CUR

Centre for Civil Engineering Research and Codes

Design manual for pitched slope protection

Ministry of Transport, Public Works and Water Management

Road and Hydraulic Engineering Division

Directorate-General for Public Works and Water Management

DESIGN MANUAL FOR PITCHED SLOPE PROTECTION

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FOREWORD

The "Guidelines for Concrete Dike Revetments" appeared in 1984 as a joint publication of the Technical Advisory Committee on Water Defences (TAW) and CUR (Report 119). The manual did not treat design aspects in detail because fundamental research into the stability of pitched slope protection was still in progress. By 1988 this research had reached a stage at which TAW and CUR set up a commission with the terms of reference to prepare a practical review of the results of the research with particular reference to design aspects.

The CUR C 74 "Concrete Dike Revetments" Research Commission, set up at the end of 1988, was given the following tasks:

- to check the design criteria obtained by research against practical experience,
- to adapt the design criteria into a practical and concise form and to promote the application of criteria still to be formulated, and
- based on the newly acquired understanding of the subject to establish a better inventory of pitched revetments.

The original aim of preparing a "Guidelines for Concrete Dike Revetments – Part 2" has been changed for the following reasons:

- In the past all official TAW publications have been referred to as "guidelines". Recently however it was decided that books which establish the state of the art should be called "manuals", the term "guidelines" now being reserved for publications which set out legal frameworks and policy (hence the title of the present manual).
- The present manual covers pitched dike revetments which can be of concrete or stone. Although block mattresses are also considered the Commission is of the opinion that the present title, "Design Manual for Pitched Slope Protection", gives a better impression of the contents than the original title.
- Since 1984, when the old "Guidelines for Concrete Dike Revetments" was published, the understanding of the subject has grown to such an extent that the present manual should not be regarded as a supplement (as Part 2) to the original guidelines but rather should be seen as an independent publication.

The Commission comprised:

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The manual was written by Ir. M. Klein Breteler of Delft Hydraulics, with contributions by members of the Commission. Thanks are expressed to all who have contributed to the preparation of the manual. CUR and TAW would like to thank the Netherlands Cement Industry Association for its financial support.

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CONTENTS

NOTATION

A	average column or block area	[m^2]
A_C	area of the wet channel (canal) cross-section	[m^2]
A_M	wet cross-sectional area of the beam of a ship	[m^2]
a	co-ordinate up the slope	[m]
B	berm width	[m]
B_s	ship's beam	[m]
b	thickness of the filter layer	[m]
b_1,b_2	thickness of Filter Layers 1 and 2	[m]
b_m	thickness of the minestone layer	[m]
b_u	thickness of the filler layer	[m]
b_w	width of the navigation channel at the water surface	[m]
C_m	model coefficient	[–]
C_M	beam coefficient	[–]
CW	market value	[Dfl]
CW_f	market value factor	[Dfl]
c	constant	[–]
c	coefficient	[–]
D	thickness of the cover layer (block thickness)	[m]
D_{b90}	grain size of the base (sand), 90 % by weight of which is less than the stated size	[m]
D_{bx}	grain size of the base (sand), x % by weight of which is less than the stated size	[m]
D_{f15}	grain size of the filter, 15 % by weight of which is less than the stated size	[m]
D_{fx}	grain size of the filter, x % by weight of which is less than the stated size	[m]
D_g	hole diameter	[m]
D_{v15}	grain size of the material in the hole or joint (washed-in material), 15 % by weight of which is less than the stated size	[m]
D_{v15g}	grain size of the material in the hole (washed-in material), 15 % by weight of which is less than the stated size	[m]
D_{v15s}	grain size of the material in the joint (washed-in material), 15 % by weight of which is less than the stated size	[m]
D_{v15kar}	$\mu(D_{v15}) - 2 \cdot \sigma(D_{v15})$ – see Section 13.4.2	[m]
D_{u15}	grain size of the filler layer, 15 % by weight which is less than the stated size	[m]
d	ruling water depth – see Figure 49	[m]
d_a	water level behind the dike relative to the crest	[m]
d_h	water depth on the berm (f_i the berm is above still water level, d_h < o)	[m]
d_k	water depth on the crest	[m]
d_o	depth of the lower and upper limit of the pitching below the still water line (if the pitching is above SWL, d_o is negative)	[m]

d_o	water level in front of the dike relative to the crest	[m]
d_s	level (relative to the still water line) where the pressure head difference on the cover layer is the maximum = the intersection of the pressure head difference front and the slope relative to the still water line	[m]
d_t	water depth at the toe of the slope, see Figure 49	[m]
e	natural logarithm base = 2.718	[–]
F	load on the toe or the anchor per metre length of dike	[N/m]
F	wind fetch	[m]
F_a	shear force per metre length of dike	[N/m]
F_g	gravity	[N]
F_i	wind fetch in the direction β_i, relative to the central orthogonal	[m]
F_n	block weight component perpendicular to the slope	[N]
F_w	shear force which can be absorbed in zones not under attack, per metre length of dike	[N/m]
F_w	force of friction	[N]
$F_\varnothing$	force on the block due to the difference in pressure head	[N]
g	acceleration due to gravity	[m/s²]
H	ruling wave height (force) (incoming waves)	[m]
H	height of equivalent secondary ship wave approaching normal to the dike	[m]
H_{cr}	height of incoming waves which cause cover layer failure (strength)	[m]
H_i	height of secondary ship waves	[m]
H_{kruin}	effective or equivalent wave height on the crest	[m]
H_k	percentage of organic material	[%]
H_s	significant wave height (force) (incoming waves)	[m]
H_{scr}	significant wave height of incoming waves which cause cover layer failure (strength)	[m]
H_{skar}	$\mu(H_s) + 1.65 \cdot \sigma(H_s)$, see Section 13.4.2	[m]
H_{so}	significant deep water wave height (incoming waves)	[m]
$H_{x\,\%}$	wave height exceeded by x % of waves	[m]
$H_{2\,\%}$	wave height exceeded by 2 % of waves	[m]
h	local water depth (general)	[m]
$\bar{h}$	average fall in water level	[m]
$\hat{h}$	maximum fall in water level at the bank	[m]
h_c	crest height relative to the still water level	[m]
$\hat{h}_f$	bow wave height	[m]
hoh	centre-to-centre distance (between holes)	[m]
I_c	Consistency Index = $(W_l - W_k)/I_p$	[%]
I_p	Plasticity Index = $(W_l - W_p)$	[%]
i	real interest base	[–]
i	hydraulic gradient (loading)	[–]
i'	hydraulic gradient on the cover layer (loading)	[–]
$i_\uparrow$	hydraulic gradient parallel to the cover layer, up the slope (loading)	[–]

$i_{\downarrow}$ hydraulic gradient parallel to the cover layer, down the slope (loading) [–]
i_{cr} maximum allowable hydraulic gradient along the interface (strength) [–]
$i_{\downarrow cr}$ maximum allowable hydraulic gradient parallel to the cover layer down the slope (strength) [–]
$i_{\uparrow cr}$ maximum allowable hydraulic gradient parallel to the cover layer up the slope (strength) [–]
i_f hydraulic gradient in a bow wave [–]
i_{max} hydraulic gradient in a stern wave [–]
k permeability of the filter ($q = k \cdot i$) [m/s]
k' permeability of the cover layer ($q' = k' \cdot i'$) [m/s]
k'_g permeability of the cover layer, holes only (joints not taken into account) [m/s]
k_m permeability of minestone [m/s]
k'_s permeability of the cover layer, joints only, (holes not taken into account) [m/s]
k_u permeability of filler layer [m/s]
k_1, k_2 permeability of Filter Layers 1 and 2 [m/s]
L block length (parallel to the axis of the dike) [m]
L_{es} consolidation length [m]
L_{gp} wave length in shallow water (based on T_p) [m]
L_k clay content = percentage of particles smaller than 2μm [%]
L_{op} wave length in deep water (based on T_p) = $gT^2_p/(2\pi)$ [m]
L_o wave length in deep water = $gT^2/(2\pi)$ [m]
L_s ship length [m]
L_t length of the slope [m]
L_{wi} length of secondary ship wave [m]
N number of holes [–]
N number of years [–]
n_{vkar} 0.3 (see Section 13.4.2) [m]
n porosity [–]
n_L length scale [–]
n_f porosity of the filter [–]
n_k ripening factor [–]
n_v porosity of the hole or joint filler (washed-in material) [–]
n_{vg} porosity of the hole filler (washed-in material) [–]
n_{vs} porosity of the joint filler (washed-in material) [–]
n_u porosity of the filler layer [–]
n_z porosity of the sand [–]
O_{90} characteristic size of an opening in the geotextile [m]
P factor dependent on the method of placing stones (P = 0.6 for rip rap, P = 1.1 for neatly placed rip rap, $P \approx$ 1.25 for pitched stones) [–]
P_b parameter characterizing the distance up the slope from the highest transition structure [–]

P_o	parameter characterizing the distance down the slope from the lowest transition structure	[–]
p	pressure	[N/m^2]
P	probability	[–]
P	probability of exceedance	[–]
q	filter velocity (discharge per m^2 of flow profile; specific discharge); during the measurement of permeability of a geotextile	[m/s]
q	filter velocity (specific discharge)	[m/s]
q	filter velocity through the cover layer (specific discharge; discharge per m^2 of pitching)	[m/s]
q_{cr}	maximum allowable filter velocity (strength)	[m/s]
S_k	silt content = percentage of particles between 2 and μm and 63 μm	[%]
SWL	still water line (level)	
s_{kar}	$\mu(s) - 2.3 \cdot \sigma(s)$ (see Section 13.4.2)	[m]
s	joint width	[m]
T	ruling wave period	[s]
T_g	thickness of geotextile	[m]
T_i	period of secondary ship waves	[s]
T_{ong}	average unloaded clearance over the length of the ship	[m]
T_p	wave period at the peak of the spectrum	[s]
T_s	loaded clearance under a ship	[m]
T_z	average wave period	[s]
$T_{1/3}$	significant wave period	[s]
t	time	[s]
t	storm duration	[s]
U	present value	[*f*]
u	wind speed at 10 m above the water surface	[m/s]
u_{cr}	maximum allowable current velocity over the crest (strength)	[m/s]
u_k	current velocity over the crest (load)	[m/s]
v_s	ship speed	[m/s]
W_L	air content of the water	[–]
W_c	compressibility of pore water containing air	[m^2/N]
W_k	weight percentage of water	[%]
W_l	Liquid Limit = water content at which a groove in clay almost closes, when the sample is dropped 25 times from a height of 1 cm onto a firm surface	[%]
W_{opt}	optimum water content of the clay	[–]
W_p	Plastic Limit = limit of rollability = the water content at which a small ball of clay can just be rolled into a thread of 3 mm diameter, without it crumbling	[%]
X	lower limit of grain size denoted for the category of filter	[mm]
x	general variable	[m]

x	local co-ordinate parallel to the axis of the dike	[m]
x_{KAR}	characteristic value of x	[..]
x_{max}	upper limit of the reliability interval of the variable x	
x_{min}	lower limit of the reliability interval the of the variable x	
Y	upper limit of grain size denoted for the filter category	[mm]
y	local horizontal co-ordinate perpendicular to the axis of the dike	[m]
y	distance between the axis of the ship and the axis of the navigation channel	[m]
y_a	distance, measured along the slope, from the still water line to the uppermost block of a block mattress (used for an anchor structure)	[m]
y_t	distance, measured along the slope, from the still water line to the (lowest) toe structure	[m]
Z_k	sand content = percentage of particles larger than 63μm	[%]
z	local vertical co-ordinate	[m]
z'	local co-ordinate perpendicular on the slope	[m]
z_{max}	stern wave height	[m]
$z_{2\%}$	ruling wave run-up = wave run-up level relative to the still water line (SWL) which is exceeded by 2 % of the waves	[m]
$(\cdot)_{max}$	largest value	[..]
$(\cdot)_{min}$	smallest value	[..]
α	slope angle (from horizontal)	[°]
α_i	coefficient in the equation for secondary waves	[–]
α_v	slope of the foreshore relative to horizontal	[°]
β	angle of wave attack relative to the structure	[°]
β_i	angle of wave attack relative to a central wave ray	[°]
Γ	relationship between the maximum pressure head difference on the cover layer and equivalent to the weight of the block = the influence factor for possible transition structures, the friction, the inertia and the entry flow $(\Gamma_1 + \Gamma_2 + \Gamma_3)/\Gamma_0$	[–]
Γ_0	factor which takes into account the effect of transition structure on the pressure head difference on the cover layer	[–]
Γ_1	friction factor	[–]
Γ_2	inertia factor	[–]
Γ_3	entry flow factor	[–]
γ_b	berm reduction factor related to the effect on wave run-up = $Z_{2\%}$, with berm/$Z_{2\%}$, without berm	[–]
γ_r	slope surface roughness reduction factor related to the effect on wave run-up (for block pitching, $\gamma_r = 1$)	[–]
γ_{sg}	geotextile factor affecting the permeability of a cover layer with holes	[–]
γ_{ss}	geotextile factor affecting the permeability of a cover layer with joints	[–]
γ_β	angle of attack reduction factor for waves related to the effect on wave run-up (for normal wave attack $\gamma_\beta = 1$)	[–]

Δ	relative volumetric mass of stone = $(\rho_b - \rho)/\rho$	[–]
$\Delta\alpha$	(infinitely) small distance	[m]
Δ_f	relative volumetric mass of filter grains = $(\rho_f - \rho)/\rho$	[–]
Δ_z	relative volumetric mass of sand grains = $(\rho_z - \rho)/\rho$	[–]
η	water level relative to the still water line	[m]
θ	slope of the pressure head front relative to horizontal	[°]
Λ	leakage length = $\sqrt{(bDk/k')}$	[m]
λ	leakage height = sin $\alpha\sqrt{(bDk/k')}$	[m]
$\mu(..)$	expectancy value	[..]
ν	viscosity of water	[m²/s]
ξ_o	breaker parameter based on the ruling wave height and period = tan $\alpha/\sqrt{H/L_o}$	[–]
ξ_{op}	breaker parameter based on the peak period of irregular waves = tan $\alpha/\sqrt{H/L_o}$	[–]
ρ	volumetric mass of the water	[kg/m³]
ρ_f	volumetric mass of the filter grains	[kg/m³]
ρ_z	volumetric mass of the sand grains	[kg/m³]
ρ_b	volumetric mass of the blocks	[kg/m³]
$\Sigma(..)$	summation	-
σ_b	$(\rho_b - \rho)\, gD + (1 - n)\,(\rho_f - \rho)gb$ = grain stress necessary required for a stable structure, caused by the weight of the blocks and the filter layer under water	[N/m²]
$\sigma(..)$	standard variation = ¼ of the width of a 95 % reliability interval $\left(\approx \frac{\text{upper limit} - \text{lower limit}}{4}\right)$	[..]
Φ	angle of internal friction of the subsoil (minimum: 30° for loose non-compacted sand, 35° to 40° for compact sand)	[°]
Φ	angle of friction between the cover layer and the layer directly underneath	[°]
ϕ_A	amplitude of the pressure head directly above the subsoil in the filter layer	[m]
ϕ	pressure head (in the filter)	[m]
ϕ'	pressure head on the cover layer	[m]
ϕ_b	pressure head under the incoming wave crest relative to the point at which the pressure head front meets the revetment slope	[m]
ϕ_g	reduction in pressure head on the geotextile during permeability measurements	[m]
ϕ_t	difference in pressure head on the cover layer	[m]
$\phi_{(z,t)}$	pressure head in the subsoil at z' at time t	[m]
ϕ_1	difference in pressure head across the cover layer as a result of the resistance to through flow	[m]
ϕ_2	difference in pressure head across the geotextile	[m]

ϕ_3	difference in pressure head across the cover layer as a result of the in and out flow resistance	[m]
ϕ_4	reduction in pressure head in joints or holes in the cover layer	[m]
Ω	relationship between the surface area of joints and holes and the total pitched area, per section of the cover layer, that is, a section of the pitched area through which water can pass	[–]

SUMMARY

This manual describes current methods for designing dike revetments of pitched blocks and block mattresses. The use of such revetments on river and canal banks is also considered briefly. In particular, guidelines are discussed for preparing designs for new revetments; methods are also given for checking existing revetments. The manual is aimed at the practical application of the results of basic research into block pitching carried out by Delft Hydraulics and Delft Geotechnics for the Technical Advisory Committee on Water Defences. Reference should be made to [BEZUIJEN, BURGER and KLEIN BRETELER, 1990] for a description of the research.

Pitched dike revetments include the following revetment systems:
- basalt and other natural rock, laid in a single layer;
- concrete blocks and columns and other small concrete elements which are laid in a single layer;
- block mattresses, made up of small concrete elements, which are fastened together by cables or via a geotextile.

Slab revetments, comprising elements larger than about 1 m^2 are not discussed here. The design methods presented take into account the properties of the cover layer and the sublayers, both of which are important for stability. The sublayers can include layers of granular material, geotextiles and/or clay layers.

Two design methods are worked out in detail:
- the Preliminary Design Method and
- the Analytical Design Method

The Preliminary Design Method considers only one method of failure, the lifting of one block out of the cover layer by wave action.

The method is based on a considerable collection of practical data, results of large scale model investigations and calculations using the Analytical Design Method and the STEENZET/1+ numerical method. The method is quick and easy to use but gives less accurate results than the Analytical Design Method for structures in which a granular layer is laid underneath the cover layer.

The Analytical Design Method is based on equations which describe the physical processes in detail. It can be used to assess the stability of the cover layer and that of the interface between the granular filter and the subsoil.

Geotechnical instability caused by wave action is treated separately.

The construction aspects are discussed principally in relation to the transition structures and the design aspects of the structure as a whole.

Analytical Design Method	Detailed design method used to determine the thickness of the cover layer and the properties of the filter – Section 8.4.1
ANAMOS	User-friendly PC-program for assessing the stability of the cover layer, the possibility of the filter sanding up and the danger of the cover layer sliding (based on the Analytical Design Method) – Section 8.4.1
Atmospheric oscillations	Periodic fluctuations in water level (long waves) with a period of 15 to 45 minutes, resulting from macroscopic turbulence in the atmosphere – Section 6.1
Basis	Sand or clay under a granular filler or a geotextile (subsoil) – Chapter 2
Block mattress	Prefabricated blocks bonded into a mattress by cables or a geotextile – Chapter 2
Block pitching	Cover layer bonded blocks – Chapter 2
Block surface	Total surface area of block measured in the largest cross section in the plane of the slope, including any holes – Section 5.2
Bonded filter	Filter in which the individual grains are bonded together by a binder – Section 5.4
Bow wave	Wave caused by the bow of a ship – Section 6.3
Breaker parameter	Relationship between the slope of the revetment and (square root of) the wave steepness which gives an indication of the way in which waves break on the slope – Section 6.2.2
Breaking	Dissipation of wave energy as a result of restricted water depth or excessive wave steepness which leads to a reduction in wave height (period tending to be unaffected) – Section 6.2.2
Breaking index	Relationship between the maximum wave height (with respect to breaking) and the water depth – Section 6.2.2
Camber	The rounding (in the vertical cross-section) of the profile of the cover layer – Section 7.7
Characteristic value	Ruling (safe) value, which, for the particular design method, leads to the possibility of failure which cannot be exceeded – Section 13.4.2
Clay or lutum content	Proportion of particles smaller than 2μm – Section 5.8
Collapsing breaker	A wave breaking on a fairly steep slope with a steep, violent, foaming and turbulent wave front (not yet plunging) – Section 6.2.2

Column	Block with an (almost) flat top and bottom and a prismatic horizontal cross-section with more than four corners – Chapter 2
Cover layer	Outer protective layer, comprising bonded blocks – Chapter 2
Cover layer thickness	Thickness of the blocks in a revetment, measured perpendicular to the slope – Section 5.2
Density	Relationship between volume and mass of material, such as concrete, grains without pores, etc, (volumetric mass, specific mass) – Section 8.2.3 and Chapter 9
Deterministic calculation method	Method used to determine the stability of a dike revetment based on (safe) values of load and structural parameters – Chapter 13
Diffraction	The phenomena which occurs when, for example, waves pass behind an island (Section 6.2.2)
Dike core	The soil, comprising sand and/or clay which must be protected against the effects of water movements, for example, by means of pitched dike revetments – Chapter 2
Diked area	Area protected against flooding by a system of hydraulic structures – Section 13.1
Drop in water level	Temporary reduction in water level in a canal during the passage of a ship – Section 6.3
Effective or equivalent wave height	Calculated value for the loads on the crest due to wave overtopping (run-up height minus crest height) – Section 6.5
Entry flow	Flow of water towards the space which grows under a block when it is moved off the slope and which leads to a lowering of pressure head relative to that under a block which has not moved – Section 8.3
Fetch length	Length of sea or lake in the direction from which the wind comes – Section 6.2.2
Filler layer	Thin layer of granular material for filling the uneven surface of the layer below the cover layer to create a smooth surface on which to place the blocks – Section 5.3.1
Filter velocity	Discharge per unit area perpendicular to the flow direction (pores + grains) (specific discharge) – Section 8.2.2
Flow resistance	Relationship between the hydraulic gradient and (specific) discharge (reciprocal of permeability) – Sections 5.2 and 8.2.3
Geotechnical instability	Collective name for three related failure mechanisms: (1) deformation of the filter and/or base into an S-profile (slip circle), (2) soil movements in the base and (3) sliding of the cover layer – Chapter 9

Geotextile	Plastic cloth applied, for example, to sand or clay to prevent them being flushed into the filter – Section 5.7
Granular filter	A layer of relatively fine granular material of limited thickness (rubble, minestone, etc) – Section 5.3
Ground area	Surface area of a block, measured in the plane of the slope at the plane of contact with the subsoil, excluding any holes – Section 5.2
Grouting mortar	Binding material for example, molten asphalt, which can be injected between the blocks – Section 5.6
Hydraulic gradient	Change in hydraulic pressure head per unit length (pressure head gradient) – Section 8.2.2
Interface	Surface of contact between the base (sand or clay) and the filter (granular filter or geotextile) – Section 8.2.1
Interlocking blocks	Prefabricated blocks which, because of their shape, lock together and cannot be lifted out of the pitching – Chapter 2
Inertia	Force needed to accelerate the movement of a block – Section 8.4.3
Internal instability	Migration of the fine fraction of a filter layer through the pores of the filter – Section 5.3.1
Joint	Space between two adjacent blocks or blocks lying one above the other in pitching, possibly filled with washed-in material – Section 5.2
Leakage height	Vertical component of the leakage length (leakage height = leakage length · $\sin\alpha$ – Section 8.2.3
Leakage length	Parameter which is dominated by the permeability ratio of filter layer and cover layer. It determines the value of the pressure head difference over the cover layer and the gradient in the filter (in case of large leakage length, the pressure head difference over the cover layer is large and the gradient in the filter small) – Section 8.2.3
Linked elements	Block mattresses or interlocking stones – Section 5.2
Liquid limit	The water content at which a groove in the clay almost seals itself, after a sample is dropped 25 times from a height of 1 cm on to a firm surface – Section 5.8
Loads	Wave heights (or pressure head differences on the cover layer or in the filter) which threaten the stability of the structure – Section 8.2.2
Loose blocks	Blocks in the revetment which are only in contact with the row of blocks immediately below and which can be lifted out of the pitching by a force which is only slightly greater than their own weight – Section 8.3

Main dikes	Principal dikes, fronted by a summer dike and a flood plain and protecting large areas of land – Section 6.6
Mean value	Averaged value – Section 13.2
Molten asphalt	Grouting mortar with a bitumen base – Section 5.6
Paved blocks	Prefabricated blocks laid in a bonded pattern with no washed-in material – Chapter 2
Paved elements	Blocks laid in a bonded pattern but not bonded together by cables, geotextiles or interlocks – Section 5.2
Period	Period between two upward-zero crossings of the water surface (zero = still water line) – Section 6.2.1
Permeability	Relationship between the specific discharge and hydraulic gradient (reciprocal of the flow resistance) – Section 5.2
Phreatic line	Water level within the filter or the dike Section 8.2.3
Piping	Transport of sand through small channels under the cover layer or a geotextile towards a sand leak in the structure – Section 8.3
Pitching	Cover layer of bonded blocks
Plastic limit	Water content at which a ball of clay can no longer be rolled into a cylinder of 3 mm diameter without breaking – Section 5.8
Plasticity Index	Liquid Limit minus Plastic Limit – Section 5.8
Plunging breaker	Breaking wave which impacts on the revetment slope – Section 6.2.2
Porosity	Relationship between the space between grains and the total volume (space + grains) of a granular material – Section 5.3.1
Preliminary Design Method	A simple method for making initial estimates of the thick ness of the cover layer based on the obvious properties of the cover layer and the subsoil – Section 8.4.1
Pressure	Force per unit area – Section 8.2.2
Pressure head	Level relative to a plane of reference, for example, the still water line, to which the water in an (imaginary) fine tube rises as a result of the local pressure (the tube is open at the top and at the bottom) – Section 8.2.3
Probabilistic method of calculating	Method used to determine the failure probability of a revetment taking into account uncertainties about the size of the loads and the strength of the of the structure – Chapter 13
Refraction	Change in direction of waves as they approach shallow water – Section 6.2.2
Residual strength	Time between initial damage and the appearance of a bare patch of the dike core (resistance to wave attack after the onset of damage) – Section 13.4

Revetment failure probability	The probability (per year) of flooding resulting from damage to the revetment structure – Section 13.3
Revetment structure	All the layers which protect the dike core of sand against erosion by water movements, comprising a cover layer with (possibly) filter layers and a clay layer – Chapter 2
Ring dike	River dike protecting the winter bed (including the flood plain) – Chapter 3
Ripening factor	Quantity of water to which the clay/lutum can be bonded relative to the quantity of clay/lutum – Section 5.8
River dike	River dike immediately bordering the summer bed – Chapter 3
Sanding pentration	a) From the base: Migration of sand through the pores of the filter, leading to the settlement of the filter and the cover layer – Section 8.2.1 b) From the beach: Increase in level of sand and silt in the cover layer and the filter, originating from, for example, the foreshore in front of the revetment – Section 8.7.3
Secondary ship waves	Short waves caused by a passing ship – Section 6.3
Seiches	(Resonant) oscillations in estuaries or rivers caused by atmospheric/oscillations or squalls – Section 6.2.2
Shoaling	Change in wave height as a function of water depth (without waves breaking) – Section 6.2.2
Silt content	Proportion of particles between 2μm and 63μm – Section 5.8
Specific discharge	Discharge per unit of surface facing directly in the direction of flow (pores and grains)(filter velocity) – Section 8.4.3
Spectrum	Division of wave energy density as a function of the period (with a broad spectrum the wave periods differ greatly) – Section 6.2.1
Spilling breaker	Gradually breaking wave (foam) – on a fairly gently sloping foreshore – Section 6.2.2
Squalls	Temporary increases in water level (Section 6.1)
Standard variation	Measure of the spread or uncertainty (about $\frac{1}{2}$ of the difference between the upper and lower limits) – Section 13.2
Still water line	Water level when waves are absent but taking into account wave and wind set up, etc – Section 6.1
STEENZET	Advanced numerical model for calculating the stability of the cover layer – Section 8.4.1
Strength	Characterized by the maximum allowable wave height (or difference in pressure head on the cover layer or the hydraulic gradient in the filter) at which the stability of the structure is just in doubt – Section 8.2.2
Sand content	Proportion of particles larger than 63μm – Section 5.8

Sieve curve	Graphical representation of the grain size distribution of granular material, with grain size on the horizontal axis and the percentage below the stated size based on mass (weight) on the vertical axis (widely graded material has a flat curve) – Section 5.3.1
Soil body	Dike core of sand and/or clay which must be protected against the effect of moving water (by means of, for example, a pitched dike revetment)
Stern wave	Wave occurring at the end of the fall in water level caused by a passing ship – Section 6.3
Sublayers	All the layers between the core of the dike (sand) and the cover layer, for example, filter layers, geotextiles and the clay layer – Chapter 2
Subsoil	Sand or clay underneath a granular filter or a geotextile (base) – Chapter 2
Summer dikes	Relatively low dikes fronted by an unprotected foreshore, and protecting relatively small flood plain areas – Section 6.6
Surging breaker	Wave on a very steep slope which has very little foam and reflects almost completely – Section 6.2.2
Toe structure	Structure at the bottom of the slope forming a transition between the foreshore or a supporting berm – Sections 7.5 and 10.4
Transition structure	Structure designed to join together two different types of revetment
Volumetric mass	Relationship between the volume and the mass of a material, for example, concrete, grains without pores (density, specific mass) – Section 8.2.3 and Chapter 9
Washed-in material	Granular material washed into the joints between the blocks to increase the interactive forces between the blocks – Section 5.5
Wave height	Difference between the highest and the lowest level of the water surface during the period between two positive zero crossings of the water surface (zero being the level of the still water line) – Section 6.2.1
Wave period	Period between two upward zero crossings of the water surface (zero being the still water line) – Section 6.2.1
Wave run-up level	Highest level, relative to the still water line, to which a wave wets the slope – Section 7.2
Wave steepness	Relationship between wave height and wave length – Section 6.2.1
Wave set-up	Increase in water level as a result of wind waves – Section 6.1
Wind set-up	Increase in water level as a result of a storm – Section 6.1

CHAPTER 1

INTRODUCTION

1.1 General

Early in the 1980's an investigation began into the stability of pitched block slope revetments; the investigation was planned to take several years. This investigation led to a fundamental understanding of stability, supplementing the practical experience acquired in the last century in dike construction and the restoration of often damaged and neglected dikes.

The investigation was undertaken by Delft Hydraulics and Delft Geotechnics and was commissioned by the Netherlands Technical Commission for Hydraulic Structures (TAW). The results of the investigation have been presented in many scientific publications. These publications mainly give the results of particular studies and there is a need for a more practical manual. In the present publication the information obtained in the investigation has been made more readily accessible and directly applicable by concentrating on practical design methods themselves rather than by the investigations leading to the design methods.

PITCHED DIKE REVETMENTS
Pitched dike revetments comprise the following systems:
- basalt and other natural stone placed in a single layer,
- concrete blocks, concrete prisms and other small concrete elements placed in a single layer,
- block mattresses, comprising small concrete elements bonded together by cables, or a geotextile.

Revetment slabs, comprising elements greater than about 1 m^2 are not included in the definition of pitched dike revetment.
The areas of application include:
- slope revetments on dikes and the banks of rivers and canals, and
- revetments on the berms and crests of dikes and banks.

In an early stage of the investigation a set of guidelines on pitched dike revetments was published, which was mainly descriptive, [Guidelines for Concrete Dike Revetments, 1984]. Sufficient progress has now been made for the present manual to be published on the subject. This manual is aimed principally at the design aspects of pitched dike revetments.

The present publication replaces the 1984 guidelines although this still fulfils a function as a qualitative description of the performance of this type of dike revetments under wave attack.
There are many references to the reports on the 1989 investigation which provide the foundations to the present investigations, for example, [Bezuijen et al., 1990]. The latter report furnishes those interested with the background to the work. The present manual has been laid out in such a way that it is not necessary to consult the earlier works when making a design. The aim of the manual is to help the designer to make the correct choice and to enable him to foresee the consequences of his selection. This is achieved by, for instance, using worked examples. The preparation of a good design, however, is more than the application of worked examples. Because of the conditions, considerations and preferences there is no one all embracing design and each slope protection design has to be made to measure. Because of this the designer must be creative in order to produce a suitable design.
Within the framework of this manual a pitched dike revetment is defined as a protection to the body of a dike against the erosive effects of waves and currents and comprises prefabricated elements placed adjacent to each other in a particular way, see Figure 1. These elements are small so that any deformations which occur in the dike body over the course of time are readily adapted to.

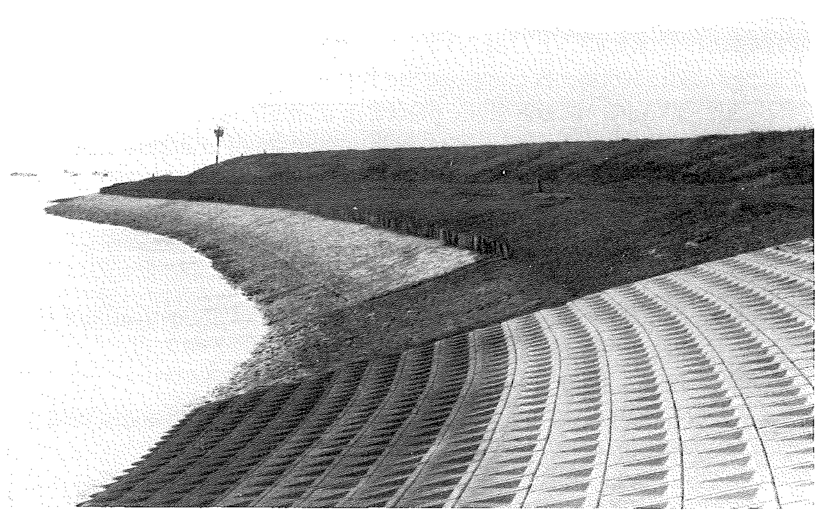


Fig. 1. A dike with a pitched revetment.

Although this definition precludes revetments of natural stone such as basalt, the design techniques described can also be applied to this type of revetment. Today, however, natural stone revetments are no longer used, except where the material is available locally, for example, when old slopes are being repitched.

The manual is written for engineers familiar with the basic concepts of hydraulic structures and who are involved in the design and management of such structures. The main emphasis in the manual is on the design of new pitched revetments on a sea or lake dike; the design of revetments for the banks of rivers and canals is only treated briefly. Assessment of the safety of existing dikes is considered within the framework of present experience. However since this topic has not been investigated comprehensively the results cannot be readily applied.

1.2 The layout of the manual

The intention of the manual is to guide the reader, step by step, to a reliable design for a new pitched dike revetment. The various revetment alternatives are described and compared, in principle, with revetment systems which are not pitched, see Chapter 2. In this connection reference is made to [Selection Methodology Guidelines, 1988, in Dutch]. In the latter publication distinction is made between paved stones, such as blocks and prisms, block mattresses and interlocking blocks, such as tongue and grooved units. The manual stresses that revetments comprise not only the cover layer of pitched stone but also, for example, any sublayers of, granular material, rubble, minestone, etc. and possibily a geotextile and/or clay layer, see Figures 2 and 3.

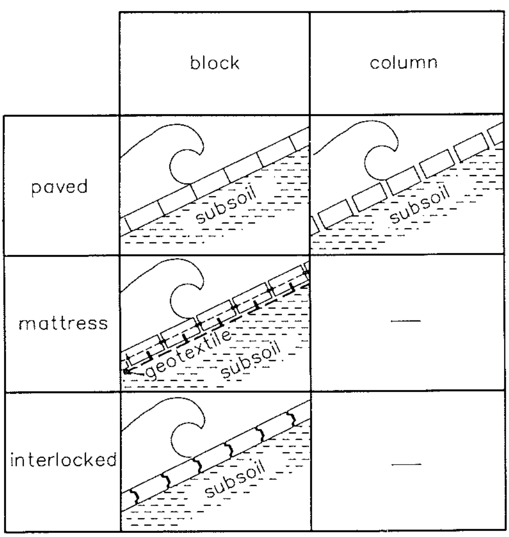


Fig. 2. Revetment types.

The areas of application, described in Chapter 3, are sea dikes, lake dikes and river and canal banks. The specifications required for revetment structures are described in Chapter 4. The materials which can be used in revetment construction, including the sublayers, are discussed in Chapter 5. Quantative design is discussed in Chapter 6, beginning with a method for determining the wave height and period for both wind and ship waves. The size of the loads which occur when the crest is overtopped and as a result of wave impact are presented very schematically.

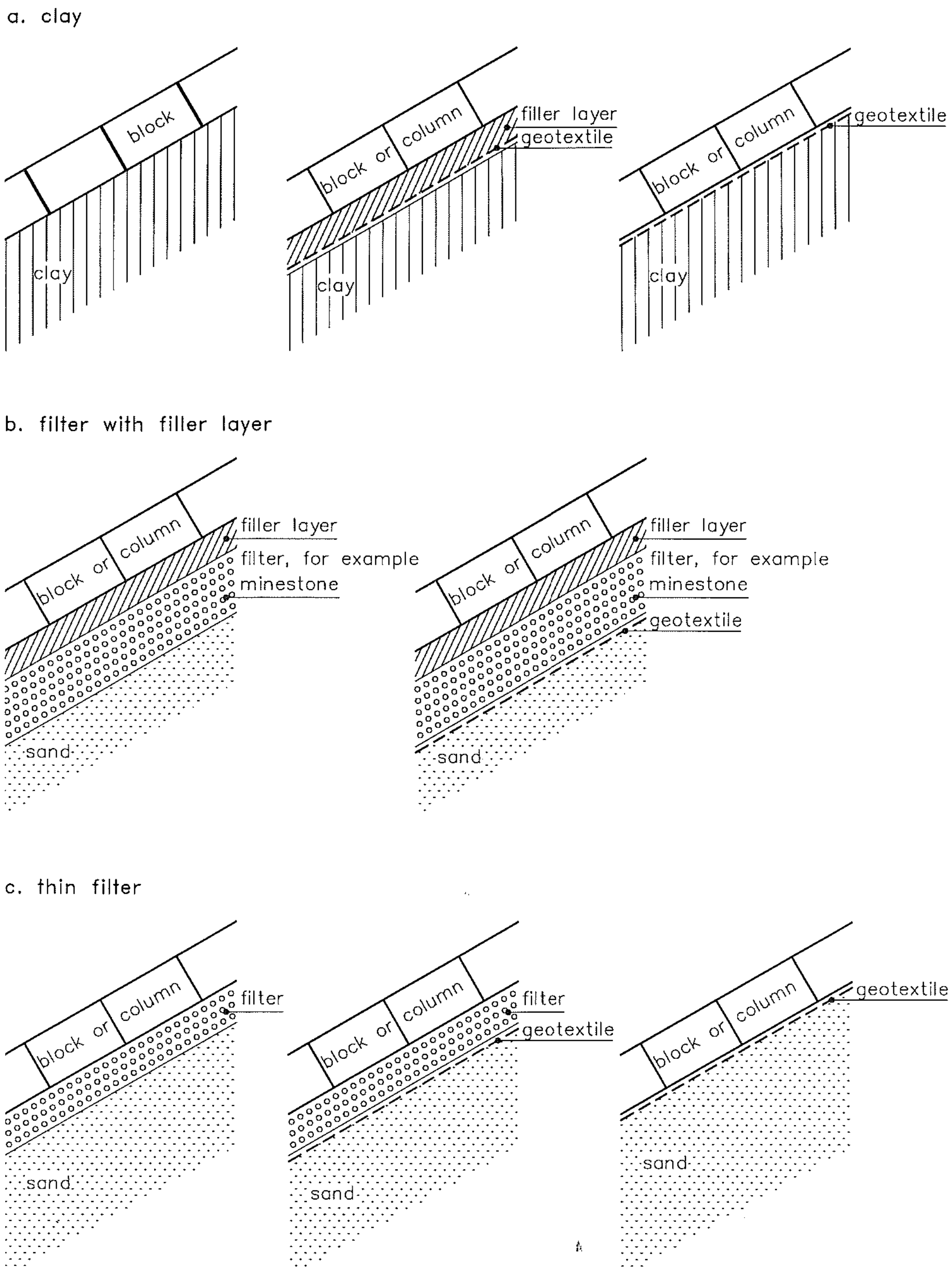


Fig. 3. Types of sublayers.

General design aspects, treated in Chapter 7, include the determination of wave run-up, the height of the berm(s), the choice of slope angle, the level at which the revetment can be replaced by grass and the application of a camber to the slope.
Chapter 8 describes the physical processes which contribute to revetment failure. The reader is then taken step by step through the design graphs and equations used to design a stable structure.
The methods discussed indicate whether or not the revetment structure designed is sufficiently stable. Optimum economic solutions are not worked out. Since a structure can also fail if the base, for example the sand core directly under the revetment, is insufficiently stable, graphs and equations are given in Chapter 9 for evaluating geotechnical stability. This information relates to the geotechnical instability caused by an unsuitable revetment structure and not that due to the dike as a whole. Chapter 10 considers the form of the transition from one type of revetment to another and also toe structures. Chapter 11 discusses the alternatives available to the designer for producing a revetment design which is stable and forms a guide to selecting the various structural components. Examples of structures which have been found to be unstable are given in Chapter 12.
Chapter 13 treats the concept of safety and differentiates between dikes, the safety of which is based on the stability of the cover layer (the pitching), and dikes the safety of which is based primarily on a thick clay layer. Parameters which are not clearly understood and for which safety factors have to be selected are indicated. This method of safety analysis is identified as an instrument for assessing the reliability of the design and for identifying those particular structural components which must be constructed with great care.
Although the manual is primarily aimed at the design of new pitched dike revetments the design approach can also be used to evaluate existing revetments; this subject is discussed in Chapter 14. The examination of the whole flood protection system is described in [Testing Guidelines, 1991]; a design example is worked out in Chapter 15. Gaps in present knowledge are discussed in the last chapter of the manual, Chapter 16. The conclusion drawn is that present knowledge, which can now be considered to be well developed, generally favours paved blocks overlying a granular filter. The terms most frequently used are listed in a glossary which follows immediately after the Contents List.

Chapter 2

REVETMENT SYSTEMS

Pitched dike revetments (block pitching) are a form of protection lying between revetments comprised of elements which are disconnected, such as rubble, and monolithic revetments, such as asphalt, see Figure 4. Individual elements of rubble typically do not support adjacent elements; in contrast, a monolithic revetment can only work as one single unit.

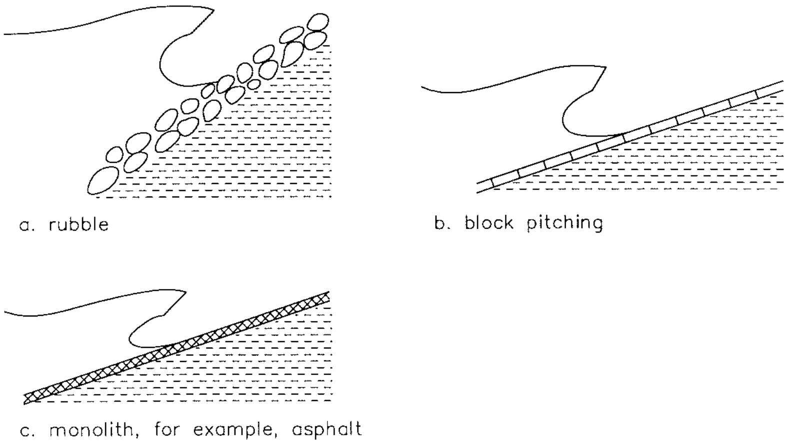


Fig. 4. Basic construction types.

The individual elements of a pitched revetment, referred to in this manual as "blocks", are placed tightly together in a smooth pattern. This ensures that external forces such as waves and currents can exert little drag on the blocks [Bezuijen et al. (1990), page 26] and also that stones support each other without any loss of flexibility when there are local subsoil irregularities or settlements.
Based on these characteristics the revetment systems considered in the manual can be summed up as follows:

- Paved blocks: prefabricated units placed in a bonded pattern, the best known forms being blocks or columns. Most block revetments are paved, and thus "fixed", but, nonetheless designs must be based on the fact that a few blocks may work loose.
- Block mattresses: prefabricated blocks joined together into a mattress by cables or a geotextile.
- Interlocking blocks: prefabricated blocks which lock together because of their shape making it difficult for individual blocks to work loose.

Examples of these types of revetment are given in Figure 5.

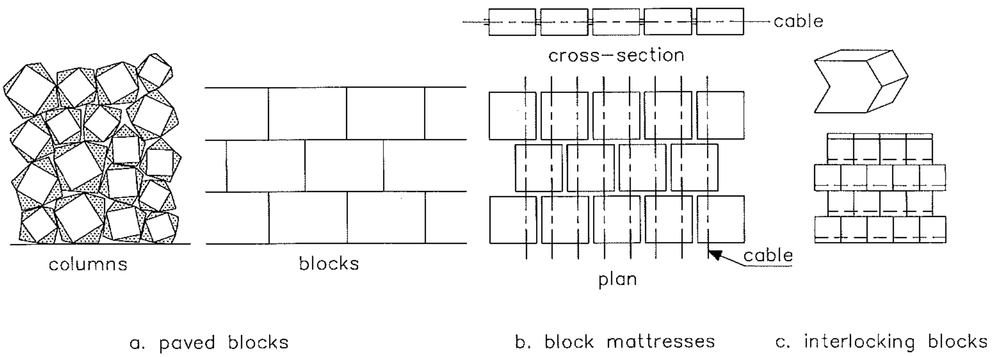


Fig. 5. Examples of three types of revetment.

This manual considers only relatively *smooth* block pitching. If obstructions, stick out of the slope or if there are edges which can be attacked by wave run-up, large hydraulic forces can be a threat to stability. These forces are not taken into account in this manual. Small irregularities do not cause problems. It can be assumed provisionally that irregularities are small if the breadth multiplied by the height is less than about 20 % of the breadth of the block multiplied by its thickness.
The subsoil on which the blocks are founded is very important for the stability of the revetment. The revetment structure is therefore subdivided into a cover layer (the blocks) and the sublayers (all the layers between the blocks and the dike core or the ground, which is generally soil).

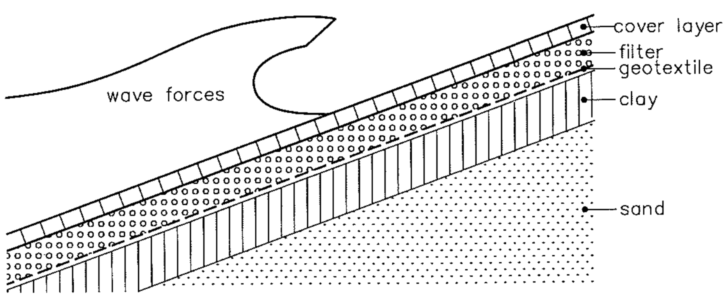


Fig. 6. General cross-section through a revetment structure.

This subdivision is shown for a general case in Figure 6. In this figure the sublayer is divided into granular layers, designated as "filter", geotextiles and a clay layer. The dike core (ground) is also referred to as the "base". This is generally sand but can also contain old dike remnants, for example, clay.

Examples of sublayer composition are listed below and illustrated in Figures 2, 3, 7 and 8:

- cover layer – filler – minestone – base
- cover layer – filler – minestone – geotextile – base
- cover layer – filler – Silex – geotextile – base
- cover layer – gravel or rubble – geotextile – clay – base
- cover layer – geotextile – base
  (only when loads are expected to be relatively small)
- cover layer – geotextile – clay – base
- cover layer – clay – base

Materials used frequently in the past, like straw mattresses and the cushion layer of bricks (Fig. 9) are left out of the consideration here. They are dealt with in chapter 10 (transition structures), where the problem of connecting a new revetment to the existing structure arises.

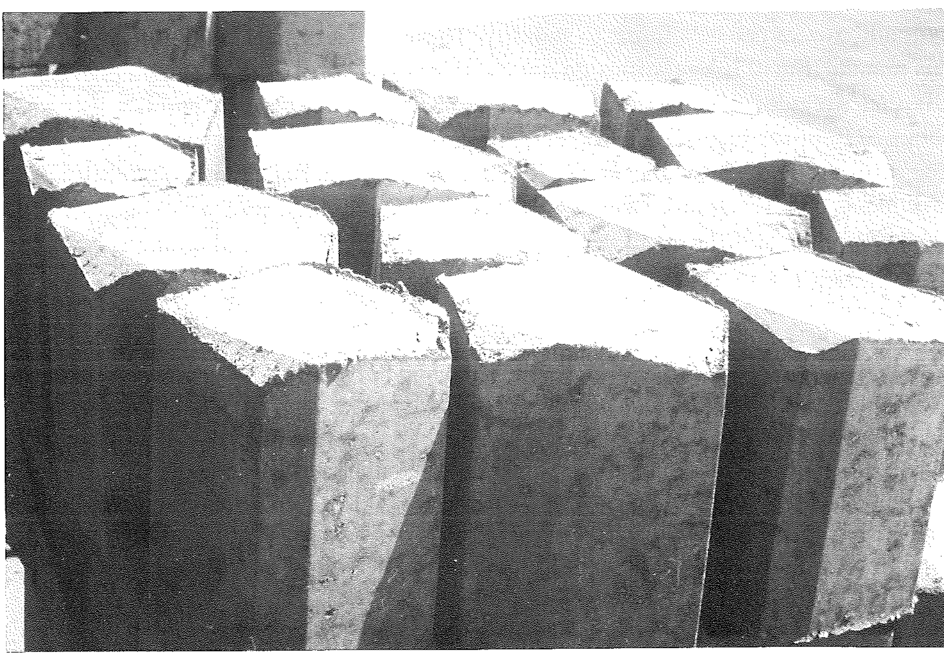


Fig. 7. Concrete columns.

Clearly there are many different types of revetment structures: rubble, various forms of pitching and monolithic types. Each type of structure has advantages and disadvantages, a number of which are listed below. For full details reference should be made to [Guidelines to selection, 1988].

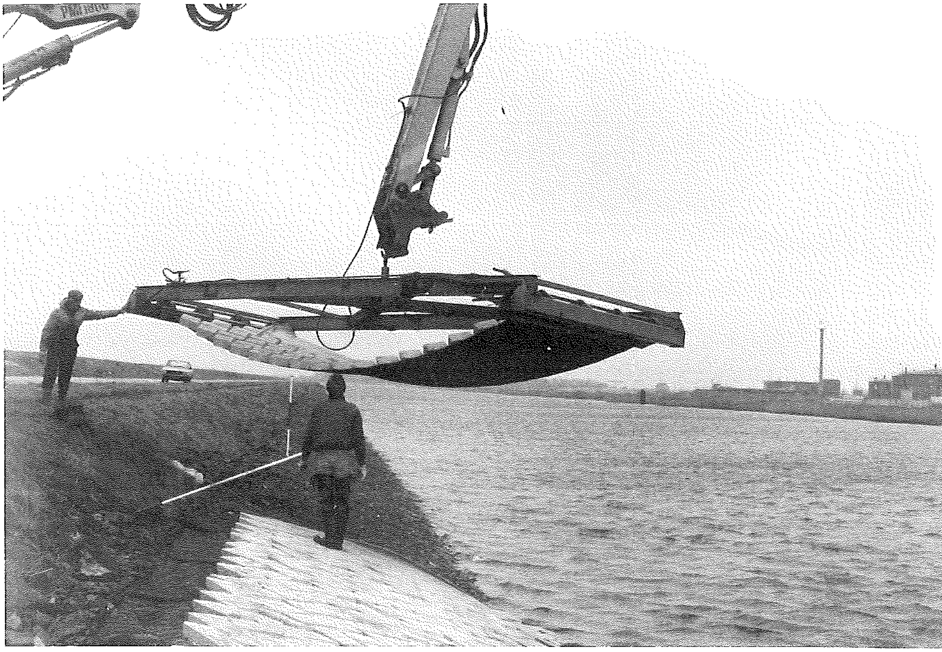


Fig. 8. A block mattress.

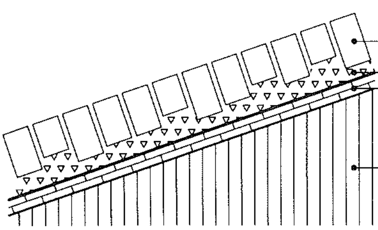


Fig. 9. Columnar basalt laid on building rubble and bricks.

The relative advantages and disadvantaged of pitched dike revetments are summarized below:

- When using rubble as protection against wave attack individual blocks must be heavier than those used in a revetment of pitched block.
- The total weight of a rubble revetment is greater.
- Because there is no local rock, rubble has to be imported to the Netherlands.
- Rubble is easier to lay than block pitching.
- Rubble can be laid underwater.
- Monolithic concrete revetments are not flexible.
- Monolithic asphalt revetments can only be laid above the tidal zone.
- Monolithic asphalt and concrete revetments are impermeable and as a result large pressure differences can develop, see Figure 10.

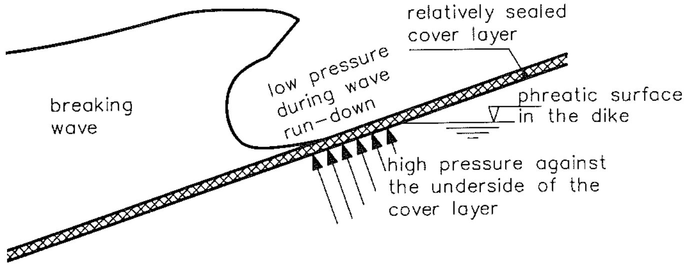


Fig. 10. Pressure difference on the cover layer under breaking waves.

It is stressed that monolithic cover layers form a water-tight protection to the subsoil and pressure differences can develop on the cover layer under wave attack, or if there is a fall in water level. These differences in pressure can cause the cover layer to fail. As shown in Figure 10, these pressure differences develop when waves break on the slope, particulary immediately before and during wave impact.

Pressure difference can also be a problem for pitched revetments of tightly placed blocks [Bezuijen et al. (1990), page 36]. Pressure differences can be greatly reduced by using blocks with holes or a form of block which, when laid, creates holes between the blocks, such as columns or blocks with cut back or tapered sides. Pressure differences threaten the stability of the cover layer because loose blocks or blocks which, accidentally, are not laid as tightly as they should be, can be loosened and lifted out of the pitching. Blocks which are not tightly laid can however be an advantage since the cover layer is then flexible and able to adapt to settlements in the subsoil. In the Netherlands this is very important because of the very weak subsoil which tends to occur. When large holes are used to reduce pressure differences there can be a risk that the filter sublayers are flushed out [Bezuijen et al. (1990), page 122]. This must be prevented by using a relatively coarse filter or a geotextile under the cover layer, see Figure 11. Both these solutions can however have a negative effect on the pressure differences and thus the stability of the cover layer.

Vegetation can also be used to prevent sublayers being flushed out. Vegetation, in the form of a grass slope on good quality clay, can only be used on its own above storm flood level.

In freshwater areas open block mattresses can be used under water in combination with vegetation (reeds and other plants). The mattresses, in this situation, hold the soil in place while the plants establish themselves. This is referred to as a "natural technique" for bank protection, see [Banks providing opportunities for natural development, 1994].

Compared with monolithic revetments, the blocks in a pitched revetment work less well together. This can, however, be improved:

- by washing a granular material in between the blocks
- by filling the space between the blocks with a good quality grouting mortar such as concrete mortar or molten asphalt.

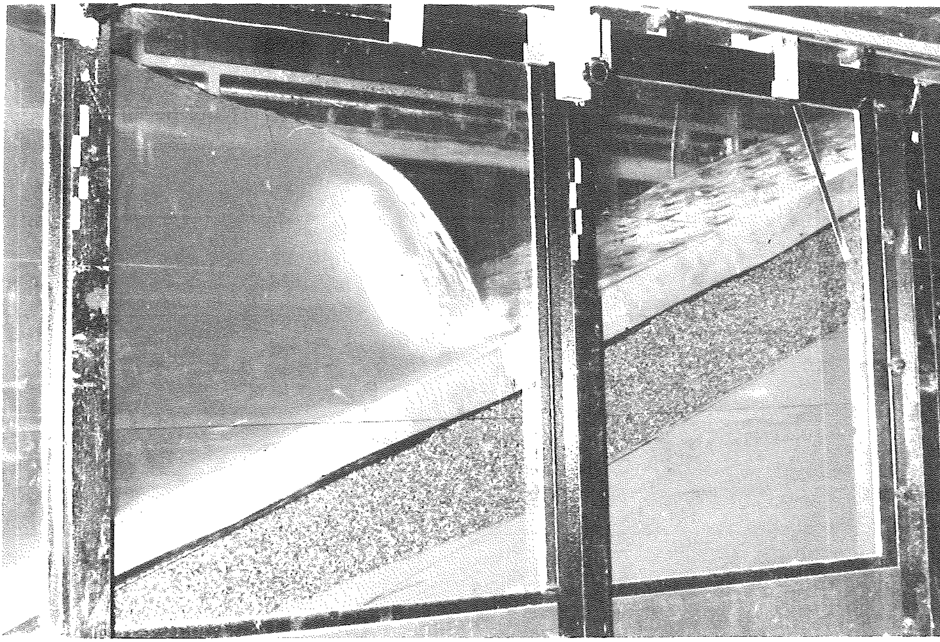


Fig. 11. Model test on a revetment with large holes in a cover layer lying on an open geotextile.

The first of these solutions is frequently used but maintenance may be required if the granular material itself is flushed out. This problem does not occur with concrete or asphalt mortar. There are other problems with this solution, however, because water permeability is reduced and because the concrete cannot adapt to settlements in the subsoil. Because of the ground composition in the Netherlands settlement can be expected. The inability to adapt to local settlement leads to cavities under the revetment which in turn leads to cracking and possible failure, see Figure 12.

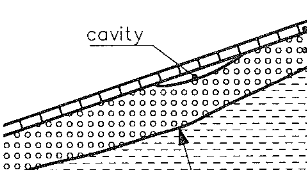


Fig. 12. Cavity under an unflexible revetment.

The different types of revetment have advantages and disadvantages and their own particular areas of application. As a result a dike or bank rarely has only one type of protection and transition structures are needed for joining the different types together. On the one hand the transition structure must prevent the movement of water from under the top layer of sand or filter material of one revetment to the other, leading to local settlement. On the other hand the transition structure prevents the cover layer against sliding and must be securely attached to it.

Chapter 3

AREAS OF APPLICATION

Table 1 summarizes the various areas in which pitched dike revetments are applied. The ruling hydraulic loads, those which are usually the most important for the design are indicated together with the other hydraulic loads. The latter, however, are rarely important for the design.

Table 1. Areas of application.

	Hydraulic loads	
Application	Ruling	Other
Sea and lake dikes	Wind waves	Currents
River dikes	Ship and wind waves	Currents
Dikes with considerable wave overtopping	Wind waves and overtopping head	Currents
River groynes and summer banks	Ship waves and currents	Wind waves
Canal banks	Ship waves	Currents

In the table the subdivision based on area of application parallels, to a large extent, the subdivision based on hydraulic loads. In general the hydraulic loads exerted on sea dikes are caused by wind waves and by ship waves on canal and river banks. River groynes and summer dikes have a special place here because these have to be able to resist both ship waves and overtopping currents. Forces due to currents are very rarely taken into account in the design of pitched bank and dike revetments. Pressure fluctuations in the flow can only threaten the revetment if the local current velocity is greater than about 2 m/s and/or if the flow is very turbulent. Such conditions can occur, for example, where the current velocity is strongly retarded by flow deflecting structures. Such high current velocities and pressure fluctuations usually can only develop on dikes exposed to severe wave action and groynes overtopped by currents. These structures are therefore treated separately. For methods for quantifying hydraulic loads reference should be made to Chapter 6.

This manual aims primarily at the application of pitched revetments on sea and lake dikes. Information about structures in other areas of application is confined to a description of the most important differences between these structures and sea and lake dikes, see Figure 13.

Fig. 13. Washed-in concrete columns on a thin filter on top of a geotextile. Four rows of "growth" blocks are placed along the upper edge of the geotextile.

In general paved and interlocking blocks are only used above low water because they cannot be laid underwater; if necessary however the lake or canal level can be lowered artificially. This restriction does not apply to block mattresses although laying under water can be much more difficult than laying above water level. Because of the width of the flood plain, river dikes can be so far from the navigation channel that loads due to ship waves are very small. These dikes are therefore not provided with a "hard" revetment except in special cases where, for example, the streamlines flow close to the dike, or if the dike is poorly aligned relative to the direction of storms. In these cases, the dike may be attacked by wind waves, see also [Guide for the design of river dikes, Vol. 1, Section 12.1.6].

In addition to the revetment categories based on hydraulic loads, categories can also be based on exceptional loads, the most important of these being:

- loads due to (drift) ice, and
- loads due to ships running aground (or collision)

Ice loads occur most frequently on lake or river dikes since freshwater freezes more readily than salt water. Ship collisions occur mostly on rivers and canals, but are less dangerous than collisions with sea dikes since inland vessels tend to be smaller. Because of their very large mass, sea-going vessels can seriously damage a dike unless the foreshore is relatively high and the ship therefore cannot reach the dike. Exceptional loads are treated in detail in Section 6.6.

Chapter 4

SPECIFICATIONS FOR REVETMENTS AND SUBLAYERS

4.1 Functional specifications

The primary function of the revetment is the reliable protection of the hydraulic structure or bank. It is assumed that vegetation can (eventually) form an integral part of a revetment structure, see Figure 14.
On this basis the following functional specifications can be set for the revetment:

1. The structure must reliably withstand the ruling or design wave conditions.
2. The structure must reliably withstand excess pressure caused by a high phreatic level in the dike.
3. The sublayers must be protected against erosion and/or flushing out.
4. The revetment and the sublayers must be able to adapt to uneven settlement of the subsoil.
5. After being damaged by exceptional forces, such as ship impact, heavy drift ice, vandalism, etc., the structure must continue to fulfil its prime function and so giving sufficient opportunity for repair.
6. Trees on or near the structure must not have a negative effect on the primary function of the revetment.
7. Sometimes a sublayer functions as a water containing layer (a sealing layer of, for example, clay).

These specifications allow the designer some freedom to define concepts, such as, "durability", "sufficient", etc. Safety plays a primary role in the definition of these concepts if the structures are to protect against flooding, for example, sea dikes. In contrast if the revetments have no safety function then economic optimization based only on construction, maintenance and repair costs are more important. In this respect, ship impacts, which occur rarely, are unimportant.
The fourth specification refers to the flexibility/adaptability of the revetment: the possibility of the revetment (and filter) being lifted after local deformation caused by cavities. A large friction force between adjacent concrete elements can be a disadvantage in this respect and can impede the settlement of individual blocks. In contrast the friction between blocks can contribute to the stability of a revetment under wave attack. The flexibility requirement here works against the attempt to create the greatest possible stability for a given cover layer thickness. In practice the best possible compromise has to be sought in which there can be no reduction in the stability specification. When translating the functional specifications into definite design criteria a distinction should be made between a revetment which will be frequently subjected to very heavy loads, for example, in the tidal zone and one which will rarely be sub-

jected to loads, for example, only at storm flood levels. Strict specifications must be set for the stability of the various structural components in the zone which will be most frequently loaded (daily to annually). These specifications must have a definite reserve with respect to loading and must guarantee reliable resistance, ageing phenomena not being allowed to affect the level of safety. Stricter durability criteria are formulated in Section 8.3 for the zone which is most heavily loaded than for structures only loaded at storm flood level. These stricter stability criteria also apply to structures which are mainly subjected to wave forces.

Fig. 14. Pitching with vegetation.

In addition to the above functional specifications, which are aimed at the primary function of the revetment, there can also be many important secondary function specifications. These depend on the following related functions or the environment:

- Landscape:
  - aesthetic form and/or colour: adaptability to the landscape,
  - visual appearance of repairs: no negative effects.
- Recreation:
  - easy to cross (gentle, non-smooth, slopes),
  - possibilities for boat moorings,
  - possibilities for constructing recreational structures, for example, landing stages, steps, berms and trailer slopes, etc.
- Shipping:
  - visual aids to estimating the position and width of shipping channels and the underwater position of bank protection,
  - wave absorption.
- Nature:
  - construction components: environmentally friendly,
  - the revetment: environmentally friendly,
  - vegetation: able to grow on the revetment.

4.2 Concrete quality

The quality requirements for non-reinforced fabricated concrete elements are set out in the Dutch Standard: "Concrete Slope Elements", [NEN 7024, 1972]. Nowadays reinforced concrete elements and insitu concrete are rarely used in dike construction and therefore standards have not been prepared for these features. Recent developments in slab revetments using open colloidal concrete can however change this situation [Eversdijk, 1990].
NEN 7024 specifies the properties of the products to be used. For slope elements, the particular properties are:

- the dimensions (length, width, depth, surface and smoothness, etc),
- the volumetric mass (especially aimed at durability, which is closely related to water penetration),
- the density (water penetration),
- chemical resistance (salt, sulphates, acids, etc.),
- physical resistance (frost, impact, wear, vehicle loads, etc.),
- durability,
- strength.

NEN 7024, 1972 gives criteria for the above properties and also indicates how the properties can be measured. The standard also refers to other subsidiary standards for basic materials (sand, gravel, cement, water, additives, etc.), concrete mixing, mix compositions, revetment element production, curing processes, methods of checking and measuring instruments. Products are generally in accordance with the standards. When products are to be supplied with a (Dutch KOMO) certificate a systematic certification system is used to control the whole production process.

This system of standards and certificates forms a very strict framework which can however be relaxed in specific cases. Obviously it must be clearly demonstrated in these cases that this can be allowed. This keeps the way open for new developments in materials, production processes and products.
The specification of compressive strength for concrete is an example of this relaxation in standards. According to the standard compressive strength must be at least 60 MPa, not excessive for interlocking elements because large forces develop on relatively small parts of the blocks. In other cases however a lower compressive strength of 30 MPa is acceptable.
Steps are currently being taken to change or modify NEN 7024. The suggestion is that the standard should not be limited to the revetment elements but that should be extended to include the construction application, the design of the revetment and certain standard construction details. Resistance to seawater depends on the type of cement used. In salt environments preference should be given to blastfurnace cement rather than Portland cement, see also [Guide to concrete dike revetments, 1989].

4.3 Construction and maintenance specifications

In addition to functional specifications, specifications are also set for construction and maintenance. These are discussed below.

Construction specifications:
- It must be possible to construct the design. This should be confirmed before the work begins because if it is found during the execution of the work that a part of the revetment cannot be constructed recourse has then to be made to emergency solutions which generally have an adverse effect on structural stability or durability.
- It should be possible to construct the revetment quickly and easily, preferably using mechanical equipment.
- In the tidal zone, the sublayers must be able to resist wave attack. In many cases if a filter layer is used the cover layer can only be placed immediately after high water.
- Ideally the revetment should be able to follow a bend in dike alignment so that vertical transition structures can be avoided as much as possible, see Figure 142.

Maintenance specifications:
- It must be possible to maintain, monitor and repair the revetment.
- The structure must be accessible. In many cases the revetment can only be reached from the land and materials and (heavy) plant have to be carried over the structure. Access roads should be provided over a reasonably flat area and the front slope therefore should not be too steep.
- Preferably it should be possible to reuse the materials used for the revetment. The management of the structure should take into account the fact that eventually it will not be possible to maintain the quality of the revetment by repair and mainte-

nance and that the structure will have to be removed. This stage should be anticipated in the design by selecting structural components which are not subject to wear and tear and which can be reused. This applies for example if the structure is subjected to differential settlement. An alternative solution is perhaps to eventually adapt the profile because the structure should not be expected to stand for centuries unchanged. In this respect interlocking stones and block mattress are at a disadvantage because:

- the connections can break if the revetment slope fails,
- in order to reuse the components a relatively large part of the slope must be broken out.

Chapter 5

MATERIALS AND THEIR SPECIFICATIONS

5.1 Introduction

Many types of materials can be used in pitched dike revetment construction and some of the main groups have already been discussed in Chapter 2. A more detailed qualitative description is given in the present chapter, material properties in relation to structural design are quantified in Section 8.4.3. The materials are subdivided primarily into type and area in which they are applied. Granular washed-in materials are therefore treated separately from granular filters. Although these are the same types of material the washed-in materials are used in the cover layer and the filters in the sublayers. Because of their different functions the properties required are not the same and must therefore be specified separately [Bezuijen et al. (1990) page 84].

5.2 Cover layers

The cover layers for the type of slope protection described here are of concrete but, in principle, other materials, such as natural stone, can also be used. As mentioned in Chapter 1,however the use of natural stone today is limited. Concrete quality is specified in Section 4.2.
There are three types of cover layer, see Section 4.2:

- Paved blocks
- Block mattresses
- Interlocking blocks

This subdivision is relevant for the design in connection with the strength of the cover layer and the loads to which it is subjected. In practice, from the point of view of design, there are only two types of cover layer, namely:

- cover layers formed of individual blocks, ("paved blocks"), and
- cover layers formed of linked blocks ("block mattresses" and "interlocking blocks").

Paved blocks

With this type of revetment it is possible for a single block to be lifted out of the slope. In practice this can only occur if, locally, insufficient material has been washed into the cover layer or if too much space has been accidentally left between adjacent blocks so that the washed-in material cannot contribute to the strength of the paving. As a result some blocks can work loose. The "weakest link in the chain" governs the design. In cover layers constructed with individual blocks this is the one loose block. The stability of the cover layer depends on the quality of the paving but also on its thickness and volumetric mass. The volumetric mass depends on the material used and can vary generally between 2000 and 3000 kg/m^3. The thickness of the cover layer cannot always be taken as the distance between the underside and the highest point of a block. Of prime importance is the relationship between the total volume of the block and the area it covers. This relationship gives the effective thickness of the cover layer, projections out of the surface being averaged out over the block surface as a whole, see Figure 15.

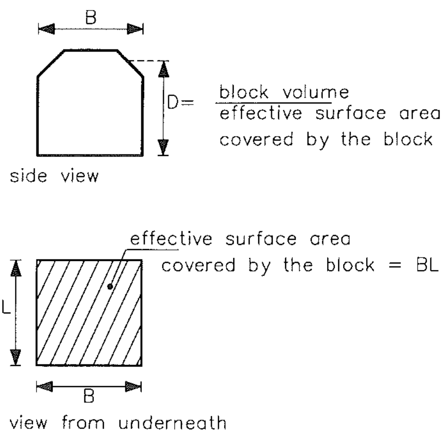


Fig. 15. Thickness of the cover layer.

Linked blocks

All the blocks in a cover layer of linked blocks must be attached to each other. Linkage can be provided by, for example, cables through the blocks, a geotextile to which all the blocks are attached, interlocking connections, etc. The weakest point in this type of cover layer is at the edges of the mattress or the interlocking, for example, at a transition structure [Bezuijen et al. (1990) page 214]. All mattresses which are not linked in a way that takes into account the large hydraulic forces which will act on the structure, can be points of weakness. The stability is then barely greater than that of individual blocks. The stability of the edges of a mattress can be increased by using larger blocks there. Blocks one and a half times the standard size can be used in order to obtain a "battered" or half block pattern, see Figure 16.

The pattern of blocks illustrated in Figure 16 assumes that there is an important interaction between individual blocks. As a result individual blocks cannot lift off the slope unless neighbouring blocks are also lifted, see Figure 17. For a satisfactory interaction between adjacent blocks the joints should generally be filled with rubble. This filling may need to be maintained. The revetment functions best when individual

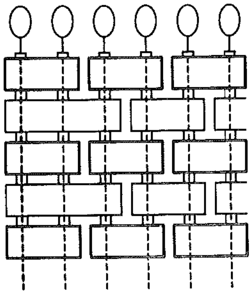


Fig. 16. A block mattress with "$1\frac{1}{2}$" size blocks on the left edge.

blocks cannot move since then movements are not transmitted to adjacent blocks. It is however necessary to specify that small movements of individual blocks should mobilize interactive forces between neighbouring blocks. In practice a movement of 5 to 10 % of the thickness of the cover layer should produce an interactive force equivalent to the weight of individual blocks. The way a block mattress functions is shown in Figure 17.

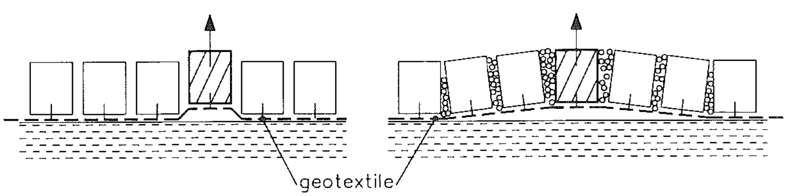


Fig. 17. The functioning of a block mattress shown schematically.

The system illustrated here is based on a geotextile to which the blocks are attached. Since the geotextile allows blocks to be displaced slightly a considerable interactive force can be mobilized and the system only works properly if washed-in material is applied. If the system does not satisfy the above specifications the cover layer should be designed as if some of the blocks are loose, for example, as an interlocking block system. In situations where individual blocks might have considerable movement the filter should also move. This however can lead eventually to serious deformations in the surface of the slope, see Figure 18. Unless there is good interaction between blocks the mattress is barely more stable than a revetment of paved blocks.

The durability of the connection between linked blocks is obviously very important. The material used (steel cables, geotextile) must therefore be highly resistant to (sea) water, sun light, plants, animals, vandalism, etc. The synthetic pins used for attaching blocks to a geotextile, see Figure 19, can, for example, become brittle at low temperatures and therefore a tough material must be used.

Fig. 18. Distortion of the surface of the slope caused by the movement of material under the mattress (test section near Lelystad).

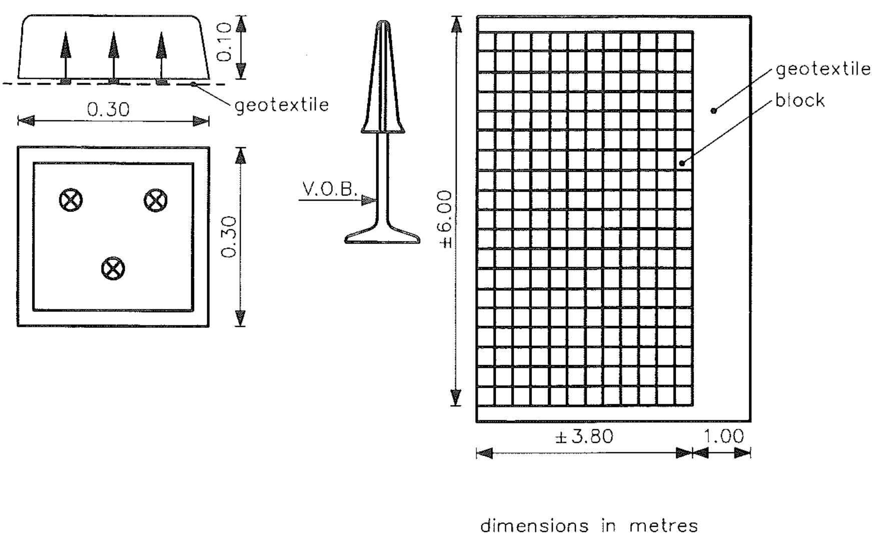


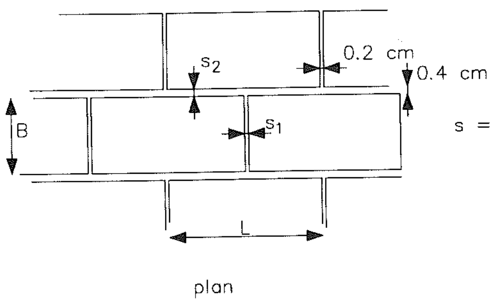
Fig. 19. Synthetic pins for attaching blocks to a geotextile.

Provided that the concrete satisfies the criteria given in Section 4.2 it should be sufficiently durable.

P e r m e a b i l i t y   o f   t h e   c o v e r   l a y e r

The pressure differences on the cover layer (loads) will largely depend on its permeability. If the permeability is high water can flow easily through, pressure differences will be small and upward forces on the blocks also small. Permeability is the reciprocal of the flow resistance and therefore high permeability implies low resistance to flow.

Permeability mainly depends on the size of the joints between the blocks and any holes in the cover layer. In stability calculations joints are characterized by their average width (s) and holes by their diameter (D_g). The average joint width should be such that the value based on the total jointed area (in the plane of the slope in a revetment with an area of several square metres) is in practice the same as that determined for an individual block, see Figure 20. In other words, the product of total joint length and average joint width must give the total joint area. For example, if blocks of 50×25 cm^2 are applied, with joints 0.4 cm wide on the long side and 0.2 cm wide on the short side, the average joint width is 0.33 cm, [the surface area of the joints is $(50 \times 0.4) + (25 \times 0.2) = 25$ cm^2; the surface area of the joints, using the average joint width, is $(50 + 25) \times 0.33 =$ approximately 25 cm^2].



$$s = \frac{\text{joint surface area per block}}{1/2 \times \text{block perimeter}} = \frac{s_1 B + s_2 L}{B + L}$$

Fig. 20. Average joint width.

The relative open area, for rectangular blocks with circular cross-section holes, Ω, can be calculated as follows (using the joint area per block + hole area per block):

$$\Omega = \frac{s\,(B + L) + N\pi D_g^2/4}{BL} \tag{1}$$

where:

s = average joint width [m]
B = block width [m]
L = block length [m]
N = number of holes [–]
D_g = hole diameter [m]

For columns the equivalent block width and length must be first calculated using the average area of the columns, A, on the the slope:

$$B = L = \sqrt{A} \tag{2}$$

where:

A = average area of a column [m]
B = equivalent block breadth [m]
L = equivalent block length [m]

For columns the joint width (s), can be calculated using the average area of the column, A, and the relative open area, Ω:

$$s = \frac{\Omega}{2}\sqrt{A} \tag{3}$$

where:

Ω = relative open area (relationship between the area between the blocks and the total area) [–]

This value has been worked out in Table 2 for typical column dimensions:

Table 2. Equivalent joint width (s) for columns, as a function of the average column area (A) and the relative open area (Ω).

		Joint width, s, [mm]		
A [m^2]	B × L [m^2]	$\Omega = 0.05$	$\Omega = 0.10$	$\Omega = 0.15$
0.040	0.20 × 0.20	5.0	10.0	15.0
0.063	0.25 × 0.25	6.3	12.5	18.8
0.090	0.30 × 0.30	7.5	15.0	22.5
0.123	0.35 × 0.35	8.8	17.5	26.3
0.160	0.40 × 0.40	10.0	20.0	30.0
0.250	0.50 × 0.50	12.5	25.0	37.5

Clearly joint and hole filler will affect the permeability of the cover layer. This washed-in material is characterized by its grain size and porosity. This subject is discussed further in Section 5.5.
Because of the affect that cover layer permeability has on structural stability it is recommended that the joint width and the grain size and porosity of joint and hole filler are selected with a factor of safety (based on the lower limit of the likely joint width, see Chapter 13).

A geotextile placed directly under the pitching should be considered as a component of the cover layer, even when it is not attached to each block, see Figure 21. There is a practical reason for this. The submerged (water) permeability of structural components and other parameters are important when calculating stability and the calculations can be simplified by considering the permeability of the cover layer and the geotextile together. This procedure is treated in the introduction to Appendix H. The properties of geotextiles are described further in Section 5.7.

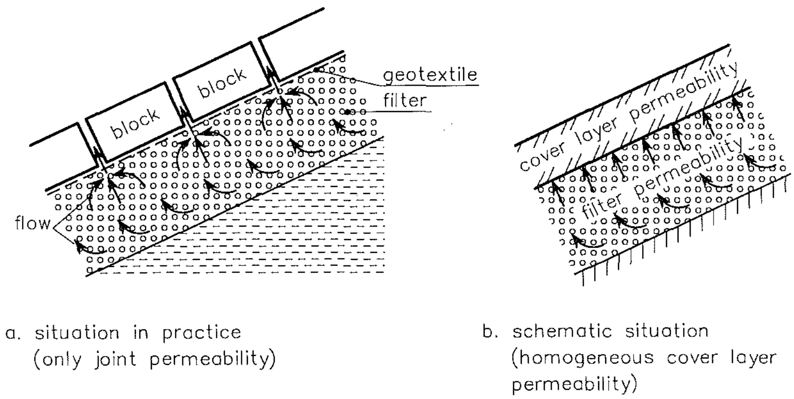


Fig. 21. Permeability schematization.

If the cover layer is laid on a granular filter, the grain size and the porosity of the filter affect the permeability of the cover layer. This is due to the complicated patterns of flow under the joints where there is a strong contraction of flow lines towards the joints, see Figure 21. Depending on the factors given above the cover layer permeability can vary depending on the details of the revetment. This is indicated in Table 3. For calculation procedures reference should be made to Section 8.4.3 and Figure 117.

Table 3. The range of permeability (k′) of cover layers laid on granular filters.

	k' (mm/s)	Type of cover layer
very large k'	60 to 100	blocks with many holes
large k'	5 to 20	washed-in columns
small k'	3 to 10	large blocks without holes
very small k'	≤ 1	blocks or columns with (partly) tightly packed joints

Provisions for ensuring a long-lasting, reliable permeability should be included in the design, otherwise the permeability can change considerably. A washed-in material which has chemical/physical properties which could lead to sealing and tight packing must be avoided. The possibility of reduced permeability caused by vegetation, sand and silting up must be taken into account, see Section 8.7.3.

5.3 Granular filters

5.3.1 *General aspects*

Function and specification

A cover layer of pitched blocks is usually laid on one or two granular filter layers, see Figure 22. If two filter layers are used, the upper layer is a filler layer (smoothing layer) on which the cover layer can be laid with a smooth finish. The filler layer is applied if the lower filter layer comprises coarse components which cannot be laid smoothly.
The functions of the filter layers are as follows:

- The transfer of loads on the cover layer to the construction base or core. This applies particularly to the distribution of local loads and the transfer of loads to any sand under the filter layer thus helping the structure as a whole to resist geotechnical instability (sliding and liquefaction).

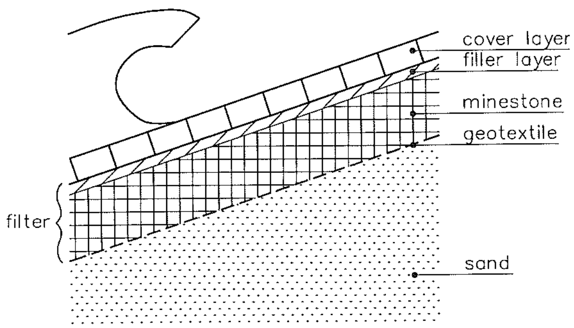


Fig. 22. Filter layers commonly used under pitched revetments.

- The prevention of pressure build-up in the core of a dike or structure.
- The prevention of material being flushed out of the base, unless this is already being prevented by a geotextile.

A number of general specifications can be set for the materials and the cover layer on the basis of the above functions.

- It must not be possible to flush the filter material through the spaces between the concrete elements of the cover layer.
- The loose-grained filter should have internal stability, implying that the fine fraction cannot be flushed through the larger pores. The wider the grain size distribution (well graded, poorly sorted) the better the internal stability.
- The filter layer should have a permanent, loose-grained character. Arching caused by uneven settling and wedging is not usually allowed since the revetment must remain in contact with the base. If contact is lost cavities will develop into which sand can move. Because of arching these cavities will not be seen from the surface but, under heavy loads, can suddenly cause the structure to fail over a large area.

- In order to prevent the build up of pressure in the core of a dike the permeability of the filter material should be greater than that of the sand core.
- Because of the method of construction it may be necessary for the filter layer to resist wave action. This can be the case when the filter material is exposed to tidal action during construction before the cover layer has been applied. The stability of the front face of the filter layer is particularly important when concrete elements are to be laid by hand on slopes steeper than 1 in 3.

In the past the requirement that the filter should prevent build up of pressure in the sand core has, perhaps, been over emphasised. This has led to the use of very permeable (coarse) filters. If the permeability of the filter is more than ten times that of the sand core this requirement is satisfied. However, from Table 6 (the permeability of sand) and Figure 119 (the permeability of filters) it can be seen that a fine filter (D_{f15} = 1 to 3 mm) can prevent the build up of pressure in the sand core. With respect cover layer stability a fine filter with large permeability is preferred to a coarse filter. A fine filter with large permeability is also preferable in connection with its possible sanding up with material from the base.

Quality guarantees

The quality of the structure should be considered at each stage of its service life. A system for monitoring the quality is therefore needed. This system should establish specifications for materials, working methods, text procedures, etc. and cover more than the inspection of the end product. It is very important to avoid mistakes during the production process [Quality Guarantees, 1986].
Material specifications should be sufficiently strict. The specification of, for example, 5 to 25 mm gravel without any reference to grain size distribution, can lead to the delivery of many different types of material. A method of monitoring deliveries involving sampling is usually necessary. Specifications should take into account existing standards for materials, grades and contract specifications. The development of regulations for materials is proceeding rapidly, both in physical and environmental areas and continuing attention should be given therefore to inventarising information and the effects of changes in this field. Reference should be made, for example, to the following Dutch publications: [Recommendations for the design of banks, 1989], [Regulations on building materials, design, 1991] or [the RAW Standard Regulations, 1990].

Design parameters

When designing the cover and filter layers the filter material can be characterized using a design grain size and porosity:

- Design grain size: D_{f15} = the theoretical sieve size, through which 15 % of the material, by weight, passes [m].
- Porosity: n = relationship between the volume of pores (between the grains) and the total volume (pores plus grains) [–].

D_{f15} and n both affect two important structural failure mechanisms: uplift of blocks and the sanding up of filters. Therefore if, in practice, the grain size is larger than specified in the design, this will have a negative effect on the stability. It is essential therefore to check that a coarse filter material is not used in the construction. Variation in the properties of the filter material, revealed during construction, can be due to the supplier delivering the wrong material but can also be due to natural sorting during transport and storage, weathering, crushing, etc.
In addition to D_{f15}, D_{f50} can also be important particularly in connection with:

- structural stability against wave action during construction
- flushing of filter material out through holes and joints in the cover layer

Table 4. Indicative values of grain size (D_{f15}) and porosity (n). (X and Y in the table are the lower and upper limit of the grain size distribution).

Material, X/Y	Grain size D_{f15}[mm]	Porosity n[–]
rubble and gravel, X/Y	$0.8{\cdot}X$ to $1.3{\cdot}X$	0.3 to 0.4
for example rubble 7/20 mm	5.6 to 9.1	0.3 to 0.4
minestone 0/70 mm	1 to 5	0.2 to 0.35
minestone 10/125 mm	5 to 15	0.25 to 0.35
Silex 5/25 mm	4 to 7	0.3 to 0.4
Silex 25/70 mm	18 to 35	0.3 to 0.4
slags 8/25 mm	6 to 12	0.3 to 0.4
slags 5/70 mm	4 to 8	0.25 to 0.35

D_{f50} should therefore be specified together with D_{f15}.
Some indicative values of D_{f15} and n are given in Table 4. In general the more widely graded materials have a smaller porosity than well-sorted materials with a steep sieve curve.

It is stressed that sieve analyses are essential to ensure that filter material are as specified. A sensitivity analysis will help to show how carefully the monitoring should be undertaken and if this is necessary for both the filler and the filter layers, see Section 13.5.

Internal stability

A granular filter is considered to have internal stability if the fine fraction cannot be flushed through the coarser grains. [Bezuijen et al. (1990), page 126]. Loss of internal stability is also referred to as suffosion or internal erosion. Internal instability can only develop when the filter has a wide grain size distribution (flat sieve curve). The smaller grains can then be flushed through the pores between the large grains. To give an indication of possible internal instability the flattest part of the sieve curve between D_{f0} and D_{f20} should be determined, see Figure 23.

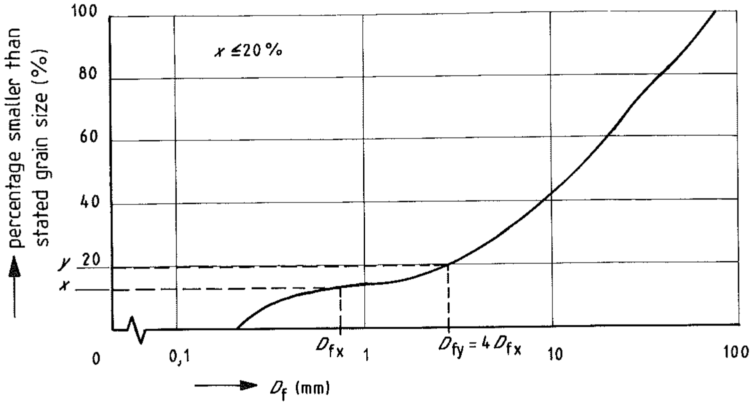


Fig. 23. Sieve curve with a flat section.

This curve should be used to determine the smallest ratio of the "percentage smaller than" values for two grain sizes which differ by a factor of 4, see Figure 23. This ratio must satisfy:

$$(y/x)_{min} > 2.3 \quad \text{with} \quad x \leq 20\ \% \tag{4}$$

with:

D_{fx} = grain size which is just smaller than that at which the slope on the sieve curve is a minimum (m)

D_{fy} = grain size which is just larger than that at which the slope on the sieve curve is a minimum (m), such that

$$D_{fy} = 4 \cdot D_{fx} \tag{5}$$

x = percentage smaller than D_{fx} ($x \leq 20$ %)

y = percentage smaller than D_{fy}

$(..)_{min}$= smallest value of ..

If internal instability does develop, $(y/x)_{min} < 2.3$, care must be taken to ensure that the grains which are flushed out are not trapped by and accumulate in other parts of the structure. It should be assumed in the stability calculations that all grains smaller than D_{fx} are lost, see Figure 24. As a result D_{f15} is increased to a value which has a "percentage finer than" about $15 + x$ on the original sieve curve, indicated by D_{f15+x} in Figure 24.

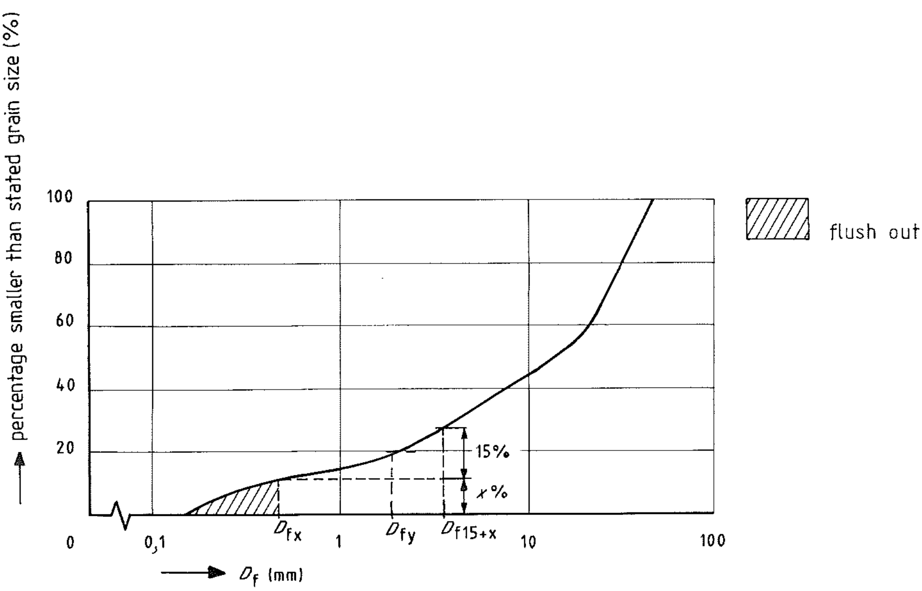


Fig. 24. Characteristic grain size after the fine fraction has been flushed out (only relevant to internal stability).

Example:

a Given: The flattest part of the sieve curve lies between D_{f10} and D_{f20}.
$D_{f5} = 0.7$ mm, $D_{f10} = 1.1$ mm, $D_{f13} = 1.8$ mm, $D_{f16} = 2.8$ mm,
$D_{f20} = 4.4$ mm, $D_{f25} = 6.5$ mm, $D_{f28} = 7.2$ mm, $D_{f50} = 11.2$ mm

b. Selection of x and y so that y/x is a minimum:
1. Assume $x = 5$, then $D_{fx} = D_{f5} = 0.7$ mm
   Find y using $D_{fy} = 4 \cdot D_{fx} = 4 \cdot 0.7 = 2.8$ mm
   Therefore: $y = 16$, for $D_{f16} = 2.8$ mm $\Rightarrow$ $y/x = 16/5 = \underline{3.2}$
2. Assume $x = 10$, then $D_{fx} = D_{f10} = 1.1$ mm $\Rightarrow$ $D_{fy} = 4 \cdot 1.1 = 4.4$ mm
   Therefore: $y = 20$, for $D_{f20} = 4.4$ mm $\Rightarrow$ $y/x = 20/10 = \underline{2.0}$
3. Assume $x = 13$, then $D_{fx} = 1.8$ mm and $D_{fy} = 7.2$ mm $\Rightarrow$ $y/x = \underline{2.2}$
4. Assume $x = 16$, then $D_{fx} = 2.8$ mm and $D_{fy} = 11.2$ mm $\Rightarrow$ $y/x = \underline{3.1}$
   Therefore $(y/x)_{min} = 2.0$

c. Conclusion: internal instability
The possible loss of all grains smaller than $D_{fx} = D_{f10} = 1.1$ mm must be taken into account. Calculations for the situation after the fine fraction has been flushed out should be made using $D_{f15+x} = D_{f25} = 6.5$ mm as the value of D_{f15}.

It is recommended that stricter criteria should be used for materials which will weather or be crushed during construction or during the service life. The number of small particles will increase as a result of weathering and crushing and, depending on the extent, use should be made of the following criteria:

$$(y/x)_{min} > 2.5 \text{ to } 3.5 \quad \text{with} \quad x \leq 20\ \% \tag{6}$$

Since there is little known about the effects of weathering and crushing on particle size, it is recommended that specific investigations are made for particular cases.

5.3.2 *Description of materials*

A relatively large number of different types of materials are used for filters and fillers. Generally the materials are quarried rock, dredged aggregates and industrial bi-products. The first group comprises both broken and unbroken stone. The most frequently used materials are discussed below.

Rubble

Rubble is finely broken natural stone originating from stonequarries. It can be obtained in many different types of stone and grades. In the Netherlands, limestone, porphyry, quartzitic sandstone (grès) are used which are obtained from Belgium. The grades of Belgian rubble which can be supplied are in accordance with Belgian standards [NBN B11-101, 1975]. The following sizes are given in these standards: 2/4 mm, 2/7 mm, 4/7 mm, 7/10 mm, 7/14 mm, 7/20 mm, 10/14 mm, 14/20 mm, 20/32 mm, 20/40 mm, 32/40 mm and 40/56 mm.
The best known rubble obtained from Germany are basalt, basaltic lava, quartzitic sandstone and grey wacke (quartzite).
The grades of German rubble which can be supplied are in accordance with German standards [TL Min-StB83, 1983] and include "Splitt" (0/5 mm, 5/11 mm, 11/22 mm and 22/32 mm), "Edelsplitt" (2/5 mm, 5/8 mm, 8/11 mm, 11/16 mm and 16/22 mm) and "Schotter" (32/45 mm and 45/56 mm). "Edelsplitt" has a higher value than "Splitt" since stricter specifications are set for grain size distribution and durability. Normally such strict specifications are not required for filter materials. Mixes can be prepared from the standard grades. The specifications which can be set for these mixes depend on the proportions required and the specifications for uncommon grades. Reference should be made to the particular standards. It should be noted that the standards referred to above are not aimed at hydraulic structure applications and therefore are unnecessarily strict on some points. This aspect is currently being studied. Examples of grades which are used as filters include 2/11 mm, 3/8 mm, 5/15 mm, 8/22 mm, 18/35 mm, 22/40 mm and 30/60 mm.

Gravel

The gravel used in the Netherlands originates generally from the lower reaches of the Maas in Limburg and the Lower Rhine in Germany. Gravel is also obtained from the

upper reaches of these rivers (the Middle and Upper Rhine and the Maas in Belgium) and also from the North Sea. Gravel is a strong and durable material and the differences in quality from the various sources are only small.
Because gravel tends to be rounded its stability, especially when poorly graded, is relatively poor. This can be a drawback causing the tendency for steep slopes to slump. In addition it can lead to the formation of "steps" in the slope, see Figure 25. Gravel is becoming increasingly difficult to obtain in the Netherlands and elsewhere because of the pressures exerted by planning authorities with respect to gravel winning. Examples of the types of gravel used for filters include 3/12 mm, 5/25 mm, 10/30 mm, 15/50 mm and 30/60 mm.

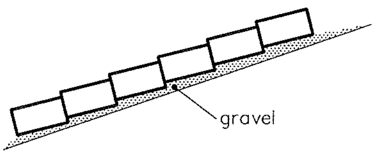


Fig. 25. "Stepped revetment" caused by the displacement of gravel on a steep slope (1 in 24).

Broken gravel

Broken gravel is produced by breaking up gravel and other grading of the broken material. The grades available in the Netherlands, quality specifications, and methods of checking are given in the [RAW Standard Regulations, 1990, Section 31.26]. There are increasing planning problems associated with winning the basic material. There are no technical reasons for crushed gravel rather than quarry rubble.

Crushed rock

According to the Dutch standard [NEN 5180, 1990) crushed natural rock should be coarser than 32 mm. Fine crushed rock, for example in the 30/60 mm grade, can in principle be used in filter layers. Quality specifications and monitoring of crushed rock can be undertaken using the Dutch [RAW Standard Regulations, 1990].

Minestone

Minestone is a bi-product of coal mining. It mainly comprises clay shale and some sandstone. Black (unburnt) minestone is supplied in the Netherlands direct from the mines or from old minestone tips in Germany and Belgium. Red (burnt) minestone can also be obtained but in finer grades. Depending on the degree of burning this material has relatively good resistance to weathering. Todate, perhaps because of the cost and quality, red minestone has not been used for hydraulic structures, although in principle there are possibilities.
The most important grades of black minestone are 0/70 mm (ungraded) and 10/125 mm (graded). The average volumetric mass of stone pieces is 2400 to 2600 kg/m^3. The density of delivered minestone (including pores) varies from 1600 to 1800 kg/m^3,

depending on the degree of compaction. Black minestone is relatively weak and sensitive to weathering and disintegrates relatively quickly when exposed to the atmosphere. The most important weathering processes are changes from dry to damp conditions and from freezing to thawing temperatures. Minestone does not disintegrate underwater and if temperatures are above freezing. Weathering separates out the clay shale into small flat lenses of various sizes. These lenses are reduced in size but, generally never less than a sieve size of 2 mm. This reduction in size by weathering reduces the permeability but never to less than that of sand.
For many years black minestone has been used in hydraulic structures. When used as a filter layer the material is sometimes first used in surrounding embankments, containing a hydraulic file. Minestone used in a filter layer is normally compacted with a bulldozer. Intensive compaction can however greatly reduce the particle size, especially if the water content is high. This can reduce the permeability considerably, especially in a liquefied cover layer. This can occur, for example, under construction roads and should be avoided. Compaction using a bulldozer has advantages over other methods because the pressures under the caterpillar tracks are relatively low and the ridges on the tracks only break up the surface slightly. The environment is an important issue with minestone which contains PAK's, sulphates and chlorides. Because of sulphates minestone cannot be applied in relatively large quantities close to small areas of stagnant water [Environmentally friendly banks, 1994]. More extensive information is given by [Laan, 1985].

Silex
Silex is a by-product of the cement industry. It comprises those components of marl which are unsuitable for the production of clinker (semi-manufactured material for Portland cement). Silex is a mixture of stone and tau (lime and calcareous sandstone). The stone is very hard and chemically inert. Tau is weak and sensitive to weathering and therefore less suitable for hydraulic structure applications.
Because of the present scarcity of limestone in the Netherlands it is profitable to use tau in clinker. The quality of Silex required for for hydraulic structure applications has in recent years, therefore, been changed. Dutch Silex is currently supplied in grades of 5/25 mm, 5/40 mm, 25/70 mm, 40/90 mm and 90/200 mm. The 5/50 mm and 40/70 mm grades can be produced to order. Silex 90/200 mm is usually too coarse for application underneath block pitching. Finer grades of Silex contain relatively little tau. There are no environmental problems associated with using Silex.

Slags
Slags are the stony materials which remain after smelting, for example, metals and phosphor. LD-slags, electro-oven slags and phosphor slags can possibly be used in filter layers. LD-slags and electro-oven slags are both steel slags and have a volumetric mass between 3100 and 3400 kg/m^3. They are angular, cubic materials which are relatively sensitive to breakage and wear. The hydration of free lime, which origi-

nates in a concentrated form in lime pits, increases the volume of the slag, causing pieces of slag to break off. This reduction in size increases with the free lime content and the coarseness of the material. The capacity of steel slags for hydraulic binding is detrimental to its application in filter layers. This reaction is due to the presence of a fine fraction. The fine fraction occurs to a limited extent even after sieving because it tends to adhere to the larger pieces. The amount of fines is increased by wear during transport, storage and rolling and compaction during construction. It is not known if or when the hydraulic binding of LD-slags has led to an undesirable sheet in the filter layer. It is recommended in all cases that the larger pieces, which are bonded together with small pieces, are removed during construction. In principle various grades of steel slag are available, for example, 8/25 mm, 10/60 mm and 5/70 mm or 15/70 mm. The last two grades are produced in the Netherlands by Hoogovens IJmuiden.
With phosphor slags there are few weathering problems and they have no tendency to bind. Because of its internal instability the 0/40 mm grade produced for railway construction cannot be used in filter layers [RAW Standard Regulations, 1990]. Another grade of phosphor slag which is available is 40/180 mm.
Environmental problems tend to develop with the use of slags, the most important being the lixiviation of chrome and manganese from the steel slags and the release of phosphor, fluor and uranium from phosphor slags. Lead slags cannot be used for environmental reasons. It is recommended that developments in environmental technology and regulations are followed closely, see [Environmentally friendly banks, 1990]. For further information reference should be made to [LAAN, 1986]. Contract specifications are given in [RAW Standard Regulations, 1990].

Building rubble

Building rubble is traditionally used as a filler layer under columnar basalt revetments. At present the application of masonry and concrete rubble in filter layers is limited because the material is not available in large quantities being generally used as a foundation for roads and for gravel production. Building rubble is occasionally contaminated with aromatic hydrocarbons and it is recommended that rubble quality is carefully monitored.

5.4 **Bonded filters**

There are two main types of bonded filters:

- cement-based, for example, sand/cement bonding
- bitumen-based, for example, bituminized sand and sand asphalt bonding.

Cement-bonded sublayers have the disadvantage that they cannot readily adapt to irregular settlement and undermining. This leads to the formation of cavities. In some cases cracks develop and, as a result, sand can be washed out of the sublayers.

Bitumen-bonded filters are less sensitive to irregular settlement because of the viscous property of the bitumen. Bitumen-based bonding is therefore more suitable than sand/cement bonding for hydraulic structures.
Sand asphalt is a mixture of sand, filler and bitumen; bituminized sand is a mixture of sand and bitumen and is a very open material because only the sand grains are bonded together. Only a limited quantity of bitumen is used and the sand grains are only covered by a thin film of bitumen (a few microns). Because it is only lightly compacted the permeability of bituminized sand is almost the same as for unbonded sand. It is recommended that a sand should be used which is the same or slightly coarser than that used for the core of the structure. For temporary works, for example, the bunds used to retain hydraulic fill, 3 % bitumen can be used but a bitumen content of 4 to 6 % is recommended for filter layers. The strength and resistance to wear is then increased although the permeability is reduced. Bearing in mind the permeability required, the material should, at the most, be only slightly compacted. The sand underneath should however be well compacted.
Bituminized sand should be mixed in a normal cement mixer (mixing temperature 125 °C to 190 °C) or in a modified drum mixer (125 °C to 140 °C). Before laying the mixture should be kept in insulated silos or tanks. It should be laid at a temperature of at least 90 °C, directly after transportation to the site. A hydraulic crane should be used for laying the mixture. When laying bituminized sand underwater a temperature of up to a maximum of 110 °C is desirable to prevent stripping (loosening of the bitumen film). The temperature should be less than this before the material can be driven over by bulldozers and lorries.

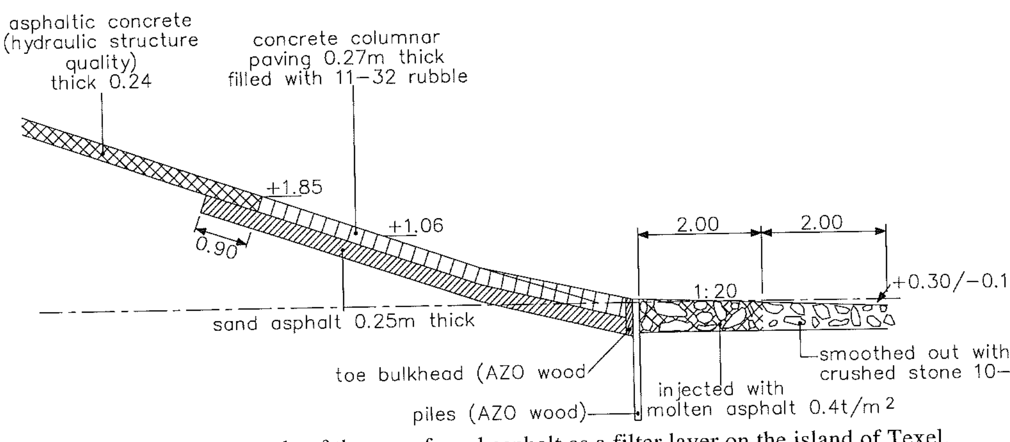


Fig. 26. An example of the use of sand asphalt as a filter layer on the island of Texel.

The material should be laid in a single layer of the correct thickness onto a smooth and compact sublayer. The minimum layer thickness is 10 to 15 cm (when laid under water the thickness should be 50 to 70 cm). Since the system has only been used for a few years its durability is as yet unknown. An example of the use of sand asphalt as a filter layer is shown in Figure 26. The figure shows a dike on Texel on which sand asphalt with a bitumen content of 6.4 % has been used, see also Figure 152. Further information can be found in [Asphalt Guidelines, 1984].

5.5 Washed-in material

Granular washed-in material is used to increase the distribution of forces between individual blocks (friction). To achieve this the material has to be forced into joints between the blocks and into the holes; steps should be taken to ensure that it remains there, see Figure 27. Although material is deliberately washed into cover layers with wide joints and/or holes (columns and block mattresses) a considerable amount of material is sometimes washed-in naturally after construction. Whatever the source of the material it ensures that the blocks are fixed in place. Use of washed-in material has, however, the following disadvantages:

- the permeability of the cover layer is reduced and, as a result, pressure differences can increase on the blocks (loads);
- the sealing of joints and holes and the growth of vegetation is encouraged and the permeability can be reduced to zero;
- on many dikes and hydraulic structures the material is washed out by tidal and wave action and has to be replaced;

Fig. 27. Washed-in columns.

- the flexibility of the cover layer is reduced and cannot adapt to settlements in the subsoil, resulting in the development of cavities.

The replacement of washed-in material is essential since the grains in the filter layer are smaller than the joints and holes in the cover layer. If the filter layer is flushed out, there is the danger that the cover layer will be undermined [Bezuijen et al. (1990) page 122]. Despite this disadvantage washed-in material apparently makes a positive contribution to cover layer stability. It is however not yet possible to quantify the positive and negative aspects of using washed-in material. In tidal zones the washing-in process can be left wholly or partly to wave action. This can be encouraged by placing material on the revetment.
The functioning of the washed in material is affected by the following:

- the material functions better, if it is angular. A disadvantage of angularity however is that it is more difficult to wash in and needs to be replaced earlier;
- the material stays more readily in place if its volumetric mass is large. It appears that apart from this fact little is known about this aspect;
- there must be a sufficiently large proportion of the coarsest fraction that can just be forced between the blocks;
- The washed-in material must not become sealed because the cover layer then becomes watertight and its flexibility reduced;
- an advantage of strong washed-in material is that cracking and wear remains limited and its permeability and angularity are retained; even so relatively weak materials have been used successfully;
- the material must have a sufficiently wide grain size distribution so that both small and large spaces between the blocks or stones are filled. With respect to permeability it is important that the fraction finer than the smallest gaps is as small as possible. For example; 5/50 mm should preferably be used for a columnar revetment in which the gaps vary between about 5 and 50 mm.

Rubble, gravel and crushed gravel can be used for washed-in material. Both burnt and unburnt minestone however are unsuitable because of their limited strength and low volumetric mass. Shells are sometimes used because, although their strength is limited and the volumetric mass low, the material tends to remain in the joints. This is due to its high angular hook resistance. At present Silex is rarely used as a washed-in material although the finer grades would be suitable because of its angularity and flinty strength. It must however, for environmental reasons satisfy very low tau content requirements and the grading must be precise. This is not possible at present. Steel slag is unsuitable because its tendency for hydraulic bonding. The possibility of the material bonding is high because of the wear caused by the movement of the concrete elements and the rolling action of the more or less loose slag. This release of fine material can lead to the slag and blocks adhering together and to the sealing of cavities in the slag. Phosphor slag are well worth considering provided that the strength is not too large.

5.6 Grouting mortars

Grouting mortars are used locally or in large areas to increase structural stability by binding blocks together. Generally mortars are applied at vulnerable locations, for example, at transition sections, see Chapter 10.
The most commonly used grouting mortars are cement mortars and molten asphalt. The first has the disadvantage that it produces a completely rigid structure in contrast to the molten asphalt. More important however the molten asphalt can penetrate into the underlying filter as well as between the blocks. A locally thicker cover layer is thus created. This situation is shown in Figure 28 for a cover layer overlying a filler layer and minestone. Because of the small grain size of the minestone penetration is less.

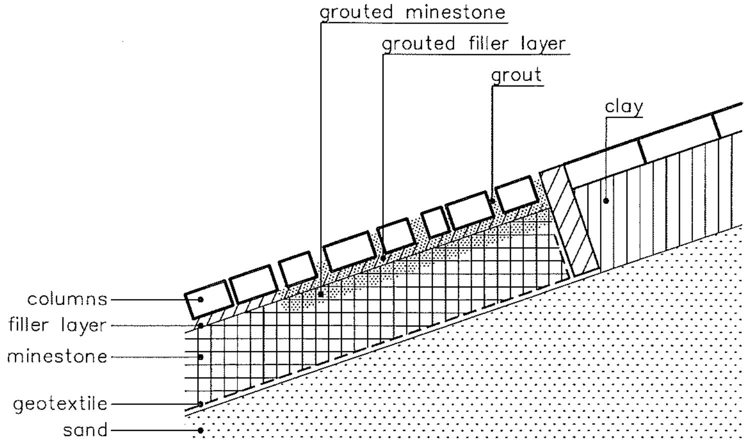


Fig. 28. Penetration of molten asphalt into the filler layer.

An important disadvantage of grouting is that a watertight cover layer is formed and large pressure differences can be created. The toe of the structure and also a strip above the transition structure must be permeable and remain so. In practice this is ensured by only grouting locally.
In old revetments the joints are often filled with rubbish which can only be grouted with difficulty. Even using a high pressure jet it is only possible to clean out the upper few centimetres of the joints which is insufficient. The revetment then has to be broken up and reset. Further information on the application of molten asphalt is given in Chapter 10 and in [Guidelines for the use of asphalt, 1984]. For information on the application of cement mortars reference should be made to CUR reports [Colloidal concrete in dike revetments, 1992].

5.7 Geotextiles

Geotextiles are laid to prevent underlying material penetrating into the upper layer and from there being flushed out [Bezuijen et al. (1990), page 89]. They are often applied between the filter and the sand. If there is clay under a granular filter a geotextile is needed to give support during construction. Sometimes a geotextile is used directly under the cover layer, for example, under blocks on sand or clay or when there are large holes in the cover layer. The material is also used in block mattresses, the blocks being mounted onto the geotextile [Bezuijen et al.(1990), page 214].
The size of openings in the geotextile (the mesh size) is important to its functioning as a dividing material. The characteristic size of the opening is determined by the sand size which has to lie on the geotextile and by carrying out standard sieve tests. The average diameter of the sand fraction, 90 % of which remains on the geotextile is, by definition, referred to as O_{90}. The O_{90} value is known for a particular geotextile by the supplier and is generally given in the specifications.
If the grain size of the material under the geotextile is larger than O_{90} the geotextile acts as a good separator. If this is not the case the grains will flush out through the geotextile if the force of water is sufficient. Water movements at the boundary between the sand and a granular filter may however be insufficient. This will depend on the permeability and the thickness of the geotextile. The values required are generally given in the specifications for the geotextile. The permeability is expressed as a filter velocity for a given loss in head across the geotextile (or vice versa).
There are many types of geotextiles, the most important being:

- *Gauze:*
  A weave comprising more or less round threads (similar to screen).
- *Strip weave:*
  A weave of flat threads, individual threads of which can often easily split.
- *Mat:*
  A weave of thick, more or less round threads, the threads of which can often easily split. A mat is generally thicker than a gauze or a strip weave.
- *Non-woven cloth:*
  A non-woven cloth of very thin threads running through each other. The cloth is often soft and easy to press flat.
- *Non-woven membrane:*
  A non-woven membrane of very thin threads running through each other pressed together into a stiff sheet. It cannot be pressed flat.

The threads are almost always made of plastic, such as polypropylene, polythene or polyamide. Natural products are rarely used. If very strict specifications are set for the strength it may be necessary to weave steel threads into the fabric.

An indication of the relationship between geotextile permeability and O_{90} is given in Table 5.

It is essential to choose the correct type of geotextile when using the material between the cover layer and a filter. The characteristic size of the opening must be such that the grains of the filler, will be stopped but not the flushed out finest fraction of the filter (minestone). If small grains (sand) are trapped, the geotextile becomes clogged and forms a sealing layer with very low permeability under the cover layer, greatly reducing the stability.

Table 5. Indicative values of O_{90} and the permeability of geotextiles.

Geotextile	Permeability (filter velocity for a loss in head of 100 mm) [mm/s]	O_{90} [mm]
Woven monofilament	100 to 500	0.1 to 1.0
Woven flat tape	10 to 100	0.05 to 0.6
Woven multifilament	5 to 50	0.20 to 1.0
Non-woven	1 to 200	0.02 to 0.20

The following is a rule of thumb for O_{90} for a geotextile directly under the cover layer:

- $O_{90} \leq D_{u15}$: to prevent the filter material being flushed out (geometrically tight or sealed)
- $O_{90} \geq \frac{1}{5} D_{u15}$: to ensure that the openings in the geotextile are not less (or barely less) than the pores in the filter.

where:
- D_{u15} = characteristic grain size of the layer directly under the geotextile [m]
- O_{90} = characteristic size of the openings in the geotextile [m]

For safety the permeability of the new geotextile should be increased by 0.25 bearing in mind the possible reduction during the working life of the structure. This safety factor only applies to a gextextile laid between the cover layer and a granular filter. For further information on geotextiles reference should be made to [VELDHUIZEN VAN ZANTEN, 1986].

5.8 Clay

Clay is often used under pitched revetments under the cover layer and also under a granular filter. It is used:

- because of its low water permeability to reduce the flow water through a dike;
- to protect the core of a dike if the cover layer is damaged;
- to limit the pressure head across the cover layer.

Clay characteristics

Clay is characterised by the following properties, see Figure 29:

- lutum/clay content, L_k[%] = percentage of particles smaller than 2 μm (mass percentage of the mineral fraction)
- silt content, S_k[%] = percentage of particles between 2 μm and 63 μm (mass percentage of the mineral fraction)
- sand content, Z_k[%] = percentage of particles larger than 63 μm, but smaller than 2 mm (mass percentage of the mineral fraction)
- water content, W_k[%] = (mass percentage of the mineral fraction).
- organic material, H_k[%] = percentage of organic material (mass of the dry material)
- Liquid Limit, W_l[%] = the water content at which a groove in the clay almost recovers after a sample is dropped 25 times from a height of 1 cm onto a firm surface
- Plastic Limit, W_p[%] = water content at which a ball of clay, when rolled into a thread of 3 mm, can be rolled no further without breaking
- plasticity index, I_p = $W_l - W_p$
- consistency index, I_c = $(W_l - W_k)/I_p$

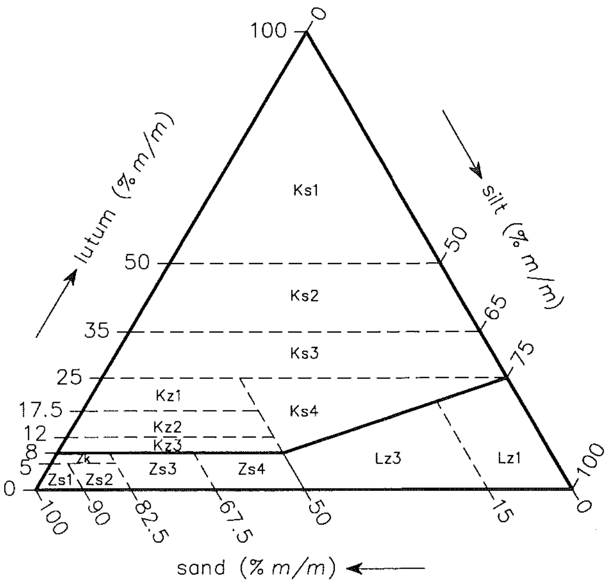


Fig. 29. The lutum/clay - silt - sand - triangle used for naming clay components. (Ks = silty clay, Kz = sandy clay, Lz = sandy loam, Zk = clayey sand, Zs = silty sand).

The Liquid Limit and the Plastic Limit are also referred to as the "Atterberg limits" of consistency.

Water tightness

Cracking, due for example to atmospheric conditions, seriously affects the water tightness of a clay layer. This is very important if the clay is near to the surface of the revetment. The sensitivity to cracking increases with the heavy content of the clay. As yet there is no way of setting specifications to guarantee water tightness. It can be assumed provisionally that a satisfactory water tightness can be achieved when a well compacted erosion-resistant clay is used which has the correct water content. The clay under a pitched revetment is generally so cracked that the permeability varies between 10^{-4} and 10^{-6} m/s.

Resistance to erosion

The resistance of clay to erosion depends mainly on the cracking and the extent to which the constituents have separated out. The extent to which this has occurred due to the atmosphere correlates well with the Atterberg limits, the organic material content, $CaCO_3$ and sand [Muijs, 1989]. This has been established for clay on dikes covered by grass. No investigations have been carried out for clay under block pitching and the Muijs criteria can therefore only be applied provisionally.
The minimum requirements to ensure some resistance to erosion are:

- organic content: $H_k < 5$ %, and
- $CaCO_3$ content: < 25 %, and
- sand content: $Z_k < 40$ %

The following limits are provisional:

- good resistance to erosion: $W_l > 45$ % and $I_p > 0.73\ (W_l - 20)$;
- average to good resistance to erosion: others;
- poor resistance to erosion: $I_p < 18$ % or $I_p < 0.73\ (W_l - 20)$

These requirements are on the safe side. For further developments in clay specifications reference should be made to the TAW-publication "Quality standards for clay in dike construction" which is to be published in 1995. Good resistance to erosion is essential for clay used directly under the cover layer.

Fig. 30. Erosion channels in the surface of clay.

W o r k a b i l i t y

Clay which is obtained from below groundwater level, for example from mud flats or saltings, should be allowed to stand and "ripen". If ground, which has been saturated with water, dries out the groundwater is lost by evaporation, initially directly from the surface and then from shrinkage cracks and vegetation. This drying process, which is accompanied by physical and chemical changes, is referred to as "ripening". Reduction in water content leads to an increase in grain stress and, as a result, the clay becomes accessible and workable.

If the clay is relatively wet (unripe) it must be allowed to ripen on a clay dump. The ripening process can be accelerated by using pumps in the clay dump. Even so clay takes many months to ripen. Use of unripe clay on a dike should be avoided because it will ripen in the structure and cracks will develop. This can reduce the resistance to erosion and result in high water permeability.

Clay can be laid on a dike when the water content satisfies the requirement that:

$$W_k \leq W_l - 0.75 \cdot I_p \tag{7}$$

The following processing and compaction specifications are recommended:

- the optimum value for the moisture content (W_{opt}) is the minimum allowable moisture content determined by the standard Proctor Test;
- the clay should be laid and compacted in layers of maximum thickness 0.4 m;

- the pressure on the clay, when compacting with a bulldozer, should not be too high; criteria will be given in this respect in the clay specification manual currently being prepared;
- clay should not be worked in freezing conditions;
- the clay should be homogenous and should not contain any contaminants.

Clay should only be laid directly under the cover layer above the average high water level. A geotextile and a (thin) filter layer are necessary for construction reasons for structures in the tidal zone.

5.9 Sand

Sand is generally used for the core of a dike. If a clay layer is laid on the sand the sand properties will not affect the functioning of the revetment and/or the filter. If a granular filter or a geotextile is laid on the sand then the sand grain size determines the dimensions of the filter and/or the geotextile.
The characteristic grain size of sand used for granular filters, is D_{b50}, the grain size exceeded by 50 % by weight. D_{b90} also influences the dimensions of the geotextiles. In general, the following relationships apply in Dutch conditions:

$$0.1 \text{ mm} < D_{b50} < 0.4 \text{ mm}$$
$$D_{b90}/D_{b50} = 1.3 \text{ to } 1.5 \tag{8}$$

In addition to the above relationships, the permeability, k, and the internal angle of friction, Φ, are important in connection with geotechnical stability.

Indicative values of permeability, for normal porosity (0.35 to 0.45) and water viscosity (1.1 to 1.3 mm^2/s), are given in Table 6.

Table 6. Indicative values for the permeability of a sand containing hardly any silt.

Grain size D_{b50} [mm]	Permeability k [mm/s]
0.10	0.06
0.15	0.14
0.20	0.24
0.30	0.54
0.40	1.00

The internal angle of friction for well compacted sand is 35° to 40°, see Section 9.2.

Chapter 6

EXTERNAL LOADS

6.1 Introduction

The external loads on a dike revetment can be subdivided into two main groups: hydraulic loads and other loads. The primary function of a dike is flood protection, even under extreme conditions. Therefore the revetment must be able to withstand hydraulic loads [Bezuijen et al. (1990), page 45]. These loads can include, see Figure 31:

- water level:
  - high tide levels and/or river flood levels;
  - temporary high water levels due to oscillations in atmospheric pressure, squalls or seiches;
  - wind set up;
  - unexpectedly low water levels.
- waves:
  - wind waves and swell;
  - ship waves.
- currents:
  - currents along the structure;
  - currents over the crest of the structure and wave overtopping.

Seiches are periodic fluctuations in water level (long waves) with a period of between 15 and 45 minutes. These are the result of macroscopic turbulence in the atmosphere and, in this way, are similar to squalls.

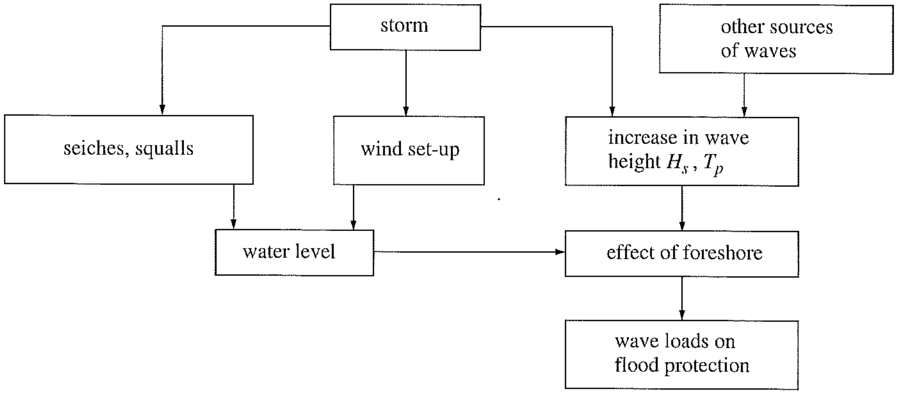


Fig. 31. The relationship between hydraulic boundary conditions resulting from a storm.

Surges are single occurrences which can be caused by:

- intense storms acting on a small area of sea;
- sudden changes in atmospheric pressure;
- changes in wind force and direction;
- earthquakes, volcanic eruptions, etc.

Both seiches and squalls can cause an increase in water level of 15 to 50 cm. A method for the calculation of this rise is given in the [Delta Commission Report, Parts 1 and 4, 1960, 1961]. Seiches and squalls can result in a considerably higher rise (resonance) in estuaries and enclosed harbour areas. These effects are referred to as seiches. Seiches can also be formed by other phenomena.
The water level (the still water line, SWL) is important to the design since it is the level at which hydraulic loads attack a pitched revetment; in itself it does not equate to a load. Loads due to wind waves are discussed in the following section. Ship waves and other loads caused by flow over the crest of, for example, a groyne or a summer dike are treated later in the present chapter.

6.2 **Wind waves**

6.2.1 *Characterisation of wave fields*

Wind waves produce significant loads on sea and lake dikes. The height of a train of wind waves is usually not constant and these waves are therefore referred to as "irregular". The characteristic wave height of the train is referred to as the significant wave height, H_s. By definition this is equal to the average of the highest one third of the waves and is generally the height estimated by eye [Bezuijen et al. (1990), page 45]. If the waves do not break the wave height distribution is similar to a Rayleigh distribution. The significant wave height is then equivalent to the height exceeded by 13.5% of the waves, $H_{13.5\%}$. The following relationships between different wave heights apply, see Figure 32:

$$H_{1\%} = 1.52 H_s \tag{9}$$

$$H_{5\%} = 1.22 H_s \tag{10}$$

$$H_{13.5\%} = H_s \text{ (significant wave height)} \tag{11}$$

$$H_{50\%} = 0{,}59 H_s \text{ (median wave height)} \tag{12}$$

where:

$H_{x\%}$ = wave height exceeded by x% of the waves [m]

The wave height in a storm is usually characterised by the value of H_S at the height of the storm and develops over a period of a quarter to half an hour.

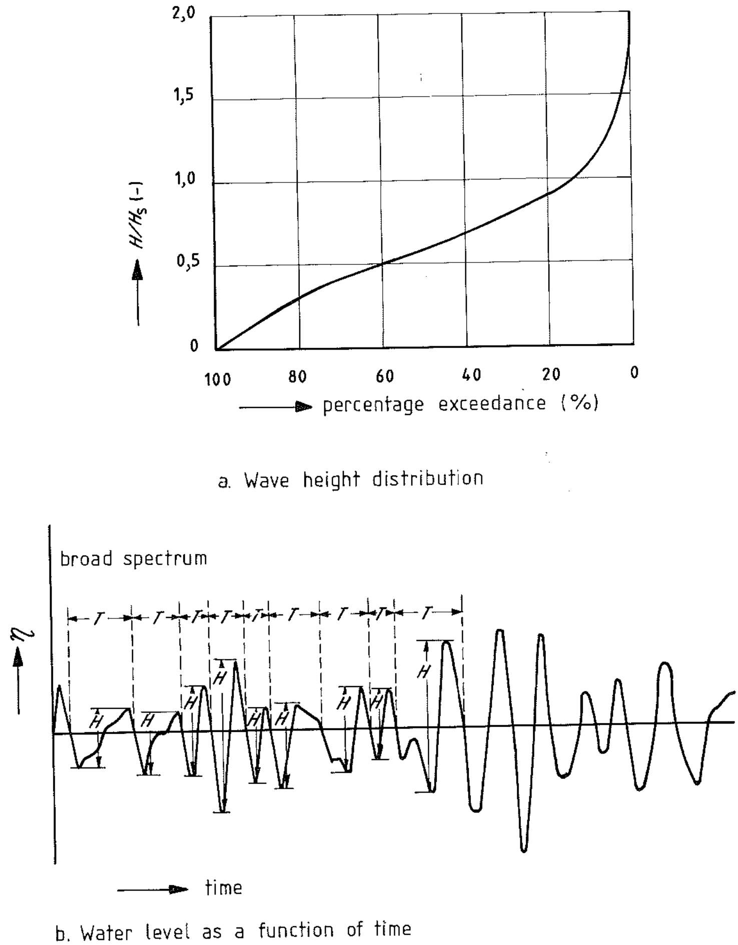


Fig. 32. Wave height and period of irregular waves.

The wave period in a train of waves is also not constant. This aspect of irregular waves is described by a wave spectrum. A wave spectrum gives a subdivision of the wave energy as a function of the period. This subdivision can be compared with a histogram of the period categories for a number of waves are set out, the higher waves being given more weight than the lower. The width of the spectrum can be seen as a dimension for the distribution of wave periods. The wave train is characterised by the wave period at the top of the spectrum (T_p). This can be interpreted as the period of the waves with the highest energy density, see Figure 33. The peak period is somewhat longer than the average wave period (T_z):

$$T_p/T_z = 1.1 \text{ to } 1.3 \qquad (13)$$

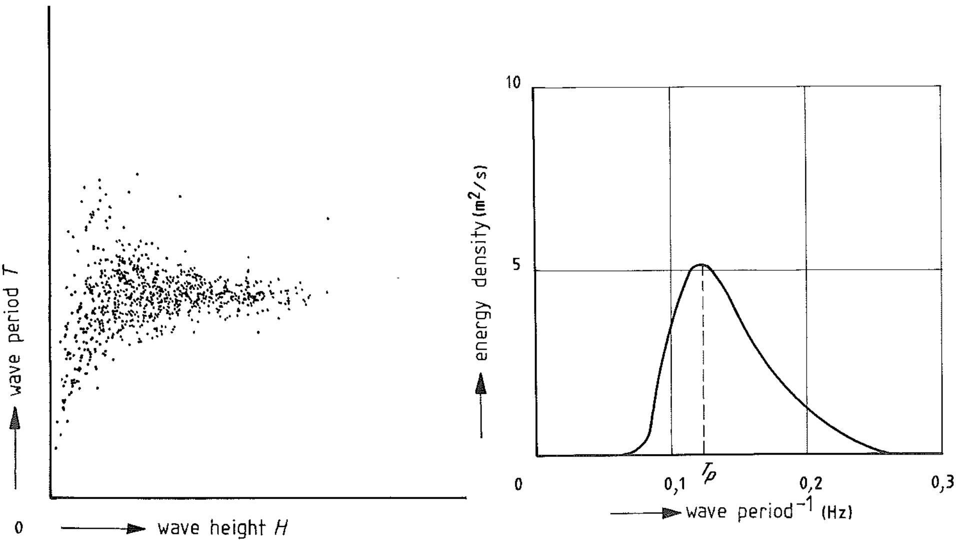


Fig. 33 Characteristic wave parameters: H_s and T_p.

A spectrum can have two peaks. This can be because one group of waves has developed in storm some distance away, the remainder being due to a local storm. The first group of waves are referred to as swell. The period of these waves is usually relatively long and the height relatively low. The stability of the structure should take into account both swell and local waves, since it is assumed that individual waves can initiate damage. In addition to wave height and period the wave length can also be important. The wave length is generally dependent on the wave period and the water depth. For deep water this can be calculated using linear wave theory [SKOVGAARD, 1974]:

$$L_{op} = \frac{gT_p^2}{2\pi} \text{ (deep water: if } h/L_{op} > 0.25) \tag{14}$$

where:

T_p = wave period at the peak of the spectrum [s]
L_{op} = wave length in deep water based on T_p [m]
h = water depth [m]
g = acceleration due to gravity [9.8 m/s^2]

The wave length in deep water is used in the design method presented in this manual to take into account the effect of wave period:

Wave steepness: H_s/L_{op}

The wave steepness thus defined is for use only in calculations. In practice it is defined as the quotient of the wave height and the wave length at the local water depth instead of in deep water. The wave length in relatively shallow water is much more difficult to calculate and in practice is not used.

6.2.2 *Design wave height and period*

The design wave height and period at a structure are, in the first instance, determined by the local wave conditions in the sea or lake and by any swell. Local wind generated waves can be estimated using the SMB method (Shore Protection Manual, 1977]. It must be stressed that this only gives a first estimate which can differ, in practice, by 20 to 40 %. Better results can be obtained by, for example, using the HISWA computer program.
According to the SMB method wave conditions are determined by four parameters:

1. *Wind speed:* u [m/s]
   The wind speed at 10 m above sea level

2. *Fetch:* F [m]
   The length of sea or lake in the wind direction determines the height to which waves can grow, see Figure 34.

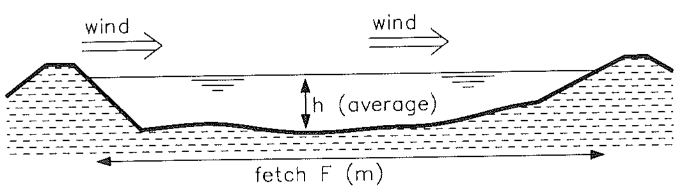


Fig. 34. Fetch F.

In most cases the fetch has to be determined by averaging the lengths from a range of directions [Guide for the design of river dikes, 1991]. The fetch in the wind direction is first determined (along the central ray in Figure 35). Then all the fetch lengths (ray lengths) in the directions between –42° and +42° relative to the central ray are determined in intervals of 6°. The average value for the particular wave conditions, is calculated using the following formula:

$$F = \frac{\sum \{F_i \cdot (\cos\beta_i)^2\}}{\sum \cos\beta_i} \tag{15}$$

where:

F = the resulting fetch

F_i = the ray length in the direction β_i relative to the central ray [m]

β_i = the angle of ray i relative to the central ray [°]

$\sum\{..\}$ = summation of all values of i

A worked example is given in Section 15.3.

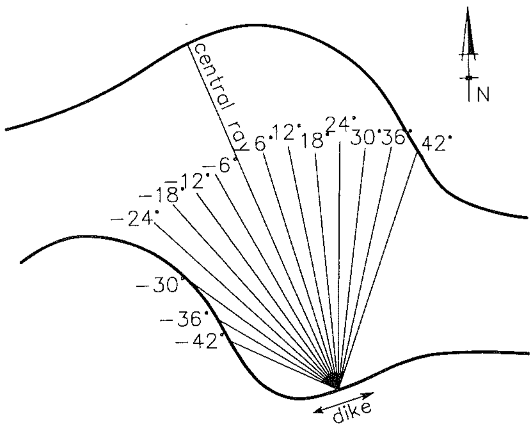


Fig. 35. Wave rays used for calculating the fetch.

3. *Storm duration:* t [s]
   Wave heights grow as the storm persists until a fully grown sea state is reached. The shorter the fetch the more quickly is the maximum wave height reached.

4. *Average water depth:* h [m]
   The deeper the water, the higher the waves can be until "deep water" is reached. Beyond this the depth no longer affects the wave conditions.

Usually the fetch in estuaries and lakes determines the wave conditions and not the storm duration because, with the limited fetch, the waves reach their ultimate height after a short time. On the sea coast the fetch is often so large that the size of the waves is determined by the storm duration. Values for the significant wave height (H_s) and period at the peak of the spectrum can be read off Figures 36 to 44 using the u, F and t parameters. The wave parameters can be interpolated for intermediate values of water depth.

Figure 44 must be used if the storm duration is relatively small. If the duration is smaller than indicated in Figure 44 a value for the fetch has to be obtained in a different way. The fetch is read from the figure for the storm duration and wind speed and then used in Figures 36 to 43 to determine the wave height and period. Generally the storm duration will be longer than the minimum used in Figure 44, in which case Figures 36 to 43 are not required.

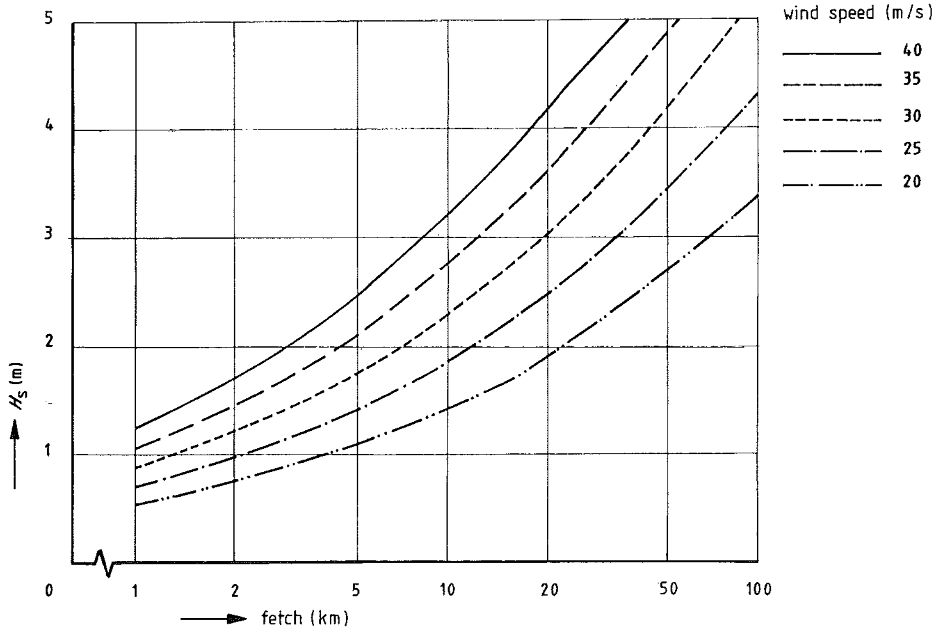


Fig. 36. Significant wave height (H_s) as a function of fetch and wind speed with a mean water depth of 50 m (providing the storm lasts longer than indicated in Figure 44).

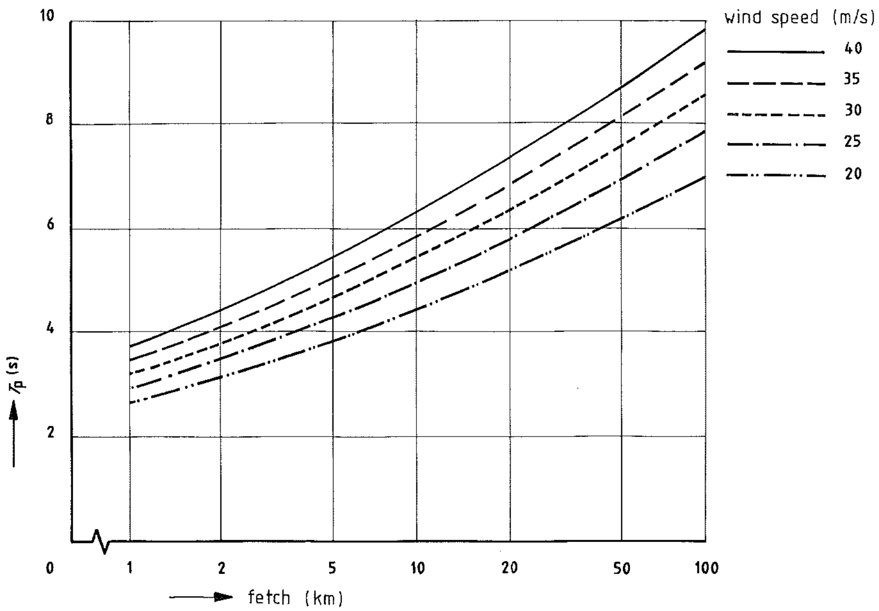


Fig. 37. Wave period (T_p) as a function of fetch and the wind velocity with a mean water depth of 50 m (providing the storm lasts longer than indicated in Figure 44).

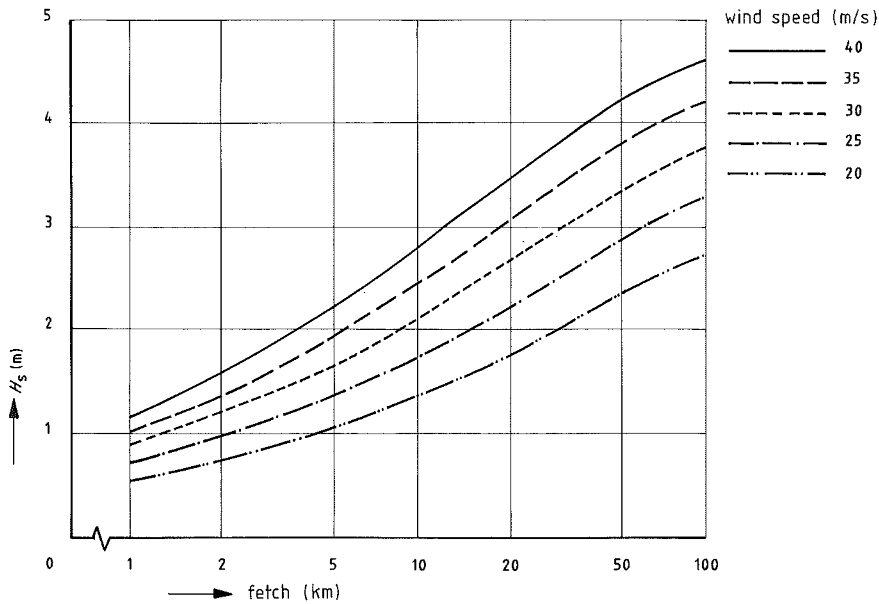


Fig. 38. Significant wave height (H_s) as a function of fetch and wind speed with a mean water depth of 20 m (providing the storm lasts longer than indicated in Figure 44).

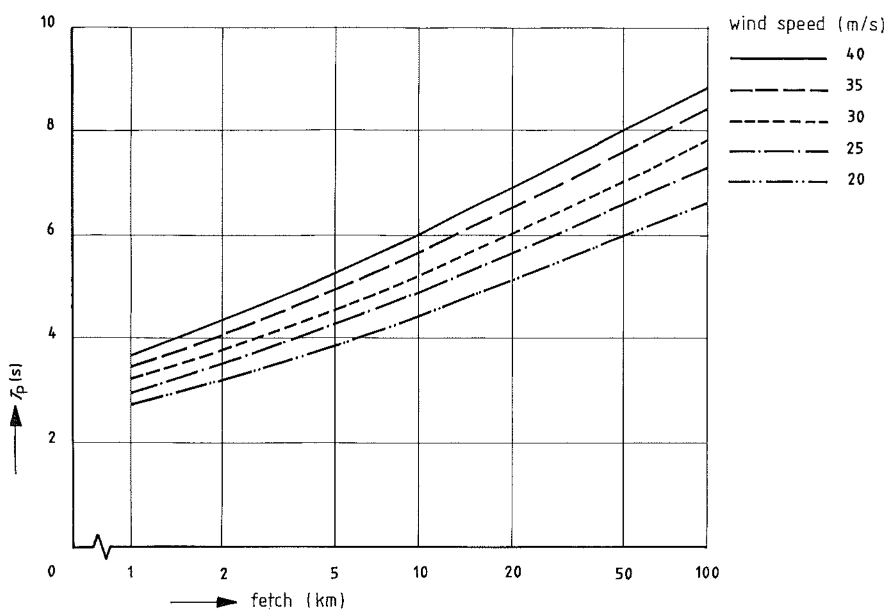


Fig. 39. Wave period (T_p) as a function of fetch and the wind speed with a mean water depth of 20 m (providing the storm lasts longer than indicated in Figure 44).

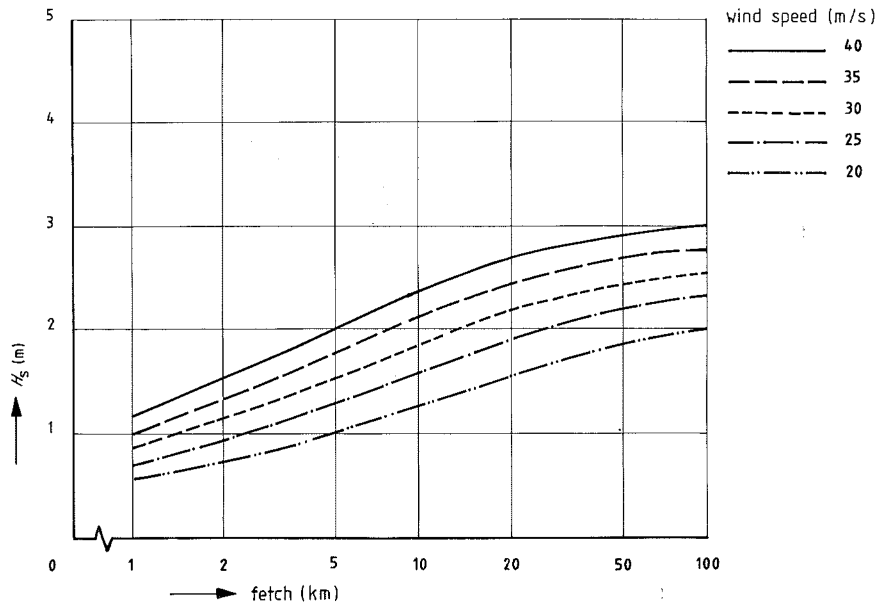


Fig. 40. Significant wave height (H_s) as a function of fetch and wind speed with a mean water depth of 10 m (providing the storm lasts longer than indicated in Figure 44).

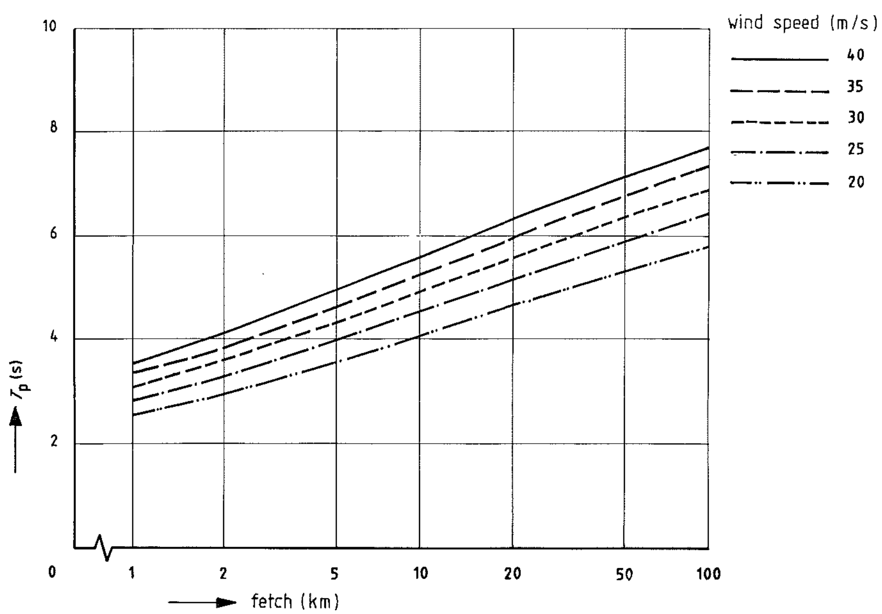


Fig. 41. Wave period (T_p) as a function of fetch and the wind speed with a mean water depth of 10 m (providing the storm lasts longer than indicated in Figure 44).

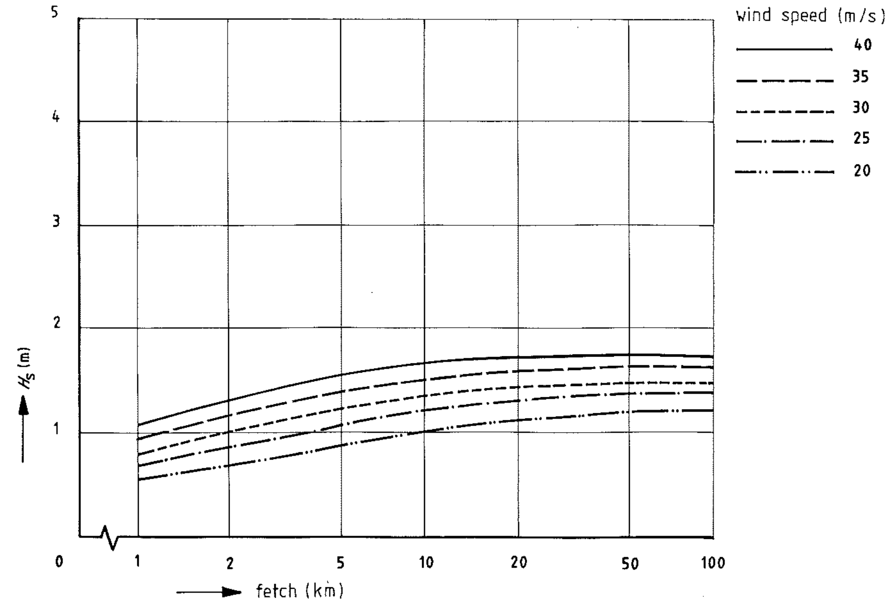


Fig. 42. Significant wave height (H_s) as a function of fetch and wind speed with a mean water depth of 5 m (providing the storm lasts longer than indicated in Figure 44).

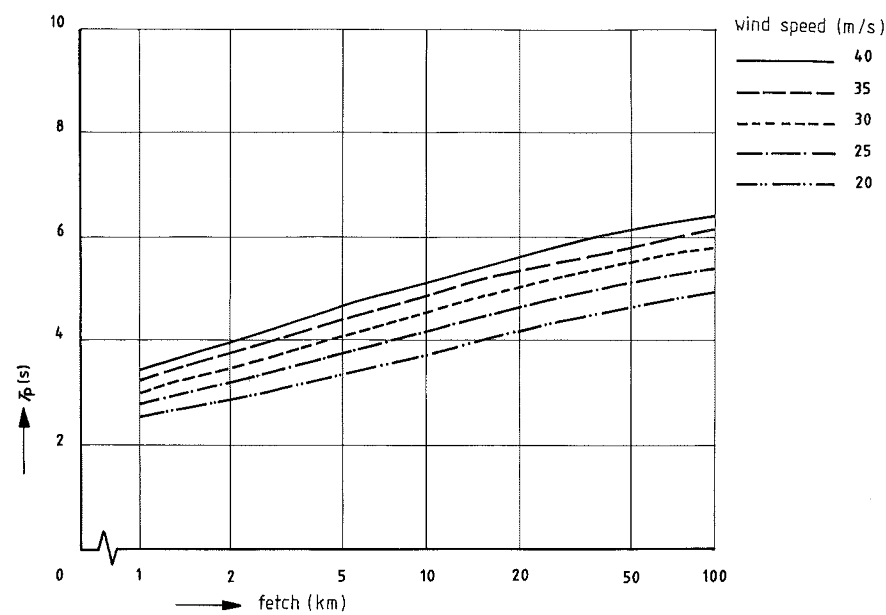


Fig. 43. Wave period (T_p) as a function of fetch and the wind speed with a mean water depth of 5 m (providing the storm lasts longer than indicated in Figure 44).

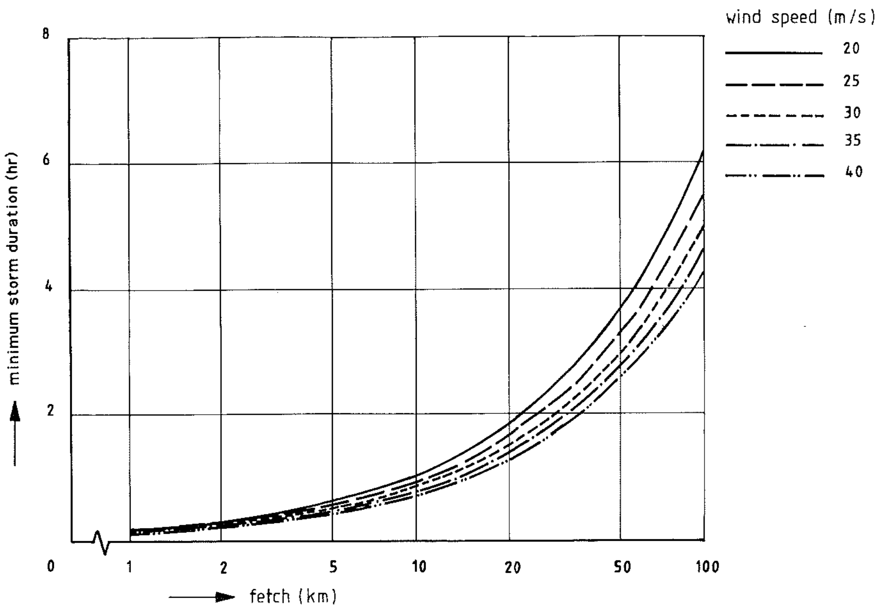


Fig. 44 Minimum storm duration required to fully develop a swell (wave height and wave period according to Figures 36 to 43).

Worked examples:

1. Given: $F = 10$ km, $h = 15$ m, $u = 30$ m/s, $t = 8$ hr

   Calculation: Sea state fully developed after 0.8 hr, see Figure 44, therefore fully developed wave parameters must be interpolated between wave heights $h = 10$ m and 20 m:
   If $h = 10$ m: $H_s = 1.8$ m, see Figure 40;
   $T_p = 4.8$ s, See Figure 41.
   If $h = 20$ m: $H_s = 2.1$ m, see Figure 38;
   $T_p = 5.2$ s, see Figure 39.

   Result: $H_s = (1.8 + 2.1)/2 \approx 2.0$ m
   $T_p = (4.8 + 5.2)/2 \approx 5.0$ s

2. Given: $F = 50$ km, $h = 20$ m, $u = 40$ m/s, $t = 3$ hr

   Calculation: Sea state fully developed after 3.8 hr, see Figure 44, therefore not fully developed. According to Figure 44 the fetch value to be used in calculations for a storm duration of 3 hr is $F = 39$ km.

   Result: Figure 38: $H = 4.1$ m
   Figure 39: $T_p = 7.5$ s

The figures can also be used to determine the wave period if the wave height is known. The calculation is as follows:

1. Determine the related wind speed, u, for values of H_s and F (using Figures 36, 38, 40 or 42).
2. Determine the related wave period, T_p, for values of u and F (using Figures 37, 39, 41 or 43).

An indication of the wind speeds which, according to the [Delta Commission Report, Part 6, 1961], lead to a water level with a probability of exceedance of 10^{-4}/year (basic level) with a certain tide, is given in Table 7. A wind speed of 24.5 to 28.4 m/s is equivalent to a wind force of Beaufort 10, 28.4 to 32.6 m/s is wind force 11 and, above 32.6 m/s, wind force 12.

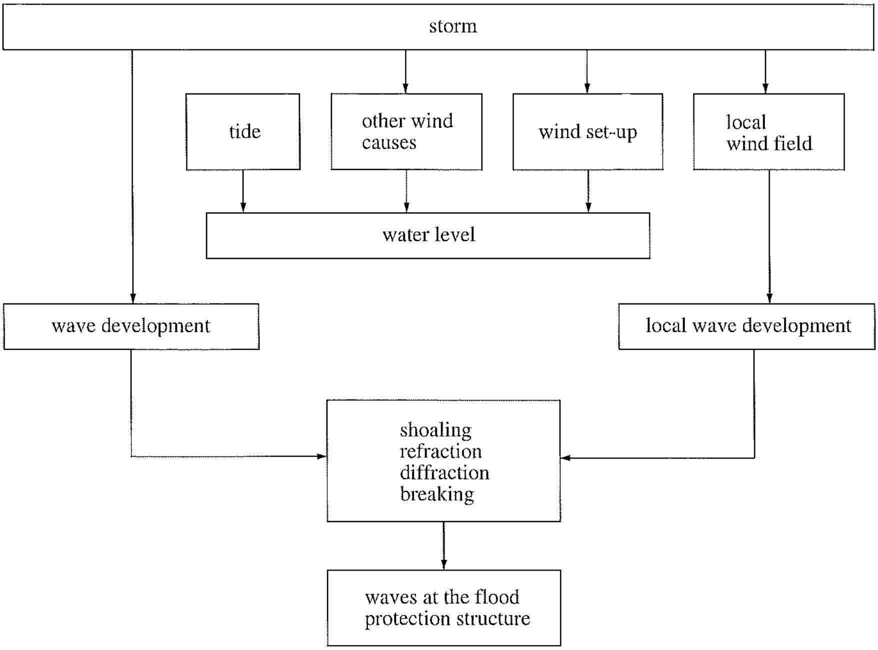


Fig. 45. Processes affecting the wave boundary conditions at a structure.

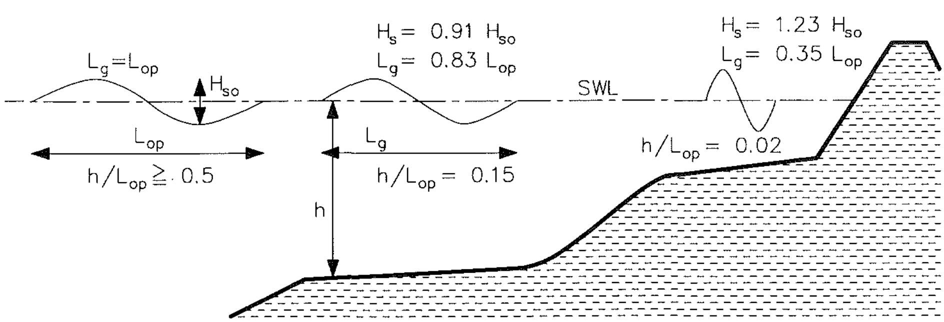
The wind speeds are linked statistically with the occurrence of a high water level. The wind speeds are, for example, higher with a storm from the southwest (with a lower water level) with a probability of exceedance of 10^{-4}/year.
A report is to be published every five years for all Dutch sea dikes for design purposes and to check boundary conditions, beginning in 1993. For the Lower Rivers Area, reference should be made to the [Lower Rivers Manual, 1989].

Table 7. Wind speeds which, with a certain tide may cause a water level with a probability of exceedance of 10^{-4}/year, compared with the flood disaster of 1953.

	probability of exceedance of 10^{-4}/year			1953 storm		
location	level [m]	wind speed [m/s]	direction [°]	level [m]	wind speed [m/s]	direction [°]
Vlissingen	5.6	31	320	4.5	26	320
Hook of Holland	5.0	32	320	3.8	29	320
Den Helder	5.0	35	300	3.2	27	300

The wave heights and period calculated using the method described above take into account the general geometry of the sea or lake near to the structure, but often the effect of the bed (the shape of depth contours) immediately in front of the structure is not discounted. The following processes can therefore have important effects on wave boundary conditions, see Figure 45.

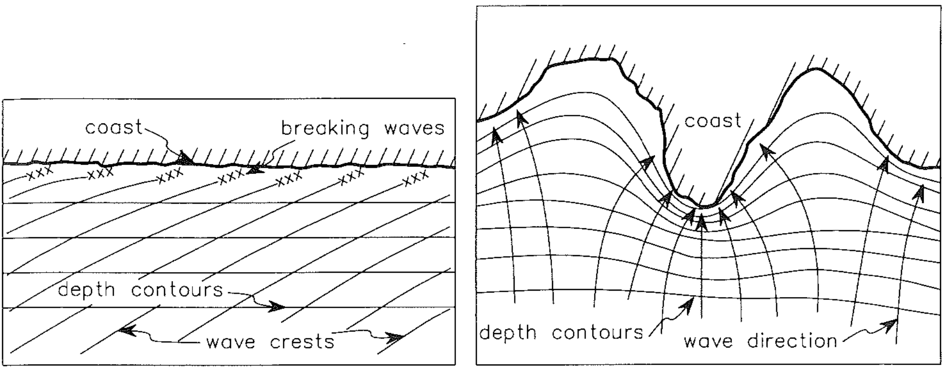
1. *Shoaling*, see Figure 46
   According to linear wave theory, when a deep water wave enters shallow water the height is first reduced (to about 90 % of the deep water wave height for $h/L_{op} = 0.15$). The heigh continues to grow with the further reduction in water depth. If $h/L_{op} = 0.15$ the wave height becomes equivalent to the deep water wave height; if $h/L_{op} = 0.02$ the wave height increases to 123 % [SKOVGAARD, 1974].



H_{so} = significant wave height in deep water
L_{op} = wave length in deep water
H_s = significant wave height in water depth, h
L_g = wave length in water depth, h

Fig. 46. Shoaling (change in wave height due to changing water depth).

2. *Refraction,* see Figure 47
When the direction of wave propagation is at an angle to the depth contours, the direction changes. With decreasing depth (relative to the direction of propagation of the waves) the angle becomes closer to 90°. As a result the wave height is generally reduced. With converging waves the height increases, for example, at a spit or headland. Refraction can also occur as a result of a non-uniform current (stream refraction).



a. straight coastline b. irregular coastline

Fig. 47. Plan view of a wave field showing refraction.

3. *Diffraction,* see Figure 48
If an island or a shoal stands in front of the coast the waves diffract so that there are waves behind the island. On the coast behind the island and the immediately adjacent stretches of coast, the waves are lower. At some points on the coast adjacent to the island the wave height can be higher than H_s (a maximum of 17% higher).

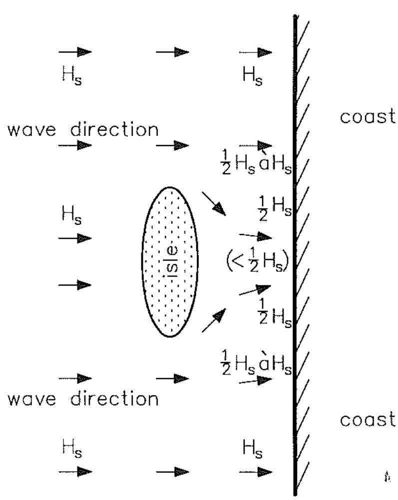


Fig. 48. Diffraction around an island off the coast.

4. *Breaking*

Waves break as soon as they are too steep or if they enter shallow water and as a result the wave steepness (H_s/L_{gp}) and the relative wave height (H_s/d) are restricted. The following can be assumed:

– as a result of wave steepness:

$$(H_s)_{max} \approx 0.1 \cdot L_{op}\sqrt{\tanh(2\pi d/L_{op})} \tag{16}$$

– as a result of water depth:

$$(H_s)_{max} \approx 0.5 \cdot d \tag{17}$$

where:

d = ruling depth in relation to breaking, see Figure 49 [m]
L_{gp} = wave length in shallow water [m]
h = local water depth [m]

$$\tanh(x) = \frac{e^x - e^{-x}}{e^x + e^{-x}} \text{ (in this case: } x = (2\pi d)/L_{op})$$

Since waves do not break immediately on entering shallow water it can be assumed that the height is determined by the water depth at some distance in front of a particular point. This distance can be assumed to be half of the deep water wave length, see Figure 49.

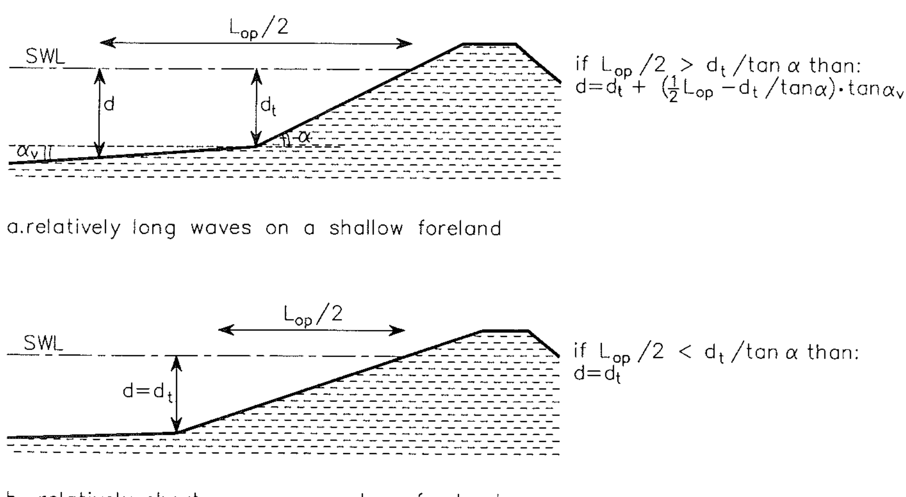


Fig. 49. Ruling water depth, d, and depth at the toe, d_t.

Depth-limited breaking gives a maximum wave height for $H_{2\%}$ of:

$$(H_{2\%})_{max} = 0.6 \cdot d \tag{18}$$

The depth-limited wave height, H_s, can be determined more accurately using the figures in Appendix A. The "rules of thumb" discussed above are described in detail in Appendix I.

The wave period is hardly affected when waves break.
The wave conditions can be *estimated* with the above rules of thumb and diagrams. It is recommended however that more accurate values are determined when preparing a design. Such values can be obtained using, for example, the ENDEC computer program [1988]. Reference should also be made to [VAN DER MEER, 1990], [DIFRAC, 1986] and [HISWA].
The loads on a dike slope depend on the way in which the waves break on the slope. This is characterised by the breaker parameter, ξ_{op}:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{(H_S/L_{op})}} \tag{19}$$

where:

ξ_{op} = breaker parameter [–]
α = slope angle [°]

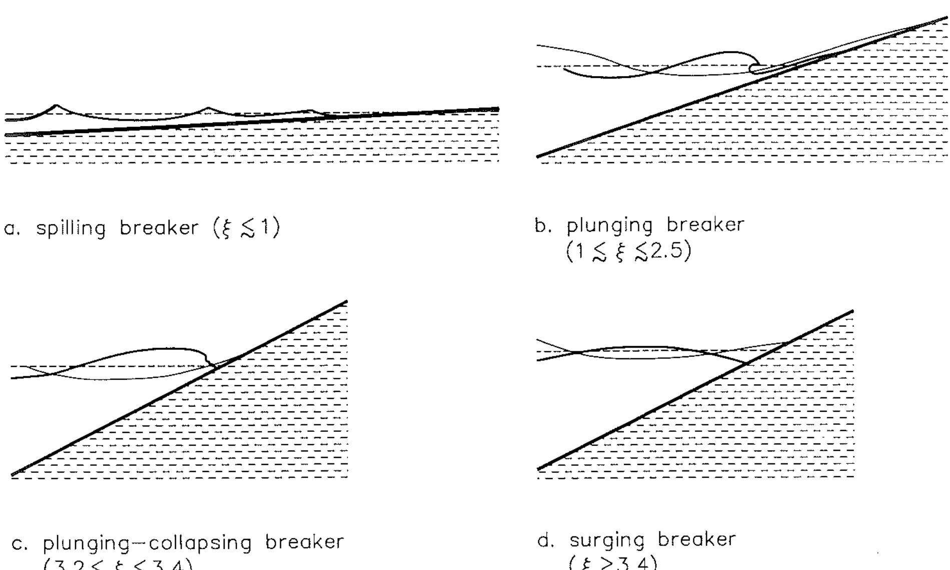
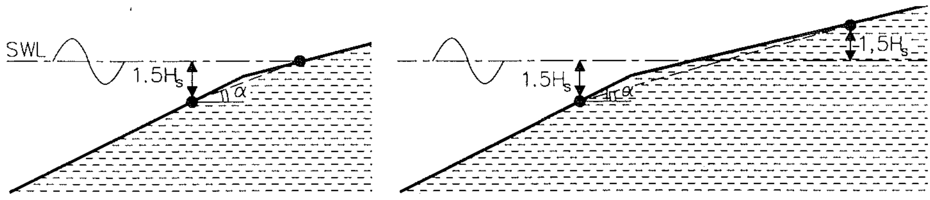


Fig. 50. Breaker types on a smooth slope.



a. slope in relation to type of breaker

b. slope in relation to wave run up

Fig. 51. Average slope angle, α.

Various types of breaking can occur, depending on the breaker parameter, see Figure 50, [Guide to Concrete Dike Revetments, 1984].
For complicated slopes, for example, where the slope above the water line differs from that below, the method of breaking and the loads on the structure are generally determined by the average slope between the water line and a point one wave height below, see Figure 51. For wave run-up however the average slope below and above the water line are important, see Section 7.2.

6.3 Ship waves

Water movements due to passing ships can be subdivided, see Figure 52, into:

– primary ship waves	– front waves
	– temporary drop in water level
	– stern waves
– secondary ship waves	– front and/or edge waves
– currents	– return flow
	– propellor jets

This section of the manual concentrates on wave phenomena, since the current speeds developed (return flow and propellor jet) are too small to affect the stability of pitched revetments (less than 2 m/s for types of vessels).
Primary ship waves are only important if the ship passes close to the bank (less than 10 times the beam of the ship), for example, close to canal banks, dikes and groynes. These waves are less important in the relatively wide channels which can occur, for example, during high floods. Secondary ship waves can propagate over hundreds of metres and can therefore attack dikes which are some distance from the navigation channel. Because of the small wave height however the wave loads are relatively small. Ring dikes are generally not provided with a hard revetment, except in situations where unusual loads can occur, for example, locally where a river flows along a dike (an estuary dike). A heavy revetment may be necessary in areas exposed to particular storm directions (wind waves, see Section 6.2). Ship-induced water movements depend on the type of ship, the passage (speed and position in the navigation channel)

and the dimensions and geometry of the navigation channel. Water movements near the slope can be calculated for given vessels and navigation channels. Advanced computing procedures are available [Damage to shipping channel cross sections, 1988]. Software packages, for example, [DIPRO, 1989], can also be used. A simplified method of calculation is presented below which can also be applied to pitched dike revetments and gives conservative results.

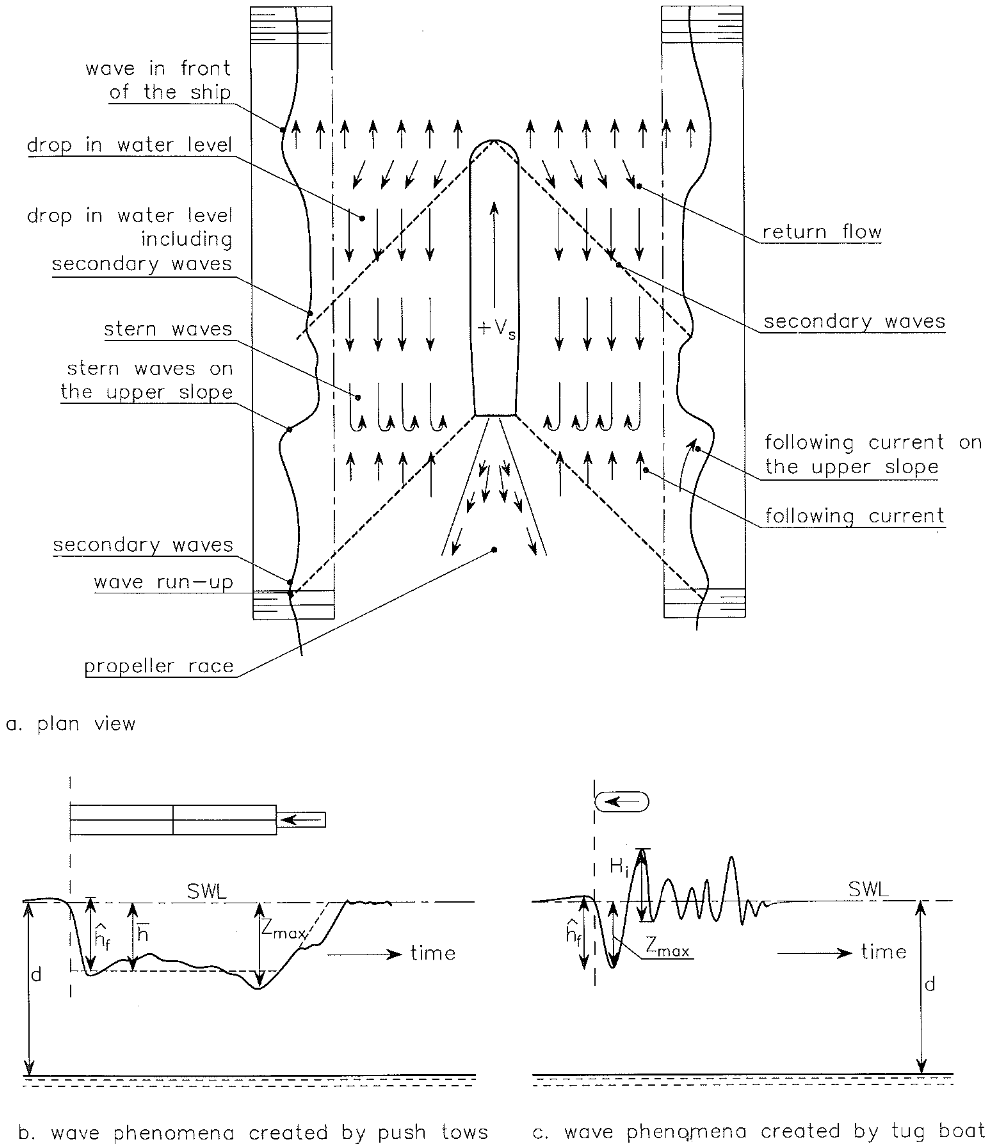


Fig. 52. Typical water movements created by passing ships.

The design loads are determined by the type of ship used on inland waterways. Inland waterway classes, types of ships and related dimensions are given in Table 8.
For locations where seagoing ships pass close to a structure the design takes into account the largest ship and the maximum allowable ship speed. Secondary ship waves are independent of ship size and therefore only the fastest ship speed needs to be considered.

Table 8. Inland waterway vessel classes in the Netherlands with the pertaining decisive vessels.

waterway class	ship type	ship length L_s [m]	beam B_s [m]	loaded draft T_s [m]	unloaded draft (assumed) T_{ong} [m]	B_s/T_s [–]	L_s/B_s [–]	L_s/T_s [–]
I	Spits	39	5.10	2.40	1.20	2.1	7.7	16.0
II	Kempenaar	55	6.60	2.50	1.40	2.6	7.6	20.0
IIA	Hagenaar	56 to 67	7.20	2.55	1.40	2.8	8.5	24.1
III	Dortmund – Ems Canal	67 to 80	8.20	2.60	1.50	3.2	8.2	26.8
IV	Rhine – Herne Canal	85	9.50	2.80	1.60	3.4	8.9	30.4
V	Large Rhine							
VI	Push tow in 2 × 2	110	11.50	3.00	1.70	3.8	9.6	36.7
	formation	185	22.80	3.30	0.60	6.9	8.1	56.1

NB. No guidelines are set for Classes V and VI. The data given above is based on the current dimensions of vessels in these classes

In order to calculate the primary and secondary ship wave characteristics values are needed for the following parameters, see Figure 53:

- ship's length, L_s,
- ship's beam, B_s,
- loaded draft, T_s, (or the unloaded draft averaged over the length of the ship, T_{ong})
- position y of the ship, relative to the axis of the navigation channel,
- the wet channel cross sectional area, A_C,
- water depth, d,
- channel width at the water line, b_w.

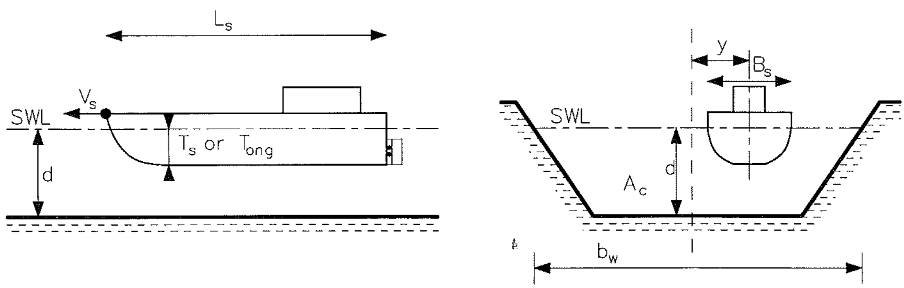


Fig. 53. Relevant dimensions of ships and waterways.

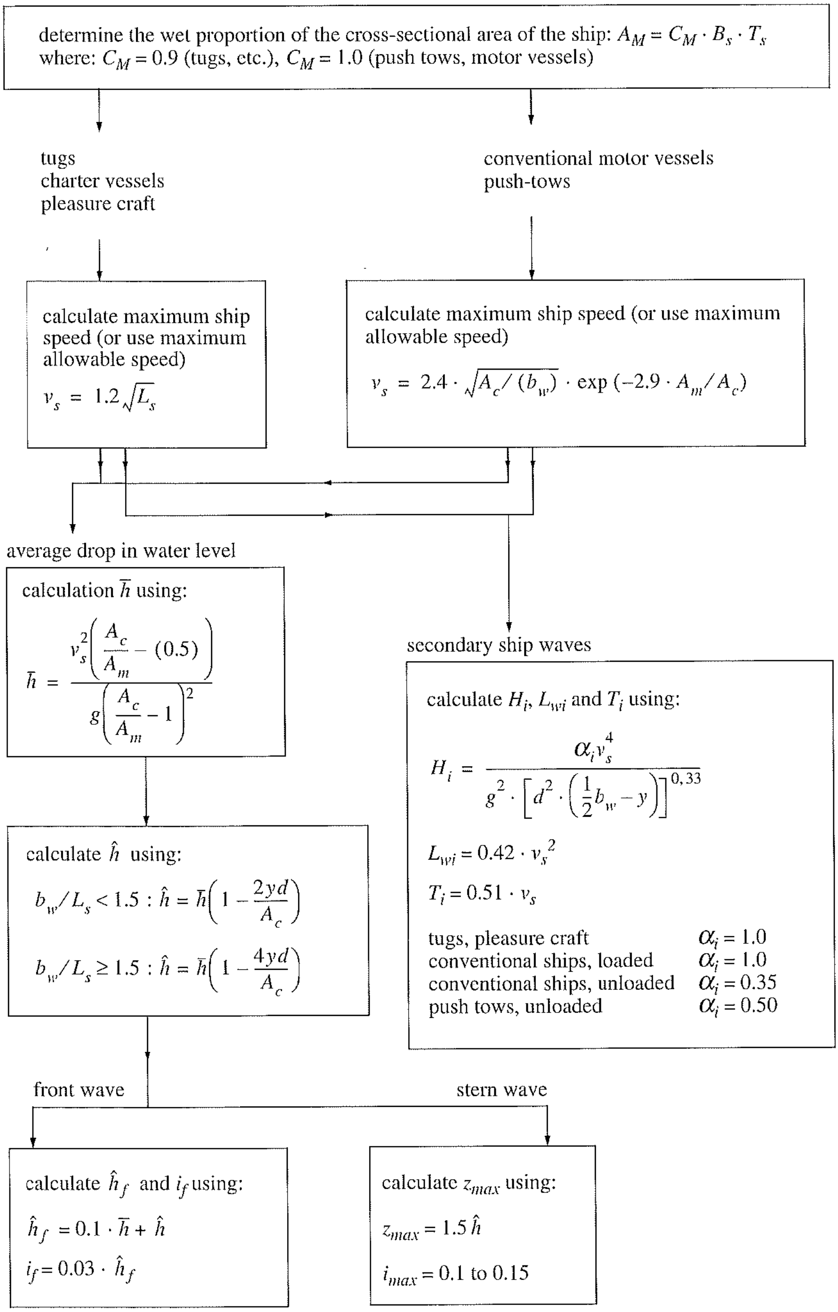


Fig. 54. Flowchart for the computation of ship-induced loads.

If the values of these parameters are known, the wave characteristics can be determined, see Figure 54. Push tows and conventional motor ships are generally responsible for primary ship waves, tugs, charter vessels, and pleasure craft for secondary waves. The calculation procedure is shown in Figure 54. Using this procedure the forces can be calculated for all types of ships; see also [Bank design recommendations, 1989].

The equations given for calculating primary wave heights should only be applied for channel widths less than 12 times the beam (bw ≤ 12Bs). When the channel is wider it can be assumed that the primary wave heights at the banks can be neglected. For secondary waves there is no width restriction attached to the equations. For secondary waves approaching the bank at an angle, the wave height can be reduced using:

$$H = H_i \sqrt{\cos\beta} \tag{20}$$

where:

H_i = wave height of secondary wave approaching at an angle [m]
H = wave height of equivalent wave approaching normal to the bank [m]
β = angle of approach of secondary waves ≈55° (normal approach: $\beta = 0°$ [°]

The average fall in water level, $\hat{h}$, the front wave height, $\hat{h}_f$, and the stern wave height, z_{max}, can vary at the bank between 0.3 and 0.5 m, although waves of up to 1 m can also occur. The duration of the drop in water level can be 20 to 60 s and that of the bow and stern wave 2 to 5 s, depending on the type of ship and ship speed. The average secondary wave height, H_i, can vary between 0.25 and 0.50 m. Maximum values can increase with small, fast moving ships up to about 1 m, with periods, T_i, varying between 2 and 3 s.

6.4 Currents with no waves

Current loads usually do not affect the stability of pitched dike revetments which are stable under wave attack. Current loads however can be important if they occur when the revetment is under heavy wave attack. The following situations can be critical:

- very turbulent currents such as those which occur in a strongly decelerating flow;
- flow over the crest of a dike or groyne when the water level is higher than crest level;
- flows of 2 m/s or more along the structure.

In some cases the thickness of the revetment is determined by factors other than waves and currents, for example, the method of construction or the need to prevent vandalism. Strongly decelerating flows can occur near outfall structures, for example, non-return sluices and cooling water outlets. The turbulence, in these situa-

tions, can produce rapidly varying pressures on the revetment, which can cause individual blocks to lift.
The flow over the crest of a dike or groyne is important particularly if the water level is higher than the crest level. Critical flow can develop over the crest which can be expressed as follows, (free discharge weir conditions, see Figure 55):

$$d_k = \tfrac{2}{3} d_o \tag{21}$$

$$u_k = \sqrt{g d_k} = 2.5 \cdot \sqrt{d_o} \tag{22}$$

where:

u_k = current velocity over the crest [m/s]
d_k = water depth on the crest [m]
d_o = water level in front of the dike, relative to the crest [m]

As soon as the water level behind the dike rises above crest level, the following submerged weir conditions apply, see Figure 56:

$$\tfrac{2}{3} d_o < d_k < d_a \tag{23}$$

$$u_k = \sqrt{2g\,(d_o - d_k)} \tag{24}$$

where :

d_a = water level behind the dike relative to the crest [m]

In the above there is a discharge coefficient related to the roughness, which approaches 1.0. For rough revetments, for example, rubble mound, this value cannot be used.

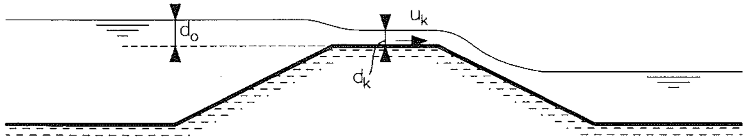


Fig. 55. Free discharge weir.

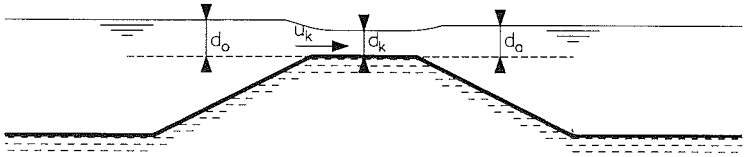


Fig. 56. Submerged weir.

Obviously the current velocity over the crest is important when designing the revetment material for the crest, see Section 8.6.

6.5 **Wave overtopping**

A dike with a low crest can fail under wave overtopping as a result of erosion of the crest and the inner slope. This can be prevented by, for example, using a pitched dike revetment. The stability of the revetment is mainly at risk on the crest where usually the adhesion between the blocks is limited. [VAN KRUININGEN, 1989]. The risk is higher under the attack of a single (high) wave than under longer term overtopping by lower (average) waves.
For calculating average overtopping discharges reference should be made to [Lower Rivers Manual, 1989]. The most damaging forces on the crest are due to the occasional higher waves which propagate over the crest. The size of the wave on the crest governs the stability of the crest pitching. This can be estimated using the run-up on a similar dike with a crest higher than that being designed. This is illustrated in Figure 57.
The wave height which should be used to calculate the loads on the crest revetment is that which is equivalent to the run-up height minus the crest height:

$$H_{crest} = z_{2\%} - h_c \tag{25}$$

where:

H_{crest} = fictitious wave height on the crest [m]
$z_{2\%}$ = ruling wave run up = wave run-up relative to the still water line (SWL) which is exceeded by 2% of the waves, see Section 7.2 [m]
h_c = crest height relative to SWL [m]

It should be noted that this equivalent wave height does not in practice correspond to the depth of water level on the crest during wave action. The equivalent wave height, as defined here, should only be used for estimating the effects of wave action on the stability of the pitching.

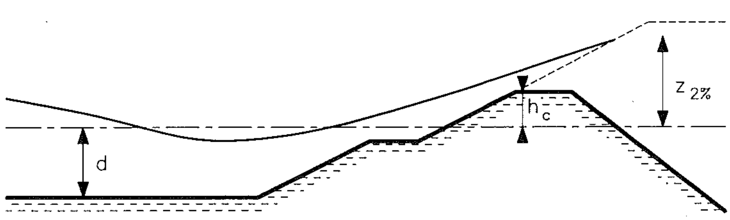


Fig. 57. Wave run-up as a criterion for the effective wave height at the crest.

Figures are included in Section 7.2 for determining the run up height. Section 8.6 should be used for designing the revetment on the crest.

6.6 **Exceptional loads**

Exceptional loads on the revetment should be taken into account in the design in addition to the hydraulic loads. These loads include, for example, see Figures 58 and 59:

- ice loads;
- ship impacts;
- impacts caused by wreckage, floating rubbish, etc.;
- recreational activities and vandalism.

These loads are discussed briefly below. Design equations cannot be developed however because insufficient is known about these loads in relation to pitched dike revetment design. Heavy ice loads, particularly from drifting ice, can seriously damage a dike revetment and toe structure. Ice movements can act on the revetment especially if there "gripping" points in a rough surface, for example, blocks which jut out from the surface or if the slope is steeper than 1:3. Small steep-sided dikes, such as those often found along river banks, can be pushed aside completely by ice.

Fig. 58. Ice loads.

Fig. 59. An example of an exceptional load.

Fresh water ice loads, in particular, form a problem for dikes because fresh water freezes more readily than salt water and also because it is generally tougher (harder) than salt water ice. Despite the considerable damage which can be caused by ice, direct flooding is rarely the result. Flooding is generally associated with high water levels and heavy wave action which can erode and breach the body of the dike after the revetment has been damaged. This rarely occurs in icy conditions because the ice on the water limits or prevents wave action. There is insufficient information available for designing hydraulic structures to resist heavy ice loads. A summary of the available information is given in [Carstens, 1980] and the subsequent reports published by IAHR (Working group on ice forces on structures). The following dike features can however be expected to improve the design:

- gentle slopes;
- smooth surfaces without any projections;
- a berm above the design water level which has a clearly defined edge.

A ship which has lost steerage can ram a dike, a situation which, more than likely, will occur in a storm when wave action is severe and when water levels are high. The ship's propeller when still driving can damage a revetment considerably. The land behind the dike is at considerable risk if a ship damages the revetment, particularly if the damage is worsened by hydraulic loads and breaching occurs. There is also a possibility that canal and river dike revetments will be damaged by ship impact. Dikes can be subdivided into:

1. Summer dikes – relatively low dikes fronted by an unprotected foreshore, and protecting relatively small flood plan areas;
2. River dikes – dikes immediately adjacent to a river or canal;
3. Main dikes – principal dikes, fronted by summer dike and a flood plain and protecting lare areas of land.

In the present context only damage to river dikes is important since summer dikes have no safety function and the possibility of a ship reaching a main dike can be ruled out. At present there are no reliable rules for designing revetments or dikes to resist ship impact or stranding.
Floating rubbish or wreckage is generally too small to seriously damage a revetment. Local damage can be caused to a revetment by recreational activities and vandalism and attention should be paid to blocks being removed, the effects of camp fires and geotextiles being punctured by fishing equipment. The design should prohibit the latter in particular since it is difficult to identify and repair. The removal of blocks and vandalism are important in connection with maintenance strategies but do not affect the flood protection function since, more often than not, repairs can be made before major storms. Loose blocks in areas from which many blocks have been removed can be grouted, however, with asphalt. Areas larger than 20 m^2 should not be injected, since this can reduce the permeability of the cover layer and thus increase the forces on the blocks considerably, see Section 5.6 and 8.2.3. Finally it should be noted that vegetation which has wooden roots (bushes and trees) can cause serious damage and maintenance should be aimed at preventing this developing.

CHAPTER 7

DESIGN AND CONSTRUCTION ASPECTS FOR THE STRUCTURE AS A WHOLE

7.1 Introduction

The design of a pitched dike revetment should be based on the structure as a whole because the shape of the slope and the pitching are important to its ability to function. The following aspects are discussed in this chapter, see Figure 60:

- slope angle;
- berms, if any, and berm level;
- toe structures;
- the slope above the pitching and the related wave run-up;
- the shape of the outer face of the dike – (convex or concave camber);
- the construction of the dike body.

Some of these features affect the stability of the pitching and also wave run-up. They also, therefore, determine the crest elevation needed for the dike.

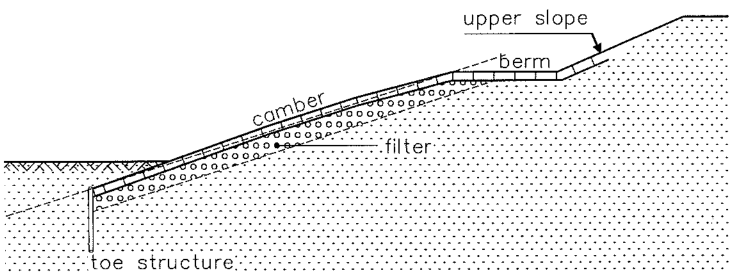


Fig. 60. Cross-section through a dike.

7.2 Wave run-up

The wave run-up on pitched dike revetments ($z_{2\%}$) varies very little from revetment to revetment because the surface is usually smooth, see Figure 61. Projections and holes, etc., have a negligible effect on the run-up [KLEIN BRETELER, 1990] and large roughness elements are an exception. These elements are discussed at the end of the present section.

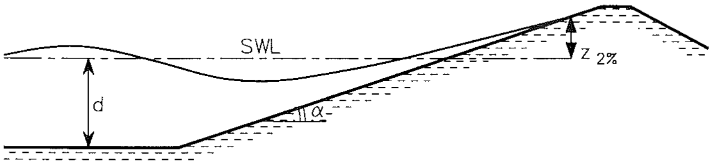


Fig. 61. The wave run-up level, $z_{2\%}$.

The relative run-up, $z_{2\%}/H_s$, can be obtained, for slopes without berms, from Figure 62. The following parameters are used in the figure:

$z_{2\%}$ = wave run-up, relative to SWL, which is exceeded by 2% of the waves [m]. This is the ruling parameter for the crest level for a dike which has no cover layer on the crest or the inner face.

ξ_{op} = the breaker parameter [–] = $\tan a/\sqrt{(H_s/L_{op})}$, see Section 6.2.

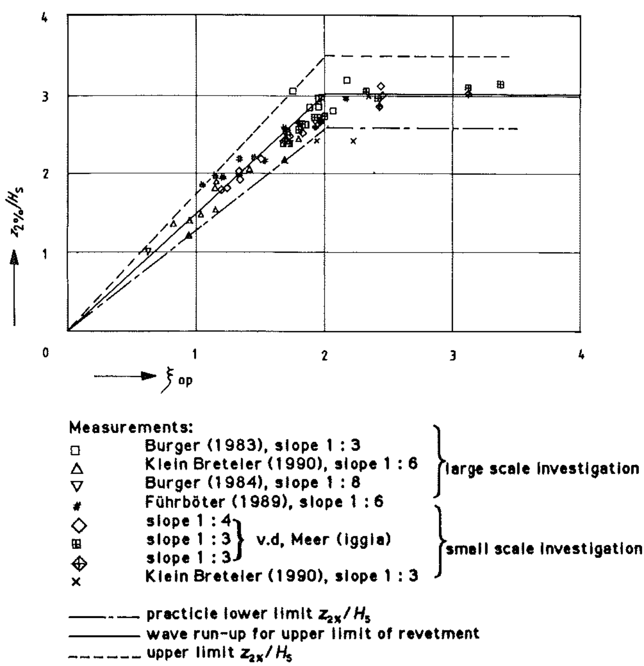


Fig. 62. Wave run-up on a slope without a berm ($\tan\alpha < 0.5$) [Van der Meer, 1991d].

Figure 62 gives the most reliable measurements recorded in small and large scale model investigations. As indicated on the figure, the measurements generally lie between the following limits:

– upper limit:

$$z_{2\%}/H_s = 1.75 \cdot \xi_{op} \text{ for } \xi_{op} < 2.0 \tag{26}$$

$$z_{2\%}/H_s = 3.5 \quad \text{for } \xi_{op} \geq 2.0 \tag{27}$$

– lower limit:

$$z_{2\%}/H_s = 1.3 \cdot \xi_{op} \quad \text{for } \xi_{op} < 2.0 \tag{28}$$

$$z_{2\%}/H_s = 2.6 \quad \text{for } \xi_{op} \geq 2.0 \tag{29}$$

The [Guide to the design of river dikes, 1991] recommends that dike crest levels are calculated using a run-up formula in which the average wave period (T_z) is used instead of T_p. According to the manual: $T_z = T_{1/3}/1.15$. $T_{1/3}$ is the significant wave period which is more or less equivalent to the peak period. Using the relationship between the peak period and the average period ($T_p/T_z \approx 1.15$) the equation in the manual can be written as:

$$z_{2\%} = 1.6\ \xi_{op}\, H_s \text{ (for } \xi_{op} < 2) \tag{30}$$

For $H_s/L_{op} = 0.04$, storms occurring frequently in the Netherlands, and $\tan\alpha < 0.4$ the equation is similar to a relationship used frequently in the past: $z_{2\%} = 8H_s \tan\alpha$. For continuity Equation (30) must be used for the present to determine the dike crest elevation in Dutch dike reinforcement works. A rule is given in Section 7.6 for determining the upper limit of the hard revetment which takes into account wave run-up: the upper limit must lie between $\frac{1}{2}H_S$ and $\frac{1}{2}z_{2\%}$ above the design water level. A run-up equation can therefore be used which is a best fit through the middle of the points plotted in Figure 62, namely:

$$z_{2\%}/H_s = 1.5 \cdot \gamma_b \cdot \gamma_r \cdot \gamma_\beta \cdot \xi_{op} \text{ for } \xi_{op} < 2.0 \text{ (waves breaking on the slope)} \tag{31}$$

$$z_{2\%}/H_s = 3.0 \cdot \gamma_b \cdot \gamma_r \cdot \gamma_\beta \quad \text{for } \xi_{op} \geq 2.0 \text{ (non-breaking waves)} \tag{32}$$

where:

γ_b = berm reduction factor [–]
(for slope without a berm: $\gamma_b = 1$)
γ_r = slope roughness reduction factor [–]
(for pitched blocks $\gamma_r = 1$)
γ_β = wave approach angle reduction factor [–]
(normal wave action: $\gamma_\beta = 1$)

The design slope for run-up is the average slope in the zone between SWL–1.5 · H_s and SWL+1.5 · H_s. Berms on the slope should not be taken into account when determining the average slope, see Figure 63. The procedure is a simplified version of [SAVILLE, 1957]:

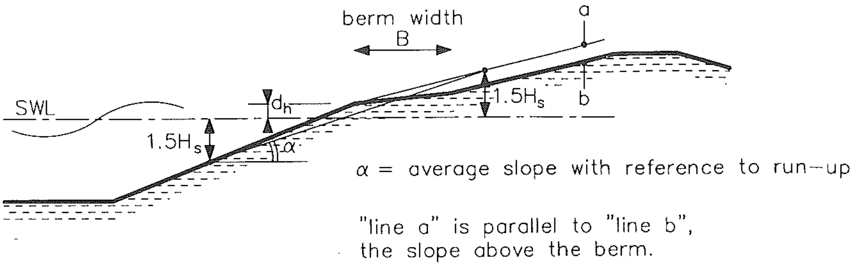


Fig. 63. Definition of average slope α, berm width B and berm depth d_h, ($d_h < 0$).

- draw a line parallel to the slope above the berm which cuts the front edge of the berm;
- mark a point on the slope below the berm which is $1.5 \cdot H_s$ below SWL;
- mark a point on Line a which is $1.5 \cdot H_s$ above SWL;
- join the two points; the slope of this line is the average slope used for to run-up calculations.

The effect of the berm depends on its width (B) and the water depth at the berm (d_h). A reduction factor (γ_b) for the run-up can be read from Figures 64 and 65. If the berm is above the water level $d_h < 0$ but an absolute value of d_h is applied in the figures. In the figures it appears that, for a particular berm width, the reduction in run-up due to the berm is a maximum and therefore nothing can be achieved by widening the berm. γ_b in this case lies between 0.60, lower limit, and 0.65, upper limit, ($\gamma_b = 0.06$ to 0.65). An example of the run-up on a complex slope is worked out below, see Figure 66.

$$H_S = 2 \text{ m}$$

$$T_p = 5 \text{ s} \Rightarrow L_{op} = \frac{g}{2\pi} \cdot T_p^2 = \frac{9.8}{2\pi} \cdot 5^2 = 39 \text{ m}$$

$$\left.\begin{array}{l} \text{below SWL: } \tan \alpha_1 = 0.33 \\ \text{above SWL: } \tan \alpha_2 = 0.25 \end{array}\right\} \Rightarrow \tan a = \frac{0.33 + 0.25}{2} = 0.29$$

$$\Rightarrow \xi_{op} = \frac{0.29}{\sqrt{(2/39)}} = 1.3$$

The run-up without a berm, see Figure 62:

$$z_{2\%}/H_S = 2.0 \Rightarrow z_{2\%} = 2 \cdot 2.0 = 4.0 \text{ m}$$

With a berm, 6 m wide and 1 m below SWL:

$$4.4 \cdot (\tan a)^{2/3} = 4.4 \cdot 0.29^{0.667} = 1.9$$

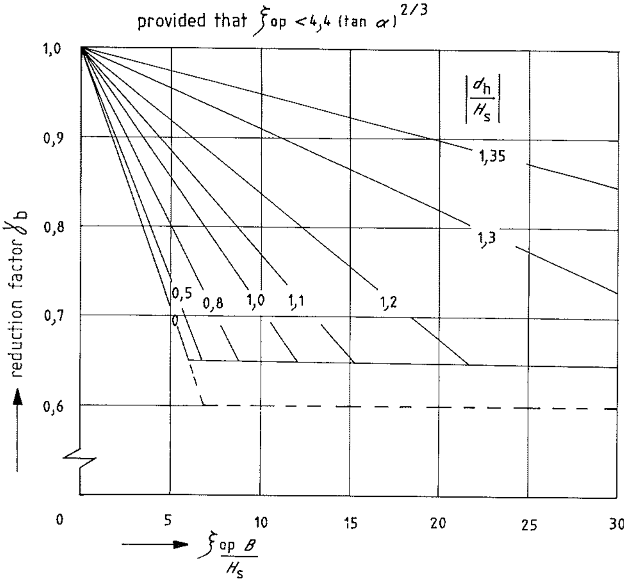


Fig. 64. Reduction factor for the effect of a berm with waves breaking on the slope: $\xi_{op} < 4.4 \cdot (\tan\alpha)^{2/3}$ [VAN DER MEER, 1991c].

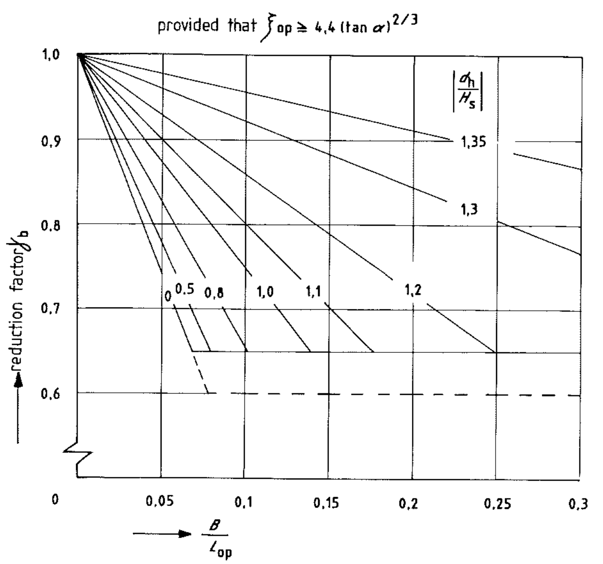


Fig. 65. Reduction factor for the effect of a berm with non-breaking waves on the slope: $\xi_{op} > 4.4 \cdot (\tan\alpha)^{2/3}$ [VAN DER MEER, 1991c).

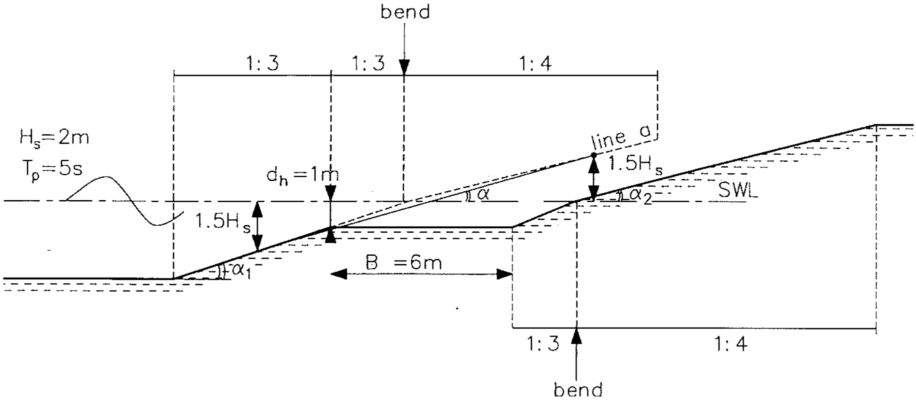


Fig. 66. Wave run-up on a complex slope.

This implies that $\xi_{op} < 4.4 \cdot (\tan\alpha)^{2/3}$ and that Figure 64 should be used.

$$\left.\begin{aligned} \xi_{op}B/H_S &= 1.3 \cdot 6/2 = 3.9 \\ d_h/H_S &= 1/2 = 0.5 \end{aligned}\right\} \Rightarrow \text{using Figure 64: } \gamma_b = 0.78$$

Run-up with a berm:

$$z_{2\%} = 0.78 \cdot 4.0 = 3.1 \text{ m}$$

Berm slopes less than 1:10 have no effect on run-up. The level at the front edge of the berm determines the berm depth, d_h, see Figure 67. If the berm is steeper than 1:10, it should be seen as the upper slope of dike face comprising two slopes (no berm). The average slope, see Figure 63, should then be used.

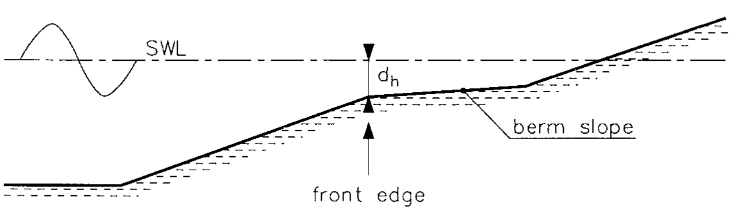


Fig. 67. Front edge of berm and berm slope.

The effect of the wave approach angle (β, where $\beta = 0$ for perpendicular waves, that is, parallel to the dike) was investigated for short wave crests by [Van der Meer and De Waal, 1990]. With this type of wave there is some spread in propagation angle and therefore the waves do not all approach from the same direction. A better picture of the waves is obtained in practice by considering the waves as coming from a single direction. In contrast to the findings of earlier investigations with long wave crests, for example the [Lower Rivers Manual, 1989], it appears that the effect of angle of approach is fairly small:

- if $\beta \leq 30°$, then:

$$\gamma_\beta = 1.0 \tag{33}$$

- if $\beta > 30°$, then:

$$\gamma_\beta = 1.12 - 0.004 \cdot \beta \tag{34}$$

where:

β = angle of approach of waves [°]

Waves with an angle of approach of 80° produce a reduction factor of $\gamma_\beta = 0.8$. The wave run-up is then 20% less than that for perpendicular waves.
The effect of a high foreshore on which waves break in front of the dike is currently being investigated [Van der Meer, 1991b]. Initial results show that run-up is only affected to a limited extent by the breaking process (about 0 to 10%). This means that run-up can be calculated using the figures and formulas given in the present chapter provided that the wave height and period at the toe of the structure are used as inputs.
As stated at the beginning of this section the run-up on block pitching can be assumed to be approximately the same as on a smooth surface. Recent measurements show that large roughness elements do, however, affect run-up [Van der Meer 1991a]. The measurements were made on a smooth slope to which cubes were attached, extending from the toe to the crest in a pattern.
The length of the cubes was about 10 to 20 % of the (significant) wave height. The centre-to-centre distance between cubes in one series of tests was about one wave height, in another series about half a wave height. The cubes occupied 4 and 11% of the surface area of the slope, respectively. Provisional results from both series of tests indicate that the run-up is less than that on a smooth slope by a factor $\gamma_r \approx 0.6$ to 0.75. Such large roughness elements can however affect the stability of the cover layer. This aspect is not discussed in the present manual.
In accordance with Figure 54, Figures 62, 64 and 65 can be used to calculate run-up with secondary ship waves by using H_i for H_s and T_i for T_p. In these figures the run-up caused by primary ship waves (bow and stern waves) does not extend above SWL.
The most recent run-up measurements will be presented in a separate TAW publication about wave run up on and overtopping of dikes, which will be published in 1995.

7.3 Slope angle

The selection of slope angle is largely based on that used for existing structures. In protected areas, for example in estuaries, a steeper slope can be used, say, 1:3 to 1:4. In exposed areas the slope is generally in the range 1:4 to 1:6. On new dikes, for example river bend cut-offs or reinforcements on a broad foreland, the choice of slope is less restricted. Along rivers and canals the slope is often 1:3 to 1:4.
The angle of the front slope has a large effect on structural stability. The stability of the cover layer (the lifting or pushing off of the revetment), the interface between a granular filter and the subsoil and the slope as a whole (loads on the sand body, see Chapter 9) are all affected. A more gentle slope can usually have a thinner cover layer and sometimes a thinner filter than a steep slope.
A gentle slope, however, has the disadvantage that the revetment covers a larger area of the slope, even though the crest level may be lower because of the smaller run-up, see Figure 68.

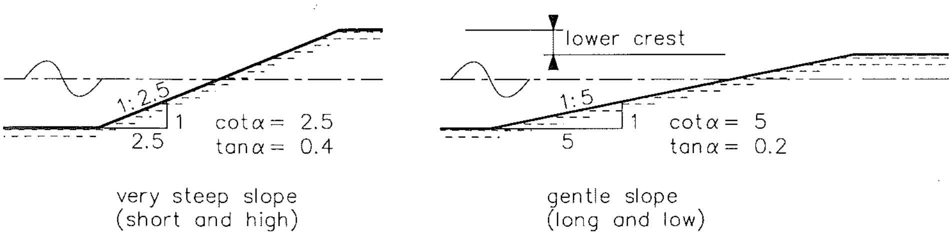
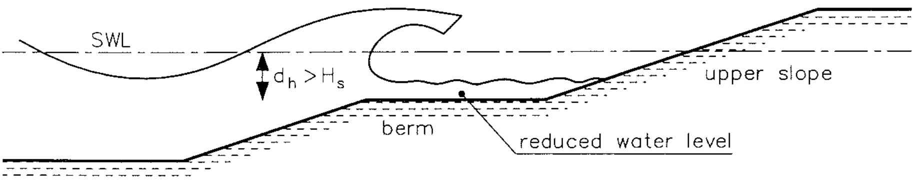


Fig. 68. Comparison between steep and gentle slopes (see also Appendix B).

An optimization procedure, which includes, for example construction and maintenance costs, can be used to indicate which slope angle should be applied for particular conditions. This aspect is also discussed in Chapter 11. A steep lower slope with a gentle slope above a berm can, for example, be considered instead of an expensive revetment. If the slope is less than 1:3 the blocks can be laid more smoothly and the work is easier. With steeper slopes the productivity falls off rapidly. Blocks can also be laid mechanically on steep slopes with few problems. Steep slopes, however, are not recommended since repair work must almost always be done by hand.

7.4 Berms

Berms on the outer face should always be constructed at about the storm flood design level. Their effect on run-up is then the maximum, see Section 7.2. Generally the berm height can vary by half the wave height from the storm flood level and still produce the optimum effect, see Figures 64 and 65. In the past, however, many dikes have been designed with berms below storm flood level. As a result, berms on older dikes are often too low to reduce the run-up to any great extent. The wave action on the slope above the berm is much less with the berm, provided that the berm depth (d_h) is less than H_s, see Figure 69 and an obvious reduction in run-up can be expected ($\gamma_b < 0.85$, see Figures 64 and 65). The slope above the berm can then be revetted with open blocks through which vegetation can grow or, if wave action is expected to be small, turf can be laid on clay. This is the subject of the study by [De Waal and Veltman, 1991].



the wave breaks on the berm and not on the upper slope

Fig. 69. Berm constructed below SWL.

In addition to breaking up the loads on the structure and reducing run-up a berm can also serve the important function of a roadway for maintenance work on the outer slope. For this function the berm should be at least 2.5 to 3 m wide and be paved so that it can bear vehicles, carrying for example, maintenance materials, bricks and clay. Paving is essential rather than a grass surface which can be destroyed by vehicles in wet conditions. Berms on the outer slope are usually constructed 2.5 to 10 m wide with the front edge at the design storm flood level. The slope of the berm is generally between 1:10 and 1:20. Outer slope berms to reduce wave forces and/or run-up are unnecessary if only limited wave action is expected.

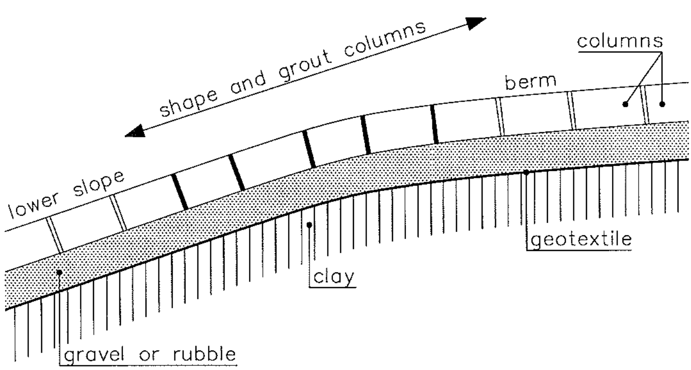


Fig. 70. The front edge of the berm (shaped and grouted).

Special consideration should be given to the edge of the berm, particularly if it is to be in a zone of heavy loads at a level between the design storm level and $2 \cdot H_s$ below, see Figure 73. Adhesion between blocks will be minimal here, since blocks on the berm itself will be almost horizontal and will only be able to bear a limited load normal to the slope. It is recommended that blocks at the edge are shaped to the slope and measures taken to strengthen this area, for example, using locally applied asphalt grouting or heavier blocks, see Figure 70.

7.5 Toe structures

The most important functions of the toe are:
- to support the revetment, and
- to prevent the revetment being undermined

Further details are considered in Chapter 10. The level of the toe structure for dikes constructed on a high foreshore which is above low water should be 25 to 50 cm below the level of the foreshore, see Figure 71. If the foreshore is below normal low water level a supporting berm should be applied. An example of a supporting berm is shown in Figure 72. The toe should preferably be constructed above low water to facilitate the construction work. Details of toe construction are given in Chapter 10.

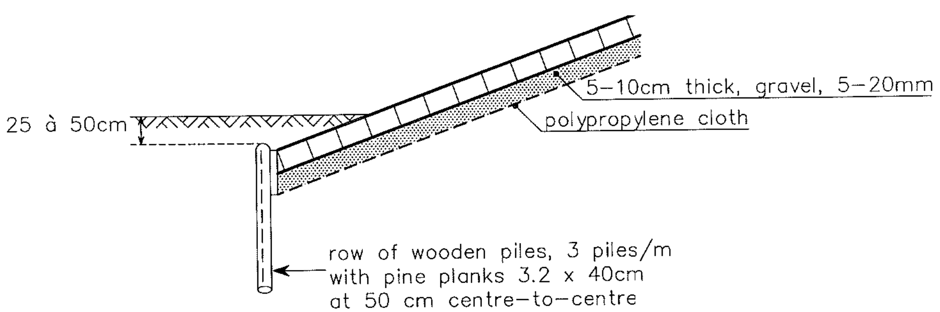


Fig. 71. An example of a toe structure with a high foreshore.

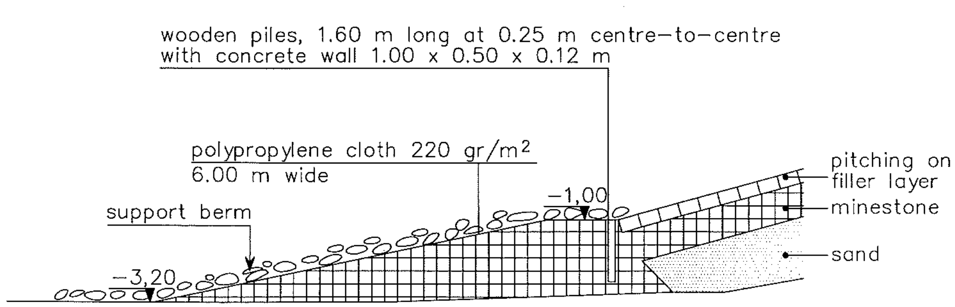


Fig. 72. Example of a berm (foreshore below normal low water level).

7.6 Upper edge of the revetment

The upper edge of the pitching is generally the weakest point of the revetment. There is relatively little friction between the blocks here since there is no fixing force from a row of blocks above. The upper edge and in fact any other transition structure must not be constructed in areas where wave forces will be high, that is between the design water level (SWL) and between SWL–H_s to SWL–$2H_s$, see Figure 73.

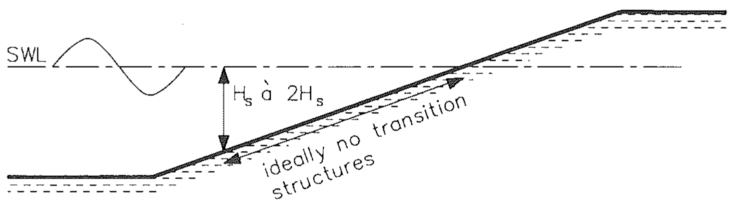


Fig. 73. Most heavily attacked section of the slope.

Fig. 74. Erosion of grass above the revetment.

If the revetment ends at the top with a grass slope, this should be sufficiently high above the normal high water level that the grass is only covered with salt water once or twice a year. The transition should also be sufficiently high that there is little or no erosion of grass at the edge of the pitching, see Figure 74. The design of the boundary is currently being investigated. Provisionally, pitched revetments should extend up to a level between SWL + $\frac{1}{2}H_S$ and SWL + $\frac{1}{2}z_{2\%}$ see Figure 75. The slope above the pitched revetment should be clad with several rows of open blocks, through which vegetation can grow, or heavy bricks set in revetment clay. Turf should only be considered for the upper slope in exceptional cases, for example, when a berm is used at a little below the design level or in areas of relatively light wave action, see Section 7.4.

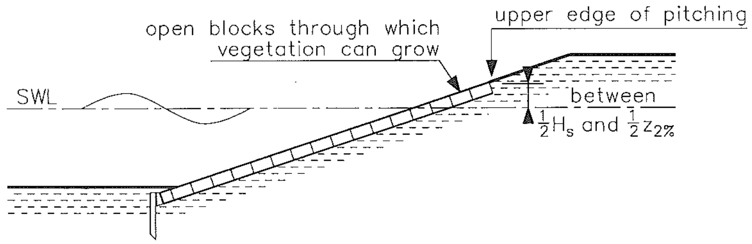


Fig. 75. The upper edge of the hard revetment, its height being at least between SWL + $\frac{1}{2}H_S$ and SWL + $\frac{1}{2}z_{2\%}$.

7.7 The shape of the slope below a berm

The slope below a berm can, in principle, be straight or have a camber, (convex, see Figure 76, or concave, see Figure 77). Soil slopes are generally laid convex.
If the slope is convex, the maximum point should be $^1/_{50}$ to $^1/_{100}$ of the slope length above the straight slope level, and should be at $^1/_3$ of the slope length down the slope, below the front edge of the berm, see Figure 76.

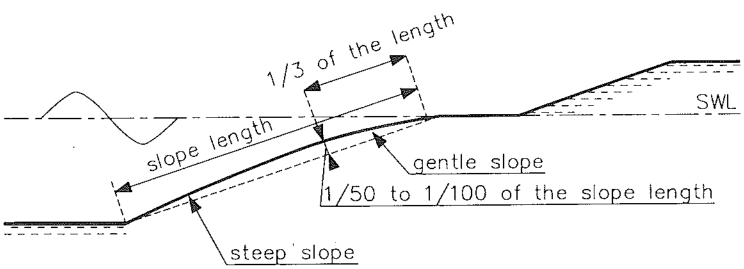


Fig. 76. A convex camber with a gentle slope immediately below SWL.

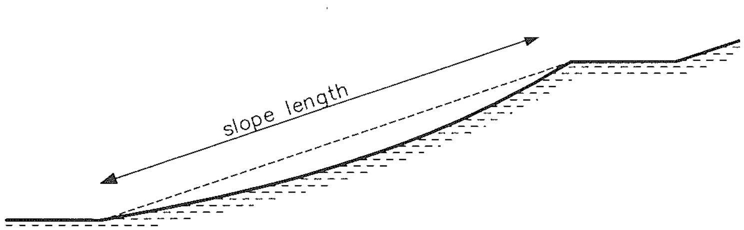


Fig. 77. Slope with a concave camber.

Both forms have advantages and disadvantages, the most important of which are listed below.
The advantages of a convex slope are:

- smaller run-up, because the more gentle slope is at the top; this is more important in terms of run-up than a more gentle slope at the bottom;
- the gentler slope is easier to construct at the berm than a steeper slope;
- blocks tend to jam together giving a stronger revetment;
- blocks are more easily reset;
- the revetment is visually more attractive.

The disadvantages of a convex slope are:

- it is less flexible because of the jamming effect and, if undermining occurs, arching can develop;
- joints are easily washed out and there is a greater possibility of loose blocks occurring.

From this it can be concluded that, despite the disadvantages, a convex revetment surface is preferable.

7.8 **Construction aspects**

7.8.1 *General*

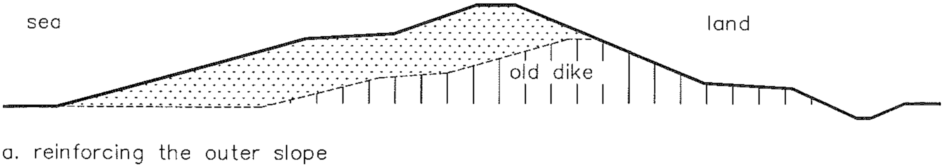
Dike construction works can be subdivided into:

- new works: a completely new dike, for example, when cutting off a bend in a river;
- reinforcement of the outer slope (water side);
- reinforcement of the inner slope (land side).

The whole outer slope, including berms, must be considered when designing a new dike or reinforcing an existing dike. When reinforcing the inner slope generally only the section up to an existing berm or the crest section needs to be considered, see Figure 78.
Dike reinforcing begins with placing the essential clay. This can involve:

- excavation of clay from sections of the old dike;
- when reinforcing the outer slope, excavation and ripening of clay from the mud flats in front of the old dike and from the foundations of the new dike;
- any excavation required in the area behind the dike;
- supply of clay to site, if necessary.

Retaining dikes are then constructed with the clay, see Figure 79. If there is deep water in front of the dike the retaining dikes can be constructed using minestone. If the foreshore is high, for example at mean tide level, sand can be pumped into place and bulldozed into retaining dikes.



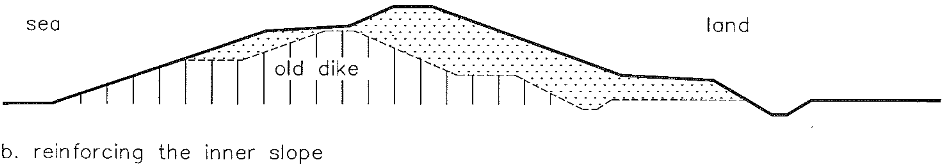


Fig. 78. Reinforcing the outer and inner slopes of a dike.

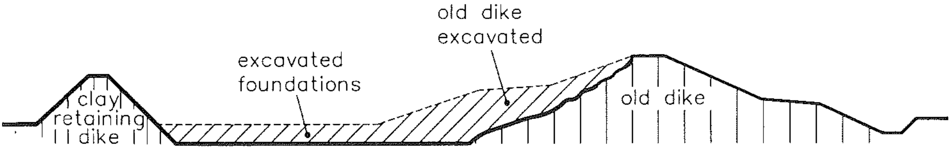


Fig. 79. Retaining dike constructed from clay excavated out of the foundations of the main dike.

Usually sand for hydraulic fill is obtained locally. The sand can be dredged and pumped directly into barges by suction dredgers. The barges then transport the sand to the coast from where it is pumped to the site along pressure lines. If barges, for example in shallow waters, cannot reach to the site, long pressure lines have to be used. For direct sand winning, without barges, pumping has to be carefully controlled because of the danger of silting (if the suction/pumping period is too long the silt fraction may separate out). Sometimes the sand is first pumped into large storage areas prior to use. Depending on the stability of the sand and the available water pressure the sand is laid in one, two, or sometimes three layers, see Figure 80. The rate of settling is checked by using settlement plates; the water pressure is checked by using water-pressure meters.

The second layer of material can be deposited between retaining dikes constructed from the sand of the first layer, etc. These temporary dikes are protected with plastic sheets which must be removed after the layer has been pumped into place, since they could form slip surfaces within the dike. After being deposited the hydraulic fill can be profiled and compacted using bulldozers. The revetment clay is then laid in two layers and also compacted using bulldozers. The total thickness of clay must be at least 80 cm on the slopes and 60 cm on inner slopes. The best quality clay, see Section 5.8, should be used for the outer slope.

Because clay on the outer slopes, is liable to be flushed out it should only be used directly under blocks above HWST (High Water Spring Tide). Clay can be used in the tidal zone if it is applied under a filter layer, with a geotextile between the clay and the filter, see Figure 155. If blocks have to be placed directly on clay, the subsurface must first be prepared and compacted. A thin layer (1 to 2 cm) of moist clay can then be laid to provide a smooth subsurface for the blocks. This clay should be laid along the dike and can be sprayed if necessary. Blocks are usually placed mechanically using a block claw, see Figure 82. If necessary the blocks can be tamped down. If minestone is used for the retaining dike, it can be re-used, after the sand has been pumped into place, as a filter layer, 0.5 to 1 m thick, see Figure 72.

Fig. 80. Hydraulic filling with sand.

In order to obtain a smooth surface for the pitching a thin filler layer of gravel or rubble is laid on the minestone. This layer should be as thin as possible, because the thickness can have a detrimental effect on the stability of the revetment, see Chapter 8. A layer thickness of 5 cm is generally sufficient.
When strengthening the inner face of a dike the old revetment, assuming it is still satisfactory, can remain in place unless its profile differs from that required. In order to obtain a good connection it is generally necessary to take up a 2 m wide strip of the old revetment and re-lay it under the new profile, see Figure 81.

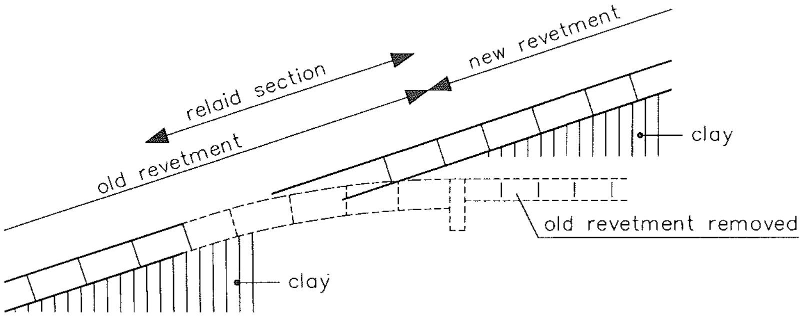


Fig. 81. Transition between old and new revetments when reinforcing the inner face of a dike.

Construction of the transition structure is discussed further in Chapter 10. Figures 82 to 84 show blocks being placed.


Fig. 82 Lifting a row of blocks with a block claw.

Fig. 83. Placing blocks with a block claw.

Fig. 84. Blocks being tamped-in.

7.8.2 *Block mattresses*

Block mattress revetments are constructed in three phases:

- construction of the dike body and any revetment sublayers (clay and/or filter layers);
- laying the mattress;
- anchoring the mattress and applying the joint filler.

The construction of the dike body is described in Section 7.8.1 above. A well-compacted slope is important in order to produce a smooth surface and thus ensure that there is a good connection between the mattress and the subsurface. When laying mattresses on banks it is strongly recommended that they are laid on undisturbed ground and that areas excavated too deeply are carefully refilled.
Before using a geotextile the slope must be carefully inspected for any projections which could puncture the material. Care must be taken when laying a mattress on a geotextile to ensure that extra pressures are not applied and that the geotextile is not pushed out of place. Geotextile sheets must be stitched together with an overlap of at least 0.5 to 1.0 m to prevent subsoil being flushed out. This is particularly important if the mattress is laid directly on sand or clay.
Block mattresses are laid using a crane and a balancing beam. The mattress must be in the correct position before it is uncoupled because it is difficult to pick up again and also time-consuming. Provided that part of the mattress can be laid above the water line, it can be generally laid very precisely and joints between adjacent mattreses can be limited to 1 to 2 cm. Laying a mattress completely under water is much more difficult. The spacing between the blocks of adjacent mattresses nonetheless should never be more than 3 cm.
Once in place, mattresses should be joined so that the edges and corners cannot bang together under the action of waves, see Figure 85. Loose corners are particularly vulnerable. In addition the top and bottom edges of the revetment should be anchored, as shown in Figures 86 and 154. A toe structure is not needed to stop mattresses sliding.

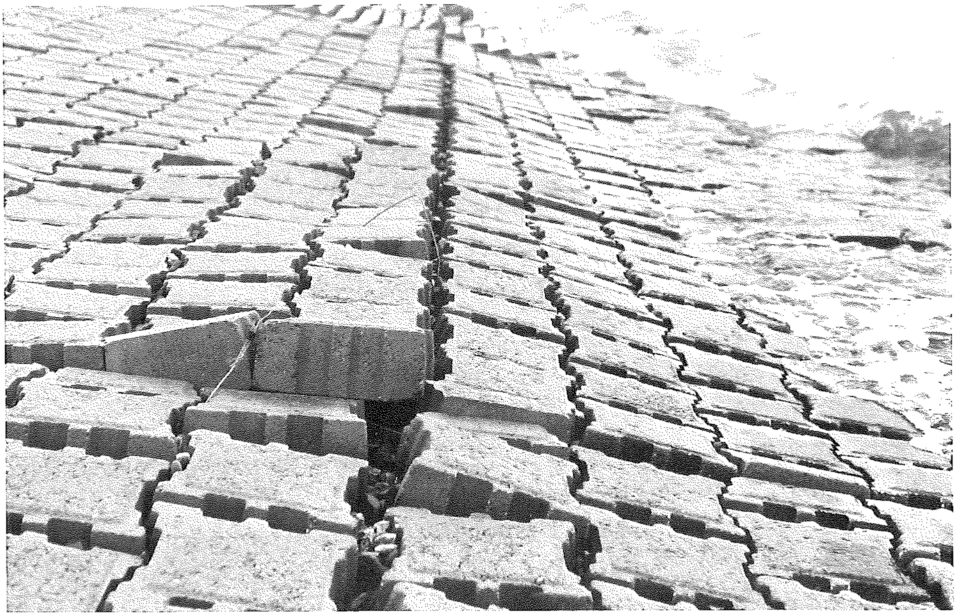


Fig. 85. The turned-up corner of a mattress – test section near Lelystad.

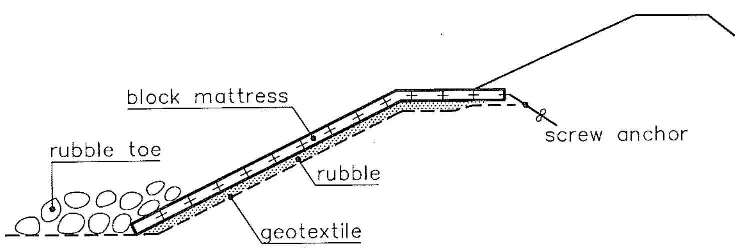


Fig. 86. Block mattress toe construction and anchorage.

Chapter 8

REVETMENT STABILITY

8.1 Introduction

The stability of the revetment (pitching and sublayers) depends on whether or not failure mechanisms can develop when it is exposed to the hydraulic design conditions. To evaluate the stability, information is therefore required about the hydraulic design conditions, the structural properties and the possible failure mechanisms. This chapter describes, qualitively, the physical background, Section 8.2, and the design criteria formulated, Section 8.3. Sections 8.4, 8.5 and 8.6 describe methods for predicting whether or not failure will occur under given hydraulic conditions.
The chapter is concluded with Section 8.7 with a discussion on toe and anchor structures, block structures laid on clay and the effect of sand and silt being carried from the foreshore into the structure.
It is stressed that answers are only given to questions about the stability of a particular structure under given loads and that none of the design methods presented leads to the optimum economic structure. Points to consider when creating the optimum design are examined in Chapter 11.

8.2 Description of the physical processes

8.2.1 *Failure mechanisms*

Depending on the composition of the structure and the type of loads to which it will be subjected one of the following failure mechanisms can be the one which governs the design. This failure mechanism will be the one which develops at a load lower than the other mechanisms [Bezuijen et al., 1990, page 35]. The failure mechanism can be:

- loose blocks being lifted out of the slope;
- the filter being sanded up by material out of the base (causing the cover layer to settle);
- the cover layer sliding down the slope because of poor toe or anchor structures;
- geotechnical instability.

As discussed in Chapter 3, failure can be caused:

- On the slope, by:
  - wind waves (sea and lake dikes), and
  - ship waves (banks, groynes and river dikes).
- On the crest, by:
  - wave action, and
  - overtopping.

Lifting out of blocks

The stability of the cover layer is threatened by uplift related to pressure head differences. These differences occur, for example, at the moment of maximum wave run-down as shown in Figure 87.

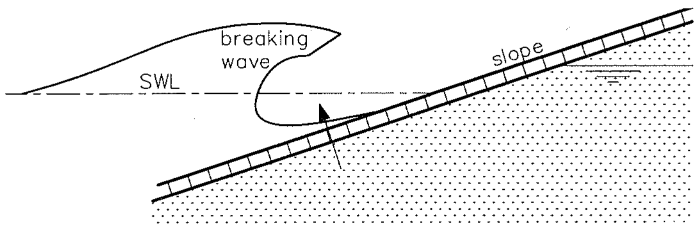


Fig. 87. Uplift at maximum wave run-down.

At this moment part of the slope (below SWL too) almost dries out, even though there is water within the structure, see Figure 88. This causes uplift forces on the underside of the cover layer. This mechanism can be produced by both wind and ship waves in both slope and crest revetments, in the latter when it is overtopped. Designs for overcoming this failure mechanism are discussed in Section 8.4 and 8.5, (slopes), and 8.6 (crests). Flow along a dike rarely governs the design for the areas considered in this manual, a cover layer 10 cm thick being able to resist currents of up to 2 m/s, see Appendix E.

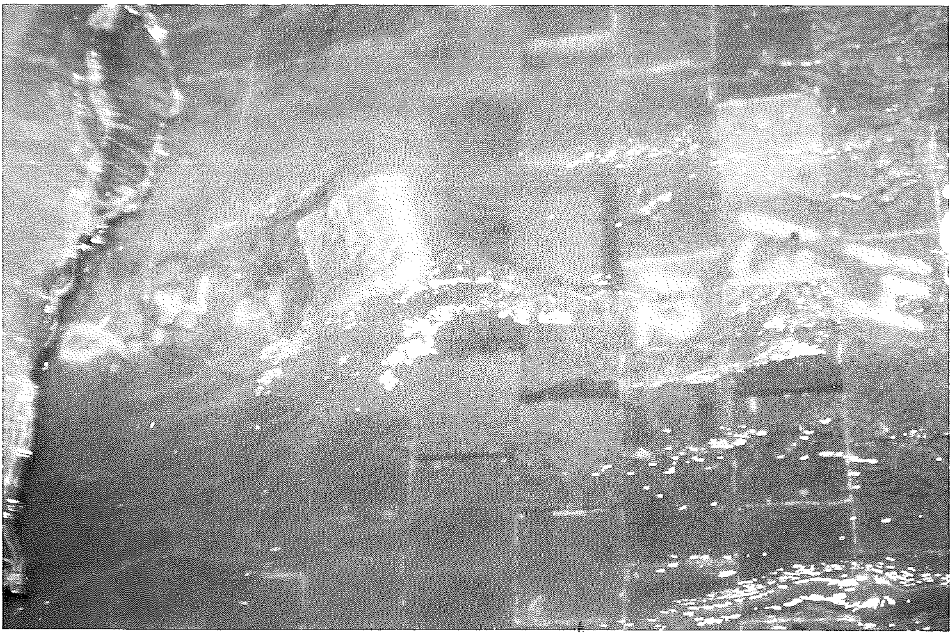


Fig. 88. A cover layer on the point of failure.

As mentioned in Chapter 2, only relatively smooth pitching is considered in this manual. The hydraulic impact forces on projections, or projecting edges are not considered. In this connection revetments with small irregularities can be considered, provisionally, to be relatively smooth, that is, the product of the breadth and the height of the irregularlity is less than about 20 % of the product of block width and thickness. Flow over a structure, such as a groyne in a river, can produce relatively high current velocities which can lead to blocks being lifted out. Designs to prevent this happening are described in Section 8.6.1.

The movement of sand into a filter

A geotextile can be applied between a granular filter layer and the sand core of a dike or bank; a geotextile is also recommended when the subsoil is clay, see Figure 89. At the interface between the filter and the base there is a danger that flow through the filter (caused by the movement of water in the cover layer) can carry grains from the base into the filter, see Figure 90. This can cause the cover layer to settle locally which can lead to a loss in cover layer adhesion.

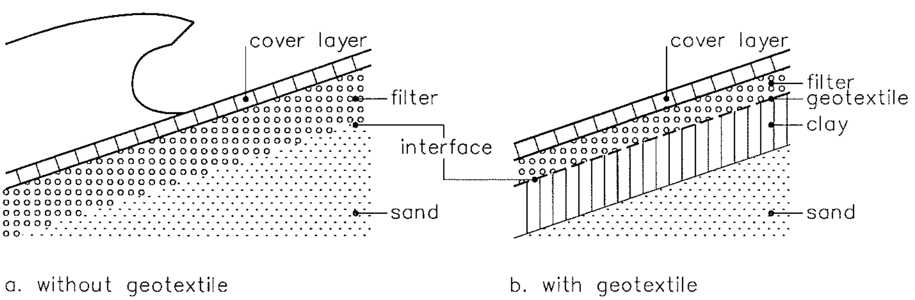


Fig. 89. Composition of a revetment and the position of the interface between the filter and the base.

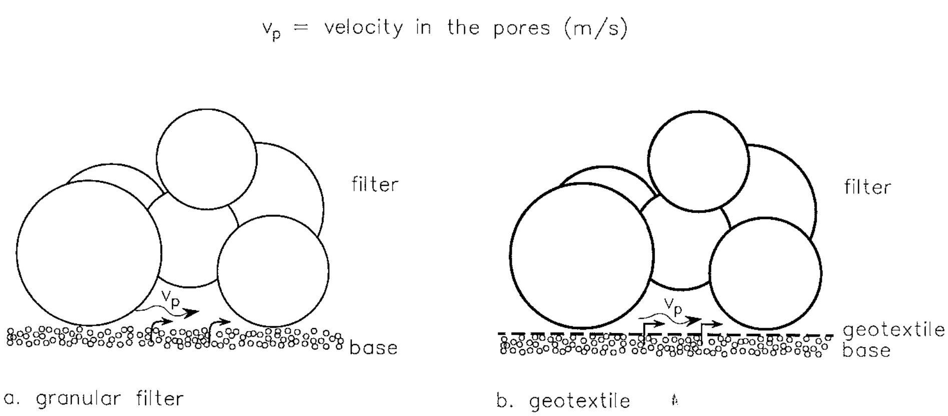


Fig. 90. Movement of sand into the filter from the base.

In principle sand can move into the filter as a result of all of the loads described below. The design of the filter and/or the geotextile are discussed in Section 8.4.3. The movement of sand from the foreshore into the cover layer and the filter is a different process. In this case the cover layer is not expected to settle, see Section 8.7.3.

Cover layer sliding

Generally the friction between the cover layer and the sublayers prevents the cover layer from sliding. Under certain conditions toe structures or, in the case of block mattresses, anchors are needed to stop cover layers sliding. Sliding can occur if a geotextile is placed between the cover layer and the sublayer, because the friction will be relatively small or if the slope is very steep. Cover layers can also slide if the friction forces are reduced temporarily by upward pressure forces. This aspect is discussed in Section 8.7.1.

Geotechnical instability

Two failure mechanisms can be involved in geotechnical instability:

- sliding due to a slip surface in the base (sand);
- weakening of the base material.

The first mechanism can develop in wave action during wave run-down. A slip circle can also form as the water flows out of the body of the dike, see Figure 91.

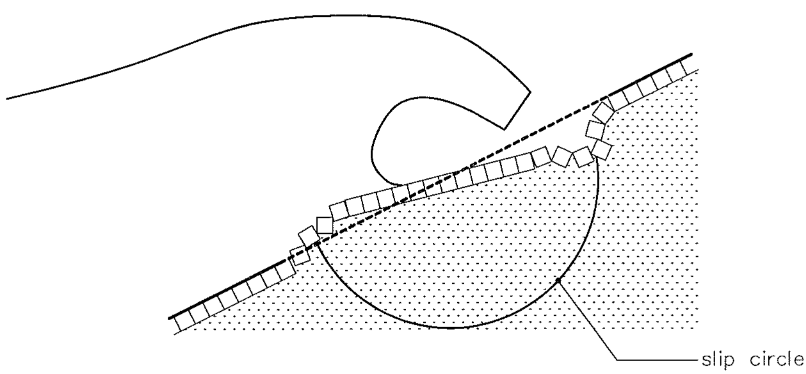


Fig. 91. The distortion of an S-profile, shown schematically.

The dike core can weaken if it has not been satisfactorily compacted and further compaction can occur under wave impact. Because this will tend to occur when the core is saturated, very high excess water pressures will develop which weaken the core. As a result, the cohesion (the internal friction between grains) is so low that the sand slides downwards and/or flows. These forms of geotechnical instability only occur under wave action; they are discussed in Chapter 9 and are not considered further in the present chapter.

8.2.2 *Loading and strength*

In general a structure fails when the loads exceed its strength [BEZUIJEN et al., 1990, pages 41 and 42]. A distinction must be made here between waves and currents. In the first instance, if only wind waves are considered the loads on the pitched dike revetment and its strength depend on whether or not the structure is considered as a whole or if individual failure mechanisms are considered.

The structure as a whole

If the structure is considered as a whole the loads can be characterized by the wave height and period taking into account the water level (depth) and the slope angle:

H_s = significant wave height at the toe of the structure [m]
T_p = wave period at the top of the spectrum [s]
d = water depth at the toe of the structure [m]
α = slope angle (relative to horizontal) [°]

In Section 6.2.2 which describes how a wave breaks on the slope, it is shown how the breaker parameter can be defined using these parameters, see Figure 50:

$$\xi_{op} = \frac{\tan a}{\sqrt{H_S / L_{op}}} \tag{35}$$

$$L_{op} = \frac{g T_p^2}{2\pi} \tag{36}$$

where

ξ_{op} = breaker parameter [–]
L_{op} = wave length in deep water [–]
g = acceleration due to gravity [≈ 9.8 m/s²]

The strength of the structure is characterized by the wave height at which it fails. This is the critical (significant) wave height, H_{scr}.
The size of H_{scr} depends on both the geometry of the structure and also the value of ξ_{op}.

Individual failure mechanisms

When considering individual failure mechanisms a decision has to be made on the particular load or strength parameter to which to relate the failure. The pressure head difference on the pitching is used when considering the *lifting out of a block or column* [BEZUIJEN et al., 1990, page 99]. In this connection the "pressure" is not considered because the "pressure head difference" can be more easily related to the flow of water. The relationship between the pressure and the pressure head is as follows [BEZUIJEN et al., 1990, page 62]:

$$\phi = \frac{p}{\rho g} + z \tag{37}$$

where:

ϕ = pressure head, see Figure 92 [m]
p = pressure, see Figure 93 [N/m^2]
ρ = volumetric mass of water (fresh water:1000; salt water:1025) [kg/m^3]
g = acceleration due to gravity [9.8 m/s^2]
z = local height relative to datum (for example, SWL) [m]

In still water the water pressure depends on the depth. This is not the case when considering the pressure head. In still water the pressure head in the water is generally equivalent to the height of the water surface above the reference level.
The pressure head under a breaking wave on a slope is shown schematically in Figure 92 and should be contrasted with the pressure on the slope shown in Figure 93.

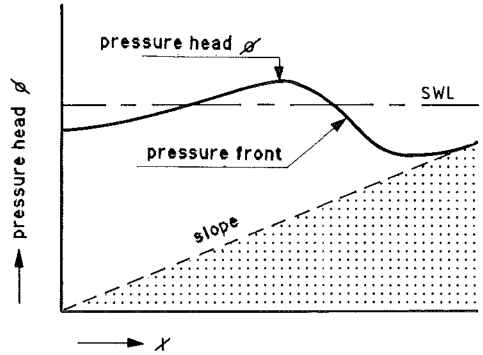


Fig. 92. Variation in pressure head on a slope.

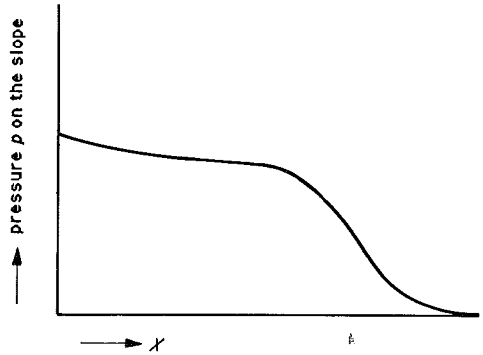


Fig. 93. Variation in pressure on a slope.

The strength of a cover layer is characterised by the difference in pressure head at which it begins to fail. The pressure head difference depends to a large extent on the thickness and volumetric mass of the cover layer.

The penetration of sand into the filter layer (from the base) is also a failure mechanism. This is a direct result of a hydraulic gradient in the filter which therefore can be considered as a load [BEZUIJEN et al., 1990, page 128]. The hydraulic gradient can be seen as the water pressure head gradient (= the change in pressure head per unit length), see Figure 94:

$$i = \frac{\partial \phi}{\partial a} = \frac{\text{fall}}{\text{distance}} \tag{38}$$

where

i = hydraulic gradient in the filter, parallel [–] to the cover layer
a = local co-ordinate, parallel to the cover layer [m]

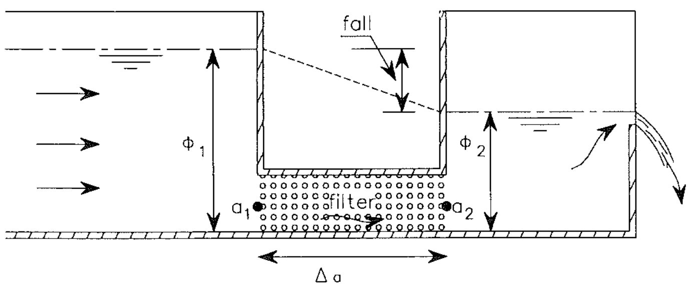


Fig. 94. Hydraulic gradient in the filter $\left(i = \frac{\phi_1 - \phi_2}{\Delta a} = \frac{\text{fall}}{\text{distance}}\right)$.

There is a close relationship between the hydraulic gradient and the water flow through the pores of the filter. The relation between the hydraulic gradient and the specific flow rate (filter velocity) through the filter is approached here with a linear relationship [BEZUIJEN et. al., 1990, page 81].

The permeability (the reciproque of the flow resistance) equals the relationship between the hydraulic gradient and the filter velocity:

$$k = q/i$$

where

k = filter permeability [m/s]
q = specific discharge (filter velocity) through the filter [m/s]

The strength of the interface is characterised by the hydraulic gradient at which grains penetrate the filter. This is referred to as the critical hydraulic gradient, i_{cr}, and

acts parallel to the cover layer. From the calculation procedure described in Section 8.4.3, it appears that i_{cr} is larger for upward flow in the filter than for downward flow at the interface, see Figure 95. A distinction has therefore to be made between:
- the critical hydraulic gradient for upward flow, $i_{cr\uparrow}$, and
- the critical hydraulic gradient for downward flow, $i_{cr\downarrow}$.

The loads are also different:

- loads due to upward flow along the interface, $i_{\uparrow}$, and
- loads due to downward flow along the interface, $i_{\downarrow}$.

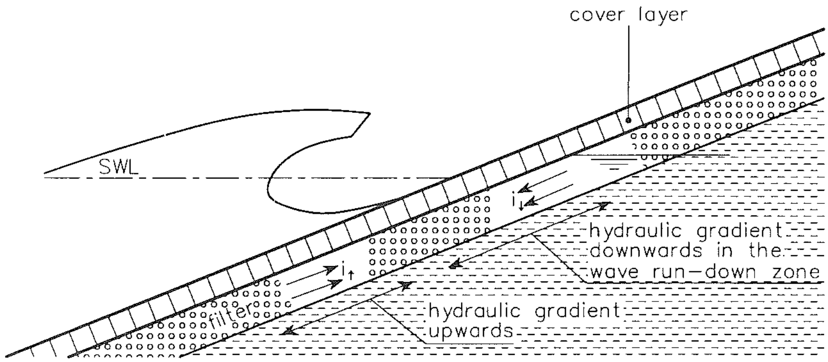


Fig. 95. Hydraulic gradient along the interface (upward/downward).

8.2.3 *Leakage length*

As discussed in Section 8.2.1, the stability of the cover layer is threatened by the difference in pressure head and blocks can be lifted out of the slope. The difference in pressure head is easy to calculate for an impermeable cover layer and wave run-down since this can be schematized as a temporary lowering of the water level, see Figure 96. In this case the maximum pressure head difference is the fall in water level, (the difference between the level of the phreatic line and the external water level, in metres of water column).

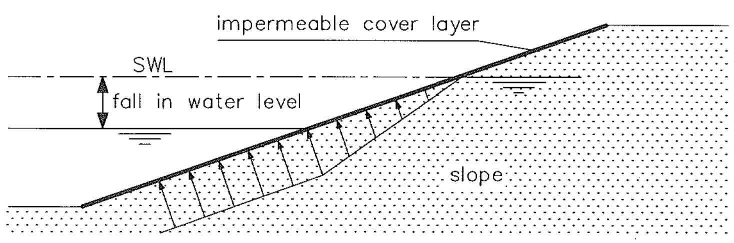


Fig. 96. Difference in pressure head due to a fall in water level under an impermeable cover layer (phreatic level in the structure assumed to be SWL).

For breaking waves and a permeable cover layer the calculations are considerably more complicated. The maximum run-down situation is shown in Figure 97. The figure shows how the water from the incoming wave passes through the cover layer and then up through the filter (Area A). The water flows out where the pressure head on the slope is low. This flow causes a difference in pressure head on the cover layer. At the same time water is flowing downwards and outwards through Area B. This flow also creates a difference in pressure head on the cover layer. This flow pattern, which lasts for about half a second per wave (about 1/10 to 1/20 of the wave period), is responsible for lifting (loose) blocks. Wave impact can, in a short period of time, damage a cover layer lying on a granular filter. The smaller the flow, the smaller the pressure head difference. The flow is limited if the thickness of the filter layer is small or if the filter permeability is low. In this situation, if the permeability of the filter and the cover layer is small, it is difficult for water flowing through the filter to escape "unhindered" through the cover layer, that is, without a large difference in pressure head.

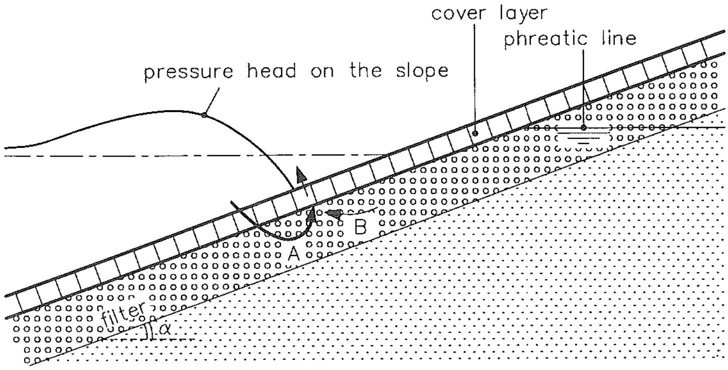


Fig. 97. Flow through the cover layer and out through the filter during maximum run-down.

This permeability relationship is also found in the most important structural parameter, the leakage length. This parameter is a measure of the pressure head difference on the cover layer for given wave forces [BEZUIJEN, et al., 1990, page 71]:

$$\Lambda = \sqrt{\left(bD \cdot \frac{k}{k'}\right)} \tag{40}$$

or, in a dimensionless form, $\frac{\Lambda}{D} = \sqrt{\left(\frac{d}{D} \cdot \frac{k}{k'}\right)}$

where

Λ = leakage length [m]
b = filter layer thickness [m]
D = thickness of the cover layer
k = permeability of the filter [m/s]
k' = permeability of the cover layer [m/s]

The leakage length clearly takes into account the relationship between k and k' and also the thickness of the cover layer and the filter layer. For the theory behind this relationship reference should be made to Appendix C. The theory is very similar to that for seepage into a polder. The pressure head difference which develops on the cover layer is larger with a large leakage length than with a small leakage length. This is mainly due to the relationship k/k' in the leakage length formula. The leakage length is generally between 0.5 and 3 m.
The [Guide to concrete dike revetments] defines the leakage length differently as:

$$\lambda = \sin\alpha \sqrt{\left(\frac{bkD}{k'}\right)} \tag{41}$$

This parameter is referred to as the "leakage height".

If a granular filter is not used when the cover layer lies on a geotextile on sand, or when blocks lie on clay, the leakage length cannot be determined because the size of b and k cannot be calculated. The physical description of the flow is different for this type of structure. The leakage length approach can be applied provided that the permeability of the cover layer is much smaller than that of the filter and the permeability of the filter is larger than about 1 mm/s, see Table 9.

Table 9. Applicability of the leakage length theory (+ = applicable, – = not applicable).

	Cover layer	
Sublayer (under the geotextile, if any)	$k' < k$	$k' > k$
granular filter, k>1 mm/s	+	–
granular filter, k<1 mm/s	–	–
sand or clay	–	–

The effect of the leakage length on the dimensions of the critical wave is apparent from the following equation which was developed from general indications found in the Analytical Design calculations discussed in Section 8.4.3 [Bezuijen et al., 1990, page 176]:

$$H_{scr} = c \cdot \Delta D \cdot \left(\frac{D}{\Lambda \xi_{op}}\right)^{0.67} \tag{42}$$

or $$H_{scr} = c \cdot \Delta D \cdot (D/b)^{0.33} \cdot (k'/k)^{0.33} \cdot \xi_{op}^{-0.67} \tag{43}$$

where

H_{scr} = (significant) wave height at which blocks are just lifted out of the slope [m]
Δ = relative volumetric mass of blocks (concrete) [–]
c = coefficient, slightly dependent on Δ, $\tan\alpha$, friction, etc.

The relative volumetric mass of concrete (Δ) is given by:

$$\Delta = (\rho_b - \rho)/\rho \tag{44}$$

where

ρ = volumetric mass of water (fresh water: $\rho = 1000$; salt water: $\rho = 1025$) [kg/m^3]
ρ_b = volumetric mass of blocks (concrete) [kg/m^3]

Equations (42) and (43) indicate the general trends and have been used together with measured data to set up the general model described in Section 8.4.2, see Figures 103 to 113. From these equations, assuming by approximation that c is constant, it appears that:

- Influence Δ: An increase in the volumetric mass, Δ, produces a proportional increase in the critical wave height. If ρ_b is increased from 2300 to 2600 kg/m^3, H_{scr} is increased by about 23%.
- Influence ξ_{op}: The breaker parameter, comprises the slope angle ($\tan\alpha$) and the wave steepness ($\xi_{op} = \tan a/\sqrt{H_s/L_{op}}$) . If the slope angle is reduced from 1:3 to 1:4 ($\tan\alpha$ from 0.33 to 0.25) H_{scr} is increased by about 20%.
- Influence D: An increase of 20% in the thickness of the cover layer, D, increases H_{scr} by about 27%.
- Influence Λ: A 30% reduction in the leakage length, Λ, increases H_{scr} by about 20%. This can generally be achieved by halving the thickness of the filter layer or by doubling the k'/k value. The latter can be achieved by approximation, by:
  - reducing the grain size of the filter by about 50%, or
  - by doubling the number of holes in (between) the blocks, or
  - by making hole sizes 1.5 times smaller, or
  - by doubling the joint width between blocks.

Changing the structural parameters changes the coefficient c slightly; the effect of these parameters can only be evaluated by approximation.
It should be noted that changing the structural geometry can mean that failure mechanisms other than blocks being lifted out may govern the stability of the structure.

8.3 Design criteria

The lifting out of loose blocks

Clearly a revetment is designed to protect the weakest point on the slope against the largest forces that are likely to occur. For revetments which do not comprise interlocking blocks this implies that an accidentally loosened block must be taken into account. Such a block is referred to below as "loose". A loose block can be lifted out of the slope by a force which is equivalent to its weight plus the friction mobilized when it is pushed along the row of blocks immediately below it on the slope, see Figure 98.

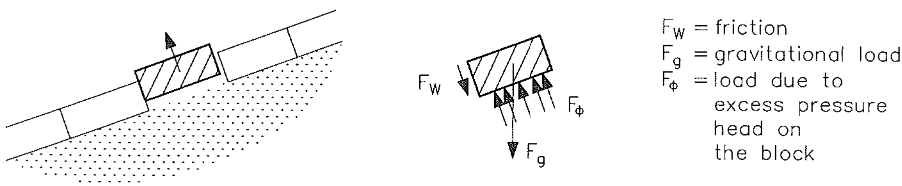


Fig. 98. Loads acting on a loose block.

For safety reasons one should anticipate on loose blocks since, even with careful laying and washing in, blocks can still work loose. Unless it can be shown otherwise in a large number of tests [Bezuijen et al. (1990), page 99] it has to be assumed that loose blocks will occur, see Figure 99. In principle block mattresses can also work loose if they are not fastened together. The edges of the mattress can be considered as loose blocks with only limited interaction with adjacent mattresses.

If mattresses are fastened together, or if the blocks (on the mattress) interlock in some way or other, the situation is different. The freedom of individual blocks to move is then restricted considerably and a block cannot be lifted out on its own. The failure mechanisms which can develop in this situation include:

- A row, normal to the axis of the dike and several blocks wide, can be lifted slightly, pushing the underlying filter: an S-profile then begins to form in rows of blocks and the connection between blocks is broken eventually.
- The upwards force on the cover layer can reduce the friction between the cover layer and the sublayer and the cover layer can then move as a whole down the slope.

It is recommended, in all cases, that block mattresses should be considered as loose blocks (conservative assumption). For mattresses fastened together this gives a safe underestimate of the stability; for mattresses not fastened together the approach gives a good approximation of the actual stability.

Cover layer failure due to blocks being lifted out is not an instantaneous process which does not occur at one wave and does occur at a following larger wave. Loose blocks remain relatively stable up to a certain wave height but small movements will occur if wave heights increase above this value. After the wave has passed however, the block will settle back into its original position. With increasing wave heights the block movements will increase and there is a possibility that the block will not return to its original position. Lifting out may then be imminent.

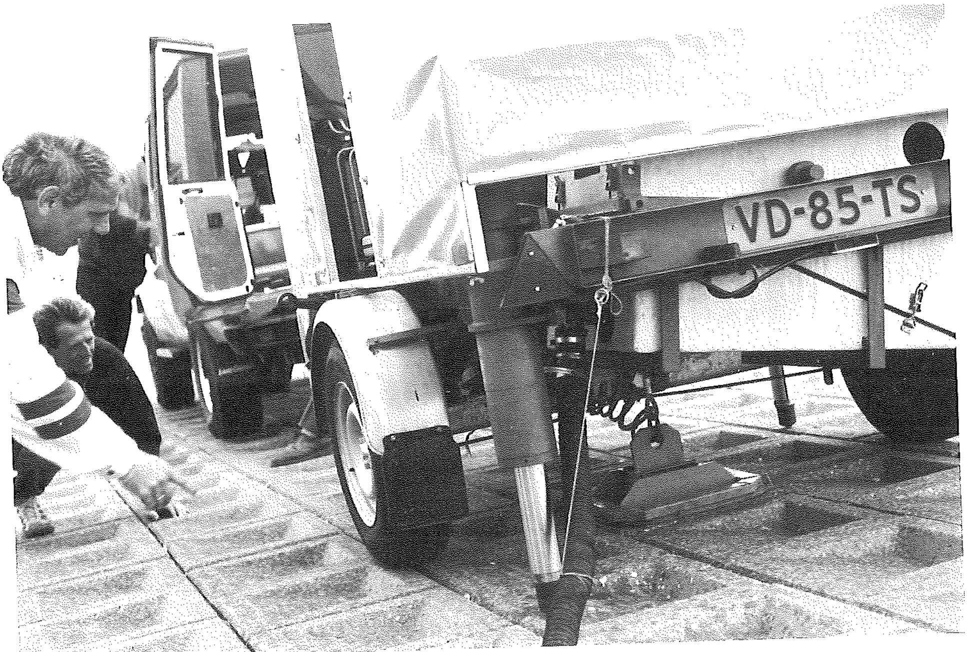


Fig. 99. Friction test using a suction cup.

A train of waves does not have the same height, see Section 6.2.1. In a train with a significant wave height of H_s, which is near to the critical H_{scr}, blocks will only move, therefore, when the waves are very high. The degree of movement to be expected is about 1 cm with one wave in 50 (on average about once every 5 minutes) and then only concerns the few blocks which are completely loose.
The following design criteria have been developed for both paved and linked revetments with the above in mind:

1. Exceptional loads (extreme wave action at the design flood storm level):
   a. No movement of blocks with individual waves of height H_s, and also
   b. Block movements should, at the most, be only 10% of the depth of the cover layer with individual waves of height between H_s and $H_{2\%}$.

   Generally criterion 1a is ruling with regard to criterion 1b.
2. Frequently occurring waves (ship waves in canals, wave action in the tidal zone):
   – no movement of blocks with individual waves of height between H_s and $H_{2\%}$.

   The size of $H_{2\%}$ can be calculated as follows, see Appendix I:

$$H_{2\%} = 1.4 \cdot H_s\text{; with a maximum value, } H_{2\,\%} = 0.6 \cdot d \qquad (45)$$

where

d = design water depth, as defined in Figure 49 [m]

The difference between the wave heights which cause block movement of 0% and 10% can be considerable. This is because a short duration load, which is larger than the weight of the block plus the friction, produces very little movement in the block. This is due to the inertia of the block and the associated movement of the water but, more particularly, to the influence of the "entry flow". By this is meant the flow in the filter which develops when, for example, a block makes one or two movements. During these movements it must be possible for water to flow out of the filter in response to the difference in pressure head on the revetment and also to fill the spaces between the moving block and filter, see Figure 100. The more quickly a block moves the faster the water must flow to fill the rapidly growing space under the block. Entry flow does not develop of its own accord because the flow resistance in the filter must be overcome. This leads to a decrease of the pressure head under the moving block. Entry flow is a process similar to that which causes the suction on an underwater block when it is lifted quickly out of the revetment. If the block is lifted very slowly out of the revetment the situation is easier to evaluate because the water has more time to flow under the block. The shorter the duration of the loads the larger is the entry flow effect. For most types of block revetment wave impacts which have a very short duration are therefore of secondary importance.

The beneficial effect of entry flow increases with the surface area of the block and is greater if the permeability of the filter is relatively small. This effect is quantified using the parameter Γ_3, see Figure 133.

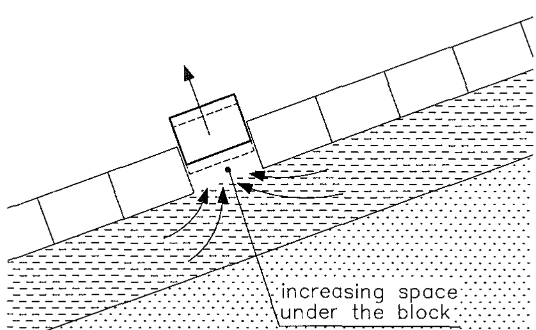


Fig. 100. Flow towards a moving block.

Sand penetration into the filter or flushing out through the cover layer

The heaviest loads on the interface usually occur immediately before wave impact. The hydraulic gradient then extends from about the level of maximum wave attack on the cover (0.5 to 1.5 times the wave height below SWL) up to the phreatic level in the filter. If the loads in this section exceed the strength (the critical hydraulic gradient) material can be transported down the interface. This transport is a maximum at about SWL and the maximum erosion of the base will occur here, see Figure 101 [BEZUIJEN, et al., 1990, page 128]. An upwards hydraulic gradient develops on the interface below the point of maximum load on the cover layer. This is much more local than the downwards gradient. Generally the length of the section affected by the upwards hydraulic gradient is about a wave height, measured along the slope. Material transport cannot occur if the grains in the base are larger than the pores in the filter or the mesh of the geotextile. This failure mechanism is very unlikely.

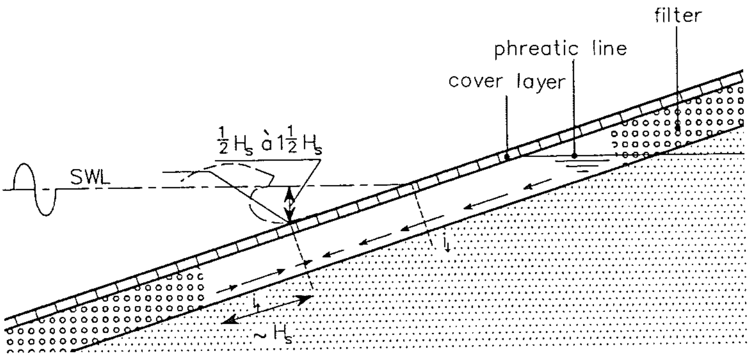


Fig. 101. Loads on the interface.

The filter or the geotextile is then referred to as being "geometrically tight or sealed". In this situation care must then be taken to ensure that toe or transition structures are also geometrically sealed, so that sand transport along small channels in the surface of the sand is impossible. This type of transport is referred to as piping under the filter or geotextile, see Figure 102. A geometrically tight filter (or geotextile) is specified in some designs, particularly if the filter layer is to be very thin ($b < \frac{1}{2}D$). The wave impact on the interface can then be very heavy without damage occurring. A granular filter can only be laid directly on clay if the filter layer is sufficiently thick and fine grained. Practical criteria for this however are not available. It is recommended that a geometrically tight geotextile is always laid between the filter and the clay. Generally it is unnecessary for individual particles to be less than the mesh size because small clay particles always tend to erode into flocs. This is taken into account in the formulas used for designing geometrically tight geotextiles for use on clay.

To summarize, the following criteria should be applied:

1. Granular filter or geotextile on sand, normal filter layer thickness:
   - downwards flow: $i_{cr\downarrow} > i_{\downarrow}$ and (strength>loads)
   - upwards flow: $i_{cr\uparrow} > i_{\uparrow}$ (specified)
2. Granular filter or geotextile on sand, small filter layer thickness:
   - geometrically sealed (specified)

3. Geotextile on clay:
   - geometrically sealed (recommended)

Fig. 102. Channel formation in the sand surface (geotextile and granular filter removed).

A precise distinction between normal, thick and thin filter layers, based on calculations, is discussed in Section 8.4.3. Criteria are also given for geometrically sealed filters and geotextiles .

8.4 **Revetment design involving wind wave loads**

8.4.1 *Review of various methods*

Various methods can be used to determine the loads and the strength. In order of increasing rigour and accuracy these are:

- the preliminary design method (only for the stability of the cover layer);
- the analytical design method (for the stability of the cover layer and the penetration of sand into the filter);

- the STEENZET/1+ numerical model (for the stability of the cover layer and loads in the filter related to the penetration of sand);
- the STEENZET/2 numerical model (for loads on the cover layer and loads in the filter related to the penetration of sand);
- model investigations (for the stability of the cover layer and sand penetration into the filter).

The preliminary design method is based on experimental data from physical model investigations and the results of calculations using the analytical design method and STEENZET/1+. This gives a direct relationship between the wave boundary conditions and the structural properties on the one hand and stability on the other. The method is fast and very easy to use but gives less accurate results than the other methods for structures with a granular filter. From the results it is only possible to assess cover layer stability; the method cannot be applied for the penetration of sand into the filter (from the base).

The analytical design method is based on equations describing the physical processes in detail and can be used to assess the stability of the cover layer and the interface between the filter and the sublayers [BEZUIJEN et al., 1990, page 101]. The method comprises a number of design diagrams from which the stability can be assessed without the use of a computer. It is more accurate than the preliminary design method, but can only be used for structures which have a granular filter under the cover layer. The method is mainly for the designer who is comparing various design options and optimizing preferred solutions. In contrast to the preliminary design method the analytical design method indicates the effect of all the structural components and aspects. It is available in the user friendly ANAMOS PC Program [DE WAAL, 1990a].

The STEENZET/1+ numerical model can be compared with the analytical design method but the results are more accurate, especially for structures with a very open cover layer and a fairly fine grained filter layer [BEZUIJEN et al., 1990, page 108]. As with the analytical design method this model can only be used if there is a granular filter under the cover layer. STEENZET/1+ is run on a normal PC but is only intended for specialists. It is recommended that results using the analytical design method are checked using STEENZET/1+. The model is less suitable for use in initial design stages.

The STEENZET/2 numerical model is an advanced computer model based on the finite element method and can be applied for structures which do not have a granular filter layer, that is, where the cover layer is laid on sand. The model only calculates the difference in pressure head on the cover layer and therefore, in order to assess stability, must be used in combination with other methods. It is not suitable for the design process but can be used to check the results of other methods. In view of the complexity of the model it is likely to be used only by a small group of specialists.

The final method discussed here for evaluating the stability of a revetment is physical model investigations [BEZUIJEN, et al., 1990, page 151]. With large scale physical models (up to 1:3), provided that wave action is perpendicular to the structure, there are very few limitations to the structural composition and wave boundary conditions which can be examined. With small scale physical models there are, in contrast, many limitations which in practice means that the stability can only be determined for particular situations. This is because the scaling laws for flow in the structure tend to contradict those for waves. A combination of small scale physical model investigations, aimed at establishing the relationship between wave boundary conditions and pressure on the slope, and numerical calculations, aimed at flow in the structure, can however be a good approach. Since the duration of a physical model investigation is always a limiting factor relatively new revetments tend to be investigated in this way.
The properties of a revetment apparently change in the course of time [STOUTJESDIJK, 1991]. In the past this aspect has been neglected in physical model investigations, see Section 8.7.3. Geotechnical instability is now taken into account as a failure mechanism in addition to the lifting of blocks and the penetration of sand into the filter. This mechanism requires a different approach and is treated separately in Chapter 9.
The preliminary and analytical design methods are described in the following sections. The use of STEENZET/1+ when the analytical design method is inadequate, the use of STEENZET/2 and the application of physical model investigations are outside the scope of this manual.
The results from all the above types of models have been verified extensively for slopes between 1:3 and 1:4. Attempts are now being made to verify the results over a wider range of slopes between 1:2.5 to 1:8. Some of the methods have been checked for very flat and very steep slopes.

8.4.2 *Preliminary Design Method*

The first step when designing or evaluating a cover layer of pitched blocks is to apply the "Preliminary Design Method". This is based on:

- results of large scale model investigations [BEZUIJEN et al., 1990, page 162];
- calculations using the Analytical Design Method;
- calculations using STEENZET/1+.

The aim of the Preliminary Design Method is to obtain, as easily as possible, an initial estimate of the thickness of the cover layer. The method simply establishes the relationship between the most important load parameter, that is, the breaker parameter, ξ_{op}, and $H_s/(\Delta D)$, H_s being the significant wave height and ΔD expressing the effective cover layer thickness. The relationship between ξ_{op} and $H_{scr}/(\Delta D)$ at the point when the lifting of a loose block is initiated depends on the type of structure. The types of structure have been subdivided principally into:

1. Paved blocks on a geotextile overlying sand.
2. Paved blocks laid directly on to clay (or on to a geotextile on clay).
3. Paved blocks laid on a granular filter.
4. Interlocking blocks laid on a geotextile overlying sand.
5. Interlocking blocks laid directly on to clay (or on to a geotextile on clay).
6. Interlocking blocks laid on a granular filter.

In addition to the main subdivision, a further subdivision has been made for pitching laid on a granular filter (Types 3 and 6). A distinction has been made between those types which are "suitable", "acceptable" and "unsuitable" with reference to the development of pressure head difference on the cover layer. The subdivision is described below and is shown in Figure 103:

a. "Appropriate" structures – cover layer lying directly on the filter:
   if: thin filter layer: $b/D < 0.5$, and
   fine filter layer: $D_{f15} < 10$ mm and
   open cover layer: joints only: $\Omega > 3\%$
   holes at centre-to-centre up to 0.3 m: $\Omega > 7\%$
   holes at centre-to-centre of 0.3 m: $\Omega > 15\%$
   and no hole or joint filler.
b. "Acceptable" structures – cover layer lying directly on the filter:
   if the requirements for Type a and b are not satisfied
c. "Unsuitable" structures – cover layer lying directly on the filter:
   if: thick filter layer: $b/D \geq 0.5$ and
   coarse filter layer: $D_{f15} > 3$ to 5 mm and
   sealed or densely compacted cover layer:
   joints only: $\Omega < 2\ \%$
   holes at centre-to-centre up to 0.3 m: $\Omega < 5\%$
   holes at centre-to-centre of 0.3 m: $\Omega < 10\%$

where

b = thickness of the filter layer laid directly under the cover layer (filler layer) [m]

D_{f15} = grain size of the filter layer laid directly under the cover layer (filler layer) which is exceeded by 15% of the material by weight [m]

Ω = relative open surface area [–]

With this subdivision the filter layer should have a permeability greater than 1 mm/s. Otherwise the cover layer (on a geotextile) should lie on sand. When several layers of granular material are used, for example, a filler layer on top of minestone, only the upper (filler) layer is important. The distinction between a coarse and fine filter layer depends on the other structural properties. The limit for Structural Type a. – appropriate, is therefore, D_{f15} = 10 mm and for Structural Type c. – acceptable, 3 to 5 mm.

The same principle applies to the distinction between sealed or open cover layers. The cover layer is seen as comprising "linked elements" in the case of block mattresses fastened together or interlocking blocks with tongue and groove-type joints, at the minimum, to the row(s) of blocks above on the slope. Columns can be schematized as rectangular blocks with wide joints, see Table 2 in Section 5.2.

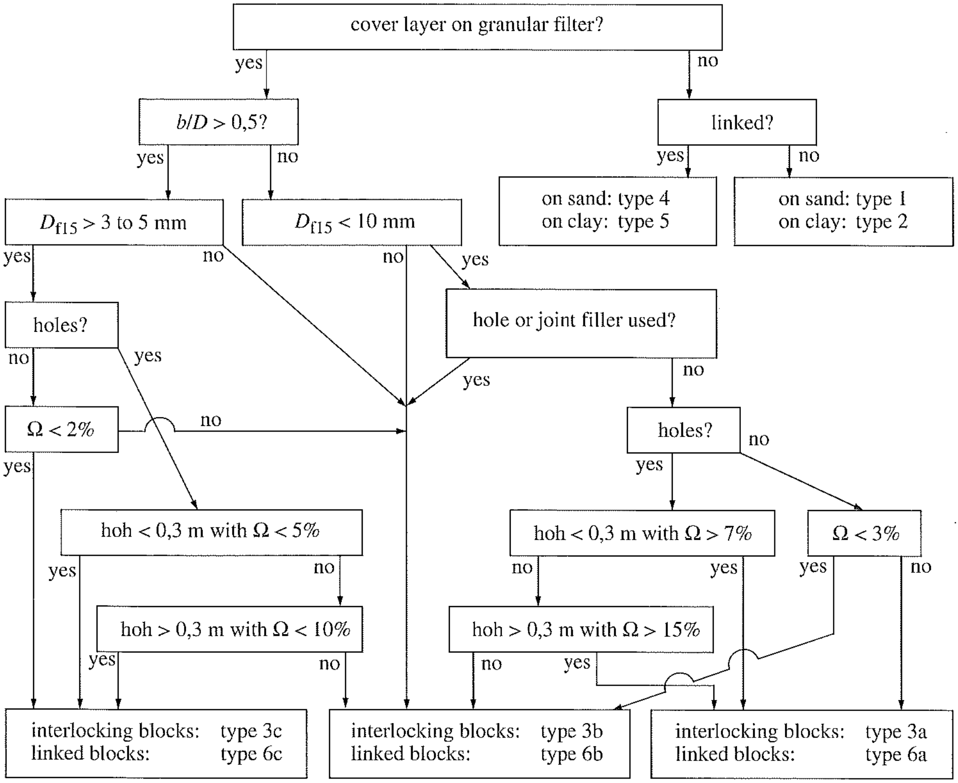


Fig. 103. Flow chart for determining the type of structure using the Preliminary Design Method (hoh = centre-to-centre).

The relative open surface area for rectangular blocks with (possibly) circular holes can be calculated using Equation 1, see Section 5.2.
The design should allow for the heaviest loads impacting at some distance below SWL. Depending on the slope angle and the wave steepness the heaviest loads will act 0.3 to 1.5 times the wave height below SWL, see Figure 128.
Once the structural type has been established the stability of the structure, as designed, can be evaluated using Figures 104 to 113. These figures, with the exception of Figures 105 (Structural Type 2) and 110 (Structural Type 5) are graphs with unbroken and dashed lines. If the load ($H_s/(\Delta D)$) for a certain value of ξ_{op} falls below the unbroken line, then the structure is stable for that load. If the value is above the dashed line, the structure is unstable. The area between the two lines is a "grey" area

in which the stability of the structure is doubtful. If this area the stability depends on factors not covered by the Preliminary Design Method and the Analytical Design Method should be applied, see Section 8.4.3. With Structural Types 2 and 5 (cover layer on clay) there is no upper limit to the grey area (no dashed line).

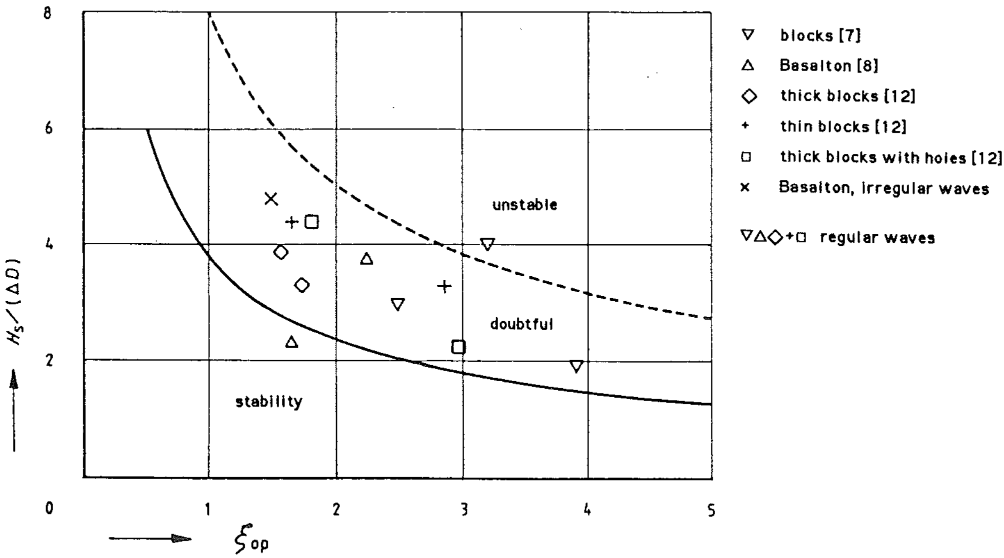


Fig. 104. Paved blocks on a geotextile overlying sand (Structural Type 1). This type of structure has only limited application if $H_s > 1$ m (see Chapter 9, Figures 137 to 140).

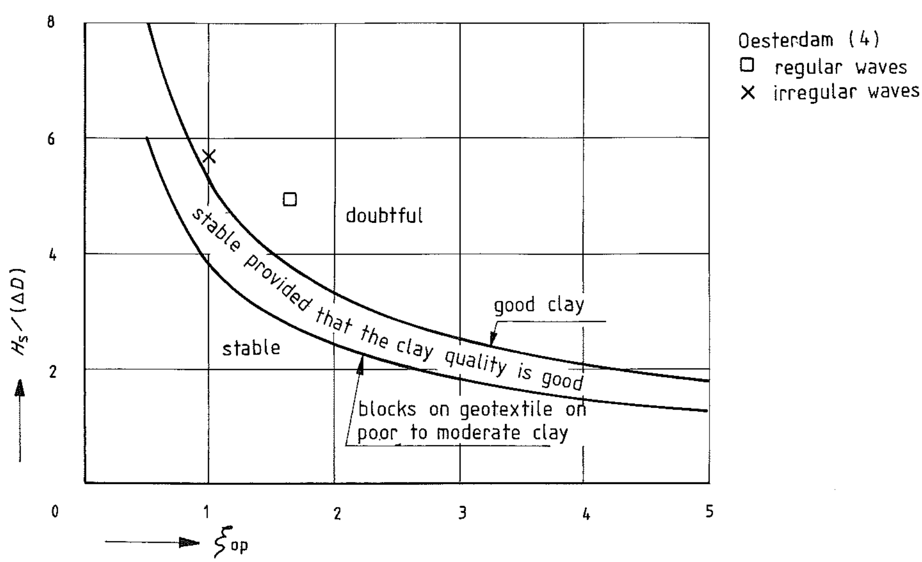


Fig. 105. Paved blocks laid directly on good clay or on a geotextile on poor to moderate quality clay (Structural Type 2, see Section 8.7.2).

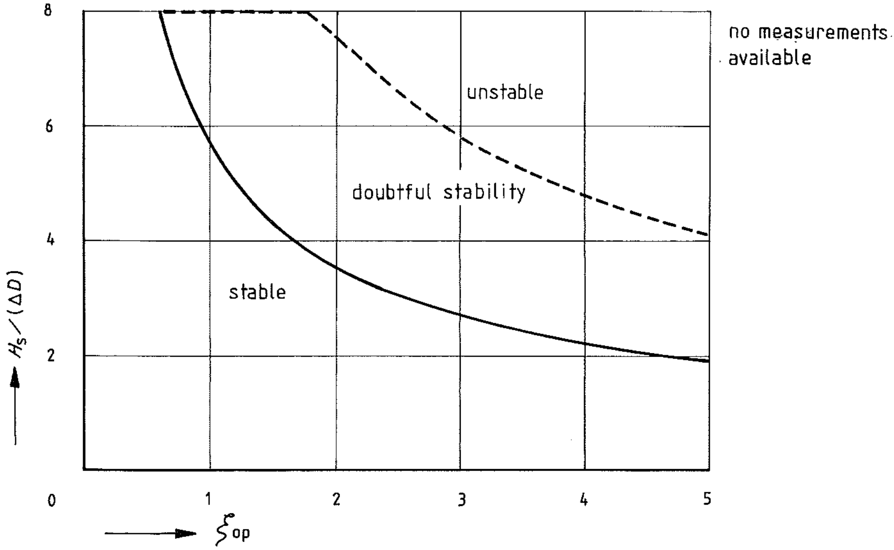


Fig. 106. Paved stones on a granular filter – Appropriate Structure (Structural Type 3a). (Result to be checked using Analytical Design Method).

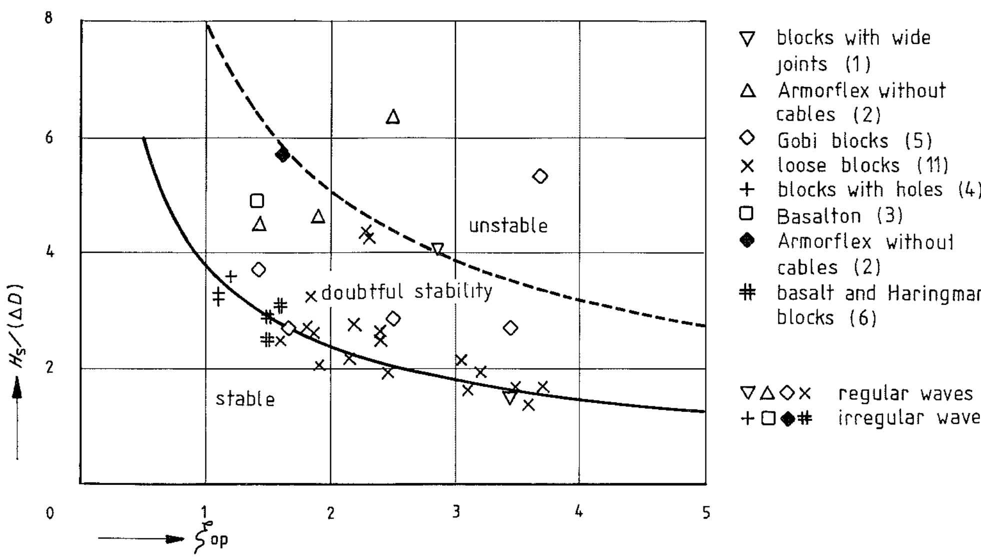


Fig. 107. Paved stones on a granular filter – Acceptable Structure (Structural Type 3b). (Result to be checked using Analytical Design Method)

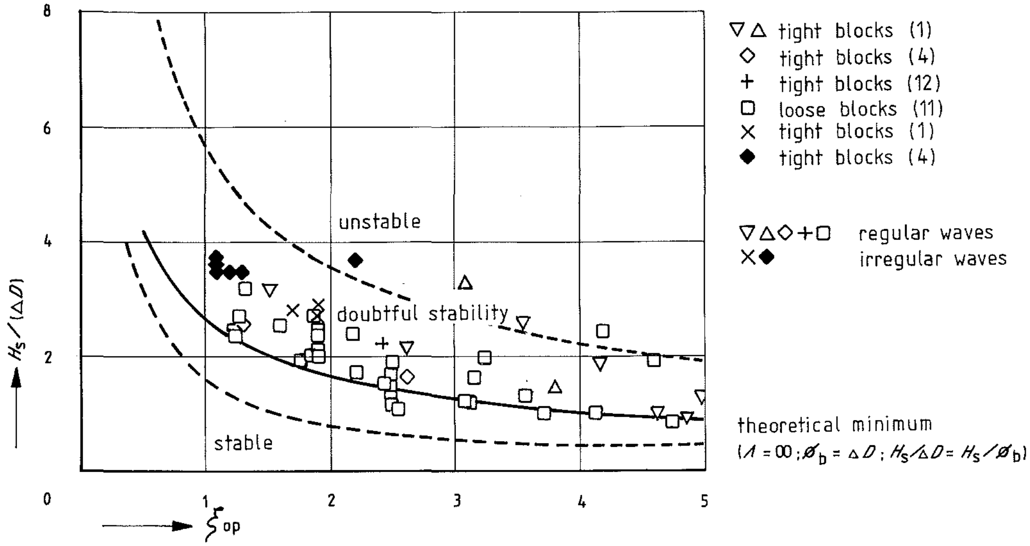


Fig. 108. Paved blocks on a granular filter – Unsuitable Structure (Structural Type 3c). (Results to be checked using the Analytical Design Method).

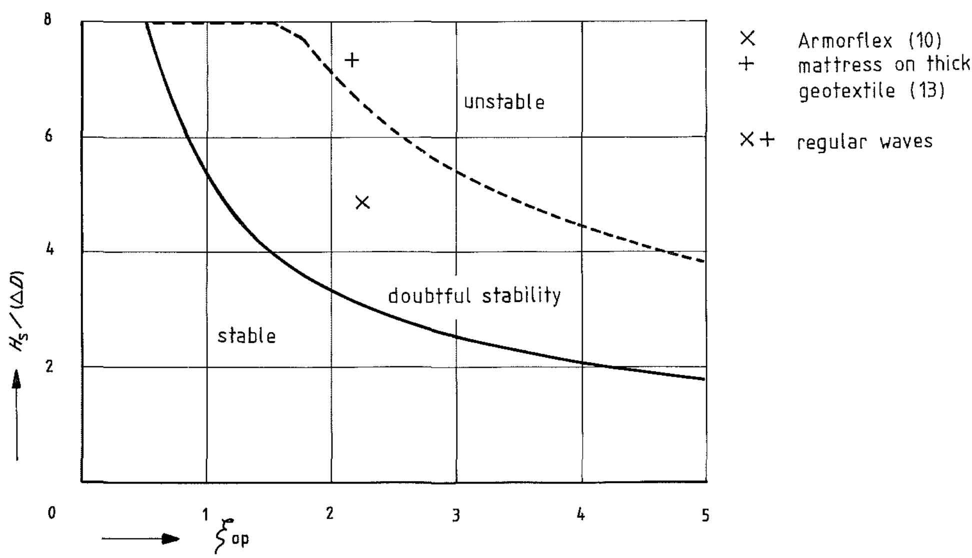


Fig. 109. Linked blocks on a geotextile on sand (Structural Type 4). This type of structure has only limited application if $H_s > 1$ to 1.5 m, see Chapter 9, Figures 137 to 140.

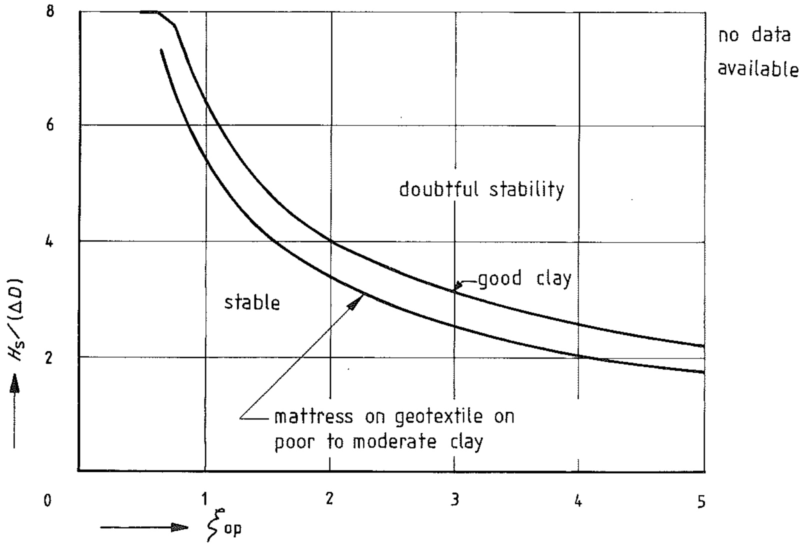


Fig. 110. Linked blocks on good clay or a geotextile on poor to moderate quality clay (Structural Type 5), see Section 8.7.2.

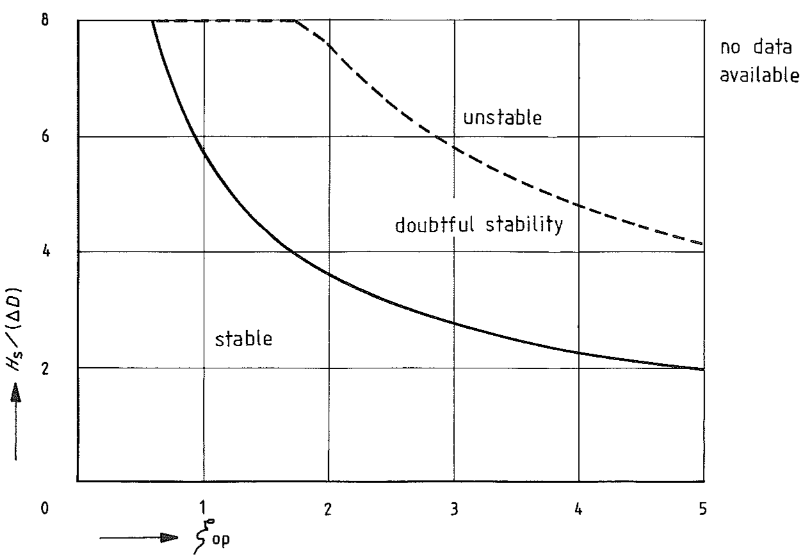


Fig. 111. Linked blocks on a granular filter – Appropriate structure (Structural Type 6a). (Results to be checked using the Analytical Design Method).

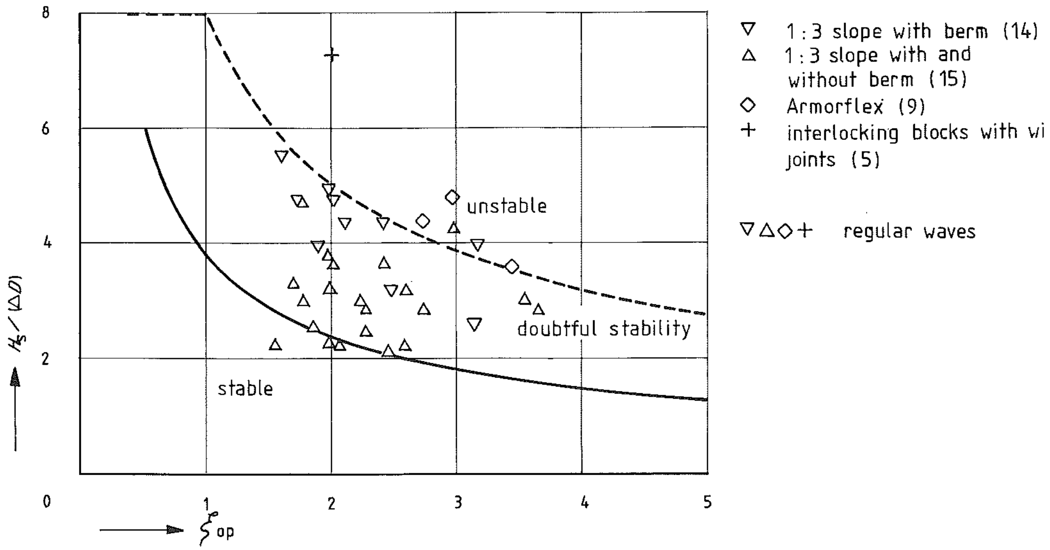


Fig. 112. Linked blocks on a granular filter – Acceptable Structure (Structural Type 6b). (Results to be checked using the Analytical Design Method).

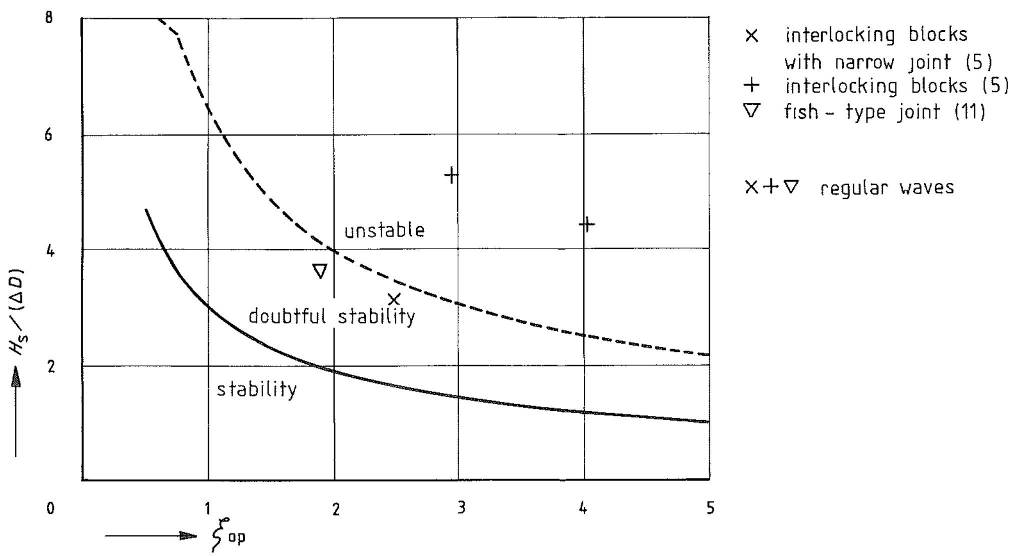


Fig. 113. Linked blocks on a granular filter – Unsuitable Structure (Structural Type 6c). (Results to be checked using the Analytical Design Method).

The results from large scale model investigations are also included in the figures. Generally the data, which have values of $H_s/(\Delta D)$ indicating the initiation of damage, lie in the grey area. Since the spread of measured data is unavoidable, some points lie outside the grey area, which is acceptable. The lines have been selected so that only a few points fall outside the grey area. All the lines are based on Equation (42) although strictly this equation is only applicable for Structural Types 3 and 6. Because there is no equation for the other structural types and because the data do not contradict Equation (42) it has been applied generally.
The numbers in square brackets on the figures refer to the following literature references:

[1] = [Burger, 1983]
[2] = [Lindenburg, 1983]
[3] = [Van der Weide et al., 1983]
[4] = [Oester Dam, 1982]
[5] = [Den Boer, 1982]
[6] = [Burger, 1985]
[7] = [Van der Meer et al., 1983]
[8] = [Lindenburg, 1988]
[9] = [Large Scale Model Study of Armorflex, 1984]
[10] = [Large Scale Modelling of Armorflex, 1983]
[11] = [Wouters, 1991]
[12] = [Führböter et al., 1988]
[13] = [Large Scale Model Studies, 1982]
[14] = [Large Scale Model Investigations, 1983]
[15] = [Two-dimensional Model Study, 1985]

Although the results of the investigations into cover layers laid directly on to sand give a reasonably consistent picture of stability the fact that such structures can also fail through geotechnical instability should not be overlooked. For this type of failure $H_s/(\Delta D)$ is not the only relevant parameter and care must be taken when using Figures 104 and 109, see also Chapter 9, Figures 137 to 140 in particular. The Preliminary Design Method should only be used to obtain a first impression of the thickness of the cover layer and it is recommended that, irrespective of the type of structure, calculations should be carried out to check the following:

a. Cover layer on a granular filter:
   - the stability of the cover layer using the Analytical Design Method, see Section 8.4.3,
   - the stability of the interface between the filter and the base, using the Analytical Design Method, see Section 8.4.3,
   - the geotechnical stability, Chapter 9,
   - the stability of the toe and anchor structure, see Section 8.7.1.

b. Cover layer on a geotextile on sand:
   - the geotechnical stability, see Chapter 9,
   - the sand-tightness of the geotextile, see Figure 116,
   - the stability of the toe and anchor structures, see Section 8.7.1.
c. Cover layer on clay:
   - inclusion of clay specification in the design, see Section 8.7.2,
   - the stability of the toe and anchor structures, see Section 8.7.1.

8.4.3 *The Analytical Design Method*

Compared with the Preliminary Design Method, the Analytical Design Method enables a more precise distinction to be made between sealed and open cover layers, fine and coarse filters, etc. In addition it can be used to assess the stability of the interface between the filter and the subsoil [BEZUIJEN et al., 1990, Chapters 3, 4, 5 and 7]. The areas of application of this method are however relatively limited. In fact it can only be applied to pitching laid on granular filters. To help with the solution of problems in other areas reference is made in this section to design methods which can be applied to other structural types. The strength of the Analytical Design Method lies in the fact that it takes into account all the relevant loading and structural aspects and compels the designer to choose from all the various parameters available, enabling him to optimize the designs of the structure. The more advanced methods such as the STEENZET/1+ and 2 numerical models and physical model investigations are less suitable for optimizing designs and are aimed more at the verification of the results of the Analytical Design Method.
A list of all the structural and loading parameters is given below. These parameters have to be quantified before the Analytical Design Method can be used to check the stability of a structure comprising a cover layer with possibly a geotextile, on one or more filter layers. The Analytical Design Method cannot be used for structures in which the a cover layer is laid directly on sand or clay, possibly with a geotextile, without a (thin) layer of granular material. The latter type of structure is considered in this section only with reference to the stability of the cover layer and the relative figure used in the Preliminary Design Method. This enables design methods for all structural types to be summarized in Figure 116. The data marked with an asterisk in the summary given below applies also to structures with no granular filter.

Input parameters:
Loading, for definitions, see Section 6.2:

H_s *significant wave height [m]
 The stability for oblique waves ($\beta \neq 0$) is similar to that for normal waves
T_p *wave period at the peak of the spectrum [s]
d design water depth, see Figure 49 [m]

Slope:

tanα *(design slope angle) [–]

The design slope angle is the average slope between SWL and one to two wave heights below this level, see Figure 114. If the design slope and wave height are much steeper at lower water levels, it is possible that heavier loads will develop in these conditions than at the maximum water level and wave height.

d_o depth of the transition structure below SWL, see Figure 115 [m]

This refers to the transition structure which is nearest to the point of maximum attack on the slope (d_s below SWL, see Figures 124 and 128).

Blocks, for definitions, see Section 5.2:

D *thickness of the cover layer [m]

B width of the block, perpendicular to the axis of the dike, down the slope [m]

For columns the equivalent width and length is calculated from the average surface area of the unit, see Table 2 in Section 5.2:

$B = L = \sqrt{A}$ where: A = average surface area of the column [m^2]

L (block length parallel to the axis of the dike) [m]

s (average joint width) [m]

For columns see Table 2 in Section 5.2

Δ *(relative volumetric mass of the cover layer material = $(\rho_b - \rho)/\rho$ [–]

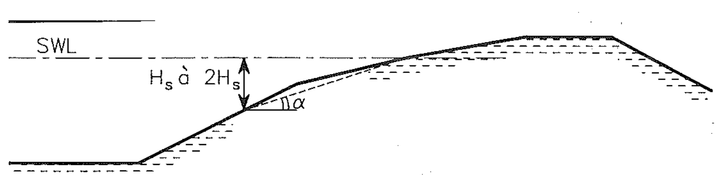


Fig. 114. Definition of the average slope angle used for designing the revetment (for wave run-up: see Figure 51).

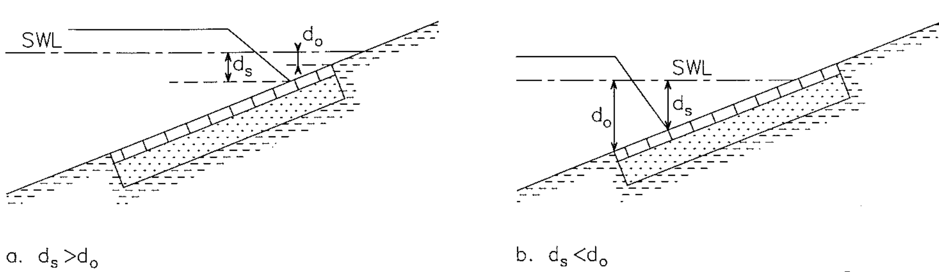


Fig. 115. Depth at the transition structure (if the transition lies above SWL then d_o is negative).

Washing-in, see also Table 4 in Section 5.3.1

D_{v15} characteristic grain size of hole filler or washed-in material 15% by weight of which is less than stated size [m]
n_v porosity of hole filler or washed-in material

Filter layers, see also material specifications in Section 5.3
b_1 (thickness of Filter Layer 1) [m]
b_2 (thickness of Filter Layer 2) [m]
$D_{f15,1}$ characteristic grain size of Filter Layer 1, 15% by weight of which is less than the stated size [m]
$D_{f15,2}$ characteristic grain size of Filter Layer 2, 15% by weight of which is less that the stated size [m]
n_{f1} (porosity of Filter Layer 1) [–]
n_{f2} (porosity of Filter Layer 2) [–]

Geotextile, see also Section 5.7:
ϕ_g head loss across the geotextile during permeability measurements [m]
q specific discharge, filter velocity: discharge per m^2 through the geotextile during permeability measurements [m/s]
T_g thickness of geotextile [m]
O_{90} *(characteristic mesh width for the geotextile) [m]

Base, see also Section 5.8 and 5.9:
D_{b90} *grain size, 90% by weight of which is less than the stated size $\approx D_{b85}$ [m]
D_{b50} *grain size, 50% by weight of which is less than the stated size [m]

When several filter layers are applied the upper layer is often a thin filter layer overlying, for example, minestone. There can also be washed-in material. When generally applying the Analytical Design Method these three materials are perhaps difficult to distinguish. The following recommendations therefore apply for a structure which includes a filler layer, minestone and a washed-in cover layer:

- For the Preliminary Design Method we make use of the filler layer parameters: layer thickness, b_u, grain size, D_{u15}, and porosity, n_u. In the calculations these parameters are specified using the general filter symbols: b, D_{f15} and n.
- The following filler layer and washed-in material grain sizes and porosity must be used to determine the permeability of the cover layer:
  - filler layer: D_{u15} and n_u (the general symbols and terminology, "D_{f15}", "n", and "filter", are used in diagrams);
  - washed-in material: D_{v15} and n_v
- The grain size of the filler layer is important (the general symbol, "D_{f90}", is used) because there is the possibility that grains can be flushed out through the cover layer.
- At places where the general filter symbols are used, (D_{f15} and n) they should be replaced by D_{u15} and n_u to determine the permeability of the filler layer.

- The following parameters should be used when calculating the leakage length:
  a. The layer thickness, b_1, and the permeability, k_1, of the minestone.
  b. The layer thickness, b_2, and the permeability, k_2, of the filler layer.
  Equation (46) must then be applied.
- D_{f15} and n for the minestone must be used to determine the stability of the interface between the filter and the base. This also applies when there is a geotextile between the minestone and the base.

Analytical Design Method calculations

Most of the equations used in the Analytical Design Method are presented in the manual in the form of diagrams so that only a ruler and a simple calculator are needed. The present section is completed with a description of the calculation method. A worked example is given in Chapter 15. The limitations of the Analytical Design Method are presented in Table 9. Figure 116 gives a flow chart which goes through the calculation, step by step. The chart refers to the stages and diagrams which are needed to determine the stability of a given structure.
The most elaborate calculation is that for a cover layer overlying a granular filter which is subjected only rarely to heavy loads (superstorm) and is a suitable case to study. The method is applied in the following sections:

a. provisional design,
b. determination of the leakage length,
c. checks on the possibility of sand penetrating the filter,
d. checks for block movements when $H = H_s$,
e. checks that any block movements will be less than $0.1 \cdot D$ if $H = 1.4 \cdot H_s$,
f. checks on geotechnical stability.

a. *Preliminary design*
   The data needed for the provisional design are first collected. This design can be made, in the first instance, using the Preliminary Design Method, see Section 8.4.2, or it can be based on experience. This initial design is necessary since the procedure only gives answers to questions on the stability of a given structure. The provisional design should take into account that:
   1. If $H_s/(\Delta D) < 1$ the cover layer is certainly stable, but probably over-designed.
   2. If $H_s/(\Delta D) > 8$ the cover layer is not stable.
   3. If, with a granular filter on sand, $D_{f15}/D_{b50} > 20$ to 40 a geotextile is probably needed to prevent sand penetration

The procedure indicated in Figure 116 should then be followed for the particular type of structure being considered.

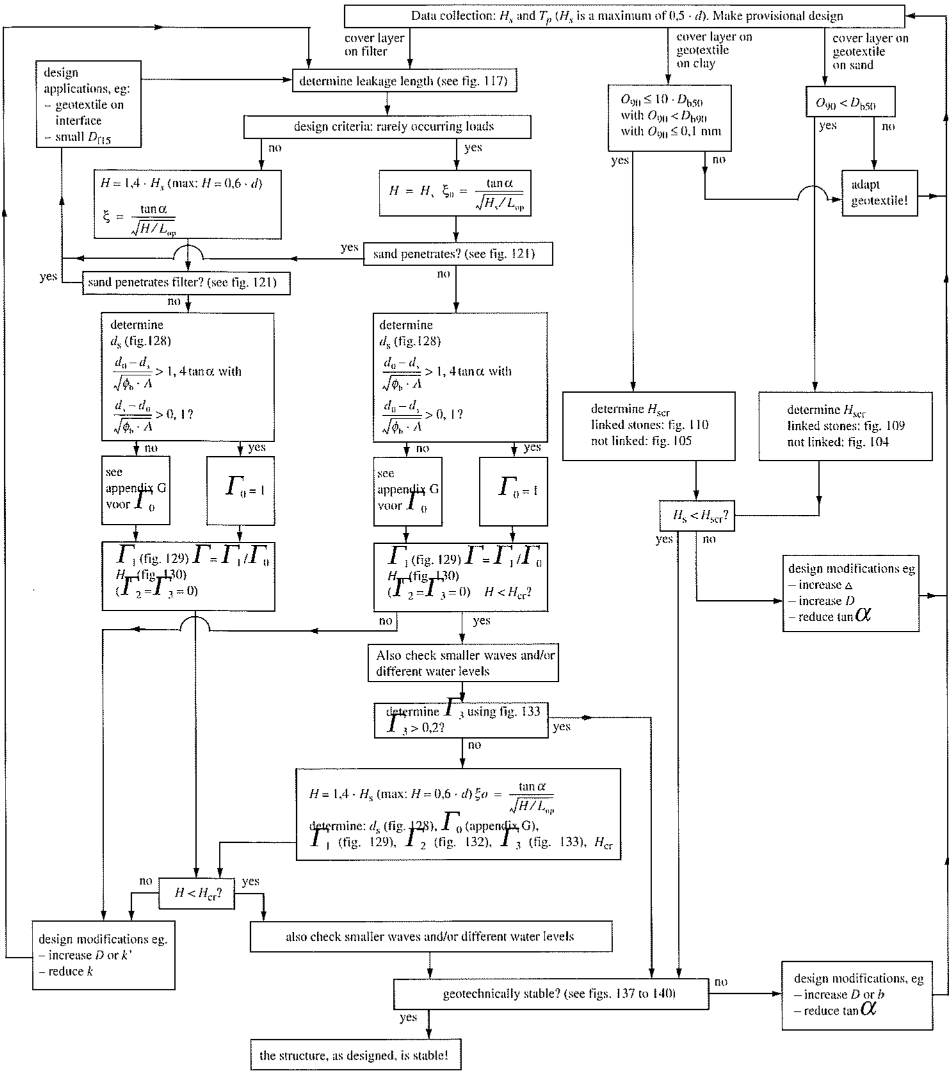


Fig. 116. Stability computation scheme using the Analytical Design Methods.

b. *Determination of the leakage length*

When considering a cover layer laid on a granular filter the first step is to determine the leakage length using the chart given in Figure 117. Distinction should be made here between paved blocks with joints, columns and blocks with holes. A cover layer of columns should be equated to one of blocks with joints, see Table 2 in Section 5.2. The permeability of the cover layer (k') should then be determined using Figure 118. For unusual cover layers, for example, blocks with holes, or chamfered sides, or when there is a geotextile between the cover layer and the filter, reference should be made to Appendix H.

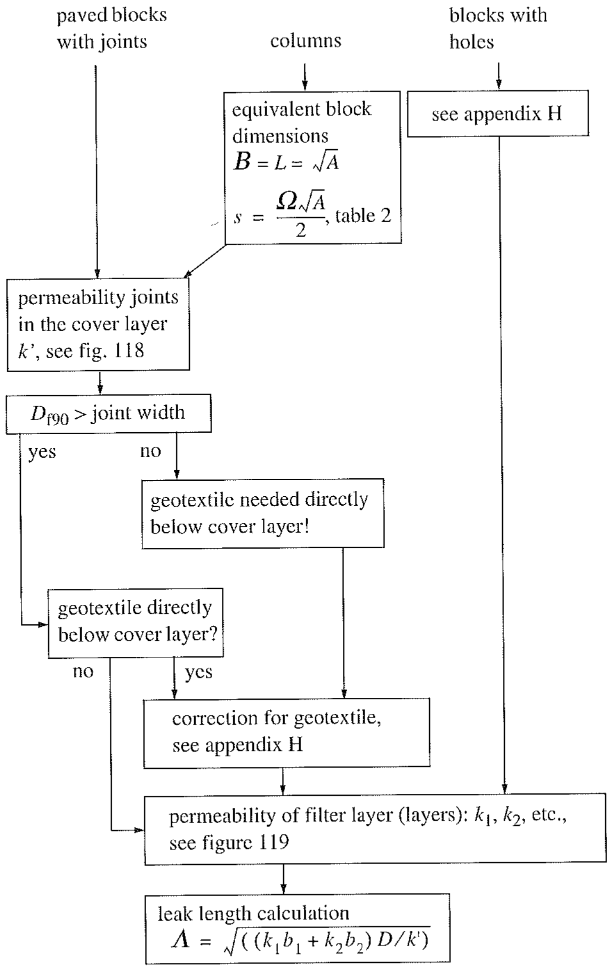
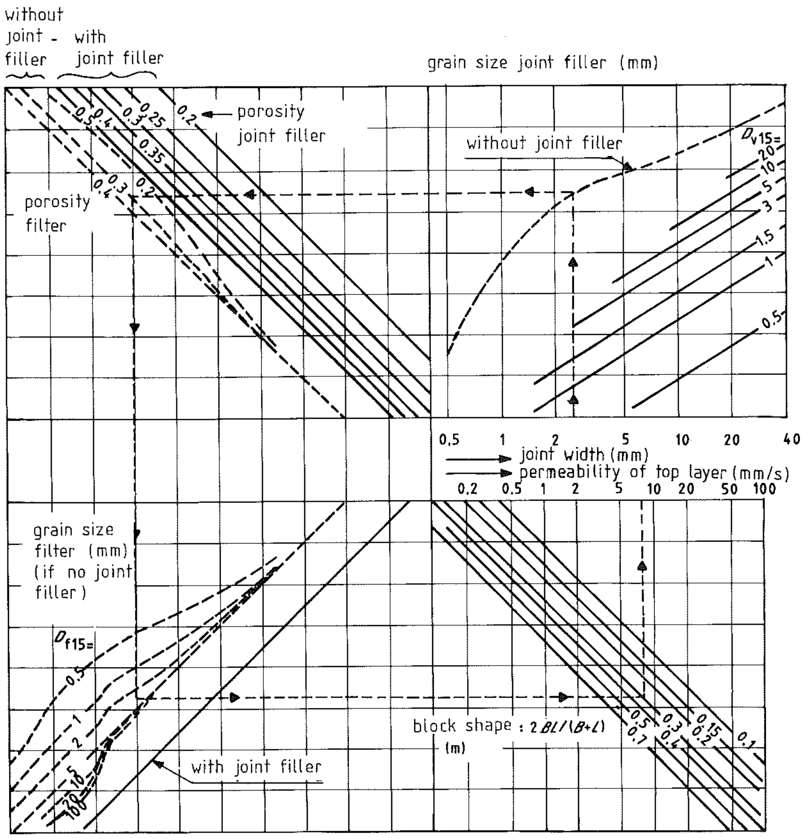


Fig. 117. Scheme for determining the leakage length.

Figure 118 should be used as follows:

- enter at the upper horizontal axis, which gives the joint width; in the example given the joint width is 2.5 mm;
- if, as shown in the example, there is no joint filler, a line should be followed vertically to the dashed line and then horizontally to the dashed line for the required porosity of the filter layer, which lies directly under the cover layer, in this case $n = 0.4$;
- proceed vertically upwards to the dashed line corresponding to the grain size of the filter layer. In the example $D_{f15} = 2$ mm;



example: joint width: s = 2.5 mm
no joint filler: (washed-in material)
porosity filter: n = 0.4
grain size filter: D_{f15} = 2 mm
block shape: B = L = 0.3 m
k' = 8 mm/s

Note: in case of joint filler the joints are filled only half.

Fig. 118. Permeability of the cover layer (joints only).

- go horizontally to the right to the block shape parameter $2BL/(B+L)$. For square blocks this parameter is equivalent to the block width (or length). In the example this is 0.3 m;
- proceed again vertically upwards to the lowest horizontal axis to read off the permeability of the cover layer. In this example this is 8 mm/s. If a joint filler is applied then the solid line should be used.When the permeability of the cover layer has been determined, the permeability of the filter layers can be evaluated in accordance with Figure 117. This is obtained, for each filter layer, using Figure

119. It is very important that each successive filter layer (below the cover layer) should have a greater permeability than the underlying layer, see Figure 120. The leakage length can then be determined using:

$$\Lambda = \sqrt{\frac{(k_1 b_1 + k_2 b_2)\, D}{k'}} \tag{46}$$

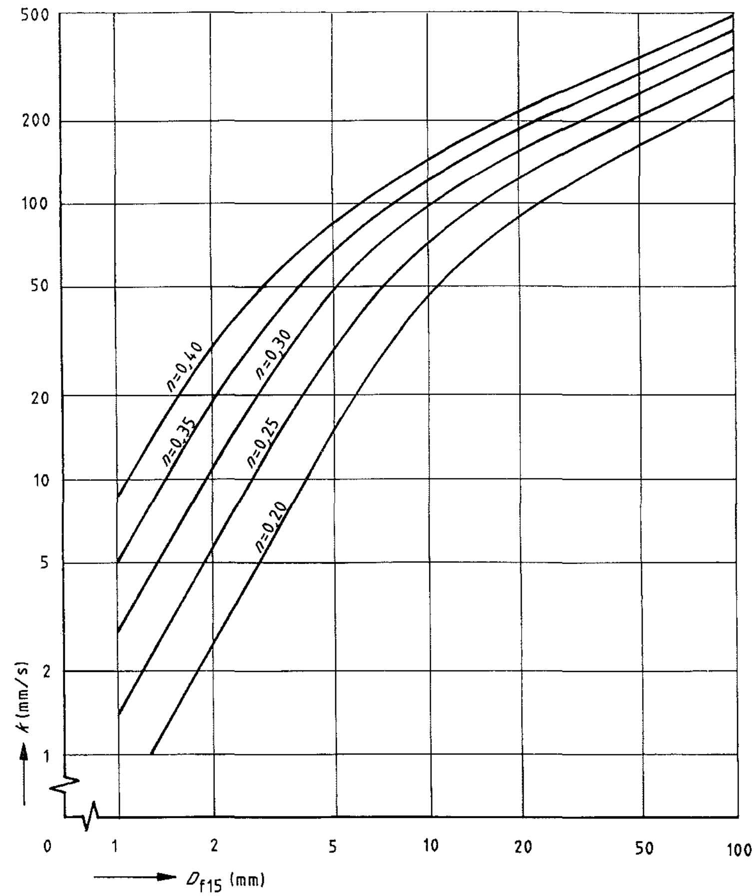


Fig. 119. Permeability of the filter (see also Table 4 in Section 5.3.1).

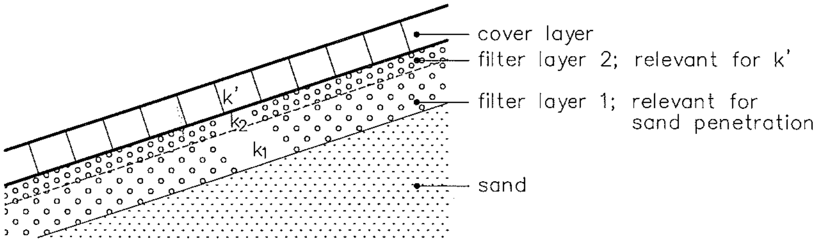


Fig. 120. Several filter layers.

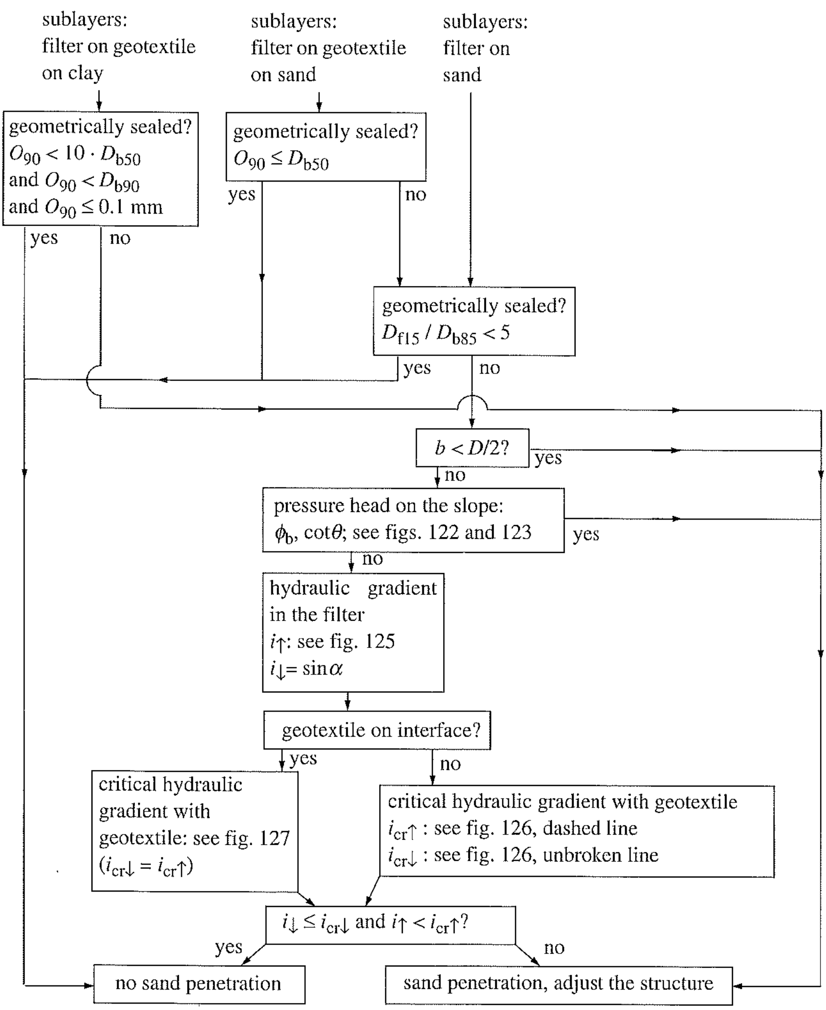


Fig. 121. Scheme for the determination of the stability with respect to penetration with sand into the filter and washing of the basis.

When the leakage length has been determined in the way indicated in the chart shown in Figure 116, the initial design can be checked against the design criteria, see also Section 8.3. For this discussion the "seldom occurring" loads are considered. The other branch of the chart is very similar. The design wave height is first set at the significant wave height: $H = H_s$ (loading). As explained in Section 6.2.2 the significant wave height in shallow water can never be greater than half the design water depth ($H_s = d/2$).

c. *Check on the possible penetration of sand into the filter*
The next step in the design is to consider the possible penetration of sand into the filter. Reference should be made to the chart given in Figure 121 where a distinction is made between structures with a base of clay under a geotextile and structures with a base of sand, possibly under a geotextile. The first step is to consider if the interface is sealed (sand tight). If this is not the case, for a sand base (and $b \geq D/2$), the hydraulic loads on the interface, that is, the hydraulic gradient in the filter and the base, may be so large that sand will penetrate the filter. The method of calculation assumes that the sand base is well compacted.

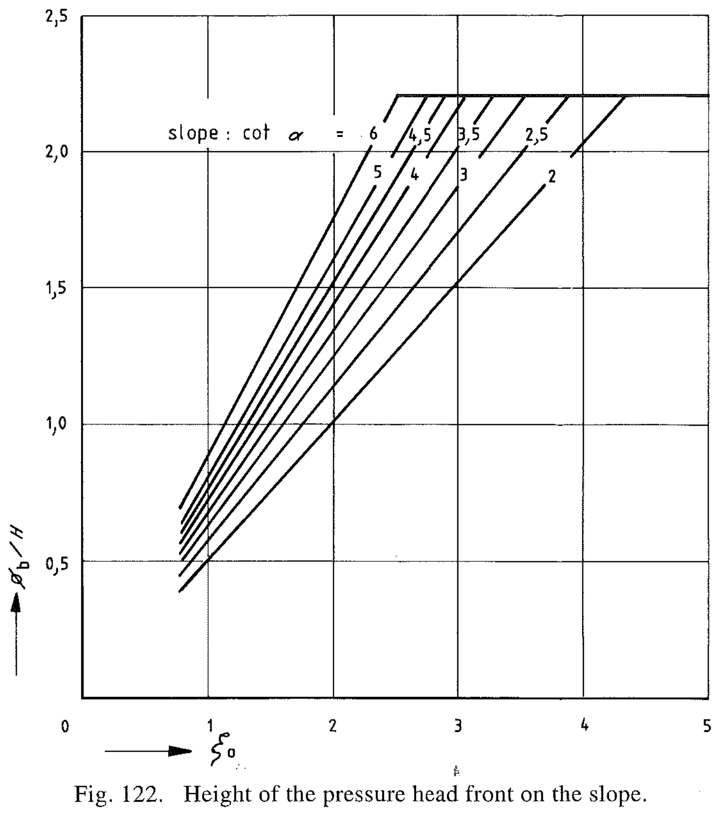


Fig. 122. Height of the pressure head front on the slope.

In order to determine the load on the interface the design pressure head at the interface must be calculated using the wave height (H), the slope ($\tan\alpha$) and the breaker parameter (ξ_{op}). Figures 122 and 123 should be used for this calculation. The pressure head on the slope is characterised by three parameters, ϕ_b, $\cot\theta$ and d_s, as shown in Figure 124. The value of d_s is needed later in the calculation, see Figure 128.

When considering the loading on the interface a distinction has to be made between:

- the hydraulic gradient up the interface, $i_\uparrow$;
- the hydraulic gradient down the interface, $i_\downarrow$

The hydraulic gradient up the interface ($i_\uparrow$) can be determined using the graph in Figure 125. The hydraulic gradient down the interface ($i_\downarrow$) is always equivalent to $\sin\alpha$ ($\approx \tan\alpha$).

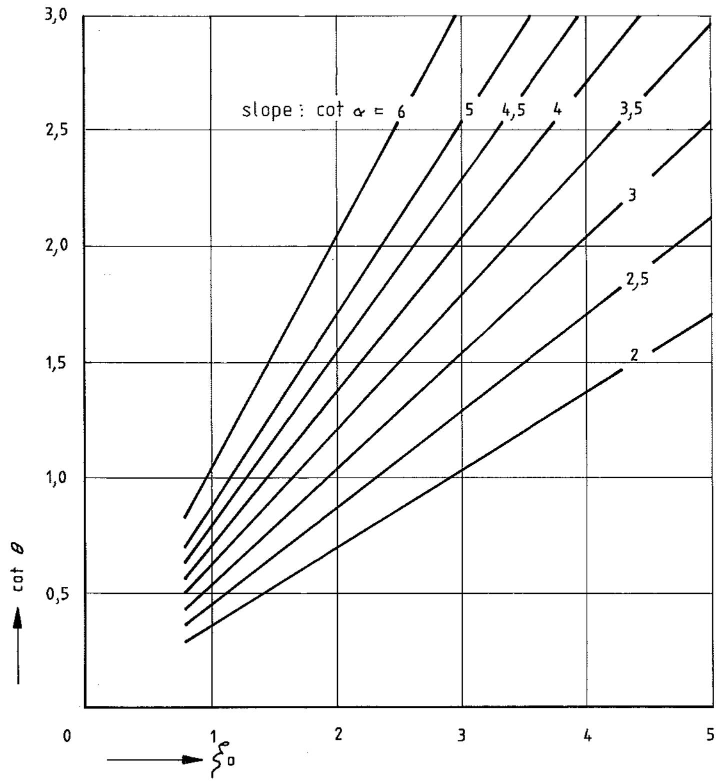


Fig. 123. Gradient of the pressure head front on the slope.

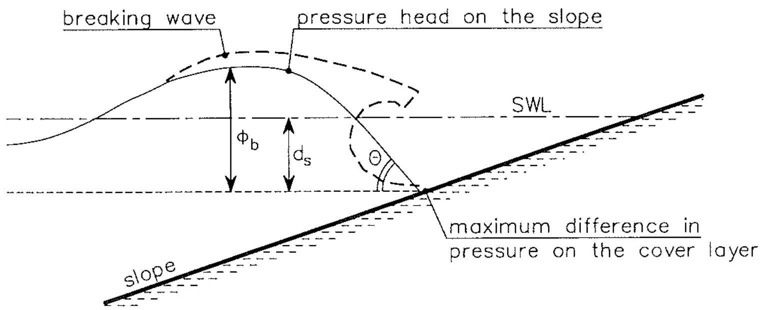


Fig. 124. The design pressure head on the slope (see also Figures 122, 123 and 128).

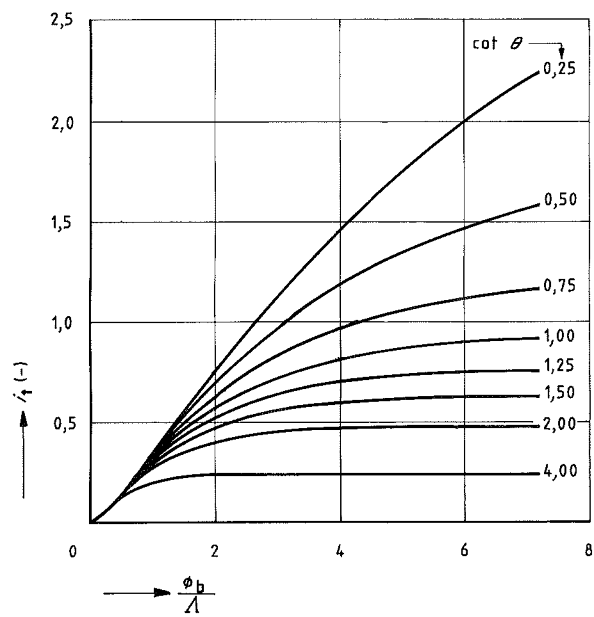


Fig. 125. Hydraulic gradient up the interface ($i_\uparrow$) (load).

When determining the critical hydraulic gradient on the interface below a geotextile at the onset of sand penetration into the filter reference should be made to the diagram in Figure 126. The diagram has to be used first to determine the critical hydraulic gradient for flow up the slope, $i_{cr\uparrow}$(the unbroken line in the third quadrant). The diagram is then used for the flow down the slope, $i_{cr\downarrow}$.

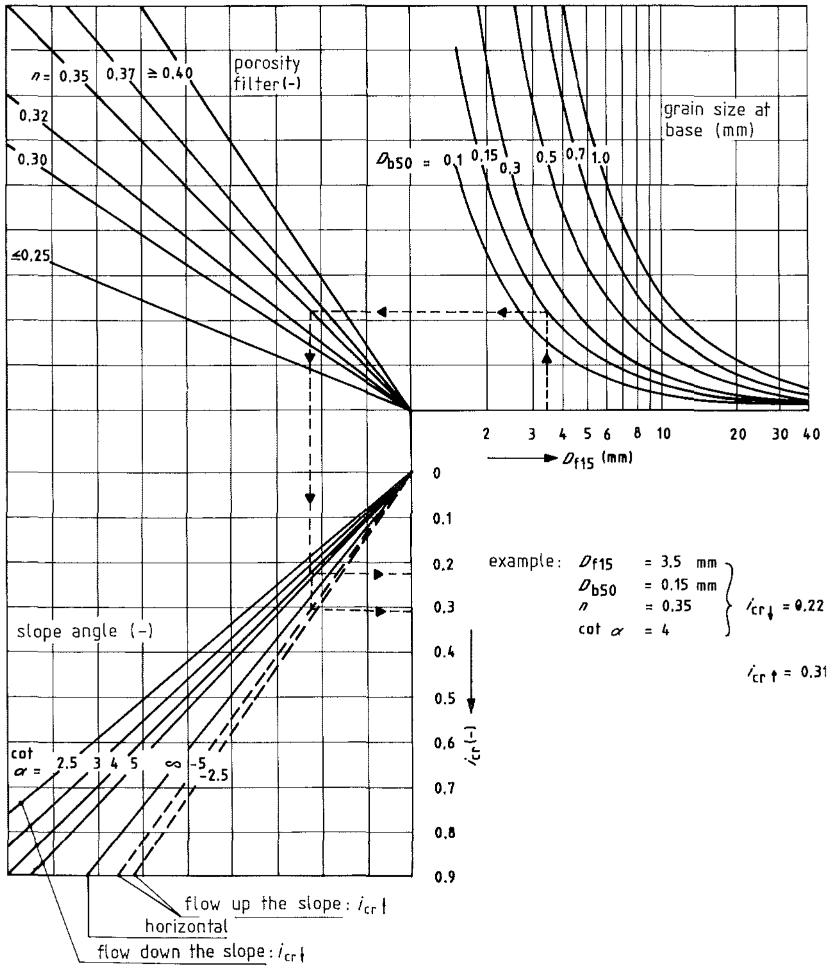
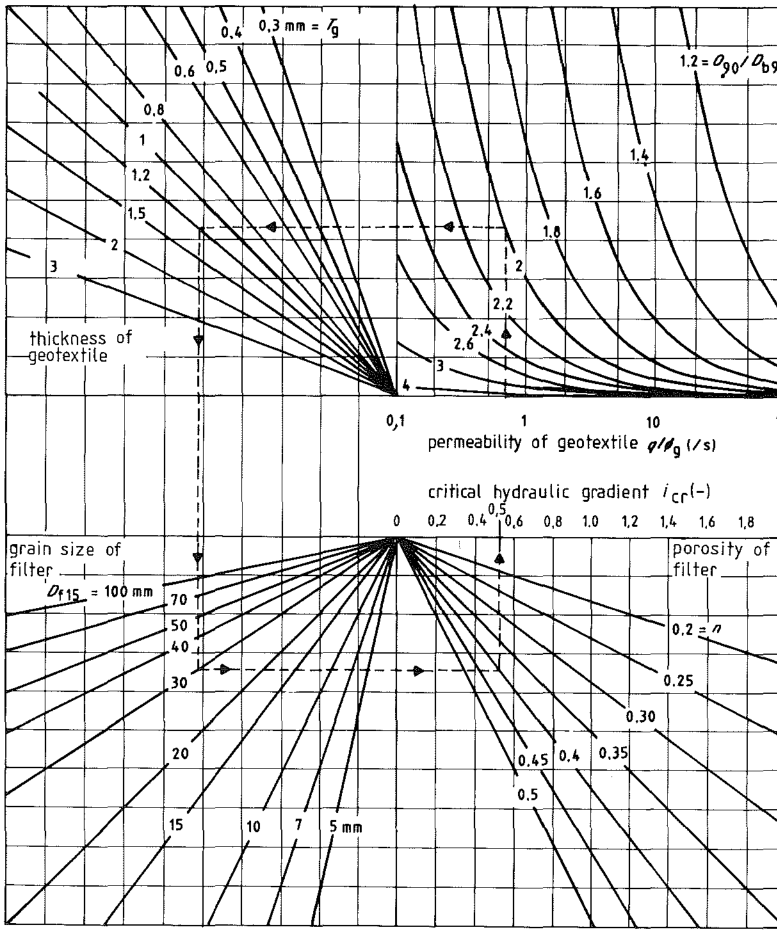


Fig. 126. Critical hydraulic gradient for an interface between a sand base and a granular filter.

Figure 127 must be used to determine i_{cr} for an interface where there is a geotextile. The distinction between upwards and downwards for this case has not yet been quantified. The figure gives a conservative value for i_{cr}. If $i_{cr\uparrow} < i_{\uparrow}$ or $i_{cr\downarrow} < i_{\downarrow}$ the interface is insufficiently stable and the structure must be adapted. After using Figure 121 the procedure follows the chart in Figure 116, which recommends that the design be adapted if sand is expected to penetrate the filter. The modifications which could be applied include the use of:

- a geotextile;
- a finer filter (D_{f15} smaller);
- a more gentle slope ($\tan\alpha$ smaller);
- etc.



example: permeability q/ϕ_g = 0.7 / s
O_{90}/D_{b90} = 2
thickness T_g = 1.2 mm
filter on geotextile: D_{f15} = 30 mm
n = 0.4

i_{cr} = 0.52

Fig. 127. Critical hydraulic gradient for an interface with a geotextile.

d. *Check on whether blocks will move if $H = H_s$*

If the interface between the filter and the base is sufficiently stable the calculation can proceed further using the chart in Figure 116 to check the stability of the cover layer. To do this it is necessary to know the level at which the maximum difference in pressure head will occur. This can be obtained using Figure 128.

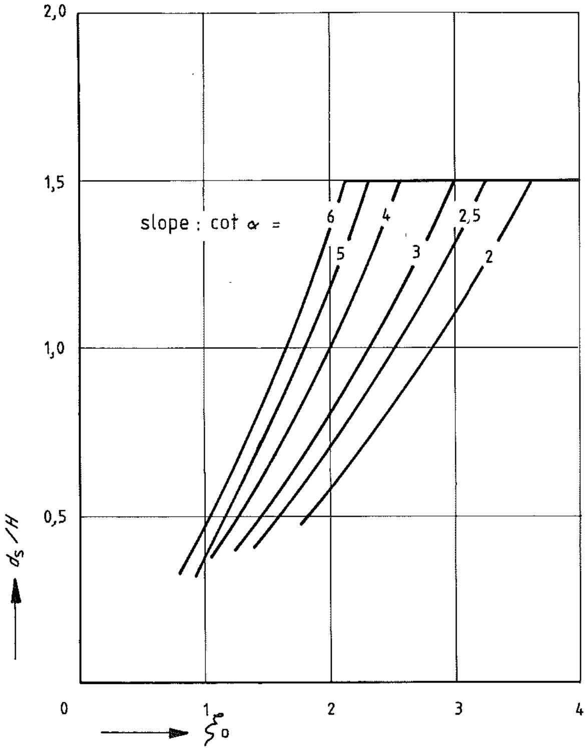


Fig. 128. Level at which the maximum difference in potential occurs.

For a transition structure the following requirements must be evaluated, (d_s is always positive but d_o can also be negative, see Figure 115):

– below the transition, down the slope

$$\frac{d_o - d_S}{\sqrt{(\phi_b \cdot \Lambda)}} > 1.4 \cdot \tan a \text{, and also} \tag{47}$$

– above the transition, up the slope

$$\frac{d_S - d_o}{\sqrt{(\phi_b \cdot \Lambda)}} > 0.1 \tag{48}$$

If both requirements are satisfied the transition structure is unlikely to affect block stability and $\Gamma_0 = 1$. If one or both requirements are not satisfied the value of Γ_0 must be determined using Appendix G.
The factor for the friction between blocks, Γ_1, see Figure 129, and the influence factor, Γ, must be determined using:

$$\Gamma = \Gamma_1/\Gamma_0 \tag{49}$$

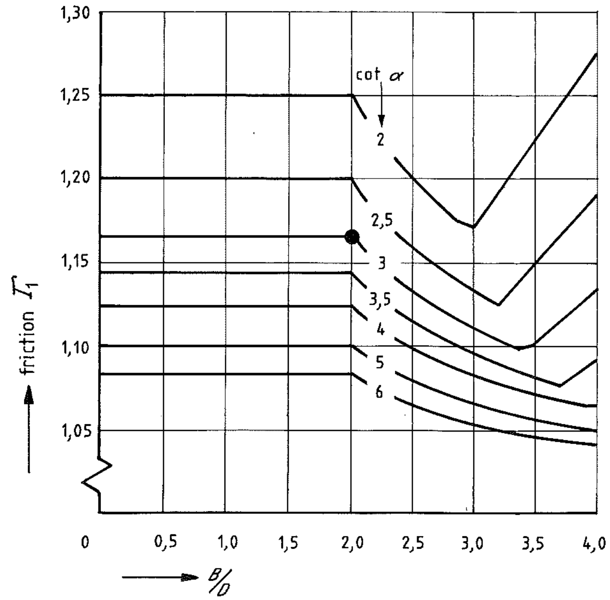


Fig. 129. Influence factor Γ_1 for the friction on a loose stone.

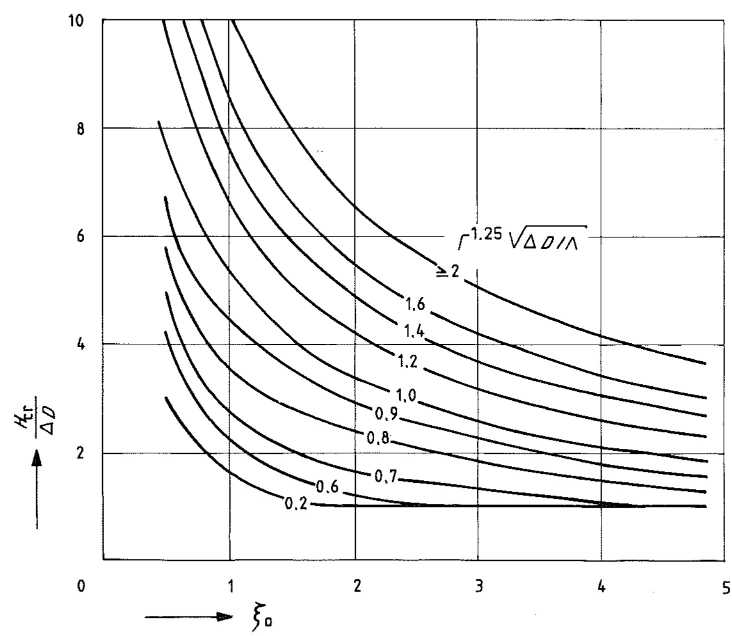


Fig. 130. Critical wave height for lifting a loose block – H_{cr} as a function of ξ_o and $\Gamma^{1.25} \cdot \sqrt{\Delta D/\Lambda}$.

The next step in the design is to determine the critical wave height using Figure 130. If the critical wave height is larger than the loading (H_s) the stability should be checked at a lower or higher water level (and/or lower wave height). This is particularly important if $\Gamma_0 \neq 1$ or if the maximum loading, see Figure 131, occurs on the slope above or below the transition structure, see Figure 131.

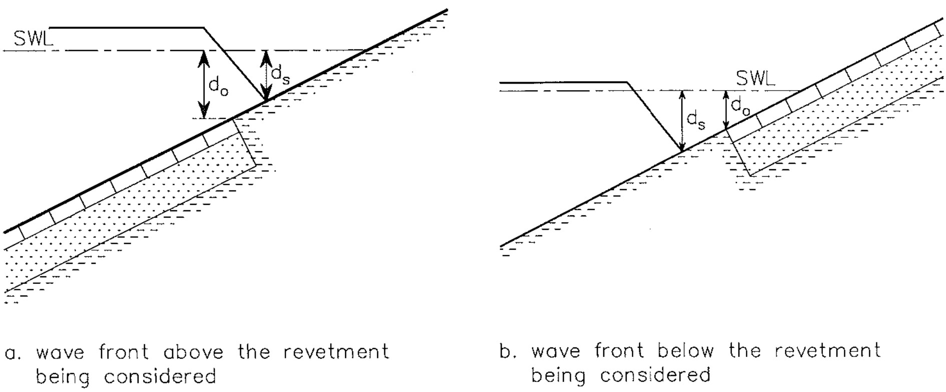


Fig. 131. The part of the slope subjected to the highest load may be above or below the revetment.

e. *Check on block movement if $H = 1.4 \cdot H_s$ is less than $0.1 \cdot D$*

The next step in the design is to determine if block movement under very high waves ($H_{2\%}$) remains less than 10% of the thickness of the cover layer. This step is only necessary if the influence factor for flow (Γ_3) is less than 0.2, see Figure 133. We proceed to step f (the geotechnical stability) if Γ_3>0.2. If this is not the case the design wave height must be determined using Figure 116:

$$H = 1.4 \cdot H_s \text{ up to a maximum of } H = 0.6 \cdot d \tag{50}$$

where

d = design water depth, see Figure 49 [m]

The parameters ϕ_b, d_s, Γ_0 and Γ_1 must then be redetermined, using higher wave heights, see Figures 122, 128 and 129 and Equations (47) and (48). If the transition structure is important then a new value for $\cot\theta$ should be found using Figure 123. Figure 132 must then be used to find the value of Γ_2, the influence factor for mass inertia. With this value and that already found for Γ_1, see Figure 133, a new value of Γ can be calculated:

$$\Gamma = (\Gamma_1 + \Gamma_2 + \Gamma_3)/\Gamma_0 \tag{51}$$

H_{cr} can then be found using Figure 130. If the design wave height ($1.4 \cdot H_s$ or $0.6 \cdot d$) is smaller than the new value of H_{cr}, the cover layer is sufficiently stable. All that remains is to check if this is also true for slightly lower wave heights and/or water levels.

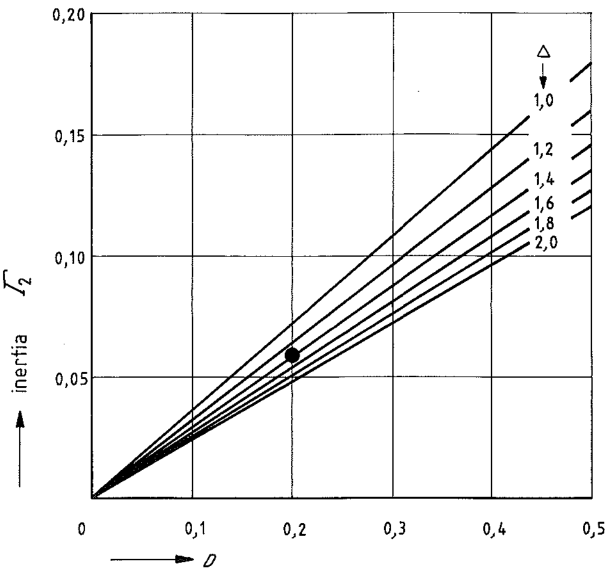


Fig. 132. The influence factor for the inertia of mass, Γ_2, if block movement is permissible.

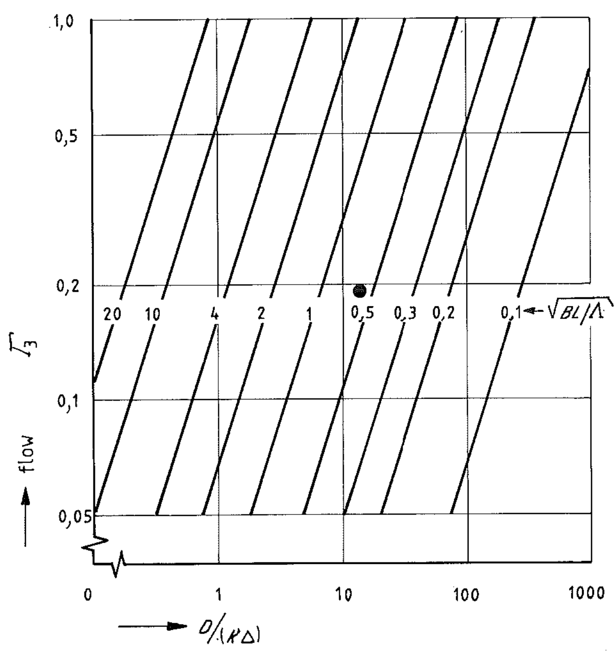


Fig. 133. The influence factor for flow, Γ_3, if block movement is permissible.

f. *Geotechnical stability*
   Finally the geotechnical stability must be evaluated, see Chapter 9, with particular reference to Figures 137 to 140.

It should be noted, that if a structure is shown to be stable by using the Analytical Design Method, it is unnecessary to check it with the Preliminary Design Method, since the Analytical Design Method evaluation is more accurate. Checks using STEENZET/1+ however are recommended if:

$$\frac{H_s}{\Lambda \cdot (\tan a)^{1.5}} > 70 \cdot \xi_{op}^{-2} \qquad (52)$$

In this case the Analytical Design Method is less accurate. The curves in Figure 130 have not been established definitively for combinations of H_s, Λ, $\tan\alpha$ and ξ_{op} which satisfy Equation (52). The curves are based on conservative (safe) assumptions. Further investigations are required to prove if the stability in this case is higher than that given in Figure 130.
The Analytical Design Method is also available in the form of the user-friendly ANAMOS PC program [De Waal, 1990a].

8.5 Design method for ship waves

The characteristics for primary and secondary ship waves near to the bank determine the stability of pitched dike revetments subjected to loads due to ship-induced water movements. The failure mechanisms make no distinction between wind waves and ship waves.
The external loads can be characterised by the relevant wave height and period, in this case, the wave height H being replaced, see [Recommendations for the design of banks, 1989], by:

- front (bow) wave: $H = \hat{h}_f$ and $\xi_{op} = 1$;
- stern wave: $H = z_{max}$ and $\xi_{op} = 1$;
- secondary ship waves: $H = H_i \cdot \cos\beta$ and $T = T_i$ where $\beta = 55°$ when the ship passes parallel to the bank.

The values of $\hat{h}_f$, z_{max} and H_i are determined using Figure 54, see Section 6.3.
The stability of the revetment can then be checked using the flow chart given in Figure 116. The calculated load due to secondary ship waves, H, takes into account the fact that the waves attack at an angle (55°). The procedure described above can also be used to evaluate designs made using the Preliminary Design Method, discussed in Section 8.4.2.

8.6 Design method for crest revetments

8.6.1 *Design for loads due to overtopping flow*

The loads on, for example, groynes or low dikes due to overtopping can be characterised using the current velocity on the crest. The size of these currents are discussed in Section 6.4. The critical current velocity on the crest, u_{cr}, can be calculated using the following conservative formula:

$$u_{cr} = 1.2 \cdot \sqrt{g \Delta D} \tag{53}$$

where

u_{cr} = current velocity on the crest at the onset of damage [m/s]
g = acceleration due to gravity [≈ 9.8 m/s^2]
Δ = relative volumetric mass of the blocks [–]
D = thickness of the cover layer [m]

This formula is based on the results of research carried out by Knauss (rubble and pitching on a spillway channel), Isbash (rubble on an outlet channel) and recent model investigations for the 'Afsluit Dike' in the Netherlands, see Appendix E.

8.6.2 *Design for wave loads*

The equivalent wave height at the crest, as defined in Section 6.5, is used when designing the crest of a dike against wave action. The criterion which the revetment must satisfy is:

$$\frac{z_{2\%} - h_c}{\Delta D} \leq 3 \tag{54}$$

where:

$z_{2\%}$ = wave run-up level relative to SWL assuming a dike of unlimited crest height, see Figure 61 [m]
h_c = crest height relative to SWL [m]

The criterion is based on the results of model investigations by [Van Kruiningen, 1989], see Appendix F. In view of the limited information available about this structure a safety factor is included in the equation.

8.7 Other design aspects

8.7.1 *Loads on toe and anchor structures*

Under heavy wave action there is always the tendency for a cover layer of pitched blocks to slide downwards over the sublayer [BEZUIJEN ea., 1990, page 277; MEIJER, 1991]. The driving force behind this is the weight component of the blocks down the slope (F_a), see Figure 134. Because of the difference in pressure on the cover layer, immediately before wave impact, a number of blocks will have reduced contact with the sublayer (F_n), resulting in lower friction. If the force of friction is less than the component of the weight down the slope, a part or all of the cover layer must be supported against sliding by the toe structure (if it comprises paved or interlocking blocks) or by an anchoring structure, if block mattresses are involved.
If the weight component down the slope is larger than the friction on only a small part of the slope, the friction between the blocks outside the wave zone can compensate for the effective loss in weight. There are then no loads on the toe or anchor structure. A procedure is given in [BAKKER en MEIJERS, 1988] for calculating the load on toe or anchor structures for pitching or block mattresses on a granular filter. Although this is an analytical solution it is fairly complicated. Simplified equations are given in [MEIJERS, 1991] which can also be used for revetments without a granular filter. The factors which have the most influence on the forces are also presented in the latter report.

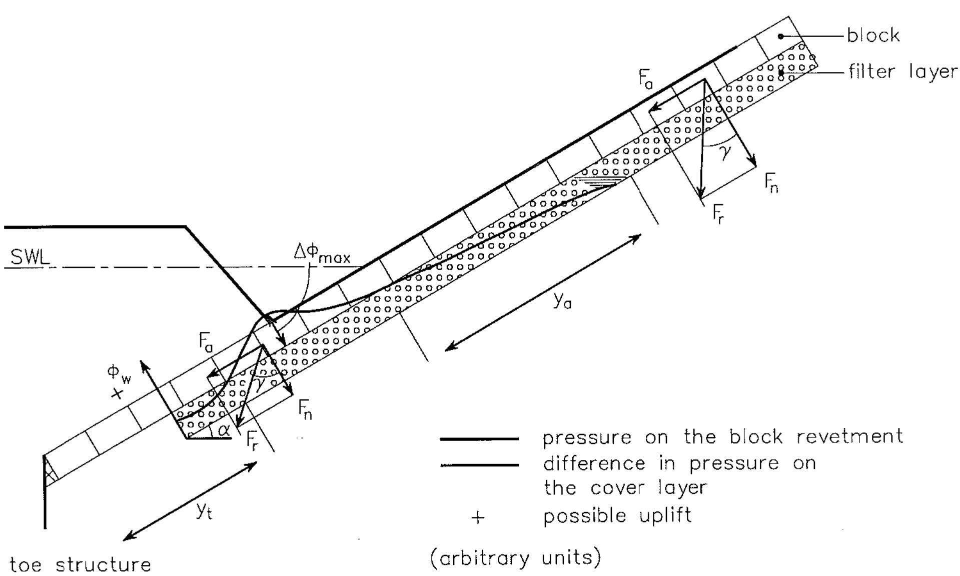


Fig. 134. Forces which may cause the revetment to slide.

A further simplification is presented in [BEZUIJEN et al.,1990, page 140], which is based on the conservative (safe) assumption that, between SWL and SWL–H_s, there is no friction between the cover layer and the sublayer. The sliding force in this area must thus be absorbed by the toe or anchor structure and/or the friction between blocks adjacent to the area.
This assumption gives conservative results provided that one of the following requirements holds:

- $\xi_{op} < 2$, or
- $H_s > \Lambda \cdot \sin\alpha$.

If neither of these requirements is satisfied H_s must be replaced by $\Lambda \cdot \sin\alpha$ (the leakage height, λ) in the following equations. The sliding force on blocks between SWL and SWL–H_s, the section in which sliding will occur, is given by:

$$F_a = \rho_b g H_S D \tag{55}$$

where:

F_a = sliding force per metre length of dike [N/m]

For the friction on a cover layer, outside the area which will slide, a distinction has to be made between paved and interlocking blocks and block mattresses.

Paved and interlocking blocks (toe structure)

The maximum force of friction between blocks below the part of the revetment which will slide (H_s below SWL) and above the toe structure is determined by the submerged weight of the blocks, $D \cdot (\rho_b - \rho)$, the slope (α), the angle of friction (friction coefficient, $\tan\Phi$) and the length of this part of the slope, y_t-H_s/$\sin\alpha$:

$$F_w = D \cdot (\rho_b - \rho)g \cdot (\cos\alpha \cdot \tan\Phi - \sin\alpha) \cdot (y_t - H_S/\sin\alpha) \tag{56}$$

Block mattresses (anchor structures)

The maximum force of friction between blocks in the section above that which will slide (SWL) and below the anchor structure is determined by the dry weight of the blocks, $D \cdot \rho_b$, the slope, α, the angle of friction (friction coefficient, $\tan\Phi$) and the length of this part of the slope, y_a

$$F_w = D \cdot \rho_b g\ (\cos\alpha \cdot \tan\Phi - \sin\alpha)y_a \tag{57}$$

where:

F_w = sliding force which can be absorbed by the zone which is not under attack, per metre length of dike [N/m]

Φ = angle of friction between the blocks and the layer directly underneath [°]. Recommended value for Φ: $\frac{2}{3}$ of the angle of friction between two layers of concrete blocks ($\Phi \approx 20°$). If a geotextile is laid between the blocks and the sublayer the value will be lower. When no data are available a value of $\Phi = 15°$ is recommended.

y_t = distance, measured down the slope, from SWL to the (lowest) toe structure [m].

y_a = distance, measured up the slope, from SWL to the highest block of the block mattress (anchor) [m].

y_t and y_a should be measured along the profile of the revetment to ensure, for example, that berms are taken fully into account. Using Equations (52) and (56) the force on a toe or anchor structure can be calculated:

$$F = F_a - F_w \qquad (58)$$

where:

F = Force on the toe or anchor structure per metre length of dike (if $F_w > F_a$, the structure is not loaded) [N/m]

Worked example for a toe structure (paved block revetments):

- *Design and boundary conditions*

Significant wave height:	$H_S = 1.5$ m
Thickness of the cover layer:	$D = 0.35$ m
Volumetric mass of blocks:	$\rho_b = 2350$ kg/m^3
Slope:	1:4 ($\sin\alpha = 0.24$)
Distance between SWL and toe:	$y_t = 14$ m
Angle of friction:	20° (blocks on a granular filter)

- *Calculation*

$F_a = \rho_b g H_S D = 2350 \cdot 9.8 \cdot 1.5 \cdot 0.35 = 12.1$ kN/m

$F_w = \mathrm{D} \cdot (\rho_b - \rho) g(\cos\alpha \cdot \tan\Phi - \sin\alpha)(y_t - H_S/\sin\alpha)$

$= 0.35 \cdot (2350 - 1030) \cdot 9.8 \cdot (0.97 \cdot 0.36 - 0.24) \cdot (14\text{-}1.5/0.24) = 3.8$ kN/m

- *Conclusion*

The force on the toe structure, $F_a - F_w = 8.3$ kN/m

From the first estimate it appears that a row of wooden piles, (three piles per m, with a pile length of 1.5 m in sand with an angle of friction of 30°), can absorb double this horizontal force. These wooden piles, applied in clay, which is not too weak, (undrained shear strength of at least 20 kN/m^2), could also absorb this force satisfactorily.
The forces calculated above apply to the design conditions. Higher loads may occur however during construction, particularly at the toe. These forces are difficult to quantify because they depend on the way the blocks are being placed, see Section 10.4.

8.7.2 *Blocks laid on clay*

The Delta Flume tests, carried out for the design of the "Oester Dam" in the Eastern Scheldt, (see Figure 105) indicated that a revetment of blocks laid on clay is more stable than one of blocks laid on a granular sublayer. Although the mechanisms which can lead to failure are not understood completely, it appears that the build up of pressure under the revetment is increased by the impermeable nature of the clay. The effect of the reduced through flow, see Section 8.3, is very clear. The blocks become, as it were, sucked on to the clay. In order to take advantage of this increased stability the clay must be specified very strictly [Bezuijen et al., 1990, page 212]. Similarly the construction and inspection procedures must also be carefully specified. Although many of these aspects are discussed elsewhere in the manual, a short description is given below in order to complete this summary. It is very important for the stability of a block revetment that there should be very little if any flow of water under the blocks. Erosion channels should therefore not be allowed to form. A number of requirements must be satisfied to prevent this occurring during construction and during the service life of the structures. If some of the following requirements cannot be met the possibility of protecting the clay surface with a geotextile should be considered. A multi-layer geotextile can be used locally to fill up irregularities. The stability is then however lower than when the blocks are placed directly on good quality clay, see Figures 105 and 110.

Clay quality

A high resistance to erosion is essential. The following provisional criteria, all of which must be satisfied, are given in Section 5.8:

- organic content: H_k < 5 % and
- $CaCO_3$ content: < 25 % and
- sand content: Z_k < 40 % and
- Liquid Limit: W_l > 45 % and
- Plasticity Index: I_p > 0.73 $(W_l - 20)$

Areas of application

The structure should not be subject to daily hydraulic loads, that is, the structure can only be used above the tidal zone.

Execution (see Section 5.8)

- Water content during construction:
  - maximum, $W_l - 0.75 \cdot I_p$;
  - minimum, the optimum water content of the standard Proctor Test (W_{opt}).
- Placing and compaction, for example by bulldozer, must be in layers of a maximum thickness of 0.4 m.

- Compaction by bulldozer. The pressure on the clay during compacting must not be too high.
- The clay must be smoothed out to ensure that there is a good connection with the blocks.
- The surface of the clay must not be made smooth using a thin layer of sand.
- A thin layer of loose damp clay, a few centimetres thick, can be scattered onto the clay for smoothing out after compaction. The slope can then be prepared along the dike and, if necessary, sprayed with water. This layer should be as thin as possible (1 to 2 cm) because it can only be compacted by the blocks themselves as they are laid.
- Care should be taken to ensure that frozen clay, clay with sand lenses and contaminated clay are not used in the construction.
- The width of joints between blocks must be as small as possible to minimise the hydraulic loads on the clay during wave attack.

Inspection

In order to check the quality of the clay which has been used it is recommended that, from time to time, the clay surface is inspected by removing a few blocks. Erosion channels can greatly reduce the stability of the structure during wave attack and can form an important contribution to the ageing process. channels which run under several blocks are of particular concern.

8.7.3 *Penetration of sand and silt from the foreshore*

The design procedures given above are particularly suitable for relatively new structures. For these structures the permeability of the filter and the cover layer is known with reasonable certainty. In the course of time however ageing processes affect the permeability. Basic investigations were carried out by [STOUTJESDIJK, 1991] into one of the possible processes, namely the penetration of sand or silt into the structure from the surrounding area, for example, the foreshore.
From the investigations it appears that the pores of the filter and the joints in the cover layer can be completely filled with silt or sand. Sanding up of the cover layer only has not been established. The permeability of both layers is therefore reduced. The conclusion is that this does not have a negative effect on the stability. Since the ageing processes are not generally understood and/or have not been quantified, this conclusion should be treated with care. Further investigations into this subject are in progress.

CHAPTER 9

GEOTECHNICAL ASPECTS OF REVETMENT DESIGN

9.1 Introduction

A dike revetment can only be stable if the ground underneath is stable. Guidelines are given therefore in this chapter for determining the stability of the ground. The stability of the ground under a revetment is closely related to the stability of the complete body of the dike. The geotechnical stability of the body of the dike is not considered in the present manual and, for this, reference should be made to the [Lower Rivers Manual, 1989].
This chapter only considers the geotechnical stability of the material under the revetment to the extent to which it is influenced by wave loads [BEZUIJEN et al., 1990, page 144]. The assumption, therefore, is that the dike and the revetment are geotechnically stable when they are not being subjected to wave loads. Sliding planes can develop in the material directly under the revetment, as shown in Figure 135. The sliding planes which can develop within the revetment itself, for example, between the cover layer and the filter, are not shown in the figure; this aspect is discussed in Section 8.7.1.
The problems related to the geotechnical stability of revetments are fairly extensive, see, for example, [DE GROOT et al., 1990] and [VAN DER GRAAF en DE GROOT, 1991]; only a brief description is possible within the scope of the present manual. Information on geotechnical aspects is important to the designer, since a change in design, which leads, for example, to increased stability of the cover layer, could, however lead to a reduction in geotechnical stability.
Two loading situations can be distinguished:

- the moment of maximum wave run-down when pressures on the slope are at a minimum;
- the moment of wave impact, when local high pressure peaks occur.

At maximum wave run-down local sliding planes can develop in the filter layers or directly below the cover layer. These can also develop during wave impact. Possible sliding planes are shown in Figure 135. A failure situation with a slip circle is shown schematically in Figure 91. These sliding planes lead to S-profiles or a total loss of stability. In addition the sand base may be affected by wave impact. The various loading situations and failure mechanisms are drawn together in Table 10 and reference made, per loading combination, to the section in the manual where they are treated.

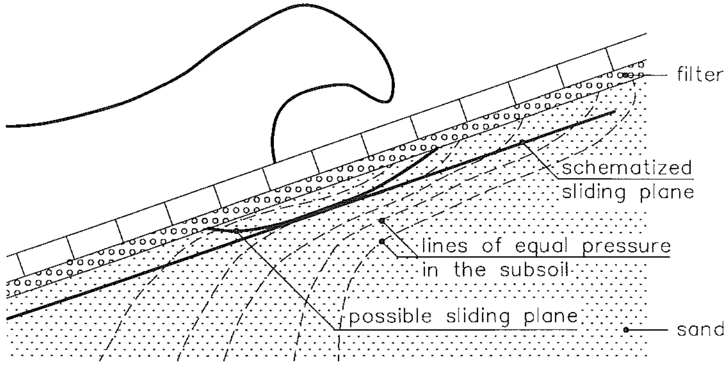


Fig. 135. Possible sliding planes and schematization for computations.

Table 10. Possible combinations of loading situations and failure mechanisms.

	Wave run-down	Wave impact
Sliding	Section 9.2	Section 9.3
Weakening	–	Section 9.3

9.2 Local sliding caused by maximum wave run-down

It is stated in Chapter 8 that the maximum load on the cover layer occurs immediately before wave impact when there is a steep wave front on the slope. This is also a dangerous situation in connection with geotechnical stability because the pressure head, on part of the slope is low, see Figure 87, and the pressure in the revetment itself cannot simultaneously adapt to this rapid reduction. As a result an upwards pressure difference develops on the cover layer, endangering its stability.
With clay subsoil, a fall in water level does not usually create a problem since clay gets its stability from its cohesion. For cohesive materials the stability is always governed by the deeper sliding planes. With such materials, the strength is often independent of depth. The driving moment, which causes the sliding, however increases with depth.
Shallow sliding planes resulting from wave run-down are therefore not anticipated. With sand subsoil the stability is only affected by shallow sliding surfaces and the effect of low pressure heads on the pitching caused by wave action is then very important. Only the stability of sandy subsoil is therefore considered below [Bezuijen et al., 1990, page 144]. Stability against sliding can be determined using a slip circle calculation. A simplified method of calculation has been developed for obtaining first impression of the stability. [Bezuijen et al., 1990]. The method gives equations for assessing the geotechnical stability of a structure when the run-down is critical.

If this is the case the design has to be adapted or additional numerical calculations applied. The following assumptions are made in the calculation:

- From numerical calculations [BEZUIJEN et al., 1988, 1990a and 1990b] it appears that, during wave attack, the flow in the subsoil is generally perpendicular to the slope. Fluctuations in pressure on the slope are assumed to be a maximum at the point where the maximum wave run-down is expected. The position of this point therefore is important to the design.
- Sliding planes are assumed to be straight. The actual situation and the assumptions are shown schematically in Figure 135.
- The friction between the cover layer and the subsoil is assumed to be negligible. Equations in which friction is taken into account are developed in [MEIJERS, 1991].

The movement of water in the subsoil (under the filter layer) cannot be calculated using the methods described in Chapter 8. There quasi-stationary flow is assumed (the flow at a particular time is only determined by the boundary conditions at that time and not by earlier boundary conditions). Generally in relatively impermeable subsoil the groundwater flow is also determined by elastic storage (the compressibility of the water and the air it contains). In a one-dimensional situation with a homogeneous subsoil and sinusoidal wave loading the solution for the variation in pressure in the subsoil [DE GROOT et al., 1988] is:

$$\phi(z,t) = \phi_A \exp\left(-\frac{z'}{L_{es}}\sqrt{\pi}\right) \cdot \cos\left(2\pi\frac{t}{T} + \frac{z'}{L_{es}}\sqrt{\pi}\right) \tag{59}$$

where:

$\phi(z,t)$ = pressure head in the subsoil at location z' at time t [m]
ϕ_A = amplitude of the pressure head in the filter layer immediately above the subsoil (for a continuous cover layer this is almost the same as the amplitude of the pressure head on the cover layer, see Figure 136) [m]
z' = co-ordinate, see Figure 136 [m]
T = wave period [s]
L_{es} = consolidation length, see Equation (60) [m]

The consolidation length is dependent on the permeability of the sand (k), the porosity of the sand (n_z), the compressibility of the water (W_c) and the wave period (T or T_p).

$$L_{es} = \sqrt{\frac{Tk}{\rho g n_z W_c}} \tag{60}$$

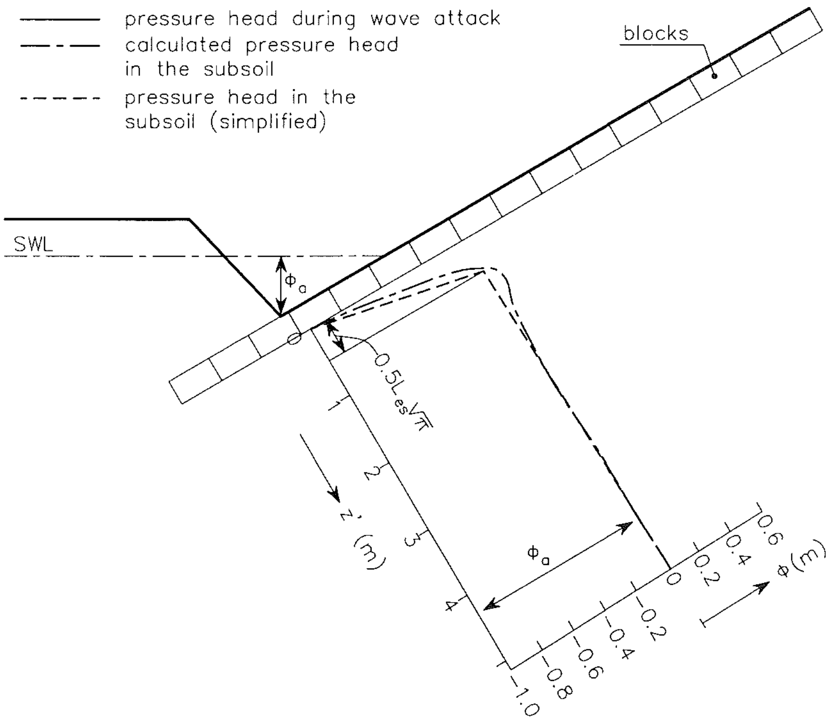


Fig. 136. Variation in pressure head in the subsoil according to Equation (59) with an air content of 5% and a permeability of 10^{-4} m/s.

where:

k = permeability of the subsoil (sand, see Table 6) [m/s]
W_c = compressibility of pore water containing air, see Equation (61) [m²/N]
ρ = volumetric mass of water [kg/m³]
n_z = porosity of the sand [–]

The compressibility of water with air can be approximated using the following equation [VERRUIJT, 1969]:

$$W_c = 5 \cdot 10^{-10} + W_L \cdot 10^{-5} \tag{61}$$

where:

W_L = air content of the water (0.05 to 0.10, see text) [–]

When the air content is greater than 0.1% the first term can be neglected, then:

$$W_c = L_g \cdot 10^{-5} \tag{62}$$

Although Equation (59) has been developed for horizontal homogeneous beds under water infinite half plane space it has been used here as an approximation for a flow situation on a slope. An example of the pressure head calculated using the equation is given in Figure 136 for an air content of 5% in the pore water ($W_L = 0.05$). The value of 5% may seem high but model investigations indicate that a value of 5 to 10% is very common under block pitching. Because of this the form of the grain skeleton can be neglected. Because of the high air content the grain skeleton is very stiff (rigid) compared with the pore water. This is in contrast to the assumption often made in consolidation calculations that water is incompressible. For dike revetments in the tidal zone the air content, for design purposes, can be even greater. In these areas very high wave loads can often occur with extreme high water levels and, as a result, the subsoil may not be completely saturated. A higher air content leads to a reduction in geotechnical stability.

The possible damping effect of a filter layer under the blocks is neglected in Equation (59). If a filter layer is applied the amplitude of the pressure head must be taken for ϕ_A. As shown in Figure 136 the results of Equation (59) can be simplified further without any great loss in accuracy. With this approximation of pressure head levels in the subsoil an equation can be developed for the minimum grain pressure in the subsoil needed to prevent geotechnical instability during wave run-down. Re-writing the relationship given in [Bezuijen et al., 1990], which describes the minimum angle of friction required, gives:

$$\sigma_b = \rho g \left\{ \frac{\phi_a}{\cos\alpha} - 0.5 \cdot (1 - n_z) \cdot \Delta_z L_{es} \left(1 - \frac{\tan\alpha}{\tan\Phi} \right) \sqrt{\pi} \right\} \tag{63}$$

where:

- Φ = internal angle of friction of the subsoil (minimum of 30° for loose, uncompacted sand, 35° to 40° for compacted sand) [°]
- $\sigma_b = (\rho_b - \rho)gD + (1 - n)(\rho_f - \rho)gb$ = the grain pressure needed for a stable structure, created by the weight of the blocks and the filter layer under water [N/m³]
- ρ_b = volumetric mass of the (concrete) blocks [kg/m³]
- α = slope angle [°]
- Δ_z = relative volumetric mass of the sand grains = $(\rho_z - \rho)/\rho$ [–]
- ρ_z = volumetric mass of the sand grains [kg/m³]
- Δ_f = relative volumetric mass of the filter grains = $(\rho_f - \rho)/\rho$ [–]
- ρ_f = volumetric mass of the filter grains [kg/m³]
- n = porosity of the filter [–]
- n_z = porosity of the sand [–]

It must be emphasized that this equation is *only valid* for toe or anchor structures which are sufficiently stable, see Section 8.7.1. By setting $\cos\alpha = 1$ the equation can be used to obtain minimum thickness of the revetment, that is, the thickness of the filter layer (b) plus the thickness of the cover layer (D):

$$\Delta D + (1-n)\,\Delta_f b = d_s - 2.8 \cdot (1-n_z)\,\Delta_z \left(1 - \frac{\tan\alpha}{\tan\Phi}\right) \cdot \sqrt{(T_p k)/(n_z W_L)} \qquad (64)$$

where:

d_s = minimum pressure head on the slope immediately before wave impact, see Figure 128. [m]
k = permeability of the sand, see Table 6 [m/s]

This equation is plotted in Figures 137 to 140. Because of the large number of variables in the equations, the following assumptions had to be made in order to produce the figures (corresponding to the "visual" situation:

- porosity of the sand: $n_z = 0.4$;
- relative volumetric mass of sand grains: $\Delta_z = 1.65$;
- air content of the sand: $W_L = 0.1$;
- mass of the filter per unit volume, including pores: $(1-n)\,\Delta_f = 1$;
- well-compacted sand : $\Phi = 40°$.

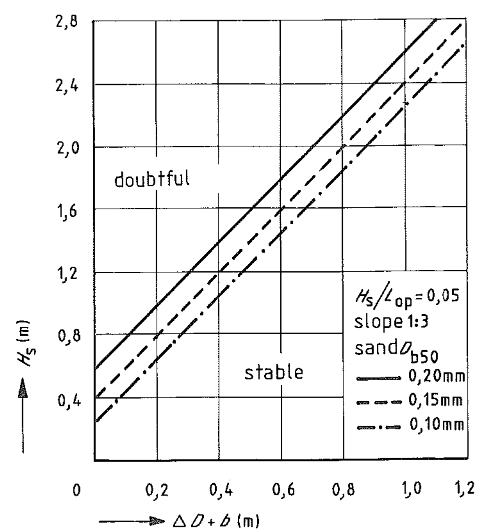


Fig. 137. Minimum thickness of cover and filter layers required for $H_s/L_{op} = 0.05$ and $\tan\alpha = 0.33$.

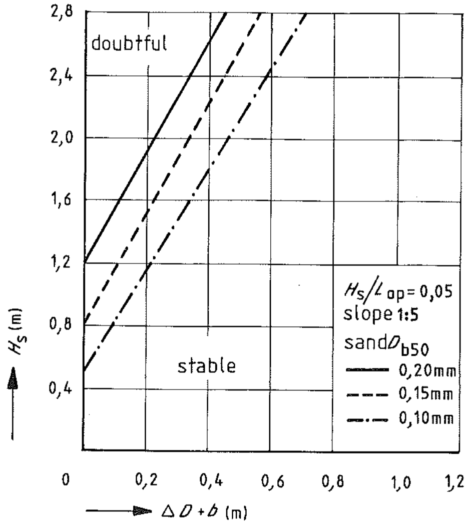


Fig. 138. Minimum thickness of cover and filter layers required for $H_s/L_{op} = 0.05$ and $\tan\alpha = 0.20$.

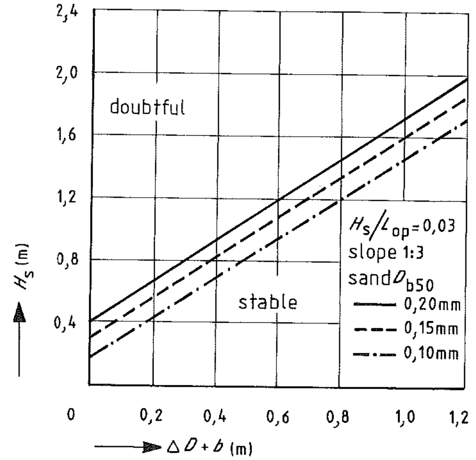


Fig. 139. Minimum thickness of cover and filter layers required for $H_s/L_{op} = 0.03$ and $\tan\alpha = 0.33$.

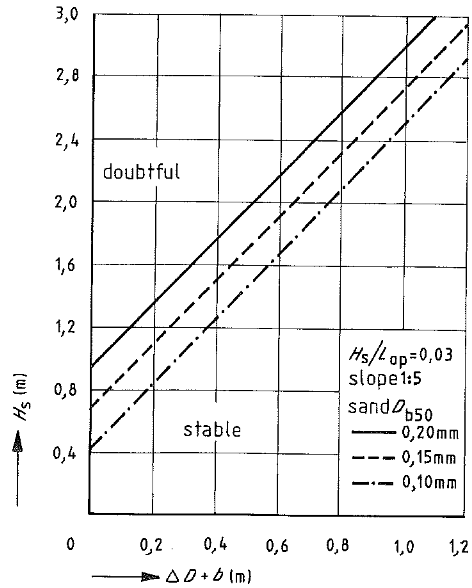


Fig. 140. Minimum thickness of cover and filter layer required for $H_s/L_{op} = 0.03$ and $\tan\alpha = 0.20$.

In other situations, the equations should be used to assess geotechnical stability. The calculation method presented below should be seen as a conservative approximation and if a pitching satisfies this requirement it will certainly be geotechnically stable. If it barely satisfies the requirement then it is recommended that a more accurate two-dimensional numerical calculation is carried out.

Calculation example

- *Initial design and boundary conditions*

Wave height:	$H_s = 1.5$ m
Wave period:	$T_p = 4.4$ s
Slope:	1:3
Well-compacted sand with	$D_{b50} = 0.2$ mm
cover layer on a geotextile on sand (thus b = o)	
thickness of cover layer:	$D = 0.35$ m
Relative volumetric mass of the blocks:	$\Delta = 1.4$

- *Calculation*

  Wave steepness: $H_S/L_{op} = \dfrac{H_S}{1.56 \cdot T_p^2} = \dfrac{1.5}{1.56 \cdot 4.4^2} = 0.05$

  $\Delta D = 1.4 \cdot 0.35 = 0.49$ m

- *Conclusion*

  In Figure 137 it can be seen that for $H_s = 1.5$ m and $D_{b50} = 0.2$ mm the minimum ΔD value required is 0.46 m. In the design $\Delta D = 0.49$ m and it can therefore be concluded that there will be no local sliding down the slope.

When assessing the geotechnical stability the air content of the water is an uncertain factor. Model investigations show that it is rarely higher than 10%. It should however be noted that the number of investigations in which the air content has actually been measured is limited. Therefore the figures in the manual are based on 10 % and it is recommended that this value is also used in the calculations. Only when the design water level applies for a long period of time, for example, on canal banks a lower air content, 0.02 to 0.05, should be applied.
From the figures it appears that a thicker granular filter gives a greater geotechnical stability. This is in contrast however to the tendency for cover layer stability (the lifting out of a block, see Chapter 8) where it appears that the thicker the filter layer the greater the loads on the blocks, that is, pressure difference on the cover layer. The conclusion is therefore that the wave loads must be distributed between the sand (shear stress) and the cover layer (pressure head difference on the blocks. Too much emphasis on one failure mechanism can divert attention elsewhere and possibly lead to failure due to a completely different mechanism.
The equations assume that only one sliding plane can develop and that the sliding of the cover layer on the filter or the geotextile can be prevented by a good toe structure or anchorage (using a block mattress). Section 8.7.1 describes how to calculate the force on a toe structure.

If there is not a good toe structure and the shear force (the component of the weight parallel to the slope) on the blocks must be absorbed by the subsoil, then sliding can develop earlier, see also [BEZUIJEN et al., 1990].
A new and less conservative method for calculation of the thickness of cover is presented in [MEYERS, 1994].

9.3 Local sliding due to wave impact

A short period local peak in pressure develops on the slope during wave impact. For asphalt revetments this peak pressure is the design load for the revetment and the subsoil, see [Asphalt Manual, 1984]. For pitched revetments however the peak is generally too short to be damaging or to affect the stability of the cover layer, see Chapter 8. In this section the effect of the peak load on the subsoil is estimated [BEZUIJEN et al., 1990, page 147] and [VAN DER GRAAF and DE GROOT, 1991].

Blocks laid on a granular material

The possibility of blocks being pushed in to a slope by wave impact can be assessed using the method of [BRINCH HANSEN, 1970]. Consider a few rows of blocks subject to wave impact; the remaining blocks are not loaded, see Figure 141. Successive approximations can be made for this situation, (Brinch Hansen applies for a smooth subsoil and, in addition, the wave impact can be schematized into a strip load on a few blocks, not the case in practice). The calculation, however, is sufficiently accurate for investigating if large deformations are to be expected as a result of wave impact.
The effect of the angle of the slope can be taken into account by taking the load perpendicular to the slope.

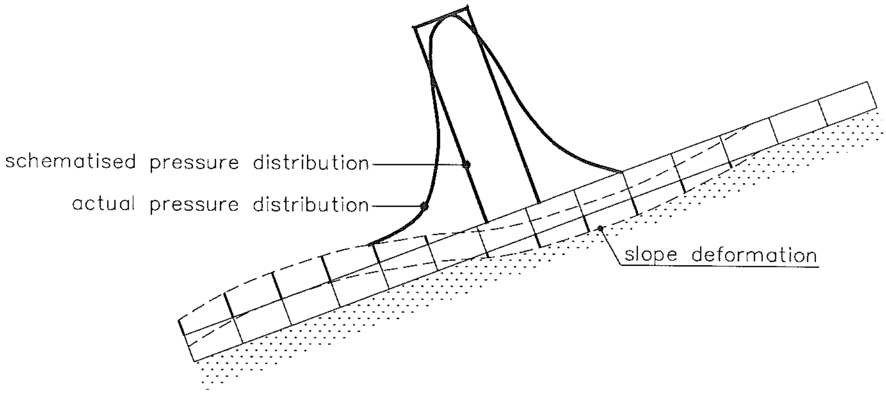


Fig. 141. Pressure distribution due to wave impact, actual and schematized, and the slope deformation which could result (S-profile).

The loading can be described using an equation for the design pressure head during the wave impact, (ϕ_{klap}), deduced from the study by [VAN VLEDDER, 1990]:

$$\phi_{\mathrm{klap}} = 12 \cdot H_s \tan\alpha \tag{65}$$

Using the assumption that $H_s/\Delta D \leq 6$ the calculations indicate that, for a 1:3 slope, this mechanism does not cause instability, provided that the internal angle of friction of the subsoil is greater than 38°. In practice this is the case when the subsoil is well compacted. Subsoil of well-compacted minestone or broken gravel has an internal angle of friction of more than 40°. In this case the mechanism will certainly not cause damage. The safety factor in the calculation is already higher than 1.3 with a very high loading. In practice the actual loading will be more or less triangular and the angle of friction needed must therefore be smaller, see Figure 141.
With loosely dumped granular material, that is, with insufficient compaction, an internal angle of friction of 30° to 35° is possible. In this case, if the thickness of the cover layer agrees with $H_s/\Delta D = 6$, the stability will be insufficient. A calculation using an internal angle of friction of 30° will give a safety factor of less than 0.5. The dike body under block pitching therefore should be well compacted. Since the loading is only short duration, local failure does not mean that large scale damage will develop on the dike, but that there will be some slope deformation, see Figure 141.
It should be noted that the results of the calculations are independent of the wave height. For a granular material, if $H_s/\Delta D$ is assumed to be constant, increasing wave height causes increased loading and the thickness and therefore the weight of the blocks should be increased. Because of the increased weight of the blocks the resistance to being pushed off the filter layer is larger. The relationship between loading and strength the cover layer remains constant for various wave heights because:

- the loading increases linearly with the wave height;
- the thickness of the cover layer increases linearly with $H_s/\Delta D$ constant;
- the strength of the subsoil is directly proportional to the weight of the overlying blocks.

The minimum angle of friction differs with the slope and values of $H_s/\Delta D$. In general however if the subsoil is well compacted, its stability will not be a problem; with poor compaction, however, deformation of the dike body may occur.
The calculation method used here can be applied for:

- a relatively stiff grain skeleton, in connection with the pore water;
- wave impact loads which (almost) only act on the cover layer and not, for example; directly on the filter layer through holes in the cover layer.

In the exceptional case in which neither of these conditions are satisfied the grain stress will be reduced under the point of wave impact due to increasing pore water pressure [HOOGEVEEN and DE GROOT, 1990]. In normal cases however the conditions will be satisfied.

Blocks laid on a geotextile on sand (liquefaction)

Continual wave impact loads on a layer of fine or medium coarse sand can cause compaction. When the material is saturated however, compaction is not possible and then wave impacts can lead to a decrease in grain pressure and possibly to liquefaction [BEZUIJEN et al., 1990, page 148].

It is very complicated to quantify this mechanism precisely; a method of approximation is given by [LINDENBERG, 1988]. The conclusion given in this manual can be simplifed: adequate compaction of granular materials (to a Proctor density of 95% or more) will remove the risk of weakening. This is certainly the case when there is a filter layer with good permeability between the blocks and the subsoil, or when there are simultaneously high wave loads and a high air content in the subsoil, for example, because of the effects of the tide.

Blocks laid on clay

Under short period wave loads clay, because of its low permeability, will not drain completely; during these conditions the water has insufficient time to flow away. The strength of the subsoil is therefore determined by its undrained shear strength which in turn is partly determined by the loading from above. In these conditions, therefore, it is not possible to give values for $H/\Delta D$ and the undrained shear strength of the clay which will ensure the stability under all wave heights. A clay with a little undrained shear strength is only satisfactory up to a certain wave height. A clay with a higher undrained shear strength therefore must be used for higher waves. Using again the Brinch Hansen calculation method it can be shown that clay with an undrained shear strength of 20 kN/m^2 will not suffer plastic deformation by impacts from waves up to 4.5 m high.

When clay is laid in accordance with the standard, with a consistency index of 0.75 or higher, see Section 5.8, the strength will always be sufficient because the undrained shear strength is 50 kN/m^2 or higher with this consistency index. When clay is laid above the high water line, this will "ripen" and dry out. The undrained shear strength can only increase under these processes. A revetment of blocks on clay, is therefore not expected to deform as a result of wave impact.

CHAPTER 10

TRANSITION STRUCTURES

10.1 Introduction

The slope protection at a site generally comprises more than one type of revetment. Below the water level it is not possible to use pitching and up the slope above the level of heavy wave action, for financial reasons, cheaper solutions are more acceptable. In practice there are other reasons for using transition structures between different types of revetments [BEZUIJEN et al., 1990, page 177]:

- When the level of an existing revetment has to be raised and the original material is no longer available; a different type of material therefore has to be used.
- If the levels of the foreshore or a channel bed in front of a dike are lowered and the revetment must be extended down the slope, and the old toe structure, for example, wooden piles, remaining in place as a support to the existing structure, cannot be removed.
- Asphalt is used on some dikes, particularly in the north of the Netherlands, to protect those parts of the slope most exposed to attack under design conditions; in order for the dike body to drain however a pitched revetment has to be used at the toe.
- After being damaged or during regular maintenance, part of a slope sometimes has to be repaired with a different type of structure.
- Structures judged to be the most suitable for straight sections of dike, for example, concrete cubes are not always satisfactorily on bends.
- Long or wide uniform sections of slope protection are sometimes undesirable, because damage can spread over relatively large areas; the slope protection in these cases is therefore sometimes divided into what are referred to as "slope sections", separated by transition structures.

Sometimes, for the reasons above a slope protection can have several types of revetment with many different transition zones.
From experience it appeared that transitions are sensitive to damage. The strength of the cover layer is locally reduced because the connection between blocks is less and blocks can be more easily loosened. The blocks at the edges of block mattresses and at the edges of an area of interlocking blocks receive less support from neighbouring blocks. The cover layer is thus weakened at these transitions. In many cases transition structures can lead to increased loading. In practice it is generally difficult to find adequate solutions. This is described in the following sections. As a result transition structures remain sensitive to damage and should be avoided as much as possible. In addition some transitions which appear to be logical on paper are difficult in practice to construct. Transitions should therefore be constructed as carefully

as possible. Construction aspects are treated in this chapter; reference should also be made to [PHILIPSE, 1989].
There are four different types of transition structures:

1. Links between different types of revetment materials and/or sublayers:
   - horizontal transitions between upper and lower slope revetments, that is, transitions across the slope, see Figure 142; transitions between pitching and grass fall into this group;
   - vertical transitions, that is, up and down the slope, between adjacent revetments.
2. The end sections of a revetment:
   - horizontal transitions – toe structures;
   - vertical transitions – connections to other civil works.
3. Structures to limit the spread of damage (compartmentation).
4. Structures at a change in slope, for example, berms.

These different functions are discussed in detail below following a section on functional specifications.
Often horizontal and vertical transition structures can be constructed in the same way and have the same function. No distinction is therefore made between the two below. Specific aspects of vertical transition structures are discussed in Section 10.7. For further information on these structures reference should be made to [DE RIJKE, 1991].

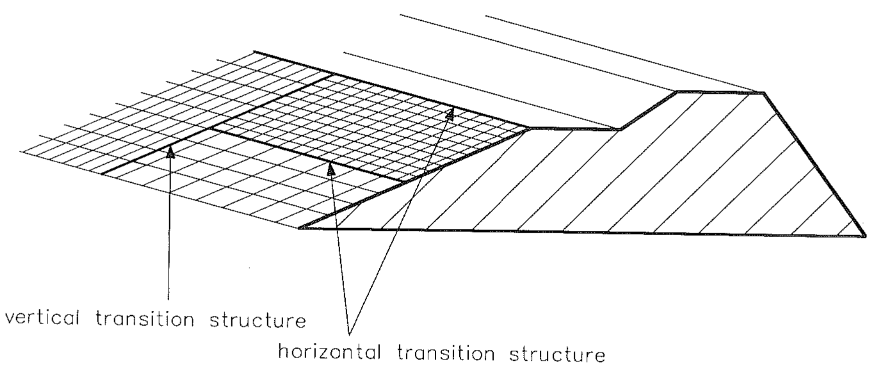


Fig. 142. Horizontal and vertical transition structures.

10.2 Specifications for transition structures

A transition structure, which has not been carefully designed, constructed or maintained, can lead to damage. Examples of this include:

- paved blocks lifted out and mattress edges which "flap";
- migration of one filter material into another leading to the cover layer settling;
- flushing out of sand, fine filter material or clay through joints or around piles which penetrate through the filter and cover layers;

- the rotting of wooden structural components;
- freezing of grouting cement mortar.

Specifications for transition structure should be divided into:

- functional specifications;
- construction specifications;
- management and maintenance specifications.

Unfortunately it is not always possible in practice to satisfy all the specifications and inevitably damage occurs at transition structures. This clearly should be avoided as much as possible.

10.2.1 *Functional specifications*

1. The transition structure must be as strong as the heaviest of the revetments being joined and in a sufficiently good state to protect the underlying dike body.
2. The water permeability of the cover layer at the transition should preferably be at least as large as the most permeable of the revetments being joined. The transition filter should be as permeable as the least permeable of the revetments. Because of the way the transition is constructed the permeability is often reduced.
3. There should be no movement of material from one layer to another or movement from the subsoil into the filter layers while the various sublayers are being laid, see Figure 143.
4. The transition structure should be as flexible as the revetments being joined so that it can readily adapt to local settlements without cavities developing.
5. The durability of the transition must be at least as large as that of the revetments being joined.
6. Transitions at toe structures should:
   - protect the toe revetment against a fall in level of the foreshore, the bed level in front of a river/canal dike, or the beach level,

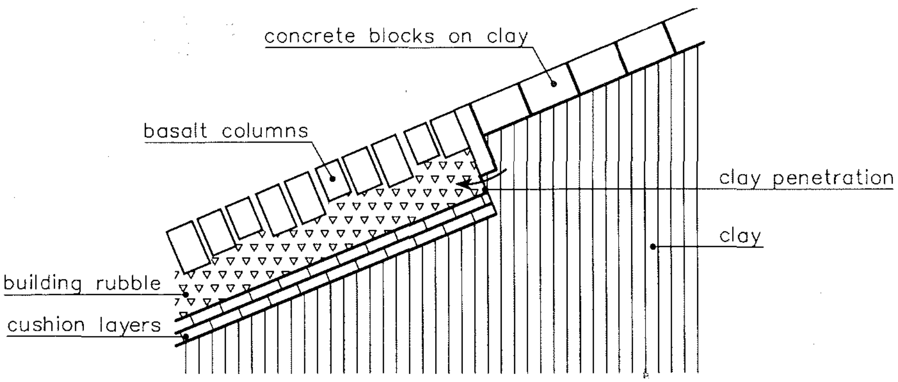


Fig. 143. Incorrect transition design, (clay can possibly penetrate into rubble).

– prevent the revetment sliding off.
– allow any groundwater in the dike to drain away.

Since, from experience, transition structures can initiate damage there should be the additional functional specification that the structures must have sufficient residual strength if a block is lost out of the revetment.
A durable transition structure can be made from various materials:

a. Concrete
This material is very durable, but is heavy and more difficult to work with than wood because piles cannot be driven through it. Similarly nails cannot be driven into it, see Figure 144.
b. Hardwood
Tropical hardwood is certainly durable enough but unfortunately it may not be able to resist heavy wave attack, see Figure 145.
c. Impregnated softwood
Only pine can be impregnated completely, for example, with creosote. A disadvantage of impregnated wood however is that it is environmentally unfriendly and poisonous materials, which degrade only slightly, are released during production, storage and service life. The extent of contamination depends on the impregnation system used [Environmental policy and the durability of wood, 1990]. The consequences and alternatives for creosoted wood are discussed in [HEIJNIS and KEIJZER, 1990].

Fig. 144. Toe structure with concrete sheet piling (during renovation).

d. Plastic
   Plastic is rarely used because it is fairly expensive and has a tendency to creep.

   The functional specifications should state that no filter or base material should be flushed out as a result of the transition structure. This problem generally occurs at vertical seams, for example, along piles which pass through the whole structure and can be avoided. This can be achieved by not breaking through the filter layer or by ensuring that any breaks are not immediately above one another, that is, by overlapping the layers. Examples of this are given in Section 10.2, see Figure 150.

10.2.2 *Construction specifications*

1. The shape of any connecting elements should be such that, after placing, it should be possible to compact the excavated foundation trench satisfactorily.
2. Excessively deep or wide excavation must be avoided, and it is very important to specify accurate dimensions for the work. Mechanical excavation with some manual work is preferred.
3. Excavation in minestone, for example, to insert a concrete retaining wall, must be avoided because of the risk of the formation of cavities which cannot be filled in later.
4. To prevent joints being contaminated with silt, plants and small stones, cement mortar should be grouted into the transition structure as quickly as possible after the blocks have been laid.
5. The joints to be grouted should not be too small, at least about 2 cm.
6. If asphalt is to be injected then clearly all the components of the structure must be heat resistant, (not the case with some geotextiles).

The above construction specifications are needed to make sure that excavation work is carried out carefully. When placing a toe bulkhead, for example, a trench has to be excavated to a depth equal to the depth of the bulkhead. Mechanical excavators tend to dig too deeply and as a result the toe structure tends to settle on the down slope side because the passive soil pressure has been slightly reduced. In addition some blocks at the structure will settle more than others because the ground adjacent to the toe is difficult to compact.
If the excavation has gone too deep it is always necessary to:

1. On the down-slope side of the toe bulkhead
   a. Refill and compact directly after pacing the bulkhead or while placing the first row of blocks.
   b. Fill with, for example, minestone or rubble (on a geotextile).
2. On the up-slope side of the toe bulkhead or along the transition structure:
   Fill to above the level of the transition structure, compact well and then excavate to the correct depth.

Fig. 145. Wooden toe structure (immediately before renovation).

Excavation to the correct depth can be achieved by first mechanically excavating to within a few centimetres of the depth required, driving the wooden piles and completing the remaining excavation, by hand, prior to placing the toe bulkhead.

10.2.3 *Management and maintenance specifications*

The management and maintenance specifications should be based on the experience that even well designed and constructed transition structures require more maintenance than the rest of the revetment, therefore:

- the number of transition structures should be kept to a minimum, and
- transition structures should ideally not be used in areas where heavy loads are expected, see Chapter 6.

10.3 Examples of transition structures between two types of revetments

Columns → blocks

Figure 146 shows an example of a transition between an old basalt revetment laid on building rubble and cushion layers and rectangular blocks laid on gravel or rubble on a geotextile. At the transition it is necessary to have a concrete retaining wall because it is not easy to place new blocks against basalt (and columns in general).

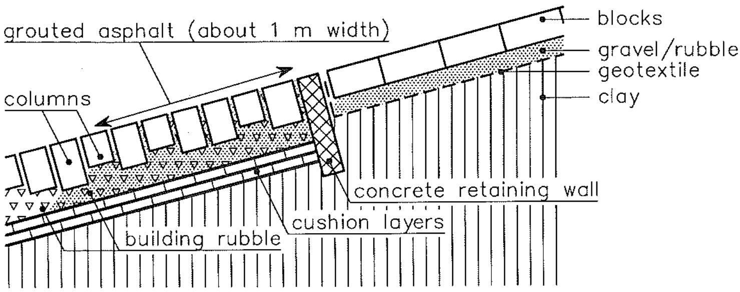


Fig. 146. Transition from columns to blocks on a filter (The alternative in Figure 147 is better).

The geotextile under the rubble is folded up against the concrete retaining wall so that it is pressed against the wall by the blocks and the rubble. This structure has two disadvantages:

- clay can be flushed out if it is badly constructed, and
- the clay is very difficult to compact after the retaining wall has been placed.

An alternative is given in Figure 147 in which the geotextile runs under the retaining wall. In contrast to the first design, the clay cannot be flushed out and in addition, it is not necessary to replace and compact the clay excavated when placing the retaining wall.

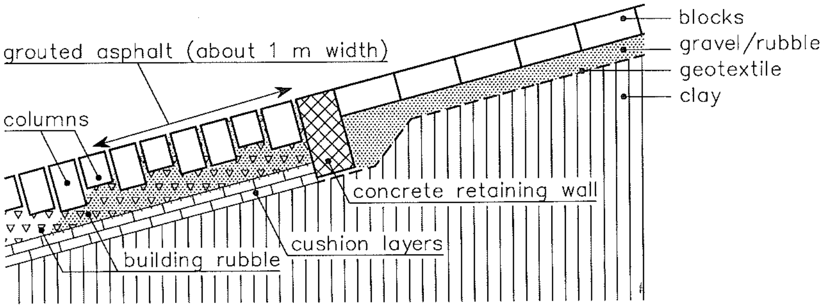


Fig. 147. Transition from columns to blocks on a filter (alternative to Figure 146).

It is not easy to set the concrete retaining wall in a straight line but it can be done using piles, for example. Piles however have the disadvantage that:

- if they rot repairs are almost impossible;
- a row can form a discontinuity if the slope settles; this can lead to cracking and less cohesion between blocks; and
- subsoil can be flushed out along a row of piles.

Examples of transition structures with blocks on clay are given in Figure 148. In this example the top row of columns must be grouted with hot asphalt, because the joint at the concrete retaining wall is often weak. This ensures that the columns are fixed, otherwise many of them would work loose. The asphalt also prevents filter material (building rubble) being washed out along the concrete retaining wall.
With old revetments, which are unlikely to settle further because the blocks are jammed, cement mortar can possibly be used. In other situations however molten asphalt mortar is generally used.

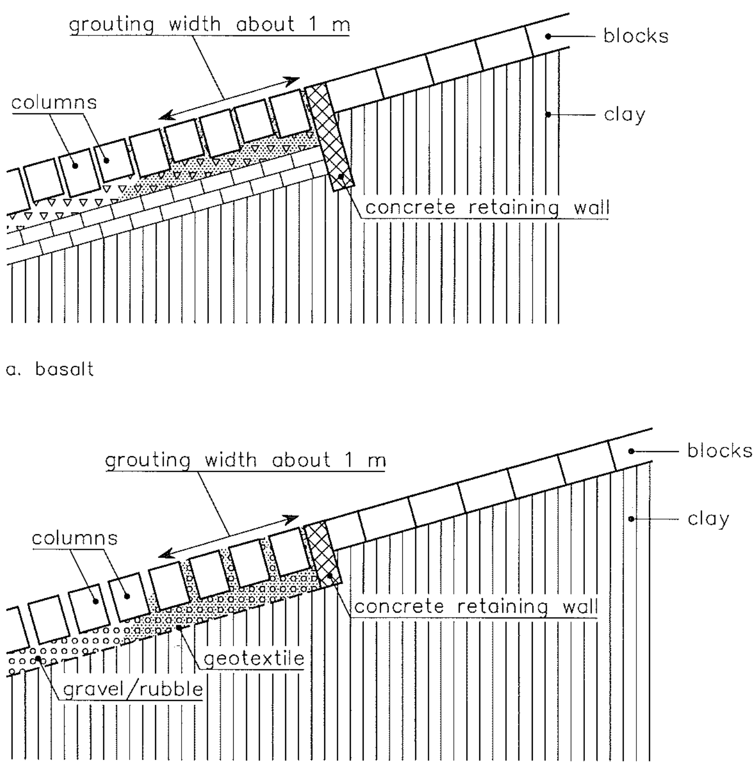


Fig. 148. Transition between columns and blocks on clay.

Because grouting mortars have a low viscosity they tend to flow away under the revetment and, as a result, the amount of mortar needed can be more than expected. Depending on the amount of material in the joints (washed-in material, sand, silt, rubbish, etc) it can be difficult to obtain reliable grouting. It may be necessary, therefore to first reset a line of columns in front of the retaining wall and then do the grouting. Grouting the columns improves the strength of the revetment and should be carried out for this reason. It is however not a complete solution because the pressure difference on the blocks in the transition will be larger than some distance away. There will also be a strip of blocks adjacent to the grouted area where the pressure difference is higher and where, therefore, the possibility of damage is higher. The largest loads will act on the row of blocks immediately above the grouted filter area. This will not be affected by the fact that the area of grouted filter is generally much greater than the grouted cover layer area. A solution to this problem is to use longer columns next to the retaining wall which are not grouted, see Figure 149. This solution is, however, fairly expensive. The space between the columns and the retaining wall must be grouted to prevent filter and base material being flushed out. The width of the strip of longer columns must be greater than the leakage length, Λ, but at least 0.5 m.

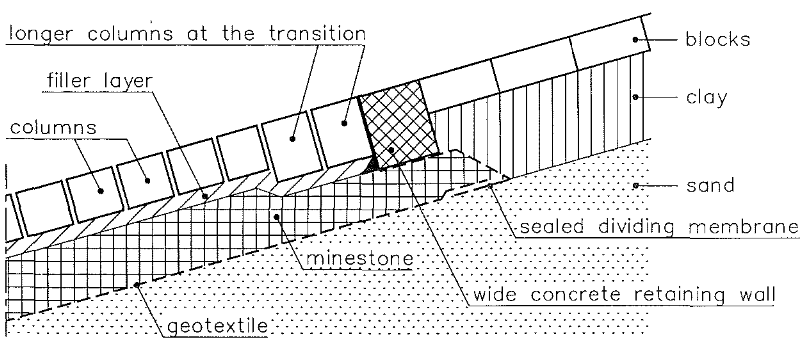


Fig. 149. Transition structure with a strip of longer columns, grouted only along the concrete wall.

Figure 149 gives an example of a transition structure between columns laid on minestone and blocks laid on clay. A wide concrete wall is used here so that it is stable during construction works. This is difficult to lay in the straight line needed for the blocks. With a crane, however, the handling of such a wide and heavy block is no problem.

Columns or blocks → columns

A concrete retaining wall is not needed if there is a durable column revetment above the existing block or column revetment at the transition. A continuous filter should however be used, see Figure 150. To ensure that the end of the geotextile overlaps the cushion layer, the upper two rows of the old basalt and the underlying layer of building rubble are removed. The geotextile is then laid on the cushion layer with a vertical piece to prevent the new filter layer migrating into the old rubble layer. The overlap between the geotextile and the cushion layer is essential to prevent clay being flushed out.

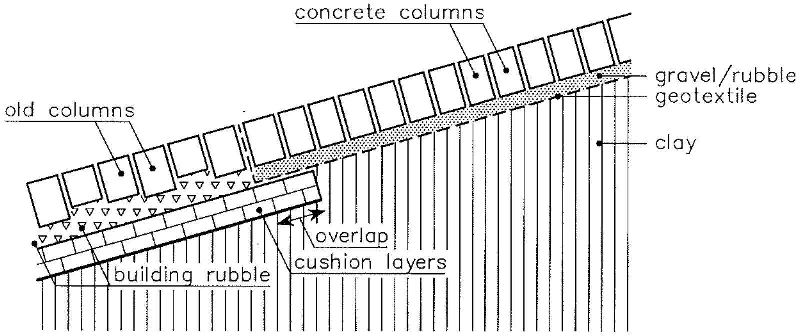


Fig. 150. Transition structure with a continuous geotextile.

This transition should preferably not be grouted with asphalt because this would stop the flow of groundwater in the filter and lead to increased pressure differences on the pitching (below the transition). This would only be the case if the leakage length of the structure above the transition (new works) is similar to or less than that below the transition or if the transition is above the design water level. If this is not the case a strip at least 1 m wide, should be grouted.

B l o c k s → a s p h a l t i c c o n c r e t e

For transitions between blocks on a filler layer on minestone and asphalt it is important for the minestone to extend a little way under the asphalt revetment as shown in Figure 151. Here there is no vertical seam through which the underlying sand can be flushed out. The minestone thickness must be gradually reduced under the asphaltic concrete in order to distribute uneven settlements over a larger area.

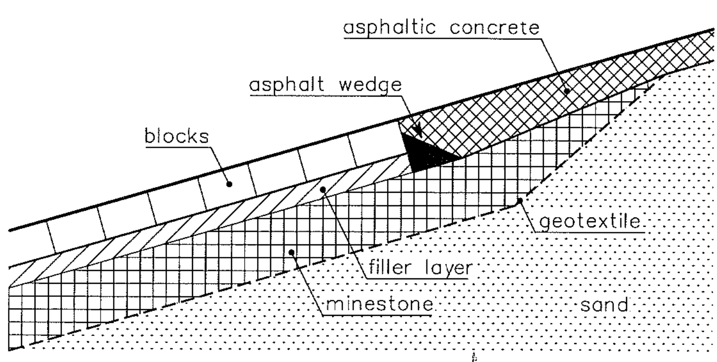


Fig. 151. Connection between blocks on minestone and asphaltic concrete.

After laying the blocks it is recommended that an asphalt wedge is laid to prevent the top row of blocks rotating and settling. The top row of blocks cannot be grouted because the joints are too small. There may therefore be relatively many loose blocks immediately below the transition structure. If necessary grouted tapered blocks can be used.

Columns → asphaltic concrete

The points discussed for transitions between blocks and asphaltic concrete also apply for transitions between columns and asphaltic concrete. The only difference is that, in this case, the top row of columns can be grouted with asphalt.
Figure 152 shows the cross section through a dike on the island of Texel in which the overlap between the filter and the asphaltic concrete can be clearly seen. The filter in this case is a bonded filter of sand asphalt.

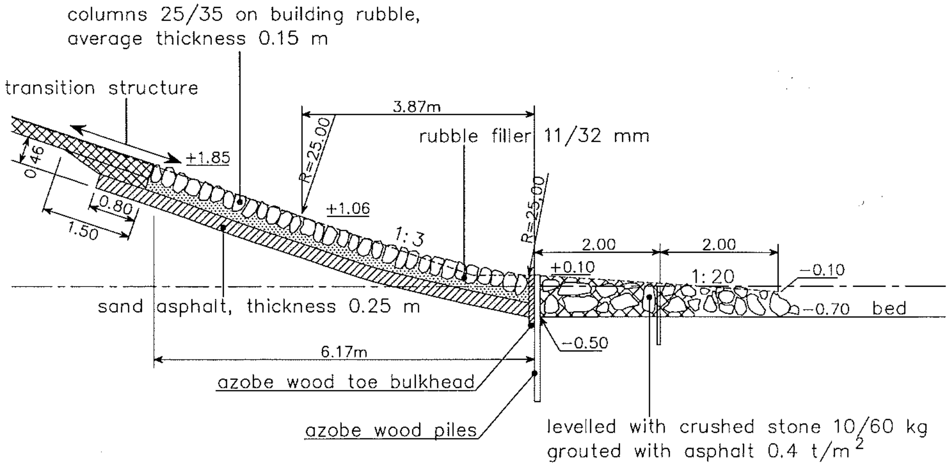


Fig. 152 Transition structure between columns and asphaltic concrete.

Blocks → grass

The transition between a block revetment and one of grass on clay should not be too abrupt. A gradual transition is important to prevent grass being grazed by cattle. Open elements through which grass can grow are used or open road quality brick-work laid on clay. An example of this type of transition is shown in Figure 153. A concrete retaining wall is used to ensure that there is a good connection with the pitched slope, a geotextile being placed against the wall. This makes it impossible for clay to be washed into the filter. This type of transition can also be made without the concrete wall. The open elements are then essential to prevent erosion at the top of the revetment. In this case the geotextile must extend about half a metre under the open elements. Sometimes the joint is secured with a row of wooden piles with a wooden bulkhead or a concrete wall. The piles are not needed structurally except when deep undermining of the grass is expected.

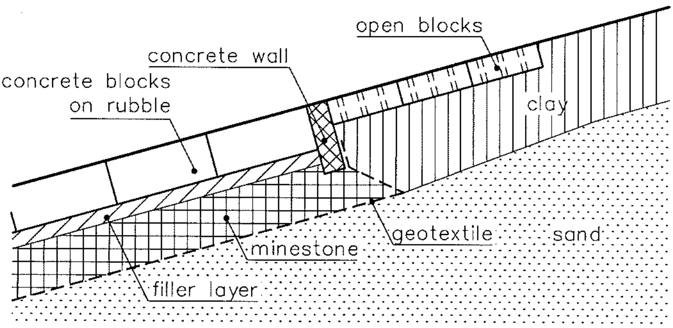
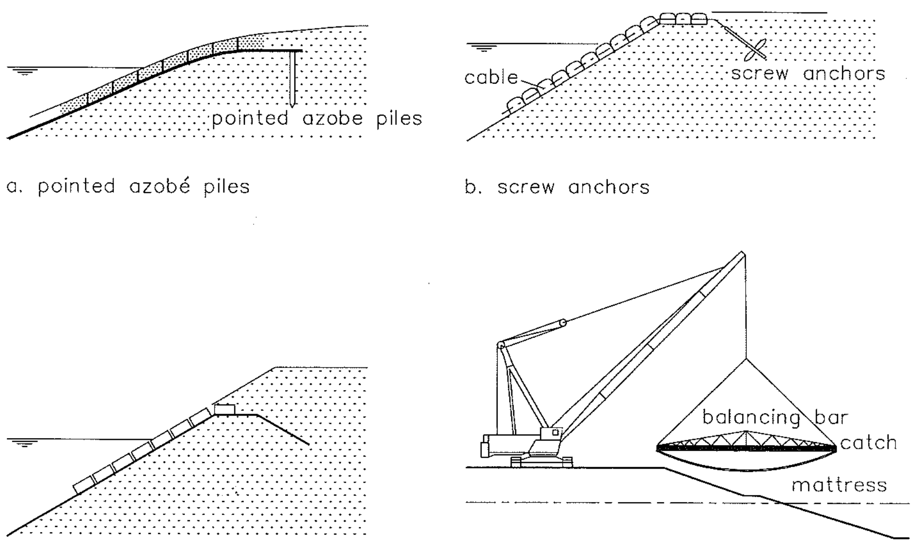


Fig. 153. Transition from blocks on minestone to grass.



c. buried geotextile

d. placing a mattress

Fig. 154. Placing a block mattress and some methods of anchoring.

Block mattresses

Each edge of a block mattresses can be seen as a transition structure. Two mattresses, lying side by side, should preferably be fastened together to make the best use of the interaction between blocks. If the mattresses are not fastened together, the corners and sides can "flap", especially if the distance between mattresses is more than about 3 cm. The stability is then barely greater than individual blocks, see Section 5.2. With bad construction the stability may be even less. If it is difficult to fasten mattresses together after they have been laid; mattresses should be used which extend in one piece from the toe to the crest of the dike. The method of fastening should take into account the large hydraulic forces which will act on the structure. The size of these forces can be calculated using the method given in Section 8.7.1, the force on the fastening being equated to that on an equivalent anchorage at the level of the fastening. The top edge of the mattress should be anchored to prevent it sliding down the slope. An example of this is shown in Figure 154. The flushing out of filter or base material can be prevented by laying a geotextile with a 0.5 to 1 m overlap under the mattresses. The design procedure for block mattresses is given in Sections 8.3, 8.4 and 8.7.

10.4 **Examples of toe structures**

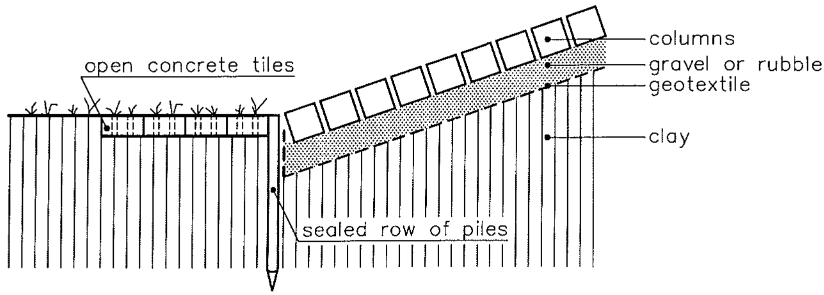
Toe structures are usually fairly heavy, on the assumption that they have to take part of the weight of the revetment, see Section 8.7.1. If the toe is too light there is a risk that it will be pushed by the revetment which will slide down the slope, widening the joints and reducing the adhesion between the blocks.
Toe structures can be divided into four different types:

1. a sealed row of piles (long columns),
2. piles with a wooden or concrete bulkhead (long blocks),
3. a sheet piled wall, and
4. a concrete retaining wall (in a berm).

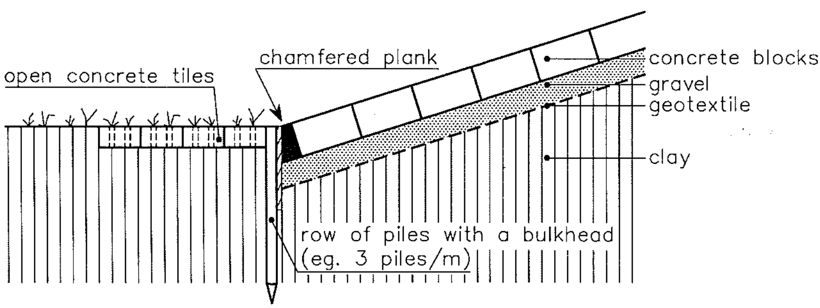
A sheet piled wall gives a watertight structure and has the advantage during construction that the toe can be pumped dry. A further advantage is that the toe is immovable. An important function of the toe is to support the revetment and to prevent it from sliding off. The piles or sheet piled walls, however, always tend to bend forward and deform when taking loads.
The form of the toe structure depends on the height of the foreshore relative to low water. From the point of view of construction and management it is recommended that the toe is not less than about 50 cm above low water.

High foreshore

Two toe structures suitable for a high vegetated foreshore are shown in Figure 155. In this context the term "high" refers to a foreshore which is above the tidal limit. The transition between the foreshore and the revetment is in the form of several rows of open blocks. These blocks will prevent cattle grazing on the toe and causing erosion by currents and waves. Because the open blocks and the foreshore can settle any settling along the toe structure itself should be uniform. Open blocks should therefore not be laid on or above any wooden piles if these are placed below the level of the toe bulkhead.



a. columns



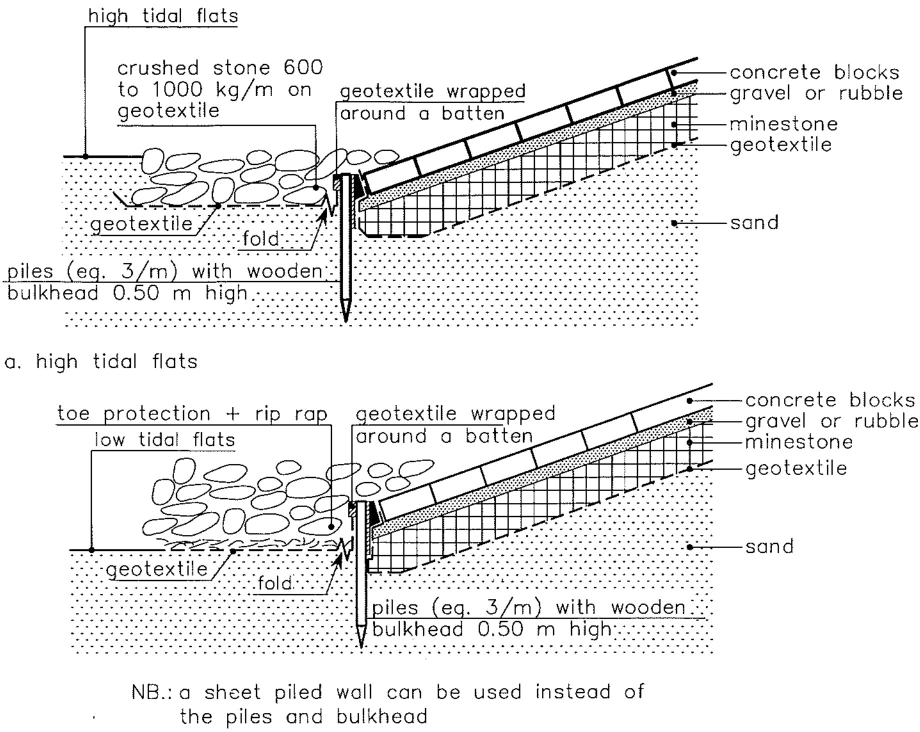
b. blocks

Fig. 155. Examples of toe structures with a high foreshore (it may be possible to omit the open concrete tiles).

A toe bulkhead should be used to enable the first row of blocks to be laid straight and close together. A chamfered plank can be used in front of the vertical bulkhead to ensure that the lowest row of blocks is laid correctly and that a stepped revetment is not formed, see Figure 25. Columns can be readily laid against a sealed row of piles.

Foreshore in the tidal zone

A buried toe structure can be used where the foreshore in a tidal zone (sea bed or tidal flats) lies at least 50 cm above mean low water. The top of the toe should be 25 to 50 cm below the level of the foreshore. Examples are given in Figure 156. The excavation for the minestone layer should preferably not be deeper than mean low water. As with a toe structure along a high foreshore, care should be taken in the tidal zone to ensure that the piles and the bulkhead will not lean over if undermined. To prevent this crushed stone should be placed on a geotextile in front of the toe to support the toe structure and stop it leaning over. The geotextile will prevent the crushed stone settling into the sand. A sealed joint at the piles or the toe bulkhead is unnecessary because it is unlikely that sand will be flushed out here. If heavy wave and current action is expected however the joint should be grouted with asphalt. The geotextile must not be attached to the bulkhead or the piles. If the method of construction requires this sufficient material should be laid (in folds). The geotextile can then be attached to a batten in a few places and this nailed to the piles.



a. high tidal flats

b. low tidal flats

Fig. 156 Examples of toe structures in the tidal zone.

The crushed stone has two functions:

- to protect the toe against erosion (if there is a possibility of flow along the toe);
- to prevent the toe leaning over (due to the weight component of the revetment down the slope).

Clearly, for the latter function, the crushed stone should be placed immediately after the piles (and the bulkhead) have been installed, or during the placing of the first row of blocks.
An alternative design is given in Figure 157 which does not include crushed stone at the toe. This structure can be applied in situations where there is no flow along the toe. A wide trench has to be excavated and, provided that there is sufficient quantity of the necessary weight, the material taken out of the trench can be conveniently used for a temporary bund to give protection during the works. After installing the piles (and the bulkhead) the trench can be filled with minestone to support the toe and prevent it leaning over.

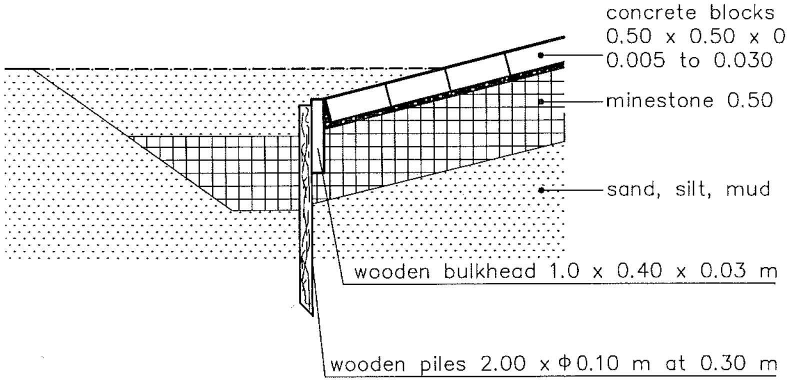


Fig. 157. Toe structure in the tidal zone with a wide trench.

A type of toe structure which is still found, but no longer constructed, is shown in Figure 158.

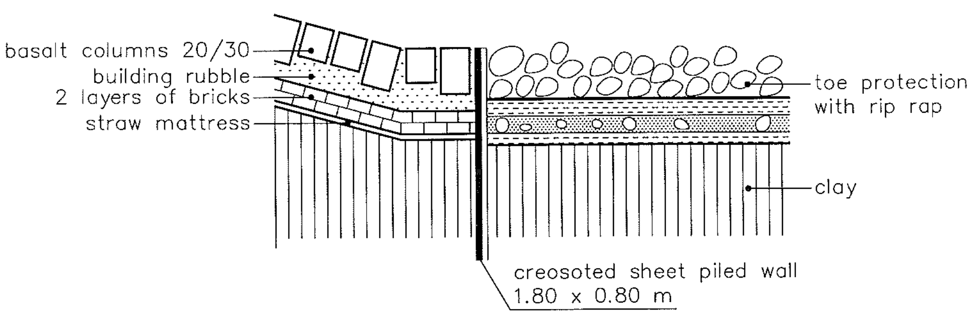


Fig. 158. Example of a type of toe structure which is no longer constructed.

Steep foreshore

When constructing a dike with a steep slope on, for example, a low foreshore rip rap has to be dumped (below low water) to form a support berm above low water. The pitched revetment is then constructed on this berm. An example is given in Figure 72 (Chapter 7) which has a row of wooden piles and a toe bulkhead. A structure in which the piles on the support berm are replaced by a concrete wall is easier to construct, see Figure 159. The concrete wall is sufficient to stop the berm falling over during construction. The wall has a sloping side against which the blocks or columns are set. The rip rap should be laid in time to prevent the concrete wall being pushed out of place by the blocks.

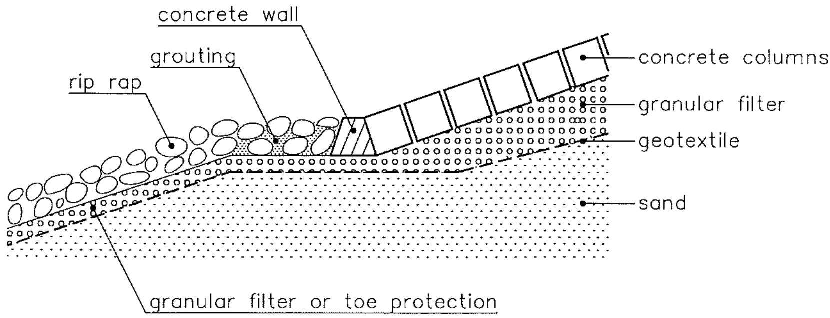


Fig. 159. An open toe structure with a concrete wall on the support berm.

It is necessary to grout the rip rap:

- to secure the concrete wall, especially on steep slopes, and
- to form a transition between a very open cover layer of rip rap and a very compact cover layer of pitching.

If a structure with a short leakage length (open cover layer) is joined directly to a structure above with a long leakage length (sealed cover layer) relatively high loads will develop on the bottom row of blocks of the structure with the long leakage length during wave action. This will only occur if the filter layer is continuous. Grouting a strip of the rip rap about 1 m wide to give the cover layer a very low permeability is sufficient to reduce the loading on the toe, Figure 160 shows a good, but labour intensive, alternative.

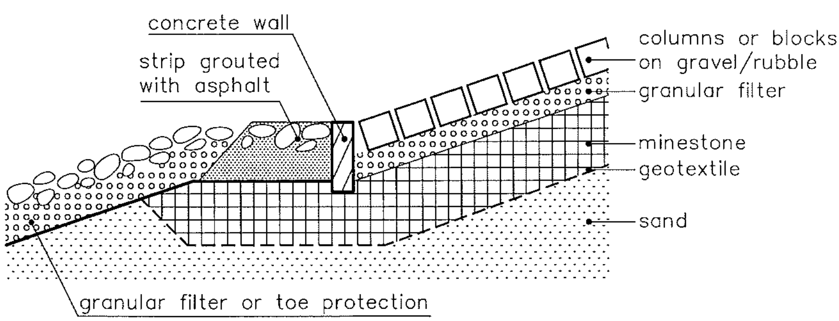


Fig. 160. Alternative type of toe constructed on a berm.

Block mattresses

A toe is not needed to support a block mattress because the anchorage at the top ensures that it does not slide down the slope. The subsoil, however, must not be allowed to flush out at the toe. In addition the mattress must not flap at the toe. This can be a problem if the toe lies between SWL and one to two times H_s below. A good toe structure for a mattress lying on a thin granular filter is shown in Figure 86.

10.5 Structures which limit the extent of damage

In the past the transition to grass was often made using a close row of piles, projecting above the revetment. The aim of this was to prevent the material being washed out of the dike, collecting on the grass and causing the turf to deteriorate. The piles also helped to break the wave run-up. When the level of the revetment was eventually being raised the piles were left in place because experience has shown that damage generally stops at a transition zone. Later, rows of piles perpendicular to the existing piles were made, thus forming compartments. Although the rows of piles clearly prevent local damage spreading there are important arguments against using piles for this purpose:

- the vibration and movement of piles due to wave forces lead to the loosening of the revetment and loss of adhesion; as a result, the revetment can be damaged more easily;
- when piles are shaken loose space is created through which the underlying material can be flushed out;
- maintenance is essential because broken piles rot and must be replaced quickly to prevent the sublayers being flushed out;
- settlement of the slope or toe structure leads to cracks and loose blocks on the slope immediately below the row of piles.

Removal of piles is usually not recommended because the holes cannot be filled satisfactorily. It is better to force the piles completely into the structure to about 20 cm below the top of the sublayers.
The spread of local damage can also be prevented by grouting strips, 0.5 to 1 m wide, with asphalt. This should be carried out immediately after placing the revetment, because the joints may become filled with rubbish and then the asphalt cannot penetrate properly, see also Section 10.3. Strip grouting can however lead to damage because the pressure difference on the strips is larger than elsewhere. The conclusion must be that "homogeneous" revetments without piles or grouted strips suffer much less damage. If damage does occur however with this type of revetment it can spread to a much larger area.

10.6 Berms

Damage-sensitive points in pitching occur where a gentle upper slope meets a steeper lower slope or where there is an outer (front) face berm. Unless special measures are adopted there will be less adhesion between blocks as a result of the open joints at the change in slope. This is not such a problem for a berm at storm flood level because the heaviest loads occur below SWL, but for lower berms additional measures are recommended.
An example of a berm with a service road is shown in Figure 161. The front edge of the berm is constructed with tapered blocks. This is not the ideal solution because

these blocks do not settle uniformly together and, because of differences in dimensions during construction the blocks do not always fit. It is better to use a concrete wall with a sloping side or to finish off the front edge in the way shown in Figure 162 and 70). This finish, when carried out together with grouting on the edge, is a good solution for columns. With blocks it is not possible to grout the edge because the joints are too narrow. The solution therefore is to use a few rows of tapered blocks. A disadvantage with a concrete wall is that the trench for the wall is difficult to compact on the down slope side.

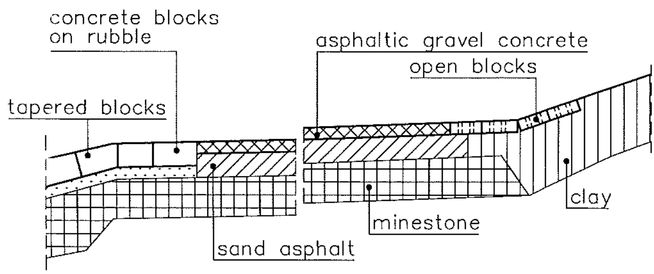


Fig. 161. Berm with a service road and an inadequate solution for the front edge (tapered blocks, see Figure 163).

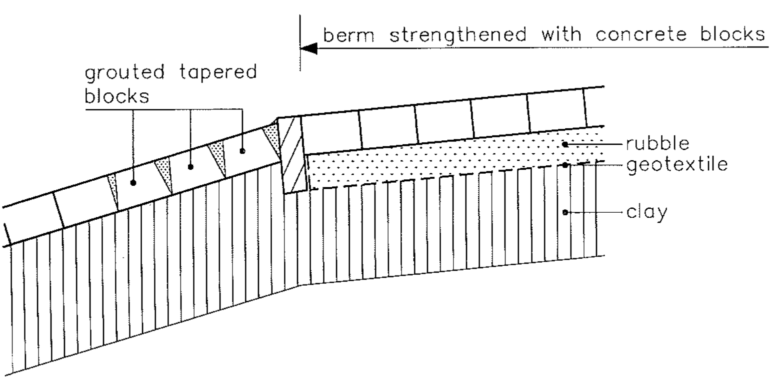


Fig. 162. The correct design of the front edge of a berm using asphalt grout and a concrete wall.

Generally berms are used as service roads and blocks laid on clay are therefore unsuitable since they have insufficient bearing capacity. A foundation layer of rubble on a geotextile is sufficient however to prevent settlement due to light maintenance vehicles. Heavy vehicles however require a filter layer at least 0.5 m thick, see Figure 163.

Fig. 163. Damage to a berm (tapered blocks).

10.7 Vertical transitions

Vertical transitions are constructed between two dike sections and also where a dike is broken through by, for example, a sluice or a bridge pile. In the first case the vertical transition can be constructed in the way described in the sections above for horizontal transitions. At the point where the revetment meets the structure the vertical seam is often not continuous and material can be flushed out from the subsoil, see Figure 164.

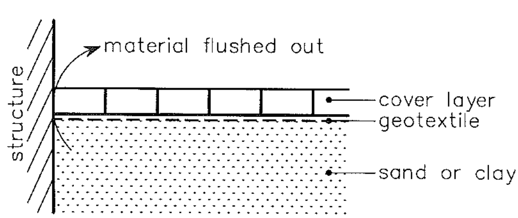


Fig. 164 A continuous seam of geotextile may cause subsoil to be flushed out along a structure.

This can be prevented by grouting a strip of the cover layer about 1 m wide with asphalt. The asphalt need not necessarily adhere to the structure because the revetment will tend to settle more than the structure. The asphalt must however prevent the filter grains being flushed out. Tapered blocks should be used locally because concrete blocks have joints which are too narrow for grouting.
Grouting alone is insufficient for revetments in which the cover layer is laid directly on sand or clay and possibly a geotextile. In this case a granular filter, about half a metre wide (gravel box) is constructed, see Figure 165. Alternatively a strip of sand asphalt can be used.

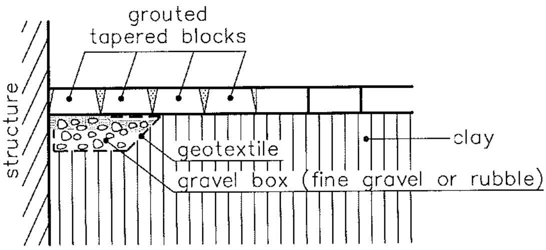


Fig. 165. Gravel box transition.

CHAPTER 11

DESIGN CONSIDERATIONS

The previous chapters have concentrated on the methods which can be applied for the design and construction of pitched dike revetments. A good design however is more than simply applying standard calculations. No two designs are the same and in every case there are conditions, considerations and preferences which mean that each design must be "made to measure". This chapter stems from the firm conviction that this manual can never be a "cookery book" for a design replacing the creativity of the designer. The manual serves only to evaluate graphically the "plusses" and "minusses" of particular alternatives.

Thirteen of the most frequent points to consider in revetment design are treated briefly below. It is assumed that the need for a slope protection has already been established, a topic treated in the [Selection Methodology, Manual 1988] and are not considered below.

The plus (+) and negative (–) signs indicate whether or not the design has a positive or negative effect on the functioning or the price of the structure. If this is unknown or if it has no effect the zero (0) sign is used. If, depending on the situation, both positive and negative effects are possible this is shown by a (±) sign.

1. *A steep versus gentle slope*
   - (+) The aim of steepening the slope is to reduce the length of the revetment. As a first approximation the slope length is taken to be inversely proportional to sinα.
   - (–) The wave run-up for breaking waves will increase on a steeper slope, being directly proportional to tanα. This increase the crest height and therefore the dike volume which is proportional to the square of its height (without berm, see also Appendix B).
   - (–) The loading will also generally increase (with increasing slope angle), which leads, amongst other things, to a need for a thicker cover layer.
   - (+) The friction between blocks increases with the slope in proportion to sineα. This implies that a less thick cover layer is needed on steeper slopes, because the cohesion between individual stones increases its strength.
   - (–) With a steeper slope it is more difficult to create a stable interface between the filter and the base. The specifications for the filter material therefore become more strict and, on very steep slopes, only a geotextile can be used.
   - (–) With a steeper slope a heavier toe structure is needed to support the revetment. This requirement is increased because there is an increased, with a steeper slope, tendency for the toe to be undermined.

(–) With a steeper slope damage spreads more quickly; and the residual strength of a steep slope is therefore less.
(–) With slopes of 1:3 to 1:2 the geotechnical stability of the slope and the revetment is critical.

2. *Application of berms on the outer slope*
Assuming a smooth slope, what are the effects of applying a berm at the design water level?
(±) The aim of the berm is to reduce the wave run-up, achieving a lower crest level and a smaller dike volume. Because of the berm however the width of the dike is increased by the berm width and, as a result, the dike volume is increased by the berm width multiplied by the berm height (per unit length of dike). Depending on the geometry and the wave conditions the effects of a berm can be positive or negative, see Appendix B.
(+) The berm can be used for a service road.
(–) The berm creates a kink in the front slope, a weak point at a heavily loaded section.
(+) The lower level of the phreatic surface in the revetment occurs at the berm level; for a cover layer of low permeability the berm therfore, increases the stability; for a more or less permeable cover layer it has little or no effect.

3. *Small versus large cover layer permeability*
(+) The aim is to reduce the upwards pressure head difference on the cover layer so that a thinner cover layer can be used; this can be achieved by increasing the permeability of the cover layer.
(–) If the cover layer permeability is large however the relationship between cover layer and filter permeability (k'/k) must be constant and not reduce with time. Investigations into this aspect are in progress, see Section 8.7.3.
(–) The flushing of the filter through the cover layer. There are three remedies for this:
a. Coarser filter
(–) leads to larger loads on the cover layer and thus a need for thicker blocks.
b. Geotextile on a granular filter directly under the cover layer:
(–) leads to larger loads on the cover layer and thus thicker blocks.
(–) there is a possibility that the geotextile will become blocked and the loads on the cover layer will be increased still further.
c. (±) Bonded filter (sand-cement or bitumenized sand).
As yet this is not a generally accepted solution and is also relatively expensive.
(–) Increasing the permeability of the cover layer enables fluctuations in external loadings to penetrate more strongly through the filters, the effects being:

(–) larger hydraulic gradients in the filter and at the filter base interface. For a geometrically sealed filter, this does not have any negative effects on sand penetration from the base. For filters which are not geometrically sealed however this can lead to interfacial instability.
(–) geotechnical instability of the slope.
(–) internal instability in the case of a broadly graded filter (in which the finer particles in granular material are flushed out between the coarser particles, see Section 5.3.1).

This sets stricter specifications for the filter and the interface and promotes, perhaps, the use of a thicker filter or a geotextile at the filter-base interface.

(+) Smaller wave run-up on the cover layer which has a rougher surface because of very large holes (in general relatively little reduction in run-up).
(–) Larger loads on the cover layer because of the very open surface and the greater roughness, see Chapter 2.

4. *A good versus poor filter permeability when the sublayer is clay*
(+) Reduction of the permeability of the filter leads to a considerable reduction in the loads on the cover layer and thus to a reduction in block thickness.
(–) For non-cohesive filter materials a reduction in permeability can be achieved by reducing the grain size or choosing a broader gradation:
    (–) risk of material being flushed out through the cover layer;
    (–) risk of geotechnical instability of the filter if the permeability is similar to that of sand;
    (–) risk of suffosion and/or flushing out of the fine fraction, whereby the effect is lost.
(–) When using clay, a cohesive sublayer, strict specifications have to be set to prevent gully formation in the surface. Because clay is difficult to characterise the specifications are, by necessity, forced to be conservative.
(+) A less permeable filter reduces the loads on the filter/base interface. A reduction in the thickness of the filter layer then becomes possible.

5. *Angular versus round filter material*

Round filter material (gravel) has disadvantages compared with angular material (rubble):
(–) In the Netherlands environmental objections exist against gravel winning.
(–) On steep slopes round matrial is less stable than angular material. Loose stones can therefore gradually settle individually forming "stepped" slopes, see Figure 25, and/or slide as a whole, leading to heavier loads on the toe structure.

6. *Thick versus thin filters*
   (+) With thinner filters smaller loads on the cover layer are smaller because of the smaller leakage length.
   (–) With thinner filters loads on the filter/base interface are larger.
   (–) A filter which is too thin can put the stability of the base at risk.
   To counteract the disadvantages of a filter which is too thin, a smaller permeability can be used, see Item 4 above, or a geotextile can be applied on the filter/base interface.

7. *(Rectangular) blocks versus (polygonal) columns*

Blocks	Columns
(±) Sealed cover layer.	(±) Open cover layer.
(+) Easy to lay mechanically in rows.	(–) Laid by hand, more difficult to lay mechanically.
(–) Difficult to lay at bends and joints.	(+) Easy to lay at bends and joints.
(–) Repairs to damage troublesome.	(+) Repairs to damage less troublesome.
(–) Washing-in to increase the strength is not generally possible.	(±) Washing-in forms part of the strength philosophy but maintenance is necessary.
(+) Smooth external appearance.	(+) Attractive appearance.
(+) Less expensive.	(–) More expensive.

8. *Concrete versus natural stone*
   If there is stone available of satisfactory quality and in sufficient quantity it can in general be used in the revetment, despite the high cost of laying by hand and maintenance/"plugging of the remaining holes with smaller stones". Concrete blocks however have specific advantages and disadvantages:
   (+) Are less expensive (because of the relatively high delivery costs of stone).
   (+) Have a consistent quality.
   (+) Have uniform thickness.
   (+) Can be laid mechanically, making for less expensive products which can be manufactured more quickly. In addition less specialised personnel are needed and working conditions are better.
   (+) Offer more choice for form and size.

   BUT
   (–) Generally have a lower volumetric mass.
   (–) Are often judged to be less attractive.

9. *Concave versus convex camber*
   In general block pitching is laid with a slightly convex camber and not a flat slope. A convex camber of between 1/50 and 1/100 is referred to as positive. The advan-

tages and disadvantages of a convex camber are discussed in detail in Section 7.7 and there seems to be a slight preference for a convex rather than concave camber.

10. *High construction costs versus high maintenance costs*
The total costs of a revetment are made up of:

a. Construction	d. Repairs to damage
b. Small (routine) maintenance	e. Modifications
c. Large scale maintenance.	f. Demolition and clearing away

In general lower construction costs imply higher maintenance costs. The designer must be aware that the choice of a particular level of safety can sometimes be a little subjective and the extent to which damage is acceptable is, amongst other things, determined by the structural function (dike or bank protection) and by the amount of residual strength in a structure after initial damage.
Depending on the form of financing (subsidization) the manager may prefer to choose between high construction costs or relatively high maintenance costs. In this case optimization of total capitalized costs for construction, maintenance and demolition is not relevant (there is rather a need to minimize non-subsidized costs). The manager must however make financial provisions for his part of the costs which therefore need to be evaluated. Various calculation methods have been developed for comparing structural alternatives and at present the discounting method is that most frequently applied. Discounting or cash value, is a method of bringing costs, incurred or earned at various times together on to a time basis using a rate of interest and anticipated price indices, the aim being to make totals comparable. The real rate of interest, i:

$$i = \text{actual rate of interest} - \text{inflation factor} \tag{66}$$

Assuming a constant real rate of interest, the cash value, CW, of a proposal with a present value of U, which is to be undertaken over N years, is calculated by multiplying in by a cash value factor, CW_f, as follows:

$$CW = U \cdot CW_f = U \cdot \frac{U}{(1+i)^N} \tag{67}$$

For a good cost comparison of various alternative solutions it is necessary to calculate the cash values of all the (future) items needed during the service life of the structure to maintain a minimum level of quality. Possible trends in structural quality with time and the associated savings in costs are shown in Figures 166 and 167.
The result of this type of cash value calculation, although interesting, can however only serve as the basis for an indicative comparison.
The technical assessment of maintenance costs and periods as well as the economic estimates of changes in interest rates and price levels have an important

effect on the results of the calculations. Generally it is not possible to express a preference for higher construction or higher maintenance costs. It is recommended therefore that investigations are carried out into the relationship between maintenance costs and structural properties, loading, etc. to enable a better evaluation to be made in the future.

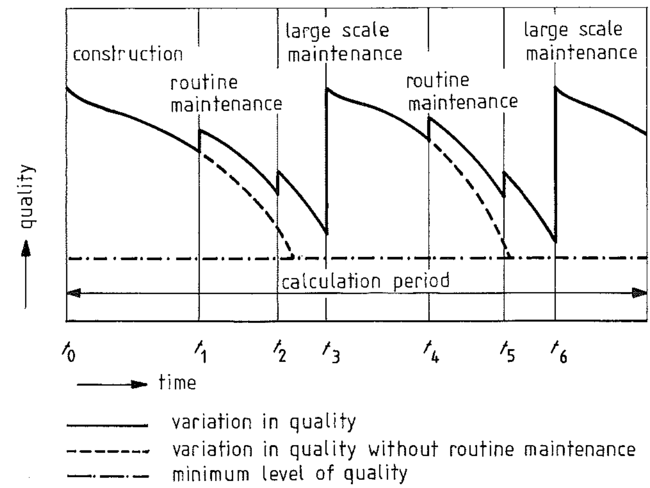


Fig. 166. Quality variation curves indicating when, during the period of the calculations, maintenance activities are planned or executed to ensure that the structure satisfies a certain level of quality. From [Leidraad Keuzemethodiek, 1988].

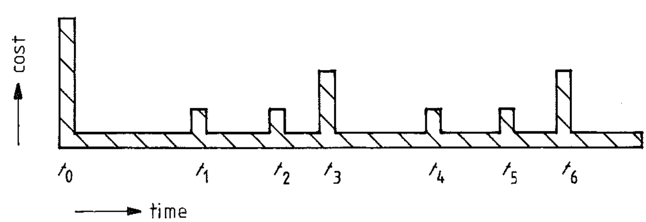


Fig. 167. Cost histogram indicating the capital required to finance structural maintenance and when this capital must be available. From [Leidraad Keuzemethodiek, 1988].

11. *The effects of ageing*

In the course of time a slope protection structure will age under the effects of wind and weather. Ageing is therefore not to be confused with the gradual failure of a poor design or a badly constructed structure or the increased need for maintenance. Ageing is the gradual change, positive or negative, in one or more of the structural parameters which affects the functioning of the structure. A good design should anticipate the effects of ageing as accurately as possible. It is however not easy to include quantifiable ageing effects in the design. Some points to consider are discussed below, see also Section 8.7.3.

Ageing of the cover layer:

(+) The adhesion between columns which have been washed-in with granular material can change (in the longer term) under the effect of wave loads. By selecting the washed-in material correctly these adhesive forces can be increased and the strength of the cover layer reinforced. If the material is not correctly selected there can be a local loss in adhesion. The choice of washed-in material must be considered very carefully at bends where joints are wider.

(±) The washed-in material can itself suffer from ageing. The performance of the material can be affected both negatively and positively if it is crushed under mechanical or thermal loads or if it becomes cemented.

(–) The permeability of the cover layer is reduced if sand or silt from nearby areas gets into the joints. As a result there can be an increase in pressure head difference on the cover layer during wave action.

(0) The presence of plants, shellfish and other animals in the joints can increase the adhesion between blocks. Roots extending into clay sublayers can increase local differences in the moisture content causing the material to dry out and crack. This indirectly leads to a reduction in stability. The vegetation may however be removed by heavy wave action and cracks in the clay layer may be quickly closed by swelling; it is impossible to quantify these aspects in the design.

(–) Woody plants can break the adhesion between blocks.

Ageing of the sublayers:

(–) Most forms of sublayer ageing are the result of faulty design or the incorrect choice of material, for example, the degeneration of the filter material, and the wearing or clogging up of the geotextile. Alternative filter materials, particularly as minestone or slags containing metals, should only be used if they comply with guarantees and quality assurance checks.

(+) The penetration of sand and silt into the filter via the (joints in the) cover layer can cause ageing. The effect is generally positive that is, not permanent, since this material will tend to be flushed out in the design conditions. The effect can therefore be neglected.

12. *The role of residual strength in the design*

Slope protection structures should be designed such that the probability of failure is acceptably small. This probability is, mainly, dependent on the extent to which the structure continues to function after initial damage, that is, its residual strength. This can be taken into account in the design, see also Section 13.4.
The following aspects are important in connection with residual design:

(0) After initial damage a cover layer of polygonal columns probably maintains its form somewhat longer than a cover layer of cubes. There are examples however where damage to polygonal columns has spread rapidly.
(0) Damage to rough slopes may spread more quickly than damage to smooth slopes.
(+) The inclusion of a geotextile in the structure delays the spread of damage.
(+) A cohesive sublayer of clay is, traditionally, the most widely used method for giving a structure residual strength. Other cohesive materials such as bitumenized sand, sand cement and sand asphalt can, in certain conditions, also contribute to the residual strength.

13. *Rough versus smooth slope*
A traditional and widely used way of reducing wave run-up and therefore crest height and dike volume is to increase the roughness of the slope. The effect under design conditions is nevertheless limited, mainly because:
(0) The largest part of the rough slope is below water and very often the run-up is on a grass slope above a storm flood berm.
(0) The freedom of choice of block shape is generally too limited to be able to make the surface sufficiently rough to satisfy the specification give in Section 7.2 .
(–) A disadvantage of increased slope roughness is that the forces on surface elements are increased and damage may occur earlier and spread faster.

Chapter 12

EXAMPLES OF DAMAGE TO PITCHED BLOCK REVETMENTS

12.1 Introduction

This chapter discusses the experience acquired over a number of years with cover layers of pitched blocks, see for example Figure 168. This experience is drawn from damage inventories prepared by the Road and Hydraulic Engineering Division of the Netherlands Ministry of Transport, Public Works and Water Management. First a summary is given of the data collected. The damage to transition structures is then discussed briefly. In conclusion the possibility of explaining the damage using the Preliminary Design Method, is considered for a number of cases.

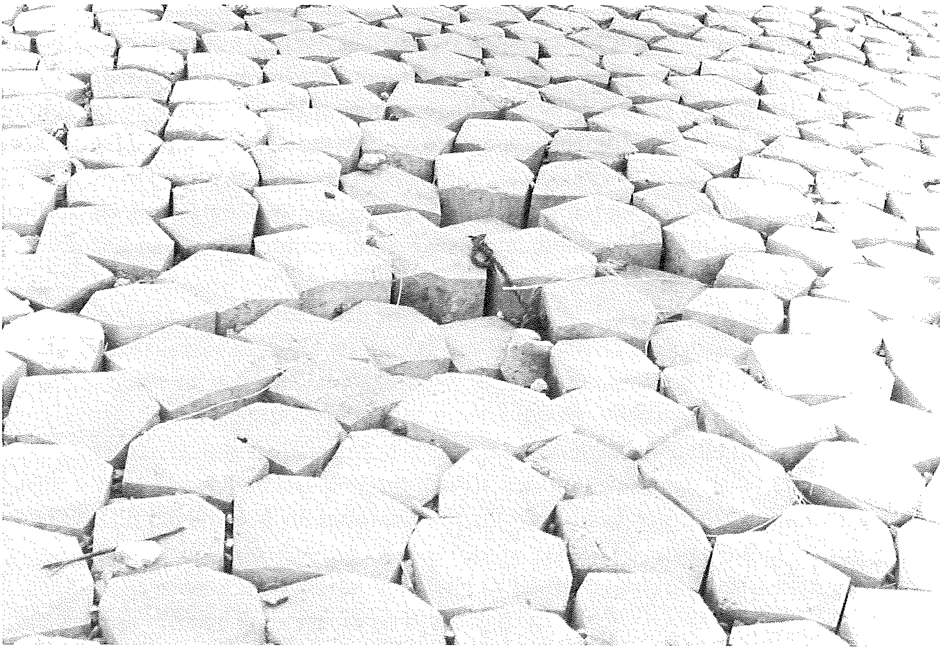


Fig. 168. A revetment of polygonal columns showing some settlement.

12.2 A review of damage inventory data

In 1975 the then Netherlands Centre for Research into Hydraulic Structures began to make an inventory of storm damage to slope revetments. This inventory was commissioned by the Road and Hydraulic Engineering Division of the Netherlands Ministry of Transport, Public Works and Water Management. Since this year to 1992 725 reports of damage to revetments were received, 85 of which had concrete cover layer

elements. Detailed information was analysed for a limited number of cases. Of the 85 revetments 71 had concrete block elements and 14 had columns. Damage to the concrete block cover layers ranged from the local erosion of clay under individual blocks to damage over an area of 1.300 m^2, the latter occurring in 1978 on a dike on the Old North Beveland Polder. In 1988 the South Wall of the Maasvlakte was damaged extensively when the design conditions were exceeded. In this case damage to the concrete column cover layer ranged from washed-in material being flushed out locally to damage over an area of 10.000 m^2.
Most of the damage to concrete elements 75 cases was reported in the 1980's, only 10 cases being reported earlier. It should be noted that ageing was not included in the inventory if areas, which had deteriorated for this reason, had been repaired before the storm damage occurred.
Often a particular section of dike was damaged on only one occasion during the period of the inventory. In some cases however a dike was damaged at various locations and by more than one storm. A revetment of concrete blocks was damaged on one section of a particular dike during nine separate storms. It appears however that generally only a limited number of revetments are damaged regularly. The areas of concrete element revetment damage in the period of the inventory is given in Table 11.

Table 11. Observed damage to concrete element revetments

area of damage (m^2)	number of areas damaged
0–10	59
10–100	20
100–1,00	4
1,000–10,000	2
total	85

12.3 Damage at transition structures

Damage often develops at transition structures. The design of these structures should therefore be prepared very carefully, see Chapter 10.
Damage can occur because, for example:

- the materials/designs are not entirely suitable (a concrete retaining wall which is not deep enough);
- the construction is incorrect (the wall is placed on coarse material at a transition instead of on clay;
- if, for example, the soil conditions are difficult compacting clay satisfactorily near to transition structure.

  These types of problems should be considered at the design stage and solved to ensure so that the transition structure is at least as strong as the adjacent slopes. In practice it is advisable to attach a higher degree of safety to transition structures and special attention should be paid, for example, to transitions between underlying filters, geotextiles, etc.

The damage inventory indicated six different types of damage which can occur at transitions. Two such types relate to transitions between tapered concrete blocks. Because of the shape there is little if any adhesion between blocks which can, as a result, be lifted out of the slope. Three types of damage relate to transitions between a basalt revetment and one of concrete blocks on clay. At two locations the clay had been eroded from under the blocks and cavities formed. At the third location cavities had developed in the basalt revetment and as a result columns were lifted out. This damaged the concrete retaining wall, the damage spreading to the concrete blocks on the clay. Finally there was one example of a concrete column revetment where the sand asphalt slope above the transition was damaged in a similar way, that is, by cavities developing underneath.

12.4 Damage case studies

A number of cases of damage are described below and, where possible, simple calculations have been carried out to determine why the damage occurred. Some of these cases are discussed in detail in Part 20 of the series of reports which deal with block revetment studies, see [Bezuijen et al., 1990, Appendix II].

a. *The harbour dike at Oterdum*
A dike revetment of concrete blocks was damaged during the storm which occurred on 2 to 3 April 1973, [Burger, 1985a]. The size of the blocks in this case is 0.5×0.5 m, with a thickness of 0.20 m, see Figure 169. The relative volumetric mass $\Delta = 1.35$. The revetment lies between NAP −0.20 m and NAP +2.93 m, at a slope of 1 : 2.8. Under the cover layer is a filter layer of minestone, 0.6 m thick. Between the filter layer and the blocks is a filler layer of 5/15 mm gravel, 0.1 m thick.
During the storm the water level rose to NAP +2.94 m. Wave heights were estimated at between 1.1 and 1.5 m with a period of 4.2 to 4.6 s. The waves approached the structure at about 45°.
The damage comprised blocks lifted partly out of the slope, blocks lifted completely out of the slope and blocks which had settled. The damage occurred mainly in the fourth and fifth row below the transition between the blocks and the asphaltic concrete.
The dike was of fairly recent construction. As demonstrated below the damage to this structure could have been expected because of the small block thickness. The revetment was repaired using blocks with holes. As far as is known, no more damage has occurred, although the block thickness, despite the holes, is in principle, insufficient.

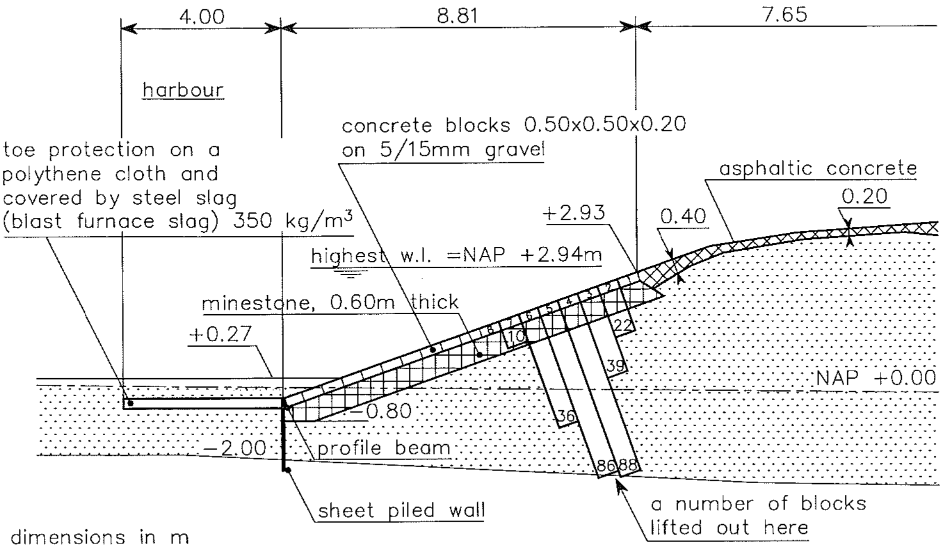


Fig. 169. The harbour revetment at Oterdum.

The most likely damage mechanism was the lifting out of loose blocks by the excess pressure under the blocks. The following parameters apply to the conditions when the damage occurred:

$\xi_{op} = 1.7$ to 1.8,
$H_s/\Delta D = 4.0$ to 5.4.

According to the Preliminary Design Methodology this structure can be considered as "usual", see Figure 103 and damage can be expected if $\xi_{op} = 1.8$ and $H_s/\Delta D$ is between 2.5 and 5.5, see Figure 107. The damage which occurred can therefore be explained using this approach.

b. *Philips Dam*

The revetment on the Philips Dam was damaged during storms on 13 November 1987 and 3 January 1988 [Evaluation of the Krammer Dam, 1988]. The damage occurred on a revetment of concrete columns constructed between NAP +0.50 and +2.00 m, see Figure 170. The revetment slope is 1:4 and the thickness of the cover layer 0.20 m (Δ estimated at 1.35). The revetment is laid on a filter comprising minestone, layer thickness 0.50 m, and rubble, layer thickness 0.20 m (grain size: 25/60 mm). This layer of rubble is considered to be too thick. A revetment of "Haringman" blocks (concrete blocks) on clay lies between NAP +2.00 m and +4.00 m. The transition between the two types of revetment is in the form of a concrete retaining wall and a strip of non-woven geotextile. During the storms of 3 January, 1988 the water level reached NAP +2.45 m. Wave were heights

estimated to be 0.85 m. The damage occurred locally, columns being more or less completely lifted out of the slope of the upper revetment. At one location a Haringman block was pushed out of the second row above the transition.
Generally the blocks were laid mechanically but at the bend in the dike they were laid by hand. As a result the joints here are wider and the filler material could be flushed out. Wider joints also occurred at the toe structure which was underdesigned. As a result the revetment could slide a short distance down the slope. The most probable damage mechanism was the lifting of a (loose) block by excess pressure introduced by the processes described above. In view of the location, the wave steepness was estimated to be 5% and, therefore:

$$\xi_{op} \approx 1.1,$$
$$H_s/\Delta D \approx 3.3.$$

According to the Preliminary Design Methodology this structure can be considered as "usual", see Figure 103, and damage can be expected if $\xi_{op} = 1.1$ and $H_s/\Delta D$ lies between 3.5 and 7.5, see Figure 107. The damage was therefore caused by waves of a lower height than indicated by the this approach.

c. *South wall (Maasvlakte)*
The concrete column revetment here was damaged extensively during storms in the period 28 February to 1 March 1988 [Hernandez et al., 1988]. The revetment lies between NAP and NAP +6.24 m at a slope of 1 : 8, see Figure 171. The revetment is 0.35 m thick and has a volumetric mass of 2.300 kg/m^3 (Δ = 1.23). The filter layer under the cover layer is formed of silex (grain size 30/90 mm) forms. The filter layer thickness is 0.40 m.
During the storm period the water level rose to NAP +2.22 m. The wave height, some distance from the structure, was $H_s \approx 3.8$ m with a period $T_p \approx 11$ s. The toe of the structure comprises a 15 m wide strip of rubble at NAP, the foreshore falling away fairly quickly to NAP –6.5 m. The rule of thumb given in Section 6.2.2 indicates a ruling water depth of 4 to 5 m, see Figure 49, and H_s = 2 to 2.5 m at the toe. During the storms an area of 10.000 m^2 of revetment was destroyed completely, see Figure 172. Because of the scale of damage it is not possible to say at precisely at what level the initial damage occurred. Steel (blast furnace) slag had been scattered on the cover layer. Because of silt and the hydraulic bonding of the slag it appears that the permeability of the revetment was lost completely, while at the same time the underlying silex filter remained fairly coarse.
The failure mechanism appeared to be the pressure underneath the revetment exerted on a part or the whole of the slope. This mechanism was possible because the cover layer had become impermeable due to the sealing effect of the steel slag and the silt.
The damage occurred under the following conditions:

$$\xi_{op} = 1.3,$$
$$H_s/\Delta D = 4.6 \text{ to } 5.8.$$

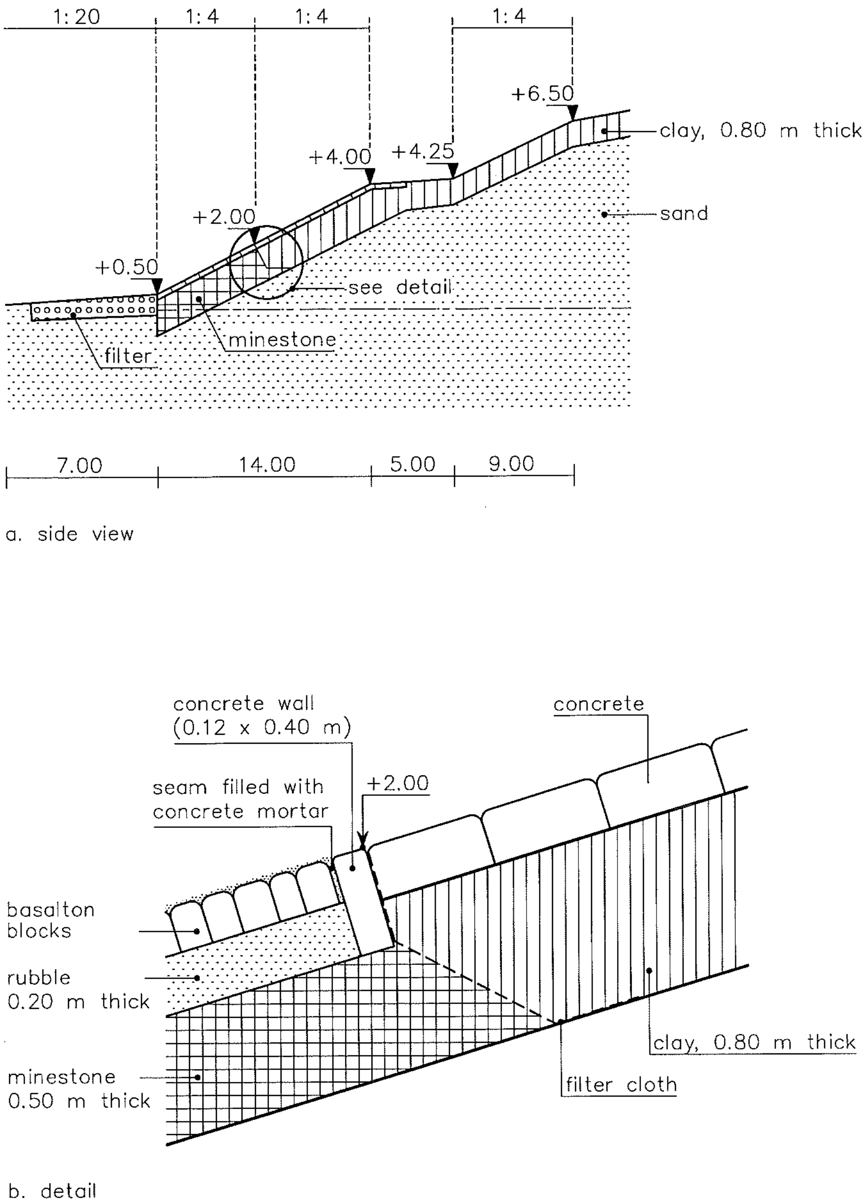


Fig. 170. Philips Dam.

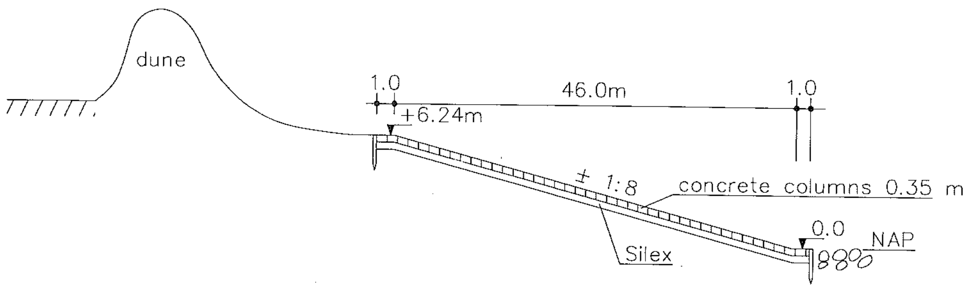


Fig. 171. South wall (Maasvlakte).

In view of the very low permeability of the cover layer the structure has to be considered "unsuitable". Because this aspect is not covered in Figure 103 the Preliminary Design Methodology has been applied. According to Figure 107, for $\xi_{op} = 1.3$, if $H_s/\Delta D$ lies between 3.1 and 6.7 damage can be expected. From the calculations it appears therefore that the structure was not designed properly taking into account the sealed cover layer and the relatively open filter layer.

Fig. 172. Damage to the Maasvlakte south wall (the rubble layer on the toe is to the left, the remains of the revetment are to the right).

d. *Perk Polder*

The flood protection works on the Perk Polder (along the Western Scheldt) are regularly damaged. Damage occurred, typically during the storms of 3 March 1984 and 20 October 1986 [JOHANSON, 1987]. The water levels in these storms rose to NAP +3.85 m and NAP +4.02 m respectively. The revetment, which was damaged, comprises concrete blocks 0.50×0.50 m^2, see Figure 173. The blocks are 0.20 m thick and are laid on a clay layer 0.80 m thick. The slope is about 1 : 3. The bottom edge of the concrete block revetment is at NAP +3.00 m and is bounded by a concrete retaining wall, 0.40 m deep. Immediately below the retaining wall there is a revetment of basalt on a filter of broken building rubble on layers of bricks. Since the flood protection works face eastwards the wind-induced wave action is limited. During a storm in 1983 observations were made from which it appeared that, for a water level of NAP +4.13 m, (fetch generated) waves of 0.6 m were able to develop. The revetment was damaged during a similar storm on 3 March 1984. Ship wave action is also important in this case, particularly in view of the frequency with which they occur. The height of these waves estimated to be up to 1.00 m. This implies that the transition is attacked regularly by ship waves during the flood tide (mean High Water of Spring Tides is NAP +2.68 m and mean High Water of Neap Tides is NAP +1.88 m).

Several blocks were lifted out of the slope during the storm and at these places erosion channels were found in the clay. Samples of the clay were tested and from the results it appeared that the sand fraction in the clay was a little on the high side. It also appeared that the concrete wall in the transition structure did not penetrate the clay everywhere. The construction of this wall and compaction of the adjacent clay had caused a problem during construction. The damage mechanism in this case was the erosion of clay under the blocks. This led to settlement in some areas and blocks being lifted out in others. The damage ultimately was caused during a storm with waves pupendicular to the dike.

e. *Oester Dam*

On 23 March 1981 a revetment of concrete blocks on the Oester Dam in the Eastern Scheldt was damaged [VAN BAALEN, 1981]. The $0.50 \times 0.50 \times 0.20$ m^3 blocks lie between NAP +1.80 m and at the time of the damage, there was a berm at NAP +5.00 m, see Figure 174. Between NAP +1.80 m and NAP +2.90 m the blocks are laid on minestone, layer thickness 0.70 m, and a filler layer of broken gravel 11/32 mm, layer thickness 0.10 m. The blocks on the upper part of the slope are laid directly on clay, layer thickness 0.80 m. The slope of the revetment is 1 : 4. The flood protection works were under construction when the storm occurred and the outer berm had not been completed. As a result water collected to a depth of 0.60 to 0.80 m above the berm.

The original design of the transition from minestone to clay, as shown in the figure was later modified. A geotextile was introduced between the clay and the minestone and concrete retaining wall was inserted as shown in Figure 170.

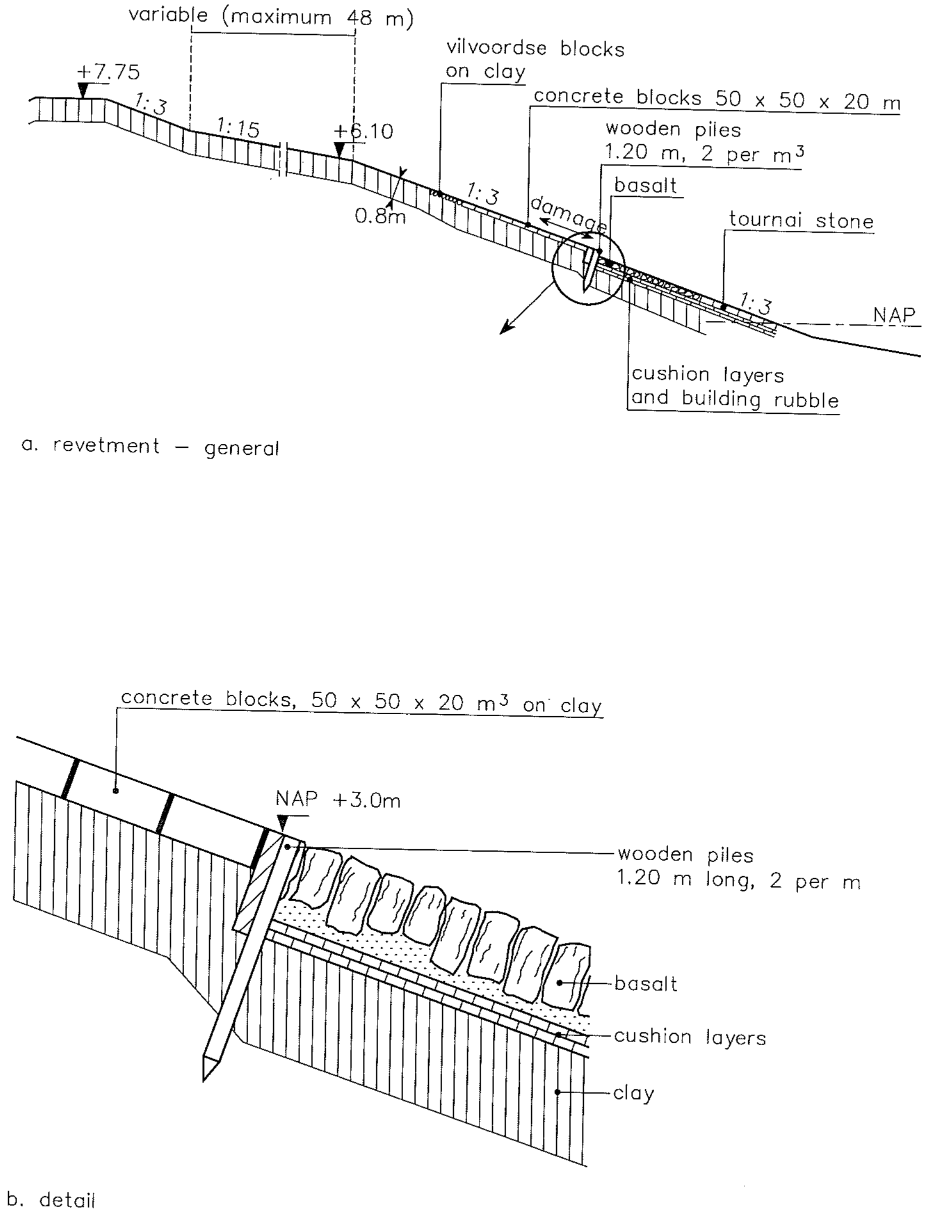


Fig. 173. Perk Polder Flood Protection Structure.

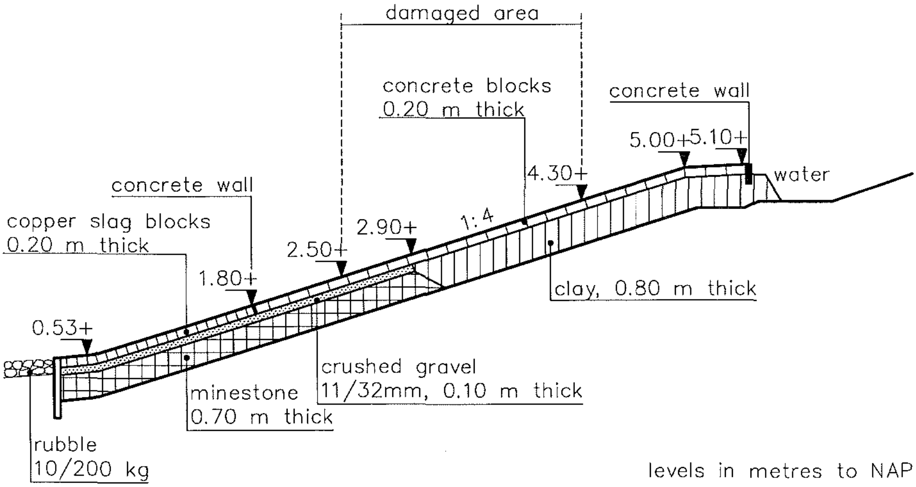


Fig. 174. The Oester Dam.

From the wave records it appears that during the storm waves reached a height equivalent to an H_s of 0.60 to 0.70 m. The water level rose to a maximum of NAP +3.25 m. During the storm about 35 blocks were lifted completely out of the slope and about 15 blocks were partly lifted out. In general the damage occurred between NAP +2.50 m and NAP +3.60 m. In the damaged area clay was flushed out. In places where the slope appeared to have been unaffected, 0.40 m deep erosion channels were found during the repair work.

One possible cause of the damage was the fact that water collected in the area where the berm had not been constructed yet. This water contributed to the erosion channels being blocked with sand which in turn led to a build up of water pressure which lifted the blocks. The damage mechanism appears to have been the lifting of the blocks by excess pressure which developed underneath, most probably, because of the erosion of the underlying clay. The resistance of the clay layer to erosion was negatively affected by amounts of sand confined in clay.

The poor quality clay was subsequently replaced.

12.5 Repairing damage

In order for repairs to be satisfactory it is important to determine and analyse the cause of failure. Sometimes the analysis reveals that the structure was unable to resist the wave action. Damage can also be the result of poor design and too light a structure being selected. This situation can also arise if the boundary conditions change. Damage can also occur if the structural quality deteriorates with time.

CHAPTER 13

SAFETY CONSIDERATIONS

13.1 Introduction

After the flood disaster in the Netherlands in 1953 it was decided to increase the level of safety provided by the flood defences several times or to monitor it more strictly. Before the disaster, for example, the height of dikes was calculated at only 0.5 to 1 m above the level of the highest observed water level.
The Delta Commission has set the design water level for flood protection works for the densely populated western part of the country at NAP +5 m at the Hook of Holland. This is in agreement with the assumption that "the most vital part of the country should be secure against storm floods which have a frequency of exceedance of 1% per century. For other parts of the Netherlands a greater frequency of exceedance has been accepted", [Report of the Delta Committee, Part 1, 1960]. Levels with a similar exceedance frequency as the design water levels at the Hook of Holland (the base level) have been set for the whole of the Netherlands together with the associated design levels. It is now realized that safety can only be kept in proportion if a certain allowable flood probability is set for areas which must be protected by hydraulic structures against flooding, see Figure 175.
The flood protection works around such an area can comprise a variety of structures, for example, dikes, dunes, sea walls and sluices. Each structure has a probability of failure and therefore an associated probability of flooding. The [Water Defences Act, draft 1988] is a step towards a safety standard based on flood probabilities. For all Dutch areas needing flood protection this law gives "the safety standard as the average probability of exceedance per year of the highest high water level to be used for calculating the primary flood protection works, taking into account the other factors which affect the flood protection capacity". In accordance with the recommendations of the Delta Commission the probability of exceedance is an average 1/10,000 per year, for example, for the Province of South Holland.
By defining safety in this way, the probability of flooding of an area surrounded by a dike is not taken into account. The possibility of setting a different probability of flooding in the near future however remains. In this case the new flood probability will not be a new safety standard, but only an interpretation in terms of the existing water level standard in which the basic assumption is the reinforcing of the level of safety to be achieved.

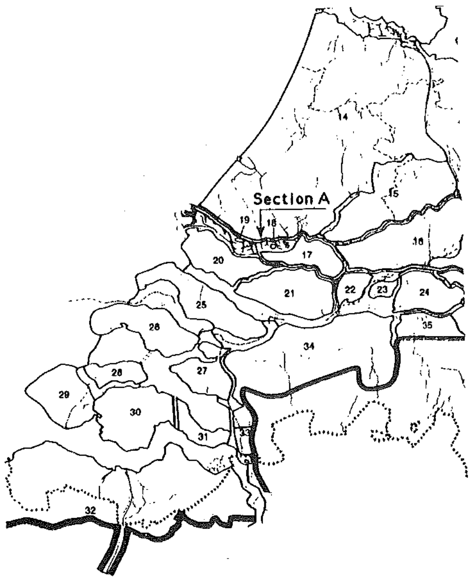


Fig. 175. Dike rings of the southwest of Holland.

It is not yet possible to base designs on flood probability because the physical processes involved have not yet been fully quantified. In order to be able to apply the safety of dike revetments in practical terms it is necessary to deduce an allowable failure probability for the structure. To achieve this the safety standard for flood protection works has been translated into an acceptable failure probability of the dike revetment (including possible filter and clay layers) in Section 13.3. Section 13.4 then examines the practical possibilities for the designer to achieve a satisfactory but not disproportionate level of safety [BEZUIJEN et al., 1990, page 185].
In conclusion, sensitivity analysis is considered in Section 13.5, enabling the results of the calculations to be evaluated. Because of the complexity of the design this chapter can only indicate points for consideration by the designer of pitched dike revetments. The given (allowable) probabilities of failure can only be indicative and aim to give guidance when safety aspects are being considered. No attempt has been made to set standards because this is still problematic. It is the designer who has to select the relevant points for a given situation from this design philosophy. Although the structure can be damaged in various ways, for example, by tourists who lift out blocks or heavy ice loads, this manual assumes that the only danger is flooding caused by extreme hydraulic loads. If there is an important flood probability which is caused by the exceptional loads given in Section 6.6, then the considerations in this chapter cannot be applied.
This distinction enables safety to be examined in a practical way for a number of cases. A generally applicable approach unfortunately is not available.

13.2 Stochastic variables

The concept of safety and failure probability is based on the fact that nothing is certain. Each parameter not only has a mean value, but also a distribution, for which no exact values can be given. Such parameters are referred to as being stochastic.
The crest height of a dike serves as an example. In the case of a protected dike compartment, with no wave action, the height is a function of the design water level (including atmospheric effects, seiches, rise in sea level, etc) and the amount of settlement of the dike since construction. Even if these uncertainties are neglected, there is still a stochastic problem.
Despite attempts to give the dike the height required, unavoidably it will be a little higher or lower (clearly the lowest point of the dike is relevant here). In this respect all that can be said is that the height of the dike which is to be built in the future will "probably" lie between certain upper and lower limits. In a deterministic design the lower limit of the crest height is selected at the design water level plus a safety margin (flood warning height), so that the highest level to be protected is, or lower than, the lowest imaginable crest height (at the cross section with the lowest crest height). The probability of overtopping is then most likely acceptably small.
In contrast if the mean value of the crest height to be built is selected on the basis of the expected water level the probability of overtopping, that is, the design conditions are exceeded, is about 50%. This is not acceptably small and therefore in this case the design should not be based on mean values.

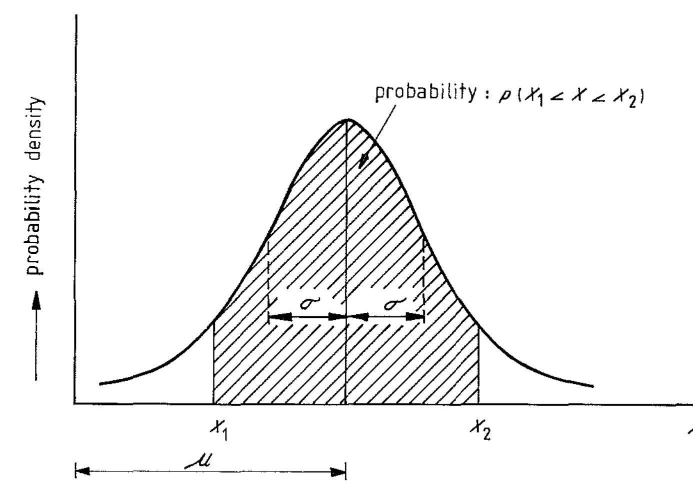


Fig. 176. Normal distribution.

Probabilistic designs are not worked out with upper and lower limits because these are, in fact, rarely used. Distribution functions are used, (a normal distribution is shown in Figure 176). The area under the distribution function, between two limiting

values of the variable, is equivalent to the probability that the variable has a value between these limits. A normal distribution is characterised by a mean value (μ) and a standard variation (σ). The difference between a value at the upper limit of a reliability interval of 95 % and the mean value is about $2 \cdot \sigma$ for a normal distribution, see Figure 176: $x_1 = \mu - 2\sigma$; $x_2 = \mu + 2\sigma$.
If a "sensitivity estimated" upper and lower limit for the value of a parameter agrees with the limits of a "reliability" interval of 95%, the mean value and the standard variation (distribution) can be calculated as follows:

$$\mu(x) = \frac{x_{\max} + x_{\min}}{2} \tag{68}$$

$$\sigma(x) = \frac{x_{\max} - x_{\min}}{4} \tag{69}$$

where:

x = parameter
$x_{\max}$ = estimated upper limit of the value of x
$x_{\min}$ = estimated lower limit of the value of x
$\mu(x)$ = mean value of x
$\sigma(x)$ = standard variation (distribution) of x

13.3 Allowable failure probability for a revetment

Section 13.1 states that a flood protection structure must be able to resist a particular hydraulic load. Since there is always a possibility that the structure will fail there is a real possibility that it cannot satisfy this requirement. This can be interpreted as a requirement that the probability of failure of a dike must be (much) smaller than the stated probability of exceedance of the water level.
This manual does not discuss dikes but only dike revetments. In order to make the correct decisions during the design of the revetment it is necessary to estimate a practical value for the allowable probability of structural failure. It is therefore important that this failure probability is properly related to the probability of failure of the dike (as a whole). The revetment structure is defined in this connection as the cover layer, together with the filter layers and the clay layer. The revetment will fail if the sand core of the dike is attacked directly by waves. The dike fails if it can no longer hold back the water.
The allowable probability of failure of the revetment cannot be the same as that of the dike because the dike can also fail for other reasons, such as overtopping. In addition the dike can still afford protection even when the sand core is under direct wave attack. The most important failure mechanisms for the revetment are summarized in Figure 177 together with dike overtopping. The figure also shows that damage to the revet-

ment can occur at relatively low water levels at the beginning of a heavy storm and extend upwards up the slope. In this respect a relatively low level is, for example, mean high water. This is explained in Section 13.4.4.

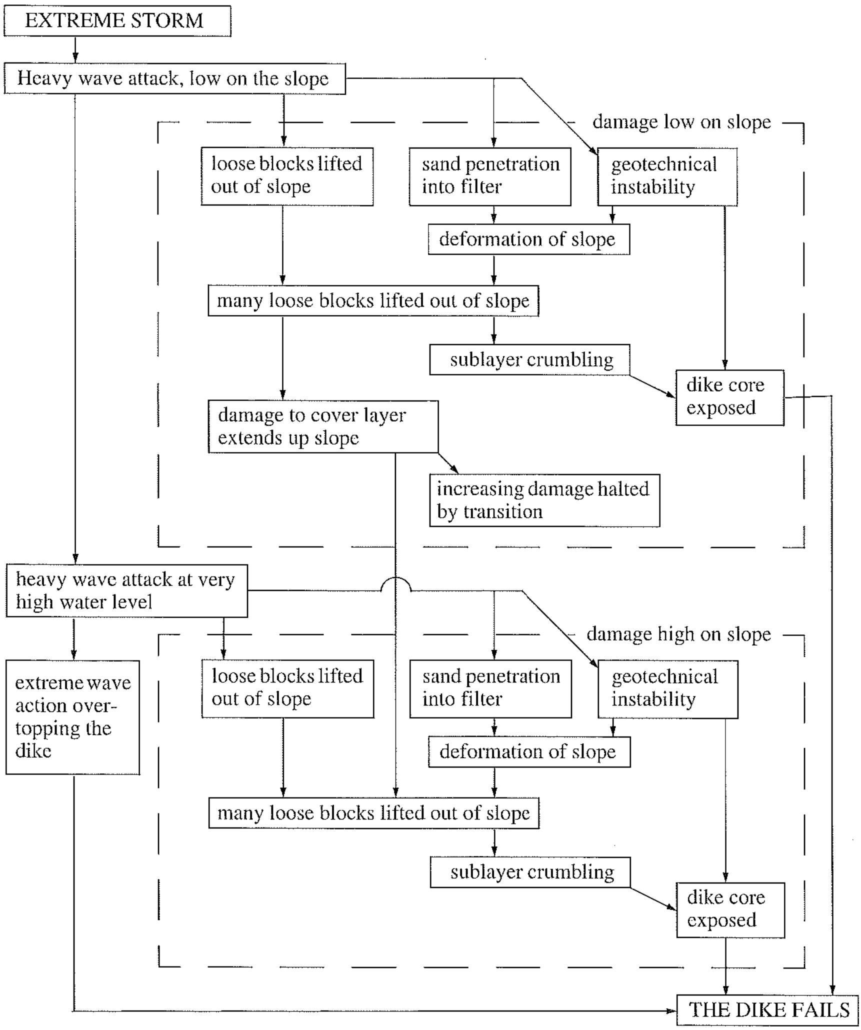


Fig. 177. The most important hydraulic load failure mechanisms.

Combination of the failure probabilities for all the mechanisms gives a failure probability for the dike (the way in which the probabilities are combined is determined by the extent to which they are interdependent). Assuming that several failure mechanisms have an unlikely probability of zero the implication is that the allowable probability of failure for the revetment is much smaller than that for the dike as a whole. From place to place, however, depending on the dike cross section, this can

vary greatly. In the exceptional case that the dike has a high residual strength after the failure of the revetment and also that the other failure mechanisms for example wave overtopping have only a very small failure probability, the allowable failure probability for the revetment itself can be larger than that for the dike as a whole. This case is not considered here.
Provisional values for the allowable probability of failure for a revetment, which is 10 to 100 times smaller than the prescribed probability of exceedance of water level, are:

1. If the design water level has an average probability of exceedance of $1 \cdot 10^{-4}$/year, the allowable probability of failure for the revetment is $1 \cdot 10^{-6}$ to $10 \cdot 10^{-6}$/year.
2. If the design water level has an average probability of exceedance of $2.5 \cdot 10^{-4}$/year, the allowable probability of failure for the revetment is $2.5 \cdot 10^{-6}$ to $25 \cdot 10^{-6}$/year.

These allowable probabilities of failure relate to a revetment on an individual dike section. A dike with a pitched revetment must therefore be subdivided into (small) sections. For a straight dike, subjected to similar loads along its length and having the same type of revetment, the total failure probability of the sections, together, is about equal to the largest failure probabilities of the individual sections. This is because the "failure of section x" depends, to a large extent, on the "failure of Section y", because the probability of failure is dominated by the probability of an extreme load and the uncertainties in the numerical model in which all sections are the same.
It is sufficient therefore to determine the largest overall probability of failure for all the sections, namely the weakest section. This failure probability will certainly not be greater than that of a theoretical section with:

- a loose block at the point where the pressure head difference on the cover layer is the largest;
- a most unfavourable value for material parameters in relation to stability.

When estimating the most unfavourable value for material parameters it should be borne in mind that the stability of a loose block is determined by the average value of the parameters of at least 20 square metres of revetment. This approach overestimates the failure probability which should not lead to excessive over designing. This implies that the *average* value of the material parameters, for example, the joint width, grain size of the filter, etc, of dike sections of about 10 to 50 m must be taken into consideration. An estimate, which is unfavourable for stability, should be made for all parameters when determining this *average*. Clearly there is uncertainty attached to these estimates which leads to a mean value and a distribution.

Example:
Twenty shipments of filter material are needed to construct a dike. The shipment with the largest average value of the grain size, D_{f15}, governs the design because the larger

the value of D_{f15}, the smaller the stability of the cover layer and the interface between the filter and the base. Material with $D_{f15} = 3$ mm is specified but, for the coarsest shipment, D_{f15} is estimated to be 3 to 5 mm. Using Equations (68) and (69) it follows that: $\mu(D_{f15}) = 4$ mm and $\sigma(D_{f15}) = 0.5$ mm.
For dikes subjected to variable loads the section with the largest load should be considered. In addition if the dike has various types of block pitching the calculation can be limited to the section subjected to the largest loads combined with the weakest strength. If the probability of failure of this section is acceptable, then the probability of failure of the revetment of the dike as a whole is acceptable. Clearly this can lead to the over-design of the more protected sections of the dike. A subdivision into smaller dike sections is recommended where this is the case. It should be noted that the acceptable probability introduced here is only indicative and its application does not guarantee an acceptable flood probability. It is only intended to be a guide to put the relevant safety aspects into perspective.

13.4 **Residual strength and design procedures**

13.4.1 *Introduction*

Although individual blocks can be lifted out of the pitching this does not necessarily lead to failure of the revetment. The term "residual strength" becomes applicable when one single block has been lifted out of the pitching [BEZUIJEN et al., 1990, page 194]. The residual strength of a revetment can be expressed in terms of the length of time between the initial damage to the structure (loose blocks lifted out of the slope) and the exposure of the sand core. This period can be very long, for example, when there is a thick layer of good quality clay which does not readily erode. It appears therefore that, in a series of events, the strongest component governs the performance of the whole.
In this section a good clay layer is frequently referred to when discussing high residual strength. The reason for this reference is the relatively frequent occurrence of these clay layers, compared with layers of similar residual strength, such as a thick layers of bituminous sand. Where reference is made in this chapter to good quality clay, materials of a similar residual strength are also implied.
If the residual strength is so large that the structure can easily resist a heavy storm after a block has been lifted out this fact should be taken into account in the design of the revetment. In such a case the cover layer does not, in fact, contribute to safety and a thinner cover layer is sufficient to protect the clay against more frequent wave action then the extreme conditions referred to in the previous section.
The decision whether or not to include residual strength in the design has far-reaching consequences which are discussed below:

- *No residual strength*
  Basing safety on the cover layer alone implies that the acceptable failure probability of the cover layer must be equivalent to that of the components of the revetment structure (cover layer, filter layer, etc). The probability of failure of the cover layer must thus be very small, for example, 1/10 to 1/100 of the frequency of exceedance of the design loads. As a result a fairly thick cover layer is needed. The revetment structure is inadequate if it can be shown that blocks are frequently, for example, once in ten years, lifted out of the slope by waves. In these cases simply replacing the blocks is insufficient because this failure must be interpreted as the failure of the structure under wave loads which are lower than the design loads.

- *With residual strength*
  If safety is based on the residual strength this must be quantified in the form of the time between initial damage and the failure of the structure. At present this is not possible but investigations have been carried out into this aspect. In addition the structure has to be repaired before another storm of significance occurs, or the residual strength must be sufficient to also resist that storm (and all the following storms before the damage is repaired). It should be borne in mind here that, during very heavy storms, many revetments may be damaged and labour, equipment and materials must be ready to repair the damage quickly.

Although it is not possible to quantify residual strength precisely there are results from large scale model investigations from which provisional conclusions can be drawn [Burger, 1984 and 1985]:

1. Blocks – filter layer – minestone – sand:
   One to two hours after one block had been lifted out, the cover layer had been undermined to such an extent that the cover layer collapsed and, in a short time, was swept away. Whether the minestone could resist the heavy wave action sufficiently is not known.
   Provisional conclusion: the residual strength was insufficient and safety probably must be guaranteed by the stability of cover layer.
2. Basalt – filler layer – brick layers – good clay:
   Within one hour of a block being lifted out a large hole developed in the cover layer. The clay layer was then able to resist heavy wave action for many hours.
   Provisional conclusion: the residual strength was sufficient and it was not necessary for the block pitching to contribute to the safety of the structure.
3. Blocks laid directly on reasonable clay:
   The first block lifted out was quickly followed by more. The depth of erosion in the clay then increased at a rate of about 5 to 10 cm/hr.
   Provisional conclusion: designs based on residual strength calculations are risky and should therefore be avoided. Safety should therefore have been mainly guaranteed by the blocks.

For the above examples, therefore it appears that there was only residual strength when there was a thick layer of good quality clay. The clay specifications are at present the subject of research. Investigations have also been carried out into the residual strength of a minestone layer. It is possible that other materials, such as bitumenized sand, have a high residual strength but this has not yet been demonstrated. Two design philosophies are presented in the following sections which differ in terms of the size of the residual strength.

13.4.2 *Design procedure for structures with a negligible residual strength*

Dikes which form part of a flood protection system and therefore have an important safety function must satisfy the requirement that, under the design hydraulic conditions, the revetment has an acceptable small probability of failure. If the revetment does not include a thick layer of good quality clay (or a similar material) the level of safety will depend on the pitched block cover layer. The probability of failure of the revetment can be determined using probability calculations. These are, however, unnecessarily complex for comparing different design phase options. A much simpler procedure is therefore set out below:

Design procedure

Using a large number of failure probability calculations [KLEIN BRETELER and DE RIJKE, 1991] drew the conclusion that the acceptable probability of failure would not be exceeded if the following design procedure is followed for block pitching:

1. Determine the mean values for all the parameters describing the structure (joint width, $\mu(s)$, grain size of the filter, $\mu(D_{f15})$, etc), taking into account the factors described in Section 13.3.
2. Determine the design loads (storm flood level, significant wave height and period), for example, for an average probability of exceedance of 10^{-4}/year and calculate the characteristic value of the significant wave height:

$$H_{skar} = \mu(H_s) + 1.65 \cdot \sigma(H_s) \tag{70}$$

3. Estimate the standard variation of the joint width, $\sigma(s)$, and the grain size of any joint filler, $\sigma(D_{v15})$, see Equation (69).
4. Calculate the characteristic values, x_{kar}:

   – joint width:

$$s_{kar} = \mu(s) - 2.3 \cdot \sigma(s) \tag{71}$$

   – joint filler (if any)

   grain size:

$$D_{v15kar} = \mu(D_{v15}) - 2 \cdot \sigma(D_{v15}) \tag{72}$$

porosity:

$$n_{\text{vkar}} = 0.3 \tag{73}$$

5. Calculate, using the Analytical Design Method, the necessary thickness of the cover layer, see Section 8.4.3, using the above characteristic values and the other structural properties.
6. Correct the calculated cover layer thickness by multiplying with the model coefficient, $C_m = 0.8$. This correction coefficient is obtained by comparing the results of the large scale model investigations with those obtained with the Analytical Design Method [KLEIN BRETELER and DE RIJKE, 1991].

Example of characteristic value calculations

Assume that the significant wave height typically lies between 1.2 m and 1.6 m and that 1.4 m is the most likely value (mean value, μ), then:

$$\sigma(H_s) = (\text{upper limit} - \text{lower limit})/4 = (1.6 - 1.2)/4 = 0.1 \text{ m}$$

$$H_{\text{skar}} = \mu(H_s) + 1.65 \cdot \sigma(H_s) = 1.4 + (1.65 \cdot 0.1) = 1.57 \text{ m}$$

It should be noted that the design procedure, recommended above, is not the only one which can give a good design. An important unsolved problem is the size of the acceptable probability of failure. Because definitive decisions have not been taken about this, no definitive design procedure can be given. In this chapter only provision recommendations and considerations are presented.

[KLEIN BRETELER and DE RIJKE, 1991] demonstrated that a good design can be obtained without using the above procedure but using the equations and diagrams in Section 8.4.3 which give conservative results. There are two design procedures using the analytical method described in Section 8.4.3:

1. *Mean values*

   Designs using mean values for all parameters and a load with a selected frequency of exceedance give a revetment with a failure probability which is 1 to 10 times smaller than the selected load frequency of exceedance (NB: $C_m = 1$).

   As an example a load with an exceedance frequency of 10^{-4}/year, results in a revetment with a probability of failure of 10^{-5} to 10^{-4}/year.
2. *Characteristic values*

   Designs using characteristic values and a model coefficient, $C_m = 0.8$, as given in the procedure above, give a revetment with a probability of failure which is 10 to 100 times smaller than the frequency of exceedance of the load.

The second method is particulary suitable for dike revetments in which safety is mainly based on the cover layer. The first method is more easily used, if sufficient residual strength can be guaranteed.

13.4.3 *Guarantee of sufficient residual strength*

Experience suggests that the residual strength of a good clay layer is so large that it can resist a storm of long duration with heavy wave action. If this is the case the need for the cover layer of pitched blocks to contribute to the safety is then open to question. In such a case the function of the revetment is aimed more at the protection of the clay against frequent wave actin. During the design storm the revetment therefore has no role (with respect to safety) because the clay which gives the dike the necessary protection and guarantees the security. After a long period of wave action the high residual strength may decrease and the revetment deteriorate. Subsequent heavy storms can then be very dangerous for the structure.
When assessing the residual strength required it is also necessary to assess when the damage can be repaired because this determines the maximum period which can be allowed between wave attacks. It should be borne in mind that, based on experience, two or three days after a heavy storm there is the possibility of another heavy storm which will be more probable than one developing after many storm-free days. This probability may be two to three times larger than of the previous storm [Delta Commission Report, Part 2, 1960].

Example
A storm with an average frequency of once in 300 years (equivalent to that of 1953) can cause damage at so many locations that a considerable number of men, material and equipment must be ready to repair all the damage on the following day. The reason for this apparent haste is that two days after the first storm a storm with a frequency of once in 100 years can occur.
The possibility of two such storm events occurring can be estimated generally at:

$$p = (2 \text{ to } 3) \cdot \frac{1}{300} \cdot \frac{1}{100} \approx 10^{-4}/\text{year}.$$

The longer the time needed to repair the damage the larger the residual strength needed. The clay layer is essential for the safety of the structure and this layer must therefore be of a satisfactory quality. It is not possible to specify the quality, unfortunately, because the investigations into this aspect have yet to start.

Cover layer design procedure
Because the structural safety is based on the residual strength of the clay layer the design loads for safety are not important for the design of the cover layer. The design of this layer can be based on loads which occur relatively frequently. A choice can be made between a thin, economical revetment which requires much maintenance and a thicker more expensive revetment which needs very little repair. The revetment should be designed on the hydraulic loads which, from economic considerations, will not cause damage. A distinction should be made between the effects of different fail-

ure mechanisms because the replacement of a block is much easier than repairing an area which has settled because of sand penetrating the filter or because of geotechnical instability. Because safety is not in question it is sufficient to carry out stability calculations using the mean values of the various parameters as inputs, see previous section, "Method 1, Mean values". This also applies for the joint width between blocks, $\mu(s)$, and the grain size of any washed-in material, $\mu(D_{v15})$. This contrasts with the design parameters for a structure which has no residual strength.

13.4.4 *Combination of large and small residual strength in one cross-section*

Dikes can be designed with different types of revetments, in the tidal zone and the slope above, see Figure 178. It is possible to provide one structure with a thick, good quality clay layer (high residual strength) and the other with no residual strength. For this type of dike it is important to be aware that damage to the cover layer can develop in the tidal zone and then, with the growth of the storm and the water level, spread. With growing storm intensity and increasing water level, the revetment can then, be as it were "rolled up" from below. This process is also shown in the chart in Figure 177.

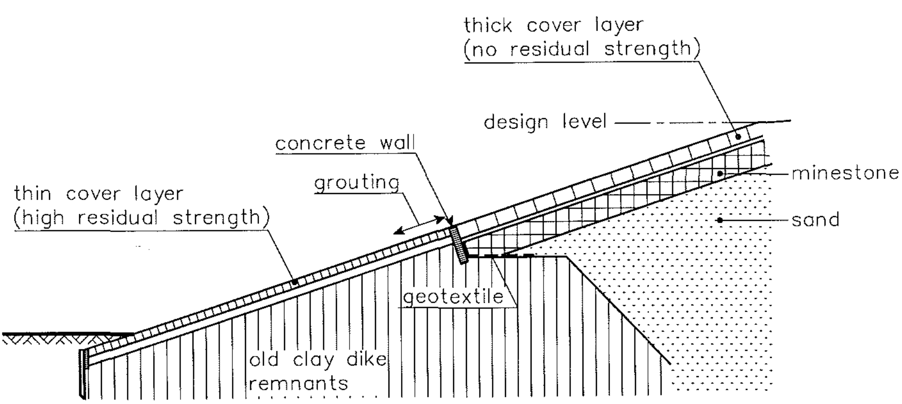


Fig. 178. Structure with a large residual strength in the tidal zone and a low residual strength at the design level.

Figure 179 shows how there can be considerable wave action at a relatively low water level in the build-up to the height of the storm. These waves attack at a level well below the design water level and are, perhaps, comparable to a once in one year or a once in ten year storm. The spread of damage can be stopped by a transition structure provided that this is sufficiently stable. Spreading will probably not be stopped if the transition is a concrete wall which is less than 50 cm high and has no row of piles.

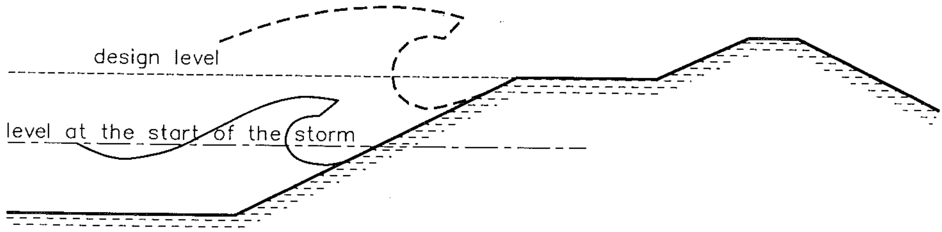


Fig. 179. Loads are applied low on the slope when the storm begins.

A structure with a high residual strength in the tidal zone can therefore only be combined with a structure with no residual strength, if the spread of damage upwards is unlikely. A heavy sealed transition should be constructed between the two revetments. This must be able to resist the sliding forces on the upper revetment and also considerable damage to the lower revetment. A row of piles with a concrete wall, or a wooden bulkhead can stop damage spreading. In addition a wide strip of grouted blocks adjacent to the transition structure can contribute to its stability.

13.5 Sensitivity analysis

The value of the calculations can be easily appraised using a sensitivity analysis. This enables the designer to judge if additional information is required about the hydraulic loads or the structural properties.
The sensitivity analysis can be divided into three parts which should be carried out in sequence:

1. Design of the structure.
2. Establishment of any parameters which are only known to a limited level of accuracy.
3. Redesign of the structure, with a modified value for one of the parameters established in Part 2.

Part 1 yields the starting points or assumptions for the sensitivity analysis and is probably carried out at the initial stages of the Preliminary Design. In Part 2 an estimate must be made of the possible range of values for the input parameters used in the design. In principle a distribution must be estimated for each parameter. In practice this part can generally be omitted for some parameters because the distribution is limited or because the parameters are known to have little effect on the design.
In Part 3 the design calculation has to be repeated for each of the input parameters. In each calculation the value of one parameter is modified and an assessment made of the effects on the ultimate design. A value which is well within the related reliability interval is therefore eventually selected.
The above procedure can be illustrated with the following example. A filler layer, thickness $b = 5$ cm (mean value), is selected. According to the recommendations in

Section 13.4.2, for structures with no residual strength and using Method 2 only the mean value is required for the thickness of the filter layer. A layer thickness of $b = 6$ cm is then selected and the structure redesigned. If it now appears that the block thickness should be much more than about 10% different to that for the 5 cm layer thickness it can be concluded that the design is sensitive to the size of this layer thickness. It is then interesting to obtain more information about this parameter. It may be possible to change the construction (method) and the material and so to determine the layer thickness more accurately.
The sensitivity analysis gives information about the reliability of the design, indicates the parameters about which more details are required and in addition identifies the structural components which should be constructed with very careful supervision.

Chapter 14

CHECKING THE SAFETY OF EXISTING STRUCTURES

14.1 Introduction

Although the design methods in this manual are principally aimed at new dike revetments, they can also be used to check existing revetments. This aspect is considered in the present chapter. The chapter is limited to guidelines for pitched block dike revetments; details are given in the [Leidraad Toetsing, 1992]. Safety checking procedures should conform with the guidelines given in the latter publication.
There are three levels at which safety can be checked. These levels relate to the various design tools discussed in Section 8.4.1:

- general checking (use of the Preliminary Design Method, Section 8.4.2),
- detailed checking (based on the Analytical Design Method, Section 8.4.3),
- advanced checking (based on the STEENZET/1+ numerical model).

These levels of testing are described in Sections 14.3, 14.4 and 14.5.
General checking is always used first because it is the easiest to apply and requires the least information about the structure. Detailed checking is only used if no preconceived opinions can be expressed about the structure, for example, the $H_s/\Delta D$ value is in the "grey" area and the stability is doubtful. A "grey" zone is defined for this method in Section 14.4 as an area in which a definitive judgement is impossible. The zone is smaller than that for general checking, as shown schematically in Figure 180. Finally advanced checking methods can be applied.
With old structures the specifications at the time of construction differ from today. Many years ago the design process, which led to a "stable" structure, was always to use a heavier revetment if the old revetment appeared to be insufficiently stable. In this way practical knowledge was built up based on damage and failure. Central to this design process was the desire to obtain a structure which, with an acceptable amount of maintenance, could withstand relatively frequently occurring storms (the design was based on the "serviceability limit state"). It is however doubtful whether the structure obtained could resist hydraulic loads with an exceedance probability of, for example, 10^{-4}/year. Today the primary specifications for a structure are different to those applied in the past.
The desire to keep maintenance within limits must be viewed within the context of the requirement that a structure should fulfil the flood defence function in very extreme conditions (flood prevention). Designs, at present, are aimed at the "ultimate limit state", the latter being subject to safety checks.

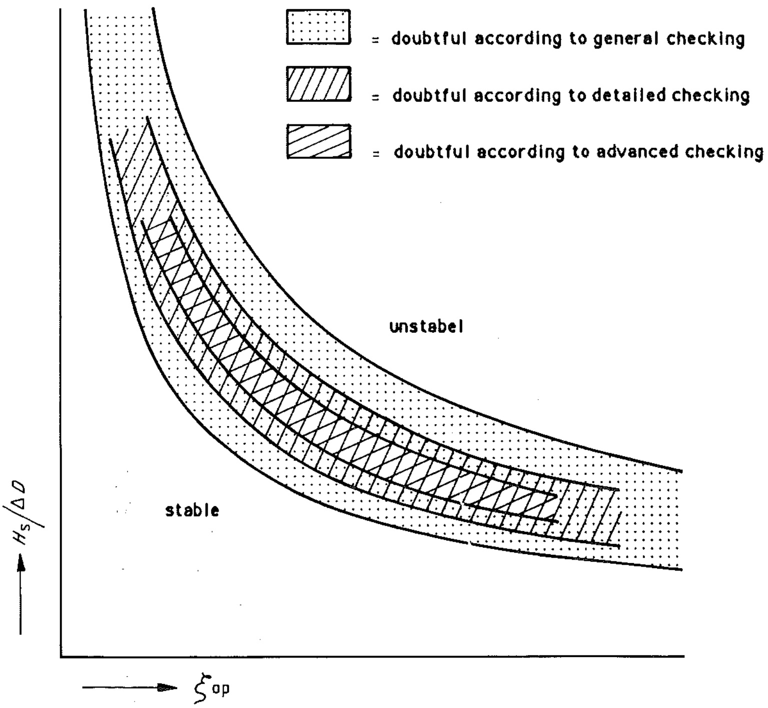


Fig. 180. Schematic representation of the "grey" zone in which stability is doubtful.

As discussed above in Section 13.3 (particularly Figure 177), a number of strength components can be identified in relation to failure via damage to the pitching. The number of dike components with residual strength, see Figure 181, depends on the composition of the revetment. In traditional designs only the first strength component, that in connection with the initial damage, was important. This always dominated the maintenance. All five strength components are important when checking safety standards, whether or not failure occurs in extreme conditions. This means, on the one hand, that much heavier loads are now considered than in the past and that, on the other hand, the strength of the flood protection works (the sum of the five strength components) to be taken into account is also much greater.
Although residual strength plays an important part in safety checking there is, unfortunately, too little known about the subject for a quantitative evaluation. Reference will be made to this aspect in the "Checking Manual" which will be published by TAW as a prospective guideline in 1995. The definitive "Checking Manual" will be published together with the new Dutch Act on Water Defences.
Because of this it is not yet possible to rigorously check a structure. Where reference is made, in the remainder of this chapter, to a structure being insufficiently stable the implication is that it can only be sufficiently stable as a whole if the residual strength is sufficient.

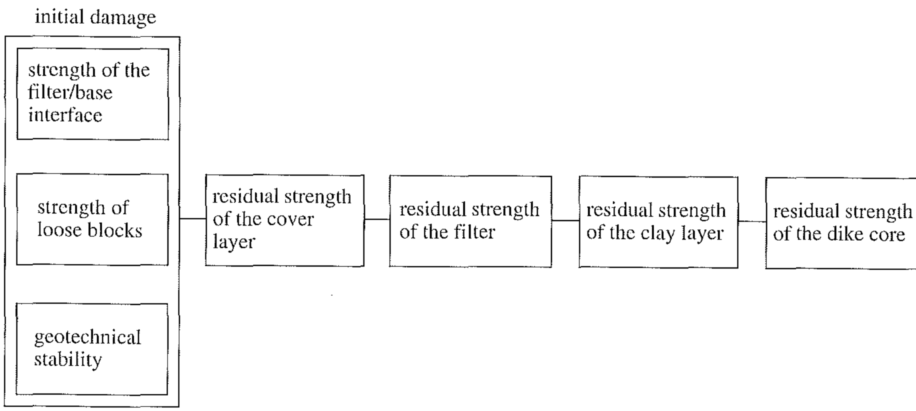


Fig. 181. Strength components.

14.2 Data based on experience

The first stage in checking the safety of a revetment is to use experience.
Information about how a revetment functions increases with periodic inspection and maintenance. This gives information about blocks which have been lifted out and also points to the functioning of the sublayers, the possible sealing of the cover layer, the extent of washing-in, etc. If experience shows that every year blocks are lifted out of the slope at various places, this is very likely to occur during the design storm. From this it can be concluded that in this case safety cannot be guaranteed by the cover layer but must obviously be entrusted to the residual strength. In such a case there is no point in checking the safety of the cover layer because this layer can be quickly damaged during extreme conditions.
During periodic inspections however there is little assessment of relatively small damage. If it is not possible to assess the cause of the damage then all aspects of the structure (cover layer, filter, geotechnical stability, residual strength) should be checked. Table 12 summarizes some types of damage which occur most frequently and indicates whether immediate repairs should be carried out and gives provisional conclusions about safety checking.
The occasional loose block indicates that blocks are loosened in extreme wave conditions and the revetment should be thoroughly checked. If loose blocks are never found this does not mean that they do not occur. Unless it can be demonstrated conclusively that there are no completely loose blocks by, for example an extraction test, a higher stability should be considered than that calculated using methods given in this manual. The way to quantify this additional stability is currently being investigated.
If water can only seep away slowly through the joints in the cover layer this can indicate low cover layer and/or sublayer permeability (due to sand or silt in the material from, for example, the foreshore). It is unfortunately, not (yet) known if low permea-

bility is always associated with low filter permeability and the absence of loose blocks. It may in fact be necessary to reconsider the composition of the filter in these situations, see Section 8.7.3.

Table 12. Summary of the most frequently occurring types of damage caused by hydraulic loads or drift ice.

type of damage	repair needed?	conclusions about checking
one block lifted out	storm season: immediate repair summer: possible postponement but repair before the storm season	if damage is every year check only the residual strength
several blocks lifted out	immediate repair	cover layer unsatisfactory, check residual strength!
local settlement of blocks	reset if difference in height is more than 5 to 10 cm; find cause!	check all structural aspects; extra attention to sublayers
washed-in material flushed out	refill if more than 10 cm of joints empty	check all structural aspects, including empty joints
loose blocks	fill with washed-in material	check all structural aspects, especially loose blocks
transitions with loose blocks	fill with washed-in material or grout	check all structural aspects, especially loose blocks
water remains in joints (low k')	almost irreparable	check all structural aspects, especially low k' and/or k
ice damage	immediate repair	check for stability after initial damage by ice flow

14.3 General checking

Cover layer

General checking of cover layer stability can be in the form of inspecting the outward appearance of the structure to establish the structural type and using one of the Preliminary Design Method Figures 104 to 115. To use this method reference should be made to Section 8.4.2. Unfortunately a distinction cannot be made yet between structures where there is an occasional loose block and those where it can be demonstrated that these do not occur.

Penetration of sand into the filter from the base

An assessment is given below of whether or not instability will develop due to sand penetrating the filter. The grain size of the filter and the sand is needed for this assessment. Samples are usually taken to obtain these grain sizes. Depending on the type of filter material (which effects the porosity) the following criteria can be applied for filters underneath block pitching:

a. *Broadly graded filter material*, such as unsorted minestone, minestone 0/70 mm, minestone 10/125 mm, Silex 0/90 mm, slags 5/70 mm, etc., see Figure 182a.
b. *Filter materials with a steep sieve curve*, such as stone slag 5/25 mm, rubble 30/60 mm, Silex 25/70 mm, etc., see Figure 182b.

Unless it can be shown, using Figures 182a or 182b and general information about the material, that it lies well within the stable area, the stability of the structure is doubtful if there is no information on the grain sizes.

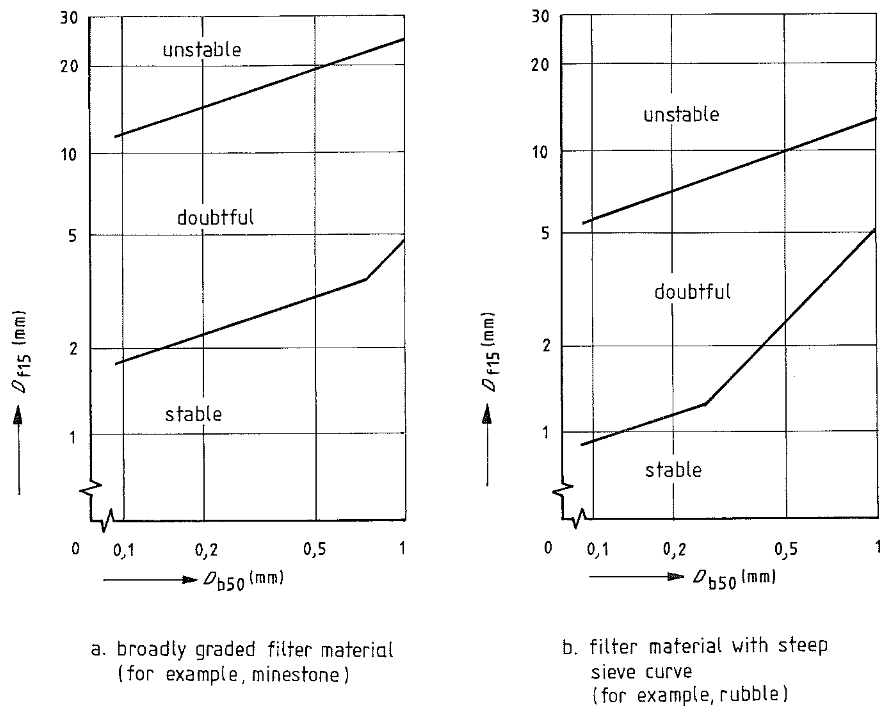


Fig. 182. General check on the stability of the filter/sand interface. (see also Section 8.4.3 and Table 4 in Section 5.3.1).

Loss of material through a geotextile

A geotextile separating the blocks from the clay or sand must be geometrically sealed:

– sand subsoil:– sufficiently stable if $O_{90} \leq D_{b50}$,
– stability doubtful: others,
– not sufficiently stable if $O_{90} > D_{b90}$,

- clay subsoil: – sufficiently stable if $O_{90} < 10 \cdot D_{b50}$ and $O_{90} < D_{b90}$ and $O_{90} < 0.1$ mm,
  – not sufficiently stable if $O_{90} > 10 \cdot D_{b50}$ or $O_{90} > D_{b90}$ or $O_{90} > 0.1$ mm,

The success of a geotextile laid between the blocks and the sand will be apparent directly after the first storm with a sufficiently high water level and waves larger than 0.5 m. If, under these conditions, the surface of the slope is still in a good state of repair it is not necessary to check the geotextile.
The above criteria also applies for checking the stability of a geotextile laid between clay and a granular filter layer. If, in this case, the stability is insufficient, the stability of the filter/geotextile/clay interface is said to be "doubtful". More flexible criteria are applied for a geotextile between a sandy subsoil and a granular filter:
- sufficiently stable: $O_{90}/D_{b90} < 1.5$,
- doubtful stability: others,
- not sufficiently stable: $O_{90}/D_{b90} > 2.5$ and $D_{f15} > 10$ mm.

Clay directly underneath the blocks
According to the Preliminary Design Method the blocks can be assumed to have a relatively high stability if they are laid on good quality clay, the high quality clay being a prerequisite. The quality of the clay must therefore be checked. To do this blocks have to be removed from various places in the slope and samples of the underlying clay removed. The clay must satisfy the specifications set out in Section 8.7.2. If the clay is unsatisfactory then the stability corresponds to that indicated for blocks laid on reasonable clay in Figures 105 and 110, provided that there is a geotextile between the clay and the blocks. Without a geotextile the stability is "doubtful" if one or more samples show the following, or if there are no samples available:

$$I_p < 18\% \quad \text{or} \quad I_p < 0.73 \cdot (W_l - 20) \quad \text{or} \quad Z_k > 40\%$$

Another way to assess clay quality is to lift some blocks out at a level near SWL which is attacked in heavy storms by severe wave action (at least half the design wave height). If there appears to be no erosion channels in the clay surface and the blocks rest firmly on the subsoil, the clay quality is sufficient in relation to the stability of the cover layer. Clay samples are then not necessary. This inspection does not however make any assessment of the residual strength of the clay layer.

Geotechnical stability
To check geotechnical stability all that is required is an assessment of the slope and existing experience with wave loads. If one of the two following criteria is satisfied the structure is acceptable.

- the slope is not steeper than 1 : 4 and no geotechnical instability has been observed under wave action on the slope at or below storm flood level;
- the total thickness of the cover layer and filter layers (including clay layers) is more than 1.2 m and the slope is not greater than 1 : 3.

If neither of these criteria is satisfied the stability of the structure is doubtful.

Residual strength

Unfortunately insufficient is known about residual strength for it to be checked, see Section 13.4. Reference should therefore be made to "Leidraad Toetsing" when it has been published.

14.4 Detailed checking

Detailed checks are only made when general checking indicates that there are dike sections with granular filters, which are of "doubtful" stability. Detailed checks are also made if general checking indicates "doubtful" geotechnical stability. This type of checking involves analytical methods, as described in Section 8.4.3.

The analytical method is used to calculate a failure wave height for the cover layer and a critical hydraulic gradient which will cause sand to block the filter (strength). The stability of the structure in relation to the wave height and the hydraulic gradient in the filter (loads) can be assessed in the following way:

a. The structure is stable if:
   1. $H < H_{cr}$ (or $H_{2\%}$, if this is relevant according to the Analytical Design Method) and
   2. $i_{\downarrow} \leq i_{\downarrow cr}$ and
   3. $i_{\uparrow} \leq i_{\uparrow cr}$,
   4. the structure is geotechnically stable in accordance with Figures 137 to 140.

b. The stability is doubtful if:
   – it can be shown to be either stable or unstable.

c. The structure is unstable if:
   1. $H > 1.5 \cdot H_{cr}$ (or $H_{2\%}$, if this is relevant according to the Analytical Design Method) or
   2. $i_{\downarrow} > 2 \cdot i_{\downarrow cr}$, or
   3. $i_{\uparrow} > 2 \cdot i_{\uparrow cr}$.

For the following structural aspects detailed checking is the same as general checking, see Section 14.3:

- a geotextile between a cover layer and the sand or clay;
- blocks on clay;
- blocks on a geotextile on sand, and;
- residual strength.

14.5 **Advanced checking**

In order to carry out advanced checking it is essential to collect as much data about a dike as possible, for example:

- the smallest force required to lift a block out of the slope;
- the permeability of the cover and filter layers;
- etc.

Small scale model tests are also desirable to accurately determine the pressure on the slope as a function of time and place. Calculations can then be made using STEENZET/1+ and STEENZET/2 based on this detailed data. If the checks give negative results, there is considerable information available on which to base advice on any structural improvements needed.

CHAPTER 15

CALCULATION EXAMPLES

15.1 Introduction

In the preceding chapters methods are presented for designing pitched dike revetments. A specific example is worked out in this chapter for a revetment for a theoretical sea dike. The design process begins with the collection of information needed to determine the wave boundary conditions at the toe of the structure. After choosing the slope, cover layer type and sublayers, the Preliminary Design Method, described in Section 8.4.2, is used to make an initial estimate of the thickness needed for the cover layer. This initial design is then checked using the Analytical Design Method described in Section 8.4.3. The optimum design can only be obtained by comparing the initial design with a number of alternatives. Ultimately the block thickness, in combination with a method of construction which is as simple as possible influences governs the final choice. The latter aspects are beyond the scope of the present manual.

It should be noted that the conclusions drawn for the example are only relevant to this particular example and cannot be applied generally.

15.2 Basic assumptions

The situation and the design assumptions for the pitched revetment are shown schematically in Figure 183. The revetment being considered is for a sea dike along an estuary, the maximum loads being wave loads from the north northwest (NNW). Data related to the design hydraulic loads are given in Table 13.

A decision has first to be taken on whether to design the revetment on the basis of extreme conditions with a probability of exceedance of 10^{-4}/year (ultimate limit state) or, if it is sufficient, to design on the basis of loads which occur relatively more frequently (serviceability limit state). This design is treated in Chapter 13.

Table 13. Hydraulic load data.

exceedance frequency [/year]	0.04	$1 \cdot 10^{-4}$
design wind speed [m/s]	28	34
water level; referred to NAP [m]	+3.2	+4.7
average bed level, referred to NAP [m]	−6.8	
bed level at the dike, referred to NAP [m]	+0.2	

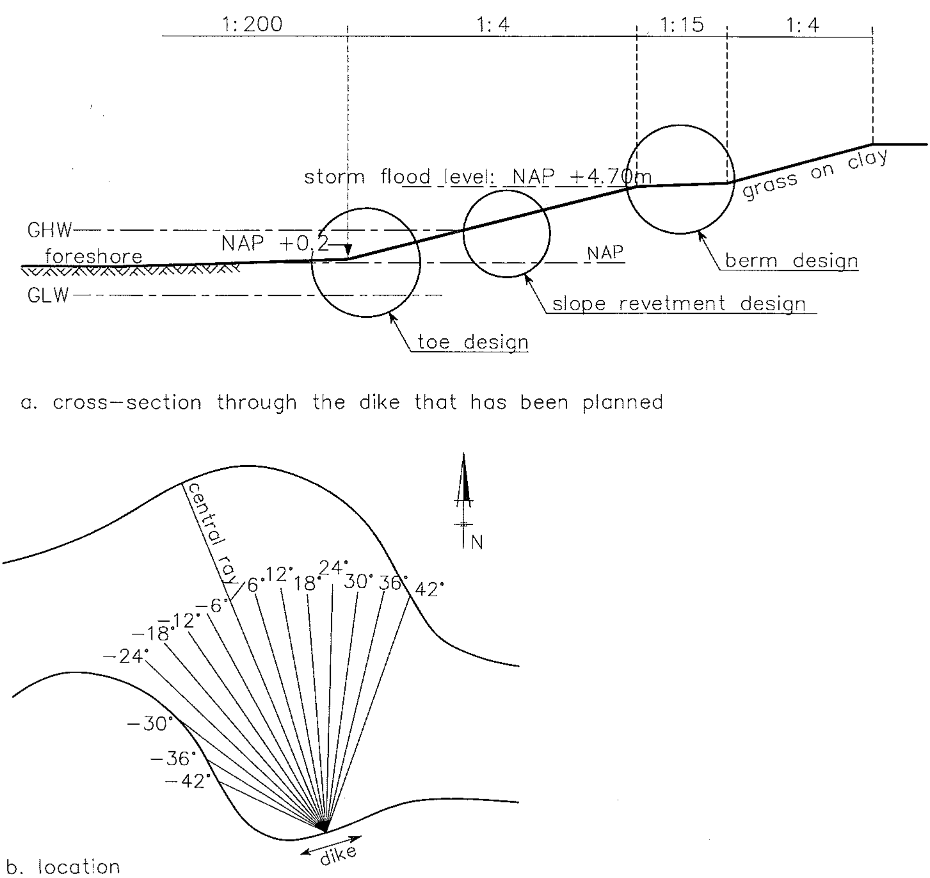


Fig. 183. Schematic diagram showing the dike and the estuary.

The design considered here is for the reinforcing of an old clay dike of good quality clay. The situation is as shown in Figures 78 and 79 where, after reinforcing, there is still a considerable body of clay behind the dike.
A design is selected in this case which takes into account a residual strength which is sufficient and which can be based therefore on a load which occurs frequently. A loading which is exceeded once in 25 years is selected (probability of exceedance 0.04 per year). This means also that the design can be based on mean values for all inputs, for example, wave height, joint width, etc. The way in which the data is obtained and how the choice is made is outside the scope of this example.
The foreshore slopes away in front of the dike at an average of 1 : 200, extending several hundred metres out from the dike. At the toe of the dike the level is at NAP+0.2 m. The flow along the toe is very small. The geometry of the estuary and the shape of the contours do not affect wave refraction and diffraction. The level of high and low water is NAP +1.80 m and NAP −1.80 m. For this example it is assumed that the dike will have a pitched revetment on a slope of 1 : 4 with an outer

face berm at storm flood level. The berm must have a hard revetment because it will be used by maintenance traffic. The submerged slope below the berm is minestone, the thickness of which has to be established. Underneath the berm is an 80 cm thick clay layer.

The minestone has a characteristic grain size, $D_{f15} = 2$ mm. For this example the porosity, n, is chosen at a fairly low level of $n = 0.2$. For the cover layer there is a choice between concrete columns and rectangular concrete cubes. The properties of the cover layer are summarised in Table 14.

Table 14. Properties of the cover layer.

property		concrete columns	concrete blocks
thickness [m]	$D =$	to be selected	to be selected
dimensions: surface area [m^2]	$A =$	0.09	0.25
length [m]	$L =$	does not apply	0.5
width [m]	$B =$	does not apply	0.5
relative open surface area [%]	$\Omega =$	≈ 15 %	≈ 0.6 %
joint width [mm]	$s =$	does not apply	1 to 2
volumetric mass [kg/m^3]	$\rho_s =$	2.400	2.400

The minestone is laid on hydraulic fill (sand) which has a characteristic grain size of $D_{b50} = 0.15$ mm ($D_{b90} = 0.2$ mm).

15.3 Calculations for the wave loads on the toe of the dike

Section 6.2.2 describes how the wave conditions in an estuary are a function of the average depth, the fetch, the wind speed and the storm duration. From Table 13 it follows that the average depth for the selected frequency of exceedance is equivalent to $h = 3.2 + 6.8 = 10.0$ m. The fetch is determined using the following formula [Guide for the design of river dikes, 1991]:

$$F = \frac{\Sigma\{F_i \cdot (\cos\beta_i)^2\}}{\Sigma\cos\beta_i} \tag{74}$$

where:

F = fetch [m]
F_i = ray length at an angle β_i to the central ray [m]
β_i = angle between ray i and the central ray [°]
$\Sigma\{..\}$ = summation of all values of i

The fetch is calculated in Table 15 for the situation in Figure 183.

Table 15. Fetch computations.

F_i [km]	β_i [°]	$\cos\beta_i$ [–]	$F_i(\cos\beta_i)$ [km]	
2.5	–42	0.74	1.36	
2.9	–36	0.81	1.88	
3.5	–30	0.87	2.65	
8.5	–24	0.91	7.07	
8.1	–18	0.95	7.30	
7.9	–12	0.98	7.59	
8.0	–6	0.99	7.91	
8.0	0	1.00	8.00	
8.0	6	0.99	7.91	
7.9	12	0.98	7.59	
7.6	18	0.95	6.87	
7.2	24	0.91	6.01	
6.6	30	0.87	4.95	
5.9	36	0.81	3.88	
5.3	42	0.74 +	2.95 +	
	$\Sigma\{..\}$ =	13.51	83.92	$\Rightarrow F = 83.92/13.51 = 6.21$ km

Assuming an average water depth of 10 m, the wave height and period can now be determined using Figures 40 and 41:

$$H_s = 1.5 \text{ m}$$
$$T_p = 4.4 \text{ s}$$

The minimum storm duration necessary to create these wave conditions can be read from Figure 44. It appears that a duration of less than one hour is sufficient to generate the maximum wave height. Because of shoaling the wave height on the foreshore will change. This will depend on h/L_{op}:

$$\left.\begin{array}{l} -\text{ wave length: } L_{op} = \dfrac{g}{2\pi}T_p^2 = \dfrac{9.8}{6.3}\,4.4^2 = 30.2 \text{ m} \\ -\text{ water depth on the foreshore: } h = 3.0 \text{ à } 4.0 \text{ m} \end{array}\right\} \Rightarrow h/L_{op} = 0.10 \text{ à } 0.13$$

From Figure 46 it appears that a small reduction in wave height can be expected with this value of h/L_{op}. This reduction is about 5 to 10% which gives the following:

$$H_s = 1.4 \text{ m}$$
$$T_p = 4.4 \text{ s}$$

Whether these wave conditions can also develop at the toe depends on the depth of water at a distance $L_{op}/2$ from the dike. The maximum possible significant wave height in the depths of water in front of the structure can be estimated using the rules of thumb given in Section 6.2.2, Equation (16) and Figure 49. In this situation:

$$L_{op}/2 > d_t/\tan\alpha, \text{ therefore: } d = d_t + (L_{op}/2 - d_t/\tan\alpha)\cdot\tan\alpha_v$$
$$= 3.0 + (30.2/2 - 3.0/0.25)\cdot 0.005 \approx 3.0 \text{ m}$$

maximum significant wave height: $(H_s)_{max} = 0.5 \cdot d = 0.5 \cdot 3.0 = 1.5$ m

The conclusion is that the significant wave height is not yet affected by breaking (because $H_s \leq (H_s)_{max}$).

15.4 Preliminary design and choice of cover layer

Using the Preliminary Design Method it is possible to make a first estimate of the thickness needed for the cover layer and, based on this, to choose between a cover layer of columns or one of rectangular blocks. A filler layer between the cover layer and the minestone is needed for both types of cover layer. The filler layer under the columns must be much coarser than that under the blocks, to prevent it being flushed out. The following preliminary designs have been selected, see Figure 184:

Filler layer under columns:	– grain size:	16 to 32 mm, $D_{u15} \approx 16$ mm,
	– porosity:	$n_u \approx 0.4$
	– layer thickness:	$b_u = 0.05$ m
Filler layer under blocks:	– grain size:	5 to 25 mm, $D_{u15} \approx 5$ mm
	– porosity:	$n_u = 0.4$
	– layer thickness:	$b_u = 0.05$ m

The layer thickness selected for the minestone is 0.5 m.

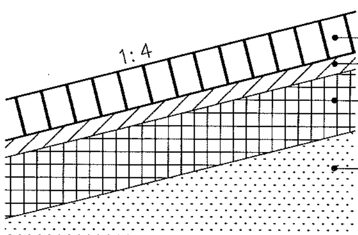


Fig. 184. Revetment structure alternatives.

The type of structure can be established using the chart in Figure 103. Part of the scheme used in this example is repeated in Figure 185.
To assess whether b/D is larger than 0.5, the thickness of the filler layer, b, must be introduced, in this case 0.05 m. At this stage the thickness of the cover layer is estimated at D = 0.2 to 0.3 m, and therefore b/D = 0.17 to 0.25.
The ruling grain size is that of the filler layer. For blocks D_{f15} = 5 mm (the right hand route in Figure 185) and for columns, D_{f15} = 16 mm (left hand route in Figure 185). The blocks cannot be washed-in and also have no holes. The chart must then be used to establish if the open surface area (Ω) is less than 3%. From Table 14 it can be seen that this is in fact the case.

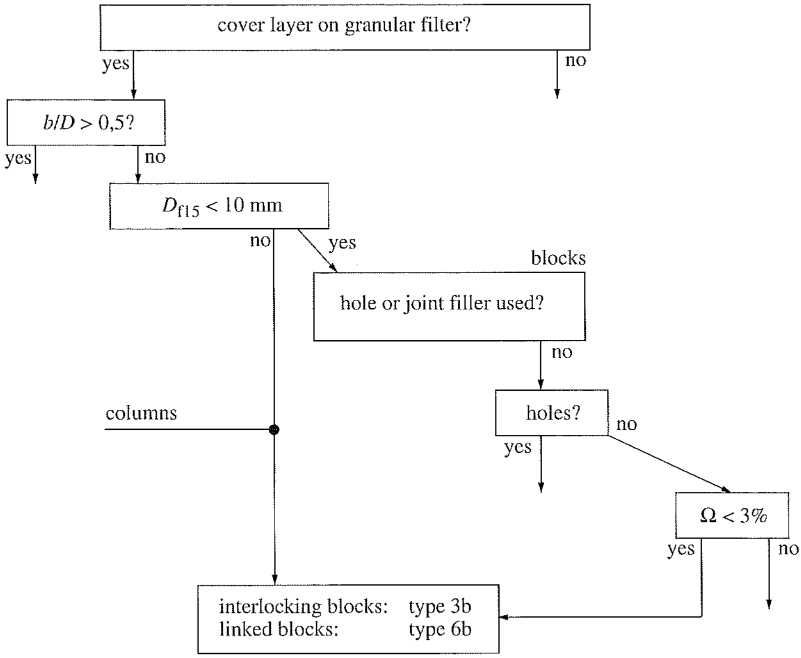


Fig. 185. Flow chart for determining the type of construction using the Preliminary Design Method (see also Figure 103).

The chart indicates Type 3b (normal structure) for both structures. This means that there is little difference in stability and that the choice of cover layer can be based on other factors, such as price, appearance, etc. Without going further into this aspect here the rectangular block alternative was selected.
The thickness required for the cover layer depends on the breaker parameter (ξ_{op}) and the specific gravity of the blocks in seawater:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{(H_s/L_{op})}} = \frac{0.25}{\sqrt{(1.4/30)}} = 1.16$$

$$\Delta = \frac{\rho_s - \rho}{\rho} = \frac{(2400 - 1025)}{1025} = 1.34$$

From Figure 107 it follows that, for $\xi_{op} = 1.16$, the cover layer may be unstable if $H_s/(\Delta D) = 3.5$ to 7.5. The necessary thickness of the cover layer is then, as shown below, 0.14 to 0.30 m:

minimum: $D = H_s/(\Delta \cdot 7.5) = 1.4/(1.34 \cdot 7.5) = 0.14$ m,
maximum: $D = H_s/(\Delta \cdot 3.5) = 1.4/(1.34 \cdot 3.5) = 0.30$ m,

A value of $D = 0.25$ m was chosen provisionally for the thickness of the cover layer. This is checked in the following section using the Analytical Design Method.

15.5 Detailed design using the Analytical Design Method

The Analytical Design Method has to be used to check that the cover layer thickness of 0.25 m is sufficient. In addition the stability of the interface between the minestone and the sand and the geotechnical stability also have to be checked.

The calculations are divided into seven sections:
1. Calculations of the design wave height and period.
2. Selection of design criteria.
3. Calculation of the leakage length.
4. Check on the stability of the interface between the minestone and the sand.
5. Check on the possibility of block movement when $H = H_s$.
6. Check that the block movements are not greater than $0.1 \cdot D$ when $H = 1.4 \cdot H_s$.
7. Check on geotechnical stability.

These calculations are discussed below. The procedure is shown schematically in Figure 116; the relevant part of this figure is repeated in Figure 186.

1. *Design wave height and period calculations*
   The design wave height and period, from Section 15.3, are:
   $H_s = 1.4$ m
   $T_p = 4.4$ s
   $\xi_{op} = 1.16$

2. *Selection of Design Criteria*
   Design criteria selection is treated in Section 8.3, which states that a distinction should be made loads which occur rarely and those which occur frequently. In the present example loads are considered which occur much less than once a year and can therefore be categorized as "rare". According to Figure 186, such loads must first be calculated using the significant wave height parameters:
   $H = H_s = 1.4$ m
   $T = T_p = 4.4$ s
   $\xi_o = \xi_{op} = 1.16$

3. *Leakage length calculation*
   According to the left hand route of the chart in Figure 186 the leakage length must next be calculated using the procedure shown in Figure 117. The relevant part of this figure is given in Figure 187.

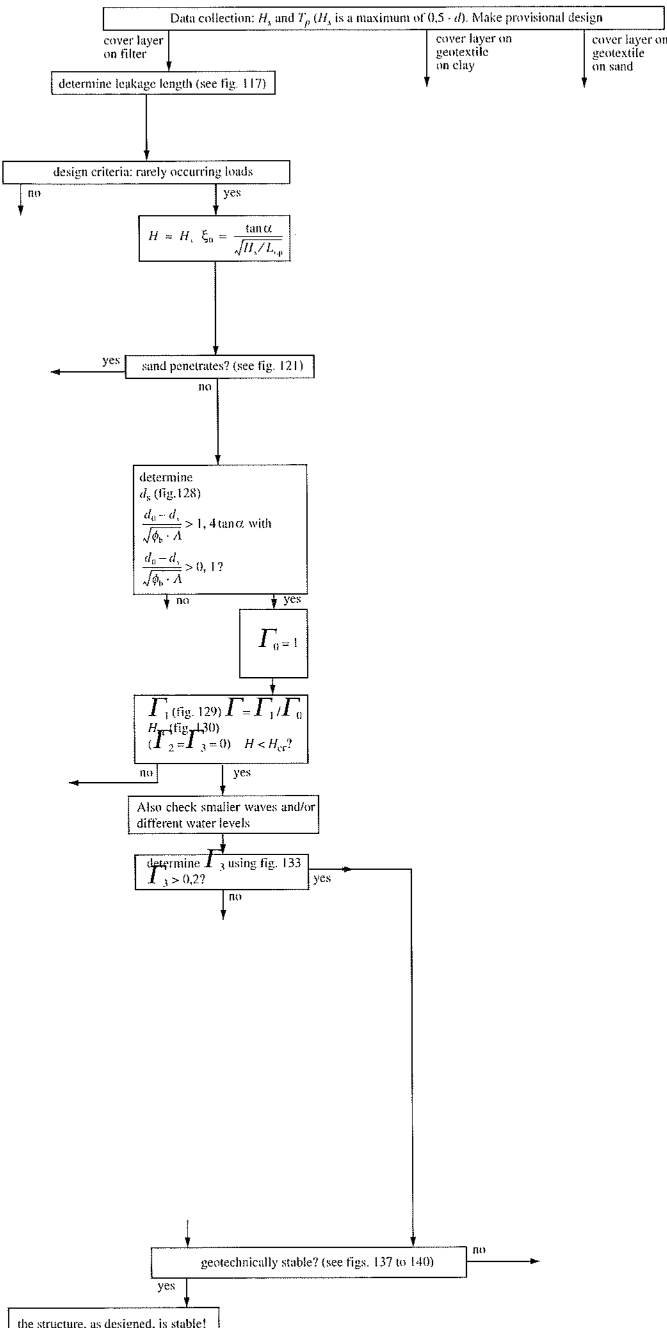


Fig. 186. Structural design chart (Example of calculation procedure, see also Figure 116).

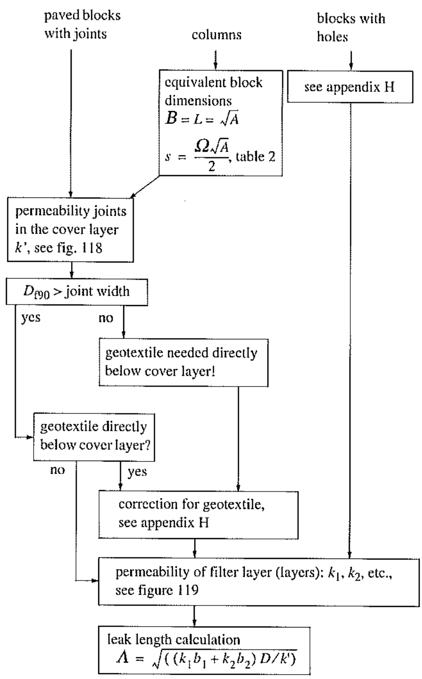


Fig. 187. Chart for determining leakage length (Example of calculation procedure, see also Figure 117).

Figure 187 indicates that the permeability of the cover layer must first be determined using Figure 118, using the following data:

- joint width: s = 1.5 mm
- no joint filler
- filter porosity: n = 0.4 (NB filler layer applied)
- filter grain size: D_{f15} = 5 mm (NB filler layer applied)
- block shape: $B = L = 0,5$ m $\Rightarrow 2BL/(B + L) = 0.5$ m

Figure 118 is repeated for this calculation as Figure 188.
The procedure is as follows:

- Enter the figure at the upper horizontal axis at a joint width of 1.5 mm.
- Follow the thin dashed line vertically downwards to the solid line (if there is no joint filler).
- Go horizontally to the solid line at n = 0.4 and then downwards to D_{f15} = 5 mm.
- Go horizontally to the line for $2BL/(B + L)$ = 0.5 m and upwards to the lowest horizontal axis.
- The permeability of the cover layer can then be read off at k' = 4 mm/s.

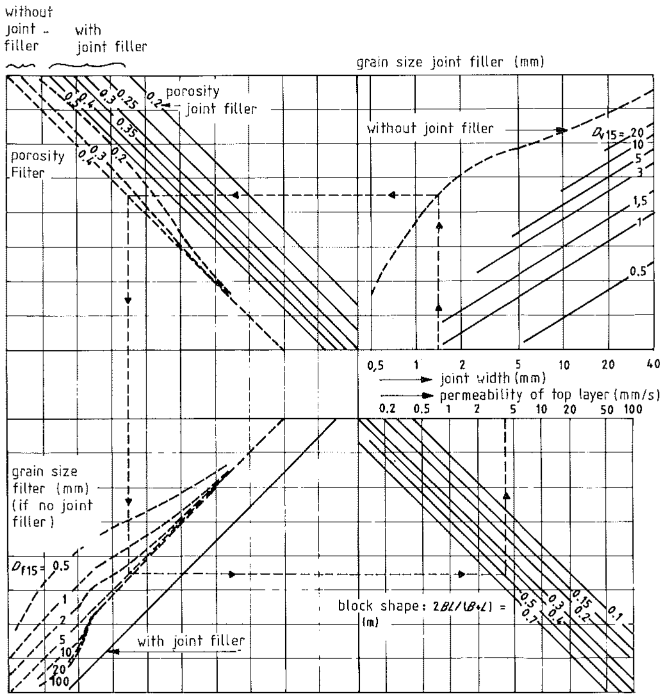


Fig. 188. Permeability of the cover layer (Example of calculation, see also Figure 118).

According to the chart in Figure 187 (D_{f90} > joint width, no geotextile under the cover layer) the permeability of the filter layers should be determined using Figure 119, repeated as Figure 189. This figure indicates how the permeability of the filler layer and the minestone can be read off:

- filler layer: $D_{f15} = 5$ mm; $n = 0.4 \Rightarrow k_u = 85$ mm/s;
- minestone layer: $D_{f15} = 2$ mm; $n = 0.2 \Rightarrow k_m = 2.5$ mm/s;

The leakage length is then:

$$\Lambda = \sqrt{\left(\frac{(b_u k_u + b_m k_m)\,D}{k'}\right)} = \sqrt{\left(\frac{(0.05 \cdot 0.085 + 0.5 \cdot 0.0025) \cdot 0.25}{0.004}\right)} = 0.59 \text{ m}$$

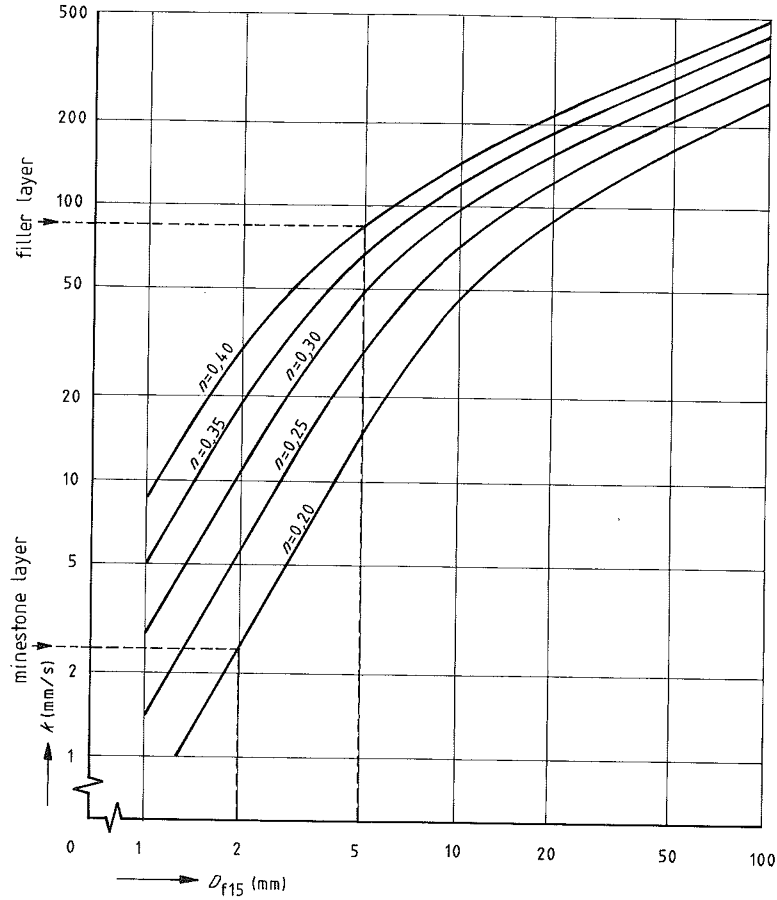


Fig. 189. Filter layer permeability (Example of calculation, see also Figure 119 and Table 4, Section 5.3.1).

4. *Check on the stability of the interface between the minestone and the sand.*
After calculating the leakage length, the sanding up of the filter is considered, see Figure 186. The procedure for this is shown in the flow chart in Figure 121, part of which is reproduced in Figure 190. As indicated in the chart a check has to be carried out to determine if the filter is geometrically sealed against sand grains, in this case the filter being the layer of minestone, see Section 8.4.3.

$$D_{f15}/D_{b85} \approx D_{f15}/D_{b90} = 2/0.2 = 10 \Rightarrow \text{not sealed geometrically.}$$

In this case the thickness of the filter layer is greater than half the thickness of the cover layer ($b > D/2$) and the chart must be used again to determine the pressure head on the slope: θ_b and $\cot\theta$.

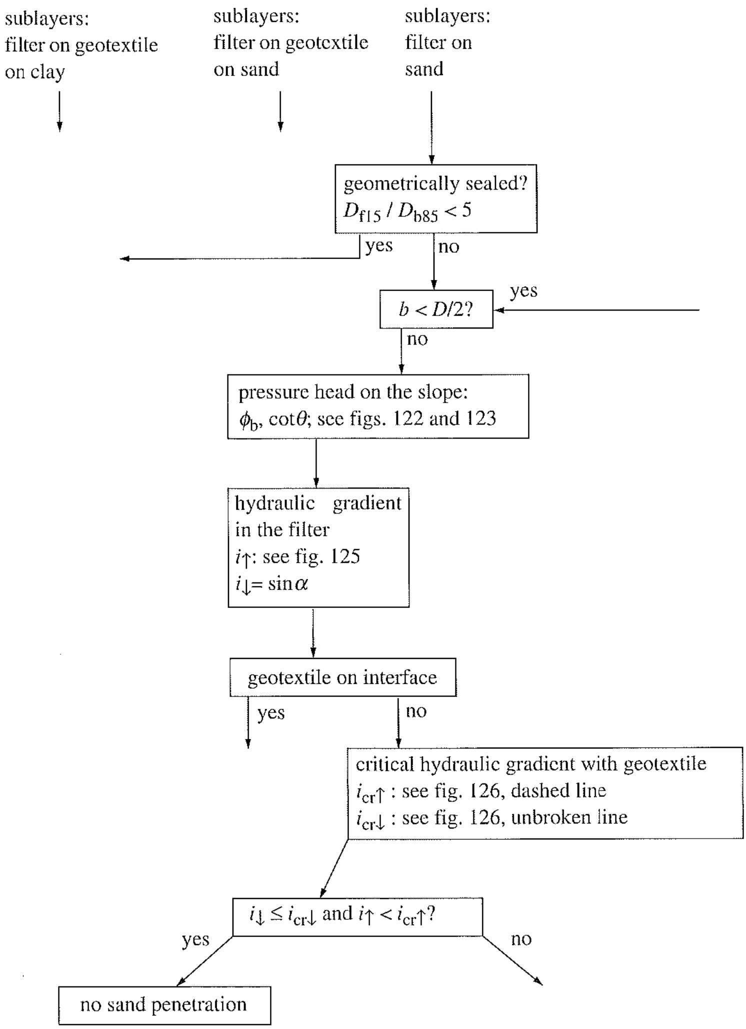


Fig. 190. Chart for determining the stability of the interface between sand and minestone (Example of calculation, see also Figure 121).

The chart used is given in Figures 122 and 123, repeated here as Figures 191 and 192. Using the figures:

$$\phi_b/H = 0.83;\ H = H_s = 1.4 \text{ m} \Rightarrow \phi_b = 0.83 \cdot 1.4 = 1.2 \text{ m}$$

$$\cot\theta = 0.8$$

The next step in Figure 190 is to determine $i_\uparrow$ and $i_\downarrow$ using Figure 125, repeated below as Figure 193:

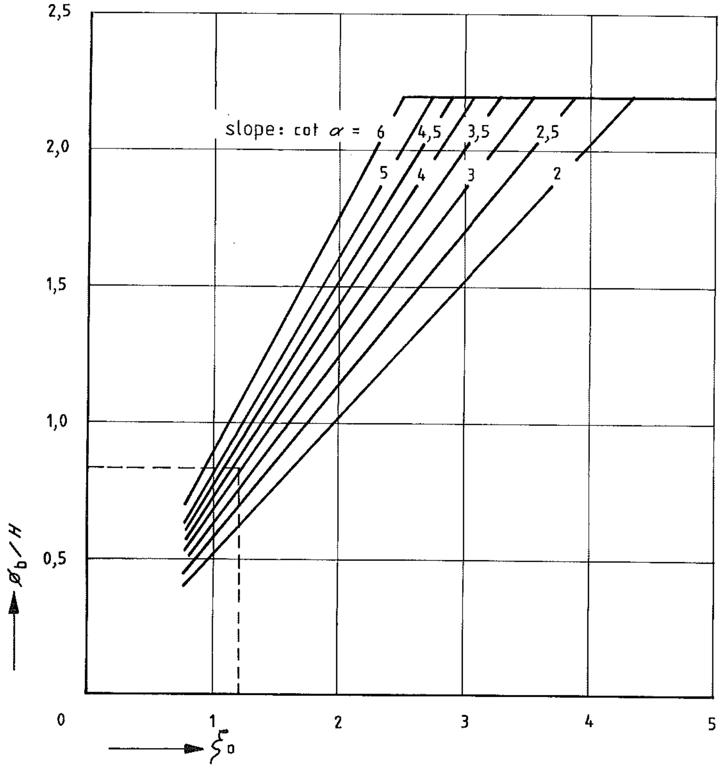


Fig. 191. Pressure head on the slope (Example of calculation, see also Figure 122).

$$\left.\begin{aligned} \phi_b/\Lambda &= 1.2/0.59 = 2.0 \\ \cot\theta &= 0.8 \end{aligned}\right\} \Rightarrow \text{(with Figure 193) } i_\uparrow = 0.6$$

$$i_\downarrow = \sin\alpha \approx \tan\alpha = 0.25$$

According to the chart in Figure 190 the maximum allowable hydraulic gradient ($i_{cr\uparrow}$ and $i_{cr\downarrow}$) must now be calculated using Figure 126. This figure is repeated in Figure 194.

The procedure is as follows:

- Enter the figure at the horizontal axis for the grain size of the filter, minestone: $D_{f15} = 2$ mm,

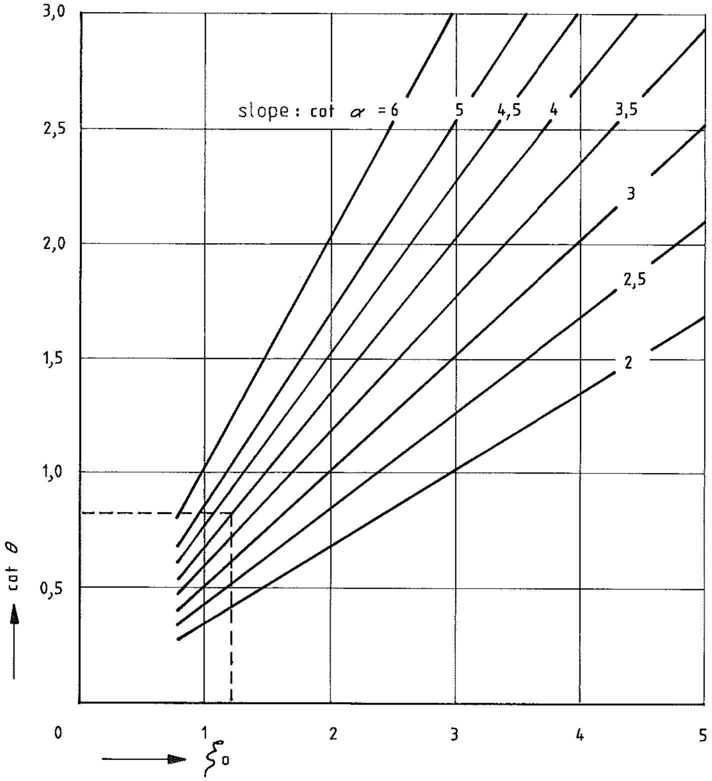


Fig. 192. The steepness of the pressure head front on the slope (Example of calculation, see also Figure 123).

- Go, via the sand grain size (D_{b50} = 0.15 mm), to the porosity of the filter (minestone: n_f = 0.2); this is outside the figure, implying that the allowable hydraulic gradient is at least equivalent to 0.9 ($i_{cr\uparrow} > 0.9$ and $i_{cr\downarrow} > 0.9$). Since this slope is less than 0.9 (that is, $i_\uparrow = 0.6$ and $i_\downarrow = 0.25$) it can be concluded that sand is unlikely to penetrate and block the filter.

5. *Check on block movement if $H = H_s$*
After checking the possibility of filter blockage the next step in the chart, Figure 186, is to calculate the wave height at which loose blocks begin to move. Figure 128 is first used to determine where the heaviest loads occur on the pitching. This figure is repeated here as Figure 195.

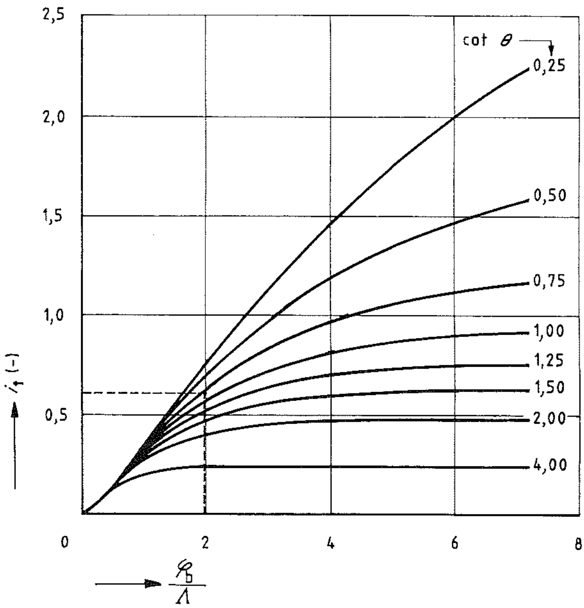


Fig. 193. The hydraulic gradient up the interface loads. (Example of calculation, see also Figure 125.

From the figure it follows that:

$$d_s/H = 0.4;\ H = H_s = 1.4\text{ m} \Rightarrow d_s = 0.4 \cdot 1.4 = 0.56\text{ m}$$

Checks have then to be made to determine if the transitions at the berm and the toe of the slope affect the pressure head difference on the pitching, see Equations (47) and (48) in Section 8.4.3:

a. Below the level of maximum load ($d_o > d_s$):

$$\text{no effects if } \frac{d_o - d_s}{\sqrt{(\phi_b \Lambda)}} > 1.4 \cdot \tan\alpha \tag{75}$$

with $d_s = 0.56$, $d_o = d_t = 3.0$, $\phi_b = 1.2$, $\Lambda = 0.59$ and $\tan\alpha = 0.25$, it follows that:

$$(d_o - d_s)/\sqrt{(\phi_b \Lambda)} = (3.0 - 0.56)/\sqrt{(1.2 \cdot 0.59)} = 2.9 \text{ and } 1.4 \cdot \tan\alpha = 0.35$$

Conclusion. The transition at the toe has no effect on the pressure head difference on the pitching.

b. Above the level of the maximum load ($d_o > d_s$):

$$\text{no effects if} \quad \frac{d_s - d_o}{\sqrt{(\phi_b \Lambda)}} > 0.1 \tag{76}$$

with $d_s = 0.56$, $d_o = 3.2 - 4.7 = -1.5$, $\phi_b = 1.2$ and $\Lambda = 0.59$, it follows that:

$$(d_s - d_o)/\sqrt{(\phi_b \Lambda)} = (0.56 - 1.5) / \sqrt{(1.2 \cdot 0.59)} = 2.4$$

Conclusion. The transition at the toe has no effect on the pressure head difference on the pitching.

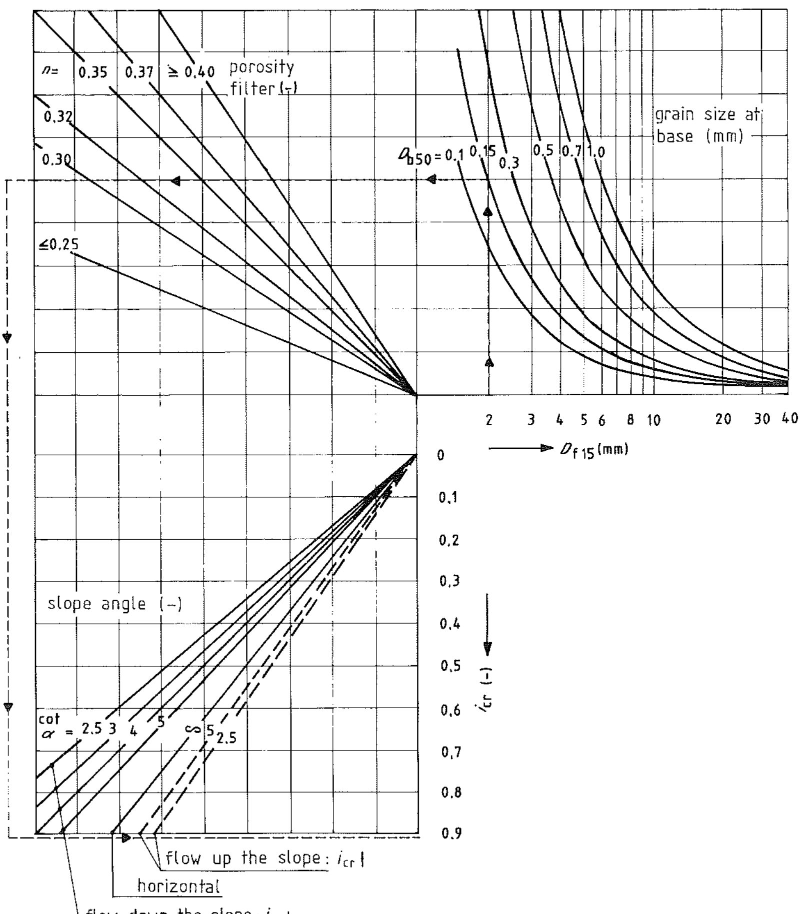


Fig. 194. Maximum permissible hydraulic gradient on the interface. (Example of calculation, see also Figure 126).

The friction factor Γ_1 can be determined from Figure 129, which is reproduced here as Figure 196. With $B/D = 0.5//0.25 = 2.0$, it follows that $\Gamma_1 = 1.13$.
The critical wave height at the beginning of movement can therefore be determined using Figure 130, see Figure 197. The following parameters can therefore be used in the calculations:

$$\Gamma^{1.25} \cdot \sqrt{(\Delta D/\Lambda)} = 1.13^{1.25} \cdot \sqrt{(1.34 \cdot 0.25/0.59)} = 0.9$$

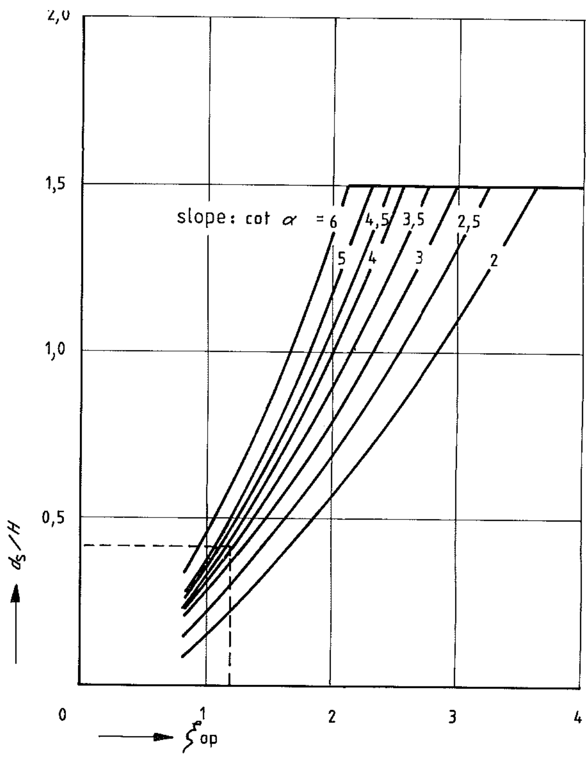


Fig. 195. The level at which maximum loads occur on the cover layer, relative to SWL. (Example of calculation, see also Figure 128).

Because the transition structures have no effect Γ is equal to Γ_1. Using Figure 197:

$H_{cr}/(\Delta D) = 4.2$, therefore
$H_{cr} = 4.2 \cdot \Delta \cdot D = 4.2 \cdot 1.34 \cdot 0.25 = 1.41$ m

The loads which develop are somewhat smaller, $H_s = 1.4$ m, indicating that the design is acceptable.

A check then has to be made to confirm that there are no block movements at lower water levels. However, since the transition structures exert no influence and significant wave heights at lower water levels will be lower, the blocks will not move.

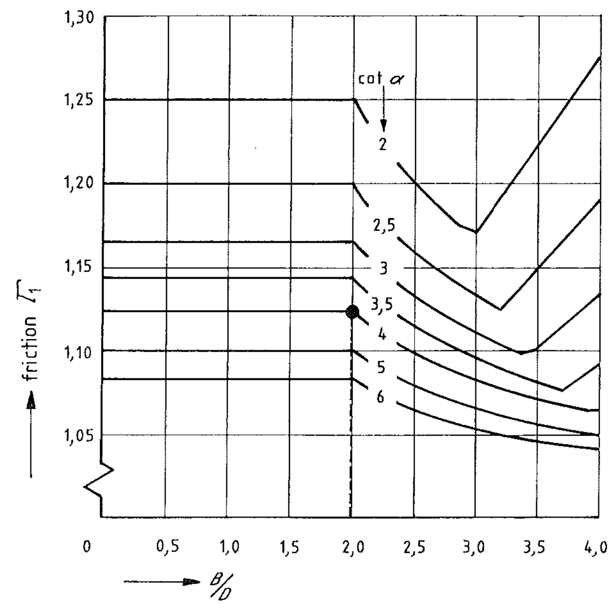


Fig. 196. Influence factors related to friction. (Example of calculation, see also Figure 129).

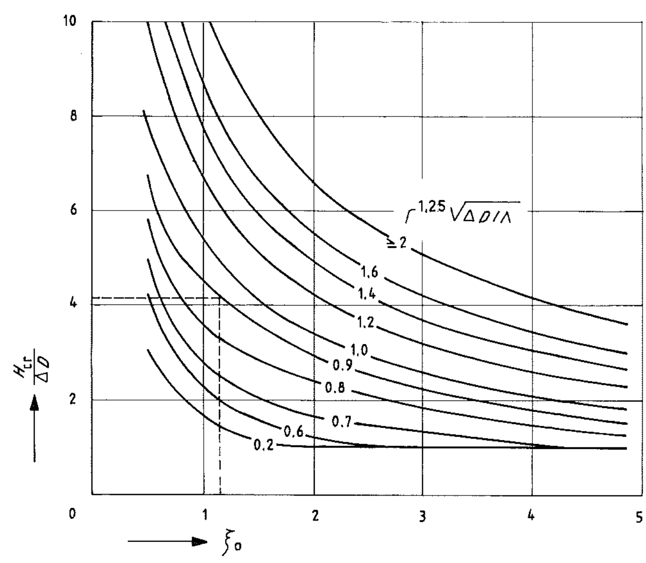


Fig. 197. Critical wave heights at the initiation of motion. (Example of calculation, see also Figure 130)

6. *Check on max 0.1 · D block movement if* $H = 1.4 \cdot H_s$

If $H = 1.4 \cdot H_s$, the block movement should be less than $0.1 \cdot D$. This check is only important if $\Gamma_3 < 0.2$. From Figure 133 (see also Figure 198) it appears that $D/(k'\Delta) = 0.25/(0.004 \cdot 1.34) = 47$ and $\sqrt{(BL/\Lambda)} = \sqrt{(0.5 \cdot 0.5/0.59)} = 0.65$.
$\Gamma_3 = 0.65$, indicating that this check is unnecessary.

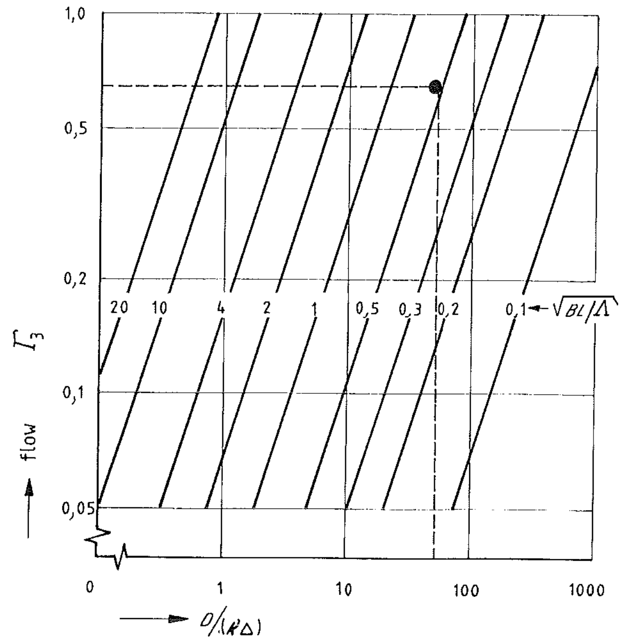


Fig. 198. Influence factors related to flow. (Example of calculation, see also Figure 133).

7. *Check for geotechnical stability*

The last step in Figure 186 is to check the geotechnical stability. Figures 137 and 138 are used for this check, since:

$$H_s/L_{op} = H_s/(1.56 \cdot T_p^2) = 1.4/(1.56 \cdot 4.4^2) \approx 0.05$$

Figure 137 is repeated as Figure 199.
With $\Delta D + b = 1.34 \cdot 0.25 + 0.5 = 0.8$ m, the figure shows that $H_s = 1.4$ m is in the stable area on the graph if the slope is 1:3 or less. In the design the slope is 1 : 4 and it can therefore be concluded that, for normal sand compaction, there is no risk of geotechnical instability.

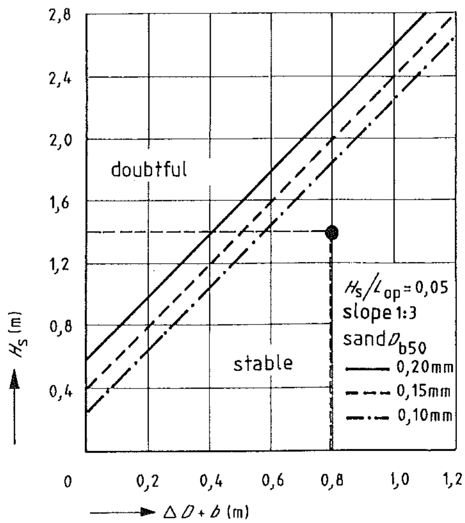


Fig. 199. Geotechnical stability if $H_s/L_{op} = 0.05$ and $\tan\alpha = 0.33$. (Example of calculation, see also Figure 137).

15.6 Toe and transition structures

In Section 15.2 it is assumed that maintenance equipment is able to ride on the berm. As discussed in Sections 7.4 and 10.6 the berm should therefore be at least 3 m wide and have a block revetment laid on a filter layer. The width of the berm affects the ultimate crest height of the dike. This aspect is not discussed further here. A berm width of B = 5 m is assumed.

The slope of the berm can best be protected with blocks laid directly on good quality clay. Because this revetment will only be applied above the highest water level and above the berm, relatively thin blocks can be used, for example, 0.15 m thick. The level of the transition to grass on clay is determined by the wave height and the run-up under extreme conditions, see Section 7.6. In this example, the loads have an exceedance frequency of 10^{-4} per year. The load can be calculated, step by step, using the data given in Table 13:

- wind speed: 34 m/s;
- average water depth in the estuary: 11.5 m;
- linear interpolation between Figures 38 and 40 gives H_s = 1.9 m; eventually to obtain a safety factor this can be increased to H_s = 2.1 m;
- linear interpolation between Figures 39 and 41 gives T_p = 4.8 s; eventually to obtain a safety factor this can be increased to T_p = 5.0 s;
- wave length: $L_{op} = \frac{g}{2\pi} \cdot T_p^2 = 1.56 \cdot 5.0^2 = 39$ m;

- depth on the foreshore in front of the dike:
  $d = 4.5$ to 5.5 m $\rightarrow d/L_{op} \approx 0.13$.
  From Figure 46 it appears that, because of shoaling, the wave height is reduced by about 5 to 10%.
  Thus $H_s = 0.93 \cdot 2.1 = 2.0$ m;
- check on wave breaking:
  $d \approx 4.5$ m $\Rightarrow (H_s)_{max} = 0.5 \cdot d = 0.5 \cdot 4.5 = 2.2$ m.
  There is therefore no reduction due to waves breaking.

The wave run-up can be calculated using Equation (31) because $\xi_{op} < 2$:

$$z_{2\%}/H_s = 1.5 \cdot \gamma_b \cdot \gamma_r \cdot \gamma_\beta \cdot \xi_{op}$$

with, in this case

$$\gamma_r = \gamma_\beta = 1 \Rightarrow z_{2\%} = 1.5 \cdot \gamma_b \cdot H_s \cdot \xi_{op}$$

with:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{(H_s/L_{op})}} = \frac{0.25}{\sqrt{(2.0/39)}} = 1.1 \qquad (77)$$

The effect of the berm follows from Figure 64, because

$$\left.\begin{aligned} \xi_{op} &= 1.1 \\ 4.4 \cdot (\tan\alpha)^{2/3} &= 4.4 \cdot (0.25)^{2/3} = 1.75 \end{aligned}\right\} \Rightarrow \xi_{op} < 4.4 \cdot (\tan\alpha)^{2/3} \qquad (78)$$

Since the berm is at SWL:

$d_b/H_s = 0$ and
with $\xi_{op}\, B/H_s = 1.1 \cdot 5/2.0 = 2.75$,
it follows from Figure 64 that $\gamma_b = 0.83$.

The run-up height is therefore:

$$z_{2\%} = 1.5 \cdot \gamma_b \cdot H_s \cdot \xi_{op} = 1.5 \cdot 0.83 \cdot 2.0 \cdot 1.1 = 2.7 \text{ m}$$

The transition between blocks laid on clay and grass laid on clay must, according to Section 7.6, be between at least $H_s/2$ above SWL (1.0 m + SWL) and half the run-up height (1.4 m + SWL) above SWL. With SWL at NAP +4.7 m the transition to grass must therefore be between NAP +5.7 m and NAP +6.1 m. The latter level is selected for the present design. The dike crest is chosen at NAP +7.8 m. The complete structure, including a solution for the toe, is shown in Figure 200. Rubble is not needed at the toe because flow is not expected along the dike. Support for the toe bulkhead is provided by the minestone filling, see also Figure 157.

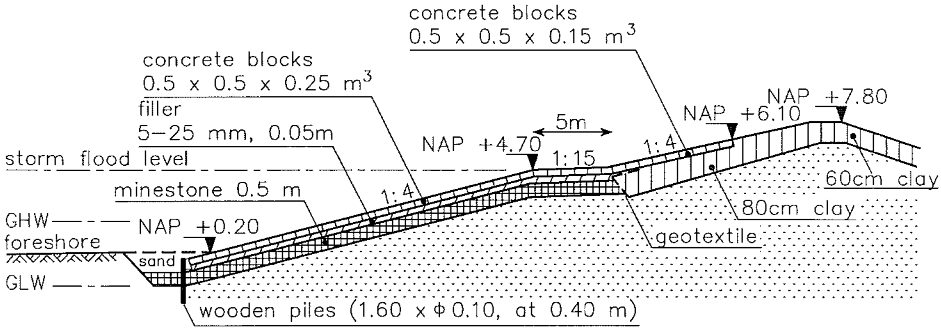


Fig. 200. Selected structure.

Chapter 16

GAPS IN PRESENT KNOWLEDGE

The results of ten year's research into the stability of pitched dike revetments are presented in this manual. Despite long periods of intensive research there are still a number of question to be answered which bar the way to a complete understanding of the physical processes related to revetment stability [BEZUIJEN et al., 1990, page 232].
Before the research was initiated information about this type of structure was so limited that it was extremely difficult to distinguish between the various types of structures which could be applied. The research has clearly shown that pitched dike revetments cannot be dismissed simply as a few stones and has produced design methods for the various types of structures. The research has also indicated how and where the various methods can be applied.
Most information has been collected about block revetments in which the cover layer lies on a granular filter, with particular reference to the initial damage caused by wind waves immediately after construction. The initial damage can be:

- single blocks being lifted out of the slope;
- the filter being blocked by sand from the base;
- geotechnical instability.

There is much research still needed to give a complete picture of the loads to be expected on block pitching during the lifetime of the structure. The most important areas in which research is still needed are:

- *Structural ageing*
  In the course of time, sand, silt and vegetation get into the joints between blocks and any holes in the slope. This reduces the permeability of the cover layer, which is detrimental to stability but increases the friction and adhesion between blocks which promotes stability. It is also possible for sand from, for example, the foreshore, to block the filter layer as well as the cover layer, again effects promoting stability.

- *Adhesion between blocks*
  At present it is assumed that the weakest element in the revetment is a loose block which only experiences friction forces exerted by the row immediately below. Extraction tests can be used to show whether or not loose blocks are likely to occur with particular types of structures. Recent extraction tests on an old slope of rectangular blocks, which were apparently rigidly adhered together, showed that there were in fact a small percentage of blocks loose above high water. Loose blocks were rarely found however in the tidal zone.

- *Residual strength*

  If one block can be lifted out of the slope, it is assumed that other blocks will be lost during the storm and that a hole will develop in the dike. For some structures however this assumption is too pessimistic, for example, when there is a good quality clay layer which will be able to resist wave attack for some time.

- *Permeability of the cover layer*

  Cover layer permeability has a large effect on structural stability. In order to predict the parameter accurately it is necessary to know the average joint width for the existing structure. This may depend on the dimensions of the blocks, their shape and the way in which they were placed, by hand or by machine.

- *Open structures on a slightly permeable subsoil*

  If the leakage length is small relative to the wave height (cover layer relatively permeable, filter layer impermeable, see Section 8.2.3) the time at which the ruling hydraulic pressure acts on the cover layer will move from immediately before to during wave impact. The failure process for this type of structure is not yet fully understood. Examples of structures for which wave impact is the ruling load are:
  - blocks with a very large open area (many holes) on a geotextile overlying a fine filler;
  - blocks or mattresses on clay (possibly with a geotextile between);
  - blocks or mattresses on a geotextile on sand.

- *Berms*

  A berm on the slope affects the water movements. How this affects the ruling pressure head on the slope is not yet fully understood.

- *Crest revetments*

  In order to determine the stability of a pitched revetment on the crest of a dike and on berms, the ruling pressure head must be quantified accurately. As yet this is not possible.

- *Irregular waves*

  The calculation methods presented in the manual are based on regular waves. A method has been developed for predicting the loads on structures due to irregular waves, the accuracy of which however is not so large. In principle STEENZET/1+ can be used with irregular waves, but the system is not yet operational.

- *Bends*

  The present methods are aimed at the straight sections of dikes. Whether the stability at bends is different is not yet known.

– *Block mattresses*
For the purposes of the Analytical Design Method block mattresses are considered in the present manual as interlocking blocks. The correctness of this assumption is questionable because, on the one hand, the mattress ensures that there is better interaction between the blocks and, provided that the edges and corners are well anchored, there are no loose blocks. On the other hand, however, the interactive forces can only be mobilized if the mattress begins to move, which in turn leads to subsoil instability. With a mattress held together by cables the interactive forces can perhaps be mobilized better than with blocks attached to a geotextile but between which there are large spaces. This however cannot be yet guaranteed.

– *Durability of asphalt products*
Asphalt products are used in pitched dike revetments for grouting transition structures (molten asphalt) and as filter layers (sand asphalt or bitumenized sand). Because repairs are expensive, filter layers in particular must be durable so that they can continue to function without maintenance and repairs. It is not yet known if sand asphalt and bitumenized sand can function longer than 10 to 20 years as filter layers.

– *Demixing, wearing and crumbling*
The properties of granular material can change before it is laid as a result of demixing, wearing and crumbling. It is not yet possible to predict the affect of these processes on the design properties of the material.

– *Safety*
A safety philosophy is presented in Chapter 13. Ultimately however the primary interest is in the probability of flooding and flood prevention. The philosophy therefore has to be developed further before it can become a design method for flood protection works as a whole and not only for stable revetments. To achieve this aim more information is needed about the above subjects listed above, particulary residual strength.

– *Maintenance costs*
When preparing a design it is possible to underestimate the maintenance costs because insufficient is known about this subject for particular types of revetments.

Many of these aspects are subjects for further research.

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D = Dutch, E = English, G = German

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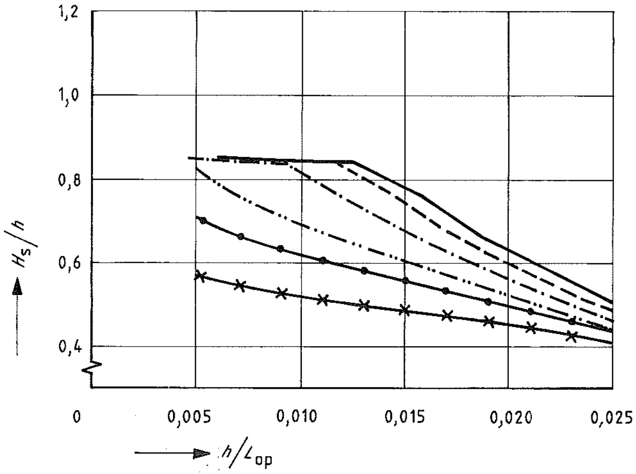
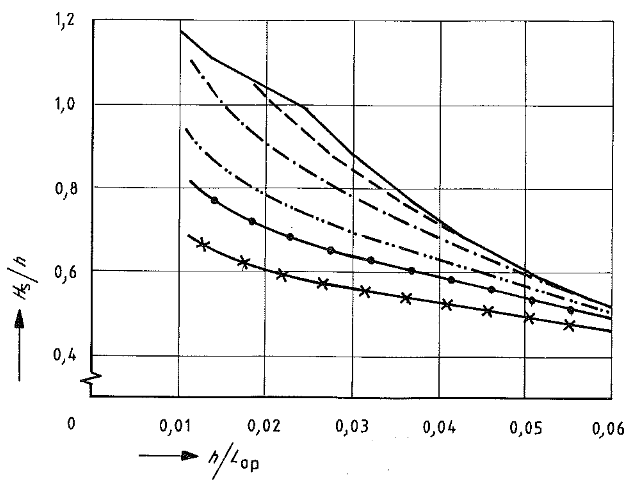
Appendix A

DEPTH-RESTRICTED WAVE HEIGHT

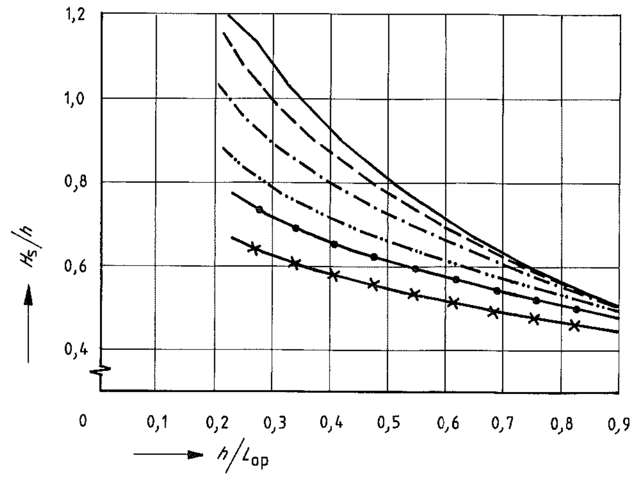
Maximum significant wave height (limited by depth)



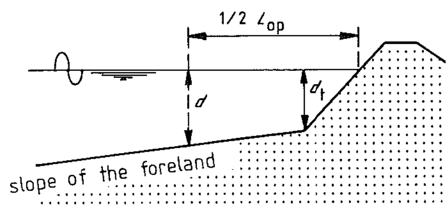
a. $H_s/L_{op} = 0,01$



b. $H_s/L_{op} = 0,03$



c. $H_s/L_{op} = 0{,}05$



slope of the foreland:

1:10
1:13
1:20
1:30
1:50
1:100 en flauwer

h = local water depth [m]
(not specifically at 1/2 L_{op} of the structure)

L_{op} = wave length in deep water = $\dfrac{gT_p^2}{2\pi}$ [m]

H_s = significant wave height in a water depth of h [m]
T_p = peak period [s]

see also Appendix I (based on ENDEC calculations)

Fig. A.1

APPENDIX B

CALCULATION OF CREST LEVEL FOR DIFFERENT FORMS OF SLOPE

Examples of calculations for different dike cross-sections (H_s = 4.7 m, T_p = 8.5 s)

slope	1 : 3	1 : 4	1 : 4 and berm
design water level	5.00	5.00	5.00
run up level	13.30	10.00	7.00
rise of sea level	0.25	0.25	0.25
seiches/squalls	0.25	0.25	0.25
settlement of dike	0.50	0.50	0.50
dike height	19.30	16.00	13.00

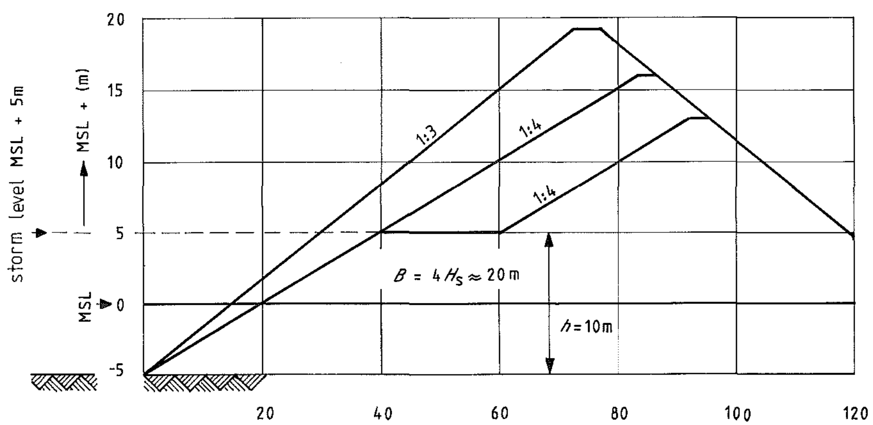


Fig. B1

APPENDIX C

LEAKAGE LENGTH

The leakage length is derived from the filter layer flow equation. The differential equation, given below, can be deduced for flow in the filter layer assuming that the flow in the filter layer is parallel to the slope and that the flow through the cover layer is perpendicular to the slope.
Consider a small section of filter, see Figure C1.

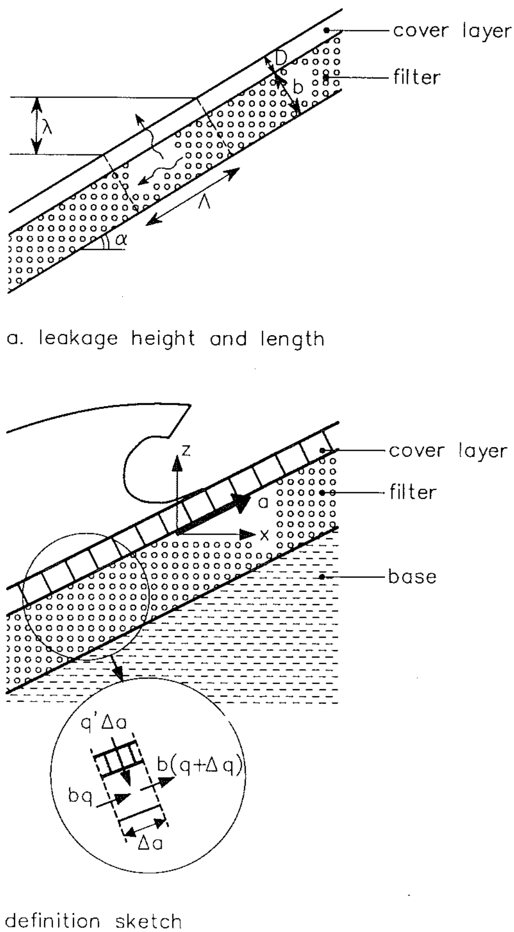


Fig. C1. Potential flow in the filter and the cover layer.

The specific discharge (q) through the filter layer can be expressed as:

$$q = -k\frac{\mathrm{d}\phi}{\mathrm{d}a} \tag{C1}$$

the specific discharge through the cover layer as:

$$q' = \frac{k'}{D}(\phi' - \phi) \tag{C2}$$

where:

q	= specific discharge through the filter layer [m/s]
q'	= specific discharge through the cover layer [m/s]
k	= the permeability of the filter layer [m/s]
k'	= the permeability of the cover layer [m/s]
b	= filter layer thickness [m]
D	= cover layer thickness [m]
ϕ	= pressure head in the filter layer [m]
ϕ'	= pressure head on the slope [m]
a	= co-ordinate down the slope [m]

For continuity, the net specific discharge in a small section of the filter layer, Δa, must be zero, see Figure C1. Thus:

$$q' = b\frac{\Delta q}{\Delta a} \tag{C3}$$

If this section, Δa, is infinitesimally small, it becomes $\mathrm{d}a$, so that by inserting Equations (C1) and (C2) into (C3), the differential equation for the flow in the filter layer becomes:

$$\frac{d^2\phi}{\mathrm{d}a^2} = \frac{\phi - \phi'}{kbD/k'} \tag{C4}$$

From this equation if appears that kbD/k', which has dimensions [m^2], is important. The root of this number is defined as the leakage length (Λ) with dimensions, [m].
In order to compare different revetment slopes, the vertical component of the leakage length is sometimes used. In these cases the leakage height (λ) is defined as:

$$\lambda = \Lambda \sin\alpha \tag{C5}$$

Appendix D

THE RELATIONSHIP BETWEEN IRREGULAR AND REGULAR FAILURE WAVE HEIGHTS

The results of large scale model investigations are included in the figures for the Preliminary Design Method, Figures 103, 116, 117, 118, 119, 121, 122, 123, 125, 126, 128, 129, 130, 133, 137. It should be noted that most of these observations were made with regular waves. The results of the regular wave tests can be translated into irregular wave results as follows:

a. The regular wave height at the initiation of damage (H_{cr}) is divided by 1.4 to obtain the effective significant wave height at this point (H_{scr}):

$$H_{cr}/H_{scr} = 1.4 \qquad \text{(D1)}$$

The size of this factor has been determined from a comparison of test results with regular and irregular waves for five different structures. The equation above has a semi-theoretical background, which was established by [De Waal, 1990].

b. The effective peak period for the irregular waves is similarly based on that for regular waves.

A measurement point on a $H/(\Delta D)$-ξ graph for regular waves can be converted to one on a $H_s/(\Delta D)$-ξ_{op} graph for irregular waves, as follows:

$$\xi_{op} = \frac{\xi}{\sqrt{1.4}} \qquad \text{(D2)}$$

$$\frac{H_s}{\Delta D} = \frac{1}{1.4} \cdot \frac{H}{\Delta D} \qquad \text{(D3)}$$

Appendix E

MAXIMUM ALLOWABLE CURRENT VELOCITY OVER THE CREST

Very few investigations have been carried out into the stability of block pitching on the crest when it is overtopped. Knauss, see [Pilarczyk et al, 1990] gives an equation for the critical specific discharge:

$$q_{cr} = 0.625 \cdot \sqrt{g} \cdot (\Delta D)^{1.5} \cdot (1.9 + 0.8 \cdot P - 3 \sin \alpha) \tag{E1}$$

where:

- q_{cr} = specific discharge per running metre at which damage is initiated [$m^3/s/m$]
- P = factor which depends on the way the blocks were laid ($P = 0.6$ for loosely dumped rubble; $P = 1.1$ for neatly placed rubble, $P \approx 1.25$ for pitched blocks)
- α = slope angle (rear slope)

This equation reduces to one for the critical velocity on the crest. As postulated by Knauss overtopping is assumed to be complete:

$$\left.\begin{aligned} q &= u_k d_k \\ d_k &= u_k^2/g \end{aligned}\right\} \Rightarrow q = u_k^3/g \tag{E2}$$

From Equation (E1) it follows that

$$u_{cr} = \sqrt{\Delta g D} \cdot (1.19 + 0.5 \cdot P - 1.88 \cdot \sin \alpha)^{0.33} \tag{E3}$$

where:

- u_{cr} = current velocity on the crest at the initiation of damage.

The value of P for block pitching is about 1.25 although there is still some uncertainty about this value. If $P = 1.25$ and $\sin\alpha = 0$ then:

$$u_{cr} = 1.2 \cdot \sqrt{\Delta g D} \tag{E4}$$

This equation is identical to that found by Isbash for loosely dumped rubble being overtopped, see [Huis in 't Veld et al., 1984]. In view of this agreement and the fact

that block pitching is almost certainly more stable than rubble, it can be concluded that Equation (E4) will give safe results.
The relevant details of model investigations carried out by [Van Kruiningen, 1989] for an overflow channel with various types of block pitching protection are as follows:

- Model details:
  - relative volumetric mass of the blocks: Δ = 1.1 to 1.15;
  - thickness of the revetment: D = 0.019 to 0.020 m;
  - rear slope: 1:2.4.
- Results:
  - specific discharge at failure: q_{cr} = 0.020 to 0.055 $m^3/s/m$;
  - current velocity at failure: u_{cr} = 0.58 to 0.81 m/s.

According to Equation (E4): u_{cr} = 0.54 to 0.57 m/s. This value agrees well with the smallest value measured in the model investigations. It should be noted that pitching on a slope where there is flow along the dike is much more stable than indicated by Equation (E4). The factor 1.2 should be increased to 1.5 to 2.5 in these situations.

APPENDIX F

CREST STABILITY WITH OVERTOPPING WAVES

Very little is known about the stability of block pitching on a dike crest when it is overtopped by waves. Model investigations were carried out by [VAN KRUININGEN, 1989] into the stability of a dike crest protected by block pitching on the Afsluitdijk. He gives the results of his investigations, see Table F1, but no general equations.

Table F1. Results of the investigations by [VAN KRUININGEN, 1989] (prototype values).

Test	H_s [m]	T_p [s]	ξ_{op} [–]	$z_{2\%}$ [m]	damage	h_c [m]	$\dfrac{z_{2\%}-h_c}{\Delta D}$ [–]
101	1.60	5.3	1.16	2.78	0	1.9	3.8
102	1.90	5.3	1.07	3.05	8 and 2	1.9	5.0
111	1.48	6.6	1.51	3.35	4	1.9	6.3
112	1.30	6.6	1.61	3.14	2	1.9	5.4
113	1.09	6.6	1.75	2.86	0	1.9	4.2
131	1.64	5.3	1.15	2.83	0	1.9	4.0
132	1.93	5.3	1.06	3.07	0	1.9	5.1
133	2.15	5.3	1.00	3.23	7	1.9	5.8
142	1.34	6.5	1.56	3.14	2	1.9	5.4

The numbers of blocks lifted out of the revetment during the test are given in the "damage" column of the table.
The experimental conditions were as follows:

- relative volumetric mass of the blocks: $\Delta = 1.15$;
- cover layer thickness: $D = 0.20$ m (prototype);
- front slope: 1:4 to 1:4.5;
- rear slope: 1:2.4;
- length scale of the model: $n_L = 10$.

The last column in Table F1 is the relationship between the effective wave height on the crest ($z_{2\%} - h_c$, see Section 6.5) and ΔD. Damage seemed to develop when this relationship was greater than about 5. Since the data in the table are based on a limited number of structures a good safety margin must be included in the design of the crest revetment. The following relationship is recommended:

$$\frac{z_{2\%} - h_c}{\Delta D} \leq 3 \tag{F1}$$

Appendix G

THE EFFECT OF TRANSITION STRUCTURES

A transition structure can either increase or decrease the difference in pressure head on the cover layer. The distance between the block subjected to the maximum load, that is, the largest difference in pressure head, and the transition structures above and below is important. This is shown in Figure G1.

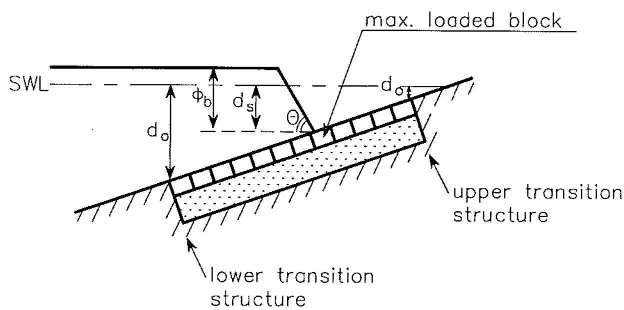


Fig. G1. Distance between the most heavily loaded block and the transition zones.

As shown by the calculations in Section 8.4.3 the effect of the transition structure becomes noticeable if one of the following conditions apply:

1. The upper transition structure is close to the most heavily loaded block, that is:

$$\frac{d_s - d_o}{\sqrt{(\phi_b \cdot \Lambda)}} \leq 0.1 \tag{G1}$$

2. The lower transition structure is close to the most heavily loaded block, that is:

$$\frac{d_o - d_s}{\sqrt{(\phi_b \cdot \Lambda)}} \leq 1.4 \cdot \tan\alpha \tag{G2}$$

Therefore $(d_o - d_s)$ and $(d_s - d_o)$ can have negative values.

Often grouting the revetment with asphalt can completely seal the cover layer (and part of the filter layer). Such a grouted strip must be classed with the transition structure, the edge of the strip being taken when measuring the distance to the most heavily loaded block. The level of the transition is defined as the level down the slope of the first non-grouted joint in the cover layer below the transition, see Figure G2. Almost always the upper or lower transition will affect the maximum pressure head difference on the cover layer. Equations (G1) and (G2) will indicate which of the two transitions will have the greatest effect on the pressure head. If neither of these equations satisfy the above conditions the smallest value of $|d_o - d_s|$ is selected. In this situation the method presented here can only be indicative and it is recommended that the results are confirmed using ANAMOS or STEENZET/1+.

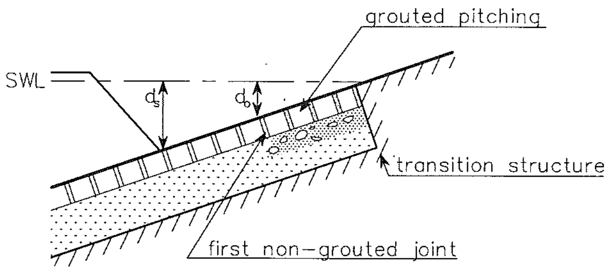


Fig. G2. Definition of d_o: grouted strip of blocks at an upper transition.

The effect of the transition is expressed in terms of an influence factor, Γ_0. The size of the factor can be determined using Figures 4, 5 or 6, the flow chart in Figure G3 indicating which figure to use. Figures G4, 5 and 6 are all composed in the same way:

- Start in the first quadrant (above, right) with, on the horizontal axis, the relative distance between the transition and the most heavily loaded block. Figure G4: $(d_s - d_o)/(\Lambda \tan\alpha)$, Figure G5: $\log[(d_o - d_s)_s/(\Lambda \tan\alpha)]$, Figure G6: $(d_o - d_s)/(\Lambda \tan\alpha)$.
- Proceed, in this quadrant, vertically to the line with the related value of ϕ_b/Λ and then horizontally to the second quadrant, (above, left).
- The second quadrant relates to the slope of the pressure head front: $\cot\theta$ the third quadrant to the revetment slope: $\tan\alpha$.
- Unbroken and dashed lines are drawn in Figures G5 and G6; the unbroken lines should be used if $\phi_b/\Lambda \geq 1$ and the dashed line if $\phi_b/\Lambda \leq 0.75$. If ϕ_b/Λ lies between 0.75 and 1, Γ_0 can be interpolated between Γ_0 at $\phi_b/\Lambda = 0.75$ and Γ_0 at $\phi_b/\Lambda = 1$.

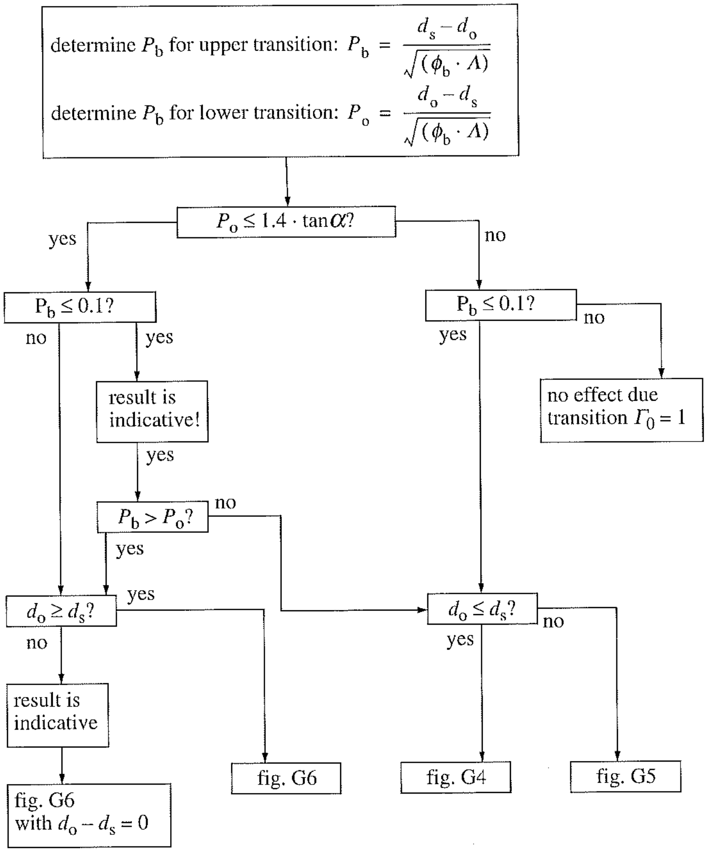
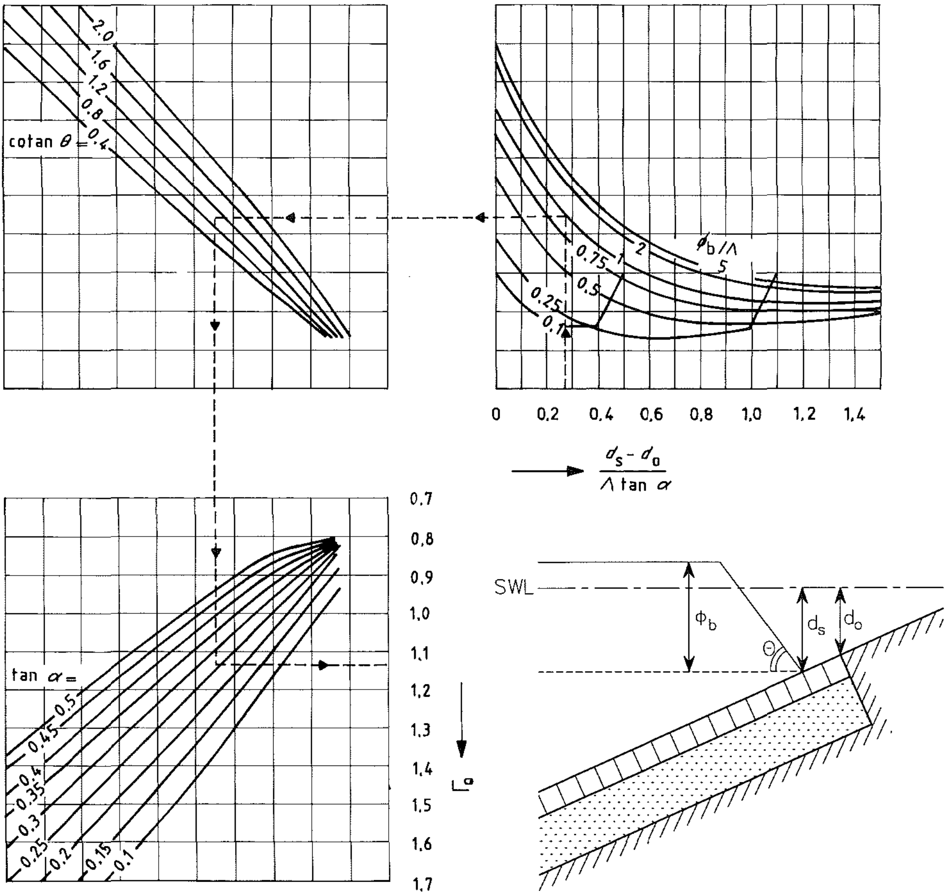


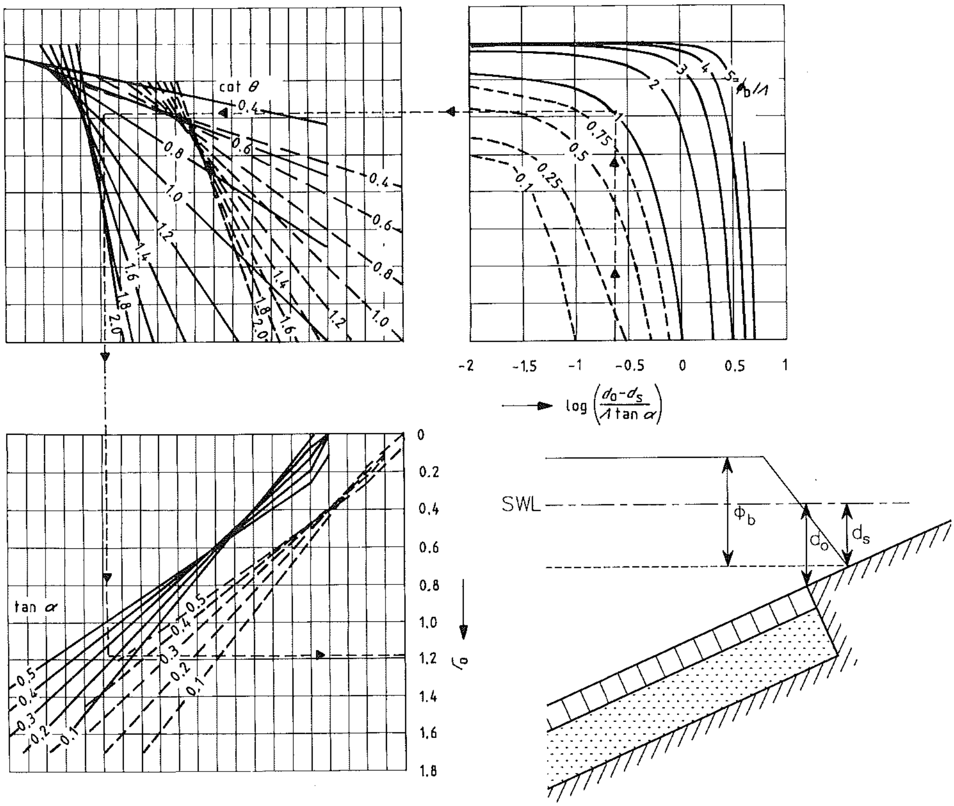
Fig. G3. Flow chart for selecting from Figures G4, G5 or G6.



example: $\Lambda = 1.5$ m cotan $\theta = 1.0$
$d_s = 0.91$ m tan $\alpha = 0.27$
$\phi_b = 1.5$ m $d_o = 0.8$ m

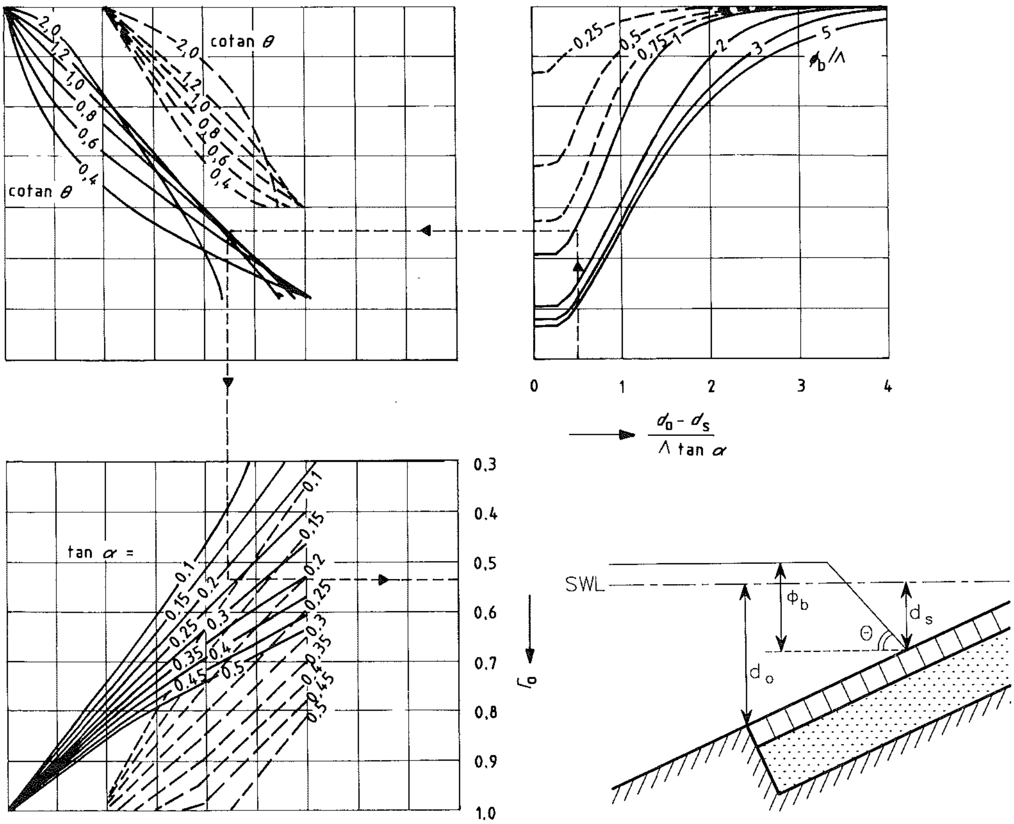
$\rightarrow \dfrac{d_s - d_o}{\Lambda \tan \alpha} = 0.29 \rightarrow \Gamma_o = 1.14$

Fig. G4. Influence factor (Γ_0) if the transition is on the slope above, that is, $d_o \leq d_s$.



example: $\Lambda = 1.5$ m, $\cot\theta = 0.92$, $d_s = 0.91$ m, $\tan\alpha = 0.25$, $\phi_b = 1.55$ m, $d_o = 1.0$ m $\Big\} \longrightarrow \log\left(\frac{d_s\ d_o}{\Lambda \tan\alpha}\right) = -0.62 \longrightarrow \Gamma_o = 1.2$

Fig. G5. Influence factor (Γ_0) if the transition is on the slope above ($d_o > d_s$). Use the unbroken line if $\phi_b/\Lambda \geq 1$ and dashed line if $\phi_b/\Lambda \leq 0.75$.



example: $\Lambda = 1{,}5$ m $\quad$ cotan $\theta = 1.0$
$d_s = 0.91$ m $\quad$ tan $\alpha = 0.25$
$\phi_b = 1.5$ m $\quad$ $d_0 = 1{,}1$ m

$$\left.\begin{matrix} \Lambda = 1{,}5\ \text{m} & \cot\theta = 1.0 \\ d_s = 0.91\ \text{m} & \tan\alpha = 0.25 \\ \phi_b = 1.5\ \text{m} & d_0 = 1{,}1\ \text{m} \end{matrix}\right\} \rightarrow \frac{d_s - d_0}{\Lambda \tan \alpha} = 0.51 \rightarrow \Gamma_0 = 0.54$$

Fig. G6. Influence factor (Γ_0) if the transition is on the slope below. Use the unbroken line if $\phi_b/\Lambda \geq 1$ and dashed line if $\phi_b/\Lambda \leq 0.75$.

PERMEABILITY OF A COVER LAYER WITH HOLES AND/OR A GEOTEXTILE

H1 Introduction

As stated in Sections 5.2 and 8.2, the permeability of the cover layer is a parameter which greatly affects the pressure head difference on the cover layer. The flow in the filter and in the cover layer is shown schematically in Figure H1 as a flow which shoots out onto the slope through the cover layer from below, after flowing through the filter parallel to the cover layer. Because of the geometry of the pitching and because the permeability of the filter is larger than that of the cover layer, the flow in the cover layer is mainly perpendicular onto the slope. The pressure difference, due to the outward flow of water, can lift blocks out of the slope.

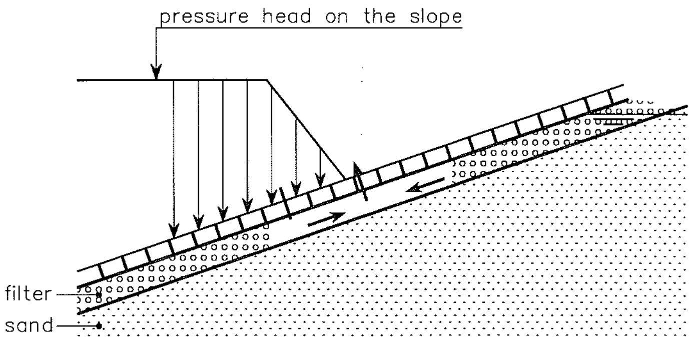


Fig. H1. Flow through the cover layer and the filter, shown schematically.

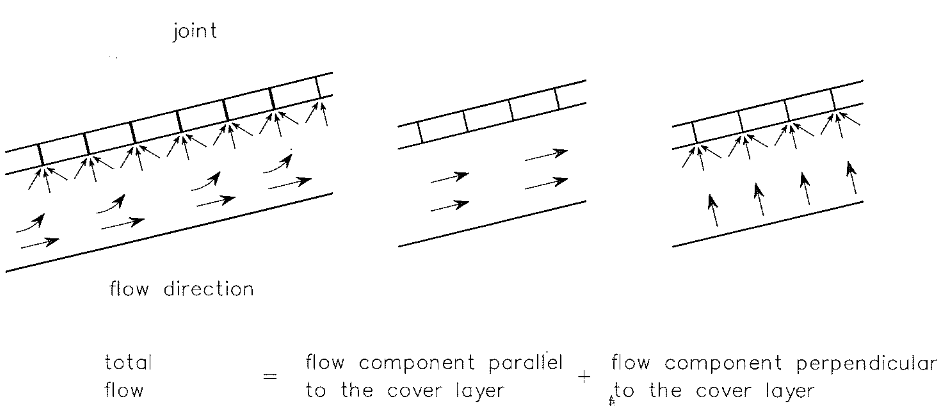


Fig. H2. Flow in the filter.

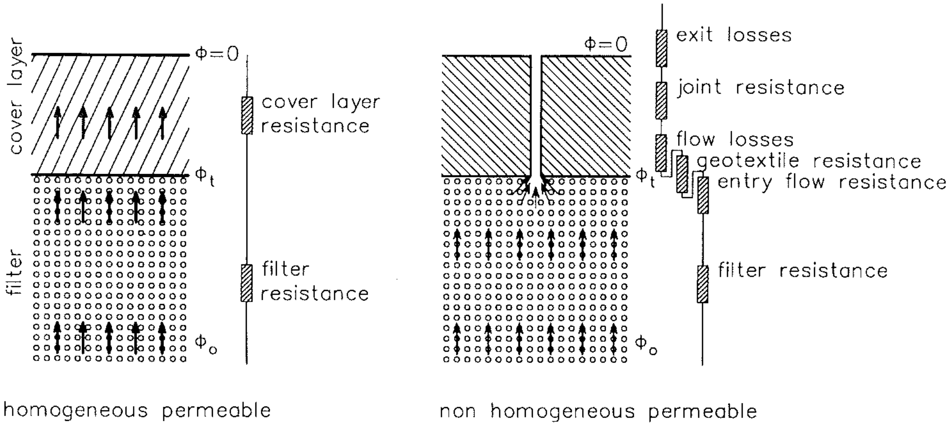


Fig. H3. Resistance components.

The place on the slope where the water flows through the cover layer outwards is shown in Figure H2. Not all the water flowing in the filter escapes. The figure also shows that the flow in the filter is divided into a component down the cover layer and one perpendicular to the surface. In this schematization the permeability of the cover layer relates only to flow perpendicular to the slope. This is shown in Figure H3.
The figure shows the various resistance components which the water must overcome as it flows upwards through the cover layer. These are:

1. *Entry flow resistance*
   Water in the filter must overcome an additional filter resistance as it flows towards the cover layer. The cross section of flow is suddenly greatly reduced because water can only flow through the joints between the blocks and any holes in the blocks. This results in additional flow resistance to that before the reduction in flow cross section, see the right hand part of Figure H3. This flow resistance is referred to as the entry flow resistance.
   The aim is to schematize and reduce the flow resistance to two components:
   - the discharge related to the entry flow resistance is equivalent to the discharge through the cover layer.
   - the head loss related to the entry flow resistance is one of the components of the pressure difference on the cover layer (which must be compensated for by the weight of the cover layer).

2. *The geotextile flow resistance*
   A relatively permeable geotextile (compared with the filter) can form a large resistance to flow under the cover layer. This is because the water only flows through a very small part of the geotextile, that is, the part directly below the joints and any holes. This leads to a relatively high flow velocity (about 10 to 100

times greater than that in the filter) and therefore a relatively high loss of head across the geotextile.

3. *Entry and exit flow resistance*
   Water flowing into the revetment approaches the geotextile through the joints between the blocks or through any holes. If there is no joint filling, washed-in material, Vena Contracta develop under the joints which cause a reduction in flow.
   Similarly water flowing out of the revetment has to contract on entering a joint or hole.

4. *The flow resistance caused by the walls of the joints or holes, and any washed-in material*
   Water flowing through a narrow joint is subjected to wall resistance. This resistance to flow is increased by granular washed-in material.
   Each of the above resistance components contributes to the pressure head difference on the cover layer:

$$\phi_t = \phi_1 + \phi_2 + \phi_3 + \phi_4$$

where:

ϕ_t = total head losses across the cover layer [m]
ϕ_1 = total head losses due to the entry flow resistance [m]
ϕ_2 = total head losses across the geotextile [m]
ϕ_3 = total head losses due to the entry and or exit resistance [m]
ϕ_4 = total head losses in joints and holes [m]

Examples of these various components are given in Table H1. From the table it appears that in this example for jointed blocks the entry flow resistance and exit resistance and the joint wall resistance are dominant. Therefore if a very permeable geotextile is placed under the cover layer, the permeability is halved and the flow resistance of the geotextile is dominant.
With washed-in columns the permeability is almost completely determined by the washed-in material, while for blocks with holes but no washed-in material the entry flow resistance is dominant, despite there being a geotextile under the cover layer. If the holes are washed-in, the filler material becomes dominant. The table gives only examples and general conclusions cannot be drawn from the information presented.

Table H1. Some examples of the contribution of different resistance components.

Component		Blocks with joints	Blocks with joints and a geotextile	Washed-in columns	Blocks with holes and a geotextile	Blocks with washed-in holes and a geotextile
Cover layer thickness [m]		0.20	0.20	0.20	0.20	0.20
Block/column width [m]		0.50	0.50	0.30	0.50	0.50
Joint width [mm]		1.50	1.50	15.00	1.00	1.00
Filter:	porosity [–]	0.40	0.40	0.40	0.40	0.40
	D_{f15} [mm]	5.00	5.00	20.00	5.00	5.00
Washed-in material:	porosity [–]	–	–	0.40	–	0.40
	D_{f15} [mm]	–	–	5.00	–	10.00
Geotextile:	head loss [mm]	0	10	0	10	10
	discharge [mm/s]	0	100	0	100	100
Number of holes		–	–	–	9	9
Hole area [cm^2]		–	–	–	5×5	5×5
Permeability [mm/s]		4.4	2.1	7.7	18.5	9.5
Component ϕ_1 [%]		8.2	2.3	0.3	70.0	16.7
Component ϕ_2 [%]		0.0	65.4	0.0	23.6	10.5
Component ϕ_3 [%]		43.8	9.8	0.5	3.5	1.6
Component ϕ_4 [%]		47.4	22.4	99.2	2.8	71.1

H2 Permeability of pitching with holes and/or a geotextile

The permeability of cover layers with holes (possibly formed by chamfered sides) and cover layers on a geotextile can be quantified using a number of diagrams. The procedure involves two to five steps, depending on whether there are only holes, only a geotextile, or both:

1. Determine the permeability of the cover layer with only joints between the blocks, that is, no geotextile or holes: k'_s.
2. Determine the factor for the effect of the geotextile and multiply it by the result of Step 1: $\gamma_{sg} \cdot k'_s$.
3. Determine the permeability of the cover layer with only holes in the blocks, that is, no geotextile or joints: k'_g.
4. Determine the factor for the effect of the geotextile and multiply it by the result of Step 3: $\gamma_{gg} \cdot k'_g$.
5. Calculate the total permeability:

$$\text{if } \gamma_{gg} \cdot k'_g < \gamma_{sg} \cdot k'_s \text{ then } k' = \gamma_{sg} \cdot k'_s + \tfrac{1}{2} \cdot \gamma_{gg} \cdot k'_g \tag{H1}$$

$$\text{if } \gamma_{gg} \cdot k'_g < \gamma_{sg} \cdot k'_s \text{ then } k' = \tfrac{1}{2} \cdot \gamma_{sg} \cdot k'_s + k'_s \cdot \gamma_{gg} \cdot k'_g \tag{H2}$$

If there is no geotextile under the cover layer, Steps 2 and 4 can be neglected and $\gamma_{sg} = \gamma_{gg} = 1$. If there are no holes, Steps 3, 4 and 5 can be neglected. The whole procedure is given in the chart in Figure H4 which also refers to the diagrams to be used for determining the permeability of the joints (k'_s), the holes (k'_g) and the factors for the geotextile (joints: γ_{sg}, holes: γ_{gg}). This chart replaces Figure 117, Section 8.4.3, for this type of cover layer.

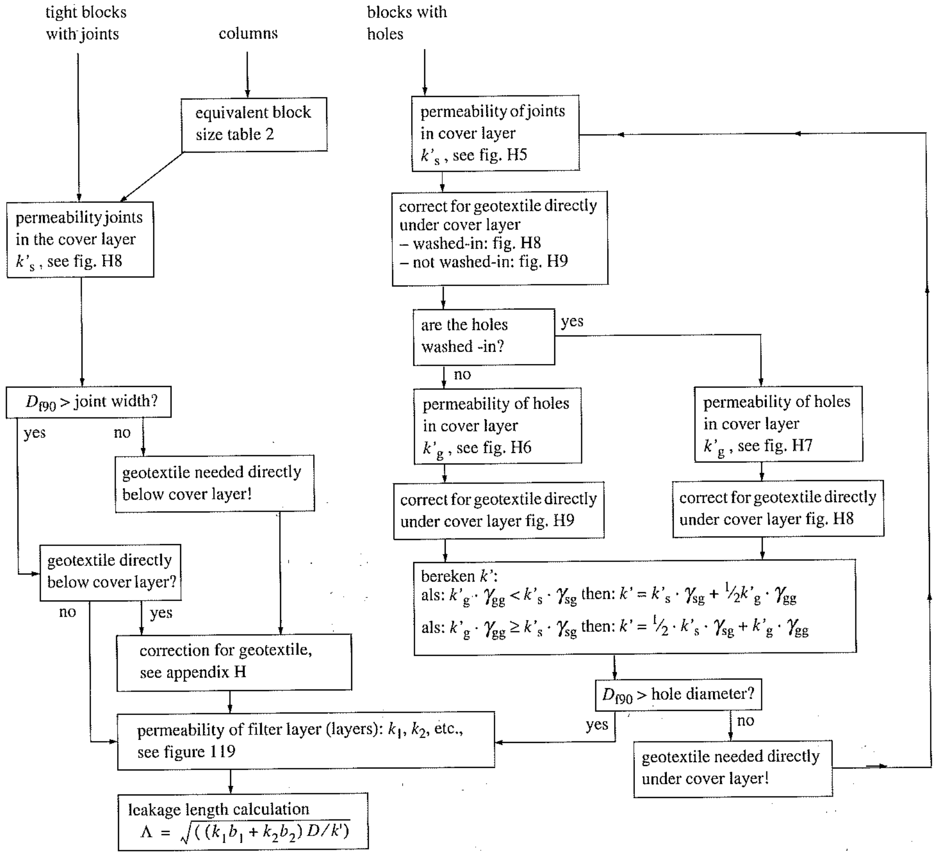


Fig. H4. Flow chart for assessing leakage length for revetments with holes and/or with a geotextile between the filter and the cover layer (replaces Figure 117, Section 8.4.3).

The following parameters can affect the permeability of a cover layer, with holes, laid on a geotextile:

- joint width (s);
- block width and length (B and L);
- block thickness (D);

– hole diameter (D_g);
– number of holes per block (N);
– permeability of the geotextile:
  – drop in head during permeability measurements (ϕ_g);
  – specific discharge during permeability measurements (q);
– porosity of the filter or the filter layer (n);
– grain size of the filter or the filter layer (D_{f15});
– porosity of the washed-in material in the joints (n_{vs});
– grain size of the washed-in material in the joints (D_{v15s});
– porosity of the washed-in material in the holes (n_{vg});
– grain size of the washed-in material in the holes (D_{v15g}).

Table H2 gives the parameters which must be known in order to determine the permeability of the cover layer.

Table H2. Information required to determine the permeability of various types of cover layer.

Cover layer	With geotextile		With holes				With geotextiles and holes			
Washed-in joints ?	no	yes	no		yes		no		yes	
Washed-in holes ?	n.a.	n.a.	no	yes	no	yes	no	yes	no	yes
Joint width, s	x	x	x	x	x	x	x	x	x	x
Block size, B and L	x	x	x	x	x	x	x	x	x	x
Thickness of the cover layer, D	–	x	x		x	–	x	x	x	x
Holes, D_g and N	–	–	x	x	x	x	x	x	x	x
Geotextile ϕ_g and q	x	x	–	–	–	–	x	x	x	x
Filler, n and D_{f15}	x	–	x	x	x	–	x	x	x	–
Joint filler, n_{vs} and D_{v15s}	–	x	–	–	x	x	–	–	x	x
Hole filler, n_{vg} and D_{v15g}	–	–	–	x	–	x	–	x	–	x

The chart given in Figure H4 is explained below using an example.

H3 Calculation Example

The permeability for a cover layer comprising rectangular blocks on a geotextile, on a filler layer, on a filter layer of minestone is calculated as an example.
Each block has a 6×6 cm^2 hole, washed-in with stones with a characteristic grain size of 7 mm and a porosity of 0.3. The joints between the blocks are only 2.5 mm wide and are not completely filled with washed-in material. The cover layer can be characterised as follows (because there is a filler layer, the details of the minestone are not relevant):

- joint width: $s = 2.5$ mm;
- block thickness: $D = 0.15$ m;
- block width and length: $B = L = 0.32$ m;
- hole diameter: $D_g = 60$ mm;
- number of holes per block: $N = 1$;
- permeability of the geotextile:
  - head loss during permeability measurements: $\phi_g = 0.03$ m;
  - specific discharge during permeability measurements: $q = 0.01$ m/s;
- porosity of filter layer: $n = 0.3$;
- grain size of filter layer: $D_{f15} = 3$ mm;
- porosity of washed-in material in holes: $n_{vg} = 0.3$;
- grain size of washed-in material in holes: $D_{v15g} = 7$ mm.

The permeability of the cover layer can be calculated using the flow diagram given in Figure H4:

1. Permeability of the joints between the blocks, without the geotextile:
   $s = 2.5$ mm
   $n = 0.3$
   $D_{f15} = 3$ mm
   $B = L = 0.32 \Rightarrow 2BL/(B + L) = 0.32$ m
   From Figure H5 it follows that: $k'_s = 8$ mm/s

2. Permeability of the joints between the blocks, with the geotextile:
   $\phi_g = 0.03$ m
   $q = 0.01$ m/s
   $B = \mathrm{L} = 0.32$ m
   $$s = 2.5 \text{ mm} \Rightarrow \Omega = \frac{s\,(B + L)}{BL} = 0.016$$
   $n = 0.3$
   $D_{f15} = 3$ mm
   From Figure H9 it follows that: $\gamma_{sg} = 0.07 \Rightarrow k'_s \cdot \gamma_{sg} = 0.6$ mm/s

3. Permeability of the holes in the blocks, without the geotextile:
   $D_g = 60$ mm
   $B = L = 0.32$ m
   $N = 1 \Rightarrow BL/N = 0.1\ \mathrm{m}^2$
   $D_{v15g} = 7$ mm
   $n_{vg} = 0.3$
   From Figure H7 it follows that : $k'_g = 4$ mm/s

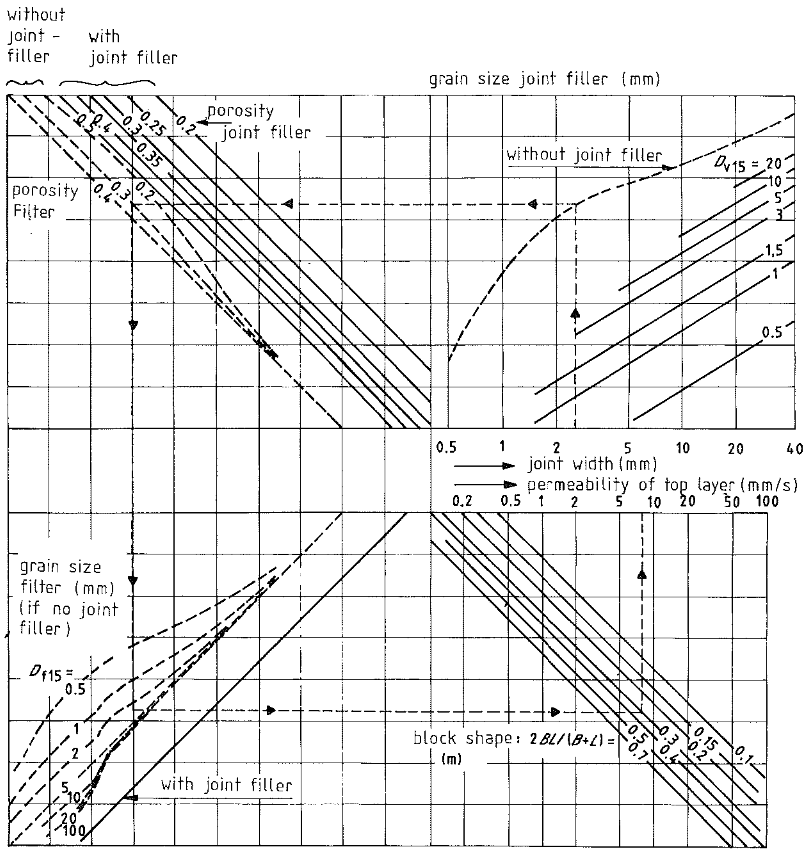


Fig. H5. Permeability of a cover layer with joints k_s', without holes or geotextile. (unbroken line – no washed-in material, dashed lines – with washed-in material)

4. Permeability of the holes in the blocks, with the geotextile:

ϕ_g = 0.03 m

q = 0.01 m/s

D_{v15g} = 7 mm

n_{vg} = 0.3

D = 0.15 m

From Figure H8 it follows that: $\gamma_{gg} = 0.33 \Rightarrow k_g' \cdot \gamma_{gg} = 1.3$ mm/s

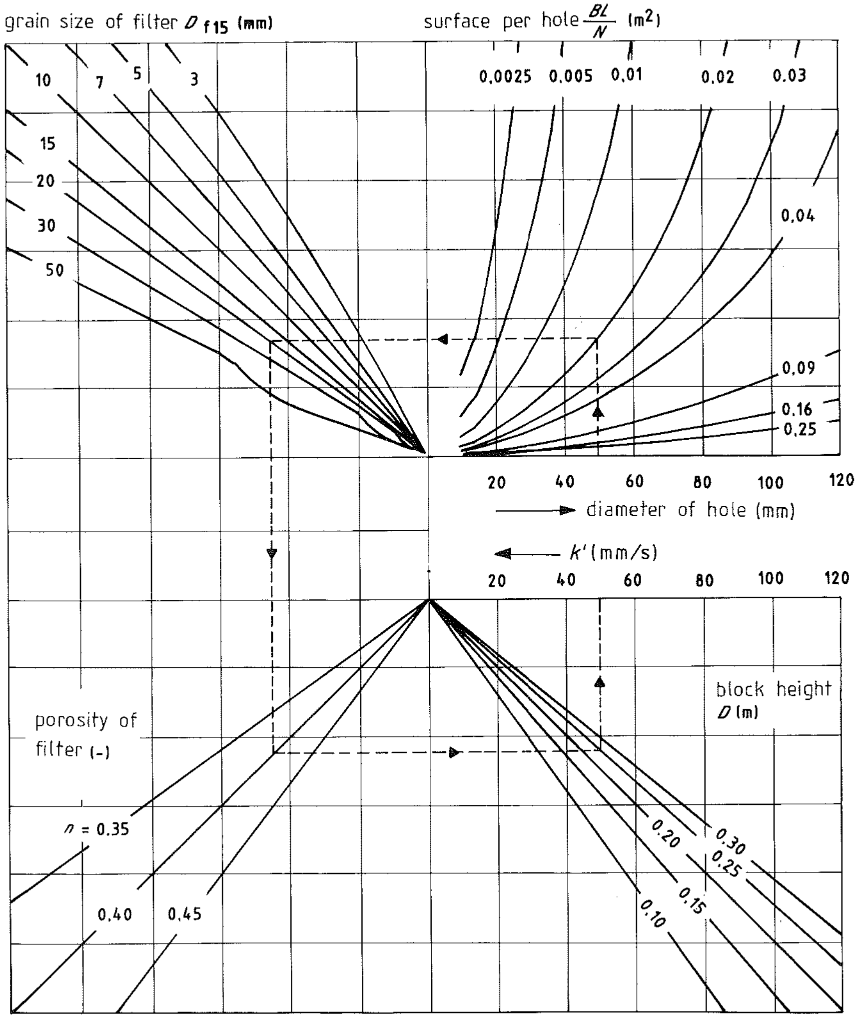


Fig. H6. Permeability of a top layer with holes without washing-in material (k_s') without possible joints or geotextile.

5. Calculation of the total permeability:

$k_s' \cdot \gamma_{sg} = 0.6$ mm/s

$k_g' \cdot \gamma_{gg} = 1.3$ mm/s

$\gamma_{gg} \cdot k_g'$ is larger than $\gamma_{sg} \cdot k_s' \Rightarrow k' = \frac{1}{2} \cdot \gamma_{sg} \cdot k_s' + \gamma_{gg} \cdot k_s' = \underline{1.6 \text{ mm/s}}$

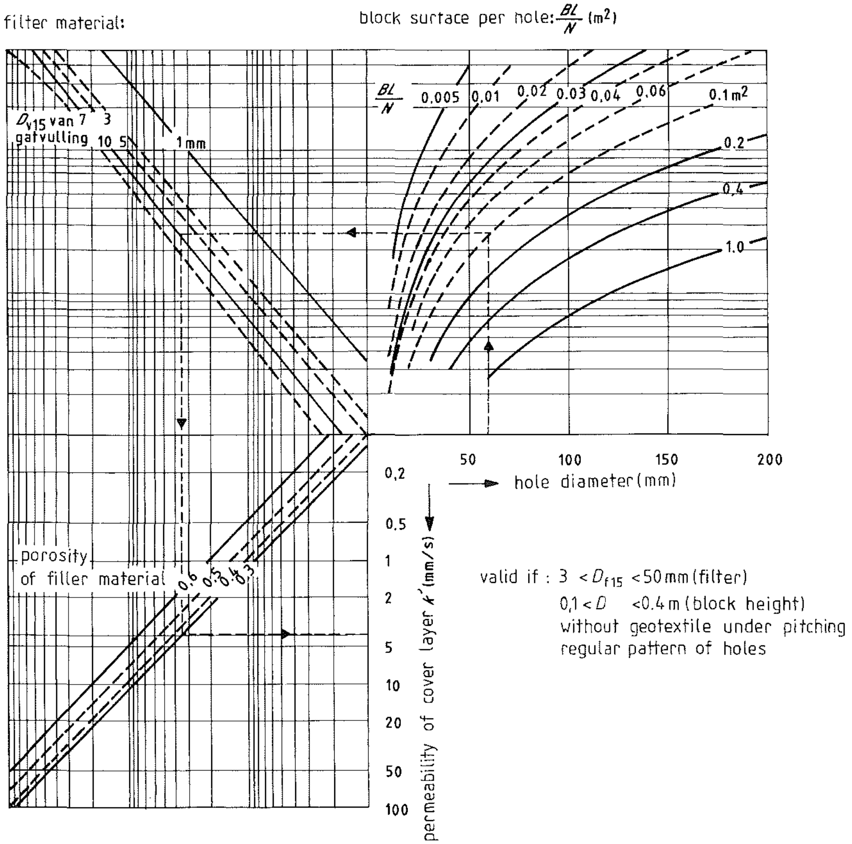


Fig. H7. Permeability of a cover layer containing holes with washed-in material (k_s') but without joints or geotextiles.

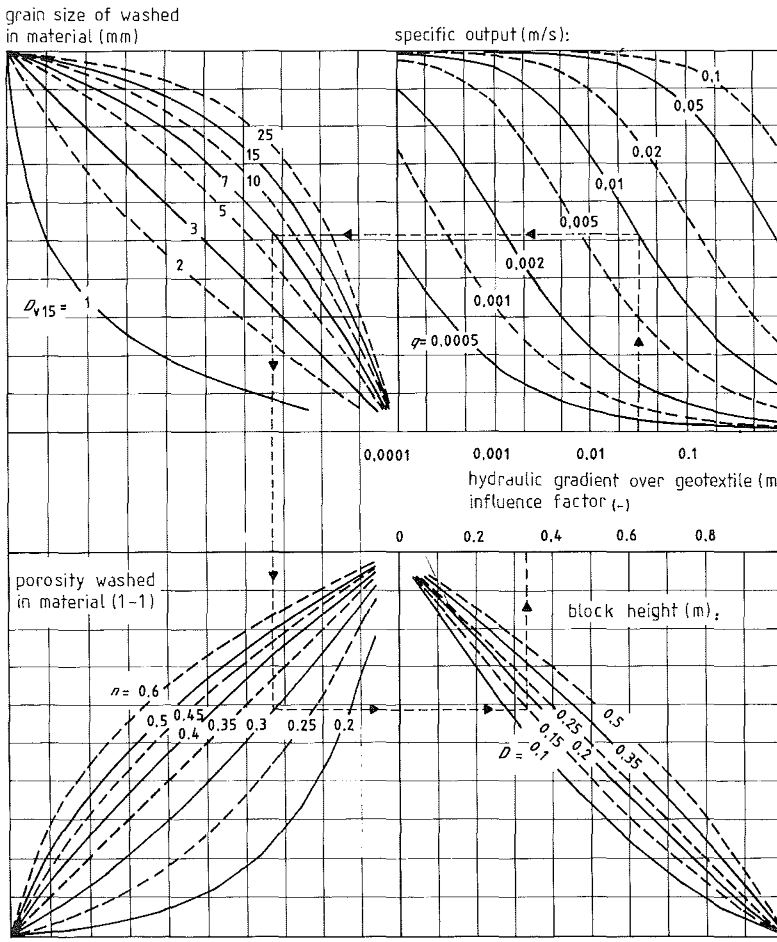


Fig. H8. Influence factor for a geotextile with washed-in material in the joints (γ_{sg}) or holes (γ_{gg}).

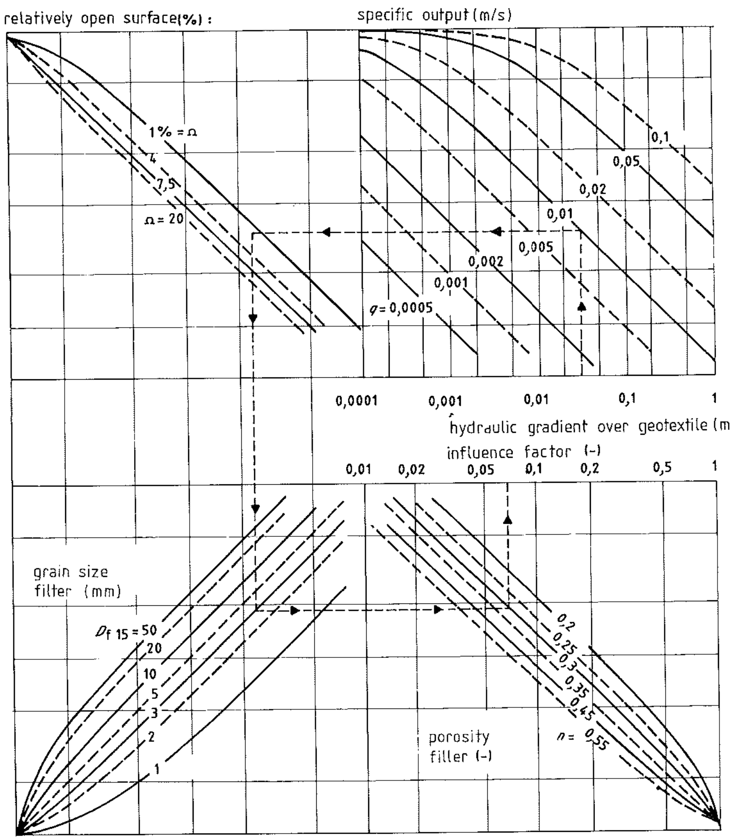


Fig. H9. Influence factor for a geotextile without washed-in material in the joints (γ_{sg}) or holes (γ_{gg}).

Appendix I

BACKGROUND TO THE "RULES OF THUMB" FOR ESTIMATING THE MAXIMUM WAVE HEIGHT IN SHALLOW WATER

Section 6.2.2 describes how the wave height is gradually reduced in relatively shallow water by the breaker process. For this process there is, by approximation, a relationship between wave height and water depth. This relationship for the significant height is different to that for very high waves, say $H_{2\%}$. This appendix considers the background to the following "rules of thumb":

$$(H_s)_{max} = 0.5 \cdot d \tag{I1}$$
$$(H_{2\%})_{max} = 0.6 \cdot d \tag{I2}$$

where:

$(H_s)_{max}$ = maximum significant wave height (breaking, because of limited water depth) [m]
$(H_{2\%})_{max}$ = maximum wave height in shallow water which is exceeded by 2% of the waves [m]
d = design water depth [m]

The relationship between $(H_s)_{max}$ and $(H_{2\%})_{max}$ is different here to that described in Section 6.2.1 because the wave height distribution due to the breaking process is not a Rayleigh distribution. The size of $(H_s)_{max}$, for example, can be seen as an upper limit for the significant wave height for a particular value of d. If the significant wave height in deeper water is smaller, then the wave height in shallower water is also less than the given maximum. The relationship between the wave height and the water depth is given in Figure I1 for waves with a deep water height of 2.5 m (and $H_{2\%} = 1.4 \cdot 2.5 = 3.5$ m). The possible effect of shoaling, etc, is neglected. The design water depth is the depth at a certain distance seawards of the structure. This distance is equivalent to $\frac{1}{2} L_{op}$ (L_{op} = deep water wave length $gT_p^2/(2\pi)$) .
This "rule of thumb" is based on the design graphs of [Van der Meer, 1990], some of which are included in Appendix A. These design graphs are based on ENDEC calculations and establish the relationship between the dimensionless maximum significant wave height, $(H_s)_{max}/h$, and the following parameters:

- wave steepness in deep water: H_{so}/L_{op},
- steepness of the foreshore: $\tan\alpha_v$, and
- relative local water depth: h/L_{op}.

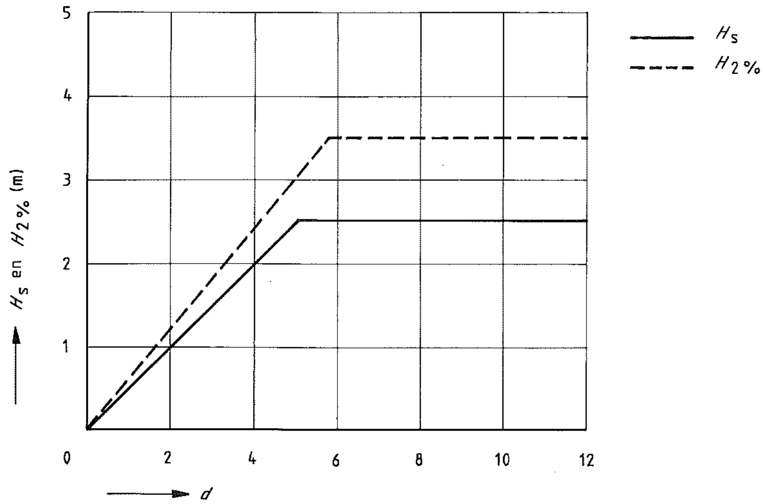


Fig. I1. Example of the relationship between the wave height and the ruling water depth (deep water: $H_s = 2.5$ m).

where:

H_{so} = wave height in deep water [m]
a_v = slope of the foreshore [°]
h = local water depth [m]
L_{op} = wave height in deep water = $gT_p^2/(2\pi)$ [m]

The "rules of thumb" only apply to situations in which:
- the relationship between the local significant wave height and the wave length in deep water (H_s/L_{op}) is between 0.01 and 0.05;
- there is a "gradual" reduction in water depth towards the structure;
- the average bed slope is less than 1:30 near to the structure (about $L_{op}/2$ seawards);
- the effects of refraction and diffraction can be neglected.

The "rules of thumb" for this situation are given in the following section.

I1 Significant wave height

Lines from the design graphs of [Van der Meer, 1990] are presented in Figures I2, I3 and I4. Because the axes have been changed they may appear to be different though representing the same relationship.

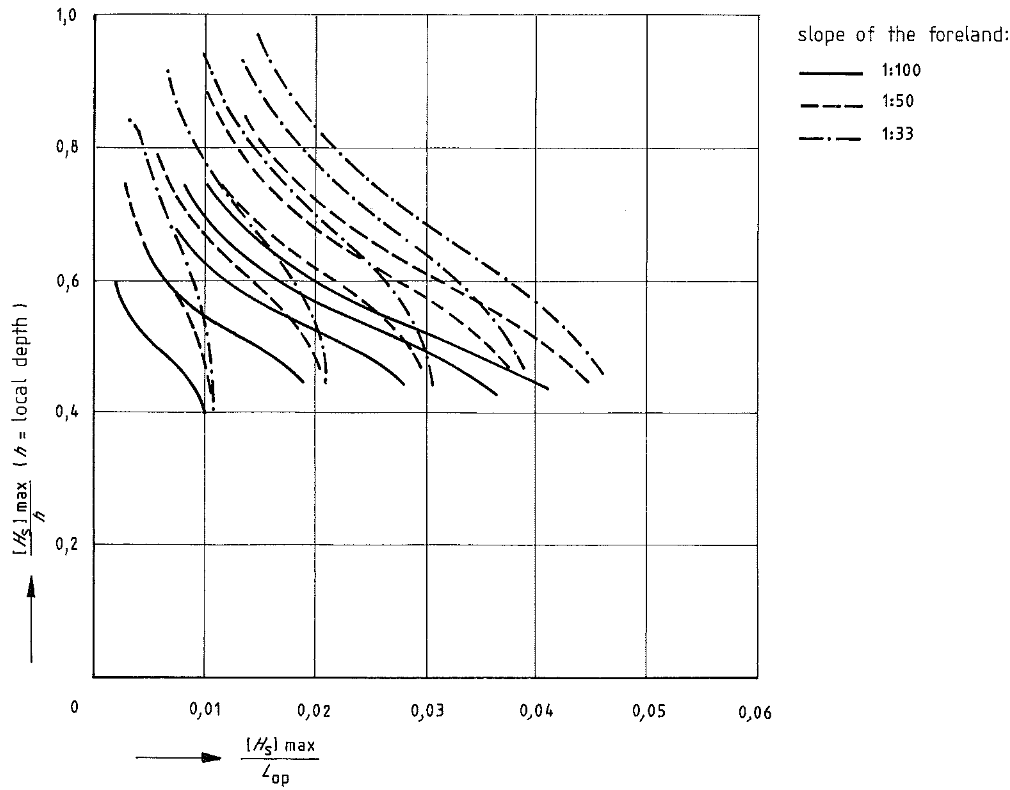


Fig. 12. $(H_s)_{max}/h$ as a function of $(H_s)_{max}/L_{op}$, (h = local water depth).

The design water depth can be selected from these figures.
The relationship between $(H_s)_{max}$ and the local water depth (h), is plotted in Figure 12, against the effective local wave steepness, $(H_s)_{max}/L_{op}$, which is based on the local deep water wave height and length. The shallow water wave height is not used here because it is difficult to calculate and the intention here is to use "rules of thumb". From the figure it can be seen that $(H_s)_{max}/h$, the breaker index, can vary between 0.4 and 1, depending on the bed slope and the wave steepness. "Rules of thumb" cannot be selected from this figure, because there is too much variation in the breaker index. What is required is a simple way of expressing the effect of the bed slope and wave steepness. In order to define a design water depth at some distance in front of the structure it was decided to assume that the breaking wave should always have some distance to travel during the breaking process. The breaking process is treated as a wave entering shallow water which only breaks some distance further on.

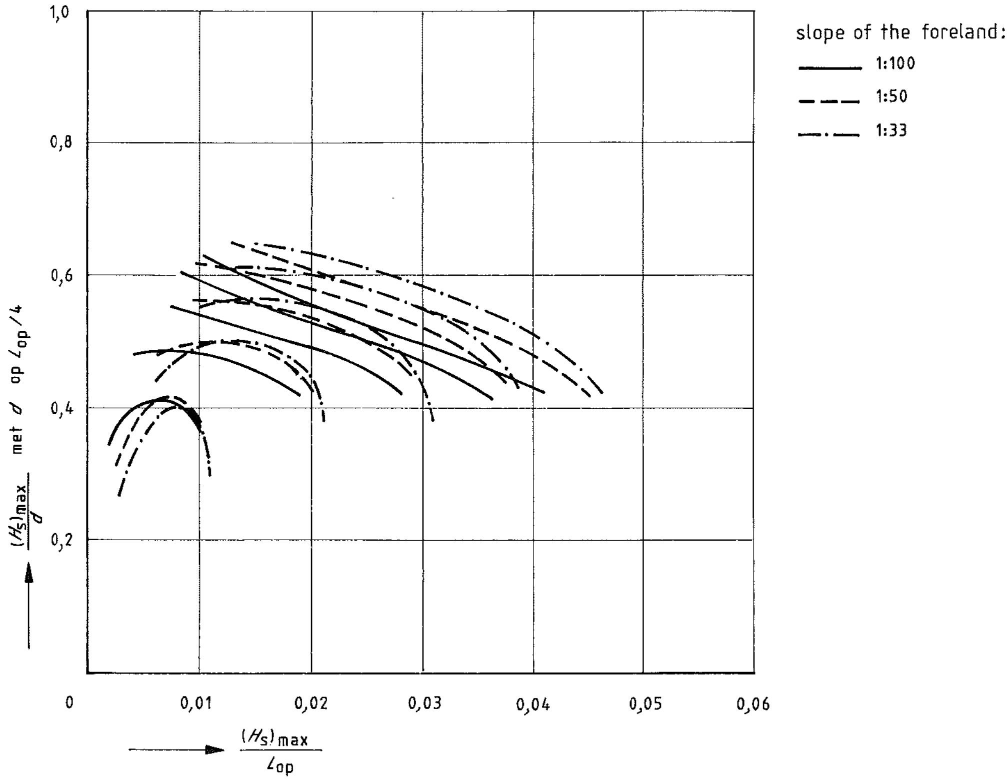


Fig. I3. $(H_s)_{max}/d$ as a function of $(H_s)_{max}/L_{op}$. (d = water depth at a distance of $\frac{1}{4}L_{op}$ from the structure).

Figure I3 uses a ruling water depth of $L_{op}/4$. In this case there appears to be a much smaller distribution than in Figure I2. The value of $(H_s)_{max}/d$ varies between 0.4 and 0.6 in the area of interest, $0.01 < (H_s)_{max}/L_{op} < 0.05$.
The ruling depth finally selected for a distance $L_{op}/2$ from the toe of the structure is presented in Figure I4. This gives a somewhat smaller distribution than Figure I3. In the area of interest, $0.01 < (H_s)_{max}/L_{op} < 0.05$, the following safe (conservative) estimate can be made for the relative water depth:

$$(H_s)_{max}/d \approx 0.5 \tag{I3}$$

where:

d = ruling water depth water depth at a distance $L_{op}/2$ from the structure [m].

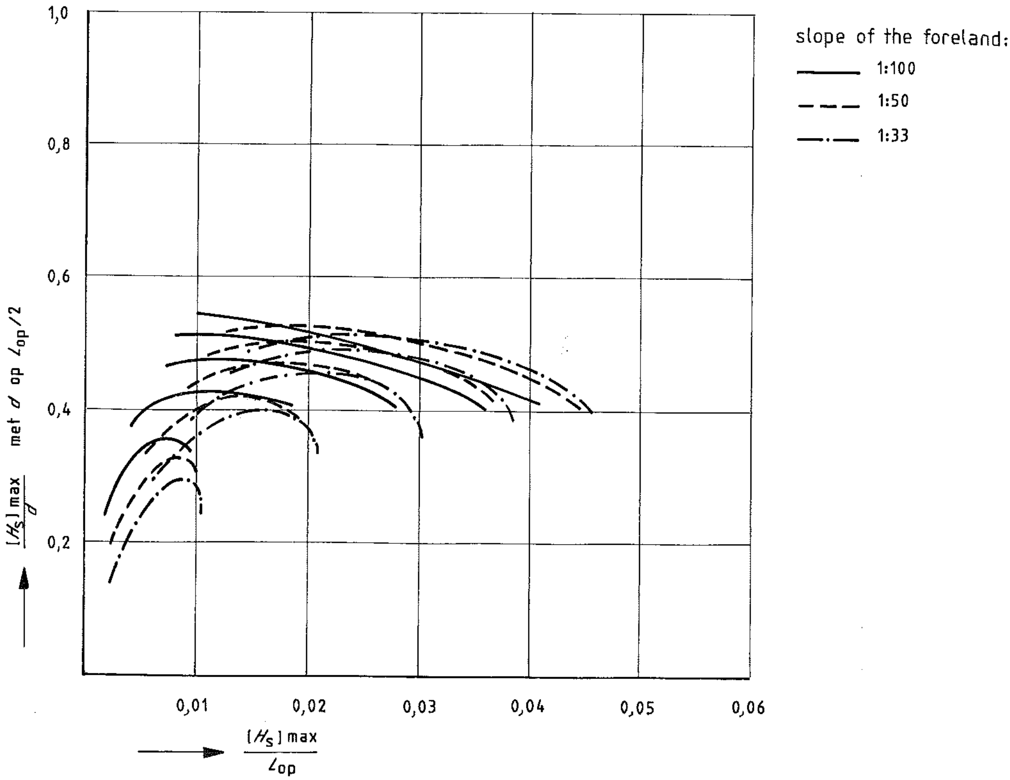


Fig. I4. $(H_s)_{max}/d$ as a function of $(H_s)_{max}/L_{op}$, (d = water depth at a distance of $\frac{1}{2} L_{op}$ from the structure).

I2 Wave height with a small percentage exceedance

The maximum wave height in shallow water, which is exceeded by 2% of the waves, $(H_{2\%})$max, can be calculated using an equation from the work of [Van der Meer, 1990]:

$$H_{1\%} = 1.517 \cdot \frac{H_s}{(1 + H_s/h)^{1/3}} \tag{I4}$$

By extrapolating the coefficient, so that the Rayleigh deep water holds, see Section 6.2.1, it follows that:

$$H_{2\%} = 1.4 \cdot \frac{H_s}{(1 + H_s/h)^{1/3}} \qquad \text{(I5)}$$

If the assumptions applied for H_s are also applied here, namely that the ruling depth occurs at $L_{op}/2$ in front of the structure, then:

$$(H_{2\%})_{max} = 1.4 \cdot \frac{(H_s)_{max}}{\{1 + (H_s)_{max}/d\}^{1/3}} \qquad \text{(I6)}$$

where:

$(H_s)_{max}/d = 0.5$ it follows then finally that:

$$(H_s)_{max}/d = 0.6 \qquad \text{(I7)}$$