Оглавление

PRACTICAL LESSON 1

A distinction is made between physical, strength and deformation characteristics of soil. Physical characteristics are divided into basic, derivative and classification characteristics.

The basic physical characteristics of soil (table 1.1).

The basic are the characteristics determined from experience.

Table 1.1

Basic physical characteristics of soil

Name	Designation	Dimension	Calculation formula
Soil density	ρ	kg/m^3	$\rho = G / V$
Specific gravity of soil	γ	kN/m^3	$\gamma = \rho g$
Soil particle density	ρ_s	kg/m^3	$\rho_s = G_s / V_s$
Specific weight of soil particles	γ_s	kN/m^3	$\gamma_s = \rho_s g$
Soil moisture	W	in fractions of one or %	$W = G_w / G_s$
Humidity on the plasticity boundary	W_p	in fractions of one or %	$W_p = G_{w,p} / G_s$
Moisture on the yield boundary	W_L	in fractions of one or %	$W_L = G_{w,L} / G_s$

Derived physical characteristics of the soil (table 1.2).

Table 1.2

Derived physical characteristics of soil

Name	Designation	Dimension	Calculation formula
Density of drysoil	ρ_d	kg/m^3	$\rho_d = \rho / (1 + W)$
Specific weight of dry soil	γ_d	kN/m^3	$\gamma_d = \rho_d g = \gamma / (1 + W)$
Porosity coefficient	e	in fractions of one or %	$e = (\rho_s - \rho_d) / \rho_d = \rho_s / \rho_d - 1$
Porosity	n	in fractions of one or %	$n = (\rho_s - \rho_d) / \rho_s = 1 - \rho_d / \rho_s$

Classification physical characteristics of soil (table 1.3).

Table 1.3

Derived physical characteristics of soil

Name	Designation	Dimension	Calculation formula
Number of plasticity	I_p	in fractions of one or %	$I_p = W_L - W_p$
Fluidity index	I_L	in fractions of one or %	$I_L = (W - W_p)/I_p$
Water saturation coefficient	S_r	in fractions of one or %	$S_r = (\rho_s W)/(\rho_w e)$
Total moisture capacity	W_{sat}	in fractions of one or %	$W_{sat} = (\rho_w / \rho_s)e$ (respond $S_r = 1$)

The dusty-clayey soils are characterized by the predominance of dusty and clayey particles in their composition and depending on the number of plasticity I_p are allocated by type according to table 1.4.

Table 1.4

Type of silty-clay soils	Number of plasticity, %
Sand dust	$1 \le I_p \le 7$
Loam	$7 < I_p \le 17$
Clays	$17 < I_p$

Dusty-clay soils are distinguished by consistency characterized by the flowability index I_L, according to table 1.5.

Table 1.5

Variety of dusty-clay soils	Flow index
Sandy loam	
Hard	$I_L < 0$
Plastic	$0 \le I_L \le 1$
Fluidity	$1 < I_L$
Loams and clays	
Hard	$I_L < 0$
Half-hard	$0 \le I_L < 0{,}25$
Hard-plastic	$0{,}25 \le I_L < 0{,}5$
Soft-plastic	$0{,}5 \le I_L < 0{,}75$
Fluid-plastic	$0{,}75 \le I_L \le 1$
?	$1 < I_L$

Coarse-clastic and sandy soils are divided into types depending on their particle-size composition according to table 1.6.

The granulometric composition of the soil is the content by weight of groups of particles (fractions) of different size in relation to the total mass of absolutely dry soil, expressed as a percentage.

Table 1.6

Classification of sandy soils depending on particle size distribution

Variety of coarse clastic soils and sands	Particle size, mm	Particle content, % bymass
Gravel	> 2	> 50
Sand		
Gravelly	> 2	> 25
Coarse	> 0,5	> 50
Medium coarse	> 0,25	> 50
Small	> 0,1	≥ 75
Dusty	> 0,1	< 75

Depending on the coefficient of porosity, there are dense, medium dense and loose sands (table 1.7).

Table 1.7

Stacking density of sandy soils

Variety of sands	Porosity coefficient e		
	Gravelly, coarse, mediumcoarse	Small	Dusty
Dense	$e < 0{,}55$	$e < 0{,}6$	$e < 0{,}6$
Medium Dense	$0{,}6 \le e \le 0{,}7$	$0{,}6 \le e \le 0{,}75$	$0{,}6 \le e \le 0{,}8$
Loose	$e > 0{,}7$	$e > 0{,}75$	$e > 0{,}8$

Sands and coarse clastic soils, according to the classification, can be low-moisture, wet and saturated with water (table 1.8).

Table 1.8

Classification of sandy soils by water saturation coefficient

Variety of coarse clastic and sandy soils	Water saturation coefficient
Low-moisture	$0 < S_r \leq 0{,}5$
Wet	$0{,}5 < S_r \leq 0{,}8$
Saturated with water	$0{,}8 < S_r \leq 1$

The design resistance of foundation soils R_0 is intended for preliminary determination of foundation dimensions and is also used as an initial value of the design resistance of soil when determining the foundation footing dimensions using the selection method.

For sandy soils, according to Set of Rules 22.13330.2016, the design resistance is determined according to table 1.9.

Table 1.9

Design resistance R_0 of sands

Sands	Value of R_0, kPa depending on stacking density	
	Dense	Medium density
Large	600	500
Medium sands	500	400
Small		
Wet	400	300
Wet and water saturated	300	200
Dusty		
Low-moisture	300	250
Hamidity	200	150
Saturated with water	150	100

The design resistance R_0 of clayey (non-sagging) soils is determined according to Set of Rules 22.13330.2016 according to table 1.10 depending on the types of soils, values of yield strength and porosity coefficient. Table 1.10 shows R_0 values for the extreme (upper and lower) values of I_L and *e*, for the intermediate values of the fluidity index and porosity coefficient R_0 is determined by interpolation.

Table 1.10

Design resistance R_0 of clayey (non-sagging) soils

Dusty-clay soils	Void ratio *e*	R_0 value, kPa, at fluidity index I_L	
		0	1
Sand dust	0,5	300	300
	0,7	250	200
Loam	0,5	300	250
	0,7	250	180
	1,0	200	100
Clays	0,5	600	400
	0,6	500	300
	0,8	300	200
	1,1	250	100

All of the above classifications are used for the relative qualitative assessment of foundation soils (coarse is better than fine; hard clay is better than soft-plastic or fluid) and for determining the design resistance R_0. Numerical values of R_0 also add a quantitative factor to the qualitative assessment (how much better or worse the foundation soil is).

EXAMPLE 1.1. DETERMINING THE TYPE OF SANDY SOIL

For sandy soils, the results of particle size analysis were obtained, shown in table 1.11. Determine the type (name) of soils.

To determine the type of the soil it is necessary to sum the data of the percentage content (table 1.11) to reduce the fraction (from the maximum size to the minimum), each time comparing the obtained sum with the limits of the weight content of particles according to State Standard 25100-2020, given in table 1.6.

Table 1.11

Results of granulometric analysis for sands

Fraction, mm	Content, % in soils		
	1	2	3
2,00	–	–	53
2,00–0,50	18,42	24,5	10,3
0,50–0,25	11,39	29,3	8,7
0,25–0,10	32,41	33,5	9,8
0,10–0,05	34,34	12,1	6,1
0,05–0,01	2,08	0,4	5,0
0,01–0,005	0,88	0,10	7,0
< 0,005	0,48	0,1	0,1

Determine the total number of particles for soils:

For particles > 2 mm	$+^0$	$+^0$	53 > 25 Consequently, thesand is gravelly
For particles > 0,5 mm	18,42 18,42	24,5 24,5	
For particles > 0,25 mm	11,39 29,81	29,3 53,8 > 50 Consequently, sand of medium size	
For particles > 0,1 mm	32,41 62,22 < 75 Consequently, the sand is dusty		

Answer. Soil 1 contains particles larger than 0,1 mm less than 75 %, which means that the sand is fines; soil 2 is a coarse sand, since the total content of particles larger than 0,25 mm is 53,8 > 50 %; soil 3 is a gravelly sand, since particles larger than 2 mm contain 53 % (over 50 %).

EXAMPLE 1.2. DETERMINE THE TYPE AND VARIETY OF COARSE SAND

Specific weight of the soil part γ_s = 26,8 kN/m^3, specific gravity of soil γ = 20 kN/m^3, natural humidity w = 24,5 %.

We calculate the coefficient of porosity:

$$e = \frac{26{,}8}{20{,}0}(1 + 0{,}01 \cdot 24{,}5) - 1 = 0{,}0668.$$

According to table 1.7, we classify the sand as coarse medium density sand, so $0{,}55 < e = 0{,}668 < 0{,}7$.

We calculate the water saturation coefficient:

$$S_r = \frac{0{,}01 \cdot 24{,}5 \cdot 26{,}8}{0{,}668 \cdot 10} = 0{,}983,$$

hence, the type of sand is water-saturated ($0{,}8 < S_r = 0{,}983 < 1$).

Answer. Soil type — coarse sand, type — medium density, type — saturated with water.

EXAMPLE 1.3. DETERMINE THE TYPE AND TYPE OF CLAYEY SOIL

For example: natural humidity w = 24,5 %, moisture content on the rolling boundary w_p = 21,5 %, moisture content on the flowing boundary w_L = 31,4 %.

Let's calculate the number of plasticity:

$$I_p = 31{,}4 - 21{,}5 = 9{,}9\ \%.$$

According to table 1.4

$$7 < I_p = 9{,}9 < 17.$$

Consequently, the type of soil is loam.

Let's calculate the fluidity index:

$$I_L = \frac{24{,}5 - 21{,}5}{31{,}4 - 21{,}5} = 0{,}3.$$

Comparing the obtained value with the data in table 1.5, we classify this clay soil as stiff clay loam.

Answer. The considered clay soil is a stiff clay loam.

EXAMPLE 1.4. DETERMINE THE DESIGN RESISTANCE OF A STIFF CLAY LOAM

Given: I_p = 9,9 %, I_L = 0,3, e = 0,75.

We determine the interpolation limits for the porosity coefficient e = 0,75, so we choose the interval from e_1 = 0,7 to e_2 = 1,0.

When I_L is constant, R_0 value for intermediate value of porosity coefficient can be calculated by the formula

$$R_0 = R_0^{e_1} - \frac{R_0^{e_1} - R_0^{e_2}}{e_2 - e_1}(e - e_1),$$

where $R_0^{e_1}$, $R_0^{e_2}$ are the values of design resistance at the porosity coefficient e_1 and e_2, respectively.

Let's calculate the value of R_0 at I_L = 0:

$$R_0^{I_L=0} = 250 - \frac{250 - 200}{1 - 0{,}7}(0{,}75 - 0{,}7) = 241{,}67 \text{ kPa}.$$

Let's calculate the value of R_0 at I_L = 1:

$$R_0^{I_L=1} = 180 - \frac{180 - 100}{1 - 0{,}7}(0{,}75 - 0{,}7) = 166{,}67 \text{ kPa}.$$

Knowing the value of R_0 for the design porosity coefficient e, we find by interpolation the design resistance for I_L = 0,3:

$$R_0 = R_0^{I_L=0} - (R_0^{I_L=0} - R_0^{I_L=1})I_L.$$

Desired value

$$R_0 = 241{,}67 - (241{,}67 - 166{,}67)\ 0{,}3 = 219{,}17 \text{ kPa}.$$

Answer. R_0 = 219,17 kPa ≈ 220 kPa.

Example 1.5. Determine the design resistance of coarse, medium-density sand saturated with water

The data on physical properties are given in Example 1.2.

Answer. According to table 1.9 we define the value of R_0 = 500 kPa.

Mineralogical and granulometric composition of soils

Construction properties of soils depend on the mineralogical and granulometric composition, structure, texture and state in the natural occurrence, in the massif. There are four main groups of minerals: primary — quartz feldspars, mica; secondary — formed as a result of weathering of magmatic and metamorphic parads, clay, flake in structure; primary sedimentary minerals — sulfates (gypsum, anhydrite), carbonates (calcite, dolomite aragonite), halides (gadite and sylvin).

Mineralogical composition of loose soils changes naturally with decreasing grains: the smaller they are, the more important are secondary. clay minerals whose content determines the physical and mechanical properties of different types of soils and methods of their study.

The most common in nature quartz and feldspars determine the strength and compressibility of soils. The same shape and size (less than 0,001 mm) particles of mica, muscovite and biotite attract different amounts of water from the air. Hygroscopic capacity and height of capillary rise on soils with white muscovite mica is 1,5...2,0 times higher than on black-brown biotite crumb.

Thus, glauconite admixture of only 3...4 % reduces strength characteristics by 3 times and increases compressibility by up to 4 times. The content of carbonates in the ground causes increase of soaking rate and filtration coefficient values.

The content of clay minerals determines the plasticity of soils, their moisture capacity. Adding 10 % montmorillonite to the soil reduces its water permeability by 10,000 times. Adding bentonite up to 10 % leads to fourfold decrease of the angle of internal friction. However, mixtures with kaolinite are almost indistinguishable under load from almost incompressible quartz flour.

The particle size distribution of soils is characterized by particle size, the quantitative ratio of which in samples is determined visually and on the basis of grain analysis, when sifting through a set of sieves (for sandy soils). The content of each fraction is determined as a percentage of the weight of the dried sample.

The content of particles of different fractions has a significant impact on the. properties of soils. Therefore, to quantify its particle size distribution, an integral curve of particle size distribution of the soil, called the curve of particle size distribution, is constructed in semi-logarithmic scale (fig. 1.1).

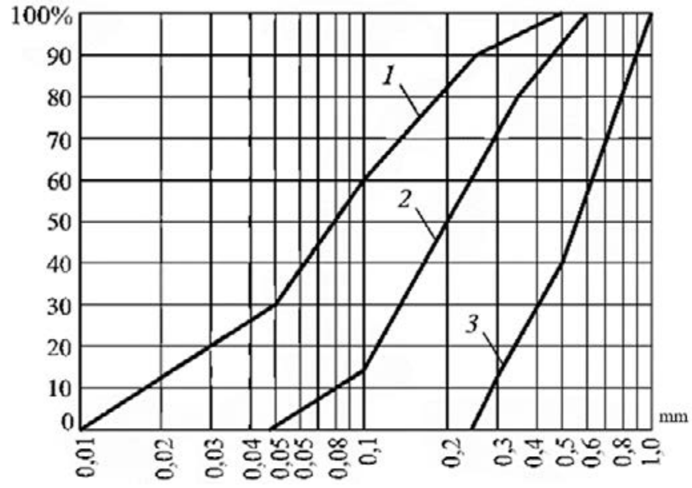

Fig. 1.1. Integral curve of grain composition of sand:
1 — dusty; *2* — small; *3* — coarse sand

With increasing heterogeneity, the slope of the grain size distribution graph becomes less and vice versa. For the numerical index of the heterogeneity of coarse-clastic and sandy soils the expression is used:

$$C = d_{60} / d_{10},$$

where d_{60} and d_{10} are the diameters of particles less than which this soil contains 60 and 10 % of particles, respectively.

Homogeneous soil has an index $C_u \leq 3$, heterogeneous $C_u > 3$.

Regulatory and design characteristics of soils

The normative and design values of the soil characteristics should be established on the basis of statistical processing of the test results according to the procedure in State Standard 20522-2012.

The normative values of the soil characteristics or parameters determining the properties of the soil mass should be taken equal to their mathematical expectations obtained on the basis of test results processing, unless other conditions determining their values are stipulated.

The calculated values of soil characteristics are determined taking into account their possible deviations to the unfavorable side from their normative values. Such deviations should be taken into account by means of partial ground reliability coefficients γ_g. The values of these coefficients may be different for different characteristics and limit states.

All calculations of foundations should be carried out using the calculated values of soil characteristics X calculated according to the formula

$$X = X_n / \gamma_g,$$

where X is the normative value of this characteristic; γ_g — ground reliability coefficient.

The coefficient of reliability for soil in calculating the calculated values of strength characteristics ϕ, with dispersed soils, as well as soil density ρare set depending on the variability of these characteristics, the number of determinations and the value of confidence probability α (according to State Standard 20522-2012). For other soil characteristics it is allowed to take γ_g equal to 1.

Confidence probability of the calculated values of soil characteristics α is taken as equal in the calculation of the foundations for the first group of limiting states 0,95, for the second group — 0,85.

PRACTICAL LESSON 2

DETERMINATION OF THE DEFORMATION CHARACTERISTICS OF SOILS

The choice of the type of laboratory and field tests and their volume is mainly determined by the experience of the geotechnical engineer. The following minimum criteria should be used when drawing up a program of laboratory tests:

1. Type of project (residential or industrial building, bridge, embankment, retaining wall, etc.).
2. The size of the projected facility
3. Magnitude of the loads to be transferred to the foundation.
4. Type of load: static load, dynamic load, seismic load, stress trajectories.
5. Limitations on the limit states: carrying capacity and deformation of the foundation.
6. Accuracy of the construction of the profile of the soil with the allocation of individual engineering and geological elements.
7. Features of soils (loess, swelling, permafrost, etc.).

The results of field and laboratory tests should give a sufficient understanding of the profile of the soil strata and physical and mechanical characteristics required for the design of buildings or structures.

The main parameter that characterizes the compressibility of dispersed soils is the total modulus of deformation E. This modulus is used when calculating the settlement of foundations by the method of layer-by-layer elementary summation and is called the standard modulus of deformation, which is determined with a reliability factor $\gamma_g = 1$. In addition to the general strain modulus, the parameters characterizing compressibility and the initial stress state of the soil include: Poisson`s ratio, ν; pre-compaction pressure, S_r; degree of recompaction, overconsolidation ratio (OCR); shear modulus, G; bulk strain modulus, K; resting lateral pressure coefficient, K_0; primary consolidation coefficient, c_v; secondary consolidation coefficient, c_α.

To calculate the settlement of foundations using the layer-by-layer summation method, only two deformation moduli, E and E_0, are required, and the second one is used only if the depth of the excavation of the designed building exceeds 5 m.

In Set of Rules 22.13330.2016 the normative value of the deformation modulus E is taken as equal to the deformation modulus determined by means of soil tests E_{pi}. The value of the strain modulus found from 5000 and 10,000 cm^2 die tests is considered reliable. Therefore, strain moduli found by other test methods are reduced to the die test, using transition coefficients, mk. In particular, the transition factor m_k, which depends on the type of soil and varies from 1 to 6, may be used in the transition from compression to punch strain modulus for buildings and structures of the level of responsibility.

For buildings and structures of I and II levels of responsibility this factor should be found from experiments, by comparative laboratory and field tests with a flat, screw die and pressure meter.

Note that the values of the die modulus of deformation depend on the area of the die: the greater the area of the die, the greater the value of the modulus of deformation.

The strain modulus found from triaxial tests is also less than the die modulus, but not as much as the compression strain modulus. In fact, a correlation with the die strain modulus is needed for it as well. In State Standard 12248-2020, under triaxial compression, it is recommended to determine the initial or tangential strain modulus using the experimental dependence $\varepsilon_1 = f(s_1)$. At the same time, the secant strain modulus can be found, which depends on the stress level (s_1). At a small level of axial strain (ε_1) the initial and secant strain moduli coincide.

The elastic strain modulus is not equal to the initial strain modulus found from triaxial tests, it can be more than ten times. Therefore, when determining the elastic modulus of deformation, triaxial tests are carried out with measuring the velocity of transverse waves, which cause almost no compaction of the soil (deformation less than 10^{-6}). In this case, however, it is not the elastic modulus of deformation that is determined, but the elastic shear modulus from the expression

$$G = \rho V_s^2,$$

where ρ is the density of the ground; V_s — velocity of the transverse wave.

Using the found value of the shear modulus, determine the elastic modulus of deformation:

$$E = 2G(1 + v).$$

Choice of test scheme

The choice of triaxial test scheme in determining the strength characteristics depends on soil conditions, conditions of external load application, type and method of buildings and structures erection.

Application of non-consolidated-undrained (NU) tests.

The strength of soils determined from NU tests under full stress is applicable only to situations where there is little drainage and consolidation. In practice, this applies to soils with a filtration coefficient (k) of less than 10^{-3}cm/s.

The undrained strength characteristic (c_u) (fig. 2.1) found in NU tests is recommended to be used when buildings and embankments on water-saturated soft plastic and flowable clay soils are erected at a fast rate.

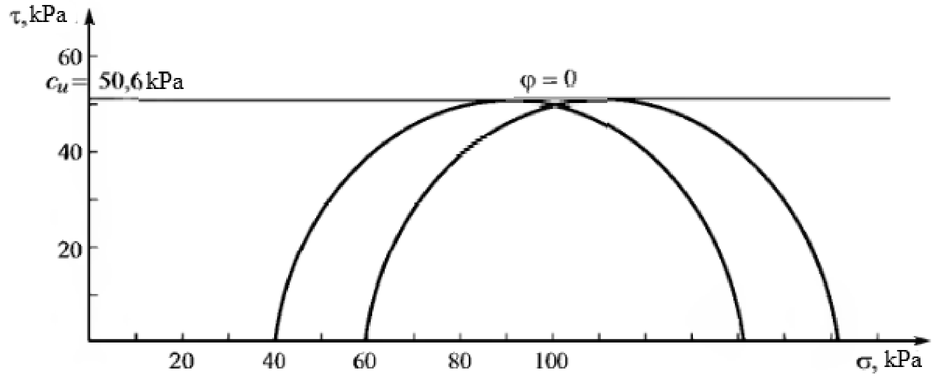

Fig. 2.1. Results of NU tests

Application of consolidated-undrained (CU) tests.

CU tests allow the determination of strength characteristics in both effective (φ′, c') and total stresses (φ, c) which are used in the following cases:

1. When reconstructing existing buildings and increasing the height of existing embankments.
2. When earthworks in the form of embankments and levees are erected in stages.
3. In the design of tank bases.

In the first and second cases the loading from a structure is applied for a long time and the density (porosity) of the foundation soil changes in time due to its compaction. In the latter case, the rate of loading (filling of the tank) and the type of soils in the base may be different, so tests should be conducted both according to the NU and CU scheme.

In contrast to NU and CD-tests, CU-tests with pore pressure measurement provide data for interpretation of strength at both full and effective stresses (fig. 2.2). Similar to the NU tests, in the CU test the soil samples are reconsolidated in the laboratory at a given all-round pressure.

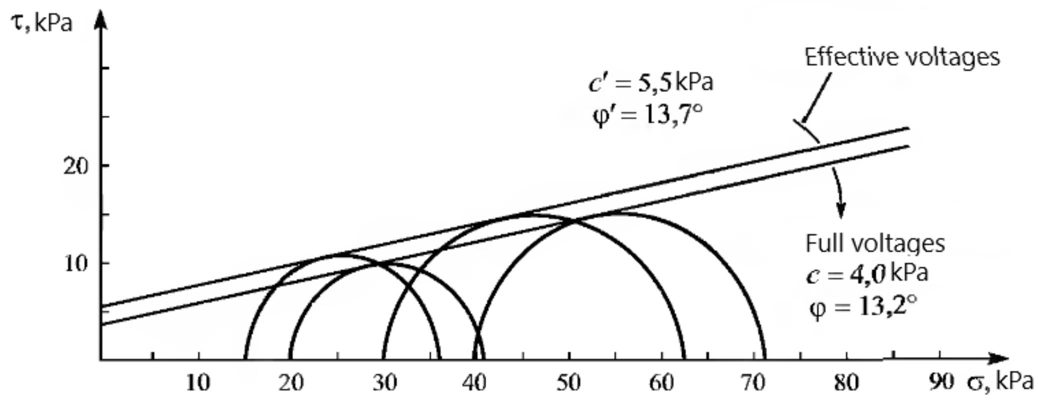

Fig. 2.2. CU test results in effective and total stresses

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