OBSERVATIONS ON AN EMBANKMENT ON SOFT ORGANIC GROUND

By

I. KABAI—I. LAZÁNYI

Department of Geotechnique, Technical University, Budapest
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1. Introduction

Recently, embankments have often to be built in peaty, marshy areas overlying soft subsoil which formerly were simply bypassed or where foundations were realized by removing first the soft soil layer.

Building earthworks on soft soil raises several problems: complete or limited ground failure under the embankment, large settlements and protracted consolidation, lateral spreading or total destruction of the embankment due to tensile cracks.

Various protective measures have been developed for overcoming these difficulties such as complete or partial displacement of the soil (by dredging or blasting); vertical drains — gravel or sand piles — in the soft layer for accelerating the consolidation and for partial load transfer to a deeper soil layer likely to have more favourable bearing values (KÉZDI, 1951; HANBSO, 1960; MOSER, 1977). Recently, the so-called "Geodrains" and nonwoven fabrics have gained a wide application. The main point in all these methods is to improve the stability of the embankment foundation partly by accelerating the consolidation, partly by reinforcing the soil (HANSBO, 1975).

Recently, the so-called step-wise or stage construction has been increasingly applied. The embankment is built up by progressive stages whereby also the overburden on the soft foundation is gradually increased, according to a schedule. Under the increasing loads, the subsoil undergoes significant consolidation and its physical properties will be improved enough by the end of construction to carry the entire load of the embankment. Up-to-date instruments are available for controlling the step-by-step construction and for monitoring the settlements, the pore-water pressures and the changes of the shear strength. Besides, inclinometers may be used to measure the horizontal displacements in a vertical borehole located at the toe of the embankment.

Experimental observations made in constructing an embankment on peaty soft foundation will be presented. On one section of an upgraded national main road traversing an area of peaty, marshy ground, the design institute UVATERV suggested to construct a major embankment by the step-wise method. The Authors were entrusted by the client to carry out the necessary laboratory and field measurements prior to, and during the construction work. The purpose of the investigations was to observe continuously the behaviour of the embankment during construction, and to determine the increase in strength caused by preloading, so that further construction could be scheduled. Besides of the direct, practical benefits obtained, the measurements yielded a great deal of interesting general observations concerning the deformation and shear strength behaviour of waterlogged soft organic peaty soils.

2. Stage construction method

It is an established fact that soft soils subjected to controlled loading may undergo a significant "hardening" i.e. a gain in shear strength (BISHOP, 1954; KÉZDI, 1975, 1976 and others). The stage construction method takes advantage of this peculiarity of soil behaviour.

The undrained shear strength and consolidation stress of normally consolidated soils are known to be linearly related (see the lines of stress vs. voids ratio and stress vs. shear stress in Fig. 1).

Beyond a given isotropic consolidation stress, the shear strength τ_u determined in a closed (undrained) system depends only on the phase composi-

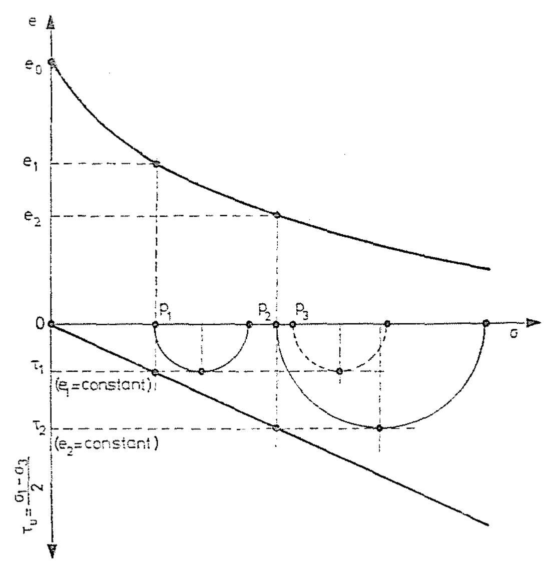

Fig. 1. Compression and undrained shear strength of normally consolidated soils vs. isotropic consolidation stress

tion of the soil rather than on the hydrostatic stress value applied in the closed system during the test to failure (see the principal stress circles belonging to void ratios e_1 and e_2 in Fig. 1).

The test method presented in the following simulates the course of the step-wise loading itself, hence it suits laboratory checking and designing of stage construction in practice.

According to practical experience, laboratory tests yield slightly higher shear strength values than do field tests. This discrepancy can be accounted for — among others — by the different states of stress.

In nature, in what is known as pure compression, the soil is subject to anisotropic consolidation stresses (also known as K_0 or at-rest condition), whereas in a laboratory strength test the initial consolidation stress state can be selected at will as isotropic or anisotropic one.

The effect of different initial stress states may be reduced by starting also the test from anisotropic consolidation stress state. Such a process is represented in Fig. 2 where the principal stress circles are shown in the system of coordinates σ, τ and the stress path in the system p, q.

An arbitrary anisotropic stress state is represented in the coordinate system p, q by point 0. In the coordinate system σ, τ the corresponding anisotropic effective principal stresses are $\bar{\sigma}_{z0}$ and $\bar{\sigma}_{x0} = K_0 \cdot \bar{\sigma}_{z0}$. Point 0 is on line q_0, corresponding to the at-rest condition, while the neutral stress is zero ($u = 0$). Producing failure in a closed system ($\Delta V = 0$) in a saturated soil subject to anisotropic stresses $\bar{\sigma}_{x0}$ and $\bar{\sigma}_{z0}$ and plotting the process vs. total stresses σ_{x0}, σ_z results in the stress path $\overline{OA_0}$. The corresponding constant undrained shear strength $\phi_u \simeq 0$ is τ_u. The result plotted in terms of effective stresses $\bar{\sigma}_x$, $\bar{\sigma}_z$ leads to point B_0 on line $\bar{q}$ corresponding to failure.

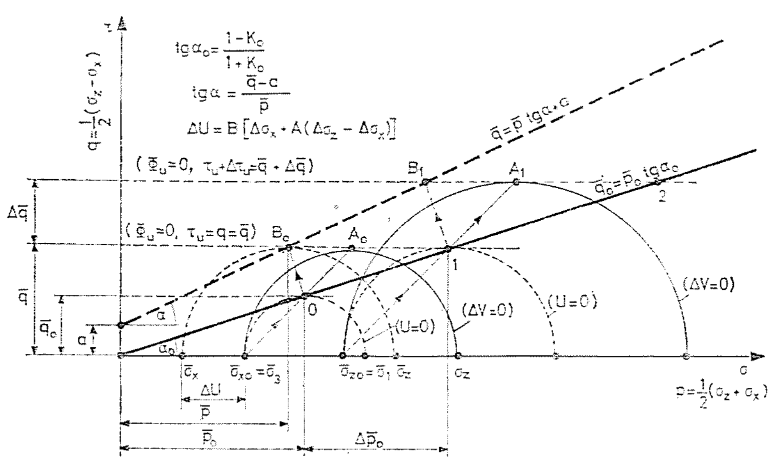

Fig. 2. Effect of preloading on undrained shear strength

In the case of an instantaneous load on the soil, when consolidation cannot take place ($\Delta V = 0$, $\varnothing_u = 0$), the critical shear stress is given by:

$$\tau_u = q = \bar{q} = a + \left(\bar{\sigma}_x + \frac{\bar{\sigma}_z - \bar{\sigma}_x}{2}\right) \sin \varnothing .$$

Relationships for $\varnothing$ and c:

$$\varnothing = \arc \sin (\mathrm{tg}\, \alpha)$$

and

$$c = \frac{a}{\cos \varnothing}$$

permit τ_u to be calculated as

$$\tau_u = c \cos \varnothing + \left(\bar{\sigma}_x + \frac{\bar{\sigma}_z - \bar{\sigma}_x}{2}\right) \sin \varnothing .$$

Clearly, the value of τ_u could also be obtained by starting from an anisotropic initial state (see GANGOPADHYAY, 1974; KHERA—KRIZEK, 1968). Omitting details, τ_u is given by:

$$\tau_u = \frac{c \cos \varnothing + \bar{\sigma}_{z0} \sin \varnothing \left[K_0 + A(1 - K_0)\right]]}{1 + (2A - 1) \sin \varnothing} ,$$

where c and $\varnothing$ are shear strength parameters determined from the effective stresses; A is *Bishop*'s pore-water pressure coefficient and K_0 the coefficient of at-rest condition ($K_0 = \bar{\sigma}_{x0}/\bar{\sigma}_{z0}$). This method assumes $u = 0$ at $\bar{\sigma}_{x0}$ and $\bar{\sigma}_{z0}$.

On the other hand, for step-wise load increments $\overline{\Delta p}_0$ of a sufficient low rate to permit complete consolidation and gradual compaction (drained test), the effective at-rest line $\bar{q}_0$ applies, and the stress path $\overline{01}$ will result (Fig. 2).

Once a consolidated state of stress depicted by point (1) is attained somewhere in the soil, the undrained shear strength (with no volume change, $\Delta V = 0$) will be increased at that point to ($\tau_u + \Delta\tau_u$) as seen in Fig. 2. Thus, the first application of load Δp_0 can increase the shear strength in the foundation by 0 to $\Delta\tau_u$ through a given — uninhibited — compaction of the subsoil. The shear strength increment of 0 to $\Delta\tau_u$ may incidentally permit another load increase. This process cannot, of course, be continued infinitely because both the values and the ratios of vertical to horizontal stresses differ at each point of the foundation. The lateral displacement will only be zero at the embankment axis, this being the only place where strict at-rest condition prevails. Besides, also the change in time of the shear strength due to creep and the development of the progressive failure have to be taken into account. Of course, these effects can only be approximated in design, so the controlled stage construction method must not be based on laboratory tests alone but it has to be completed by field checks.

3. Investigation of deformations

Application of the described principles was tested in the laboratory on samples of organic peaty soils. The identification and compaction data (w_L, w_p, w, ϱ_n) are represented in Fig. 3.

The samples were subjected to compression (oedometer) tests in order to determine the degree and timely course of the compression (primary and secondary consolidation) and the time-dependence of the pore-water pressure, using the instrument seen in Fig. 4. This instrument developed at the Department of Geotechnique, Technical University, Budapest, differs from the usual laboratory devices by accommodating soil samples of different heights (h/d = 0.3 to 1.1). Completed by a filtering stone inserted into the sample top face, and a pore-water pressure sensor, it suits determination of the pore-water pressure directly at the top surface of the saturated soil sample.

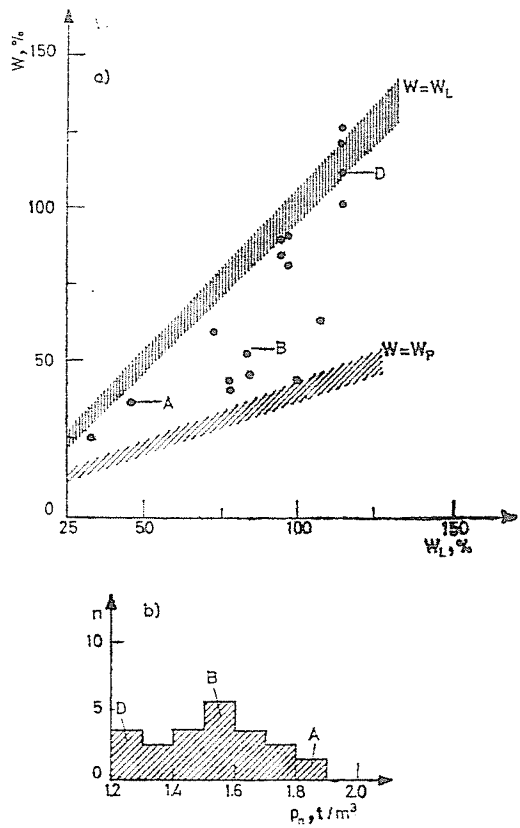

Fig. 3. Physical characteristics of the tested soils. a) relationship between liquid limit w_L and natural water content w; b) frequency distribution of wet bulk density ϱ_n; A — organic clay; B — organic, peaty clay; D — clayey peat

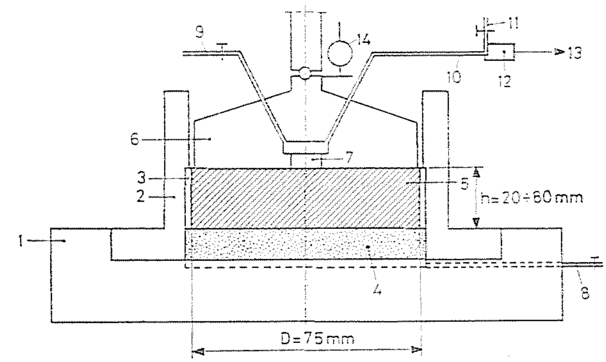

Fig. 4. Oedometer developed at the Department of Geotechnique. 1 — base plate; 2 — stiff confining ring; 3 — sampling ring; 4, 7 — filterstones; 5 — soil sample; 6 — loading platen; 8 — drain outlet; 9 — water inlet to deaeration; 10 — water outlet; 11 — air vent; 12 — pressure gauge transducer; 13 — pressure gauge recorder; 14 — strain gauge

The compression tests have proven the extreme compressibility of the tested soil. The values of the dry bulk density ϱ_d and of the modulus of compression M are shown in Fig. 5. The M values are given for two successive load increments ($\sigma_z = 0$ to 100 kPa and $\sigma_z = 100$ to 200 kPa) clearly demonstrating the significant, favourable effect of preloading.

The consolidation tests were rather instructive by pointing out the significance of the secondary time-effect.

The principal components of the compression of a soil under a given load are known to be the immediate compression and the deformation due to primary and secondary consolidations. These effects may be distinguished in

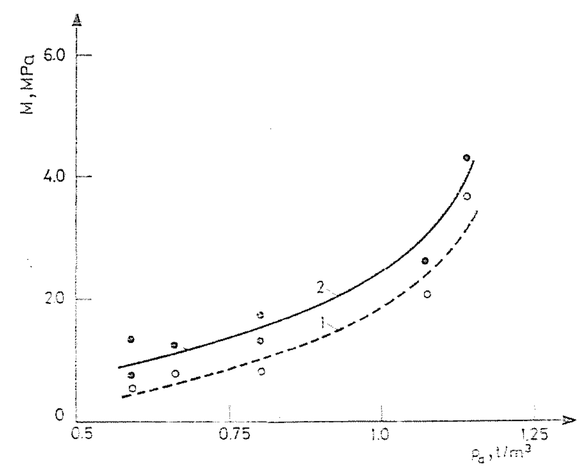

Fig. 5. Relationship between the compression modulus M, dry bulk density ϱ_d and vertical normal stress σ_z. (1 $\sigma_z = 0 \div 100$kPa; 2) $\sigma_z = 100 \div 200$kPa

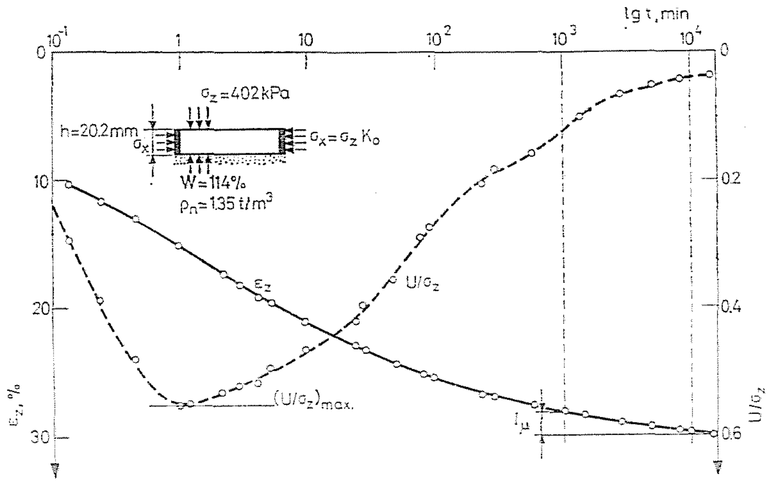

Fig. 6. Specific compression ε_z and pore pressure ratio u/σ_z vs. time ($\log t$) in a partially drained system

laboratory tests by the square-root or the logarithmic plotting methods (KÉZDI, 1976).

A typical test result comprising specific compression ε_z, ratio of neutral stress to vertical stress u/σ_z vs. time ($\log t$) has been plotted in Fig. 6. The measurement of the pore-water pressures made it possible to separate primary consolidation lasting as long as there is pore-water pressure acting at all, from secondary time-effect. The tests showed primary consolidation to be rather exactly described by *Terzaghi*'s theory of linear consolidation, as seen in Fig. 7. But deformation continues even after primary consolidation is

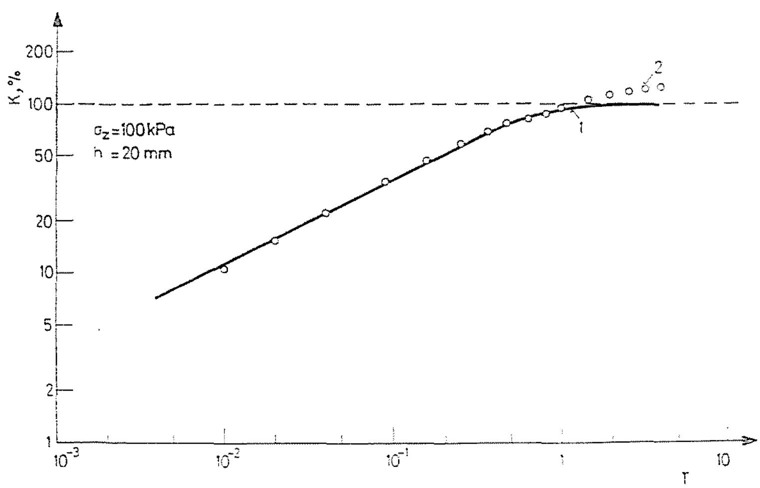

Fig. 7. Relationship between the degree of consolidation $\varkappa$ and time factor T; 1 — curve furnished by *Terzaghi*'s theory of linear consolidation; 2 — measurements

complete ($\varkappa = 100\%$), and the secondary time-effect may clearly be distinguished. (It is interesting to note that the delimitation of primary consolidation by measurement of the neutral stresses and by square-root and logarithmic plotting yielded closely agreeing results.)

The experiments pointed to the following regularities or laws of secondary consolidation:

a) The process follows a logarithmic law; its semi-logarithmic plot is a straight line. To describe the process of deformation, the index of secondary time-effect I_μ defined as the specific compressive strain ε_z for a logarithmic time cycle (e.g. 10 to 100 minutes, 1 to 10 years etc.) will be introduced (see Fig. 6).

b) Several tests suggest that the value of index I_μ increases as the logarithm of load (Fig. 8). This relationship seems to be supported by the findings of other researchers (Akai, 1963; Watanabe, 1964).

c) Tests conducted with soil samples of different heights have shown the secondary time effect to be independent of the sample (or soil layer) thickness (Fig. 9).

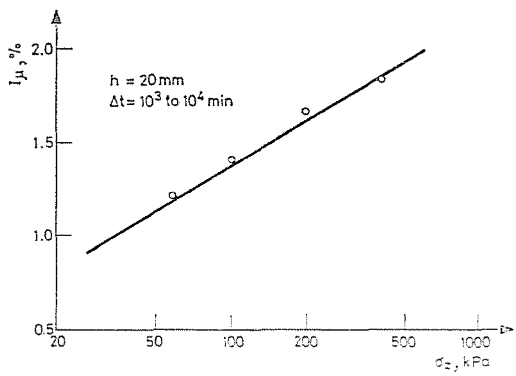

Fig. 8. Slope of the line of secondary consolidation, index of secondary time effect $I\mu$ vs. vertical stress σ_z. (Measurements on soils presented in Fig. 3)

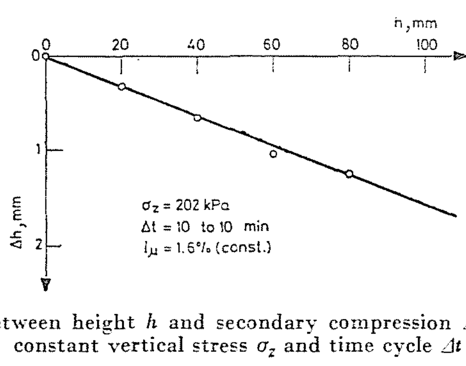

Fig. 9. Relationship between height h and secondary compression Δh of the soil sample at constant vertical stress σ_z and time cycle Δt

d) In an actual laboratory test, the measured ratio u/σ_z is normally less than unity even "at the instant" of load application. It is at its maximum shortly after load application is complete, as shown by line u/σ_z in Fig. 6.

The u/σ_z maxima measured in our tests as a function of sample height h and load σ_z are seen in Fig. 10. These observations seem to agree with the laboratory tests by KARLSON—WIEBERG, 1978; MURAKAMI, 1977; and with the field test results by SHOJI—MATSUMOTO, 1976.

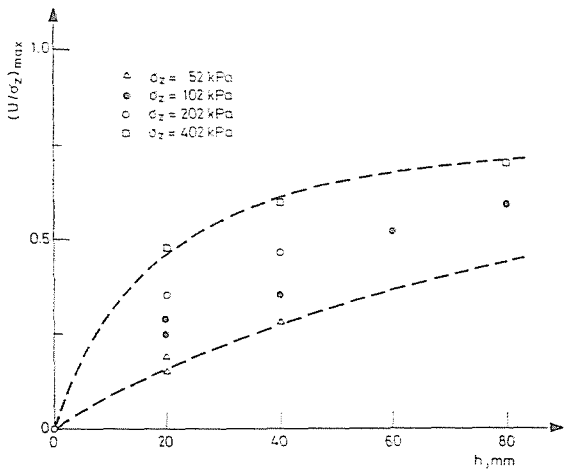

Fig. 10. u/σ_z maxima measured in laboratory in partially drained consolidation tests vs. stress σ_z and sample height h

This apparent time lag in pore-water pressure build-up may be attributed to the fact that on the one hand, strictly there exists no instantaneous load, and on the other hand, the pore-water pressure build-up depends on the deformation rate of the soil skeleton, governed by the viscous properties of the soil, thus, also on the magnitude and rate of loading.

4. Laboratory strength tests

The shear strength of soft, organic, peaty soils may greatly increase because of the significant compression and the change in soil structure under load. To investigate this problem, triaxial compression tests, as well as direct shear tests were made on undisturbed samples taken from shafts, this kind of sampling being the least disturbing for the structure. On the area investigated, the groundwater rises from time to time up to, or even above, the ground surface, therefore field vane borer tests have also been performed to obtain data on the in-situ shear strength of the deeper soil strata (see Chapter 5).

The triaxial compression test may start from either the isotropic or any anistropic consolidation stress state, in closed (undrained) or open (drained) system. The least favourable assumption for the embankment is the undrained system. For practical reasons, in the case of soil samples of small diameter ($d \simeq 38$ mm), it was more expedient to start the test from the isotropic consolidation state, and to increase the load to failure in undrained system ($\Delta V = 0$), with simultaneous measurement of the pore-water pressure.

One of the test results is seen in Fig. 11. Line $\overline{01}$ represents the isotropic consolidation state, $\overline{12}$ the stress path in terms of total stresses, and $\overline{13}$ the stress path in terms of effective stresses. Lines A and B are the stress paths at failure in terms of total and effective stresses, respectively.

Shear strength parameters $\varnothing$ and c calculated from triaxial tests are seen in Fig. 12 where the curves A and B correspond to the shear strength parameters expressed in terms of total and effective stresses, respectively. The result clearly reflects the difference due to the neutral stresses.

The tests revealed a close relationship between the mean principal stress at failure σ_a and the corresponding neutral stress u. The test results are shown in Fig. 13. The experiments demonstrate clearly that an abrupt load increase to a sufficiently high value may lead in an undrained system to a critical neutral stress ratio

$$\frac{u}{\sigma_a} \rightarrow 1$$

and eventually to failure. In construction, this state must not be allowed but kept safe controlled with a sufficient margin of safety. In this respect, it has, however, to be pointed out that, in reality, the process takes place in a drained or partially drained system, since even abrupt loads may cause

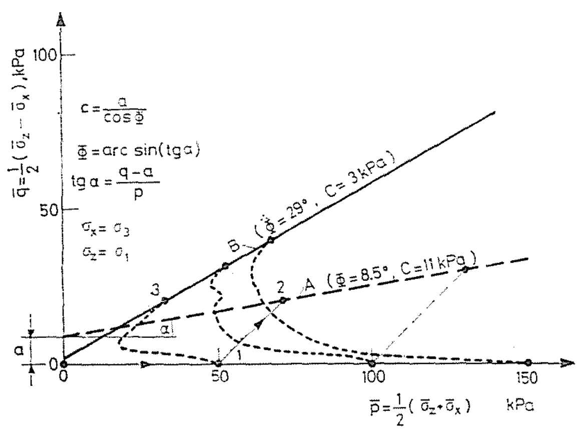

Fig. 11. Stress paths and shear envelopes vs. total (A) and effective (B) stresses

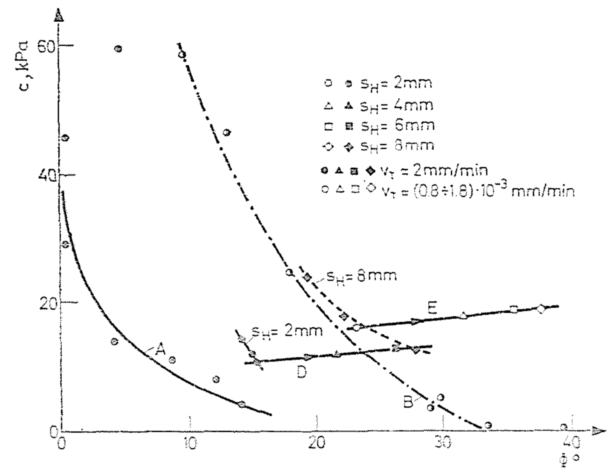

Fig. 12. Variation of shear strength parameters — triaxial compression test results. A — vs. total stresses; B — vs. effective stresses; D — direct shear test by quick loading and shear; E — slow shear of soil sample consolidated under vertical load

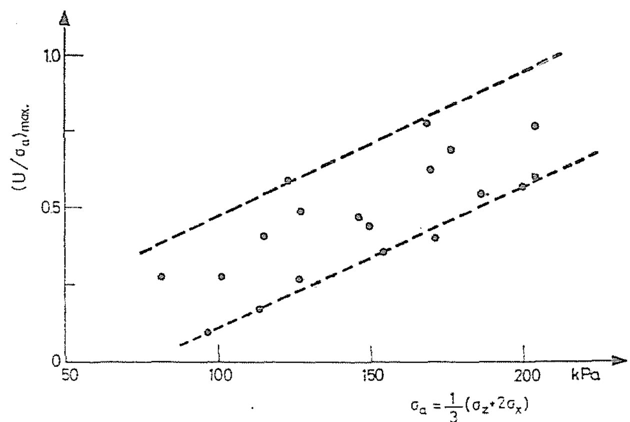

Fig. 13. u/σ_a values obtained in triaxial compression test in closed system vs. σ_a

some volume change and compaction. And when a soft soil is loaded sufficiently slowly, the resulting large compaction produces a significant strength increase.

This phenomenon has been evidenced by direct shear tests, one of which is represented in Figs 14 and 15.

Test (1). After placing the saturated soil sample into the shear box, the vertical normal stress σ is applied immediately and at a rapid rate of

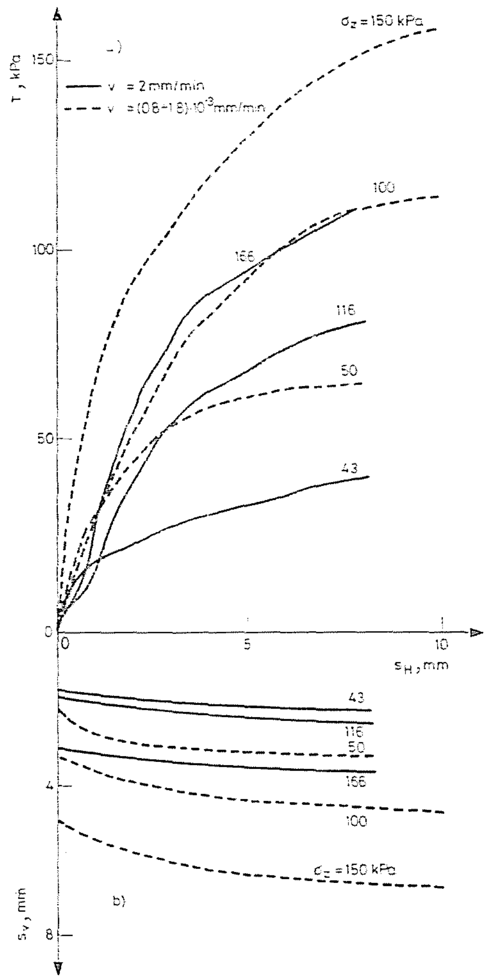

Fig. 14. Direct shear strength tests. a) shear stress τ vs. shear displacement S_H; b) horizontal S_H vs. vertical displacement S_V

2 mm/min up to failure. Vertical compression S_V of the sample under stress and its variation due to shear are represented in Fig. 14b.

Test (2) differs from test (1) by having the soil sample consolidated under normal stress σ until the rate of vertical deformation decreased to $S_V = 0.01$ mm/h. Thereafter the shear stress is increased gradually, at an average rate of $V - (0.8$ to $1.8) \times 10^{-3}$ mm/min. Also these test results (S_H, τ and S_H, S_V) are shown in Fig. 14.

These two tests significantly differ by shear stresses referred to some constant S_H value.

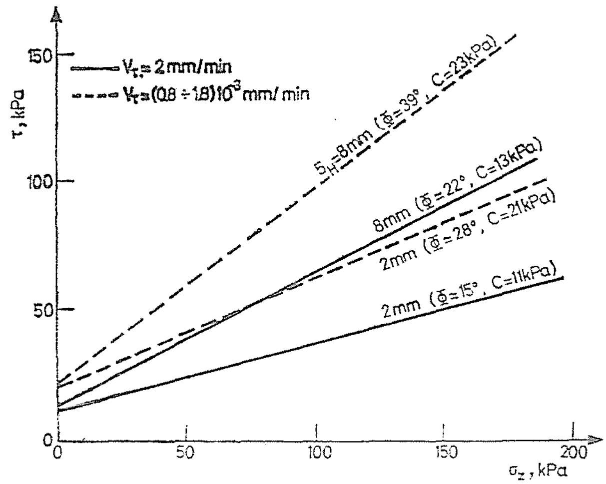

Fig. 15. Direct shear test results

For comparison of the two types of tests, the shear strength envelopes for shear displacements of $S_H = 2$ mm and $S_H = 8$ mm have been plotted in Fig. 15. The test results clearly point to the favourable effect of compaction to increase the shear strength. On the other hand, caution is necessary not to increase the displacement due to shear forces in soft, organic soils beyond an adequate safety margin referred to the critical state.

Of course, this safe limit also depends on time, for loading can only be increased as long as the creep of soil under a given load tends to diminish within a scheduled period of time, otherwise the situation may again turn critical after a while (ŠUKLJE, 1969; MITCHELL, 1976).

Using the stress/strain curves in Fig. 14, shear strength parameters Ø and c have been calculated for constant shear displacements S_H: 2, 4, 6 and 8 mm, and plotted in Fig. 12 (lines D and E). Note that a relatively large shear displacement is required to mobilize a certain level of shear strength (see points at displacements of 2, 4, 6 and 8 mm on lines D and E).

These tests demonstrated the possibility to increase sufficiently the shear strength of soft, saturated soils by significant compression under a sustained load, provided it is increased in such a way as not to produce at the same time critical state (failure).

The laboratory tests have been supplemented by field investigations. The in-situ shear strength of the soft foundation layer has been determined by field vane tests prior to, and during, the subsequent stages of construction, thereby providing a check on the predicted effect of preloading.

Two types of vane borers have been applied (Fig. 16). The conventional Swedish vane borer is known to have proven for determining the shear strength

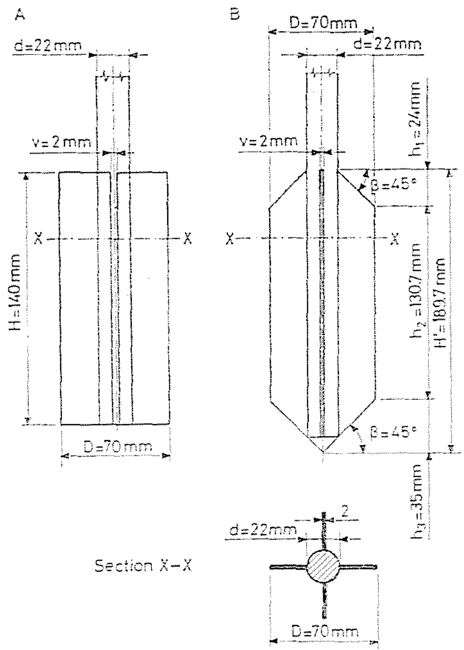

Fig. 16. Types of vane borers applied in field tests

of soft materials, and also the experiences in Hungary were favourable (Carlson, 1948; Kézdi, 1976). (See vane borer type A in Fig. 16).

The application of these shear vanes in fibrous materials is likely to be difficult and to entrain disturbances of soil structures. This prompted the development of special bicurved vanes for testing peats (Helenelund, 1975). In our tests, the simplified shear vane type B in Fig. 16 has also been applied (Kabai–Lazányi, 1978, 1980).

The vane borer test corresponds to an undrained shear test with no volume change on a soil sample consolidated in anisotropic stress state. Thus, the stress circle given by point 0 in Fig. 2 represents the state at rest before the shear, while point A_0 corresponds to rapid shear with no volume change ($\Delta V = 0$).

From the equilibrium between moments of external and internal forces acting on the vane borer, the shear strength of the soil is:

$$\tau = KM = K'M$$

where, with notations in Fig. 16,

M = torque on the shaft of the vane borer;

$$K = \frac{6}{7\pi D^3},$$

$$K' = \frac{1}{\pi\left[\frac{D^3}{6\cos\beta} - \frac{d^3}{12\cos\beta} + \frac{D^2 h_2}{2}\right]}.$$

K and K' refer to vane types A and B, resp., in Fig. 16 (see CARLSON, 1948; KABAI—LAZÁNYI, 1978, 1980).

On the site of investigation, at a depth of 4 to 5 m under the ground level, the foundation of the road embankment was found to consist of very loose, soft, peaty clay with vegetable fibres.

The purpose of the shear vane tests was to establish

1. the variation of the shear stress measured by either of the two types of shear vanes;

2. the increase in compressive strength of the peat due to compression under the increasing load of the road embankment.

The relationship between shear stress τ and rotation α for the two types of shear vanes has been plotted in Fig. 17.

The peak and final shear strength values τ_{max} and τ_V measured by the two different types of vane before the construction of the embankment are shown in Fig. 18.

Results from the two kinds of vane tests are in very close agreement.

Final shear strength values τ_V vs. depth lie along the same curve. Also the peak shear strength values τ_{max} scatter within the same range.

Thus, no error attributable to the type of the shear vane has been found. Practically, the two models are equivalent when used in the same soil type.

At the same time, the modified device B is superior by its ease of being driven into the ground and by its lower resistance to pulling out, whereas the vanes of the original probe have often broken off in the same ground.

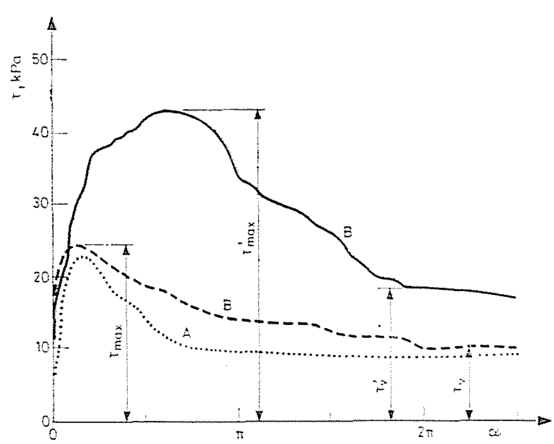

Fig. 17. Relationship between shear strength and rotation

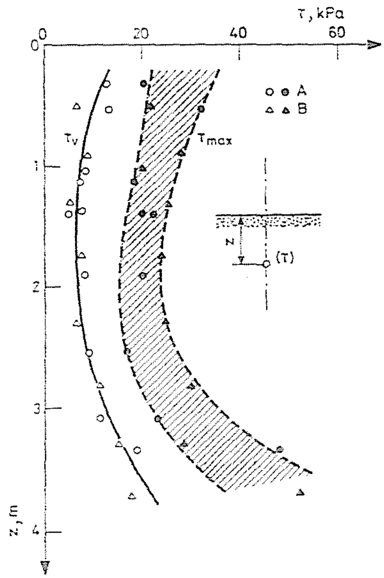

Fig. 18. Shear strength prior to construction of the embankment vs. depth

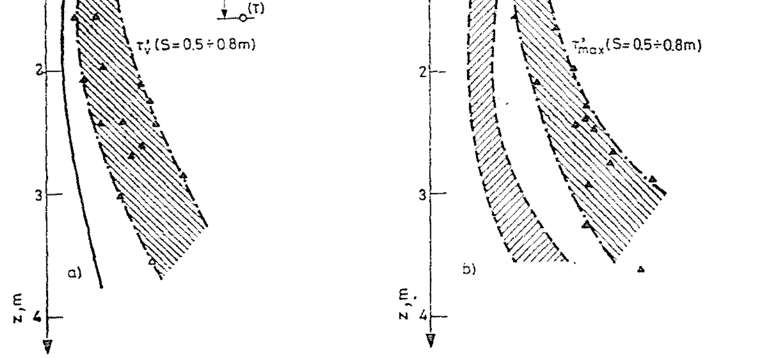

Fig. 19. Variation of shear strength prior to, and after, construction (τ_{max}, τ_v and τ'_{max}, τ'_v, respectively). *a)* Final shear strength values τ_v, τ'_v; *b)* peak shear strength values τ_{max}, τ'_{max}

The modified vane borer was also applied for checking the increase in shear strength due to the embankment construction. A typical shear diagram is represented in Fig. 17 together with the peak and final values τ'_{max} and τ'_v, resp., of the shear strength.

At the embankment section tested, the settlement under load amounted to $s \simeq 0.5$ to 0.8 m, much affecting the shear strength. The results are seen in Figs. 19a and 19b, showing the values of τ_v and τ'_v, and of τ_{max} and τ'_{max}, respectively, together with their ranges of scatter.

Thus, field tests made with the vane borer justified at the same time the suitability of this method for a rapid check of earthwork construction as well as for shear strength comparisons. Also the effect of short-time preloading on the soft subsoil may readily be checked. Its application permits an efficient control of the construction rate of earthworks (embankments) under critical conditions.

In stability tests, the obtained shear strength values have to be reduced in dependence on the plasticity index of the soil (BJERRUM, 1972; KÉZDI, 1976).

Summary

Physical properties of soft, peaty, very compressible organic soils can highly be improved by gradual, controlled preloading during construction.

Laboratory tests furnish essential and reliable information for the preliminary design through strength tests. Determination of the extreme τ_u values from the expected failure pattern under the embankment has, however, to be checked by field measurements.

The important compression due to secondary time effect is likely to be controlled by a linear law, as evidenced by the described measurements, generalization requires, however, further analyses.

Measurement of settlement, pore-water pressure, shear strength, lateral displacement and heave is crucial for the step-wise loading and field-controlled construction.

In soft, organic, peaty soils, the shear vane may be a simple and efficient tool for determining the strength increase upon preloading. The obtained shear strength values have, however, to be reduced. The measuring instrument is outstanding by its simplicity and by the possibility of furnishing prompt and directly comparable results.

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Senior Assistant Dr. Imre KABAI
Associate Prof. István LAZÁNYI
H-1521, Budapest

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