CHAPTER 114

NEW CRITERIA FOR GRANULAR FILTERS AND GEOTEXTILE FILTERS UNDER REVETMENTS

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Abstract

New criteria for the design of filters under revetments are introduced and verification tests are described. These criteria are based on a hydrodynamic limit instead of the usual geometric limit, assuming a similarity between the flow in open channels and flow in the pores at an interface between filter and base. First a theory was developed for a granular filter only, and later this theory was extended to include the influence of a geotextile. The verification experiments are outlined, and the applicability of the derived formulae is discussed.

Introduction

For the design of coastal structures, such as dike revetments or a breakwater, several aspects have to be considered. Not only the stability of the cover layer is important, but also the stability of the foundation layers, i.e. the filter layers has to be assured. In this paper we restrict ourselves to the filter layers. Until recently the design of these filter layers was based on strict rules demanding geometrically tight filters: the pores in the filter material have to be smaller than the particles of the subsoil to be protected.

The geography of the Netherlands with vast area's of its surface beneath sea level urge a major infrastructure in order to protect its population against flooding hazards. Many dike revetments are necessary, so it is of

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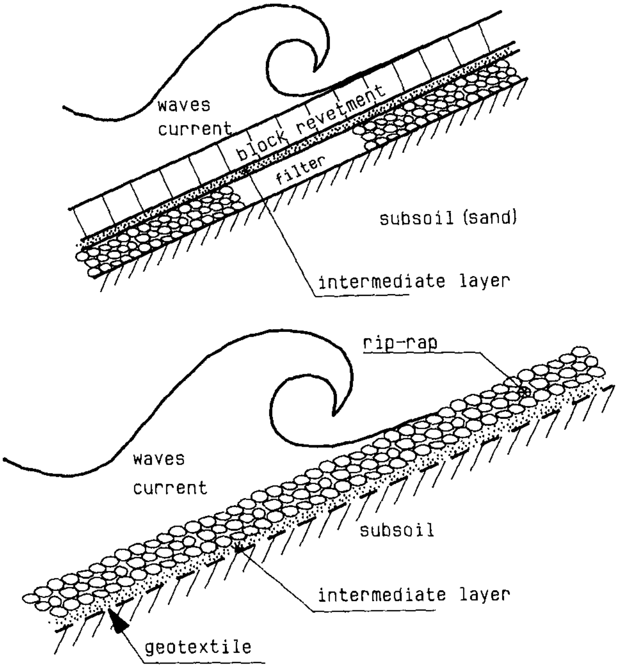


Fig 1. Wave attack on revetments

vital interest to be able to dimension these structures both as safe and as economic as possible. Mainly they can be outlined as in Fig. 1.

For the case we are concerned with, e.g. filter layers between cover layer and base, under a revetment, damage to the filter layer leads to inadmissible settlements of blocks in the cover layer. As a result of research concerning revetments, it was found that the hydraulic gradients in this type of construction are related to the slope gradient. These gradients are relatively small except within the direct vicinity of the surface of the structure.

For transport of grains, forces to activate particle movement have to be present, such as water flowing through the soil skeleton. When hydraulic gradients are small, the "no transport" demand can thus be achieved as well by controlling the pore water velocity; i.e. limiting the permeability of the filter material. In the Netherlands much effort has been put in research to derive new filter rules, based on a combination of geometric and hydrodynamic effects. This has led to filter criteria which might be a factor four less stringent concerning the ratio of filter to base grain size. After verification of these filter criteria for granular material, the theory was extended to incorporate the influence of a geotextile, at the interface between filter and base material. Assuming that the geotextile has a damping effect on the hydraulic load on the base material, an extended design formula was derived and calibrated with experimental research.

Theory for granular filter interface

Traditionally filter criteria are based on geometric considerations which yield criteria which are based on limiting the diameter of the pore-channels connecting the pore holes in the soil skeleton. These constrictions in the filter must be that small, that the characteristic grain size of the base material is larger, so that base particles cannot pass these constrictions.

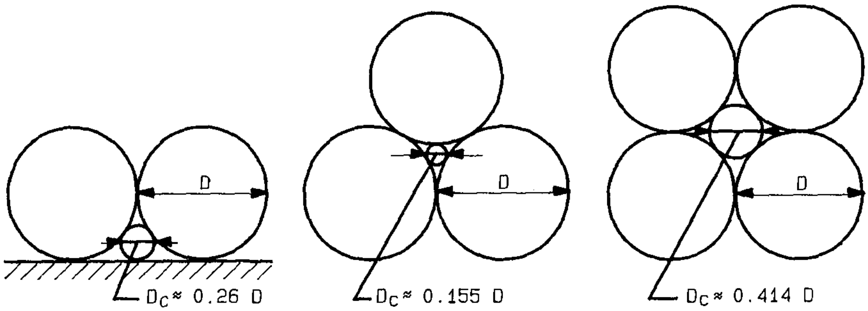


Figure 2 The characteristic constriction size is a function of grain size, and packing of the soil skeleton

In this paper we will distinguish between filter and base material by using D for the diameter of the filter material and d for that of the base material.

According to Kenney (1985), the characteristic constriction in the filter material is

$$D_c^* <= 0.2\ D_{15} \tag{1}$$

and the characteristic grain size of the base material is d_{85}. This leads to a design rule which was conducted first by Terzaghi which says that the ratio D_{15}/d_{85} must be smaller than 4 or 5.

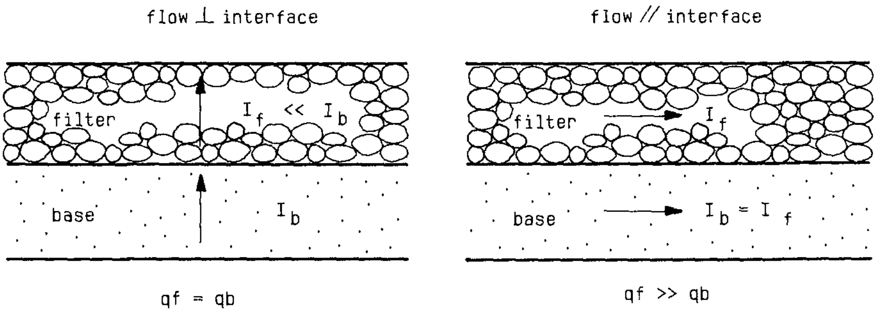


Figure 3 Hydraulic gradients

Yielding; $$\frac{D_{15}}{d_{85}} < 5 \qquad (2)$$

Though later researchers like Thanikachalam (1974) and others have conducted criteria including effects of the grain size distribution, geometric filter criteria are generally tight.

In order to introduce hydraulic criteria we have to classify the loading conditions, see Fig. 3. Traditionally filter experiments to verify for geometric penetration where performed applying flow perpendicular to the filter interface. However, in practical circumstances, the hydraulic loading is often dominated by the hydraulic gradient parallel to the filter interface. As the pore water velocity is determined by the higher permeability in the coarser filter, the hydraulic gradient parallel will lead to higher velocities in the filter than in the base material. On the contrary with perpendicular loading the velocities are approximately the same since continuity must hold.

If the diameter of the pore holes in the filter is large with respect to the grain size of the base-material, the hydraulic loading on the base material can be related to the loading of a sand bottom in an open channel. It is assumed that, given a granular filter with pores much wider than the grains in the base layer, the critical shear over the sand interface is equal to the critical shear in a channel with the same bed material.

Shields related the critical shear stress to the diameter of the base material. As the critical shear stress is related to the water velocity, a relation between the filter velocity and the diameter of base material can be derived, adding corrections for hydraulic conditions in a filter. The threshold of sediment motion

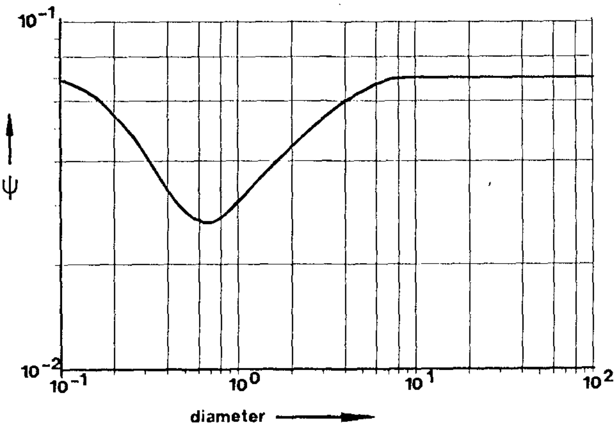


Figure 4 Shields parameter.

in open channels according to Shields is;

$$\tau_{cr} = \Psi_s \Delta g D_{50} \rho \tag{3}$$

Where;

τ_{cr} = critical shear stress
Ψ_s = Shields parameter
Δ = relative density of sand grains $\left(\frac{\rho_s - \rho}{\rho}\right)$
g = acceleration of gravity
D_{50} = grain size corresponding to 50% by weight of finer particles
ρ = mass density of water
ρ_s = mass density of sand

By introduction of the shear velocity $V_* = \sqrt{\tau/\rho}$, equation (1) can be rearranged to yield;

$$V_{*cr} = \sqrt{\Psi_s \Delta\ g\ D_{b50}} \tag{4}$$

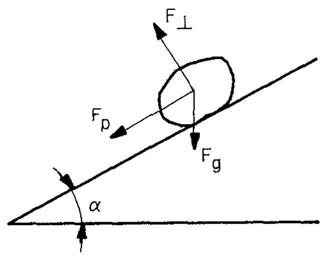
Where V_{*cr} = the critical shear velocity.

Formula (4) was extended in order to calculate the water velocity at threshold of sediment transport for steady flow parallel to a horizontal interface between a filter layer and a base layer, assuming a ratio between the shear velocity and the pore water velocity;

$$q_{cr} = n\ V_{pcr} = \frac{n}{\kappa} \sqrt{\Psi_s \Delta g d_{50}} \tag{5}$$

Where κ is the ratio V_*/V_p between shear velocity, and pore water velocity, and q is the filter velocity. The magnitude of coefficient κ was determined experimentally; κ will be discussed in the next section.

For as far as the inclination concerns, assuming that at threshold of particle motion, there is an



$F_\perp$: perpendicular

F_p: Parallel force

F_g: Gravitational

Figure 5 Equilibrium of forces

equilibrium of three forces, see Fig. 5, this leads to:

$$F_p + F_g \sin \alpha = \tan \phi \left\{ F_g \cos \alpha - F_\perp \right\} \tag{6}$$

With ϕ : natural angle of repose of single grains of the base material
α : slope angle

Furthermore the gravitational force, F_g is virtually neutralized when fluidisation occurs:

$$i_f = \Delta(1 - n_b) \tag{7}$$

Where n_b is the porosity of the base material.

Since the shear forces are proportional to V^2, the critical filter velocity q_{cr} can be written as:

$$q_{cr} = \frac{n}{\kappa} \sqrt{s^{\Delta g d_{50}} \left(\frac{\sin(\phi - \alpha)}{\sin\phi} \right) - \frac{i_\perp}{\Delta(1 - n_b)}} \tag{8}$$

where one can identify a part that is related with the hydraulic conditions, a part that accounts for the slope angle, and a part which accounts for the perpendicular hydraulic gradient. A more detailed derivation is given in Bezuijen, Klein Breteler and Bakker (1987)

Finally a permeability relation is needed to relate the filter velocity to the grain size of the filter material. As this filter material is relatively coarse, we do have to account for the turbulence intensity. Therefore we suggest a non linear permeability relation such as proposed by Forchheimer:

$$i = a\,q + b\,q^2 = a\,q\,(1 + \frac{b}{a}q) \qquad (9)$$

with;

$$a = 160\ \frac{\nu}{g}\ \frac{(1-n)^2}{n^3\ D_{15}^2}$$

$$b = \frac{2.2}{g\ n^2\ D_{15}}$$

where;

ν = kin. visc.of water: 1.2 10^{-6} [m^2/s] (13^o)

This relation which is a quadratic equation in the filter velocity, can be solved directly, and this gives a secant linear expansion in the permeability

$$k^s = \frac{-a + \sqrt{a^2 + 4bi}}{2bi} = \frac{a}{2bi}\left\{\sqrt{(1 + \frac{4bi}{a^2}} - 1\right\} \qquad (10)$$

if $4bi/a^2 \ll 1$ than $\sqrt{(1 + 4bi/a^2}$ can be approximated by $1 + 1/2\ (4bi/a^2)$; the secant permeability becomes than $1/a$; The Darcy result.

If $4bi/a^2 \gg 1$, than $\left\{\sqrt{(1 + 4bi/a^2)} - 1\right\} \simeq (2/a)\sqrt{bi}$

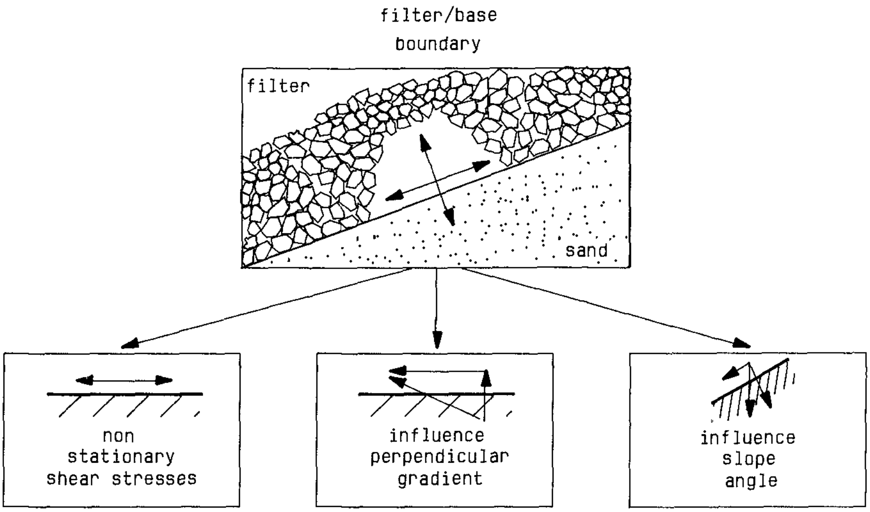


Figure 6 Loading conditions

So than $K^s = 1/\sqrt{bi}$, the result for turbulent flow.

Also experimentally, by measurement it is relatively simple to identify a laminar part, and a turbulent part. Dividing the filter velocity by the hydraulic gradient, one gets;

$$i/q = a + b\ q \tag{11}$$

By measuring the hydraulic gradient for two values of the hydraulic discharge, a and b can be derived relatively simple.

Verification by model tests for granular filters

The formulae (5) and (8) have been verified by means of laboratory experiments, using non cohesive base materials with steep sieve curves.

In order to verify formula (8), it was tested for the different load components. First stationary flow was verified, both on a horizontal bed, and on sloping beds. Then test where carried out with a perpendicular gradient added to the horizontal flow, and finally the effect of cyclic flow was tested.

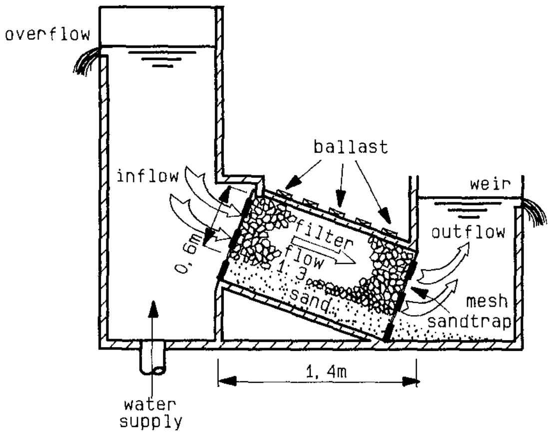


Figure 7 Delft Hydraulics Filter-box, with inclined interface

All these tests were performed in facilities where the sand transport could be measured and where the stability of the interface could be observed visually, except the tests with cyclic flow (visual observation only). The hydraulic loading was increased until there was particle transport. For the stationary flow, both on a horizontal bed, as well as for the inclined beds, the

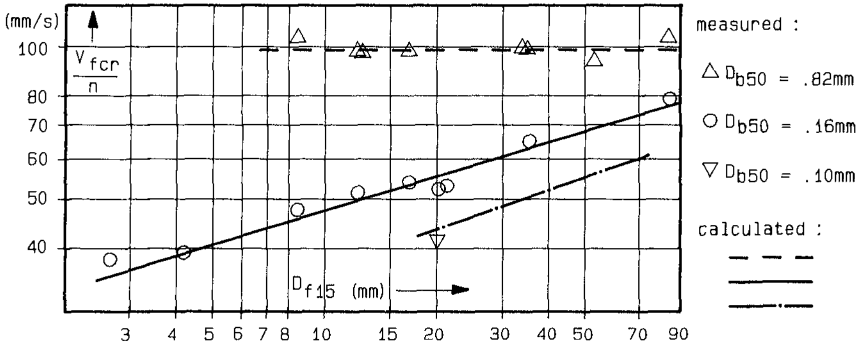


Figure 8 Filter-box test results for stationary flow parallel to a horizontal interface

filter box of Delft Hydraulics was used. see Fig 7. Tests were performed using different base materials with diameters ranging from 0.10 mm up to 0.82 mm and different ratio's D/d. The hydraulic gradient was increased step by step until considerable erosion took place. Sand transport of 0.2 gr/s/m2 of dry sand was considered to be critical.

In Fig. 8 some results of the experiments in the filter box are given. The values for the coefficient κ where determined from this figure. As κ becomes constant for larger diameters of the base material the critical filter velocity becomes constant. For the finer base material the critical filter velocity is still a function of the filter material, as this material determines the turbulency conditions. This yielded the following relations for κ;

$$\kappa = \frac{V_*}{V_p} = 0.8\ Re_{cr}^{-0.2} \qquad \text{for } 0.1 < d_{50} < 0.3 \text{ mm}$$

$$\kappa = 0.20 \qquad \text{for } 0.5 < d_{50} < 1.0 \text{ mm}$$

$$\kappa = 0.35 \qquad \text{for } d_{50} > 2 \text{ mm}$$

For $0.1 < d_{50} < 0.3$ mm in formula (8) q_{cr} is a function of Re_{cr}. and therefore a function of q_{cr}, so formula (8) is an implicit relation, however by rearranging q_{cr}, an explicit relation can easily be found.

From tests where a perpendicular gradient was added it was concluded, that though there is a lot of scatter in the results, the proposed formula gives a safe estimate of the critical horizontal filter velocity.

Under practical circumstances the stationary perpendicular gradient is often small, and can than be neglected. For the non stationary component, especially for smaller particles, the stability is enhanced. Since if a particle would move perpendicular to the interface, the removal of the particle from the bed would create an under pressure between particle and subsoil, reducing the gradient, thus giving a greater stability than suggested by the fluidisation gradient.

In order to investigate the influence of a cyclic component in the water velocity, tests were performed in the pulsating water tunnel of Delft Hydraulics. Tests were performed with a wave period of 2 s. which is relatively small compared to wave periods experienced on dikes and bank protection. The experimental results indicated, that for a wave period of at least 2 s. the critical velocity does not differ significantly from the stationary result. The experimental results where discussed in more detail by Bezuijen, Klein Breteler and Bakker (1987)

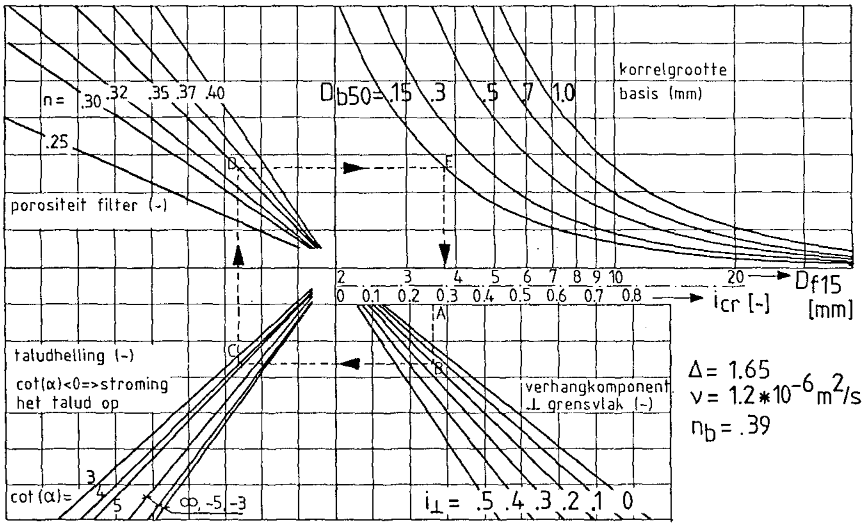


Figure 9 Design diagram for granular filter interface

Finally wave loading tests on a revetment in the Scheldt flume of Delft Hydraulics was performed. In these tests the combination of all loading aspects were present, and the filter was build based on the afore mentioned filter criteria for a critical design. The tests were used to derive wave loading characteristics too. The verification was both visually, and by measurements of particle transport. No transport of base material was noticed.

Based on the derived theory a design diagram was made. Assuming that hydraulic conditions are known, one can use this, either to find the admissible hydraulic conditions for a given base material, and ratio between filter and base material, as is indicated in Fig. 9, or one can use it the other way round, and derive the filter ratio, when hydraulic conditions are given.

Extension of theory to geotextile interfaces

Once thus far the question arose whether the theory could be extended to include the influence of a geotextile. In comparison to an interface without one, the geotextile reduces the hydraulic loading because the boundary layer of the water flow cannot reach the grains of the base material, so a damping effect will exist. Apart from that, the geotextile, even when it is not geometrically tight, will provide an obstacle for transporting particles.

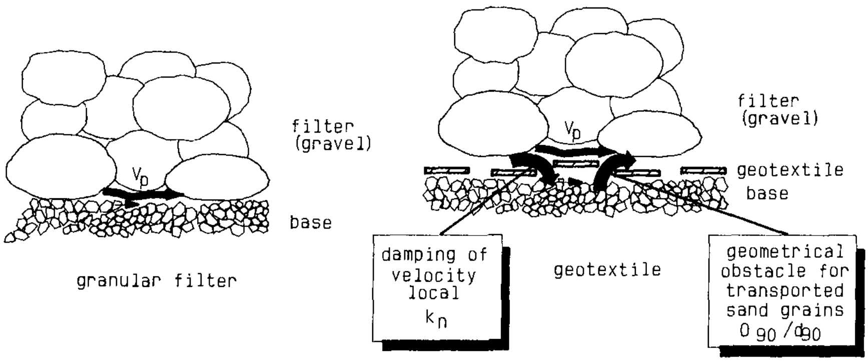


Figure 10 Principle of erosion at interface

Relevant parameters for the stability of particles underneath a geotextile might be;

- the characteristic opening size O_{90} of the geotextile.
- the permeability k_g of the geotextile, as this determines whether turbulences in the pore holes in the filter can penetrate to the base material surface, and

- the thickness T_g of the geotextile, which is related to this damping and has a geometric effect too.

The water partly flows just along the geotextile, and gives less shear on the underlaying particles. As we assume that the pore water velocity gives the driving force for particle transport, the presence of a geotextile will reduce this pore water velocity. This reduction is a function of the opening size distribution, the permeability, and the thickness of the geotextile, see Fig. 10.

In order to derive a unified theory, the earlier proposed formula for a horizontal bed (without the effect of perpendicular flow) was extended using dimension analysis. An extended formula was derived, see Klein Breteler (1990), where three dimensionless coefficients where introduced in order to quantify these results;

$$q_{cr}=\left(C_1\left(\frac{T_g}{d_{90}}\right)^{C_2}\left(\frac{d_{90}}{O_{90}}\right)^{C_3}\left(\frac{w}{k_g}\right)^{C_4/m}+\frac{n}{\kappa}\right)\sqrt{\Psi_s \Delta g d_{50}} \quad (12)$$

Where C_i are positive coefficients

The geotextile/soil geometry is characterized by the dimensionless thickness of the geotextile, T_g/d_{90}. The critical filter velocity q_{cr} increases with decreasing O_{90}/d_{90} ratio, or with increasing T_g/d_{90}, leading to T_g/d_{90} as a parameter. The geometric effect is characterized by the ratio d_{90}/O_{90}. The ratio w/k_g, where w is the fall velocity of a base particle, is characteristic for flow damping at the threshold of sand motion.

The formula was arranged in such a way, that all parameters will be zero, if no geotextile is present. The formula reduces than to that for granular material (5).

Geotextile interface experiments

Formula (12) was calibrated with experimental testing, quantifying the coefficients C_i. Tests were performed with mesh netting, tape fabric geotextile, and the thicker mat alike geotextile. In these tests base materials ranging from 100 μm up to 200 μm were used under the condition of horizontal flow through a rock layer on a horizontal geotextile.

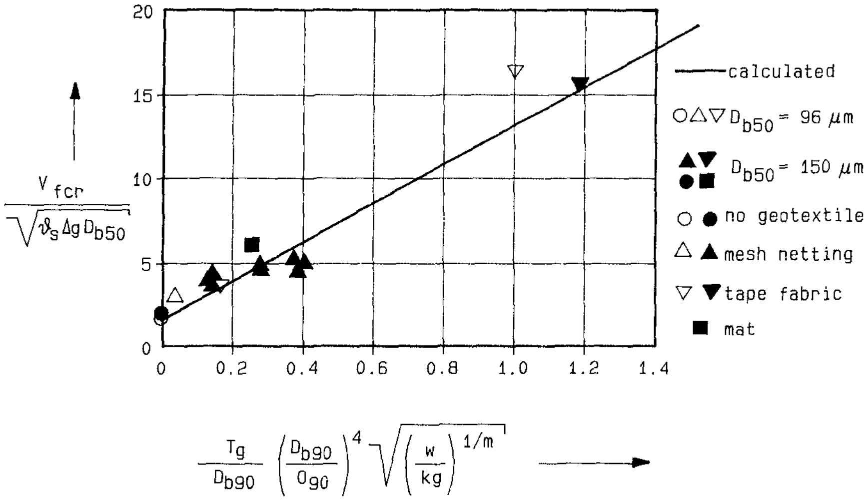


Figure 11 Test results and empirical formula

Since the number of tests performed is relatively small, it is not possible to determine more definite values for the coefficients C_i. Using trial and error methods, acceptable values for C_i where established; $C_1 = 12$, $C_2 = 1$, $C_3 = 4$ and $C_4 = 1/2$, with the test results. This yielded the following formula;

$$q_{cr} = \left(12 \left(\frac{T_g}{d_{90}}\right) \left(\frac{d_{90}}{O_{90}}\right)^4 \left(\frac{w}{k_g}\right)^{1/2m} + \frac{n}{\kappa}\right) \sqrt{\psi_s \Delta g d_{50}} \qquad (13)$$

In Fig. 11 the test results are gathered in such a way that one can judge the overall behavior of the derived formula. The experiments where described in more detail by Klein Breteler (1990).

Concluding Remarks

The derived formulae give a large extension to the range of applicability for filter materials, because the ratio of filter material and base material can be extended considerably.

The extension of the derived formulae to include the presence and influence of a geotextile filter makes them more generally applicable. The extended formula can, for example be applied to the design of bottom protections. As the feasible extension to include the influence of the inclination, such as included in the formula for granular filters only, has not been been verified yet.

For other types of structures such as breakwaters, with large elements at the surface structure the use of the criteria should be restricted to that part of the structure for which the interface is not in the direct vicinity of the surface of the structure. Since in the large "pores" between these elements, the dynamic effects of the water flow are not negligible.

A limiting factor to the applicability is that hydraulic loading conditions have to be known, or need to be established, thus for the further introduction of the derived filter criteria a parallel development of design tools to predict the hydraulic loading, in various constructions, is needed.

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