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DESIGN CRITERIA FOR GEOTEXTILES BEYOND THE SANDTIGHTNESS REQUIREMENT

CRITERES DE DESSIN POUR DES GEOTEXTILES EN PLUS DES EXIGENCES DE RETENTION DU SOL

ENTWURFSREGELN FÜR GEOTEXTILIEN JENSEITS DES BEREICHS DER SANDDICHTIGKEIT

Conservative design criteria for geotextiles applied for erosion control are based on the requirements for sandtightness. When these are satisfied the erosion of the base material is prevented even under extremely high hydraulic loads. For very fine base materials, however, it is difficult to meet these requirements. The present design criteria are generally considered to be too conservative and the present investigation forms the first stage in the establishment of less stringent design rules, taking into account the influence of hydraulic loads.
Model tests have been performed to determine the critical filter velocity in the granular material on top of the geotextile at which the base material under the geotextile begins to erode. The model measurements are used to test and verify the theory presented.

Konservative Entwurfsregeln für Geotextilien, die zur Verhinderung der Erosion des Untergrundes angewendet werden, beruhen auf der Forderung der Sanddichtigkeit. Wenn dies Forderung erfüllt ist, kann die Erosion des Basismaterials auch unter extrem hohen hydraulischen Beanspruchungen verhindert werden. Für feinkörnigen Untergrung ist es jedoch schwierig, diese Forderung zu erfüllen. Die derzeitigen Entwurfskriterien werden allgemein als zu konservativ angesehen. Die vorliegende Untersuchung stellt den ersten Schritt in der Formulierung weniger strender Entwurfsregeln dar, die den Einfluß hydraulischer Belastungen berücksichtigen. Es wurden Modellversuche zur Bestimmung der kritischen Filtergeschwindigkeit im körnigen Material oberhalb des Geotextils durchgeführt, bei denen das Basismaterial unterhalb des Geotextils erodiert zu werden begann. Die Messungen am Modell werden dazu verwendet, die vorgelegte Theorie zu überprüfen und zu erhärten.

1. INTRODUCTION

Geotextiles are frequently applied in civil engineering. The most important application areas are bank and bed protection, and foundations. The main purpose of the fabric in these constructions is to prevent the fine base material being washed out. Geometrical sand tight geotextiles are used in practice; this means, that sandtightness is reached because the sand particles can not pass the apertures in the geotextile purely based on their dimensions. This kind of sandtightness is called geometrical sandtightness and is independent on the magnitude of the hydraulic load.
The present conservative design rules based on this principle originate from the criterion that erosion of the base material through the geotextile can not be allowed even under extreme (unrealistic) hydraulic loading conditions. For the application of these rules it is not necessary to have proper knowledge of the hydraulic loads in the structure, which inevitably leads to overdimensioning. Design rules based on the principle of sandtightness are determined by various authors (see (1) and (3)). Those for woven fabrics on non-cohesive soils according to Ogink (1) are presented as a short review:

- Static load conditions : $O_{90}/d_{90} < 1$
- Dynamic load conditions : $O_{98}/d_{15} < 1$

The characteristic aperture size, O_{90}, is defined as follows:
O_{90} is equivalent to the average sand diameter of the sand fraction, 90% of which remains on the geotextile after a sieve test under defined conditions.
In practice, however, there are situations in which it is very difficult to meet the requirements of this conservative design rule. As an example can be mentioned:

- geotextiles on a subsoil of very fine material (very fine sand, silt, etc.)
- geotextiles which forms part of a "block mattress". During construction the total weight of the mattress hangs on the geotextile. For this function, if no expensive measures are taken, a network of strong and thick threads is required with a relatively open structure wich is contradictory to the dense structure required for sandtightness.

To widen the areas of application of geotextiles an in ship-induced water level fluctuations. Strong cyclic flows are caused by wind-induced wave attack and occur, vestigation has been initiated to determine a sandtightness criterion related to the hydraulic load which occurs in practice.
The hydraulic loads to be resisted can be of various kinds. Pure flow attack occurs, for example, in irrigation canals as a result of the discharge. The hydraulic load in a navigation channel results mainly from the for example, in the underlying layers of sea dikes or in the foundations of closure works.
The various flow situations which can occur can be schematized as follows:

- steady flow parallel to the plane of the geotextile
- cyclic flow parallel to the plane of the geotextile

- steady flow perpendicular to the geotextile
- cyclic flow perpendicular to the geotextile

Of course, combinations of all four situations may occur. The situation of steady flow parallel to the plane of the geotextile has been studied first and is treated in the following chapters. As is shown the transport mechanism at the interface between the base material and geotextile is comparable with that in granular filters. Many tests have already been carried out on granular filters (2) to find a relationship between the hydraulic load and the initiation of movement of the base material. The purpose of these tests was to reduce the total thickness of a granular filter and consequently the costs. The theoretical background derived for the granular filters has been tested and verified by measurements in the present series of tests.

The presentation of the theoretical background is seen as a first start to the derivation of design rules for geotextiles.

The theory presented and tests discussed serve to clarify, that the applicability of a geotextile increases if the hydraulic load is taken into account.

2. THEORETICAL BACKGROUND

In practice geotextiles are often applied at the interface between coarse granular material (filter) and much finer base material, the purpose being to protect the base material against erosion. In the present investigation attention is concentrated on a situation in which a non-cohesive sand with a steep sieve curve is protected by a geotextile with large O_{90}/d_{90} ratio.

Since the mesh width of the geotextile is chosen to be much larger than the sand grains the geotextile will only be able to prevent erosion up to a critical water velocity in the filter. The hydraulic gradient, i, in the filter causes a pore velocity, v_p, between the filter grains. Due to the relatively low geotextile permeability compared to the filter permeability the flow activated in the geotextile will be slower than that in the filter.

At a critical ppore velocity, v_{pcr}, the sand grains under the geotextile will start to move. Pore velocities higher than v_{pcr} will eventually lead to unacceptable erosion.

The failure mechanism of a geotextile with large O_{90}/d_{90} ratio, described above, is comparable to the failure mechanism of a granular filter with pores much larger than the sand grains being protected.

A well designed granular filter is able to prevent scour by means of one or both of the following mechanisms:

- The water motion in the granular filter is slower than that in the medium above it, which means that the filter decreases the hydraulic load.
- The pores between the grains of the filter are sufficiently small to prevent the passage of sand grains.

Conventional design criteria for granular filters rely completely on the second mechanism. Model tests on granular filters have been carried out which demonstrated that the conventional design criteria are too stringent in many cases leading to overdesign. A filter which is not geometrically sand tight can, in fact, perform its duty up to a certain filter velocity (and accompanying pressure gradient in the filter).

The basic idea behind the new design criteria is that there is a similarity between the flow in open channels and the flow in the pores of a filter at the threshold of sediment transport. It is assumed that, given a granular filter with pores much wider than the grains in the base layer, the bed shear over the sand interface is equal to the shear in a channel with the same bed material (at the threshold of sand transport).

The threshold of sediment motion in open channels has been investigated very thoroughly by various scientists in the past. Shields (4) for example, found the following (empirical) formula for the critical shear:

$$\tau_{cr} = \Psi_s \Delta\, g\, d_{50}\, \rho \tag{1}$$

τ_{cr} = critical shear (N/m^2)
Ψ_s = Shields parameter
Δ = relative density of sand grains, = $(\rho_s-\rho)/\rho$
g = gravity (m/s^2)
d_{50} = grain size corresponding to 50% by weight of finer particles (m)
ρ = mass density of water (kg/m^3)
ρ_s = mass density of sand (kg/m^3)

The coefficient Ψ_s was determined empirically by Shields. It depends among other things on the grain size and specific weight of the base material.

ψ_s, valid for a sand bottom with submerged density Δ = 1.65 and viscosity $\nu = 10^{-6}$ m^2/s is plotted against d_{50} in Figure 1:

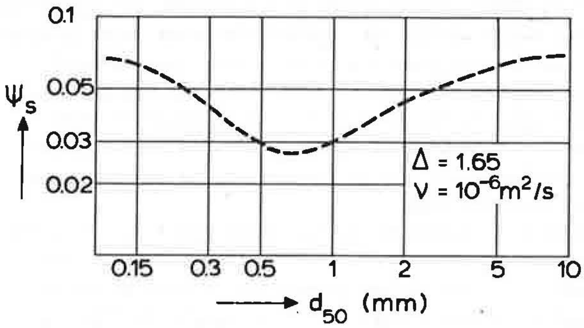

Figure 1. Shields parameter related to d_{50}

By introducing the shear velocity, $v_* = \sqrt{\tau/\rho}$, the Equation (1) can be rearanged as follows:

$$v_{*cr} = \sqrt{\Psi_s \Delta\, g\, d_{50}} \tag{2}$$

Based on this equation an equation for the critical filter velocity in a granular filter can be derived, assuming a simple v_*/v_p ratio:

$$v_{fcr} = n \cdot v_{pcr} = \frac{n}{\kappa} \sqrt{\Psi_s \Delta g d_{50}} \qquad (3)$$

v_{fcr} = critical filter velocity (m/s)
n = porosity of filter
κ = coefficient = v_*/v_p
v_{pcr} = critical pore velocity (m/s)

The magnitude of κ, determined empirically, is:

$\kappa = \frac{v_*}{v_p} = 0.8/Re^{0.2}$ for $0.1 < d_{50} < 0.3$ mm

$\kappa = 0.2$ for $0.5 < d_{50} < 1$ mm

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

$$Re = \frac{v_f \cdot d_{f50}}{\nu}$$

Re = Reynolds number
d_{f50} = d_{50} of filter material (m)
ν = water viscosity (m^2/s)

Substitution of the permeability law for granular material according to Cohen de Lara (5) into the formula derived for v_{fcr}, (3), leads to the determination of the critical hydraulic gradient, i_{cr}:

$i_{cr} = \frac{3.5 \Psi_s \Delta d_{50}}{d_{f15} \cdot n^3}$ for $0.1 < d_{50} < 1$ mm

$i_{cr} = \frac{\Psi_s \Delta d_{50}}{d_{f15} \cdot n^3}$ for $d_{50} > 2$ mm

Calculated values of i_{cr} are presented in Figure 2 together with the results of model tests. The close agreement between measured and calculated values is clearly seen.

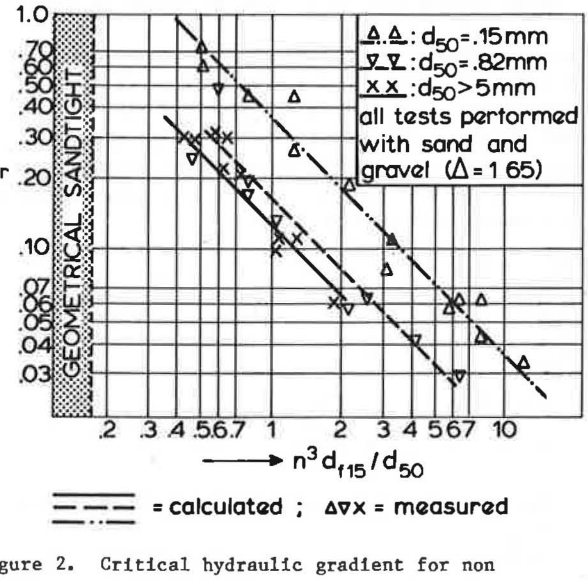

Figure 2. Critical hydraulic gradient for non geometrical sandtight granular filters

The same basic approach has been followed for a geotextile with a large O_{90}/d_{90} ratio. It is assumed that the water velocity is decreased considerably by the geotextile before the flow attacks the sand surface under the geotextile. The threshold of sediment transport will therefore be reached at relatively higher filter velocities compared to the situation without a geotextile. The introduction of a coefficient, κ_g, has been considered which contains the v_{pcr}/v_{*cr} ratio in the same way as the coefficient used for granular filters (see Equation (3)) and which takes into account the influence of the geotextile on the local water velocity at the base layer surface.

The coefficient κ_g depends probably on the following geotextile and base material characteristics:

$$\kappa_g = \frac{v_{*cr}}{v_{pcr}} = f(k_n, T_g, O_{90}, d_{90})$$

where
f(...) = function of...
k_n = normal geotextile permeability (m/s)
T_g = thickness of geotextile (m)

Model investigations have been performed to determine the coefficient κ_g.

3. MODEL SET-UP AND TEST PROGRAM

Model tests have been performed on various geotextiles which focussed on measuring the critical hydraulic gradients and filter velocities. The critical hydraulic gradient, i_{cr}, is the gradient at which the base material motion begins. It is measured in the filter layer overlying of the geotextile. The critical filter velocity is the velocity in the filter layer at which the base material begins to move.
The tests performed can be considered as the first step towards obtaining a full set of design rules, applicable to all geotextiles in all circumstances. For practical reasons a non-cohesive base material was used with a grain size d_{50} = 0.15 mm and steep sieve curve (C_u = d_{60}/d_{10} = 1.5; d_{90} = 0.22 mm). The test rig is shown in the figure 3.

The choice of the filter material placed on top of the geotextile was based on the consideration that the influence of the geotextile on the critical hydraulic gradient would be very distinct. The filter material was a round gravel with grain diameters in the range 17 mm to 35 mm (d_{f50} = 24 mm; C_u = 1.2). From earlier experiments (2) it was known that this grain size range would guarantee a negligibly low critical hydraulic gradient for the situation without geotextile. Test no. 4 was performed with gravel with d_{f50} = 14.3 mm (C_u = 1.35) to confirm that the critical filter velocity does not depend on the grain size of the filter material.

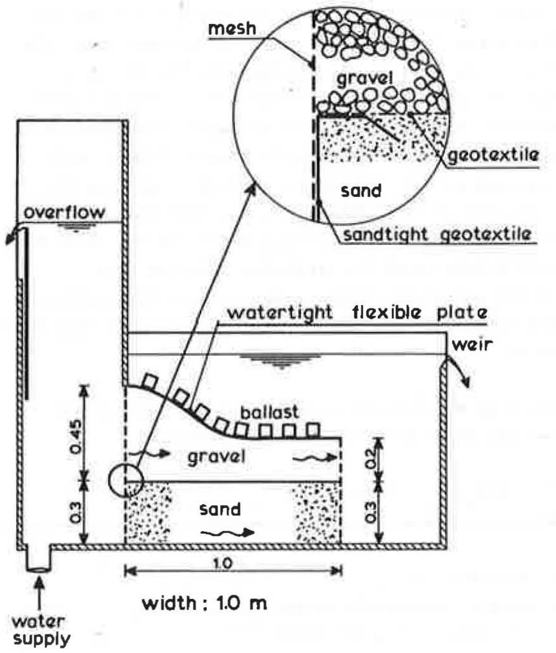

Figure 3: Test rig (schematized).
measures in m.

The test section was designed in such a way as to minimise boundary effects (see Figure 1). The filter velocity in the first half of the test section was considerably lower than in the second half of the section due to the decreasing filter layer thickness in the flow direction. This model set-up ensured that incipient sand motion took place away from the model boundaries. The suitability of this set-up was investigated by using (unpainted) yellow sand in the first 10 cm of the test section and (unpainted) dark grey sand in the remainder of the section. These sands had exactly similar sieve curves. It was observed that the yellow sand eroded only when flow conditions were far beyond the threshold of sediment motion. The vertical hydraulic gradients in the base layer in the second half of the test section were very low and consequently did not influence the critical filter velocity. The top and bottom of the test section were watertight. 700 kg of ballast were mounted on top of the test section.
A geometrically sandtight geotextile at the upstream and downstream boundaries of the test section (see Figure 2) ensured an undisturbed inflow in the base layer and outflow at the downstream end.
During each test the hydraulic gradient was increased step by step until considerable erosion took place underneath the geotextile. Each step lasted half an hour after which the eroded sand was sucked away from the bottom of the downstream end of the test rig. The test consisted of about ten steps and was completed without interruption.

The sediment transport was plotted against the filter velocity and against the hydraulic gradient. As a criterion for threshold of sediment transport a transport rate of $150 \cdot 10^{-6}$ kg/s/m was chosen. This criterion corresponds roughly with the criterion used by Shields.

The back wall of the test rig was fitted with 16 piezometer tubes which supplied the information required about the flow in the filter and base material. The discharge through the filter was measured by using a Rehbock weir.
The test program and typical test results are listed in table 1.

Table 1, Test results

Test nr.	geo-textile	Type	O_{90}	O_{98}	T_g	k_n	k	v_{fcr}	i_{cr}	$\frac{O_{90}}{d_{90}}$
			mm	mm	mm	mm/s	m/s	mm/s	-	-
1	N66336	A	0.37	0.39	0.45	6.8	0.11	63	0.35	1.68
2	N66339	A	0.40	0.42	0.72	6.0	0.11	65	0.35	1.82
3	N66373	A	0.52	0.55	0.68	5.0	0.11	52	0.23	2.36
4	N66373	A	0.52	0.55	0.68	5.0	0.085	49	0.32	2.36
5	R425	B	0.44	0.53	1.2	1.6	0.11	140*	1.6*	2.00
6	8147	C	0.65	0.74	2.0	2.1	0.11	78	0.48	2.95
7	-	-	-	-	0	-	0.11	21	0.05	-

N = Nicolon
R = Robusta
A = Mesh-netting
B = Tape fabric
C = Mat
\- = No geotextile
k_n = filter velocity through geotextile without sand or gravel at $i = h/T_g = 1$.
k = permeability of gravel $(v/\sqrt{i})$ (m/s)
h = potential head over geotextile (m)
* The maximum hydraulic gradient which could be created was insufficient to cause the threshold transport rate of $150 \cdot 10^{-6}$ kg/s/m during Test 5. The given values have been found by extrapolation.

The normal permeability of the geotextile was measured using a perspex pipe (cross-sectional area 19.62 cm^2). The geotextile was placed in the pipe and the discharge through the pipe, v, and the potential head across the geotextile, h, were measured (provisioned Dutch Standard (1)). The results yield the following permeability formulas for the fabrics tested:

N66336: $v = 6.8 \cdot 10^{-3} \cdot i$

N66339: $v = 6.0 \cdot 10^{-3} \cdot i$

N66373: $v = 5.0 \cdot 10^{-3} \cdot i$

R 425 : $v = 1.6 \cdot 10^{-3} \cdot i^{0.70}$

8147 : $v = 2.1 \cdot 10^{-3} \cdot i^{0.73}$

v = discharge velocity through the geotextile (m/s)
i = h/T_g
h = potential head across geotextile (m)
T_g = geotextile thickness (m)

From the test results it is clear that the critical filter velocity, v_{fcr}, is not dependent on the grain size of the filter (see Tests 3 and 4 in Table 1). This is in agreement with the theory presented in Chapter 2. Consequently it is obvious to focus the attention on the critical filter velocity. The critical hydraulic gradient can be calculated from the critical filter velocity by using a permeability law for granular material.

Since the flow in the filter can be characterized by the filter velocity (concerning the threshold of sand motion) it is clear that there is a resemblance between the threshold for flow in a granular filter and that in an open channel. As has been described extensively the incipient sand motion takes place when a certain critical shear velocity at the interface (bottom) is exceeded. From Shields we know the magnitude of the critical shear velocity:

$$v_{*cr} = \sqrt{\Psi_s \, \Delta \, g \, d_{50}} \tag{4}$$

Presently it is stated that, at the threshold of sand transport, the shear velocity at the sand surface, which is protected by a geotextile, equals v_{*cr} as given by Equation (4).

Based en Equation (3) (the derivation of the critical filter velocity, v_{fcr}, for granular filters) use has been made of the assumption, that there is a relationship between v_{fcr} and v_{*cr} as follows:

$$v_{fcr} = n \, v_{pcr} = \frac{n}{\kappa_g} v_{*cr} = \frac{n}{\kappa_g} \sqrt{\Psi_s \, \Delta \, g \, d_{50}} \tag{5}$$

The value of v_{*cr} for the base material used in the present investigation has been calculated as follows:

$$\Psi_s = 0.063 \text{ (Shields)}$$

$$v_{*cr} = \sqrt{\Psi_s \, \Delta \, g \, d_{50}} \tag{6}$$

$$= \sqrt{0.063 \cdot 1.65 \cdot 9.8 \cdot 0.15 \cdot 10^{-3}}$$

$$= 0.012 \text{ m/s}$$

The coefficient κ_g in Equation (5) equals the ratio v_{*cr}/v_{pcr} and includes the damping influence of the geotextile. The coefficient κ in Equation (3) can be regarded as a boundary condition for κ_g in the situation that there is no geotextile:
$k_n = \infty$, $T_g = 0$ and $O_{90}/d_{90} = \infty$.

The influence of the geotextile on the critical filter velocity is basically characterized by:
- the flow damping of the geotextile
- the geotextile/soil geometry

Both of these basic characteristics are discussed below:
- The normal hydraulic gradient, $i = h/T_g$, in the geotextile at a discharge velocity v, which equals v_{*cr}, has been suggested as a dimensionless characteristic parameter for the geotextile flow damping at the threshold of sediment transport. The dimensionless flow damping parameter can be derived by using a general permeability law for a geotextile ($v = k_n \cdot i^m$):

$$\left(\frac{v_{*cr}}{k_n}\right)^{1/m} \tag{7}$$

  It is obvious that the v_{fcr}/v_{*cr} ratio increases with increasing flow damping.
- The geotextile/soil geometry is characterized by the O_{90}/d_{90} ratio. Obviously the v_{fcr}/v_{*cr} ratio decreases with increasing O_{90}/d_{90} ratio.

The above theory leads to the following formula for critical filter velocity:

$$v_{fcr} = \left(A_1 \left(\frac{v_{*cr}}{k_n}\right)^{A_2/m} \cdot \left(\frac{O_{90}}{d_{90}}\right)^{-A_3} + \frac{n}{\kappa}\right) \sqrt{\Psi_s \, \Delta \, g \, d_{50}} \tag{8}$$

in which A_1, A_2 and A_3 are positive coefficients. Equation (8) becomes similar to equation (3) in the case of no geotextile. This can be seen by substituting $k_n = \infty$ and $O_{90} = \infty$ into eq. (8).

Since the number of tests performed in the present investigation is only small, it is not possible to give accurate values for coefficients A_1, A_2 and A_3, which would lead to a design formula for geotextiles. On the other hand it is possible to verify the theory which has lead to the choice of the given basic parameters $(v_{*cr}/k_n)^{1/m}$ and O_{90}/d_{90}.
The parameters are presented in Table 2. The values of A_1, A_2 and A_3 have been determined by trial and error, various values being considered. With $A_1 = 4$, $A_2 = \frac{1}{2}$ and $A_3 = 1$ a formula was found which agrees very well with the results of the tests.

Test No.	Geo-textile	v_{fcr}	$\frac{v_{fcr}}{v_{*cr}}$	$\left(\frac{v_{*cr}}{k_n}\right)^{1/m}$	O_{90}/d_{90}	$\sqrt{\left(\frac{v_{*cr}}{k_n}\right)^{1/m}} \cdot \frac{d_{90}}{O_{90}}$
		mm/s	-	-	-	-
1	N66336	63	5.1	1.8	1.7	0.80
2	N66339	65	5.2	2.0	1.8	0.78
3	N66373	52	4.2	2.4	2.4	0.66
4	N66373	49	4.0	2.4	2.4	0.66
5	R425	140	11.3	17.8	2.0	2.1
6	8147	78	6.3	10.9	3.0	1.1
7	-	21	1.7	0	∞	0

Table 2. The critical filter velocity, v_{fcr}, related to geotextile and subsoil characteristics.

The values given in Table 2 have been used to plot Figure 4.

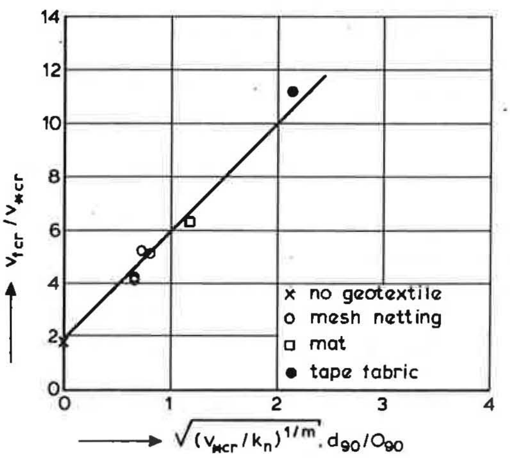

Figure 4. The critical filter velocity, v_{fcr}, as a function of geotextile and subsoil characteristics.

From Figure 4 it is clear that there is good agreement between the straight line, which is drawn on the basis of Equation (8), and the measurements. It can be concluded, therefore, that the given basic parameters, $(v_{*cr}/k_n)^{1/m}$ (flow damping) and O_{90}/d_{90} (geometry), include all of the geotextile characteristics relevant to the incipient sand motion phenomenon.
However, it is stressed again that the values of A_1, A_2 and A_3 presented are first approximations based on a small number of tests. Further basic research must be undertaken to conform the approach used.

4. CONCLUSIONS

The following conclusions can be drawn, even though only a number of tests have been undertaken on woven geotextiles:

- Geotextiles with large O_{90}/d_{90} ratio can be used for erosion control provided that the hydraulic load is limited.
- The transport mechanism at the interface between the geotextile and the base material appears to be comparable to the transport mechanism in granular filters.
- The critical filter velocity at the threshold of sediment motion of the base material seems to be independent of the grain size of the granular material laid on top of the geotextile.
- The critical filter velocity (v_{fcr}) depends on the critical shear velocity, v_{*cr}, of the base material and the geotextile properties. A first approximation of this relationship has been presented.

It is very clear that the theory presented must be tested and verified with a more extensive series of measurements. Only then will it be possible to give a proper set of design rules for geotextiles.

The tests to be carried out in the near future should cover:

- different geotextiles, especially non-wovens.
- different base material.
- different flow conditions.

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