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PENETRATION TESTS ON SOILS

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Wright-Patterson Air Force Base, Ohio

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PENETRATION TESTS ON SOILS

by

V. F. Razorenov

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All figures, graphs, tables, equations, etc. merged into this translation were extracted from the best quality copy available.

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U. S. BOARD ON GEOGRAPHIC NAMES TRANSLITERATION SYSTEM

Block	Italic	Transliteration
А а	*А а*	A, a
Б б	*Б б*	B, b
В в	*В в*	V, v
Г г	*Г г*	G, g
Д д	*Д д*	D, d
Е е	*Е е*	Ye, ye; E, e*
Ж ж	*Ж ж*	Zh, zh
З з	*З з*	Z, z
И и	*И и*	I, i
Й й	*Й й*	Y, y
К к	*К к*	K, k
Л л	*Л л*	L, l
М м	*М м*	M, m
Н н	*Н н*	N, n
О о	*О о*	O, o
П п	*П п*	P, p
Р р	*Р р*	R, r
С с	*С с*	S, s
Т т	*Т т*	T, t
У у	*У у*	U, u
Ф ф	*Ф ф*	F, f
Х х	*Х х*	Kh, kh
Ц ц	*Ц ц*	Ts, ts
Ч ч	*Ч ч*	Ch, ch
Ш ш	*Ш ш*	Sh, sh
Щ щ	*Щ щ*	Shch, shch
Ъ ъ	*Ъ ъ*	"
Ы ы	*Ы ы*	Y, y
Ь ь	*Ь ь*	'
Э э	*Э э*	E, e
Ю ю	*Ю ю*	Yu, yu
Я я	*Я я*	Ya, ya

* ye initially, after vowels, and after ъ, ь; e elsewhere. When written as ё in Russian, transliterate as yё or ё. The use of diacritical marks is preferred, but such marks may be omitted when expediency dictates.

FOLLOWING ARE THE CORRESPONDING RUSSIAN AND ENGLISH DESIGNATIONS OF THE TRIGONOMETRIC FUNCTIONS

Russian	English
sin	sin
cos	cos
tg	tan
ctg	cot
sec	sec
cosec	csc
sh	sinh
ch	cosh
th	tanh
cth	coth
sch	sech
csch	csch
arc sin	$\sin^{-1}$
arc cos	$\cos^{-1}$
arc tg	$\tan^{-1}$
arc ctg	$\cot^{-1}$
arc sec	$\sec^{-1}$
arc cosec	$\csc^{-1}$
arc sh	$\sinh^{-1}$
arc ch	$\cosh^{-1}$
arc th	$\tanh^{-1}$
arc cth	$\coth^{-1}$
arc sch	sech^{-1}
arc csch	csch^{-1}
rot	curl
lg	log

Set forth in this book are the theoretical principles and the technological features of the study of the physicomechanical conditions of soils by the methods of penetration, sounding and vane shear. The proposed methods can be successfully used in conjunction with geological engineering investigations having in mind the design and structure of various engineering projects.

The recommended methods of penetration tests provide a high degree of accuracy in determining the indexes of the physical properties of soils and offer considerable objectivity in checking the accuracy and reliability of the obtained results. The proposals in the book are based on the methods of penetration for determining the consistency and limits of plasticity of cohesive soils, the angle of internal friction of noncohesive soils, and are also based on of a method of combined tests by penetration and vane shear in order to disclose the indexes of friction and cohesiveness of cohesive soils.

The methods of static and dynamic sounding of soils are presented in the book in less detail and relate, mainly, to the study of the surface layers of the soil using a penetrator point with a diameter considerably larger than the diameter of the probe.

In the book considerable attention is allotted to the application of methods of penetration and vane shear in the investigation of the correlation between the physical and mechanical properties of soils; numerous examples are given of the successful use of the proposed design diagrams for revealing such a correlation in cohesive soils having either disturbed and undisturbed structure. The recommended design diagrams turn out to be convenient for the normalization of the resistances of earth foundations.

The book is illustrated with numerous examples of soil test data by the methods of penetration, sounding and vane shear and it is intended for scientists, design and planning engineers and builders, and also for engineers and technicians, who deal with research on soil properties.

Tables - 14, illustrations - 76, bibliographies - 74.

PREFACE

In recent years in engineering geology practice relatively new methods have been extensively used in the study of the physicomechanical properties of cohesive and noncohesive soils by the penetration, sounding and vane shear. The comprehensive use of the three methods in the study of soils under natural conditions, supplementary research on soils by the methods of penetration and vane shear under laboratory conditions, and the rational combination of the new methods with the traditional methods of soil testing, primarily results in a considerable reduction in the time, cost and labor expense of the investigations; secondarily it will insure greater accuracy, reliability and objectivity in determining the number of the quantitative characteristics of the physicomechanical properties of the soil and finally, it will make it possible to organize an effective and objective checking of all steps in the engineering geology surveys, and a checking of the accuracy and reliability of the revealed characteristics.

This work consists of an experiment of the systematic presentation of the basic results of conducted theoretical and experimental research in developing methods for the determination of the quantitative characteristics of the physicomechanical properties of soils by penetration, sounding and vane shear. Laid down as the basis of the research are investigations covering many years, the features of new methods, conducted under the guidance of the author at the All-Union Research Institute of Hydrogeology and Engineering geology (VSEGINGEO), and under the guidance of Prof. Ye. V. Platonov and the author at the Poltava Construction Engineering Institute.

These investigations were made in close collaboration and according to a coordinated program, with the active and direct participation of the engineers: G. V. Zhornik, V. G. Zabara, N. L. Zotsenko, G. M. Pankratovo, I. N. Skrylya, V. G. Khilobok and V. D. Shitov. Experimental investigations on checking individual positions and proposed recommendations were made on various soils in Kherson, Poltava, Moscow, Yaroslavl, Novgorod, Leningrad and Crimea oblasts and in the Karelian ASSR.

Also used in the book are the results of theoretical and experimental research of Soviet scientists in developing new methods for soil tests, and some material was drawn from foreign specialists.

The author expresses his deep gratitude to Prof. Ye. V. Platonov and to all participants engaged in this work for their continuous assistance and attention, which was reflected during the preparation of this book, and the author is also endebted to the reviewer, candidate of science and technology L. N. Vorobkov, for a number of valuable councils, taken into consideration during the editing of the manuscript.

INTRODUCTION

The research methods on the physical and mechanical properties of soils by penetration, sounding and vane shear in the USSR have undergone especially intense development in the last 10-15 years.

At the present time no less than 20 scientific research and project-prospecting institutes of the Soviet Union are actively participating in the development of procedures and technology for field and laboratory methods of soil testing by penetration, static and dynamic sounding, vane shear, in the study of methods for interpretating the obtained results, in designing new instruments, installations and equipment.

The method of penetration tests, in its contemporary development, is distinguished by the following features.

1. In connection with the study of soils the method of penetration tests for depth and for a lower height of the conical penetrator, does not have limitations and it easily overlaps the range of change in the mechanical properties running from fine silt to rock fragments.

2. Penetration instruments, the types and dimensions of penetration points, naturally, have defined limits of application. Conventional instruments are calibrated, for example, for investigating soils with characteristics of mechanical properties (resistivity to penetration) ranging from 0.02-0.05 to 6.5-15 kg/cm^2, i.e., they are intended for use in soils with liquid to solid consistencies, inclusively.

3. The results of penetration tests are the objective characteristics of the mechanical properties of the soils being investigated and therefore, within certain limits, they do not depend on the force of penetration, but taking into account the constants of penetrator points - on the expansion angle of the employed conical penetrator points.

4. The principle of the invariance of the results of penetration tests provides the possibility for objective checking on the accuracy and reliability of the determination of the penetration characteristics of the mechanical properties of soils.

5. The results of penetration tests represent the peculiar resistance characteristics of soils subject to shearing. Thus, among the results of penetration tests, for example, using conical and ball penetrators and among the characteristics of friction and cohesiveness of cohesive soils, simple functional dependences exist.

6. Combined tests of cohesive soils by methods of penetration and vane shear in many instances make it possible completely simply and to sufficiently specify the angle of internal friction and the specific cohesion of cohesive soils.

7. From the results of penetration tests the angle of internal friction of noncohesive soils is determined directly.

8. Penetration method in soil testing is characterized by a high volume of samples and an increased accuracy of the results obtained. Thus, the method of penetration can be successfully used for the mass determination of the characteristics of the physicomechanical properties of the soil under field as well as laboratory conditions.

9. The sufficiently high accuracy of penetration tests makes it possible to establish comparatively simply a correlation or even functional interrelationships between the characteristics of the penetration and the physical properties of soils. In cohesive soils with disturbed structure the disclosure of such interrelationships is available at any commerical soil mechanics laboratory.

10. The disclosure of the noted interrelationship makes it possible to solve simply and correctly the other actual problem in establishing the interrelationship between the characteristics of friction and cohesion and the characteristics of the physical state of cohesive soils.

11. According to the results of penetration tests, without any supplementary definitions, the consistence of cohesive soils and the degree of the density of noncohesive soils can be established.

These and many other important features of the method of penetration tests are examined in this book.

The development of the method of penetration tests in the Soviet Union became especially intensive at the end of the 40's after the publication of the works of Acad. P. A. Rehbinder, who showed the effectiveness of the application of penetration for investigating the structural and mechanical properties of dispersed systems. As a result of these studies P. A. Rehbinder formulated the principle of invariance of the results of penetration tests, which at the present time can be considered as checked under the various soil conditions.

During this period as a result of A. M. Vasiliev's studies a new standard, No. 5184-49, was introduce' for determining the boundary of yield by the method of pe tration with the aid of a bob cone and P. O. Boychenko's first works were published based on the methods of penetration determination of the consistence of cohesive soils.

At this time Prof. N. A. Tsytovich's known investigations dealt with the determination of the cohesion of viscous (virtually noncompacted) cohesive soils by the method of the ball bearing probe. Thus, already in the beginning of the 1950's in the USSR a set of procedures was successfully applied according to penetration determination of the physicomechanical conditions of the soil.

Further theoretical and experimental research for the substantiation of the method of penetration tests and the determination of the features of its application in various soils were conducted by N. N. Agranat and Prof. M. P. Volarovich Prof. V. G. Berezantsev, Doctor of Geological and Mineralogical Sciences I. M. Gor'kov, Prof. S. P. Nichiporenko, Prof. A. A. Nichiporovich, Prof. F. D. Ovcharenko, Prof. M. F. Shirokov and many other Soviet specialists.

In 1955 Prof. V. G. Berezantsev established a very simple, and apparently, a sufficiently precise functional dependence between the results of penetration tests and the characteristics of the friction and cohesion of a cohesive medium. These analyses served as a basis for further proposals according to the methods for the combined testing of cohesive soils.

In the 1960's penetration tests began to be successfully used for determining:

a) the mechanical properties of various construction materials, including ceramics and glass;

b) the modulus of the total deformation of soils, relative subsidence of loess, the angle of internal friction and the degree of compactions of sands;

c) the correlation and functional dependences between the physicomechanical conditions of the soil.

Application of the mechanized penetration equipment was designed and made using cone penetrators up to 500 mm high (All-Union Scientific Research Institute of Geology-VSEGINGEO) and spherical punches up to 1000 mm in diameter (the former Academy of Construction and Architecture-Belorussian Soviet Socialist Republic b. ASiA BSSR) and laboratory penetration soil probes under conditions of uniaxial and triaxial compression (Poltava Civil Engineering Institute).

Prototypes of various field and laboratory penetrometers were used.

Up to 1953-1955 the analyses of the hydroelectric power project (V. A. Durante) involved the application of methods of static and dynamic sounding during engineering geological investigations. In this case for accurate readings of the soil resistance at the point of the cone penetrator, excluding the frictional forces, which act along the lateral surface of the rods, a string dynamometer was used.

Research institutes for foundations and underground structures (S. A. Shashkov) almost simultaneously conducted comprehensive experimental research on dynamic sounding in sandy soils.

In 1960-1965 various mechanized equipment was devised for static sounding to depths of 10-20 and even 30 m, at the institutes of VSEGINGEO and the TsKB, Gosgeolkom (Central Design Office, State Geological Commission), Fundamentproyekt (State Institute for the Planning of Foundations and Substructures), TsNIIS (All-Union Scientific Research Institute of Transportation Construction, Ministry of Transportation Construction) Mintransstroy, DIIT (Dnepropetrovsk Institute of Railroad Transportation Engineers), Bashkir Scientific Research Institute for construction and other organizations. The important and interesting feature of some of this equipment is the application of a complex of penetration-well logging methods, i.e., apart from the soil resistance at the point of cone penetrator the specific weight and soil moisture are simultaneously determined, within the limits of the depth of sounding, by the method of radioactive isotopes.

In these devices probing depths up to 10-13 m are developed, and therefore, the devices themselves, by necessity, are quite complex and bulky. For probes to a depth of several meters light weight structures have been developed.

The conducted research works and the accumulated experience on the application of methods of static and dynamic sounding make it possible to confirm the high effectiveness of these methods. In comparison with the conventional drilling operations, in many instances, a considerably higher accuracy in the analysis of the sections is obtained with a noticeable savings in labor and materials.

Currently underway are intense research programs on the use of sounding methods for the direct determination of the strength and deformation characteristics of soils, the determination of their bearing capacity etc. There is no doubt that the successful and comprehensive solution of these tasks is a matter of time. One of the trends in the procedure for solving the assigned task amounts to a means of increasing the diameter of the cone penetrator in comparison with the diameter of the rod in order to create conditions, under which the insertion of the probe would be accompanied by the forcing of the soil into the forming cavity rather than by compaction. In this case the characteristics of the penetration and sounding tests will approximately agree which offers the possibility to make use of the solutions of the method of penetration for the analysis, generalization and interpretation of the results of static and dynamic sounding.

Dedicated to the research on the features of the method of vane shear are the works of the Moscow Highway Institute (MADI) (S. I. Rokas), DIIT (O. M. Reznikov), TsNIIS, Mintransstroy (M. K. Druzhinin and A. M. Gorelik), VSEGINGEO (V. D. Shitov), Poltava Construction Engineering Institute: (ISI (G. V. Zhornik), Leningrad Academy im. Mozhay (P. I. Eyzler) and other organizations.

As a result of these analyses made in recent years, original designs for field and laboratory soil testing equipment by vane shear and for combined soil tests by penetration and vane shear have been proposed.

The materials from these analyses, supplemented by the works of foreign specialists, indicate the objectivity and accuracy of vane shear, towards the development of the invariance of the obtained characteristics during the testing of plastic soils, towards the possibility, under specific conditions, to determine resistivity to vane shear with a specific cohesion of the soil, etc.

The proposed work is an attempt to generalize the available material according to the experimental and analytical investigations in the area of the new methods for determining the quantitative characteristics of the physicomechanical conditions of the soil. Within the scope of this book greater attention is allotted to the analysis of the features of the methods of penetration and vane shear. This is also logical, since the intense development of the methods of static and dynamic sounding, the complex use of well logging methods and the already obtained recommendations urgently require a special and more detailed presentation.

The author realizes the possibility for individual errors and deficiencies in the proposed experiment of a brief systematic presentation of the new methods of research on the physicomechanical conditions of the soil, and therefore, the author has requested that those organizations and those individuals using these methods, send their comments, proposals and advice on the further improvement and development of methods of penetration, sounding and vane shear. Letters should be directed to: Moscow, Zh-17, B. Ordynk, 32, All-Union Scientific Research Institute of Hydrogeology and Engineering geology (IVSEGINGEO).

CHAPTER I

THE PROCEDURE FOR PENETRATION SOIL TESTS

Determination by the penetration method. Penetration is defined as the method of studying the physical and mechanical properties of soils by determining the resistance of soils to the penetration of points of various forms and sizes. In the case where the depth of indentation of a point does not exceed its height, it involves a further account of intrinsic penetration. If the depth of indentation of a point exceeds its height, then it is proper to refer to the sounding of soils.

The proposed distinction in the terminology reflects the fundamental difference in graphs of the "force of penetration (sounding) P - the depth of indentation of point h," which characterizes the conditions of the deformation of soil during the process of penetration or soundings (Fig. 1a and b).

During the process of penetration tests the cross-sectional area of the cone penetrator during its entry into the soil continuously increases. Thus far the depth of indentation of the point will be less than its height, $h < h_{кон}$ the graph of the dependence $h = f(P)$ represents, for example in cohesive soils, the square of the parabola (Fig. 1a) by the equation:

$$P = Rh^2 + P_0 \text{ кг}, \qquad (1)$$

where R and P_0 - parameters of the equation.

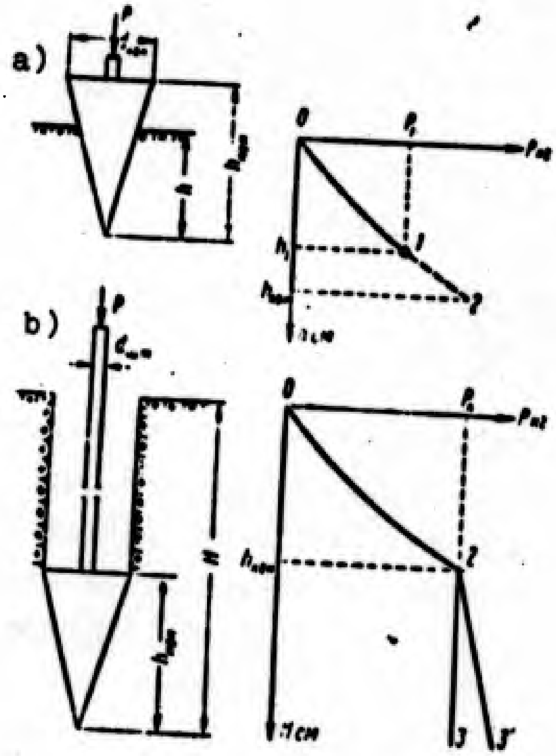

Fig. 1. Diagrams and typical graphic representations of the results of soil data by the methods of penetration and sounding. a) penetration; b) sounding.

With depth of indentation of a point which exceeds its height $h > h_{кон}$ the cross-sectional area of the point remains constant; therefore, a further increase in the force of sounding in cohesive soils turns out to be infeasible. The graph of the dependence $h = f(P)$ has a sharp break (Fig. 1b). Instead of a parabolic dependence (1) a simple dependence $P = P_{макс}$, is established, i.e., the force of sounding in uniform cohesive soils exhibiting considerable cohesion apparently, does not depend on the depth of indentation of the point (straight line 2-3 in Fig. 1b).

In noncohesive (sandy) soils with uniform characteristics of mechanical properties throughout the depth of sounding, the conditions $h > h_{кон}$ and $d_{кон} >> d_{шт}$ (diameter of cone $d_{кон}$ is considerably greater than the diameter of shaft $d_{шт}$) lead to a linear magnification in the force of sounding along with the depth of indentation of the point (straight line 2-3' in Fig. 1b).

Under the conditions of penetration tests the physical process of the failure of cohesive soils to a considerable degree depends on their consistence. Thus, for instance, during a change in the moisture content a water-saturated clay soil from the liquid limit to the shrinkage limit the physical process of failure in the soil essentially changes two times.

Approximately in the range of the plastic state of the soil, i.e., in the interval where the moisture content at the liquid limit W_f to the moisture content at the plastic limit W_p (actually this range is substantially higher and it is characterized by the values of the resistivity of penetration R approximately from 0.05-0.1 to 5-10 kg/cm^2), a sufficiently distinct displacement of a certain amount of soil to the surface is observed during the entry of cone penetrator.

According to the studies of a number of authors, at moisture contents higher than W_f during the entry of the indentor the displacement of the soil no longer occurs, but a migration of a thin layer of soil adjacent to the indentor part of the cone is observed along the forming cone impression [32, 33, 34, 43].

At moisture contents less than a certain critical value, which is locat d in the interval between the plastic and shrinkage limits, the entry of coniform indentor into soil is frequently accompanied by the phenomena of disturbance of the continuity of the surface layer of soil.

Methods for expressing the results of penetration soil tests. The selection of the generalized characteristics of the results of penetration tests is conducted on the basis of solving the axisymmetric problem of the theory of limiting equilibrium [3, 4].

As is known, in the absence of a surcharge weight, i.e., with the position of circular punch on the surface of the soil, the maximum strength of the foundations σ is determined from the binomial formula

$$\sigma = U_{T}\Delta r + N_{T}c \ \text{т/м}^{2}, \qquad (2)$$

where U_T and N_T - dimensionless "coefficients of the bearing capacity of the soil" [51], which depend only on the angle of internal friction of the soil $\phi°$; c - specific cohesion of the soil in t/m^2; Δ - specific weight of the soil in t/m^3; r - radius of the circular punch in m.

The variations of the formulas for calculating the maximum strength of earth foundations are connected with the form of the functional dependences between the coefficients U_T and N_T and the angle of internal friction of the soil $\phi°$.

Prof. V. G. Berezantsev's solution (1958) was obtained from the condition that in the state, which immediately precedes the shear of the soil over the sliding surface, the configuration of the compacted condensed core under the punch remains strictly constant. The compacted core on the basis of the experimental data is accepted in the form of a cone with an angle of expansion α, equal to 90° [4]. Thus, the tabular values of the coefficients U_T and N_T, calculated by Prof. V. G. Berezantsev for the angles of internal friction from 16 to 42°, can be used for an analysis of the results of penetration tests with a conical point $\alpha = 90°$.

The experimental check on the reasoning for a possible application for solving axisymmetrical problems of the theory of limiting equilibrium when processing the results of static sounding was performed for the first time by V. A. Yaroshenko [57]. For this purpose comparative tests were conducted on air-dried and slightly moist sands using cylindrical, flat ($\alpha = 180°$) and conical ($\alpha = 60°$) punches, 3.56 cm in diameter. The sounding reached a depth down to 150 cm. The degree of convergence of the test data was estimated with the aid of the ratio of maximum strengths $\sigma_{кон}/\sigma_{штамп}$. The absolute values of resistances to penetration by the probe varied with depth and in different tests from 10 of up to 239 kg/cm^2.

On the average for the ratio of $\sigma_{кон}/\sigma_{штамп}$ 0.983 was obtained i.e., the test data completely coincided.

The comparative field tests on cohesive soils using punches and conical points with an angle of expansion of 30°, were completed into 1966 under the guidance of the author. One hundred and twenty-eight soundings were made in water-saturated silty clays to depths of 1.8-2.2 m.

On the average for 53 determinations the sounding probes, obtained within the range of the corresponding uniform layers during the penetration of conical points and punches, turned out to be identical, but their ratio $P_{кон}/P_{штамп}$ was 1.017 with an accuracy factor of the tests at 2.4% and a coefficient of linear correlation at 0.98.

Thus, the conditions soil of displacement, and then according to the probe depth and soil compaction as a result of the wedging action of conical point or the compacted core, which is formed flat punch, turned out to be completely identical.

Consequently, the known solutions of the axisymmetric problem of the theory of limiting equilibrium can, in fact, be used for the analysis of the results of penetration and sounding.

For this purpose the mean pressure at the bottom of the circular punch is expressed as a value of the applied force $\sigma = \frac{P}{\pi r^2}$, and the radius of the punch r on the basis of the simplest geometric considerations is linked to the height of the compacted core under the punch, or, that it is linked to the penetrating depth of the conical point h, i.e., $r = h \, tg \, \alpha/2$. (In a particular case with $\alpha = 90°$ $r = h$).

After substitution and small conversions we will have

$$P = U_0 \Delta h^3 + N_0 c h^2 \; \tau, \tag{3}$$

where $U_0 = U_T \pi \, tg^3 \alpha/2$ and $N_0 = N_T \pi \, tg^2 \alpha/2$. In connection with Prof. V. G. Berezantsev's solution $U_0 = U_T \pi$ and $N_T \pi$.

Equation (3) already turns out to be completely convenient for the analysis of results of penetration tests on clayey and sandy soils.

<u>Case I.</u> **The angle of internal friction approaching zero** ($\phi \to 0$). In this case the coefficient U_0 is also approaching zero, since $U_T \approx 0$, while coefficient N_0 represents a constant value, not equal to zero.

Equation (3) can be simplified and converted into the form

$$P = N_0 c h^2 \; \tau. \tag{4}$$

If we accept the value of the "resistivity of penetration R," as an objective characteristic of the results of penetration tests, equal to the ratio of the force of penetration P, to the square of the depth of indentation of the conical point h^2, then equation (4) acquires the following form:

$$R = \frac{P}{h^2} = N_0 c \text{ T/m}^2. \tag{5}$$

For the investigated state of the soil N_0 = const., the specific cohesion c = const., and therefore even R = const. Consequently, in an uniform clayey soil the resistivity of penetration possesses the property of invariance, i.e., it does not depend on the value of the force of penetration [43]. With an increase in the force of penetration the depth of indentation of the point will, of course, increase; however, the ratio of the force of penetration to the square of the corresponding depth of indentation of the point remains constant.

The taken condition, $\phi \to 0$, as is known, is observed, in the first place, in tests on bentonite and, in the second place, on clayey (cohesive) soils with a liquid and liquid-plastic consistence.

<u>Case II</u>. **Angle of internal friction greater than 10-15°.** Under these conditions the binomial formula (3) with the introduction of the concept of the resistivity of penetration turns out to be equal to

$$R = U_0 \Delta h + N_0 c \text{ T/m}^2. \tag{6}$$

It is easy to see that in this case the greater or lesser degree of R depends on the ratio $U_0\Delta h/R$, i.e., it depends first of all on the value of specific cohesion of the soil and the depth of indentation of the point.

According to Table 13 the construction norms and regulation (SNiP), II-B.1-62, on the standard characteristics of the angles of internal friction and specific cohesion, taking into account the values of coefficients U_T and N_T, obtained by V. G. Berezantsev [4], it is not difficult to calculate the variability of the ratio $U_0 \Delta h/R$ for different types and states of clay soil.

According to SNiP, the standard values of the angles of internal friction vary from 15 to 25°, the value of specific cohesion - from 9.4 to 0.6 t/m^2, the specific weights of the soils in a state, close to field capacity - from 1.85 to 2.16 t/m^3. In this case the ratio of the coefficient N_0/U_0 is equal to 3.0-2.0. For penetration soil tests under laboratory conditions the maximum indentation depth of the conical point can be accepted as 0.050 m.

Under the indicated boundary conditions the calculations of the ratio $U_0 \Delta h/R$ revealed the following:

a) for heavy clay loams ($w_p \geq 18.5\%$), and for those which are more clayey the ratio $U_0 \Delta h/R$ have fractions of several percent more;

b) for sandy loams and loams ($12.5 \leq w_p < 18.5\%$) the ratio $U_0 \Delta h/R$ changes from fractions of one percentage to 3%;

c) for light sandy loams ($w_p < 12.5\%$) the ratio $U_0 \Delta h/R$ do not exceed 4%.

Consequently, the effect of the first term in equation (6) is completely unessential, and therefore, in all cases a very ideal invariance of the resistivity to penetration will be observed. Only in isolated cases in field penetration tests on light sandy loams at a depth more than 0.25 m will the ratio $U_0 \Delta h/R$ exceed 15%.

From the results of the conducted analysis the following conclusions can be drawn.

1. In cohesive soils of firm consistence the value of the specific cohesion increases considerably. Thus, under these conditions the invariance of the resistivity to penetration is exhibited most distinctly.

2. In cohesive soils with undisturbed structure, the effect of the structural cohesion as a general value of the resistance of soil is especially significant. Thus, the more pronounced the structural conditions of the soil, less the effect of the first term in the equation (6), and the more ideal the invariance is the specific resistance to penetration.

3. The value of the resistivity to penetration is a convenient tool for the characteristic results of penetration tests in the majority of cohesive soils under different conditions.

Case III. **Angle of internal friction greater than 30°, but the specific cohesion close to zero.** In dry and completely water-saturated sands the specific cohesion is equal or close to zero. In this case the binomial equation (3) can again be simplified as

$$P = U_0 \Delta h^3 \; T. \qquad (7)$$

Equation (7) determines the dependence of the mechanical properties of sands on the value of the natural pressure q = Δh. The resistivity to penetration R increases proportionally to the depth of indentation of the coniform point or, the same is proportional to the diameter of the circular punch

$$R = \frac{P}{h^2} = U_0 \Delta h \; T/M^2. \qquad (8)$$

It is obvious that the use of a variable of the resistivity to penetration in the calculations in certain cases is unsuitable Thus, in sandy soils as an objective characteristic of the results of penetration tests it is advantageous to accept the value of the "index of penetration U," equal to the ratio of the force of the penetration P to the cube of the depth of indentation of the conical point h^3:

$$U = \frac{P}{h^3} = U_0 \Delta \; т/м^3. \tag{9}$$

From formula (9) it follows that in uniform sands the index of penetration exhibits the property of invariance, i.e., just as in the ratio of the resistivity to penetration in cohesive soils, the results of tests do not appear to depend on the value of the forces of penetration.

On the right side of formula (9) is product U_0 times the specific weight of the sand Δ. If we introduce the dimensionless quantity of the generalized index of penetration $U_0 = \frac{U}{\Delta}$, into the examination then formula (9) is converted to the form

$$U_0 = \frac{P}{\Delta h^3}. \tag{10}$$

Thus, the generalized index of penetration U_0 is proportional to coefficient U_T in binomial equation (2) and therefore, it depends directly on the angle of internal friction of the sand.

Sands, and in particular, sandy loams, acquire a certain cohesion during wetting. Besides that, under conditions of an undisturbed structure these soils possess a specific structural cohesion. In similar transient states of soils when considerable angles of internal friction (>25°) and sufficiently noticeable values of specific cohesion (>0.1 t/m^2) are observed, a breakdown of the invariance for both R and U is possible.

Under these conditions the possibility for using the characteristics R and U, if necessary, can be checked by comparing the degree of convergence of results of experiments with the conditions of invariance of the specific resistance and the index of penetrations.

For this transient case, based on N. L. Zotsenko's proposal, it is convenient to make use of formula (6) and to plot the graph $R = f(h)$ at the coordinates of "the specific resistivity to penetrations R - the depth of indentation of conical point h." In this case the slope of the mean straight line to the vertical axis represents the index of penetration $U = U_0\Delta$, and the segment detached by the mean straight line on the axis of the abscissae is equal to the initial value of the specific resistance to penetration $R_0 = N_0c$.

The dependence between the results of penetration tests is at different points. The use of the proposed characteristics of resistivity to penetration and the index of penetration makes it possible to easily solve the problem of the possibility for using penetrometers of different designs. From the condition of invariance it follows that the values R and U do not depend principally on the accepted method of registration of the resistance to penetration.

It is possible to measure the depth of indentation of the point with a fixed weight using a movable system of the penetrometer or, on the contrary, to record the force of penetration using an assigned constant depth of indentation of the point, and in either case the value of the resistivity to penetration and the index of penetrations will be identical.

The values of the resistivity to penetration obtained by the different points using static penetrometers and or even strikers, are proportional. Consequently, with equally accurate measurements, and with some exceptions, the type of the indentor has no significance.

This important property of resistivity to penetration is the simple result of the proportional dependence between that proposed by Acad. P. A Rehbinder of the characteristic maximum shear stress $\tau_{\text{пред}}$, and the resistivity to penetration [43]

$$\tau_{\text{пред}} = \rho_{\alpha} R \ \text{kg/cm}^2, \qquad (11)$$

where ρ_{α} - represents the constant of the indentor depending, for example, with the conical points, only on the angle at the apex α.

In the following presentation let us designate the value of the resistivity to penetration obtained with the aid of any nonstandard points, by q:

$$q = \frac{P}{h^2} \ \text{kg/cm}^2. \qquad (12)$$

On the basis of dependence (11) the maximum stress to shear turns out to be equal to:

$$\tau_{\text{пред}} = \rho_{30} R = \rho_{\alpha} q \ \text{kg/cm}^2. \qquad (11a)$$

Consequently, the actual resistivity to penetration disclosed using different indentor, turn out to be proportional:

$$R = k_{\alpha} q \ \text{kg/cm}^2, \qquad (13)$$

where

$$k_{\alpha} = \frac{\rho_{\alpha}}{\rho_{30}}. \qquad (14)$$

It must be noted that even abroad in recent years the completely determined failure was projected from the traditional method of expressing the results of penetration tests through the depth of indentation by conical point h. Approximately after 10 years from the publication of P. A. Rehbinder's studies at the Swedish Hansbo Geotechnical Institute [64], and R. Karlsson [66, 67] first began the application of the new method of expressing the results of penetration tests through $\tau_f = k(P/h^2)$, whereupon the coefficient of proportionality k is acknowledged depending only on the apex angle of the cone. The value τ_f is linked with the resistance to weakened shear, and the proportionality factor k is determined experimentally by means of the parallel testing of soils by the method of vane shear.

In R. Karlsson's report to the Fifth International Congress on soil mechanics and foundations (No. 1/29) the results of penetration tests were determined with the aid of the value of "parametrical shear strength" $\tau_{\text{пар}} = P/h^2$, i.e., directly with the aid of the resistivity to penetration $R = \tau_{\text{пар}}$ [66]. In another report by H. Scherrer (No. 1/55), dedicated to the procedure of penetration determination of the liquid limit, the value of the "balanced pressure" $\sigma = (1/\pi)\cdot(P/h^2)$, proportional to the resistivity to penetration was used. In these studies a conical point with angle at the apex of 90° was used therefore, the lateral surface of the submerged part of the cone was equal to $S_{\text{бок}} = \pi h^2$ [72].

Thus, it was reported that even abroad the infeasibility of further application of the wanting physical sense "of penetrometric" measurements is gradually being realized; for example, the depth soundings using the insertion of the cone at this arbitrary load over an arbitrarily preset time" [P. A. Rehbinder, 1956], and as a result the Soviet specialists unanimously proposed analogous methods for estimating the results of penetration tests.

Coefficient of penetration. In many instances it turns out to be very convenient to express the results of penetration tests in the form of the dimensionless relative resistivity to penetration or in the form of the coefficient of penetration N, equal to the ratio of the resistivity for the investigated condition of the soil R to the resistivity of the consistency at the liquid limit of soil which is in the state of complete water saturation R_f

$$N = \frac{R}{R_f}. \tag{15}$$

The dependence (13), as such, is also valid for the liquid limit of the soil, i.e., $R_f = k_\alpha q_f$. In summation, the coefficients of penetration obtained using different points under one or another condition will be substantially identical:

$$N = \frac{R}{R_f} = \frac{q}{q_f}. \tag{16}$$

In this connection the imperative need becomes completely obvious for expressing the test data, plotted with the use of nonstandard indentors, with the aid of dimensionless values of the coefficients of penetration.

Literature and experimental data on the constants of conical indentor. The theoretical values of the proportionality factors between the maximum shear stress $\tau_{\text{пред}}$ and the specific resistance to penetration R were calculated by Academician P. A. Rehbinder [43], N. N. Agranat and Professor M. P. Volarovich [1], M. F. Shirokov, by V. G. Berezantsev [3] and several other authors.

According to P. A. Rehbinder, the maximum stress to shear with every moment of a sufficiently gradual insertion of the conical point is calculated as a component of the total load on indentor along the indented surface of the cone. The proposed constant of the standard coniform point $\rho_{30} = 1.108$, according to the conditions of his conclusion turns out to be valid only at sufficiently small values $\tau_{пред}$ (approximately up to 0.001-0.010 kg/cm^2), "i.e., under conditions of pronounced flow along the lateral surface of the cone during its insertion."

N. N. Agranat with M. P. Volarovich, and N. N. Agranat and with M. F. Shirokov on the basis of M. F. Shirokov's solution for the critical equilibrium of the medium under the coniform point, proposed sufficiently agreeing analytical and graphic methods for determining the maximum stress to shear [1].

Under the conditions for the development of the phenomena for pushing the soil during the insertion of the indentor, the value of the constants of the conical points ρ_α, according to N. N. Agranat and M. P. Volarovich, and the proportionality factors $k_\alpha = \frac{\rho_\alpha}{\rho_{30}}$ are presented in Table 1 [1].

Table 1. The constants ρ_α and the proportionality factors k_α for conical points.

Apex angle of the conical point $\alpha°$	Values based on N. N. Agranat and M. P. Volarovich		Proportionality factor k_α according to the condition of invariance of the force of penetration
	the constant of indentor ρ_α	proportionality factor $k_\alpha = \rho_\alpha/\rho_{30}$	
20°	2.07	2,16	2,309
30°	0,959	1	1
40°	0,533	0,557	0,542
45°	0,416	0,435	0,418
60°	0,214	0,224	0,215
80°	0,099	0,103	0,102
89°20′ — 90°	0,083	0,087	0,072

N. N. Agranat and M. F. Shirokov have noted that the value of the maximum stress to shear, calculated according to the results of penetration tests, with the constants of the indentors entered in Table 1 completely coincide with $\tau_{\text{пред}}$, obtained by other methods (with uniform shear between the coaxial cylinders and with cones or with a narrow clearance between two plates) for a number of dispersed systems within the limits from 0.005 to 0.100 kg/cm^2.

From the published material of comparative penetration tests it follows that the theoretical values of the coefficients ρ_α and k_α can also be used with considerably larger values of resistivity to penetration. In order to explain the degree of convergence of the calculated and experimental values of the coefficients of proportionality k_α let us make use of the results of studies, made at the Swedish Geotechnical Institute on the penetration of highly plastic clays with undisturbed structure [64].

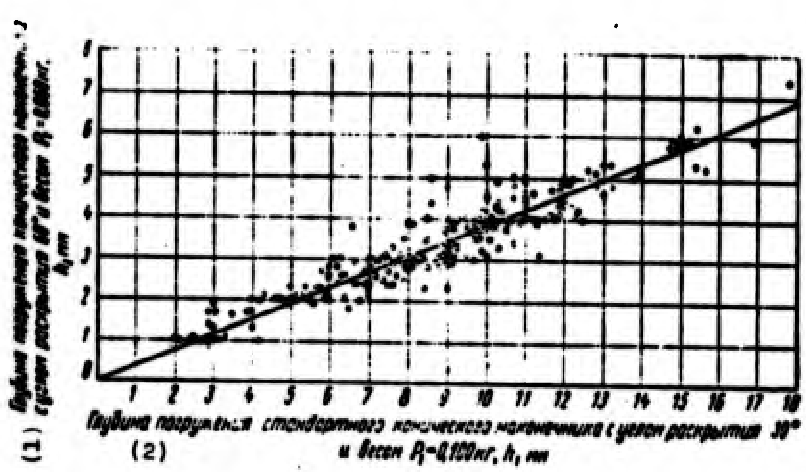

Fig. 2. The experimental graph of the proportional dependence between the results of penetration tests with conical points with angles of expansion at 30° and 60°. KEY: (1) Depth of indentation of conical point with a angle of expansion of 60° and a weight of P_2 = 0.060 kg, h_2 mm. (2) Depth of indentation of the standard conical point with an angle of expansion at 30°, and a weight of P_1 = 0.100 kg, h_1 mm.

Figure 2 shows more than two hundred comparative results of the measurement of the depth of indentation of two conical points with angles of the apex at 30 and 60° and with their weight, $P_1 = 0.10$ kg and $P_2 = 0.06$ kg respectively. Testings encompassed the entire range of the plastic condition of the soil with consistencies from the liquid state ($h_1 = 1.5$ cm, $R_{мин} = 0.044$ kg/cm^2) to the solid state ($h_1 = 0.2$ cm, $R_{макс} =$ $= 2.500$ kg/cm^2).

From the results of these tests it follows that, in the first place, the ratio $h_2/h_1 = \text{const}$, i.e., between the appropriate depth of indentation of two different points there is a proportional dependence, and, in the second place, the value of the proportionality factor is $h_2/h_1 = 0.384$.

Taking into account the dependences (11a) we will find that the proportionality factor k_α for a conical point with an angle of the 60° is equal to

$$k_\alpha = \left(\frac{h_2}{h_1}\right)^2 \frac{P_1}{P_2} = 0{,}384^2 \frac{0.10}{0.06} = 0{,}246,$$

which is in complete agreement with the result of the theoretical solution $k_\alpha = 0.224$, indicated in Table 1.

The determination of the proportionality factor k_α from the condition of invariance of the forces of penetration and sounding. All of the conducted experimental research in full view testifies that in uniform and water-saturated cohesive soils with an identical cross-sectional area of the submerged part of the conical point, the forces of penetration will be identical or close to it. Thus, under the condition $r = \text{const}$, the forces of penetration or soundings turn out to be invariant with respect to the angle of expansion of the conical point $P = \text{const}$. Thus, from formula (12) with the use of geometric ratios $r = h \operatorname{tg} \alpha/2$, it is easy to obtain the dependence for the precise determination of the proportionality factor k_α with respect to the conical point with the angle of expansion 30°

$$k_\alpha = \frac{\mathrm{tg}^3 15^\circ}{\mathrm{tg}^3 \frac{\alpha}{2}}. \tag{17}$$

From the data in Table 1 the full agreement of the values of the proportionality factors k_α, calculated according to N. N. Agranat and M. P. Volarovich [1] and according to formula (17) (the difference does not exceed 4-6%) is obvious. For a conical point with an angle of expansion $\alpha = 90°$ the value $k_\alpha = 0.072$ is more correct since the proportionality factor 0.087 relates to a smaller angle $\alpha = 80° 20'$.

During penetration tests of three-phase soils with conical points having an angle of expansion $\alpha \geq 60°$, the incidental phenomena of soil compaction and the formation of a ground cone frequently appear. Under these conditions the values of the proportionality factors k_α, given in Table 1, turn out to be uncertain.

Invariance of the resistivity to penetration. By invariance of the resistivity to penetration is meant the independence of the results of penetration tests in uniform soils on the value of the force of penetration. If we consider the ratio of the constants of the conical points, then the values of the resistivity to penetrations within the specified boundaries also turn out to be invariant with respect to the angle at the apex of the indentor.

The property of invariance of the resistivity to penetration discloses the exceptional possibilities for checking the accuracy and reliability of the results of penetration tests during the testing process itself. From the condition of invariance of the resistivity to penetration the independence of the obtained results directly ensues from the particular, technological features of the penetration tests, which makes it possible to insure the objective determination of the indexes of the mechanical properties of the investigated medium which, in turn, depend only on its grain size and state.

The invariance of the resistivity to penetration is exhibited in cohesive soils only with the observance of the following basic conditions:

a) with relative uniformity of the soil throughout the depth of penetration;

b) in the absence of a change in the initial physical condition of the soil during the process of penetration tests;

c) in the absence of technological disturbances or errors in tests.

The concept of invariance in the results of penetration tests was noted by Prof. N. N. Ivanov and P. P. Ponomarev back 1932 [25], and it was discussed by A. A. Inozemtsev in 1940 and was finally developed in the works of Acad. P. A. Rehbinder and his colleagues in 1948-1949 [43]. These investigations showed that the invariance of the maximum stress to shear can be distinctly observed in soils in a liquid consistence with changes in loads at a ratio of 1:100 and even 1:1000, and with a change in the apex angle of the conical point from 45 to 90°.

The study of the invariance of the values of the maximum stress to shear was examined in works of P. A. Rehbinder and N. A. Semenenko for dispered systems with a $\tau_{пред}$ about 0.001 kg/cm^2 [43], in the works of N. N. Agranat and M. P. Volarovich for peats with values of $\tau_{пред}$ up to 0.1 kg/cm^2 [1], in the works of

A. M. Vasiliev and Prof. S. P. Nichiporenko for clayey soils in the plastic state with values of $\tau_{\text{пред}}$ from 0.25 to 5.3 kg/cm^2 [9, 32].

The most detailed studies of water-saturated Spondyl clay with disturbed structure and with 36% moisture content were conducted by S. P. Nichiporenko using three conical points with an apex angle of 30, 45 and 60°. Penetration tests were carried out under the action of four stages of loading within the interval of 0.2-1.8 kg. The obtained results confirm the full validity of the conclusion of S. P. Nichiporenko about the fact that "with different cones and variable loads virtually coinciding values of the plastic strength are obtained $\tau_{\text{пред}}$" [32].

Sufficiently thorough and precise penetration tests of slightly dried samples of Odintsovskiy and Novobakhmutskiy clays with disturbed structure were made by I. A. Al'perovich, P. N. Berenshteyn and G. S. Bloch. According to the results of these tests the invariance of the resistivity to penetration even at very considerable (for soils) values (485 and 837 kg/cm^2) were quite clearly revealed.

The high degree of invariance of resistivity to penetration can be observed based on the results of several hundreds of penetration tests, made by the Swedish Geotechnic Commission in clayey soils with undisturbed structure back in the 1920's [50, 64]. Tests were conducted using conical points with an apex angle of 60° and with six different forces of penetration: 0.01; 0.03; 0.06; 0.1; 0.2 and 0.3 kg (Fig. 3). Plotted along the axis of the abscissae are the depth of indentation of a cone, 0.06 kg in weight, and along the axis of the ordinates - the depth of indentation of the other points. (In Fig. 3 the scale along the axis of the ordinates is reduced by 4 times in comparison with the scale along the axis of the abscissae).

Fig. 3. The method of checking the invariance of the resistivity to penetration by expressing the results of the comparative tests through the depth of indentation of conical point.
KEY: (1) Depth of indentation of conical points with a weight of 0.01 kg, 0.03 kg, 0.10 kg, 0.20 kg, 0.30 kg h mm.
(2) Depth of indentation of conical point with a weight P_1 = 0.06 kg, h mm. [г = g = grams]

The studies in question encompassed a range of resistivity to penetration (with recalculation to standard conical point) from 0.003-0.013 to 1.23-1.71 kg/cm^2, i.e., within a interval of consistencies from liquid to semisolid, inclusively.

For any specified condition of the soil according to the condition of invariance, the specific resistances to penetration do not depend on the value of the vertical load and therefore:

$$R = \frac{P_0}{h_0^2} = \frac{P_1}{h_1^2} = \text{const.} \tag{18}$$

From this, the ratio of the depths of indentation is

$$\frac{h_1}{h_0} = \sqrt{\frac{P_1}{P_0}} = k_h = \text{const} \tag{19}$$

or

$$h_1 = k_h h_0 \text{ см.} \tag{20}$$

The lines $h_1 = k_h h_0$ in Fig. 3 are presented with proportionality factors, calculated for the specific weights of the corresponding points. It is quite evident that the experimental points are sufficiently closely grouped along theoretical straight lines, which confirms the condition of the invariance of the resistivity to penetration.

Examination of the laws governing the location of the experimental points which is given somewhat more attention makes it pc sible to note the following:

for cones 0.01 and 0.03 kg in weight, i.e., those lighter than the standard indentor having a weight of 0.06 kg, the experimental points are grouped in curves, which have an insignificant concavity upwards;

for cones with a weight of 0.1; 0.2 and 0.3 kg, i.e., those heavier than the standard indentor, the experimental points are grouped in curves, which have an insignificant concavity downward.

Such results characterize the specific errors of the conducted penetration tests, connected with a worsening of the mechanical properties of the soils in the surface sections of the samples. In summation, the less the weight of the conical point, the less the depth of its indentation, and the more significant of depth of indentation of the indentor. A worsening of the mechanical characteristics of the surface sections of the samples occurred because of the partial failure of the structure of the soil, apparently when preparing, the samples.

The specific trend towards conducting penetration tests with different loads has been observed in some of the foreign research. Thus, for instance, given in R. Karlsson's work are interesting graphic representations of the dependence of the resistivity to penetration obtained using several values of vertical load, on the depth of indentation of conical points [67]. Four typical diagrams for a standard conical point with an apex angle of 30° are shown in Fig. 4. The condition of invariance of resistivity to penetration $R = \text{const}$ is depicted in these graphic representations in the form of a straight line, parallel to the axis of the abscissae. It is quite evident that in the majority of tests at depth of indentation less than 6-8 mm, the resistivity to penetration turned out to be overstated. For the analysis of the reasons for the similar phenomenon R. Karlsson's graphs were replotted using more suitable coordinates P-h^2. It turned out that in a part of the experiments (for example, during the testing of silt with $w = 141\%$) a certain amount of surface drying of the samples was observed. At the same time the actual accuracy of the measurement of the depth of indentation of the point (about 0.5 mm) resulted into much error in the measurement with h up 6-8 mm.

The graphic method of checking the invariance of the resistivity to penetration. A simple and demonstrative method for checking the invariance of the resistivity to penetration is based on the replotting of the graph of the parabola at the coordinates of the "vertical force of penetration P - the square of the depth of indentation of the conical point h^2" [32].

For this purpose penetration tests are made with 6-8 progressively increasing vertical loads as a minimum, and with a measurement of the corresponding depth of indentation of the point through equal time intervals of approximately 1-1.5 min.

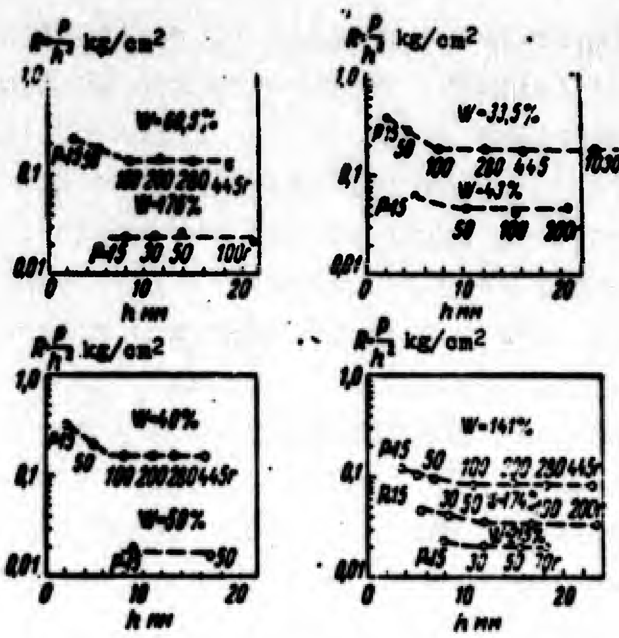

Fig. 4. The method of checking the invariance of the resistivity to penetration by plotting the graph of the dependence R = f(h). h in mm - depth of indentation of conical point; R in kg/cm² - resistivity to penetration.
[г = g = grams]

The invariance of the resistivity to penetration is considered ideal, if the results of the penetration tests with different values of load, are depicted at coordinates P-h^2 in the form of a straight line, passing close to the origin of the coordinates (Fig. 5).

Using such a method for an evaluation of the test data, the resistivity to penetration R is represented by the slope tangent of the mean straight line to the vertical axis, and it is determined from the formulas

$$R = \frac{P_1 - P_2}{h_1^2 - h_2^2} \text{ kg/cm}^2, \qquad (21)$$

where P_1, h_1^2 and P_2, h_2^2 are designated as the coordinates of any two points, located on the straight line, or

$$R = \frac{P - P_0}{h^2} \text{ kg/cm}^2, \qquad (22)$$

where the correction to the value of the force of penetration P_0 is the abscissa of the point of intersection of the mean straight line with the horizontal axis.

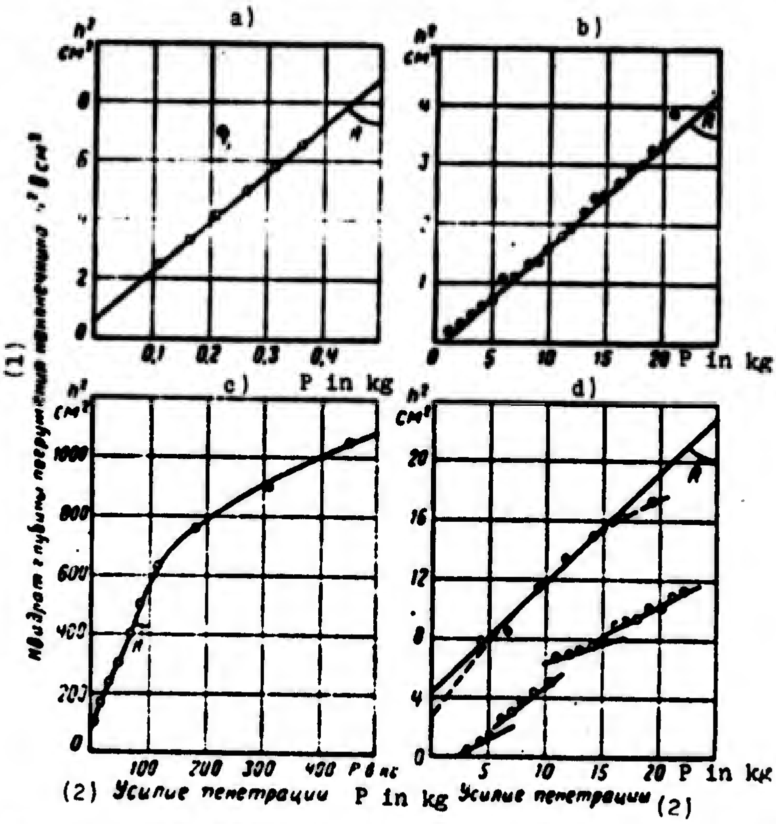

Fig. 5. The method of checking the invariance of the resistivity to penetration by leveling the parabolic graph of the dependence of the "force of penetration - the square of the depth of indentation of conical point." a) and b) graphs of the ideal invariance R; c) break in the graphs of invariance R during testing of stratified soils; d) break in the graphs of invariance R as a result of technical shortcomings of penetration tests.
KEY: (1) Square of the depth of indentation of the point h^2 in cm^2; (2) Force of penetration.

The less the scattering of the experimental points is relative to the mean straight line, the more uniform the investigated soil and the more precise and the more thorough the penetration that is made (Fig. 5a and b). If the penetration tests are made in stratified soils with a marked distinction in the characteristics of the mechanical properties, then the invariance of the resistivity to penetration is unconditionalitydisrupted (Fig. 5c).

In particular, e.g., if the resistivity of penetration of the lower layer R_2 is greater than that of the surface layer R_1, in $P-h^2$ coordinates we observe a bending of the curve in the direction of the abscissa (Fig. 5c).

The greater the difference $R_2 - R_1$, the sharper the break in the invariance of the resistivity to penetration and the steeper the slope of the corresponding deflection of the graph from a straight line.

Examples of two typical cases of the occurrence of technical errors in penetration tests are presented in Fig. 5d. The upper graph represents an attempt to approximate a curvilinear dependence in the form of a straight line of the graphic representation of the invariance. In this case the straight line crosses axis of the ordinates at point $h^2 = 4.5$ cm^2, which is 30% of the maximum ordinate of the linear part of the graph ($h^2_{макс} = 15$). Of course, this case by no means corresponds to the condition of the passage of the straight line of the graphic representation of invariance near the origin of the coordinates. Thus, the test data are doubtful.

The lower graph in Fig. 5d testifies to the failure of the penetrometer, namely, with respect to the wedging of the movable system of the penetrometer in the guide. Similar test data should, of course, be rejected.

When testings the samples of cohesive soils having a disturbed structure, it is comparatively simple to obtain a high degree of invariance of resistivity to penetration. As an example, Fig. 6 gives the results of penetration tests of 12 samples of loess-like clay loams with disturbed structure, taken at the village of Novotroitsk, Kherson Oblast.

The test samples were selected from three different excavations at a depth of 0.5-3.5 m from the surface.[1]

In Fig. 6 for experiments 1, 3, 5, 7, 8, 9 and 10, bright and dark points represent the values of resistivity to penetration obtained when testings the soil samples from the end and from the lateral cut surface of the cutting ring. The high degree of convergence of parallel results characterizes the complete uniformity of the soils achieved with careful remolding of the samples and by their diurnal storage in a hydrator (exsiccator with water) for leveling off the moisture content.

For a comparison 7 graphs were plotted in Fig. 7 for the invariance of resistivity to penetration which was revealed when testings samples of clay loam soils having undisturbed structure. The samples were taken from the same excavations as those previously.

From these data it follows in Fig. 7 that the ideal invariance of resistivity to penetration is distinctly observed under conditions of solid consistency, in the tests of samples of the clay loam soils with undisturbed structure, which are in a three-phase state.

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[1]The average plasticity index of the investigated soils is equal to $M_p = 18.1$, and the average moisture content at the liquid limit is $w_f = 38.6\%$.

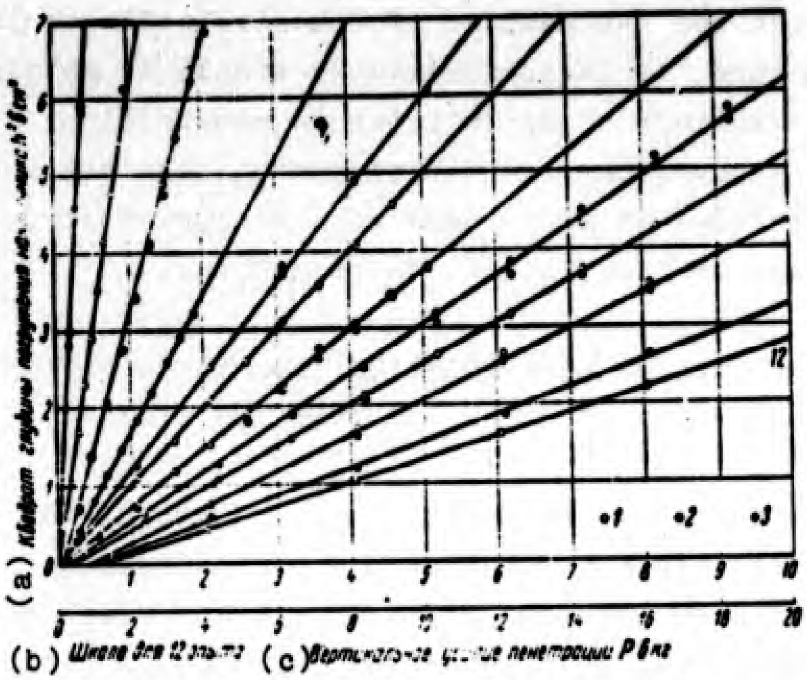


Fig. 6. Graphs of the ideal invariance of resistivity to penetration plotted according to the results of laboratory tests on 12 samples of loess-like clay loams with disturbed structure. 1 - test data from the butt end of the cutting ring; 2 - test data from the side of the cutting ring; 3 - coinciding test data. KEY: (a) Square of the depth of indentation of the indentor h^2 in cm^2; (b) Scale for 12 experiments; (c) Vertical force of penetration in P in kg.

Figure 8 gives the test data on four samples of Iol'diyev clay, which was found in the natural state under conditions of cryptoliquid consistency (M_p = 42.5, w_f = 62.5%, γ = 2.76 g/cm^3). Tests were made on samples from the city of Kem', Karelian ASSR under conditions of undisturbed structure (2, 3 and 4) and after remolding sample 1.

From the location of the experimental points relative to the mean graphs 2, 3 and 4 in Fig. 8, it follows that the surface layer of samples of the undisturbed structure to a depth of 1.7-2 cm had increased values of the resistivity to penetration.

This circumstance, unfortunately, which is frequently encountered in laboratory practice, is connected with the insignificant drying of the samples even under conditions when the necessary measures for storage are used.

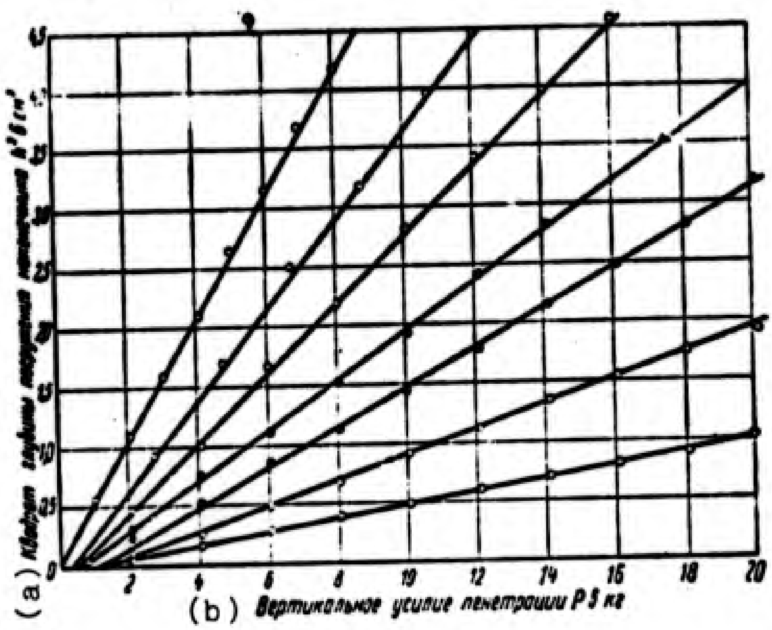
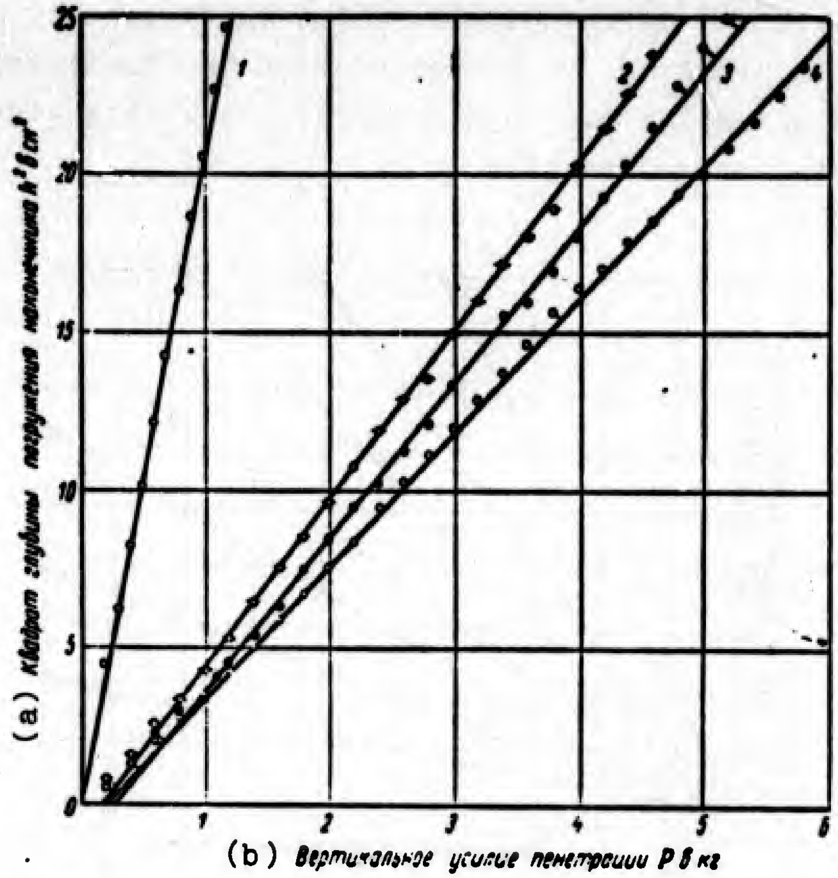


Fig. 7. Graphs of the ideal invariance of resistivity to penetration plotted according to the results of laboratory tests of 7 samples of loess-like loams with undisturbed structure.
KEY: (a) Square of the depth of indentation of the point h^2 in cm^2; (b) Vertical force of penetration P in kg.

Iol'diyev clays, as is known, differ by their high coefficients of structural strength. Thus, these and other analogous tests confirmed the conclusion of P. A. Rehbinder and his colleagues about the strict invariance of resistivity to penetration in the tests of clays with pronounced structural bonds.

Two examples from the practice of field penetration tests on homogeneous soils are shown in Figs. 9 and 10. The results shown above of laboratory penetration tests on cohesive soils

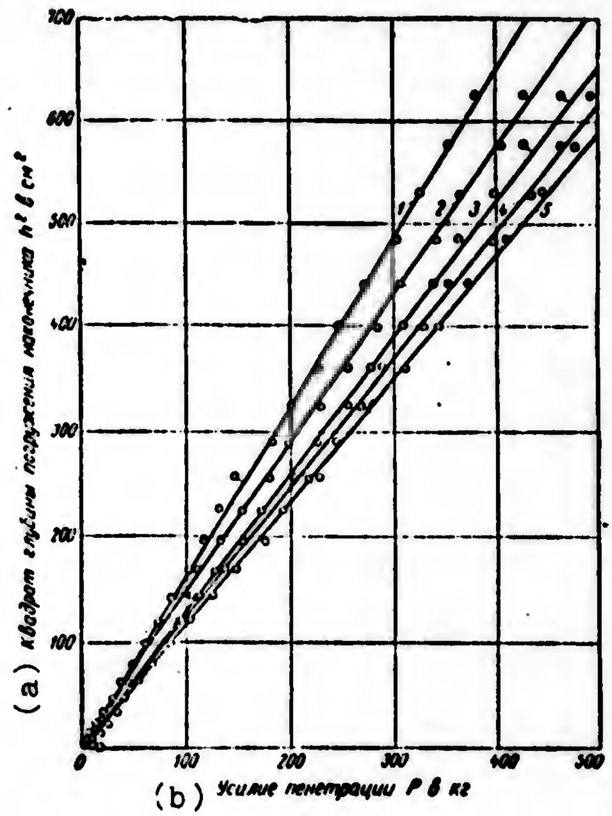


№ опыта (c)	1	2	3	4
W in %	66	66		
[illegible] in g/cm^3	0.969	0.987		
R in kg/cm^2	0.05	0.188	0.204	0.238

Fig. 8. Graphs of the ideal invariance of the resistivity to penetration plotted according to the results of the laboratory tests on samples of Iol'diyev clay with disturbed (experiment 1) and undisturbed (experiments 2, 3 and 4) structure.
KEY: (a) Square of the depth of indentation of the point in h^2 in cm^2; (b) Vertical force of the penetration in P in kg; (c) No. of the experiment.

were made during a progressive application of a vertical load. In this case tests were carried out with the aid of a MP-1 VSEGINGEO penetrometer at the constant rate of insertion of the point from 5 to 25 cm/min. Coniform indentor point with the angle of expansion of 30°, and a height of 25 cm was used.

Fig. 9. Graphs of the ideal invariance of the resistivity to penetration R, plotted according to the results of the field tests on fine silty and clayey sediments with undisturbed structure. 1 - R = = 0.617 kg/cm²; 2 - R = 0.706; 3 - R = 0.766; 4 - R = 0.813; 5 - R = 0.853 kg/cm².
KEY: (a) Square of the depth of indentation of the point h² in cm²; (b) Force of penetration P in kg.



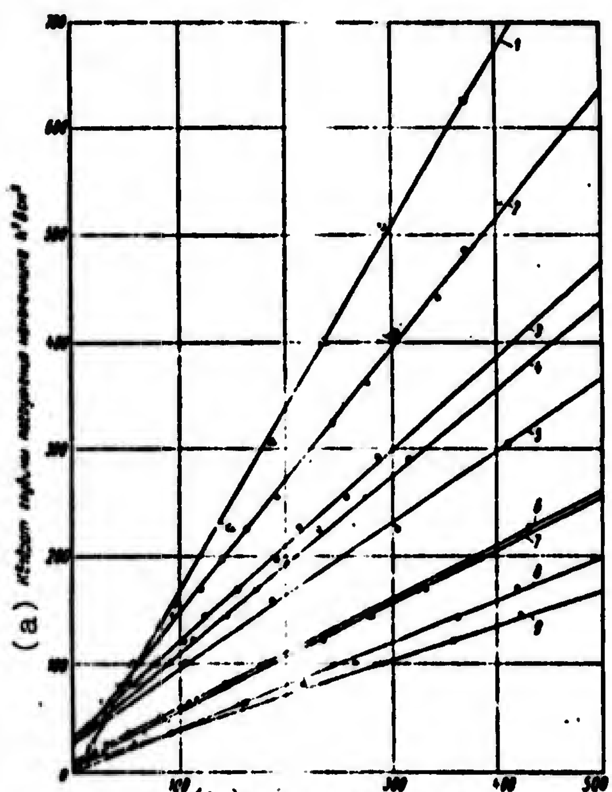


Fig. 10. Graphs of the ideal invariance of the resistivity to penetration R, plotted according to the results of the field tests of a soil horizon consisting of a clay loam mantle. 1 - R = 0.597 kg/cm²; 2 - R = 0.823; 3 - R = 1.14; 4 - R = 1.294; 5 - R = 1.494; 6 - R = 2.01; 7 - R = 2.023; 8 - R = 2.588; 9 - R = 3.085 kg/cm²
KEY: (a) Square of the depth of indentation of point h² in cm; (b) Force of penetration P in kg.

Figure 9 shows the high degree of invariance of the resistivity to penetration of fine silts and clays of the Sivash estuary, obtained from test data in the Nevotroitsk Rayon, Kherson Oblast. The resistivity to penetration of fine silty and clayey sediments with undisturbed structure change within the range of 0.62-0.85 kg/cm^2, which corresponds to the compact-plastic consistency of fine silty and clayey sediments.

The moisture content of the silty and clayey sediments in this area changed within the range of $w = 29-32.8\%$ with a dry unit weight $\delta = 1.21-1.3$ g/cm^3 and a degree of water saturation $G = 0.69-0.74$.

In the following example Fig. 10 gives typical graphic representations of the ideal invariance of the resistivity to penetration obtained during field tests of soil horizon consisting of a clay loam mantle with disturbed and undisturbed structure ($M_p = 10.8$, $w_f = 30.7\%$, $\gamma = 2.66$ g/cm^3).

These tests were conducted at Nakhabino settlement, Moscow Oblast during a change in the moisture content from 15.6 to 23.6%, a dry unit weight from 0.94 to 1.88 g/cm^3 and a degree of water saturation from 0.37 to 0.96.

From the examination of examples of the graphs of the invariance of the resistivity to penetration, one can draw the following conclusions.

1. During penetration tests of homogeneous cohesive soils with disturbed and undisturbed structure throughout the range from liquefaction to solid consistency inclusively, a high degree of invariance of the resistivity to penetration is observed.

The completeness of the graphs $P - P_0 = Rh^2$ lies in the fact that, in the first place, the experimental points are hardly dispersed and are sufficiently densely grouped along the mean straight line and, in the second place, the plotted straight line passes near the origin of the coordinates, i.e., the absolute term in the equation of the parabola P_0 reverts to zero.

2. The completeness of the graphs of the invariance of the resistivity to penetration makes it possible:

a) to confirm the complete homogeneity of the soil throughout the depth of penetration;

b) to determine the resistivity to penetration with a high degree of accuracy;

c) to indicate the absence of technical errors in the conducted tests, and therefore, to reveal the objectivity of the obtained results.

The breakdown of the invariance of the resistivity to penetration serves as an index of either the technical errors in testing, or the heterogeneity of the soil within the limits of the investigated depth.

3. During penetration tests of cohesive soils under conditions of three-phase state, a breakdown in the invariance of the resistivity to penetration is not observed. Consequently, any significant soil compaction during the process of testings does not occur. This important feature during testings is observed when using conical points having an angle of expansion of 30°.

4. Theoretical considerations about the development of the high perfection of the invariance of the resistivity to penetration were confirmed during tests on soils at solid consistency or with undisturbed structure.

The testing of the invariance of the resistivity to penetration by means of progressive increments of vertical force and a corresponding depth of indentation of the point in view of the aforementioned, is the necessary requirement, as a rule, of penetration tests. The errors in testings, possible disturbance of the structure of the samples, unavoidable drying of samples under prolonged storage, the presence of contaminents in the soil and other factors can lead to considerable distortions of the numerical test data.

Meanwhile, the plotting of the graphs of the invariance of the resistivity to penetration is made under laboratory conditions, as a rule, in the course of testings. This makes it possible in due time to determine the systematic and gross errors in testings, to take measures to eliminate shortcomings, to terminate unsuccessful or clearly defective testings. On the contrary, the completeness of the graphs of invariance R testifies to the successful procedures in testings.

Because of this the absence of checks on the invariance of the resistivity to penetration should be considered as the inadmissible carelessness and breakdown of the required technology of penetration tests.

The sharpness of this position can even be explained by the fact that, as will become evident further on, the determination of the real characteristics of the mechanical properties of stratified soils (for example, slightly dried samples) according to the results of penetration tests with one stage of loading, is absolutely impossible. Thus, according to the results of penetration tests it is necessary to either establish the homogeneity of test sample or by means of supplementary analysis, to disclose the layers with uniform characteristics of mechanical properties.

The field penetration tests with the aid of hand tools, nevertheless, are conducted with one stage of loading. In this case an additional requirement is advanced to satisfy the need for testing at a sufficient depth of indentation of conical point.

The essence of the question is not difficult to explain from the diagram, given in Fig. 11. Let us convert calculated formula (22) to the form:

$$R = \frac{P - P_0}{h^2} = R_1 - \frac{P_0}{h^2} \text{ kg/cm}^2 \qquad (23)$$

or

$$R_1 = R + \frac{P_0}{h^2} \text{ kg/cm}^2. \qquad (24)$$

Thus, for case 2, presented in Fig. 11, the current value $R_1 = \frac{P}{h^2}$ is always more than the corrected value of the resistivity to penetration ($R_1 > R$) to the $\frac{P_0}{h^2}$, which with an increase in the depth of indentation of the point constantly decreases. In the case 1 when $P_0 < 0$ (Fig. 11) analogously we will find that $R_1 < R$, whereupon R_1 also unlimitedly approaches R with an increase in the depth of indentation of the indentor h.

It is possible to accept the condition that the difference $R_1 - R$ in absolute value would not exceed a certain permissible value ΔR, commensurable with the accuracy of the penetration testings:

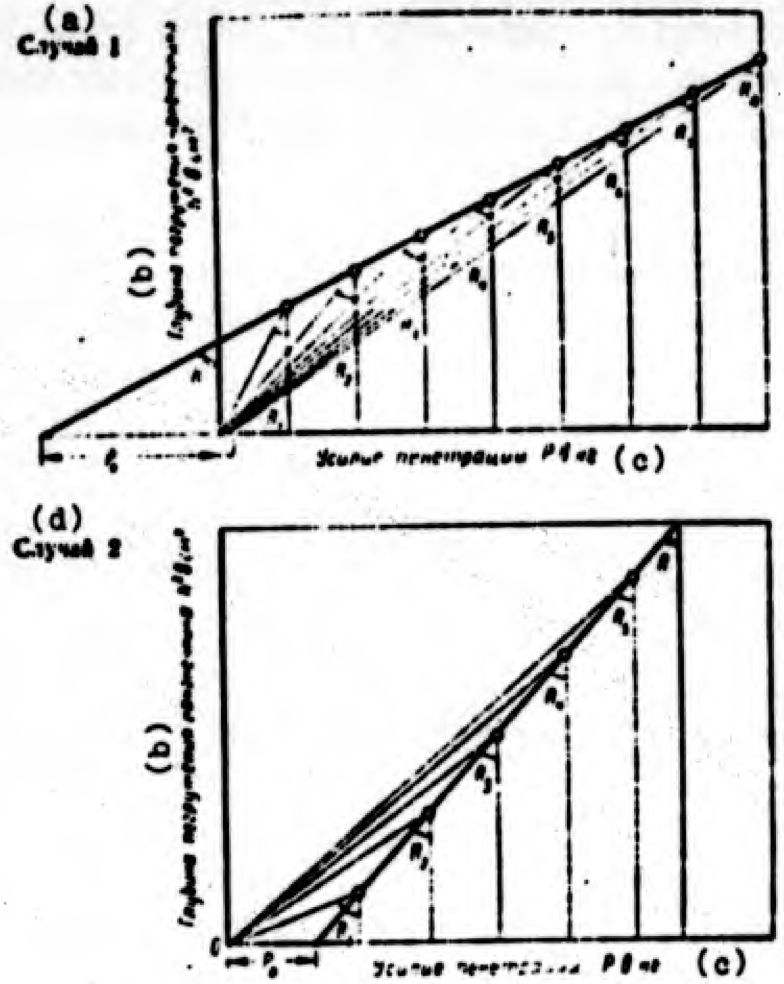


Fig. 11. The diagram of the change in the values of the resistivity to penetration $R_1 = \frac{P}{h^2}$, obtained with one stage of load, depending on the depth of indentation of the indentor.
KEY: (a) Case 1; (b) Depth of indentation of the indentor h^2 in cm^2; (c) Force of penetration P in kg; (d) Case 2.

$$\Delta R > R_1 - R = \frac{P_0}{h^2} \ \text{kg/cm}^2, \tag{25}$$

whence

$$h > \sqrt{\frac{P_0}{\Delta R}} \ \text{cm}. \tag{26}$$

For example, when testing soil having semisolid consistency ($R = 0.85 \div 1.90$ kg/cm^2) it is possible to consider that $\Delta R = \pm 0.05R$, i.e., $\Delta R = 0.043 \div 0.095$ kg/cm^2, a $P_0 = 0.25$ kg. In this case in accordance with the inequality (26) the effective depth of indentation of the indentor should be greater than 1.6-2.4 cm.

Under this condition the error in the determination of the resistivity to penetration from the formula $R = P/h^2$ under the assumption that $P_0 = 0$ will not exceed $\pm 5\%$ from the revealed value R.

The data on the maximum allowance P_0 to the measured force of penetration P can be gradually accumulated according to the results of experiments on the testing of the invariance of the resistivity to penetration.

The given considerations made it possible to develop the following simple recommendations regarding the conducting of field penetration tests in uniform cohesive soils using manual penetrometers with a constant height of conical indentor, h = const. During tests of soils with solid consistency the height of conical point should be more than 30 mm, and under conditions of plastic consistency - either more or equal to 55 mm, and with liquid consistency of the soil - more or equal to 100 mm.

The need for the introduction of a positive correction P_0 to the values of the force of penetration P is caused by three basic reasons:

a) by a certain delay in the development of the phenomena of pushing of the soil during the insertion of the indentor to a shallow depth. The conditions of maximum equilibrium do not appear immediately, but only during the insertion of the point of the indentor to the certain minimally required depth. Thus, in cohesive soils the increased resistance to the penetration of the indentor appears first and the graph of the invariance of the resistivity to penetration acquires a shift to the right of the origin of the coordinates;

b) by the effect of the structural cohesion of the soil through the intensive development of the phenomena of soil displacement. In summation, following from Figs. 6-8, with an increase in the resistivity to penetration the value of positive correction P_0 gradually increases.

During penetration tests of clay suspensions the graphic representations of the invariance, plotted at coordinates P-h^2, pass through the origin of the coordinates, and therefore, parameter $P_0 = 0$. On the contrary, during penetration tests of loess with undisturbed structure I. M. Gor'kov always observed "the initial section of the elastic resistance to deformation, whose value corresponds to the load on the cone of 150-300 g" [19]. Consequently, in this case the occurrence of correction P_0 is connected with the presence of the structural bonds in the tested soil samples;

c) by possible systematic errors of the penetration tests. In this case correction P_0 can have a negative value, i.e., the graph of the invariance of the resistivity to penetration can transverse the axis of the abscissae from the left of the origin of the coordinates. Usually, this occurs with an overestimate of the value of the first stage of load. As a result of a drop in the mobile system of the penetrometer in the development of the force of inertia the depth of indentation of the point turns out to be exaggerated on the first and several subsequent stages of loading.

Analysis of the results of penetration tests of stratified cohesive soils. During the insertion of the point of the coniform indentor into a layer with different mechanical properties a more or less significant breakdown of the invariance of the resistivity to penetration is immediately observed.

The validity of this position was specially checked by V. D. Shitov in checking the completely uniform samples of the loess-like clay loams with disturbed structure from the region of Poltava city (M_p = 11.1, w_f = 31.1%, γ = 2.64 g/cm^3). Figure 12 depicts the results of penetration tests of four different samples of clay loams with liquid-plastic, soft-plastic, stiff-plastic and semisolid consistencies. The samples were tested in a flat cylinder with a wooden pad at the base, which completely eliminated the wall effect of the cylinder.

From the test data it follows that the breakdown of the invariance of the resistivity to penetration for the standard conical point with the angle of expansion of 30° occurs at the point of contact of the indentor with the wooden pad. The development of a similar maximally clear and specific picture of soil displacement is further testimony in favor of the use of a standard conical point.

The parallel penetration tests on clay loams using a conical point with with a an apex angle of 60° led to completely different results (Fig. 13). It is easy to see that the breakdown of the invariance of resistivity to penetration begins in this case long before the direct contact of the indentor point with the pad (for a comparison here the results of pentration tests using a standard conical point were presented).

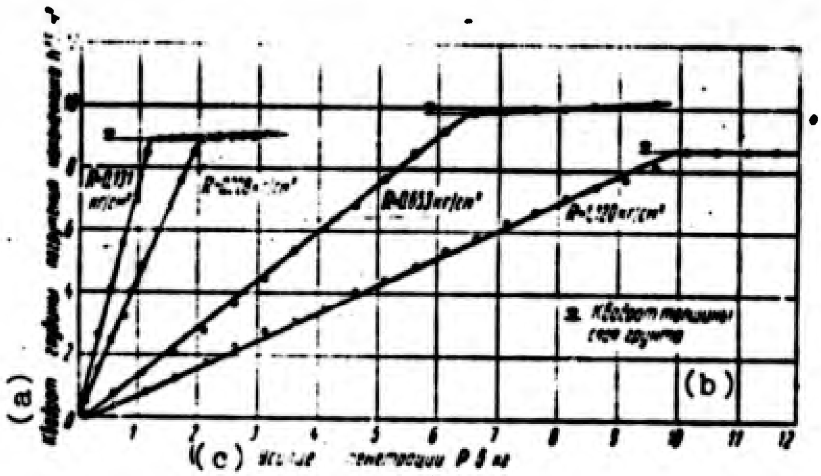
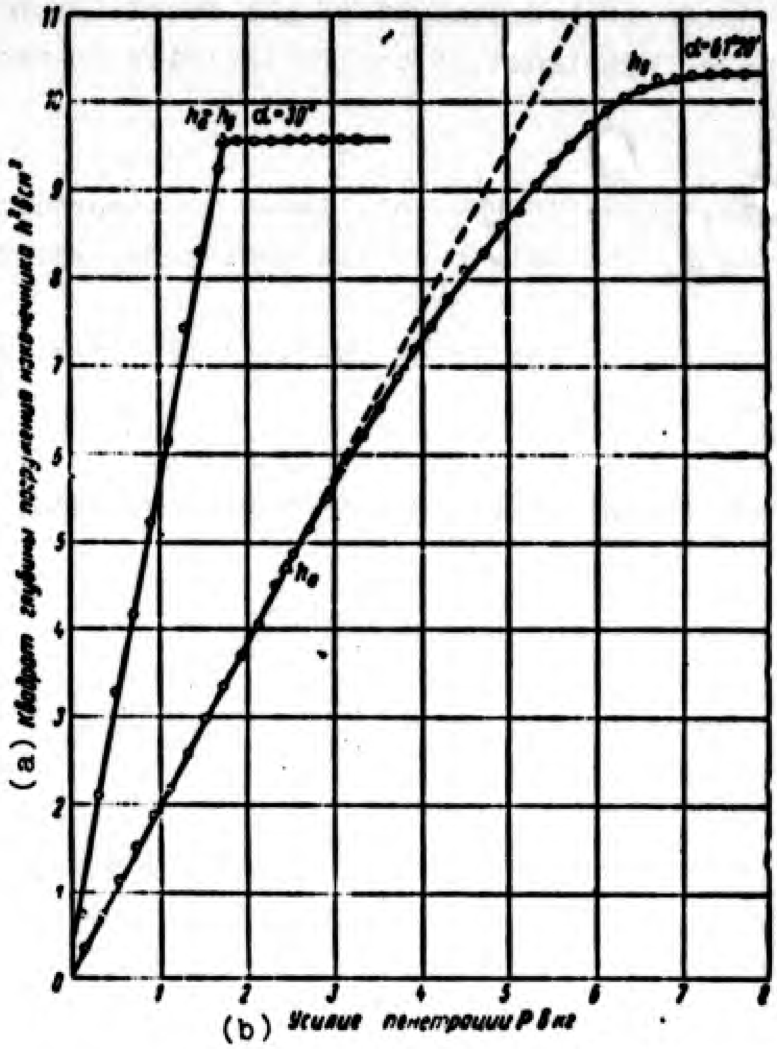


Fig. 12. Graphs of the breakdown of the invariance of the resistivity to penetration revealed during the laboratory tests of a loess-like clay loam using a standard conical point.
KEY: (a) The square of the depth of indentation of the indentor h^2 in cm^2; (b) Square of the thickness of the soil layer; (c) Force of penetration P in kg.
Designation: $кг/см^2$ = kg/cm^2

The premature breakdown of the invariance of resistivity to penetration obtained for a indentor with the angle of 61°20' can be explained in the following manner. When using these points during their insertion a soil cone impression having a sharper angle with the apex is formed. The breakdown of the invariance R occurs at the contact point of the soil cone impression with the wooden pad.

For the analysis of the obtained results let us introduce the designation:



(c) Показатели	(d) Углы раскрытия наконечника	
	α = 30°	α = 61° 20'
(e) h_a в см	3,00	2,17
(f) h_b в см	3,00	3,2
(g) α грунтового конуса	—	43°50'
(h) R_1 и q в кг/см²	0,194	0,527

Fig. 13. Graphs of the breakdown of the invariance of resistivity to penetration revealed during the laboratory tests on loess-like clay loam using different conical indentors.

KEY: (a) Square of the depth of indentation of the indentor h^2 in cm^2; (b) Force of penetration P in kg; (c) Indexes; (d) Expansion angles of the indentor; (e) h_a in cm; (f) h_b in cm; (g) α soil cone impression; (h) R_1 and q in kg/cm^2.

$h_{кон}$ - depth of indentation of the steel indentor, when breakdown of the invariance of the resistivity to penetration in cm is noted;

$H_{сл}$ - thickness of the soil layer in the cylinder, or which is the same, the height of the soil cone impression;

$\alpha_{гр}$ and α - expansion angles of the soil cone impression and the steel indentor.

Then, from the geometric relationships we have

$$\mathrm{tg}\frac{\alpha_{гр}}{2} = \frac{h_{кон}}{H_{сл}}\mathrm{tg}\frac{\alpha}{2}. \tag{27}$$

In this case with $h_{кон}$ = 2.17 cm, $H_{сл}$ = 3.2 cm and α = 61°20' we will find that $\alpha_{гр} \approx 44°$.

Direct testing with conical point having α = 45° actually showed a very insignificant breakdown of the invariance of the resistivity to penetration near the base of the soil layer. Consequently, at expansion angles of 30-40° the formation of soil cone impressions is not observed and therefore the breakdown of the invariance of the resistivity to penetration occurs only with a direct (contact) of the indentor with another soil layer.

It is obvious that this important circumstance can insure a high degree of accuracy for finding the position of the second layer of soil.

After the explanation of this feature of penetration tests let us turn to the simplest, most adequate and frequently observed diagram of a two-layered base, depicted in Fig. 14.

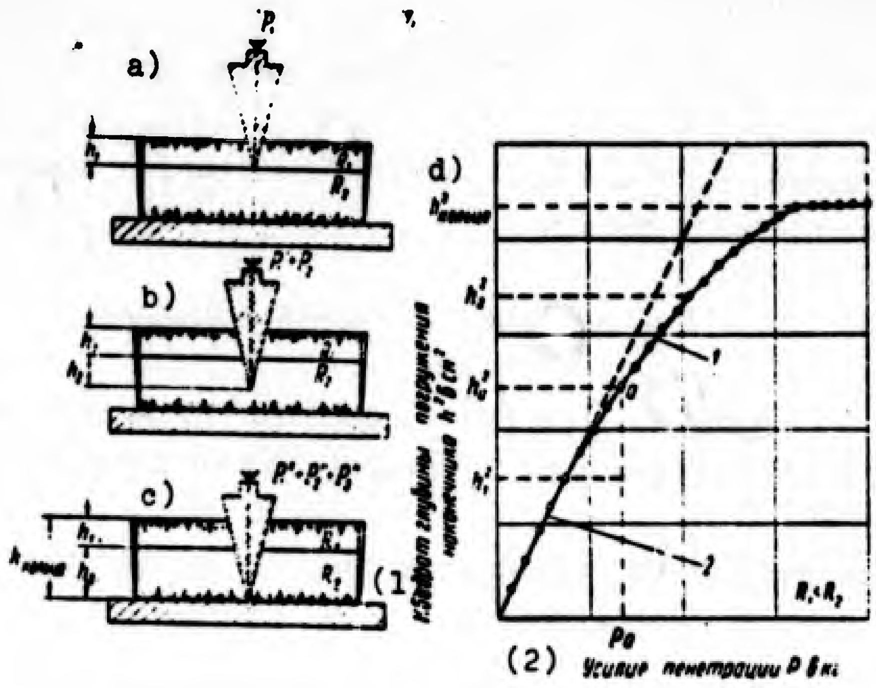


Fig. 14. Diagrams of penetration tests of a sample of stratified soils a, b, c) stages of insertion of the indentor; d) typical graph of the breakdown of the invariance of the resistivity to penetration obtained during the testing of stratified soils; 1 - parabola; 2 - straight line.
KEY: (1) Square of the depth of the insertion of the indentor h^2 in cm^2; (2) Force of penetra- P in kg.

If the surface layer of soil with a resistivity to penetration R_1 should have a greater depth of indentation of the indentor ($h_1 > h$), then, according to dependence (22), when $P_0 = 0$ we will obtain

$$P = R_1 h^2 \text{ kg.}$$

However, in accordance with the diagram in Fig. 14b, a part of the point of the indentor with a height of h_2 has penetrated into the second layer with the resistivity to penetration R_2. Thus, in this case the condition of equality of the force of penetration P to the reactance of the cohesive soil can be expressed by the dependence

$$P = R_1 h^2 + (R_2 - R_1) h_2^2 \text{ kg} \qquad (28)$$

However, $h_2 = h - h_1$. After substitution and some transformations will will obtain

$$P = R_2 h^2 - 2(R_2 - R_1) h h_1 + (R_2 - R_1) h_1^2 \text{ kg} \qquad (29)$$

where h_1 - distance from the surface to the second layer of soil, or which is the same, the thickness of the first layer of soil.

The analysis of the dependence (29) provides a basis for the following conclusions:

1) in the case when the conical point passes through two different layers of soil, the graph of the proportional dependence $P = R_1 h^2$, characterizing the ideal invariance of the resistivity to penetration, within the limits of first layer R_1 transforms to a parabola with equation (29) (Fig. 14d). Thus, the invariance of the resistivity to penetration R_1 at coordinates $(P-h^2)$ is unavoidably disrupted;

2) the second layer of soil according to the initial condition is uniform. Thus, during the gradual insertion of the indentor into the second layer the ideal invariance of the resistivity of the second layer to penetration will be detected. Consequently, it is necessary to reconstruct the parabola (29) at those coordinates for which the variable of the depth of indentation of the indentor h will not be included when using the second term in equation (29);

3) it is easy to show that if we determine the average value of the resistivity to penetration $R_{cp} = P/h^2$, during the penetration of stratified soils then this value represents the weighted mean characteristic of the resistivity of the separate layers to penetration.

The determination of the resistivity of the second layer R_2 to penetration can be made according to the results of penetration tests using the different methods developed by V. D. Shitov [41].

The invariance of the penetration index. In uniform sandy soils having a value of specific cohesion, close to zero ($c \to 0$), as already mentioned, the resistivity to penetration during the indentation with depth increases proportionally: $R = Uh$ kg/cm^2.

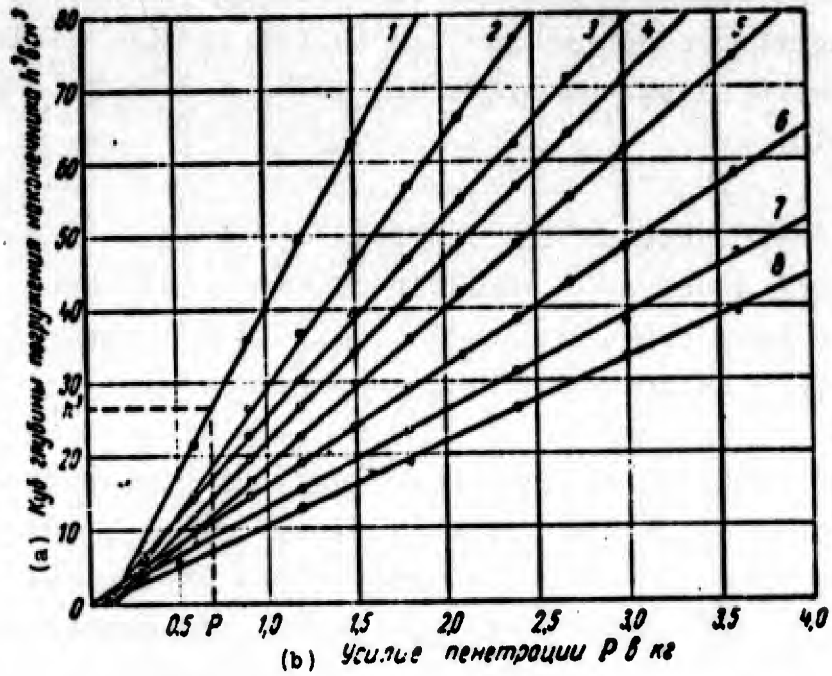
Thus, under these conditions based on the proposal of N. L. Zotsenko and the author the penetration index U and the dimensionless generalized penetration index $U_0 = U/\Delta$ are taken as an objective characteristic of the results of penetration testings.

In plotting the invariance of the penetration index at coordinates of the "force of penetration P - cube of the depth of indentation of the cone h^3," the penetration index U represents the slope of the mean straight line to the vertical axis and it is determined from the formula

$$U = \frac{P - P_0}{h^3} \text{ kg/cm}^3, \qquad (30)$$

where P_0 by analogy with the test data on cohesive soils represents the correction to the value of the force of penetration, depending on the properties of the sand and the experimental conditions.

Numerous tests confirmed the very ideal invariance of the penetration indexes U and U_0. Figure 15 as an example gives the results of 8 laboratory tests on samples of fine-grain sand, having moisture contents from 5 to 28% and bulk densities from 1.5 to 1.7 g/cm^3.



№ опыта (c)	1	2	3	4	5	6	7	8
l'_0	14.8	18.5	23.7	25.7	29.6	38.4	46.4	52
Δ в г/см³ (d)	1.5	1.51	1.53	1.56	1.57	1.61	1.66	1.7

Fig. 15. Graphs of the ideal invariance of penetration indexes obtained during the laboratory tests of moistened samples of fine-grain sand.
KEY: (a) Cube of the depth of indentation of the point h^3 in cm^3; (b) Force of penetration P in kg; (c) experiment; (d) In g/cm^3.

Shown in another example in Fig. 16 are the results of 8 field penetration testings 1-8, conducted by N. L. Zotsenko in a sand borrow pit near the town of Poltava with the aid of a MP-1 VSEGINGEO penetrometer model. Tests were made using conical point having an angle of expansion of 30° and a height of 25 cm.

The moisture content of the sand in the borrow pit changed from 7.5 to 10%, the volume weight of the sand changed from

1.73 to 1.98 g/cm^3. From these tests the values of the penetration indexes from 0.037-0.051 kg/cm^3 (straight lines 1 and 2 in Fig. 16) to 0.123-0.145 kg/cm^3 (straight lines 7 and 8) were obtained.

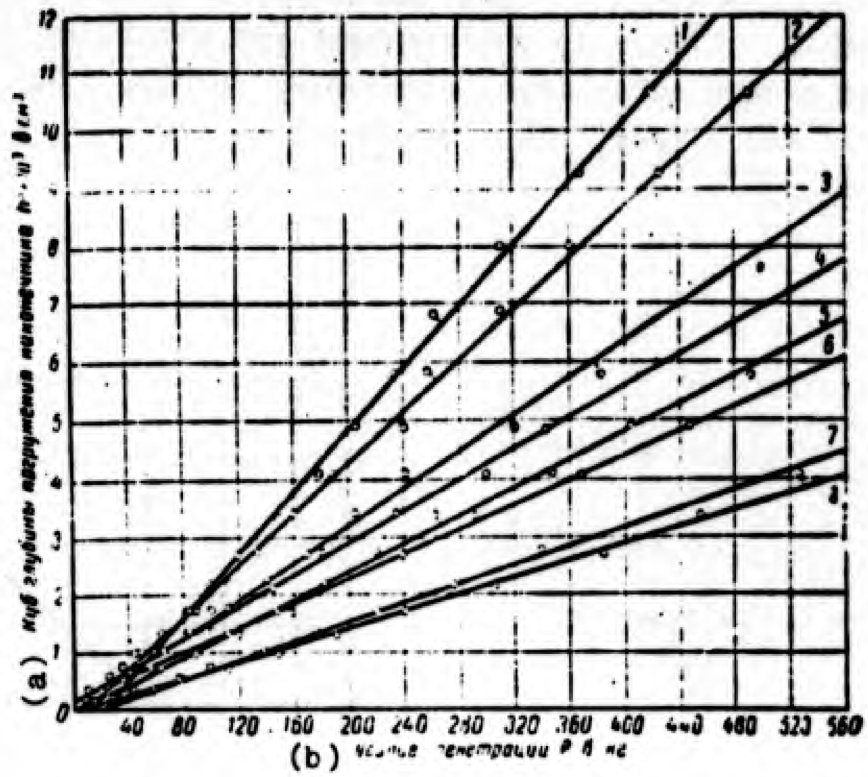


Fig. 16. Graphs of the ideal invariance of the penetration indexes obtained during the field tests of the sand.
KEY: (a) Cube of the depth of indentation of the point $h^3 \times 10^3$ in cm^3; (b) Force of penetration P in kg.

If within the limits of the depth of indentation of the point a layer of sand iwht other indexes of mechanical properties, should be detected, then this will cause breakdown of the graph of the invariance of the penetration index (Fig. 17). Represented in this example the mean values from 17 parallel penetration tests, made along the shoreline of the Dnieper river in the area of Kremenchug city. It is easy to see that down to a depth of 19.75 cm (point in Fig. 17), i.e., to $h^3 = 7.7 \cdot 10^3$ cm^3, the invariance of the penetration index U = const. Beyond this depth the invariance of the penetration index is sharply disrupted and the parabolic curve is deflected to the vertical axis. The breakdown of invariance U was the consequence of a considerable decrease in the strength properties of the sand below the ground water level (according to direct measurements the ground water level (GWL) changed from -17 to -22 cm).

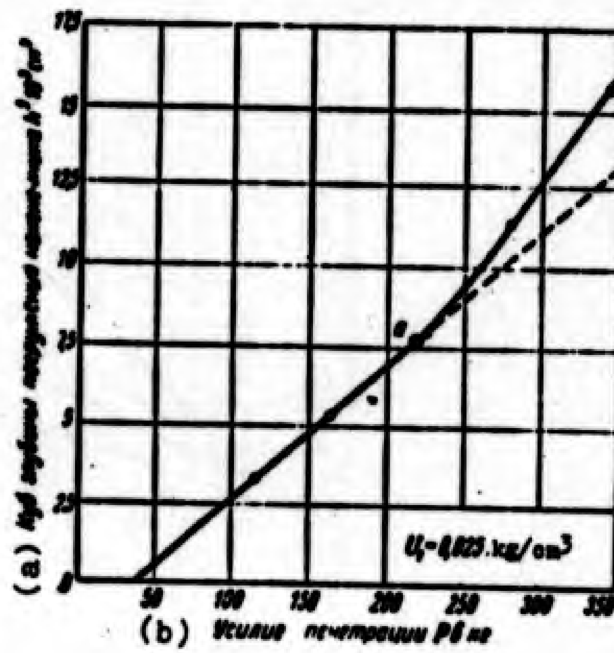


Fig. 17. Graph of the breakdown of the invariance of the penetration index obtained during field tests of stratified fine-grain sand. KEY: (a) Cube of the depth of indentation of the point $h^3 \cdot 10^3$ cm^3; (b) Force of penetration P in kg.

The first surface layer of sand was found to be in a three-phase state. The average value of the penetration index of this layer was calculated according to the results of 47 determinations and it turned out to be equal to $U_1 = 2.50 \cdot 10^{-2} \pm 0.06 \cdot 10^{-2}$ kg/cm^3 with the index of accuracy of the testings $\rho = 2.4\%$ and the degree of uniformity of sand according to p. 5.4 of the Construction Norms and Regulations SNiP II-B.1-62 $k_1 = 83\%$.

The control penetration tests on sand were carried out after flooding the site with a sheet of water 20-30 cm. deep. On the basis of 70 determinations of values for P = f(h), obtained during 19 penetration tests on the sand under the layer of water, an average value $U_2 = 1.13 \cdot 10^{-2} \pm 0.016 \times 10^{-2}$ kg/cm^3 was established. Under these conditions the index of the accuracy of testings comprised $\rho = 1.4\%$ with the degree of uniformity of the sand $k_2 = 88.5\%$.

Analysis of the results of penetration tests of noncohesive soils. The analysis and the interpretation of the results of penetration tests of sands can be carried out on the basis of the known diagram about the basic cases of limiting condition of a sand base [4, 5, 35].

As already mentioned, the limiting condition of a sand base according to the condition of strength during central loading appears at the moment of the termination of the formation of the compacted core under the flat punch. Thus, during penetration tests using a conical point the limiting condition appears at any depth of indentation of the point less than its own height.

The physical process for the onset of the limiting condition of a sand base with a progressive increase in the vertical load on the flat punch can be characterized by at least three main cases, corresponding to the different nature of the deformation and degree of development of the slip surface.

In the first case, the limiting condition which has been sufficiently studied in the axisymmetric task of the theory of limit equilibrium (V. G. Berezantsev [4, 5], M. V. Malyshev). is characterized by a fully developed slip surface, which appears on the surface of the base, and it is exhibited in the form of soil displacement and a depression from the punch.

In the third case of deformation the bases generally are associated with soil compaction below the bottom of the punch due to the action of the compacted core whereupon the slip surfaces are only outlined (at the apex of the core). The third case of the onset of the limiting condition of a sand base will be most frequently exhibited during sounding under conditions of equality of the diameter of the indentor to the diameter of the rod $d_{кон}/d_{шт} = 1$ [54, 57].

In the second, intermediate case of deformation the bases are the result of the interaction of the zones of shear below the bottom of the punch with the compaction zones higher than its bottom, whereupon the slip surfaces have one degree of development or another, are offset by the thickness of the soil and the bases do not appear on the surface.

The onset for one of the cases of a limiting condition of a sand base depends on the initial relative indentation of the punch H/d and the relative density of the sand. In connection with the conditions of penetration tests on sandy soils when H = 0, as a rule, the first case of limiting condition will be observed in the form of the total shear of soil over the continuous slip surface which appears on the surface at an angle of $45° - \phi/2$.

On the basis of the above-stated, the angle of internal friction of sand ϕ is determined using Prof. V. G. Berezantsev's solution [4]. For a conical point with the angle of expansion of 90°, the dependence $U_{090} = \pi U_T = f(\phi)$ is accepted according to the data in Table 2 and the graphic representation in Fig. 18.

For penetration tests on sandy soils, it is better to use a standard coniform point with an angle of expansion of 30° in order to avoid compaction. In this connection there arises the additional task of determining the proportionality factor $k_{\alpha U} = U_\alpha/U_\beta$ between the penetration indexes U_α and U_β, obtained during the testing of sandy soils using conical points with different expansion angles α and β.

Table 2. The values of dimensionless coefficients U_T, U_{090} and U_0.

Angle of internal friction ϕ	Coefficients		
	U_T	$U_{090}=\pi U_T$	$U_0 = 0{,}019\pi U_T$
16	4,2	13,2	0,25
18	5,6	17,6	0,34
20	7,2	22,6	0,43
22	10	31,4	0,6
24	14	44	0,84
26	19	59,7	1,15
28	25,2	79,1	1,52
30	34,6	108,6	2,09
32	48,8	153,2	2,94
34	69,2	217,3	4,17
36	97,2	305,2	5,86
38	142,6	447,8	8,6
40	216	678,2	13,02
42	317	995,4	19,12

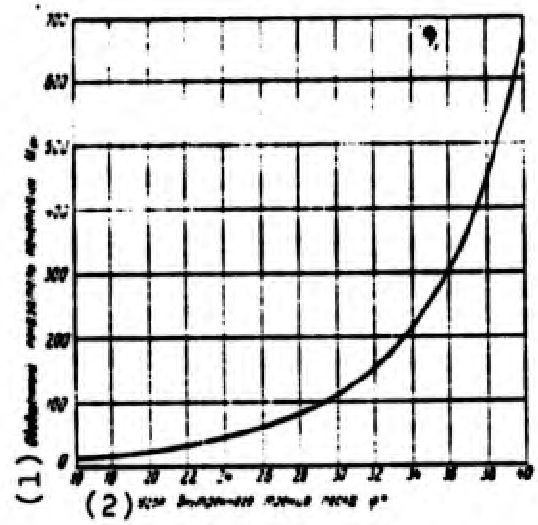


Fig. 18. Graph of the theoretical dependence between the generalized penetration index U_{090} (for a conical point with an angle of expansion of 90°) and the angle of internal friction of sandy soils (according to V. G. Berezantsev). KEY: (1) Generalized penetration index; (2) Angle of internal friction of sand ϕ°.

This task can be solved according to two versions depending on the degree of the account of the actual values of the specific cohesion of sandy soils during the determination of the penetration index U. The carried out comparisons made it possible to establish that the specific cohesion at several hundredth kg/cm^2 with small depth of indentation of the point (5-10 cm) leads to the noticeable overestimate of the penetration index. (In this case at coordinates "$P-h^3$" the experimental points acquire a regular deflection from the mean straight line). Thus, during the first stage in using the results of penetration tests on sandy soils the convergence of the angles of internal friction was observed, revealed according to the results of penetration and single-plane shear, in cases when the proportionality factor k_α = 0.087 (Table 1), was applied which N. N. Agranat and M. P. Volarovich obtained in connection with the conditions of penetration tests on cohesive soils [41].

The second version for a solution of the assigned task, proposed by N. L. Zotsenko and the author, consists of the following. It follows from formula (2), the value of the first term $U_T \Delta r$ is determined only by radius r and does not depend on the expansion angle of the conical point. Consequently, in sandy soils, in the absence of cohesion, the penetration indexes U_α and U_β are determined from uniform formulas:

$$\left.\begin{aligned} U_\alpha &= U_\tau \pi \Delta \operatorname{tg}^3 \frac{\alpha}{2}; \\ U_\beta &= U_\tau \pi \Delta \operatorname{tg}^3 \frac{\beta}{2}. \end{aligned}\right\} \quad (31)$$

Thus, the proportionality factor $k_{\alpha U}$ is obtained equal to:

$$k_{\alpha U} = \frac{U_\alpha}{U_\beta} = \frac{\operatorname{tg}^3 \frac{\alpha}{2}}{\operatorname{tg}^3 \frac{\beta}{2}}. \quad (32)$$

The values of $k_{\alpha U}$ when α = 30° for several basic expansion angles of a conical point β are given in the first graph in Table 3.

Table 3. The calculated and mean experimental values of the proportionality factors $k_{\alpha U}$.

Angles in degrees of the expansion of conical points	Computed values of $k_{\alpha U}$	Experimental values of $k_{\alpha U}$	Mean values of the penetration index U in kg/cm³
30	1	1	0.032
45	0.27	0.252	0.127
60	0.1	0.095	0.337
90	0.019	0.022	1.433

The determination of the angle of internal friction of sandy soils is made according to the results of penetration tests in the following sequence:

a) when testing sandy soils it is compulsory to record the dependence $h = f(P)$ with 6-8 stages of loading;

b) when testing dry and completely water-saturated sandy soils having disturbed structure, the penetration indexes are determined by means of constructing graphs of the invariance U at coordinates P-h^3. In the remaining cases it is more preferable to construct graphs of the invariance U at coordinates of the "resistivity to penetration $R = P/h^2$ - the depth of indentation of point h";

c) the volume weight of the sand is established and the average value of the generalized penetration U_0 index is determined;

d) taking into account the proportionality factor $k_{90U} = 0.019$, U_{090} is calculated. Further on, according to the graphic representation in Fig. 18 or in Table 2, the angle of internal friction of sand ϕ is found.

The method of determining the angle of internal friction of sandy soils from the results of laboratory tests by a circular punch was proposed for the first time by N. N. Sidorov in 1956.

The correctness of the utilized calculated value $k_{\alpha u}$ was tested by means of comparative penetration testings on coarse-grained sand, made with the aid of a MP-1 mechanical device. In all, 29 tests and 162 separate determinations of the dependence $h = f(P)$ were conducted . The test data were processed by N. L. Zotsenko at coordinates R-h. . The investigated sands did not differ in uniformity; therefore, the index of accuracy of penetration testings amounted to only 5-10%. Nevertheless, from the data in Table 3 there is completely sufficient convergence of the calculated and mean experimental values of the proportionality factors $k_{\alpha U}$.

With a volume weight of sand $\Delta = 2.05$ g/cm^3 the generalized penetration index $U_0 = 15.6$. Consequently, according to Table 2, the angle of internal friction amounts to about 41°. The obtained result coincides with the data in Table 13 of SNiP II-B.1-62 - with a void ratio of 0.51-0.60 coarse-grained sand has a standard angle of internal friction, equal to 40°.

CHAPTER II

THE PROCEDURE FOR TESTING THE SURFACE LAYERS OF SOIL BY SOUNDING

The field of application of the proposed procedure. When inserting the conical point to a depth which exceeds its height, the parabolic dependence (1) between the acting force and the depth of indentation of the point appears to be disturbed.

New relationships between the force of sounding and the depth of indentation of the point in connection with testings cohesive and noncohesive soils are examined in this chapter. The field of application of the recommended procedure is restricted to the following conditions.

a) Soil tests by static sounding are achieved to a depth of several meters or, in any case, are made under conditions whereby the frictional forces, acting along the lateral surface of the rods, are not significant.

In this case the acting force P is almost completely transmitted to the point of the probe ($P \approx P_{зонда}$) and therefore the need does not arise for the separate recording of forces P and $P_{зонда}$.

b) Soil tests by sounding are made using diameters of the points exceeding the diameter of the rod $\frac{d_{кон}}{d_{шт}} > 1.6$. With such a relationship, apart from eliminating or considerably reducing the frictional forces along the lateral surface of rod, conditions are created for displacing the soil and forming a cavity whereby the resistivity to penetration and the sounding (see below) turn out to be identical.

The conditions of static sounding of soils when $d_{кон} = d_{шт}$, to a known degree simulate the work of piles which are not examined in this book.

The recommended methods of static sounding can be found and even applied using the engineering geology surveys of the surface layers of soil for road and airfield construction, for checking the quality of the seal of the subgrade, embankments and dams of hydro-engineering constructions, for building power lines and for other purposes.

Methods of expressing the test data of soils by static sounding. The generalized characteristics of the results of static sounding of soils are selected by solving the axisymmetrical task of the theory of limit equilibrium [4, 5]. Under the condition of the action of surcharge weights $q = \Delta H$, the trinomial equation of the theory of limit equilibrium by analogy with equation (3) takes the form

$$P = U_0 \Delta h_{кон}^3 + V_0 \Delta H h_{кон}^2 + N_0 c h_{кон}^2 \quad \tau, \qquad (33)$$
$$V_0 = V_т \pi \, tg^2 \frac{\alpha}{2};$$

where

$V_т$ - dimensionless bearing capacity factor of the soil which depends only on the angle of internal friction of the soil; $h_{кон}$ - height of the conical part of the point of probe in m; H - depth of sounding, which is the same as the thickness of the layer of the surcharge weight. For example, if the angle of internal friction is close to zero or, in any case, does not exceed 5-10°, but specific cohesion c is more than 1-2.5 t/m^2, then effect of the first two members in equation (33) with the depths of sounding up to 6-10 m turns out to be relatively insignificant. Under these conditions, and also in cases when the angle of internal friction exceeds 10-15°, the specific cohesion turns out to be very significant ($c > 5$-$7.5\ t/m^2$), just as the objective characteristic of the results of static sounding can be conveniently taken as the "resistivity to sounding" Q, which represents the ratio of the force of sounding at the investigated horizon P to the square of the height of the conical part of the point of the probe $h^2_{\text{кон}}$:

$$Q = \frac{P}{h^2_{\text{кон}}} \approx N_{\bullet}c \quad t/m^2, \text{or } kg/cm^2.$$

It is easy to see that the dependence (34) is the limiting case of formula (5) when $h = h_{\text{кон}}$.

The invariance of resistivity to sounding lies in the fact that in the layer of a uniform cohesive soil the force of sounding does not change, and consequently, the resistivity to sounding remains constant $Q = \text{const}$.

In certain cases one cannot rule out the possibility of expressing the results of sounding testings in cohesive soils in the form of values of arbitrary mean pressure along the base of the conical point (specific head resistance to sounding)

$$\sigma = \frac{P}{\pi r^2} \text{ kg/cm}^2, \qquad (35)$$

where r - radius of the base of the point.

For the recommended conical point with an angle having an apex of 30° from simple geometric relationships we will obtain

$$Q = 0{,}226\sigma \ \text{kg/cm}^2, \tag{36}$$

or

$$\sigma = 4{,}433Q \ \text{kg/cm}^2. \tag{37}$$

Formulas (36) and (37) should be used when recalculating the values of the mean pressure σ in the resistivity to sounding Q and vice versa.

The invariance of the mean specific pressure lies in the fact that in the layer of uniform cohesive soil the value σ remains constant:

a) during a change in the diameter which is the same as, the height of the conical point;

b) during a change in the apex angle of the conical point.

The condition of invariance σ can be disrupted when sounding three-phase soils in the case where the diameter of the conical point is equal to the diameter of the rod, $d_{\text{кон}} = d_{\text{шт}}$. In this case a change in the diameter of the point can lead to a different degree of compaction of the soil during the insertion of the probe; this will cause a difference in the values of the mean specific pressure σ.

For an analysis of the results of static sounding, made in noncohesive soils, the dependence (34) is unsuitable. In this case it follows from the trinomial equation (33), the mechanical properties of noncohesive soils increase linearly with an increase in natural pressure. For example, in uniform sand with a specific cohesion, equal to zero and $h_{\text{кон}}$ = const, the absolute term in equation (33) is made equal to $U_0 \Delta h^3_{\text{кон}}$ = const. It is convenient from a further examination to exclude and accept the index of sounding V as an objective characteristic of the results of sounding testings, equal to the ratio of differences in the forces of sounding $P_2 - P_1$ to the product of the corresponding difference in the depths of sounding to the square of the height of the conical point $(H_2 - H_1)h^2_{кон}$ (proposal of N. L. Zotsenko and the author).

$$V = \frac{P_2 - P_1}{(H_2 - H_1)\, h^2_{кон}} = V_0 \Delta \text{ t/m}^3\text{, or kg/cm}^3. \qquad (38)$$

Formula (38), naturally, is valid when $H_1 > h_{кон}$ and $H_2 > H_1$. If we accept that $H_2 = h_{кон}$, and $H_1 = 0$ (in this case $P_1 = 0$), then formula (38) is converted in formula (9), proposed for determining the index of penetration $U = \frac{P}{h^3_{кон}}$.

Just as for the case of penetration, in the further analysis of the results of sounding in sands, it is convenient to utilize a dimensionless quantity of the "generalized index of sounding V_0," equal to the ratio of the index of sounding V to the volume weight of sand Δ:

$$V_0 = \frac{V}{\Delta} = \frac{P_2 - P_1}{\Delta (H_2 - H_1)\, h^2_{кон}}. \qquad (39)$$

The invariance of the index of sounding in the layer of uniform sand is exhibited by virtue of the fact that with an increase in the depth of indentation of the point, the force of sounding increases linearly, i.e., the intensity of an increase in the resistance of the sand with depth of sounding proves to be constant. Thus, the index of sounding V, which takes into consideration this feature of the change in the mechanical properties of sands, remains invariable.

The conditions of the deformation of a sand base during the insertion of probe were studied by V. A. Yaroshenko for the very typical case $d_{кон}/d_{шт} = 1$, which corresponds to the conditions of the insertion of a pile. With certain portions of the schematization, these conditions according to V. A. Yaroshenko, are reduced to the following [57].

1. The insertion of the probe occurs as a result of the displacement of sand in the zone of least resistance. Depending on the initial density of the sand and the relative penetration of the probe H/d_{KOH} the zones of least resistance are located above or below the base of the conical part of the probe.

2. In unconsolidated sands the driving of the probe occurs even at the surface because of the interaction of the zones of shear and zones of compaction of the sand below the base of the conical part of the probe; this is accompanied by a certain amount of depression on the surface of the soil.

3. In sands of average and high density the insertion is first accompanied by a displacement of the soil at the surface (1st case of a limiting condition). Then, with relative penetration $H/d_{KOH} > 1.5-2$ the displacement ceases and the probe buries itself because of the interaction of the zones of shear and the zones of compaction above the base of the conical part of the probe (2nd case of a limiting condition). In this case, the displaced volumes overcome all of the increasing weight of the surcharge weight and the resistivity of the base "increases almost directly proportional to an increase in depth."

4. Under the conditions of the uniform, incompressible medium, the resistivity of the base should increase constantly and linearly with an increase in the depth of sounding. Under actual conditions an intense compaction of the sand occurs. Thus, if the resistance of the surcharge weight becomes greater than the stresses, necessary to compact the sand in the area below the base of the conical part of the probe, then further driving will be achieved as a result the "interaction of the small zones of displacement and zones of compaction located below the base of the conical part of the probe" (3rd case of deformation of the base).

5. Soil compaction in the area below the base of the conical part of the probe unavoidably gives rise to the formation of an unloading ground arch over the area of compaction, which provides, at a certain depth of indentation of the probe, stabilization on the effect of the weight of the surcharge, i.e., constancy to the force of sounding.

The higher the initial density of the sand, the greater the stresses, necessary for its intense compaction, and the greater the relative penetration of the probe at which the stabilization of the effect of the weight of the surcharge occurs. In very dense sands the critical depth of sounding $H_{кр}$, which corresponds to condition P = const, is observed when $H/d_{кон} > 100$.

The above-mentioned presentation of the conditions of deformation of a sand base makes it possible to draw the following conclusions:

a) the objective characteristic of sounding tests in sandy soils - the index of sounding V - applicable in the 1st and 2nd cases of a limiting condition at depths of sounding, not exceeding $H_{кр}$;

b) in dense sands with the diameters of the conical points at 5-7.4 cm the critical depth of sounding amounts to at least several (2-3) meters. With an increase in the diameter of the conical point of the probe, the critical depth of sounding $H_{кр}$ increases correspondingly;

c) under conditions when the diameter of the conical point is considerably greater than the diameter of the rod $d_{кон}/d_{шт} \gg 1$, a zone of least resistance greater than the base of the conical part of the probe point is artificially produced. In this case the critical depth of sounding can also be increased.

As a whole it is necessary to state that the application of the characteristics of resistivity to sounding Q and the index of sounding V provides the successive relationship with the characteristics of the resistivity to penetration R and the penetration index U. When $H \leq h_{кон}$ the formulas (34) and (38) can be converted into formulas (5) and (9), respectively.

Determination of the angle of internal friction in noncohesive soils. As Yu. G. Trofimenkov correctly noted, "until now there are comparatively few experimental data on the dependence between the resistance of the soil to the insertion of a cone and the angle of internal friction of the soil. This can be explained mainly by the difficulty in determining the angle of internal friction of sands in their natural state" [49]. Thus, it should be noted that the existing proposals according to the methods for determining the angle of internal friction of sands are the first approximation, and therefore, need further refinement and development.

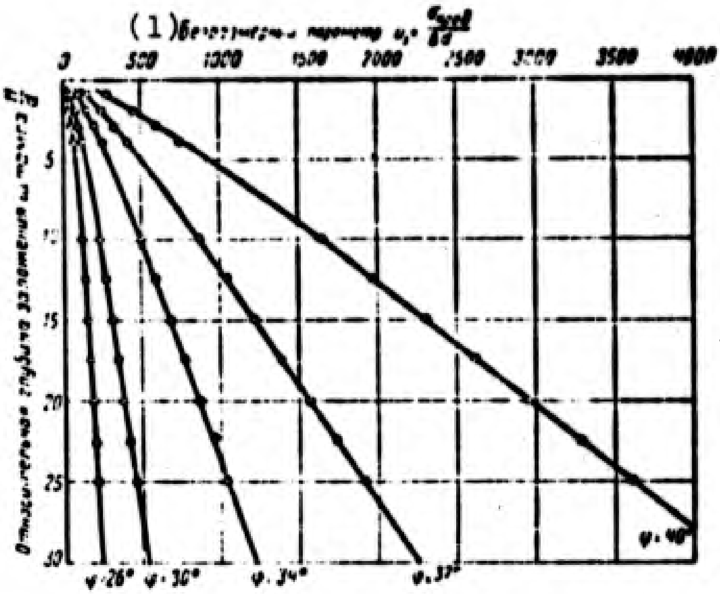
The theoretical dependence for determining the ultimate pressure $\sigma_{пред}$ in the 2nd case of the limiting condition of a sand base, which corresponds to the interaction of the zones of shear and zones of compaction above the bottom of the punch, was obtained by V. G. Berezantsev in the form [4]:

$$\sigma_{пред} = U_1 \Delta d \text{ kg/cm}^2, \qquad (40)$$

where the dimensionless bearing capacity factor of the soil U_1 is a function of both the angle of internal friction $\phi°$, and the relative depth of the punch impression H/d (Fig. 19).

The approximate dependence (40) was revealed by V. G. Berezantsev on the basis of a binomial formula $\sigma_{пред} = \frac{U_1}{2}\Delta d + V_1 q_{тр}$ by determining the intensity of surcharge weight at the level of the bottom of the punch $q_{тр}$ taking into account the frictional forces, which act along the cylindrical surface, and which confine the volume of soil located above the areas of shear of the base.

Value $q_{тр}$ is expressed in the form $q_{тр}=\Delta d f(\varphi^\circ, H/d)$; therefore, the dimensionless coefficient U_1 is equal to: $U_1=\frac{U_2}{2}+V_1 f(\varphi^\circ, H/d)$ [4].



(2)

Fig. 19. Graphs of the theoretical dependence between dimensionless parameter U_1 and the relative depth of the punch impression H/d, obtained by V. G. Berezantsev for the second case of a limiting condition under the conditions ϕ° = const.
KEY: (1) Dimensionless parameter $U_1 = \frac{б_{кред}}{\Delta d}$; (2) Relative depth of the punch impression $\frac{H}{d}$.

It is easy to see that when H/d > 8 the graphs of dependence $U_1 = f(H/d)$ when ϕ° = const, in the first place, are linear and, in the second place, that the constancy of the angular coefficients of the graphs of the linear dependence $U_1 = f(H/d)$ corresponds to the constancy of the angle of internal friction of the investigated sand ϕ° = const.

Thus, as an objective characteristic, which determines the change in the angle of internal friction of sand, as already mentioned, it is most correct to accept the slope of straight lines $U_1 = f(H/d)$ to the axis of the oridnates, i.e.

$$V_0' = \frac{U_1'' - U_1'}{H_2/d - H_1/d} = \frac{\sigma''_{пред} - \sigma'_{пред}}{\Delta (H_2 - H_1)} = \frac{4(P_2 - P_1)}{\pi d^2 \Delta (H_2 - H_1)}. \qquad (41)$$

The theoretical graph of dependence $V_0' = f(\phi°)$, plotted according to V. G. Berezantsev, is represented in Fig. 20 (curve 1).

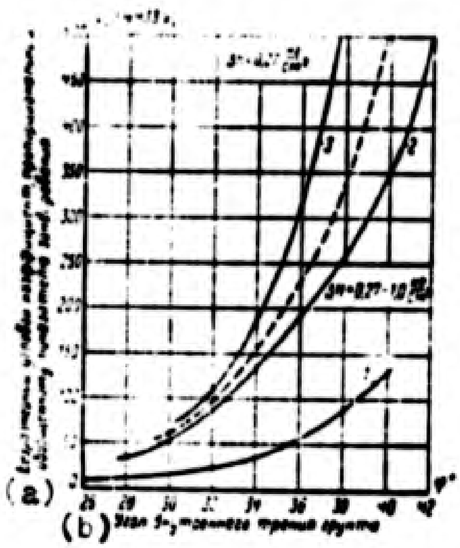


Fig. 20. Graphs of the theoretical dependence between the dimensionless coefficient V_0' and the angle of internal friction of sandy soils for the second case of a limiting condition. 1 - According to V. G. Berezantsev; 2 and 3 - according to V. A. Yaroshenko, taking into account the curvilinearity of the envelope of Mohr's limiting circles.
KEY: (a) Dimensionless angular coefficient proportional to the generalized sounding index; (b) angle of internal friction of the soil.
Designation: кг/см^2 = kg/cm^2.

The generalized index of sounding V_0 is proportional to value V_0', i.e., $V_0 = \pi\ tg^2\ \alpha/2\ V_0'$. For a conical point with the angle of expansion of 30° we will have:

$$V_0 = 0{,}226\, V_0' \qquad (42)$$

or

$$V_0' = 4{,}433\, V_0. \qquad (43)$$

The conducted experimental investigations, however, showed that the angle of internal friction in sandy soils depends completely on the intensity of surcharge weight [5, 57]. Ignorin this circumstance, according to V. A. Yaroshenko, can lead to a miscalculation of the limit loads by as much as 500%. Thus, when determining the angle of internal friction in sandy soils from the results of static sounding it is necessary to consider the curvilinearity of the envelope of Mohr's limiting circle [57]. In this case with an increase in the surcharge weight the angle of internal friction of sand will decrease and when $\Delta H \geq 1$ kg/cm^2 a constant value $\phi^{\circ}_{\text{мин}}$ = const will be reached.

In the first approximation, V. A. Yaroshenko accepted the logarithmic dependence $\phi_1 = \phi - 7.25 \lg (\Delta H)_1$ with a constant angular coefficient 7.25. In this case, for example, when $\sigma =$ $= 11$ kg/cm^2, $\phi_0 = 26°$, at $\sigma = 150$ kg/cm^2, $\phi_0 = 34°$ and at $\sigma = 480$ kg/cm^2 $\phi_0 = 40°$. The nature of dependence $\sigma = f(\phi°)$ for the case ΔH = const in V. A. Yaroshenko's work is not explained. According to the table of the accepted values of σ and $\phi°$, based on V. A. Yaroshenko's work [57], it is possible to judge that the ratio on the average is $\frac{\lg \phi_2/\phi_1}{\lg \sigma_2 \sigma_1} = 0.106$.

The reorganization of dependences $\sigma = f(\Delta H, \phi°)$, recommended by V. A. Yaroshenko, with the coordinates of the "mean pressure along the base of the point σ - pressure of the surcharge weight ΔH" at $\phi°$ = const, determined the strict linear dependence of the obtained graphs, which corresponds to the theoretical relationships, presented in Fig. 19. As an example in Fig. 21 a family of graphs $\sigma = f(\Delta H)$ is presented for the different values of the angles of internal friction of sand ϕ_1 = const with $\Delta H = 0.27$-1 kg/cm^2. An analogous system of graphs with large values of dimensionless angular coefficients V_0' exists when $\Delta H \leq 0.27$ kg/cm^2.

The empirical graphic representations of the dependence $V_0' = f(\phi°)$, plotted according to V. A. Yaroshenko for $\Delta H = 0.27$-1 kg/cm^2 and $\Delta H < 0.27$ kg/cm^2, are depicted in Fig. 20 (curves 2 and 3).

Taking into account the certain conditionality for the separation of the dependences $V_0' = f(\phi°)$ for the intensities of the surcharge weights greater and less than 0.27 kg/cm^2, apparently, it will be advantageous to make an estimate of the variability of the angle of internal friction of the sand prior to refinement for the averaged graph $V_0' = f(\phi^\circ)$, shown in Fig. 20 with a dotted line.

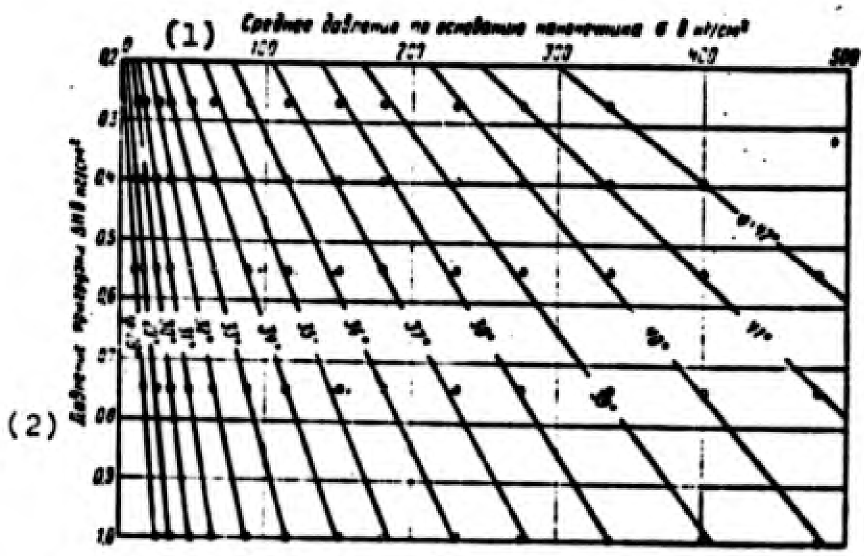


Fig. 21. Calculated graphic representations of the dependence between the mean pressure along the base of the conical point and the pressure of the surcharge weight under the conditions ϕ° = const (according to V. A. Yaroshenko).
KEY: (1) Mean pressure along the base of point σ in kg/cm^2; (2) Pressure of the surcharge weight ΔH in kg/cm^2.

Analysis of the results of the static sounding of noncohesive soils. For the purpose of an experimental check on the theoretical regularities about the change in resistance to sounding in sand according to depth experimental investigations were initiated in 1962-1963 on the shoreline of the Dnieper river in the area of Kremenchug city under the guidance of the author. Typical graphic representations of $h = f(P)$, obtained by N. L. Zotsenko, are given in Fig. 22a. The sounding was made using a mechanized MP-3 model with a conical point having a height of $h_{кон} = 9$ cm, a diameter of $d_{кон} = 4.83$ cm when $d_{кон}/d_{шт} = 1.5$. The initial sections of the sounding were not recorded to a depth of 15-22.5 cm in these tests.

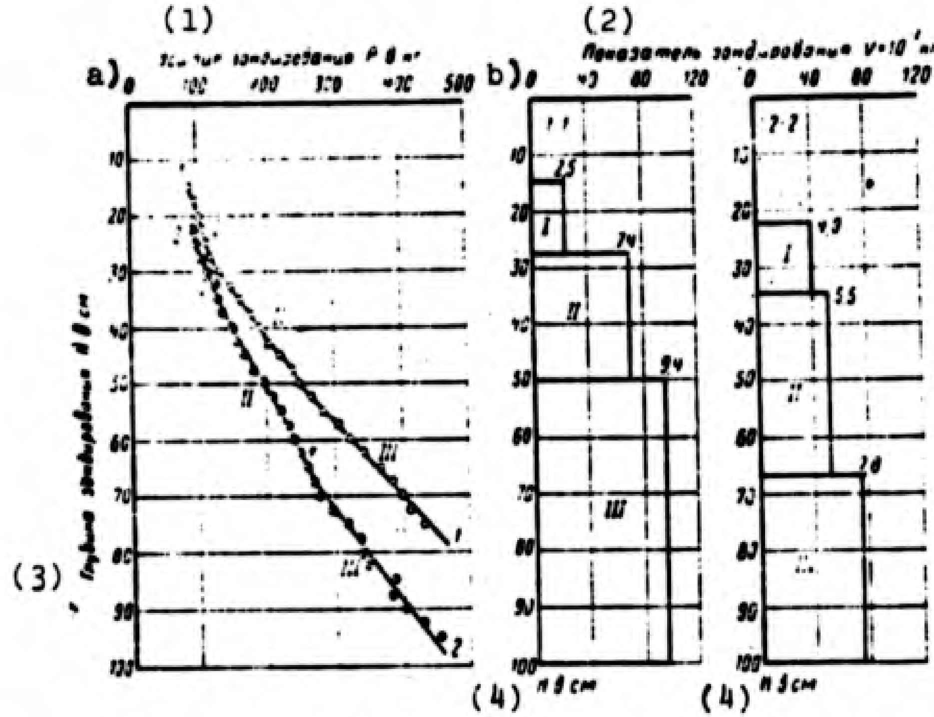


Fig. 22. Graphs and diagrams of the static sounding of the surface layer of a fine-grained sand, obtained by N. L. Zotsenko.
KEY: (1) Force of sounding P in kg; (2) Sounding index $V \cdot 10^{-2}$ kg/cm.^{3}; (3) Sounding depth H in cm; (4) H in cm.

It is easy to see that the average results of the sounding of the surface layer of sand are actually depicted in the graph of Fig. 22a in the form of two conjugate segments of straight lines 1 and 2, which characterize the linear increase in the force of sounding with an increase in natural pressure within the limits of uniform layer.

With a change in the grain-size distribution and volume weight of the sand skeleton the slope angle of the corresponding segment of the straight line to the vertical axis changes regularly.

The very essential feature of these analyses is the absence of an especially compacted sand during the insertion of the probe. Thus, for instance the filling of the cavity with sand was always observed which is formed during the insertion of the point using a diameter substantially greater than the diameter of the rod.

The volume of the cavity to the height of the conical point was 88.5 cm^3, or 53.5% of the volume of the sand being displaced.

In accordance with the graphs in Fig. 22a the results of the sounding are interpreted in the form of three separate layers I, II, and III, which were determined by the values of the sounding index from 2.5 to 9.4×10^{-2} kg/cm^3, which is the same as the values of the generalized sounding index from 12.5 to 47 (with the mean volume weight of the sand $\Delta = 2.01$ g/cm^3).

The average results of the sounding according to the separate layers are depicted in Fig. 22b in the form of diagrams of static sounding. For example, at a depth of sounding of 50-100 cm, i.e., when ΔH = 0.1-0.2 kg/cm^2, the sounding index changed in the range of 7.8×10^{-2} to 9.4×10^{-2} kg/cm^3. Consequently, the generalized sounding index V_0 = 39-47 or auxiliary coefficient V_0' in formula (43) was 173-208. Based on the dashed-line graph in Fig. 20 we find that the angle of internal friction of sand $\phi°$ = 34°30' to 35°30'.

Analogous results of static sounding of sands are given in G. Plantema's report (G. Plantema, report No. 2/6) at the IVth Congress on Soils Mechanics and Foundation Engineering [70].

The presentation of this report was included in the review by Prof. N. N. Maslov. The tests were carried out in a steel drum with a diameter of 1 m and a height of 0.5 m using a penetrometer with a conical point having $\alpha = 60°$. The results of the sounding in dry, moist and water-saturated sand at the coordinates of the "specific pressure σ - depth of sounding H" are depicted as clear straight lines (Fig. 23). As a characteristic of the results G. Plantema's sounding takes the "angle of deflection" of the straight line from the vertical axis α_0, measured in degrees. The sounding index V is equal to the product of the constant coefficient π tg $\alpha/2 = 1.813$, times the tangent of the "angle of deflection α_0," i.e., $V = 1.813$ tg α_0.

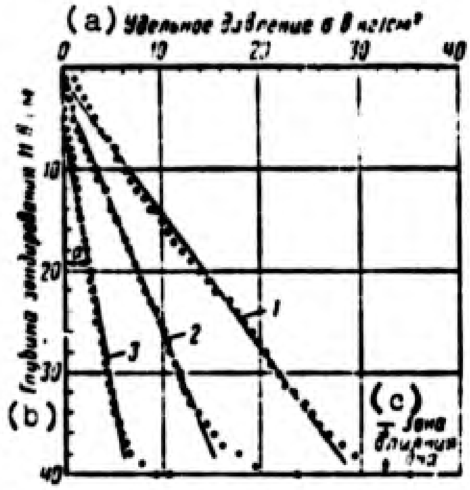


Fig. 23. Graphs of the static sounding of a fine-grained sand, obtained by G. Plantema. 1 - Moist; 2 - dry sand; 3 - water-saturated sand. KEY: (a) Specific pressure σ in kg/cm^2; (b) Depth of sounding H in cm; (c) Zone of the bottom effect.

Linear increase in the mean pressure at the tip of the pile model and the head resistance of the metallic casing d = 32.5 cm was noted by Yu. I. Kovalev during the driving of the pile and casing in the sandy soil to depths of 1.2 and 5.2 m, respectively.

In Prof. N. A. Tsytovich's survey on materials for the Vth International Congress on Soil Mechanics and Foundation Engineering (Paris, 1961) the importance of the "uniquely placed" experiments of Prof. Zh. Kerizel' in his research on the ultimate strength of a sandy soil at the tip of probes and piles of different diameters is noteworthy. These analyses were presented in report No. 38/12 for the Vth Congress and were highlighted in the report at the Conference in Mexico [68] on deep placement of foundations.

The investigations were carried out with extreme care in a special experimental reinforced concrete pit having a diameter of 6.4 m and a depth of 10.25 m, with a volume of 335 m^3. The layering of the pit with sand was achieved by three foremen using vibrators which provide the minimum deviation from the specified degree of volume weight of the soil skeleton - δ. The service life of one filling of the pit was approximately 2 months. The considerable attention given to this question on the part of Zh. Kerizel' regarding the four year old experiments by the foremen in filling the pit also made it possible to achieve a high degree of perfection. Thus, for instance, in one of the last fillings an accuracy within the limits of $\delta = 1.680 \pm 0.005$ g/cm^3 for compacting the sand was obtained, which is illustrated in a report by Zh. Kerizel' using the graph $\delta = f(H)$, plotted from the results of 9 parallel determinations in each of the 37 investigated layers [68].

The soundings were made in uniform sands with three different values for volume weights of the skeleton, $\delta_{\text{мин}} = 1.58$ g/cm^3, $\delta_{\text{ср}} = 1.685$ g/cm^3 and $\delta_{\text{макс}} = 1.78$ g/cm^3. Steel piles, 45, 110, 216 and 320 mm in diameter, were driven in the sand base with the aid of a hydraulic jack having a load-lifting capacity of 200 t.

In another series of experiments piles with a diameter of 600, 1000 and even 1500 mm were driven. With the use of a second hydraulic jack both the full force of the sounding, as well as the force necessary to drive in only the tip of the pile were measured.

The basic results of the conducted analyses with piles 45-320 mm in diameter are represented in the interpretation by Zh. Kerizel' in Fig. 24. Along the axis of the abscissae the conditional mean pressure necessary per unit of cross-sectional area of the tip of the pile, expressed in bars is plotted.[1] The main conclusions based on the results of these analyses are as follows.

The results of static soundings, made in sands, unconsolidated ($\delta_{\text{мин}} = 1.58$ g/cm^3) and at an average density ($\delta_{\text{ср}} = 1.685$ g/cm^3),

---

[1]Let us recall that 1 bar $= \frac{10^5 \text{ N}}{\text{m}^2} = 10$ N/cm^2. In accordance with the system of unity SI 1, N $= 1$ kg·m/s^2, therefore, 1 kG = = 9.80665 N. Thus, approximately, 1 bar $\approx$ 1 kG/cm^2.

turned out to be almost analogous. In both series of experiments the rapid attainment of the critical depth of sounding $H_{кр}$, corresponding to the stabilization of the effect of surcharge weight is noted, i.e., to the constancy of the force of sounding $P = \text{const}$ or $\sigma = \text{const}$. In this case the clear development of the invariance of the values of the mean pressure is observed - with a change in the diameter of pile 2.9-5.2 times that of the value of the mean pressure completely coincided (experiments of the 1st series) or almost coincided (experiments of the 2nd series).

The results of static soundings, produced in sands of average density and in compacted sands ($\delta_{макс} = 1.75$ g/cm^3), were substantially different. In compacted sands the critical depth of sounding fully depends on the diameter of the pile, whereupon the greater the diameter of the pile, the greater is $H_{кр}$.

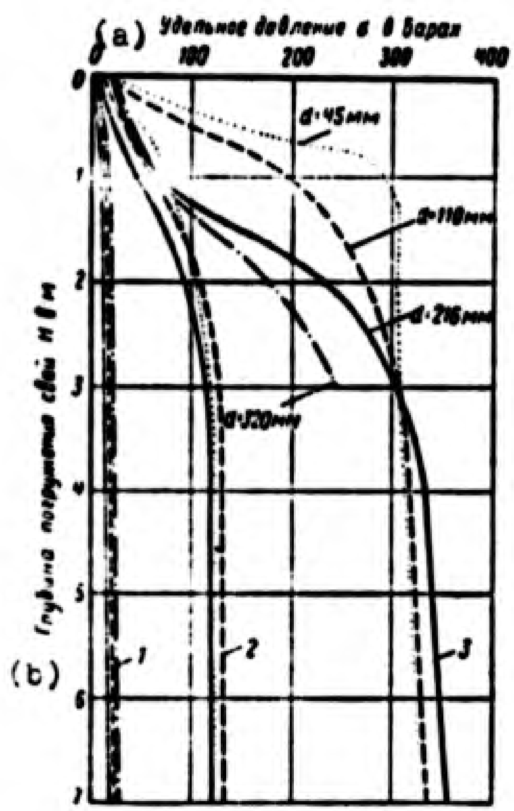


Fig. 24. Graphs of the static sounding of sand obtained by Prof. Zh. Kerizel'. 1 - Test in unconsolidated sand, $\delta = 1.58$ g/cm^3; 2 - Test in a sand of average density, $\delta = 1.685$ g/cm^3; 3 - test in compact sand, $\delta = 1.78$ g/cm^3. KEY: (a) Specific pressure σ in bars; (b) Depth of the pile driving H in m.

Consequently, in compacted sands the invariance of values σ unlike two previous series of experiments prior to the achievement of the characteristic depth of sounding, is not observed. After the achievement of $H_{кр}$, the values of $\sigma_{макс}$ proved to be sufficiently close.

The values of the critical depths of sounding $H_{кр}$, and of the maximum values of the mean pressure $\sigma_{макс}$, which are revealed upon reaching $H_{кр}$, and other characteristics are given in Table 4.

Table 4. The basic results of experimental research on the static sounding of piles.

Series of experiments	Volume weight of the sand skeleton, δ in g/cm^3	Angle of internal friction ϕ°	Characteristic depth of sounding, $H_{кр}$ in m	Maximum value $\sigma_{макс}$ upon reaching $H_{кр}$ in bars	$\Delta V'_0 = \frac{\sigma_2 - \sigma_1}{H_2 - H_1}$	V'_0
1	1.58	35	1.8	20	1.15	7
2	1.685	38.5	3.2	120	1.3	8
3	1.75	42	4* 1**	340	9.5*	54*

*For results of sounding of a pile 216 mm in diameter.

**For results of sounding of a pile 45 mm in diameter.

The values of coefficients $\Delta V'_0$ and V'_0 [see formula (41)] were calculated by Zh. Kerizel' for the characteristic of the intensity of an increase in the head resistance of sand at depths which exceed $H_{кр}$. In this connection Zh. Kerizel' notes that in the 1st and 2nd series of experiments the intensity of the increase is "almost imperceptible" and it becomes "more significant" only for the 3rd series of experiments. It is easy to see that in this case the coefficients proportional to the sounding index V, and to the generalized sounding index v_0 serve as the objective of the characteristics of the results of static sounding. Thus, an analogous approach to an estimate of the results of static sounding of sands has also been found in foreign research.

The results of driving piles 600, 1000 and 1500 mm in diameter (report of the Vth Congress by Zh. Kerizel') were partially published by N. A. Tsytovich [51], Yu. G. Trofimenkov, L. N. Vorobkov, et al. [49]. One of the main conclusions for this part of the experiments lies in the fact that during the driving of piles of greater diameter in sands of average density and in compact sands a linear increase in head resistance of sounding is distinctly observed with depth, i.e., strict constancy of the values of sounding in index and generalized sounding index. Moreover, in both cases the characteristic depth of sounding within the limits of the depth of the experimental pit (7 m) was not reached. The fundamental form of these experimental graphs completely coincides with the theoretical graphic representations $U_1 = f(H/d)$ for the second case of a limiting condition presented in Fig. 19.

The examined theoretical reasons and the given examples of the test data provides a basis for the following recommendations regarding the interpretation of the results of static sounding in sandy soils.

1. Depending on the relative density of sand, relative depth of indentation of the probe and the ratio of the diameter of the probe to the diameter of the rod, the insertion of the probe occurs because of the interaction of the zones of shear and the zones of compaction above as well as below the base of the conical part of the probe.

The existing development of the theory of the static sounding of sandy soils makes it possible to fully interpret the first and second cases of a limiting condition, characterized by a linear increase in the force of sounding with the depth of indentation of the probe. In this case the values of the generalized sounding index V_0, interrelated with the angle of internal friction of the soil by the graphs depicted in Fig. 20 are taken as the objective characteristics of the results of sounding.

2. For the sounding of heterogeneous sandy soils the graphic representations of the obtained results are depicted at the coordinates of the "force of sounding P - depth of indentation of the probe H" as a system of conjugate segments of straight lines. The constancy of the generalized sounding index V_0 within the limits of the selected uniform layers makes it possible to plot the resultant test data at coordinates V_0 - H - for the diagram of static sounding. In this case the ordinates of points of the conjugate separate straight lines determine the depths of the selected uniform layers.

3. The quite frequently observed case of soil compaction below the base of the conical part of the probe is objectively characterized by the constancy of the force of sounding, usually independent or barely depending on the depth of indentation of the probe. In this characteristic case of sounding Q, and the value proportional to it - the conditional mean pressure at the base of the conical point of the probe σ, is exhibited.

Analysis of the results of static sounding in cohesive soils. The penetration tests turn out to be especially effective during the analysis of the mechanical properties of comparatively uniform cohesive soils. In this case the test evaluation is reduced to plotting linear graphs of the invariance of resistivity to penetration, which in a number of cases can be successfully made during the course of testing itself. Under conditions of a stratified base the test evaluation, as shown above, turns out to be more laborious and is connected with the reorganization of graphs $P = f(h^2)$ or $P = f(h^3)$ at new coordinates.

In comparison with penetration tests in cohesive soils by the methods of static and dynamic sounding, an incomparably greater depth was reached. It is natural that in connection with these methods the case of stratified soils is more typical, when within the limits of the depth of soundings several layers with more or less different characteristics of mechanical properties can be isolated.

In this connection the presentation of methods for interpreting the results of static sounding in cohesive soils should begin with the analysis of variability of the values of resistivity to sounding using a stratified base.

As an example given in Fig. 25 are the results of static sounding, made with a mechanized MP-1 VSEGINGEO model in the surface layer of a clay loam mantle in the vicinity of Nakhabino settlement, Moscow oblast. The sounding was carried out using a conical point with the angle of expansion of 30° and a height of $h_{кон} = 9$ cm.

The initial segment 0-0 on the graph of Fig. 25a carried the force of sounding within the limits of accuracy of measurement ~10 kg. In segment 0-1 a parabolic dependence, characteristic for the case of the inertion of a conical point under conditions of penetration was observed. At point 1 the sounding was initiated with a constancy of the applied force. Segments 2-3 and 4-5, equal to the height of the indentor, characterize the transient parabolic dependence which appears during the insertion of point into a soil layer of greater strength. Segments 3-4 and 5-6 represent the results of sounding in a uniform soil.

In the investigated example three different layers of soil are maked off with $Q_1 = 1.02$ kg/cm^2, $Q_2 = 3.05$ kg/cm^2 and $Q_3 = 4.34$ kg/cm^2 (Fig. 25b).

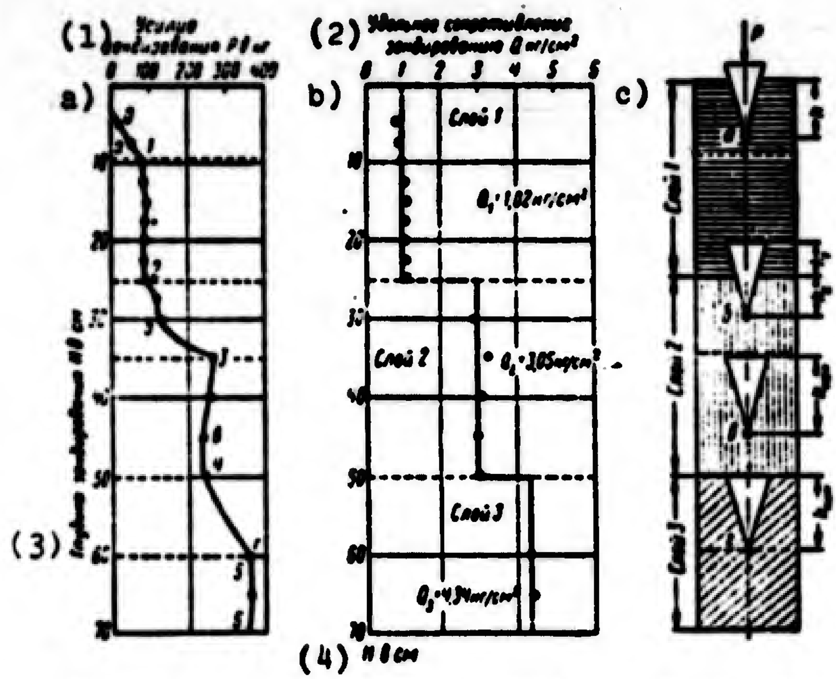


Fig. 25. The static sounding in the surface layer of a clay loam mantle. a) Graph; b) projection diagram; c) diagram.
KEY: (1) The force of sounding P in kg; (2) Resistivity to sounding Q kg/cm^2; (3) Depth of sounding H in cm; (4). H in cm.
[Слой = Layer]
Designation: кг/см^2 = kg/cm^2.

The equation of the transient parabolic dependence, observed under conditions when there are two different layers of soil within the limits of the height of the indentor, is revealed, according to G. V. Zhornik's proposal, by analogy with a similar case under conditions of penetration. As soon as the tip of the conical indentor is driven to a height h_2 into a layer of soil with other characteristics of mechanical properties, the force of sounding is subject to a change (point 6 in Fig. 25a).

In this case between force P and the resistivity to sounding the following parabolic dependence can be established in the adjacent layers Q_1 and Q_2:

$$P = Q_1 h_{\text{кон}}^2 + (Q_2 - Q_1) h_2^2 \text{ kg}. \qquad (44)$$

At $h_2 = 0$ we obtain the equation (34) $P_1 = Q_1 h^2_{кон}$ (point в in Fig. 25a). At $h_2 = h_{кон}$ we will have the same equation (34) for the resistivity to sounding Q_2, namely: $P_2 = Q_2 h^2_{кон}$ (point г in Fig. 25a).

The calculation of values of Q_2 within the limits of the transition zone, equal to the height of the cone, if necessary can be conducted according to the formula

$$Q_2 = \frac{P - Q_1 (h^2_{кон} - h^2_2)}{h^2_2} \text{ kg/cm}^2. \qquad (45)$$

The parabolic form of the graph within the limits of transition zone serves as the criterion of the beginning of a new layer. Knowing that the height of the transition zone is equal to the height of the cone, it is easy to establish, for example, the position of points 2 and 4 from the graphic representation in Fig. 25a.

Consequently, the determination of the levels of the individual layers does not usually produce difficulties.

Under normal conditions when $h_{слня} > h_{кон}$ the transient segment is equal to the height of the cone; at $h_{слоя} < h_{кон}$ the transient segment increases considerably and, as can be easily shown, it is equal to $h_{кон} + h_{слоя}$.

Figure 26 gives an example of the development of the invariance of resistivity to sounding within the range of the uniform layers of soil. Under these conditions, as already mentioned, the force of sounding remains constant, and thus, a change in the natural pressure does not exert a noticeable effect on the results of sounding.

The soundings were made using a mechanized MP-1 VSEGINGEO model in the surface layer of a clay loam mantle of stiff plastic and semisolid consistencies ($Q = 0.77$–1.22 kg/cm^2). A conical point, 8.65 cm high, was used.

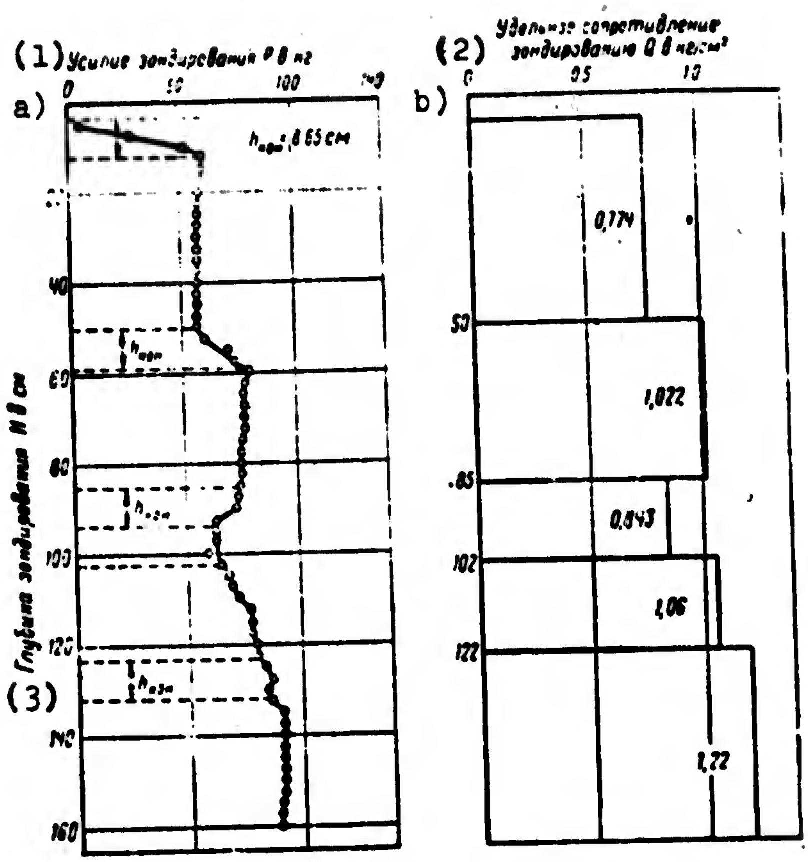


Fig. 26. Static sounding of a surface layer of clay loam (conical indentor $h_{кон}$ = 8.65 cm).

a) Graph; b) projection diagram.
KEY: (1) Depth of sounding P in kg; (2) Specific resistance to sounding Q in kg/cm^2; (3) Depth of sounding H in cm.

From the examination of the graph and projection diagram of the sounding in Fig. 26 it was shown that in these tests a sufficiently high accuracy of measurement of the forces and depth of indentation of the point was obtained. Specifically, for example, the height of all segments with the development of the transient parabolic dependence (44) between the separate layers coincided well with the height of the conical points.

The given theoretical reasons and the examined examples make it possible to recommend the following simple criterion for the objective interpretation of the results of static sounding in cohesive soils.

1. In the surface layer of soil, equal to the height of a conical point ($h \leq h_{кон}$), the initial section of the penetration characterized by the parabolic configuration of the graph $P = Rh^2 + P_0$, concave to the horizontal axis is easily observed.

When $h = h_{кон}$ the sounding begins, and the calculation of R and Q according to formulas (5) and (34) leads to coinciding results.

2. Within the limits of the uniform layer of cohesive soils the force of sounding remains constant and therefore the resistivity to sounding does not change.

3. At the interface between two uniform layers with different characteristics of mechanical properties a transient section, equal to the height of the conical point and characterized by a parabolic configuration of the graph is always distinctly observed.

The transient section appears as a result of the passage of the cone through two layers simultaneously.

4. If within the limits of the depth of sounding there are intermediate layers of soil, less than the height of the conical tip $h_{слоя} < h_{кон}$, then the value of transient section can turn out to be greater than the height of the cone and equal to $h_{кон} + h_{слоя}$. In order to determine the resistivity of a layer of soil to sounding formula (45) is utilized.

5. In certain cases one cannot rule out the possibility where a regular increase or decrease in the characteristics of mechanical properties may be deserved in a group of separate consecutive layers of cohesive soils commensurable with the height of the conical point.

In these cases the results of sounding tests can be interpreted in the form of the overall layer with the disclosed regularity of an increase or decrease in the values of resistivity to sounding.

Comparison of the results of sounding tests in cohesive soils. The accuracy and objectivity of the results of sounding tests in cohesive soils using different expansion angles of the conical indentors, different diameters of indentors $d_{кон}$ and different ratios of the diameters of the indentors and rod $d_{кон}/d_{шт}$ were checked by the following two methods:

a) by means of comparing the results of a group of parallel soundings and b) by comparing the results of sounding with the results of penetration tests from the surface of the soil and at separate selected horizons in the excavations.

As an example Fig. 27 depicts four graphs of the results of static sounding of the surface layer of water-saturated, fine clayey and silty deposits (M_p = 17.6, w_r = 48.7%), made by L. D. Martynova using conical points, $d_{кон}$ = 79 mm in diameter and with expansion angles 30, 60, 90 and 120°. It is easy to see that within the limits of the depth of sounding two uniform layers from 0 to 80 cm and from 80 to 175-190 cm can be distinguished. The control determinations of the volume weight Δ and moisture content w completely confirmed the results of sounding - in the first layer mean values Δ_1 = 1.8 g/cm^3 and w_1 = 40.4% were obtained; in the second layer, Δ_2 = 1.69 g/cm^3 and w_2 = 52.6%.

From Fig. 27 it also directly follows that the presence of the invariance of the force of sounding with respect to the angle of expansion of the conical point - within the boundary of the uniform layer of soil, the forces of sounding hardly change.

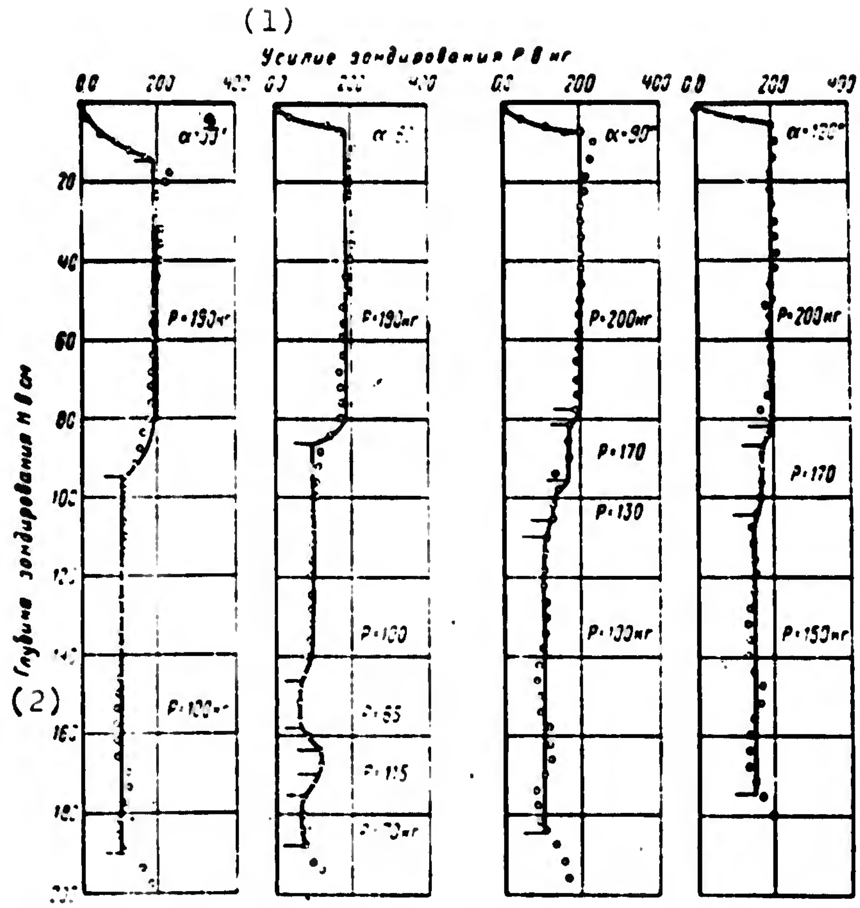


Fig. 27. Graphs of the parallel static soundings of the surface layer of the fine silty and clayey soils made using conical indentors with different expansion angles.
KEY: (1) Force of sounding P in kg; (2) Depth of sounding H in cm.
Designation: кг = kg.

The mass sounding tests of the surface layer of fine silty and clayey deposits (the sounding ran for 453 linear m) made it possible to test the possibility of using formula (17) for calculating the proportionality factors $k_\alpha = Q_{30}/Q_\alpha$ (when sounding in cohesive soils) or $k_\alpha = V_{30}/V_\alpha$ (when sounding in noncohesive soils). A comparison of the calculated and mean experimental values of the proportionality factors k_α, obtained when sounding in fine silty and clayey sediments under conditions $d_{\text{нон}}$ = const, is given in Table 5.

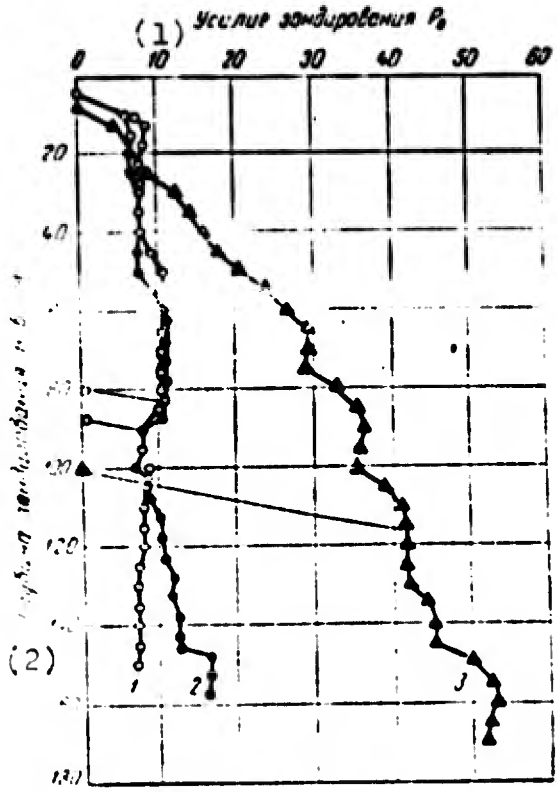
Table 5. Calculated and mean experimental values of the proportionality factors of k_α.

Vertex angle of the conical indentor in deg.	Values of the coefficient k_α according to the condition of invariance P	Mean experimental values of k_α	Number of determinations
30	1	1	53
37	0.641	0.623	38
45	0.418	0.393	51
60	0.215	0.221	66
75	0.122	0.129	56
90	0.072	0.068	82
120	0.024	0.025	68

The mean coefficient of the variation in the mean experimental values of k_α amounted to $V = \pm 13.2\%$. From the data in Table 5 a complete agreement of the calculated and experimental values of the proportionality factors of k_α follows.

The parallel soundings of the fine silty and clayey deposits using conical indentors with angle of expansion of 30°, but with different diameters (71, 79, 98, 118 and 140 mm) also confirmed the sufficient convergence of the values of resistivity to sounding Q. In connection with graphs $H = f(P)$ in Fig. 27 the resistivity to the sounding of the first layer of soil during the testing using indentors with different diameters amounted to $Q_1 = 0.87 \pm 0.16$ kg/cm^2 (coefficient of variation $V = 18.4\%$); for the second layer, $Q_2 = 0.58 \pm 0.12$ kg/cm^2 ($V = 20.7\%$).

Consequently, in water-saturated cohesive soils when $d_{кон}/d_{шт} > 1.6$, a doubling of the diameter of the indentor does not produce a change in the values of resistivity to sounding, $Q = \text{const}$.



Графики (3)	$d_{кон}/d_{шт}$	$h_{кон}$	$d_{кон}$	$d_{шт}$
		в см		
1 и 2	1,76	8,65	4,76	2,7
3	1	5,1	2,69	2,7

Fig. 28. **Graphs** of the parallel static soundings of a sandy loam soil layer obtained when using conical indentors at different ratios of $d_{кон}/d_{шт}$.

KEY: (1) Force of sounding P_0; (2) Depth of sounding H in cm; (3) Graphs. [в см = in cm; и = and]

The sounding soil tests when $d_{кон} = d_{шт}$ or $d_{кон}/d_{шт} = 1$ lead to completely different results even at relatively shallow depths of sounding (up to 1 m).

Following from Fig. 28, as a result of two sounding tests of the sandy loam soil layer when $d_{кон}/d_{шт} = 1.76$ the presence of sufficient uniform soils to a depth up to 160 cm was established. Meanwhile, the third parallel sounding, made when $d_{кон}/d_{шт} = 1$, at the interface between the two first soundings, led to a considerable exaggeration of the required force. For example, when H = 150 cm the force of sounding in the first two tests was from 13 to 30% of the maximum force in the third comparative test.

An analogous phenomenon was observed when sounding in Cambrian clay in the region of Gorelovo settlement in Leningrad oblast under conditions of $\frac{d_{кон}}{d_{шт}} = \frac{2.45}{2.45} = 1.0$ and $\frac{d_{кон}}{d_{шт}} = \frac{4.71}{2.45} = 1.92$ (Fig. 29).

In the first case a clearly expressed linear dependence $Q = Q_0 + k_Q h$, which characterizes an increase in the force of sounding is observed because of an increase in the frictional forces along the lateral surface of the rod.

The revealed nature of the dependence makes it possible to calculate easily the mean specific force intensity of friction over the lateral surface of the rod, which according to the results of these tests, actually proved to be approximately constant and was 0.35-0.5 kg/cm^2.

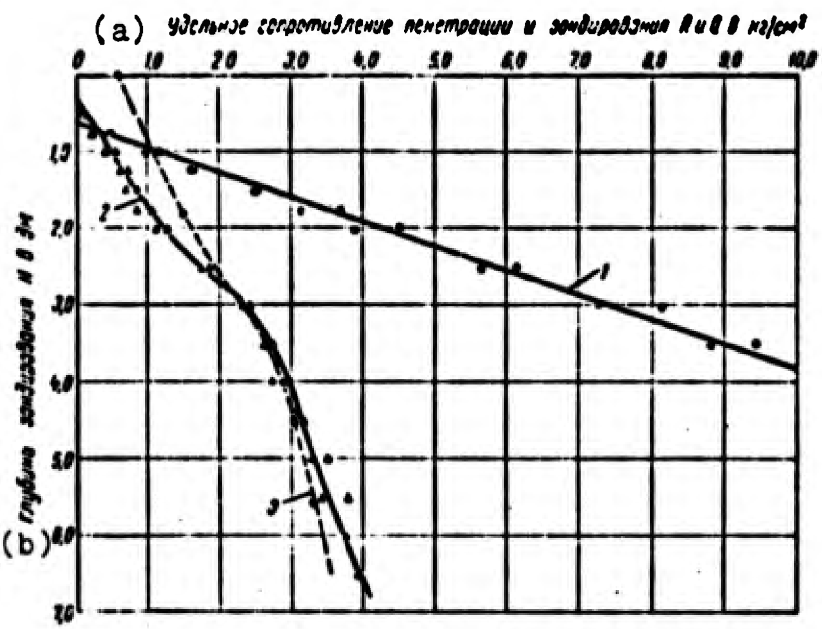


Fig. 29. Graphs of parallel static soundings and penetration tests in the surface layer of Cambrian clay, obtained when using conical indentors with a different ratio of $d_{кон}/d_{шт}$. 1 - Sounding when $d_{кон} = d_{шт}$; 2 - the same when $d_{кон} > d_{шт}$; 3 - penetration at the exposed horizons.
KEY: (a) Resistivity to penetration and sounding R and Q in kg/cm^2; (b) Depth of sounding H in dm.

It is quite obvious that the values of resistivity to sounding, calculated for this case according to formula (34), are knowingly erroneous and have nothing in general with the real values of Q at the appropriate depths. In this case it is easy to be convinced, once having compared the obtained graph with the values of the resistivity to penetration R, established on the surface of the soil and at depths of 16, 37 and 56 cm, with the aid of mechanized and manual hydraulic penetrometers after excavating the pit (Fig. 29).

On the other hand, in the second case when $d_{кон} = 1.92\ d_{шт} = 4.71$ cm the frictional forces along the lateral surface of the rod were completely absent. Thus, the resistivity to sounding, obtained when using this indentor ($d_{кон} = 4.71$ cm, $h_{кон} = 8.88$ cm), following from Fig. 29, completely coincided with the results of a layer-by-layer determination of the resistivity to penetration R (let us note that the regular increase in the specific resistivity to sounding with depth is in this case the consequence of a regular decrease in the moisture content from 24.6 down to 20.7%, and corresponding, an increase in the volumetric soil skeleton from 1.59 to 1.71 g/cm^3).

The simplest mathod for the elimination or, in an extreme case, for a considerable reduction in the frictional forces along the lateral surface of the rod is an increase in the diameter of the conical indentor by 1.6-1.9 times in comparison with the diameter of the rod. O. M. Reznikov, who recommended the use of the ratic $d_{кон}/d_{шт} = 1.7$ in sounding, came to the same conclusion.

Methods of expressing the soil test data from dynamic sounding. Dynamic sounding is the method of studying the physicomechanical conditions of the soil by determining the resistance of soil to the insertion of probe under the action of the consecutively increasing number of shocks of a hammer with a fixed weight, falling from a fixed height.

The features of the method of dynamic sounding are sufficiently examined in detail in the studies of V. A. Durante [23], Yu. G. Trofimenkov, L. N. Vorobkov, et. al [49], S. A. Shashkov [52, 53, 54] and other authors.

These works of Soviet specialists are sufficiently well known: therefore, set forth below are only certain recent positions according to the methods for interpreting the conducted tests.

The improvement of the procedure for interpreting the results of dynamic sounding was made taking into account the observed analogy between the process of inserting the probe and the features of deformation of cohesive soil during cyclic loading.

The known diagram of the deformation of a uniform cohesive soil during cyclic load is represented in Fig. 30 [17]. Under the action of loads P_1 and P_2 less than the limiting value $P_{пред}$, an equilibrium of an elastically compacted state of the cohesive soil will always be achieved, with which a subsequent increase in the number of cycles of the load does not increase the deformations of the soil, i.e., an increase in the deformation approaches zero (curves 1 and 2).

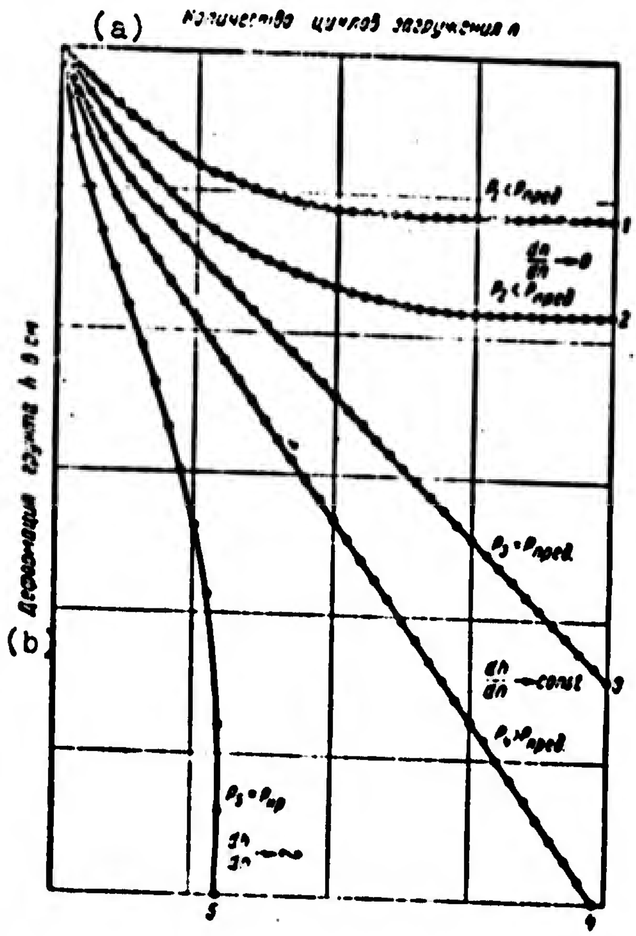


Fig. 30. Diagrams of the deformations of a uniform cohesive soil during cyclic loading under conditions of the absence of considerable soil compaction. 1 and 2 - Stabilization of the deformations of the soil; 3 and 4 - undamped or prolonged undamped deformations of soil; 5 - progressing deformations of the soil.
KEY: (a) Number of cycles of load n; (b) Deformation of the soil h in cm.

The ultimate load $P_{пред}$ is called the smallest load, at which the stabilization of the deformations of the soil no longer occurs. In this case as a result of the development of shear in the soil, an increase in the deformations from one cycle becomes a constant value (curve 3). A further increase in the load leads to a further increase in the deformations from one cycle (curve 4).

Under the action of a critical load $P_{кр}$ complete failure of soil occurs and the noted regularity is disrupted. Under these conditions an increase in the deformations of the soil from every subsequent cycle increases continuously.

Unlike the given diagram, dynamic sounding is accomplished with a constant impact impulse, and with variable characteristics of the mechanical properties of cohesive soils. Thus, the results of dynamic sounding are interpreted taking into account the following features.

1. The results of dynamic sounding are depicted at the coordinates "depth of indentation of the probe H - accumulating number of blows of the hammer n." In this case the graph $H = f(n)$ represents a system of conjugate segments of straight lines (Fig. 31). The mean increase in the depth of indentation of the probe from one blow is

$$S = \frac{H_2 - H_1}{n_2 - n_1} \text{ cm/blow} \qquad (46)$$

(i.e., the mean deformation intensity of a cohesive soil during sounding) or the average number of blows of the hammer N, necessary to drive the probe 1 dm:

$$N = 10\frac{n_2 - n_1}{H_2 - H_1} \text{ blows/dm} \qquad (47)$$

are the objective characteristics of the mechanical properties of the soil within the range of the selected uniform layers. In following presentations the values of N are called the "indexes of dynamic sounding."

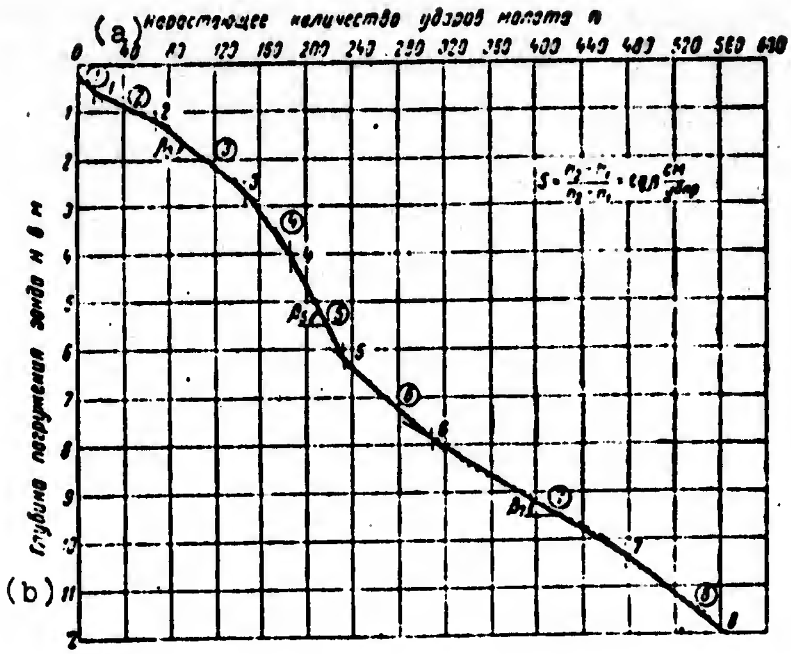


Fig. 31. Combined linear graphic representations of the dynamic sounding of loess-like clay loams with undisturbed structure.
KEY: (a) Increasing number of blows of the hammer n; (b) Depth of penetration of the probe H in m.
[см/удар = cm/blow].

2. The objective criterion of a uniform layer of soil throughout the depth of sounding is the criterion of constancy of the angular coefficient S = const or N = const in the following equations of the linear dependences

$$H_i = H_0 + S(n_i - n_0) \text{ cm} \tag{48}$$

or

$$n_i = n_0 + 0{,}1\, N(H_i - H_0) \text{ blows.} \tag{49}$$

3. A change in the angular coefficient S or N in the equations of linear dependences (48) or (49) means a change in the characteristics of the mechanical properties of soils throughout the depth of sounding, which in general is connected with changes in the composition and physical condition of the soil. The connecting points of the separate straight lines in Fig. 31 (1-8) identifies the position of the selected layers (the number of the layer is shown in the figure by a small circle) with an accuracy to within one-half of the height of the conical point, i.e., approximately ±2.5-3.5 cm.

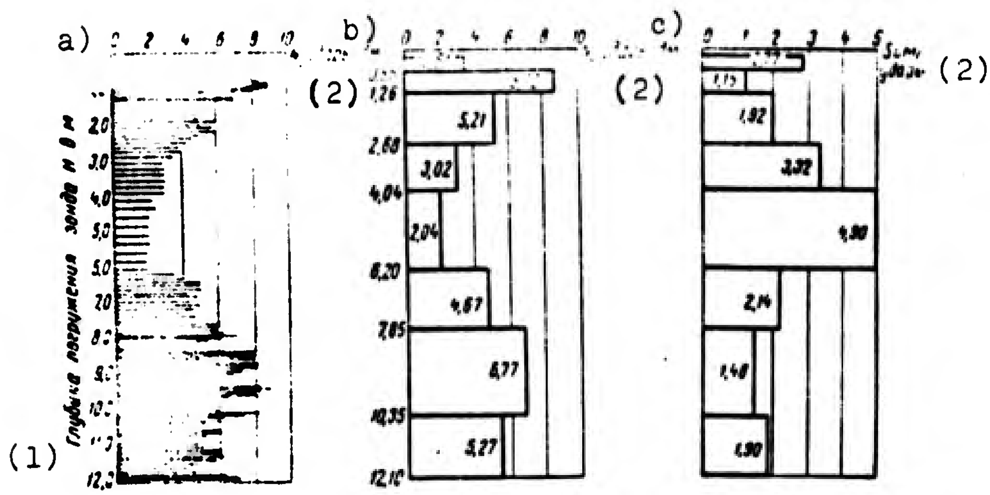


Fig. 32. Versions of projection diagrams of dynamic sounding. Designated along the axis of the abscissae are: a) number of blows, necessary to drive the probe 1 dm; b) average number of blows, necessary to drive the probe 1 dm; c) average increase in the depth of indentation of the probe from one blow.
KEY: (1) Depth of penetration of the probe H in m; (2) blows/dm.

4. The general results of dynamic sounding are represented in the form of a projection diagram of the average number of blows N, necessary to drive the probe 1 dm (Fig. 32b) or they are represented in the form of a projection diagram of the average deformation intensities of the soil from one blow S (Fig. 32c).

Both forms of projection diagrams are virtually equivalent. Depending on tradition or practice projection diagrams of the mean indexes of only N or S can be utilized in a further analysis.

The existing method on the technical specifications and on recording the initial processing of the results of dynamic sounding is illustrated by projection diagrams $N = f(S)$, presented in Fig. 32a. In this case the deformation intensity of the soil is estimated visually based on the frequency of horizontal lines along the vertical axis, and the values of the index of dynamic resistance of the soil throughout the depth of sounding N are depicted to scale in the form of a family of parallel lines of different length. From a comparison of the projection diagrams of dynamic sounding, depicted in Fig. 32a and b, it is not difficult to conclude that the existing method of processing the results of sounding is characterized mainly by the scattering of separate values of N_i relative to the average characteristics of N, presented in Fig. 32b.

For a determination of the degree of scattering of the studied characteristics of N_i within the boundaries of a uniform layer the ratios are first calculated

$$k_N = \frac{N_i}{N}, \tag{50}$$

making it possible to mutually compare the obtained results of dynamic sounding for a separate hole, for a group of holes or for even a specific section of sounding. A measure of the scattering of results of sounding is the value of the root-mean-square deviation from an average of the separate values of the sampling population:

$$\sigma = \pm\sqrt{\frac{(k_N - \bar{k}_N)^2}{n}}, \tag{51}$$

where n - number of conducted measurements: at $n < 30$ under the radical in the denominator of formula (51) one should set $(n - 1)$ - $\bar{k}_N = \frac{\Sigma k_N}{n}$ - average value of the ratio k_N.

If $|\sigma| \leq 0.1$, then the investigated soils are very uniform; at $0.1 < |\sigma| \leq 0.2$ - uniform. The condition $|\sigma| > 0.2$ serves as an indication of the determined deficiencies in conducting dynamic soil tests. For example, friction of the lateral surface of the rod against the soil was observed, the number of conducted measurements was insufficient when distinguishing the separate uniform layers of soil, i.e., in determining $\bar{k}_N$ errors are allowed, the diameter of conical point turned out to be too small, etc.

Experimental check of the proposed method of interpreting the results of dynamic sounding. The foregoing dependences, developed under direct and active participation of G. V. Zhornik, were subjected to experimental check in 1964-1965 during the geological engineering investigations in the Kakhov and Rogachik irrigation tracks of Kherson oblast, conducted by Ukrgiprovodkhoz (Ukrainian State Institute for the Planning of Water Management Structures and Rural Electric Power Plants) upon the initiative of Ye. V. Ripskiy.

The dynamic sounding was made using four reference sections, 140-150 km^2 in area, spaced 50 to 80 km apart. The sounding was carried out in the loessial aeolian-deluvial complex using a device installed on SBU-150 model drilling rigs (weight of hammer is 60 kg, height of fall, 90 and 100 cm). All tests are made using conical points with the angle of expansion of 30°, diameter of 68 mm, height of 126.7 mm with the ratios of the diameters of the point to the rod, $d_{кон}/d_{шт} = 1.6$. A part of the testings was done with a UBP-15 device. In all, 185 holes comprising 3080 m were drilled.

Shown in Fig. 31 are the results of dynamic sounding, made in loess deposits having a solid consistency with N = 2.04-15.3 blows/dm. Variations of projection diagrams of the soundings which are characterized by the occurrence of 8 uniform layers, are presented in Fig. 32.

The first system of control for the results of dynamic sounding consisted of the determination of the degree of scattering of the sounding indexes N_1 relative to the mean value N.

Given as an example in Fig. 33a and b are two distribution curves of the sampling population of the ratios $k_N = N_1/N$, obtained during the sounding of 4 drill holes down to 6 m in one of the groups (Fig. 33a - 229 values of k_N) and during the sounding of 12 holes, drilled at three separate sites (Fig. 33b - 682 values of k_N).

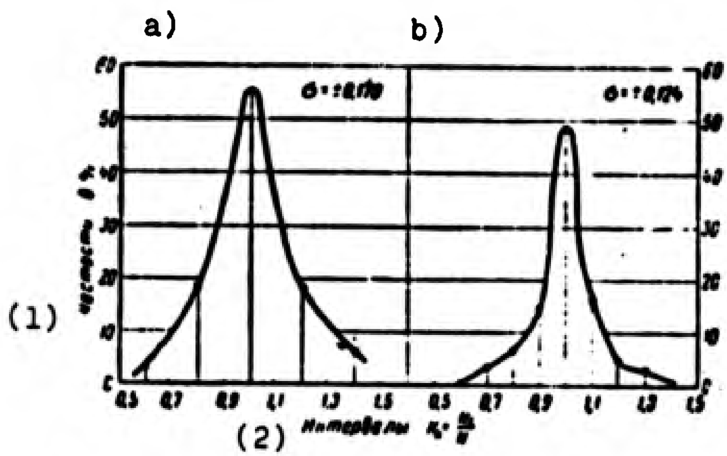


Fig. 33. Graphs of the distribution of the sampling population of the ratios N_1/N for dynamic sounding holes;
a) 4 holes; b) 12 holes.
KEY: (1) Frequencies in %;
(2) Intervals $k_N = N_1/N$.

The values of the root-mean-square deviation from the average in the selected samplings comprised $\sigma = \pm 0.17$ (Fig. 33a) and $\sigma = \pm 0.124$ (Fig. 33b). The average value of the coefficient $\bar{k}_N = 1$. Thus, the coefficient of variation in the results of dynamic sounding under investigation comprised ±17 and 12%, respectively.

The analysis showed that the sampling populations, given in Fig. 33, obey the law of normal distribution, and therefore, the individual deviations in values N_1 observed in Fig. 32a have a random nature.

The studies of the accuracy of the method of dynamic sounding and the proposed method of interpreting the obtained results were extended into 1965. Given in Table 6 are the values of the standard deviation of the ratios $k_N = N_1/N$ from the average σ_1 and the standard deviation of the arithmetic mean $\bar{\sigma}$, differentiated for the different values of the dynamic sounding index N.

Table 6. Values of σ_1 and $\bar{\sigma}$ with different dynamic sounding indexes N.

Mean values of N in blows/dm	σ_1 in %	$\bar{\sigma}$ in %	Index of accuracy of the tests P in %	Number of determinations
3	16.5	6.1	6.1	710
5	12.2	4.3	4.3	300
7	12.7	5.6	5.6	173
9	11	4.1	4.1	220
11	10.4	4	4	215
13	10.4	3.9	3.9	290
15	10	3.9	3.9	125
17	9.3	4.2	4.2	30

The cited data were disclosed based on the results of dynamic soundings for 5 main drill holes, bored to a depth of 20.5-23.5 m and arranged at angles and at the center of a square by a section 2.5 × 2.5 m, and for 4 supplementary drill holes, arranged in the form of a second square several tens of meters on the side.

The analysis of the obtained materials gives a basis for the following conclusions.

1. At values of the dynamic sounding index of more than two, the standard deviation of an individual determination does not exceed ±18% from the ratio of N_1/N and when $N > 7$ it is usually ±10-12% from the ratio N_1/N.

Thus, the proposed method of interpreting the results of dynamic sounding actually facilitates the differentiation of layers with uniform indexes of mechanical properties.

2. The standard deviation of the arithmetic mean $\bar{\sigma}$, following from Table 6, amounted to only ±4-6%, which characterizes a sufficiently high degree of accuracy of the determination of the arithmetic mean value of the dynamic sounding index.

3. The regular decrease in the standard deviation σ_1 with an increase in N is the consequence of the accepted method of recording the resistance to dynamic sounding by the direct count of the blows (method of "impacts"). In this case, actually, the greater the dynamic sounding index, the less the relative error of an individual determination.

With 1-2 blows per 1 dm when driving the probe, the relative error of a separate determination can exceed the permissible limits. Under these conditions it turns out to be especially expedient to assign a constant number of blows for assurance and to record directly the depth of indentation of the probe (method of "settlings").

It is obvious that in soils with plastic and semisolid consistencies the method of settling leads to a minimum relative error of measurement. Consequently, if dynamic sounding is achieved predominantly in soils of low strength, when $N < 5\text{-}7$, then it is advantageous to utilize the method of recording the settlings of the probe. In soils with solid consistency, when $N > 7\text{-}8$, it turns out to be more preferable to use the method of recording the blows of a hammer.

In summation, it can be stated that the proposed procedure for the initial processing of the results of dynamic sounding in cohesive soils facilitates:

a) the differentiation of uniform layers with constant characteristics of the mechanical properties ($|\sigma| < 0.2$);

b) a sufficiently high degree of accuracy for determining the average characteristics of S or N within the limits of the uniform layer;

c) the completely unequivocal determination of the location of uniform layers.

The second system of control for the results of dynamic sounding includes an estimate of the degree of reproducibility of the results in parallel soundings. For this, in all of the sections several sites, 2.5-4 m on the side were selected in whose vertexes 4 parallel soundings were carried out.

Four graphs of the average deformation intensities of the soil during the process of parallel dynamic soundings are given on Fig. 34. The corresponding values of the sounding indexes at separate selected levels (between conjugate points 1-6) are presented in Table 7.

Table 7. The results of parallel dynamic soundings in loess deposits having a solid consistency.

No. of layers	Depths of separate layers H in m	Average thickness of the layer ΔH in m	Dynamic sounding indexes, i.e., the average number of blows N per 1 dm of driving of the probe				Average count of the number of blows
			hole 1	hole 2	hole 3	hole 4	
1	0.18-0.62	0.45	3.1	3.6	4	3	3.4
2	0.62-1.23	0.61	7.4	8.7	6.9	8.4	7.8
3	1.23-2.61	1.38	5.8	5.2	5.2	5.5	5.5
4	2.61-3.80	1.19	---	3	3.6	3.6	3.4
5	3.80-6.25	2.25	2.7	2	1.9	2	2.4

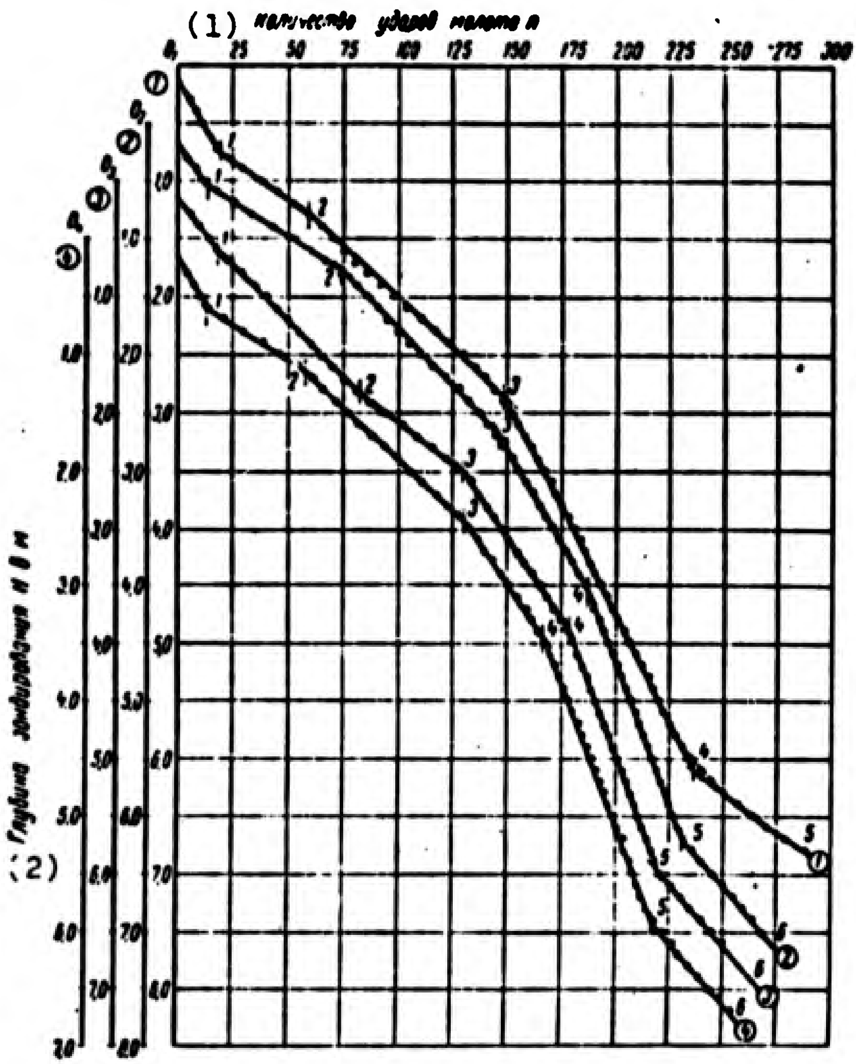


Fig. 34. The graphs of parallel dynamic soundings of a loess-like clay loam with undisturbed structure.
KEY: (1) Number of blows of the hammer n; (2) Depth of sounding H in m.

The completely acceptable convergence and the necessary objectivity of the obtained results are quite obvious. The separate deviations are connected with the displacement of the second layer at a depth of 1.82-2.5 m (hole 3) and with the more increased resistance of soil at a depth of 6.1 m (hole 1).

Parallel soundings at the other sites led to approximately the same results. The sometimes observed differences are related to the actual variability in the physico-mechanical conditions of the soil within the boundaries of site.

The third, most laborious, system of control for the results of dynamic sounding includes a comparison of the average values of the resistivity to penetration R with the average values of the sounding index N. A comparison of the results was made on 29 horizons in 5 excavations with a change in resistivity to penetration from $R_{мин} = 1.65$ kg/cm^2 to $R_{макс} = 20.65$ kg/cm^2, i.e., with a ratio $R_{макс}/R_{мин} = 12.5$.

The graph of the proportional dependence $R = 1.02\ N$ kg/cm^2 is given in Fig. 35. The correlation analysis of this dependence revealed a high value of the correlation coefficient $r_к = 0.955$, an insignificant error in the correlation coefficient $\eta = 0.016$ and the presence of a very close connection between the results of dynamic sounding and static penetration (index of the closeness of the relation $r_к - 3\eta = 0.906$).

Consequently, the third check of the results of dynamic sounding also confirmed the sufficient accuracy and necessary authenticity of the obtained results. However, the maximum depth of excavation during this study did not exceed 5.5 m. Thus, even the conclusion about the sufficient convergence of results of static penetration and dynamic sounding relates only to this depth. A continuation of these studies in 1965-1966 revealed the following.

(1)

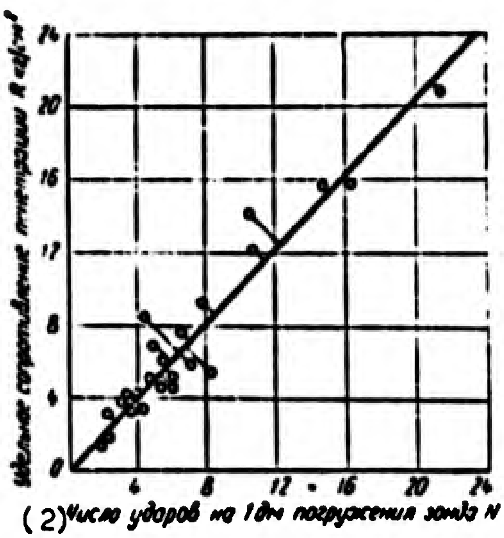


Fig. 35. The graph of the comparison of the values of the dynamic sounding index and the resistivity to penetration. KEY: (1) Resistivity to penetration R kg/cm^2; (2) Number of blows per 1 dm of driving of the probe N.

To the depth of sounding of about 8-9 m a proportional dependence with a proportionality factor, equal to unity, holds between mean values of the resistivity to penetration and the dynamic sounding indexes. Consequently, at depths of sounding down to 9 m, with the diameter of the rods at 42 mm and the ratio at $d_{кон}/d_{шт} = 1.6$ a noticeable mutual rotation of rods under the action of impact does not occur, under conditions when the resistivity to penetration does not exceed 5-6 kg/cm^2.

At depths of sounding greater than 9 m or at shallower depths, but with relatively higher values of resistivity to penetration (>10 kg/cm^2) the mutual rotation of the rods at the nipple connections is feasible. In summation, considerable frictional forces occur, acting along the lateral surface of the rods, which lead to the sinking of the probe at too slow a rate with every blow and, as it was explained, they cause an exaggeration of the dynamic sounding indexes by 1.5; 2, and in certain cases even 3 times. It is logical that in this case the results of dynamic sounding cannot be used for differentiating geological sections.

In 1966 during the geological engineering investigations conducted in the Crimea by O. N. Padalkin and A. A. Yakimenko under the guidance of the author, control dynamic soundings were made to a depth of 22 m with partial preliminary borings to a depth of 8, 12, 16 and 20 m. The dynamic sounding indexes obtained from sounding in the surface soil and after the preliminary boring of the leading hole to the specified depth were compared. During the sounding of the selected horizon through the hole drilled in advance, the frictional forces along the lateral surface of the rods were almost completely eliminated. Thus, the agreement of the dynamic sounding indexes (within the limits of the usual accuracy of sounding of $\pm 2\bar{\sigma}$, i.e., approximately ±8-12%), obtained with the boring and without the boring of a hole, was considered as proof of the practical feasibility of conducting dynamic soundings to a depth of 20-25 m. In summation, it was explained that with the proper thoroughness in coupling the individual rods, with periodic tightening of the joints by turning the column of rods, and with strict adherence to the fundamental rules of the test procedure the method of dynamic sounding can turn out to be completely effective and provide research data on the variability of the mechanical properties of cohesive soils having solid consistency.

CHAPTER III

THE PROCEDURE FOR TESTING COHESIVE SOILS USING VANE SHEAR AND COMBINED SOIL TESTS

Vane shear is a method of studying the physical and mechanical properties of soils by determining the resistance of the soil to the rotation of various winged or multiple points with four small mutually perpendicular blades (Fig. 36).

Fig. 36. Points for testing cohesive soils by the methods of penetration and vane shear: a) multiple conical point with small blades in the base of the cone; b) multiple conical-wing point; c) wing point.

Numerous studies by different authors showed that if the depth of indentation of the multiple point is less or equal to the height of the small blade, then vane shear under the action of the external torque is achieved using cylindrical and a single circular surface. With a further increase in the depth of indentation of the point still a second area of a circle formed by the small blades during rotation is added to this surface of vane shear [30, 44, 61].

The method of expressing the soil test data by the method of vane shear. In general when testing the soil by the method of vane shear the dependence of the angle of rotation of the multiple or wing point β on the value of the applied torque $\beta = f(M)$ is determined. On the basis of the simplest representations of the equality of maximum external torque $M_{\text{макс}}$ to the product of resistivity to vane shear τ by the static moment of the surfaces of shear k_τ one can determine the value

$$\tau = \frac{M_{\text{макс}}}{k_\tau} \text{ kg/cm}^2 \qquad (52)$$

With a depth of indentation of the point, equal to its height $h = h_{\text{нак}}$, the constant of point k_τ (static moment of a cylindrical and single circular surface) is determined from the formula

$$k_\tau = \frac{\pi D^2}{2}\left(\frac{D}{6} + h\right) \text{ см}^3, \qquad (53)$$

where D - diameter of the cylindrical surface of vane shear, equal to the width of two small blades of the multiple or wing point, in cm.

If the depth of indentation of the point is substantially greater than its height ($h \gg h_{\text{нак}}$), then as already mentioned, the constant of the point k_τ increases and turns out to be equal to:

$$k_\tau = \frac{\pi D^2}{2}\left(\frac{D}{3} + h\right) \text{ см}^3. \qquad (54)$$

The relative constancy of the force of sounding is included in the workings of the second cylindrical surface based on an increase in the torque of vane shear.

If soil tests by vane shear are made with the aid of the multiple conical point with small blades in its head (see Fig. 36a), then the constant of point k_τ is determined from the formula:

$$k_\tau = \frac{\pi D^2}{2}\left[\frac{1}{6}\left(D - \frac{d^2_{кон}}{D^2}\, d_{кон}\right) + h_{кр}\right] \text{см}^3, \tag{55}$$

where $d_{кон}$ - diameter of the base of the cone in cm.

Making the value k_τ more precise when using formula (55), is not substantial.

Under conditions of testing clay soils directly at the surface (at the sites, in exposures, excavations, etc.) the dead weight of the soil within the limits of the depth of indention of the point naturally can be disregarded. Thus, the resistivity of the soil to vane shear under these conditions can be identified with sufficient reliability with the cohesion of the soil $c = \tau$.

The validity of the stated position was checked by S. I. Rokas by means of comparative soil field tests using vane shear and displacement of soil columns with the aid of a frame.

The results of comparative tests made by V. D. Shitov and B. P. Slavinskiy can also serve as a certain illustration of this position. Samples of loess-like clay loam with disturbed structure ($M_p = 16.3$, $w_f = 38.9\%$, $\gamma = 2.69$ g/cm^3) were tested using an LP-1 penetrometer with a vane shear attachment and a VSV-2 design model of the hydroelectric power project for rapid shear.

Three parallel samples were shorn under loads of 0.5; 1 and 2 kg or 1, 3 and 5 kg directly following the application of vertical pressure. From the graphic representation of shear strength the specific cohesion of soil was determined, which was compared with the average specific cohesion, obtained when testing two-three samples according to the method of vane shear.

The results of the comparative tests are represented in Fig. 37. The high convergence of the values of soil cohesion obtained by the different methods, is also connected with the careful control in correlating the physical characteristics of the samples in the parallel tests.

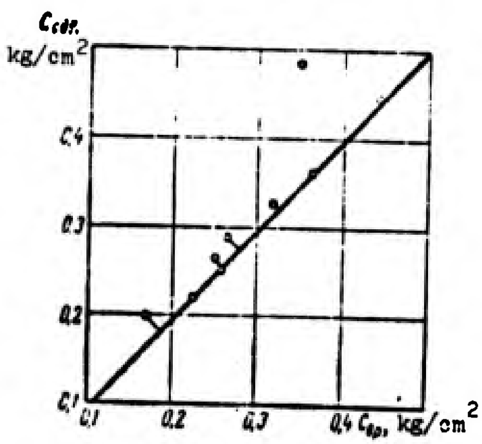


Fig. 37. The graph of the comparison of the values of the specific cohesion of the samples of loess-like clay loam, obtained by vane shear and by shear using a VSV-2 model. Plotted along the axis of the abscissae are the values of the specific cohesion, obtained by the method of vane shear; along the axis of the ordinates - by the method of simple shear.

Comparative tests on clays (M_p = 18-31), clay loams and sandy loams, made by O. M. Reznikov under laboratory conditions are also of interest. According to the results of 46 comparative tests on clays and 49 tests on clay loams, O. M. Reznikov came to the conclusion that "in clays, heavy and medium-textured clay loams in the absence of natural pressure (testing near the surface, on the bottom of excavations and trenches) vane shear gives the same data on the strength of soils as the conventional laboratory methods of testing."

However, in sandy loams and light-textured clay loams in the opinion of O. M. Reznikov, distorted representations of the values of the resistance of the soil to shear are obtained, which is the consequence of the volumetric strains which occur in the zone of shears, and respectively, the change in the stress condition which occurs in this zone [44].

The invariance of resistivity to vane shear. The invariance τ with respect to the dimensions of the multiple point is a significant property of resistivity to vane shear. This means that in uniform cohesive soils with an increase in the linear dimensions of the point the torque of vane shear will, it goes without saying, increase. However, the ratio of the torque of vane shear to the corresponding constant of the multiple point, i.e., resistivity to vane shear, will be approximately identical.

Figure 38 gives as an example the results of three different tests on fine silty and clayey sediments of the estuary, made by using a wing point under laboratory conditions with $k_\tau = 35.9$ cm^3 (point 1); under field conditions with the aid of a manual universal hydraulic VSEGINGEO penetrometer using a wing point with a constant $k_\tau = 176.38$ cm^3 (point 2) and under field conditions with the use a MP-1 model equipped with a wing point having a cone $k_\tau = 2404$ cm^3 (point 3). In these tests, carried out by G. V. Zhornik and V. D. Shitov, the range of change in the static moment of the surfaces of vane shear is overlapped 67 times, which naturally characterizes the high degree of perfection in developing the invariance of resistivity to vane shear.

Tests were conducted with a moisture constant of $w = 37.4\%$, volume weight of the skeleton $\delta = 1.22$ g/cm^3 and the degree of water saturation $G = 0.832$.

The invariance of resistivity to vane shear is considered ideal, if the test data for vane shear at different values of k_τ

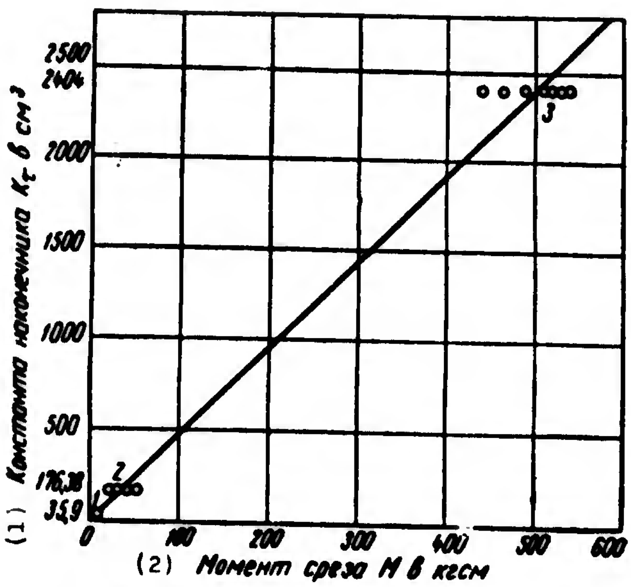


Fig. 38. The graph of the ideal invariance of resistivity to vane shear.
KEY: (1) The constant of the indentor k_τ in cm^3; (2) The moment of shearing M in kgcm.

are depicted at coordinates $M - k_\tau$ in the form of a straight line passing near the origin. Graphically, the resistivity to vane shear - the cohesion of the soil - is represented by the slope of the averaging straight line to the vertical axis, and therefore in general, is calculated from the formula

$$c = \frac{M_2 - M_1}{k_{\tau_2} - k_{\tau_1}} \text{ kg/cm}^2, \qquad (56)$$

where M_1, $k_{\tau 1}$ and M_2, $k_{\tau 2}$ are the coordinates of any two points, located on the straight line.

According to O. M. Reznikov, the invariance of resistivity to vane shear is exhibited only in clays and clay loams, at least heavy and medium-textured. In sands and, in particular, in sandy loams, the resistance to vane shear depends on the diameter of the rotating vanes and turns out to be less, the greater the geometric dimensions of the vane point are.

It is obvious that the presence of the i variance of resistivity to vane shear serves as the obje(ive criterion of the accuracy and reliability of the conducted tests. Thus, the checking of the invariance of the values τ in all cases is desirable, and during the testing of sandy loams and light clay loams, it is the necessary requirement for the correctly conducted tests.

The features of field and laboratory tests on cohesive soils by the method of vane shear. The maximum moment of resistance of soil $M_{\text{макс}}$ is determined under field conditions at a constant rate of rotation of the multiple point in the soil, usually not exceeding by one-tenth of an angular degree per second [61] or, at worst, one-fourth or one-half of an angular degree per second.

At the maximum angle of shear which varies for different soils and for their condition, approximately from 8 to 35°, the overall duration of the tests will amount to 1.3 to 5.8 min for the first conditions, 0.4-2.3 min for the second conditions. And in both cases the physical properties of the soils during tests hardly change at all.

As a result of the tests usually with the aid of recorders, the graphs of the dependence $\beta = f(M)$ are recorded according to the soil type presented in Fig. 39 (tests on the silty and clayey sediments of the Sivash estuary). In the majority of cases the value of maximum torque $M_{\text{макс}}$ is quite distinctly displayed in the graph of vane shear after which the torque decreases somewhat.

Under laboratory conditions the maximum moment of resistance of the soil to vane shear $M_{\text{макс}}$ is determined by means of a progressive increase in the torque by equal stages. The soil tests can be made under conditions of a rapid, continuous load prior to the onset of shear at intervals between the stages at 10 s and with 10-15 stages of load, or with stage-by-stage loading with

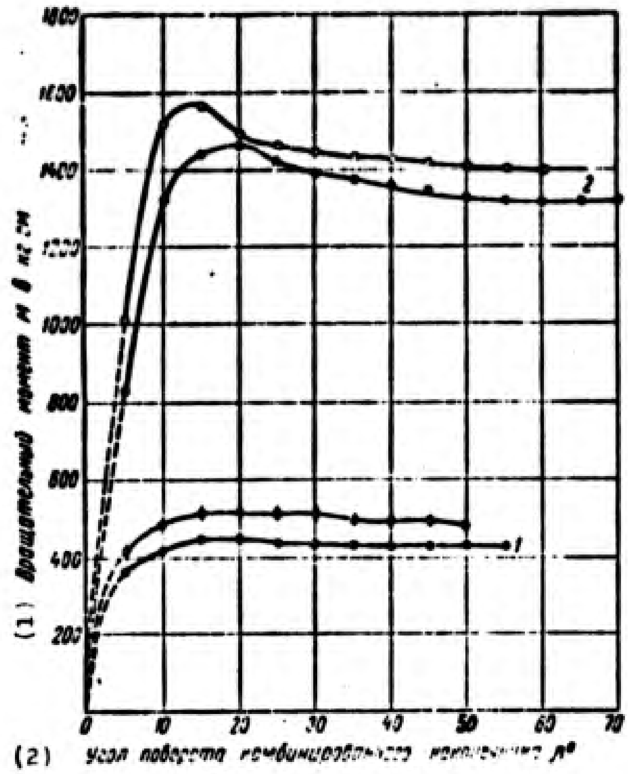


Fig. 39. Typical graphic representations of vane shear, obtained during the field tests of fine silty and clayey sediments: 1 - testing with an indentor when $k_{\tau} = 2404\ cm^3$; 2 - the same, when $k_{\tau} = 10933\ cm^3$.
KEY: (1) Torque M in kg cm; (2) Angle of rotation of the multiple point β°.

the application of the subsequent stage of loading after the conditional damping of the angle of rotation of the point by the previous stage of loading. The test data are depicted in the form of graphs of the dependence of the angle of rotation of a multiple point on the torque M (this graph is analogous to the graph $\beta = f(M)$ which represented point 1 in Fig. 39).

Under conditions of rapid loading a "short-term or standard strength" is revealed according to M. N. Gol'dshteyn [17], which corresponds to the standard velocity of loading and which characterizes the upper ultimate strength of the soil being investigated. This procedure for testing is most convenient during the study of the mechanical properties of soils with weak structure, where the application of the condition of the damping of deformations can lead to the phenomena of soil compaction.

As numerous tests have shown, the conical point with four vanes in the base of the cone turned out to be most successful (Fig. 36a). This indentor, proposed by A. V. Grigor'yev and P. I. Eyzler, assumes that in one of the versions, the rotation of the vanes of the point is independent of the conical part. Its main and distinctive feature lies in the fact that with the aid of this indentor it appears to be feasible to conduct the combined tests by penetration and vane shear approximately in one and the same volume of soil. This circumstance provides increased accuracy of the results of the combined tests, and because of this, there is merit in the proposed design.

The principles in the procedure for determining the characteristics of friction and of cohesion of cohesive soils. In 1955 Prof. V. G. Berezantsev on the basis of an examination of the conditions of limit equilibrium of soil during the driving of a conical point, for the first time proved theoretically the presence of the proportional dependence between the specific cohesion of the soil and the value of the resistivity to penetration [3]

$$c = k_\phi R \text{ kg/cm}^2, \qquad (57)$$

where k_ϕ - function of proportionality which depends on the apex angle of the conical point α and the dimensionless coefficient M_ϕ which is, in turn, the function of the angle of internal friction of the soil $\phi°$.

Equation (57) and dependence $M_\phi = f(\phi°)$ were obtained by V. G. Berezantsev taking into account the following positions [3, 4].

1. The conditions of limit equilibrium of a cohesive medium which possesses internal friction were investigated. The limit equilibrium of this medium at every point is determined by Coulomb's equation:

$$\tau_{пред} = p\,\mathrm{tg}\,\varphi + c \quad \mathrm{kg/cm^2}, \qquad (58)$$

where $\tau_{пред}$ - tangent, and p - normal component of the stress, which acts on the site of the slip.

2. In proportion to the development of the areas of limit equilibrium the slip surfaces are formed from the sites of the slip.

The surfaces, which are tangent to the plane of action of the principal stresses, form in the direction of the larger principal stress at every point of angle $+\mu$; they represent the surface of the first family. The surfaces of the second family have an angle of deflection $-\mu$ (Fig. 40a).

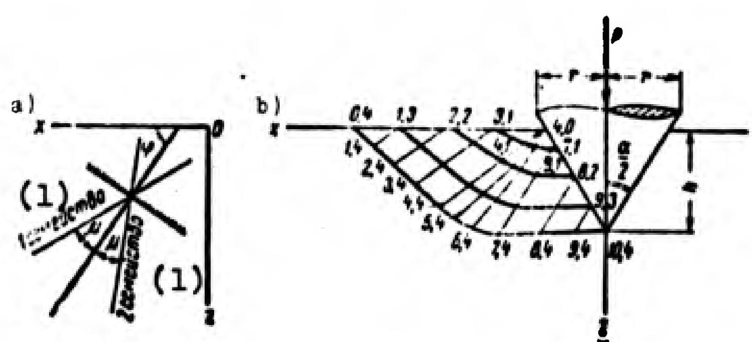


Fig. 40. Design diagrams of the limit equilibrium of a cohesive medium for the conditions of penetration tests using a conical point: a) diagram of the slip surfaces of the first and second families; b) the diagram of the lines of sliding.
KEY: (1) family.

3. The effect of the volume weight of the connected medium in the sheared areas in comparison with the effect of the value of cohesion is not substantial. Thus, the construction of the grid of lines of sliding and the calculation of the functions of limit equilibrium were made not allowing for the volume forces.

4. The construction of the network of lines of sliding in plane XOZ (Fig. 40b) and the determination of the functions of limit equilibrium was conducted with the aid of recursion formula for three areas: I, II and III. Area I is restricted to coordinates 0.4; 4.4; 4. Area II is determined by the angle between the tangents to lines 4; 4.1 and 4; 6.1, which is equal to $\alpha/2$. Area III (4; 6.4; 10.4) is restricted to a forming cone. At points 4 (in direction 6; 7.1; 8.2; 9.3 and 10.4) the intensities of the normal pressure of the cone to the medium were calculated.

5. In the apex of the conical point the stresses are theoretically equal to infinity. Thus, the construction of the network of lines of sliding was plotted up to point 10.4, close to the apex, which has an abscissa $r_0 \approx 0.05r$.

6. The value of the force of penetration P was determined from the equilibrium condition by means of a projection of the intensities of normal pressures over the lateral surface of the conical indentor on axis OZ. The formula for the calculation of the dimensionless proportionality factor k_ϕ acquired the form

$$k_\varphi = \frac{c\,\mathrm{tg}^{\prime}\frac{\alpha}{2}}{M_\varphi}. \qquad (59)$$

The coefficients M_ϕ calculated by V. G. Berezantsev for three angles of internal friction of the soil, and the coefficients k_ϕ for a standard conical indentor ($\alpha = 30°$) are given in Table 8.

Table 8. The values of coefficients M_ϕ and k_ϕ.

Angle of internal friction in deg	Dimensionless coefficients	
	M_ϕ	k_ϕ
0	16	0,87
10	21,5	0,646
20	37	0,376

The graph of the dependence of the angle of internal friction of the soil on the coefficient k_ϕ, plotted according to the data in Table 8, is presented in Fig. 41.

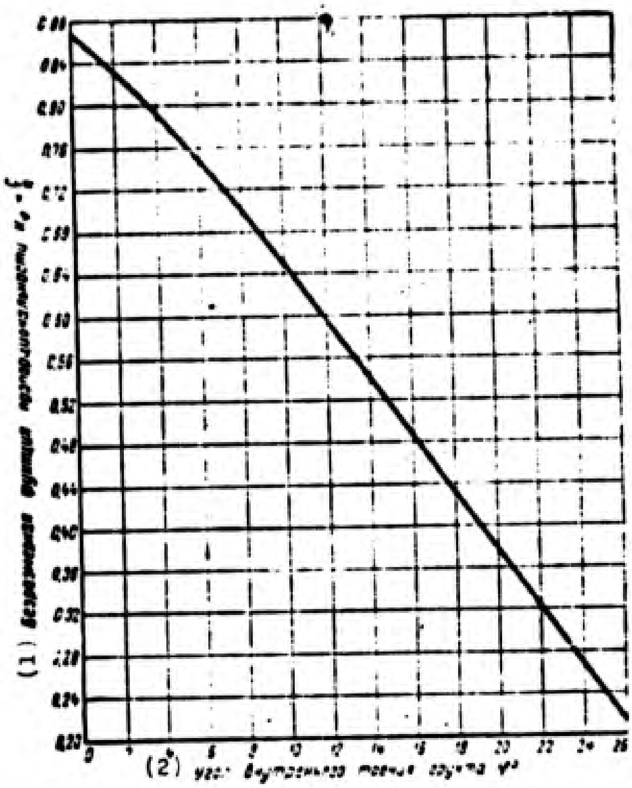


Fig. 41. The graph of the theoretical dependence between the dimensionless function of proportionality k_ϕ and the angle of internal friction of cohesive soils $\phi°$ (according to V. G. Berezantsev).
KEY: (1) Dimensionless function of proportionality k_ϕ = C/R; (2) Angle of internal friction of the soil $\phi°$.

Thus, during the determination of the specific cohesion of the soil the method of vane shear and of resistivity to penetration according to the results of penetration tests, appears to be feasible easily and simply by establishing the angle of internal friction of the soil.

The procedure for the use of penetration tests for determining the cohesion of soil was expressed for the first time in 1947 by Prof. N. A. Tsytovich and was published later in a number of his works. Prof. N. A. Tsytovich proposed to utilize a ball bearing punch for determining the specific cohesion in the plastic, almost noncompactive clayey soils, with the angle of internal friction $\phi \leq 5\text{-}7°$ or the "equivalent" cohesion during the testing of different cohesive soils. Ball bearing punches with a diameter of 20 mm for laboratory determinations up to 1000 mm for field tests are used. S. S. Vyalov and N. A. Tsytovich have shown that by utilizing the test data where a ball bearing punch is used, it is possible principally to calculate the bearing capacity of cohesive soils [11].

Later, in 1953, V. A. Baldin proposed a procedure for combined soil tests by penetration and vane shear for an approximate estimate of soil conditions. For this purpose V. A. Baldin developed and successfully applied a universal penetrometer and ShchP-1 densimeter probe. The further development of the idea of using the combined tests for determining the lithologic composition of soils was realized by S. I. Rokas under the guidance of Prof. V. F. Babkov.

Various methods of combined tests on cohesive soils. Combined tests on cohesive soils can be made by three different methods using various indentors.

1. Tests can be made by a multiple conical point with vanes in the base of the cone (Fig. 36a).

The sequence of such tests is set forth below. In this case the resistivity to penetration R is directly determined from the test data.

These tests can be made both under field and laboratory conditions. They provide a high degree of accuracy of the determinations and therefore they are considered as the basic ones.

2. The tests are carried out using a multiple conical-wing indentor; the point of the cone and of the cutting tip of the vanes are located at the same level or the cutting tips of the vanes are somewhat protruded (Fig. 36b). In this case at first the given resistivity to penetration q_0 is determined. The resistivity to penetration for a standard point R is calculated from the formula $R = k_{\alpha 0} q_0$. The constant of the multiple point $k_{\alpha 0}$ is established preliminarily according to the results of the comparative penetration tests using standard and multiple points.

These simpler tests are expedient for use under field conditions during engineering geological investigations, which do not require the higher degree of accuracy of the determinations.

3. The tests are made only under field conditions on a specified geometric grid for the location of the experimental points, using separately standard conical and vane (Fig. 36c) points. Penetration tests with a conical point are made in the apexes of squares (Fig. 42a) 25 cm on the side ($h_{кон} = 5.5$ cm) or 30 cm ($h_{кон} = 10$ cm). In the center of every square the tests for vane shear are carried out with the use of a vane point. The processing of the results of combined tests is made using the diagram presented in Fig. 42b; every test for vane shear is compared with the results of four penetration tests using a conical point.

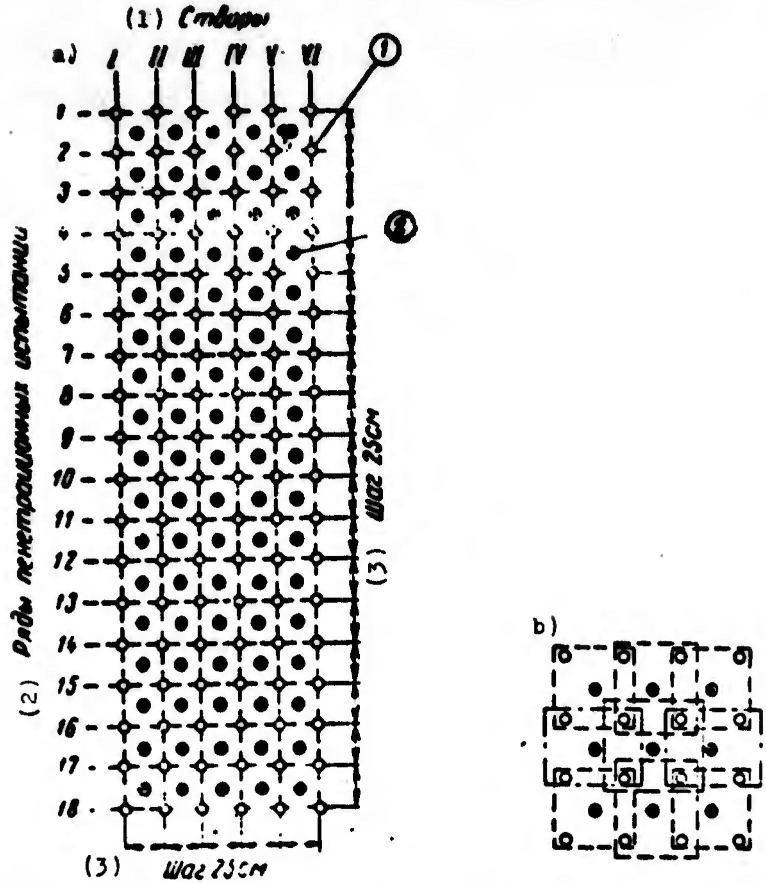


Fig. 42. Diagrams of the conducted combined tests using conical and vane points at a standard site: a) lay-out diagram of the experimental points; b) diagram of processing the results of the combined tests; 1 - penetration tests using a standard conical point; 2 - test using a vane point.
KEY: (1) Alignments; (2) Series of penetration tests; (3) Spacing 25 cm.

The separate tests are recommended for use in soils of semisolid and solid consistencies, since the insertion of the vane point is accompanied by minimum disturbance of the natural structure of the soil.

The determination of cohesion and the angle of internal friction from the results of the combined soil tests by the methods of penetration and vane shear is made using a multiple point with vanes in the base of the cone (see Fig. 36a) in the following sequence.

1. At first the conical part of the multiple point is driven in and the dependence between the depth of indentation and the applied value of vertical force $h = f(P)$ is determined.

2. Then, the head of the point with the vanes is forced downward and the test for vane shear is conducted with the determination of the dependence between the angle of rotation of the point and the applied value of the torque $\beta = f(M)$.

3. The combined tests thus are made consecutively approximately in just one layer of soil, which makes it possible not only to determine the values of the resistivity to penetration R and the specific cohesion c, but also to obtain sufficiently accurate value of their ratio $k_\phi = c/R$.

4. The combined soil tests are repeated at different points of the investigated site. According to the test data the dependence $c = f(R)$ is plotted at the coordinates: specific cohesion c - resistivity to penetration R (Fig. 43). The proportionality factor k_ϕ is represented by the slope of a straight line to the vertical axis, drawn from the origin through the corresponding group of experimental points.

(Presented in Fig. 43 is the averaging straight line 0-3, corresponding to $k_{\phi 3} = 0.493$ and $\phi_3^0 = 15°40'$, and an additional four straight lines, which characterize the range of change in the angle of internal friction of the soil.)

5. With a decrease in the moisture constant and an increase in the volume weight of the soil skeleton the angle of internal friction and the cohesion of the soil gradually increase. This feature can be taken into account by separating out several groups of experimental points and by the determination of the minimum and maximum values of k_ϕ and c on the graph at coordinates c-R. (For example, in Fig. 43, of 176 results of combined tests, 155

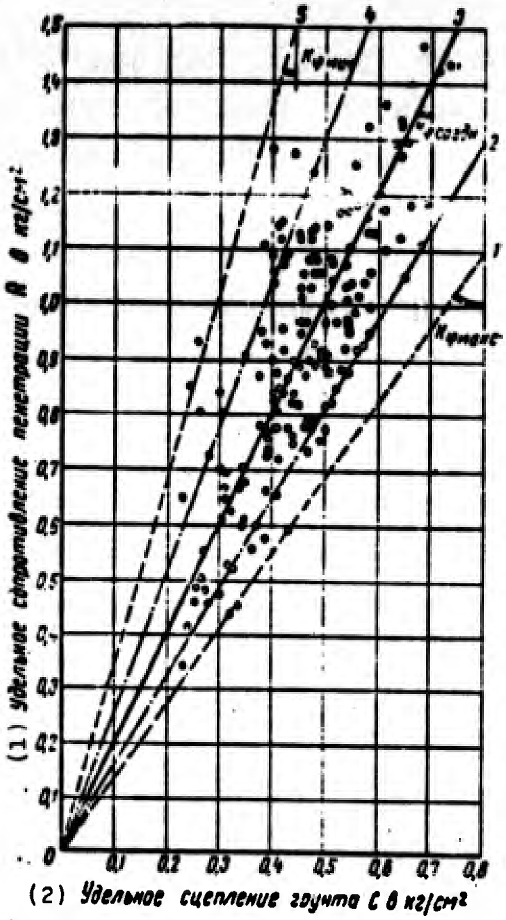


Fig. 43. Graphs of the combined tests on loess-like clay loam.
KEY: (1) Resistivity to penetration R in kg/cm^2; (2) Specific cohesion of the soil c in kg/cm^2.

experimental points, or 88% fall between two straight lines, 0-2 and 0-4, which correspond to $k_{\phi 2} = 0.615$ and $k_{\phi 4} = 0.383$ and which determine the angles of internal friction $\phi_2^0 = 11°15'$ and $\phi_4^0 =$ $= 19°40'$.)

6. According to the fixed proportionality factors $k_{\phi 1}$, $k_{\phi 2}$, etc., based on the graphic representation in Fig. 41, one can find the values of the angles of internal friction ϕ_1^0, ϕ_2^0, etc. The minimum and maximum values of cohesion of the soil are displayed directly on the graphic representation at coordinates c-R.

The control procedure for the quality of combined tests. The control for the quality of combined tests is achieved on the basis of the following prerequisites.

The numerous penetration tests on different soils using a conical point with vanes in the head (see Fig. 36a) and using a conical vane point (see Fig. 36b) revealed the presence of a clear invariance of the so-called given resistivity to penetration

$$q_0 = \frac{P - P_{нач}}{h^2}\ \text{kg/cm}^2, \qquad (60)$$

where $P_{нач}$ - initial resistance of the soil to cutting by the four vanes of the point. This value is found from the experiment and is accepted conditionally as equal to the abscissa of the point of intersection of the straight line of the graphic representation of the invariance with the horizontal axis.

At coordinates P-h^2 the given resistivity to penetration is represented by the tangent of the angle of the plotted straight line to the vertical axis. Figure 44 gives examples of graphs of invariance q_0, obtained during the laboratory penetration of samples of blue Cambrian clay with disturbed structure by a conical-vane point (M_p = 20.3, w_f = 46.5%, γ = 2.74 g/cm^3).

It is not difficult to see that after the driving of the point equipped with vanes into the soil ($h^2 \geq$ 1-2 cm^2) an ideal invariance q_0 is observed in the range of 0.172-5.87 kg/cm^2, i.e., during a 34-fold change in the strength properties of the soils.

As a result of the frictional forces along the lateral surface of the vanes which increase with an increase in the depth of indentation of the point, the given resistivity to penetration q_0 proves to be more than the resistivity to penetration R. Under specific conditions between these values the proportional dependence is distinctly exhibited

$$R = k_{\alpha 0} q_0 \ \text{kg/cm}^2, \qquad (61)$$

where $k_{\alpha 0}$ = const < 1 - constant of the multiple point, which is easily revealed based on the results of the comparative tests.

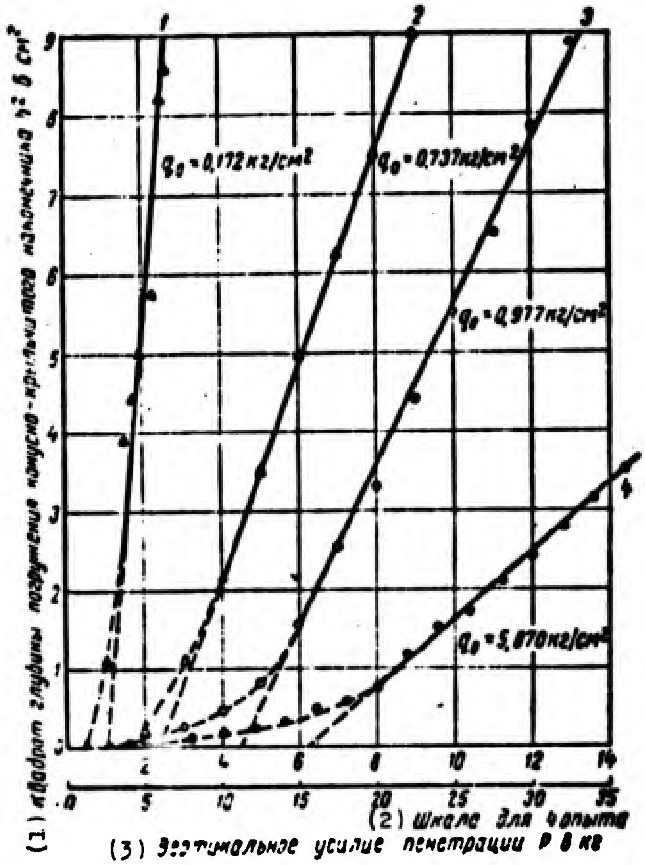


Fig. 44. Graphs of the ideal invariance of the given resistivity to penetration obtained during laboratory tests on samples of Cambrian clay.
KEY: (1) Square of the depth of indentation of a conical-vane point h^2 in cm^2; (2) Scale for 4 experiments; (3) Vertical force of penetration P in kg. [кг/см² = kg/cm²]

Figure 45 at coordinates R-q_0 shows one such comparative graph, obtained based on the results of 34 tests, made in a loess-like clay loam with disturbed and undisturbed structure within the range of liquid to semisolid consistencies.

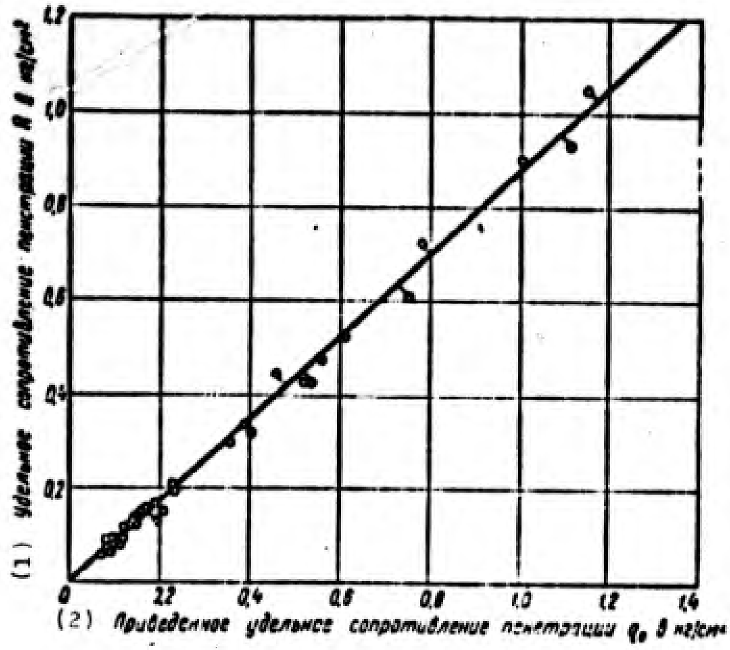


Fig. 45. Graph of the proportional dependence between the values of the given resistivity to penetration q_0 and the resistivity to penetration R. KEY: (1) Resistivity to penetration R in kg/cm^2; (2) Given resistivity to penetration q_0 in kg/cm^2.

In these laboratory tests a conical point $h_{кон}$ = 2.04 cm high with vanes in the head whose height is $h_{кр}$ = 1.98 cm and whose diameter is $D_{кр}$ = 4 cm when $k_{т}$ = 66.07 cm^3 was used. As can be seen from the graph in Fig. 45, the scattering of experimental points around the averaging straight line is completely insignificant and therefore it is easy to determine the constant of the point

$$k_{\tau_0} = \frac{R}{q_0} = 0{,}874.$$

At coordinates P-h^2 the graphs of the dependence P = $f(h^2)$, obtained when driving the conical point with vanes in the head into the soil (see Fig. 36a), have the following features. First, within the limits of driving the conical part of the multiple point $h^2_{\text{кон}}$ a section of the ideal invariance of the resistivity to penetration is noted. According to the general rule, the slope tangent of this graph to the vertical axis determines the value of the resistivity to penetration R. Then, the transient curvilinear section of the increased resistance of the soil is observed; this resistance arose during the driving of the cutting point with the four vanes. This part of the graph is completely analogous to the dashed part of the graphs $P = f(h^2)$, depicted in Fig. 44. Further on the linear part of the graph $P = f(h^2)$ is again displayed which characterizes the ideal invariance of the value of the given resistivity to penetration. The slope tangent of this part of the graph to the vertical axis is equal to the value q_0.

The given resistivity to penetration is computed according to the formula

$$q_0 = \frac{P - P_{\text{нач}}}{h^2_{\text{кон}} - h^2_{\text{нач}}} \text{ kg/cm}^2. \qquad (62)$$

During the driving of the vane part of the multiple point into soils of semisolid and solid consistencies, the development of the phenomena of the local disintegration of the soil is possible. In this case a more or less considerable breakdown of the invariance of the given resistivity to penetration q_0 and an especially considerable distortion of the initial conditions $q_0 > R$ and $k_{\alpha 0} = \text{const}$ is observed.

It frequently turns out that $q_0 < R$, but the proportionality factor $k_{\alpha 0}$ becomes variable and greater than unity: $k_{\alpha 0} \neq \text{const} > 1$.

The results of similar tests, naturally, should be discarded and in the course of a further analysis, should not be taken into consideration.

Thus, for the objective control of the results of combined tests, which are being made, for example, using a multiple conical point with vanes in the head, it is necessary to determine: the resistivity to penetration R during the driving of the conical part of the point; the given resistivity to penetration q_0, obtained during the driving of the cone using the vane part of the point.

If the ratio $R/q_0 \approx$ const ≤ 1, then, the test data by vane shear will not contain systematic errors. Otherwise, the value of cohesion of the soil will turn out to be understated, and the results of the combined tests will be distorted.

Examples of research on the variability of the characteristics of the friction and cohesion of cohesive soils by the method of combined tests. Two examples of the application of a method of combined tests for investigating the variability of the specific cohesion and the angle of internal friction of cohesive soils are shown in Figs. 43 and 46.

Presented in Fig. 43 are the results of field combined tests, made in a loess-like clay loam (M_p = 14.5, w_f = 33.3%, γ = 2.65 g/cm^3) with disturbed structure, using a conical-vane point (expansion angle of the cone is 30°, D = 4.72 cm, h = 4.73 cm and k_τ = 193 cm^3). The proportionality factor k_α = 0.432 was determined preliminarily according to the results of the comparative penetration tests using a conical-vane and standard conical points. The tests were carried out in the surface layer of a plot of about 1.5 hect. in the region of Odessa city.

From the examination of the test data the following conclusions are:

a) the basic group of experimental points (88%) falls within the range of change in the angle of internal friction of the soil of 11°15' to 19°40' and with a specific cohesion of 0.26 to 0.66 kg/cm^2;

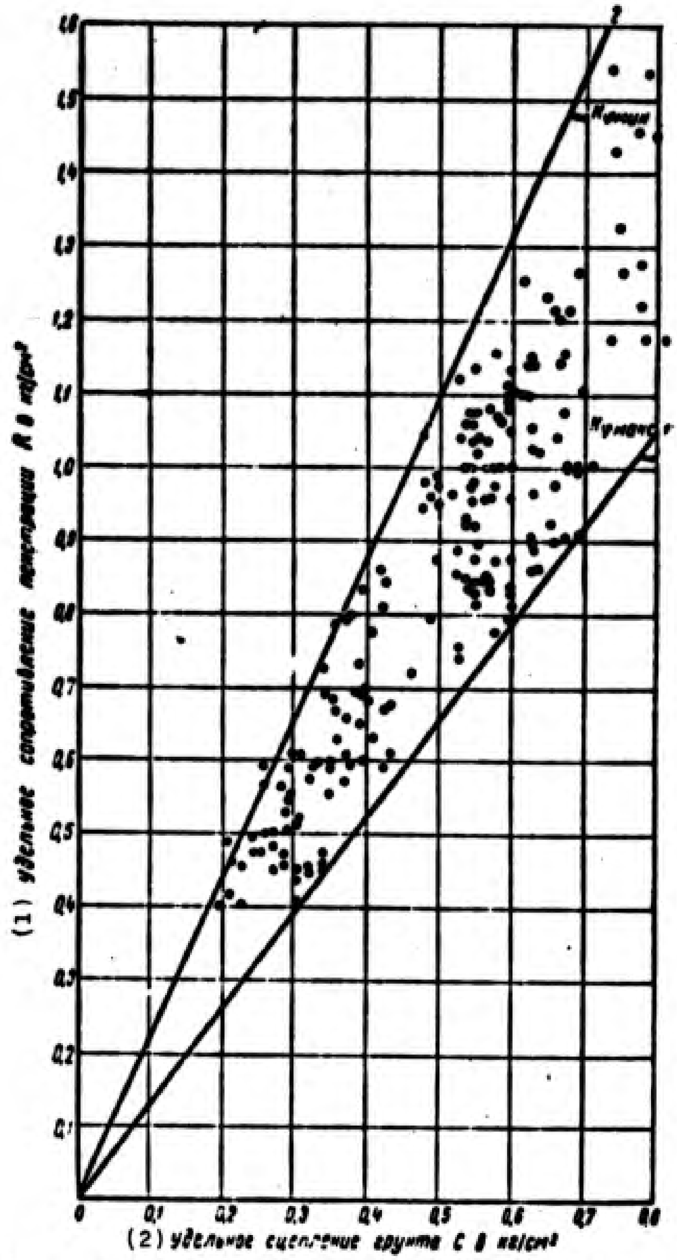


Fig. 46. Graphs of the combined tests on Cambrian clay.
KEY: (1) Resistivity to penetration R in kg/cm^2; (2) Specific cohesion of the soil c in kg/cm^2.

b) the mean value of the angle of internal friction of the soil comprised 15°40' with an average specific cohesion of 0.46 kg/cm^2;

c) the minimum values of resistivity to penetration are equal to $R_{мин}$ = 0.45-0.5 kg/cm^2 and the specific cohesion $c_{мин}$ = 0.26-0.32 kg/cm^2.

Let us note that $R_{мин}$ and $c_{мин}$ correspond to the moisture content of the clay loam, w = 23.4% and volume weight of the skeleton, 1.41 g/cm^3.

Below, in Chapter IV, the procedure for establishing the interconnection between the physical characteristics of the soils and the resistivity to penetration will be shown. Thus, the revealed minimum, average and maximum characteristics of friction and the cohesion of the soils, in accordance with the procedure of Prof. N. N. Maslov [26, 27], can be connected with the determined values of the volume weight and moisture content of the loess-like clay loam.

In order to check the convergence of the results of the field and laboratory multiple tests, samples of the investigated clay loam with disturbed structure were delivered to the laboratory. The combined tests were repeated with a manual universal penetrometer of another design (VSEGINGEO spring penetrometer) and with a new conical-vane point (expansion angle of the cone 22°30', D = 4.57 cm, h = 5.46 cm, $k_т$ = 204 cm^3). Prior to the test, the soil at a specified moisture content was arranged in three layers in a cylindrical drum of the standard compaction type with a volume of 7.8 dm^3 and using the identical number of blows to compact each layer to the necessary volume weight of the soil. The laboratory tests were carried out using 7 different values of volume weight and moisture content of the soil which were established under field conditions. For the second indentor it was established that k_α = 0.65.

By plotting the graph of the dependence c = k_ϕR at coordinates c-R, the average value k_ϕ = 0.512 was determined, which corresponds to the angle of internal friction $\phi = 15°$. Thus, the average results of the field and laboratory combined tests completely coincided.

The second example, depicted in Fig. 46, demonstrates the results of the field combined tests, conducted in the surface layer of Cambrian clay with undisturbed structure ($M_p = 20.3$, $w_f = 46.5\%$, $\gamma = 2.74$ g/cm^3) in the area of Gorelovo station of Leningrad Oblast the tests were conducted using a conical-vane point whose height is $h = 3.55$ cm and with a diameter of the vane of $D = 4.31$ cm with a $k_\tau = 125$ cm^3 and using a standard conical point $h_{кон} = 55.3$ mm high. In these tests two universal hydraulic VSEGINGEO penetrometers were used. The tests were made at separate sites according to the diagram, presented in Fig. 42. Thus, during the examination of the test results it was found that the average of four values of the resistivity to penetration corresponded to each value of the specific cohesion of the soil which were determined during the penetration tests in the apex of the square test area.

The Cambrian clays at the sites were found to be in a state of stiff plastic and semisolid consistency ($R = 0.4$–1.65 kg/cm^2). The specific cohesion changed from 0.2 to 0.8 kg/cm^2. Following from Fig. 46, two extreme rays (1 and 2) determine the boundary values of the function of proportionality $k_{\phi макс} = 0.758$ and $k_{\phi мин} = 0.456$, which corresponds to a change in the angle of internal friction from 5°30' to 17°.

Following from the given examples, the combined tests by the methods of penetration and vane shear offer considerable scope to the direct determination of the indices of friction and cohesion under both field and laboratory conditions in clay soils ranging from liquid to semisolid consistencies, inclusively. The possibility of using the method of vane shear in soils of solid consistency is connected with specified limitations, examined in the previous section.

Versions of the combined soil tests. Under conditions of solid consistency of the soil, penetration tests can be augmented by failure tests on samples under unconfined compression. As is known, the ultimate pressure during unconfined compression $p_{\text{пред}}$ and the specific cohesion c are interconnected by the condition

$$p_{\text{пред}} = k_c c \text{ kg/cm}^2, \qquad (63)$$

where

$$k_c = 2 \operatorname{tg}\left(45 + \frac{\varphi}{2}\right). \qquad (64)$$

Taking into account the dependence (57) we will find that between the ultimate pressure during unconfined compression and the resistivity to penetration there is a proportional dependence

$$p_{\text{пред}} = k_c k_\varphi R \text{ kg/cm}^2, \qquad (65)$$

whereupon the product of the proportionality factors $k_c k_\phi$ is determined only by angle of internal friction of the soil.

Consequently, the determination of the characteristics of friction and cohesion of cohesive soils of solid consistency can be made using the results of the combined tests by penetration and unconfined compression.

The method of penetration tests can also be successfully used for checking the results of the determinations of the indices of friction and cohesion of cohesive soils by the method of unconfined compression. The essence of the proposed concept is easily revealed during small transformations of Coulomb's equation (58). After the substitution of its value for the specific cohesion c, expressed through the resistivity to penetration according to formula (57), we will have

$$\frac{\tau_{\text{пред}}}{R} = k_\varphi + \frac{p}{R} \operatorname{tg} \varphi. \qquad (66)$$

At the dimensionless coordinates $\tau_{\text{пред}}/R$ and p/R the system of intersected straight lines determines equation (66), depending only on the angle of internal friction of the soil ϕ. The preliminary construction of such a nomograph with the use of graph $k\phi = f(\phi)$ in Fig. 41 does not present difficulties. The working section of the nomograph is included within the intervals $p/R < 1$ and $p/R > 1.8$.

The angle of internal friction and the cohesion of cohesive soils are determined from the results of the combined tests of one-two samples by the methods of penetration and simple shear. After conducting the penetration tests and the determination of R the sample is loaded to any convenient pressure p, at which the ratio p/R falls on the working section of the nomograph. Further on, the ultimate shear stress $\tau_{\text{пред}}$ is determined and the ratio $\tau_{\text{пред}}/R$ is calculated. On the nomograph the experimental point is designated with coordinates $\tau_{\text{пред}}/R$ and p/R, and by means of interpolation between the adjacent straight lines the angle of internal friction of the soil is determined. After the determination of k_ϕ from formula (57) the cohesion of the soil is found.

The checking of the proposed method when $\phi < 22°$ gave completely favorable results.

CHAPTER IV

THE PROCEDURE FOR ESTABLISHING A CORRELATION BETWEEN THE PHYSICO-MECHANICAL PROPERTIES OF COHESIVE SOILS

In the USSR and abroad in recent years a large number of works has been published, dedicated to the development of methods for establishing an objective interrelationship between the physical and mechanical properties of cohesive soils.

Even at the present time one can no longer doubt the fact that between the characteristics of the strength or deformation properties and the characteristics of the physical condition of the soil, under specific conditions, one can obtain a correlation or even functional dependences. The extreme indicative criteria of the achieved successes are the first experiments in the development of generalized and regional norms of the standard and design characteristics of clayey soils stated in SNiP II-B.1-62 and in other publications [55, 56].

With the use of the method of penetration tests it turns out to be possible to completely solve simply and effectively some of the soil science tasks by determining the consistency, plastic limits and other characteristics of cohesive soils. For this purpose first of all let us examine the fundamental principles of the procedure for establishing an interrelationship between the physical and mechanical properties of cohesive soils. First let us present the features in the development of such an interrelationship in connection with the conditions of complete water-saturation of cohesive soils. Then, let us analyze the basic design diagrams for a three-phase condition of the soil.

Phase relationships in soils. As is known, with a specific constant of the degree of water saturation of the soil G_{01} = const between the unit weight δ_1 and the moisture content of the soil w_0 there is a hyperbolic dependence:[1]

$$\delta_1 = \frac{\gamma}{1 + \frac{\gamma w_0}{\gamma_w G_{01}}} \text{ g/cm}^3, \qquad (67)$$

where γ and γ_w = 1 g/cm^3 - unit weight of the soil and water, respectively.

For practical use it is more convenient to present the dependence (67) in the form of the equation which links the

[1]In all future presentations the following designations are used. The first, even index for designations of the degree of water saturation G_{01}, G_{23}, G_{45} etc. determines the moisture content of the investigated condition of the soil w_0, w_2, w_4 etc.; the second, odd index - the unit weight of the soil δ_1, δ_3, δ_5 and etc. An analogous system is used for the designations of the resistivity to penetration R_{01}, R_{23}, R_{45} etc., coefficients of penetration N_{01}, N_{23}, N_{45} etc. and any other characteristics of the mechanical properties of soils. In the water-saturated condition of the soil when G_0 = 1 the unit weight δ_0 and moisture content w_0 are interrelated values; therefore, the designations of the degree of water saturation and characteristics of the mechanical properties of soils have one index: G_0, G_1, G_2, etc., R_0, R_1, R_2, R_3 etc.

"volume weight of the soil skeleton," $1/\delta_1$, i.e., the volume of the soil with the weight of the soil skeleton, equal to unity, and the moisture content of soil w_0:

$$\frac{1}{\delta_1} = \frac{1}{\gamma} + \frac{w_0}{\gamma_w G_{01}} \text{ cm}^3/\text{g}. \qquad (67a)$$

With such an expression of the phase relationships in the soil between the volume weight of the soil skeleton and the moisture content with the constancy of the degree of water saturation of the soil G_{01} = const. there is a linear dependence. Thus at coordinates $w - 1/\delta$ the graphs of the dependence (67a) for cases $G_0 = 1$, G_{01} = const, G_{pf} = const, etc. represent a system of straight lines which intersect at a point on the axis of ordinates with ordinate $1/\gamma$ (Fig. 47).

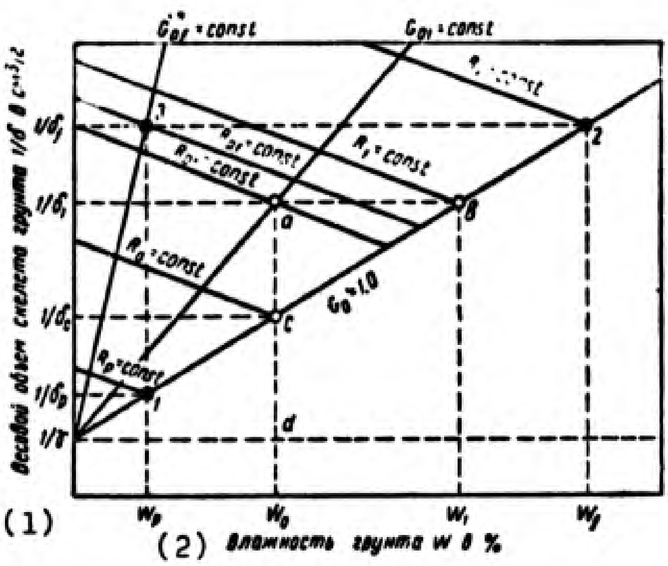


Fig. 47. The design diagram of the phase relationships in the soil for the case of the constant degree of water saturation of the soil, G_{01} = const. The system of parallel lines characterizes the design diagram for the case of the constancy of the resistivity to penetration, R_{01} = const. 1, 2, 3 - points which determine the indicative indexes of the soil.
KEY: (1) Volume weight of the soil skeleton $1/\delta$ in cm^3/g; (2) Moisture content of the soil W in %.

The convenience and clarity of the graphs of dependence (67a) can be seen from the following example. If in Fig. 47 one designates point a with coordinates w_0 and $1/\delta_1$, then the volume of the mineral part of the soil per unit of its weight, i.e., the unit volume of the soil skeleton, is equal to (ordinate of point d):

$$V_{\text{гр}} = \frac{1}{\gamma} \text{ cm}^3/\text{g}.$$

The volume of water in the pores of the soil per unit weight of the soil skeleton comprises (difference in the ordinates of points c and d):

$$V_{\text{воды}} = \frac{1}{\delta_0} - \frac{1}{\gamma} \text{ cm}^3/\text{g}.$$

The volume of air in the pores of soil per unit weight of the soil skeleton is made equal (difference in the ordinates of points a and c and b):

$$V_{\text{возд}} = \frac{1}{\delta_1} - \frac{1}{\delta_0} \text{ cm}^3/\text{g}.$$

The volume by weight of the soil skeleton $1/\delta_0$ under conditions of complete water saturation $G_0 = 1$ is determined according to the dependence

$$\frac{1}{\delta_0} = \frac{1}{\gamma} + \frac{w_0}{\gamma_w} \text{ cm}^3/\text{g}. \tag{67b}$$

Thus, directly from the graphic representation in Fig. 47 it is possible to determine the components of the phase relationships for virtually any values of moisture content and volume by weight of the soil skeleton.

The dependence between the physico-mechanical properties of cohesive soils under conditions of complete water saturation. The dependence between the physical and mechanical properties of soils is most simply revealed for the case of complete water saturation ($G_0 = 1$) of cohesive soils with disturbed structure.

Back in 1948 the outstanding Soviet scholarly, Prof. N. M. Gersevanov theoretically predicted that between the void ratio ε, or, by the same token the moisture content of soil $w = \frac{\varepsilon\gamma_w}{\gamma}$

and the ultimate strength of the soil to shear $\tau_{пред}$ there is a logarithmic dependence [12]. In accordance with N. M. Gersevanov's proposal the equation of linear dependence (58) between the normal pressure p and the resistance of the soil to shear $\tau_{пред}$ is first expressed in the form

$$\tau_{пред} = p\left(\operatorname{tg}\varphi^{\circ} + \frac{c}{p}\right) \text{ kg/cm}^2. \tag{68}$$

For the initial branch of consolidation compression of the soil, the specific cohesion is proportonal to the compacting pressure

$$c = \mu p \text{ kg/cm}^2. \tag{69}$$

Thus

$$\tau_{пред} = k_{\mu} p \text{ kg/cm}^2, \tag{70}$$

where

$$k_{\mu} = \mu + \operatorname{tg}\varphi^{\circ} = \text{const}. \tag{71}$$

The equation of the consolidation compression of the soil, as is known, has a logarithmic nature:[1]

$$\varepsilon_1 = \varepsilon - \frac{1}{A} \ln \frac{p_1}{p}. \tag{72}$$

where ε represents the void ratio when the pressure $p = 1$ kg/cm^2; ε_1 and p_1 - current coordinates of the compression curve.

---

[1]The logarithmic nature of equation (72) was substantiated by N. M. Gersevanov as an "immense number of experiments, conducted by different researchers on sandy and clayey soils" [12]. The simple theoretical derivation of this dependence is based on the physical premises, that under conditions of consolidation compression between a change in the volume of pores of the the soil and the normal pressure, there is an inversely proportional dependence.

After substituting dependence (70) in equation (72), according to N. M. Gersevanov, we will obtain

$$\varepsilon_i = \varepsilon - \frac{1}{A} \ln \frac{\tau_{\text{пред}\,i}}{\tau_{\text{пред}}}. \tag{73}$$

Only in 1962, i.e., after 14 years N. M. Gersevanov, a Professor at Princeton university (USA), V. Ye. Schmid, repeated N. M. Gersevanov's derivation about the existence of a logarithmic dependence between the void ratio and the resistance of the soil to shear.

On the possibility for calculating on a slide rule taking into account the transient modulus ln N = 2.303 lg N and after introducing the designation $1/A_1 = 2.303^1/A$,we will have

$$\varepsilon_i = \varepsilon - \frac{1}{A_1} \lg \frac{\tau_{\text{пред}\,i}}{\tau_{\text{пред}}}. \tag{74}$$

Under conditions of complete water saturation of the soil, the values of the void ratios can be replaced by values of the moisture content:

$$\varepsilon = \frac{w\gamma}{\gamma_w};$$

$$w_i = w - \frac{\gamma_w}{A_0} \lg \frac{\tau_{\text{пред}\,i}}{\tau_{\text{пред}}}\ \%, \tag{75}$$

where

$$A_0 = \gamma A_1\ \text{g/cm}^3.$$

Equation (75) is basic for the analysis of the interrelationship the physicomechanical properties of water-saturated cohesive soils with disturbed structure.

It is obvious that the logarithmic nature of dependence (75) should also be exhibited in the relationship of both components of the resistance of soil to shear.

Thus, there are grounds for considering that between the moisture content and the specific cohesion of soil there is also a logarithmic dependence of the form

$$w_i = w - \frac{\gamma_w}{V_0} \lg \frac{c_i}{c} \ \%. \qquad (76)$$

The logarithmic dependences (75) and (76) at a semilogarithmic scale, as is known, are equalized as a straight line. Thus, the study of the features of the development of these and other analogous dependences in the following presentation will be carried out only at this scale. At coordinates $w - \lg \tau_{пред}$ and $w - \lg c$ (i.e., at a semilogarithmic scale) an angular coefficients γ_w/A_0 and γ_w/V_0 represent the tangents of the slope of the corresponding straight lines to the vertical axis.

In accordance with equation (57) $c = k_\phi R$ between the specific cohesion and the resistivity to penetration there is a proportional dependence with a variable angular coefficient $k_\phi \neq$ const. By means of substitution in equation (76) instead of the ratio c_1/c of the equal relationship $k_{\phi 1}R_1/k_\phi R$ we will obtain

$$w_i = w - \frac{\gamma_w}{V_0}\left(\lg \frac{R_i}{R} + \lg \frac{k_{\phi i}}{k_\phi}\right) \ \% \qquad (77)$$

or

$$w_i = w - \frac{\gamma_w}{r_0} \lg \frac{R_i}{R} \ \%. \qquad (78)$$

where the angular coefficient γ_w/r_0 is equal to

$$\frac{\gamma_w}{r_0} = \frac{\gamma_w}{V_0}\left(1 + \frac{\lg k_{\phi i}/k_\phi}{\lg R_i/R}\right). \qquad (79)$$

In the following section it will be shown that for the investigated soil the ratio of the logarithms $\frac{\lg k_{\phi 1} k_\phi}{\lg R_1/R}$ is a constant value, not depending on the characteristics of the physical condition of the soil.

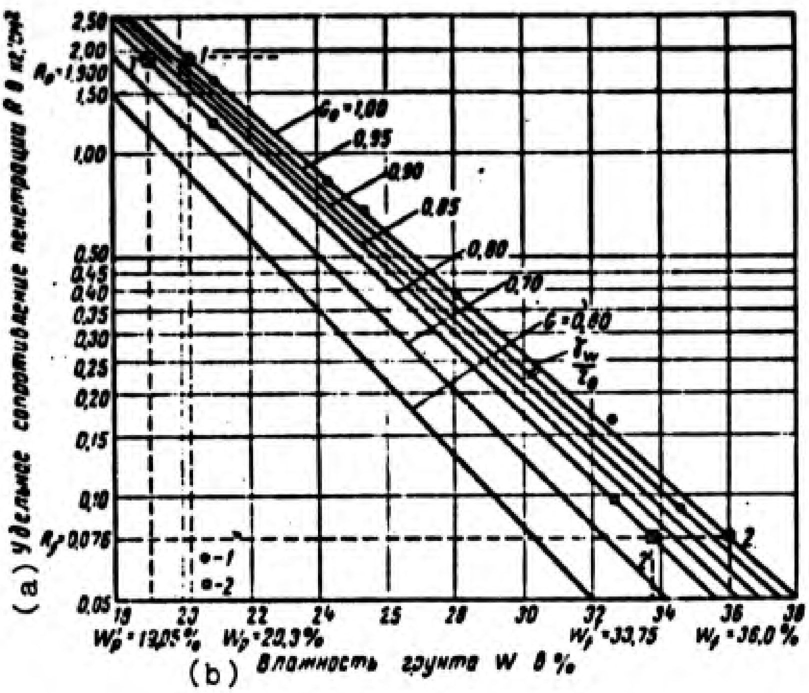


Fig. 48. The design diagram of the linear dependence between the moisture content of the soil and the logarithm of the resistivity to penetration for the case of the constant degree of water saturation of the soil $G_0 = 1$; $G_{01} = 0.95$; $G_{23.} = 0.9$ etc. (at a semilogarithmic scale).
KEY: (a) Resistivity to penetration R in kg/cm^2. (b) Moisture content of the soil w in %.

Thus, the angular coefficient γ_w/r_0 for the investigated soil in accordance with equation (79) is also a constant value. Thus, there is a possibility of stating that under conditions of complete water saturation of cohesive soils, according to equation (78), between the moisture content and the logarithm of the relative resistivity to penetration there is a simple linear dependence (Fig. 48).

In the selection ratio of values w and R in equation (78) of no limitations, naturally it does not apply. As a beginning

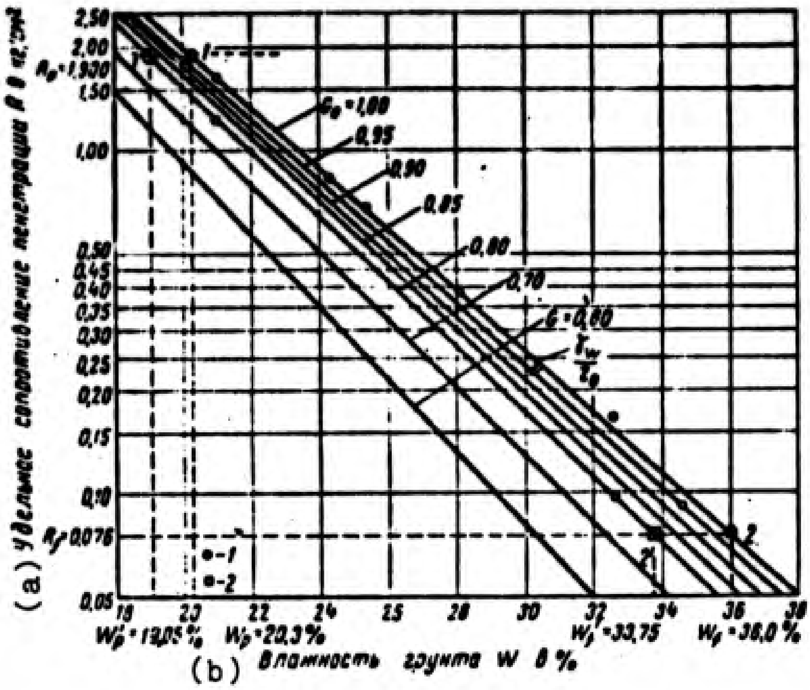


Fig. 48. The design diagram of the linear dependence between the moisture content of the soil and the logarithm of the resistivity to penetration for the case of the constant degree of water saturation of the soil $G_0 = 1$; $G_{01} = 0.95$; $G_{23} = 0.9$ etc. (at a semilogarithmic scale).
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Thus, the angular coefficient γ_w/r_0 for the investigated soil in accordance with equation (79) is also a constant value. Thus, there is a possibility of stating that under conditions of complete water saturation of cohesive soils, according to equation (78), between the moisture content and the logarithm of the relative resistivity to penetration there is a simple linear dependence (Fig. 48).

In the selection ratio of values w and R in equation (78) of no limitations, naturally it does not apply. As a beginning for analyzing line 1-2, depicted in Fig. 48, any convenient point can be taken for the calculations at coordinates w, R.

Usually for practical calculations it is convenient to utilize a point with ordinate R = 1 kg/cm^2. In this case equation (78) assumes the form

$$w_l = w - \frac{\gamma_w}{r_0} \lg R_l \ \%. \tag{80}$$

The slope tangent of line 1-2 to the vertical axis, i.e., angular coefficient γ_w/r_0 in equations (78) and (80), is calculated from the formula

$$\frac{\gamma_w}{r_0} = \frac{w - w_l}{\lg R_l/R} \ \%. \tag{81}$$

Prof. N. M. Gersevanov's classical solution, reproduced above in the form of equations (68) - (73), was derived during the first stage of research on the problem of cohesive soils with disturbed structure. However, as evident from the following, the field of application of this solution can be expanded, and the corresponding equations of logarithmic dependences (74), (75), (76), (78) and others can be successfully used in a number of cases for the analysis of the interrelationship of the physicomechanical properties of cohesive soils with undisturbed structure.

Experimental data on the logarithmic dependence between the physicomechanical properties of water-saturated cohesive soils. As foretold by N. M. Gersevanov the logarithmic dependence between the void ratio (or, in other words moisture content) and the shear strength of water-saturated cohesive soils with disturbed structure was fully confirmed in the experimental research of recent years.

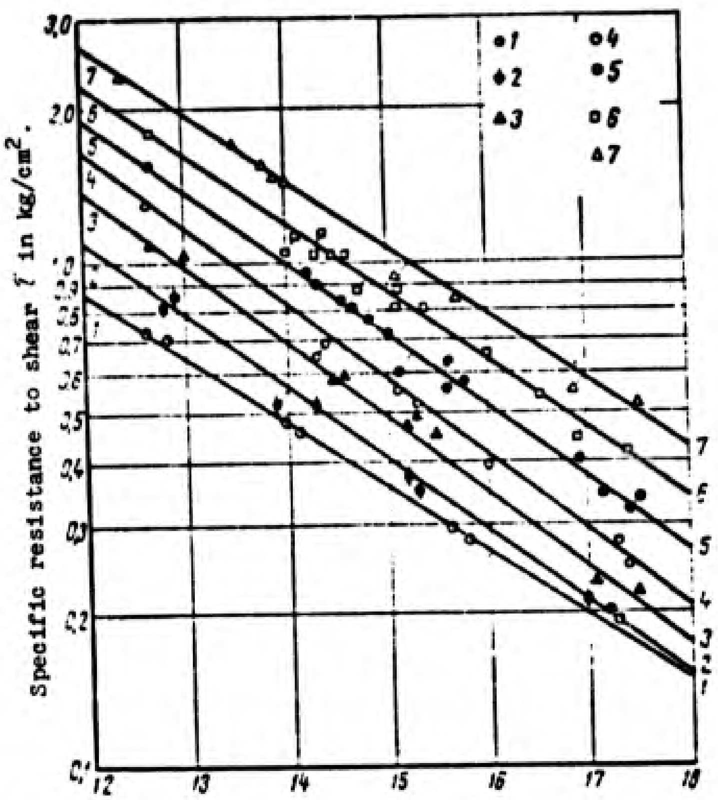


Fig. 49. Graphs of the linear dependence between the moisture content and the logarithm of the resistivity of soil soil to shear, obtained during the testing of samples of water-saturated sandy loam at pressures. 1 - 0.1 kg/cm² 2 - 0.25 kg/cm²; 3 - 0.5 kg/cm²; 4 - 1 kg/cm²; 5 - 2 kg/cm² 6 - 3 kg/cm² and 7 - 5 kg/cm² (according to Prof. N. N. Maslov and V. D. Kazarnovskiy) (at a semilogarithmic scale).

Very interesting analyses were made by professor N. N. Maslov and V. D. Kazarnovskiy [28] in research on the resistance of soils to shear depending on moisture content with different vertical loads. The results of one of a series of shear tests on sandy loam are given in Fig. 49. V. D. Kazarnovskiy and N. N. Maslov, the author of a very interesting and promising method for studying the resistance of soils to shear on the basis of the characteristics of "density-moisture content" [26, 27], and based on the materials of these and other analogous results of experiments, drew the conclusion about a sufficiently ideal development of a logarithmic dependence (75) for a number of investigated soils using different loads of compaction [28].

Analogous results were obtained by Professor M. N. Gol'dshteyn and S. S. Babitskaya [16] during soil tests using a DIIT shear box according to consolidated-undrained methods for shear. At a semilogarithmic scale using coordinates ε - lg $\tau_{\text{пред}}$ linear graphic representations of equation (74) were made with loads from 1 to 4 kg/cm^2 (Fig. 50).

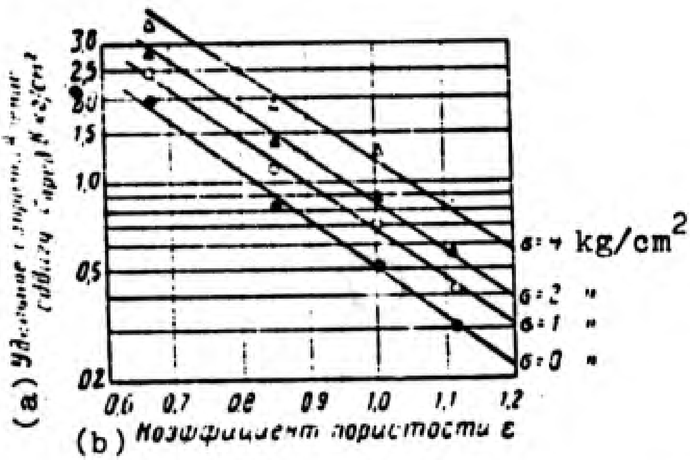
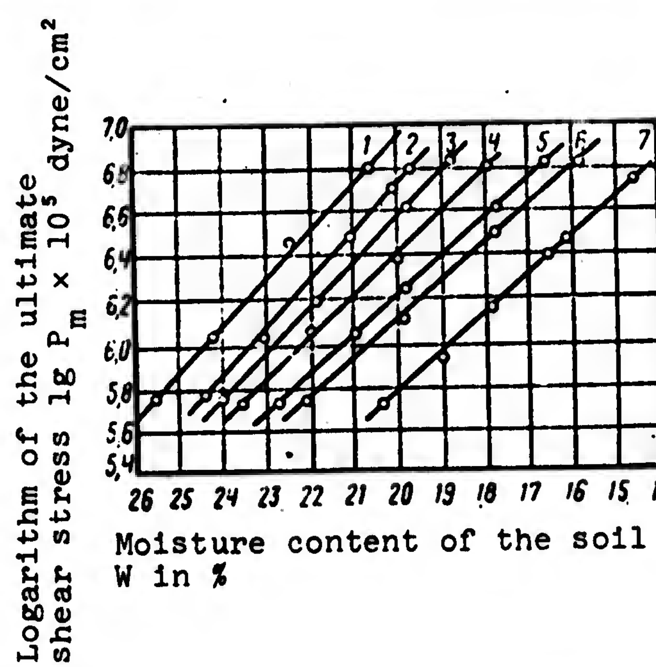


Fig. 50.. Graphs of the linear dependence between the void ratio and the logarithm of resistivity of water-saturated soil to shear, produced at various pressures (according to Prof. M. N. Gol'dshteyn and S. S. Babinskiy) (at a polylogarithmic scale).
KEY: (a) Resistivity to shear in kg/cm^2; (b) Void ratio ε.

The linear dependence between the moisture content of seven different clayey soils with disturbed structure and the logarithm of the ultimate shear stress $P_m = 0.658\ P/h^2\ 10^5$ dynes/cm^2[1] was most clearly revealed by Prof. F. D. Ovcharenko (Fig. 51). In this connection F. D. Ovcharenko noted that with the graphic representation of dependence $P_m = f(1-w)$ a number of curves typical for exponential functions [33, 34] was obtained.

The high accuracy of the development of the linear dependence (80) is easily outlined in another example of the results of penetration tests, made with samples of 13 different water-saturated soils having plasticity indexes from 4.6 to 53 (Fig. 52). In the construction of the graphs the results of the author's experiment using a micropenetrometer are used and also the materials from the studies of I. M. Gor'kova [18], P. O. Boychenko [6] and A. M. Vasiliev [9].

Fig. 51. Graphs of the linear dependence between the moisture content of cohesive soils and the logarithm of the ultimate shear stress $\tau'_{пред} = \rho_\alpha R$, obtained during penetration tests on loess 1, clay loam 3, loess-like clay loam 5, clays 2, 4, 6, and a Chernozem 7, under conditions of complete water saturation (according to Prof. F. D. Ovcharenko) (at a polylogarithmic scale).



[1]The constant of the indenter point $\rho_\alpha = 0.658$ is accepted according to P. A. Rehbinder [43] for an expansion angle of 45°, whereupon P is expressed in grams, and h - in mm. Let us recall that 1 dyne = $1 \cdot 0^{-5}$ n, or 1 dyne ≈ $0.10 \cdot 10^{-5}$ kgs.

With a micropenetrometer the author conducted tests on Nikopol'skiy loess 3, Dushambinskiy clay loam 5, Kudinovskiy clay 9 and Crimean silty and clayey sediments 14. With the aid of a plastometer, P. A. Rehbinder and I. M. Gor'kova completed experiments on Khvalynskaya clay 10, a chernozem 11 and a bentonite from Gilyaba 13. P. O. Boychenko, with the use of a consistometer, tested morainic clay loams 2 and 6, Cambrian clay 7, deluvial loam 8 and a Chasov'yarskaya 12 clay. A. M. Vasiliev, using Vicat's modified probe, conducted experiment on a sandy loam 1 and on a Nikopol'skiy loess 4.

A kg/cm^2

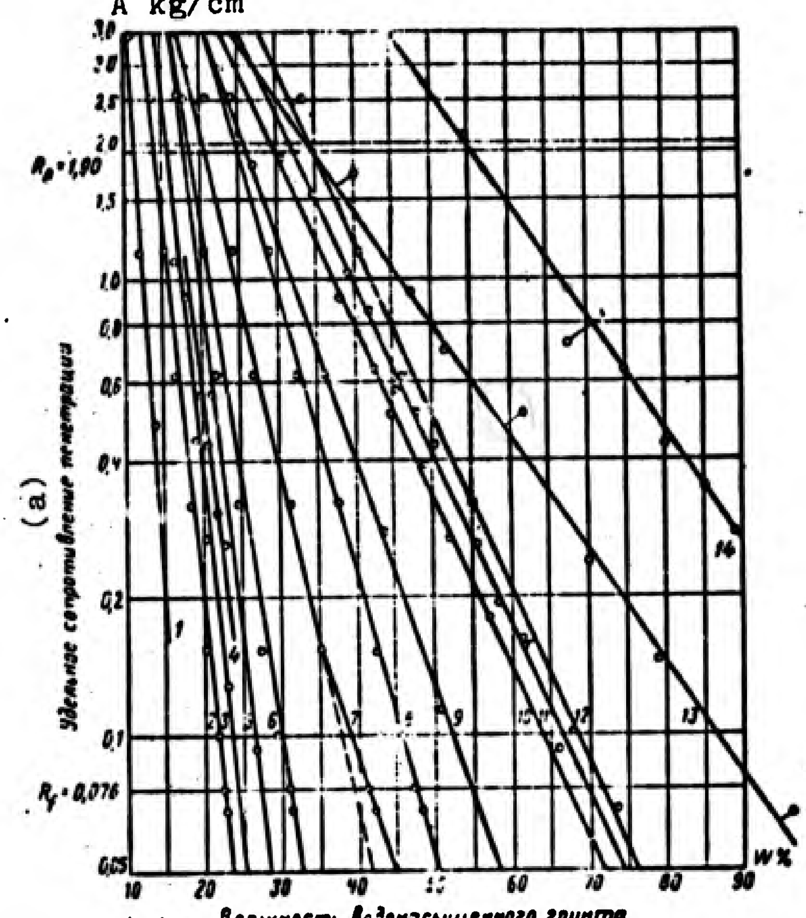


Fig. 52. Graphs of the linear dependence between the moisture content and the logarithm of the resistivity to penetration, obtained during penetration tests of different cohesive soils under conditions of complete water saturation (at a semilogarithmic scale).
KEY: (a) Resistivity to penetration. (b) Moisture content of the water-saturated soil.

It is quite characteristic that the results of the penetration tests on Nikopol'skiy loess 3 and 4, carried out by different workers and with different instruments, completely coincided. The plasticity indexes of this soil, proportional to the slope tangent of lines 3 and 4 to the vertical axis, as one would expect, were obtained identically (Fig. 52).

It would be possible to give dozens of other analogous examples, drawn from the works of Soviet and foreign specialists (M. V. Ivanova, Ye. F. Vinokurov, M. P. Volarovich and S. N. Markov,[1] I. D. Belovidov,[1] D. Mokhan, (W. Black) [59, 60], J. Mitchell [69] G. Sherrer [72], and many others). However, apparently, there is no need to do this, since all other examples completely repeat the graphs in Figs. 49-52.

In this connection it can be stated that there is full validity and sufficiently high accuracy of the development of the linear dependence between the moisture content or the void ratio of water-saturated soils and the logarithm of the characteristics of the mechanical properties revealed by the different methods of mechanical testings.

The possibility for using the logarithmic dependence (76) for the description of the interrelationship between the moisture content and the logarithm of the specific cohesion of the soil can be seen according to the results of experimental research by L. Bjerrum [58], R. Seval'dson, (T. Wu, A. Douglas and R. Goughner) [74] and certain other authors. As an example reproduced in Fig. 65 are the results of experiments by T. Wu,

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[1] In the studies of Prof. M. P. Volarovich, S. N. Markov, I. D. Belovidov ("new physical methods in the study of peat." Gosenergoizdat, Moscow-Leningrad, 1960) clear logarithmic dependences are revealed between the moisture content and the characteristics of the mechanical properties of peat (ultimate shear stress $\tau_{\text{пред}} = \rho_{\alpha}R$ and ultimate pressure under uniaxial compression $p_{\text{пред}}$).

A. Douglas and R. Goughner [74], visually characterizing the presence of a logarithmic dependence (76) for cohesive soils with disturbed and undisturbed structure. The formulation of these results at semilogarithmic scale was done by B. Ranganatham [71], who notes that graphs $w = f(\lg c)$, placed in Fig. 65, agree "with the well known and long established linear dependence" between the moisture content and the logarithm of specific cohesion of the soil.

The constancy of the angular coefficient γ_w/r_0 in equations (78) and (80), i.e., the constant proportions $\frac{\lg k_{\phi 1}/k_\phi}{\lg R_1/R}$ in formula (79) was tested by the author according to SNiP II-B.1-62 and according to materials b. of the SRI on construction at Sverdlovsk city on the standard characteristics of the friction and cohesion of cohesive soils of quaternary deposits and eluvial cohesive soils of the Urals [55, 56]. As is known, in Table 13 of the SNiP II-B.1-62 and in the table of regional norms there are standard (mean statistical) values of the specific cohesion c, angle of internal friction $\phi°$ and the modulus of deformation E for separate intervals of moisture content at the plastic limits w_p and for the void ratios ε.

The calculations were made in the following sequence.

1. According to the tabular values of the angles of internal friction of the soil ϕ based on the auxiliary graph $k_\phi = f(\phi°)$ (Fig. 41) the proportionality factor k_ϕ was determined.

2. On the basis of dependence $R = \frac{c}{k_\phi}$ according to the tabular values of specific cohesion and the known coefficients k_ϕ the resistivity to penetration R for the assigned intervals w_p and ε were calculated.

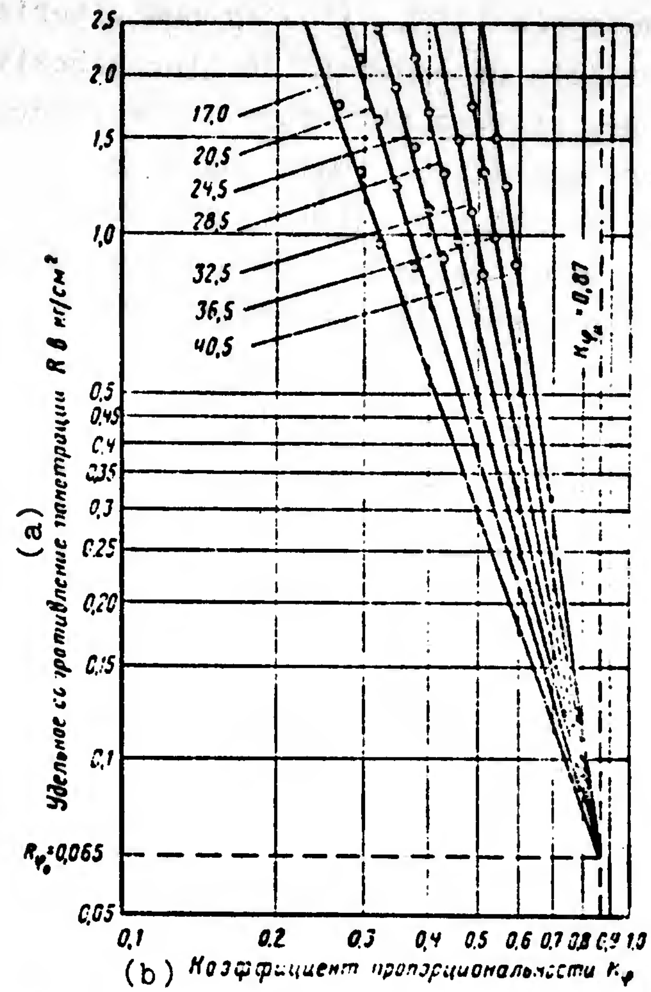

Fig. 53. Family of graphs of linear dependence $\lg k_\phi = f(\lg R)$ for eluvial soils with different values of the plastic limit (according to V. B. Shvets) (at a logarithmic scale).
KEY: (a) Resistivity to penetration R in kg/cm². (b) Proportionality factor K_ϕ.

As an example in Fig. 53 at coordinates $\lg k_\phi - \lg R$ a family of graphs of dependence (82) is given, plotted from the specified data of V. B. Shvets for the case of the constancy of the plastic limits of the soil, w_p = const [56]

$$\lg \frac{k_{\varphi i}}{k_\varphi} = \left(1 - \frac{V_0}{r_0}\right) \lg \frac{R_i}{R}. \qquad (82)$$

It is easy to see that the ratios $\frac{\lg k_{\phi 1}/k_\phi}{\lg R_1/R}$ represent tangents of the angle of the plotted straight lines to the vertical axis in Fig. 53, and therefore for any of the investigated soils this ratio is a constant value.

Let us incidentally make one important observation. With certain portions of the schematization it is possible to consider that all graphic representations of dependence (82) in Fig. 53 intersect at one point with abscissa $k_{\phi_0} = 0.87$, which corresponds to the "boundary of friction," i.e., to such a condition in eluvial soils, in which the angle of internal friction is equal to zero, $\phi° = 0$ (see graph $k_\phi = f(\phi°)$ in Fig. 41). In this case the resistivity to penetration at the boundary of friction is $R_{\phi_0} = 0.065$ kg/cm^2, and therefore the equation of lines in Fig. 53 take the form

$$\lg k_{\varphi i} = -0{,}06 + \left(1 - \frac{V_0}{r_0}\right)(1{,}187 + \lg R_i). \tag{82a}$$

The angular coefficients $1 - V_0/r_0 = \frac{\lg k_{\phi 1}/k_\phi}{\lg R_1/R}$ for each of the straight lines in Fig. 53 are determined directly from the graphic representation. For exàmple, for line 3 which characterizes dependence (82a) for a light clay loam, when $w_p = 24.5\%$ and $M_p \approx 10.2$, the angular coefficient $1 - V_0/r_0 = -0.274$. Consequently, equation (82a) can be converted to the form

$$\lg k_{\varphi i} = -0{,}385 - 0{,}274 \lg R_i. \tag{82b}$$

Dependence (82b) and other analogous dependences, which can be obtained for different values of the plastic limit w_p, have already doubtless practical value. It turns out that the condition of the independence of angular coefficients γ_w/r_0 and γ_w/V_0, and respectively their ratio of V_0/r_0 from the characteristics of the physical condition of the soil provide considerable scope for solving some actual engineering geology problems.

For example, the combined tests on cohesive soils by the methods of penetration and vane shear make it possible easily and simply, to disclose the angular coefficient $1 - V_0/r_0$. Consequently, according to the earlier developed graphs of the dependence (82) for soils of determined stratigraphic and genetic formations, the determination of the plasticity index of the soil and the corresponding moisture content at the liquid and plastic limits is possible.

On the contrary, if are known the indicative criteria of the investigated soil (plasticity index M_p and moisture content at the liquid limit w_f or plastic limit w_p), then according to the results of the penetration tests the characteristics of the friction and cohesion of the soil can be determined.

For the confirmation of the expressed positions in Fig. 54 at a logarithmic scale the results of the combined testings of 29 water-saturated samples of a loess-like clay loam with disturbed structure (M_p = 12.7, w_f = 33.9%, γ = 2.68 g/cm^3) are given, made by V. G. Zabara by the methods of penetration and vane shear using a LP-1 probe with an attachment for vane shear [41]. Plotted along the axis of the abscissae are the values of the specific cohesion c, along the axis of the ordinates - the value of the resistivity to penetration R. On the graph at all experimental points the values of the moisture contents in percentages by weight, are designated and besides that, the plastic limits (R_p = 1.90 kg/cm^2) and liquid limits (R_f = = 0.076 kg/cm^2) are designated by dotted lines.

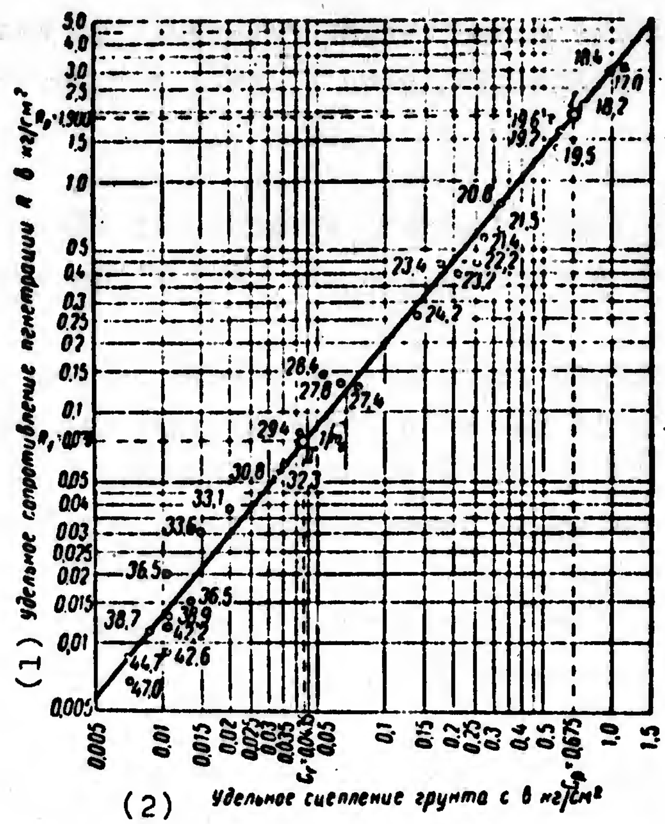

Fig. 54. Graph of the linear dependence between the logarithms of the specific cohesion and resistivity to penetration, obtained during the testing of samples of loess-like clay loam under conditions of complete water saturation. KEY: (1) Resistivity to penetration R in kg/cm². (2) Specific cohesion of the soil c in kg/cm².

From equations (76) and (78) it directly follows that the tangent of the angle of the plotted line to the vertical axis is equal to the ratio of the angular coefficients

$$\frac{1}{m_0} = \frac{\gamma_w/r_0}{\gamma_w/V_0} = \frac{\lg c_1/c}{\lg R_1/R}. \qquad (83)$$

The angular coefficient $1/m_0$ was calculated by V. G. Zabara by the method of least squares, and it turned out to be equal to 0.853. The coefficient of the linear correlation amounted to $r_к = 0.98$, which corresponds to a functional interrelationship.

The results of the experimental research, an example of which is presented in Fig. 54, give a basis for the following conclusions:

a) under conditions of complete water saturation of cohesive soils with disturbed structure the theoretical dependence $c = k_{\phi}R$;

b) at a logarithmic scale when a change in the consistency of the soil from liquid to solid, inclusively, (in Fig. 54 the ratio $R_{\text{макс}}/R_{\text{мин}} = 444$), the angular coefficient $1/m_0 = V_0/r_0$ remains strictly constant. Thus, when $G_0 = 1$ for the investigated clay loam in terms of any known value of resistivity to penetration, it is easy and simply to determine the angle of internal friction and the cohesion of the soil;

c) with a decrease in the moisture content the cohesion and the angle of internal friction increase regularly. For example, for extreme points in Fig. 54 when $w = 44.7\%$ $c = 0.007$ kg/cm^2 and $\phi = 3° 30'$; and $w = 18.4\%$ $c = 1$ kg/cm^2 and $\phi = 21°30'$.

On the basis of the foregoing it is possible to propose a rational diagram of the engineering geological by investigations, by which, in the first place, the interrelationships of the physical and mechanical properties of the soils are expressed in the form of values of the resistivity to penetration, and, in the second place, by which the angular coefficient $1 - V_0/r_0$ is established. It is obvious, that in this case in accordance with the procedure of Prof. N. N. Maslov [26, 27] the dependences between the physical characteristics and the angle of internal friction and that of the separately specific cohesion of the investigated soil are incidentally determined.

The method for solving the first part of this very interesting problem is examined in the following section.

The dependences between the physicomechanical properties of cohesive soils under conditions of a three-phase state. A more complex dependence exists between the physical and mechanical properties of cohesive soils under conditions of a three-phase state. On the basis of research and generalization of the available experimental materials it turned out that one can conclude that equation (75), (78) and others, which determine the logarithmic dependences between the physicomechanical conditions of the soil under conditions of complete water saturation ($G_0 = 1$), represent the quotient in the case of analogous equations of the logarithmic dependence for the conditions of the constant degree of water saturation of the soil, G_{01} = const < 1.

In connection with the results of penetration tests it is possible to propose the following basic equation of a design diagram:

$$w_l L_0 = w - \frac{\gamma_w}{\gamma_0} \lg \frac{R_l}{R} \%, \qquad (84)$$

where coefficient L_0 is the exponent of the water saturation, of the soil, $L_0 = f(G_{01})$. For the case of complete water saturation of the soil when $G_0 = 1$ = const, the auxiliary coefficient $L_0 = 1$ and equation (84) can be converted into an already known equation (78)

For an illustration of this position several calculated graphs of the linear dependences between the moisture content of the soil and the logarithm of the resistivity to penetration for different values of the degree of the water saturation of soil, G_{01} = const are depicted in Fig. 48. Below it will be shown that with a decrease in the degree of water saturation of the soil, the values of the auxiliary coefficient L_0 will increase. Consequently, with a decrease in G_{01} the angular coefficient γ_w/r_0L_0 will also decrease. Thus, in Fig. 48, the slope tangents of the corresponding lines to the vertical axis gradually diminish.

For the expansion of the structure of the coefficient L_0 let us examine:

a) resistivity to penetration R_0 of a water-saturated soil with a moisture content w_0;

b) resistivity to penetration R_{01} of a three-phase soil with a moisture content w_0 and volume weight of the soil skeleton $1/\delta_1$, i.e., with a maximum moisture capacity w_1.

Then, the fundamental equations (78) and (84) obtain acquire the form

$$w_0 = w - \frac{\gamma_w}{r_0} \lg \frac{R_0}{R} \ \%; \qquad (78a)$$

$$w_0 L_0 = w - \frac{\gamma_w}{r_0} \lg \frac{R_{01}}{R} \ \%. \qquad (84a)$$

By subtracting equation (78a) from equation (84a) we obtain

$$w_0 (L_0 - 1) = \frac{\gamma_w}{r_0} \lg \frac{R_0}{R_{01}} \ \%. \qquad (a)$$

In order to determine the logarithm of the ratio R_0/R_{01} let us assume that in the case of a constant moisture content of the soil (w_0 = const) an increase in the volume weight of the soil skeleton $1/\delta_1 - 1/\delta_0$ is proportional to an increase in the logarithms of the corresponding values of the resistivity to penetration, i.e., a dependence is exhibited, analogous to the case of complete water saturation of the soil:

$$\frac{1}{\delta_i} - \frac{1}{\delta} = -\frac{1}{r} (\lg R_i - \lg R) \ см^3/г, \qquad (85)$$

where $1/r$ - proportionality factor. The minus sign indicates that with an increase in the volume weight of the soil skeleton the value of the resistivity to penetration will decrease.

When $R_1 = R_0$, $1/\delta_1 = 1/\delta_0$ and $R = R_{01}$, $1/\delta = 1/\delta_1$ we will obtain

$$\lg \frac{R_0}{R_{01}} = \frac{1/\delta_1 - 1/\delta_0}{1/r}. \qquad \text{(b)}$$

However, in accordance with the dependence (67b)

$$\frac{1}{\delta_1} - \frac{1}{\delta_0} = \frac{w_1 - w_0}{\gamma_w}, \qquad \text{(c)}$$

therefore

$$\lg \frac{R_0}{R_{01}} = \frac{w_1 - w_0}{\gamma_w/r}. \qquad \text{(d)}$$

After setting up the dependence (r) in equation (a) and the small transformations when $G_{01} = w_0/w_1$, we will finally obtain

$$L_0 = 1 + \left(\frac{1}{G_{01}} - 1\right) \frac{\gamma_w/r_0}{\gamma_w/r}. \qquad (86)$$

For the investigated soil the angular coefficients γ_w/r_0 and γ_w/r represent constant values. When $G_{01} = G_0 = 1$, the auxiliary coefficient $L_0 = 1$. With a decrease in the degree of water saturation of the soil, the coefficient L_0 will actually increase. For example, the graphs of dependence (84a) in Fig. 48 were calculated when $\dfrac{\gamma_w/r_0}{\gamma_w/r} = 0.27$.

The system of equations (84a) and (86) is the general form of equations of the interrelationship between the physical and mechanical properties of cohesive soils under conditions of a three-phase state.

For research on the effect of the moisture content and volume weight of the soil skeleton on the variability of the characteristics of the mechanical properties of soils in connection with the features of different cases of the engineering geological investigations, let us examine the method of constructing the design diagrams of the interrelationship of the physico-mechanical conditions of the soil for the following cases: the constancy of the characteristics of the mechanical properties of soils, R_{01} = const.; the constancy of the moisture content of the soil, w_0 = const.; the constancy of the volume weight of the soil skeleton, δ_1 = const.

The design diagram of the variability of the mechanical properties of soils for the case R_{01} = const. Among the variants of design diagrams of the interrelationships of the physico-mechanical properties of cohesive soils with disturbed and undisturbed structure, special attention will be given to the use of the design diagram for the case of the constancy of the characteristics of the mechanical properties of the soils R_{01} = = const., R_{23} = const. and etc.

By substituting the value of the coefficient L_0 from equation (86) into equation (84a) a new equation of the interrelationship between the volume weight of the soil skeleton $1/\delta_1$ and the moisture content of the soil w_0 for the case N_{01} = = $\frac{R_{01}}{R}$ = const is established:

$$\frac{1}{\delta_1} = \Delta_N - k_N w_0 \quad \text{cm}^3/\text{g}, \tag{87}$$

where

$$\Delta_N = \frac{1}{\gamma} + \frac{w_f}{\gamma_w B_{\kappa p f}} - \frac{1}{r} \lg \frac{R_{01}}{R} \quad \text{cm}^3/\text{g}; \tag{88}$$

$$k_N = \frac{1 - B_{\kappa p f}}{\gamma_w B_{\kappa p f}} \quad \text{cm}^3/\text{g} \tag{89}$$

and

$$B_{\kappa pf} = \frac{\gamma_{w}/r_0}{\gamma_{w}/r}. \qquad (90)$$

The concept and content of the conventional designations $B_{\kappa pf}$ will be clear from the following.

The angular coefficient k_N in linear equation (87), as it follows from the formula (89), is a constant value. Consequently, at coordinates moisture content w - volume weight of the soil skeleton $1/\delta$; the case R_{01}/R = const. is characterized by a system of parallel lines which were regularly shifted at the origin with an increase of the resistivity to penetration (Fig. 47).

From formula (88) it follows that the law governing the shift of lines R_{01}/R = const. to the origin is logarithmic. Thus, from the examination of the design diagram in Fig. 47 a important position immediately follows that for all other cases of G_{01} = const., w_0 = const and δ_1 = const. among one of the characteristics of the physical condition of the soil and the logarithm of the coefficient of penetration lg R_{01}/R there is a linear dependence (see Figs. 48 and 55).

The design diagram for the case R_{01}/R = const., presented in Fig. 47, can be considered as a basic nomograph-data sheet of the investigated soil which establishes the correlation between the physical and mechanical properties of cohesive soils.

Design diagrams of the variability of the mechanical properties of soils for the cases w_0 = const and δ_1 const. The design diagrams of the interrelationship between the physical and mechanical properties of soils for the cases w_0 = const. and δ_1 = const. are presented in Fig. 55. The corresponding equations of the calculated diagrams are easily obtained from the basic equations (84a) and (86).

and

$$B_{\kappa pf} = \frac{\lg w/f_0}{\lg w/f}. \qquad (90)$$

The concept and content of the conventional designations $B_{\kappa pf}$ will be clear from the following.

The angular coefficient k_N in linear equation (87), as it follows from the formula (89), is a constant value. Consequently, at coordinates moisture content w - volume weight of the soil skeleton $1/\delta$; the case R_{01}/R = const. is characterized by a system of parallel lines which were regularly shifted at the origin with an increase of the resistivity to penetration (Fig. 47).

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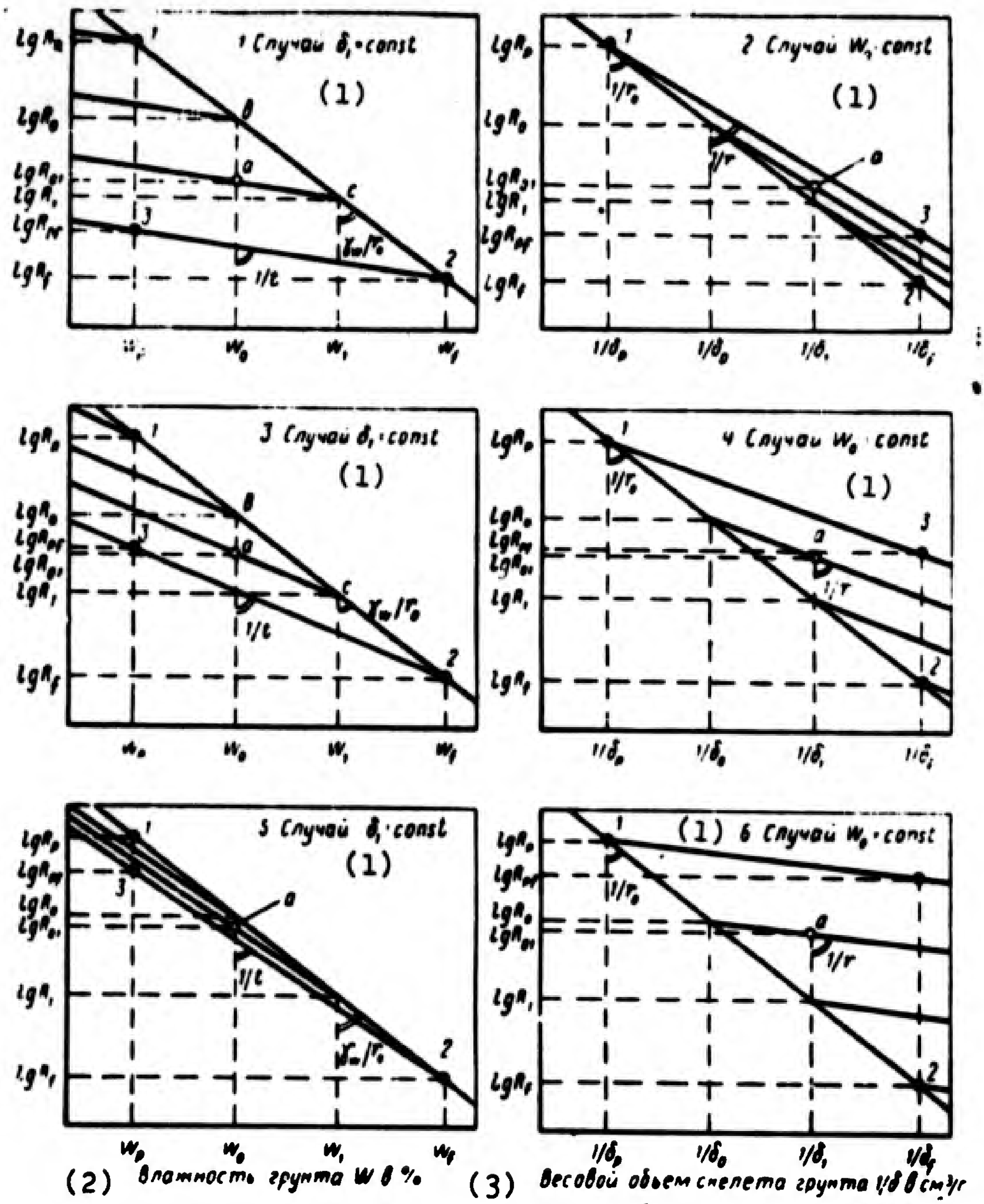

Fig. 55. Design diagrams of the linear dependences between the moisture content and volume weight of the soil skeleton (abscissa) and the logarithm of the resistivity to penetration (ordinate) for the cases δ_1 = const. and w_0 = const., respectively. Cases 1 and 2 correspond to the test data of slightly hydrophyllic soils, cases 3 and 4 - hydrophilic soils, and cases 5 and 6 - strongly hydrophyllic soils. 1, 2, 3 - points which characterize the indicative criteria of cohesive soils (at a semilogarithmic scale).
KEY: (1) Case. (2) Moisture content of the soil W in %.
(3) Volume weight of the soil skeleton $1/\delta$ in cm^3/g.

For example, for the case w_0 = const., at first the moisture content of the soil w_0 on the left side of equation (84a) is expressed through the volume weight of the soil skeleton $1/\delta_1$ and the degree of water saturation of the soil G_{01}. Based on formula (67a) we will have

$$w_0 = \gamma_w\left(\frac{1}{\delta_1} - \frac{1}{\gamma}\right)G_{01}. \tag{67c}$$

The product $G_{01}L_0$ taking into account dependences (86) and (90) is equal to:

$$L_1 = G_{01}L_0 = 1 - (1 - G_{01})(1 - B_{\text{кр}}), \tag{91}$$

whereupon when G_0 = 1, the auxiliary coefficient L_1 = 1. As it follows from dependence (91), with a decrease in the degree of water saturation of the soil, coefficient L_1 will also decrease.

Analogously, the moisture content w for the case G_0 = 1, which corresponds to the resistivity to penetration R, is expressed in the form

$$w = \gamma_w\left(\frac{1}{\delta} - \frac{1}{\gamma}\right). \tag{67d}$$

After substituting dependences (67c) and (67d) in equation (84a) and reducing by γ_w we will obtain

$$\left(\frac{1}{\delta_1} - \frac{1}{\gamma}\right)L_1 = \left(\frac{1}{\delta} - \frac{1}{\gamma}\right) - \frac{1}{r_0}\lg\frac{R_{01}}{R}\ \text{cm}^3/\text{g}. \tag{92}$$

The system of equations (92) and (91) determines the general dependence between the physicomechanical properties of cohesive soils for the case w_0 = const., w_2 = const. and etc.

Another particular method of expressing this interrelationship immediately follows from equation (87) and dependences (88) and (89). After some regrouping of the terms which are included in equation (87) we will arrive at the already known dependence

$$\frac{1}{\delta_1} = \frac{1}{\delta_w} - \frac{1}{r} \lg \frac{R_{01}}{R} \qquad (85a)$$

where

$$\frac{1}{\delta_w} = \frac{1}{\gamma} + \frac{w - w_0(1 - B_{\text{кр}1})}{\gamma_w B_{\text{кр}1}}. \qquad (93)$$

The absolute term in equation (85a) $1/\delta_w$ determines the volume weight of the soil skeleton with which the investigated soil with the different values of the moisture content w_0 or w_2 or w_4 etc. has the coefficient of penetration $R_{01}/R = 1$.

At a semilogarithmic scale the graph of equation (85a) is depicted as a straight line. The angular coefficient $1/r$ represents the slope tangent of this line to the vertical axis.

The value of the angular coefficient $1/r$ is determined only by the lithologic-genetic features of the soil and it does not depend on the characteristics of its physical properties. Thus, as can be seen from Fig. 55 (diagrams 2, 4 and 6), at coordinates "volume weight of the soil skeleton $1/\delta$ - logarithm of the resistivity to penetration lg R," the graphs of dependence (85a) for the different values of the moisture content of the soil represent a family of parallel lines which adjoin the secant 1-2. The latter characterizes the graph of dependence (92) for the conditions of complete water saturation soil when $G_0 = 1$.

The design diagram of the interrelationship of the physicomechanical conditions of the soil for the case w_0 = const. is most expedient for the grapho-analytic working method of the results of penetration tests made on samples of cohesive soils of approximately identical moisture content but with substantially different values of volume weight of the soil skeleton.

The particular method of expressing the interrelationship between the physicomechanical properties of the soils for the case δ_1 = const. also directly follows from equation (87) and dependences (88) and (89). After the repeated regrouping of the members entering equation (87) we will obtain the new equation of the linear dependence between the moisture content of the soil and the logarithm of the relative resistivity to penetration:

$$w_0 = w_i - \frac{1}{t} \lg \frac{R_{01}}{R}\ \%, \qquad (94)$$

where

$$w_i = \frac{w - w_1 B_{\kappa p l}}{1 - B_{\kappa p l}}\ \%; \qquad (95)$$

$$\frac{1}{t} = \frac{\gamma_w}{\gamma_0 (1 - B_{\kappa p l})}\ \%. \qquad (96)$$

The absolute term in equation (94) determines the moisture content, at which the investigated soil with different values of volume weight of the soil skeleton δ_1 or δ_3 or δ_5 etc. has the coefficient of penetration R_{01}/R = 1.

Just as in the preceding case, the value of the angular coefficient 1/t does not depend on the characteristics of the physical properties of the soil. Thus, at coordinates "moisture content of the soil w - logarithm of the resistivity to penetration lg R," the graphs of dependence (94) at different values of the volume weight of the soil skeleton they represent a family of parallel lines adjoining secant 1-2 (Fig. 55, diagram 1, 3 and 5). The secant is the graph of dependence (84a) for the conditions of complete water saturation of the soil when G_0 = 1.

The angular coefficient 1/t represents the slope tangent of the family of parallel lines to the vertical axis.

From expressions (90) and (96) the following important dependence is easily established:

$$\frac{r_0}{\gamma_w} = \frac{r}{\gamma_w} + t\ \%. \tag{97}$$

Thus, the angular coefficients r_0/γ_w, r/γ_w and t in equations of the correlation between the physical and mechanical properties of three-phase and two-phase cohesive soils with disrupted and undisturbed structure are also interrelated.

Depicted in Fig. 55 are three versions of the graphs of dependences (84a), (86) and (92), (91) for the cases δ_1 = const. and w_0 = const. The two upper graphs (diagrams 1 and 2) correspond to the test data of slightly hydrophyllic soils ($B_{kpf} > 0.46$), which have a variability of the mechanical properties which barely depends on the change in moisture content and basically is determined by a change in the volume weight of the soil skeleton. The two lower graphs (diagrams 5 and 6) correspond to the test data of strongly hydrophyllic soils ($B_{kpf} < 0.28$). In soils of this type the variability of mechanical properties in a decisive way depends on a change in the moisture content and barely depends on a change in the volume weight of the soil skeleton.

In accordance with the features of the design diagrams, depicted in Fig. 55, let us note that diagram 2 for the case w_0 = const. and diagram 5 for the case δ_1 = const. are not recommended for use with the grapho-analytic working method of the results of experiments.

The condition of constancy of angular coefficients γ_w/r and 1/t. The recommended design diagrams naturally to a certain degree simplify the more complex correlations between the physical and mechanical properties of soils, and because of this, commonly have specified limitations in application.

Special and sufficiently laborious experimental research showed, that the constancy, for example, of the angular coefficient γ_w/r, which determines the intensity of change of the mechanical properties of the soil under conditions of a three-phase state, is quite distinctly exhibited only when observed under a specified condition.

In connection with cohesive soils with disturbed structure it is necessary that the initial volume weight of the soil skeleton from which the samples will be molded for the test work; have the specified conformity with the accepted value of moisture content. In this case the greater the moisture content of the soil, the less will be the initial volume weight of the soil skeleton.

In an opposite case under conditions of the measured constancy of the initial unit weight of the soil skeleton for individual batches of soil which have substantially different moisture contents, the situation might arise where subsequent compaction of the samples by static loading up to an assigned value of the unit weight will result in otherwise less characteristic values of the specific resistivity to penetration. In this case different values of the mechanical properties of the soils can be produced; in other words where there is one and the same physical state of compaction of the samples (w_0 = const $1/\delta_1$ = const).

In summation, at coordinates $1/\delta$ - lg R a system of regularly diverging lines with substantially variable values of the angular coefficient γ_w/r will be obtained. However, even under these conditions the dependence $1/\delta_0 = f(\lg R_0)$ when $G_0 = 1$ remains constant. Thus, the particular values of the angular coefficients γ_w/r_1 can be predicted in advance for the different values of the moisture content utilizing the data on the initial the value of the volume weight of the soil skeleton.[1]

This phenomenon is found in a specified analogy with three basic cases of soil compaction, examined V. G. Khilobok and the author in materials of the V-th conference on strengthening and compacting of soils [42].

Under field conditions during fill construction of embankments and dams in water-engineering constructions, filling of automobile and railroad grades and other earth construction under the action of gravitational settling and traffic the necessary conformity between the moisture content and the initial volume weight of the soil skeleton of the fill is always and completely rapidly established. Thus, during the analysis of dependences $1/\delta_1 = f(\lg R_{01})$ when $w_0 = \text{const.}$ with the use of samples taken from the body of earth dams a strict constancy of the angular coefficient γ_w/r was distinctly observed.

Determination of the coefficient of consistency of cohesive soils. According to p. 2.10 of the SNiP II-B. 1-62, the consistency of cohesive soils under conditions of complete water saturation is measured by the index of consistency $B_н$, determined according to the formula

$$B_н = \frac{w - w_p}{w_l - w_p}. \tag{98}$$

During the mass calculations it is mnemonically more conveniently, when with an increase in the resistivity to penetration, the characteristics of consistency also increase, but does not decrease. Thus, in the present work the characteristic of the coefficient of consistency $M_н$, proposed by Z. A. Makeyev is frequently utilized:

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[1]Examples of such nomographs were published by V. G. Zabara, V. D. Shitov and the author in materials of the All-Union Conference on the methods of engineering geological and hydrogeological substantiation of the irrigation systems in loess regions (Kiev, 1966).

$$M_{\kappa} = \frac{w_f - w}{w_f - w_p}. \tag{99}$$

The index and coefficient of consistency of soils are related by the dependence $B_{\kappa} = 1 - M_{\kappa}$.

If in expressions (98) and (99) instead of the values of moisture content w, w_p and w_f we substitute the right side of equation (80) with corresponding values of the resistivity to penetration R, R_p and R_f, then it is easy to obtain the important dependences for determining the characteristics of consistency according to the results of penetration tests:

$$B_{\kappa} = \frac{\lg R_p/R}{\lg R_p/R_f} \tag{100}$$

$$M_{\kappa} = \frac{\lg R/R_f}{\lg R_p/R_f}, \tag{101}$$

where R_p and R_f - resistivity to penetration at the plastic w_p and liquid limits w_f.

In accordance with the determination of the angular coefficients γ_w/r_0 and γ_w/r as the slope tangents of the corresponding graphs of the linear dependences (78) and (85a) to the vertical axis, for example for the conditions of complete water saturation of the soil taking into account formula (81), we will have

$$\gamma_w/r_0 = \frac{w_f - w_p}{\lg R_p/R_f}\ \%. \tag{81a}$$

For the conditions of constant moisture content of the soil let us turn to the design diagrams, presented in Fig. 55 (cases 2, 4 and 6). Let us introduce into the examination the value of the resistivity to penetration R_{pf} for point 3, which corresponds to the physical condition of the soil with the moisture content at the plastic limit w_p and with the volume weight of the soil skeleton at the liquid limit $1/\delta_f$. The relative resistivity to penetration in this condition of the soil with disturbed structure $N_{pf} = R_{pf}/R_f$ in the following presentation is called the index of hydrophilic behavior.

Then, analogously with dependence (81a) for conditions w_p = const. further on we will obtain

$$1/r = \frac{1/\delta_f - 1/\delta_p}{\lg R_p/R_{pf}} \text{ cm}^3/\text{g} \tag{102}$$

or

$$\gamma_w/r = \frac{w_f - w_p}{\lg R_p/R_{pf}} \ \%. \tag{103}$$

The ratio of the angular coefficients γ_w/r_0 and γ_a/r in accordance with formulas (90) and (100) is equally characteristic to the index of consistency $B_{\kappa pf}$ at point 3 with the moisture content at the plastic limit w_p and with the volume weight of the soil skeleton at the liquid limit $1/\delta_f$:

$$B_{\kappa pf} = \frac{\gamma_w/r_0}{\gamma_w/r} = \frac{\lg R_p/R_{pf}}{\lg R_p/R_f}. \tag{104}$$

In the practice of soil mechanics analyses, unfortunately, one does not usually consider that the calculation of the index and coefficient of the consistencies in formulas (98) and (99) can be exclusively for the conditions of complete water saturation of the soil.

Besides, the dependences (98) and (99) themselves represent, as is known, unsuccessful computed formulas. The determination of the consistency as the ratio of the differences of sufficiently close values unavoidably leads to considerable errors.

Meanwhile, the direct determination of the index and coefficient of the consistencies according to the results of penetration tests does not impose limitations. The remarkable property of the formulas (100) and (101) lies in the fact that they can be utilized for determining the index and coefficient of consistencies of soils with both disturbed and undisturbed structure and in water-saturated soils, as well as soils in three-phase state.

In fact, the consistency of soil serves as the physical characteristic of the degree of mobility of the soil particles under the effect of external forces and is proportional to the characteristics of the mechanical conditions of the soil in the investigated state. If the studied characteristics of the mechanical properties of the soil are equal, then this means that there is equality of the corresponding consistencies. Thus, in a general case of a three-phase condition of the soil, the generalized coefficient of consistency of the soil $M_{к01}$ with a moisture content w_0 and a volume weight of the soil skeleton $1/\delta_1$ is determined from the formula[1]

$$M_{к01} = \frac{\lg R_{01}/R_f}{\lg R_p/R_f}. \qquad (105)$$

Formula (105) can be presented in a form, making it possible to calculate the generalized coefficient of consistency of the soil in a three-phase state bassed on known characteristics of moisture content w_0, volume weight of the soil skeleton $1/\delta_1$ and the indicative criterion of the soil M_p, w_f, and $N_{pf} = R_{pf}/R_f$ (or coefficient $1/r$) without conducting penetration tests.

First, the dependence (105) is presented in the form

$$M_{к01} = \frac{(\lg R_0 - \lg R_f) - (\lg R_0 - \lg R_{01})}{\lg R_p - \lg R_f} = M_{к0} - \frac{\lg R_0 - \lg R_{01}}{\lg R_p - \lg R_f}. \qquad (106)$$

In order to determine the difference in logarithms $\lg R_0 - \lg R_{01}$ it is necessary to make use of the design diagram for the case δ_1 = const., presented in Fig. 55 (cases 1, 3 and 5).

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[1]Under conditions of complete water saturation of the soil at moisture contents w_0, w_1 w_2 etc., the coefficient and the index of the consistency are designated by $M_{к0}$, $M_{к1}$, $M_{к2}$ etc. or $B_{к0}$ $B_{к1}$, $B_{к2}$ etc. Under conditions of a three-phase state the first even index determines moisture content w_0, w_2, w_4 etc., the second off index - the volume weight of the soil skeleton $1/\delta_1$, $1/\delta_3$, $1/\delta_5$ etc. For example, $M_{к01}$, $M_{к23}$ or $B_{к01}$, $B_{к23}$ etc.

From the similarity of triangles bca and 123 it is easy to obtain

$$\frac{\lg R_0 - \lg R_{01}}{\lg R_p - \lg R_{pf}} = \frac{w_1 - w_0}{w_f - w_p} = M_{\text{к}0} - M_{\text{к}1} \tag{107}$$

or

$$\lg R_0 - \lg R_{01} = (M_{\text{к}0} - M_{\text{к}1})(\lg R_p - \lg R_{pf}). \tag{108}$$

The coefficients of consistency $M_{\text{к}0}$ and $M_{\text{к}1}$ are calculated from formula (99) for the appropriate moisture contents w_0 and w_1.

After the substitution of dependence (108) in dependence (106) and after small conversions we finally obtain

$$M_{\text{к}01} = M_{\text{к}0} - (M_{\text{к}0} - M_{\text{к}1})(1 - M_{\text{к}pf}) \tag{109}$$

or

$$M_{\text{к}01} = M_{\text{к}1} + (M_{\text{к}0} - M_{\text{к}1}) M_{\text{к}pf}, \tag{110}$$

where

$$M_{\text{к}pf} = \frac{\lg R_{pf}/R_f}{\lg R_p/R_f}. \tag{111}$$

By substituting expressions $M_{\text{к}0} = 1 - B_{\text{к}0}$ and $M_{\text{к}1} = 1 - B_{\text{к}1}$ in dependence (109) it is easy to obtain the formula for determining the generalized index of consistency of three-phase soils with disturbed structure $B_{\text{к}01}$ with moisture content w_0 and a volume weight of the soil skeleton δ_1.

$$B_{\text{к}01} = B_{\text{к}0} + (B_{\text{к}1} - B_{\text{к}0}) B_{\text{к}pf}, \tag{112}$$

where $B_{\text{к}1} > B_{\text{к}0}$ and $B_{\text{к}pf} < 1$.

The determination of the consistency of a three-phase soil according to formulas (98) and (99) leads to erroneous results. Under conditions of a three-phase soil the generalized consistency of the soil, i.e., the actual resistivity of the soil to external forces, will always be less favorable. This feature of a three-phase soil is also revealed by dependences (109) and (112).

Let us complete further conversions of dependence (109). For this purpose let us express the coefficients of consistency $M_{к01}$ and $M_{кpf}$ in formulas (105) and (111) through ratios of the corresponding logarithms of the relative resistivity to penetration. After certain conversions we will obtain

$$\lg R_{01}/R_f = M_{к0} \lg R_{pf}/R_f + M_{к1} \lg R_p/R_{pf}. \tag{113}$$

Equation (113) establishes the functionsal relationship or correlation between the physical and mechanical properties of cohesive soils in the three-phase state.

Under conditions of complete water saturation of the soil when $R_0 = R_{01}$ and $M_{к0} = M_{к1}$ equation (113) is simplified to :

$$\lg R_0/R_f = M_{к0} \lg R_p/R_f. \tag{114}$$

From equation (113) it is possible to see the absolute inadmissibility, and unfortunately, even more so the applied comparison of the coefficient or index of consistency in the state of complete water saturation of the soil $M_{к0}$ or $B_{к0}$ with any characteristics of the mechanical properties of this soil in the three-phase state R_{01}, $\tau_{пред}$, $P_{пред}$ and etc. Such comparisons are completely incompetent, and the sometimes obtained "regularities," as a rule, do not have a cognitive value.

The indicative criteria of cohesive soils. The indicative criteria of cohesive soils are called any convenient characteristics, necessary and sufficient for the complete indication (identification) of the studied soil for the purpose of establishing a correlation between its physical and mechanical properties.

It follows from equation (78), for the determination of the correlation between the physicomechanical properties of cohesive soils under conditions of complete water saturation it is necessary to determine preliminarily two boundary conditions; for example, two parameters of equation (78): w and γ_w/r_0 (let us recall that the value R can be accepted equal to unity). Thus, for research on water-saturated soils it suffices to assign two indicative criteria of cohesive soils.

Under conditions of the three-phase state in accordance with equations (84) it is necessary to establish preliminarily not two, but three boundary conditions; for example, the parameters of this equation w, γ_w/r_0 and L_0. Consequently, in general, in the research on the variability of the mechanical properties of three-phase soils it is necessary to disclose preliminarily three indicative criteria of cohesive soils.

Depending on the assigned task and the volume of the experimental research the following two versions are especially recommended as indicative criteria.

The first version of the indicative criteria is calculated for the use of a grapho-analytical working method on the test results. After constructing the graphs of the correlation of the physicomechanical properties of the soils according to the diagrams, presented in Figs. 47, 48 or 55, first of all two angular coefficients in any convenient combination, are selected according to the conditions of the construction of the graphs:

or γ_w/r_0 and γ_w/r;
or, γ_w/r_0 and $1/t$;
or, sometimes, γ_w/r and $1/t$.

The first two indicative criteria of the soil characterize the intensity of the change of the mechanical properties under conditions $G_0 = 1$ const. (coefficient γ_w/r_0) and w_0 = const. (coefficient γ_w/r). The indicative criterion $1/t$ determines the intensity of the change of the mechanical properties of soils under conditions $1/\delta_1$ = const. (let us recall that $r_0/\gamma_w = r/\gamma_w + t$).

All three angular coefficients determine a relative change in the characteristics of the mechanical properties of cohesive soils.

The official role of the third indicative criterion of cohesive soil includes absolute coordinates in the transition. As a beginning of the analysis one of the following combinations can be accepted.

For any known physical condition of the soil, assigned in the form, for example, moisture content w_0 and the degree of water saturation G_{01}, resistivity to penetration R_{01} is determined.

Or, on the contrary, any convenient value of the resistivity to penetration R_{01} can be assigned and the corresponding physical characteristics of soil w_0 and G_{01} can be determined.

In the practice of the use of the proposed design diagrams, the most convenient case turned out to be that when the value of the resistivity to penetration $R_{01} = R_0 = 1$ kg/cm^2 is assigned and the moisture content of the soil w_0 with the degree of the water saturation of soil, equal to unity, $G_{01} = 1$ is determined. Such a selection of the indicative characteristics turns out to be especially convenient during the determination of the correlation of the physicomechanical properties of cohesive soils under conditions of undisturbed structure.

The second version of the indicative criteria is more preferable when an analytical method of working out the results of the experiments; for example, when using a least squares method. In this case the coordinates of three characteristic points of the graph of dependence $R_{01} = f(M_p, w_f, N_{pf}, w_0, \delta_1, \gamma)$ can be accepted as an indicative criterion:

1) R_f, w_f and $G_0 = 1$ - resistivity to penetration R_f at the liquid limit of the water-saturated soil w_f;

2) R_p, w_p and $G_0 = 1$ - resistivity to penetration R_p at the plastic limit of the water-saturated soil w_p;

3) R_{pf}, w_p and δ_f - resistivity to penetration R_{pf} with moisture content at the plastic limit w_p and with the volume weight of the soil skeleton at the liquid limit δ_f, i.e., when the degree of water saturation $G_{pf} = w_p/w_f$.

Under conditions of a disturbed structure the resistivity to penetration at the liquid and plastic limits, R_f and R_p are assigned. Thus, in accordance with the traditional practice the following components of the coordinates of the characteristic points of the graph are accepted as indicative criterion of cohesive soils with disturbed structure: the plastic limit of clayey soils, i.e., moisture content at the liquid w_f and plastic w_p limits of consistencies.

These two indicative criteria with known values R_f and R_p are completely sufficient for determining the variability of the mechanical properties of soils under conditions of complete water saturation.

As a third indicative criterion the value of the resistivity to penetration R_{pf}, is utilized expressed in the form of the dimensionless coefficient of penetration - index of hydrophilic behavior $N_{pf} = R_{pf}/R_f$. In this case the coordinates of data points of the physical condition of the soil w_p and $1/\delta_f$ on the diagrams of Fig. 55 (point 3) turn out to be known already.

Generalized design diagram for the case of the constancy of the characteristics of the mechanical properties of soils. During engineering geological investigations in soils of one stratigraphic and genetic formation, the characteristic coefficient of the consistency of the soils $M_{\text{к}\ pf}$ can be taken as constant, $M_{\text{к}\ pf} = \text{const}$.

As a result, with the use of equation (110) at coordinates "coefficient of the consistency of the soil $M_{\text{к}}$ - index of the consolidation of the soil k_d," the generalized nomograph-data sheet of the soils with disturbed and undisturbed structure can be constructed (Fig. 56), which characterizes the constancy of the generalized coefficient of consistency $M_{\text{к}01}$ and, consequently, also the constancy of the mechanical properties of soils during a simultaneous change in the coefficient of consistency $M_{\text{к}}$ and the index of consolidation k_d.

The index of consolidation k_d, proposed by Prof. V. A. Priklonskiy, counted equal to the coefficient of the consistency $M_{\text{к}1}$ with the moisture content which corresponds to the complete moisture capacity of the soil:

$$k_{\text{д}1} = M_{\text{к}1} = \frac{\varepsilon_f - \varepsilon_1}{\varepsilon_f - \varepsilon_p} = \frac{w_f - w_1}{w_f - w_p}. \qquad (115)$$

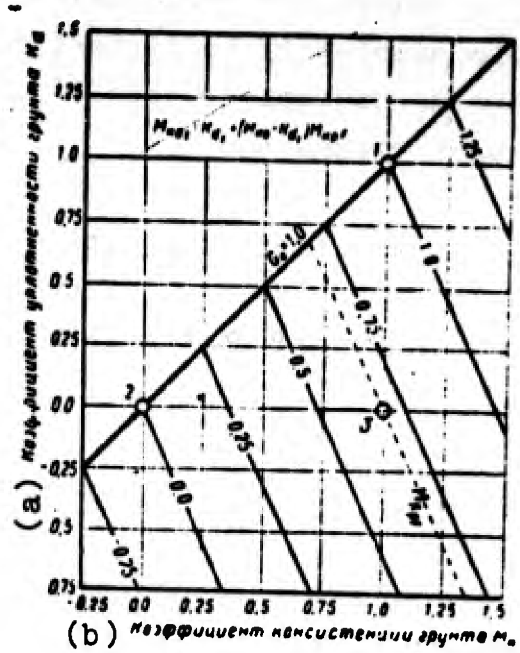


Fig. 56. Generalized design diagram at relative coordinates $M_{к} - k_{d}$ for the case of the constancy of the consistency and characteristics of the mechanical properties of soils. KEY: (1) Coefficient of consolidation of the soil k_{d}. (2) The coefficient of the consistency of the soil $M_{к}$.

The chief characteristic of the generalized design diagram of the correlation of the physicomechanical properties of the soils lies in the fact that in this case the characteristics of the mechanical properties turn out to be independent of the degree of dispersion of the soil which determines the values of the moisture content at the plastic and liquid limits.

Literature on the logarithmic dependence between the physicomechanical properties of three-phase soils. The design diagrams of the correlation of the physicomechanical properties of cohesive soils were published by the author in 1963 [38]. The publication preceded the survey of literature sources for the purpose of determining the materials which make it possible to confirm or to refute the disclosed regularities. These investigations, supplemented by the sources of 1964-1966, make it possible to note the following.

The proposed design diagrams of the correlation of the physical and mechanical properties of cohesive soils with a high degree of accuracy coincide with the results of various experimental research in soils with disturbed structure based on the determination of the characteristics of strength by the methods of unconfined compression, indentation of a punch, and penetration.

Thus, for instance, in the materials of the American Society of Civil Engineers in 1954 the results of experiments were published on unconfined compression of 46 samples, moulded from clay with indicative characteristics $M_p = 18$, $w_f = 39\%$, $\gamma = 2.72$ g/cm^3 [69].

The results of these tests were presented by J. Mitchell at a semilogarithmic scale at coordinates $\varepsilon - \lg p_{пред}$ and $w - \lg p_{пред}$. The graphoanalytical working method of the results of the experiments in diagrams w_0 = const, ε_1 = const and G_{01} = const revealed a very close spacing of the experimental points around the averaging straight lines with a distinct agreement of the calculated and average experimental values of the moisture content for conditions $G_0 = 1$ [38]. It is interesting that J. Mitchell himself mentioned very considerable scattering of experimental points and the absence of a clear correlation between the initial void ratio of the tested samples ε and the ultimate pressure during unconfined compression $p_{пред}$ [69].

As is known, in the USA, Canada, England and other countries the method of an empirical estimation of the mechanical properties of soils in a conditional state has wide application according to the results of laboratory tests of samples using a punch. As the index of the bearing strength of the soil (California Bearing Ratio - CBR) the mean pressure put to a punch 50 mm in diameter, at which after 2 min the relative settling of the punch $\lambda = 0.05$ or absolute settling $h = 0.25$ cm, is reached. The test data - CBR - is expressed in percentages from the standard bearing strength of crushed stone accepted according to this method as equal to $p_{ст} = 70$ kg/cm^2.

Between the coefficient of penetration and the CBR there is proportional dependence within certain limits. Thus with constant values of the volume weight of the soil skeleton δ_1 = const, δ_3 = const, etc. between the moisture content of the soil and the logarithm of the CBR a clear linear dependence should be exhibited.

The checking of this position was realized according to results of 88 determinations of the CBR, made on samples of London clay by the English Highway Research Laboratory [31].

The distinctive feature of these tests is the sufficiently large range of change in the moisture content - from 10 to 30%, the volume weight of the soil skeleton - from 1.44 to 1.68 g/cm^3, which led to changes of the CBR from 4 to 97.3%, i.e., by 24 times.

The processing of the results of the experiments at coordinates w - lg CBR also led to a sufficiently high convergence of the results of the calculation and the experimented data [38]. It turned out, for example, that the index of the hydrophilic behavior of the investigated differences in the London clay amounted to only N_{pf} = 4.85, i.e., the test samples belong to a group of slightly hydrophilic soils.

It is worthy to mention that in the journal "Geotechnique" No. 1 for the year 1961 (W. Black [59] with reference to E. Davis, Geotechnique, Vol. 1, No. 4, 1949) presents at a semi-logarithmic scale, without a designation of the experimental points, the graphic representation of the dependence w = f(lg CBR) with δ = const, completely agreeing with the results of the processings of these experiments according to the design diagram for the case δ_1 = const (case 1 in Fig. 55). Later on W. Black

[60] published at a semilogarithmic scale the graphs of the dependence $w = f(\lg CBR)$ with $\delta = const$ for 6 different cohesive soils. Thus, it is possible to state that in the published works of the foreign specialists separate fragments of the proposed design diagrams are encountered.

The proposed design diagrams of the correlation of the physical and mechanical properties of cohesive soils turn out to be especially convenient and effective for the normalization of the calculated resistances of earthen foundations.

As an illustration of the possibilities for the proposed dependences, let us show that the table of the arbitrary resistances of cohesive nonconsolidated soils as the foundation of bridges and ducts, recommended according to the technical specifications of design [TUP] of the [construction norms] SN 200-62 [48], can be replaced by a simple calculated formula according to equation (113). In Table 75 TUP SN 200-62 the values of the arbitrary resistances of earthen foundations σ, as is known, are presented for sandy loams, clay loams and clays depending on the index of consistencies B_{κ} (or on the coefficient of consistency M_{κ}) at an interval of 0.4-1 and a void ratio of the soil ε at an interval of 0.5-1.1.

For purposes of simplification in the normalization in the first approximation, it is possible to consider that the indicative characteristics σ_p, σ_{pf} and σ_f are constants. Then, in accordance with equation (113) the values of the arbitrary resistances of the earthen foundations should satisfy the following requirements:

a) the coefficient of the first term of equation (113) $\lg \sigma_{pf}/\sigma_f$ is constant. Consequently, the family of graphs $\lg \sigma_{01} = f(M_{\kappa 0})$ when $\varepsilon_1 = const$, $\varepsilon_3 = const$, etc. (that is, $M_{\kappa 1} = const$ $M_{\kappa 3} = const$ and etc.) at coordinates $M_{\kappa} - \lg \sigma$ should represent a family of parallel lines.

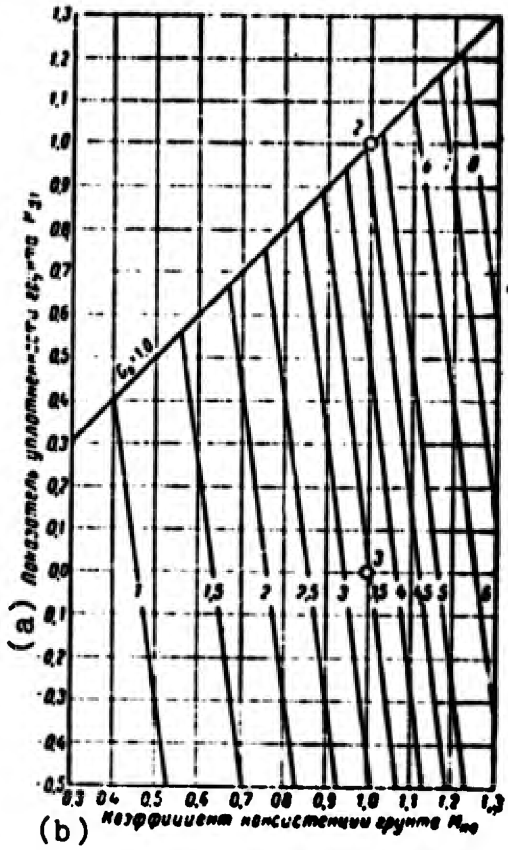


Fig. 57. Generalized nomograph for determining the values of the arbitrary resistances of earthern foundations according to the coefficients of the consistency and consolidation of the soil (according to TUP SN 200-62).
KEY: (1) Index of consolidation of the soil k_{dl}. (2) Coefficient of the consistency of the soil $M_{\kappa 0}$.

The values of the arbitrary resistances σ, presented in the Table of the TUP completely satisfy this requirement;

b) with an increase in the void ratio ε_1 the coefficient of consistency $M_{\kappa 1}$ decreases [see formula (115)], which leads to a decrease in the value of the arbitrary resistance σ_{01}.

It is possible to show also that this position is completely satisfied by the data from the standard table of arbitrary resistances.

However, the accepted structure of the construction of the norms of the design of natural foundations, namely, depending on the index of the consistency and the void ratio, cannot be considered as sufficiently successful. The determination of the arbitrary resistance of the foundation σ is considerably simplified, if we resort to the relative characteristics of the index of consolidation k_{dl}. According to the extensive studies of P. O. Boychenko [6] let us assume that between the plasticity index M_p and the liquid limit w_f, there is a linear dependence of the form [39, 41]

$$M_p = 0{,}586\,(w_f - 9{,}67)\ \%. \tag{116}$$

Then according accepted to the values of the plasticity index M_p in the table of the TUP one can easily calculate the corresponding values w_f, and then from the known values of the void ratios ε_1, determine the values of the coefficients of consolidation $k_{dl} = M_{\kappa 1}$.

The results of the made calculations allowed the use of the diagram of the correlation of the physicomechanical properties of the soil at relative coordinates $M_{\kappa 0} - k_{dl}$ according to type, presented in Fig. 56.

After the plottings were made the generalized equation of the correlation of the form was established:

$$\lg \sigma_{\mathrm{el}} = -0{,}444 + 0{,}980\,M_{\kappa 0} + 0{,}128\,k_{dl}. \tag{117}$$

Equation (117) determines the entire totality of the graphs of equal consistencies, or, in other words, the graphs of equal characteristics of the mechanical properties of soils presented in Fig. 57.

Thus, instead of the table of arbitrary resistances of cohesive soils, the more convenient and more demonstrative nomograph of the variability of arbitrary resistances σ, depending on the change in the coefficients of consistency $M_{\kappa 0}$ and coefficients of consistency $M_{\kappa 0}$ and consolidation k_{di} (Fig. 57) can be used. The values of the arbitrary resistances of the foundations are designated in the nomograph 1; 1.5; 2 kg/cm^2, etc.

The general idea of the degree of convergence of 47 tabular values of the arbitrary resistance of sandy loams, clay loams and clays (when M_p = 5-20) with the results of the calculation according to equation (117) can be obtained from the following data. In 87% of the cases the disagreement between the tabular and calculated values of the arbitrary resistances of the soils does not exceed ± 0.25 kg/cm^2. The reason for the disagreement is obvious and connected with the poor understating of the tabular values σ, with the coefficients of consistency 0.4-0.6 and in a number of cases, with $M_{\kappa 0}$ = 0.9-1 in comparison with the general law governing a change in the mechanical properties of the soils.

As a whole it may be concluded that the proposed design diagram, in particular, equation (117), completely successfully and quite accurately generalizes the totality of the tabular values of the arbitrary resistances of clayey soils accepted in TUP SN 200-62, and therefore it can be recommended for practical use when designing the foundations of railroad constructions.

The proposed design diagram makes it possible to restrict considerably the volume of prospecting works and improve the procedure for determining the calculated resistances of soils according to the results of engineering geological investigations.

CHAPTER V

INVESTIGATIONS OF THE PHYSICOMECHANICAL PROPERTIES AND INDICATIVE CRITERIA OF SOILS BY THE METHODS OF PENETRATION AND SOUNDING

The procedure for the penetration determination of the generalized coefficient of the consistency of cohesive soils. For the practical use of dependence (105) for the purpose of the penetration determination of the generalized coefficient of the consistency of cohesive soils $M_{\kappa 01}$, it is necessary to establish in advance the boundary conditions, namely the resistivity to penetration for the liquid limit R_r and for the plastic limit R_p.

Let us turn to the literature sources. The resistivity to penetration at the liquid limit of the soil R_r based on initial data of ΓOCT 5184-64 [ΓOCT = GOST = All-Union State Standard] for a bob cone with the angle of expansion 30° and a weight P = 0.076 kg is equal to $R_r = 0.076/1^2 = 0.076$ kg/cm^2.

The resistivity to penetration for a standard conical point at the plastic limit R_p according to the analyses of various authors virtually coincides. According to the values taken by P. O. Boychenko P = 0.3 kg and $h_p = 0.4$ cm, $R_p = 1.875$ kg/cm^2 [6, 7]; according to A. M. Vasiliev, when P = 0.076 kg and $h_p = 0.2$ cm, $R_p = 1.9$ kg/cm^2 [8]; according to I. M. Gor'kovye -

1.810 kg/cm^2 [18] and only according to F. I. Voronov when $P = 0.2$ kg and $h_p = 0.3$ cm it is somewhat higher: $R_p = 2.22$ kg/cm^2 [10].

By accepting the boundary conditions $R_f = 0.076$ kg/cm^2 and $R_p = 1.9$ kg/cm^2 in all subsequent calculations, we will have the coefficient of penetration $N_f = 1$ for the liquid limit and $N_p = R_p/R_f = 25$, or lg $N_p = 1.398$ for the plastic limit. For these boundary conditions the generalized coefficient of consistency of the cohesive soils with disturbed structure, computed according to equation (105), is equal to

$$M_{к01} = 0{,}715 \lg R_{01}/R_f, \qquad (118)$$

or for the resistivity to penetration R_{01}, expressed in kg/cm^2,

$$M_{к01} = 0{,}8 + 0{,}715 \lg R_{01}. \qquad (119)$$

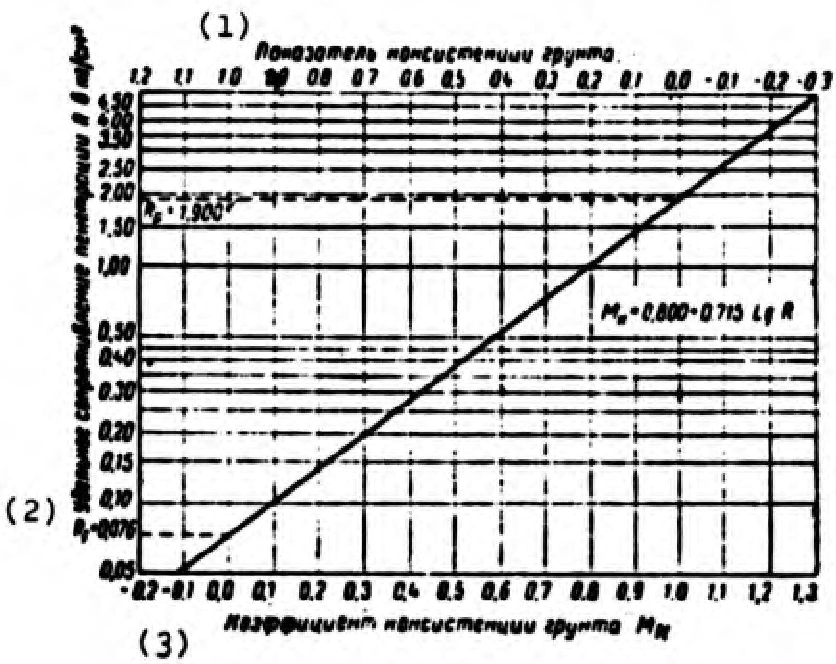


Fig. 58. Graph for determining the coefficient of consistency $M_н$ and the index of consistency $B_н$ of cohesive soils with disturbed structure according to the values of resistivity to penetration (in semilogarithmic scale) obtained from the experiment.
KEY: (1) Index of consistency of the soil; (2) Resistivity to penetration R in kg/cm^2; (3) Coefficient of consistency of the soil $M_н$.

The graph of equation (119) in semilogarithmic scale is presented in Fig. 58. For example when $R_{01} = 0.5$ kg/cm^2 $M_{\kappa 01} = 0.58$.

The determination of the consistency of cohesive soils according to the results of penetration can be conducted in accordance with the data in Table 9.

Table 9. The boundary values of the consistency and the characteristics of penetration tests of cohesive soils.

Consistency of cohesive soils	Coefficient of consistency M_κ	Index of consistency B_κ	Resistivity to penetration R in kg/cm^2	Coefficient of penetration $N = R/R_f$
Solid	>1	<0	>1.900	>25
Semisolid	0.75-1	0-0.25	0.850-1.900	11.18-25
Stiff plastic	0.50-0.75	0.25-0.5	0.380-0.850	5.0-11.18
Soft plastic	0.25-0.5	0.50-0.75	0.170-0.380	2.24-5
Liquid-plastic	0-0.25	0.75-1	0.076-0.170	1.0-2.24
Liquid	<0	>1	<0.076	<1

Let us note that the presence of the dependence (118) or (119) is identical to the condition under which all the water-saturated cohesive soils with disturbed structure and with an identical value of resistivity to penetration R_{01} have just one consistency $M_{\kappa 01}$ or $B_{\kappa 01}$.

For the first time this position was experimentally disclosed by P. O. Boychenko by means of numerous comparisons of values of M_κ for different clay soils with disturbed structure at the identical depths of indentation of a standard conical point of 0.3 kg in weight. As a result of such a comparison the obtained empirical dependence $M_\kappa = f(h)$ was utilized by P. O. Boychenko for the "direct" estimation of the consistency of soils. It turned out that the results of the determination of the coefficient of consistency $M_{к}$ in P. O. Boychenko's graphic representation coincide quite closely with the results of the calculation according to equation (119). (The difference does not exceed 0.06-0.07). The reason for the certain disagreement lies in the fact that almost all samples of soil utilized for plotting the empirical graph were tested under conditions of the three-phase state with the degree of water saturation from $G_{01} = 0.8-0.85$ to $G_{23} = 0.92-0.95$ [39].

Equations (118) or (119), and also the graph of dependence $M_{к} = f(\lg R)$, presented in Fig. 58, can be used for determining the "equivalent" consistency of cohesive soils with undisturbed structure.

The data in Table 9 on the change in the boundary values of resistivity to penetration R_{01} or of the coefficient of penetration $N_{01} = R_{01}/R_{f}$ are the basis for the composition of other analogous tables or graphs $M_{к} = f(Q)$ for the purpose of determining the generalized consistency of cohesive soils with disturbed and undisturbed structure according to the results of static and dynamic sounding.

In fact, in accordance with the initial dependences (12) and (34) let us recall that, in the first place, among the test data of the mechanical properties of cohesive soils by the methods of penetration, static and dynamic sounding using points of the correct geometric form, there is proportional dependence $R = k_{\alpha 1}q$; $R = k_{\alpha 2}\sigma$; $R = k_{\alpha 3}N$, etc., and, in the second place, the resistivity to penetration is equal to the resistivity of sounding $R = Q$, if the effect of the natural pressure is not too considerable.

Thus, during the determination of resistivity to sounding Q in the recalculation for a conical point with the angle of expansion of 30°, the values of the characteristics of the consistency of the soil $M_{к}$ and $B_{к}$ are directly determined from the graphic representation in Fig. 58 or by the data in Table 9.

According to the empirical dependence $R = 1.02\ N$, the proportionality factor between the resistivity to static penetration R and the index of dynamic sounding N for an arrangement with the weight of the hammer ~ 60 kg and with the height of fall ~ 90 cm is equal to $k_\alpha = 1.02$. Consequently, even in this case, the graph in Fig. 58 and the boundary conditions, reduced in Table 9, can be used directly for determining $M_к$ or $B_к$ for the results of dynamic sounding.

The substitution, for example, of the dependence $R = k_u \sigma$ in equation (119) does not change the value of the angular coefficient 0.715. Thus, the empirical graphic representations $M_{к01} = f(\lg \sigma_{01})$ published in technical literature should be parallel to the basic graph of the dependence (119), depicted in Fig. 58.

As an example let us examine Table 10 for the boundary values of the mean pressure on the tip of the conical point depending on the consistency of the soil, proposed by Yu. G. Trofimenkov [49].

Table 10. The boundary values of the indexes of penetration and sounding depending on the consistency of the cohesive soils.

Consistency of the cohesive soils	Boundary values $M_к$	Mean values of $M_к$	Resistivity to penetration R in kg/cm²	Coefficient of penetration m	Mean pressure in kg/cm² according to Yu. G. Trofimenkov			$\frac{\sigma_{cp}}{R}$
					σ_{min}	σ_{max}	σ_{cp}	
Solid	>1	—	>1.9	>25	—	—	>100	—
Semisolid	0,75—1	0,875	1.27	16,71	50	100	75	59
Stiff plastic	0,50—0,75	0,625	0,568	7,48	20	50	35	61,5
Soft plastic . . .	0,25—0,5	0,375	0,254	3.34	10	20	15	59
Liquid-plastic . . .	0—0,25	0,125	0,114	1,5	—	—	<10	—

It is not difficult to see that the ratio of the average values of σ_{cp} and the resistivity to penetration R was obtained approximately the same way, i.e., at a semilogarithmic scale the graphs of $M_к = f(\lg R)$ and $M_к = f(\lg \sigma)$ are actually parallel.

If the coefficient of penetration for the liquid-plastic consistency of the soil is accepted as unity, then the subsequent values of the coefficients of penetration increase by geometric progression: 1.0; 2.2; 5.0; 11.2 with a denominator of 2.2.

Thus, the corresponding values of the characteristics of penetration and sounding with an increase in every subsequent step of the coefficient of consistency, should be approximately doubled. This dependence, for example, also holds in the table, recommended by R. Pek, U. Khenson and T. Tornburn (subgrades and foundations. Gosstroyizdat, Moscow, 1958) for determining the consistency of cohesive soils based on the number of blows in the "standard test for penetration."

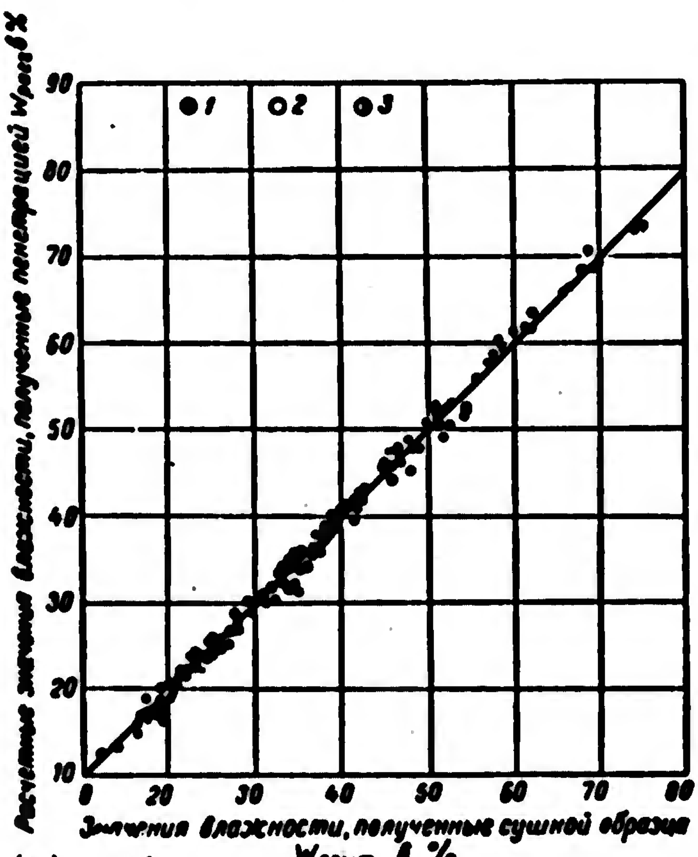
In the materials of the Ist All-Union Conference on Rock Mechanics (Alma-At, 1966) results are given of the investigations by Prof. M. M. Protod'yakonov on the rational classification of rocks. M. M. Protod'yakonov considers that it is expedient to construct the rational classification according to the law of geometric progression with a progression ratio equal to 1.5-1.6. Thus, it can be stated that the substantially different methods of investigating the mechanical properties of soils and rocks produce coinciding results.

The determination of the mosture content of water-saturated cohesive soils with disturbed structure. From the determination of the coefficient of consistency $M_{\kappa} = \frac{w_f - w}{M_p}$ it follows directly that the moisture content of a water-saturated soil w depends linearly on the value M_{κ}:

$$w = w_f - M_p M_{\kappa} \ \%. \quad (120)$$

Consequently, with the known plastic limits w_f and w_p the determination based on the results of the penetration of the coefficient of consistency M_{κ} for water-saturated cohesive soils with disturbed structure means a simultaneous determination of the mositure content.

The comparison of the results of more than 150 moisture content tests according to the proposed method with a control moisture content test by drying the samples according to GOST 5179-64 for 19 types of clay soils [6, 9, 18], which differ by degree of dispersion, plastic properties (from $M_p = 5$ to $M_p = 53$) and by mineralogical composition, are presented in Fig. 59. Along the axis of the abscissae are plotted the values of the moisture content obtained by drying the samples, along the axis of the ordinates - determined by penetration. It is not difficult to see that the spread of the values of the moisture content from the mean value does not exceed ±1.5% the moisture content by weight, but in 40% of the cases the corresponding values of the moisture content turned out to be identical.



(a) (b)

Fig. 59. Graph of a comparison of the values of the moisture content by weight, obtained by drying selected soil samples and determined according to the results of penetration tests 1 - according to experiments with a micropenetrometer; 2 - according to experiments based on P. A. Rehbinder's plastometer; 3 - according to experiments based on P. O. Boychenko's consistometer.
KEY: (a) Computed values of the moisture content obtained by penetration W_{pacr} in %; (b) Values of the moisture content obtained by drying the sample $W_{опыт}$ in %.

The high convergence of the calculated and experimental values of the moisture content is the consequence of a strict observance of the condition of complete water saturation of the soil. The breakdown of this condition would unavoidably entail more or less considerable errors in the determination of the moisture content of a soil from the results of penetration.

In general in determining the moisture content it is not compulsory to carry out the preliminary determination of the plastic limits of the soil being investigated. It suffices in advance to obtain the data on the values of the resistivity to penetration R_1 and R_2 at two moisture contents of the samples w_1 and w_2 ($w_1 > w_2$). Then, the calculation of any moisture content approximately in the range of the plastic consistency of the soil w_i is made based on the value of resistivity R_i according to equation (80) obtained from the experiments, whereupon the angular coefficient γ_α/r_0 is computed according to formula (81) with the use of characteristics w_1, R_1 and w_2, R_2.

The principles of the procedure of the penetration determination of the criteria of cohesive soils. All-Union State Standards No. 5183-64 and 5184-64 pertaining to the methods of the laboratory determination of the limits at which a soil sample can be rolled (plastic limit), w_p and the liquid limit w_f were introduced for first time into use at the beginning of 1950 and at that time they undoubtedly had progressive value. Abroad only 11 years later were the first data given in the reports of the Vth International Congress of the Mechanics of Soils and Foundations in favor of the transition to the penetration method of determining the liquid limit. Meanwhile, A. M. Vasiliev [8] developed and GOST proposed a very simple procedure for determining w_f with the aid of a "bob cone" which for several years after its introduction, found acknowledgement and replaced the previous method for determining the liquid limit by blending two halves of a soil sample in a cup [62].

However, at the present time it should be recognized that the procedure for determining the limit at which a soil sample can be rolled is morally antiquated and leads to an obvious and considerable overestimate of w_p in coarsely dispersed soils [24].[1]

The procedure for establishing the liquid limit has certain technological deficiencies [7].

As a result, the used methods cannot insure the required accuracy of the determinations of the plastic limits and therefore, do not facilitate meeting the specifications of economical solutions when designing the footings of artificial structures.

For the basic improvement of the existing methods of determining the plastic limits it is necessary:

1. To forego the traditional procedure for determining the limit at which a soil sample can be rolled. It is advantageous to accept P. O. Boychenko's proposition about the determination of the limit at which a soil sample can be rolled by the method of penetration [6, 7].

2. To reject the method of gradual selection of the specified consistency of the soil during the determination of

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[1]P. N. Yerofeyev [24] has noted the invariance w_p from the plasticity index M_p and the grain-size distribution in coarsely dispersed marine sediments when $M_p < 30-35$. A similar, completely inadmissible failure of the standard procedure, according to P. N. Yerofeyev and A. B. Shpikov, is the "consequence of the overestimate of the moisture content of the soil during the determination of the limit at which the soil sample can be rolled because of the water bonds which compensate for the deficiency of colloidal bonds in coarsely dispersed soils and which maintain constant cohesion in them" [24].

the liquid limit, at which a "bob cone" 0.076 kg in weight is driven in the soil to a depth of 1 cm. As the analysis showed considerable errors in the determination of the liquid limit [7] are connected with the procedure for selecting the assigned consistency.

Since the penetration method of determining the liquid limit has excellently withstood the test of time, it is necessary to only improve the technological methods of determining this characteristic.

3. To simplify the technique of preparing the soil for tests, in particular, to the preliminary removal of the sand fractions of the soil larger than 1 mm as undesirable. A similar technological method, which is a negative feature of the existing procedure, frequently leads to a distorted representation of the actual properties of the investigated soil.

4. To introduce into the new procedure for determining w_p and w_f the possibility of a control and automatic-control for the accuracy and reliability of the results of the determinations by means of required checking of the invariance of the resistivity to penetration.

5. To improve the design of laboratory penetrometers, utilized for determining the plastic limits. The accuracy of recording the depth of indentation of a conical point should be not less than 0.1 mm. In this connection one ought to forego the use of the "bob cone," which does not provide the necessary accuracy in determining the liquid limit.

6. To differentiate the new procedure for determining the plastic limits. Depending on the stage of the engineering geological investigations and classification of the engineering constructions, it is advantageous to utilize 2 or 3 particular

procedures for a single complex, which have different labor costs and which provide for the determination of the plastic limits with varied guaranteed accuracy. In this case, the most "simplified" procedure should result in the accuracy of the determination of w_p and w_f approximately at a level of the existing GOST.

From the aforementioned it follows that as the basis of the new procedure for the determination of plastic limits it is necessary to propose the method of penetration tests.

The determination of the plastic limits of cohesive soils by the method of two penetrations. On the basis of the generalization of the results of studies by A. M. Vasiliev [8, 9], P. O. Boychenko [6, 7], I. M. Gor'kova [18], M. V. Ivanova and G. G. Saatchana, F. I. Voronov [10] and other authors, taking into account the foregoing features of the method of penetration tests and the proposed design diagrams of the interrelationship of the physicomechanical properties of the soil, several versions of the procedure for the penetration determination of the plastic limits of cohesive soils can be recommended [36, 38, 39, 41].

The method of two penetrations is based on the following premises.

1. The plastic limits w_p and liquid limits w_f are determined from the results of penetration tests using two samples as the minimum with any two values of moisture content w_0 and w_2, which fall approximately within the range of the plastic limit.

2. The samples are tested under conditions, close to complete water saturation, $G_{01} \geq 0.96$.

3. The resistivities to penetration equal to $R_p = 1.9$ kg/cm^2 and $R_f = 0.076$ kg/cm^2 correspond to the values of the moisture content at the plastic limits w_p and liquid limits w_f for the consistencies of the soil.

4. Under conditions of complete water saturation of the soil there is a strict linear dependence (80) between the moisture content and the logarithm of the relative resistivity to penetration.

Thus, at a semilogarithmic scale the corresponding graphic representations are depicted as straight lines (Fig. 48, 55).

At the assigned boundary conditions R_p and R_f the angular coefficient γ_w/r_0 in equations (78) and (80), according to formula (81a), obtains the form

$$\gamma_w/r_0 = 0{,}715\, M_p\ \% \tag{121}$$

or

$$M_p = 1{,}398\, \gamma_w/r_0\ \%. \tag{122}$$

Consequently, in water-saturated cohesive soils with disturbed structure the plasticity index M_p is proportional to the slope of the straight line (80) tangent to the vertical axis, and therefore it is a measure of the intensity of the change in the mechanical properties of soils under conditions of complete water saturation. In this connection the reason for the remarkable constancy of the slope angle of the graphs $w = f(\lg P_m)$ when $G_0 = 1$, plotted by F. D. Ovcharenko (Fig. 51) becomes clear. In these studies the plasticity index of various clays were approximately identical.

Taking into account the foregoing, the plastic limits are determined graphically in the following manner. According to the experimental points at coordinates $w - \lg R$, a straight line results (Fig. 48). The intersection of the plotted straight line with two horizontals $R_p = 1.9$ kg/cm^2 and $R_f = 0.076$ kg/cm^2 determines points 1 and 2, whose abscissae correspond to the plastic and liquid limits.

Analytically, the plasticity index M_p and the liquid limit w_f are calculated from formulas [36, 41]:

$$M_p = 1.398 \frac{w_0 - w_2}{\lg R_2/R_0} \tag{123}$$

and

$$w_f = \frac{k w_0 - w_2}{k - 1}\ \%, \tag{124}$$

where

$$k = M_{\kappa 2}/M_{\kappa 1}. \tag{125}$$

If $w_2 < w_0$, then $R_2 > R_0$ and $M_{\kappa 2} > M_{\kappa 0}$. Formulas (123) and (124) are derived by solving two equations of the form (80) taking into account the obtained experimental data on w_0, R_0 and w_2, R_2.

Given as an example in Fig. 48 are the results of penetration tests by V. G. Zabara in determining the plastic limits of a heavy clay loam, from the area of B. Lepetikha, Kherson oblast.

According to the graphic plotting, shown in Fig. 48, the plastic limit proved to be equal to $w_p = 20.3\%$ and $w_f = 36\%$.

The correctness of the determinations for the indicative criteria of the soils is checked:

a) by testing a third sample;

b) by determining the volume weight of samples Δ_1 as a check on the degree of water saturation of the soil:

$$w_1 = \gamma_w [1/\Delta_1 (1 + w_0) - 1/\gamma]; \tag{126}$$

c) by determining the invariance of the resistivity to penetration.

In 1962-1965 a number of organizations had undertaken research on testing the penetration method for determining the plastic limits. As one would expect, the application of the method of two penetrations gave sufficiently satisfactory results. For example, according to A. B. Shpikova, when testing a large number of samples of saline soils virtually full agreement of the liquid limits was obtained using the methods of two penetrations and GOST 5184-64. In this regard it was noted that during the parallel determinations of the plastic limits from the method of two penetrations and the limits at which a soil sample can be rolled, according to GOST 5183-64, in 98% of more than 100 cases of conducted comparisons, a correlation of the plastic limits within the limits of ±1.5% moisture content by weight was observed.

Such a high degree of convergence of the results, of course, cannot be the general rule. One should consider the fact that in certain cases, the plastic limits, determined by the method of penetration and by rolling the soil sample in threads, will be substantially different.

The standard methods of determining the plastic limits according to GOST 5183-64 and 5184-64, as known, allow for deviations in the values of the plasticity index up to 4% moisture content by weight. The proposed method of two penetrations provides for increased accuracy of the determinations. It is possible to consider that the disagreement in the results of the determinations of more than ±1-1.5% moisture content by weight testifies to the defects in the required technology of the working procedure.

For the possible wider application of the penetration method of determining the plastic limits under field conditions and for solving a number of practical problems which do not require increased accuracy in determining the plastic limits or problems associated with the investigation of soils of a specific genetic type, where the values of the plastic limits are closely connected with the percentage of the clay fraction, it is possible to introduce certain simplifications into the method of two penetrations.

In order to do this it is necessary to establish preliminarily numerical values for the coefficients $M_{\kappa a}$ and w_a in the empirical equation of the linear dependence between the plastic limits of cohesive soils:

$$M_p = \frac{1}{M_{\kappa a}}(w_f - w_a)\ \%. \qquad (127)$$

After the substitution of dependence (127) in equation (120) and after solving it relative to w_f the basic dependence is obtained for determining the liquid limit from the results of penetration tests at one moisture content of a water-saturated soil with disturbed structure (method of one penetration) [39, 41]:

$$w_f = \frac{w_0 - w_a/M_{\kappa a} M_{\kappa 0}}{1 - 1/M_{\kappa a} M_{\kappa 0}}\ \%. \qquad (128)$$

In the first approximation it is possible to accept $1/M_{\kappa a} = 0.586$, $w_a = 9.67\%$ and $w_a/M_{\kappa a} = 5.67$. The determination of w_f from the known moisture content w_0 and from the coefficient of consistency $M_{\kappa 0} = f(\lg R_0)$, can be conveniently derived with the aid of a radiant nomograph [39, 41]. After establishing w_f through formula (99) the plastic index of the soil is found.

The results of the comparison of the values of the plastic limits determined by using methods of two and of one penetration (with averaged coefficients $M_{\kappa a}$ and w_a) revealed the possibility for an error of up to 6% in determining w_{Γ}, moisture content by weight (specifically, for loess deposits and Khvalynskiy clays). Thus, the use of the indicated averaged coefficients for various soil conditions in no way can be recommended.

Investigation of the interrelationship of the physicomechanical properties of cohesive soils with disturbed structure under conditions of a three-phase state. The high accuracy of penetration tests makes it possible to establish comparatively easily the interrelationships between the physical and mechanical properties of cohesive soils. Under conditions of a disturbed structure investigated during the construction of embankments and dams of water-engineering projects and of highway and railroad projects, the determination of such an interrelationship even at this stage of the analysis already represents a comparatively simple task. In this case, following from the design diagrams, presented in Fig. 55, the indicative criterion of cohesive soils with disturbed structure are simultaneously disclosed: plasticity index M_p, liquid limit w_{Γ} and the index of hydrophilic behavior $N_{p\Gamma}$.

Given as an example in Fig. 60 at coordinates $1/\delta$ - lg R are the results of penetration tests of 13 samples of a clay loam from the area of Sadovoye settlement in the Crimea, made by V. G. Zabara. The obtained values of resistivity to penetration represent averages from two tests on the butt ends and along the face of the cutting ring. The check on the invariance of resistivity to penetration $R_{\text{тор}}$ and $R_{\text{рез}}$ was made during 6-8 stages of load as the minimum.

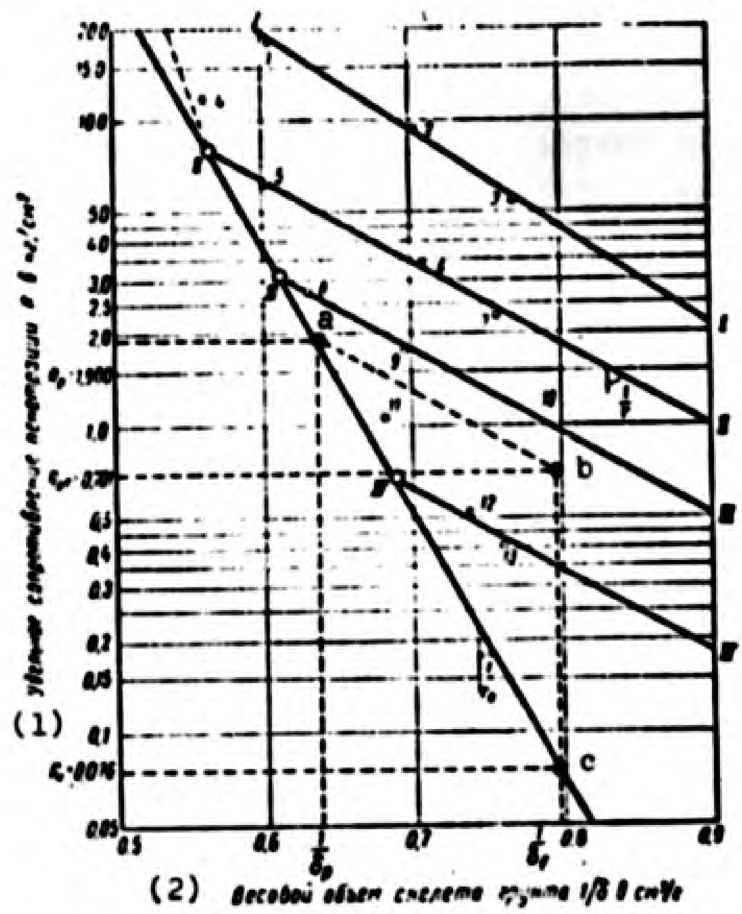


Fig. 60. The parallel graphic representations of the linear dependence between the volume of a loess-like clay loam skeleton, by weight with disturbed structure and the logarithm of the resistivity to penetration for a case of constant moisture content, w_0 = const, w_2 = const, etc. (at a semilogarithmic scale). a, b, and c - Points which characterize the indicative criteria of the loess-like clay loam: $1/\delta_p = 0.636$ cm^2/g; $1/\delta_r = 0.796$ cm^3/g; $R_{pr} = 0.701$ kg/cm^2 or $N_{pr} = 9.22$.

KEY: (1) Resistivity to penetration, R in kg/cm^2; (2) Volume of the soil skeleton, by weight, $1/\delta$ in cm^3/g.

The test evaluation was made by a graphic-analytical method in the following sequence.

1. The test data were plotted in the field of the graph using coordinates volume of the soil skeleton by weight - logarithm of resistivity to penetration. (For the convenience of constructing the graphs a semilogarithmic coordinate system is used). The location of each experimental point was determined by two coordinates $1/\delta_1$, lg R_{01}, $1/\delta_3$, lg R_{23}, etc. The number of each tested sample was written at the experimental point. The values of the moisture contents of the corresponding samples w_0, w_2, w_4, etc., are given in Table 11.

Table 11. Values of the moisture contents and minimum volumes by weight of the soil skeleton of the test samples.

(1) № опорных прямых	(2) № образцов	w_0 в %	$w_{ср}$ в %	$1/\delta_{мин}$ в см³/г
I	1 2 3	14,7 14,7 14,7	14,7	0,517
—	4	18,4	18,4	0,554
II	5 6 7	19,4 19,6 19,4	19,5	0,565
III	8 9 10	23,4 24,3 24,2	24	0,61
—	11	30,6	30,6	0,676
IV	12 13	31,6 32,0	31,8	0,686

KEY: (1) Reference lines; (2) Samples.
Designation: cm3/r = cm3/g.
[в = in]

2. The plotting of the graphs of dependence (92) was made for the case of constant moisture content of the soil w_0 = const. Thus, through groups of experimental points with close values of moisture content by weight, 4 average straight lines are presented in Fig. 60.

The closer the experimental points are grouped around the average line, consequently, the more accurate are the results of the penetration tests and determinations of the physical properties of the test samples, and less is the effect of the extraneous factors on the investigated dependence.

3. Four reference lines, plotted for average moisture contents 14.7; 19.5; 24 and 31.8%, according to the design diagram in Fig. 55, should be mutually parallel. The parallelism of the average straight lines is one of the checks on the correctness of the construction of the nomograph.

4. The convergence of the experimental results and the correlation of the plotted straight lines is determined according to the closeness of the precalculated positions I, II, III and IV with minimum values of the volume by weight of the soil skeleton $1/\delta_{\text{макс}}$ about the average straight line, plotted for the case of complete water saturation of the soil, $G_0 = 1$.

The minimum values of the volume by weight of the soil skeleton are calculated for the average values of the moisture content according to formula (676) with $1/\gamma = 0.37$ cm^3/g and are presented in Table 11.

As indicative criteria of the graphs of dependence (92) the values of the angular coefficients $1/r_0 = 0.1145$ cm^3/g and $1/r = 0.369$ cm^3/g and a volume by weight of the soil skeleton $1/\delta_0 = 0.668$ cm^3/g when $R_0 = 1$ kg/cm^2.

For cohesive soils with disturbed structure it is advantageous to simultaneously determine the plastic limits $1/\delta_p = 0.636$ cm^3/g and $1/\delta_f = 0.796$ cm^3/g (plasticity index $M_p = 16$) and the index of hydrophilic behavior $N_{pf} = 9.22$. The latter is found through the value of resistivity to penetration of point b $R_{pf} = 0.701$ kg/cm^2. To plot it from point a, a broken straight line is drawn, parallel to the reference line, and from point c it is restored to the perpendicular.

The values of the plastic limits and the index of hydrophilic behavior N_{pf} can be determined by calculation without graphics. For example, at first according to formula (90) $1 - M_{\text{н}_{pf}} = 0.31$ is established. Further, on the basis of dependence (118) $\lg N_{pf} = 0.965$ is computed, whence $N_{pf} = 9.22$.

The high accuracy in plotting the nomograph of the dependence (92), presented in Fig. 60, was the consequence of the high quality of the prepared samples. Only the accuracy of the results of penetration tests of two water-saturated samples (4 and 11) falls off somewhat. From the results of the plotting it follows that under conditions of $G_0 = 1$ when $R_0 > 8$ kg/cm^2 the angular coefficient $1/r_0$ decreases noticeably (broken part of the graph).

A very interesting nomograph, $1/\delta_1 = f(\lg N_{01})$, when $w_0 = \text{const}$ was made by V. G. Khilobok during the examination of the results of penetration tests of 58 samples of a sandy loam with disturbed structure, from Vartemyaka village, Leningrad oblast (Fig. 61).

The tests were conducted with the moisture content of the samples at approximately 7 to 31% under conditions when the sandy loams were of solid ($N_{01} > 25$) as well as liquid ($N_{01} < 1$) consistencies. In this case the previously expressed position (see Chapter I) about the change in the form of failure of the soil during an especially considerable change in the consistency of the soil was completely confirmed. It is possible, for example, to assume that under conditions of complete water saturation of the soil with $N_0 > 76$, i.e., knowingly with solid consistency of the sandy loam, shearing of the soil will unavoidably be observed (in the conducted tests this phenomenon was not noted, since the maximum value of the coefficient of penetration amounted to $N_{\text{макс}} = 38$). Within the range of plastic consistency, i.e., in this case in 48 tests when $N_{01} = 0.65$-76, soil displacement was excellently exposed. And only when $N_{01} < 0.65$, i.e., definitely with a liquid consistency of the soil, was the phenomena of soil displacement along the forming cone revealed.

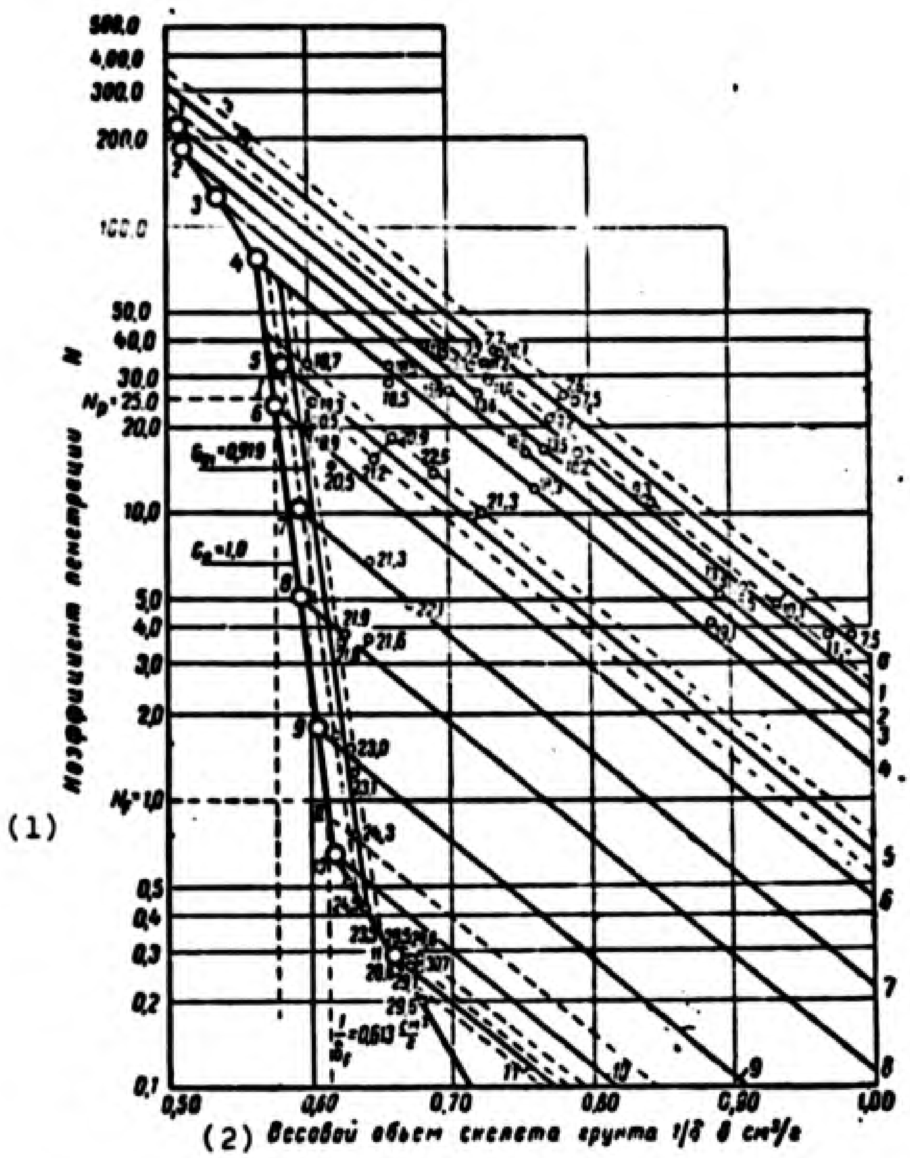


Fig. 61. Parallel graphic representations of the linear dependence between the volume by weight of a sandy loam skeleton with disturbed structure and the logarithm of the coefficient of penetration for the case of a constant moisture content w_0 = const, w_2 = const, etc., (at a semilogarithmic scale).
KEY: (1) Coefficient of penetration N; (2) Volume by weight of the soil skeleton 1/δ in cm^3/g.

Because of this the dependence $w_0 = f(\lg N_0)$ for the case $G_0 = 1$ produced a characteristic change in the angular coefficient which was doubled.

When testing a group of samples under conditions of a constant moisture content the slope angle of the parallel graphs $1/\delta_1 = f(\lg N_{01})$ was completely and uniquely determined according to a system of the first five straight lines, depicted in the nomograph of Fig. 61, and it was equal to $1/r = 0.247$ cm^3/g. The sequence of further plottings of the nomograph is analogous to the previsou example.

The plasticity index of the soil is easily found through the intersection of two horizontal lines with boundary conditions $N_f = 1$ and $N_p = 25$ with a straight line 5 – 7 – 9 – 10. We find that $1/\delta_f = 0.613$ cm^3/g and $1/\delta_p = 0.577$ cm^3/g, or $M_p = 3.6$.

For the determination of the index of hydrophilic behavior N_{pf} let us make use of formula (90), which we will rewrite in the form

$$r/\gamma_w = B_{\kappa pf} r_0/\gamma_w = (1 - M_{\kappa pf}) r_0/\gamma_w = (1{,}398 - \lg N_{pf}) 1/M_p. \tag{129}$$

Whence

$$\lg N_{pf} = 1{,}398 - r/\gamma_w M_p. \tag{130}$$

At the known value of the angular coefficient $\gamma_w/r = 0.247$, we will find that $\lg N_{pf} = 1.252$ or $N_{pf} = 17.86$.

Such a high value of the index of hydrophilic behavior is, apparently, the characteristic feature of sandy loams. Just as an insignificant value of the plasticity index determines the exceptional effect of the moisture content on the variability of the characteristics of the mechanical properties of water-saturated soils, so also in this case the high value of the index of hydrophilic behavior testifies to this feature of sandy loams under conditions of three-phase state.

Let us note that in these analyses the method of compacting the samples in order to produce the specified value of the volume by weight of the soil skeleton, did not influence the results of the penetration tests. The samples were compacted in a standard compaction device with the aid of a falling load and also compacted in a GGP-30 preliminary compacting device during static loading. In plotting the nomograph in Fig. 61, a distinction in the methods of compacting the samples was not disclosed.

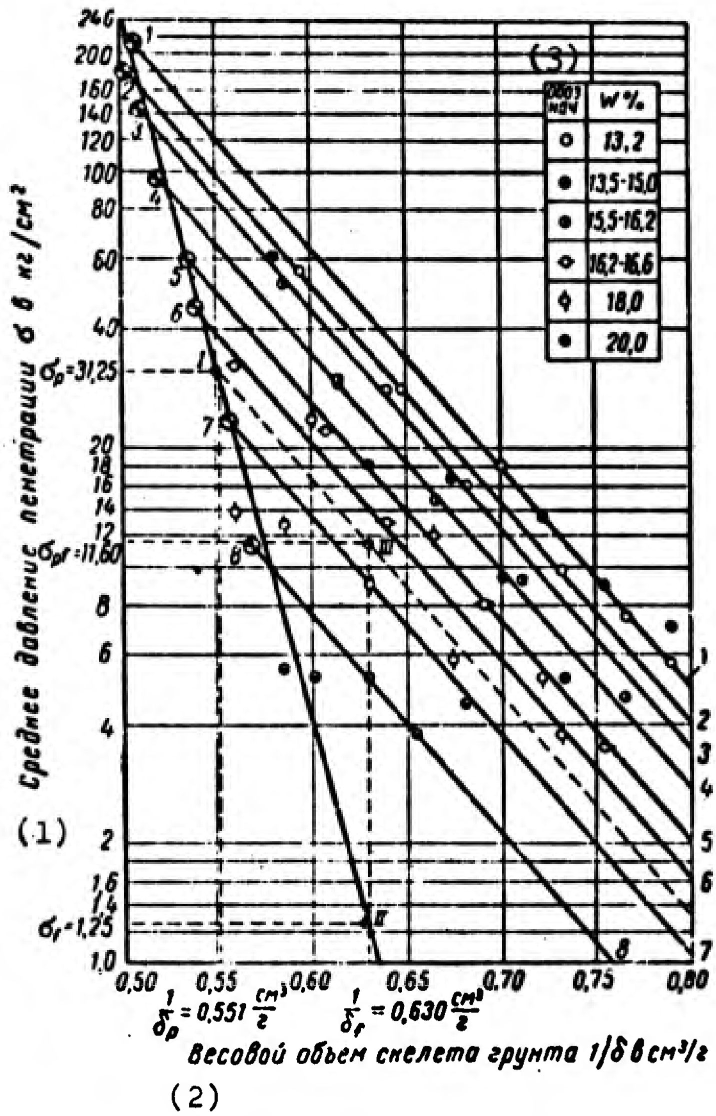


Fig. 62. Parallel graphic representations of the linear dependence between the volume by weight of the clay loam skeleton with disturbed structure and the logarithm of the mean pressure of penetration for the case of constant moisture content w_0 = const, w_2 = const, etc. (at a semi-logarithmic scale). I, II, and III - points which characterize the indicative criteria of the clay loam. KEY: (1) The mean pressure of penetration σ in kg/cm²; (2) Volume by weight of the soil skeleton $1/\delta$ in cm³/g; (3) Symbol.

Nomograph $1/\delta_1 = f(\lg \sigma_{01})$ when w_0 = const, presented in Fig. 62, is constructed according to the results of penetration tests, made by Ye. A. Yakovleva at TsNIIS Mintransstroya. The peculiarity of these tests lies in the fact that unlike all previous examples of penetration tests approximately 40 samples were tested consecutively by three indenters with areas of 0.5; 1 and 2 cm^2. The measured forces of penetration P were averaged on the basis of the principle of invariance, and namely it was assumed that $P_{F=1} = 2P_{F=0.5} = (1/2)P_{F=2}$. For example, in one of the tests it was found that $P_{F=0.5} = 6$; $P_{F=1} = 11$ and $P_{F=2} = 26$. Then, $\Sigma P = 2 \cdot 6 + 11 + (1/2)26 = 12 + 11 + 13 = 36$. The mean pressure on the base of the indenter was 1 cm^2 in area, thus, equal to:

$$\sigma = \frac{36}{3 \cdot 1.0} = 12 \text{ kg/cm}^2.$$

The results of penetration tests of a clay loam along 1589 km of the Chu-Badam railroad line ($M_p = 8$, $w_f = 26\%$, $\gamma = 2.7$ g/cm^3) were grouped by Ye. A. Yakovleva into six different moisture contents, given in the table in Fig. 62. After plotting all the obtained results, w_0, $1/\delta_1$, σ_{01},; w_2, $1/\delta_3$, σ_{23}, etc. in the field of the graph at coordinates $1/\delta - \lg \sigma$, it turned out that the best generalization is obtained by constructing eight parallel lines with average moisture contents of 13.6 (1st line); 13.2 (2nd line); 14.1; 15; 16.6; 16.9; 18.7 and 20% (8th line).

According to the results of the graphic construction it is evident that the precalculated positions 1, 2, 3, ..., 8 which determine the minimum values of the dry unit weight of the soil, are quite closely grouped around the limiting straight line I–II. As already mentioned, the observed feature is the objective criterion of the correctness and accuracy of the construction of the nomograph in Fig. 62.

The angular coefficients of the straight line 1-8 and the families of parallel lines according to the plotting are eaual to $1/r_0 = 0.0565\ cm^3/g$ and $1/r = 0.183\ cm^3/g$, respectively. Consequently, the plasticity index $M_p = 1.398 \cdot 5.65 = 7.9$. Then, in accordance with formula (130) $\lg N_{pf} = 1.398 - 0.079/0.183 = 0.9675$, or $N_{pf} = 9.28$.

Distinctly revealed in the example in question was the important law governing the proposed design diagram - the numerical values of the plasticity index M_p and the indexes of hydrophilic behavior N_{pf} can be directly determined from the results of penetration tests with the aid of any designs of penetrometers using points of various forms and dimensions. As already mentioned, the plasticity index and the index of hydrophilic behavior are the characteristics determining a relative change in the mechanical properties of the soils. Thus, the determination of these characteristics can also be conducted on the basis of research on a relative change in the results of penetration tests on cohesive soils.

In other words, the fundamental constancy of the angular coefficients of the design diagram γ_w/r_0, γ_w/r and $1/t$, that determine the indicative criteria M_p and N_{pf}, is the consequence of the presence of the proportional dependence between the results of penetration tests, made using different points and probes $R = k_{\alpha 1}q$, $R = k_{\alpha 2}n$, etc. (n - number of blows when testing the soil with a DORNII striker). Thus, in the denominator of formula (81) and in other analogous formulas the ratio of the corresponding characteristics of the results of penetration tests remains constant: $R_1/R = q_1/q = n_1/n$, etc.

There is another matter with regard to the determination of the liquid limit w_f. In order to establish it knowledge is necessary on the absolute value of the proportionality factor $k_\alpha = const$, which is constant for the utilized indenter and probe. In this case let us make use of the information about the plastic limits for the investigated clay loam according to GOST 5183-64 and 5184-64.

The liquid limit $w_f = 26$ corresponds to the dry unit weight of the soil $1/\delta_f = 0.26 + 0.37 = 0.63$ cm^3/g. Consequently, the mean pressure of penetration on the base of indenter at point II is equal to $\sigma_f = 1.25$ kg/cm^2. Whence, the proportionality factor $k_\alpha = 0.076/1.25 = 0.0608$. The plastic limit $w_p = 26 - 7.9 = 18.1\%$, which coincided with the results of the determination based on the standard $w_p = 18\%$.

Table 12. Classification of the cohesive soils with disturbed structure according to the characteristics of the hydrophilic behavior.

Categories of cohesive soils	$M_{\text{к}\,pf}$	$B_{\text{к}\,pf}$	N_{pf}
Slightly hydrophilic	<0.54	>0.46	<5.7
Hydrophilic	0.54-0.72	0.28-0.46	5.7-10.2
Highly hydrophilic	>0.72	<0.28	>10.2

The numerals I and III in Fig. 62 designate two other characteristic points of the indicative criteria of the soil at a mean pressure of penetration $\sigma_{pf} - 9.28 \cdot 1.25 = 11.6$ kg/cm^2 and $\sigma_p = 25 \cdot 1.25 = 31.25$ kg/cm^2.

The analogous and completely successful processing of the results of penetration tests from Ye. A. Yakovleva was also made on three other types of soils. For example, for a clay loam along 616 km of the Oktyabr'skiy R. R. with a proportionality factor $k_\alpha = 0.0608$ according to the results of plotting revealed the following characteristics of the plastic limits: $M_p = 9.7$; $w_f = 31.8\%$; $w_p = 22.1\%$. Based on the standard. $M'_p = 11$, $w'_f = 32\%$ and $w'_p = 21\%$, etc.

Investigations of the variability of the specific cohesion of cohesive soils with disturbed structure under conditions of a three-phase state. The design diagram of dependence $1/\delta_1 = f(\lg c_{01})$ for the case of a constant moisture content w_0 = const completely coincides with the corresponding design diagram $1/\delta_1 = f(\lg R_{01})$, presented in Fig. 55, in connection with the values of the resistivity to penetration. For the confirmation of this position Fig. 63 gives as an example the results of the experimental research by V. G. Zabara for determining by the method of vane shear the specific cohesion in 11 samples of a clay loam with disturbed structure, taken from the region of Sadovoye settlement in the Crimea ($M_p = 16$, $w_f = 42.6\%$, $\gamma = 2.7$ g/cm^3). The example in question supplements the results of penetration tests of this clay loam, depicted in Fig. 60 since the equality of the moisture contents for the reference lines with an identical number has been reached (see Tables 11 and 13).

Samples in the cutting ring whose diameter is 6.18 cm and whose height is 5 cm (by volume 150 cm^3) were tested with the aid of a vane point ($k_\tau = 56.6$ cm^3) in a LP-1 laboratory device with an attachment for vane shear [41]. Primary attention was directed to the maximum correlation of the moisture contents by weight in the separate groups of samples intended for compaction by static load up to different values of the dry unit weight of the soil. This important requirement was satisfied by preparing the samples from each group taken from one general lot with a specified moisutre content. To facilitate the analysis, the numbers of the samples are written in Fig. 63 at every experimental point. The mosture contents of the separate samples, the average moisture contents and the minimum dry unit weights of the soil for the corresponding reference lines are given in Table 13.

Based on this principle methods were developed for determining the indicative criteria of soils from the results of penetration tests using three samples as the minimum under conditions of a three-phase state [38]. For a complete and comprehensive determination of the interconnection between the physico-mechanical properties of cohesive soils with a disturbed structure, it is necessary to carry out penetration tests using 9 samples as the minimum with $G_{01} < 1$ and three different values of the moisture content by weight.

At the beginning of 1967 more than 50 nomograph data sheets were obtained for the interconnection of the physicomechanical properties of cohesive soils with disturbed structure to the plasticity index from 3.6 to 42.5 (iol'diyevaya clay). This made it possible to propose a classification of soils according to the values of the characteristic coefficient of consistency $M_{\kappa_{pf}}$ and the index of hydrophilic behavior $N_{\rho f}$ (Table 12).

The minimum value of the index of hydrophilic behavior N_{pf} is recorded at about 3 (lacustrine-glacial deposits in the area of Valday City, iol'diyeaya clays in the region of Kem', Karelian ASSR). The maximum value of the index of hydrophilic behavior noted in the conducted analyses was approximately 18 (sandy loams in the region of Vartem'yaka village, Leningrad oblast).

Let us recall that in slightly hydrophilic soils the mechanical properties are almost exclusively determined by the dry unit weight of the soil and virtually do not depend on the moisture content. In highly hydrophilic soils, on the contrary, the variability of the mechanical properties depends basically on a change in the moisture content of the soil.

Table 13. Values of the moisture contents and the minimum dry unit weights of the tested samples.

Lines	Samples	w_0 в %	w_{cp} в %	$1/\delta_{макс}$ в $cм^3/г$
I	1 2	14,7 14,7	14,7	0,517
II	3 4 5	19,4 19,6 19,4	19,5	0,565
III	6 7 8	23,4 24,3 24,2	24	0,61
—	9	30,6	30,6	0,676
IV	10 11	31,6 32	31,8	0,688

Designation: $cм^3/г$ = cm^3/g.
[в = in]

The high quality of the working procedure for preparing the samples and conducting the tests provide the necessary clarity and effectiveness to the constructed nomograph. It is easy to see that:

all the experimental points 1-11 are quite closely grouped around the averaging reference lines;

all four reference lines are mutually parallel and the precalculated positions with the minimum values of the dry unit weight of the soil I-IV are arranged on the limiting straight line, which determines the dependence $1/\delta_0 = f(\lg c_0)$ for the conditions of complete water saturation of the soil $G_0 = 1$.

At known values $1/\delta_p = 0.636$ cm^3/g and $1/\delta_f = 0.796$ cm^3/g the specific cohesion $c_p = 0.68$, $c_{pf} = 0.275$ and $c_f = 0.0285$ kg/cm^2 can be accepted as indicative criteria. In this case the following values of angular coefficints are found: $1/V_0 = 0.116$ cm^3/g and $1/V = 0.407$ cm^3/g. In addition, when $c = 1$ kg/cm^2 and $G_0 = 1$, we will have $1/\delta_0 = 0.617$ cm^3/g.

The indicative criteria in the form of angular coefficients turn out to be especially convenient for comparing the nomographs of the interconnection $1/\delta_1 = f_1(\lg R_{01})$ and $1/\delta_1 = f_2(\lg c_{01})$ (Figs. 60 and 63). For the investigated clay loam when $G_0 = 1$, the ratio of angular coefficients is $V_0/r_0 = 0.987$. Consequently, under conditions of complete water saturation of the soil at the coordinates, cohesion of the soil c - resistivity to penetration R, it is not difficult to plot the graph $\lg c_0 = f(\lg R_0)$, analogous to the graphic representation in Fig. 54. Thus, according to the values of resistivity to penetration R from the experiment it is possible to determine specific cohesion c, and then the ratio $k_\phi = c/R$, i.e., the angle of internal friction of the soil ϕ.

Since the nomographs in Figs. 60 and 63 make it possible to establish the angular coefficients $1/r$ and $1/V$, then with the use of formula (104) it is possible to determine the indexes of consistency $B_{\kappa\ pf}$ (for penetration tests) and B_{cpf} (for tests by the method of vane shear). Then, for conditions of constant degrees of water saturation of the soil G_{01} = const at coordinates lg c to lg R a system of straight lines with variable angular coefficient V_0L_{1c}/r_0L_{1R} can be constructed. Auxiliary coefficients L_{1R} and L_{1c} are calculated from formula (91) for the corresponding values $B_{\kappa\ pf}$ and B_{cpf}. In summation, it is possible to utilize the results of penetration tests for determining the indexes of friction and cohesion of three-phase soils with disturbed structure.

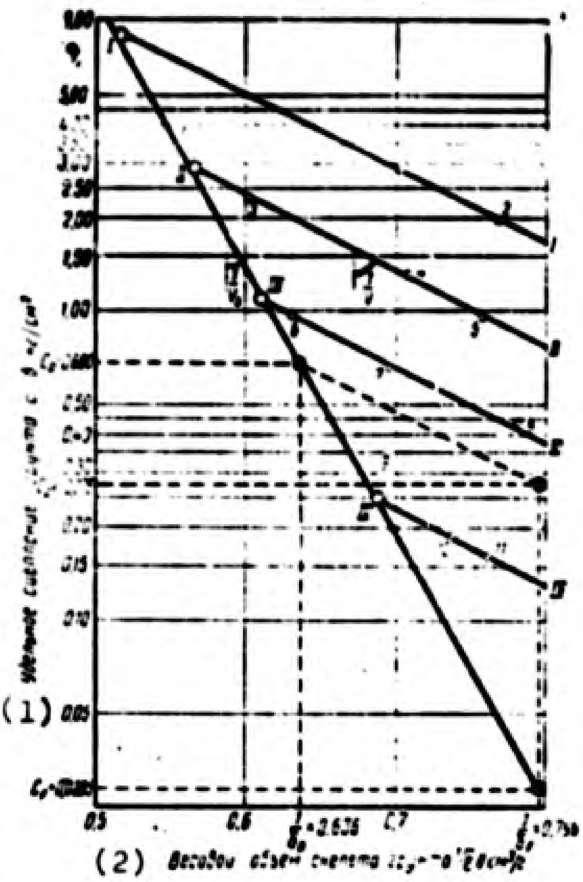


Fig. 63. Graphs of the linear dependence between the dry unit weight of loess-like clay loam with disturbed structure and the logarithm of specific cohesion for the case of constant moisture content w_0 = const (at a semilogarithmic scale). KEY: (1) Specific cohesion of the soil c in kg/cm^2; (2) Dry unit weight of the soil 1/δ in cm^3/g.

The procedure for determining the structural properties of cohesive soils. The effect of the structural properties on the value of the resistivity to penetration at identical values of the dry unit weight and moisture content of the soil can be conveniently described with the aid of variables of the coefficient of the structural strength:

$$k_{ot} = \frac{R_{osc}}{R_{os}}. \tag{131}$$

In the numerator of this formula with the designation of the resistivity to penetration by index c, the conditions of the undisturbed structure, and therefore, the presence of the structural properties of cohesive soils are emphasized. Unlike the conditions of a disturbed structure, the resistivities to penetration at the plastic limits R_{pc} and liquid limits R_{fc} are unknowns and will be subject to a special determination.

Under the three characteristic conditions of the soil, namely: with moisture contents at the plastic limits and at the liquid limits and with the moisture content at the plastic limit and the volume weight at the liquid limit, the coefficients of the structural strength will be equal to:

$$k_p = \frac{R_{pc}}{R_p} = \frac{R_{pc}}{1.900} = 0.526 R_{pc}; \tag{132}$$

$$k_f = \frac{R_{fc}}{R_f} = \frac{R_{fc}}{0.076} = 13.18\, R_{fc}; \tag{133}$$

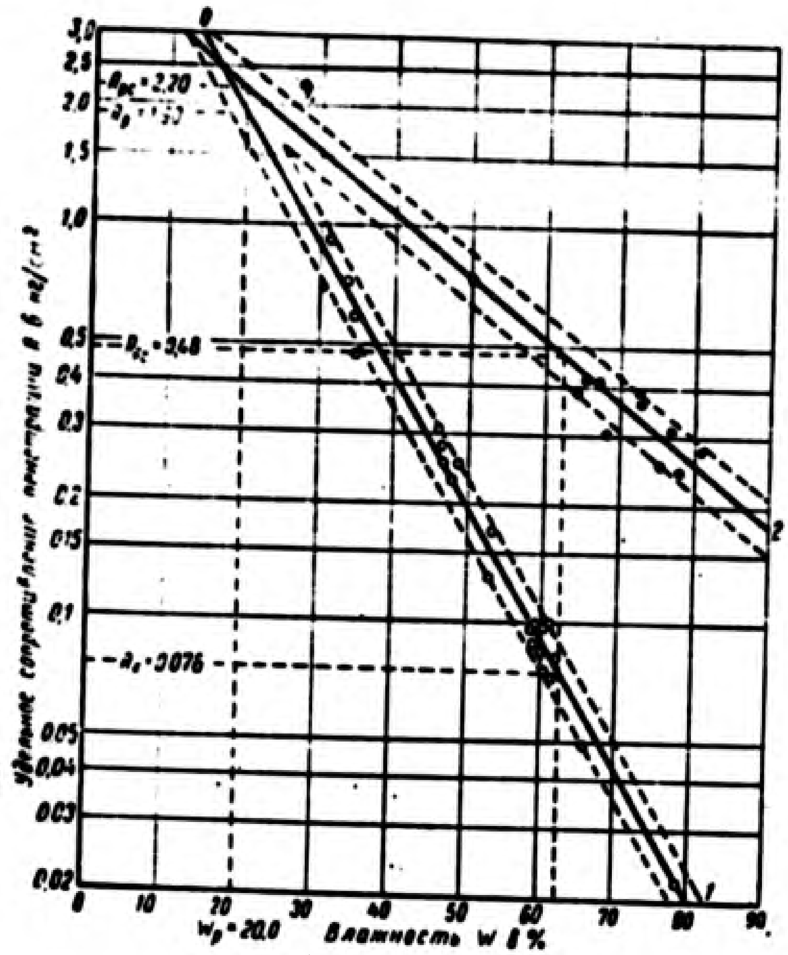
$$k_{pf} = \frac{R_{pfc}}{R_{pf}}. \tag{134}$$

It is easy to show that if the three coefficients of the structural strength are known, for example k_p, k_f and k_{pf}, then the mechanical properties of the three-phase soils with undisturbed structure, in particular, the resistivity to penetration R_{01c}, can be revealed with sufficient accuracy based on the calculation of the mechanical properties of three-phase soils under conditions with disturbed structure. In this case the coefficient of structural strength k_{01} is quite simply determined by calculating the specified physical condition of the soil, which is characterized by a moisutre content w_0 and by a dry unit weight δ_1 [37].

Investigations of the interconnection of the physicomechanical properties of water-saturated cohesive soils with undisturbed structure. The determination of the interconnection between the physicomechanical properties of cohesive soils with undisturbed structure requires meticulousness of the working procedure.

If in soils with disturbed structure it is comparatively simple to establish the functional interconnections, then under conditions of an undisturbed structure only correlation dependences usually can be obtained as a result of the unavoidable scattering of the results of the parallel experiments. In this case one cannot exclude the fact that the obtained value of the linear correlation coefficient will testify to the failure of the conducted analyses.

In 1963 in the region of Kem' city and with V. D. Shitov's participation field and laboratory penetration tests were conducted on iol'diyeva clays with disturbed and undisturbed strucutre (M_p = 42.5, w_f = 62.5%, γ = 2.76 g/cm^3). The tests were carried out under field conditions at separate sites, 1.8 × 3.5 m in dimension, at depths from 0.5 down to 1.65 m from the surface.



(1)

(2)

Fig. 64. Graphs of the linear depencence between the moisture content of pol'diyeva clay with disturbed (0-1) and undisturbed (0-2) structure and the logarithm of the resistivity to penetration for the case of complete water saturation of the soil G_0 = 1 (at a semilogarithmic scale)

$$M_p = 42.5;\quad k_f = \frac{R_{fc}}{R_f} = 6.32;\quad k_p = \frac{R_{pc}}{R_p} = 1.16.$$

KEY: (1) Resistivity to penetration R in kg/cm^2; (2) Moisture content W in %.

Parallel to this under laboratory conditions samples with disturbed structure were tested with the necessary checks on the invariance of resistivity to penetration with 10-12 and more stages of load. The tests were made using a laboratory LP-1 penetrometer.

The results of the conducted experiments confirmed the presence of a linear dependence between the moisture content of iol'diyeva clay with disturbed and undisturbed structure and the logarithm of the resistivity to penetration. Following from Fig. 64, made by V. D. Shitov at coordinates w - lg R, the graphs of the test data are depicted in the form of two straight lines, 0-1 and 0-2, that intersect near the plastic index of the soil. Consequently, the position, at which the coefficient of structural strength at the plastic limit approaches unity, turned out to be valid.

The variables of the coefficients of structural strength turned out to be equal: at the liquid limit $k_f = 6.32$ and at the plastic limit $k_p = 1.16$.

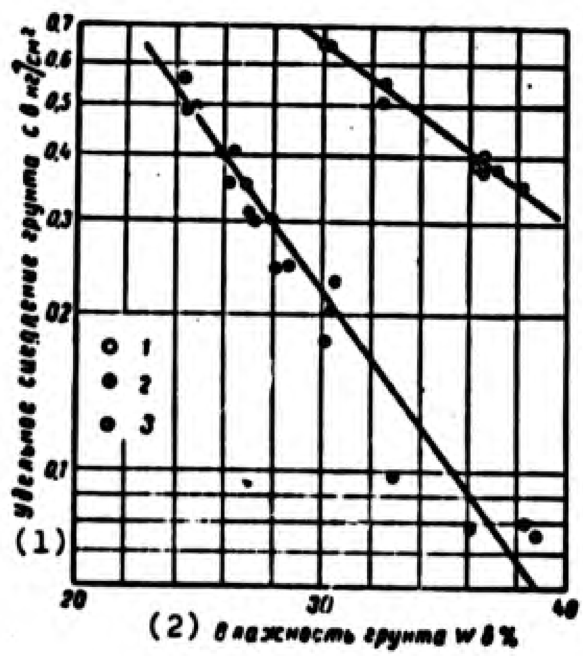


Fig. 65. Graphs of the linear dependence between the moisture content of the clay and the logarithm of specific cohesion for the case of complete water saturation (at a semilogarithmic scale). 1 - Undisturbed structure; 2 - disturbed structure; 3 - flocculent structure (according to T. Wu, A. Douglas and R. Gugner).
KEY: (1) Specific cohesion of the soil c in kg/cm^2; (2) Moisture content of the soil W in %.

Given for a comparison in Fig. 65 at a semilogarithmic scale when processing B. Ranganatham's work [71] are the results of the research by Prof. T. Wu, A. Douglas and R. Gugner [74] on the determination of the interconnection between the moisture content of a clay with disturbed and undisturbed structure (M_p = 21.2, w_f = 44.2%, w_p = 23%, based on the American standard w_F = 55%) and the specific cohesion of the clay. It is easy to see that in these analyses a completely analogous regularity was established. The coefficient of structural strength at the plastic limit proved to be, however, considerably greater k_p = 181 when k_f = 8.1.

(1)

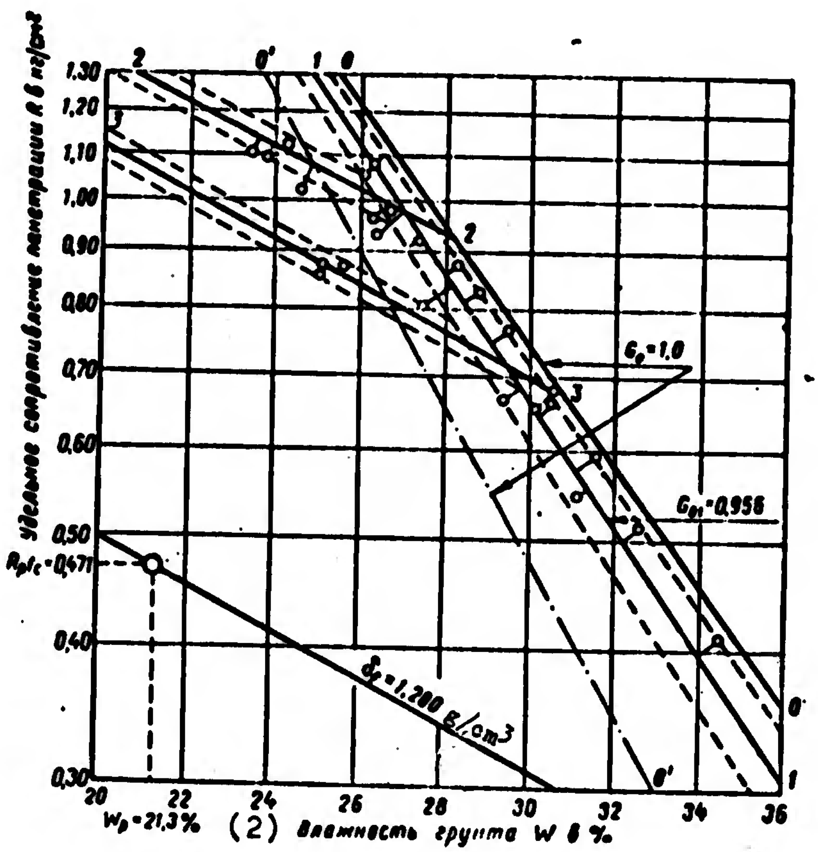


Fig. 66. Graphs of the linear dependence between the moisture content of lacustrine-glacial clay with undisturbed structure and the logarithm of the resistivity to penetration of cases G_{01} = 0.956 (graph 1-1) and δ_1 = const (graphic representation 2-2 and 3-3) (at a semilogarithmic scale).
KEY: (1) Resistivity to penetration R in kg/cm^2; (2) Moisture content of the soil W in %.

In the following example in Fig. 66, at coordinates w - lg R the results of the laboratory penetration tests of samples of lacustrine-glacial clay with undisturbed structure from the area of the town of Valday, more typical conditions of complete water saturation of the soil, are examined, (M_p = 20.4, w_f = 41.7%, N_{pf} = 3.66, γ = 2.75 g/cm^3).

The test data are interpreted in the form of an averaging straight line 1-1, which connects up the basic group of 18 experimental points and which characterizes the interrelationship of the physicomechanical properties of the investigated clay for the case of the average degree of water saturation G_{01} = 0.956.

In accordance with the equation (84a) the slope tangent of line 1-1 to the axis of the ordinate is equal to γ_w/r_0L_0. Directly on the graph in Fig. 66 we find that γ_w/r_0L_0 = 18.33%.

The remaining 7 experimental points were basic for drawing two parallel lines 2-2 and 3-3 which characterize the interrelationship w = f(lg R) for the case of a constant dry unit weight of the soil δ = 1.56 g/cm^3 and δ = 1.49 g/cm^3. The slope tangent of these straight lines to the vertical axis, designated in the design diagram in Fig. 55 through 1/t in accordance with the plotting, is equal to 1/t = 48.56%.

In order to determine the indicative criteria of the clay the following dependences are utilized:

$$\gamma_w/r_0 = 18{,}33L_0; \tag{135}$$

$$L_0 = 1 + (1/G_{01} - 1)(1 - M_{kpf}) \tag{136}$$

and

$$\gamma_w/r_0 = {}^1/tM_{kpf}. \tag{137}$$

The latter formula follows directly from expressions (97) and (90).

By equating the dependences (135) and (137), we obtain

$$18{,}33L_0 = 48{,}56\,M_{\kappa pf}.$$

After substituting its value for coefficient L_0, according to dependence (136), when $1/G_{01} - 1 = 0.046$

$$1 + 0{,}046\,(1 - M_{\kappa pf}) = 2{,}649\,M_{\kappa pf}.$$

Whence $M_{\kappa\ pf} = 0.388$, or $1 - M_{\kappa\ pf} = 0.612$. Further, in accordance with formula (137) we find: $\gamma_w/r_0 - 0.388 \cdot 48.56 = 18.84\%$.

Line 1-1 intersects on the horizontal with mark R = 1 kg/cm^2 at a point with the abscissa $w_R/L_0 = 26.85\%$. Consequently, the absolute term in equation (84a) when R = 1 kg/cm^2 is equal to $w_R = (1 + 0.612 \cdot 0.046)26.85 = 27.6\%$.

In summation, equations (84a) and (136), which characterize the interrelationship of the physicomechanical properties of lacustrine-glacial clay with undisturbed structure under conditions of a three-phase state, are derived in the form:

$$w_0 L_0 = 27{,}6 - 18{,}84 \lg R_{01}; \qquad (138)$$

$$L_0 = 1 + 0{,}612\left(\frac{1}{G_{01}} - 1\right). \qquad (139)$$

Investigations of the interrelationship of the physicomechanical properties of loess-like clay loams with undisturbed structure under conditions of a three-phase state. In 1964-1965 in the Kakhova and Rogachik irrigation areas of Kherson oblast extensive field and laboratory penetration tests of loess-like clay loams with disturbed structure were conducted for the purpose of determining the interrelationship between the physical and mechanical properties of soils. The analyses were made under the direct and active participation of I. N. Skryl' and V. D. Shitov using separate horizons in experimental excavations down to 3.5-5.5 m in depth and using samples selected by the cutting annuli by the volume 200 cm^3.

The penetration tests of soil samples with disturbed structure were carried out parallel to the determination of the indicative cr.teria used by V. G. Zabara.

The accuracy and reliability of the results of the laboratory penetration tests on samples with undisturbed structure was checked by the methods given in Chapter VI. Let us only note here that the test data of 31 samples from 193 soil samples with M_p = 12-19, delivered to the laboratory, contained various errors and therefore they were excluded from further analysis.

In connection with research on the variability of the mechanical properties of the loess-like clay loams with undisturbed structure, tested at a water saturation from 0.32 to 0.77, the design diagram for the case of constant moisture content of the soil, w_0 = const turned out to be more preferable. Accepted as the indicative criteria of soil were two angular coefficients γ_w/r_0 and γ_w/r and a dry unit weight of the soil $1/\delta_0$ when $G_0 = 1$, and resistivity to penetration $R_0 = 1$ kg/cm^2. In order to set up the equations of the interrelationship between the physicomechanical conditions of the soil dependences (90)-(92) were utilized.

The results of penetration tests of 162 samples were divided into three separate groups according to the number of reference sections, in which the samples were taken (sections up to 100 km^2 in area were spaced 60-80 km apart). For each of the sections graphic representations of the dependence (92) $1/\delta_1 = f(\lg R_{01})$ with w_0 = const, w_2 = const, etc., were plotted.

For the purpose of checking the possibility in principle of determining the interrelationship between the physicomechanical properties of the loess-like clay loams with undisturbed structure, the results of the experimental research on the first two sections were, in turn, subdivided into two parts. In the first part the results of penetration tests were selected, especially uniform samples, whose resistivitity to penetration at the butt end and on the cut surface virtually coincided (the discrepancy between $R_{\text{тор}}$ and $R_{\text{рез}}$ did not exceed 12%). The second part included all the remaining experiments on less uniform samples.

Thus, for instance, 82 samples were tested, which were selected from the first section, located in the Novotroitska area of Kherson oblast. The results of 55 tests were considered of especially high quality and were used for the construction of the nomograph, presented in Fig. 67. The remaining 27 tests were included in the nomograph, presented in Fig. 68. At this stage of processing any additional rejection of the results of experiments was not permitted

The procedure for constructing the nomographs coincides in detail with the accepted method, examined during the discussion of the results of penetration tests, depicted in Figs. 60, 61, and 62. The slight difference, typical for conditions of undisturbed structure, consists only in the fact that the precalculated position of the minimum dry unit weights of the soil produced a certain scattering relative to the limiting lines 1-8 (Fig. 67) and 1-7 (Fig. 68).

From the examination of the plotted nomographs it is not difficult to conclude that the angular coefficients 1/r completely coincided and were 0.310 cm^3/g, but the angular coefficients $1/r_0$ differ within the limits of the accuracy of the plottings 0.052 and 0.05 cm^3/g. Thus, after the determination of this feature a consolidated nomograph $1/\delta_1 = f(\lg R_{01})$ was constructed, which determines the interrelationship of the physicomechanical properties of loess-like clay loams under conditions of undisturbed structure.

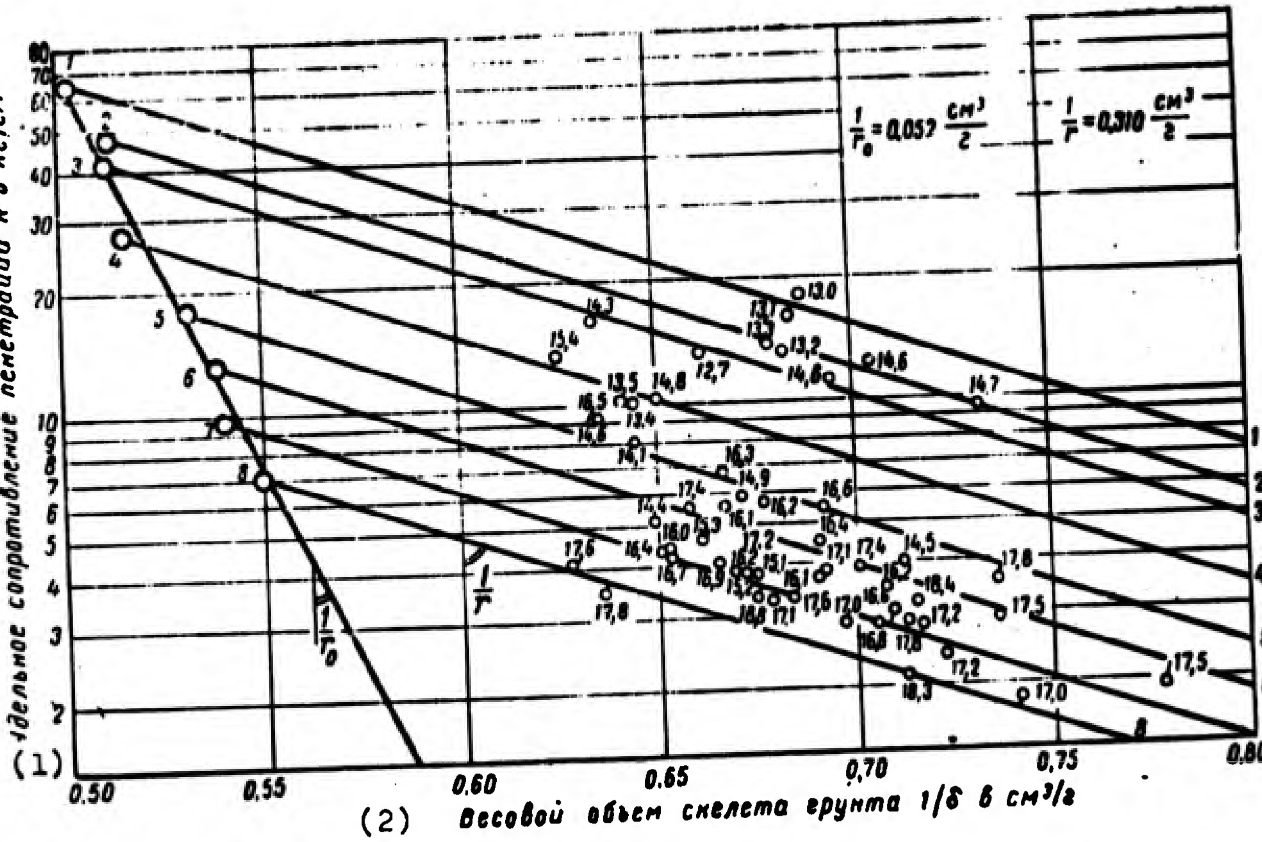


Fig. 67. Parallel graphic representations of the linear dependence between the dry unit weight of a loess-like clay loam with undisturbed structure and the logarithm of the resistivity to penetration for the case of a constant moisture content w_0 = const., w_2 = const., etc., (at a semilogarithmic scale). The graphs are plotted according to the test data of very uniform samples from the first reference section.
KEY: (1) Resistivity to penetration R in kg/cm^2; (2) Dry unit weight of the soil 1/δ in cm^3/g.

Designation: см^3/г = cm^3/g.

The characteristic index of consistency $B_{\text{н pf}}$ for the soils of the first section in accordance with dependence (90) is equal to $B_{\text{н pf}}$ = 0.052/0.31 = 0.168. Consequently, as already mentioned, according to Table 112, the investigated loess-like clay loams relate to highly hydrophilic soils. In fact, from graphs in

Fig. 67 is not difficult to establish that, for example, when $1/\delta_1 = 0.670\ cm^3/g$ in the case of a change in the moisture content from 13% (line 1-1) to 16.5% (line 6-6), i.e., altogether only up to 3.5%, the resistivity to penetration changes from 20 to 5 kg/cm^2, i.e., four-fold. It is logical that under these conditions the assertion about the decisive effect of the moisture content on a change in the mechanical properties of loess-like clay is completely correct.

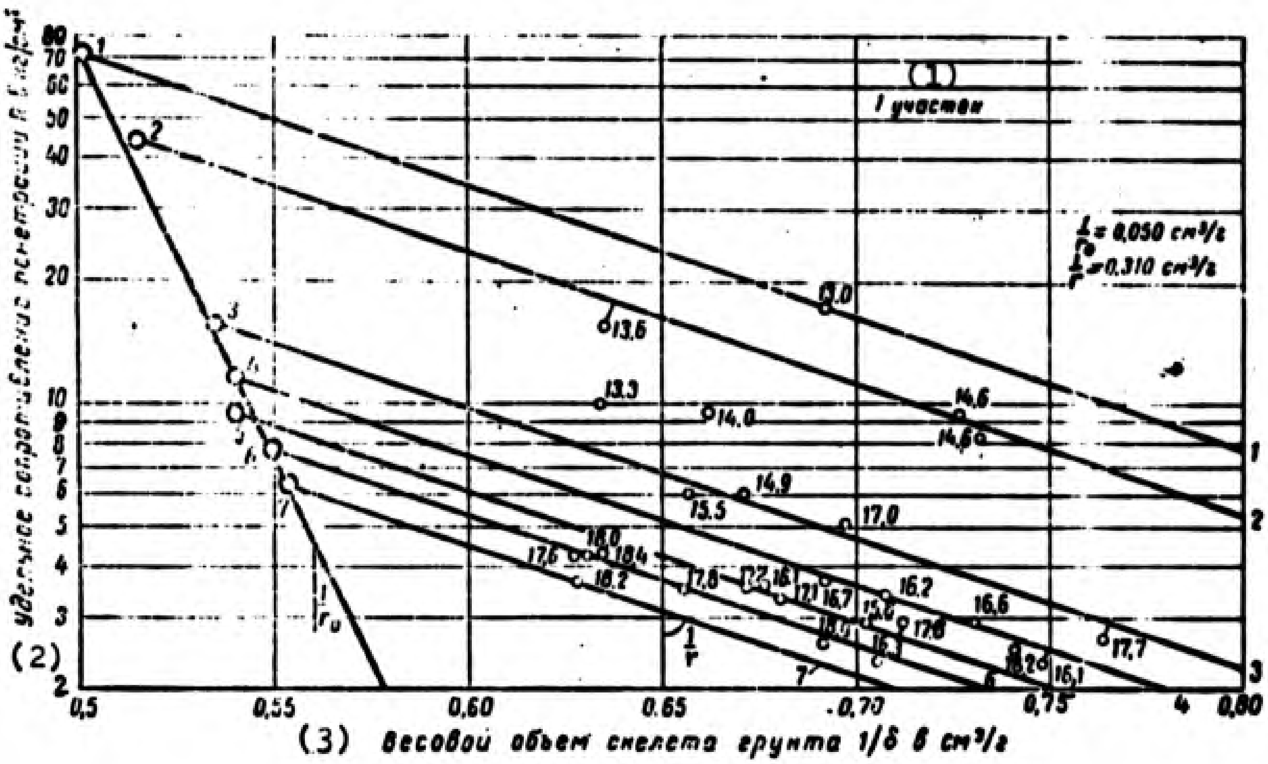


Fig. 68. Parallel graphic representations of the linear dependence between the dry unit weight of a loess-like clay loam with undisturbed structure and the logarithm of the resistivity to penetration for the case of a constant moisture content w_0 = const, w_2 = const, etc., (at a semilogarithmic scale). The graphs are plotted according to the test data of less uniform samples from the first reference section.
KEY: (1) 1 Section; (2) Resistivity to penetration R in kg/cm^2; (3) Dry unit weight of the soil $1/\delta$ in cm^3/g.
Designation: $см^3/г = cm^3/g$.

Analogous nomographs for the case of a constant moisture content of the soil with the plasticity index M_p = 12-19 were completely and unequivocally constructed for two other reference sections. In the comparison of the nomographs a very interesting feature of the investigated soils was explained. It turned out that the indicative criteria of the soils γ_w/r_0, γ_w/r and $1/\delta_0$ for three consolidated nomographs were approximately identical (for the second section γ_w/r = 0.287 and γ_w/r_0 = 0.05; for the third section γ_w/r = 0.312 and γ_w/r_0 = 0.043).

The results of the conducted analysis provided a basis for the following conclusions:

a) in sufficiently uniform cohesive soils with undisturbed structure the interrelationship between the physical and mechanical properties of soils are completely and unequivccally revealed;

b) the proposed design diagram of the interrelationship of the physicomechanical properties of cohesive soils can be used for uniform cohesive soils with undisturbed structure in generalizing and interpreting the results of the experiments;

c) with a sufficient and considerable number of penetration tests the graphoanalytic working method provides for the completely acceptable accuracy of the determination of the indicative criteria of soils with undisturbed structure.

Investigations of the interrelationship between the strength and deformation properties of cohesive soils. In technical literature it was repeatedly noted that between the impressions of punch h and the average specific pressure on its base p a parabolic dependence is observed of the form

$$p = kh^m \text{ kg/cm}^2. \quad (140)$$

In order to maintain the dimensionality dependence (140) it is more correct to present it in the form

$$p = k(h/h_0)^m \text{ kg/cm}^2, \tag{141}$$

where the shrinkage factor k, dimensionless exponent m and the certain impression of the punch $h_0 = 1$ cm represent the parameters of the equation. From the formula (141) it follows that the shrinkage factor k has a dimensionality kg/cm^2 and is numerically equal to the specific load p with the impression of the punch $h = h_0 = 1$ cm.

Within the range of the plastic state for cohesive soils with disturbed structure the characteristics of mechanical properties change exactly 25 fold, whereas under conditions with undisturbed structure they change 5-10 fold.

During the change in the characteristics of the mechanical properties by several fold the exponent m changes comparatively insignificantly and therefore can be approximately accepted as constant. Under these conditions, when h = const, it was established earlier [41], that the ratio E_1/E_2 is equal to the ratio of the corresponding values of resistivity to penetration R_1/R_2, i.e.,

$$E_1/E_2 = R_1/R_2. \tag{142}$$

The ratio [142] turns out to be especially convenient for extrapolating soil tests by the punch method. Assuming the ratio E_0/R_0 is approximately constant, and by introducing the designations $k_E = E_0/R_0$, we will find that the modulus of total deformation is proportional to the resistivity to penetration

$$E = k_E R \text{ kg/cm}^2. \tag{143}$$

The checking of dependence (143) was done according to V. B. Shvets' data on normal values of the modulus of deformation for the eluvial soils of the Urals, given in the table of regional norms [56]. Earlier in Chapter IV it was already noted that the computed values of the resistivity to penetration were determined by the author according to the standard values of the characteristics of friction and the cohesion of soils assigned to these norms.

A comparison of the values of resistivity to penetration R and the modulus of total deformation E is presented in Fig. 69. Individual points with coordinates R_i', E_i are sufficiently closely grouped along the averaging straight line E = 156.8R. The coefficient of linear correlation is equal to $r_н$ = 0.969, the error in the correlation coefficient η = 0.012 and the index of the closeness of the relationship $r_н - 3\eta$ = 0.933.

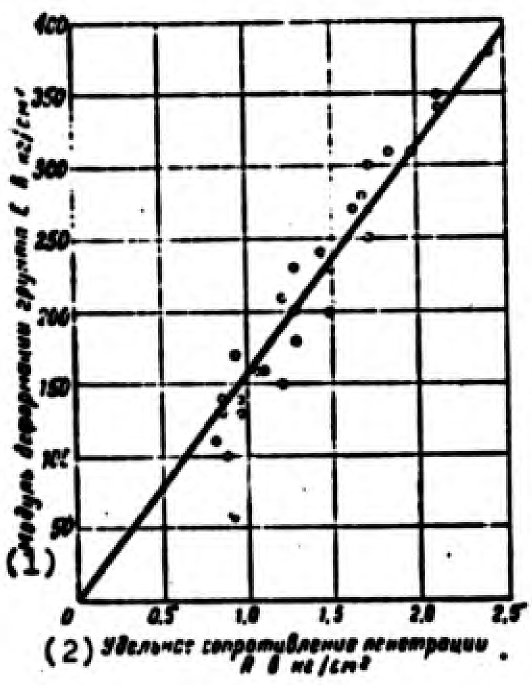


Fig. 69. Graph of the proportional dependence between the modulus of total deformation of eluvial soils and the resistivity to penetration (according to V. B. Shvets).
KEY: (1) Modulus of deformation of the soil E in kg/cm^2; (2) Resistivity to penetration R in kg/cm^2.

Thus, a very close connection between the strength and deformation properties of eluvial soils is revealed.

Thus, if one were to determine the average resistivity to penetration in the first section around the experimental punch, within the limits of the depth of the active region, then the modulus of total deformation for the second section would be located according to dependence (143) according to the results of the second group of penetration tests.

It is of interest to emphasize that back in 1945 M. N. Troitskaya noted the presence of a proportional dependence between the modulus of deformation of sands and clay loams and the number of blows of the load using a DORNII striker. In I. V. Dudler's comparatively recent analyses [22] linear dependences were obtained, while in certain cases there were also proportional dependences between the modulus of total deformation of alluvial sands and the index of dynamic sounding N. In this case the coefficients of linear correlation were from $r_{\text{к}} = 0.81$ to $r_{\text{к}} = 0.96$.

The determination of the degree of compactness and the dry unit weight of sandy soils.

Since 1964 under the guidance of the author field and laboratory investigations were carried out for the purpose of establishing the interrelationship between the dry unit weight of the sand and the indexes of penetration and sounding. Taking into account the known literature sources [22, 23, 52-54, 63, 70] and the results of the conducted experiments the following is noted.

In sandy soils, the disturbed and, apparently, undisturbed structure can reveal the dependence between the characteristics of the physicomechanical properties. Thus, for instance, under conditions of disturbed structure and a constant degree of water saturation of the sand between the unit weight and the logarithm of the index of penetration linear dependences are exhibited of the form

$$1/\delta_i = 1/\delta - 1/v \lg U_i/U \ \text{cm}^3/\text{g}, \qquad (144)$$

where $1/v$ - angular coefficient, which represents at coordinates $1/\delta$ - $\lg U$, the slope tangent of the plotted straight line to the vertical axis.

As an example Fig. 70 gives the results of penetration tests, completed by N. L. Zotsenko on samples of fine-grained sand with a moisture content $w = 3.5\%$ (line 1), with a hygroscopic moisture content $G_0 \approx 0$ (line 2) and under conditions of complete water saturation $G_0 = 1$ (line 3).

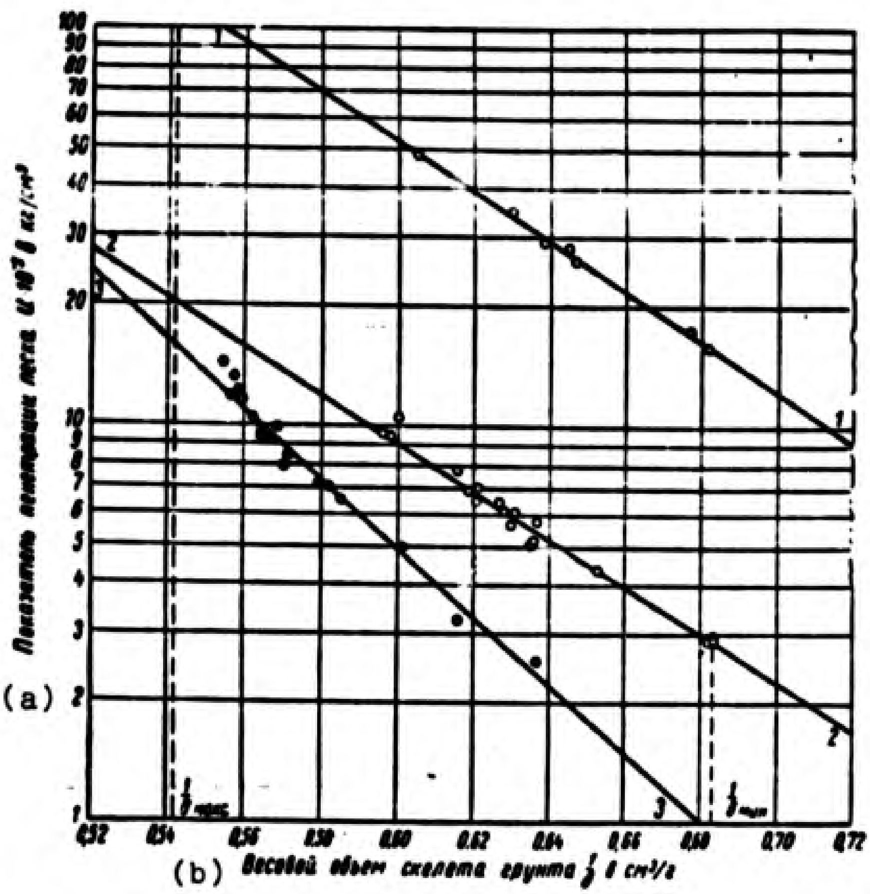


Fig. 70. Graphs of the linear dependence between the unit weight of fine-grained sand and the logarithm of the index of penetration. 1 - Moist sand; 2 - air-dried sand; 3 - water-saturated sand (at a semilogarithmic scale).
KEY: (a) The index of penetration of sand $U \cdot 10^{-3}$ in kg/cm^2; (b) Unit weight of the soil $1/\delta$ in cm^3/g.

In these tests the checking of the invariance of the index of penetration was conducted using 10-14 stages of loading. The tests were made with the aid of an LP-1 penetrometer with a standard coniform point. Complete water saturation of the samples in the third series of test was reached by vacuuming the samples under the bell of a vacuum pump of Komovskiy design. The samples of sand with different values of the dry unit weight (from 0.55 to 0.685 cm^3/g) were tested in rings whose diameter is 108 mm and whose height is 110 mm.

The basic test data are given in Table 14.

Table 14. The characteristic values of the physicomechanical properties of sand and the basic data of the correlation analysis.

State of the sand	Degree of water saturation G	Boundary values of the dry unit weight in cm^3/g		Boundary of the values of the index of penetration in kg/cm^3		Correlation coefficient r_K	Error in the correlation coefficient η	Characteristic of the closeness of the relationship $r_K - 3\eta$
		$1/\delta_{макс}$	$1/\delta_{мин}$	$U_{макс}$	$U_{мин}$			
Dry . . .	0	0,542	0,683	$20.2 \cdot 10^{-3}$	$2.86 \cdot 10^{-3}$	0,97	0,01	0,94
Moist . .	—	0,587	0,682	$63.1 \cdot 10^{-3}$	$15.9 \cdot 10^{-3}$	0,98	0,017	0,94
Water-saturated	1	0,542	0,637	$16.3 \cdot 10^{-3}$	$2.36 \cdot 10^{-3}$	0,97	0,013	0,93

[макс = max; мин = min]

In Table 13 and in Fig. 70 using the designations $1/\delta_{макс}$ and $1/\delta_{мин}$ the dry unit weight of the sand are determined at maximal dense and unconsolidated states in cm^3/g, and $U_{макс}$ and $U_{мин}$ - maximum and minimum values of the index of penetration in these states.

The equations of lines 2 and 3, presented in Fig. 70, take the form:

a) for the dry state of the fine-grained sand

$$1/\delta = 0.759 - 0.166 \lg U \text{ cm}^3/\text{g};$$

b) for the water-saturated state of the fine-grained sand

$$1/\delta = 0{,}679 - 0{,}113 \lg U \text{ cm}^3/\text{g}.$$

The standard deviation of the unit weight of the sand, according to the conducted tests, comprised only ±0.006 cm^3/g. Consequently, in both cases when $G_0 \approx 0$ and $G_0 = 1$, the accuracy of the penetration determination of the unit weight of the soil is in no way inferior to the accuracy of the determinations by the cutting ring method.

The degree of compactness of sandy soils D, as is known, is determined from the formula:

$$D = \frac{1/\delta_{мин} - 1/\delta_i}{1/\delta_{мин} - 1/\delta_{макс}}. \qquad (145)$$

where $1/\delta_1$ - unit weight of the sand in the investigated state.

After the substitution of their values into the formula (145) instead of the values of the unit weights of the sand, expressed with the aid of equation (144) through the indexes of penetration, we will obtain

$$D = \frac{\lg U_i/U_{мин}}{\lg U_{макс}/U_{мин}}. \qquad (146)$$

For example, in the first approximation, assuming $U_{макс} = 20.2 \cdot 10^{-3}$ kg/cm^3 and $U_{мин} = 2.86 \cdot 10^{-3}$ kg/cm^3, we will obtain the following simple formula for determining the degree of compactness of dry fine-grained sands according to the results of the penetration tests:

$$D = 3 + 1{,}18 \lg U. \qquad (147)$$

In this formula the index of penetration has the required dimensionality kg/cm^3.

Let us recall that in comparison with the clayey soil, the sandy soils have the two following features. First, the values of the unit weights of (void ratios) of the sand are restricted to the minimum and maximum values $1/\delta_{\text{макс}}$ and $1/\delta_{\text{мин}}$, corresponding to the densest and unconsolidated compositions of the sand. Thus, in spite of the known conditionality of these boundaries and some deficiencies in the procedure for determining them, one ought to consider the fact that physically it is not possible to obtain sands more dense or more unconsolidated than the revealed values $1/\delta_{\text{макс}}$ and $1/\delta_{\text{мин}}$ (see dotted lines in Fig. 70).

In the second place, in clayey soils an increase in the moisture content always leads to a reduction in the characteristics of the mechanical properties of the soils. On the contrary, in sandy soils with the excess of hygroscopic water the mechanical properties of medium- and fine-grained sands, as a rule, increase and only after reaching maximum cohesion do they begin to decrease.

Further analyses in this direction, obviously, will make it possible to develop concrete recommendations based on the determination of the degree of compactness and unit weight of the different sandy soils according to the results of penetration and sounding.

CHAPTER VI

THE PRINCIPLES OF THE TECHNOLOGY OF SOIL TESTING BY THE METHODS OF PENETRATION, SOUNDING AND VANE SHEAR

Checking the results of mechanical soil tests. The statement of the problem of the experimental determination of the correlation between the physicomechanical conditions of the soil with disturbed and undisturbed structure brought about the need for the introduction of supplementary measures for checking the quality of the test data, obtained by the methods of penetration, sounding and vane shear.

Before turning to the in-depth analysis of the test data, one ought to attempt to evaluate objectively the quality of the carried out work, to disclose, and then eliminate a certain part of the results, containing rough or systematic errors.

During penetration tests under laboratory conditions the basic errors are connected with the possible disturbance of the natural composition of the soil which was permitted during sampling, transportation, storage and preparation of the samples. However, during testing, for example, of morainic soils cases are not eliminated where the presence of the different inclusions, commensurable with the depth of indentation of the indentor, determines the infeasibility of testings similar soils under laboratory conditions.

If samples with disturbed structure are tested then the obtained results can contain errors because of defects in the technique of preparing the samples - the nonuniform distribution of the moisture content in the various portions of the sample, the slight drying of the samples from the surface, the nonuniform packing of the samples, etc.

The many years of experience in laboratory penetration testing made it possible to propose a completely effective system of checking the quality of the tests, although it is somewhat unwieldy.

For this purpose all of the samples, as a rule, are tested from two sides of the cutting ring for the necessary testing of the invariance of the resistivity to penetration. The degree of difference in the results is determined from the formula

$$v_R = \frac{R_{\text{тор}} - R_{\text{рез}}}{R_{\text{тор}} + R_{\text{рез}}} 100\%, \tag{148}$$

where $R_{\text{тор}}$ and $R_{\text{рез}}$ are the value of the resistivity to penetration on the part of the butt end and cut surface of the cutting ring. (Penetration tests of samples in the laboratory are made only under the condition of confining the sample in a steel casing or cutting ring, since otherwise test data will be underestimated because of the wedging effect of the conical point).

If the absolute value v_R is less than 18%, then as a desired result the arithmetic mean $R = (R_{\text{тор}} + R_{\text{рез}})/2$ accepted. With $v_R > 18\%$ the penetration tests are repeated with a new sample or, after establishing the thickness of the soil layer in the ring with sharply contrasting different characteristics of mechanical properties, the determination of the volume weight and moisture content for each layer of soil is carried individually. Generally, one should accept as a rule that the penetration tests under laboratory conditions are undesirable, if more one-fourth of the tested samples result in $v_R > 18\%$.

If $v_R < 12\%$, then the tested samples differ by the high uniformity of the mechanical properties.

An example of the comparison of the results of penetration tests of the loess-like clay loams with undisturbed structure from Kherson oblast, conducted from two sides of the sample, is presented in Fig. 71. It follows from the figure that the majority of the test data falls in the range $v_R \leq \pm 15\%$.

As a result of the disturbance of the invariance of the resistivity to penetration 6 samples were excluded from further examination (3%).

The check on the constancy of the moisture content of the test samples is achieved by a comparison of the values of the moisture content obtained by the drying of the samples selected under field conditions during the cutting of the samples - $w_{пол}$ and it was found that during the repeated moisture content tests of these samples after completion of the penetration tests - $w_{лаб}$. It is obvious that the convergence within the limits of ± 1-2% (absolute) mean values of the moisture content $w_{лаб}$ and $w_{пол}$, obtained from two parallel determinations, will testify to the effectiveness of the accepted methods of storing the samples.

An example of the comparison of the results of parellel moisture content tests of 259 samples of loess-like clay loams with undisturbed structure is presented in Fig. 72. It is easy to see that the overwhelming majority of the results (namely 89% of the tested samples) were in disagreement in values of moisture contents not more than $\pm 1\%$ (absolute), which characterizes the necessary care required in conducting the investigations. Only in 28 parallel moisture content tests was disagreement from ± 1 to 2% of the moisture content by weight. The calculations showed that the root-mean-square error in a ratio of $w_{лаб}/w_{пол}$

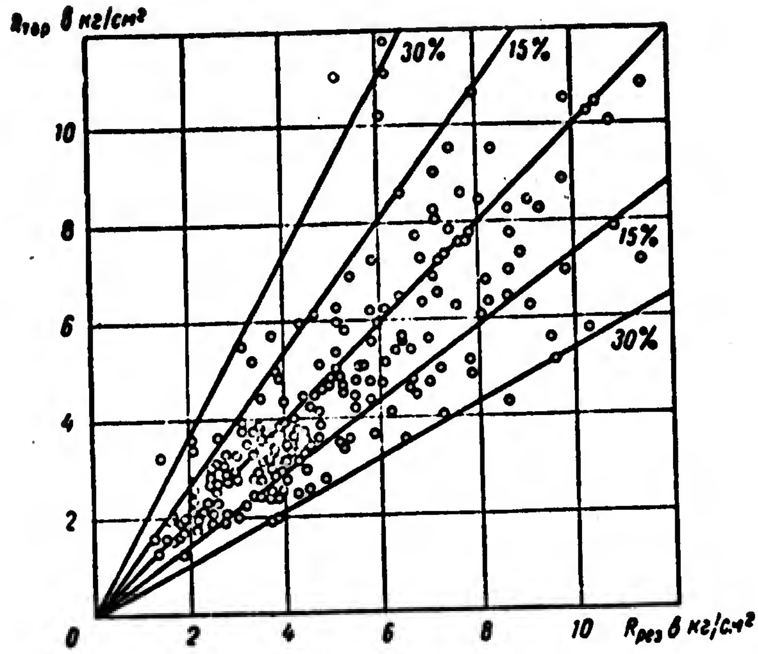


Fig. 71. Graph of the comparison of the values of the resistivity to penetration obtained on one and the same sample of the loess-like clay loam with undistured structure during tests on the cut sides - $R_{рез}$ and butt end - $R_{тор}$ of the cutting ring.

Designation: в кг/см^2 = in kg/cm^2.

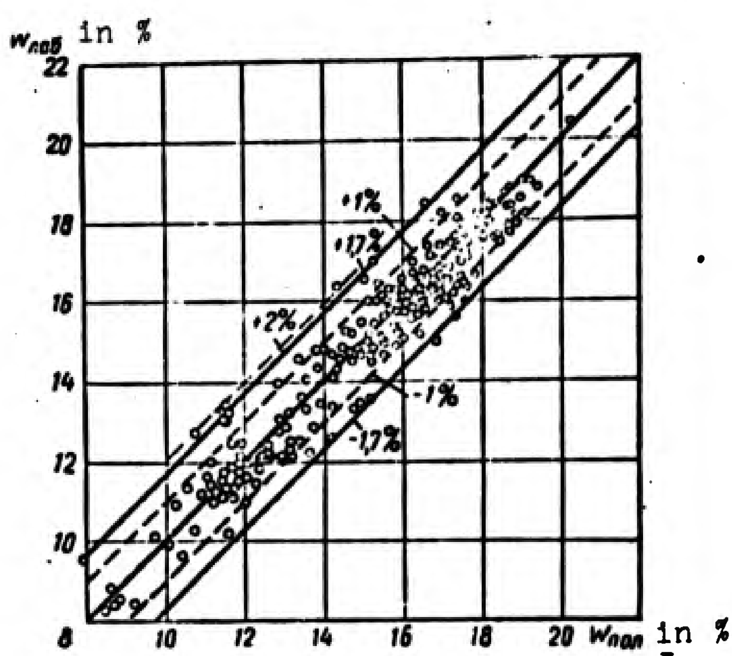


Fig. 72. Graph of the comparison of the values of the moisture content of a loess-like clay loam, determined by air-drying the samples selected under the field $w_{пол}$ and laboratory $w_{лаб}$ conditions.

was equal to 0.045. Thus, within the range of change in the moisture content of 14 to 19%, the absolute values of the root-mean-square error in the moisture content tests fall within the interval of ±0.6-0.9% (absolute).

Hence, it is not difficult to conclude that during the process of transport and storage of the samples the constancy of the moisture content was completely retained. In this matter, however, it is not surprising, since the penetration tests of the samples, as a general rule, were made within days after their selection in the excavation.

Based on the materials of a comparison of $w_{поле}$ and $w_{лаб}$, presented in Fig. 72, rejection of the results of the penetration tests naturally was not conducted.

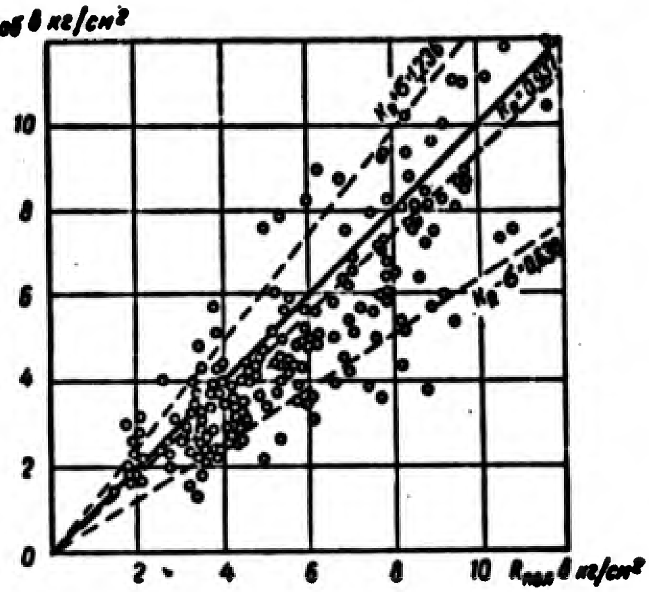
The check on the quality of the cutting and dressing of the samples, after storage of the samples during their transport to the laboratory is conducted by means of the comparison of the results of the field and laboratory penetration tests. For this purpose the mean values of the laboratory penetration tests $R_{лаб} = \frac{R_{то} + R_{рез}}{2}$ are compared with the mean values from four field penetration tests $R_{пол} = \frac{R_1 + R_2 + R_3 + R_4}{4}$, made around a sample in the excavation.

Presented in Fig. 73 are the results of the comparisons $R_{пол}-R_{лаб}$ from the same series of field and laboratory tests on loess-like clay loams with undisturbed structure. The basic derivations according to the results of this comparison amount to the following:

a) root-mean-square deviation of the ratio $\frac{R_{лаб}}{R_{пол}}$ was ±0.299;

Fig. 73. Graph of the comparison of the average values of the resistivity to penetration obtained during tests under field $R_{пол}$ and laboratory $R_{лаб}$ conditions.

Designation: в кг/см^2 = = in kg/cm^2.



b) on the average $R_{лаб}$ = 0.937 $R_{пол}$, i.e., a certain reduction is observed in the results of the laboratory penetration tests in comparison with the field;

c) at moisture contents less or equal to 12-13%, a systematic understating of the results of the laboratory penetration tests was observed as a result of the volumetric disturbance of the structure of the sample during its cutting.

In order to evaluate the degree of convergence of the results formula (148) and criteria $v_R \leq 18\%$ were utilized. Based on the results of this comparison the test results of an additional 10 samples (5.2% of a light clay loam with a plasticity index $M_p \approx 12$ were rejected. Thus, from the comparison of the results of the field and laboratory penetration tests in this case considerable difficulty in selecting low moisture samples of light loess-like clay loams during resistivity to penetration $R > 15$ kg/cm^2 was revealed.

The correctness of establishing the volume weight of the sample is checked by means of a comparison of the values of the volume weights obtained in the parallel determinations made at every horizon in the excavation. During 4-6 parallel determinations Δ at every horizon the deviation in the various values from the average is somewhat varied in the different soils. During the analysis of loess-like clay loams a condition was accepted, whereby the absolute value of the difference $\delta_1 - \delta_{cp}$ did not exceed 0.025 g/cm^3.

In summation, the test data of an additional 7 samples (3.6%) were excluded from further examination. Besides that, only according to indirect criterion were the test data of an additional 6 samples (3.1%) considered inconsistent.

As a whole, as already mentioned in Chapter V, of 193 samples with the a plasticity index from 12 to 19 the test data of 162 samples, or 84% were used for the construction of the compound nomograph. Consequently, even with the specified thoroughness of the soil mechanics determinations and penetration soil tests these or different errors, underlying the required determination are unavoidable.

The given procedure for checking the accuracy and the reliability of the results of penetration tests for production targets it is all the more expedient to be selective, having considered the real possibility for the emergence of errors.

For example, during the resistivity to penetration $R < 10$ kg/cm^2 the volumetric disturbances of the structure of samples of loess-like clay loams do not appear during their cutting. Consequently, the checking must be achieved there, where it is actually necessary, for example, during the testing of low moisture clay loams when $R \geq 10$ kg/cm^2, etc.

Statistical methods of checking the accuracy of the test data. The need for the use of statistical methods for checking during the determination of the strength properties of soils is provided for in the project "Instructions for determining the characteristics of soils (standard and calculated)," published in the journal "The basis, fundamentals and mechanics of soils" in 1966.

These instructions are completely valid, since the statistical methods of checking the degree of scattering of the studied characteristics make it possible not only to not evaluate the accuracy of the obtained results, but also to determine the necessary number of parallel tests.

In Chapter II with the presentation of the recommended method for interpreting the results of dynamic sounding certain methods of such an analysis have already been noted. First, within the limits of the exposed uniform layer of soil auxiliary coefficients k_i are calculated which represent the ratio of the measured characteristics, R_i or Q_i or τ_i, etc, to the calculated characteristic for a uniform layer or site, R or Q, or τ [see formula (50)].

The standard deviation of the various tests is computed according to the formula

$$\sigma_i = \pm\sqrt{\frac{\overline{(k_i-\bar{k})^2}}{n}}. \qquad (149)$$

where $\bar{k} = \frac{\Sigma k_i}{n}$ - mean value of the ratio k_i for each different layer or site; n - number of carried out measurements within the limits of each selected layer.

As already mentioned, when $n < 30$ under the radical in the denominator of formula (149) one should take $n - 1$.

The coefficient of a variation in the ratios k_i is determined from the formula

$$v = \frac{\sigma_i}{\bar{k}}. \qquad (150)$$

The results of the carried out calculations can be presented in the form of graphs of normal distribution. In order to do this, along the axis of abscissae are plotted the values of the ratios k_1 at intervals of 0.1, and along the axis of ordinates - the rate n_1/n, i.e., the relative quantity of coinciding values k_1 in the interval. As an example Fig. 74 shows a graph of normal distribution, plotted from the results of 846 field penetration tests, made in one of the reference sections of Kherson oblast. The root-mean-square deviation amounted to $\sigma_1 = 0.132$ with the coefficient of variation $v = 13.2\%$ and the index of accuracy $\rho = 1.3\%$.

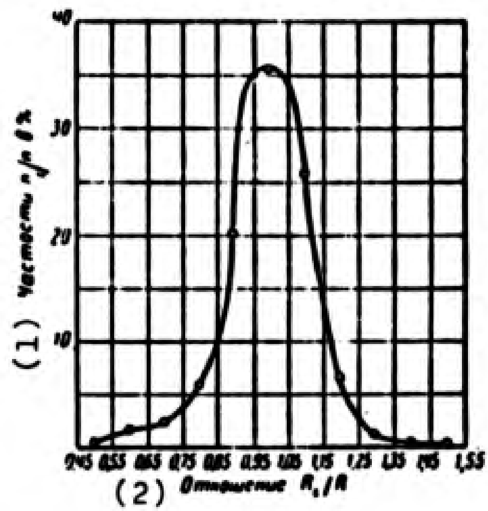


Fig. 74. Graph of the distribution of the stratified samplings of ratio R_1/R, plotted according to the results of the field penetration tests on loess-like clay loams with undisturbed structure. KEY: (1) Rates n_1/n in %; (2) Ratio R_1/R.

The degree of uniformity of the studied characteristics of the soil is established in accordance with recommendations in p. 5.4, SNiP II-B.1-62:

$$k_{\text{од}} = (1 - v)\,100\,\%. \qquad (151)$$

The soils are considered to be completely uniform, if $k_{\text{од}} > 80\%$.

The standard deviation of the arithmetic mean ratio $\bar{k}$ is equal to:

$$\bar{\sigma} = \pm \frac{\sigma_k}{\sqrt{n}}. \qquad (152)$$

The index of accuracy of tests is found as the ratio

$$\rho = \pm \frac{\bar{\sigma}}{\bar{k}} 100\%. \qquad (153)$$

The results of the carried out analysis are expressed in the form

$$k = \bar{k} \pm \bar{\sigma}. \qquad (154)$$

For the transition to the absolute values of the studied characteristics the initial dependence (50) $k_1 = R_1/R$ or $k_1 = Q_1/Q$, etc., is applied. It goes without saying that the calculations according to formulas (149)-(154) can be made in connection with one uniform layer or site, utilizing the absolute values R_1, Q_1, τ_1, etc.

Upon completion of field tests the determined value of the index of accuracy ρ is assigned. The number of parallel tests is selected from the condition that the actual index of accuracy calculated from formula (153) would not exceed the assigned value.

The methods of penetration and sounding are calculated in mass application. Thus, the accuracy of the test data within the limits $\pm$1-5% is comparatively not difficult to obtain.

The effect of the velocity of penetration, sounding and vane shear on the test data. The existing designs of laboratory penetrometers, with a few exceptions, are calculated by conducting tests during the staged application of the vertical force. The obtained typical graph of the results of penetration tests in this case is presented in Fig. 75 (as an example the test data of a sample of loess-like clay loam during the resistivity to penetration $R = 8.05\ kg/cm^2$ are selected). At coordinates of the observation time t - depth of indentation of the point h during different stages of vertical force $P_1 = 2.2$ kg, $P_2 = 4.2$ kg, etc., graphs $h = f(t)$ are plotted which differ by the relatively larger depth of indentation of the point in sections a-b with a short duration time of the action of the load and, on the contrary, by the relatively shallow depth of indentation of the point in sections b-c during a relatively longer time of action of the load. Graph $h = f(t)$ for the second section at a semi-logarithmic scale usually straightens into a straight line, which indicates the development of creep strain in the soil.

The noted feature of graphs $h = f(t)$, which are obtained during penetration tests, served as the principle for the selection of the boundary between the two sections as a maximum depth of indentation of the point [43, 41]. As a rule, the maximum observation time at every stage of load does not exceed several minutes. Thus, the penetration tests at this stage of load can cease, if after the latter 30 s or 1 min the observation of an increase in the depth of indentation of the point amounts to less than 0.01 cm. Frequently, according to the experience of the previous tests constant duration holdings at every stage of load are assigned for example, 1 or 2 min. At each stage after the expenditure of holding time the depth of indentation of the point is recorded in the record book, and the subsequent stage of load is applied to the indenter. This test procedure

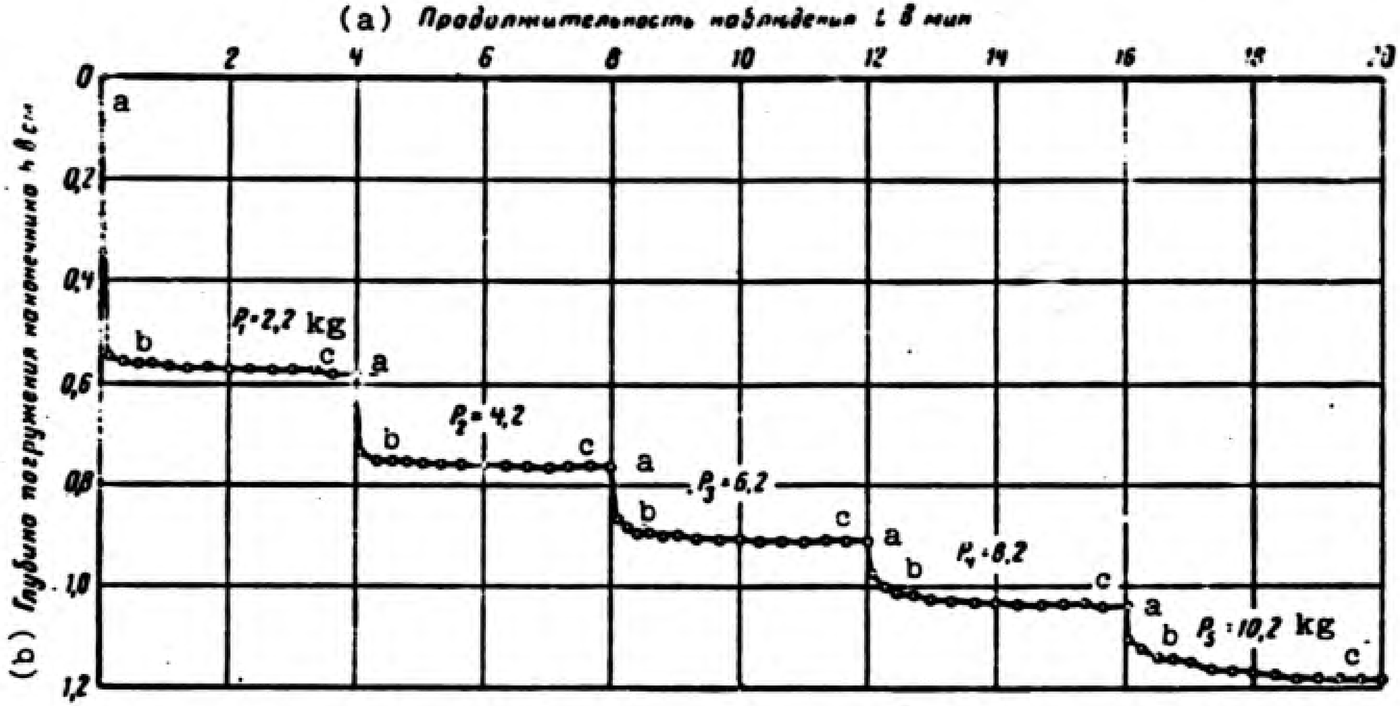


Fig. 75. Graphs of the dependence of the depth of indentation of the conical point on the sampling time during the action of the staged load of a constant value.
KEY: (a) Observation time t in min; (b) Depth of indentation of the point h in cm.

makes it possible to construct the graph of the invariance of the resistivity to penetration directly during the process of the testing.

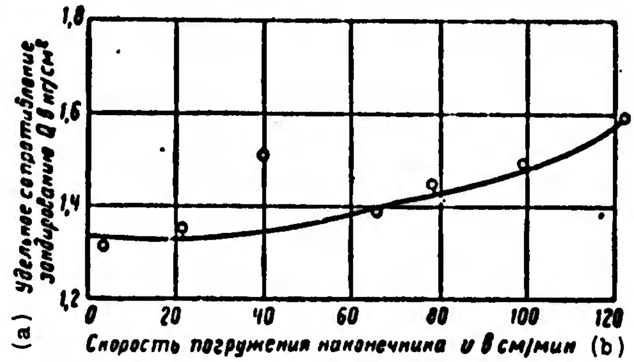
Under field conditions penetration tests by manual instruments are conducted with a more or less certain constant velocity of insertion of the point into soil on the order of 0.5-1 cm/s. In the installation of static penetration or sounding the assigned constant velocity of the insertion of the point into the soil is comparatively simply provided by a reducer device.

The effect of the velocity of penetration on the test data was studied experimentally by Prof. V. F. Babkov [2], S. I. Rokas [47] et al.

According to the results of these tests a conclusion can be formulated about the need for the standardization of the velocity of penetration soil tests. In the opinion of S. I. Rokas, the velocity of penetration should not exceed 1-1.2 cm/s, i.e., 60-70 cm/min. With a further velocity increase the resistance of the soil begins to increase noticeably. Increase reaches 20-30% with velocities of 180-192 cm/min. According to V. F. Babkov, a further velocity increase to 565 cm/min can lead to a resistance increase up to 50% [2].

The effect of the velocity of sounding on the test data was studied by N. L. Zotsenko directly under field conditions in the region of the Novotroitsk settlement, Kherson oblast with the aid of a MP-1 probe. The tests were made in a loess-like clay loam with undisturbed structure (M_p = 13, w_f = 38.5, γ = = 2.7 g/cm^3) with the use of a conical point 9 cm high with $\frac{d_{кон}}{d_{шт}}$ = 1.8. The graph of the results of comparative tests in connection with a uniform layer of soil at a depth from 60 down to 80 cm is presented in Fig. 76. In these tests the degree of uniformity was from 86 to 96% with an index of accuracy of the tests from 0.8 to 1.5%.

Fig. 76. The graph of the dependence of the resistivity to sounding on the velocity of the insertion of the conical point.
KEY: (a) Resistivity to sounding Q in kg/cm²;
(b) Velocity of the insertion of the point v in cm/min.



It follows from Fig. 76 that at a velocity from 4 to 40 cm/min the resistivity to sounding remained constant. With a velocity increase of the sounding by 3 times, from 40 to 120 cm/min, the resistivity to sounding increased to 19%. Under conditions of sounding the effect of a velocity increase of sounding by the obtained results began to appear at a somewhat lower velocities in comparison with the penetrations. Undoubtedly, this problem requires further study. Thus, henceforth before it can be improved one should consider a velocity increase of penetration and sounding of more than 40-50 cm/min as undesirable. V. F. Babkov and A. S. Smirnov arrived at approximately the same conclusion about accepting the maximum velocity insertions of the point for a field penetration device at 60 cm/min.

In relation to the maximum permissible velocity of vane shear, the known literature sources contain quite limited and inconsistent information. At the Swedish Geotechnical Institute, for example, they consider that if the maximum speed of vane shear does not exceed 0.1 deg/s, then the values of resistivity to vane shear at the different velocities of shear less than maximum, will be approximately identical [61]. O. M. Reznikov notes that the maximum angular velocity of vane shear $\omega_{макс}$ should not be standardized, but the maximum velocity of the application of torque $\Delta M/\Delta t$ = const should be. In this case the values of the maximum resistance of the soil corresponding to "short-term" or "standard" strength, almost free of the effect of creep, will be determined. Virtually, the duration of the tests by the method of vane shear amounted to 15-20 s at angles of shear from 3 to 30° in O. M. Reznikov's experiments. Thus, in DIITe, just as at the institute, VSEGINGEO, tests were carried out at considerably higher velocities of vane shear.

Meanwhile, in the operating instructions of TsNIIS, for the determination of the resistance of soils to shear in borings [30] it is noted that "rapid" shear is completed in 5 to 15 min during the scheduled time.

During this stage of research on the problem the angular velocity in the interval 0.25-0.5 deg/s in the first approximation, can be accepted as the basis. A further increase in the angular velocity of cutting based on a number of considerations, and among other things, according to the conditions of recording the test data, would be premature.

Technological features of penetration tests under laboratory conditions. During penetration tests of samples at liquid, liquid-plastic and soft-plastic consistencies, the minimum weight of the mobile system of the penetrometer should not exceed 0.05-0.1 kg. In a contrary case during the first and partially during the second stage of loading, certain exaggeration of the depth of indentation of the conical point will be observed because of the development of the force of inertia of the drop. Thus, on the graph of the invariance of the resistivity to penetration, the first two experimental points will be made to separate out.

In this sense the recommended technique for penetration tests, not allowing for the development of the force of inertia of the drop of the point, differs significantly from the proposals of the Swedish Geotechnical Institute in penetration soil tests by dropping cone (Fall - Cone Test) [50, 64, 65, 66, 67] or from P. O. Boychenko's proposals on the application of a falling cone 0.3 kg in weight [6, 7].

Thus, for instance, T. Kal'stenius [65] quite correctly notes that the equation of the equilibrium conditions for the results of penetration tests by the falling cone takes the form

$$Pk_{дин} = N_{\Delta}\,\Delta h^{3} + N_{c}\,ch^{2}\ \text{kg}, \qquad (155)$$

where $k_{дин} = 1 + 1.63\ h^{0.23}$ = "dynamic factor," depending on the depth of indentation of the point; N_{Δ} and N_c - auxiliary coefficients.

It is not difficult to see that when $k_{дин} = 1$, T. Kal'stenius' fundamental formula coincides with formula (3), stemming from the solution of the axisymmetric problem of the theory of limit equilibrium.

The second most widespread error of laboratory penetration is connected with the surface effect of the test specimens. During the testing of the samples with disturbed structure appreciable errors in the values of the resistivity to penetration during the surface drying of the sample are possible. For example, during the formation of a layer of slightly dry soil with a thickness altogether of only 1-2 mm, the error in values of R can reach several tens of percentage. In this case the use of formula (29), derived for the analysis of the results of penetration tests, is extremely inconvenient. Thus, the dressing of the sample should be done only immediately before the testings and measures taken to protect the sample from drying.

During the testing of samples with undisturbed structure, taken with a cutting ring, on the contrary, it is possible to encounter a phenomena of a certain breakdown of the structural bonds in the surface layers of the sample during its dressing. Thus, one should consider the fact that the results of penetration tests, made with the depth of indentation of the point up to 0.5 cm. can contain considerable errors. Under these conditions the depth of indentation of the point should not be less than 1.5-2.5 cm.

The third possibility of more or less considerable error is connected with the phenomena of the thixotropic strengthening of the soils. Prof. B. M. Gumenskiy [20] focused his attention on the necessity of accounting for this circumstance. According to B. M. Gumenskiy's analyses, the ultimate shear stress $\tau_{\text{пред}} = \rho_a R$, for example, of samples of hydrous mica clay, after preparation amounted to 0.04 g/cm^2 after 2 hours of preparation, - 0.116 g/cm^2 after 4 hours and after 9 hours - 0.324 g/cm^2.

Thus, B. M. Gumenskiy recommends to begin the penetration tests only after the complete termination of the thixotropic strengthening of the prepared samples. Based on the materials of a series of tests for this, a holding of not less than 9-16 hours is necessary.

B. M. Gumenskiy's proposal is especially advisable during the study of the correlation between the physicomechanical properties of the soils. As for the procedure for determining the plastic limits, additional holding of the samples is not provided for in GOST 5183-64 and 5184-64. Thus, when using a method of two penetrations the technique of preparing the samples would not differ from the existing standards.

During the testing of samples with undisturbed structure cutting rings whose diameter is 71.4 mm and whose height is 50 mm are utilized, i.e., a volume of 200 cm^3. For samples with disturbed structure, prepared in the laboratory, the cutting rings 56.4 mm in diameter and 40 mm is height are used, i.e., a volume of 100 cm^3. These rings are supplied with a sharpened guide and an adapter for a height of 20 mm. The guide and adapter uniform packing of the samples during sampling to an assigned density.

Conclusion

The given theoretical premises and given examples provide a basis for considering that the recommended methods for studying the physicomechanical properties of soils by penetration, sounding and ratio shear, even during their actual development, deserve attention and in many instances can be successfully used with engineering geological investigations for the purpose of designing and building various engineering structures.

Penetration methods of studying the physicomechanical properties of soils make it possible:

1) to determine the properties of soils under conditions when the known traditional methods of research on soils turn out to be ineffective;

2) to organize parallel and mutually supporting research on the mechanical properties of soils both under field, and laboratory conditions. The doubtless advantage of such an organization of the work is the standarization of the utilized probes and instrumentation, the uniformity of the procedures and the methods of study;

3) to carry out the mass studies for the complete and comprehensive solution of the assigned task;

4) to insure the necessary and sufficiently high accuracy of the results of the conducted tests, and therefore, to insure the research on the characteristics of the mechanical properties of soils relative to a change in their physical state;

5) to objectively check the accuracy and reliability of the test data;

6) to evaluate the variability of the disclosed characteristics by methods of mathematical statistics.

For the successful realization of these and many other positive features of the recommended methods it is necessary:

to insure the incorporation of the already developed methods, technological methods and known instruments in the practice of engineering geological investigations;

to develop and improve the theoretical premises of the recommended procedures and methods of the studies;

to organize the development and release of new, highly productive field and laboratory instruments and installations.

The author hopes, that this text will facilitate the popularization of the new methods of engineering geological investigations and that is will turn out to be useful in the practical mastery of the methods of the study of the physicomechanical properties of soils by penetration, sounding and vane shear.

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