

DESIGN AND PRACTICE OF SCOUR AND EROSION COUNTERMEASURES IN WATERWAYS

Michael HEIBAUM

Member of ISSMGE, BAW – Fed. Waterways Engineering and Research Institute
(Kussmaulst. 17, 756187 Karlsruhe, Germany)
michael.heibaum@baw.de

It is essentially for waterways, to guarantee always easy and safe navigation. Therefore scour and erosion protection of bank and bed of waterways is one major task to meet this requirement. In waterways, additionally to the well known actions of currents and waves, ship induced loads act on banks and bottoms have to be considered. Only few manuals provide assistance in designing appropriate countermeasures. Additionally, the installation of scour and erosion countermeasures often is hindered by currents and wave action, so special installation methods have to be chosen. Besides hydraulic stability of the chosen countermeasure, geotechnical stability and durability has to be verified. The contribution discusses the numerous aspects to be taken into account while designing scour and erosion countermeasures and appropriate cross references are given without repeating the design approaches published there.

Key Words : Waterway, bank protection, ship induced load, scour and erosion countermeasures

1 INTRODUCTION

Scour and erosion protection is essential for waterways because of two reasons. Firstly, the cross section necessary for easy navigation has to be guaranteed. So no deposition of material in the navigation channel due to sediment or bedload transport or slope instabilities shall hinder navigation. Secondly, waterways often lead through densely populated areas, at least in Middle Europe, so stable banks of rivers and canals are of particular importance. Therefore any conceivable damages or failures have to be ruled out by corresponding protection measures. If the dimensions of a river allow for bank erosion or stream instabilities without affecting the navigation channel and the riparian life, protection may not be needed - this case is not considered here, nor are treated any flow altering devices to reduce the actions or effects on bank and bottom of waterways.

The performance of erosion countermeasures in waterways is similar to other inland and coastal scour and erosion protection methods, but moreover the de-

sign and the execution have to take account of certain loads that are caused by the interaction of water craft and waterway. Wave action may be stronger at the coast and flow load may be higher in rapid-flowing rivers where navigation is not possible. But the effect of propeller wash and bow thrusters or the disproportionately increasing load on banks whenever ship speed approaches a critical level and some other loads have to be taken into consideration that are not met without navigation. German recommendations "GBB"¹⁾, for instance, deal with these specific actions on banks and bottoms of waterways and provide approaches to design countermeasures. Other than ship induced loads are covered by several guidelines like the scour manuals of Hoffmans & Verheij²⁾ or Melville & Coleman³⁾. Just recently a report has been finished on scour relevant loads on bridge piers and corresponding countermeasures⁴⁾.

Special attention has to be paid on the stability of banks. In this context, special attention has to be paid to the interaction of hydraulic and geotechnical actions and resistances. Any bank damage will endanger

shipping and any risk of embankment damage at impounded rivers and canals with a water level elevated above ground includes the risk of flooding the hinterland. Local instabilities may trigger scouring and, vice versa, scour may result in local and global bank failure. Therefore the resistance of the bottoms and banks of rivers and canals to hydraulic actions must be verified in each case and protection methods must be provided to achieve adequate resistance and stability.

2 DESIGN CONCEPT

The design of countermeasures in waterways has to cover both hydraulic and geotechnical aspects. For geotechnical analyses an appropriate extreme load constellation with a very low probability of occurrence is determined. Unless explicitly stated otherwise, the analysis must demonstrate that the limiting equilibrium state is maintained under the relevant combination of actions. A certain factor of safety is required as to the overall stability of banks according to the national standards.

Hydraulic design should rather be based on a cost-benefit analysis. The requirements regarding the probability of occurrence of the actions to be used in the design are less stringent for hydraulic analyses than for geotechnical analyses. One aspect, for example, is to determine the stone size of an armour layer required to provide resistance to movement on exposure to currents and wave loads. The displacement of individual stones – despite accumulating over time – does not jeopardize the stability of revetments or embankments. One of the most important parameters in addition to the structure of the armour are the volume of shipping and fleet composition as regards maintenance costs. The number of stones that are displaced from a revetment increases with the volume of traffic. However, cost-benefit analyses require comprehensive and detailed data on the cost of maintaining the various types of armour, which depends on the volume of shipping and fleet composition. Such data are not (yet) available. Therefore the only feasible approach hitherto is to define limit values of hydraulic actions or effects of actions.

3 HYDRAULIC ACTIONS ON THE BANKS AND BOTTOMS OF WATERWAYS

Bank and bottom of rivers and canals are exposed to the following hydraulic actions that can occur alone or

may be superimposed:

- currents
- waves
- drawdown
- inflow of groundwater

Currents and waves can cause erosion of the bottoms and banks of a canal or river while rapid drawdown or a considerable inflow of groundwater may result in sliding or loosening of the soil (including hydraulic heave) which in turn eases erosion and scour formation.

(1) Currents

Only turbulent currents are of significance for waterways as they can cause erosion, depending on the particle size of the material present in the banks and beds. Highly turbulent currents occur, in particular, in

- riverbeds with constrictions and obstructions
- the tail water of weirs
- the propeller wash
- the return flow due to shipping
- in the slope supply flow.

Vortices develop from the alteration of the flow regime through obstructions or constrictions. The emerging turbulent flow will remove material without sufficient resistance surrounding the obstruction. Consequently, a scour hole develops. The same holds for highly turbulent flow in the tail water of weirs.

The propulsion-induced flow velocities near the bed are greatest for

- ship's propellers with large diameters and high design pitch ratios,
- ship's propellers designed for high rotational speeds or high performance,
- unducted propellers with middle rudders located behind them, owing to the division of the jet caused by the angular momentum,
- if propagation of the propeller wash is limited, e.g. in the vicinity of a quay wall,
- for small dynamic underkeel clearances.

The load case will be relevant e.g. for a vessel starting off, making full use of its installed engine power. Moving vessels generate lower impacts. The main propulsion of a vessel causes significant loads on the bank when the main rudder directs the propeller jet towards the bank, e.g. to leave a mooring or when turning.

The bow thruster jet, if directed towards a bank, can cause local scour and hence a great deal of damage to the bank. Any revetment design that includes the loads produced by bow thrusters may result in overdimensioning when compared to a design that only

covers the other loads. It will need to be considered whether such damage should be repaired during maintenance work or if it is advisable to choose revetments of connected elements in the affected areas.

The mean return flow velocity increases with the ship speed, the displacement by the vessel and the reciprocal value of the effective cross-sectional area. The local return flow in the vicinity of the bed and bank will exceed the mean value. Significant local lowering of the water level may occur at these points, resulting in a further increase in the return flow velocity owing to the narrowing of the flow cross-section. This effect is most noticeable between the ship's side and a sloping bank when a vessel sails close to the bank

The slope supply flow accompanies running waves close to a bank and refills the depression caused by drawdown from astern. In limiting cases, the slope supply flow velocity may even reach the same speed as the vessel. This occurs when the momentum of high transversal stern waves causes the wave celerity to increase to such an extent that the wave threatens to "overtake" the vessel, resulting in a breaking of the wave. This effect is most pronounced in narrow cross-sections and when vessels sail close to a bank at a speed approaching the critical ship speed. The higher speeds at which empty vessels or tugs may sail means that this case may be relevant to the design.

(2) Waves

Waves that create significant load on bed and banks of waterways are generated by sailing vessels and strong winds. However, they can also be caused by the operation of weirs, locks and power stations (surge / drawdown). Ship-induced waves are divided into primary wave system and secondary waves. The primary wave system (drawdown, transversal stern wave) develops in the vicinity of the vessel and moves at the same speed (Figure 1). Secondary waves can travel a long way from the vessel and then behave in the same way as free waves.

It is the running waves that are usually responsible for the displacement of stones in bank revetments. When vessels approach critical speed the transversal stern wave may break and form a running wave (like a moving hydraulic jump). Very high transversal stern waves occur in the following situations in particular:

- eccentric paths, in particular those close to the banks, in which case the wave length will decrease and the wave steepness, and thus the wave height, will increase;
- empty vessels, that exhibit a stern-heavy trim;

- towboats, tugs and recreational craft sailing alone that generate large diverging waves at the bow that may be superimposed on the transversal stern wave;
- vessels travelling close to the critical ship speed
- recreational craft designed for planing but which displace water when accelerating to sliding speed, in which case the transversal waves of the bow and stern wave systems are superimposed

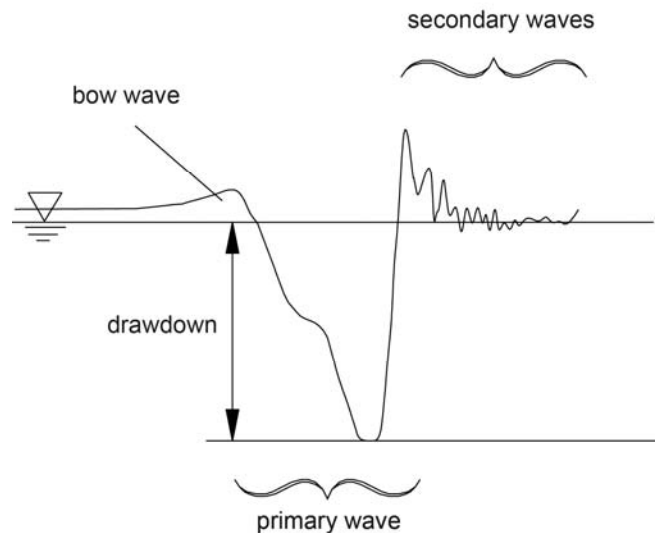


Fig. 2 Secondary wave system

The (secondary) waves generated by the discontinuities and pronounced curvature of the ship's contour are divided into diverging waves and transversal waves. They originate primarily at the bow and stern and give rise to interferences that diverge at stern along a line at a certain angle, where the highest waves occur. The diverging wave system is focused on a narrow strip along that line. The highest waves at the bank are generally caused by the diverging wave systems when vessels sail far from the bank and by the transversal wave systems when vessels sail close to the bank (Figures 2 and 3).

(3) Drawdown

Falling water levels may affect the stability of banks and beds of waterways. To evaluate the stability it is important to establish whether the pore water in the underlying soil is able to follow the changes in the water level of the river or canal without significant excess pressures being generated. A comparison of the drawdown rate of the water level and the hydraulic

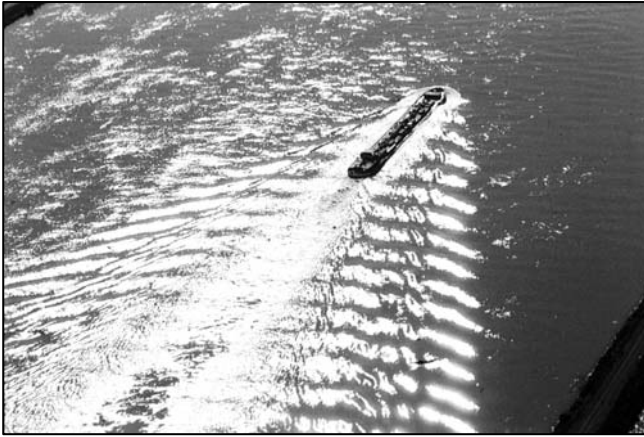


Fig. 2 Secondary wave system

conductivity of the soil can initially provide a conservative estimate of whether excess pore water pressure is being generated. Drawdown rates less than the hydraulic conductivity result in only small gradients of the pore water flow, so the associated flow force can be neglected with respect to the bank stability. Rapidly falling water levels cause excess pore water pressures in the soil, affecting the (geotechnical) stability

of the bank.

Ship induced drawdown comprises:

- Squat = drawdown of a vessel depending on the draught and the relation of the cross sections of vessel and waterway. This type of drawdown occurs only in a limited waterway cross section and spreads to the bank with decreasing magnitude.
- Bow and stern waves of sailing vessels, particularly of unladen vessels, passenger crafts and recreational crafts sailing at high speed.
- Other drawdown is created by:
 - Tides
 - Receding flood waves (immediately after the flood peak has passed)
 - Drawdown waves due to the operation of weirs, locks and power stations
 - Failure of embankments
 - Wind waves

In comparison with ship-induced waves, wind-induced waves may be relevant to revetment design if a canal is wide and the fairway is far from the bank.

(4) Groundwater inflow

Groundwater will flow into a river or canal if the

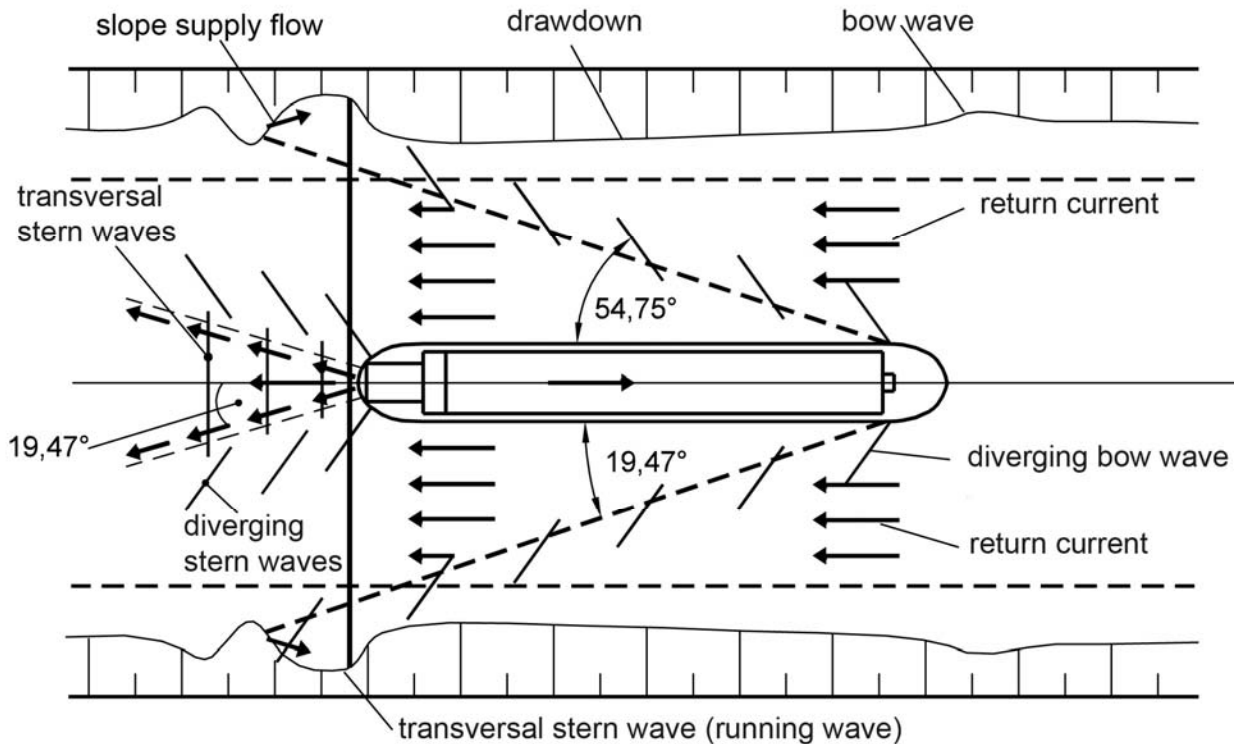


Fig. 3 Superimposition of primary and secondary wave systems (according to GBB¹⁾)

groundwater table in the slope is higher than the still-water level of the waterway. With such boundary condition a higher hydrostatic water pressure acts in the subsoil of the bank, giving rise to flow forces in the direction of the river or canal. All geotechnical design calculations must take such actions into account.

If groundwater flows out of an unprotected slope, the limit state for local slope stability will be reached at a slope inclination of $\beta \leq \phi'/2$ (β being the slope angle and ϕ' the effective angle of shearing resistance of the soil). Any outflow of groundwater from the surface over a fairly long period of time will cause erosion and should therefore be avoided.

4 DETERMINING HYDRAULIC ACTIONS

For the design of protection layers in waterways the following hydraulic design parameters are required:

- maximum wave height,
- maximum flow velocity,
- maximum drawdown,
- maximum drawdown velocity and resulting excess pore water pressure.

The values of these parameters can be determined either by measurement or by calculation. Measurement data are preferable to calculated values as the latter are subject to the following limitations:

- The calculation methods are based on assumptions regarding the relevant physical processes, that work with simplified fundamental equations and use simplified geometrical data for the boundary conditions.
- Empirical corrections to the design approaches only apply to the scope for which measurement data is available.
- Experience is not available on individual design situation (e.g. for the wave heights caused by recreational craft).

It is for this reason that often more than one equation is available.

As regards the accuracy of the calculation methods it should be noted that the loads resulting from the primary wave field can be determined more accurately than those arising from the secondary wave field, the slope supply flow and wind waves. Particularly it is very difficult to determine the loads due to the propulsion units of ships as the latter depend largely on the ship design.

5 EXCESS PORE WATER PRESSURE AS A FUNCTION OF RAPID DRAWDOWN

Natural surface water and pore water in the subsoil are not an ideal, incompressible fluid. Small microscopic air (more generally: gas) bubbles are dispersed in the water, so the fluid shows a certain compressibility. Compressible pore water causes a delayed reaction of the pore water pressure on any pressure change at the boundaries if the hydraulic conductivity of the subsoil is lower than the velocity of that pressure change. An example is the rapid lowering of the surface water level, resulting in an excess pore water in the adjacent soil. Due to this phenomenon, bank stability is affected by the interaction of surface water and pore water.

The time of delay depends on the permeability and the saturation of the subsoil⁵⁾. Figure 4 shows the development of excess pore water pressure for a river

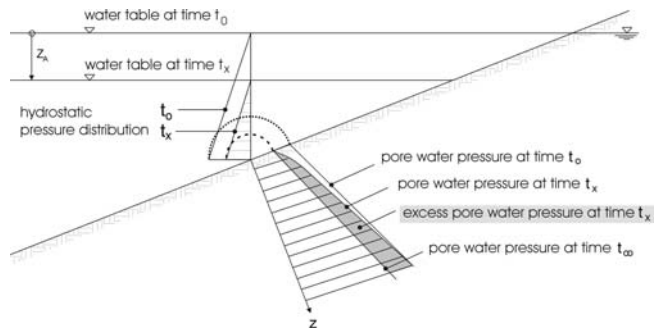


Fig. 4 Pore water pressure distribution due to sudden draw down

bank subjected to a sudden drawdown.

The time dependent pore water pressure distribution after a sudden draw down may be found by measurements or by an unsteady (mostly finite element) calculation. Immediately after the draw down, the potential difference is large in a very small zone below the surface (Figure 5a), thus a strong seepage force or a large excess pore water pressure is developing. The width of the zone increases with time until the final state is reached and all excess pore water pressure has disappeared (Figure 5c). In the case of an ideal fluid with no compressibility, the development of the pore water pressure is different (Figure 5b): The pressure change progresses with the velocity of sound. Only the ground water level remains at its place for the first moment. So only in the area of the water exit the final

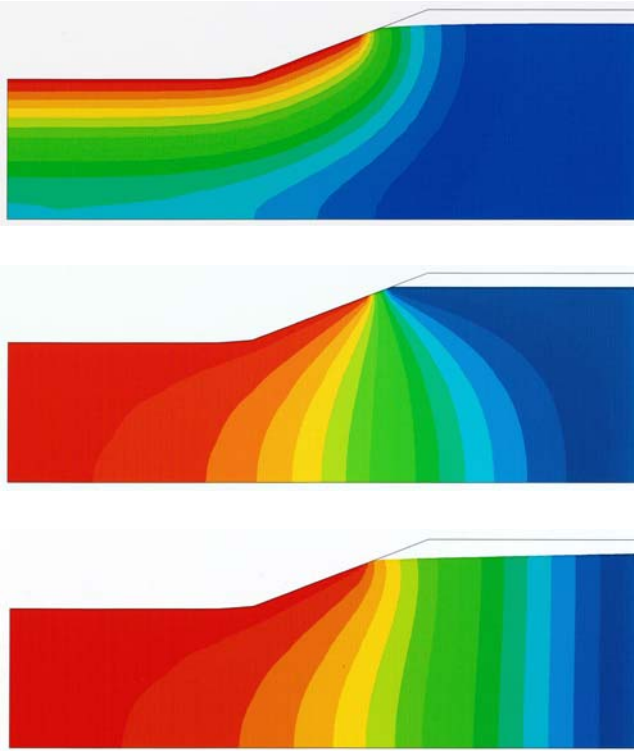


Fig. 5 Hydraulic potential shortly after sudden drop of the water table - top: compressible fluid, mid: incompressible fluid; bottom: final (steady) state

steady state is not yet reached, as even an ideal fluid needs some time to flow out of the pore volume.

The distribution of the excess pore water pressure with depth may be simplified by an exponential function with only one parameter. This pore pressure parameter "b" is dependent on the soil permeability, the compressibility of the soil and the compressibility of the pore water as shown in formula (1). The excess pore water pressure with depth is given by formula (2) (Schulz & Koehler⁶⁾),

$$b^2 = \pi \cdot \gamma_w (n \cdot \beta_w + E_s^{-1}) \cdot (2 \cdot k \cdot t_a)^{-1} \quad (1)$$

$$\Delta u(z) = z_a \cdot \gamma_w (1 - e^{-b \cdot z}) \quad (2)$$

with:

- γ_w unit weight of the water
- n pore volume
- β_w compressibility of the pore water
- E_s compressibility of the soil
- k permeability of the soil
- t_a time of pressure drop
- z_a amount of draw down
- z depth

Often "b" is found by measurement and not by formula (1). For many practical calculations, a design

chart has been developed⁷⁾ dependent on soil permeability and time of pressure drop (Figure 6). That chart gives sufficiently exact values for a mean pore volume of $n \approx 45\%$, the range of soil compressibility $5 < E_s < 50$ MPa and saturation of $85 < S < 95\%$. With increasing depth and decreasing pore volumes, "b" becomes smaller. Further information is given in Heibaum & Koehler⁸⁾.

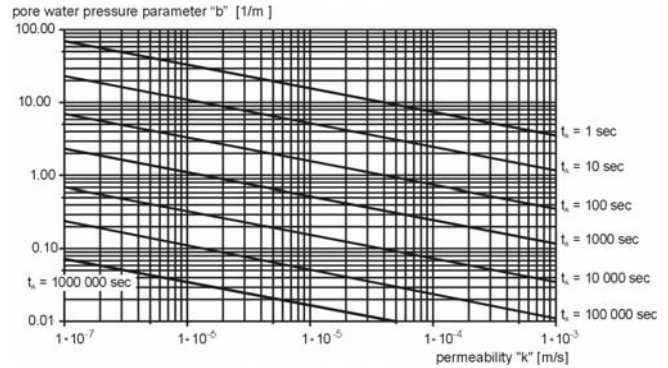


Fig. 6 Design chart: $b = f(k, t_a)$

6 SCOUR AND EROSION COUNTERMEASURES

(1) General Aspects

To protect bank and bottom of waterways from erosion a cover layer is needed that resists the hydrodynamic actions. Additionally, the surface of any cover material has to be resistant against abrasion. Depending on the configuration of the protective system, transient layers in between the armour and the subsoil are required, such as filter layer, cushion layer, impervious lining etc. The armour layer may consist of single elements, mattresses or continuous blankets (Figure 7).

(2) Permeable vs. impermeable protection

A cover layer to prevent scouring may be permeable or impermeable. Impermeable systems can be acceptable only in those cases when the development of excess pore water pressure or uplift below the layer is ruled out. Uplift may be caused by a high groundwater table compared to the level of the surface water. An (unsteady) excess pore water pressure below a permeable cover layer develops due to rapid draw-down according to paragraph 5.



Fig. 7 Armour layers (top left to bottom right: riprap, partial grouted riprap, bitumen grouted riprap, stone mattress, grid concrete mattress, concrete cushion mattress, cable connected concrete blocks, articulated concrete blocks, paved concrete/basalt columns)

(3) Cover Layer Elements

a) Single elements

Riprap (or more generally: armourstones) is maybe the most often used material for protection systems to stop or to reduce scouring. Solutions at reasonable costs can be attained if this material is sufficiently available. Often riprap is considered a temporary countermeasure only, which is wrong. This opinion may originate from the use of stones that are too small or from armour layers without a filter beneath to hinder winnowing.

Concrete elements are used when natural material is not available to the necessary extent. The production costs are much higher but the transportation distance is usually rather short; sometimes they may even be produced on-site. There is a large variety of concrete elements that are used as an armour layer. They will not be listed here in detail, but there are three groups to be mentioned: Elements of many different shapes that are used like riprap (A-Jacks, Tripods, Dolos, etc.), blocks that are placed regularly ("paved") and elements that are mutually connected.

Geosynthetic containers are multi-purpose elements made from woven or nonwoven geosynthetic fabric. They can be manufactured according to any demand, concerning raw material, size, shape, filtration capac-

ity, strength etc. The containers are prefabricated, thus providing a constant quality, and filled on or near the site. Often they result in lower costs compared to traditional construction methods which is of great importance in developing countries. They allow easy adaptation to local conditions in respect of preparation, filling, transport and installation. Until now, there is a large number of examples in operation that prove the long term stability of both the material and the entire structure.

The resistance of all single elements against hydrodynamic forces increases with the weight of the element. But increasing the weight, which is usually linked with an increasing diameter, means increasing the layer thickness, too. And since the voids between larger elements also become larger, the elements of the layer below (filter or cushion layer) have to be larger for not being eroded through the cover layer. Maybe even an additional layer is necessary. Very large stones and an increasing layer thickness may be incompatible with the water depth, so other solutions have to be sought.

b) Connected elements

To minimise the thickness of the armour layer but to provide the best resistance possible against the hydraulic actions, cover layers with connected elements are often recommended. The general idea is to use smaller and often cheaper elements, but to gain high resistance against the hydraulic load by connecting them to larger elements, to mattresses or to continuous layers. Examples are gabions, stone or concrete mattresses and open stone asphalt or mutually interlocking concrete elements, respectively.

Gabions are made of riprap or even smaller stones filled in a wire mesh basket, thus creating one larger element out of several smaller ones. Gabions are very versatile elements concerning the shape of the single element as well as the shape of the whole cover layer. Gabions may be prefabricated or - in the dry only - filled in place. Stone mattresses are similar to gabions and manufactured like these. They are usually thinner than gabions, but cover a larger area. If placed in the dry, they can be connected to each other thus creating a continuous armour layer. Mattresses are prefabricated when they are placed in the wet, but the area of one mattress is limited due to the weight. Due to the wire mesh, gabions and stone mattresses are susceptible to ship collision!

Cable connected systems are similar to stone mattresses. Like these their size is limited when placed in the wet. In the dry they can be assembled continu-

ously, depending on the system. All cable connected systems, gabions and stone mattresses are endangered by corrosion and abrasion of the cables or the wire mesh, so the long term stability of both steel and polymer wires is limited. Abrasion is due to sediment transport or due to the relative movement of wire or cable and armour element. Gabions and stone mattresses cannot be filled so tightly that the stones do not agitate at all under the hydraulic load. In this respect, concrete blocks fixed on a geosynthetic fabric may perform better. Size and placement is similar to stone mattresses.

The oldest form of a mattress is the fascine mattress, i.e. willow bundles with a diameter of 10 to 40 cm tied together. In the beginning only fascines and brushwood were combined to a mattress. Later a woven geotextile was used as the base with the fascine bundles tied on it, if necessary a brushwood layer was added (Figure 8). Fascine mattresses are prefabricated according to the desired geometry on land, then they are pulled to the desired position and sunk by dumping the armour material upon.



Fig. 8 Preparing a fascine mattress (including additional brushwood layer)

A cover layer from paved stones (natural stones or concrete blocks) also shows an increased resistance against hydraulic loads while remaining limited in thickness, since the stones are assembled with nearly no joints. But not being connected to each other bears the risk that the high resistance is lost if only one element is missing.

Open stone asphalt is a very well known continuous cover layer in coastal protection, but can be placed only in the dry. In the wet prefabricated mattresses of open stone asphalt are used.

Continuous layers also may be created by so called geosynthetic mattresses filled with concrete or mortar. They can be placed continuously, since the fabric is sewn together as needed and then the mattress is filled. Mattresses of uniform thickness are inflexible and impermeable. To achieve a certain flexibility and permeability, mattresses consisting of columns and rows of "pillows" are used. The seams between the concrete filled pillows provide the necessary permeability of the layer and the desired flexibility for good adjustment to any deformation of the subsoil.

Another form of continuous layers can be achieved by mutually interlocking concrete elements. Placement in the wet needs high effort, so usually such cover layers are assembled in the dry, or they are pre-assembled to mattress-like structures and treated like these.

Partial grouting of riprap produces conglomerate-like elements by which the resistance against hydraulic load is increased but sufficient flexibility is still provided. Grouting has proved its long term stability and its ability to keep the costs of armour structures low. There are two general grouting procedures: either the grout is distributed more or less uniformly or the grouting is done line by line or spot by spot, the latter resulting in conglomerate-like elements (also called pattern grouting). With the correct amount and the correct distribution of the grout the armour layer still remains flexible to adapt to ground deformation.

c) Bank stabilisation by vegetation

The use of vegetation with or without structural elements – so called bioengineering - is used increasingly to reduce bank erosion or for bank stabilisation. Grass has been used successfully as a dike revetment to prevent erosion of the dike material by hydraulic forces⁹⁾. But there is still a lack of knowledge about the properties of the natural materials being used in relation to force and stress generated by flowing water and ship induced loads. Till now, it cannot be recommended to rely on bioengineering countermeasures as the only technique when there is a risk of damage to property or structure, or where there is potential for loss of life if the countermeasure fails. But there is a significant ecological benefit when combining structural countermeasures and vegetation like plants fitting in partial grouted riprap or geotextile mattresses including pre-raised plants. Geosynthetic reinforcement of vegetative revetments has been applied successfully to increase resistance.

7 HYDRAULIC DESIGN OF ARMOUR LAYERS

(1) Armourstones

Experience has shown that the resistance of riprap cover layers to erosion due to the actions of waves or currents is affected by the following parameters:

- stone weight or size (grain size) and dry density
- thickness of the stone layer
- pore size and thickness of the substructure

Many methods of designing such cover layers are based on determining the required mean weight of the stones G_{50} . The required stone size is calculated after initially determining the nominal stone size D_{n50} from G_{50} and the selected dry density ρ_s by assuming a cube. Other design methods use sizes of screened stones such as D_{50} or D_{85} determined with a sieve with square perforations.

The determining load parameters to design scour and erosion countermeasures are the flow velocity and the wave height. To determine the required stone size for riprap at bridge piers, numerous design approaches are available, e.g. Melville & Coleman³⁾ (including abutments and general scour), Lagasse et al.¹⁰⁾ or the detailed report of Lauchlan¹¹⁾. Lauchlan¹¹⁾ and Melville & Coleman³⁾ demonstrate the large scatter of published equations to determine the riprap size, which shows the inherent uncertainty of these approaches.

Stone sizes required for bank and bottom protection of waterways due to ship induced loads and wind waves are discussed in GBB¹⁾. Formulae are given for effects of transversal stern waves, secondary waves, wind waves, propulsion flow, flow parallel to the bank and slope supply flow.

To determine the stone size necessary to protect banks against wave loads, detailed design approaches are given in Pilarczyk¹²⁾. It has to be mentioned particularly that in the approach for rock armour layers a notional permeability factor is introduced to take into account the porosity of the sublayers of the armour layer and its influence on the wave action on the protection system according to the proposal of van der Meer¹³⁾.

The uses of the design equations for the required stone size often presuppose a minimum construction thickness d_D of the cover layer. This results from the boundary conditions of the various model tests on which the design methods are based. The following recommendation is found rather often:

$$d_D = (1.5 \cdots 2.0) \cdot D_{n50}$$

Stability can be increased to a certain extent by increasing the thickness of the armour layer, if a non-uniform riprap is used³⁾.

(2) Geosynthetic containers

The use of geosynthetic containers for protection against hydraulic impact is a young method, so few design approaches are available. A comprehensive collection of design approaches has been published by Pilarczyk¹⁴⁾. Only recently design approaches have been developed by Recio¹⁵⁾ for stacked containers loaded by wave impact and overtopping.

(3) Connecting armour elements

Obviously there is a certain uncertainty with most of the equations to determine the appropriate armour-stone size, so it might be beneficial to think about additional resistance against the hydraulic load in addition to size, weight and shape of armourstones. Connecting the single elements of a protection layer has proved to increase the strength of an armour significantly.

Connected elements provide armour as a result of having created larger elements. Some can be considered like large armourstones, e.g. gabions or fascines with rubble core, including the condition that the element is packed in such a manner that it behaves like one body. Other element like stone mattresses or concrete mattresses show increased resistance due to the larger area that is covered by a mattress, but this may be neutralised by the reduced thickness.

a) Mattresses

Stone mattresses are containers constructed of wire mesh and filled with riprap. Therefore the same quality requirements hold as stated above. The advantage of the wire mess mattress is that smaller stones can be used. Lagasse et al.⁴⁾ have compiled formulae for the permissible shear stress and the design shear stress for the use of stone mattresses in pier scour countermeasures, but it may be extended to other application when a relevant local design velocity can be determined.

Maynord¹⁶⁾ proposes to use the filling rock diameter for the design of stone mattresses. Simons et al.¹⁷⁾ found that the critical shear stress required to move a stone within the wire mesh casing of a stone mattress is more than twice that for the same stone diameter outside the mattress. Even if these approaches are successful by including appropriate parameters, the initial idea of forming a larger element by confining smaller ones is disregarded completely. The basic idea is that

the larger element (gabion or stone mattress) can resist the hydraulic load much better than a single fill element and the casing made of wire mesh or else has to be sufficiently robust to keep the fill inside even if the fill material is dislodged.

For articulated grout filled mattresses Lagasse et al.⁴⁾ propose a design procedure. The main destabilizing factors are drag and lift forces acting on the mattress. The application requires pressure relief holes ("weepholes") to avoid any uplift force due to pore water pressure beneath the mattress. A filter is required to hinder winnowing through the weepholes. Taking into account that the weepholes have a rather limited area compared to the area of the whole mattress and that a filter will furthermore reduce the effective open area, a certain excess uplift pressure will remain.

Fascine mattresses are used for bank and bottom protection in large rivers. They are covered by armourstones that have to be designed as riprap (paragraph 7.1).

b) Interlocked elements

Similar to mattresses, hydraulic resistance is provided by articulating concrete elements. Such systems consist of preformed concrete units that interlock, are held together by steel rods, or are bonded to a geotextile fabric. General hydraulic design approaches are not known, in most cases the necessary information is provided by the manufacturer because of the many variations in shape, size and weight. Application remarks are given by Lagasse et al.⁴⁾. A problem may arise due to the fact that such protection layers are of limited thickness only. So the weight might be not sufficient to stabilise the underlying soil (geotechnical stability) and the vulnerability as to mechanical damage is increased. Design proposals concerning wave attack are published by Klein Breteler & Bezuijen¹⁸⁾.

c) Partial grouting

Field tests and experience have shown that partially grouted cover layers provide adequate resistance to all known hydraulic actions occurring on waterways, if the grouting has been performed correctly. For successful grouting the riprap size should not be too large (grout will flow through the large voids) and not too small (grout can not penetrate into the voids). An appropriate grading is CP_{90/250} according to EN 13383-1³¹⁾. Using smaller stones it becomes more difficult to achieve the desired distribution with depth while the voids of larger stones are too big to retain the grout and the contact area of two stones is too small. With

respect to tests, hydraulic design is not required in this case provided that the maximum flow velocity does not exceed 8 m/s¹⁾. Laboratory tests at the Braunschweig University, Germany, proved stability up to that flow velocity (test report unpublished). Quantity and quality of grout should be chosen according to MAV¹⁹⁾ and being tested according to RPV²⁰⁾ (both 1991 editions available in English – new German edition 2008). To maintain sufficient permeability, less than 50% of the voids are filled with grout.

(4) Continuous layers

Continuous layers like asphalt concrete, open stone asphalt, mastic and fully grouted riprap will withstand nearly all hydraulic forces occurring in waterways unless the layer remains intact and does not show a breach where the hydraulic forces can proceed to the ground or under the layer. Heavy wave action will cause impact forces on the layer, resulting in bending stress. For bituminous revetments, an approach is given by van Herpen²¹⁾.

8 DESIGN OF FILTERS

The majority of filter rules implies laminar flow more or less perpendicular to the filter-base interface. That approach holds for flow towards a drain or a well. Looking at filters in scour countermeasures, the main flow is parallel to the interface, but there is a secondary reversing flow perpendicular to the interface of unknown intensity which is decisive for the design of filters in waterways. In German canals with standard cross section, a passing ship induces an unsteady reversing flow perpendicular to the interface of about 10-20 Hz. This value may change with the size of the vessels and the cross section of the waterway. As this secondary flow is still not sufficiently understood, it is proposed to use geometrical sandtight granular filters following the criteria of Cistin / Ziems or to install geotextile filters designed accordingly. An overview of filter rules applying to scour and erosion countermeasures and is given by Heibaum²²⁾.

When trying to obey filter rules in scour preventing armour layers the question arises as to the validity with regard to the particle size. All filter rules were developed for particle sizes from coarse silt to fine gravel. Only few are based on test with coarser grained material, but even in those cases diameters of about 70 mm were the maximum. Armourstones are much larger, so are the voids, which results in turbulent flow at the interface of filter and armour. No tra-

ditional filter criterion does really cover the boundary conditions that apply to scour and erosion protection measures using armourstones. It was found empirically that some geometrical filter rules can be applied²²⁾.

Hydraulic criteria should be used with caution, since the necessary parameters – hydraulic gradient and/or hydraulic conductivity – are difficult to determine or the assumptions made in the approach may not suit the actual site. Nevertheless, in some cases, hydraulic filter criteria have lead to satisfactory results, but there is still no general valid approach. Also new proposals have been issued: Hoffmans et al.²³⁾ propose a shear stress based approach for further discussion. The simultaneous instability of filter and base is assumed. The approach shows that for flow parallel to the filter layer the relation of the decisive grain diameters following the design of Terzaghi or Cistin/Ziems is much too strict for usual turbulence intensities. The authors assume that with increasing turbulence the shear stress based approach will come closer to the geometrical sandtight filter. The approach does not take into account any other flow than parallel to the filter. So the situation in situ, where groundwater flow and surface flow interact, is not covered. Anyway, the approach shows - similarly to classical filter rules like Cistin/Ziems - an increasing diameter ratio for filter and base when the grain size distribution of the filter material is broader than the base material.

It is an advantage of geotextile filters that they can be used (nearly) irrespective of the void size of the armour. Tests are recommended for geotextile filters in severe applications (e.g. MAG²⁴⁾). For a good estimate, the filter rules of Holtz et al.²⁵⁾ or Lafleur²⁶⁾ can be applied.

9 STABILITY OF PROTECTION LAYER AND SUBSOIL ("GEOTECHNICAL STABILITY")

A distinction between the local and global stability of permeable and impermeable revetments must be made in the geotechnical design of cover layers. The design must ensure local stability for the load case in which excess pore water pressure occurs as a result of rapid drawdown of the water level, and the required mass per unit area of the revetment must be determined. The global stability of the water-side slope must be checked in any case.

Usually Bishop's method is used to calculate slope stability. To introduce the excess pore water pressure it would be wrong to use in the verification the water pressure according to the height of the phreatic line over the point of the circular slip surface considered, as is done by many calculation programs. Instead, the pore water pressure according to the (unsteady and non-linear) flow field has to be taken into consideration.

A finite element deformation calculation allows to estimate a zone with a high probability of sliding due to excess pore water pressure (Figure 9), rather parallel to the surface. This limit state is often termed "local stability" in contrast to the "global stability" of slopes with deep seating (more or less circular) failure mechanisms.

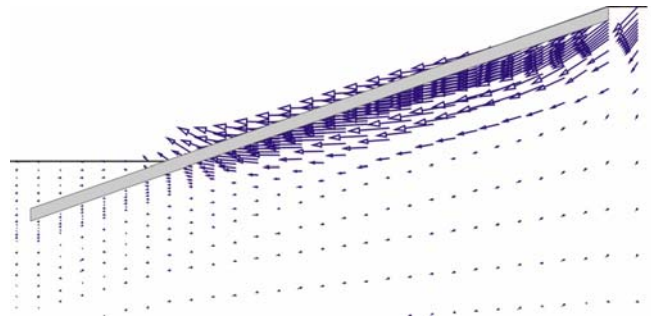


Fig. 9 Deformation vectors indicating slope failure

(1) Local stability of permeable armour layers

To verify the local stability, the required weight per unit area of permeable revetments has to be determined. The rapid drawdown of the water level of a river or canal is always accompanied by excess pore water pressures in the soil close to the surface of the bed and banks of the canal or river as discussed in paragraph 5. In the case of permeable revetments

- sliding may occur along a failure surface in the ground parallel to the slope at a certain critical depth below the revetment as shown on Figure 9 (not relevant for partial grouted riprap) or
- hydrodynamic soil displacement may occur directly below the revetment if the revetment has an insufficient weight per unit area, depending on the degree and velocity of drawdown.

Geotechnical analyses for both types of failure must always be carried out.

Usually, an armour layer has a certain toe support, e.g. toe blanket, embedded toe or sheet pile wall support, that allows for a certain toe support force that may be taken into consideration when verifying local stability. Two failure mechanisms are possible for supported slope revetments (riprap armour):

- The revetment shears off in a horizontal joint passing through the upper edge of the toe of the revetment.
- Failure of the toe of the revetment.

Corresponding approaches are given in GBB¹⁾. Suspended revetments as an alternative to toe support are also discussed.

The internal bond of partially grouted cover layers is sufficient to transfer forces to lateral, unloaded areas. As the extent of dynamic hydraulic actions on cover layers due to passing ships is limited, verification that the cover layer provides safety against sliding and shearing is not required. An embedded toe (toe extension) is required to provide protection against scour in unprotected bottoms.

(2) Local stability of impermeable armour layers

Impermeable revetments must be designed to withstand the maximum excess water pressure if the ground water level may be higher than the lowered water level in the canal. The weight per unit area of impermeable revetments must be great enough to prevent sliding on the slope or uplift at the bed. If the slope lining is capable of transferring longitudinal forces to the toe, the weight must sufficiently high to prevent uplift¹⁾.

10 PLACEMENT OF PROTECTION MATERIAL BELOW THE WATER LEVEL

(1) Granular Material

Any granular material, armourstones as well as granular filters, may be placed by an excavator or by dumping the material from a special pontoon (Figure 10). It is the advantage of dumping that a large amount of fill can be placed in short time. But it is the disadvantage that only narrowly graded grain size distributions may be used, since otherwise there will be segregation while falling through the water. The coarse material will reach the bottom first and the fines will be on top - just the opposite to what is desired.

If only narrow grain size distribution can be used for granular filters, nearly always more than one layer has



Fig. 10 Stone dumping pontoon

to be installed. Each layer needs a minimum thickness to guarantee the necessary filtration length and to avoid bare spots caused by the irregular surface of the subsoil. If there are rather steep slopes of the scour hole, it has to be verified that the filter material will not slide. Often the current velocity disables the placement of a grain filter at all.



Fig. 11 Geotextile filter placement under water

(2) Geotextiles and Mattresses

Special handling equipment and techniques is required to place geotextile fabric, particularly in underwater applications to avoid excessive wrinkling and turning over of the sheet (Figure 11). The density of synthetic fabric often is lower than water, and air bubbles are trapped in between the fibres, that make the geotextile sheet float even if the density is higher than water. The placement depth with such equipment is limited to approximately 20 m. As with grain filters,

any current will hinder a proper placement of a geotextile filter. In mild currents up to ca. 0,8 m/s, a "sandmat" - a sand fill up to 9 kg/m² in between two geosynthetic cloths – can be installed.

Floating complicates also the placement of geosynthetic concrete mattresses because they are filled in place only after installing the geotextile sheets. Other mattresses are assembled in the dry and placed by special cranes. Fascine mattresses are prefabricated in the dry, pulled to the installation position and drowned by dumping riprap upon.

Steep banks of a scour hole require sufficient friction of geotextile and subsoil. Nonwovens exhibit a higher angle of friction than wovens, so the fabric can be chosen according to the local demands.

(3) Geosynthetic containers

Containers are placed by hand, by an excavator, by side dumping vessels or by split barges. Care has to be taken that an area to be protected is covered completely (which does not apply to current training structures, for example). Special equipment allows for very precise placement of containers. When using geosynthetic containers to form a filter layer, the amount of fill should not exceed 80% of the theoretical volume, since tightly filled containers will not adjust themselves to the subsoil, to structures or to the neighbouring geocontainers.

(4) Grouted armourstones

For the placement of grout below the water table special equipment is needed. Only to a water depth less than 1 m grouting can be done by hand. The grout must not segregate when poured or dumped on the riprap. This is achieved by chemical additives or by high speed centrifugal mixing (colloidal mortar). The consistency of the grout should be such that the amount of fill is decreasing from top to bottom. The system and the grout chosen have to prove their ability by tests, since the grout behaves different in the dry and under water and is depending on the water quality. Usually test boxes with a base of at least 1 m² are installed under water, filled with armour layer material and lifted again after being grouted to check the result.

11 DURABILITY

Factors that affect a material's durability include its ability to resist abrasion, chemical attack, biodegradation, wet/dry cycles, freeze/thaw cycles, etc.

Durability of stone depends on the geology of the rock. Sedimentary rock is usually stratified and subject to failure through shear stress, impact, chemical deterioration, or changes in water content. Sedimentary armour stones generally are more easily worn down by abrasion. Any armour stone that develops small cracks may eventually fracture due to freeze/thaw cycles, irrespective of the type of rock. In Europe, the standard EN 13383-1³¹⁾ defines gradings based on stone diameter up to 250 mm and based on weight from 10 to 15000 kg. Physical requirements like density, resistance to wear and breakage, and chemical requirements are listed as well as durability aspects, e.g. resistance to freezing and thawing and to salt crystallization. Corresponding test methods to these requirements are given in the standard EN 13383-2³²⁾. This standard replaces numerous national regulations and can be recommended for worldwide use.

Quartz sand is very durable, but sand mixtures with a high carbonate content from shell material will be more vulnerable to chemical attack if the water is acidic. Also shell particles are not as hard as quartz and are more susceptible to abrasion.

Concrete is considered to be durable while asphalt, according to the Coastal Engineering Manual²⁷⁾ is not considered to be a durable material because it has low strength in both compression and tension, it is subject to chemical reaction, its stiffness changes with temperature, and it is not resistant to impact or abrasion.

Geotextiles and many plastics are generally resistant to chemical and biological attack, but will deteriorate when exposed to ultraviolet radiation and might be degraded by abrasion. The rate of UV-deterioration can be reduced by adding inhibitors. Usually geotextiles are covered with other layers (armour or soil). But there are successful examples of hydraulic structures like groynes and revetments built of geotextile containers without protection^{28) 29)}.

Care must be exercised in placing the overlying stone layers to avoid puncturing the geotextile. The choice of a nonwoven geotextile for geocontainers will minimise the risk of damage during placement due to its high extensibility. By allowing large deformations it will be able to withstand the impact load when hitting the ground as well as when the stones are dumped upon.

Since there is always some space for the armourstones of a protection layer to rock on the geotextile, the fabric must be resistant against abrasion. In Germany, all fabric that is used as filters in waterways has to pass tests to confirm sufficient resistance

against puncturing and abrasion (MAG²⁴), which is tested according to the corresponding guidelines (RPG³⁰), both available in English).

Another important factor providing durability of a scour and erosion countermeasure is flexibility. Flexibility is the property of a material that allows it to bend without breaking. Flexibility can also be used to describe the response of the whole protection structure. For instance, the individual armourstones have no flexibility, but the entire armour layer is capable of movement and settlement to a new position without undue loss of functionality.

Materials and structures with good flexibility will help absorb cyclic and impact loads, but continual flexing might eventually lead to fatigue failure, plastic deformation, and crack formation. The relative flexibility of structures makes them more suitable for withstanding earthquake loads with usually only minor settlement or damage to the armour layers. Monolithic-type structures are less likely to survive seismic loading unscathed. Sufficient flexibility allows a structure to follow ground deformation, e.g. caused by suberosion, internal erosion, piping or other hydrodynamic soil transport in the ground, while rigid structure will be undermined. So it is strongly recommended to use flexible structures for scour and erosion protection measures.

12 SUMMARY

In waterways, special attention has to be paid to the stability of banks to guarantee the necessary channel cross section to allow for easy and safe navigation. In rivers and estuaries often, and always in navigational canals, a bank protection, and sometimes also a bottom protection has to be installed. To design riprap protection, a large number of design equations is available. Much fewer and less approved approaches are available for alternative scour and erosion protection, therefore the design should be always on the safe side. In many applications, the filter design has been neglected, resulting in short lifetime of scour countermeasures. One reason might be the problem of placing a granular or geosynthetic filter under current and wave actions. But placing problems can be overcome with an appropriate effort. If also durability requirements are met, scour and erosion protection in waterways can be accomplished successfully using the materials and equipment available today.

REFERENCES

- 1) GBB: Principles for the design of bank and bottom protection for inland waterways. Mitteilungsblatt der Bundesanstalt für Wasserbau Nr. 88. Karlsruhe: Eigenverlag 2005 (<http://www.baw.de/vip/publikationen/Mitteilungsblaetter31216/index.php?id=88>)
- 2) Hoffmans, G.J.C.M.; Verheij, H.J.: Scour Manual. Rotterdam: Balkema 1997
- 3) Melville, B.W.; Coleman, S.E.: Bridge Scour. Highlands Ranch: Water Resources Publications LLC 2000.
- 4) Lagasse, P.F., Clopper, P.E.; Zevenbergen, L.W.; Girard, L.G.: Countermeasures to protect bridge piers from scour. NCHRP report 593. Washington: Transportation research board 2007
- 5) Koehler, H.J.: The influence of hydraulic head and hydraulic gradient on the filtration process. In Brauns, Heibaum, Schuler (Ed.): Filters in Geotechnical and Hydraulic Engineering. Balkema, Rotterdam, 1993, pp.225-240.
- 6) Schulz, H.; Köhler, H.-J.: A soil mechanical design approach for permeable revetments on inland waterways. ECSMF, Amsterdam 1999
- 7) Bezuijen, A.; Köhler, H.-J.: Filter and revetment design of water imposed embankments induced by wave and draw-down loadings. DeGroot, Den Hoedt, Termaat (Ed.): Geosynthetics: Applications, Design and Construction. Rotterdam: Balkema 1996, pp.1007-1023
- 8) Heibaum, M.; Köhler, H.-J.: Excess pore water pressure and slope stability. In: Proceedings GeoEng2000, Int. Conf. on Geotechnical and Geological Engineering, 20.-24.Nov.2000, Melbourne, Australia
- 9) Seijffert, J.W.: Grass covers and reinforcement measures. In: Pilarczyk, K. (Ed.): Dikes and revetments. Rotterdam: Balkema 1998, pp. 289-302
- 10) Lagasse, P.F.; Zevenbergen, L.W.; Shall, J.D. ; Clopper, P.E.: Bridge scour and stream instability countermeasures. Hydraulic Engineering Circular No. 23 (HEC-23). FHWA NHI-01-003, Washington: Federal Highway administration, 2001
- 11) Lauchlan, C.S.: Pier scour countermeasures. Scholl of Engineering report No.590. University of Auckland, 1999
- 12) Pilarczyk, K. (Ed.): Dikes and revetments. Rotterdam: Balkema 1998
- 13) Van der Meer, J.W.: Conceptual design of rubble mount breakwaters. Delft Hydraulics publication No. 483
- 14) Pilarczyk, K. (Ed.): Geosynthetics and Geosystems in hydraulic an coastal engineering. Rotterdam: Balkema 2000
- 15) Recio, J.A: Hydraulic stability of geotextile sand containers for coastal structures. PhD-Thesis Univ. of Braunschweig, Germany, Faculty of Architecture, Civil Engineering and Environment. 2008
- 16) Maynard, S.T.: Gabion-mattress channel-protection design. Journal of hydraulic engineering Vol. 121 No.7, 1995, pp. 519-522
- 17) Simons, D.B.; Chen, Y.H.; Swenson, L.J.: Hydraulic tests to develop design criteria for the use of Reno mattresses. Report, Colorado state university, Fort Collins, 1984
- 18) Klein Breteler, M.; Bezuijen, A.: Design criteria for placed block revetments. In: Pilarczyk, K. (Ed.): Dikes and revetments. Rotterdam: Balkema 1998, pp. 217-248
- 19) RPV: Guidelines for Testing Cement Bonded and Bituminous Materials for Grouting of Armourstones on Waterways,

- Karlsruhe: Bundesanstalt für Wasserbau (Fed. Waterways Engineering and Research Inst.) Eigenverlag, 2008
- 20) MAV: Code of Practice "Use of Cement Bonded and Bituminous Materials for Grouting of Armourstones on Waterways" Karlsruhe, Germany: BAW - Bundesanstalt für Wasserbau (Fed. Waterways Engineering and Research Inst.) 2008
 - 21) Herpen, J.A. van: Bituminous revetments. In: Pilarczyk, K. (Ed.): Dikes and revetments. Rotterdam: Balkema 1998, pp. 249-288
 - 22) Heibaum, M.: Geotechnical filters - the important link in scour protection In: Chiew, Y.-M.; Lim, S.-Y.; Cheng, N.-S.: 2nd International Conference on Scour and Erosion, November 14-17, 2004, Singapore: Nanyang Technology Univ, Maritime Research Centre; pp.13-28
 - 23) Hoffmans, G.J.C.M; Adel, H.den; Verheij, H.J: Shear stress concept in granular filters. In: Briaud, J.L.: Scour of Foundations. Proc. Int. Symp. 19. Nov. 200, Melbourne, Australia, pp.372-379
 - 24) MAG: Code of Practice "Use of Geotextile Filters on Waterways (MAG)". Karlsruhe, Germany: BAW - Bundesanstalt für Wasserbau (Fed. Waterways Engineering and Research Inst.), 1993.
(<http://www.baw.de/vip/publikationen/merkblaetter.php.html>)
 - 25) Holtz, R.D.; Christopher, B.R.; Berg, R.R.: Geosynthetic Engineering. Richmond: BiTech Publishers Ltd. 1997
 - 26) Lafleur, J.; Eichenauer, T.; Werner, G.: Geotextile filter retention criteria for well graded cohesionless soils. In: Lafleur, J; Rollin, A.L. (Eds.): Geofilters'96 – Proceedings. Montreal: Ecole Polytechnique 1996, pp. 429-438
 - 27) U.S. Army Corps of Engineers. Coastal Engineering Manual. Engineer Manual 1110-2-1100, Washington: U.S. Army Corps of Engineers, 2002.
 - 28) Hornsey, W.P.; Heerten, G.; Jackson, L.A., Restall, S.J.: Case study showing the growth and development of geotextile sand containers: an Australian perspective. In: Geotextiles and Geomembranes Vol. 20, 2002, pp. 231-342
 - 29) Heibaum, M.; Oberhagemann, K.; Faisal, M.A.; Haque, S.: Geotextile bags for sole permanent bank protection. Proc. 4. European Conf. on Geosynthetics, Edinburgh, 2008
 - 30) RPG: Guidelines for Testing Geotextile Filters in Navigable Waterways (RPG). Karlsruhe, Germany: BAW -Federal Waterways Engineering and Research Institute, 1993. (<http://www.baw.de/vip/publikationen/sonderinfo/richtlinie.php.html>)
 - 31) EN 13383-1. Armourstone – part 1: specification. European standard 2002
 - 32) EN 13383-2. Armourstone – part 2: test methods. European standard 2001