Stability of Slurry Trenches in Inhomogeneous Subsoil

Stabilité des Murs Emboués en Tranchée dans un Sous-Sol Inhomogène

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SYNOPSIS The numerical calculation of the stability of slurry trenches commonly is based on two safety conditions. Firstly the earth wedge beside the trench must be prevented from sliding downwards (external stability) and secondly the dropping off of single grains or grain groups must be excluded. In the presented paper it is endeavoured first of all to complete the methods of calculating the internal stability. The conventional criterion requires the representative single grain to 'floate', i.e. it is prevented from sinking down vertically and owing to this the suspensions are usually overdimensioned. The recent investigations confirmed pratical experiences which demonstrated sufficient stability also by the use of essentially lower concentrated suspensions. It is shown additionally, that in cases of locally instable coarse grained zones negatively inclined stable slopes are possible. Considering the depth, the thickness of the layers and the soil properties, stability criteria and advice for the composition are developed.

INTRODUCTION

Up to now two criteria are commonly used in the evaluation of the stability of slurry-trenches. Firstly no sliding of soil wedges may occur towards the trench. Secondly the single grains or grain groups are to be fixed in their initial position by the suspension, cp. Wenz (1963), Morgenstern and Tahmasseb (1965), Grewe (1965), Lorenz (1966, 1967), Weiss (1967). The first condition up to now has been considered fulfilled, when the hydrostatic retaining force is bigger than or equal to the active earth pressure. According to the second condition the effective yield limit τ_W of the retaining suspension has to be high enough to make the vertical sinking of the representative single grain impossible. This grain diameter is commonly fixed at 25 % of the grain distribution.

At first the so called membrane hypothesis is not correct in all cases. According to this hypothesis the hydrostatic fluid pressure always acts membranlike in the surface of the earth wall. As frequently observed, however, depending on the data of subsoil and suspension as well as geometrical conditions the suspension penetrates the earth wall up to a certain measure and/or certain filtrate formations may occur. Fig. 1 shows the principals of such penetrations and/or filter cake developments.

In the above system the critical case of the dropping off of single grains or grain groups is characterised by type 2 for coarse grained soils. In all other cases the representative grain is finer and on the other hand it is additionally fixed by a filter membrane built on the trench side.

Quantive statements in that field were made by Müller-Kirchenbauer (1972). According to these the final distance of penetration l is calculated by

$$l = \frac{h}{J_o} \qquad [cm] \qquad (1)$$

Hereby, according to fig. 2, h means the suspension pressure height and J_o is the stagnation gradient.

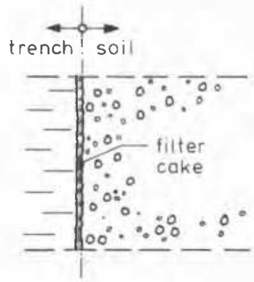
The stabilizing fluid pressure does not act membranelike in the surface area, but volumetrically inside the penetrated volume. The specific volumetric retaining force s yields

$$s = J_o \cdot \gamma_F \qquad [M_p/m^3] \qquad (2)$$

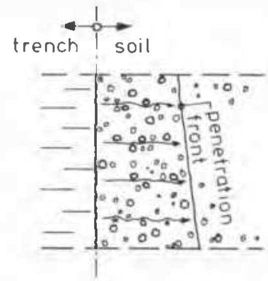
whereby γ_F = unit weight of suspension. The absolute retaining force is given by

$$S = V \cdot J_o \cdot \gamma_F \qquad [M_p] \qquad (3)$$

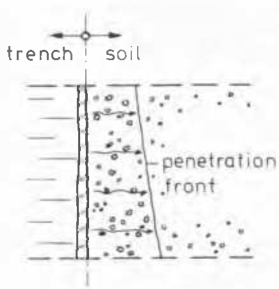
where V means the penetrated volume considered. According to equation 2 in various layers i.e. for different grain size distributions s shows very different values (see chapter: STAGNATION GRADIENT).



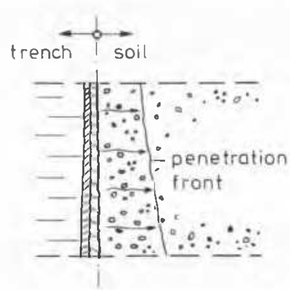
Type 1: True filter-cake development in fine pored soils. Suspension solid contents are filtered out in earth wall surface. The retaining forces act membranlike.



Type 2: Pure penetration in coarse grained soils, no surface filtration.



Type 3a: Combination of penetration and filtration. The coarse solid particles are filtered out on the surface.



Type 3b: Like type 3a. In a second state, however, before the coarse filter a further finer filter layer is separated.

Fig. 1: Types of penetration and filtration on liquid retained earth walls

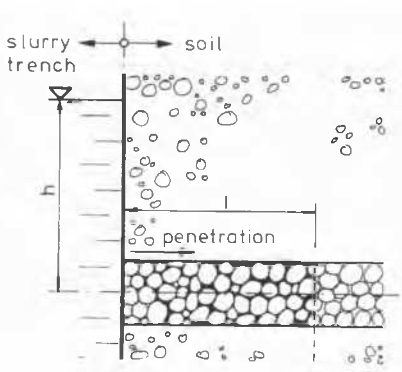


Fig. 2: Penetration of a plastic fluid (bentonite suspension) into porous medium

From equation 3 results, that the share of the penetrated volume beyond the potential sliding surface does not support the effective retaining force. This effective retaining force may be in so far lower than the hydrostatic force.

In the following we define, that the external stability (stability against sliding of the total earth body) is given sufficiently.

Methods and considerations of judging the internal stability (stability against progressive dropping off of soil material and failure processes caused thereby) are developed and explained.

INTERNAL STABILITY

In the following we presuppose the external stability, i.e. the effective hydrostatic retaining force is bigger than or at least equal to the sum of the earth pressures to be expected.

We consider the soil zone surrounding the trench containing the limit layer exposed to the danger of dropping off. In this penetrated zone the distance of penetration results from formula 1 and specific volumetric retaining pressure from formula 2.

For such cases up to now only the safety against vertical sinking of the representative grain has been stated. Especially in the very coarse layers between, the above assumption leads to high suspension stiffness which causes difficulties on the site (intensive dirt concentration, high increase of unit weight and problems in driving upwars the suspension by concrete).

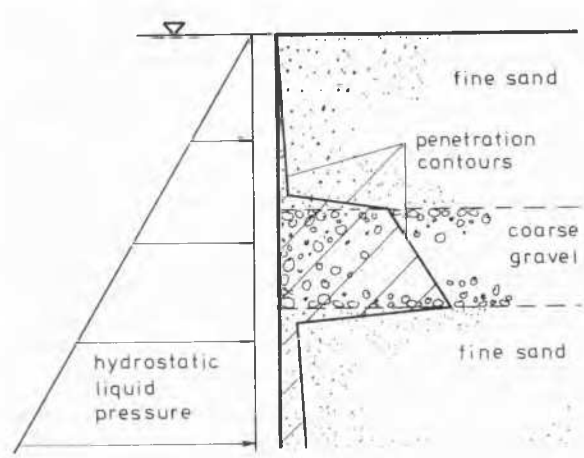


Fig. 3: Penetration contours in inhomogeneous subsoil

According to fig. 3 and 4 more precise consideration requires that:

a.) the soil zone beside the trench must be stable under the influence of the specific volumetric retaining pressures
b.) the representative grain has not to move out of the slope.

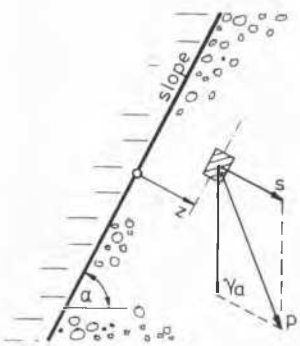


Fig. 4a:
Application of the specific volume forces inside the injected zone

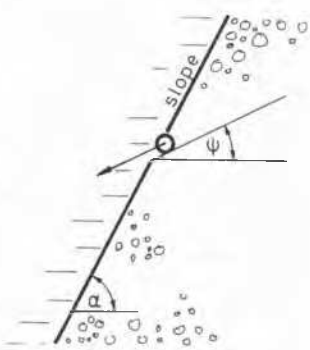


Fig. 4b:
Possible direction of the initial motion of single grain

Firstly we assume an indefinite extension of the slope. The specific volume-forces acting in the penetrated region are shown in fig.4a. The first stability condition mentioned means we have to determine the angle α. at which the slope volume is just plastified, i.e. the Rankine-state is reached.

We use the following facts:

a.) the gravitation γ_a (i.e. here the soil unit weight under uplift) and the specific retaining pressure s are to be added vectorially to a new specific volume-force p , which acts all over the considered zone.
b.) In the Rankine-state two sliding plane systems exist. One of them runs parallelly to the slope surface and the other shows an inclination of $90^\circ - \varphi$ against the slope, i.e. against the first system.
c.) The inclination of the first sliding plane system coincides with the direction of the specific volume force p.

According to the principals shown in fig. 4, we find for the normal stress σ_z and the shear stress τ_{zx} in the first sliding plane system:

$$\sigma_z = \gamma_a \cdot z \cdot \cos\alpha + J_o \cdot \gamma_F \cdot z \quad [\frac{Mp}{m^2}] \quad (4a)$$

$$\tau_{zx} = \gamma_a \cdot z \cdot \sin\alpha \quad [\frac{Mp}{m^2}] \quad (4b)$$

For the failure state follows:

$$\tan\varphi = \frac{\tau_{zx}}{\sigma_z} \quad (5)$$

$$J_o \cdot \frac{\gamma_F}{\gamma_a} \cdot \sin\varphi = \sin(\alpha - \varphi) \quad (6)$$

With these equations respectively using the Mchr's circle construction shown in fig. 5 it is possible to determine the necessary stagnation gradient especially for the most common case i.e. $\alpha = 90^\circ$. On the other hand from fig. 6 the angle α may be won for a given J_o and φ.

STABILITY OF SINGLE GRAIN

In the chapter above the slope body has been considered as an homogeneous one. This is true all over the injected zone except for the last thin layer in the surface facing the trench, when we investigate the stability of the single grain.

Two significant cases of that kind are shown in fig. 7. The direction of the initial motion according to the principal drawn in fig. 7a is inclined neither vertically nor parallelly to the slope surface. According to Müller-Kirchenbauer (1972) this direction corresponds to the angle $\psi = \alpha + \beta - 90^\circ$, whereby β, which depends on the subsoil structure, is to be taken into account from 35° to 40°. That means for a certain combination of τ_W and ψ that the diameter D of the just fixed grain is always bigger than the critical diameter D_V calculated conventionally against vertical sinking. The relation between α and D/D_v is shown in fig. 8 by using the symbols of fig. 7.

D_v is to be calculated by Weiss (1963):

$$D_v = \frac{3}{2} \tau_w \cdot \pi \cdot \frac{1}{(\gamma_s - \gamma_F)} \quad [cm] \quad (7)$$

and D by Müller-Kirchenbauer (1972):

$$D = \frac{D_v}{\sin\psi} \quad [cm] \quad (8)$$

According to these considerations for $\psi = o$ a suspension-injected slope-body cannot fail by dropping off, i.e. for example for $\beta = 35^\circ$ the minimum inclination α would be reached by $\alpha = 55^\circ$. This is correct as far as the internal slope-body is stable according to equation 6 and/or fig. 5 and fig. 6.

We get a further variant by the penetration of solid dirted suspension. This solid content is normally filtered out totally or at least partially on the surface and builds up a thin layer with an increased stagnation gradient.

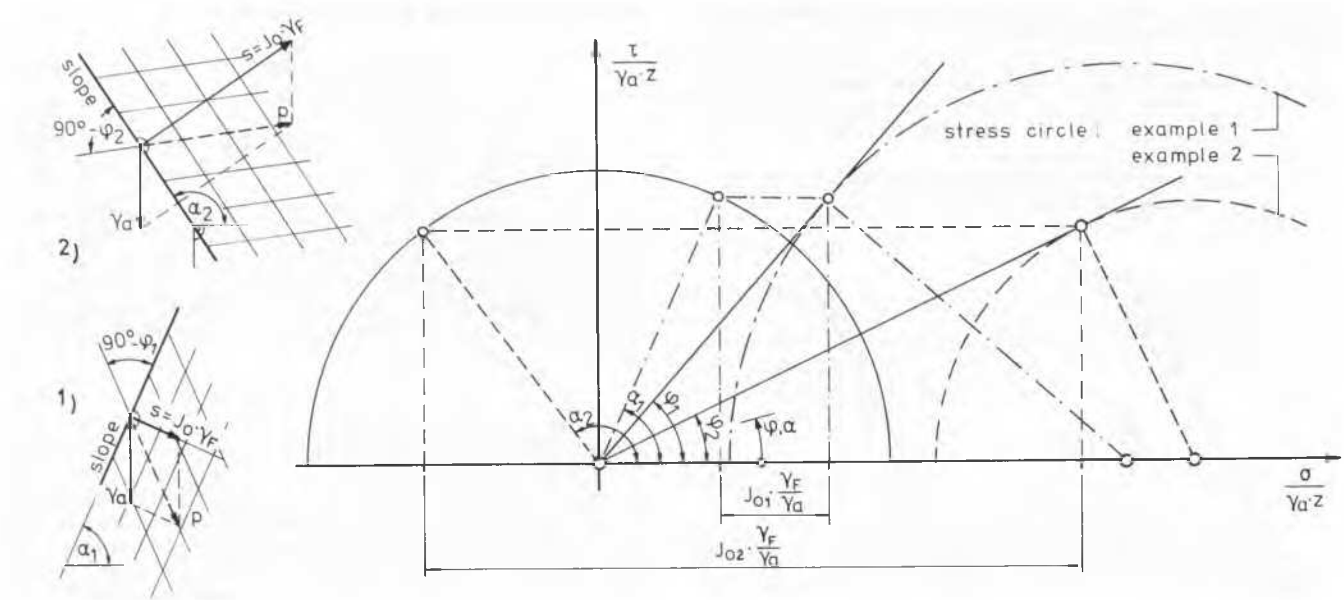


Fig. 5: Relation between angle of internal friction φ, slope angle α and stagnation gradient J_o

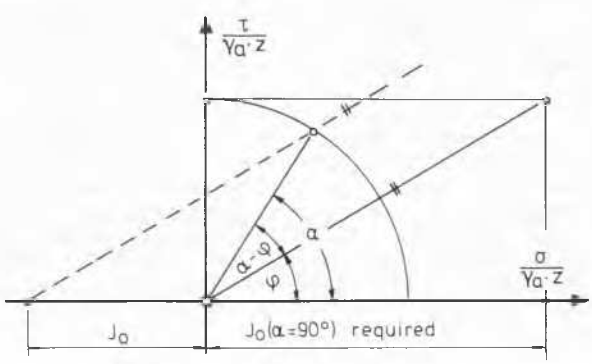


Fig. 6: Determination of the possible angle of the slope α for given φ and J_o

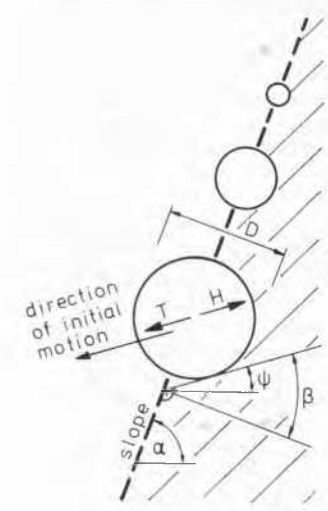


Fig. 7a: Direction of the initial motion when the single grain dropping off

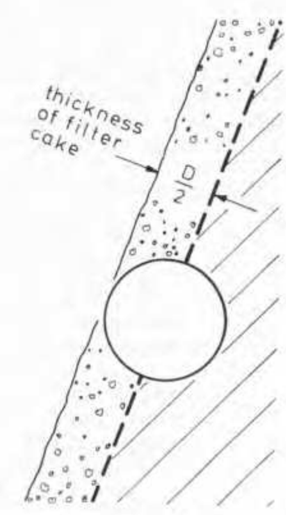


Fig. 7b: Critical thickness of filtercake for fixing the representative single grain

As evident from fig. 7b in that case the single grain consideration is not yet valid and the stability proof according to equation 7 could be applied to the mentioned thin layer itself. On the other hand the filtercake grain distribution is essentially finer than the soil's,i.e. the filtercake stagnation gradient is accordingly much higher than the soil's.
That means, that only a critical filter thickness in the order of magnitude of the half representative diameter (see fig. 7b) is necessary in order to obtain a sufficient fixing effect. Furthermore in coarser layers as shown in fig. 3 the distance of penetration as well as the penetrated suspension volume are comparatively big and, therefore,the filter development is improved. After the evaluation of many site situations the filter formation is mostly intensive enough, to reach the mentioned critical thickness. Provided that the penetration of suspension together with the total grain size distribution without filter development is excluded, a quantitative calculation of the filter thickness is possible following Müller-Kirchenbauer (1972), but not always necessary.

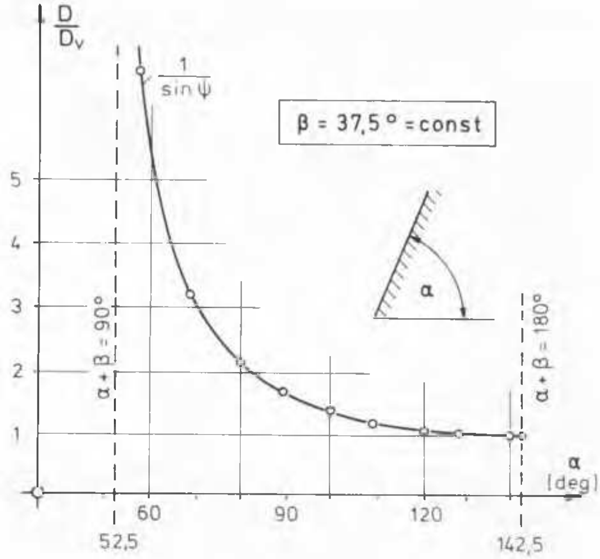


Fig. 8: Relation between the quotient D/D_v and the angle of the slope

Another important qualitative proof of filter occurance is based on the grain size distribution of the dirt. The latter begins at the coarse side at the diameter D_v (see equation 7) shows at first a relatively steep inclination and depends, furthermore, on the grain size distribution of the trenched layers. As a very safe criterion for filter occurance according to Terzaghi may be calculated

$$d_{15} \leq 5 \cdot D_v \qquad [cm] \qquad (9)$$

whereby d_{15} is the 15 % diameter of the considered soil material in the wall. On the other hand test evaluations have demonstrated that the modified criterion

$$d_{15} \leq 8 \cdot D_v \qquad [cm] \qquad (1o)$$

is practicable.

Independent of the latter considerations the dropping off stability against clean suspensions has been investigated in a special apparatus as pointed out in fig. 9. Hereby the model slope under the influence of the retaining pressure could be turned to all inclinations up to 18o° against the original vertical position (i.e. the built in position). In addition the turbulence caused by the grats under site conditions was simulated by a rotating propeller. It is remarkable that for inclinations up to 9o° against the original axis, i.e. for the most common case of vertical slope, no drop off has been observed except when the stability has been proved by equation 6.

These results indicate that the physical bases of the mentioned developments contain certain reserves, which can not be explained totally at the time being. Additional friction effects not taken into account up to now, could possibly be the explanation.

Fig. 9: Test apparatus for stability of inclined penetrated slopes

OVERHANGING SLOPE

In most cases - as mentioned above - the inclination $\alpha = 9o^o$ is usually applied. On the other hand for practical applications it is certainly adventageous to obtain further information about the consequences of local failures i.e. local slope inclinations with $\alpha \leq 9o^o$. It can be demonstrated, that for certain constellations such failures do not develop upwards to the ground surface i.e. they remain limited to the thickness of the coarse layer between. Such a situation is given in fig. 1o.

In layer II there exists a penetration according to type 2 and in layers I and III a membrane corresponding to type 1 as shown in fig. 1. Using Coulomb's hypothesis and assuming momolithic sliding wedges we are able to calculate the safety against the sliding downwards along the failure planes, inclined by ω to the horizontal.

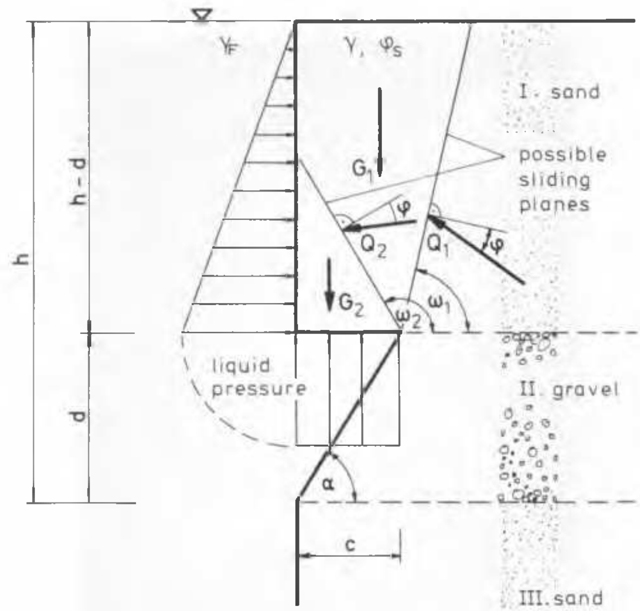


Fig. 1o: Application of the forces on the overhanging earth body

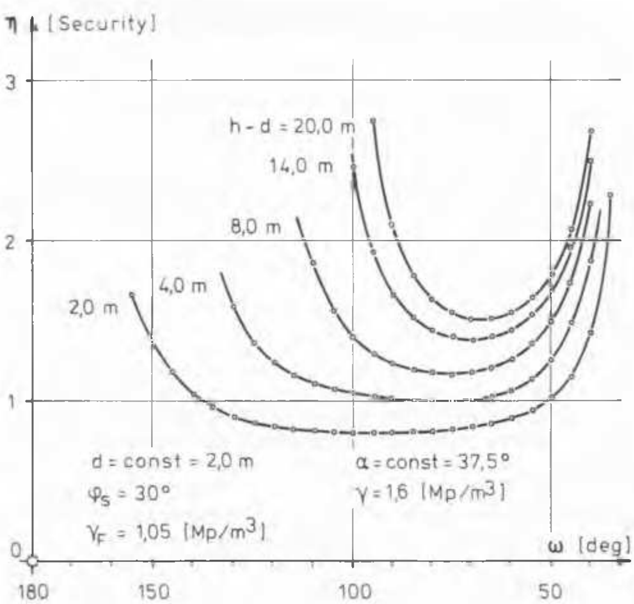


Fig. 11: Relation between assumed sliding plane inclination and calculated safety factor according to fig. 1o

Several trial calculations confirmed:

a.) The critical sliding plane (showing the minimum safety factor for a certain constellation) is always inclined with $\omega \lesssim 90^\circ$.

b.) In the order of magnitude $\eta = 1$ the shape of the curve $\eta = f(\omega)$ is very flat and in the range around $\omega = 90^\circ$ η-values hardly vary (see example fig. 11).

c.) According to a.) and b.) ω_{crit} can be taken into account for the vertical sliding plane inclination as a simplified calculation principle.

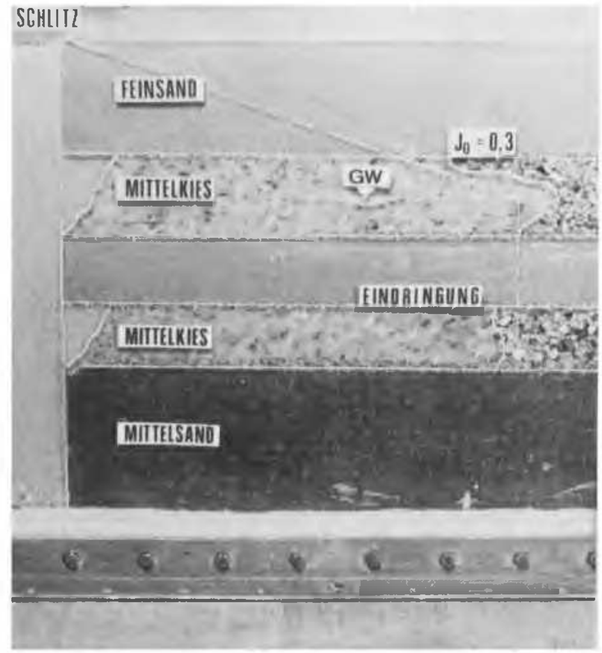


Fig. 12: Example for model test result showing two stable overhanging slopes

STAGNATION GRADIENT

The stability conditions explained in the chapter INTERNAL STABILITY cannot be calculated without the knowledge of the appropiate stagnation gradient. Firstly it is possible to determine the J_o-value experimentally using a special permeameter, demonstrated by Müller-Kirchenbauer (1968, 1972), the principle of which is shown once more in fig. 13. On the other hand one can proceed by calculating in a similar way to Weiss (1963) on the basis of an equivalent capillary system. Contrary to Weiss, who defined 17 constants in all recommending that these should be singly determined experimentally, we are trying to compile the influences as far as possible and in that way to limit the test effort.

In the simplest way we obtain the distance of penetration l for a capillary system by

$$l = \frac{h \cdot R \cdot \gamma_F}{2 \cdot \tau_F} \quad [cm] \qquad (11)$$

whereby h: pressure height along l

R: equivalent pore radius

γ_F: unit weight of suspension

τ_F: effective yield limit of suspension.

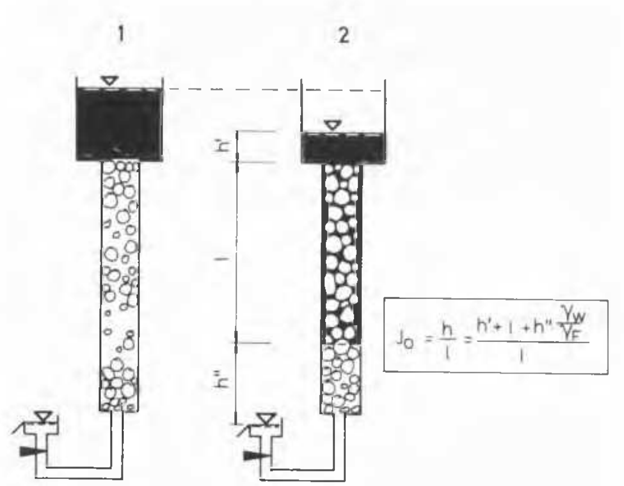


Fig. 13: Experimental determination of the stagnation gradient

State 1: Situation before the beginning of the test
State 2: Situation after stagnation

In the same system for the stagnation gradient yields:

$$J_o = \frac{h}{l} = \frac{2 \cdot \tau_F}{R \cdot \gamma_F} \qquad (12)$$

As is well known for the Darcy's permeability a similar consideration is sometimes used. Expressing the permeability k by the equivalent system, it is to be written:

$$k = A \cdot \frac{n \cdot R^2 \cdot \gamma_w}{8 \cdot \eta} \quad [cm/s] \qquad (13)$$

whereby A: Constant, containing the geometrical differences between real and changed system
γ_w: unit weight water
n: porosity
η: dynamic viscosity

For R we obtain:

$$R = \sqrt{\frac{k \cdot 8 \cdot \eta}{A \cdot n \cdot \gamma_w}} \quad [cm] \qquad (14)$$

Applying equation 12 to a real pore system we have to complete

$$J_o = \frac{h}{l} = C_1 \cdot \frac{2 \cdot \tau_F}{R \cdot \gamma_F} \qquad (15)$$

whereby C_1 contains the physical an geometric differences of the real and capillary system against a plastic fluid such as a bentonite suspension. By setting equation 14 into 15, yields:

$$J_o = \frac{h}{l} = C \sqrt{\frac{\eta \cdot \gamma_w}{2 \cdot k \cdot n}} \cdot \frac{\tau_F}{\gamma_F} \qquad (16)$$

Herein C contains the influence shown by Weiss (1963) and τ_F the minimal yield limit determined by rotation viscosimeter for stagnation state. C replaces among other influences the differences between the yield limit τ determinable in the viscosimeter and the effective yield limit in the pore system, slipping effects between fluid and pore channel surface etc..

To determine the order of magnitude of the factor C numerous tests were carried out and evaluated, for which all the parameters in equation 16 were determined. As stated for different kinds of bentonite the frequency boundary of the factor C - independent of the chosen concentration and of the grain size distribution is relatively narrow. The average C-values for several investigated kinds of bentonite lie between o,45 and o,9o. This remains basically valid, when the permeability is estimated by the modified empirical formula of Hazen ($k = c \cdot d_w^2$).

Therefore, using the average C-value for estimating J_o we get a relatively small error, which is remarkable, because permeability is involved.

Without detailed discussion of this remarkable coincidence we shall briefly name some thinkable reasons for these observations. Firstly the stagnation gradient depends reciprocally only on R^1 and not as the permeability on R^2. Secondly the influence of turbulance disappears because we are considering stagnation problem for plastic media and not a flow problem of Newtonian fluids. Therefore we can also use Hazen's estimation to express the pore channel radius in coarse pore systems, in which Darcy's law is no longer valid.

CONCLUSION

The presented paper deals with the internal stability of slurry trenches in the critical state, i.e. before being filled up with concrete.

At first the influences causing the progressive dropping off of the earth material out of the stabilized wall are investigated. It is shown that the conventional proof against vertical sinking of single grain is overdimensioned and causes too high concentrations of suspension. Furthermore this proof was always related to the coarsest representative grain of all the trenched layer sequence, even when the really coarse layer was not very thick.

In this paper the penetration and filtration behaviour of the suspension against relatively coarse grain systems was taken into account. Using the dimensionless stagnation gradient on the basis of the Rankine's state methods were developed to determine the angle α, under which a penetrated soil can be considered as stable, even when the conventional proof against simple sinking does not stand. Furthermore it is demonstrated that under certain conditions which can be proved by Coulomb's hypothesis, for a certain thickness a local failure zone can be tolerated and need not cause another upwards directed failure development. Simplified methods were suggested in order to determine the stagnation gradient J_o which is necessary for the realisation of the above-mentioned calculation.

The results obtained were controlled by experiments.

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