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USE OF GEOTEXTILES IN HYDRAULIC CONSTRUCTIONS IN THE DESIGN OF REVETMENTS
BEMESSUNG VON DECKWERKEN UNTER BERÜCKSICHTIGUNG VON GEOTEXTILIEN
DIMENSIONNEMENT DE REVETEMENTS DE RIVE EN CONSIDERATION DE GEOTEXTILES

1. Introduction

A natural bank of an area of water adjusts itself as a result of wave motions to a very flat angle of slope which fulfills the limit of equilibrium. Vertical slopes in a clay or silty soil can only remain stable as long as the cohesion is in operation. In general it can be supposed that the least of soils remain cohesive over a long period of time in the transition area between water and bank surface. This is the reason for the collapsing of such banks in blocks and levelling off after rupture in the water exchange area to an almost horizontal beach. This can be observed in all natural areas of water.
In time flowing waters change their bed depending on their own efficacy of erosion.It has for a long time been necessary to protect the cultivated areas from the water erosion by installing bank protections.
The banks of navigable waterways are subject to an increased load through the flow of displacement caused by the passing ships. Apart from the velocity of the back flow there is the relatively rapid lowering of the water level which is one of the main special loads which endanger those parts of the bank area lying under water.It is obvious that banks in the direct influence area of such water motions caused by navigation have to be pro-protected by constructions which have enough stability. This paper puts forward some of the ideas which are seen to be relevant in the construction of embankment protections and have a direct relation to the use of geotextiles in bank protection.

2. Construction methods of bank protection

The development of the construction methods of bank protection can simply be sketched out as follows:
Formerly an embankment which was threatened by erosion caused by wave movements was usually stabilised by rubble mound. The relevant idea here was that the stones themselves resulting out of their particle size and weight would be able to withstand erosion. Yet practical experience with such bank protection was characterised by a considerable amount of maintenance due to the fact that the stones had to be continually replaced. This maintenance work decreased only when the protective embankments reached often a thickness of more than 2 metres. The reason for such an intensive maintenance was usually due to the lack of filter stability between the subsoil and the riprap, either due to faulty planning or inexpert construction of the bank protection.By using graded grain particle filters under the riprap less maintenance was needed. This was an important factor to be contemplated by the construction.
Problems always recurred where the graded filters were built in under water. Often the disintegration of the built in filter was overseen. This disintegration however had serious consequences on the stability of the upper protective layers because the filter effect between bank protection and subsoil was lost.This problem could only be avoided by building in uniform-sized graded grain filter in several layers. This lead to high production costs.
The filter problem in the building of bank protection seemed to be solved with the development of geotextiles. The graded filter was replaced by geotextile filters and the revetment was built out of riprap similar to those of the particle grain filter. Because of the omission of the particle filter layers the bank protection became lighter but therefore they were not stable enough to resist the even increasing demands made on them by naivgation. This manifested itself mainly in the increase of deformation of the bank protection. The consequence of which would lead to the complete destruction of the revetment through gliding of the loose stone rubble on the geotextile plane. There then followed a period of the so-called composite or bonded construction. On the one hand an adhesion could be attained through subsequent areal sealing of the loose stone riprap. The partly sealed rubble bank protection was developed - experts adopted the name cling riprap - in which cement or bituminous bonding material could establish an areal adhesion within the protective layers without losing to any great extent the water permeability of the originally loose stone rubble. On the other hand different kinds of precast stone riprap became common in which precast concrete stones were combined through suitable constructive details to form an areal entity. The stability of the uppermost layer of the bank protection was thus ensured. Nevertheless none of these constructions were wholly stable because the subsoil under the geotextiles was subject to deformation and often lead to the occurrence of small cavities under the bonded stone riprap. These circumstances finally caused the collapse of the riprap structure. It was attempted to counter this phenomenon by the use of a novel type of double or multilayered geotextile in which the layer underneath of the structure was supposed to have a roughing, later a prefiltering function.
It is indeed now possible to build revetments which have a durability of many years. This capability is based on the results of practical empirical trials in different riprap constructions which have been undertaken independent from each other over a number of years in different waterways. It must be warned against a general transference of this partly good experience on to other waterways with different navigational loads and subsoil conditions. Failures cannot be excluded where the harmonious combination of subsoil, riprap and other marginal conditions are not present. It seems therefore more than necessary to search for a theoretical proof and verification of the experience made in the construction of bank protection.

3. Stability concept

"Stability of riprap construction" means here the capability of the riprap to absorb the hydrodynamic forces resulting from the water movements without deformation of the construction itself and/or of the subsoil. The following requirements should be fulfilled:

- The whole stability of the slope of which the uppermost layer the riprap represents has to be secured. The proofs necessary for this are wellknown from standards and specifications.

- The stability of the riprap elements has to be ascertained within the uppermost protective layer. There are enough dimensioning data from model-scale tests (e.g. Hudson-Criteria). Also there are further constructive methods (e.g. adhesive design) which may lead to an emelioration of the factor of safety.

- The stability of the slope's subsoil in a shallow depth underneath the riprap has to be observed, i.e. the stress condition of these subsoil areas near the slope surface must be kept in a condition, which remains far enough apart from failure condition (Mohr-Coulomb stipulation).

The first of these abovementioned stipulations can be fulfilled through pure soil mechanical proofs according to the standard i.e. DIN 4084. The second condition can be ensured respectively by suitable model tests, existing experience or through special constructive methods. The third stipulation means that the increase of shearing stress caused by the rapid draw-down of the waterlevel makes it necessary to put up a counterweight against the rising force caused by the groundwater discharging out of the embankment, so that the shear strength will not be exceeded and even a certain stability exists to counteract the condition of ultimate equilibrium as defined by the Fellenius rule. This evidence follows definite geotechnicle rules which have to be stipulated. The last of the abovementioned design principles applied to the method of design was originally conceived and used successfully by Köhler,(1980),(1), for practical investigations to establish several constructions of riprap on navigable waterways. The most important outset dimension is the applicable waterpressure in the subsoil of the slope caused by the rapid change in waterlevel in front of the slope as a result of the shipping passing by. Investigations in more detail are required.

4. The flow condition in the subsoil

Seepage flow as a result of the change in the water level has been investigated into by Schnitter and Zeller (1957),(7). Assuming the water level in the slope under the riprap is balanced then it is possible using these investigations to ascertain the position of the slope spring after a rapid draw-down of the water level as long as the permeability of the porous riprap medium itself allows a lowering without delay. The term 'slope spring' is used here to define that area of the slope which lies under the emerging point B of the seepage flow. In the area of the slope spring the flow lines run almost parallel to the slope and produce the greatest shear stress caused by flow (see Fig. 1).
Underneath the water level the surface of the slope is an equipotential surface. After a rapid lowering of the water level in front of the slope the water level in the slope subsoil can only follow in a delayed form depending on the permeability k of the slope's soil. An unsteady potential flow develops where the flow lines in the slope area under the lowered outside water level are directed perpendicular to the area of discharge, i.e. are directed

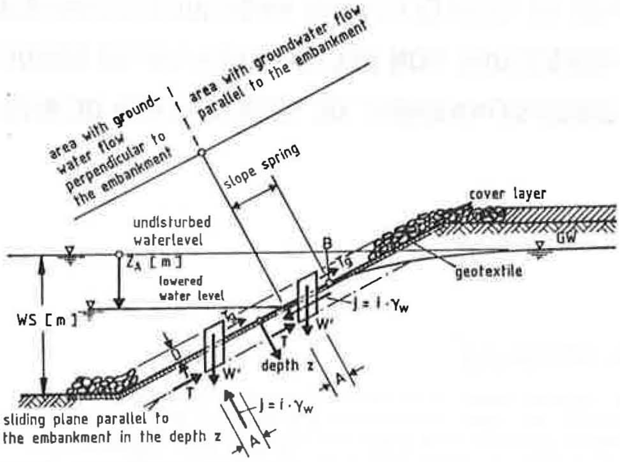


Fig. 1 : embankment elements

perpendicular to the slope inclination. The hydraulic gradient resulting out of the instationary potential pressure field reduces therewith the normal stress in a potential plane lying parallel to the slope surface in a certain depth z. The shear stress from the submerged unit weight of the soil remains unaffected from this and is not reduced. The shear stress in the subsoil increases and the safety factor against soil flow decreases. In the design of bank protection this must therefore be taken into special consideration. Through the acting of the hydraulic gradient directed perpendicular to the slope surface often there exists far less safety against soilflow on planes lying parallel to the slope as it is given in the generally known condition of instability caused by the flow lines parallel to the slope.
Whereas the parallel slope flow is purely a case of slope geometry it is much more difficult to specify the outward flow force. With an even steady lowering of the water level the resulting instationary hydraulic gradient may be determined through a one-dimensional consideration analogue to the consolidation theory. In reality two-dimensional movements of water occur in navigable waterways which lead to a three-dimensional pore water pressure response in the subsoil. The mathematical description for the distribution of pore water pressure in a slope does not as yet exist in general. Completed solutions for river beds are known for the two-dimensional system which were acquired in connection with the dam-project of the Osterschelde (Barden,1979,deGroot an Sellmeijer, 1979),(2),(3).The mathematical description of the process is made even more difficult by the fact that the compressability of the aerated pore water changes during the lowering of the water level.In the BAW (Köhler, 1980),(4), investigations were initiated which have lead to a near description of the pressure course in a one-dimensional view. In comparison with the lineal consolidation theory a series of most recent tests confirmed the original investigations which lead to the conclusions that the time taken for the balance of the pore water pressure after a rapid draw-down of the water level depends very much on the continuous change of the compressability of the areated pore water (Schneider,1985),(5). Supposing as from the first approximation that the rapid draw down caused by the passing of a ship may quite

correctly be taken as a one-dimensional occurrence then the state of unsteady flow can also be dealt with as a one-dimensional problem.In such a case the duration and place of the change in pore water pressure in the subsoil can be approximated by using the following function:

$$p(t,z) = z_A \cdot \gamma_W \, (1 - e^{(-\varepsilon(t)\cdot z)}) \qquad (1)$$

$$\text{mit} \quad (t) = K\cdot(c_v(t)\cdot t)^{-1/2}$$

In the equation (1) z is the depth perpendicular to the slope surface counted positive in the depth direction perpendicular to the slope's angle, t is the time after the rapid draw-down of the water level of the amount z_A. ε (t) is the change of pore water pressure over a certain length of time. c_v is the consolidation coefficient which fluctuates owing to the varying compressability of both of the partaking pore media water and air respectively. K is a constant of proportionality which could vary with the factor t.
The correlation of the practical results with these theoretical estimations was not very satisfactory - therefore the approximate estimation of the exponential function will be recoursed in the following:

$$\Delta p(t,z) = z_A \cdot \gamma_W \cdot a(t) \cdot e^{(-b(t)\ z)} \qquad (2)$$

Here a(t) and b(t) represent the functions which were gained out of the test results about regression analysis with the exponential function. Figures 2 and 3 represent the plot of the curves for these dimensions for a certain type of soil.

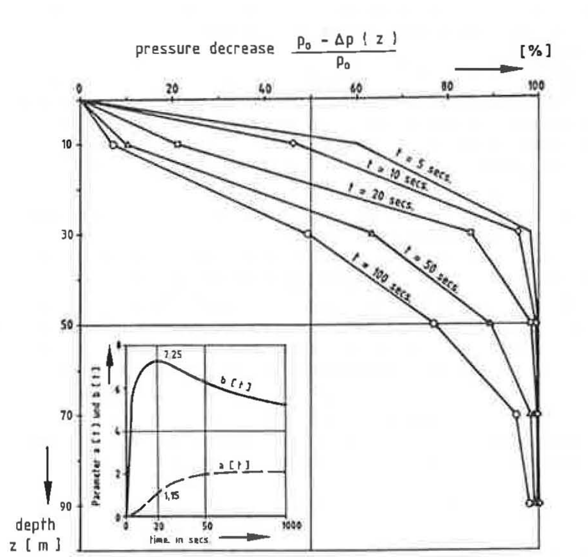


Fig. 2 : Pore water pressure decrease as a function of the depth z

Fig. 2 shows the results of a test carried out showing the delayed loss of pressure in a test cylinder with a diameter of 47 cm to simulate the rapid draw down of the water level of the amount of z_A = 85 cm within the time of t = 10 seconds. The material investigated consisted of light non-uniform medium sand with an uniformity coeffecient C_u = 5,9 and an effective grain diameter d_{10} = 0,116 mm.

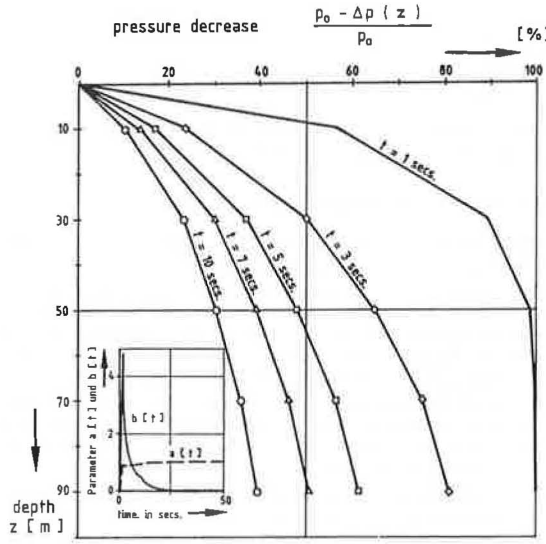


Fig. 3 : Pore water pressure decrease as a function of the depth z.

Fig. 3 shows the curve plot of an investigation made with very uniformly graded medium sand (C_u = 1,5 and d_{10} = 0.336 mm) where the draw-down value was z_A = 65 cm within a period of time t = 10 secs.).
By dividing equation (1) through γ_W and differentiating to the point of time t = constant after z, it follows that in the depth z the prevailing local hydraulic gradient i resulting from the rapid lowering of the water level to the observed period of time t:

$$i = \varepsilon \cdot z_A \cdot e^{(-\varepsilon\cdot z)} \qquad (3)$$

Relating the empirical presentation of the dependency of the time on the act of lowering the water of the equation (2) results that the local hydraulic gradient (4) is:

$$i = b(t)\cdot z_A \cdot a(t)\cdot e^{(-b(t)\cdot z)} \qquad (4)$$

Inserting for example from Fig.2 the values a and b gained during the test at the time of t = 20 seconds after the commencement of the draw-down of the water level in equation (4) it then follows that the local hydraulic gradient in the depth z = 0,1 m is for example i = 3,43. It is easy to see from this result that the stress on the grain skeleton of the subsoil can take on a considerable dangerous value.This is even more so on soils having much less permeability.
By stability analysis however the average of the hydraulic gradient should be used which is in operation respectively between the observed depth z and the embankment surface. The integration of the equation (3) leads to an extensible mathematical integral-exponential function, so that it seems sufficiently exact to show the average hydraulic gradient i_m as follows:

$$i_m = \frac{p(t,z)}{\gamma_W \cdot z} = \frac{z_A(1 - e^{(-\varepsilon(t)\cdot z)})}{z} \qquad (3a)$$

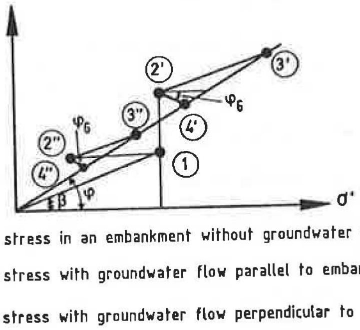
respectively

$$i_m = \frac{p(t,z)}{\gamma_W \cdot z} = \frac{z_A(1 - a(t)\, e^{(-b(t)\cdot z)})}{z} \qquad (4a)$$

The equation (4a) will be needed later for the calculation of the revetment stability.

5. Operation mode of geotextiles

It has been shown (Schulz 1984),(6), that in the treatment of the local stability of a dam slope in which way a geotextile reacts on the stress condition of a soil element under a bank protection. It is hereby assumed that for the condition of equlibrium the geotextile lies evenly and is capable of transferring the tensile force to the higher points of the embankment above the water level. It must be ensured that this be regarded in the design of bank protection.The geotextile is loaded by the weight of the riprap and therefore normal stress reacts on the soil element directly under the geotextile.
As a result of the roughness of the geotextile a shear stress is transferred to the soil as soon as the pore water pressure conditions causes local soil flow, in which small yet sufficiently separate soil elements start to move in the slope direction of the embankment. The supposition of an extra acting force as one of the holding forces against slope slide parallel to the embankment plane is only there acceptable where little deformation of the bank subsoil may be accepted without risk so that a holding force can be established to support the stability of the rip-rap. This is possible through the mobilisation of friction between geotextile and subsoil, in which the stress condition of an endangered soil element may be kept below the critical stress condition as well as with the aid of an even smaller surcharge of a light revetment as without the use of geotextiles.



(1) State of stress in an embankment without groundwater flow
(2') State of stress with groundwater flow parallel to embankment
(2") State of stress with groundwater flow perpendicular to embankment
(3'), (3") } State of stress after loading by cover layer without geotextile
(4'), (4") } State of stress after loading by cover layer with use of geotextile

Fig. 4: State of stress conditions

The following idea should be contemplated to show the mode of action of so-called supplementary layers as the lowest layer of a multilayered geotextile (special questions to inhomogenous and layered subsoil conditions are not as yet to be taken into consideration):
In the beginning the geotextile lies with the supplementary layer on the embankment. The surcharge from the covering layer effects to a greater or lesser degree an impression in the uppermost soil zone depending on the granulometric mixture, density or consistency of the underlying soil. If this condition cannot be directly attained through the effect of the static load of the riprap, then the covering layer under stress from the navigation is exposed to a continuous repeated water level change. The dynamic loads set free on the slope soil from the unsteady potential flow cause failure conditions in the subsoil.This happens in limited local areas where the load of the riprap has not yet full contact to the subsoil area. As a result of this abovementioned,especially in non-cohesive soil materials, soil redistribution arises, i.e. a deformation of the subsoil or local soil flow is initiated until the porosity offered by the additive layer is filled with sufficient soil material. Further particle movements are then impossible when a sufficiently dimensioned riprap load prevents further local failure conditions.In the sense of soil mechanics the establishing of such a state of equilibrium causes the supplementary soil filled layer to be viewed as a thin soil layer itself in which not only friction but also cohesion takes effect. This cohesive force may be taken into account in the question of stability as long as the material properties of the geotextile remain sufficient i.e durable. Prerequisite for the flow of soil material into the additional layer is that the effective pore size of the additionl layer is in correct proportion to the relevant grain diameter of the inflowing soil materials i.e. sufficient in size. In cohesive soils the process of inflowing of soil particles takes a longer time under unsteady loading conditions until the abovementioned equilibrium condition is reached.
In which degree this additional layer may actually be filled with soil particles depends doubtlessly on the chosen structure of this layer.The greatest local slope pressure directly on the interface between site soil and filter layer can appear as shown in investigations carried out by rapid draw-down tests (see Fig. 2). Therefore it is especially important to mobilise a shear strength in this transition layer.
This positive view of some of the practical uses of geotextile filters with additive layers may thus be explained as long as the abovementioned conditions are fulfilled but cannot be transferred in general because different modes of action between soil and filter layers are influenced by the inhomogeneity and grain size distribution of the subsoil especially when conditions of different soil layers could for instance cause impermissible reduction of a certain requested water permeability of the revetment layers, e.g. clogging effect.

6. Conditions of equilibrium in a soil element

Figure 1 shows a soil element in the area of a slope spring and one in the area underneath the lowered water level. Those loads acting on both soil elements assume the effective submerged weight W' and the force of flow S :

$$W' = A\cdot(z\cdot\gamma' + D\cdot\gamma_D) \qquad (5)$$

$$S = i\cdot\gamma_W\cdot A\cdot z \qquad (6)$$

For the soil element in the area of the slope spring there is the hydraulic gradient i given through the slope

geometry with the slope angle ß , in the area of the soil elements underneath the lowered outside water level the hydraulic gradient is given through the average value of the local gradient, which is to be calculated by the equations (3a) and (4a) respectively.
From a soilmechanical point of view the limit of equilibrium is ascertained and the shear parameter in the sliding planes lying parallel to the slope surface in which the failure or collapse of the embankment soil element is just reached, is required. In this view of limit equilibrium the mobilised friction force along the geotextile is taken into consideration.
It has the value:

$$T_g = A \cdot D \cdot \gamma_D' \cdot \cos\beta \cdot \tan \varphi_g' \qquad (7)$$

In this equation φ_g' describes the drained angle of internal friction between the subsoil and the geotextile. All other influencing factors can be seen in Fig. 1. The shear stress resulting from the submerged unit weight of the soil elements is:

$$T = W' \cdot \sin\beta \qquad (8)$$

The required resisting shear force req T which is necessary for the limit of equlibrium can be brought into relationship with the present shear parameter by the aid of the Fellenius-rule:

$$\text{req } T = W' \cdot \cos\beta \frac{\tan \varphi'}{FS} + \frac{A \cdot c'}{FS} \qquad (9)$$

This equation describes the drained internal angle of friction φ', c' is the drained cohesion. FS is the factor of safety according to the Fellenius rule, which sets the present shear strength against the required shear strength.
For the estimation of the condition of equilibrium two volume elements of the thickness 1 are shown which are sliding in a parallel plane to the slope surface in a perpendicular distance point of the depth z . For the soil element lying in the area of the slope spring the following equilibrium conditions are valid:

$$T + S = \text{req } T + T_g \qquad (10)$$

Because the assumed condition of limit equilibrium should lie under the actual limit condition, stability is introduced by mobilisation of the friction force of the geotextile so that the sum of the equations (5),(6) and (7, divided by FS),(8) (9) put in equation (10) gives:

$$A(z\gamma'+D\gamma_D'+\gamma_W z)\sin\beta = \frac{A}{FS}(D\gamma_D'\cos\beta\tan\varphi_g' + \ldots$$
$$\ldots(z\gamma'+D\gamma_D')\cos\beta\tan\varphi' + c') \qquad (11)$$

The solution can be expressed as follows:

$$FS = \frac{(z\cdot\gamma'+D\cdot\gamma_D')\cos\beta\tan\varphi'+D\cdot\gamma_D'\cos\beta\tan\varphi_g'+c'}{(z\cdot\gamma' + D\cdot\gamma_D' + \gamma_W\cdot z)\sin\beta} \qquad (12)$$

It should be considered that for the soil element underneath the lowered outside water level, the outward directed flow force reduces the normal force in the sliding plane parallel to the slope. The equation (9) has to be modified as follows:

$$W'\sin\beta = \text{req } T + \text{req } T_g \qquad (13)$$

The conditions of limit equilibrium are in this case:

$$\text{req } T = (W'\cos\beta - S)\frac{\tan\varphi'}{FS} + \frac{A\cdot c'}{FS} \qquad (14)$$

Through inserting the equations (5),(6),(7) and (13) in equation (14) the safety factor of stability can be attained:

$$FS = \frac{((z\gamma'+D\gamma_D')\cos\beta - i\gamma_W\cdot z)\tan\varphi'+D\gamma_D'\cos\beta\tan\varphi_g'+c'}{(z\cdot\gamma' + D\cdot\gamma_D')\sin\beta} \qquad (15)$$

Through a suitable choice of the thickness and the submerged unit weight of the riprap, in this case the value of the thickness D and the unit weight γ_D' have to be varied, the minimum safety factor, which is derived from the equations (12) and/or (15) has to be found by substituting different values of the depth z. The safety factor FS which is necessary can be determined according to the standards e.g. DIN 4084.

7. Example for practical use

The application of the formula under consideration is shown on the dimensioning of the thickness D of a bank protection on a light non-uniform medium sand (C_u=5,9), (see Fig. 2) for the embankment area underneath the lowered water level.
The following characteristic index describe the geometry of the slope and the properties of the subsoil.

Inclination of slope:	1:n = 1:3 (β = 18,43°)
Draw-down value:	z_A = 85 cm
Subsoil:	γ' = 10 KN/m^3
	φ' = 30 °
Geotextile:	φ_g'= 30 °

A loose stone rubble with a thickness of D = 30 cm is planned with a submerged unit weight of γ_D' = 12 KN/m^3.
The average hydraulic gradient i_m is determined through equation (4a).
From equation (15) follows the factor of safety FS of one of the bank soil elements parallel to the slope in the depth z,
for example z = 0,1 m:

with i_m = 3,77 it follows FS = 1,59

and for z = 0,3 m:

with i_m = 2,46 it follows FS = 0,63

The result shows that a bank protection of a thickness D = 30 cm with a submerged unit weight of γ_D' = 12 KN/m^3 does not suffice to hinder the condition of failure in the soil zone under consideration.
After further development of the calculation the minimum stability is found by varying the factors z and D , so

that becomes the actual FS = required FS , in which the required safety factor FS can be stipulated according to soil mechanical criteria.
In the example above, the smallest factor of safety shown was at FS = 0,62 for a depth of z = o,33 m. Hence follows that sufficient safety was not found until the bank protection layer was D = 0,60 m.
Practical solutions follow also for direct comparison calculations with the abovementioned equations, if they are solved using the seepage force per unit volume j ($j = \gamma_W \cdot i$).

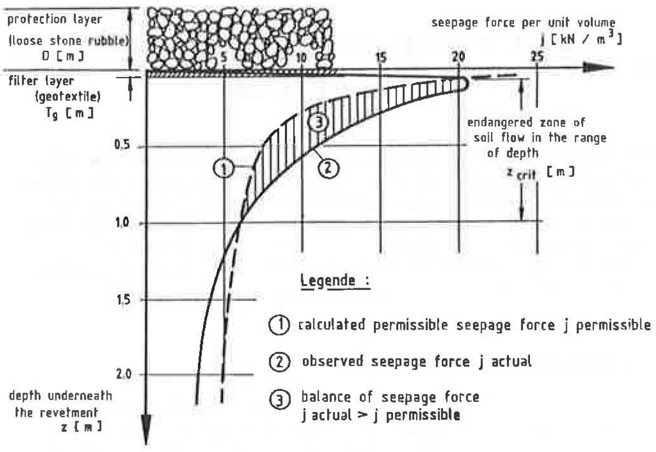


fig. 5 : seepage force unit volume j underneath a revetment as a result of rapid draw down

The graphic plot in Fig. 5 emphasises in a general form the hydraulic gradients of the calculated permissible values and the observed values of i using the equations (15) and (4a) in which the endangered zone of soil flow in the range of depth $z_{critical}$ underneath the embankment is to be shown.

8. Summary

Because unprotected banks of navigable waterways are, in the desired slope angle ß, not safe against soil erosion and soil flow, they have had to be stabilised for a long time through revetment constructions.Through the introduction of geotextiles a novel method of bank protection has been developed, which lead more and more to the use of geotextile filters as means of adhesion in constructions as distinguished from the classical ways of loose rubble stone layers bedded on graded grain filter layers.
The use of geotextiles also caused problems which were based on the fact that the riprap layers became lighter i.e. thinner. It was necessary to consider the problem in the sufficiency of the filter effectiveness. Besides this filter function a static function may be fulfilled, when the geotextile lies even and as an entity on the slope surface and is sufficiently anchored above the water exchange area. The stability of such a bank protection was deduced from the occurrence of the pressure gradient in the area of lowered water level and the emergence of the slope spring under draw-down conditions. For the dimensioning of a stabilised revetment simplified, but reasonably important suppositions were derived about the delayed decrease of pore water pressure in the subsoil as a result of the rapid draw-down effect of lowered water level by passing shipping. Out of the comparison with given examples it could be shown, that the design concept allows relevant examination of the design safety factor for sufficient stability against soil flow of the bank protection. The actual conditions lead to three-dimensional pore water pressure distribution in the subsoil which are not easily to determine in mathematical equations. Therefore an approximation of the decrease of the pore water pressure by adapting an exponential function lead to a useful design concept for the dimensioning of revetments. The problem of wave impact on the bank protection has not been dealt with in this design concept. In principle the resulting commencement of the pressure distribution can be introduced into the calculations in a similar way. However not sufficient fundamentals about the instationary pore water pressure distribution in the subsoil of bank protection under wave attack exist as yet to make such claims.

9. Literature

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