- **regional *in situ* stress regime** – faulting and folding suggests quarried blocks will tend to split
- **weathering grade** – geologically weathered rock decomposes faster in service
- **groundwater conditions** – water flowing or seeping from the quarry walls suggests weathered seams
- **discontinuities** – *in situ* block sizes, stone shapes and integrity
- **production methods** – non-blasting methods generate fewer internal cracks than aggregates blasts
- **set-aside** – stones cured by storing for several months before selection will rarely split
- **shape as seen in stockpiles** – mean blockiness and aspect ratio
- **armourstone integrity as seen in stockpiles** – proportion of stones with visible flaws after known set-aside period
- **sampling** – to obtain representative material for laboratory tests
- **block integrity testing** – full-scale destructive testing.

Rock samples are tested in a laboratory and results interpreted for the site conditions. This will allow an informed prediction of the service life of armourstone to be made (see Section 3.6), based on knowledge of rock mechanics and weathering properties of the various rock types during engineering service conditions. The expected pattern and rate of degradation of the stones should then be considered in design, in addition to damage caused by storms. With an estimate of quarry yields, a more inclusive local scenario-based design can provide better whole-life costing outcomes and the materials specification can be written accordingly. The effort will be in proportion to the project scale and risk. In general, sources that yield large blocks will have satisfactory physical and weathering resistance properties, but this is not always the case. Furthermore, sources are inherently variable, so rock quality testing is necessary.

The systems approach to quality evaluation can also work within the framework of EU or other statutory or policy constraints, provided the potential stone sources are known prior to design. A design based on selected armourstone category test requirements (eg for physical, mechanical and resistance to weathering properties) without investment in evaluating the quarry and making a service life prediction, is possible but may not be optimal. In the EU, evaluation of armourstone from suppliers is simplified by the provision of certified test results and production control documentation. This will help the quality of armourstone sources to be assessed as nominally “excellent”, “good” or “marginal”, on the basis of hand-sized specimens and aggregate-sized test material. Producers with significant supplies of armour-sized gradings for sale may also declare certain test results, so designers can consider “marginal” and even “poor” property materials in appropriate circumstances. In many cases, supplementary full-scale integrity testing of armour stones (Dupray *et al* (2004), see Section 3.8.5) will greatly increase confidence in assessing the relative suitability of several nearby sources.

In practice, the evaluation of the two aspects, namely size and quality, is often carried out simultaneously and can interact with the design process and decision-making in many ways, as illustrated in Figure 3.7. Note that Step 3 is not applicable if there are no stocks of armourstone available at the quarry. In this case, trial blasting may be required. Alternatively, if blasts are performed for other applications such as aggregate production, sorting may allow selection of suitable material to provide the information.

Box 3.10 *In-service degradation model for general wear of armourstone: illustrative example of two methods (contd)*

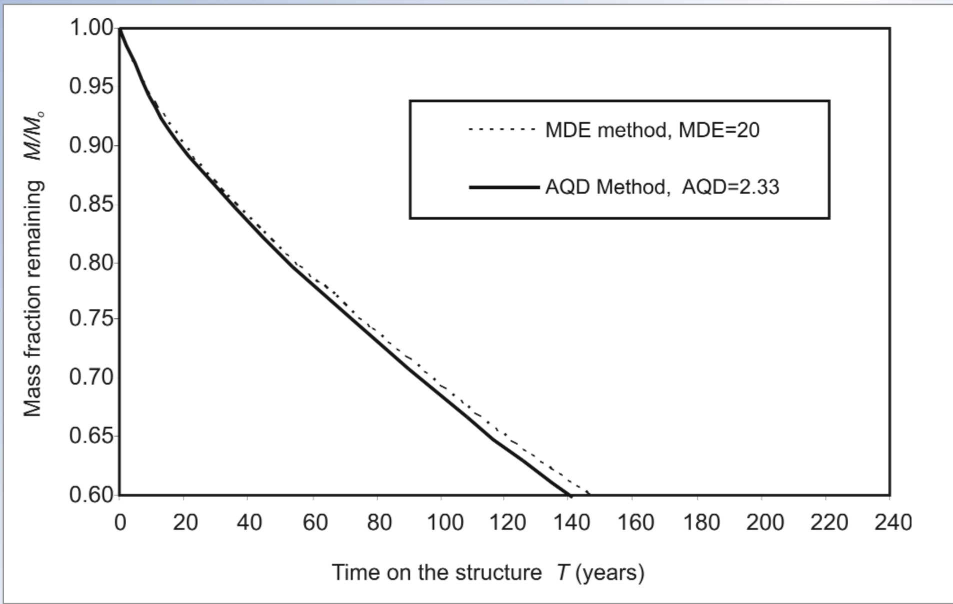

Figure 3.32 *Service life predictions for the general in-service degradation model, using MDE and AQD methods. The very close agreement shown is somewhat fortuitous and not likely to be reproduced in practice (courtesy J van Meulen)*

3.6.6 Modelling degradation due to breakages

Minor and major breakages affect the mass distributions and also, to some extent, the shape of armourstone. For example, during the survey of a contract using a 6–10 t grading of armourstone, Laan (1992) observed that degradation related to transport and handling led to a production of pieces smaller than 3 t whose mass represented 9 per cent of the original material. In addition, he observed that the M_{50} decreased from 8.5 t to 7.6 t. In a different situation, 1–3 t armourstone gradings, from different sources and exposed to different levels of quality control, were exposed to repeated routine handling events associated with stockpiling and loading. Dupray *et al* (2004) observed in each case a mass of small fragments, say smaller than 100 kg, totalling 5–8 per cent of the initial consignment and that the initial M_{50} decreased by 14–21 per cent in certain cases.

This section discusses the effects of minor and major breakage on mass distribution, their quantitative contribution to mass distribution changes, and how to assess these changes from test results.

3.6.6.1 *Effects of minor breakages and major breakage*

Minor and major breakages have different effects on mass distribution of armourstone.

Minor breakage produces small fragments originating from breakage of stone edges or crushing of armourstone corners (see Figure 3.11). It has a limited effect on the values of D_{n50} or M_{50} compared with the effect of major breakage. Minor breakage modifies the mass distribution in the sense that the fragments appear in the form of a tail and a vertical shift of the lightest part of the grading curve, as shown on Figure 3.33. The amount of fines generated is expressed by the parameter F_o (%).

Fragments resulting from minor breakage during transport or handling or structural loadings may be removed by further selection or by wave or current action. In this latter case

Table 3.40 *Quarry inspection sheet for quality control by the client (contd)*

Production of the quarry

Average global production of the quarry [________] thousand t per year

Is grading pre-selection performed? [Y/N]

Standard coarse gradings (mm)	$CP_{45/125}$	$CP_{63/80}$	$CP_{90/250}$	$CP_{45/180}$	$CP_{90/180}$
Ratio of the production (%)					
Available in stock (thousand t)					
Standard light gradings (kg)	LM_{5-40}	LM_{10-60}	LM_{40-200}	LM_{60-300}	LM_{15-300}
Average mass controlled [Y/N]					
Quarry yield (%)					
Available in stock (thousand t)					
Standard heavy gradings (tonne)	$HM_{0.3-1}$	HM_{1-3}	HM_{3-6}	HM_{6-10}	HM_{10-15}
Average mass controlled [Y/N]					
Quarry yield (%)					
Available in stock (thousand t)					

Other gradings ____

____

Production control

visual [_] bulk weighing on weighbridge [_] individual weighing [_] mechanical sorting [_]

other: ____

Quality of the production in the stocks

Integrity Good [_] Acceptable [_] Marginal [_] [___] % (by number) of blocks with major breakage

Resistance to minor breakage: Good [_] Acceptable [_] Marginal [_]

Shape: Equant [_] Tabular [_] Elongated [_] [___] % (by number) of blocks out of spec

Durability: weathering [Y/N] Signs of freeze-thaw damage: [Y/N] Signs of Sonnenbrand: [Y/N]

Other:____

Identification

Date: [_______]

Inspectors: Name [________________________] Function: [________________________]

Name [________________________] Function: [________________________]

Quarry rep: Name [________________________] Function: [________________________]

Name [________________________] Function: [________________________]

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Discharge relationships and velocities

In the case of short-crested dams – and in the other direction assuming an infinitely long dam perpendicular to the mean current direction – a set of conventional discharge relations can be used to find the **specific discharge**, q (m³/s per m).

- **Vertical closure method**

Originally, the relationships given by the Equations 5.84 to 5.86 were applied to weirs, which can be considered as an early construction stage during a vertical closure:

$$q = \mu h_b \sqrt{2g(H - h_b)} \qquad \textbf{subcritical flow} \qquad (5.84)$$

$$q = \mu\, 2/3 \sqrt{2/3(gH^3)} \qquad \textbf{supercritical flow} \qquad (5.85)$$

$$q = \sqrt{C'} \sqrt{h_3^3\, 2g\left((h_1/h_3)^3 - 1\right)} \qquad \textbf{through-flow} \qquad (5.86)$$

where:

H	=	upstream water level above dam crest level (m)
h_b	=	downstream water level relative to dam crest (m)
μ	=	discharge coefficient (-); see separate sub-section later in this section and Table 5.15
h_1	=	upstream water depth (m)
h_3	=	downstream water depth (m)
C'	=	resistance factor (a specific type of discharge coefficient) (-).

NOTE: The values of h_1 and h_3 must be measured relative to the original bed for a vertical closure (see Figure 5.21) and relative to the sill for a combined closure (see Figure 5.24).

For **through-flow** the resistance factor C' is written in terms of a through-flow resistance coefficient, C (-), and the effective length, L_s (m), of the structure in flow direction. L_s can be determined with Equation 5.87:

$$L_s = B + \left(2d - 0.67(h_1 + h_3)\right)\cot\alpha \qquad (5.87)$$

which is then used to calculate the resistance factor, C' (-), according to Equation 5.88:

$$C' = 1/3\, \frac{n_v^5\, D_{n50}}{C\, L_s} \qquad (5.88)$$

where n_v is the porosity of the rockfill (-); D_{n50} is the median nominal size of the armourstone (m); and C is the through-flow resistance coefficient (-), where $C = f(Re)$, the average value and range of which is included in Table 5.15 – lower row. For definition of other terms, see Figure 5.22.

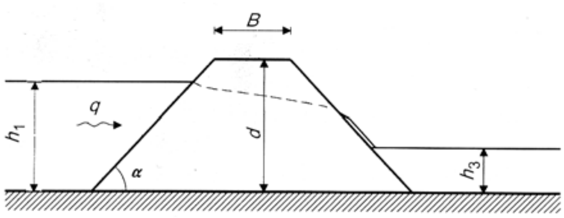

Figure 5.22 *Definition sketch for flow through a dam*

bed. In Table 5.53 formulae are presented for a fully developed velocity profile and a non-developed profile, Equations 5.221 and 5.222, respectively.

Table 5.53 *Design guidance for parameters in the Pilarczyk design formula (Equation 5.219)*

Characteristic size, D	• armourstone and rip-rap: $D = D_{n50} \cong 0.84 D_{50}$ (m) • box gabions and gabion mattresses: D = thickness of element (m) **NOTE:** The armourstone size is also determined by the need to have at least two layers of armourstone inside the gabion.
Relative buoyant density, Δ	• rip-rap and armourstone: $\Delta = \rho_r/\rho_w - 1$ • box gabions and gabion mattresses: $\Delta = (1 - n_v)(\rho_r/\rho_w - 1)$ where n_v = layer porosity ≅ 0.4 (-), ρ_r = apparent mass density of rock (kg/m³) and ρ_w = mass density of water (kg/m³)
Mobility parameter, ψ_{cr}	• rip-rap and armourstone: $\psi_{cr} = 0.035$ • box gabions and gabion mattresses: $\psi_{cr} = 0.070$ • rock fill in gabions: $\psi_{cr} < 0.100$
Stability factor, ϕ_{sc}	• exposed edges of gabions/stone mattresses: $\phi_{sc} = 1.0$ • exposed edges of rip-rap and armourstone: $\phi_{sc} = 1.5$ • continuous rock protection: $\phi_{sc} = 0.75$ • interlocked blocks and cabled blockmats: $\phi_{sc} = 0.5$
Turbulence factor, k_t	• normal turbulence level: $k_t^2 = 1.0$ • non-uniform flow, increased turbulence in outer bends: $k_t^2 = 1.5$ • non-uniform flow, sharp outer bends: $k_t^2 = 2.0$ • non-uniform flow, special cases: $k_t^2 > 2$ (see Equation 5.226)
Velocity profile factor, k_h	• **fully developed logarithmic velocity profile:** $k_h = 2/\left(\log^2(1+12h/k_s)\right)$ (5.221) where h = water depth (m) and k_s = roughness height (m); k_s = 1 to $3D_{n50}$ for rip-rap and armourstone; for shallow rough flow ($h/D < 5$), $k_h \cong 1$ can be applied • **not fully developed velocity profile:** $k_h = (1+h/D)^{-0.2}$ (5.222)
Side slope factor, k_{sl}	The side slope factor is defined as the product of two terms: a side slope term, k_d, and a longitudinal slope term, k_l: $k_{sl} = k_d\, k_l$ where $k_d = (1 - (\sin^2\alpha/\sin^2\phi))^{0.5}$ and $k_l = \sin(\phi - \beta)/(\sin\phi)$; α is the side slope angle (°), ϕ is the angle of repose of the armourstone (°) and β is the slope angle in the longitudinal direction (°), see also Section 5.2.1.3.

Escarameia and May

Escarameia and May (1992) suggested an equation that is a form of the Izbash equation (see Section 5.2.1.4) in which the effects of the turbulence of the flow are fully quantified. This can be particularly useful in situations where the levels of turbulence are higher than normal (see Section 4.3.2.5): near river training structures, around bridge piers, cofferdams and caissons, downstream of hydraulic structures (gates, weirs, spillways, culverts), at variations in bed level, at abrupt changes in flow direction. This Equation 5.223 gives the relationship between the median armourstone size, D_{n50} (m), and the hydraulic and structural parameters; and it provides an envelope to the experimental data that were used to derive it and is valid for flat beds and slopes not steeper than 1V:2H. The laboratory data were further checked against field measurements of turbulence in the River Thames with water depths between 1 m and 4 m.

$$D_{n50} = c_T \frac{u_b^2}{2g\Delta} \tag{5.223}$$

where c_T is the turbulence coefficient (-) and u_b is the near-bed velocity, defined at 10 per cent of the water depth above the bed (m/s).

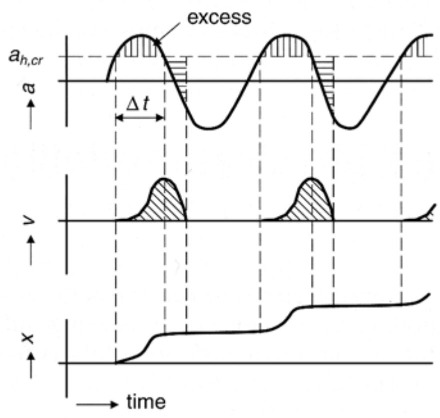

Figure 5.132

Integration of time history of accelerations in order to find displacements

An example of the assessment of the residual displacements is given in Box 5.31.

Box 5.31 *Evaluation of displacements resulting from an earthquake*

The procedure discussed above for the assessment of the residual displacements (see Figure 5.131) is demonstrated for an earthquake with the following basic characteristics:

- number of excitations: N_e = 15 (sinusoidal cycles)
- period of excitation: T = 0.5 s
- peak acceleration: a_h/g = 0.25 or a_h = 2.5 m/s².

The duration of the earthquake, T_e (s), following from $T_e = N_e T$, amounts to: T_e = 7.5 s. Further, the state of (relative) excess pore pressures is in this example characterised by p^* = 50 per cent. This pressure level is assumed constant during the period of T_e = 7.5 s. These conditions may correspond, for example, to an earthquake magnitude of M = 7 (Richter scale) or slightly higher.

The results are presented in Table 5.64. The data in the second column have been derived from Table 5.63 by interpolation with regard to a_h. The resulting residual displacements Δx are directed downward along the slope.

Table 5.64 *Residual displacement, Δx, for a range of example structure slopes (φ' = 35°, p^* = 50%) after an earthquake characterised by: a_h/g = 0.25, T = 0.5 s, N_e = 15*

Slope tanα (-)	Relative threshold acceleration $a_{h,cr}/g$ (-)	Effective acceleration time ($a_h > a_{h,cr}$) Δt (s)	Residual displacement Δx (m)
1:3	0	0.25	1.7
1:4	0.075	0.20	0.7
1:5	0.125	0.17	0.4
1:7	0.185	0.12	0.1

The results indicate that the total residual displacements along the slopes considered in Table 5.64 will be rather limited as long as the pore water pressure level is 50 per cent or less. As a consequence of the assumptions made during the analysis, the presented displacements are even conservative. Referring to Table 5.63, at the end of the earthquake, when a_h = 0, it holds that $F_{min} \geq 1$ for slopes not steeper than 1:3. This means that the displacement will reach its maximum directly after the shaking has stopped.

Finally, it should be emphasised that in this assessment made, the main uncertainty is the pore pressure percentage, p^* (-) that may be generated and should be used as a parameter in the analysis. In the case of fine, loosely packed sand, the pore pressure percentage may easily exceed 50 per cent during an earthquake characterised by M = 7, with a_h/g = 0.25. A special aspect of the behaviour of sand under cyclic loading is that the pore pressure response becomes very sensitive for more load cycles once p^* has reached a level of 50 per cent. This means that complete liquefaction may then rather easily occur.

With a 1:3 slope, for excess pore pressures considerably exceeding p^* = 50 per cent, the safety factor F_{min} < 1 at the end of the earthquake; a condition that will last until the pore pressure has been dissipated below the critical value associated with F_{min} = 1. It will be clear that, due to additional deformations following the earthquake (as a kind of indirect response), the resulting residual displacement might be much larger than the primary response given in Table 5.64. In the worst case, a complete failure or flow slide takes place.

Box 5.33 *One-phase 2D vertical modelling of water motion in a rubble structure*

In the Netherlands, a model has been developed, MBREAK/ODIFLOCS that describes the 2D vertical water motion in a rubble structure under wave attack including turbulence, inertia, unsteadiness and water depth effects (see Section 5.4.5). The boundary conditions are wave run-up and wave pressures on the slope, which have to be determined by experiments. The program calculates the phreatic water table by a finite-difference (FDM) scheme, and then the porous flow field by a FEM scheme. The result is a pressure and a velocity field under a varying water table inside the structure. Important aspects are the location of intense flow and the significant set-up of the internal water table. The model can be used for wave transmission analysis. Figure 5.138 presents calculated phreatic surfaces at different times.

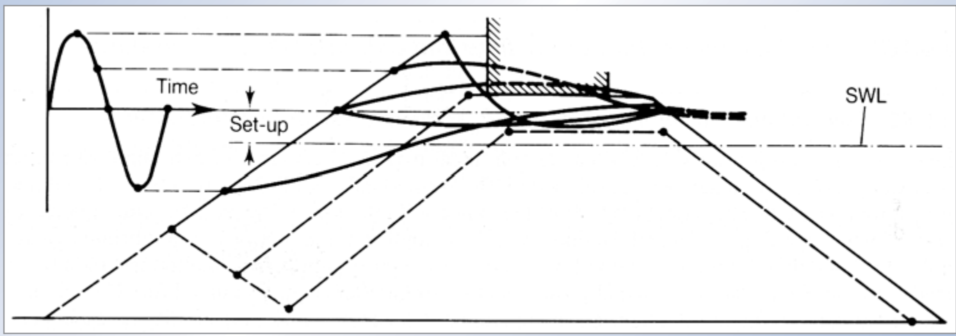

Figure 5.138 *Phreatic surfaces under wave action*

5.4.4 Geotechnical properties of soil and rock

5.4.4.1 *General*

Application of the principles of Sections 5.4.2 and 5.4.3.1 to geotechnical design requires:

- a reliable description of the soils, rocks and rockfills, and other materials of the project (this section)
- a precise description of the actions
- a representative geotechnical model to quantify the limit states, including adequate methods for analysing the stability and deformations of the soil and structures, such as calculation methods, simplified models, rules based on experience (see Section 5.4.3).

5.4.4.2 *Correspondences and differences between soil and rock*

The so-called *properties* of soils, rock and rockfill are not often a direct description of their structure and behaviour but part of a model, which is limited to some of their features. Most models have been validated by experience but unexpected events may still happen because of the differences between nature and the commonly used models. Geotechnical insight into the conception of structures and projects is therefore highly advisable.

All soils and rocks are geological materials with different positions in the transformation cycle of the Earth crust. Soils are loose particulate materials, which become denser with time, whereas rocks are continuous stiff materials, which are progressively fractured, eroded, dissolved and transformed into soils. The properties of the soils and rocks may vary within wide limits (up to a factor of $10 \cdot 10^9$) and it is very important to correctly identify those of the soils and rocks existing at the site of the project. Consequently, the knowledge of the geological history of the site or the region may help in the definition of reliable soil and rock properties.

Soil mechanics make a strong distinction between fine and coarse soils, with a limit dimension of particles at 60–63 or 80 μm, depending on the local standards. Fine soils have smaller particles (down to 1 μm for colloidal clays), with smaller voids between the particles but very large variations of the total volume of voids between the loosest (soft) and the

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Rivers are dynamic entities in which the hydraulic loadings are constantly changing and the channel boundaries vary with time. The design of a revetment cross-section, which is the basis of all the river training works described in this manual, should consider the very dynamic environment in which the designed works are constructed. Design constraints are listed below and are discussed in this section:

- scour
- river morphology
- hydrology and flow regulation
- wind generated waves
- local currents and turbulence
- water level changes due to tides and wind
- ship-induced currents and waves
- ice loads
- geotechnical boundary conditions.

Scour

River training works should be designed to resist scour, in particular erosion of the bed adjacent to the river training structure. Scour can be localised, general or a combination of both. Different terms are used to describe the various forms of scour such as *bend scour, constriction scour*, etc. The use of these terms can be inconsistent and the designer should carefully check the physical phenomenon behind the wording. The designer is recommended to adopt the terms used in a particular reference document such as *Manual on scour at bridges and other hydraulic structures* (May *et al*, 2002) or *Scour manual* (Hoffmans and Verheij, 1997).

The expected scour near the structure during construction and during service is one of the most important aspects to consider during design. Most failures of river training structures result from an underestimation of the depth of scour. Joint occurrence of local scour and critical morphological conditions should be considered.

To account for scour and morphological changes, the designer has three options for the toe depth of an envisaged structure (see Section 8.2.7.3 and Section 5.2.3.3):

- a **toe constructed sufficiently deep**, at or below the anticipated maximum scour level
- a **toe above the maximum anticipated scour level**, but with a flexible toe protection that can respond to scour and thereby protect the revetment from being undermined. This is best achieved with dumped armourstone. Alternatives such as gabion and fascine mattresses are less flexible and may not be able to respond to local scour. However they can be used where scour is **expected to be moderate** and/or without localised deep scour holes
- a **falling apron** may also be used instead of a flexible mattress (see Section 8.2.7.4).

River morphology

River training works have an impact on river morphology but are also affected by morphological changes of the river. A well designed revetment should resist the forces of erosion during service of the structure, provided appropriate maintenance is undertaken. However river training works are localised and rarely attempt to constrain the whole cross-section of the river or channel. Movements of the channel bed and banks may continue in non-protected areas. Bed and bank movements are more important near the boundaries of the training works, such as the toe of a revetment.

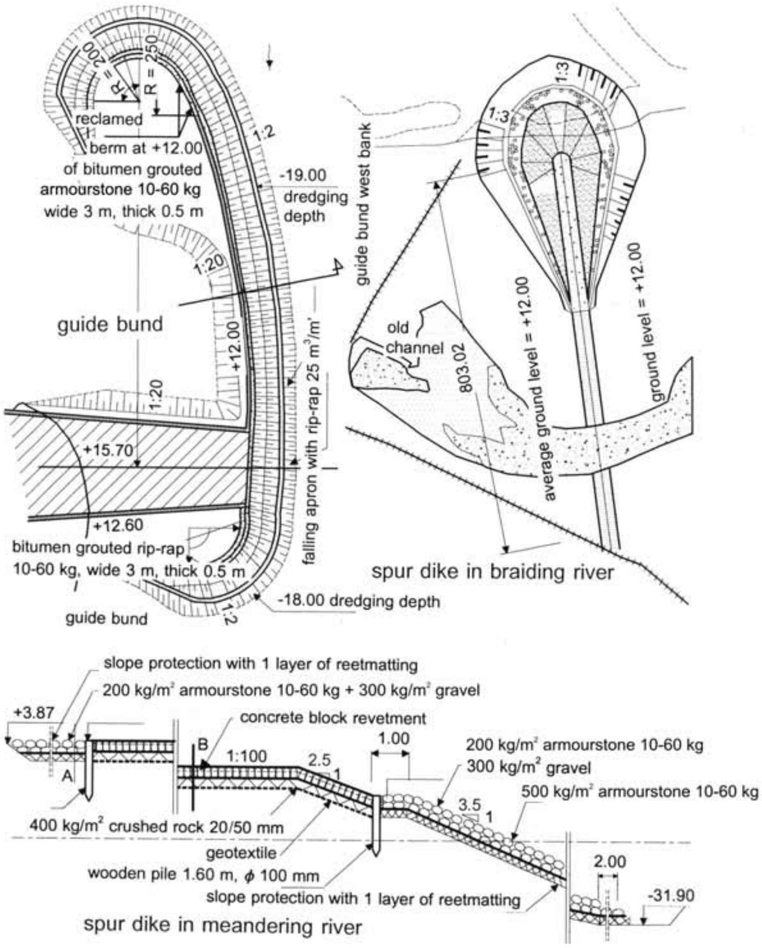

Figure 8.25 *Examples of heads of spur-dikes and end of guide bunds*

8.2.7.2 *Crest level and width of spur and longitudinal dikes*

The crest level of spur-dikes and longitudinal dikes with a function of channel stabilisation or constriction in meandering rivers is, apart from economic reasons, determined by navigation requirements, flood discharge factors and construction practicalities, eg placing stones on the revetment crest in the dry. This implies that the crest should be dry at normal (ie non-flood) water levels that might be expected during the construction period. The highest level is determined by the flood plain level, as at high river levels current concentration and erosion behind the structures should be avoided. The crests of spur-dikes may slope towards the river, typically 1:100 to 1:200.

The crest level of guide bunds as used in bridge projects is often much higher. Such guide bunds should keep the flow away from bridge abutments and bridge approaches and should not be subject to overflow. The height of the crest can be determined by the design water level for the whole project. Overtopping by waves may be acceptable and freeboard in this case is only required as a safeguard to unexpected settlements and to cater for inaccuracies in water level calculations.

8.2.7.3 *Stability of revetment toes*

River training works may be exposed to various types of scour, depending on the nature of the river and the type and location of the structure: local, general, constriction, confluence, bend or protrusion scour (see Section 8.2.6.1 and Section 5.2.3). Not all of these types of scour will develop at any particular structure, nor do they have the same magnitude. A complicating factor is that to some extent the types of scour are inter-dependent or partly correlated. More detailed guidance on scour and engineering works to counter it can be found in Hoffmans and Verheij (1997), May *et al* (2002) and Sumer and Fredsoe (2002).

As soon as all of the calculations for the joint scour and its consequences have been verified and are to an acceptable probability of exceedance, the designer should decide what counter-measures are to be taken. There are three different solutions for the problem of scour at the toe:

Case 1: No significant scour – no need for protection. The revetment has its toe at the meeting point between the slope and the riverbed level and no appreciable scour, ie scour that endangers the stability of the revetment, is expected.

Case 2: Significant scour – bed protection provided to resist scour. The revetment has its toe at the meeting point between the slope and the riverbed level but appreciable scour is expected and appropriate protection measures should be taken on the bed.

Case 3: Significant scour – toe of revetment is extended into the bed in anticipation of future scour: the revetment toe is placed in a trench, excavated in the riverbed, flood plain or foreshore at the time of construction, to form a falling apron.

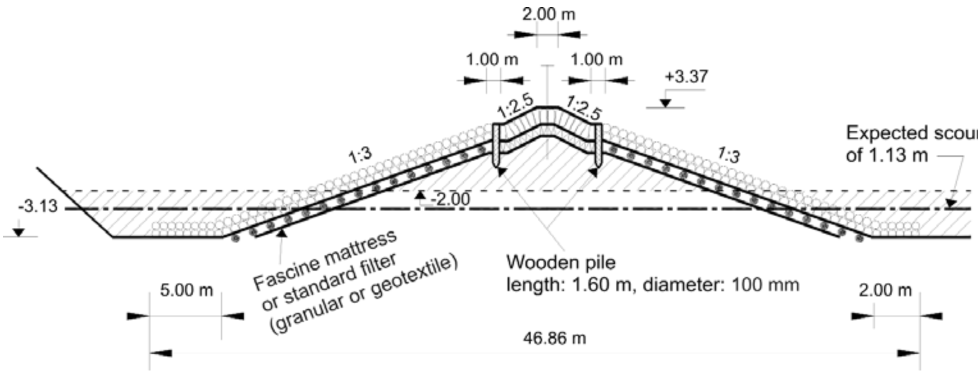

Figure 8.26 *Toe of spur-dike showing provision for scour*

No appreciable scour can generally be found along inner bends of meandering rivers and along the stems of spur-dikes. Extension of the revetment cover layer over a few metres on the horizontal riverbed is usually sufficient. In many cases, this horizontal protection is already provided by the edge of the fascine mattress or the filter layer (see Figure 8.26). When there is a risk of erosion of soil through the cover layer, extension of the filter layer should be investigated.

When there is a risk of appreciable scour or if it is expected in front of the structure, suitable measures should be taken. The designer should start by assessing the future scour depth (Hoffmans and Verheij, 1997 and May *et al*, 2002). Depending on the outcome and the local circumstances, the designer should decide if the situation is case 2 or case 3 as defined above or a combination of both. In case 2, a falling apron may be recommended (see Section 8.2.7.4). In case 3, the revetment may be extended downward in an excavated trench (see Figure 8.27). In Figure 8.27, the lower part of the revetment and the falling apron have all been placed under water. When the geotextile filter is placed under water, fascines may be added to help the placing (see Section 9.7.1.2).

Box 8.7 *Calculation of discharge in a V-shaped weir with vertical slot using armourstone*

The discharge relation of this type of fishway is given by Equations 8.4 and 8.5 (WL|Delft Hydraulics, 1998) for two different water depth situations (see Figure 8.50 for parameter definitions):

for $h_1 \geq 1.25\ H_b$:

$$Q = C_{SI}\ \mu_I\ (\frac{4}{5})^{\frac{5}{2}} \sqrt{\frac{g}{2}}\ \tan(\frac{\theta_1}{2})\ (h_1 - H_b)^{2.5} + \\ + C_{SII}\ \mu_{II}\ \frac{2}{3} (\frac{2}{3} g)^{0.5}\ 2H_b\ \tan(\frac{\theta_2}{2})\ (h_1 - \frac{1}{2} H_b)^{1.5} + \\ + 0.8\ b_{vs}\ P\ \sqrt{2g\ (h_1 - h_2)} \tag{8.4}$$

for $h_1 < 1.25\ H_b$:

$$Q = C_{SII}\ \mu_{II}\ (\frac{4}{5})^{\frac{5}{2}} \sqrt{\frac{g}{2}}\ \tan(\frac{\theta_2}{2})\ (h_1)^{2.5} + 0.8\ b_{vs}\ P\ \sqrt{2g\ (h_1 - h_2)} \tag{8.5}$$

where:

μ = discharge coefficient depending on the upstream energy head above the apex, the crest width and the geometry of the crest (-); for this type of weir: $\mu_I \approx 1.1$ and $\mu_{II} \approx 0.6$

C_S = correction factor for subcritical flow depending on the value h_1/h_2 (-); for this type of weir: $0.75 < C_S < 1$

H_b = height of the kink relative to the apex (m)

h_1 = upstream water level relative to the apex level (m)

h_2 = downstream water level relative to the apex level (m)

P = height of the vertical slot (m)

Q = discharge (m^3/s)

θ = opening angle of the V-shape (°) = $\tan(\theta_i/2) = n_i$; for this type of weir $n_1 = 7$ and $n_2 = 3$

b_{vs} = width of the vertical slot (m)

8.5.3 Scour protection of bridge piers

One of the main causes of bridge collapse is scour which can undermine the foundations of bridge piers. To easily avoid this problem, it is recommended to construct the foundations of the bridge deeper than the maximum anticipated scour depth, wherever practical. However, there are circumstances where this option is not viable, due to the costs associated with creating deep foundations in difficult conditions. In this case, the provision of some form of scour protection can provide an acceptable alternative approach.

Scour in rivers can occur as a result of a number of processes, introduced here (see also Section 8.2.6.1). For a more comprehensive reference on scour and ways of protecting against it, the reader is referred to the *Manual on scour at bridges and other hydraulic structures* (May *et al*, 2002) and/or the *Scour Manual* (Hoffmans and Verheij 1997).

- **natural scour:** This includes general lowering of the bed as part of a long-term or seasonal response to flow conditions. It also includes scour due to channel migration, ie where a deep water channel moves laterally towards bridge piers with shallower foundations. Natural scour also includes bend scour, the tendency for deeper bed levels on the outside of a bend
- **contraction or constriction scour:** This results from confining the width of a channel and thereby accelerating the flow, eg as a result of constructing bridge piers
- **local scour:** Caused by an obstruction in the flow, such as a bridge pier or a spur head.

These aforementioned scour processes can occur simultaneously, resulting in greater scour depths than would result from any process alone.

- reduce
- transfer
- share
- insure
- accept.

Each of these options is discussed in Simm and Cruickshank (1998), with the optimum control strategy depending on the risk, the ability to manage it and the organisations involved.

Clients budgeting their works, and contractors looking to control their costs, should make estimates of any additional costs and/or time they may incur on coastal, river and estuary engineering construction projects that are attributable to risk factors. Many methods of estimating project budgets are available. The simplest involves preparing risk registers in which each risk is represented by a single probability and the cost of the consequences. More complex methods can extend this principle to a systematic description of all risks based on separate identifiable consequences: minimum, most likely, maximum. When the number and/or interaction between risk elements are such that a hand calculation would be difficult or time-consuming, use can be made of the Monte Carlo analysis technique for calculating a risk distribution from a given set of risk elements. See Vrijling (2001) and Schiereck (2001) for further guidance.

9.5.3.2 *Protecting the operative*

Health and safety provisions

In addition to the health and safety issues discussed in Section 2.6, a few typical items are discussed here. Coastal and fluvial construction sites are often situated in remote parts of the world where few welfare facilities exist. In such areas few or no records may be kept of lost time due to sickness and general poor health and absenteeism caused by unsociable hours and poor working conditions. It is therefore essential to decide how:

- best to provide basic welfare and cleaning facilities for the operatives
- to protect against disease and contamination risks, eg contaminated dredged material, Weil's disease
- tidal working or unsociable hours might affect operatives' health
- to manage overall site health and safety
- to ensure the safety of any operations by lone workers in remote areas
- to avoid fatigue and stress and to manage mitigation measures.

Specific health and safety provisions (Cruickshank and Cork, 2005)

- Platforms and gangways
- ladders
- site tidiness
- illumination
- weather conditions
- first aid equipment
- protective clothing and equipment including personal buoyancy equipment
- visibility of other personnel by operators of large equipment
- means of access: water transport
- access over partially completed structures

1 2 3 4 5 6 7 8 9 10

The use of wheel loaders to place stone in bulk is limited to gradings up to 300 kg, ie for the placement of core material, and in some cases for the secondary layers. Wheel loaders with buckets tend to scoop up surface material when digging into a stockpile, which may result in contamination. If the bucket is replaced with forks, larger stones can be handled individually without contamination.

Excavators

All excavators (see Figure 9.14) should have heavy-duty, waterproofed undercarriages, which will improve their life. Biodegradable oil should be used whenever possible in the hydraulic systems of excavators working in pollution-sensitive environments, so that problems do not arise if a hydraulic hose breaks. It is important that all the excavators carry oil spill kits to mitigate the effects of leaks of engine oil or diesel. Plant refuelling should take place in a compound away from the beach or riverbank that is equipped with bunded tanks and quick-release hoses. Long-reach equipment (see Figure 9.22) is often used to extend the period of tidal working, but this reduces the excavator's capacity, necessitating the use of larger machines.

Figure 9.14 *Excavator working on the crest (courtesy J D Simm)*

Table 9.4 relates the minimum excavator mass to the various stone gradings.

Table 9.4 *Excavator mass in relation to stone mass*

Armourstone grading	Excavator mass for handling (t)
Core material	15
1–3 t	20
3–6 t	30
6–10 t	45
10–15 t	60
15–20 t	70

Notes:

1 The tabulated data refer to operations with 360 degrees excavators on a horizontal floor, viz quarry handling; in such situations the tabulated gradings are valid for reaches up to 9 m.

2 When placing stones in rock structures, ie on slopes, the lifting capacity is substantially smaller than the above data and should be determined by using load charts according to the specifications of the manufacturer.