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On the Challenges of Scour Prediction

by

Gijs Hoffmans[1] and Henk Verheij[2]

ABSTRACT

In principle, empirical structure-specific formulas are still used to predict scour. Turbulence and probability distributions for relevant parameters, such as critical shear stress, are taken into account only incidentally. This lecture describes the Dutch approach to scour by presenting a concept for a generally applicable, structure-independent scour prediction formula, as well as the results of turbulence research and a stochastic approach to the transport mechanism. Examples of projects and research carried out are presented to illustrate the possibilities afforded by and also the need for an integrated approach. Finally, some aspects of scour that require further research are defined.

INTRODUCTION

Prior to the design of hydraulic structures it is necessary to establish boundary conditions. Information is required about water levels, flow velocities and soil mechanical aspects, and, in addition, the expected local bed levels under extreme conditions should also be known. The bed level in a river varies in time, due to discharge variations as well as continuing aggregation by sedimentation or degradation by erosion. Moreover, the hydraulic structure may influence the bed level, because the flow field changes and this may initiate the formation of a lower bed-level. Three types of phenomena contribute to the bed level, viz.: 'general scour', 'natural scour' and 'local scour'. General scour is the continuous degradation of the bed level due to human changes in the river basin and the river. In some cases aggradation may occur. Causes of this include the building of reservoirs and changes in land use. Natural scour is the result of natural river processes due to local flow field changes; examples are bend scour, confluence scour, constriction scour and protrusion scour (e.g. scour upstream of a sudden constriction). Local scour is the result of flow field changes due to the presence of a structure.

Today natural and local scour are still predicted by using empirical formulas. In the scour manuals each structure has its own chapter and formula (see for instance the Scour Manual by Hoffmans & Verheij, 1997). Coefficients take into account particular conditions, such as the shape of the bridge piers, or the heads of the spurs. Sometimes the affects of turbulence and stochastics are also taken into account. The same holds for general scour, where empirical sediment transport formulas are most often used, although stochastic predictors are available (van Rijn, 1993). Very recently, transport prediction near the threshold for motion was successful using a stochastic approach taking into account hiding-exposure and hindrance effects (Kleinhans & van Rijn, 2002).

In this lecture results of Dutch research on scour and scour-related items will be considered.

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Firstly, the Dutch approach to scour prediction will be discussed, viz. the development of a generally-applicable (e.g. structure-independent) formula for natural and local scour, including turbulence and probability distributions for all parameters and an integration of all aspects involved. In the next section turbulence will be discussed, followed by a treatment of the transport mechanism. Subsequently, examples of the Dutch approach are presented. Finally, the needs for further research are presented.

DUTCH SCOUR APPROACH

Scour is the result of erosion of bed material, i.e. in conditions in which the bed shear stress is greater than the critical one (see section transport mechanism). In principle, the presence of a hydraulic structure influences the flow field and thus the bed shear stress, which may eventually result in scouring. It is believed that the blockage of a river, resulting in accelerating flow upstream and adjacent to the hydraulic structure and decelerating flow with a higher turbulence level downstream of the structure, determines the shear stress acting on the bed material. In other words: the relative geometrical parameters (relative to the water depth or river width) determine the bed shear stress and its turbulence and not the absolute values of geometrical parameters. This makes it possible to assume that a structure-independent scour relation could be determined which could replace the many formula currently used.

Scour research in the Netherlands is therefore focussed on the following issues ('Dutch scour approach'):

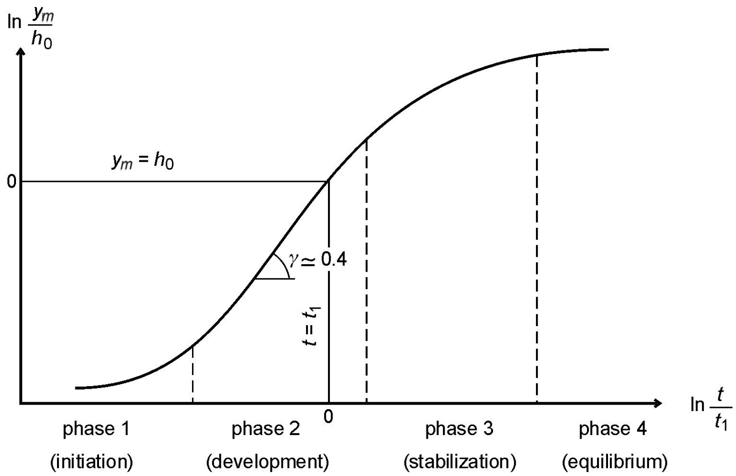
- Development of a generally applicable, structure-independent, scour formula.
- Implementation of turbulence.
- Implementation of stochastics to account for uncertainties in transport mechanism, values of parameters and coefficients.
- Numerical scour prediction by using morphological models, taking into account the mutual influence of flow field, scouring and morphology.

Firstly, the concept of a generally applicable scour formula will be presented. The second and third issues will be treated in subsequent sections. The last item will be illustrated by examples of projects carried out by WL|Delft Hydraulics.

A first attempt to derive a generally applicable scour formula was presented by Hoffmans & Verheij in the Scour Manual (1997). An improved version reads (Fig. 1):

$$y_m = y_{m,e}\ f(t) \quad \text{with} \quad y_{m,e} / L_s = \prod_{i=1}^{n} f_i \quad \text{and} \quad f(t) = 1 - \exp\left(-\lambda\left(t/t_1\right)^{\gamma}\right) \qquad (1)$$

where y_m = time-dependent scour, $y_{m,e}$ = final equilibrium scour, $f(t)$ = function describing the time-dependent scour development, L_s = characteristic length, f_i = coefficients for various influences (for instance: f_1 = flow velocity relative to initiation of motion, f_2 = influence of current and wave loads and their mutual relevance), t = time, t_1 = characteristic time scale in which $y_m = L_s$ and $\lambda = -\ln(1 - L_s/y_{m,e})$ and γ is a constant. The formula accounts for the subsequent scour phases: initial phase, phase of development, stabilisation phase and equilibrium phase.



This concept was presented by Breusers *et al.* (1977) and adapted by various researchers (e.g. HEC-18, 1995; Escarameia & May, 1999; Melville & Coleman, 2000; Sumer *et al.*, 1993; Whitehouse, 1998; Cardoso & Bettess, 1999).

However, most of these relations focus on one dominant process in a particular situation and often the time factor is not included.

Fig. 1 - Scour development process($L_s = h_0$, e.g. downstream of sill)

Bos *et al.* (2002) present an application of the above-mentioned relation for a submerged gravity based structure attacked by the combined action of currents and waves.

The characteristic length L_s is usually related to the flow depth (h_0) or to geometrical properties such as the diameter of a bridge pier or the length (b) of a spur. Breusers *et al.* (1977) had already included an empirical formula to describe the scour process around slender and large structures. We propose:

$$L_s = h_0\left((1-m)^{-2/3} - 1\right) + b\tanh(h_0 / b) \qquad (2)$$

in which $m = b/B$ in which B is the width of the river. The first term represents the constriction of the channel by the presence of an abutment or a bridge pier. For relatively large depths (say $b/h_0 < 0.5$) or slender piers the length scale is equal to b, whereas the length scale equals the flow depth for shallow water conditions ($b/h_0 > 1.5$) or large structures. For intermediate structures $0.5 < b/h_0 < 1.5$ the relation between either L_s and b or L_s and h_0 is not unambiguous. We believe that the influence of the width (l_s) to (L_s) is important in predicting the scour process. Table 1 shows different types of structures that are related to values of the ratios l_s/h_0 and b/h_0. By applying this concept the length scale (L_s) has to be smaller than $y_{m.e}$.

The governing parameters in (1) depend on the flow characteristics, the characteristics of the bed material, or geometrical values. For example f_1 is a function of:

$$f_1 = \alpha_{\ell,1} U_\ell / U_c - 1 \quad \text{with} \quad \alpha_{\ell,1} = 1 + 3r_{0,\ell} \qquad (3)$$

This allows the incorporation of turbulence, for instance via the local turbulence parameter $r_{0,\ell}$ (see next section) and a stochastic approach with respect to U_ℓ (local depth-averaged flow velocity) and U_c (local and depth-averaged critical velocity).

Based on earlier research activities of Breusers *et al.* (1977), the time scale (t_1) can be expressed for both two and three-dimensional flow (Fig. 4):

$$t_1 = \textbf{constant} \cdot L_s^2 \;/ (\alpha_\ell U_\ell - U_c)^{4.3} \quad \text{with} \quad \alpha_\ell = 1.5 + 5r_{0,\ell} \qquad (4)$$

Other definitions are also possible, for instance, the one proposed by Escarameia & May (1999), who define t_1 as the time in which 50% of the final equilibrium scour depth is reached.

The parameter γ determines the erosion velocity during the stage in which the scour develops. Obviously, relevant aspects are the critical bed shear stress (sand results in a faster scour process than clay), the flow complexity (2D or 3D) and the geometry. For two-dimensional flow behind sills $\gamma = 0.4$. In the literature values in the range from 0.2 to 0.8 are presented (Hoffmans & Verheij, 1997), however, these values are not related to a proper length scale L_s.

Recently, research has been started in the Netherlands into breaching in cohesive soils within the framework of establishing dike safety and inundation risk. In this respect critical flow velocities of cohesive soils have also been studied (Verheij, 2002).

Although the basic principles, related to turbulence and a stochastic approach to the initiation of motion are already known, at present research aimed at further develop the generally applicable scour formula is in progress.

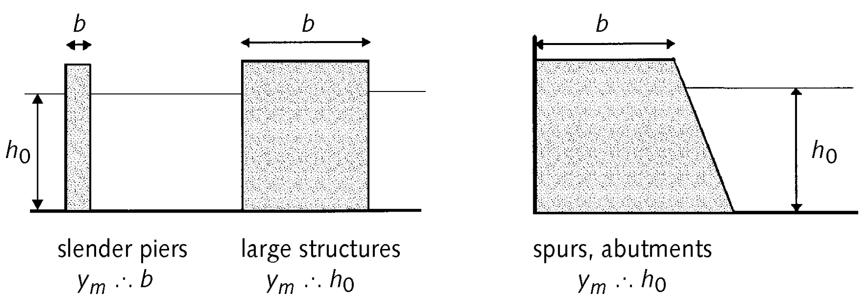


Figure 2 – Value of L_s for different structures

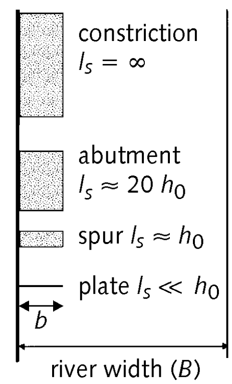


Fig 3 Definition of l_s

TURBULENCE

In a turbulent flow the particles of fluid move erratically producing eddies. It is difficult to follow the paths of individual particles. However, the behaviour of the fluid can be examined by considering average conditions. The (mean) bed shear stress (τ_0) and the standard deviation (σ_0) of the instantaneous bed shear stress are often used to express the loading on bed particles (see also section Transport Mechanism). In this section a definition of a depth-averaged turbulence-intensity (r_0), which has been successfully applied in predicting the scour process downstream of the storm surge barrier in the Eastern Scheldt will be discussed.

The bed shear stress is defined as:

$$\tau_0 = \rho u_*^2 \tag{5}$$

where u_* is the bed shear velocity and ρ is the fluid density. For uniform flow the depth-averaged flow velocity (U_0) is (Chézy equation):

$$U_0 = u_* \frac{C}{\sqrt{g}} \tag{6}$$

where C is the Chézy coefficient and g is the acceleration of gravity. The depth-averaged turbulence intensity (r_0) is defined as:

$$r_0 = \frac{1}{U_0 h_0} \int_0^{h_0} \sigma_u(z)\mathbf{d}z \tag{7}$$

where h_0 is the flow depth, z is the vertical distance and σ_u is standard deviation of the longitudinal flow velocity. Applying the definition of the turbulence energy k (or the turbulent kinetic energy) as function of the vertical:

$$k(z) = \tfrac{1}{2}\left(\sigma_u^2(z) + \sigma_v^2(z) + \sigma_w^2(z)\right) \tag{8}$$

the depth-averaged turbulence intensity can also be written as:

$$r_0 = \frac{\sqrt{\frac{1}{h_0}\int_0^{h_0} k(z)\,\mathbf{d}z}}{U_0} \tag{9}$$

	l_s		
b	$<< h_0$	$\approx h_0$	$>> h_0$
$<< h_0$	slender piers		
$\approx h_0$		wide piers	
$>> h_0$	spur modelled as plate	spur	abutment constriction

Table 1 - Classification of structures

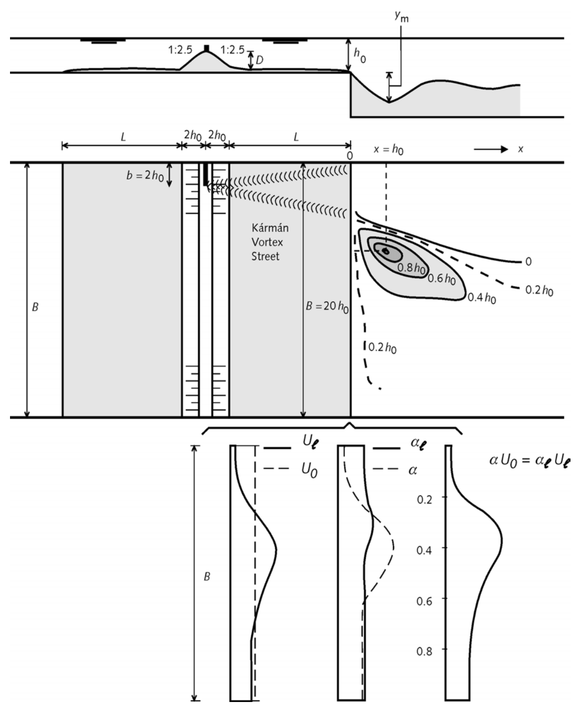


Fig. 4 – Two-dimensional flow field for a horizontal constriction

For uniform flow this can easily be verified when using model relations proposed by Nezu (1977). Following Nezu, the turbulence parameters σ_u, σ_v (= standard deviation in transverse direction) and σ_w (= standard deviation in vertical direction) could be approximated by:

$$\sigma_v(z) = 0.7\ \sigma_u(z) \qquad \sigma\ \ (z) = 0.55\ \sigma_u(z) \qquad \sigma_u(z) = 1.92\ u_* \exp\left(-z/h_0\right) \tag{10}$$

Combining (6), (7) and (10) r_0 is for *uniform flow* (Hoffmans, 1993):

$$r_0 = 1.21 \frac{u_*}{U_0} = 1.21 \frac{\sqrt{g}}{C} \tag{11}$$

For hydraulically smooth conditions (C = 75 $m^{1/2}/s$) r_0 = 0.05 and for hydraulically rough conditions (C = 35 $m^{1/2}/s$) r_0 = 0.10.

Non-uniform flow measurements of Van Mierlo & De Ruiter (1988) showed that the turbulence energy (k_m) in the centre of the mixing layer (with horizontal axis) grows rapidly to a maximum and vanishes where the new wall boundary layer is well developed. The turbulence energy (k_0) (close to the bed) then approaches an equilibrium value, which consists largely of turbulence generated at the bed (Fig. 5 and 6).

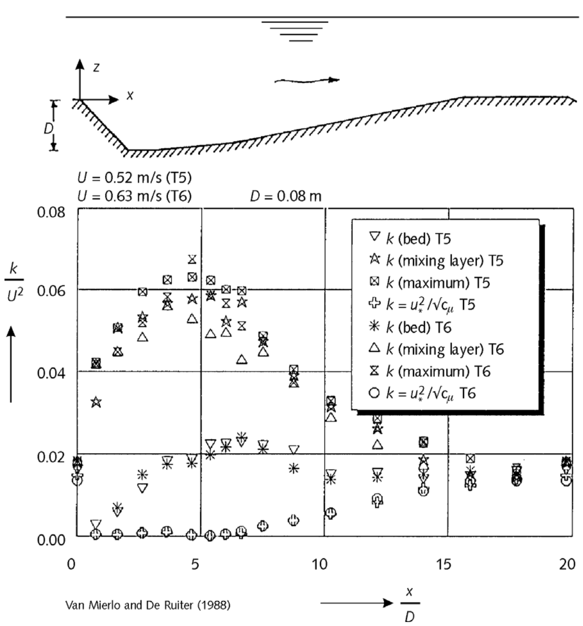


Fig. 5 – Measurements of k as function of x above an artificial dune

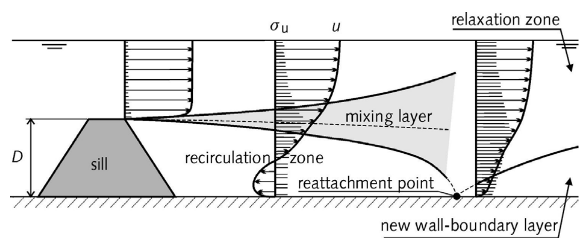


Fig. 6 - Flow downstream of sill

Downstream of the point of reattachment, the turbulence energy (k_η) in the relaxation zone decreases gradually and becomes small compared to the turbulence energy (k_0). Earlier studies of Hoffmans (1992) have shown that in a scour hole the turbulence energy (k_b) near the bed can be represented by a combination of the turbulence energy (k_η) and the turbulence energy (k_0).

$$k_b(x) = \omega k_\eta(x) + k_0(x) \qquad \text{with} \qquad k_0(x) = \frac{u_*^2(x)}{\sqrt{c_\mu}} \tag{12}$$

with c_μ (= 0.09) a coefficient used in k-ε-models and x is the longitudinal coordinate. It should be noted that in the deceleration zone $k_0 \ll \omega k_\eta$ ($\omega \cong 0.3$); this can be ascribed to the small flow gradients close to the bed.

To analyse the decay of the turbulence in the relaxation zone, an analogy with the decay of the turbulence energy and the dissipation in grid turbulence can be used (Launder & Spalding, 1972). When the zone downstream of the point of reattachment is considered and the production and diffusion terms in the transport equations of the turbulence energy and the dissipation are neglected, k_η can be given by (Booij, 1989):

$$k_\eta(x) = k_m \left(\frac{x - x_R}{\lambda_r} + 1 \right)^{\alpha_k} \tag{13}$$

where x_R ($\cong 6D$) is the x-coordinate where the flow reattaches the bed, D is the height of the sill, λ_r ($\cong$ ½ c_λ h_0/β_m) is a relaxation length, c_λ ($\cong$ 1.2) is a relaxation coefficient, β_m ($\cong$ 0.09) is the angle of the mixing layer and α_k ($\cong$ -1.08) is a coefficient that is directly related to the turbulence coefficients used in k-ε-models. The hypothesis of self-preservation (Townsend, 1976) requires constant turbulence energy in the mixing layer up to the point where the boundaries have reached the surface and the bed. An appropriate value is:

$$k_m = C_k U^2 \tag{14}$$

in which C_k ($\cong$ 0.045) is a coefficient and U is the depth-averaged flow velocity above the sill. In analogy to (13) the turbulence energy averaged over the depth, from which r_0 can be determined downstream of a sill, can be given by:

$$\frac{1}{h_0} \int_0^{h_0} k(x,z)\mathbf{d}z = \beta_k k_\eta(x) + c_0 u_*^2(x) \tag{15}$$

where β_k ($\cong$ 0.5) and c_0 ($\cong$ 1.45) are constants. If the geometry of the tests consists of a horizontal bed where the flow is sub-critical above a sill, the following relation for r_0 can be deduced by combining (9), (12), (13), (14) and (15), (Hoffmans, 1993):

$$r_0 = \sqrt{0.0225 \left(1 - \frac{D}{h_0}\right)^{-2} \left(\frac{L - 6D}{6.67 h_0} + 1\right)^{-1.08} + 1.45 \frac{\sqrt{g}}{C}} \quad \text{for} \quad L > 6D \tag{16}$$

For reasons of safety, the length L of the bed protection will always extend beyond the point of reattachment. More than 250 experiments were used to calibrate and verify (16). In these laboratory experiments both the hydraulic conditions and the geometrical parameters were varied.

TRANSPORT MECHANISM

Particle transport or scouring occurs when there is no balance between loading (instantaneous bed shear stress) and strength (instantaneous critical bed shear stress). When the load is less than some critical value, the bed material remains motionless. The bed can then be considered as fully stable; but when the load over the bed attains or exceeds its critical value, particle motion begins. The beginning of motion is difficult to define and this can be ascribed to phenomena that are random in time and space. Usually particle transport is modelled by formulas that include the difference either in velocity (or shear stress) or in turbulence or both. Based on the concepts of Shields, Grass (1970) and Raudkivi (1998) a stability formula is discussed for describing the initiation of motion.

Shields published his experimental results for the initiation of movement of uniform granular material on a flat bed, later known as the Shields-criterion although Rouse proposed the well-known curve (Fig. 7).

Several researchers investigated the influence of turbulence on bed load. As given by Kalinske and Einstein, the instantaneous velocity varies according to a Gaussian distribution. Grass

(1970) extended these ideas. The weakness of these classical stochastic models is that they do not incorporate modern turbulence knowledge. For example, measurements show that for uniform flow the influence of sweeps, which are directed towards the bed, and ejections, which are moving away from the bed, is not included in the Gaussian distribution (Fig. 8). Sweeps and ejections contribute most to the turbulent shear stress.

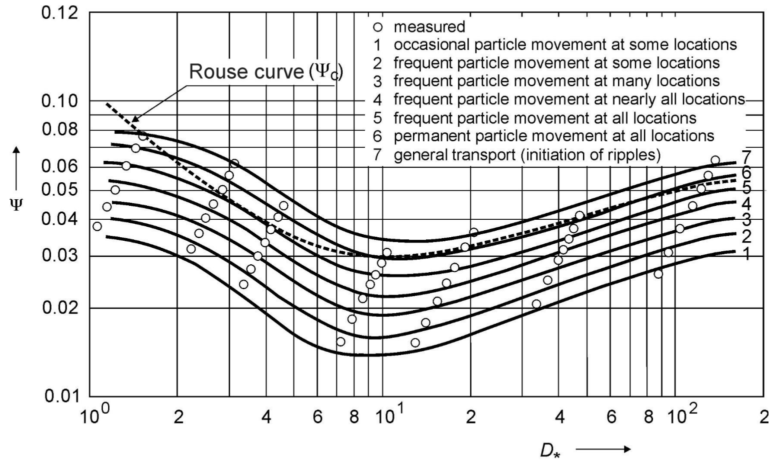


Figure 7 – Shields Diagram ($D_*=d_{50}(\Delta g/\nu^2)^{1/3}$

In the Shields diagram, the influence of fluctuating shear stresses on bed particles is not directly specified. In the sixties WL|Delft Hydraulics studied the initiation of movement of bed material in detail and distinguished 7 qualitative criteria. These introduced criteria all lie in the broad belt as originally given by Shields thus confirming the earlier research activities of Shields

However, there is more to the difference than the enhanced skew in the instantaneous bed shear stress. Under non-uniform flow conditions, there is no clear relation between the instantaneous sediment transport and the instantaneous bed shear stress. Near-bed measurements of turbulent correlations (Reynolds stresses) are estimates of momentum flux, but are only related to the force acting on the bed when it is averaged over a long period of time. This being so, some assumptions are made in order to model important design parameters for both turbulence and scouring.

When dealing with the concept of Grass, the exact shape of the distribution of loading and strength is irrelevant because a characteristic bed shear stress can be defined, this being a time-averaged value and a fluctuating term that originates from the turbulence near the bed. The characteristic value is a value that is higher or lower than the time-averaged value. Usually characteristic values are expressed as a mean value and a fraction or a multiple of the standard deviation. In fact, the problem of bed stability will now be expressed in terms of the magnitude of this fluctuation. In addition to the random nature of the load, another random variable in the process of initial instability is determined by the strength of the particles close to the bed.

To make an adaptation to non-uniform flow it is useful to analyse the influence of the turbulence in the vicinity of the bed on uniform flow. For this exercise the concept of Grass (1970) can be applied, this being based on statistical assumptions for both the loading and strength parameters (Fig. 9). The characteristic bed shear stress ($\tau_{0,k}$) and the characteristic strength, which is the critical bed shear stress ($\tau_{c,k}$) can be respectively written as:

$$\tau_{0,k} = \tau_0 + \gamma\sigma_0 \qquad \tau_{c,k} = \tau_G - \gamma\sigma_c \tag{17}$$

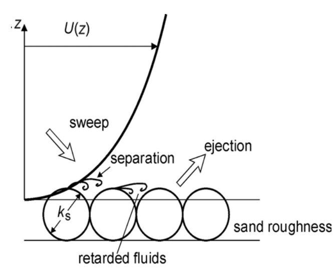
where γ is determined by an allowable transport of the bed material, σ_c is the standard deviation of the instantaneous critical bed shear stress and τ_G is the time-averaged critical bed shear stress according to Grass. A specific transport will occur if $\tau_{0,k} = \tau_{c,k}$; this will be elucidated later.

If the characteristic loading near the bed is equal to the characteristic strength (thus $\tau_{0,k} = \tau_{c,k}$)

and if $\sigma_c = \alpha_c \tau_G$ with $\tau_G = \Psi_{c,G} \Delta \rho g d_{50}$ (analogous to the Shields concept) and assuming $\gamma_{strength} = \gamma_{load} = \gamma$, a general relation for the upper layer of bed protection follows:

$$\Delta d_{50} = \frac{\tau_0 + \gamma \sigma_0}{\Psi_{c,G} \rho g (1 - \alpha_c \gamma)} \tag{18}$$

where d_{50} is the medium grain size, g is the acceleration of gravity, α_c is a coefficient representing the variation of the material characteristics, ν is kinematic viscosity, ρ is the fluid density, Δ is the relative density and $\Psi_{c,G}$ mobility parameter according to Grass. For uniform flow ($\sigma_0 \cong 0.4\tau_0$) Grass found that a bed of nearly uniform sand ($\alpha_c \cong 0.3$) was completely stable for $\gamma = 1$ and for $\gamma = 0$ a significant transport of sediment particles was observed. Based on his experiments, he reported that for $\gamma = 0.625$ the criterion of Shields was met for the initial movement of sands up to a size of 250 μm. In his opinion the $\gamma = 0.625$ criterion was also in agreement with observations of Vanoni and Tison when using the Rouse curve as a basis for the critical shear stress prediction. The critical bed shear stress τ_G is approximately 1.54 times higher than the time-averaged bed shear stress and thus 1.54 times higher than the mean critical value according to Rouse.



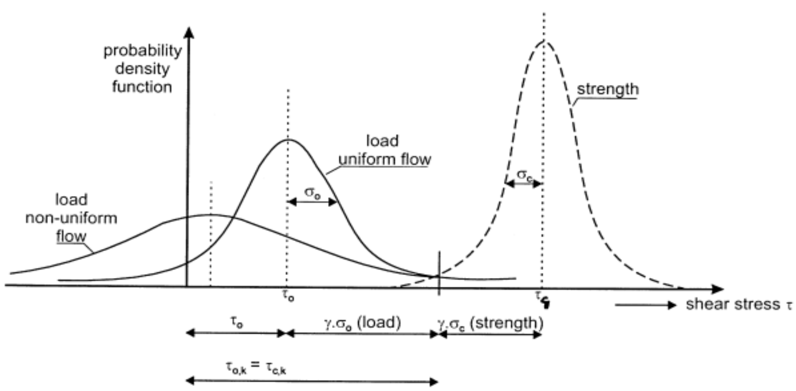


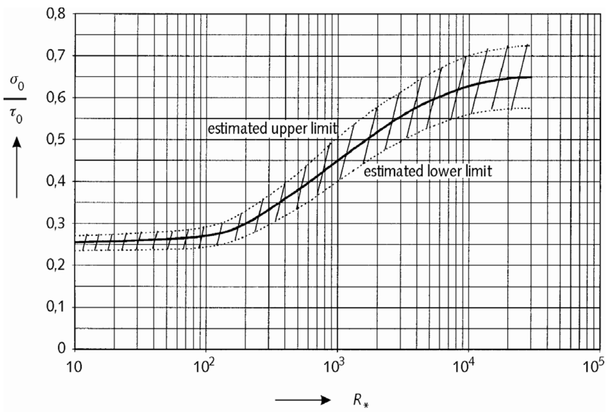
Fig. 8 Sweeps and ejections   Fig. 9 Probability functions of loading and strength parameters

With equation (18) the influence of particle gradation on the stability of the bed material can be explained in a qualitative way. A broadly graded material has more fines than a more uniform material. Consequently a broadly graded material is given an average grain size that is larger than for uniform material. These predictions correspond with observations in flume experiments.

Raudkivi (1998) pointed out that the beginning of movement is a function of the mean bed shear stress, its turbulence intensity, particle size and its distribution. For uniform flow the production of energy is at maximum near the bed and determined by the roughness (size and distribution). However, downstream of a sill, the turbulence intensity can be ten times that of the downstream uniform flow of the same depth.

In laminar flow when there is no turbulence (Reynolds number $R_* << 0$), σ_0 equals zero. In these flows the forces acting on the particles are marginal if compared to their weight. Hence all particles will be at rest ($\gamma >> 1$). Following De Ruiter (1980) the standard deviation of the instantaneous bed shear stress depends on the Reynolds numbers for uniform turbulent flow (Fig. 10).

For non-uniform (turbulent) flow there is no unique relation between σ_0 and R_*. The turbulence generated is strongly dependent on the geometry of the hydraulic structure and to a lesser extent to the roughness of the bed.
Following Raudkivi, equal conditions for the initiation of motion can be distinguished for both uniform and non-uniform flow (Fig. 11).



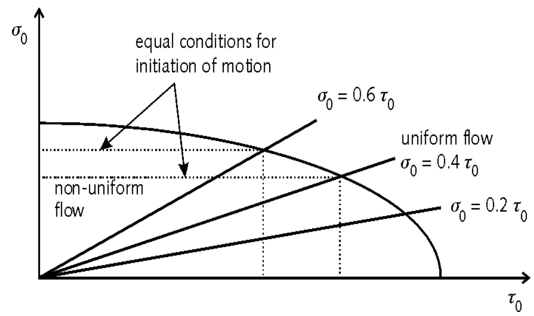


Fig- 10 - σ_0 as function of R_* (uniform flow) Fig-11 Equal conditions for initiation of motion

The area between $\sigma_0 = 0.2\tau_0$ and $\sigma_0 = 0.6\tau_0$ represents all combinations of uniform flow in which the particles are entrained by the bed shear stress. Particles can also be moved by turbulence alone at zero bed shear stress. For example, on a bed covered with ripples or dunes the bed shear stress is zero at the point of reattachment, yet the particles are in an agitated movement. Based on the experimental observations of Raudkivi, the subsequent hypothesis is introduced.

$$\tau_0 + \gamma\sigma_0 = f(\gamma)\frac{\rho}{h_0}\int_0^{h_0} k(z)\mathbf{d}z \tag{19}$$

whence follows:

$$\Delta d_{50} = 0.7\frac{(r_0 U_0)^2}{\Psi_{c,S}\, g} \tag{20}$$

where $\Psi_{c,S}$ is the critical mobility parameter according to Shields. About 90 experiments with small Froude numbers were analysed to validate (20) for flow directly downstream of a sill (Figures 5 and 6). In the tests at model scale both the hydraulic conditions and the geometrical parameters were varied. Moreover two types of transport were simulated, $\Psi_{c,S}$ = 0.032; occasional particle movement at some locations and $\Psi_{c,S}$ = 0.055, frequent particle movement at all locations. In the predictions the turbulence intensity was calculated by (16) in which the source term was neglected (Hoffmans, 2001). To evaluate the accuracy of the computed and measured values of the strength (Δd_{50}), a discrepancy ratio has been used, defined as:

$$r = \frac{(\Delta d_{50})_{\mathbf{computed}}}{(\Delta d_{50})_{\mathbf{measured}}} \tag{21}$$

About 80% of the experiments lie in the range of $0.75 < r < 1.33$.

Escarameia & May (1992) carried out tests with six different stone sizes on a flat bed at various turbulence levels. An adjustable sluice gate was designed and installed in the flume to produce a hydraulic jump with associated turbulence upstream of the test section. The tail water depths were controlled by means of a flap gate and a valve at the downstream end of the flume (Fig. 12).

The turbulence intensities were measured with a 3D ultrasonic current meter. Fig.13 shows the relation between the loading and the strength of about 30 experiments. The loading is represented by combination of a local turbulence intensity TI(10) at 10% of the flow depth and a depth-averaged flow velocity. The strength is given by $\sqrt{(\Delta g d_{50})}$. The upper and lower boundaries in Fig.13 reflect the transport for respectively $\Psi_{c,S} = 0.06$ and $\Psi_{c,S} = 0.03$. Though the transport was not explicitly measured, all experimental results lie within a range that could be expected. Since the local turbulence intensity at 10 % almost equals the depth-averaged turbulence intensity, (20) could be used to calculate the stability of the bed protection in a preliminary phase. For underflow situations it is recommended that a model relation for r_0 in analogy of (16) should be deduced. In addition, it is advisable to validate equation (20) for horizontal constriction, flow around bridge piers and abutments.

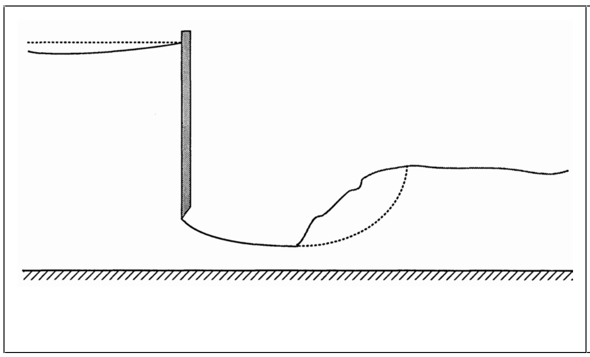


Fig. 12 - Hydraulic jump behind sluice-gate

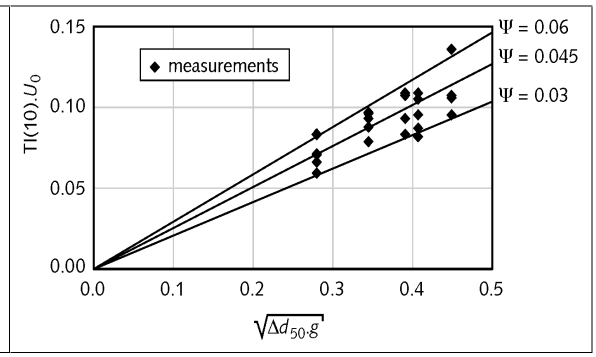


Fig.13-Relation between load and strength

EXAMPLES OF DUTCH SCOUR MODELLING

Where possible the approach described above will be applied. In this section examples of consultancy and research projects carried out by WL|Delft Hydraulics are presented.

Numerical prediction of protrusion scour

Near a factory the river width will be reduced due to the construction of a quay wall over a length of about 350 m from 220 m to 140 m at the upstream end and to 100 m at the downstream end (Fig.14 and 15). WL|Delft Hydraulics has been asked to predict the expected scour depth resulting from the construction of the structure on the right side in a river which will block about one third of the original channel width (Delft Hydraulics, 2002).

The river discharge is 2500 m^3/s, resulting in an upstream flow velocity of 2 m/s with a flow depth of 5 to 6 m. It should be noted that even if no construction is carried out the width of the river reduces from 220 m to about 130 just downstream of the proposed structure. This results in an increased flow velocity of 3 m/s.

At the left bank a “hard point” consisting of a locally more resistant bank is present and therefore protrusion scour may be expected. Protrusion scour occurs when the flow impinges on a bank or (river engineering) structure that is protruding into the flow (“hard points”), see Figure 16. The flow is forced inward and thus concentrated within a smaller width, increasing its velocity and hence its sediment transport capacity increases. The strong and sudden concentration of flow lines may result in scour of the riverbed upstream of the protrusion. The upstream location of the scour hole distinguishes protrusion scour from local scour in which the scour hole is usually found downstream of the structure and the scour process is related to eddies and vortices inclusive a high turbulence level. We observed major scour holes upstream of hard points for the first time as a different type of scour during the Meghna River project (Haskoning, 1992).

The scour prediction for the construction of the quay wall is based on a mathematical simulation of the situation using Delft3D (http://www.wldelft.nl/). The size of the grid cells is 5 x 4 metres near the corners of the structure and in the narrow part downstream. A period of 2 days was simulated morphologically. The results are shown in Fig. 14 and 15.

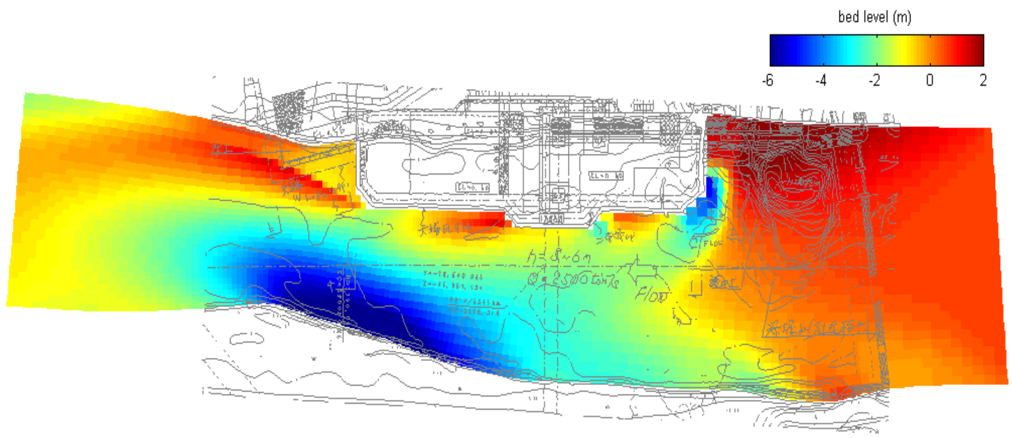


Figure 14 - Bed levels after 2 days of continuous discharge of 2500 m3/s.

(flow direction from right to left)

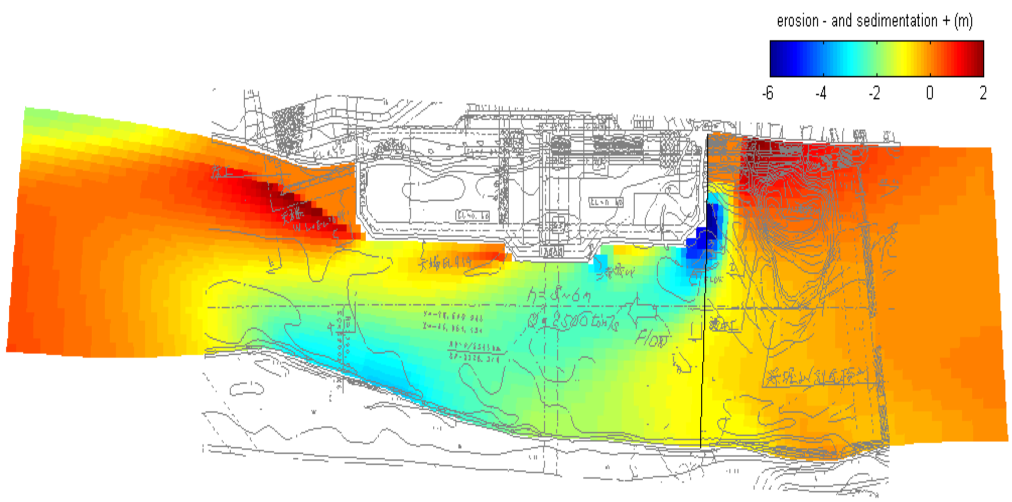


Figure 15 - Erosion and sedimentation pattern

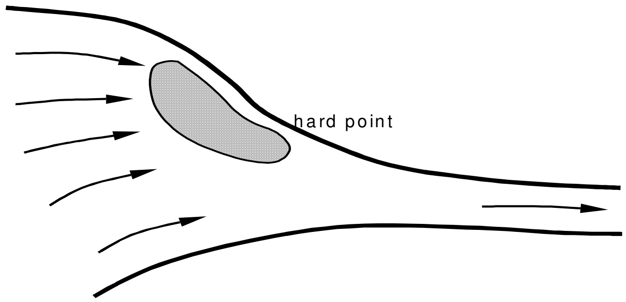


Figure 16 - Principle of protrusion

The constriction caused by the structure results in a general increase in the flow velocities alongside of it, up to 4 m/s. Vortices develop downstream of the structure. Near the upstream edge of the construction no flow separation was observed.

A scour hole with a depth of 6 m relative to the initial bed level quickly develops at the upstream edge of the structure. From the scour hole a channel forms that connects it with the existing scour hole close to the left bank. This scour hole deepens from 4 m to 7 m below the initial bed level. Furthermore, the simulation indicated that downstream of the structure close to the right bank sedimentation will occur despite the vortices and the high turbulence. In general, a local scour hole should be expected at this location.

Local scour holes and macroscale river morphology

Scour studies often use a two-step approach with a far-field and a near-field model. The far-field model is usually a mathematical model, whereas the near-field model is usually a physical model or an empirical scour formula. This approach ignores the feedback from the local scour hole in the near-field model to the morphology in the far-field model. Manuals on river engineering and scour do not deal with this feedback. However, the feedback may change the approach flow conditions in such a way that the local scour hole becomes deeper. This implies that the strong interaction between local scour and river morphology on a large scale is not taken into account.

Mosselman & Sloff (2002) discussed this phenomenon. For this purpose they carried out morphological simulations with the Delft3D model for a 15 km long reach of the River Waal in the Netherlands. As the mathematical model does not yet possess the functionality to create local scour, an initial scour hole is included in the initial bed topography close to the right bank at 7 km downstream of the upper model boundary and this is maintained by continual sediment extraction. Figure 17 shows the channel attraction as a continuous pool develops along the right bank from the scour hole to the next bend. The fairway constriction and bend tightening is also reproduced. Figure 18 shows an additional phenomenon downstream of the scour hole, viz. a pattern of forced bars. In conditions with a regular pattern of bars and channels this pattern constitutes the approach conditions for a local scour hole, and hence determines the scour depth. However, the very presence of a local scour hole itself affects the pattern of bars and channels in return. As a result, the approach flow conditions may change in such a way that the local scour hole becomes deeper. Thus, a feedback arises which may lead to deeper scour.

The foregoing shows the need for a two-dimensional approach. This is more emphasized if we consider the following aspects. In a one-dimensional approach, the bed erodes during the rising limb of the flood hydrograph (Fig.19). Conversely, the bed aggrades during the falling limb. The result is a lower riverbed during floods. Two-dimensional effects, however, may produce the opposite effect (Fig.20). Main low-water channels may experience sedimentation when the flood is conveyed over a much larger flow width. These channels are incised during the fall of the flood, partly by retarded scour across shallow channels. Thus, the riverbed can be higher during floods. Measurements of this phenomenon have been documented for the Jamuna River in Bangladesh (Delft Hydraulics, DHI & EGIS, 1996).

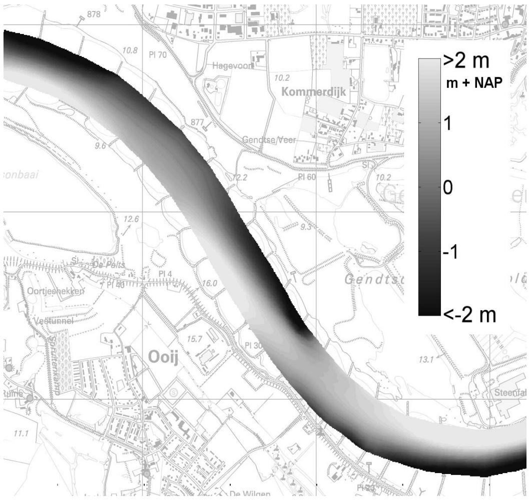


Figure 17 - Computed bed topography after 2 years. For case with continual sediment extraction to maintain scour hole (flow from right to left).

Figure 18 – Computed total erosion and sedimentation.

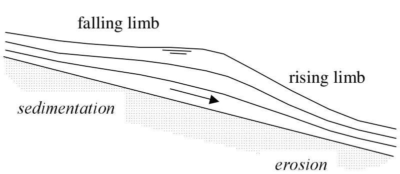


Fig. 19 - Longitudinal profile of flood Wave

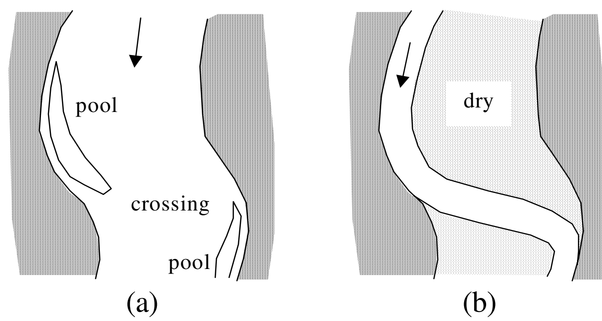


Fig. 20 - Bed topography during flood (a) and low flow (b)

Summarizing: scour studies for the design of structures on alluvial rivers require a two-dimensional approach, including a feedback from near-field models to far-field models, because channel attraction, bend tightening, channel narrowing (constriction) and downstream superimposition of forced bars due to local scour holes affect macro-scale river morphology. Manuals on river engineering and scour usually ignore this effect, which may lead to underestimates of scour depths.

Stability of scour hole slopes

Scouring is of no importance as long as no structures are threatened by the instability of the scour hole slopes. Scour holes of up to 50 m deep were considered to be acceptable near the

Eastern Scheldt barrier. However, a special schedule of regular dumping of gravel and stones were designed to keep the scour hole slopes gentle enough to remain stable (Davis & De Groot, 1983).

In most cases with limited scour hole depth (up to 5 to 10 m) and sandy bed, the natural slope of ca 1V : 1.5H is stable. Outflowing pore water, e.g. due to relatively quick descent of the water level may reduce the critical slope angle to 1V: 2H or 1V: 2.5H. Sometimes a weak clay or peat layer causes a similar reduction of the critical slope angle. The deeper scour holes, however, are often faced with two phenomena which cause a reduction of the critical slope angle to 1V : 4H or even more gentle: liquefaction of sand and breaching.

The sand in rivers and estuaries is often loosely packed, and thus liquefiable. Liquefaction flow slides may occur in scour hole slopes. The looser the sand and the deeper the scour hole, the more gentle is the slope critical for flow slides (Silvis & De Groot, 1995). A sophisticated analysis requires knowledge about the sensitivity to liquefaction of the sand and about its relative density (Stoutjesdijk *et al.*, 1998).

Breaching is a process well analysed on behalf of suction dredging in sand (Van Rhee & Bezuijen, 1998): sand is dredged away from the toe of a slope, causing a steep and unstable part of the slope. The instability propagates upstream more or less slowly and a sand water mixture flows downstream. The process stops, if no further dredging is done, either when the instability has reached the top of the slope or earlier in case of a too gentle slope. Whether the existing slope is too gentle and which slope results at the end, depends on the height of the slope, on the grain sizes and on the size of the initial instability. The steeper the slope, the smaller the sand grain sizes and the larger the initial instability, the more gentle the critical slope and the more gentle the resulting slope.

The same process of breaching or retrogression erosion, i.e. gradual retrogression of a steep slope, may occur in sand filled canyons or other deep natural under water slopes, but cohesive soils are also vulnerable for this process (Van den Berg *et al.* 2002). It may also occur in deep scour holes if a natural initial instability occurs, such as a (small) flow slide in some loosely packed sand layers or a local instability of a clay layer which has been undermined by the gradual scouring process.

Figure 21 shows an example of a bank collapse due to scouring and breaching, but designers should know that the same can happen at bed protection around bridge piers, downstream of sills and other structures.

Besides material parameters the stability of scour slopes also depends on hydraulic parameters. With higher flow velocities and more turbulent conditions, much steeper slopes are found (Hoffmans & Verheij, 1997).

Interaction of scour and bank protections

Bank protections attract the main flow in river channels, because of the formation of scour holes, which are local areas of reduced flow resistance due to the larger flow depth. This results in convergence of the flow towards the local scour holes that may become deeper than without the presence of a protected bank. To explain this, consider non-protected meander bends in homogeneous soils that have smooth curves (Fig.22-A). The bend migrates in a transverse (extension) and a downstream direction (translation). In front of the bank a scour hole due to bend scour exists.

A bank protection hinders the extension (Fig.22-B) and a deeper scour hole develops compared to a non-protected bank. This scour hole attracts the flow and slows down or stops the downstream translation. Thus, the smooth curvature of the meander bend changes, resulting in deeper scour, because the approach channel impinges more perpendicularly on the bank and the curvature of the bend becomes even more pronounced.

In addition, another mechanism also plays a role when the outer bank of a rapidly migrating channel is stabilised: the input of bank erosion products stops and consequently the supply of sediment to the outer-bend pool stops. As a result the scour hole will deepen.

Figure 21 – Bank collapse due to scouring

Clearly, this phenomenon should influence the design level of the toe of the bank protection in order to prevent undermining.

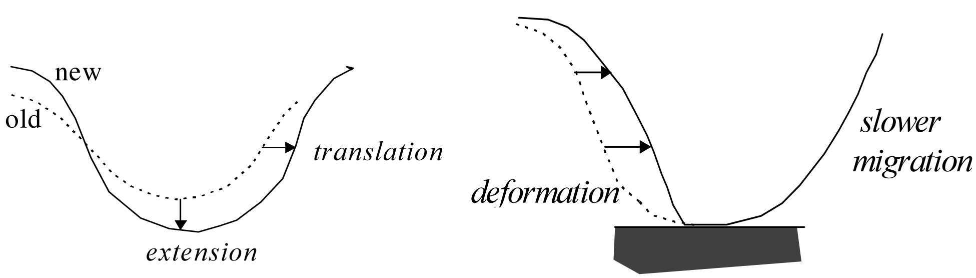


Figure 22- Meander bend deformation due to local resistance against erosion (A: smooth bends in homogeneous soil; B: hindered meandering by a bank protection).

Probabilistic approach

Usually, scouring is a combination of different types of scour. For instance, a combination of bend scour and local scour may occur near a spur. For many combinations the mode of combination is unclear, while, in addition, a lot of uncertainty exists about exact design values of parameters and coefficients. Nevertheless, the expected scour depth is a design parameter with a high economic impact. For this reason it is recommended that a joint probability analysis, taking into account the coherence of the types of scour involved and paying sufficient attention to expected values of parameters and coefficient, should be performed.

The combination of different types of scour is the first aspect that is briefly discussed. For instance for local scour and bend scour, there are two possible ways to determine the expected scour:

1. Adding both types of scour: $h_{total} = \alpha_{bend} h_0 + \alpha_{local} h_0 = (\alpha_{bend} + \alpha_{local})\, h_0$ (22)
2. Multiplying both coefficients: $h_{total} = \alpha_{bend} \alpha_{local}\, h_0$ (23)

Probably, the first approach is correct. Mesbahi (1992) proved this with small-scale experiments. Based on approximately 1000 experiments both at model and prototype scale Hoffmans (1995) showed that the total scour around bridge piers and abutments in channel flow is the sum of constriction scour and local scour (see relation 2). Some empirical formulas also include different types of scour. However, for most of the combinations of types of scour, we do not have a formal methodology to determine the scour. Thus, for the time being sound engineering judgement is required and a need for more research into this issue.

To overcome some of the problems it is possible to carry out a probabilistic approach. Then, the joint scour depth for a range of exceedance probabilities may be determined, for instance with a level II probabilistic method. To illustrate the possibilities: suppose that we will know the equilibrium scour depth for a slender structure, then:

$$y_{m,e} = L_s f_1 \cong b\left(\left(1+3r_0\right) U_\ell / U_c - 1\right) \tag{24}$$

Each parameter is characterized by a probability distribution, for instance expressed by the average value μ and standard deviation σ. However, it is not sufficient to use just r_0; it has to be expressed as:

$$r_0 = \sqrt{c_s + 1.45\sqrt{g}/C} \tag{25}$$

where c_s is a structure-dependent turbulence factor (see e.g. relation (17). Furthermore, the flow velocity U_ℓ should be related to the local unit discharge q and its fluctuations, and not the total discharge Q. Finally, also coefficients such as 1.45 in the formula for r_0 should be defined with both an average and a standard value.

After a probabilistic calculation has been carried out for a range of selected scour depths the probabilities of exceedance can be determined. This enables designers to choose the most appropriate design scour depth, which is in accordance with other failure probabilities of the structure. In addition, a probabilistic approach also gives insight into the parameters that have the greatest influence on the scour depth.

CONCLUSIONS: SOME NEEDS FOR RESEARCH

The possible consequences of scour, viz. the instability of one or more bridge piers and hence the superstructure, or a dike breach in combination with an inundation, require a well-funded design, which takes into account accepted risk levels and a state-of-the-art calculation of the expected scour depth. For a first estimate rules of thumb may be used and these may be optimised by applying two-dimensional morphological calculations.

In this lecture both aspects are discussed. Firstly, the concept of a generally applicable scour formula, that is independent of the type of structure is presented. In addition to geometrical parameters and coefficients, the formula also takes into account turbulence and the stochastic aspects of bed material characteristics. Obviously, validation of the formula under different hydraulic conditions and local situations requires data and time and consequently, the new scour formula is not ready for immediate use. For instance, it is only for sills that a formula for the relative turbulence r_0 has been derived, while the formula for Δd_{50} has not been validated for all types of non-uniform conditions. Aspects such as combined scour and the scour of cohesive material are not included because the existing knowledge is either insufficient or even not available.

Secondly, examples of consultancy and research projects in which the need for the application of 2D-mathematical, morphological models is shown are presented. The examples show the strong interaction between local scour and the macroscale river morphology, and the necessity for feedback between the mutual results. The mathematical models still require improvements, although some progress has been made, in particular for general and natural scour (see the presented examples). For example, the formation of a local scour has not yet been implemented in our own model Delft3D.

Summarizing, the following needs for research can be mentioned:

- Validation of the structure-independent formula for scour caused by different structures.
- Extension of equation (16) or (25) for the relative turbulence r_0 to provide a formula that is also valid for other structures.
- Validation of the stability formula (20) for horizontal constriction, flow around bridge piers and abutments, and flow in the ship's propeller jets.
- Determination of how local scour and natural scour should be combined: by just adding the separate values or by multiplying the coefficients.
- Breach processes (in sand and cohesive material) and their influence on scour, as well as determination of parameters influencing scour in cohesive materials.
- Implementation in mathematical models (and further validation) of the stochastic and fractionwise approach in order to calculate transport at the threshold of motion.

Finally, the Dutch approach is to treat scour as an integrated problem in which large-scale morphology, all types of scour, the process of breaching and soil mechanics each have a role. We strongly recommend this approach in order to prevent the underestimation of expected scour depth, which may result in undesired consequences of scour.

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