INTERNATIONAL INSTITUTE FOR HYDRAULIC AND ENVIRONMENTAL ENGINEERING

DELFT NETHERLANDS

A. Zanen

Re-edition 1981

revetments

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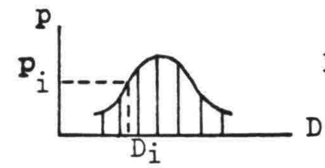
CONTENTS

LIST OF SYMBOLS

$\bar{u}$	mean flow velocity (m/s)
τ	shear stress (N/m^2)
S	tractive force (N)
g	acceleration due to gravity (m/s^2)
h	depth (m)
I	slope (-)
D_m	characteristic diameter (m)
D_{15}	15% of weight of a sample is finer
p	percentage on sieve (%)
ρ	density (kg/m^3)
ρ_w	density of water
ρ_s	density of material
Δ	relative density (-) $\quad \Delta = \dfrac{\rho_s - \rho_w}{\rho_w}$
C	Chezy roughness coefficient ($m/s^{1/2}$)
α, β, f	coefficients
W	mass (kg mass)
K	slope factor
α	side slope of bank (degrees)
φ	angle of repose (degrees)
R	resultant force (N)
n	$\dfrac{\text{voids}}{\text{total volume}}$
ε	voids ratio $\quad \varepsilon = \dfrac{n}{1-n}$
H	wave height (m)
L	wave length (m)
γ	specific weight $\quad (\gamma = \rho \cdot g)$
γ_m	specific weight of material
γ_w	specific weight of water



$$D_m = \frac{\Sigma p_i . D_i}{100}$$

1. Introduction

1.1 General

A rather important aspect of hydraulic engineering is the concern for the equilibrium and the stability, and eventually the fixation, of the boundary between the media soil, air and water.
For instance, at the boundary between air and water waves may be formed which will break as soon as they reach shallow waters. The surf so formed may give rise to erosion problems.
At the boundary between air and soil erosion or sedimentation due to wind action may occur. Both may be harmful and necessitate protective measures, such as, promoting the development of vegetation cover (direct protection) or of hedges and rows of trees to decrease the wind force (indirect protection). Erosion and sedimentation due to wind may also have useful effects in that they contribute to the formation of sand dunes which form natural levees along many coasts. Here, the civil engineer sometimes has to interfere with natural processes in order to promote an equilibrium between the sand-feeding beach and the dune vegetation withholding sand.
The subject of revetments is concerned with the boundary between water and soil. Erosion and sedimentation, which are sign of a disturbed equilibrium, may be common and harmless phenomena. The uneven surface of the earth is gradually leveled by mechanical and chemical forces, lowering mountains, filling up lakes and marshlands, forming deltas, etc. This process may be retarded by the influence of a vegetation cover; and often a state of equilibrium will be approached. However, such equilibrium may be disturbed, either by natural causes or by human interference. Then accelerated erosion may start, not only resulting in the loss of fertile land but also causing sedimentation in other areas, blocking rivers and canals and covering fertile riparian lands with sand. It is the task of the engineer and the agronomist to control this type of erosion in order to obtain equilibrium again. In addition, the civil engineer may contribute by designing and constructing protective works, in the mountains which are known as "head water control works" (Wildbachverbauung).

In civil engineering there are many well-known examples of disturbance of equilibrium. Especially in natural alluvial rivers the process of meandering, braiding, formation of new channels, sedimentation of existing channels, eroding and erecting banks, forming sand banks and islands are excellent examples of both erosion and sedimentation. Similar processes occur in tidal rivers, in estuaries and along the coasts. They may also occur in man-made water courses like irrigation and navigation canals, around bridge piers, breakwater and downstream of weirs.

The theory of these processes is dealt with in the lectures on silting and scour, sediment transportation, river engineering, coastal engineering, etc. Equilibrium is always connected with the supply of solid material to a certain area and the transport of that material from the same area.

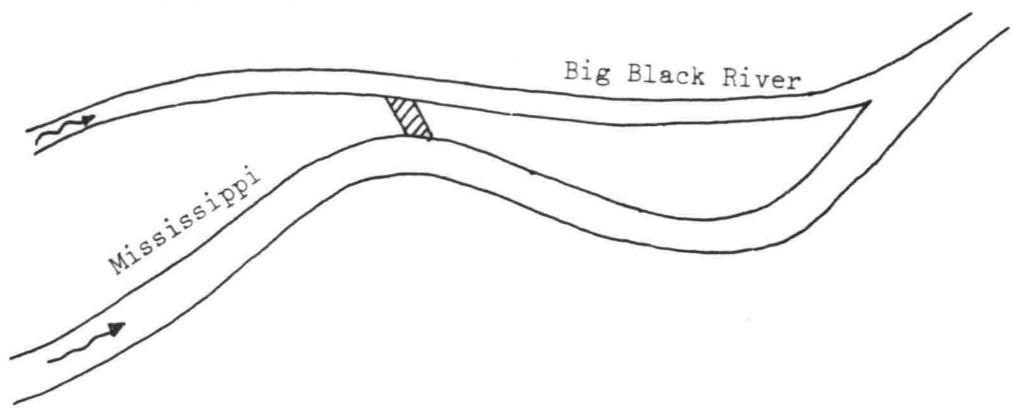
As soon as the supply to and the transport from a particular area becomes unbalanced, the equilibrium is disturbed and either erosion or sedimentation will occur. It is obvious that human interference in such processes will only be considered in cases where human interests are endangered or where benefits can be obtained for human society. Eroding forces are not only the currents turbulence and wave action, but also the forces related to ground-water flow.

In case no measures can be taken to provide balance between the supply and the transport capacity of sediments and if the transport capacity dominates, erosion is liable to occur. Such erosion can be prevented by fixation of the soil by revetment. These lectures will deal with the application, design and construction of revetments.

1.2 Making use of erosion or sedimentation

Before tackling the problems related to revetments, it may be interesting to show in a few examples that sometimes deliberate use can be made of erosion or sedimentation to reach certain ends.

The first example pertains to the Mississippi River:



In 1929 there was a dike break about 4.5 km upstream of the confluence of the Big Black River and the Mississippi. After some time the Big Black River completely took advantage of this short cut to the Mississippi. After two seasons the newly formed course was navigable. This prompted the idea that if many of the bends were eliminated then the high head water could move along more quickly, thus decreasing the chance of flooding. In a period of 8 years, 16 bends were cut off which shortened the river by about 250 km. The principle followed was that a small pilot channel was dredged through which the water could follow a shorter course. Nature did the rest.
At the so-called Diamond cut-off, about 9.5 millions m^3 were dredged and stored in the old river arm. The rest (approximately 40 millions m^3) was removed by erosion. Here the so-called "agitation dredging" was also applied. According to this method the bed material is merely worked loose so that it can be easily transported by the current. Such a method, however, does have its disadvantages. The 40 millions m^3 of sand may cause sedimentation somewhere downstream. For navigation, the advantage is the shortening of the travelling distance and easier manoevring along bends. A disadvantage may be the increase of the flow velocity hindering the upstream traffic.

This cheap but unrefined method can be applied to rivers with high discharge and with not too steep slopes and where the depth of the river causes no problems for navigation.
A totally different example is the use of the eroding power of flowing water to maintain a channel needed to release excess water. Before a few sea-arms along the Netherlands coast were closed, (Deltaplan) they consisted of deep channels which came into existence due to the fact that, at every tide, great quantities of water were flowing in and out. The larger the basin to be filled, the stronger the current and the larger the cross-sectional area of the channels. After such a sea-arm is closed, the situation is totally changed. The currents in and out do not exist any longer and the deep channels may be sanded up by the sand moving along the coast resulting from surf and tidal currents. Generally such morphological changes can be accepted, but such changes should be prevented as much as possible in the Haringvliet estuary. That is why the outlet sluices in the closure dam have a width of approximately 1 000 m. In periods of high discharge of the rivers, this "tap" is set open to release surplus water. If the channels at the seaside of the dam would be sanded-up, the outflowing water could experience too much resistance. As a consequence the capacity of the outlet sluices would decrease. It is the policy to apply a sluicing programme such that shoaling in the channels is prevented as much as possible by short but powerful outlet currents during low tide. If this proves to be insufficient, then additional aid can be given by dredging or pumping. However, the most important work must be done by the erosive force during sluicing.
A common example of making use of sedimentation is the purposeful inundation of marshy land in order to improve its condition for possible future use for human needs.

1.3 Application of revetments to prevent erosion

As has been mentioned, the application of revetments should only be considered in cases where:

i) human interests are endangered by erosion processes, and
ii) no possibility is available to influence the supply of sediment to a certain place or area such that supply and transport balance each other.

A number of such cases are mentioned below as examples:

Rivers:
In natural alluvial rivers it may be necessary to prevent further meandering at a certain place because civil engineering works are threatened (abutments of bridges, inlet sluices, pumping stations, irrigation- and road systems) or even villages and towns located close to the river. In such cases either fixed points will be created or a continuous protection over a certain length of the river bank. Such revetments will have to protect the entire bank of the bed of the river and usually even part of the river bed to prevent undermining and collapse of the entire revetment.
Also regulation- and normalization works require local protective measures in the form of revetments.

Canals:
In navigation canals current and waves are usually determined by ships travelling along the canal. Erosion presents a particular danger where, due to the combined action of waves and gravity, sediment particles are activated. Since this phenomenon does not usually occur at any great depth, protection of the slopes can be confined to a limited depth below the water surface.
In irrigation canals or canals conducting water towards a water power station, a choice has to be made between two solutions. The cross-sectional area of the canal can be designed so that the current remains below the critical value for moving the sediment particles. Consequently, no protection of the canals, bed and banks is needed.
The other solution is to restrict the cross-sectional area of the canal and to revet the bed and banks. Among other factors, the comparison of cost estimates of both solutions may determine

the ultimate choice of the solution.
In navigation canals, which also serve water management purposes, the current may also be taken into account for the design of the revetment. Based on economic considerations either the dimensions of the canal can be designed so that the current remains below its critical value and only a restricted part of the bank needs protection, or the dimensions of the canal are designed only to meet navigation requirements and eventually revetting of bed and banks is accepted.
It should be mentioned that revetments serve other purposes in addition to that of protection. Lined irrigation canals can be made water tight in order to prevent the loss of irrigation water by ground-water flow. In water power canals a smooth revetment of bed and banks may decrease the roughness, thereby preventing loss of energy.
Around and especially downstream of a bridge pier, the decelerating flow with high turbulence and eddies endangers the stability of the pier. Here protection is always needed. Around the end of a groyne, a breakwater or a harbour mole, the same danger exists. In the case of weirs, usually the water head will cause the water to pass the weir as supercritical flow. The transition to subcritical flow occurs under loss of energy with a hydraulic jump. This means that the downstream river bed and banks may be subject to strong turbulent flow. The occurrence of erosion too close to the weir should be prevented in order to guarantee its stability.
A special application of a temporary bed protection is shown in the case of the closure of a channel subject to tidal effects. During the operation one will be confronted with increasing flow velocities in the remaining open part of the closure dam. Such parts of the bed warrant that careful protective measures are taken in order to prevent severe erosion during the execution of the work. This is one of the major problems to be solved in the framework of the Netherlands' delta works.

2. Basic assumptions and theoretical considerations

2.1 Basic assumptions

It is obvious that an interrelationship exists between the cross-sectional area of a canal (for navigation as well as for transport of water) and hydraulic phenomena such as current, waves, etc. In the design stage several alternative cross sections may be considered; for each of these cross sections a comparative study should be made of the extent and construction of the revetment.

It is assumed here that such a study has led to a solution and that the magnitude of the various hydraulic phenomena are known so that only the design of the protective construction (revetment) has to be worked out.

If the revetments are designed for river works, it is also assumed that data concerning current and waves are known and are available for the definite design.

Strictly speaking, the design of the side slope of banks belongs to the discipline of soil mechanics. However, special attention is paid to the influence of outflowing ground water on the stability of the slope. This is felt to be necessary since this aspect is often underestimated and has led to disappointments.

2.2 Basic theoretical considerations

2.2.1 Resistance against current

For proper judgement whether the unprotected soil can resist a certain current or what should be the specifications (weight or size) of the stone component of a revetment, it is necessary to know the relation between size (and density) of the material and either the flow velocity of the current or the shear stress exerted on the bed by the current. For that purpose many investigations have been made.

Distinction is made between non-cohesive and cohesive soils.

a. Non-cohesive soils

In the formulae for the flow velocity usually the mean flow velocity $\overline{u}$ in the vertical is used.

The shear stress is $\tau = \rho_w . g. h. I$

From a great number of experiments Lane found:

$$\tau_{critical} = 700 \, D_m \quad (\tau \text{ in N/m}^2, \; D_m \text{ in m})$$

D_m is the characteristic diameter.
That means that a condition of equilibrium is:

$$\rho_w . g. h. I \leqslant 700 \, D_m \quad \text{or} \quad \frac{h. I}{\Delta . D_m} \leqslant 0.043$$

Shields found $\quad \dfrac{h. I}{\Delta. D_m} \leqslant 0.03$

Taking the average of both:

$$\frac{h. I}{\Delta . D_m} \leqslant 0.036, \text{ one finds } D_m \geqslant \frac{28}{\Delta} . h. I$$

Expressed in flow velocity (using Chézy's equation), one finds

$$D_m \geqslant \frac{28}{\Delta} . \frac{\bar{u}^2}{C^2}$$

Very often D_m is related to the velocity head $\dfrac{\bar{u}^2}{2g}$;

then:

$$D_m \geqslant \frac{\alpha}{\Delta} . \frac{\bar{u}^2}{2g} \text{ , in which } \alpha = \frac{560}{C^2} .$$

In this case the Chézy coefficient C (roughness) has to be estimated, which may not be easy. Also u_{max} may exceed $\bar{u}$ considerably due to fluctuations.
Often for the coefficient α a safe value of 0.50 to 0.80 is taken.

Another approach is based on the concept of flow pressure and the forces acting from one particle on the other.
By using theories which are valid for cube-form and spheres, Isbash found for mixtures and shapes of particles common in nature:

$$D_m \geqslant \frac{0.7}{\Delta} . \frac{\bar{u}_1^2}{2g} \text{ , in which } \bar{u}_1 \text{ stands for}$$

the mean flow velocity for a depth h = 1 m.
For depths larger than 1 m a correction factor is to be introduced.

Assuming a distribution of the flow velocity in the vertical obeying a sixth grade parabola:

$$\bar{u}_h = \bar{u}_1 \cdot h^{1/6}$$

The correction factor, $h^{1/6}$, can be read from Figure 2.1

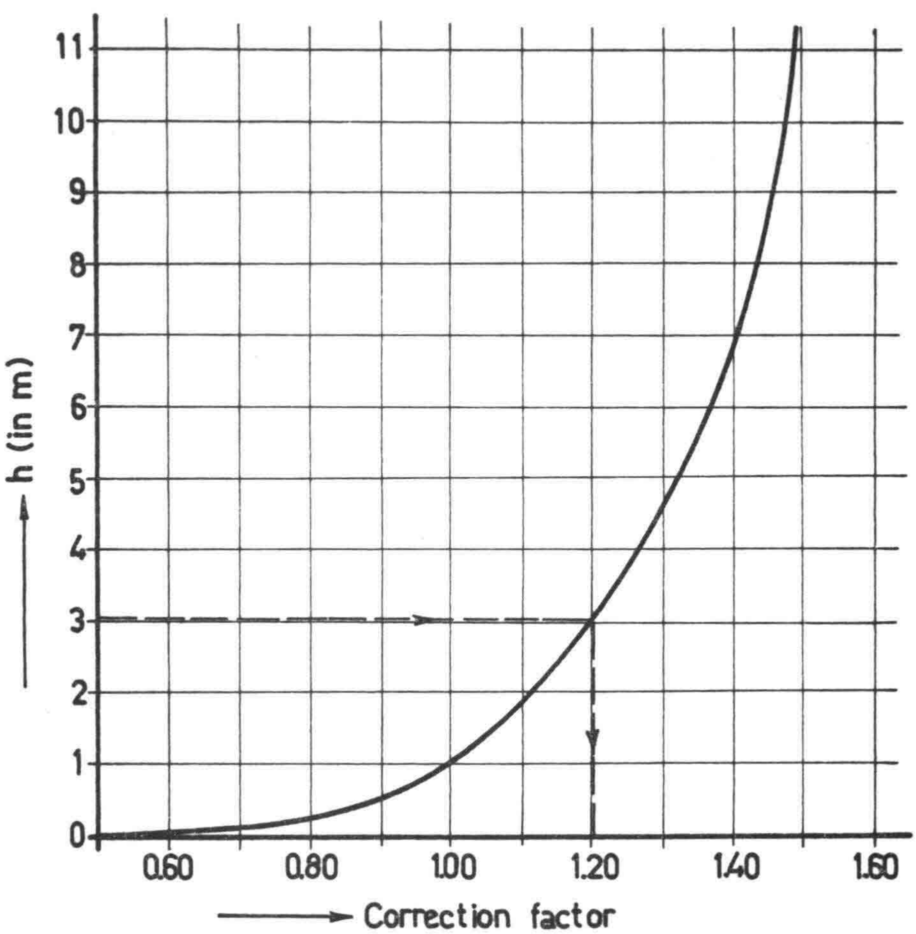


Figure 2.1 Correction factor $h^{1/6}$

In investigations conducted by the U. S. Bureau of Reclamation the turbulence of the flow was included.

They found: minor turbulence $D_m \geqslant \frac{0.2}{\Delta} \cdot \frac{\bar{u}^2}{2g}$

normal turbulence $D_m \geqslant \frac{0.5}{\Delta} \cdot \frac{\bar{u}^2}{2g}$

major turbulence $D_m \geqslant \frac{1.4}{\Delta} \cdot \frac{\bar{u}^2}{2g}$

From the above we see that in the equation

$$D_m \geqslant \frac{\alpha}{\Delta} \cdot \frac{\bar{u}^2}{2g}$$

for the factor α is found:

Lane/Shields 0.3 to 0.5

Isbash 0.7/ (corr. factor)^2 (see figure)

for instance:

$$h = 3 \text{ m} \qquad \alpha = \frac{0.7}{(1.2)^2} \simeq 0.5$$

U. S. Bur. of Reclamation 0.2 to 0.5 to 1.4, depending on the turbulence.

In Table 2.1 the values of ρ_s and Δ are given for materials of various nature:

	ρ_s	$\Delta = \frac{\rho_s - \rho_w}{\rho_w}$
sand, gravel	2650	1.65
concrete	2200	1.2
reinforced concrete	2400	1.4
asphalt concrete	2300 to 2400	1.3 to 1.4
mine stone	2200 to 2500	1.2 to 1.5
granite	2500 to 3100	1.5 to 2.1
basalt	2700 to 3200	1.7 to 2.2
masonry	1750 to 2150	0.75 to 1.15
copper slags	2750	1.75
lead slags	3000 to 4000	2 to 3

Table 2.1 Values of relative density Δ for various materials.

As the size of stone is usually expressed in mass (kg) and not in diameter, conversion is necessary from one unit into the other. Assuming that generally the mass of a stone(not being a cube nor a sphere) can be expressed as $W = 0.5 \rho_s D_m^3$, from Figure 2.2 the mass of a single stone can be read as a function of D_m and Δ (representing influence of density).

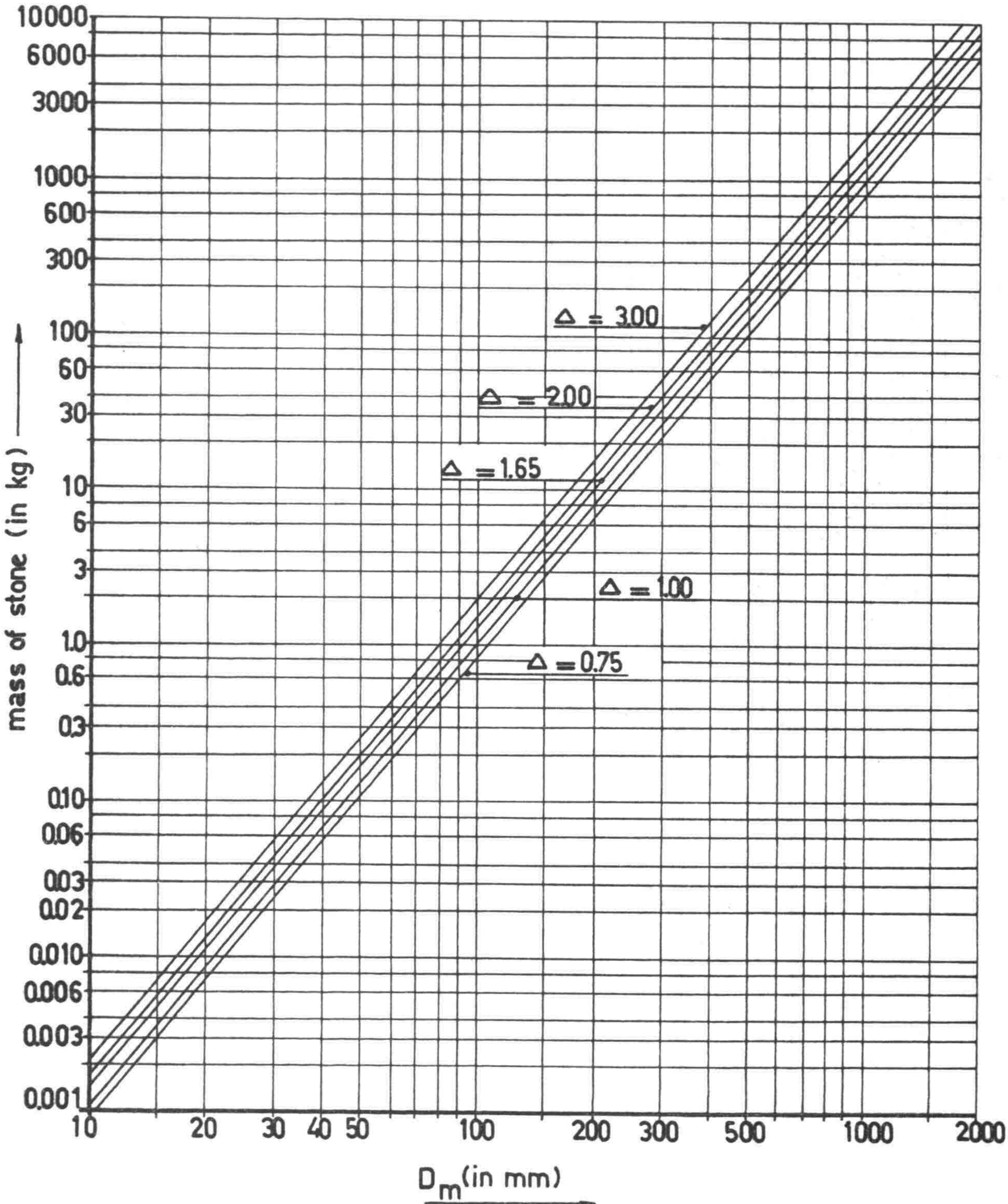


Figure 2.2 Conversion from diameter to weight (mass)

Considering the above theory one should realize that the mass W of the stone, which is required to resist a certain current, is proportional to the 6^{th} power of $\bar{u}$. In other words, an increase of the flow velocity from 1 to 2 m/s requires stones weighing 64 times as much!

The above theory holds for particles of sand, gravel or stone on a horizontal bed. Particles on the surface of a slope (side slope of a bank) are also subject to the influence of gravity(in addition). Such particles will start moving sooner than particles on a horizontal bed. This, of course, has to be taken into account by introducing the so-called slope factor K.
K is the ratio of tractive force required to move a particle on the slope to that required to move the same particle on a horizontal bed. ($K < 1$) !!
Before considering the determination of the slope factor more in detail, a remark should be made on the distribution of the shear stress along the slope.
According to the definition $\tau = \rho . g. h. I$ the shear stress at a point (i), equals $\tau_i = \rho . g. h_i . I$, which leads to a distribution of shearing stresses along the slope as shown in Figure 2. 3; zero at the water surface and maximum ($\rho . g. h. I$) near the bed (point A), as is shown in diagram a of the figure.

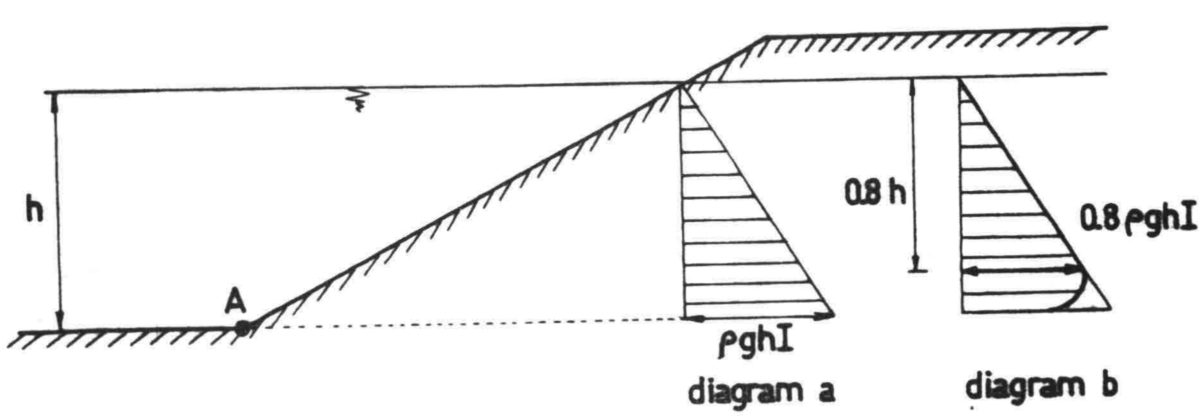


Figure 2. 3 Distribution of shear stress.

In practice the real shear stress deviates from the theoretical one due to the fact that the distribution of the flow velocity is not uniform over the cross section of the water course because of the wall roughness.
From experience it is known that the true distribution of the shear stress is well approximated, as indicated in diagram b of the figure.
In the point of the slope at 0.8 h below the water surface the shear stress reaches its maximum, which is approximately $0.8\ \rho \cdot g \cdot h \cdot I$.

Determination of the factor K
In Figure 2.4 (a cross section and a front view) the forces are indicated acting on a particle on the slope.

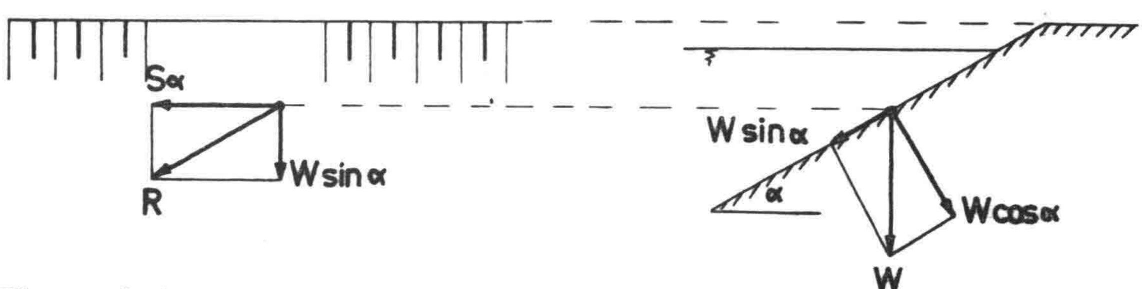


Figure 2.4 Diagram of forces

Along the surface of the slope we have the shear force S_α (due to the current) and the component $W. \sin\alpha$ of the gravitational force W (weight).
The resultant force $R = \sqrt{S_\alpha^2 + W^2 \sin^2\alpha}$
Normal to the slope the component $W. \cos\alpha$ acts.
A condition of equilibrium is:

$W \cos\alpha . \tan\varphi \geqslant R$ in which φ is the angle of repose (internal friction) of the soil.

or:

$$W^2(\cos^2\alpha \, . \tan^2\varphi - \sin^2\alpha) \geqslant S_\alpha^2$$

$$S_\alpha \leqslant W\sqrt{\cos^2\alpha \, . \tan^2\varphi - \sin^2\alpha}$$

$$S_{bed} = S_\alpha \text{ for } \alpha = o = W \tan\varphi$$

By definition: $$K = \frac{S_\alpha}{S_{bed}} = \sqrt{\frac{\cos^2\alpha \tan^2\varphi - \sin^2\alpha}{\tan^2_\varphi}} =$$

$$= \sqrt{\frac{\cos^2\alpha \sin^2\varphi - \sin^2\alpha \cos^2\varphi}{\sin^2_\varphi}} =$$

$$= \sqrt{\frac{\cos^2\alpha \sin^2\varphi + \sin^2\alpha . \sin^2\varphi - \sin^2\alpha}{\sin^2_\varphi}} =$$

$$= \sqrt{\frac{\sin^2\varphi - \sin^2\alpha}{\sin^2_\varphi}} = K = \sqrt{1 - \frac{\sin^2\alpha}{\sin^2_\varphi}}$$

If $\alpha > 0^\circ$: $K < 1$

The diagram shown in Figure 2.5 enables determination of the slope factor K as a function of the side slope α and the angle of repose φ .

For instance: α = 2 (horizontal) to 1 (vertical) | From diagram:
φ = 30^o | K = 0.43

From Lane's equation, which gives a direct relation between the shear stress and the particle diameter, we can easily find that the diameter needed on the slope is $0.8 \times \frac{1}{0.43} = 1.86$ x as large as for a particle on the horizontal bed.
(0.8 because of the max. shear stress on the slope being 0.8 x shear stress on bed; $\frac{1}{0.43}$ due to the slope factor for the given values of α and φ).

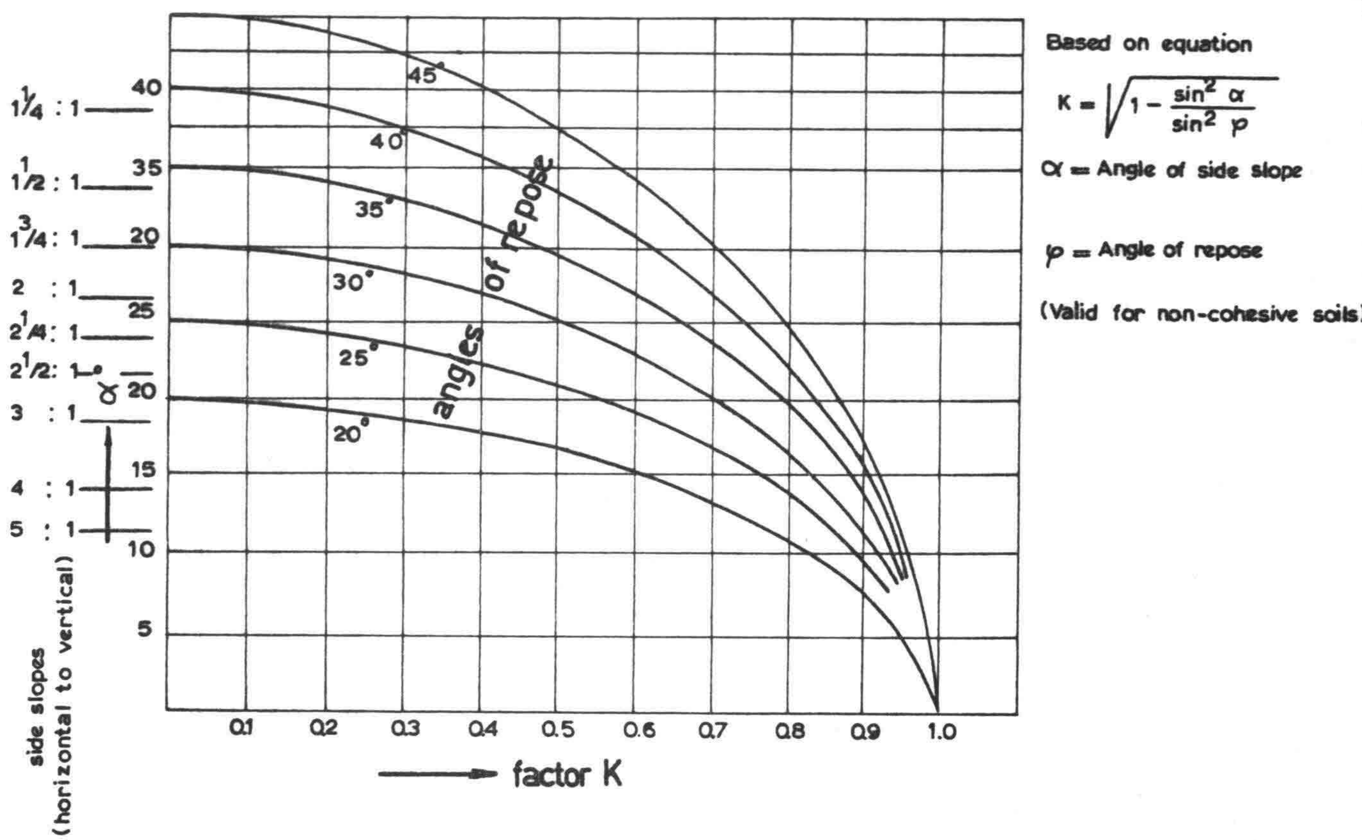


Figure 2.5 Slope factor K as f(α, β)

When using an equation like $D_m = \frac{\alpha}{\Delta} \cdot \frac{\bar{u}^2}{2g}$ (valid for horizontal bed), we find:

$$D_{m(slope)} = \frac{\alpha}{\Delta} \cdot \frac{0.8\,\bar{u}^2}{2g} \cdot \frac{1}{K},$$ which leads to

the same result.

Remark (and warning!):

In literature sometimes a slope factor is used to apply on the mean flow velocity $\bar{u}$. In such cases K is defined as

$$\sqrt[4]{1 - \frac{\sin^2 \alpha}{\sin^2 \varphi}}$$

As has been mentioned before the mass W of the stone required to resist a certain current is proportional with the 6^{th} power of the mean flow velocity $\bar{u}$.

It is obvious that the determination of $\bar{u}$ in the vertical considered should be done with maximum accuracy. If such accurate determination appears to be difficult, it is necessary to make a safe estimate.

To this end Lane made some indicative suggestions, as is shown in Table 2. 2

degree of sinuosity	percentage of critical shear stress required to initiate movement as compared with straight canals	corresponding percentage of mean velocity $\bar{u}$
straight canal	100	100
slightly sinuous river	90	95
moderately sinuous river	75	87
very sinuous river	60	78

Table 2. 2 Influence of sinuosity.

It means, for instance, that in a "very sinuous" river the diameter of the stone should be $\frac{1}{0.6} \simeq 1.7$ x as large, and the weight $(1.7)^3 = 4.6$ x as much, compared with a straight canal.

b. Cohesive soils

For cohesive soils no theoretical approach exists yet to determine the critical flow velocity.
One has to use empirical data obtained from experiments, usually done in laboratory flumes.
Table 2. 3 and 2. 4 show the results of experiments done by Etcheverry and by Fortier and Scobey respectively.

Material	Mean flow velocity (m/s)
Sandy loam	0. 75 to 0. 83
Average loam, alluvial soil	0. 83 to 0. 90
Firm loam, clay loam	0. 90 to 1. 13
Stiff clay soil	1. 20 to 1. 50

Values are valid for a depth of 3 ft(0. 90 m)

Table 2. 3 According to experiments by Etcheverry.

Material	Mean flow velocity (m/s)
Silt loam, noncolloidal	0. 60
Alluvial silt, noncolloidal	0. 60
Ordinary firm loam	0. 75
Stiff clay, very colloidal	1. 12
Alluvial silts, colloidal	1. 12

Values are valid for a depth of 3 ft(0. 90 m)

Table 2. 4 According to Fortier and Scobey.

A difficulty is that the nature of the soil is not expressed in objective parameters.
Table 2. 5 shows the results of experiments performed in U. S. S. R.

Compactness of bed material :	Loose	Fairly compact	Compact	Very compact
Voids ratio:	2.0 to 1.2	1.2 to 0.6	0.6 to 0.3	0.3 to 0.2
Material	Critical mean flow velocity (m/s)			
Heavy clayey soils	0.40	0.85	1.25	1.70
Clays	0.35	0.80	1.20	1.65
Lean clayey soils	0.32	0.70	1.05	1.35

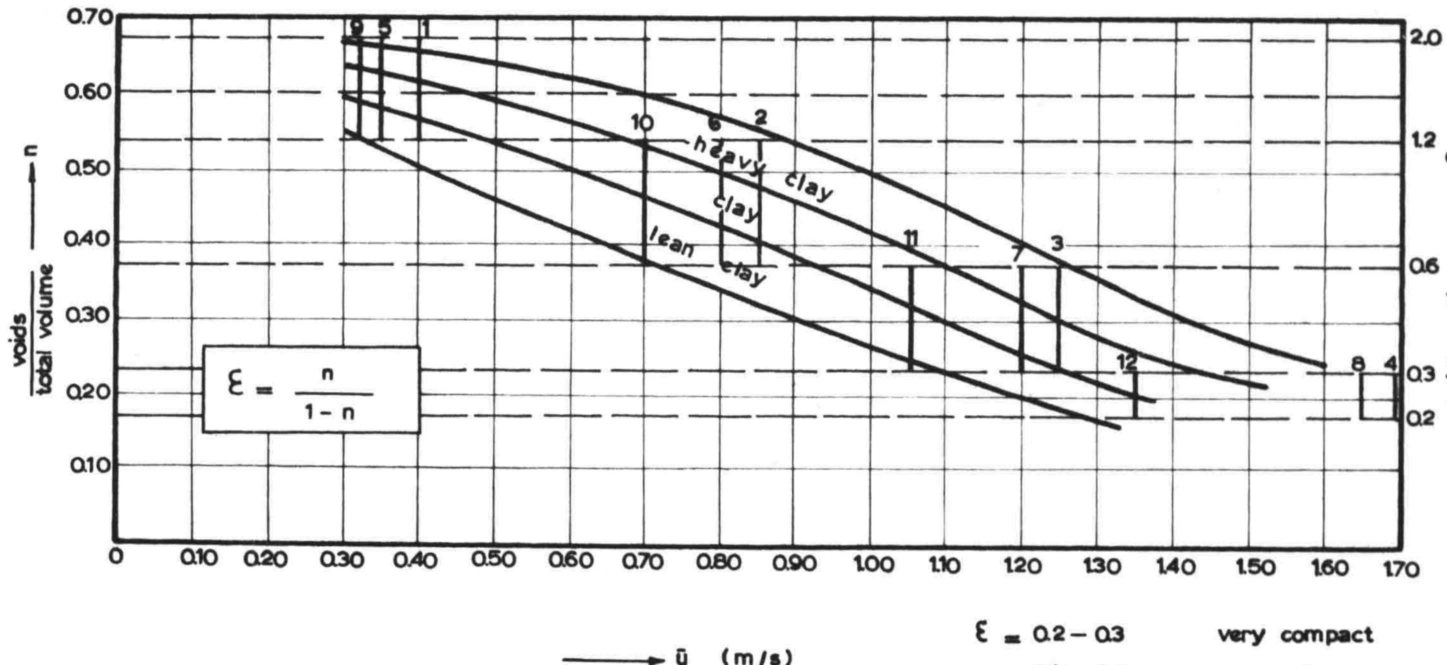
Table 2.5 Russian-experiments.

Values are valid for a depth of 1 m.

Here the same difficulties arise; although, in this case at least the compactness of the soil is defined more precisely in the parameter "voids ratio".

The remaining subjective element is that one has to decide whether the material under consideration belongs to one of the categories "heavy clay", "clay" or "lean clay".

In Figure 2.6 an attempt is made to indicate the critical flow velocity $\bar{u}$ as a function either of $\underline{n}$ $\left(\frac{\text{voids}}{\text{total volume}}\right)$ or of ε (voids ratio).
The relation between both parameters is : $\varepsilon = \frac{n}{1-n}$.



ε =	0.2 – 0.3	very compact
	0.3 – 0.6	compact
	0.6 – 1.2	fairly compact
	1.2 – 2.0	loose

1 to 4 results of U.S.S.R. tests for heavy clay
5 to 8 " " " " " clay
9 to 12 " " " " " lean clay

Figure 2.6 Critical flow velocity for cohesive soils

In the diagram all the known experiments are used and the Russian-tests are indicated separately. The figures are valid for:

i) a depth of 0. 90 or 1. 00 m,

ii) a horizontal bed, and

iii) a straight canal.

As all figures are only valid for that small depth, for deeper water the correction factor (see Figure 2. 1) has to be introduced. For slopes the slope factor K (see Figure 2. 5) has to be applied. Finally, it is recommended to introduce a factor for the sinuosity of the channel, as shown in Table 2. 2.

One should keep in mind that, because of the arbitrary selection of the category to which the cohesive soil belongs, the figures should be used only to provide a first indication of the resistance of the soil against the current. As soon as the application becomes important from the "capital investment" point of view, it is recommended to check the resistance of the material under consideration on undisturbed samples in a laboratory. Another possibility, which offers a satisfactory practical approach, is to use the available data and to construct one or several test sections. This enables verification under natural conditions, and conclusions leading to a satisfactory design.

2. 2. 2 Resistance against wave action

In Figure 2. 7 a wave is shown schematically (height H m; length L m), moving in the direction of a stone-covered slope.

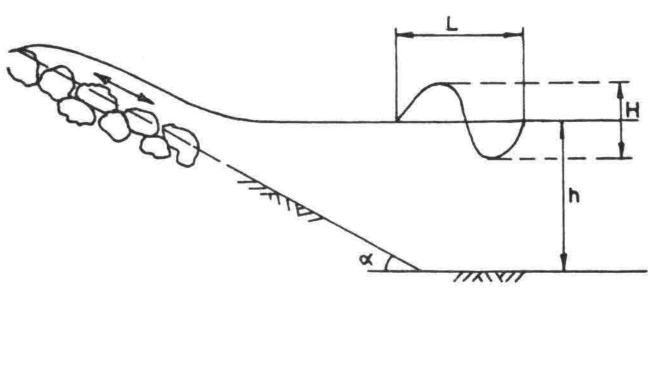


Figure 2.7 Wave attack on a slope

At the slope the wave will break, loosing energy in the form of heavy turbulence. The remaining energy causes a wave uprush against the slope and finally a flowing back of the water along the slope. It is clear that for such complicated phenomenon a strict theoretical approach is almost impossible. Therefore, one has to rely on empirical formulae borne in hydraulics laboratories and verified in nature. Two well-known formulae are mentioned below (Iribarren and Hudson).

<u>Iribarren:</u> $W \geqslant \dfrac{f. H^3 . \rho_{st}}{\Delta^3 (\cos\alpha - \sin\alpha)^3}$, where f is a coefficient related with the characteristics of the wave, the depth and the angle α of the slope, as shown in <u>Figure 2. 8.</u> W is expressed in kg mass.

<u>Hudson:</u> $W \geqslant \dfrac{H^3 . \rho_{st} . \tan\alpha}{3. 2. x \Delta^3}$

The coefficient 3. 2. is valid for smooth quarry stone. For concrete blocks of special shape, as for instance tetrapods, the coefficient may rise to a magnitude of 10 or more.

<u>Example:</u> H=1 m h=2. 5 m $\tan\alpha = \dfrac{1(\text{vert.})}{3(\text{horiz.})}$ $\rho_{st} = 2600$ kg/m^3 L= 2. 5 m.

Iribarren: $W \geqslant \dfrac{0.028.\ (1)^3.\ 2600}{(1.6)^3.\ (0.633)^3} = 71$ kg.

Hudson: $W \geqslant \dfrac{(1)^3.\ 2600.\ 1/3}{3.2.\ (1.6)^3} = 66$ kg.

The order of magnitude is the same. As the influence of the side slope is more integrated in Iribarren's formula than in Hudson's, the Iribarren formula appears to be preferable. Furthermore, for slopes steeper than tan = 1/2, Iribarren's formula leads to heavier stones, which requires its application for safety reasons.

From the equations it becomes clear that the relative density Δ is very important. It means that comparing material such as gravel with density $\rho_s = 2600$ (and $\Delta = 1.6$) with the material lead slags with density $\rho_s = 3600$ (and $\Delta = 2.6$), the mass of the pieces of stone needed to remain in rest differs by a factor 3. 1.

It means that a protective layer of lead slags requires much less material (in volume, but also in weight) than a layer of gravel. Of course, the availability of the material and the estimated cost determine which material should be selected for application. Although Iribarren's formula has been verified in nature, for application of significant importance it is normal practice to use the calculated values as an estimate to be checked in an experienced hydraulics laboratory.

Remark : For the case that the stones of the protective layer are exposed to current as well as to wave attack, theory does not provide a reliable solution. Here either laboratory experiments or test sections should be used to arrive at a satisfactory solution.

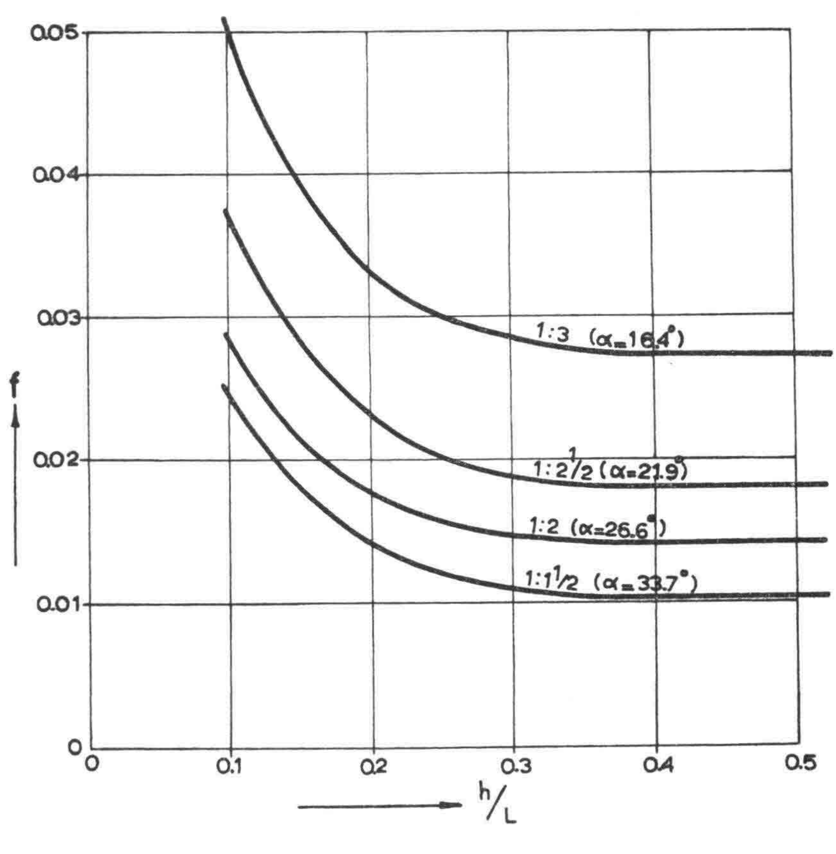


Figure 2.8 Factor f of Iribarren equation

2. 2. 3. Influence of ground-water flow

It will be clear that ground-water flow from the water course to the adjacent land will improve the stability of the soil particles on bed and bank. Conversely, a ground-water flow towards the water course will endanger that stability.

In reality the latter case is very likely to occur. For instance, at the end of the wet season the water level in a river will be high, the soil will be saturated and the ground-water table will be elevated.

When the rains stop, the water level in the river will usually drop much faster than the ground-water table. Then a temporary situation is created whereby the ground water will flow towards the river, as indicated in Figure 2. 9.

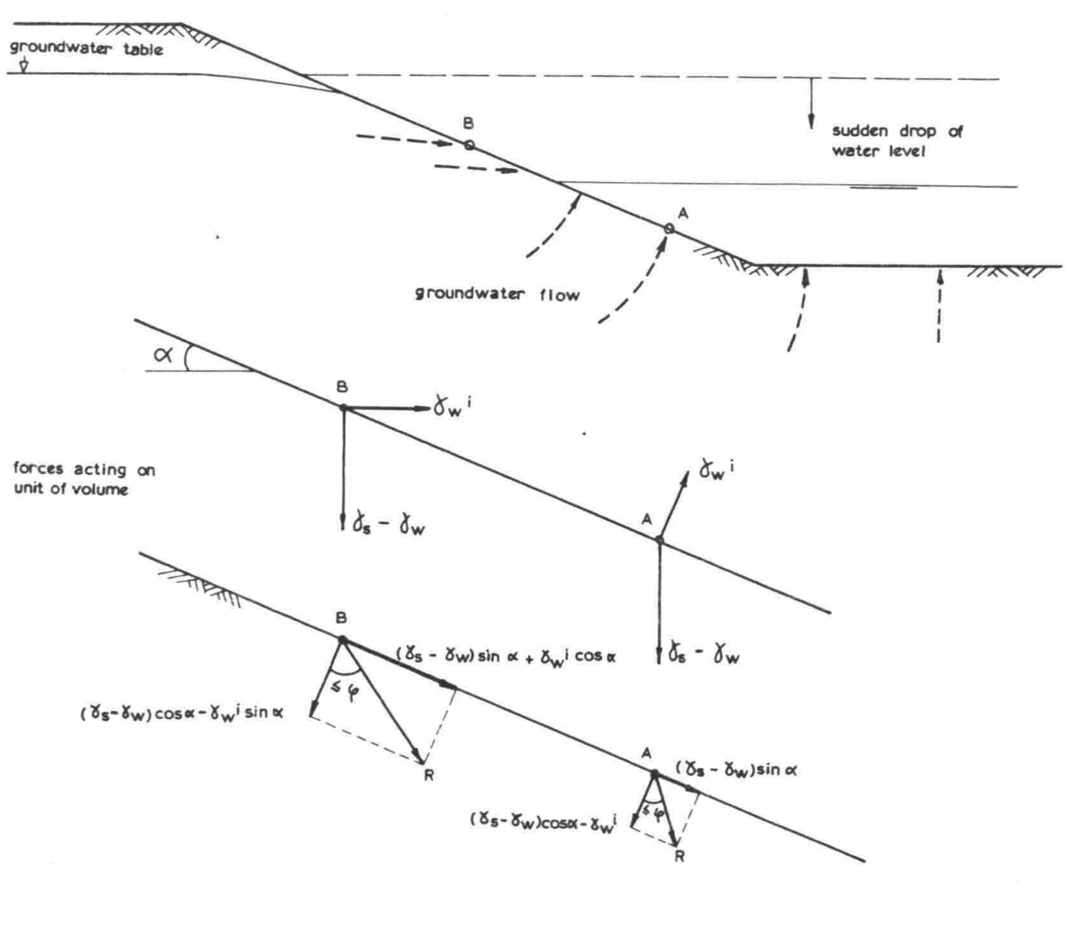


Figure 2. 9. Ground-water flow towards a canal.

Above the water surface of the river the ground water will flow through the bank almost horizontally; below the water surface the direction of the flow will be normal to bank and bed.

Also, under tidal circumstances or due to the surface drop caused by navigation, flow towards the water course will occur. It is necessary to distinguish two cases, i. e. the condition of a soil particle subject to outflowing ground water below and above the level of the open water.

a. Forces acting on a soil particle A below the water surface (see Fig. 2. 9). Here the concept "flow pressure" has to be introduced. The force due to the flow pressure is not dynamic (as the name might suggest), but static. The force acts in the direction of the ground-water flow is equal to $\gamma_w . i$ on the unit of volume.

The i is numerically equal to the difference in pressure on the front and the back side of that unit of volume. It can be derived from an flow net, which should be drawn for that purpose. Obviously the pressure gradient is largest where the velocity of the ground-water flow is largest (and the squares of the network are smallest). To investigate the condition of equilibrium, we use the components of all forces both along and normal to the slope.

For particle A the force along the slope equals $(\gamma_s - \gamma_w) \sin \alpha$; the force normal to the slope equals $(\gamma_s - \gamma_w) \cos \alpha - \gamma_w . i.$

The condition of equilibrium is:

$$\boxed{\tan \varphi \geqslant \frac{(\gamma_s - \gamma_w) \sin \alpha}{(\gamma_s - \gamma_w) \cos \alpha - \gamma_w . i}}$$

By taking the simplifying approximation that the density of wet soil $\gamma_s = 2$ and the density (specific weight) of water $\gamma_w = 1$, the condition becomes $\tan \varphi \geqslant \frac{\sin \alpha}{\cos \alpha - i}$

After determination of i (flow net), γ_s , γ_w and the angle of repose φ of the soil, the steepest possible slope α can be determined.

In the (normal) case that, in addition to the outflow of ground water there is also a current parallel to the bank, the following equation can be deduced:

$$D_m \geq \frac{\beta}{\Delta} \cdot \frac{\bar{u}^2}{2g} \cdot \left[\frac{1}{\sqrt{(\cos \alpha - \frac{i}{\Delta})^2 - \frac{\sin^2 \alpha}{\tan^2 \varphi}}} \right]$$

in which β varies between 0.7 and 1.4 depending on the turbulence.

Then a relation has been found between the size D_m of the soil particle and the steepest slope α .

b. Forces acting on a soil particle B above the water surface (see Fig. 2.9). From the drawing we can read the components of the forces along the slope and normal to the slope.

The condition of equilibrium is:

$$\boxed{\tan \varphi \geq \frac{(\gamma_s - \gamma_w) \sin\alpha + \gamma_w \cdot i \cos \alpha}{(\gamma_s - \gamma_w) \cos \alpha - \gamma_w \cdot i \sin \alpha}}$$

As the phreatic line lies in the boundary soil - air: $i = \tan \alpha$
Applying the same simplification as under a and considering Figure 2.10, we see that:

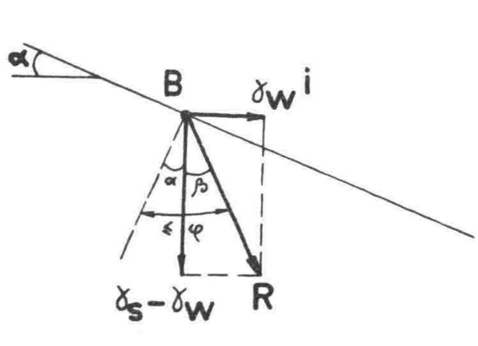


Figure 2.10
Diagram of forces

$$\tan \beta = \frac{\gamma_w \cdot i}{\gamma_s - \gamma_w} \quad \text{or} \quad \frac{\tan \alpha}{1}$$

or: $\beta \simeq \alpha$

Condition of equilibrium is that the angle which the resultant force R makes with the normal to the slope $\leq \varphi$
Or: $\alpha + \beta \leq \varphi$ or $\alpha \leq \frac{\varphi}{2}$

If, for instance, $\varphi = 30^\circ$, α becomes 15° or approximately 1 (vert) to 4 (hor), which is a rather gentle slope. If the slope is made steeper, the outflowing ground water may cause erosion.

Obviously the above considerations are only valid for non-cohesive soils. In case of cohesive soils the cohesive force will enable a steeper slope which is still in rest. In such cases the influence of cohesion can either be seen as an extra safety, or the construction of test sections with steeper slope should be considered. Also experience with already existing slopes under similar conditions can be used as a guide.

3. Consideration related to the design of revetments

3.1. General

If, based on considerations described in section 1.3, it is decided that either side slopes of banks, the bed or local areas of the boundaries of a water course are to be protected against erosion by application of revetments, and if nature of the hydraulic phenomena is known and analyzed quantitatively(see section 2.1), such basic data can be used for the design of the revetment. The theoretical considerations concerning the resistance against current, wave attack and the influence of ground-water flow, as described in section 2.2, provide the necessary tools to arrive at a satisfactory design.

Apart from studying circumstances, hydraulic phenomena and the nature of the soil, there are a number of factors which are vital to the selection of the type of revetment as well as the construction method.

Such factors may be:

- Possible variations in water level and, eventually, how fast or how slow they will be,
- Can the construction be made under dry conditions of does a part of it have to be made underwater,
- Will the maintenance of the construction, especially the part underwater, be possible and how can it be done,
- If the revetment constructed under dry conditions differs from the underwater construction, where should the transition point be and which measures should be taken to ensure he stability of that part of the construction which is vulnerable by nature.

- What sort of materials are available,
- Are there destructive ingredients in the water of mechanical, organic or chemical nature, which threaten the durability of certain materials,
- Which type of equipment can be used,
- How much experience, manpower and skill is available for construction and maintenance,
- Should the revetment be watertight or should it be given a draining function (filter) to prevent dangerous overpressure of water behind the construction,
- The cost estimate of various alternative solutions, taking into account the construction costs as well as the maintenance costs and the estimated lifetime of the construction.

After consideration of all of these theoretical and practical factors, a selection should be made. In section 3. 2 the main types of revetments are shown and their particular characteristics and possibility of application are discussed.
Section 3. 3 presents a detailed account of the stability aspects of the bank protection as a whole as well as of its parts.
In section 3. 4 special attention is paid to the requirements for the design of a filter construction. Finally, section 3. 5 includes remarks on the execution of the work, on the materials which can be used, and their durability and field of application.

3. 2. Types of revetments

A. Canals

In canals the water level usually fluctuates within rather close limits. The current will be comparatively weak and, therefore, the revetment will be needed to protect the side slopes against wave attack (taking into account any outflowing ground water).
If the possible variation of the water level in the canal is investigated and the characteristics of the waves are determined, it is possible to locate the part of the slope which requires protection against wave attack.

In Figure 3. 1 the highest and lowest possible water levels are drawn.

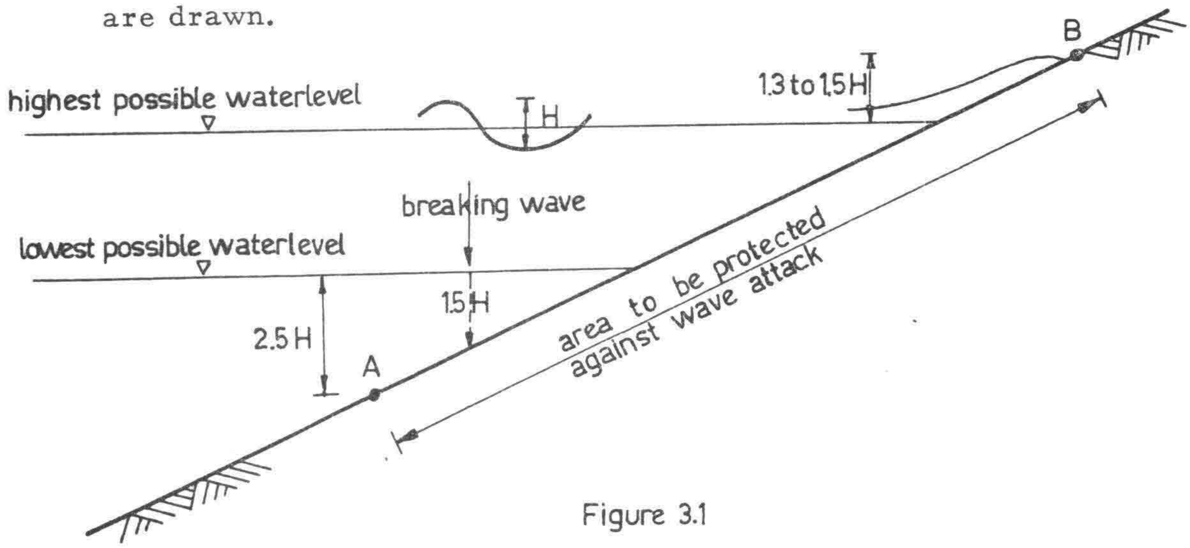


Figure 3.1

It is obvious that the upper end of the revetment is related to the highest water level; whereas, the lower end is concerned with the lowest water level. The lectures on "Navigation Canals" are dealing with the determination of the hydraulic effects accompanying ships travelling through a canal, i. e. return flow, surface drop and ship waves. Such phenomena are influenced by the size of the ship, its travelling speed and the dimensions of the canal.

A wave approaching the bank will enter shallow water near the bank and will break under loss of energy in the form of heavy turbulence, which attacks the underwater slope. Combined with the lowest water level, this phenomena determines the lower end of the revetment (point A).

The remaining part of the wave will cause an uprush against the slope and, consequently, a backflow of the water (Fig. 2. 7). Combined with the highest water level, this determines the upper end of the revetment(point B).

Point A is usually taken at approximately 2. 5 x H below the lowest water level; point B at approximately 1. 3 to 1. 5 x H above the highest water level.

If in the design of the cross section of the canal care is taken that the magnitude of the currents is kept below the critical value for the natural soil, protection of the part A B of the slope will suffice.

Warning: In practice it is very common that attempts are made to economize the design by decreasing the length(A-B) of the revetment. It must be kept in mind, however, that point A should never be brought to a higher level. Erosion of the slope below the raised point A may cause destruction of the entire revetment. It is preferable to try saving on the upper part of the revetment by bringing point B down to a slightly lower level. If it later appears that too much erosion and too much maintenance is involved, the revetment can easily be repaired or extended.

Three different types of protective constructions (revetments) can be distinguished. They are shown schematically in Fig. 3. 2.

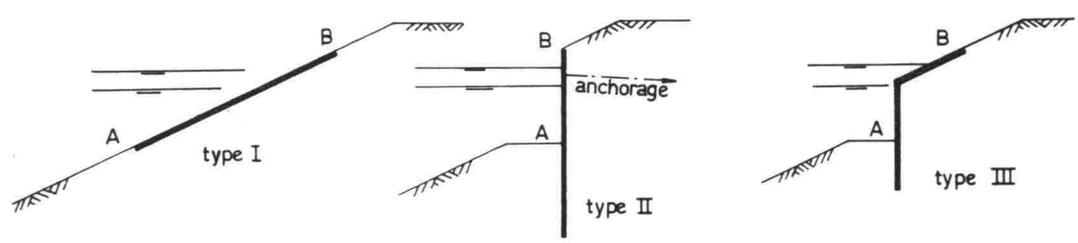


Figure 3.2

In type I the entire part of the slope attacked by the wave is revetted.

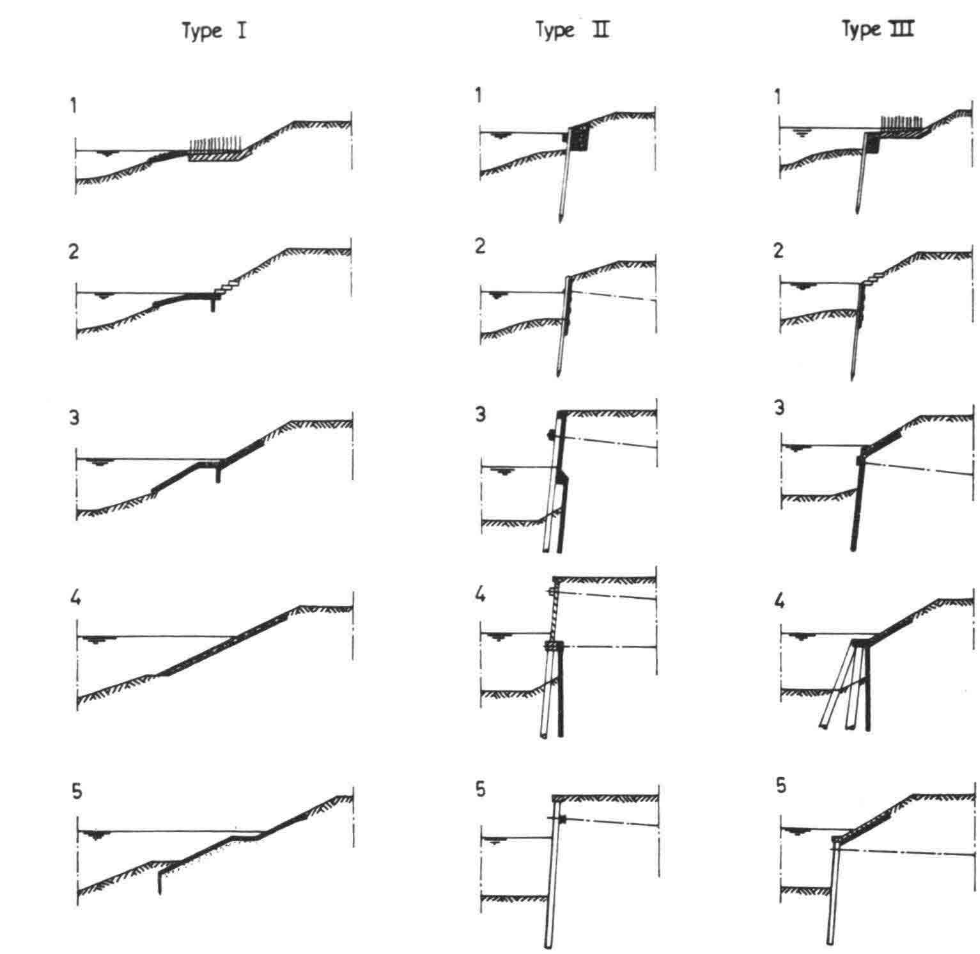
In type II the wave is reflected against a vertical wall, usually consisting of sheet piling. The part A B of the sheet piling replaces the part of the slope under attack.

Type III is a combination of types I and II. The waves are partly reflected by a sheet piling; while, the slope above the top of the sheet piling is protected against wave uprush effects.

In selecting the type to be used, consideration is given to cost, available space, secondary functions, available materials and soil properties.

In urban districts, where space may be limited, type II or III would be appropriate. Also in industrial areas along a canal type II may provide the necessary quay accommodation. In rural

districts, where sufficient room is available for a sloped bank, usually type I or III will be selected, also from an esthetic point of view.
Where stone can be obtained from nearby quarries, application of type I seams attractive.
The sheet piling of type II has to resist the wear and tear of climatological effects, especially at the part which is subject to alternate wet and dry exposure. The joints between the planks should be soil tight. Furthermore, it has to resist considerable soil pressure and, in case of steel sheet piling, even water pressure. Usually anchorage is necessary and strong and expensive profiles will be required.
The sheet piling of type III is shorter and the soil pressure is less. It will remain wet and may be constructed of wooden planks of which the joints are reasonably soiltight but not water-tight. Consequently, no appreciable water pressure can develop. Application of concrete sheet piling is possible, but the joints may allow soil particles to penetrate through the wall and special preventive measures (application of a nylon sheet behind the planking) should be taken.
When considering the application of type I, it has to be investigated whether the construction and maintenance of the revetment can be made under dry conditions (e. g. by a temporary lowering of the water level). If that is impossible, the revetment should be composed of two different constructions of which the lower part is to be constructed and maintained under wet conditions.
In Figure 3. 3 a survey is given of the three types and of their development from the time that the speed of the ships and the phenomena involved were of minor importance, to the present time with ships travelling at the highest speed which is economically justified (specified more in detail in the lectures on "Navigation Canals").



1 Reed berm and dumped stone
2 Fascine mattress and blocks
3 Fascine mattress and stone pitching
4 Permeable revetments (prepact)
5 Impermeable revetment (asphalt concrete)

1 Stakes with filter
2 Stakes with planking
3 Sheetpiling and planking
4 Sheetpiling with concrete wall
5 Anchored sheetpiling

Combination of types I and II

Figure 3.3

B. Rivers

If the slopes of a river bank have to be protected, then the current is always involved. This means that the bank must be protected right down to the river bed or even deeper. This is due to the fact that the elevation of an alluvial river bed usually fluctuates with the regime of the river so that, at the time of the construction of the revetment, the bed may have a higher elevation than at a later stage.
Furthermore, the presence of the revetted river bank itself may cause development of a deeper bed locally. This phenomenon is dealth with in the lectures on "Rivers".
In Figure 3.4 two methods are shown to overcome this difficulty.

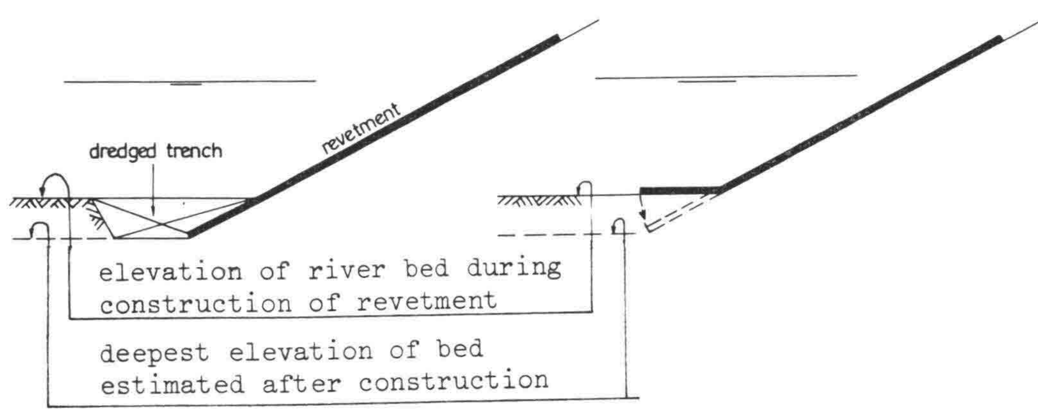


Figure 3.4

After estimation of the deepest elevation of the river bed to be expected and after construction of the revetment, the room for constructing the lower part of the revetment can be made either by dredging a trench, or the construction can be made flexible so that the revetment can adapt itself to the alterations of the river bed.
One of the characteristics of rivers is the wide range of fluctuation in the water level. There is always the question of how to determine the upper end of the revetment. Usually, local experience provides the information to decide at which elevation the revetment should be extended under normal conditions.

3.3 Stability aspects

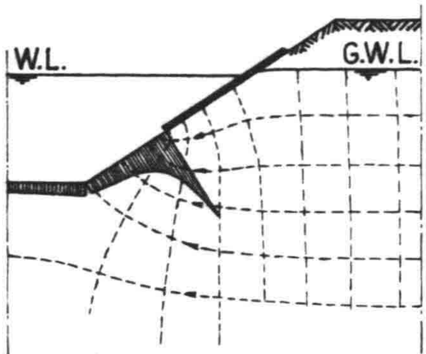
3.3.1 General

The stability of the bank itself should be considered to be the main function. Its design is mainly a matter of soil mechanics. Consideration of the forces such as dead weight, top load, possible vibrations, water pressure, flow pressure, etc. on the one hand, and the properties of the soil on the other hand should lead to the design of a safe slope. It should be kept in mind that the revetment itself may act as a sort of top load.
The influence of outflowing ground water is dealt with already in section 2.2.3.
One of the most important requirements which a revetment has to meet is that it beds down properly onto the bank and can amply adapt to any changes in the subsoil such as settlements, etc. This is necessary to prevent undermining of the construction, which might lead to a sudden collapse, and to avoid the risk of the revetment slipping as a result of reduced friction between revetment and bank.

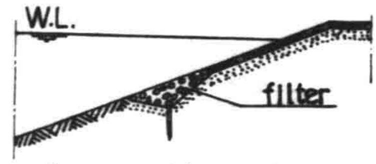
3.3.2 Stability of the revetment as a whole

The stability of the revetment as a whole should be considered especially if it consists of a coherent construction; whereas, the stability of parts of the revetment(see section 3.3.3) requires special attention if it is composed of loose elements.
As the water level in the water course, be it a canal or a river, is always subject to fluctuations (rapid and slow) and there is a time lag of the ground-water level to adapt itself to such fluctuations, the possibility of overpressure always has to be considered. It is clear that such overpressure has to be expected in the case of a watertight (impermeable) construction. However, it is also a consideration in the case of an open (permeable) construction due to rapid fluctuations of the water level.
Figure 3.5 shows how water pressure can be determined graphically. It concerns the revetment of a navigation canal and various examples of impermeable and permeable revetments.

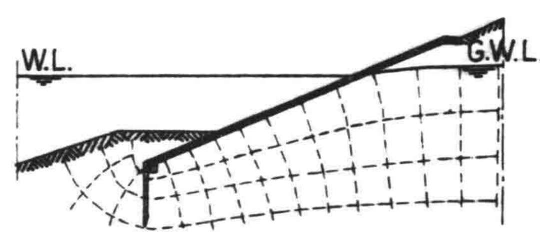
Where the bank is exposed to current and waves or where damage might endanger economic interests, extension of the revetment to a higher level is justified.
The design of a revetment for a river bank is usually like type I (Fig. 3. 2) with extensions to or into the river bed. The revetment often consists of an underwater construction and an upper part constructed under dry conditions. The transition between both constructions is dependent upon the water level during the execution of the work and also upon the necessary of having sufficient time every year to maintain the part constructed under dry conditions.
It is obvious that the construction of the lower part under wet conditions, executed in flowing water, requires both experienced and skilled labourers and suitable equipment. Very often such requirements are difficult to satisfy. Therefore, it may be necessary to adapt to particular circumstances by developing special equipment and construction methods.



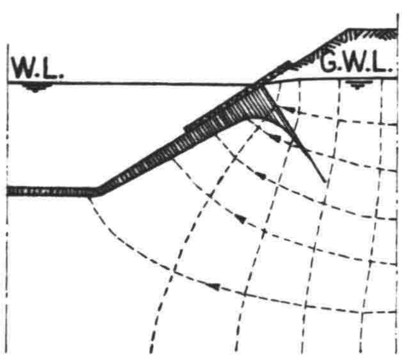
impermeable revetment
(concentration of flow lines)



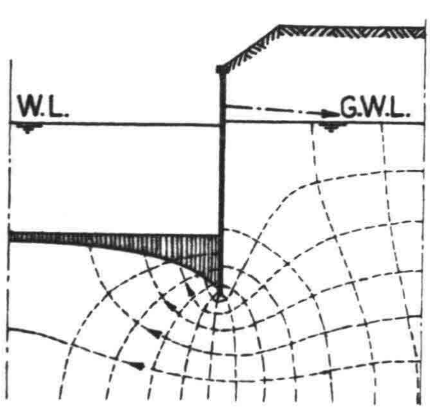
impermeable revetment
(improved by application of a filter)



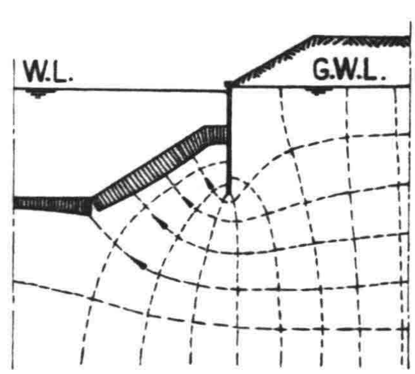
impermeable revetment
(improved by spreading of flow lines)



permeable revetment



sheetpiling (type II)



sheetpiling (type III)

Figure 3.5

Sheet piling constructions are also indicated.
From these examples it can be learned that, by proper design of the revetment, the flow lines of the flow net can be spread, thus causing less flow pressure.
The problem of equilibrium of a sheet piling constructions can easily be solved by means of a calculation of the stability and the profile needed with or without anchoring.
In order to secure the equilibrium of a coherent revetment, two criteria have to be satisfied, i. e. the critical shear stress criterion and the uplift criterion.
Considering all forces acting on a unit of area of the revetment (see Fig. 3. 6.), those criteria provide the following equiations:

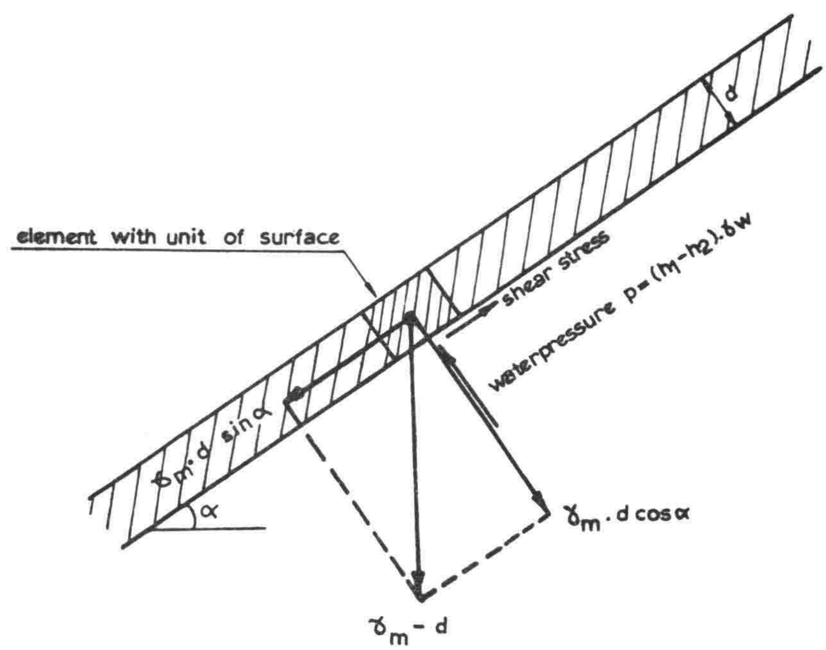


Figure 3.6

Shear stress criterion:

$$\gamma_m \,.\, d \sin \alpha < \tan \varphi \left[\gamma_m .\, d \cos \alpha - (h_1 - h_2).\, \gamma_w\right] \quad \ldots\ldots (1)$$

Uplift criterion:

$$(h_1 - h_2).\ \gamma_w < \gamma_m .\ d \cos \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The resultant water pressure $\rho = (h_1 - h_2).\ \gamma_w$, which is the difference in pressure between the bottom side and the surface of the revetment (at or below the water surface), is specified more in detail in Figure 3. 7, in which stagnant ground water is supposed.

In case of ground-water flow the value of Δh will be smaller and can be determined by using the orthogonal network. Above the water surface in the canal, $\rho (= h_1 \cdot \gamma_w)$ is smaller.

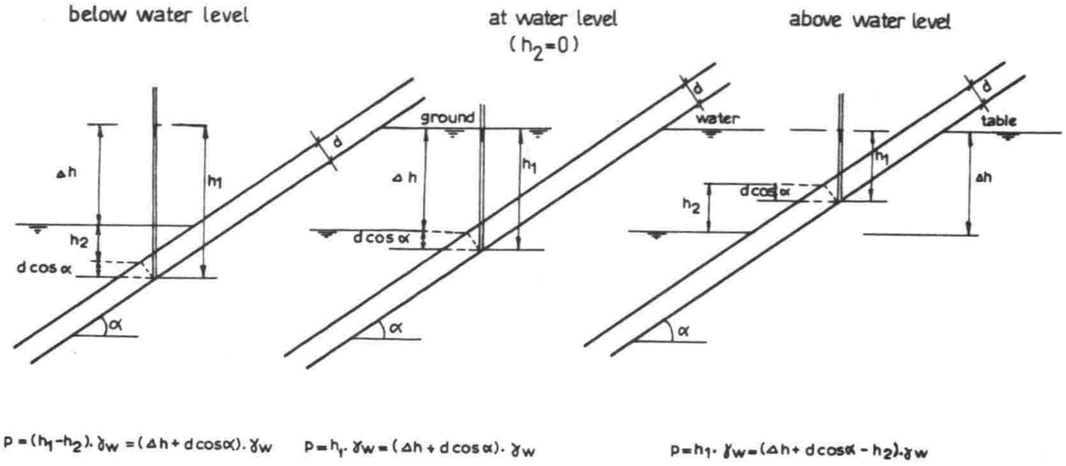


Figure 3.7

The shear stress criterion has to be satisfied in the material of the revetment, at the boundary between soil and revetment and in the soil underneath. Therefore, the smallest of the values of $\varphi_{material}$, φ_{soil} and the coefficient of friction between soil and revetment should be used in equation(1). Considering a coherent construction, it should be noted that the rapid fluctuations of the water level only cause overpressure locally and, therefore, are of less importance for stability aspects. They develop tension in the revetment material and cause deformation. If the material is viscous, such as asphalt concrete, etc., such deformations should be avoided to a certain extent in order to prevent fatigue of the material, which will gradually lead to its destruction. When a material such as stone cast with grouting asphalt is used local reduction of the frictional resistance need not be considered.
It is the slow fluctuations of the water level which may endanger the stability as they usually cause overpressure against the entire revetment. To satisfy both criteria it may be necessary to increase the thickness d (and therefore, the weight) of the revetment.

3. 3. 3 Stability of parts of the revetment

Open (permeable) constructions often consist of loose material such as gravel or dumped stone. They are subject to local overpressure which depends only on the rapid fluctuations of the water level in the canal (the ground-water table adapts itself to the slow fluctuation), and should resist tractive forces due to current, turbulence, flow pressure, wave attack, etc. Reference is made to Section 2. 2.
A permeable revetment should also meet the requirement that no soil particles can escape through it. It is also necessary that the construction continues to act as an effective filter, which means that certain permeability requirements have to be met. This subject is dealt with seperately in Section 3. 4.
The shear stress and uplift criteria, mentioned before, are essentially the same as indicated in section 3. 3. 2. Because of the nature of a filter construction, which consists of two or more layers of loose material of various size, those criteria should be written in general terms as follows:

Shear stress criterion:

$$\sin\alpha\left[\sum_{i=1}^{n}\left\{d_i(1-m_i).\ \rho_i.g+d_i.\ m_i.\ \rho_w.\ g\right\}-\rho_w.g.\sum_{i=1}^{n}d_i\right]<$$

$$<\tan\varphi\left[\cos\alpha\sum_{i=1}^{n}\left\{d_i(1-m_i).\ \rho_i.g+d_i.m_i.\ \rho_w.g\right\}-\right.$$

$$\left.-\rho_w.g\left(\Delta h+\cos\alpha\sum_{i=1}^{n}d_i\right)\right]$$

Uplift criterion:

$$\rho_w.g\left(\Delta h+\cos\alpha\sum_{i=1}^{n}d_i\right)<\cos\alpha\left[\sum_{i=1}^{n}\left\{d_i(1-m_i).\rho_i.g+d_i.m_i.\rho_w.g\right\}\right]$$

in which:

d_i = thickness of i^{th} layer

$m_i = \dfrac{\text{voids}}{\text{total volume}}$ of i^{th} layer

n = number of layers

ρ_i = density of material of i^{th} layer

Δh = difference in water level only due to rapid fluctuations

g = acceleration due to gravity

In Figure 3.8 examples are shown of impermeable as well as permeable revetments.

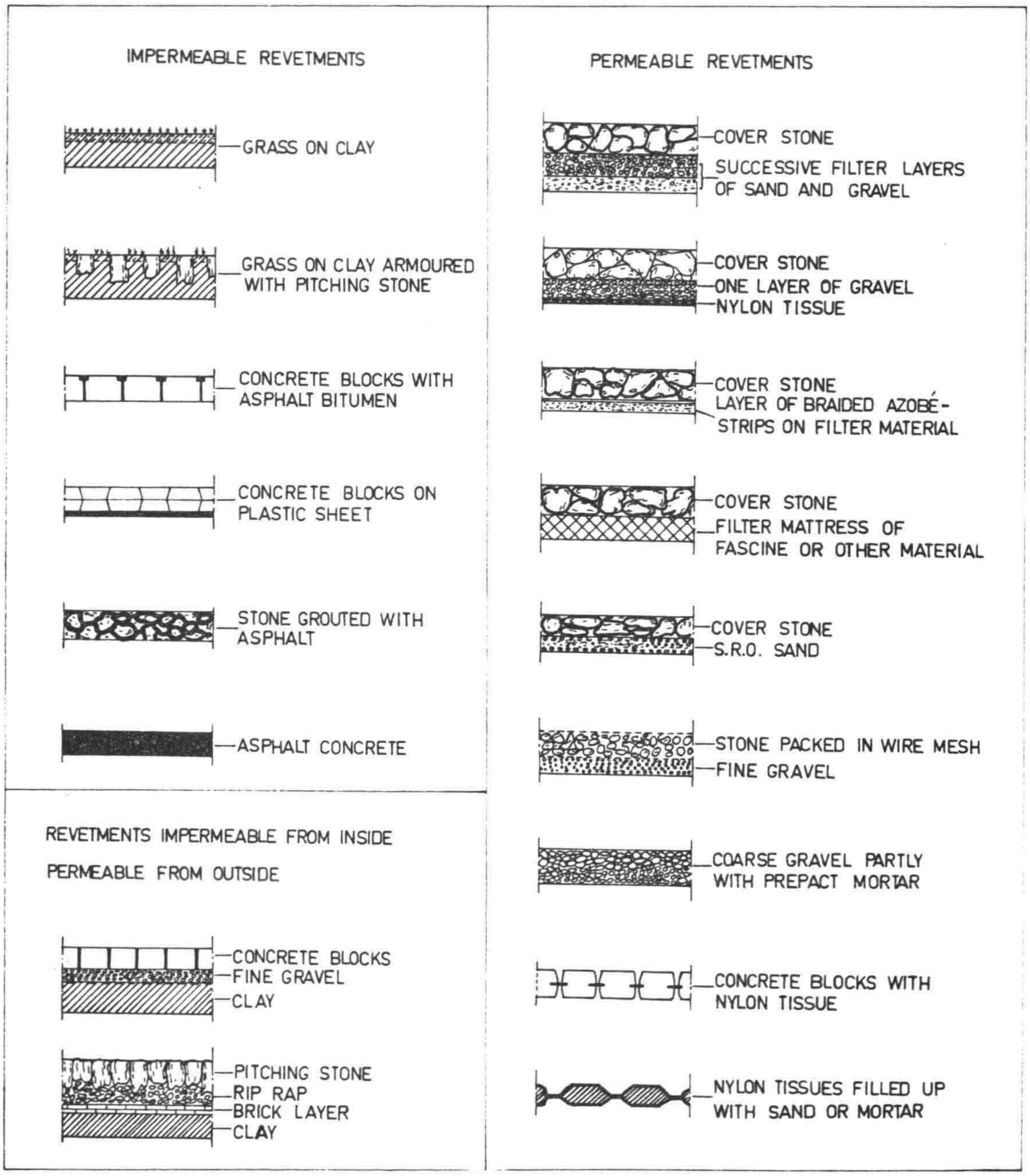


Figure 3.8

Examples of revetment - constructions

3.4 Filters

Apart from the stability requirements indicated in section 3.3, a permeable construction consisting of loose material composed of two or more layers of various particle size (see Fig. 3.9) has to meet certain permeability requirements in order to guarantee that the revetment has and maintains its effective filter properties.

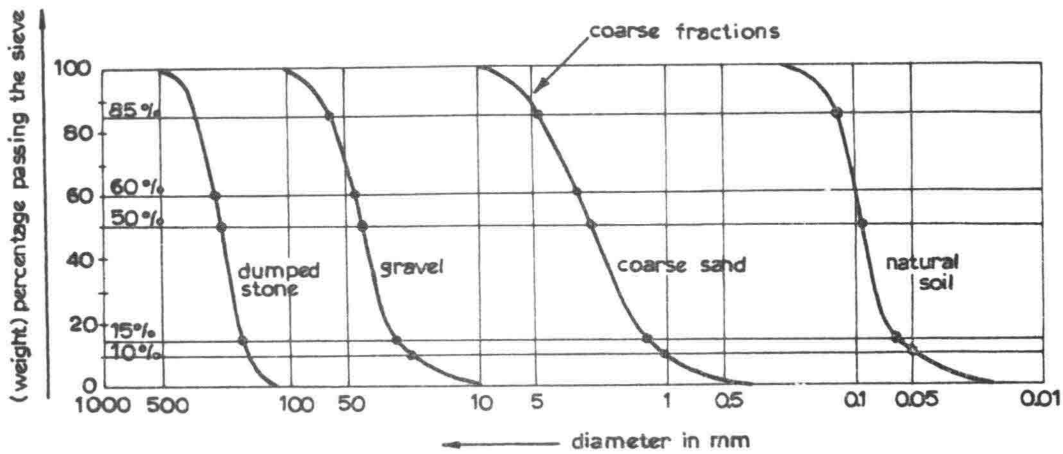


Figure 3.9

Two main requirements can be distinguished:

Criterion I : The finer particles of a layer should be prevented from being sucked through the layer on top of it;

Criterion II: The permeability of a layer should at least be equal to the permeability of the layer underneath, in order to ensure that pressure variations due to fluctuations of the water level in the water course can penetrate easily into the revetment, thus preventing significant overpressure.

Based on many investigations in models and in nature, those criteria can be formulated in mathematical terms in which the fractions of the material of two consecutive layers are related.

Criterion I leads to two requirements:

I a : $\dfrac{D_{15}\,(\text{upper layer})}{D_{85}\,(\text{layer below it})} \leq 5$	
I b : $\dfrac{D_{50}\,(\text{upper layer})}{D_{50}\,(\text{layer below it})} = \underline{a}$	(depending on shape and gradation of the particles)

The quotient a is to be specified as follows:

homogeneous round particles (as gravel)	a = 5 to 10
homogeneous square particles (as mechanically broken material)	a = 10 to 30
graded material	a = 12 to 60

Remark: These figures are approximations. In a laboratory test, as described below, they can be determined as belonging to the particular material to be used for the layers.

Criterion II leads to the requirements:

$$\boxed{\text{II a} : \frac{D_{15}\,(\text{upper layer})}{D_{15}\,(\text{layer below it})} = \underline{b}}$$, depending on the category.

The quotient b is to be specified as follows:

gravel	b = 5 to 10
mechanically broken material	b = 6 to 20
graded material	b = 12 to 40

Same remark as for factor a

$$\boxed{\text{II b} : D_5\ (\text{upper layer}) > 0.75\ \text{mm}}$$

Apart from the two main criteria mentioned, there are a few secondary criteria which also have to be met:

Criterion III: The sieve curve of the natural soil and the first layer should be more or less parallel;

Criterion IV: Regarding inaccuracy of placing the layers the thickness of the layers is subject to a minimum, e.g.:

for coarse sand	d = 10 cm
for gravel	d = 20 cm
for dumped stone	d = 1 to 2 times the size of the largest stones, depending on current, turbulence and wave attack.

In case the layers have to be placed under water or when the subsoil is uneven, it is a safe policy to increase the thickness of the layers. The extent to which this can be done is dependent upon the skill of the labourers and the available equipment for placing such layers.

Criterion V: To prevent segregation of the material when it is dumped into water, it is recommended to mix the material with 3 to 10% water beforehand and to place it in layers with a maximum of 10 cm thickness, while the quotient:

$$\boxed{\frac{D_{60}(\text{ upper layer })}{D_{10}(\text{layer below it })} \leq 20 \text{ (Requirement III)}}$$

To check whether a filter construction designed according to the various requirements meets the main criterion II and, if necessary, to adapt the factors a and b, the following laboratory test can be used:

As indicated in Figure 3. 10 the various layers are placed upside down in an apparatus.

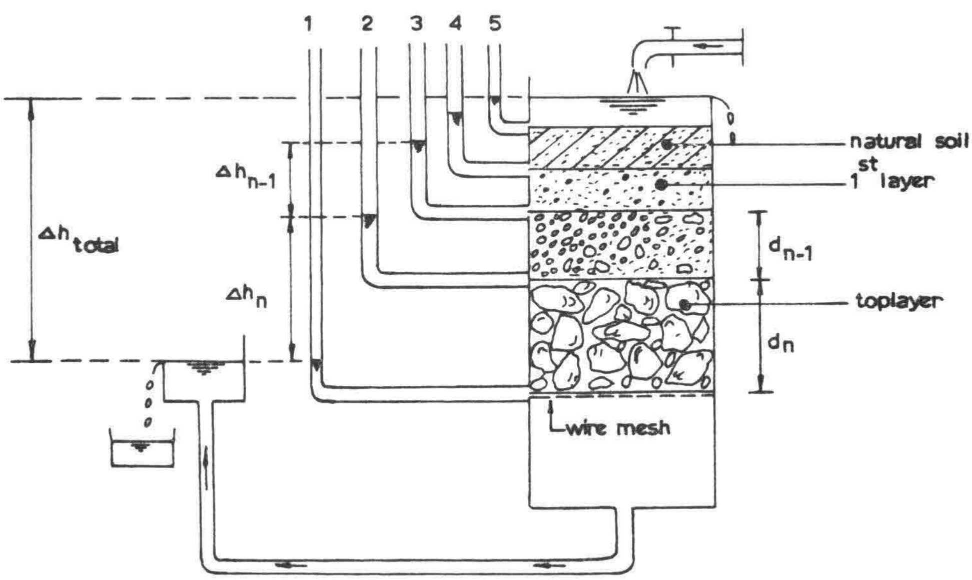


Figure 3.10

A ground-water flow from the surface to the bottom is created by a variable head Δh_{total}. At each boundary between layers manometers are placed to measures the water pressure at that particular place. So the distribution of the water pressure through the filter is known.

This distribution should be such that:

$$\frac{\Delta h_n}{d_n} < \frac{\Delta h_{n-1}}{d_{n-1}} \leqslant \frac{\Delta h_{n-2}}{d_{n-2}}$$, etc. to ensure proper permeability.

By varying Δh_{total}, rapidly and slowly, one can see how fast the pressure through the various layers adapt to the new situation and where developing water pressure have to be taken into account.

<u>Example of the design of a filter composed of layers consisting of loose material</u>

From a number of samples taken from the natural soil the mean grain-size distribution is determined and drawn in a diagram(see <u>Fig. 3.11</u>). It is assumed that the qualifications of the top layer, regarding the tractive force of the current, the slope factor, wave attack, etc., are determined as described in section 2.2, and that it has to consist of dumped stone of 5 to 40 kg, which is also dependent on its availability. The corresponding particle diameters (160 to 300 mm) are indicated in the Figure.
In this way the two boundaries of the filter are determined and, using the various requirements, it has to be investigated whether one or more subsequent layers are needed.
The factors <u>a</u> and <u>b</u> are determined using the laboratory test described and are indicated in the Figure.

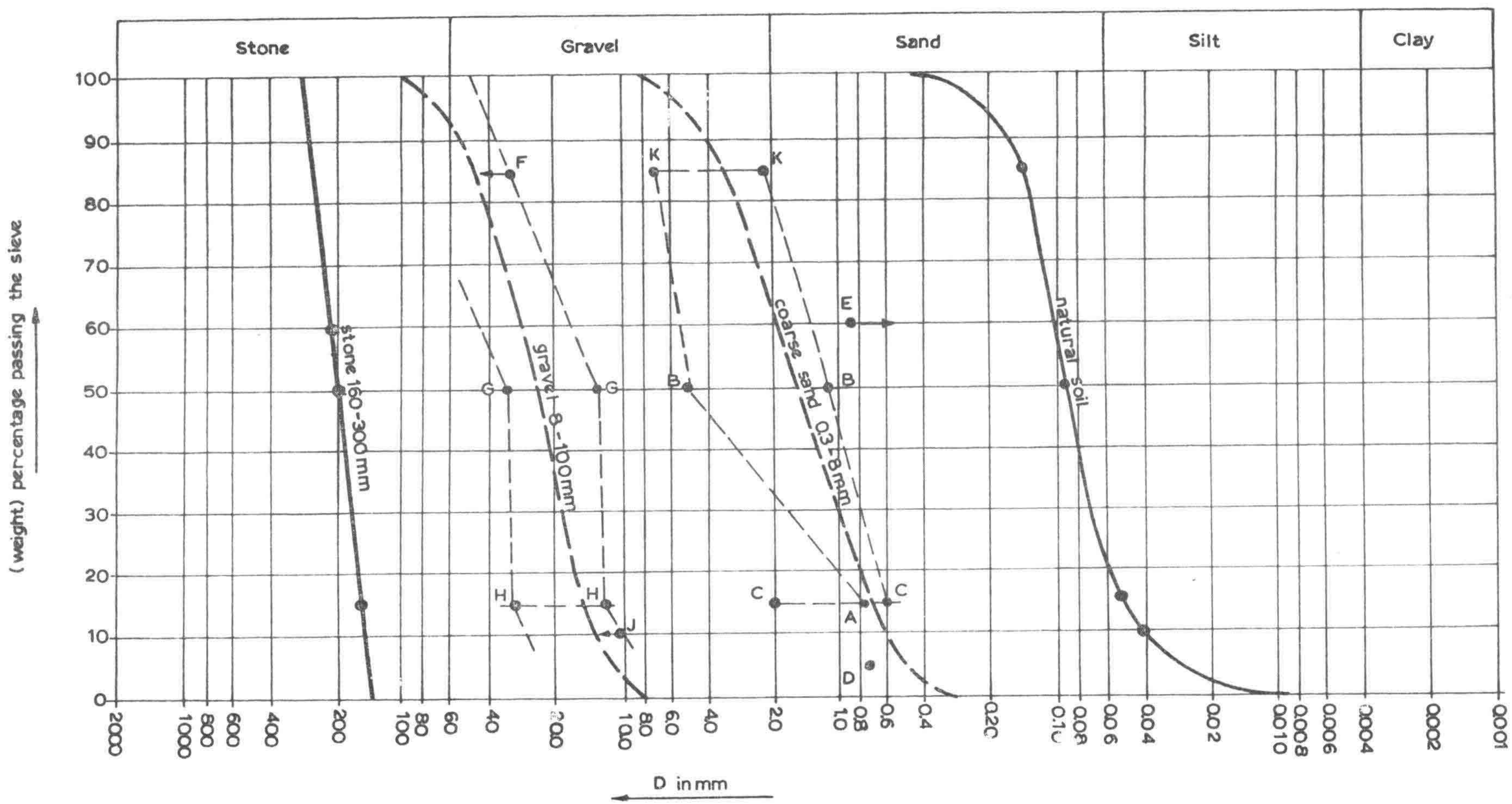
Starting from the natural soil, some limits can be determined which are valid for the grain-size distribution of the first layer.

<u>Requirement I a :</u> $$\frac{D_{15}\text{ (first layer)}}{D_{85}\text{ (natural soil)}} \leqslant 5$$

As D_{85} (natural soil) = 0.15 mm, D_{15} (first layer) $\leqslant 5 \times 0.15 = 0.75$ mm.
This point is indicated in the Figure as <u>point A.</u>

<u>Requirement I b :</u> $$\frac{D_{50}\text{ (first layer)}}{D_{50}\text{ (natural soil)}} = 12 \text{ to } 58$$

As D_{50} (national soil) = 0.09 mm, D_{50}(first layer) should be between the limits (12 to 58) x 0.09 mm = 1.1 to 5.2 mm. (<u>Points B</u>)



$$\frac{d_{15}\ n^{th}\ \text{layer}}{d_{85}\ (n-1)^{th}\ \text{layer}} < 5$$

$$\frac{d_{50}\ n^{th}\ \text{layer}}{d_{50}\ (n-1)^{th}\ \text{layer}} \text{ is for } \frac{\text{coarse sand}}{\text{natural soil}} = 12 \text{ to } 58; \quad \frac{\text{stone}}{\text{gravel}} = 7 \text{ to } 15$$

$$\frac{d_{15}\ n^{th}\ \text{layer}}{d_{15}\ (n-1)^{th}\ \text{layer}} \text{ is for } \frac{\text{coarse sand}}{\text{natural soil}} = 12 \text{ to } 40; \quad \frac{\text{stone}}{\text{gravel}} = 5 \text{ to } 15$$

Determinded by laboratory tests

Figure 3-11

Requirement II a: $\dfrac{D_{15}\text{ (first layer)}}{D_{15}\text{ (natural soil)}} = 12 \text{ to } 40$

As D_{15} (natural soil) = 0. 05 mm, D_{15} (first layer) should be between the limits (12 to 40) x 0. 05 mm = 0. 6 to 2. 0 mm. (Points C)

By combining requirements I and II a, it can be concluded that D_{15} (first layer) should have a value between the limits 0. 6 and 0. 75 mm.

Requirement II b : D_5 (first layer) > 0. 75 mm. (Point D)
It will be clear that this requirement cannot be satisfied within the limits of D_{15}.

Requirement III : $\dfrac{D_{60}\text{ (first layer)}}{D_{10}\text{ (natural soil)}} \leqslant 20$

As D_{10} (natural soil) = 0. 042 mm, D_{60} (first layer) should be $\leqslant$ 0. 85 mm.(Point E)

Thus, a number of limits have been determined which are valid for the grain-size distribution of the first layer.

The next step is to start from the top layer (n^{th} layer), of which the qualifications are known, and to determine limits for the grain-size distribution of the $(n-1)^{th}$ layer, using the same requirements.
One finds:

Requirement I a: D_{85} $(n-1)^{th}$ layer $\geqslant \dfrac{170}{5} = 34$ mm. (Point F)
Requirement I b: D_{50} $(n-1)^{th}$ layer $= \dfrac{200}{6 \text{ to } 15} = 13 \text{ to } 33$ mm. (Points G)

Requirement II a: D_{15} $(n-1)^{th}$ layer $= \dfrac{170}{5 \text{ to } 15} = 11.5 \text{ to } 34$ mm. (Points H)

Requirement III : D_{10} $(n-1)^{th}$ layer $\geqslant \dfrac{220}{20} = 11$ mm. (Point J)

Thus, a number of limits which are valid for the grain-size distribution of the $(n-1)^{th}$ layer are determined.

From the Figure the conclusion can already be drawn that the filter has to consist of three layers in the categories of coarse sand, gravel and stone respectively.
It is necessary, however, to check whether the requirements are also met between the layers of coarse sand and gravel (1^{st} and 2^{nd} layer).

For instance: $\dfrac{D_{15}\ (2^{nd}\ \text{layer})}{D_{85}\ (1^{st}\ \text{layer})} \leqslant 5$ or

$D_{85}\ (1^{st}\ \text{layer}) = \dfrac{11.5 \text{ to } 34}{5} = 2.3 \text{ to } 6.8$ mm. (Points K)

and so on.

After determination of the limits which are valid for the grain-size distribution of the material of each of the layers, it should be investigated whether stone, gravel and mixtures of gravel and coarse sand, satisfying those limits, are available.
Let us assume that, for the 1^{st} and 2^{nd} layer, material can be purchased with grain-size distributions as indicated in the Figure.
Comparison of those distributions with the limits calculated shows that almost all requirements are met, as well as criterion III.
It should be noted that D_5 (1^{st} layer) is approx. 0.5 mm, which is less than the required 0.75 mm.(Point D)
Such minor deviations usually are accepted. For differences of more importance, application of nylon fabric can be considered.
Also requirement III (point E) is not met. It means that, when the material of the 1^{st} layer is dumped under the water, segregation of the material has to be feared. Therefore, the appropriate filter construction should be constructed under dry conditions.

The ultimate result is:

1^{st} layer	10 cm coarse sand
2^{nd} layer	20 cm gravel
3^{rd} layer	30 cm stone

A great variety of solutions based on the filter principle are applied. For instance, one or more layers can be replaced by different filter material, such as fascine mattresses with or without reed layers, nylon mattresses consisting of double cloth forming cells which are filled with sand or mortar, asphalted cloth, filter mats of braided aluminium or Azobe strips or woven plastic threads with or without a reed layer underneath. Often the difficulty arises that fine soil particles can penetrate through the 1^{st} layer. The pores should not be larger than $0.5 \times D_{85}$ of the natural soil. If this cannot

be realized, only some intermediate layers may be replaced.
It is also possible to compose a filter layer of special bituminized sand, the so-called S. R. O. -sand (Special Road Oil), which provides a stable, flexible and permeable layer.
An entirely different solution is obtained by omitting the top layer or layers and providing the underlying material with such cohesive properties that it can resist all forces. For example:

- Stone packed in heavily galvanized chain link wire mesh,
- A stone layer treated with prepact mortar or mastic asphalt without affecting the permeability for water,
- Stone packed in brushwood, etc.,
- Concrete blocks connected with cast-in synthetic textile strips, etc.

All these constructions have the disadvantage that their wearability is determined by the packing material or the binding agent.
Above water level maintenance and repair can easily be done; under water this factor may be decisive for the solution to be selected.

3.5 Materials

3.5.1 General

Naturally, the designer of a revetment should be familiar with the kinds of materials which can be applied and the properties of such materials. Experience on traditional materials which are in local use can easily be collected. As far as the "new" materials are concerned, such as asphalt and synthetic materials, there is an increasing interest in the application of them under certain conditions.
Hereafter some information is given on bituminous and synthetic materials. Since these materials and their application methods are still under development, such information may become outdated within a few years. Therefore, it is recommended to solicit expert advice when selecting the most suitable material for the construction under consideration, as well as for the method of execution.

3.5.2 Bitumen and Asphalt

Bitumen: The enormous development in the application of bitumen for civil engineering structures has changed the status of this material from a waste product of the distillation of crude oil into that of a valuable end product of the refineries. Crude oil is a mixture of many light to heavy fractions of hydrocarbons. By distillation under increasing temperature and vacuum, these fractions (petrol, kerosine, lubricating oil, etc.) gradually disappear and the residue (bitumen) becomes less and less viscous at the higher temperature. After cooling to a normal temperature, bitumen of softer and harder qualities is obtained successively.
Characteristic properties of bitumen are:

i) softness.
Softness is determined by the penetration of a standard pencil into bitumen at 25 ^{o} C. Thus, soft, medium and hard bitumens are distinguished. Obviously, soft bitumen will more easily deform than hard bitumen.

ii) elasticity and plasticity.

At lower temperatures and when subject to sudden forces, bitumen behaves as an elastic material. At higher temperatures and when subject to loads of long duration, bitumen behaves as a very viscous liquid. It is, therefore, called thermoplastic material.

The temperature (in $^{\circ}$C) at which the transition from elastic to plastic occurs is indicated as the softening point. Soft bitumens have a softening point between 30° and 50°C; hard bitumens up to 75°C. The warmer the climate and the steeper the slope, the harder the bitumen that should be used. The industry has succeeded in making bitumens which are less sensitive to temperaure. They have a higher softening point for the same penetration. These "blown" bitumens are all worked up after heating (135°C for soft bitumen to 240°C for hard bitumen). This is necessary to obtain the required properties (adhesion, sealing, filling up, etc.). This heating may have disadvantages; therefore, bitumens have been developed which can be applied without previous heating.

Cut-back bitumens:

Bitumens can be liquified by the addition of a volatile distillate. When the cut-back is exposed to the air, the distillate evaporation will occur only in the case of a thin layer, exposed to not too cold air. In wet surroundings they can seldom be applied. Rapid, medium and slow curing cut-backs are distinguished by the rate of evaporation.

Emulsion-bitumen:

Emulsion-bitumen (e. g. Shell-Eshalite) is an emulsion of bitumen particles in water, with the addition of an emulsion liquid. It can be applied without heating. Evaporation of the water leaves a film of bitumen on the surface.

Asphalt:

There has to be a stable framework in order to apply bitumen in thicker layers. The adhesive forces of the bitumen then provide a link within this framework.

Stability:

The strength properties now depend on the properties of both

the bitumen and the framework. The framework normally consist of mineral matter such as filler, e.g. limestone, cement, sand, gravel and chippings. Sometimes materials like fibre, cotton, etc. are applied. If the quantity of bitumen is limited, the framework forms a material which behaves in accordance with the laws of soil mechanics. However, there are two major differences. In the first place the bitumen has a certain lubricating effect, decreasing the angle of repose to about 75% compared with the same aggregate without bitumen. This angle is about 24^{o} for bituminous sand, 27^{o} for sand asphalt and 30^{o} for asphalt concrete.
(Slopes respectively 1 to 2 1/4, 1 to 2 and 1 to 1 3/4).
The second difference from the behaviour of soil is the initial resistance of the bitumen. It depends not only on the properties of the bitumen but also on the combination with the minerals. The application of harder bitumen, with a larger initial resistance, offers the possibility of increasing the maximum slope of asphalt constructions.
By increasing the quantity of bitumen in the mixture, the lubricating effect will also increase. Thus, a part of the normal stress is absorbed by the "liquid" bitumen, leaving the remainder for the sand particles. A resultant factor is that the shear resistance decreases. If the quantity of bitumen is further increased the mixture will behave more and more like a liquid and the angle of repose will decrease to 10^{o} or even 5^{o}.
Therefore, especially in applying asphalt on slopes or under long-duration forces, the mixture should not be made too rich in bitumen. However, this quantity must be sufficient enough to obtain a good adhesion between the particles of the aggregate.

Percentage of bitumen:

For adhesion it is necessary to heat bitumen and mineral matter before mixing. A good adhesion is obtained when the quantity of bitumen in the mixture is sufficient to form a film around the grains. Thus, the finer the aggregate the greater the quantity of bitumen needed.

Practice has shown that the minimum necessary <u>weight</u> of bitumen is about:

3 %	of	the	weight	of	the	stone fraction,
6 %	"	"	"	"	"	coarse sand,
9 %	"	"	"	"	"	fine sand,
12 %	"	"	"	"	"	very fine sand (75 to 200 micron),
14 %	"	"	"	"	"	filler (75 micron).

A minimum of bitumen is needed for a coarse aggregate of homogeneous grain size. However, the application of such an aggregate has some disadvantages. In the first place the conglomerate will be pervious since the porosity of a homogeneous system of grains is about 20 to 40 %. Only a part of these voids is filled by the bitumen. Penetration of water and air will then occur and, possibly, the bitumen film will be oxidized.
This may result in loss of plasticity (blown bitumen) and in cracking, or the bitumen will be stripped from the mineral(soft bitumen). As time goes by every asphalt will lose part of its plasticity, but porosity will accelerate this phenomenon. In the second place, a conglomerate with a high porosity and a low bitumen content will have almost no plasticity. Consequently, the capacity for deformation will disappear and ruptures will easily occur. Therefore, in the case of a homogeneous sand, a greater percentage of bitumen should be applied than is indicated by the quantities given above. However, no more bitumen than is absolutely necessary should be added because the plasticity may soon be too dominant, even for relatively big porosity (5 to 10%).

<u>Composition of the minerals:</u>

It is much better to decrease the porosity by mixing grains of different diameter; thus, the space between the stones is filled with coarse sand, the coarse sand with fine sand and the fine sand with basic filler.

All the grains of the aggregate are surrounded by the minimum quantity of bitumen. A small additional quantity may be added in order to improve the workability; but, under normal temperature conditions the conglomerate must remain stable. Since the specific weight of bitumen is one (soft bitumen slightly less, hard bitumen slightly more) and that of a normal mineral is 2.65, asphalt structures will vary in specific weight between 1.8 (high porosity) and 2.3 (low porosity). In practice, an experienced laboratory has to be available for assistance, particularly when large quantities are involved, to determine the composition of the aggregate as well as the amount of bitumen to be added.

Types of asphalt:

Many types of asphalt are applied in civil engineering. They differ in type and quantity of bitumen and in the application of filler.

a. asphalt mastics, asphalt cement or fillerized bitumen.
   These materials consist of a mixture of bitumen and filler (e.g. basic limestone dust) and are often used in thin layers as a binder. Since plasticity and adhesion are the main objectives, a large percentage of bitumen is used (soft qualities on flat slopes to blown bitumen on steep slopes).
   A proportion of 40% bitumen and 60% filler is sometimes used.

b. bituminous sand.
   It is a mixture of sand and bitumen. In cold climates it is hot mixed; in hot climates sometimes a mixture of non-heated sand and cut-back is used. The percentage of bitumen depends on the grain diameter of the sand. (6 to 10% bitumen is normal.) Bituminous sand has the consistency of moist soil. It is laid to a thickness of a minimum of 10 cm up to a maximum of 30 to 40 cm and is compacted by tamping. However, the porosity remains high.
   The main purpose of a bituminous sand layer is to serve as a foundation for a top layer, which gives protection and impermeability.
   Bituminous sand has no plasticity; it soon cracks in the case of setting. If air penetrates, the bitumen film is removed (stripped) after some time. It can only be applied without protection under temporary conditions. When the application of asphalt for revetments was introduced, bituminous sand was rather popular. Nowadays it is considered to be out-of-date.

c. sand asphalt.

This consists of a hot mixture of bitumen, filler and sand. The mixture is simular to bituminous sand. It is still not pourable, but a small addition of filler (5% for instance) allows for an increased percentage of bitumen. The material is still porous, but it is tougher and more resistant. It is often applied as a foundation layer where formerly bituminous sand was used.

d. sheet asphalt.

This is a pourable sand asphalt, made by increasing the amount of filler and bitumen and carefully grading the sand. It is used for filling the joints in the stone pitching. If sand is available which allows for a good grading, sheet asphalt may consist of approximately 80% sand, 10% filler and 10% bitumen.

e. asphalt concrete.

This is a hot mixture of stone, filler and bitumen, graded in such a way that the conglomerate is practically impervious (2 to 5% pores). The maximum dimensions of the stones must be smaller than 1/4 to 1/3 of the thickness of the layer. A top layer of asphalt concrete of 5 cm for a light attack (canals) and of 10 cm for a heavy attack (seawalls) is usual in the Netherlands. Asphalt concrete is compacted by rollers. Since the influence of the rollers is limited, the depth of the layer will seldom be more than 10 cm.

Surface treatment

Surface treatment of asphalt structures can have different purposes: e.g. to protect a soft or open asphalt against corrosion; to make it less pervious; to prevent the softening of the surface during hot weather; or to give a lighter colour in order to prevent absorption of heat.

In all of these cases, a thin film (1 to 2 kg/m^2) of hot soft bitumen is sprayed onto the surface, covered by fine chippings, gravel, sand or shells and compacted by rolling. In case a stronger surface treatment is desired and, consequently, more bitumen per m^2, harder bitumens (especially on slopes) or fillerized bitumens will have to be applied.

Tack coat

A tack coat is a thin film of bitumen giving strong adhesion between two different parts of the construction. The film must be thin to prevent sliding. Application is not possible, therefore, on a very absorbent surface. Such a surface would first have to be coated with a primer. A soft bitumen or a cut-back is often applied in a quantity of about 0.5 kg/m^2.

3.5.3 Synthetic materials

The selection of a synthetic material to be applied as part of a revetment is closely related with its function.
If, for instance, a plastic membrane is needed to line the side slopes and, if necessary, the beds of canals, reservoirs, etc. in order to contain the water or to reduce infiltration of seepage into canal or reservoir, the use of impervious foils is necessary. If, on the other hand, the synthetic material has to serve as part of a filter construction, and, therefore, has to be pervious to water and impervious to soil, either perforated foils or nylon tissues are to be selected.

Foils.

Usually foils are made from polythene or polyvinyl chloride. The use of plastic materials has been studied by the Bureau of Reclamation. These studies reported that: because canal linings are in contact with moist soil, whether they are applied as a surface covering or as a buried membrane, they are subject to rapid decomposition by bacterial and micro-organism action. An extensive study was made of the effect of micro-organisms on various types of materials. Included in the studies were several plastic films and plastic-treated materials which were proposed as buried membrane and exposed-type canal linings for seepage control. The test in which the materials were baried was continued for a number of years. The majority of the materials tested were regular plastic films or slightly modified ones.

The results of the long range tests were:

- plastic films are highly resistant to bacteriological deterioration;
- tensile strength and flexibility of plastic films were virtually unaffected by burying under moist soil;
- reinforced plastics, where organic materials such as paper, canvas, and burlap are involved, are not desirable because vegetable fabric materials are vulnerable to micro-organism attack unless completely and carefully saturated with resin;
- plastics, even thin 1 - 1/2 mil films, are essentially watertight materials, but films less than 8 mils in thickness have low puncture resistance when placed over a rough subgrade and covered with angular material;
- plastic films deteriorate quite rapidly when exposed to sun and weather;
- a trial test indicated that canal slopes should be flatter than 1 (vert.) to 1 1/2 (hor.) in order to provide sufficient stability to a buried plastic membrane lining , and that ordinary weed growth will not penetrate a plastic film.

Vinyl film has the advantage that it can be jointed readily with a solvent; while, polyethene is jointed by a controlled heat-pressure process.

Filter materials.

Application of these pervious-to-water materials is becoming more common. A great variety of materials and systems exist:

- perforated foils;
- fabrics consisting of thin round threads (usually polyethene or polypropene). They are characterized by a very regular pattern and an open structure;
- fabrics consisting of flat threads, each composed of a few fibres cut from foil. The permeability is less;
- mats consisting of threads composed of many thin polypropene fibres. They have an irregular pattern;
- cloth consisting of nylon threads composed of very thin fibres. This material is very thin and very flexible;
- membrane consisting of threads, sometimes reinforced by a skelet texture. This material has a very irregular pattern.

When making a selection for a particular application, the following properties are important to consider:

- permeability to water and to soil;
- tensile strength in various directions;
- elasticity;
- resistance against tearing, high temperature;
- puncture resistance;
- immunity to biological and chemical effects (ultraviolet light);
- available lengths and widths.

Since there are many materials on the market, it is felt as a necessity to formulate standards for characterizing the various properties and control tests to check whether a certain material meets the requirements set for a particular application.
In order to arrive at this end research is being done in the Netherlands. Some preliminary results of that research regarding permeability requirements are:

- A method is found to determine the characteristic opening of a sheet. In dry conditions a great number of limited fractions are sieved for some time. The sieve is the synthetic fabric itself. From a curve that can be drawn, the values O_{max} as well as other O-values were derived. (O = opening).
  These values reproduce a property of a synthetic fabric. The method is universal and independent of the nature and the structure of the synthetic sheet.
- The resistance of a synthetic sheet against penetration of sand particles depends on the grain size distribution of the soil to be protected, the structure of the fabric, the construction of the revetment including the synthetic sheet and the current condition. Under dynamic load conditions all sand particles smaller than O_{max} pass through the synthetic filter. Under static conditions only smaller particles pass through the filter.
- The starting point for filter design is that the next layer should be more permeable for water than the previous layer, taken in the direction of the ground-water flow. If not, over-pressures will occur. For a certain synthetic sheet the relation is fixed between the pressure drop over the sheet and the filter velocity. The permeability to water appears as a

property of the sheet.

Since synthetic sheets are offered by the international concerns in many countries, international contacts are made in an effort to make collective standards of the procedures. Such standards may also serve as a guide to manufacturers of synthetic materials to be used for civil engineering constructions.

4. Examples of revetments

4.1 General

Examples of traditional revetments in various parts of the world are given in the publication "River training and bank protection" (Bangkok, 1953), of the United Nations (Flood Control Series No. 4).
Reference is made to Section 3 of that Report which deals with bank protection. In addition to some theory, methods of bank protection are discussed and a great variety of constructions applied in the region are described (China, Japan, India, the Philippines).
Section 4 deals with river works and bank protection applied in U. S. A., Europe, Australia and New Zealand.
Since the above mentioned U. N. -report dates back to 1953, some more recently constructed revetments are also discussed.

4.2 Application of plastic filter for drop structures (much turbulence!)

For drop structures leading water into the Lost River in Minnesota, in the past difficulties were met. This was due to the fact that the grain size distribution of the available sand and stone was not suitable to construct an appropriate filter. The filter material leached out through the rip-rap, resulting in undercutting of the revetment. Now, the top gravelly-type of filter is replaced by a plastic filter cloth. It results in a filter consisting of a layer of sand on a foundation material, a plastic cloth on top of it, covered by a top layer consisting of riprap material as shown in Figure 4.1.

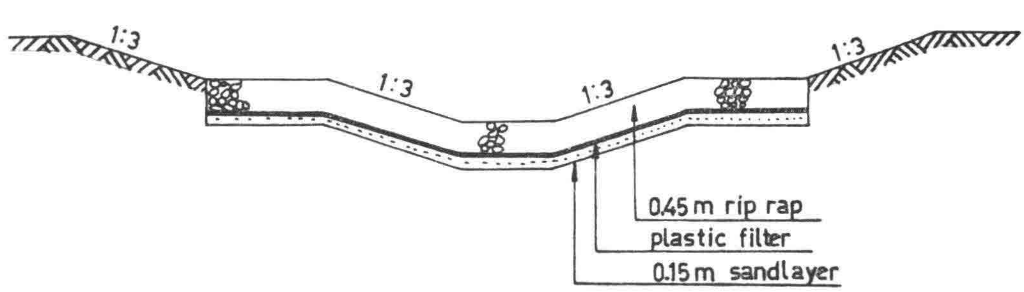


Figure 4.1

The plastic filter must be a cloth which a woven from monofilament yarns of polyvinylidene chloride resin.
The sand layer consists of pit-run sand and gravel which is uniformly graded from the 1 1/2 in. to the no. 200 sieve size.
The riprap must meet the following specifications:

0 to 10 % :	0 to 10 lb	(approximately	0 to 5 kg)
75 to 85 % :	10 to 120 lb	("	5 to 50 kg)
15 to 25 % :	120 to 250 lb	("	50 to 100 kg).

The construction is made under dry conditions.

4. 3 Bank protection of navigation canals and rivers

a. North Sea Canal

In the past the classic construction consisted of a fascine mattress covered with rubble (placed under water) for the part below the water level. Above the water level it consisted of a wooden sheet piling as toe construction of a basalt stone pitching on layers of rubble and clay.
A recently constructed revetment (1969) is shown in Figure 4. 2

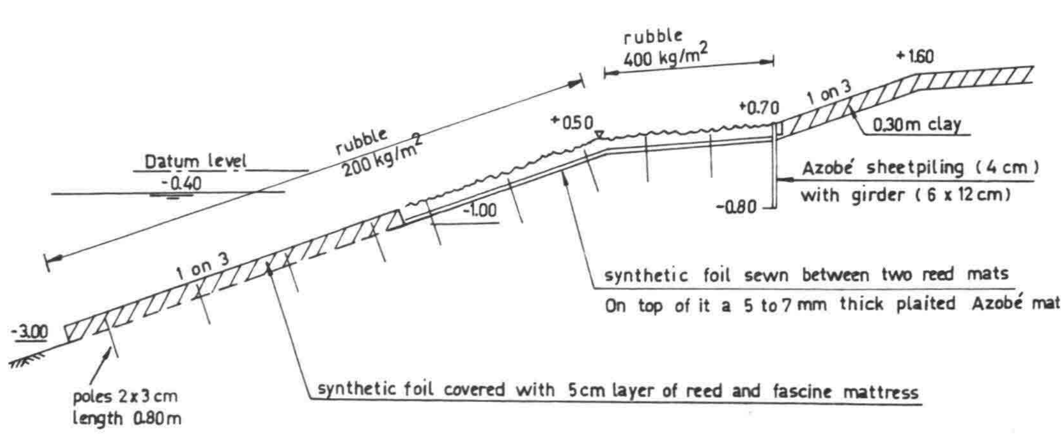


Figure 4.2

Since the canal had to be widened to a considerable extent, the new revetment could be constructed in a building pit under dry conditions. The toe construction, consisting of sheet piling with Azobe-girders, is fitted at 1.10 m above the water level with a clay layer up to 2.00 m above the water level. Below the toe construction on the subsoil a polypropylene foil is placed, sewn between two reed mats above the water line. On top of it a 5 to 7 mm thick plaited mat of Azobe is made. Below the water line the foil is covered with a 5 cm thick layer of reed and fascine mattress. The berm and the slope are loaded with rubble (400 kg/m^2) on the berm; 200 kg/m^2 on the slope).
The revetment is fixed to the bank with poles.
Since the canal is wide, the attack by current and waves is limited. Little maintenance is required. It consists of dumping rubble where necessary.

b. Hartel canal
Along this inland navigation canal, equipped for the largest pushed convoys, an open revetment construction has been chosen. One of the reason for this choice was the relatively high ground-water table of the adjoining lands in relation to the water level of the canal.
The former construction (solution a) consisted of a slope 1 on 4 covered by a 30 cm thick gravel layer, which over a depth of about 20 cm was injected with prepact cement mortar. The gravel was coarse ($>$ 30 mm). Between the levels - 1.00 m and + 1.00 m 50 to 55 litres of mortar was used per m^2 to compensate for the wave attack on this part of the slope. Between the levels - 1.00 m and - 1.75 m 35 to 40 litres mortar per m^2 was used. A disadvantage of this filter construction is that the level of the canal had to be temporarily lowered to - 2.50 m during construction. Although the application of mortar seems to be based on a sound principle, some damage was incurred at a later stage. This is thought to be caused by the conflicting requirements of a certain penetration depth and maintaining permeability.
As soon as the penetration depth is less than 0.20 m and the construction is no longer pervious, overpressure of water gives rise to damage both at and below the water level.

Furthermore, the cement mortar may be subject to mechanical or chemical erosion and, last but not least, the prepact construction is not flexible enough to adapt to the subsoil which is conflicting with one of the basic requirements of a revetment of this type.
Based on that experience, the construction executed later in the same canal is changed and shown as solution b, which can be made under wet conditions.
For this purpose two constructions based on the filter principle were applied. One or more of the filter layers, however, was replaced functionally either by a filter cloth or by a bitumen sand, as indicated in Figure 4. 3.

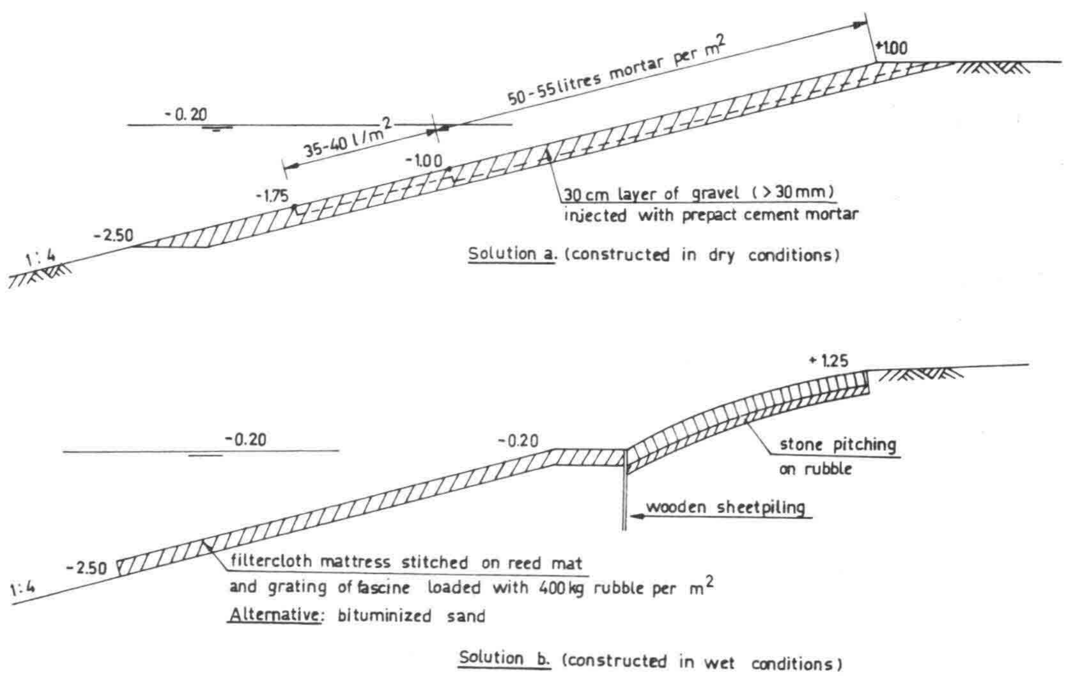


Figure 4.3

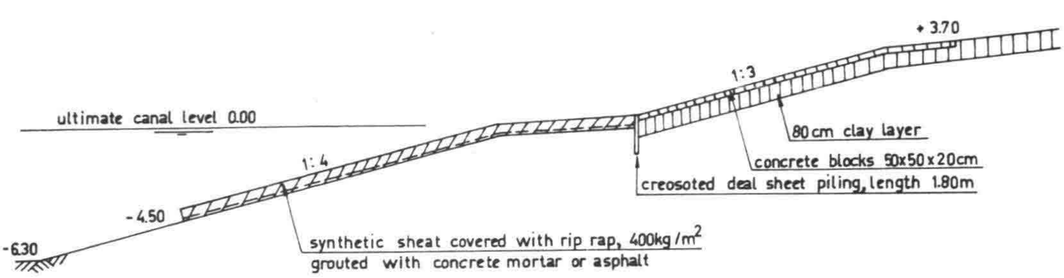
For the filter cloth a polypropene band fabric was used. The requirements for this with regard to impermeability to soil and permeability to water, as well as tensile strength, were determined by laboratory measurements. This band fabric is made rigid by a reed mat stitched on one side and then made in the form of mattresses measuring about 800 m^2 by means of a grating of fascines. The mattresses thus obtained have sufficient floating capacity and rigidity to be transported by water to the construction site.
When placed into position they are sunk and covered with light stone up to a total weight of about 400 kg/m^2.
In the case of application of bituminized sand "special road oil" (SRO) was chosen as a binding agent. This is a flux asphalt bond with a special addition to ensure the permanent fixation qualities in combination with a smaller addition of shell-lime to the mineral. The preparation and processing of this SRO-sand is done with a specially equipped working vehicle. This revetment material has good qualities of permeability and is sufficiently flow resistant with the exception of the direct wave attack areas, where a protection with dumped stone has proved to be necessary.

c. Scheldt - Rhine Canal (1972)
According to whether they have been constructed in dry or wet conditions, the banks of this canal are provided with the following revetments:

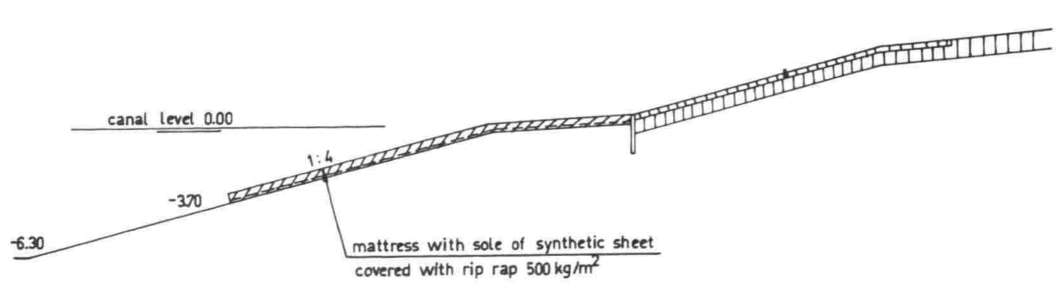
i) in the part constructed in dry conditions between the levels - 4.50 m and 0.00 m synthetic sheet material is used, covered with gravel or riprap and treated either with concrete or asphalt.
The slope between the levels 0.00 m and + 3.70 m is protected by means of concrete blocks (Fig. 4.4)

ii) if constructed in wet conditions the part below the water level is covered with a mattress with a sole of synthetic material on which gravel or riprap is dumped.

In order to arrive at this design, model research was carried out in a hydraulics laboratory to study the stability of the banks and the revetment. In the model (scale 1 : 25) ships of various sizes, including push-tow formations, were sailed along the built-in revetment. The hydraulic phenomena were measured. The conditions regarding the distance of the ships from the revetment, the travel speed, overtaking manoeuvres, etc. were varied as much as possible. The results of the tests, after assessing the damage to the revetment, are applied to the design of the actual revetment. In order to determine possible scale effects of the model, a prototype of one of the model ships was used to perform measurements in a prototype canal.



Constructed in dry conditions



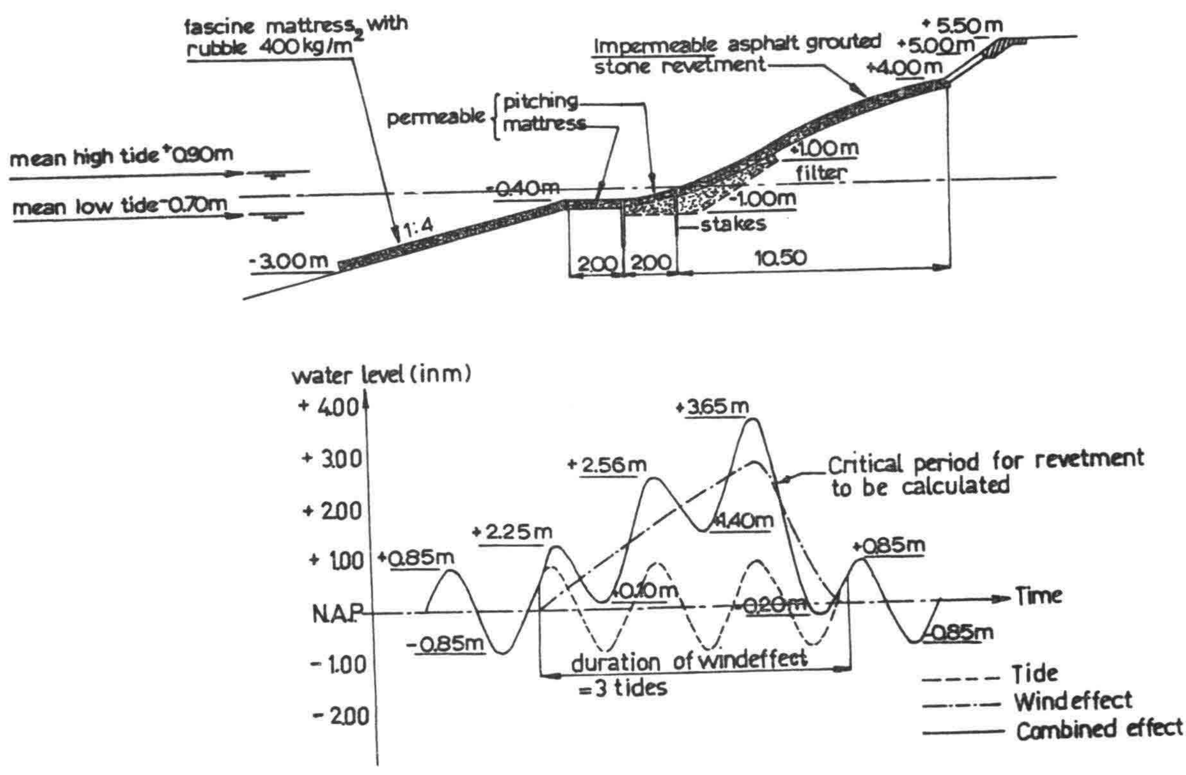
Constructed in wet conditions

Figure 4.4

d. Caland Canal

In this harbour and canal area the fluctuating water level due to tidal influences is decisive for the design of the bank protection.

Until recently an open pitching revetment on rubble filter was used predominantly for the part of the bank above low-tide level; while, on the part below it a light brushwood mattress of limited length was used. Partly because of economic reasons, it was changed over in some cases to a closed bituminous revetment above the mean water level. Before deciding on this change various possibilities were examined, which have resulted in a solution as indicated in Fig. 4.5. For the investigation use was made of an electric model.



Schematical stormsurge as a base for calculation.

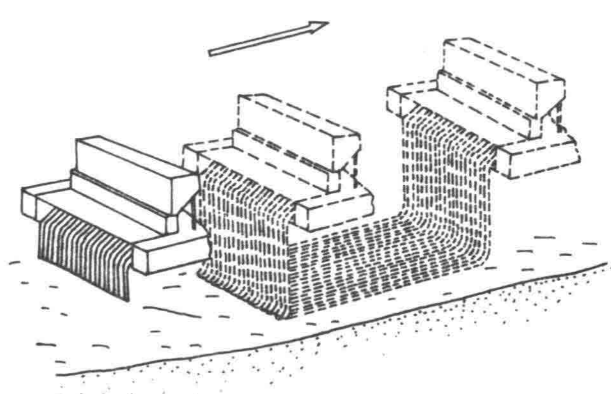
Figure 4.5

This construction, which was built-up of riprap penetrated with liquid asphalt to the level + 4.00 m with a cover of asphalt concrete to + 5.00 m, resulted from the design criterion of a tide during a storm tide as shown in the figure, from the shear stress-criterion and from the uplift-criterion for closed revetments. When using grouted riprap with a stable skeleton, the uplift criterion only applies to that part because with this material no viscous deformations can occur when the shear stress criterion is exceeded.

e. Bank slope and bed revetment applied in Japan

Due to a shortage of brushwood for fascine mattresses and the high costs of stone material, the so-called "sand-bag mattress" method was developed and applied in seaport areas and along rivers.

Bags manufactured from synthetic films are filled by special sand filling equipment and are placed by moving the equipment, as shown in Fig. 4.6 .



Sand-bag mattress laying

Figure 4.6

The shape of the sand-bag mattress is indicated in Fig. 4.7 .

Dimensions of mattress

Figure 4.7

There are no restrictions on the length; therefore, it can be adapted to fit the conditions. Nylons with mesh 0.1 mm and 0.2 mm are used. If necessary cement mortar can be added to the sand in order to prevent damage to the mattress in case of punctures in the nylon bags.

An example of application of this method for protection of a river bank is illustrated in Figure 4.8.

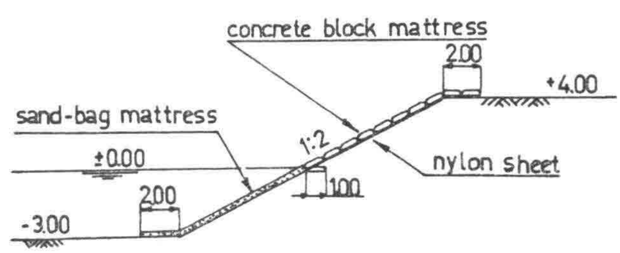


Figure 4.8

The sand-bag mattresses are applied under water because the synthetic resin, although coated, is less durable when exposed to the ultraviolet rays of the sun. Therefore, concrete block mattresses are applied above the water level. In this case 10 pieces of concrete blocks (vertically) and 5 pieces (horizontally) are connected using nylon rope.
Nylon canvas and filter materials are used to prevent drawing out of sand through the joints between the concrete blocks.

f. Yssel (branch of the river Rhine)

First the classic construction is shown in Fig. 4.9.
Under water it consists of a rubble covered fascine mattress; above the water level a stone pitching with toe construction is applied (see detail). In order to satisfy the need to construct bank protections faster and to become less dependent on river discharge (water level), new methods were developed.

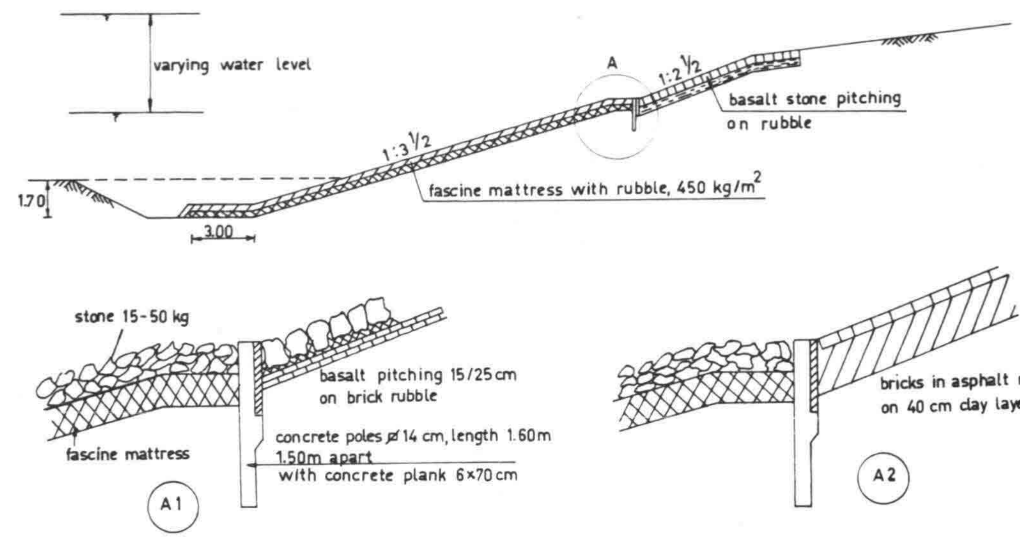


Figure 4.9 (classic construction)

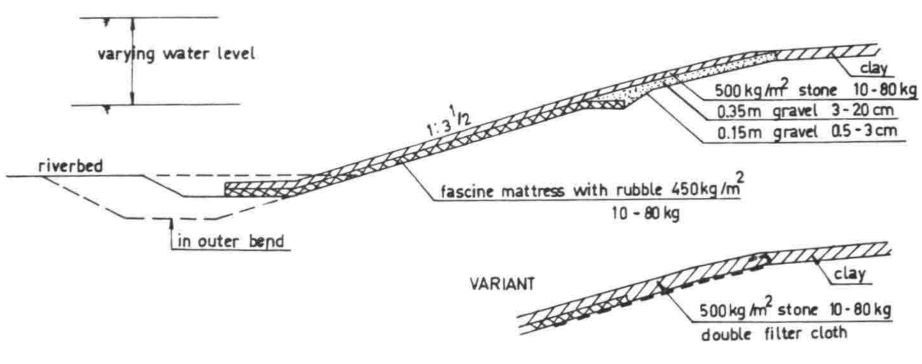


Figure 4.10

Figure 4.10 shows a construction in which the stone pitching is replaced by a rubble layer on filter. As shown in the variant, the filter layers can be replaced functionally by filter cloth.

Finally, excellent results were obtained by applying a filter construction on the entire slope. In the actual example the filter consisted of a 20 cm layer minestone, a 30 cm layer gravel 3-20 cm, covered by 500 kg/m^2 rubble stone 10-80 kg. The difficulty was to place such layers accurately on the underwater slope. Because many kilometers of bank had to be protected, special equipment was designed and constructed. The installation, shown in Figure 4.11, is placed on a pontoon on which a crane, a silo and a system of conveyer belts are assembled.

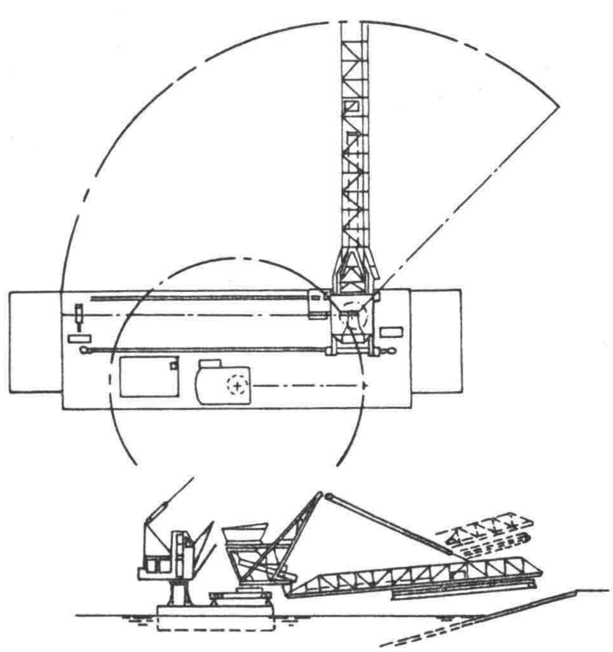
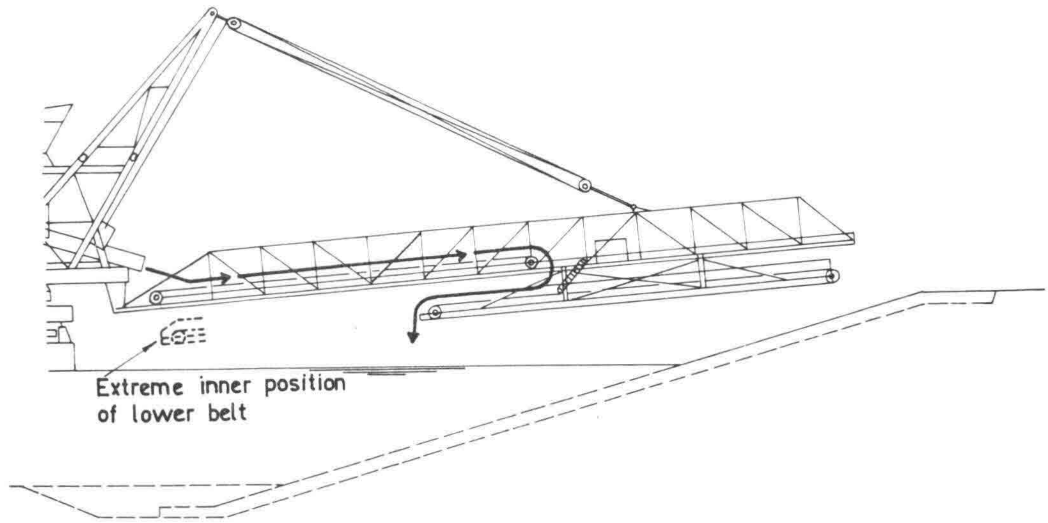


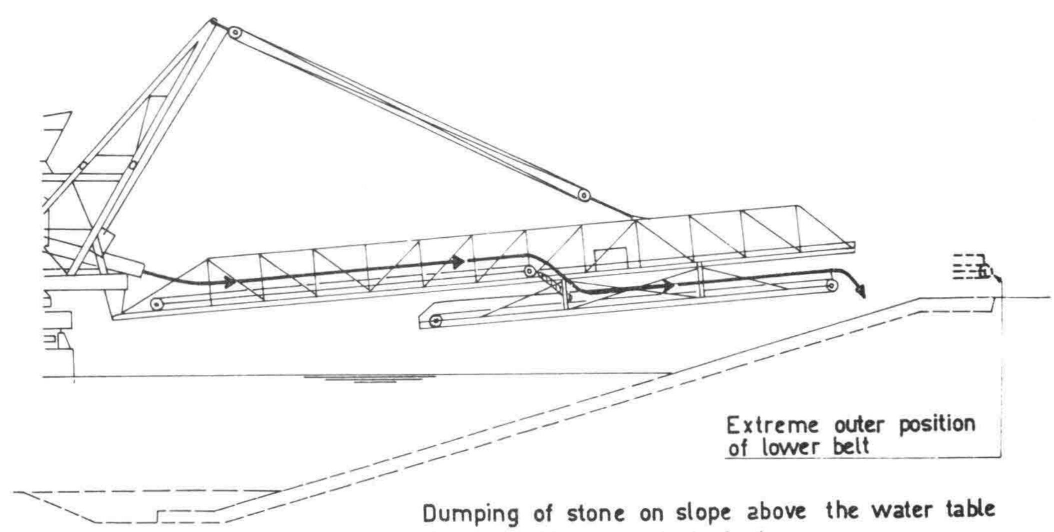
Figure 4.11

The material is transported to the site by boat and deposited in the silo by the crane. There is a vibrating chamber under the silo which can be adjusted to feed the required amount of dumping material to the conveyer belt. This belt is attached to a bridge-shaped steel construction reaching over the slope to be protected. The dumping material falls through the adjustable outlet of the hopper onto the upper conveyer belt, which moves at a constant speed of 0.50 m/s. From this belt it is fed onto the lower conveyer belt which operates at double speed. In this way obstruction in the feeding from the upper to the lower belt is prevented. The lower belt moves along the full length of the steel bridge and can run forward or in reverse as required.
When it has been established that the layer of material dumped above the water line is in accordance with the specifications, dumping can then be continued below the water line. The installation of adjustable switches ensures fully automatic control of the lower belt.
No matter how important the mechanization itself may be, the necessity to employ costly manual labour is no longer required. The main advantages are:

- that it is impossible that any part of the underwater slope can be insufficiently covered (if done manually such a guarantee can hardly be made),
- owing to the fact that the amount dumped can be accurately controlled, a protective covering of more uniformity can be dumped. In practice, this means that the specifications can be met using less material since excess dumping to compensate for possible irregularities becomes unnecessary. In other words: Employing a mechanical stone dumper not only saves labour costs, but yields more economic results owing to the very efficient use of the materials. The procedure of manipulating the conveyer belts is shown more in detail in <u>Figure 4.12</u>.



Dumping of stone on under-water slope
(lower belt moves anti-clockwise)



Dumping of stone on slope above the water table
(lower belt moves clockwise)

Figure 4.12

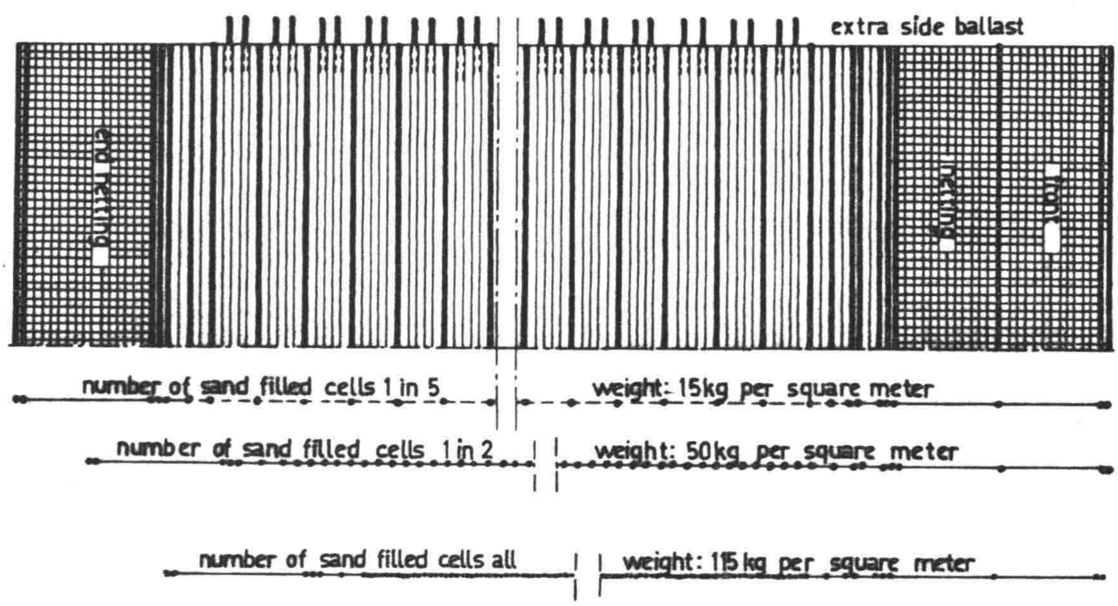
4.4 Bed protection

a. In the framework of the Delta Works in the Netherlands large areas of the bed of sea-arms had to be protected against erosion. In order to obtain the required capacity new methods were developed and new materials were used in place of the classic fascine-mattress method.

Nylon mattresses were applied successfully.

Material and systems:

The material consisted of polyamide in the form of fabrics from which mats are made. The cloth (tensile strength 125 - 350 kg/5 cm; the yarn being 210 - 840 - denier nylon 6) is double woven so that uninterrupted rows of cells (tubes) are formed, as shown in Figure 4.13 .



Nylon mattress

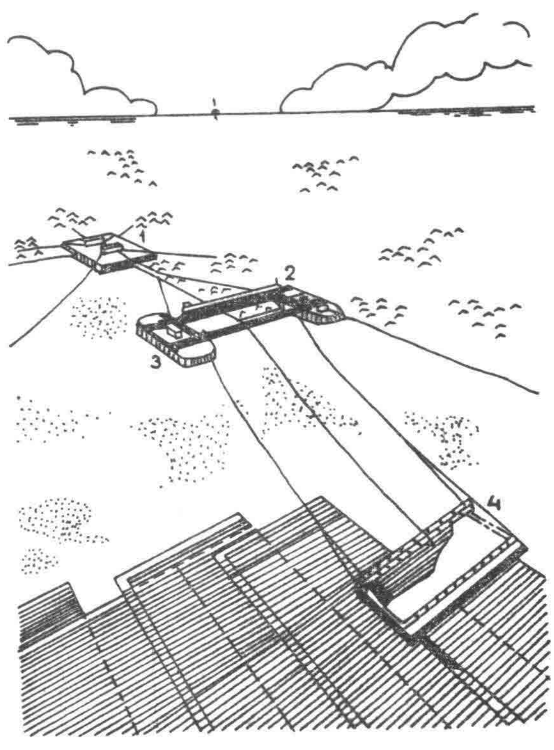
Figure 4.13

The fabric is impervious to sand (grain size 0.1 - 0.2 mm) and permeable to water. In order to resist flowing water (up to 1. to 1.50 m/s) the mattress is ballasted by filling the tubes (ø 7.5 cm) with sand. The number of tubes to be filled depends on the ballast required. (See Figure 4.13 : 15 to 115 kg/m^2). In exceptional cases where more ballast weight is required, the cells are made larger.

Loss of stability of the mattress generally occurs at the edges. Therefore, the edges are additionally weighed; e. g., with two sand-filled nylon tubes attached to the cloth by means of netting. If the mattress is laid carefully it adapts itself well to the profile of the bed. Its resistance to mechanical damage is limited.

Equipment:

Obviously special equipment is needed to place the mattresses on the bed with the required precision. For that purpose the mattress is folded up and placed on a pontoon which consists of a floor between two heavy rolls. The pontoon is moved and controlled by two barges coupled by a frame. After unlocking, the pontoon is lowered to the bed and moved over it. The front netting is then automatically drawn over the front roll so that the pontoon moves over the part already laid, smoothing the mattress over the bed. (Fig. 4. 14) .



1 towing pontoon
2 supply roll and filling chute
3 barges with hoisting frame
4 roller trolley

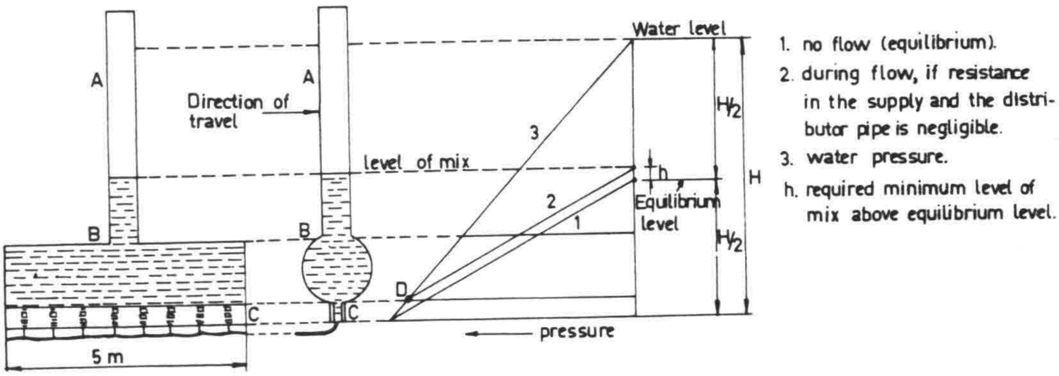
Figure 4.14

Then the pontoon is hoisted up to the frame of the barges and the next length of mattress is prepared on the pontoon floor. To this end the barges are provided with a sand silo from which the ballast sand is pumped into the cells by means of hydraulic fill.

The pontoon is 10 m long and 17.5 m wide; its carrying capacity is 40 tons. In this way nylon mattress of 16.5 x 100 m^2 are placed. After having obtained sufficient experience, a group of 20 skilled labourers on the site were able to attain a capacity of approximately 10.000 m^2 per week.
This type of mattress is also used for the protection of banks (below the water surface). The method of laying has to be adapted. It may be advisable to fill all the tubes with lean sand asphalt mortar to secure the ballast completely.

b. If the current may exceed 1.5 m/s, the mattress described above cannot be applied. Also when mechanical damage is to be feared, other methods have to be used. The protective cover may consist of a layer of rubble stone. If such a layer is asphalt-mastic grouted, a layer is obtained which can resist strong currents and is not liable to mechanical damage. Since such a layer is also impervious to water, care should be taken that its weight exceeds any possible overpressure of water under the layer.
The asphalt mastic consists of a mixture of sand (50 - 60 %), filler (10 - 20 %) and bitumen (about 20 %, penetration 10 - 500).
For bed protection in deep water, equipment is developed which makes it possible to work at depths up to 30 m and a maximum slope of 1 in 5 to 1 in 10.
On the one hand the mastic mix must have a low enough viscosity (i.e. 5.10^2 to 5.10^3 poises) to permit gravity flow through a pipe at about 120° C (higher temperatures would cause steam to form underwater).
On the other hand, after cooling to water temperature, the viscosity must be high enough to prevent excessive flow of the mix. Laboratory and practical tests have proved that it is possible to obtain mixtures meeting such requirements.
Equipment:
Essentially the equipment is simple (Fig. 4.15).
The mix is supplied through pipe A; in B it is distributed over a width of 5 m, which has proved to be a suitable size; at C it issues in jets through eight pipes. With proper dimensions and with a suitable number of these pipes, the jets will unite on a

flat bed to form a layer of uniform thickness or will evenly grout a stone layer.



Sketch and pressure diagram of apparatus

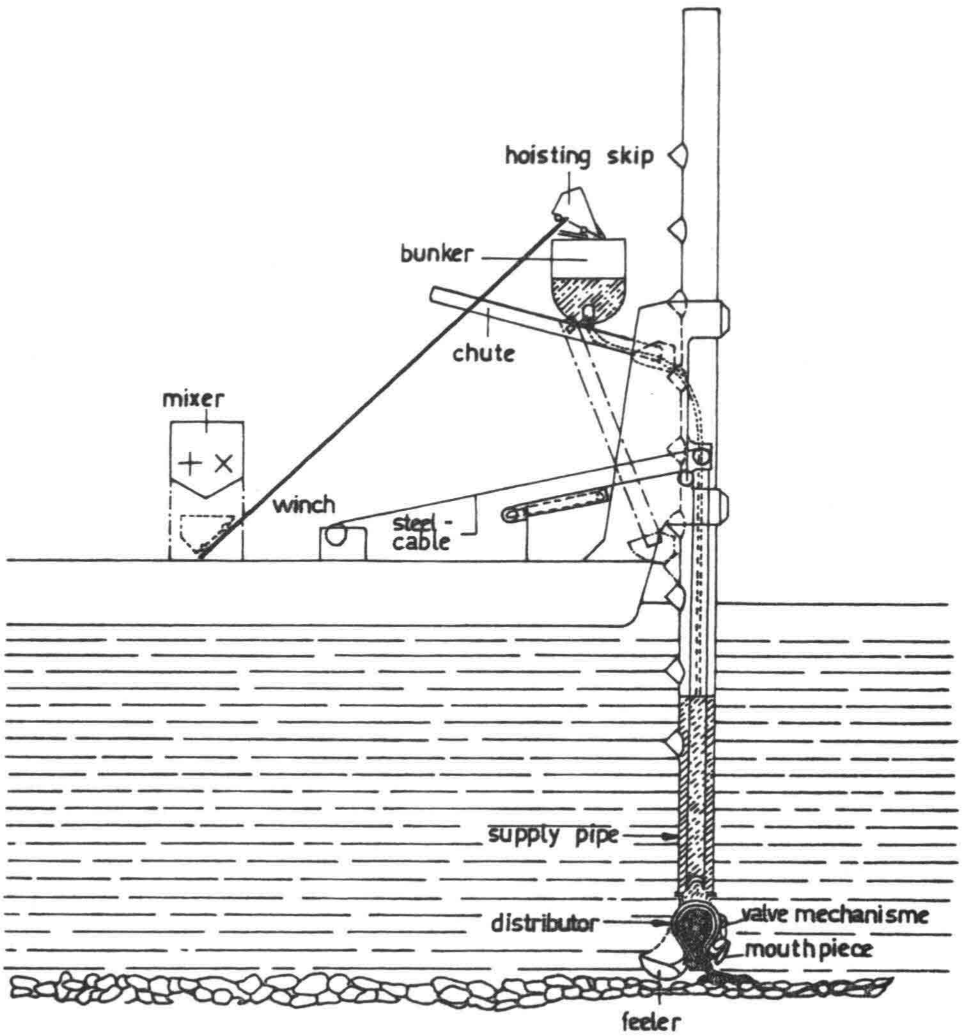
Figure 4.15

By making the orifice pipes narrow the viscous resistance is concentrated in the pipes. This serves the double purpose of producing uniform flow and keeping the water out, thus preventing steam formation inside. Regarding the latter reason the pressure in point D must be higher than outside at the same level. This is achieved by keeping the level of the mix above the equilibrium level. The equilibrium level is about halfway up the apparatus because the specific gravity of the mix is nearly 2.

The orifices can be closed with a valve.

The apparatus (weight when filled 70 tons) is fastened in the vertical position to a ship carrying an asphalt mixing plant (Fig. 4.16). It can be moved up and down by means of steel cables and a powerful winch so that any depth between 2 and 20 m can be reached. A feeler, operating a hydralic lifting mechanism, ensures that the nozzle is always at the same distance from the bottom. A mastics kettle (serving as a bunker) between mixer and apparatus ensures a continuous supply.

In actual operation the bunker is first filled to capacity. Then the apparatus, being still entirely above water, is filled through the lowest charging opening while the valve at the bottom is shut. The apparatus is then lowered to the bottom after the lower charging openings have been shut. Then the bottom valve is opened and the ship is moved in the direction required, mastics being supplied through a higher opening depending on the depth at which it must be supplied. The chute from bunker to charging opening can move so as to permit filling through the same opening over a certain vertical range (5 m).



Sketch of assembly of apparatus

Figure 4.16

c. An interesting development are mattresses to which the ballast is fixed. Dumping of ballast material, after placement of the mattress, is only necessary under extreme conditions.
Several systems are investigated. Hereafter one application is described; i. e. the so-called concrete-block mattress.
A reed layer is fixed to the polypropene filter cloth (750 gr/m^2) in order to protect it against damage. Concrete blocks are fixed

to the mattress by pens. These cover about 50 % of the total area of the mattress, as shown in Figure 4. 17.
The mattress is manufactured in a factory and winded on a cylinder. The cylinder with mattress is then transported in floating condition to the site and there fixed to a pontoon. The mattress if placed using an anchored sinking beam and moving the pontoon slowly in the direction of the current (Fig. 4. 18). Mattresses of 30 x 200 m^2 are placed in this way. Application of this method is considered as a promising development.

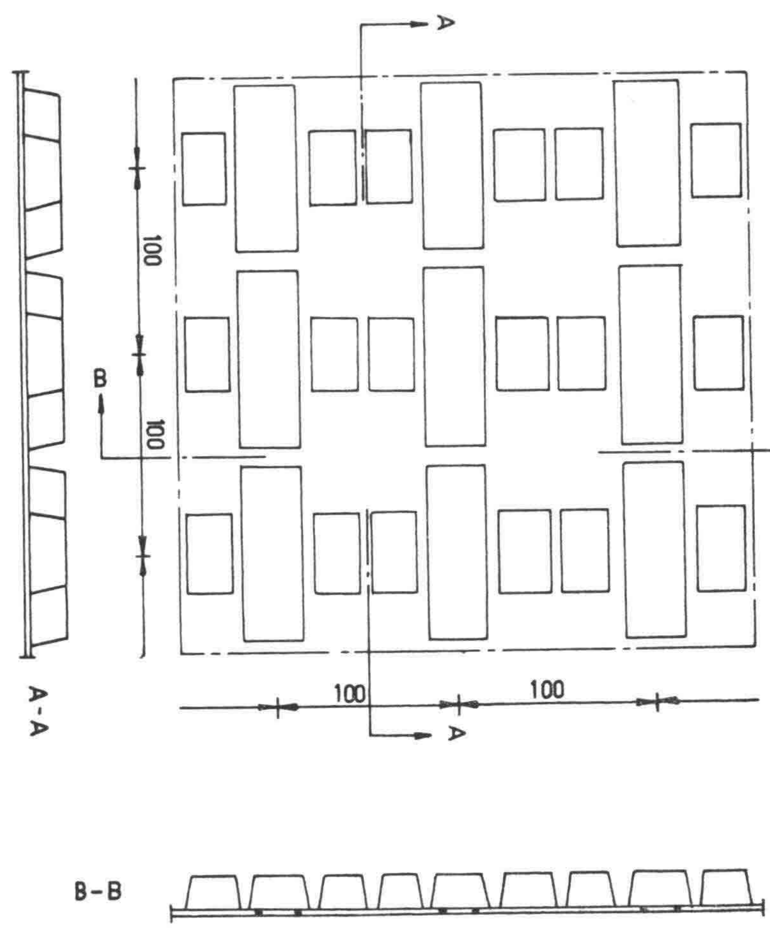


Figure 4-17

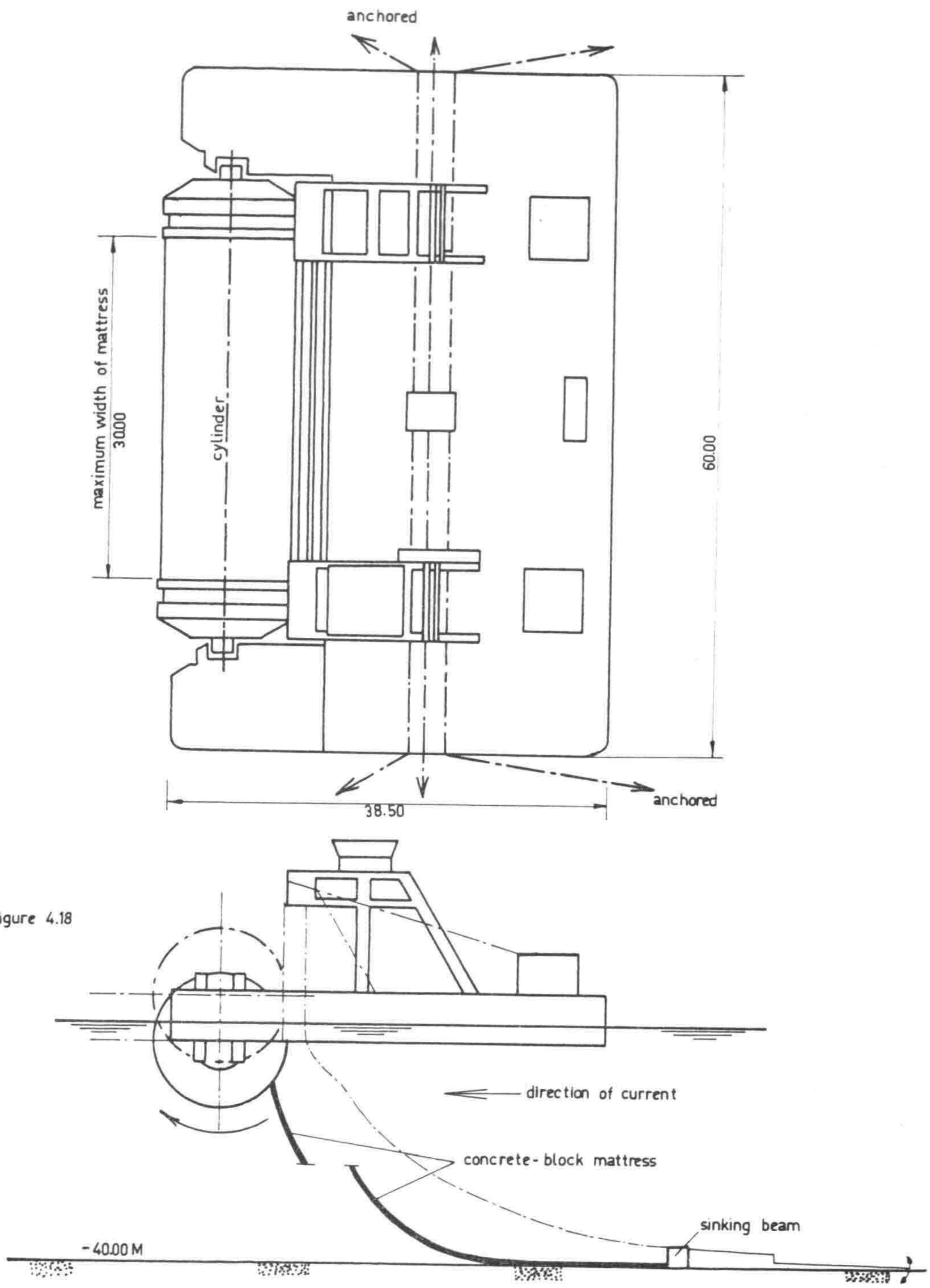

anchored
maximum width of mattress
30.00
cylinder
60.00
38.50
anchored
direction of current
concrete-block mattress
sinking beam
-40.00 M

Figure 4.18