present Report will encourage comprehensive studies on large-scale models. Perhaps certain sites could be selected for more precise data collection, while other secondary field sites could be chosen from which to obtain order-of-magnitude estimates on scour depth.

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