COMPARISON OF LABORATORY IN SITU SOIL TESTS

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Abstract. The paper describes methods for soil deformation and strength properties measurement, based on various technologies of laboratory and in-situ tests. Test results from one of the investigated sites are compared. Test techniques influence on the measured deformation and strength parameters values are discussed.

1. Introduction

The main objective of geotechnical investigations is delivery of data sufficient for quality designing of foundations of buildings and structures. Evidently, the safety of design solutions depends on both analytical methods and the quality of geotechnical investigations within the selected construction site. Also the quality of geotechnical investigations depends on many factors such as drilling and sampling techniques, preparation of samples, methods of laboratory and in situ tests and on the experience of geologists and on the requirements of codes and regulations that is equally important.

Applicable Russian standards (State Standards - GOSTs, Construction Codes – SNiPs) have recently been harmonized and actualized for international application. In fact, this procedure mostly consisted in rewriting former regulations (e.g. GOST 20276 [4]), in which some new statements were included with no international experience taken into account. E.g., Table 1 contains Eurocode 7 requirements [15] for the quality of cored soil samples while Table 2 contains the recommends methods of laboratory and in situ tests. Recommendations of Eurocode 7 in Table 1 are similar to those in Appendix E and Ж of СП (Construction Code) СП 47.13330 with the only difference in that this СП does not contain requirements to the quality of cored samples and sampling techniques.

As to Russian innovations, the main requirements to geotechnical investigations are introduced by the new СП 47.13330 in chapter 6 “Engineering Geological and Engineering Geotechnical Investigations”. In spite of its “novelty” it contains requirements from Chapter 6.3.5. The first among them “Methods of drilling shall ensure soil approbation and respective accuracy in establishing the boundaries of soil strata.” There is nothing about the quality of samples for determining their physical and mechanical properties. Soil stratification alone is determined. Then the next statement «… Application of auger and vibration drilling with sampling is allowed to support the engineering investigation of their sampling method …» that means again the impossibility of their application in practical auger and vibration drilling for sampling. This is explained by indistinct wording, and it is impossible to justify their application for an expert.

Table 1. Sample quality classes for laboratory tests and the applied categories for sampling cores.

Soil properties	1	2	3	4	5
Unchangeable soil properties					
particle size	*	*	*	*	*
humidity	*	*	*		
density, relative density, permeability, compressibility, strength.	*	*			
	*				
The properties that can be determined:					
sequence of strata:	*	*	*	*	*
strata boundaries	*	*	*	*	
strata boundaries – generally	*	*			
strata boundaries – specifically					
plastic limits, particle density, organic content	*	*	*	*	
humidity	*	*	*		
density, density ratio, porosity, permeability	*	*			
compressibility, strength					
	*				
Sampling class as per EN ISO 22475-1	A				
			B		
					C

Although ASTM 1452 [13] and ASTM 6151 [14] are applicable in the USA for sampling with solid and hollow augers, in Russia, so far, column drilling method is the most “popular” and is related to class C (Table 1) in Eurocode 7. The sampling C class means that the application concerns only permanent soil properties, to which grain composition may be related.

2. Objective of the paper

The objective of the paper is to demonstrate various methods for evaluating soil strength and deformation properties, now applied internationally while in the Russian Federation applied on a limited scale.

In Russia laboratory tests are mainly carried out as per State Standard ГОСТ 12248 [5]) while field tests are mainly performed by CPT (State Standard ГОСТ 20276). Dynamic penetration tests are less popular and, as a rule, only if CPT is not possible. Auger shear tests were outdated long ago. 5000 cm^2 flat test plates are not welcome by the Customer and are seldom applied. Screw plate tests are more frequent, if the Customer agrees, in spite of their mandatory application, stipulated by Construction Code СП 22.13330 [8].

In 2013 году ООО «НПП Геотек» (LCC R&D and Production Company "NPP Geotek") launched a soil investigation program within the bounds of its territory. The purpose was to create a template data bank of soil data for laboratory and field tests, performed by different methods. Later this data bank will be a geological model for verification of various subsoil analytical techniques and for comparison of new soil test methods with standard ones i.e., deformation and strength test methods, stipulated by respective standards [4, 5, 15].

The research program includes the following stages:

1) soil sampling down to 20 m; so far the tests were done down to 12 m depth;
2) soil physical properties assessment for geological cross sections, according to soil classification;
3) soil strength and deformation properties determination in laboratory by compression, by one-plane shear and by tri-axial compression;
4) soil stratification to 20 m depth with static, dynamic and drilling penetration tests;
5) screw plate and continuous auger tests to determine soil deformation modulus and non-drained clay soils strength;
6) 600 - 5000 cm^2 flat plate tests on a geotechnical element;
7) high-stiffness dilatometer tests;
8) laboratory and field tests data correlations for determining soil strength and deformation properties.

3. Test methods

Sampling was done manually to 1,5 m depth and then with a sampler at great depth.

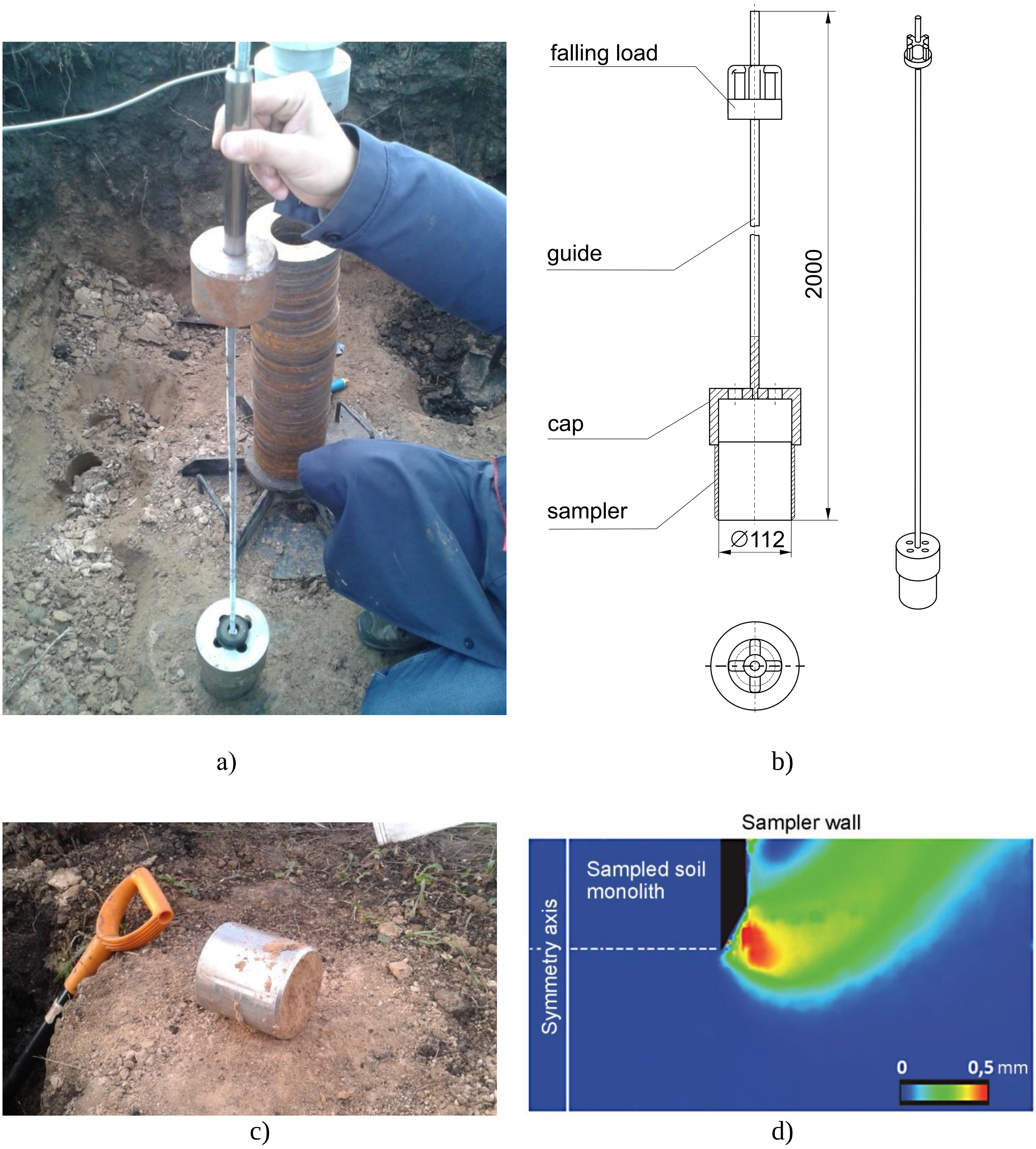


Fig. 1. Coring of soil monoliths with a sampler

Fig. 1 shows a device, developed for sampling in pits during survey of footings or for soil investigation in the bottom of excavation.

The device consists of a replaceable sampler, a cap, a guide and a 2 kg falling load. (рис. 1б). The sampler dimensions enable sampling for the whole complex of laboratory tests: uniaxial test, compression test, one-plane shear, tri-axial compression. The angle of the cutting edge and the wall thickness enable sampling with minimum influence on in-situ soil. In order to evaluate this event a PIV (photo imaging velocimetry) experiment was staged [20]. Fig. 1e shows shows that shear deformations appear at the cutting edge then propagate into the soil mass, but do not destroy the sampled soil monolith.

Soil monoliths sampling procedure with a sampler is, as follows:
1) preparation of the surface;
2) pre-lubricated sampler installation;
3) sample aligning with a level gauge;
4) cap mounting;
5) sampler settlement with a falling hammer;
6) soil is cut and extracted;
7) packaging.

The laboratory tests complex complies with ГОСТ 12248 [5], ГОСТ 5180 [2], ГОСТ 25100 [7], ГОСТ 20522 [6] and includes:

1. Compression tests with static and kinematic loading and stress relaxation mode test.
2. Consolidated-drained test in one shear plane conditions.
3. Consolidated-drained tri-axial test for a given compression path.

Field test complex, manufactured in accordance with ГОСТ 19912 [3] and ГОСТ 20276 [4]:

1. CPT.
2. DPT.
3. 2500 and 5000 cm^2 flat test plates.
4. 600 cm^2 screw plate test plate

Simultaneously there were conducted tests, based on application of little known methods such as drilling penetration and stiff dilatometer tests.

4. Engineering geological conditions and physical parameters

There were drilled three holes on the site to 12 m depth, soil monoliths were cored by rotary drilling and a set of laboratory tests was performed in order to determine soil physical and mechanical parameters. Tables 2, 3 show the obtained physical parameters, one of which is shown on Fig. 14.

Table. 2. Physical parameters

Name as per ГОСТ 25100	In situ relative water content, W, %	Soil density, ρ g/cm^3	Void ratio, e	Water saturation ratio, S_r	Plasticity index, I_p	Liquidity index, I_L
Clay loam semi-hard, EGE-2	21	1,86	0,752	0,74	14	0,01
Clay semi-hard, EGE-3	21	1,96	0,683	0,84	20	0,18
Clay stiff, EGE-4	23	2,00	0,674	0,93	20	0,34
Medium grain medium density water saturated sand, EGE-5	19	1,98	0,601	0,85	-	-

Table 3. Sand grain size composition

Over 10 mm	10 - 5 mm	5 - 2 mm	2 - 1 mm	1 - 0,5 mm	0,5 - 0,25 mm	0,25 - 0,10 mm	0,10 - 0,05 mm	0,05 - 0,01 mm	0,01 - 0,005 mm	меньше 0,005 mm
9,3	5,0	2,1	6,0	10,9	32,9	17,7	16,1	-	-	-

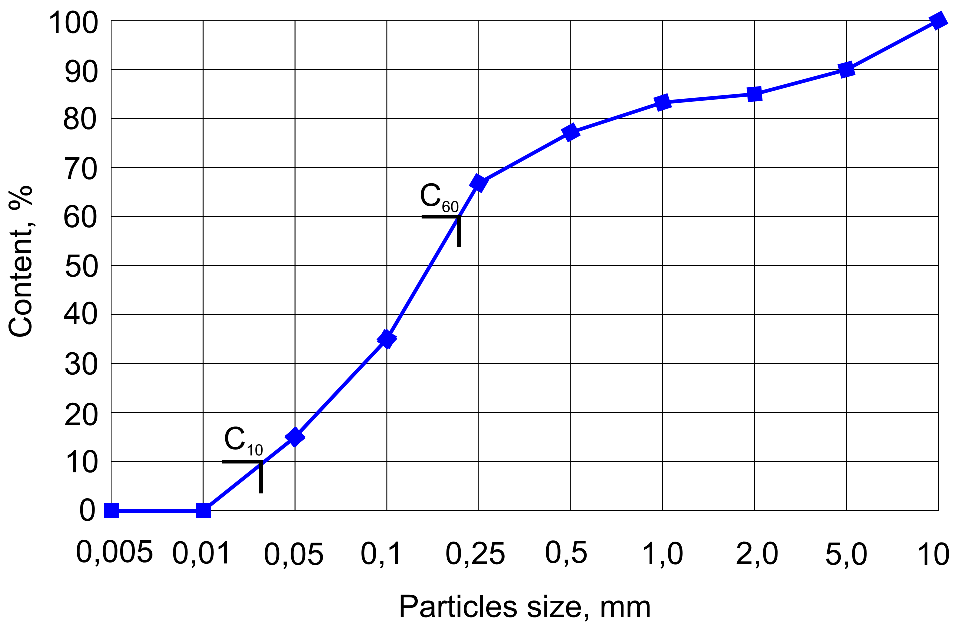


Fig. 2. Cumulative grain size curve

5. Laboratory soil tests

All tests complied with requirements of State Standards ГОСТ 25100, ГОСТ 5180, ГОСТ 12248 and accounted for formation history of sediments in some cases.

5.1. Compression tests

The paper presents only the results of compression tests with static (stepwise) loading, providing arbitrary loading type i.e., stepwise, kinematic and with stress relaxation (рис. 3а). The test data for the case of continuous loading will be published later in a separate paper.

a) b)

Fig. 3. Soil samples test facilities soil: a - compression, b – one-plane shear

The undisturbed samples tests enabled determination of secant compression deformation modulus (E_k) within up to 600 kPa pressure range. Then compression deformation modulus values within 100-200 kPa range follow, based on the fact that this range was applied for evaluating coefficient of transition to plate test deformation modulus (see Table.5.1 Code of Practice СП 22.13330). This value was also compared with values from other test methods in order to determine the deformation modulus. Notably, the international practical experience of determining correlation dependences employs oedometer modulus rather than the compression deformation modulus (E_{oed}). E.g., EN 1997-2:2007 has an equation for oedometer deformation modulus evaluation from CPT:

$$E_{oed} = \alpha q_c, \tag{1}$$

with q_c as measured cone tip resistance value. The transfer coefficient α depends on the soil type.

The Russian practice uses a similar equation (Trofimenkov and Vorobiov) [12], СП 47.13330 [9]), however, they relate it to normative (derived) deformation modulus, but what is its value? The notion "normative" value is clear from ГОСТ 20522, which states that the design (characteristic) parameter value is calculated by dividing the parameter with a safety factor for soil, which is equal to unity for the derived (normative) value. Meanwhile, it is valid for any parameter such as compression, oedometer or tri-axial deformation modulus. Therefore, the reference to normative (derived) value does not interpret the assumed deformation modulus value.

Fig. 22 shows variation (profile) of compression module versus depth.

Compression in the compression tests was applied for determining the pre-consolidation pressure (σ'_p), the over-consolidation ratio (*OCR*). $OCR=\sigma'_p / \sigma'_{v0}$ characterizes the formation history of soil deposits and is defined as maximum historical pressure σ'_p ratio over the existing soil proper weight σ'_{v0}.

OCR is used to classify clay soils, which are grouped by their values in normally consolidated (*OCR* = 1) and over-consolidated (*OCR* > 1). Notably, this parameter is not included in ГОСТ 25100 as a classification index. Classification of soils as normally consolidated and over-consolidated enables to determine deformation and strength parameters more correctly, because in this case laboratory tests account for the history of formation.

Typical of clay loam compression test data for the first engineering geological element (EGE) is given on Fig.4.

The mean value of three measurements of pre-consolidation is σ'_p = 129 kPa, which exceeds the soil weight stresses almost six times. *OCR = 6*. The clay loam is evidently over-consolidated. The pre-consolidation profile is shown on fig. 24.

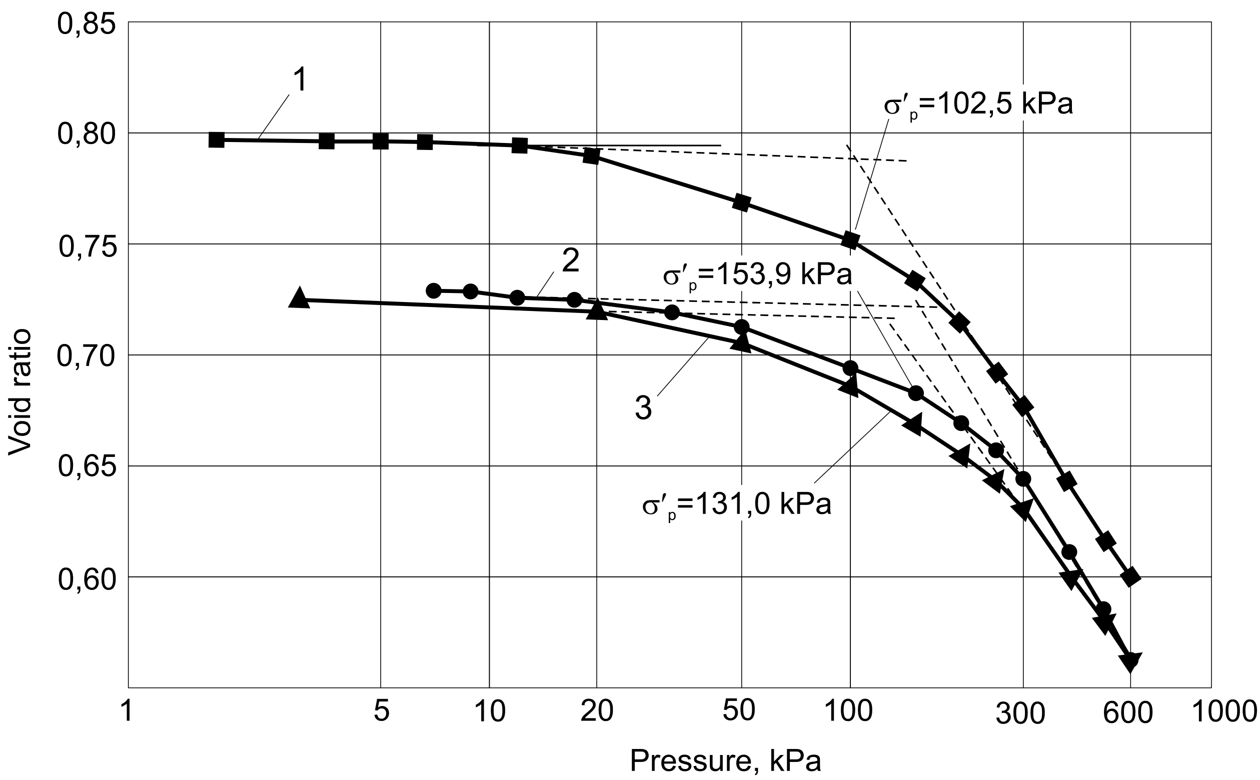


Fig. 4. Compression curves in semi-logarithmic scale.

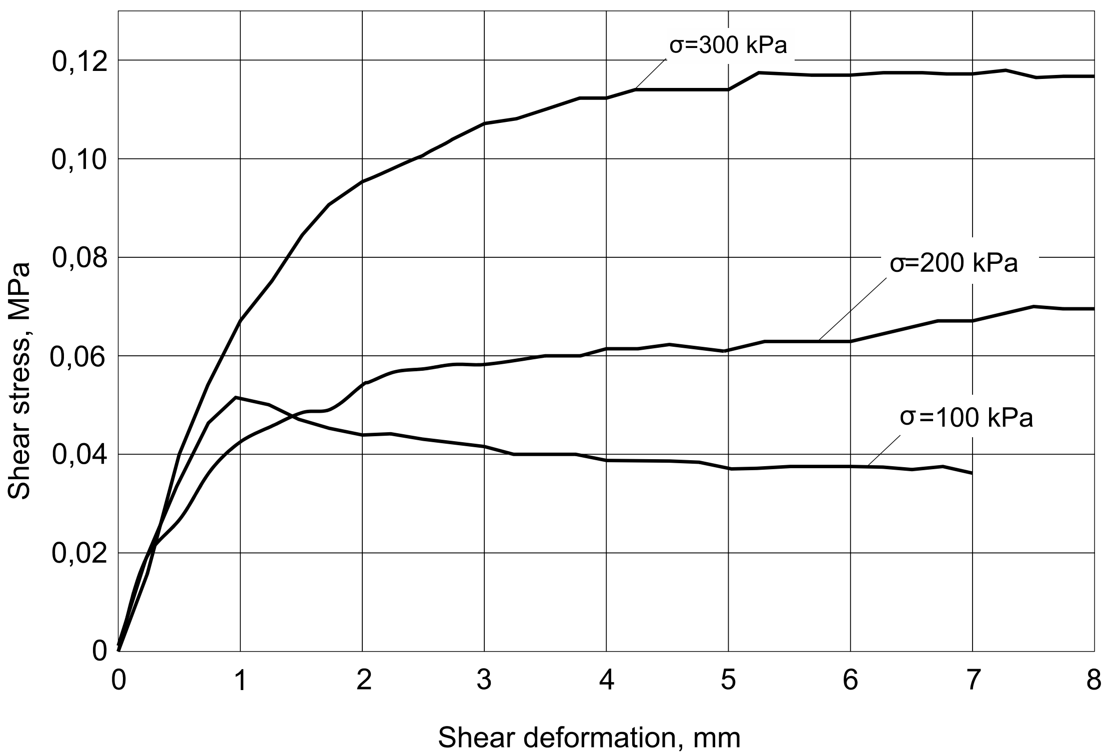
Table 6 presents oedometric modulus values versus normal pressure. As is evident from Table 6, the oedometric modulus grows with normal pressure growth and differs in that from tri-axial deformation modulus.

5.2. One-plane shear tests

These tests enabled determination of drained strength parameters φ and c. During consolidated drained tests, the formation history of investigated soils with $OCR > 1$ was applied. Soil was compacted in a pre-compaction device at σ'_p normal pressure stage as per Table 5.1 of ГОСТ 12248. E.g., for clay loams this Table recommends the following normal pressure (σ) values: 100, 200 and 300 kPa. The pre-compaction pressure was σ'_p = 90 кПа. In this case the pre-compaction device normal pressure was, as follows: $\sigma = 100 + \sigma'_p$; $\sigma = 200 + \sigma'_p$; $\sigma = 300 + \sigma'_p$

Fig. 5 demonstrates an example of stiff clay loam typical test data. For comparison, the test followed the compression pressure as per ГОСТ 12248 with the account of the pre-compaction compression pressure value, obtained from compression tests. The limit direct line on Fig.5 originates from shear test data with values of normal pressure, recommended by ГОСТ 12248 while 2 characterizes the soil strength, accounting for soil in situ compaction. Internal friction angle values are practically unchanged while specific cohesion is three times different.

a)



b)

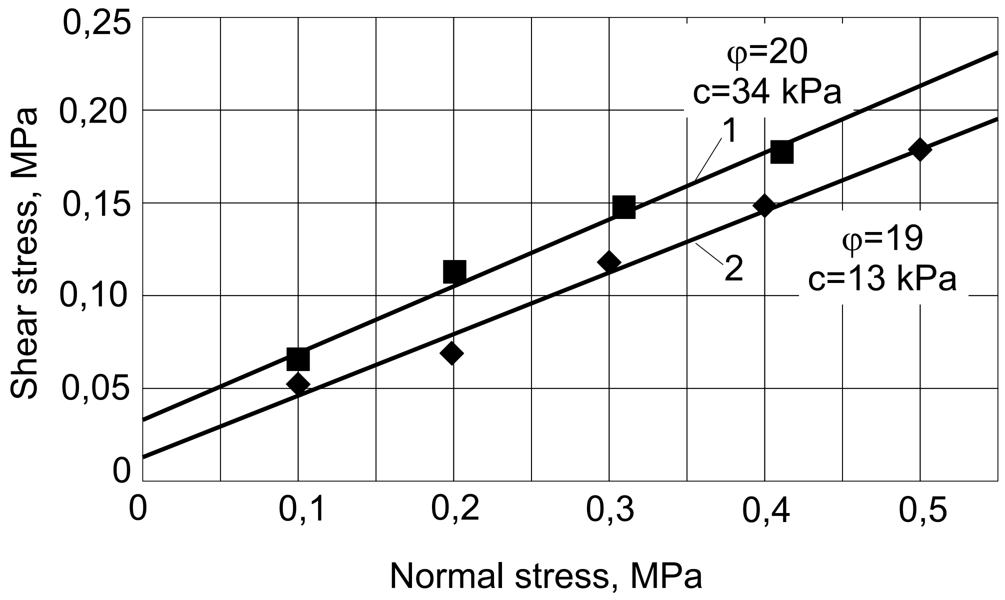


Fig. 5. Consolidated-drained shear: "a" as deformations, generated by shear stress; "b" as Coulomb limit straight line; 1, 2 with and without pre-compaction pressure taken into account, respectively.

5.3. Tri-axial compression tests

The tests were done in a tri-axial apparatus, allowing for static and kinematic tests (Fig. 6).

The static loading tests enabled determination of drained strength φ and c, initial and secant deformation moduli. Over-consolidated clay samples were re-consolidated by hydraulic pressure, equal to pre-compaction pressure. When strength parameters of over-consolidated clays were determined, the consolidation pressure value was taken from Table. 5.6 (GOST 12248) with its formation stress history taken into account. The results of consolidated-drained tests of the clay loam are given on Fig.7 and Fig. 22b.

Fig. 6. Triaxial compression apparatus

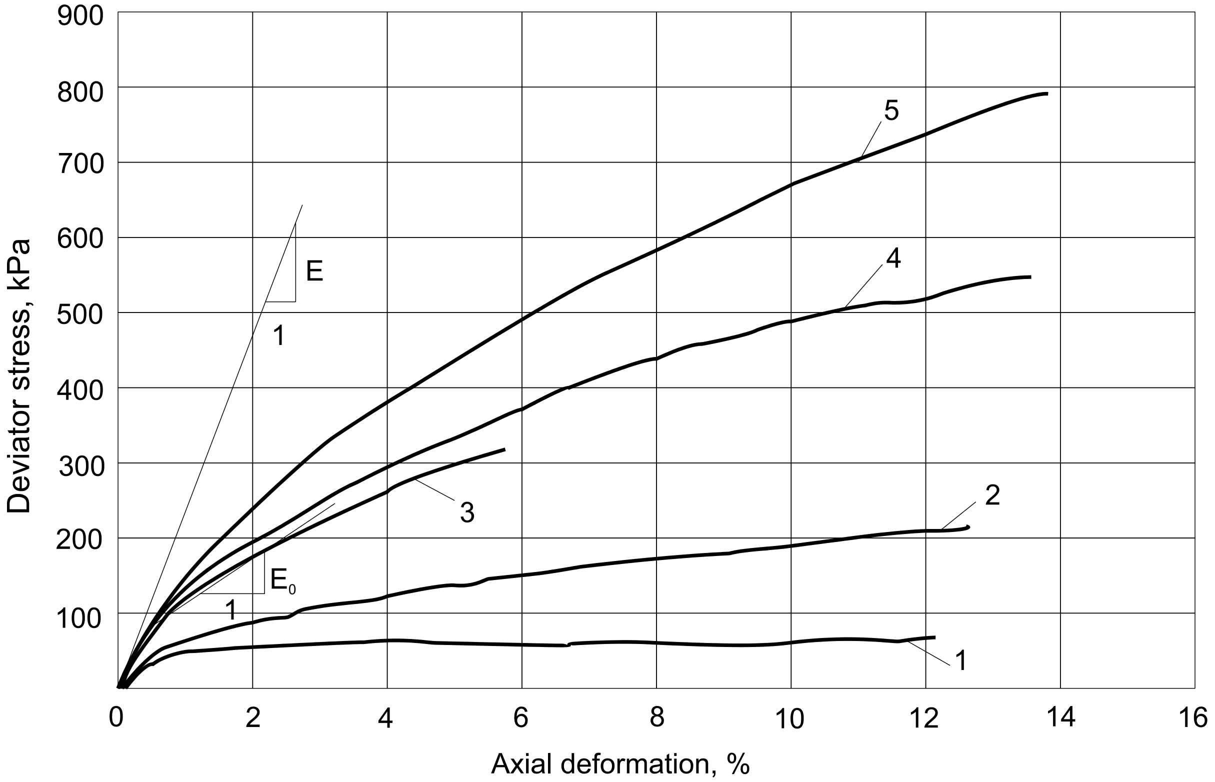


Fig. 7. Consolidated-drained tri-axial compression, axial deformation dependence on axial stress; hydrostatic pressure 1 – 15 kPa; 2 – 105 kPa; 3 – 200 kPa; 4 – 300 kPa; 5 – 600 kPa

Table 6 shows deformation modulus values. Initial (elastic) modulus (E) is determined as tangent at the initial portion of axial stress - axial deformation curve (Fig. 7) while deformation modulus (E_o) is determined as secant for respective intervals of the maximum principle stress.

6. Soil in situ tests (Field soil tests)

Field soil tests comply with standard techniques of ГОСТ 19912, ГОСТ 20276 (CPT and DPT, flat and screw plate tests) and two “forgotten” Russian methods: drilling and flat dilatometer tests. In order to harmonize Russian and international techniques it is proposed to introduce the following classification of field test methods.

Field soil tests names are shown as international abbreviations, as follows:

- static penetration tests without pore pressure measurement as CPT (Cone Penetrometer Test); the one with pore pressure measurement as CPTu; the one with elastic waves velocity measurement as SCPTu (Seismic Cone Penetrometer Test);
- dynamic penetration test with a sampler as SPT (Standard Penetration Test);
- Marchetti dilatometer as DMT (Dilatometer Test) and SDMT (Seismic Dilatometer Test);
- pressure meter as PMT (Pressuremeter Test);
- flat plate test as PLT (Plate Load Test);
- Swedish vane plate as WST (Weight Sounding Test);
- auger shear – VST.

In Russia three methods are used, which differ from international ones, therefore, the following abbreviations are proposed:

- винтовой штамп – RST (Russian Screw Test);
- жесткий дилатометр – RBT (Russian Blade Test);
- drilling penetration – RDT (Russian Drilling Test);
- dynamic cone penetration test RSPT (Russian Standard Penetration Test).

Such classification is useful for advertising Russian national methods of field soil tests.

6.1. Static penetration tests

Standard penetration test (CPT) is currently the most popular field soil test, applied practically in all geotechnical investigations, because of its low cost, speed (1-1.5 hour per 30 m), large data volume. The standard test procedure envisages application of a system for continuous penetration of a cone, using drilling rods 1.0=1.5 m long with 20 mm/s.

The tests with a complete set of sensors (for measuring force, lateral pressure, pore pressure, tilt, acceleration) e.g. SCPTU enables determination of the following soil properties and parameters:

– soil type;

– soil strata thickness;
– undrained strength of cohesive soils c_u;
– undrained angle of internal friction φ;
– shear elastic module G;
– relative density ratio I_D;
– over-consolidation coefficient OCR;
– lateral pressure coefficient K_o;
– consolidation ratio in horizontal direction c_h;
– filtration coefficient k_f;
– parameters of pile bearing capacity;
– parameters of settlements and bearing capacity of shallow footings;
- parameters of water-saturated sand soils liquefaction.

These investigations involved a standard tensometer probe (CPT), designed by LLC «Geotest» (www.geotest.ru) without pore pressure measurement. The probe enables measuring the following parameters:

- *tip resistance* q_c. Force F_c, applied to the cone, is measured and divided by the cone projection area A_c:

$$q_c = \frac{F_c}{A_c}. \tag{2}$$

CPT application experience demonstrates that for sand q_c tip resistance correlates with angle of internal friction φ', relative density I_D and with apparent lateral pressure of soil weight σ'_{ho}. In clay soils q_c relates to undrained strength c_u and apparent compaction pressure σ'_p.

- *friction over probe lateral surface* is determined as ratio of measured axial force F_s over the friction sleeve area A_s:

$$f_s = \frac{F_s}{A_s}. \tag{3}$$

Friction resistance relates to the tip resistance (Lunne et al. [17]) by applying friction ratio

$$FR = \frac{f_s}{q_t} \times 100,\%. \tag{4}$$

High values $(3-4\% < FR < 10\%)$ are specific for clay soils due to high cohesion and low friction $(FR < 1-5\%)$ for sands and low water content clays.

Tip resistance, friction forces and friction ratio are shown on Figs. 19 a,b,c.

6.2. Dynamic penetration of soils

This penetration test is conducted in order to determine soil resistance by means of a steel cone and a steel cylinder dynamic penetration into soil, in the latter case disturbed samples are cored for soil classification. Such technique is called

"SPT" worldwide. SPT tests are performed in order to determine soil resistance by means of dynamic penetration of a steel cone or cylinder into soil, in the latter case disturbed soil samples are cored to classify soil. This test technique is abbreviated as «SPT».

In the USA, the SPT-tests comply with ASTM D 1586, AASHTO T 206 requirements, in the European Union ISO 22476-3 and Eurocode 7 (EN 1997-2).are applied. In Russia, the tests comply with the requirements of ГОСТ 19912.

Dynamic penetration tests require heavy hammers and conventional drilling rigs with suspended equipment. Fig. 8 shows a drilling rig with a GEOMASH (ГЕОМАШ) suspended automatic hammer for testing.

Dynamic penetration tests involve continuous driving the probe into soil with the help of falling or vibrating hammer. The probe penetration depth is measured versus the specified hammer blow count (In dynamic vibration penetration tests probe penetration rate is measured). The blow count in dynamic penetration depends on the penetration depth per one blow count (10-15 cm). The blow count is assumed, depending on the soil composition and condition within 1-20 blows range, The measured data is applied to calculate tentative soil dynamic resistance:

$$p_d = \frac{AK_1K_2n}{h}, \tag{5}$$

with A as specific penetration energy, K_1 as energy loss ratio in one hammer blow

Fig. 8. Dynamic penetration tests with mounted "GEOMASH" equipment

on an anvil and elastic energy of rods; K_2 as energy loss ratio for rods friction against soil, determined depending on the force for the rods rotation; *h* as penetration depth; *n* as number of blows in a count..

The calculated values are used to plot a stepped graph of conventional dynamic resistance variation versus probe depth. The results of such tests are presented on Fig. 19 d.

GOST 19912 recommends dynamic penetration tests, combined with other types of geotechnical investigation for solving the following tasks:

а) delineation of geotechnical elements (thicknesses of strata and pockets, extension boundaries of soils of different composition and condition);

б) evaluation of spatial variability of soil composition and properties variability;

в) determination of rock and coarse-grained soil strata top depths;

г) quantification of physical mechanical parameters of soils (density, deformation modulus, angle of internal friction and cohesion, etc.);

д) determination of soil compaction and strengthening versus time;

е) selection of the sites for tests and sampling for further investigation of soils physical and mechanical properties..

In fact, this GOST (State Standard) enables solution of the first three problems. Soil physical and mechanical properties evaluation techniques, soil compaction and strengthening assessments are missing. Notably, the previous edition of this ГОСТ 19912 in item «г» (“d”) states: «rough estimate of soil physical and mechanical properties was not given» i.e., quantitative evaluation was not given.

As per i. 4.5 of GOST 19912 the quantitative evaluation of physical and mechanical soil properties should be done, based on statistically proved dependencies between indices of soil resistance to probe penetration and the values of soil parameters, determined with the help of other standard techniques. This requirement is similar to the requirements of EN 1997-2 [15], in which the characteristic values of soil mechanical parameters are received from field and laboratory tests correlations. Moreover, EN 1997-2 stipulates that the monoliths, sampled in dynamic penetration, are allowed for grain size composition of soils that corresponds to class 5 of monoliths quality (see Table 1).

6.3. Flat plate tests

The main advantage of this type of tests is that they are done deep in the soil massif. However, stiff plate tests require their meticulous installation in soil so that the whole bottom would be in contact with the soil surface. This requirement is difficult to achieve when tests are done down in holes. Therefore, for depths over 3 m the deformation modulus tests are done with screw rather than flat plate.

The main disadvantage of field plate tests is the necessity to use complicated loading systems with 25 ton and more load. Practically different types of support systems are used: anchor screw piles with a crossbeam, supported by the pit walls or by the loading platform. In the case of flat plate soil tests in holes it is very difficult to clean the bottom of hole, but due to smaller test plate area there is no need to apply complicated support systems. The drilling rig proper weight is sufficient in most cases.

a) B)

Fig. 9. 5000 cm^2 plate test: a – overview; б – test plate in the loading system: 1 – displacement sensor; 2 – load sensor; 3 – electro-mechanical drive.

The value of deformation modulus depends on the size (area) of the test plate [10]. Results of deformation modulus evaluation, obtained with 5000 cm^2 E_{5000} and 600 cm^2 E_{600} test plates, are in Table 4. E_{5000}/E_{600} values scatter is 1,2 – 2,0, this is the evidence of soil proper weight impact on the test plates with such areas.

Table 4. E_{5000} / E_{600} ratios

Genetic soil types	Values of $m = E_{5000} / E_{600}$ for e		
	e = 0,4–0,7	e = 0,7–1,0	e > 1,0
Alluvial	1,25	1,50	1,75
Deluvial	1,90	2,00	2,10
Eluvial	1,20	1,40	1,60

Evidently, such problem of transfer coefficients will be essential in other test methods with transfer coefficients evaluation for other field test data in order to receive deformation modulus values, corresponding to E_{5000}..». This module (E_{5000}) is a “template” and is recommended for subsoil analysis of structures of the first and second level of importance (Code of Rules СП 22.13330).

The value of deformation modulus is:

$$E = \frac{(1-\nu^2)\omega d \Delta p}{\Delta s}, \qquad (6)$$

with ν as Poisson ratio equal to 0,3 for sands and sand loams, 0.35 for clay loams and 0.42 for clays; ω as dimensionless coefficient equal to 0,8 for circular plate; d as plate diameter; Δp as pressure increment on the test plate; Δs as test plate settlement increment, corresponding to Δp.

Notably, equation (6) stems from the solution of the boundary value problem for a stiff plate on perfectly elastic space (subsoil) for the case of no residual deformations. In reality, soil elastic behavior occurs only for very small settlements is difficult to measure in the field due to flexibility of the loading system. Therefore, the deformation modulus value, obtained from equation (6) is not elastic and therefore called deformation modulus. The value of deformation modulus could be several times less than that of elastic modulus. However, the name of soil modulus from plate tests is “elastic” worldwide. In some cases, e.g. in stiffness (compressibility) evaluations of roadbeds, including subsoil, this module has different names i.e., “equivalent elastic modulus”, because it characterizes summary compressibility of several layers of materials: asphalt concrete, ballast and embankment soil.

Fig. 10 shows a typical load-settlement curve from a test with plate depth equal to 1 m. The plate sits on top of the first stiff clay loam layer (EGE-2). Test data was processed with no account for soil proper weight stresses influence due to small surcharge value equal to 1,0x12,0= 12 kPa.

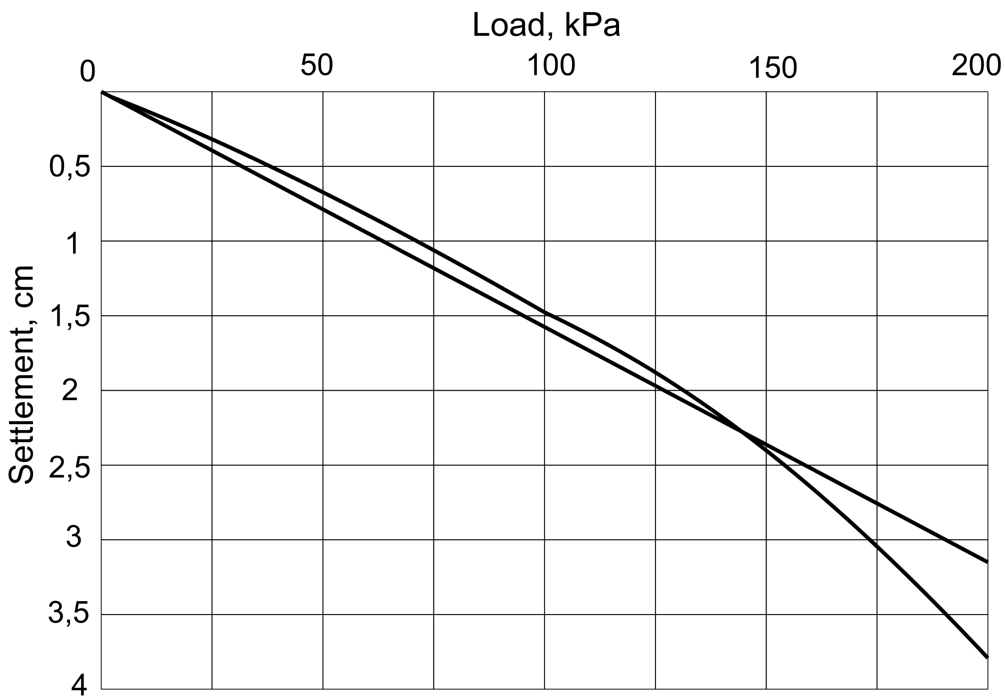


Fig.10. 2500 cm^2 test plate settlement versus applied load

Test plate deformation modulus within 0-150 kPa pressure range (secant line on Fig.10) is equal to 20,8 MPa. Oedometric deformation modulus within the normal pressure range 100-200 kPa is equal to 5,5 MPa, compression deformation modulus is equal to 3,3 MPa with Poisson ratio ν = 0,36. Experimental transition coefficient (m_k) is equal to 6,21. From Table 5.1 of (Code of Practice) СП 22.13330 with porosity ratio e = 0,752 get m_k equal to 3,0. Thus the table value of transition coefficient is underestimated times 2,07.

6.4. Screw plate tests

As per the State Standard “ГОСТ 2027 screw plate tests are conducted in order to determine soil deformation modulus, but could also be applied for determining undrained cohesive soil strength as is done with a flat plate test for determining undrained cohesive soils strength worldwide.

The screw plate is moved down below the bottom hole or frequently with no hole. In the tests in the hole the depth of screw plate below bottom hole shall be 30-50 cm depending on the soil type.

In the completed experiments the test plate was loaded with the help of an electrical mechanical device, enabling automatically control the constant rate of the assigned loading stage in the course of its settlement stabilization. The vertical load was measured with a load gauge and registration hardware, produced by LLC NPP “Geotek” (see Fig. 12). The accuracy of settlement measurements is 0,01 mm. Test plate settlement was measured as the mean arithmetic value of readings of three displacement sensors.

The test plate was loaded stepwise. The criterion of assumed stabilization was the test plate velocity below 0,1 mm during the time, specified in ГОСТ 20276. Fig. 11 shows a typical diagram of plate settlement versus load.

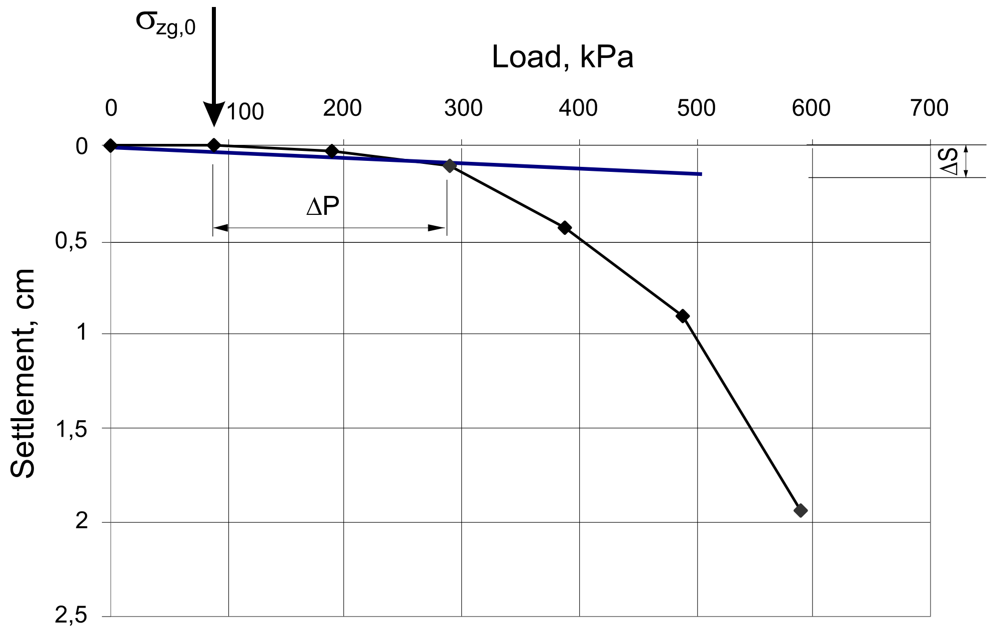


Fig. 11. Dependence of 600 cm^2 screw plate settlement versus load.

Soil deformation modulus E (MPa) is calculated for the linear portion of the diagram from equation:

$$E = (1-\nu^2)K_p K_1 D \frac{\Delta p}{\Delta S}, \quad (7)$$

with ν as Poisson ratio, equal to 0,27 for coarse soils; equal to 0,30 for sands and sand loams; equal to 0,35 for clay loams; to 0,42 for clays; K_p as coefficient, depending on the plate depth h/D (h as plate depth below soil surface, cm); D as the plate diameter, см); K_1 as coefficient, equal to 0.79 for stiff circular plate; Δp as the increment of pressure on the plate (MPa); ΔS as plate settlement increment, related to Δp, cm, of the averaging line.

Coefficient K_p is assumed to be 1 for soil plate tests in excavations, pits and pipes. For screw plate tests in drilled holes below the bottom and in the soil with no drilling coefficient K_p depends on h/D ratio value (ГОСТ 20276).

The calculated deformation modulus values are given on Fig. 22b.

6.5. Drilling soil tests

The method includes soil tests with some parameters measured [1] in the course of hole drilling with solid or hollow auger. In the latter case it is possible to sample soil monoliths (ASTM D 1452 and ASTM D 6151) for field and laboratory tests in order to find correlations between field and laboratory data.

The tests were performed, as follows. A hole was drilled with the help of a solid 135 mm diameter auger, with a three-piece drill bit (see Fig. 12) to the given depth. Then the auger was lifted 0.5 m and rotated idle several turns in order to remove friction forces between the auger and the soil. Then the auger was lowered to the bottom hole and screwed 20 cm lower, then the axial loading device was connected with the tailpiece of the upper auger.

In order to measure the auger settlement there were used three sensors, fixed on three plugs around the hole. The first pressure stage was equal to the weight of the soil above plus the weight of the augers. The deformation modulus was determined as per ГОСТ 20276 at the initial rectilinear segment of relationships on Fig.13. The plate area was 150 cm^2. The test data at various depths is shown on Fig.22б.

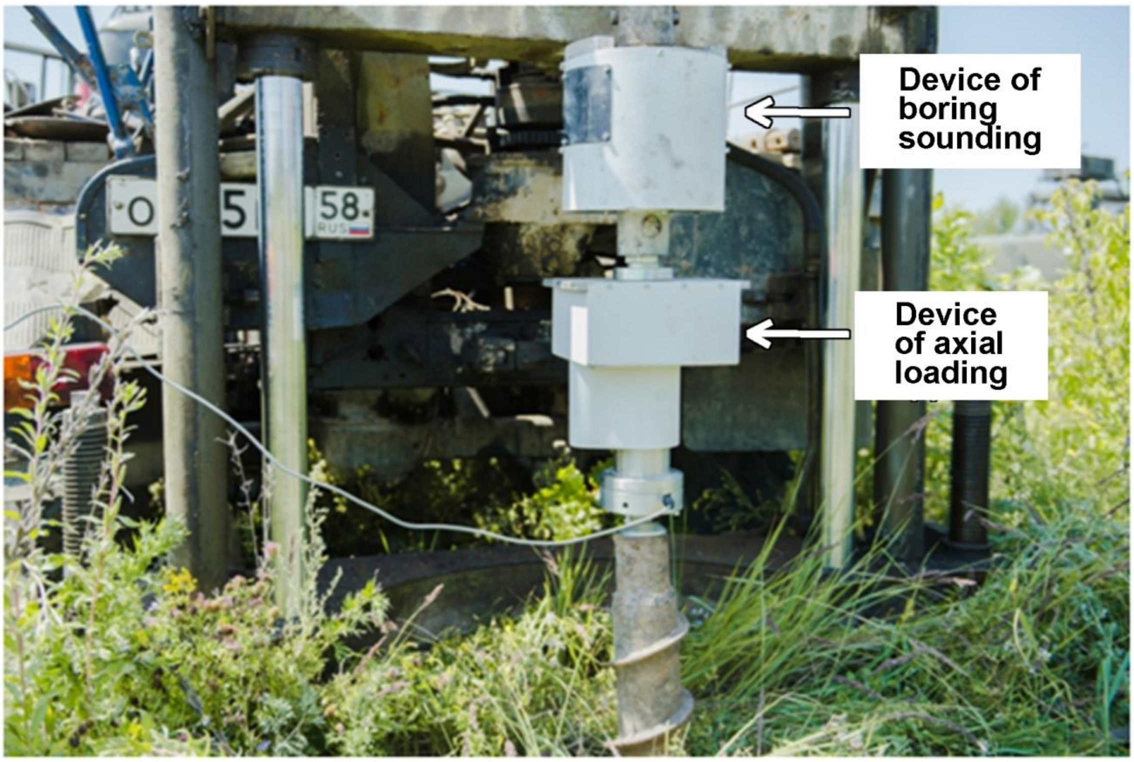


Fig. 12. The devices for registering drilling penetration and axial loading parameters.

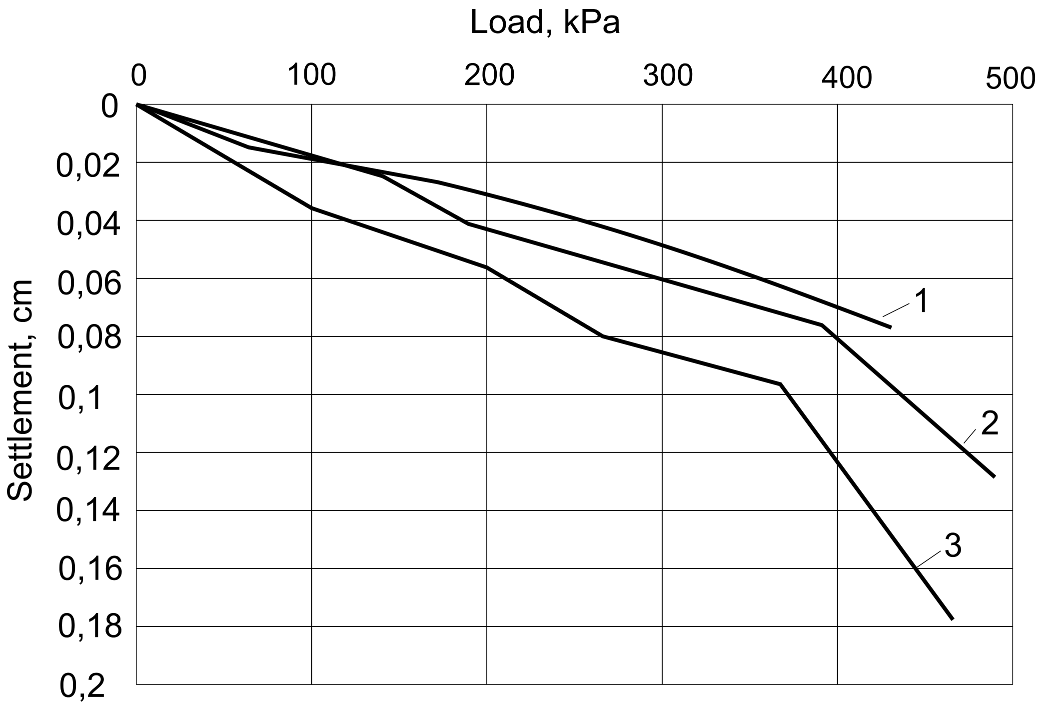


Fig. 13. Auger settlements versus pressure dependence at the depth: 1 – 3,0 m; 2 – 4,5 m; 3 – 5,0 m

Fig. 14 shows the results of soil penetration tests for different methods. As is seen on Fig.14 the parameters variation versus penetration depth is roughly the same. Also at 6-8 m depth drilling and static penetration identified a higher stiffness layer of clay soil.

6.6. Stiff probe tests

As different from Marchetti dilatometer, having a flexible membrane the measuring portion of the Russian dilatometer is stiff. This enables measurement in both gravely and clay soils and measurement of lateral (horizontal) in situ stresses σ_x, σ_y. After pushing the probe into soil to a given depth stress relaxation takes place (Fig.17) for 15-30 minutes for sand and 30-60 min for clay soils, thereafter the reference values of lateral pressure and pore pressure p_w are registered. The latter is possible in the case, when the probe is equipped with a pore pressure sensor. In the performed tests pore pressure was not monitored.

Stratigraphic index	Engineering Geological Element (EGE)	Absolute elevation	Depth	Thickness	Description of soils		Ground water table depth
solQIV	1	168,18	0,30	0,30	Top soil		No water
	2	166,48	2,00	1,70	Clay loam semi-hard		
	3	161,88	6,50	4,50	Clay stiff plastic		
	4	158,48	10,00	3,50	Clay yellow-brown semi-hard with mica content		
aQIV	5	157,48	11,00	1,00	Sand fine dark brown dense		

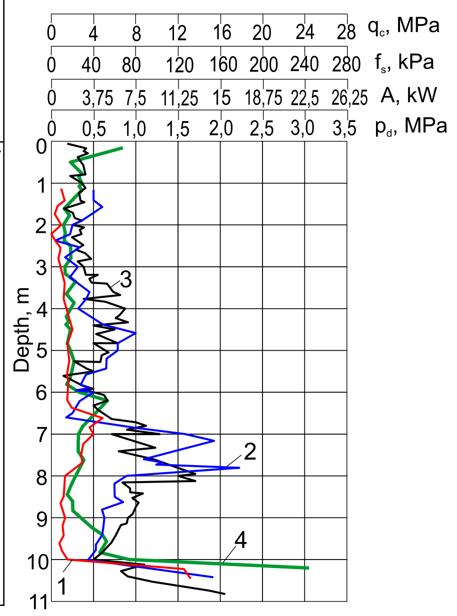


Fig. 14. Lithological column for borehole #1 (on the left) and penetration results (on the right): 1 – tip resistance; 2 – friction forces; 3 – drilling penetration; 4 – dynamic penetration

Stiff dilatometer design is different from that of Marchetti dilatometer, whose operating element is a thin steel membrane that expands under the pressure of liquid or air to a certain degree. In the stiff dilatometer the operating element is a pressure gauge, installed on the side surface of the probe.

The stiffness of the pressure gauge is dozens of times greater than that of soil. In V.V.Sidorchuk design [11] the pressure cells are hydraulic load cells of D.S.Baranov design. Because of high stiffness of the cells it is applicable in different soils, gravely soil inclusive.

Fig.15 shows a defect caused by the probe driving in fill. A 0.5 mm thick protection brass shell was broken off. However, this did not affect functionality of the probe.

Fig.15. Pressure cell protective shell damage

If Marchetti dilatometer had been used in this case then its operation membrane would have been broken. Therefore, Marchetti dilatometer applies to fine-grain homogeneous soils with no inclusions.

Fig. 16. Tests with a high-stiffness probe (RBT): a – general view; b – probe, equipped with one sensor

The manner of lateral pressures relaxation, following dynamometric probe penetration in clay soils shows the following:

– in clay soils the main portion of relaxation (85%) occurs in the course of 10-30 minutes;

– stress decay rate versus time ($K_з = \sigma_t / \sigma_{t_0}$) shows that during 30-55 minutes of exposure time $K_з$ varies within 44 – 83% range.

Table. 5. Stiff dilatometer test data

Depth, m	Test point number	Exposure time, minutes	Lateral stresses, σ_{ho}, kPa		$K_з$	Vertical stresses, σ_{vo}, kPa	Ratio $\sigma_{h0} / \sigma_{v0}$	Defor-mation modulus, MPa
			$\sigma_{t=0}$	σ_t				
1,2	1	54	176	102	0,58	24	4,25	4,45
2,2	1	46	203	168	0,83	43	3,91	7,32
3,2	1	53	219	169	0,77	63	2,68	7,37
4,2	1	47	441	284	0,64	82	3,46	12,38
5,2	1	54	515	227	0,44	102	2,23	9,89
6,2	1	46	517	245	0,47	120	2,04	10,68

– measured values of lateral (σ_{ho}) residual stresses in the end of the relaxation process, related to vertical in situ stresses (σ_{vo}) in clay soils are times 2,0 – 4,25 greater than in situ vertical stresses. Evidently, for clay soils on the site, having void ratio over 0.6, such considerable excess indicates that the soils were subjected to an external load;

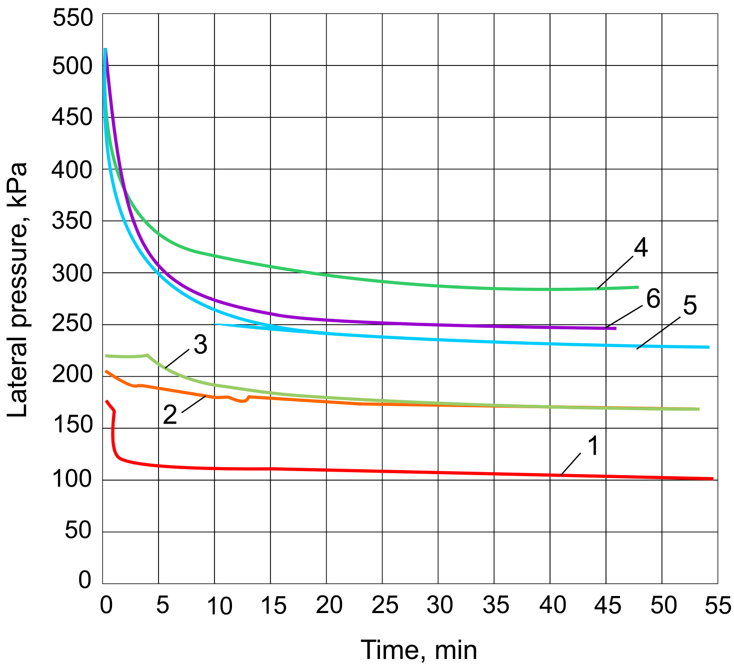


Fig.17. Relaxation of horizontal stresses: 1 – 1,2 m depth; 2 - 2,2 m depth; 3 –3,2 m depth; 4 – 4,2 m depth; 5 – 5.2 m depth; 6 – 6.2 m depth

– the distribution of measured residual lateral stress on Fig. 17 shows that these stresses grow versus depth especially starting from 3,2 m depth.

Lateral pressures graph versus depth is shown on Fig. 19e.

Deformation modulus is determined from solution, obtained by Z.G.Ter-Martirosyan et al. [11]:

$$E = \frac{\sigma}{1-\nu^2}\frac{h}{b}, \tag{8}$$

with σ as lateral stress after relaxation of stresses; ν as Poisson ratio; h and b as half-thickness and half-width of the probe respectively.

From Table 5 the values of deformation modulus as per equation (8) differ from the deformation modulus from plate tests. At 1,5 m depth the plate test modulus is equal to 20,5 MPa while the dynamometric modulus varies within 4,45 – 7,32 MPa range within 1,2 to 2,2 m depth range i.e., it is much less than the test plate deformation modulus. However, this modulus is negligibly different from compression modulus (see Table 6) equal to 4,9 MPa within 50-100 kPa range. This pressure range was adopted from horizontal stress equal to 102 kPa (see Table 5).

6.7. Generalization and interpretation of test data

Laboratory tests

During laboratory and field test data analysis the authors came across the fact that the determined deformation modulus value can be interpreted differently, depending on the test method employed.

EN 1997-2 [15] gives two recommended methods for determining compressibility parameters. The oedometer deformation modulus, denoted as E_{oed}, is determined from compression tests. The second method is tri-axial compression test, from which we obtain the initial elastic modulus, called “Young or elastic modulus *E*” in international publications. In Russia in ГОСТ 12248 it is called “deformation modulus” and is denoted as elastic deformation modulus *E*. In СП 47.13330 it is called “normative deformation modulus”. In СП 22.13330 it is called “deformation modulus”. However, depending on the deformation rate this deformation modulus may be either elastic or non-elastic. In the latter case, its name is “deformation modulus”. It is rather be called “tangent modulus”, which at small deformations coincides with the elastic deformation modulus. Hence, these moduli shall be denoted differently. Tsytovich N.A (1963) proposed to denote the elastic deformation modulus as E while to call the non-elastic one as modulus of total deformation *E*. Evidently, the return to the past would remove the indicated contradiction.

Also, ГОСТ 12248 method recommends to determine both oedometer deformation modulus and compression deformation modulus from the equation:

$$E_k = E_{oed} \times \beta, \quad (9)$$

with β as coefficient, depending on Poisson ratio.

The values of coefficient $\beta < 1$, hence compression deformation modulus is always less than the oedometer modulus, Table 6.

Fig. 7 shows that the linear dependence between deformation and stress is valid for deformations, not exceeding 1%, while the deformation, corresponding to soil failure stays within 8 to 15% range. As has been stressed before, the elastic modulus corresponds to small deformations, when soil behavior is “elastic” i.e., in the case in question it should be determined for deformations less than 1%. In order to differentiate it from compression deformation moduli it is better to assign it as E_{TX} with index TX, denoting tri-axial test conditions. As is seen from Table 6 te initial i.e., elastic tri-axial deformation modulus is equal to 19,9 MPa and is not equal to compression deformation moduli.

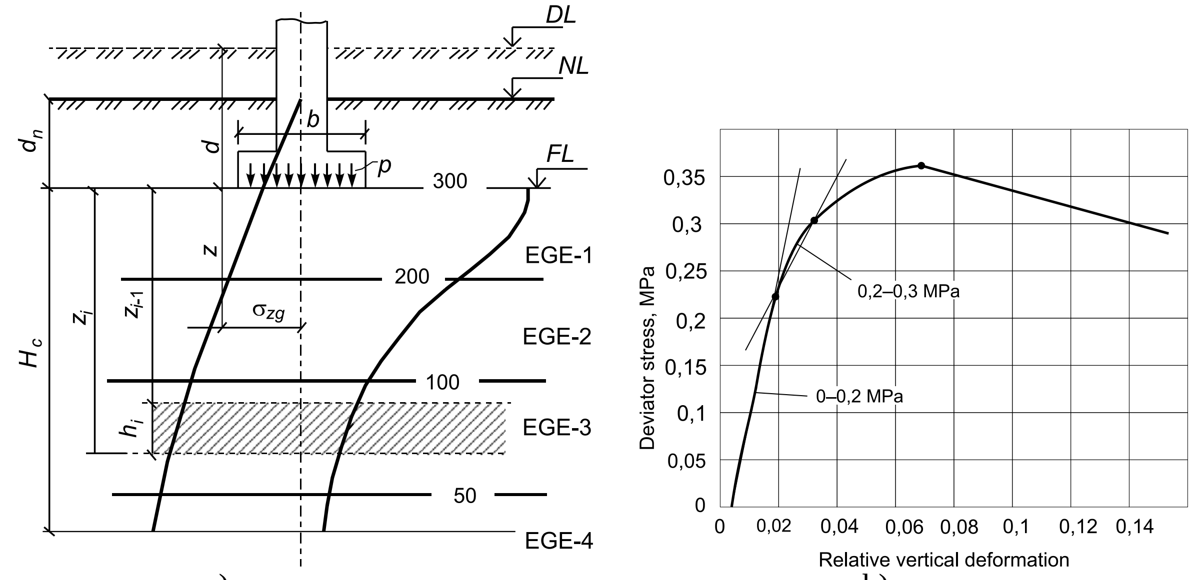
Table 6. Values of deformation moduli (clay loam, $h =$ 1,5 м)

Pressure range, kPa	Oedometric modulus, E_{oed}, MPa	Compression modulus E_k, MPa ($\beta = 0{,}6$; $\nu = 0{,}30$)	Tri-axial modulus, MPa E_{TX}	Test plate modulus, MPa E_{PLT}
0-50	3,6	2,1	19,9	21,0
50-100	8,2	4,9	9,9	
100-200	8,6	5,2	5,9	
200-400	10,9	6,5	3,8	

Deformation moduli, obtained from compression tests (E_{oed}, E_k) have smaller values than those, obtained from tri-axial tests (Table 6). There arises a question, which modulus should be a reference to other test methods. Table 6 shows that compression moduli grow if normal pressure increases, meanwhile tri-axial moduli decrease if stress deviator decreases.

Vertical stresses, caused by soil proper weight, linearly increase versus depth, which fact is due to the mean stress $\sigma_{cp} = (\sigma_1 + 2\sigma_3)/3 = \sigma_c = \gamma \times h$ growth, with γ as soil specific gravity, h as sampling depth; σ_c as hydrostatic pressure in the tri-axial compression apparatus. Because the tri-axial pressure, applied to the soil sample, is equal to vertical pressure from soil in situ weight, the initial elastic deformation modulus also grows versus depth. This is also evident from the fact that soil “stiffness” increases versus depth.

As deformation modulus value depends on the stress level, therefore, during tri-axial laboratory tests data processing its value shall correspond to the level of extra pressures in the subsoil of the designed structure. Fig. 18a shows soil proper weight stresses (σ_{vo}) versus depth profile and extra stresses (σ_{zp}) versus depth profile, caused by external load.



a) b)

Fig. 18. The settlement scheme (a) и data of triaxial tests (б)

In the center of each engineering geological element (EGE), loaded in situ with vertical pressure σ_{vo}, caused by external uniformly distributed load *p*. As is seen on Fig. 18a, soil proper weight generated stresses linearly grow versus depth while stresses, caused by the load, non-linearly reduce versus depth (Fig.18a). Therefore, in order to find the value of deformation modulus follow the procedure below.

1. Sample soil monoliths for each EGE at 2 m depth intervals. Assume each 2 m interval as an elementary soil layer for settlement analysis as per СП 22.13330.

2. Perform consolidated drained tests (GOST 12248) for each layer with soil monoliths, sampled at respective confining pressure $\sigma_c = \gamma \times h$. If soil water is present then hydrostatic uplift pressure into account shall be taken into account..

3. Evaluate additional stresses in soil for the given footing pressure as per СП 22.13330 and plot these stresses profile over the whole of investigated depth

4. Evaluate the secant deformation modulus for each σ_{zp} value (Fig. 18a) within the whole variation range at each depth and in each EGE.

E.g., EGE-1 has 200-300 kPa range; for EGE-2 the stress range is 100-200 kPa, etc.

5. The secant deformation modulus within the stress range from zero to the proportionality limit of dependence $\varepsilon_1 = f(\sigma_1)$ is equal to the elastic or initial deformation modulus (E). Subsequent values of deformation modulus within respective stress ranges are non-elastic and the soil compressibility has several moduli E_o of summary deformation

6. The pressure *p* on subsoil from the structure weight is predefince, therefore, the geological report summary table of soil physical and mechanical properties contains the elastic modulus value while a separate table contains the values of total deformation. The number of all the deformation values depends on the number of loading stages on the tested sample. Hence, the elastic modulus remains con-

stant at any investigated depth, this modulus shall be used for comparison with other test methods.

Strength properties of clay deposits depend on their formation history and on the type of stress state (via shear or tri-axial compression) that manifests in different friction angles and cohesion. Table 7 compares the data from direct shear test and triaxial compression tests. As is seen on Fig.56 and Table 7 the angle of internal friction in one-plane shear tests is 3-4^{o} less than in tri-axial compression tests. The history of pre-consolidation formation with the account of pre-compaction manifests itself in cohesion increase and does not affect the value of the angle of internal friction.

Table 7. Coulomb-Mohr strength parameters of semi-hard clay loam

Test conditions	Angle of internal friction, φ, degrees		Cohesion, c, kPa	
	Direct shear	Triaxial compression	Direct shear	Triaxial compression
Without formation history	16-17	20-21	16,7	21,6
Acconting for σ_p = 90 kPa		20-21		34-55
Accounting for σ_p = 130 kPa		21		70,2

Field tests

Figs. 19 – 24 show the results of the field tests as curves on. These curves show variation of some parameters along the depth of the investigated soil massif, these parameters were obtained by direct measurements: as CPT tip resistance and side friction; summary work in drilling tests; lateral stresses from soil weight. Other parameters are computed, using data and of direct measurement and respective equations.

This is friction ratio, conventional soil dynamic resistance, vertical stress from soil weight, deformation moduli, non-drained strength and pre-compaction pressure. The latter parameter was received from soil compression tests.

The data presented on Figs. 19 a,b,c,d are applied to delineate soil strata from the analysis of measured parameters variation (q_c, f_s, A, p_d) versus depth. The general trend of the parameters variation is roughly identical with the exception of dynamic penetration test data at 6-8 m depth. Fig.19 shows the most pronounced parameters of static and drilling penetration tests.

Fig.14 describes soil types, as stipulated in State Standard ГОСТ 25100, and penetration data, obtained by different methods. Evidently, all method show almost identical results, as is stipulated in ГОСТ 25100. However, at 3,5 – 5 m; 6,5 – 8 m depth the penetration shows presence of soils of different “stiffness”, which was not identified by conventional classification. It could rather be explained by selection of 1,0 thick sampling intervals, therefore, these interlayers were just omitted.

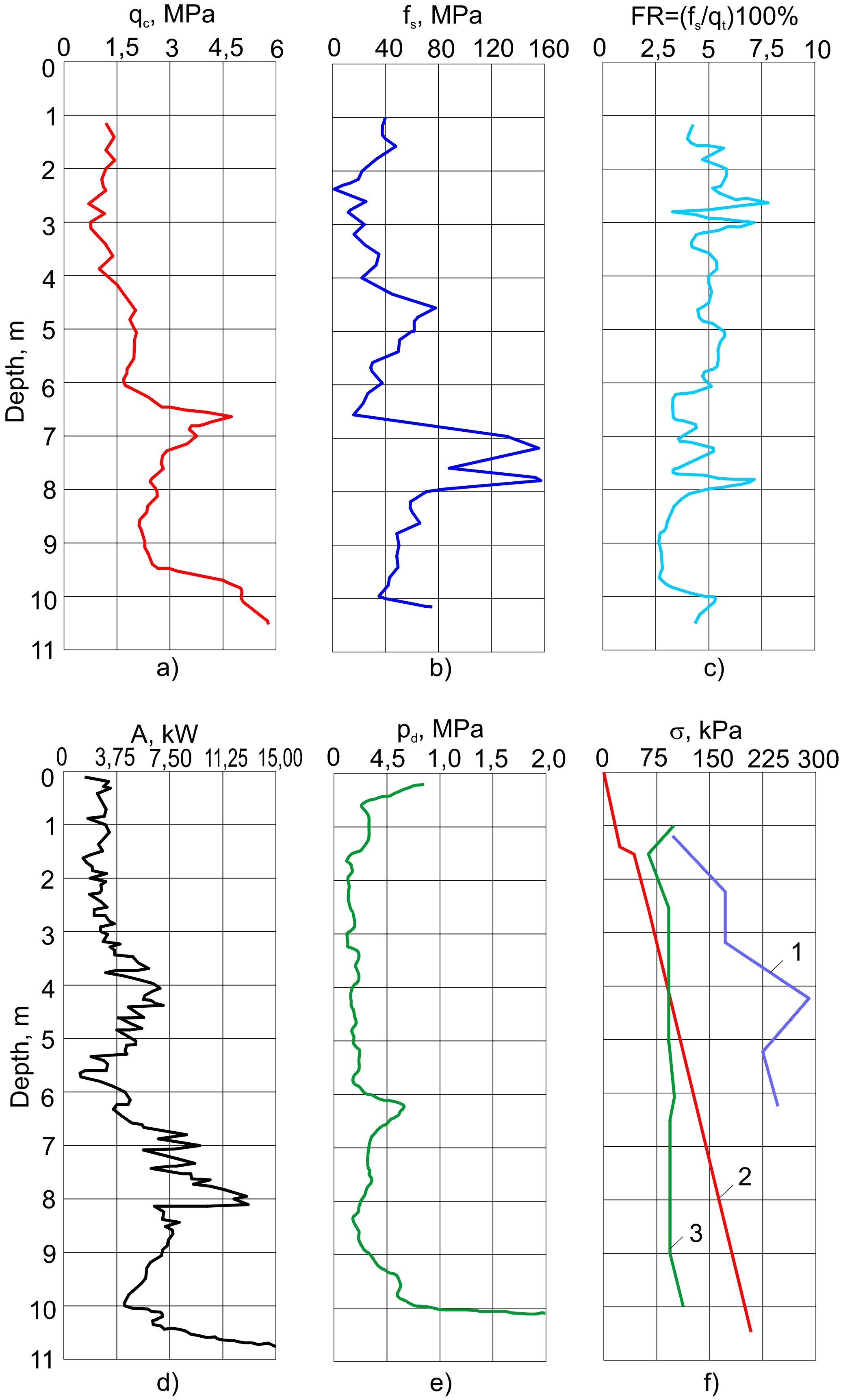


Fig. 19. Profiles, obtained with different field test methods: a – tip resistance (CPT); b – side friction (CPT); в – friction ratio (CPT); г – drilling penetration; д – dynamic penetration; e – dynamometric penetration; 1 – lateral pressure; 2 – soil weight stresses; 3 – pre-compaction pressure (compression) tests.

Figs. 20-21 and Table 8 show results of static penetration tests for soil type identification, proposed in by Robertston [21,22], Olsen and Mitchell [19]. The input parameters on the nomograms are tip resistance and friction forces. There are other similar nomograms, proposed after 1990, but they require pore pressure measurement done in this research. Supposedly, additional measurement that of pore pressure yields better results.

Several researchers developed nomograms for identifying the soil type, two of these nomograms are shown on Figs. 20-21. On these diagrams the bold type marks the values, obtained with the help of tip resistance and friction ratio evaluation. The Robertson nomogram (1990) needs computation of other related parameters [21] and of other earlier ones. The difference consists in tip resistance and friction forces normalization by vertical stresses, caused by the soil proper weight. This enables application of the nomogram for any depth.

$$F_r = \frac{f_s}{q_t - \sigma_{v0}} 100\% \tag{10}$$

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \tag{11}$$

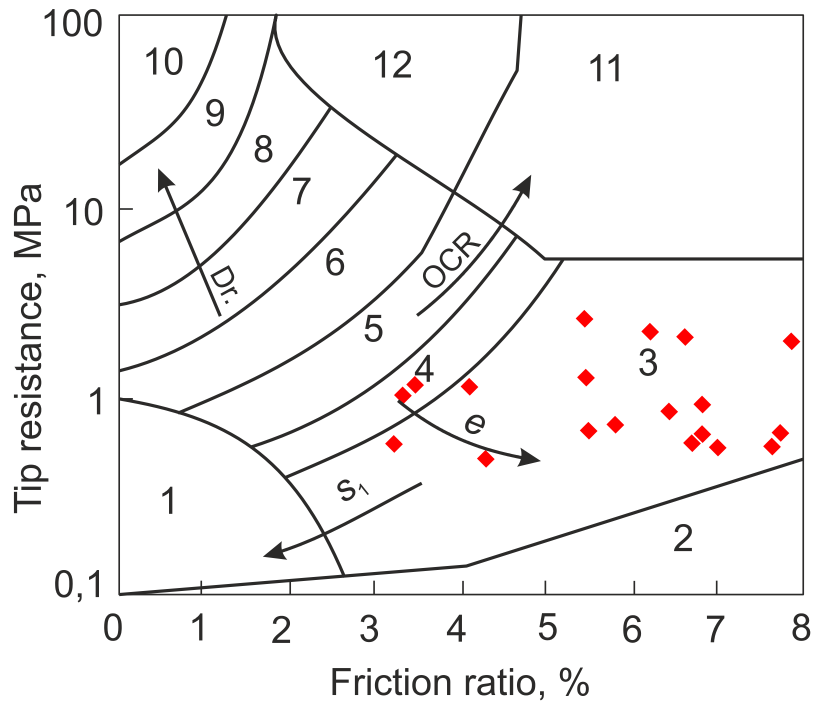


Fig. 20. Soil classification versus depth in 0,6 to 6,5 m range, by Robertson et al. [20]: 1 – sensitive finely dispersed; 2 – organic mineral soils; 3 – clay; 4 – clay loams; 5 – sand loams; 6 – silty and sandy loams; 7 – silty sands and sandy loams; 8 – silty sands; 9 – sands; 10 – gravely and coarse-grained sands; 11 – firm clays; 12 – over-consolidated soils

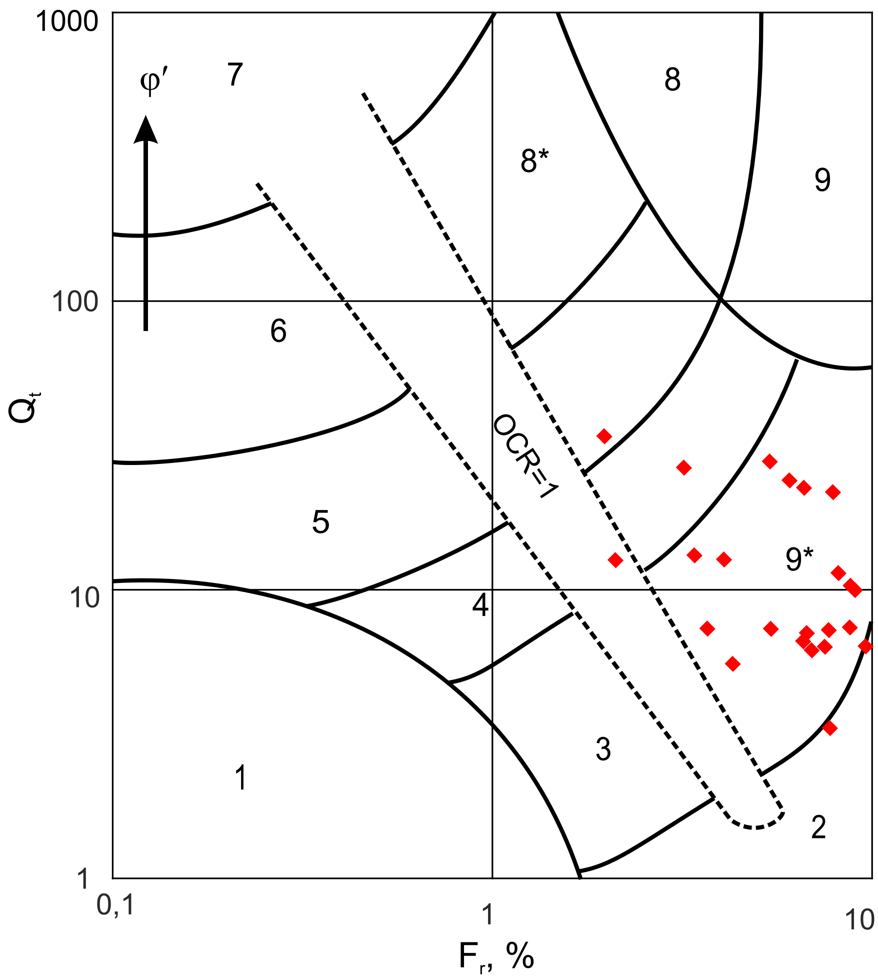


Fig. 21. Soil classification versus 0,6 to 6,5 m depth, by Robertson et al. [21]; legend: 1 – fine-dispersed soils; 2 – organic mineral soils; 3 – clay; 4 – clay loams; 5 – sand loams; 6 – sands and silty sands and sandy sand loams; 7 – gravely and coarse-grained sands; 8* –stiff sands; 9* – stiff clays (* – highly over-consolidated or cemented).

Equations, proposed by Robertson et al. in 1986, show the worst compatibility with GOST 25100 classification. Table 8 presents classification, based on nomograms of other authors.

Table 8. Soil classification table by different CPT data processing methods

Classification source	EGE number	
	2	3
ГОСТ 25100 from results of laboratory tests	Semi-hard clay	Stiff plastic clay
Robertson et al. diagram. (1986)	Clay – clay loam	Clay loam, sandy loam
Robertson et al., diagram (1990)	Over-consolidated clay	Over-consolidated clay and clay loam
Olsen & Mitchell's diagram (1995)	Over-consolidated clay and clay loam	Over-consolidated organic mineral clay and over-consolidated clay
GOST 25100	Clay loam, over-consolidated, hard	Stiff plastic partly over-consolidated clay

Fig. 22b shows deformation moduli values, determined by different laboratories and field tests. The deformation moduli values are essentially different from each other. As was expected the maximum values of are given by the tri-axial compression method and tests with flat and screw plate. Oedometer deformation modulus (E_{oed}) is close to the values of deformation modulus (E_{RBT}), obtained with stiff dilatometer soil tests. The values of deformation modulus (E_{CPT}) from CPT, according to Tables И2, И5 [9] in some cases are also close to the values of oedometer deformation modulus. This confirms the statement about СП 47.13330 tables for measuring the normative (derived) oedometer deformation modulus from CPT rather that test plate deformation modulus (E_{PLT}).

There is the following relationship of CPT on drained angle of internal friction:

$$\varphi' = 13{,}5 \lg q_c + 23, \qquad (12)$$

with q_c as cone tip resistance, MPa.

This equation is true for sand above ground water table and cone tip resistance within 5 – 28 MPa range.

Robertson & Campanella [22] proposed the following empirical equation for determining the angle of internal friction:

$$\varphi' = 35° + 11{,}5 \lg\left(\frac{q_c}{30\sigma'_{vo}}\right), \qquad (13)$$

with q_c as cone tip resistance, σ'_{vo} as apparent stress from soil proper weight.

The apparent angle of internal friction can be determined by applying the normalized cone tip resistance value $q_{c1} = (q_c / p_a)/(\sigma'_{vo} / p_a)^{0,5}$ with p_a as the reference stress value, equal to atmospheric reference equal to atmospheric pressure (100 kPa). The equation below was proposed by Kulhawy и Mayne [16]. It was developed with the help of statistical analysis of data from the tests, conducted on non-cemented sand in a test box.

$$\varphi' = 17{,}6° + 11{,}0° \lg(q_{c1}). \qquad (14)$$

Fig. 23 demonstrates an example of the graph, corresponding to the last equation for EGE-5 (sand). This graph shows data of drained angle of internal friction, determined by the method of one-plane shear test. The value of internal friction, obtained in laboratory was equal to 39°. This value was greater than respective CPT values. However, it is quite easy to modify equations (12-14) by applying the results of completed tests and to use them further for investigation of similar sand deposits. The sand grain-size composition is shown in Table 3.

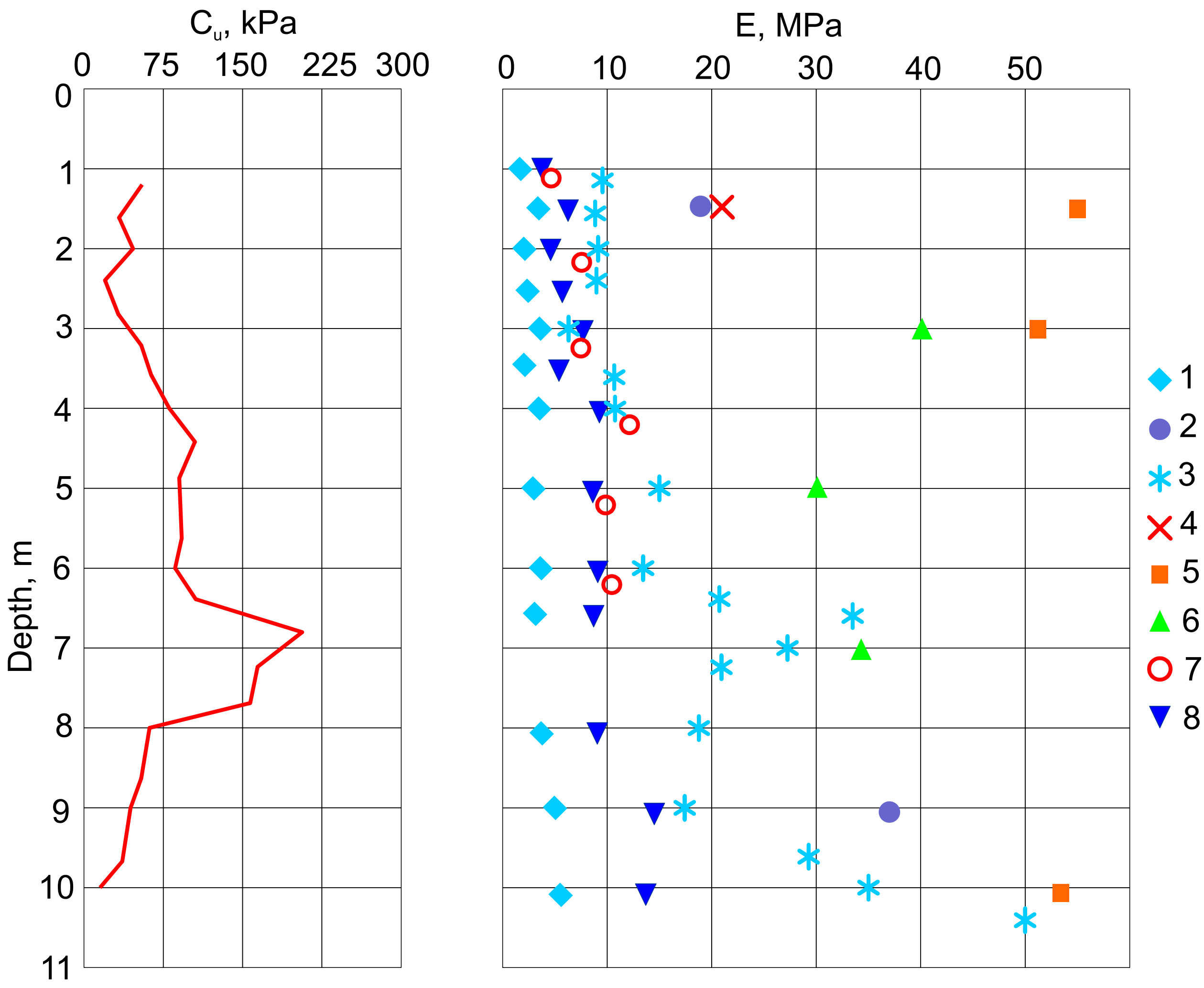


Fig.22. Soil parameters variation versus depth: a – non-drained strength; б - deformation modulus: 1 – compression modulus (E_k); 2 – tri-axial compression modulus (E_{TX}); 3 - CPT deformation modulus (E_{CPT}); 4 – 2500 cm^2 area flat test plate deformation modulus (E_{PLT}); 5 – 600 cm^2 screw plate deformation moduluso (E_{RST}); 6 – continuous auger deformation modulus (E_{RDT}); 7 – stiff dilatometer deformation modulus (E_{RBT}); 8 – oedometric deformation modulus (E_{oed})

As was noted earlier, many investigations were done in order to correlate oedometer deformation modulus with the cone tip resistence. The general relationship is (1). Differences depend on the value of α factor. The transfer factor α depends on the type of soil, as is seen in Table 9.

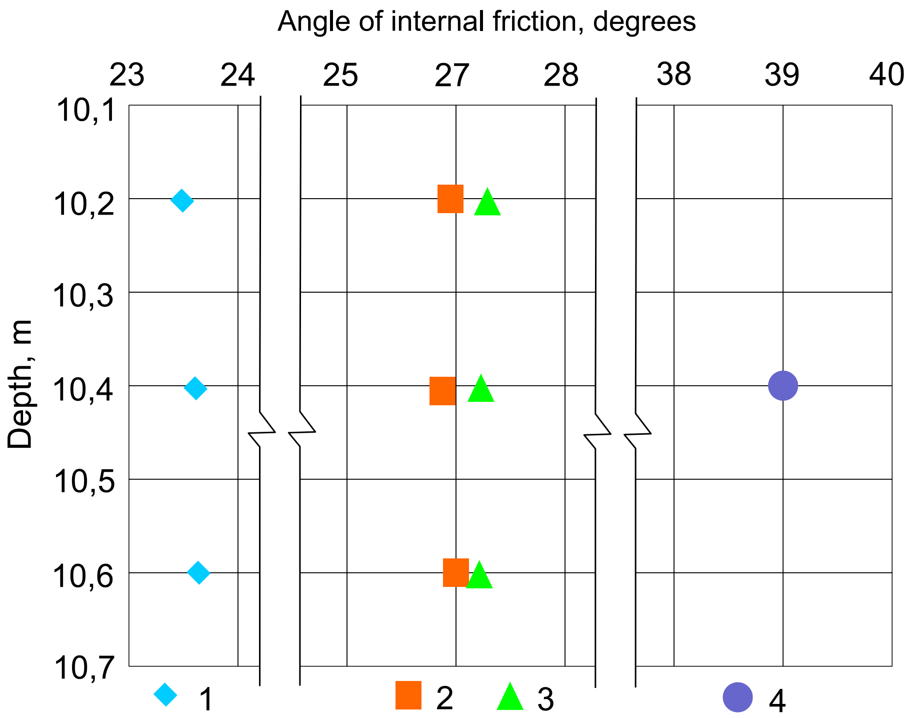


Fig. 23. Apparent angle of internal friction for EGE-5, calculated with correlation equatins: 1 – (12); 2 – (13); 3 – (14); 4 – one-plane shear.
“Угол внутреннего трения, град.” – angle of internal friction, degrees. “Глубина” - depth

Table 9. Rough estimates of oedometer deformation modulus for clay soils [24]

q_c (MPa)	$E_{oed} = \frac{1}{m_v} = \alpha q_c$	
$q_c < 0,7$ $0,7 < q_c < 2,0$ $q_c > 2,0$	$3 < \alpha < 8$ $2 < \alpha < 5$ $1 < \alpha < 2,5$	Low plasticity clay(CL)
$q_c > 2,0$ $q_c < 2,0$	$3 < \alpha < 6$ $1 < \alpha < 3$	Low plasticity muds (ML)
$q_c < 2,0$	$2 < \alpha < 6$	High plasticity muds and clays (MH, CH)
$q_c < 1,2$	$2 < \alpha < 8$	Organic muds (OL)
$q_c < 0,7$ $50 < w < 100$ $100 < w < 200$ $w > 200$	$1,5 < \alpha < 4$ $1 < \alpha < 1,5$ $0,4 < \alpha < 1,0$	Peat and organic clays (P_t,OH)

Table 10. Coefficient α from equation $E_{oed} = \alpha \cdot q_c$ with $\nu = 0,35$ for clay loams and $\nu = 0,42$ for clays

Soil name	Stiff clay loam	Stiff clay	Stiff plastic clay
Average value of coef-ficient α	5,50	6,69	11,09

If analytical solutions are applied, e.g. appendix B in ENV 1997-2, then undrained strength of cohesive soils can be determined from the following equation:

$$c_u = \frac{q_c - \sigma_{vo}}{N_k}, \tag{15}$$

with q_c as CPT cone resistance; σ_{vo} as soil proper weight stress; N_k as coefficcient, determined by tests.

Mean value N_k as function of plasticity index for soils with $I_p > 10$ can be evaluated from the following equations:

$$N_k = 19 - \frac{I_p - 10}{5}. \tag{16}$$

Apparent pressure due to precompaction can be linked up with undrained strength (c_u) of clay soils by the following equation [18]:

$$\sigma'_p = \frac{c_u}{(0{,}11 + 0.0037 I_p)}, \tag{17}$$

with I_p as plasticity index.

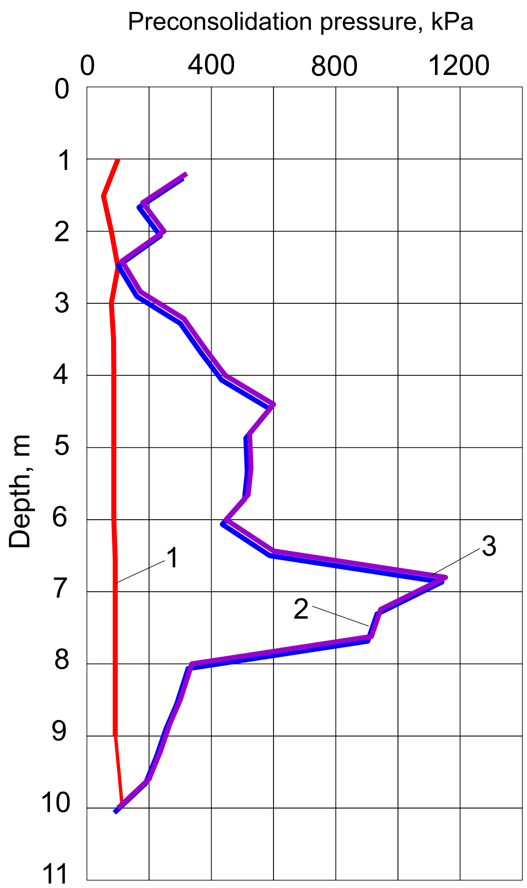


Fig. 24. Apparent pre-compaction pressure: 1 – from compression test data; 2 – from CPT data as per equation (17); 3 – from CPT data as per equation (18)

Pre-compaction pressure can be evaluated as per CPT data and the following equation [23]:

$$\sigma'_p = 0{,}33(q_c - \sigma_{vo})\,, \tag{18}$$

with q_c as probe tip resistanc зонда; σ_{vo} as total vertical stresses due to soil proper weight.

Comparison of pre-compaction pressure, found from correlation equations (17) and (18) with laboratory test data shows their considerable difference. Coincident of the correlation values is due to the fact that parameter c_u, in equation (17) was also determined via cone probe resistance q_c, as is evident from equation (15).

Conclusions

The laboratory and in situ soil tests prompt the following conclusions, a part of which is disputable:

1. The values of deformation and strength parameters depends on the test method. It is impossible to obtain equal values of soil deformation and strength by different laboratory and in situ soil tests..

2. There are two moduli, determined by the tests, either elastic deformation modulus or deformation modulus::

- elastic deformation modulus should be measured by tri-axial tests at small stress values, not exceeding proportionality limit of “axial deformation – axial stress”;

- deformation modulus should be determined from tri-axial compression tests, depending on the pre-specified stress range, using “axial deformation – axial stress”;

- deformation modulus and elastic deformation modulus coincide for small values of stresses and deformations;

- elastic deformation modulus depends of in-situ stresses and grows versus the test depth.

3. Elastic deformation modulus, obtained from tri-axial tests, is a reference value for correlating equations between compression and field tests. Fig 22b shows that the value of this modulus corresponds to the oedometer and plate test deformation modulus range.

4. The compression test method yields two moduli: compression and oedometeric ones. Currently СП 22.13330 and СП 47.13330 recommend the compression modulus for evaluating the normative (characteristic) deformation modulus, obtained from screw plate test, or flat plate test, or pressuremeter test. In order to avoid excessive computations of coefficient β or possible error in Poisson ratio it is better to apply oedometric rather than compression deformation modulus value.

Such a simple method enables the following advantages:

- application of one deformation moduli instead of the two, namely the oedometric modulus deformation moduli, as is done in the international practice;

- oedometric modulus is more realistic, because it is obtained by direct measurements without soil lateral expansion;

- oedometric modulus is applied in many geotechnical software codes of subsoil base analysis, e.g. PLAXIS;

- oedometric modulus is easier applied as a reference for evaluating correlations, e.g. between CPT data: tip resistance, friction forces, stresses for soil proper weight and oedometer deformation modulus. Application of the tri-axial elastic modulus for this purpose is much more difficult and more expensive, because it requires consolidated-drained tests.

5. Determination of deformation moduli, angle of internal friction and cohesion requires application of laboratory and field tests from specific theoretical solutions, applied to subsoil analysis. It is mandatory to correlate soil parameters measurement techniques with analytical and numerical methods of subsoil analysis, used by design engineers. In the first case, these are solutions, recommended by current regulations while in the second case these are soil models in respective computer codes.

6. The parameter, recommended for the analysis (deformation modulus,

strength parameter, etc.) shall be selected by cautious assessment of similar parameter, obtained with different laboratory and field tests. There should be applied at least two laboratory and one field test method. E.g., in order to determine Mohr-Coulomb strength parameters it is recommended to apply laboratory one-plane shear, tri-axial test and field ring-shear methods. In order to determine module of deformation it is recommended to apply compression pressure method, tri-axial compression method, flat and screw plate and pressuremeter.

7. Drilling penetration method is similar to CPT and DPT with respect to strata allocation.

8. Application of a test plate along with Schleicher elastic solution (6) enables evaluation of the deformation modulus value with subsequent evaluation of deformation moduli, obtained with other field and laboratory tests.

9. Application of the stiff dilatometer is effective both in cohesive clay mineral

and organic-mineral soils and other non-cohesive sand and coarse-grained soils for determining the initial stress state in in-situ disperse soil massifs.

With known values of vertical and horizontal stresses from soil proper weight,

itis possible both to evaluate both the lateral pressure ratio at rests and the oedometer deformation modulus and to implement anisotropic consolidation of soil samples in tri-axial compression apparatus. Presently, GOST 12248 recommends application of isotropic consolidation (three-dimensional compression) with the known initial stress state. Notably, isotropic consolidation is better suitable for water-saturated organic-mineral soils, in which the initial stress distribution is close to hydrostatic.

Application of known correlation relationships for determining parameters of

various soils from CPT data is possible only for their correction versus laboratory test data of the same varieties of soils. It is better to apply domestic regional correlation equations.

10. Coulomb-Mohr strength parameters of over-consolidated soils (OCR>1)

should be determined with the account of their stress history in situ. In order to do it is necessary to determine pre-consolidation pressure σ_p value from compression test data and then to account for this pressure as additional to the values of normal pressures as is recommended by Table 5.1 GOST 12248. Anyway the normal pressure stages shall not be less than pre-consolidation pressure value.

10.Application of Robetrson nomograms is justified for practical application to determine soil types right in situ. They need adjustment to particular regions of the Russian Federation as per GOST 25100.

References

1. Болдырев Г.Г. Буровое зондирование грунтов (Drilling penetration soil tests) / Г.Г. Болдырев, Р.Г. Кальбергенов, Л.Г. Кушнир, Г.А. Новичков // Инженерные изыскания. – 2012. – №12. С. 38-42.

GOST 5180-84. Грунты. Методы лабораторного определения физических характеристик. 16 С. (Soils. Laboratory methods for determination of physical parameters).

2. GOST 19912-81. Грунты. Метод полевого испытания динамическим зондированием. 18 С. (Soils. Field dynamic penetration test).

3. GOST 20276-2011. Грунты. Методы полевого определения характеристик прочности и деформируемости. 93 С. (Soils. Field soil strength and deformability tests).

4. GOST 12248-2010. Грунты. Методы лабораторного определения характеристик прочности и деформируемости. 162 С. (Soils. Laboratory strength and deformability tests).

5. GOST 20522-2011. Грунты. Методы статистической обработки результатов испытаний. 34 С. (Methods of statistic test data processing).

6. GOST 25100-2010. 30 С. Soils. Classification..

7. СП 22.1330-2011. 166 С. Subsoils of buildings and structures..

8. СП 47.13330. Инженерные изыскания для строительства. 2012. 119 С. Engineering investigations for construction.

9. Лушников В.В. Усовершенствование методов определения деформационных и прочностных свойств элювиальных глинистых грунтов Урала: дис. канд. техн. наук / В.В. Лушников. – Свердловск, 1966. Improvement of the Urals elluvial clay soils deformation and strength properties tests.

10. Тер-Мартиросян З.Г., Кятов Н.Х., Сидорчук В.Ф. Экспериментальные и теоретические основы определения напряженно-деформированного состояния грунтов естественного сложения. Инженерная геология, №4, 1984, с. 13-25. Experimental and theoretical basics for determining stress and deformation state of in situ soil.

11. Трофименков Ю.Г. Полевые методы исследования строительных свойств грунтов / Ю.Г. Трофименков, Л.Н. Воробков. – М., Стройиздат, 1981. – 215 с. Field methods for soil properties investigation.

12. ASTM D 1452. Standard Practice for Soil Investigation and Sampling by Auger Borings. – 2 p.

13. ASTM D 6151. Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling. – 13 p.

14. EN 1997-2:2007 Eurocode 7: Geotechnical design - Part 2: Ground investition and testing. – 202 p.

15. Kulhawy F.H., Mayne P.H. Manual on estimating soil properties for foundation design. Report EL-6800. Palo Alto, USA: Electric Power Research Institute (EPRI), 1990. – 250 p.

16. Lunne T., Robertson P.K., Powell, J.J.M. Cone penetration testing in geotechnical practice. Blackie Academic. Chapman-Hall Publishers, U.K.; available from EF Spon. Routledge Pub., New York, 1997. – 312 p.

17. NAVFAC DM-7.1. Design manual soil mechanics, 1986. – 389 p.

18. Olsen R. S., Mitchell J. K.CPT stress normalization and prediction of soil classification.” Proc., Int. Symp. on Cone Penetration Testing, CPT 95, Linkoping, Sweden, 1995, pp. 257–262

19. PIVTEC GmbH. PIVview User Manual. Homepage: http://www.pivtec.com.

20. Robertson P.K., Campanella R.G. Guidelines for Geotechnical Design Using CPT and CPTU. University of British Columbia, Vancouver, Department of Civil Engineering, Soil Mechanics Series 120, 1985. – 512 p.

21. Robertson P.K., Campanella R.G., Wightman A. SPT-CPT Correlations. Journal of Geotechnical Engineering, Vol. 109, No. 11, 1983, pp. 1449-1459.

22. Sabatini, P. J., R. C. Bachus, P. W. Mayne, J. A. Schneider and T. E. Zettler, Geotechnical Engineering Circular No. 5 – Evaluation of Soil and Rock Properties, (Publication No. FHWA-IF-02-034), 2002. – 385 p.

23. Sanglerat G. The Penetration and Soil Exploration. Elsevier, Amsterdam, 1972. – 464 p.