

**MINISTRY OF EDUCATION AND SCIENCE of UKRAINE
O. M. BEKETOV NATIONAL UNIVERSITY
of URBAN ECONOMY in KHARKIV**

Methodical recommendations
for practical classes, for making calculations
and graphic work
and organizing independent work
on an academic discipline

**«CONSTRUCTION IN DIFFICULT ENGINEERING-GEOLOGICAL
CONDITIONS»**

*(for full-time foreigner students
second (Master's) level of higher education
specialty G19 Building and Civil Engineering)*

Kharkiv
O. M. Beketov NUUE
2025

Methodical recommendations for practical classes on an academic discipline «Construction in Difficult Engineering-Geological Conditions» (for full-time foreigner students second (Master's) level of higher education specialty G19 Building and Civil Engineering) / O. M. Beketov National University of Urban Economy in Kharkiv ; comp. : I. V. Khrapatova. – Kharkiv : O. M. Beketov NUUE, 2025. – 40 p.

Compilers: I. V. Khrapatova

Reviewer PhD in Eng. sciences Tabachnikov S.V.

Recommended by the department of architecture of buildings and structures and architectural environment design, record № 1 on September 25, 2025.

CONTENTS

Introduction	4
Practical lesson 1 Assessment of engineering and geological conditions of the construction site	5
Practical lesson 2 Foundation on natural subsiding soil	9
Practical lesson 3 determining settlement and subsidence of the foundation	13
Practical lesson 4 foundation at $R = P_s$	20
Practical lesson 5 Foundation on water-saturated subsiding soil	20
Practical lesson 6 The foundation is on pre-compacted soil	21
Practical lesson 7 Foundation on a soil cushion	24
Practical lesson 8 Calculation and design of pile foundations	26
List of recommended sources	37

INTRODUCTION

The methodological recommendations have been developed in accordance with the program of the discipline “Construction in difficult engineering and geological conditions”, approved for students majoring in G19 “Construction and Civil Engineering”. In accordance with the content of the discipline, practical classes, tests, and exams are conducted. In addition, calculation and graphic work is required.

The methodological recommendations have been compiled in accordance with the practice of conducting these classes at the O. M. Beketov National University of Urban Economy in Kharkiv. The main objective of the methodological recommendations is to assist students in performing practical work in the discipline “Construction in difficult engineering and geological conditions.”

Using the initial data provided by the instructor for the practical work, the student records all calculations in a notebook. The practical work is considered complete when the student submits the notebook with all calculations, diagrams, and constructions to the instructor and answers a series of theoretical questions on the relevant topic.

PRACTICAL LESSON 1

ASSESSMENT OF ENGINEERING AND GEOLOGICAL CONDITIONS OF THE CONSTRUCTION SITE

1. For clay soils, we determine the following indicators:

- Plasticity index

$$I_p = w_L - w_p,$$

where w_L – Liquid limit:

w_p – Plastic limit.

The designation of dusty-clay soil by plasticity index is determined according to Table 1.1.

Table 1.1 – Designation of dusty-clay soil by plasticity index

Name of the dusty clay soil	Plasticity index I_p
Sandy loam	$0,01 \leq I_p \leq 0,07$
Loam	$0,07 < I_p \leq 0,17$
Clay	$I_p > 0,17$

If $I_p < 0,01$ the soil should be classified as sandy soil.

- Liquidity index:

$$I_L = \frac{w - w_p}{w_L - w_p}.$$

The consistency of dusty-clay soils according to I_L is determined by Table 1.2.

Table 1.2 – Types of dusty-clay soils according to I_L

Types of silty-clay soils	Liquidity index I_L
Sandy soils:	
Solid	$I_L < 0$
Plastic	$0 \leq I_L \leq 1$
Liquid	$I_L > 1$
Loams and clays:	

Solid	$I_L < 0$
Semi-solid	$0 \leq I_L \leq 0,25$
Tough plastic	$0,25 < I_L \leq 0,5$
Soft plastic	$0,5 < I_L \leq 0,75$
Fluid plastic	$0,75 < I_L \leq 1$
Liquid	$I_L > 1$

Conclusion: When $I_L < 0.75$ (fluid plastic and liquid clay soils) cannot serve as a base.

2. For sandy soils, we determine the following indicators:

- Void ratio :

$$e = \frac{\rho_s}{\rho} \cdot (1 + w) - 1.$$

The variety of sands by density is determined according to Table 1.3.

Table 1.3 – The variety of sands by density

Names of sandy soils	Porosity coefficient e		
	dense	medium density	loose
Gravelly, coarse and medium coarse	$e < 0,55$	$0,55 \leq e \leq 0,70$	$e > 0,70$
Fine	$e < 0,60$	$0,60 \leq e \leq 0,75$	$e > 0,75$
Dusty	$e < 0,60$	$0,60 \leq e \leq 0,80$	$e > 0,80$

- Degree of saturation:

$$S_R = \frac{w \cdot \rho_s}{e \cdot \rho_w},$$

де ρ_w – щільність води.

The name of sandy soil by water saturation is determined according to Table 1.4.

Table 1.4 – Names of sandy soil by water saturation

Varieties of soils	Degree of saturation S_r
Low water saturation (low moisture)	$0 < S_r \leq 0,5$
Moderately saturated (wet)	$0,5 < S_r \leq 0,8$
Saturated	$0,8 < S_r \leq 1,0$

Conclusion: Loose sand cannot serve as a foundation.

The calculation results are summarized in the table below:

Table 1.5 - Characterization of soil conditions at the construction site

№ layer	e	S _r	I _p	I _L	Characterization of soils
1					
2					
3					
4					
5					

2 Determining the type of soil conditions based on subsidence

- Characteristics of subsiding soil saturated with water to the degree

$$S_{r,sat} = 0.9$$

$$\rho_{sat} = \frac{\rho}{1+W} \cdot \left(1 + \frac{S_{r,sat} \cdot e \cdot \rho_w}{\rho_s} \right);$$

$$W_{sat} = \frac{S_{r,sat} \cdot e \cdot \rho_w}{\rho_s}.$$

- Building a dependency graph $\varepsilon_{sl,i}$ by P_i simultaneously determine the initial subsidence pressure P_{sl} , which corresponds to $\varepsilon_{sl,i} = 0.01$.

Example.

Data:

P, MPa	$\varepsilon_{sl,i}$
0.1	0,039
0.2	0,068
0.3	0,087

Determine the initial subsidence pressure P_{sl}

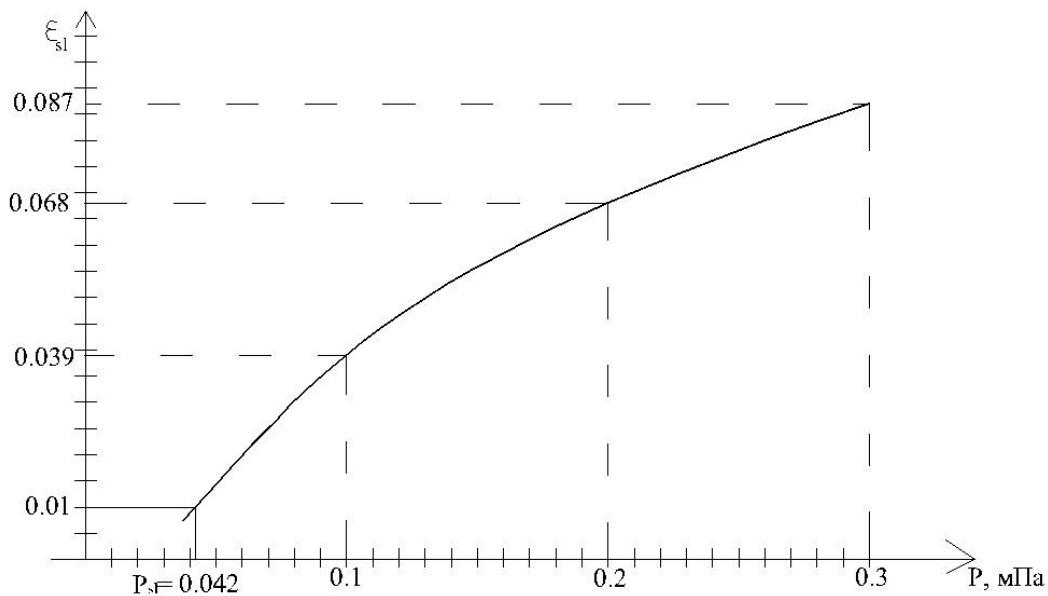


Fig. 1.1 - Example of determining the initial subsidence pressure

We calculate the values and build a graph of the stresses in the layer of subsiding soil from its own weight when it is fully saturated with water, taking into account possible cutting or filling when planning the territory. Mark the point corresponding to the value on the graph. Divide the layer of subsiding soil below this point into calculation strips, the thickness of which h_i should be no more than 2 m, and the number no less than 2. Soil conditions of type I in terms of subsidence include soil layers in which one of the following conditions is met:

- The initial settlement pressure is greater than the stress from the soil's own weight, $p_{sl} > \sigma_{zg}$, within the entire subsiding layer;

$p_{sl} > \sigma_{zg}$ в межах всієї просідаючої товщі;

- The initial subsidence pressure is less than the stress from the soil's own weight $p_{sl} < \sigma_{zg}$ within a layer no more than 2 m thick at depth;

- The calculated subsidence from the soil's own weight, determined by the formula

$$S_{sl,g} = \sum \epsilon_{sl,g} \cdot h \cdot k_{sl,i}, \text{ at } k_{sl,i} = 1 \text{ taking into account a change in depth } \epsilon_{sl} \text{ of less than 5 cm. [4]}$$

If none of the conditions are met, the soils are classified as type II soil conditions in terms of subsidence.

PRACTICAL LESSON 2

FOUNDATION ON NATURAL SUBSIDING SOIL

1 Determining the depth of the column foundation

A building without a basement – the depth of the foundation is determined from the surface of the natural relief, which in this case coincides with the conditional mark 0.000 (Fig. 2.1). Since the building is heated, the depth of the foundation for the columns in the middle row does not depend on the depth of soil freezing, while the depth of the foundation for the columns in the outer row does depend on it.

2 Determining the dimensions of the foundation base

We are designing a reinforced concrete rigid foundation. We will assume the approximate ratio of the dimensions of the foundation base to be $\eta = \frac{l}{b} = 1,4$, then the width of the foundation sole, m:

$$b = \sqrt{\frac{N \cdot 1000}{\eta \cdot (R - \gamma_{mt} \cdot d - q)}}, \quad (2.1)$$
$$l = 1,4b.$$

N – load, MH (we take the larger value of the two combinations),

γ_{mt} – the average calculated value of the specific weight of the soil and foundation material = 20 kN/m³,

d – foundation laying depth, counting from the planning mark;

b та l – width and length of the foundation sole;

q – load on the floor, for an industrial building 20 kN/m².

R – the estimated soil resistance, kPa.

First, let us take Table 2.1.

Table 2.1 – Estimated R0 support for subsidence soils

Soils	R_0 , kPa (kg/cm^2), soils			
	natural composition with density in the dry state p_d , T/M^3		compacted with density in the dry state p_d , T/M^3	
	1,35	1,55	1,60	1,70
Sandy loams	300(3) 150(1,5)	350(3,5) 180(1,8)	200(2)	250(2,5)
Loams	350(3,5) 180(1,8)	400(4) 200(2)	250(2,5)	300(3)

Note: the numerator shows the values of R_0 , which refer to non-soaked loamy soils with a degree of moisture $S_r \leq 0.5$; in the denominator - R_0 values, which refer to the same soils with $S_r \geq 0.8$, as well as to soaked silty soils.

Find b_0 according to formula (1) taking into account R_0 .

Determine R_1 by the formula:

$$R = \frac{\gamma_{c1} \cdot \gamma_{c2}}{k} \cdot [M_\gamma \cdot k_z \cdot b \cdot \gamma_{II} + M_q \cdot d_1 \cdot \gamma'_{II} + (M_q - 1) \cdot d_b \cdot \gamma'_{II} + M_c \cdot c_{II}], \quad (2.2)$$

where γ_{c1} , γ_{c2} – coefficients of working conditions, which take into account the type and condition of the base soil and the stiffness of the structural scheme of the structure, accept by table. 2.2,

Table 2.2 - Working conditions coefficients

soils	Coefficient	Coefficient for structures with a rigid structural scheme when the ratio of the length of the structure or its section to the height L/H is equal to	
		4 and more	1,5 and less
Large-clastic with sandy aggregate and sand, except small and dusty	1,4	1,2	1,4
The sands are fine	1,3	1,1	1,3
Dusty sands:			
- low moisture and wet,	1,25	1,0	1,2

- saturated with water	1,1	1,0	1,2
Clay, as well as large-fragmented with clay aggregate with a Liquidity index of the soil or aggregate $I_L \leq 0,25$	1,25	1,0	1,1
The same with $0,25 < I_L \leq 0,5$	1,2	1,0	1,1
The same with $I_L > 0,5$	1,0	1,0	1,0

k - is a coefficient that takes into account the degree of reliability of determining the characteristics of the base soil;

$k=1$, if the soil strength characteristics (φ_{II} and c) are determined experimentally;

$k=1.1$, if the characteristics of soil strength (φ_{II} and c) are taken according to DBN ;

k_z – a coefficient that takes into account the features of the joint operation of a wide foundation with the base;

If $b < 10$ м $k_z=1$; if $b > 10$ м $k_z=z_0/b+0,2$ ($z_0=0,8$);

M_γ, M_q, M_c – coefficients that depend on the angle of internal friction φ of the soil and the shape of the foundation in plan;

c_{II} – specific cohesion of the bearing soil layer.

d_b – basement depth – the distance from the planning level to the basement floor (for buildings with a basement width $B \leq 20$ м and a depth of more than 2 м, $d_b=2$ is accepted, for a basement width $B > 20$ м, $d_b=0$ is accepted).

$\gamma_{II}, \gamma'_{II}$ – average values of specific unit weight of soils above and below the base of the foundation, kN/m^3 .

$$\gamma' = \frac{\sum \gamma_i h_i}{\sum h_i}; \quad (2.3)$$

$$\gamma'_{II} = 10 \cdot \frac{\rho_1 \cdot h_1 + \rho_2 (d - h_1)}{d}. \quad (2.4)$$

Table 2.3 - Coefficients M_γ , M_q , M_c

Angle of internal friction φ , deg.	Coefficients M_γ , M_q , M_c			Angle of internal friction φ , deg.	Coefficients M_γ , M_q , M_c		
	M_γ ,	M_q ,	M_c		M_γ ,	M_q ,	M_c
0	0	1,00	3,14	23	0,69	3,65	6,24
1	0,01	1,06	3,23	24	0,72	3,87	6,45
2	0,03	1,12	3,32	25	0,78	4,11	6,67
3	0,04	1,18	3,41	26	0,84	4,37	6,90
4	0,06	1,25	3,51	27	0,91	4,64	7,14
5	0,08	1,32	3,61	28	0,98	4,93	7,40
6	0,10	1,39	3,71	29	1,06	5,25	7,67
7	0,12	1,47	3,82	30	1,15	5,59	7,95
8	0,14	1,55	3,93	31	1,24	5,95	8,24
9	0,16	1,64	4,05	32	1,34	6,34	8,55
10	0,18	1,73	4,17	33	1,44	6,76	8,88
11	0,21	1,83	4,29	34	1,55	7,22	9,22
12	0,23	1,94	4,42	35	1,68	7,71	9,58
13	0,26	2,05	4,55	36	1,81	8,24	9,97
14	0,29	2,17	4,69	37	1,95	8,81	10,37
15	0,32	2,30	4,84	38	2,11	9,44	10,80
16	0,36	2,43	4,99	39	2,28	10,11	11,25
17	0,39	2,57	5,15	40	2,46	10,85	11,73
18	0,43	2,73	5,31	41	2,66	11,64	12,24
19	0,47	2,89	5,48	42	2,88	12,51	12,79
20	0,51	3,06	5,66	43	3,12	13,46	13,37
21	0,56	3,24	5,84	44	3,38	14,50	13,98
22	0,61	3,44	6,04	45	3,66	15,64	14,64

d_1 – depth of foundation laying for non-slab structures from the planning level or the adjusted depth of laying external and internal foundations from the basement floor, m.

4. We specify the value of b_1 at R_1 .

We find b_2 at R_2 .

The difference between b_2 and b_1 , R_2 and R_1 is less than 10%, then we accept:

$$b_2 \text{ i } l_2 = 1,4b_2.$$

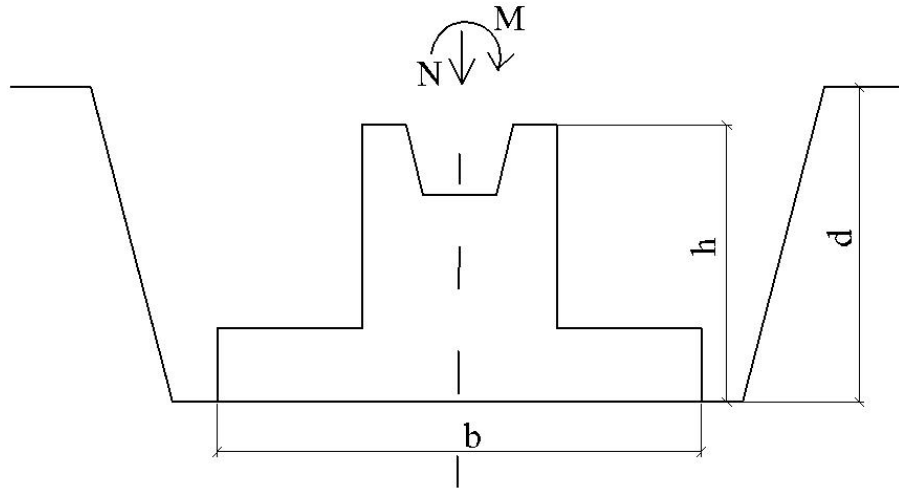


Figure 2.1 - Natural foundation

Checking the edge stresses

$$p_{\max} = \frac{N}{b \cdot l} + \gamma_{\text{mt}} \cdot d + q + \frac{M}{W} \leq 1,2R \quad (2.5)$$

$$p_{\min} = \frac{N}{b \cdot l} + \gamma_{\text{mt}} \cdot d + q - \frac{M}{W} > 0; \quad (2.6)$$

$$W = \frac{b \cdot l^2}{6} .$$

$$p_{\text{cp}} = \frac{N}{b \cdot l} + \gamma_{\text{mt}} \cdot d + q \leq R . \quad (2.7)$$

PRACTICAL LESSON 3

DETERMINING SETTLEMENT AND SUBSIDENCE OF THE FOUNDATION

1. Determination of foundation settlement by the method of layer-by-layer summation

The layer-by-layer summation method is based on the following assumptions:
the soil is a solid, isotropic, linearly deformable body;

settlement is caused only by the effect of vertical stress σ_z , other stress components are not taken into account;

lateral expansion of the soil at the base is impossible;

the stress σ_z is determined under the center of the foundation sole;

when determining the stress σ_z , the difference in compression of individual soil layers is neglected;

foundations do not have rigidity;

deformations are considered only within the compressible layer, H_c ;

the lateral expansion of the soil is taken into account by the coefficient β , which is equal to 0.8 regardless of the type of soil.

Due to the complexity of determining the law of stress reduction with depth, the soil stratum is divided into a number of layers, within which the pressure can be considered constant without a large error. Then the total settlement can be determined using the expression for the elementary layer, summing up the settlement of individual layers within the compressible stratum.

The calculation is performed in the following order:

1. The contour of the foundation is applied to the geological section.
2. Plot the stress σ_{zg} from the self-weight of the soil.
3. Determine the pressure p acting on the sole of the foundation
4. The layer below the base of the foundation is divided into elementary layers with a thickness of $h=0.4b$ (to facilitate interpolation). Within each type of soil, a whole number of elementary layers must be allocated, so the last elementary layer can be less than $0.4b$.

5. Determine the stress damping coefficients along the depth $\alpha = f(\frac{2z}{b}, \frac{\ell}{b})$ depending on the depth z and the ratio ℓ/b , where ℓ is the longer side of the foundation.

6. Build a plot of additional vertical stresses $\sigma_{zp} = \alpha \cdot p$

7. The lower limit of the compressible layer of the base is taken at the depth $z = H_c$, where the condition $\sigma_{zp} = k\sigma_{zg}$ is fulfilled, where $k = 0.2$,

$$\sigma_{zg} = \sum_{i=1}^n \gamma_i \cdot h_i$$

8. We build a graph of vertical stresses from the self-weight of the soil removed in the pit to the level of the foundation sole, at a depth of z:

$$\sigma_{zg} = \alpha_k \cdot \sigma'_{zg,0}$$

$\sigma'_{zg,0}$ is the vertical stress from the self-weight of the soil removed from the pit at the level of the base of the foundation and is equal to

$$\sigma'_{zg,0} = \gamma_{rp} \cdot d_n$$

9. We define the total settlement as the sum of the settlements of individual elementary layers according to the formula, m:

$$s = \beta \sum_{i=1}^n \frac{(\sigma_{zp,i} - \sigma_{zg,i}) \cdot h_i}{E_i} + \beta \sum_{i=1}^n \frac{\sigma_{zg,i} \cdot h_i}{E_{e,i}} \quad (3.1)$$

where β is a dimensionless coefficient equal to 0.8;

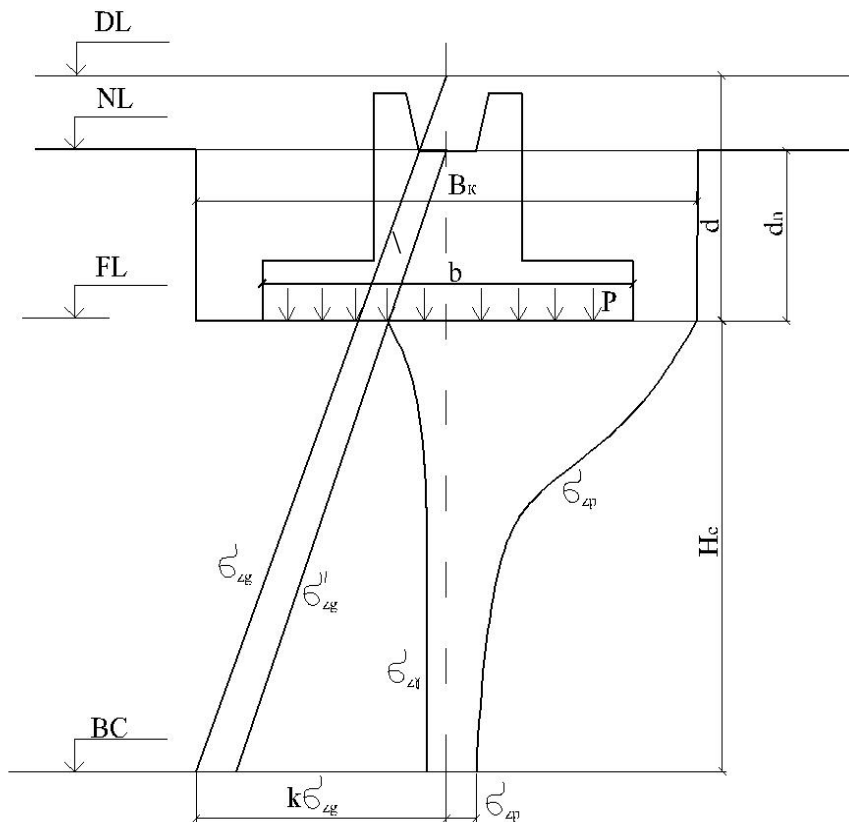
E_i – deformation modulus of the i-th soil layer along the branch of primary loading, kPa;

$E_{e,i}$ is the modulus of deformation of the i-th soil layer along the branch of secondary loading (modulus of elasticity), in the absence of data, it is possible to take $E_{e,i} = 5E_i$, kPa

When calculating the settlement of foundations erected in pits with a depth of less than 5 m, the second appendix in the settlement formula is not taken into account.

Table 3.1 - Coefficient α

ζ	Coefficient α for foundations							
	round	rectangular with the ratio of sides $\eta = l/b$,						strip ($\eta \geq 10$)
		1,0	1,4	1,8	2,4	3,2	5	
0	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
0,4	0,949	0,960	0,972	0,975	0,976	0,977	0,977	0,977
0,8	0,756	0,800	0,848	0,866	0,876	0,879	0,881	0,881
1,2	0,547	0,606	0,682	0,717	0,739	0,749	0,754	0,755
1,6	0,390	0,449	0,532	0,578	0,612	0,629	0,639	0,642
2,0	0,285	0,336	0,414	0,463	0,505	0,530	0,545	0,550
2,4	0,214	0,257	0,325	0,374	0,419	0,449	0,470	0,477
2,8	0,165	0,201	0,260	0,304	0,349	0,383	0,410	0,420
3,2	0,130	0,160	0,210	0,251	0,294	0,329	0,360	0,374
3,6	0,106	0,131	0,173	0,209	0,250	0,285	0,319	0,337
4,0	0,087	0,108	0,145	0,176	0,214	0,248	0,285	0,306
4,4	0,073	0,091	0,123	0,150	0,185	0,218	0,255	0,280
4,8	0,062	0,077	0,105	0,130	0,161	0,192	0,230	0,258
5,2	0,053	0,067	0,091	0,113	0,141	0,170	0,208	0,239
5,6	0,046	0,058	0,079	0,099	0,124	0,152	0,189	0,223
6,0	0,040	0,051	0,070	0,087	0,110	0,136	0,173	0,208
6,4	0,036	0,045	0,062	0,077	0,099	0,122	0,158	0,196
6,8	0,031	0,040	0,055	0,064	0,088	0,110	0,145	0,185
7,2	0,028	0,036	0,049	0,062	0,080	0,100	0,133	0,175
7,6	0,024	0,032	0,044	0,056	0,072	0,091	0,123	0,166
8,0	0,022	0,029	0,040	0,051	0,066	0,084	0,113	0,158
8,4	0,021	0,026	0,037	0,046	0,060	0,077	0,105	0,150
8,8	0,019	0,024	0,033	0,042	0,055	0,071	0,098	0,143
9,2	0,017	0,022	0,031	0,039	0,051	0,065	0,091	0,137
9,6	0,016	0,020	0,028	0,036	0,047	0,060	0,085	0,132
10,0	0,015	0,019	0,026	0,033	0,043	0,056	0,079	0,126
10,4	0,014	0,017	0,024	0,031	0,040	0,052	0,074	0,122
10,8	0,013	0,016	0,022	0,029	0,037	0,049	0,069	0,117
11,2	0,012	0,015	0,021	0,027	0,035	0,045	0,065	0,113
11,6	0,011	0,014	0,020	0,025	0,033	0,042	0,061	0,109
12,0	0,010	0,013	0,018	0,023	0,031	0,040	0,058	0,106



Conventional designations are as follows:

DL – planning mark;

NL – mark of the surface of the natural relief; FL – foundation sole mark;

B.C. - the lower limit of the compressible thickness;

d and d_n - the depth of laying the foundation sole from the level of the natural relief and the planning level;

p – average pressure under the sole of the foundation;

N is the net effect of external vertical forces on the edge of the foundation.

Fig. 3.1 Scheme for the determination of settlement by the method of layer-by-layer summation

Table 3.2 – Calculation of settlement

[illegible]
$$\Sigma S =$$

2. Calculation of subsidence

We perform the calculation in the following sequence:

1 We determine the thickness of the subsiding soil, in which it can be saturated with water to the degree of humidity $S_R = 0,9$.

We accept that the water can rise to the level of the foundation sole.

2 The entire layer is divided into calculated strips, preferably of the same thickness as during the settlement calculation $h_i \leq 0,4b$, but not more than 2 m and not less than 2 pcs.

3 We calculate the values and build a graph of the stresses in the subsiding soil layer from its own weight when the layer above is saturated with water, taking into account possible cutting or backfilling when planning the territory:

$$\rho_{sat} = \frac{\rho}{1+W} \cdot \left(1 + \frac{S_{r,sat} \cdot e \cdot \rho_w}{\rho_s} \right),$$
$$\sigma_{g,sat,i} = 10 \cdot \rho_{sat} \cdot h_i.$$

4 We determine the total stresses in the middle of the height of the calculated strips from the soil's own weight and from the pressure transmitted by the foundation:

$$\sigma_{\Sigma,i} = \sigma_{g,sat,i} + \sigma_{p,i}.$$

5 Using a graph of the change in $\varepsilon_{sl,i}$ from p_i , built earlier to determine the type of soil conditions relative to $\varepsilon_{sl,i}$, corresponding to the total pressures $\sigma_{\Sigma,i}$.

6 We calculate the subsidence of the base for each elementary layer of the subsiding soil, and then sum up the obtained values:

$$S_{sl} = \sum \varepsilon_{sl,i} \cdot h_i \cdot k_{sl,i}.$$

$k_{sl,i} = 1$, if $b \geq 12$ m, if $b \leq 3$ m, according to the formula:

$$k_{sl,3} = 0,5 + 1,5 \cdot \frac{p_{cp} - p_{sl}}{p_0},$$

where $p_0 = 100$ kPa.

If $3 < b < 12$ m – by interpolation.

$$k_{sl} = k_{sl,3} - \frac{k_{sl,3} - 1}{9} \cdot (b - 3).$$

The results of the calculations are also summarized in Table 3.2.

Table 3.2 - Calculation of base subsidence

$z_i = n \cdot h_i$, m	$\sigma_{zp} = \alpha \cdot p$, kPa	$\sigma_{g,sat,i}$, kPa	$\sigma_{p,i} + \sigma_{g,sat,i}$ kPa	σ_{Σ}^{cp} , kPa	ε_{sl}	h_i , m	k_{sl}	$S_{sl,i}$, m

$$\sum S_{sl} =$$

Total deformation of the base $S' = S + S_{sl}$.

PRACTICAL LESSON 4 FOUNDATION AT $R = P_{sl}$

This option is designed for cases where the subsiding soil may become saturated due to a rise in the groundwater level and/or saturation from external sources. All calculated soil characteristics are taken as the same as in the previous calculations, except for $R = p_{sl}$.

Similarly, we find the dimensions of the foundation base using formulas (2.1, 2.2). We check the edge stresses using formulas (2.5, 2.6). We do not calculate the settlement, as it is definitely within the normal range.

PRACTICAL LESSON 5 FOUNDATION ON WATER-SATURATED SUBSIDING SOIL

Elimination of the subsidence properties of soils can be achieved by soaking them. To completely eliminate subsidence during pre-soaking, the soil is also subjected to dynamic effects (hydroblasts, directional vibration, etc.).

Calculation characteristics required for the determination of b and R –

$N, \eta, \gamma_{mt}, d_1, q, \gamma_{cl}, \gamma_{c2}, k, k_z, \gamma_i, d_b$ the same as in the previous calculations, others take into account the change in soil properties during water saturation.

$R_{o,sat}$ determined according to the table B. 3;

$$c = c_{sat}, \quad \gamma = \gamma_{sat} = 10 \cdot \rho_{sat}.$$

Using φ_{sat} for determining M_γ , M_q , M_c according to Table 2.3.

Find the dimensions of the foundation sole using formulas (2.1) and (2.2).

Determine R_{sat} according to the formula:

$$R_{sat} = \frac{\gamma_{cl} \cdot \gamma_{c2}}{k} \cdot [M_\gamma \cdot k_z \cdot b \cdot \gamma_{II} + M_q \cdot d_1 \cdot \gamma'_{II} + (M_q - 1) \cdot d_b \cdot \gamma'_{II} + M_c \cdot c_{sat}].$$

Specify the value b_1 .

Find the value $R_{sat,2}$ at b_1 .

We find $R_{sat,3}$ i b_2 .

The discrepancy between b_2 i b_1 , $R_{sat,2}$ i $R_{sat,3}$ less than 10%, then we accept it: b_2 i $l_2 = 1,4b_2$.

Edge stresses are checked using formulas (2.4, 2.5).

Settlement is determined and is often greater than permissible, since the calculation uses E_{sat} . Subsidence of the lower layer due to its own weight is not determined if soil saturation occurred before the construction of the building, and is determined if it occurs during operation.

PRACTICAL LESSON 6

THE FOUNDATION IS ON PRE-COMPACTED SOIL

Elimination of subsidence properties of soils can be achieved by compacting them with heavy rammers.

The design of a foundation compacted by a heavy rammer includes the following stages:

- We specify the density of dry soil after tamping and the depth of laying the foundation:
- $\rho_{d.s.} = 1.65 \text{ t/m}^3$, $d = 1,8 \text{ m}$.

- We determine the lowering of the tamped surface, then the thickness of the compacted layer and the depth of excavation of the pit, after that - the diameter of the rammer and its weight.

- Lowering of the tamped surface, m

$$\Delta h = 1,2 \cdot (H_2 - d) \cdot \left(1 - \frac{\rho_d}{\rho_{d.s.}}\right),$$

where $\rho_d = \frac{\rho}{(1 + W)}$.

The thickness of the compacted layer, m

$$h_s = H_2 - d + \Delta h.$$

Depth of pit excavation, m

$$h_k = d - \Delta h.$$

Rammer diameter, m

$$d_s = h_s / k,$$

where for sandy loams and loams $k = 1,8$, for clays $k = 1,5$.

Rammer weight, t

$$Q = 1,5 \cdot d_s^2 \cdot \frac{\pi}{4}.$$

Optimum moisture during tamping

$$w_0 = w_p - (0,01 \div 0,03).$$

The height of the rammer $H = 8 - 10$ m, the number of blows per track 10-12 times.

Characteristics of loess loams after tamping

$$\rho_{t.s.} = \rho_{d.s.} (1 + w_0),$$

$$e_{t.s.} = \frac{\rho_s (1 + w_0)}{\rho_{t.s.}},$$

$$S_{r.t.} = \frac{\rho_s \cdot w_0}{e_{t.s.} \cdot \rho_w}.$$

Design of the foundation on a pre-compacted base.

- We find the width of the sole of the foundation by the formula at $\rho_{d.s.} = 1.65 \text{ t/m}^3$ and R_0 for compacted subsidence soil, m.

If $\varphi_{t.s.}$ i $c_{t.s.}$ for compacted soil have not yet been determined, then R is determined according to the formula.

If $d \leq 2\text{ м}$:

$$R_1 = 0.25 \cdot R_0 \cdot (d + 2) \cdot [1 + k_1(b - 1)], \text{ kPa}$$

If $d \geq 2 \text{ м}$: $k_1=0.05$, $k_2=0.2$ – for loams and sandy loams, $k_2=0.2$ – for clays

$$R = R_0 \cdot [1 + k_1(b - 1)] + k_2 \cdot \gamma_{II}(d - 2)$$

- Уточняемо b_1 i R_2 .
- The difference between b_2 and b_1 , R_2 and R_1 is less than 10%, we accept b_2 and l_2 .
- We perform a check of the marginal stresses.
- We determine the dimensions of the pit, m:

$$b_s = b + 0,5(b - d_s),$$

$$l_s = l + 0,5(l - d_s).$$

- The required amount of water for moistening, m^3 :

$$Q_w = \rho_{d,s}(w_0 - w) \cdot b_s \cdot l_s \cdot \frac{h_s}{\rho_w}.$$

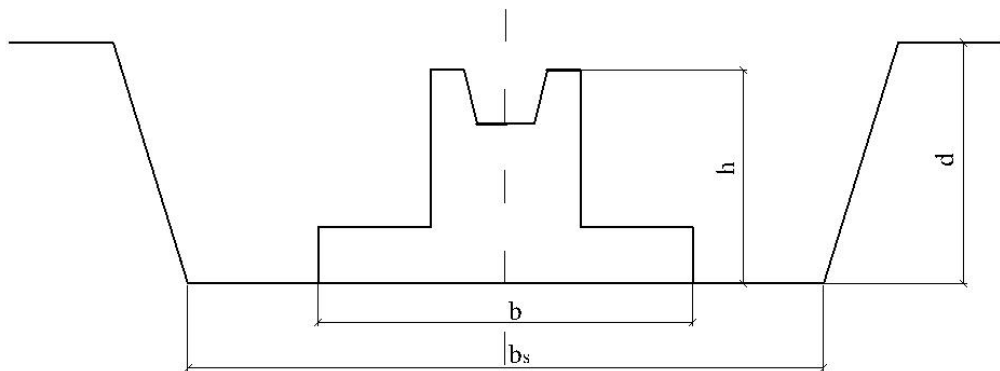


Figure 5.1 - Foundation on pre-compacted soil

PRACTICAL LESSON 7

FOUNDATION ON A SOIL CUSHION

The soil cushion replaces the subsiding soil completely (cutting) or only in the upper part (hanging). In the latter case, the overall deformation of the base, settlement, and subsidence must be within acceptable limits. Soil cushions are made with a thickness of 1.5-5 m, sometimes 10-12 m, and, as a rule, from local clay soil. In soils with subsidence, the cushions are poured in layers 0.1-0.35 m thick and compacted with rollers; when compacted with heavy tampers, the layers can be thicker.

The moisture content of the soil of the cushion should be optimal w_0 during tamping and $w_0 = w_p$ during rolling, the moisture content of the replacement soil can be any.

The dry soil density of the cushion should be $\rho_{d.s.} = 1.65 \text{ t/m}^3$ in liquidation of subsidence and $\rho_{d.s.} \geq 1.7 \text{ t/m}^3$ in the case of creating a waterproof screen after sealing.

When designing a soil cushion, the following are initially set $\rho_{d.s.}, d, R_0$.

We find the preliminary dimensions of the foundation base using formula (2.1), m. We find the estimated soil resistance R_0 using formula (2.2).

We find R_1 at $d \leq 2m$ by formula (5.10), kPa.

We clarify b_1 i R_2 . In case of discrepancy between b_2 and b_1 , R_2 and R_1 less 10%, accept b_2 .

We check the edge stresses using formulas (2.4, 2.5).

The thickness of the cutting cushion is taken, m:

$$h_s = H_{sl} - d.$$

Dimensions of the lower part of the cushion, m:

$$b_s = b(1 + 2k_h),$$

$$l_s = l + 2bk_h.$$

$k_h = 0,3$ at $p = 150 - 200 \text{ kPa}$, $k_h = 0,35$ at $p = 250 - 300 \text{ kPa}$, $k_h = 0,4$ at $p = 350 - 400 \text{ kPa}$.

In addition, the dimensions of the soil cushion should be larger than the foundation dimensions by at least 0.4 m at the bottom and 0.6 m at the top/

Next, the admissibility of the pressure transmitted by the cushion to the underlying soil is checked. To do this, we determine the area of the conditional foundation, m²:

$$A_z = \frac{N + G_f}{\sigma_{zp}},$$

where N – load on the foundation, kN,

$G_f = b \cdot l \cdot (\gamma_m \cdot d)$ - is the weight of the foundation and soil at its edges, kN.

σ_{zp} – additional stresses at depth $z = h_s$ from the load on the foundation, kPa:

$$\sigma_{zp} = \alpha \cdot P_0;$$

where p_0 is the average pressure under the foundation sole, kPa:

$$p_0 = \frac{N}{A} + d\gamma_{mt} + q.$$

Then we find it:

$$a = (l - b) / 2,$$

$$b_z = \sqrt{A_z + a^2} - a.$$

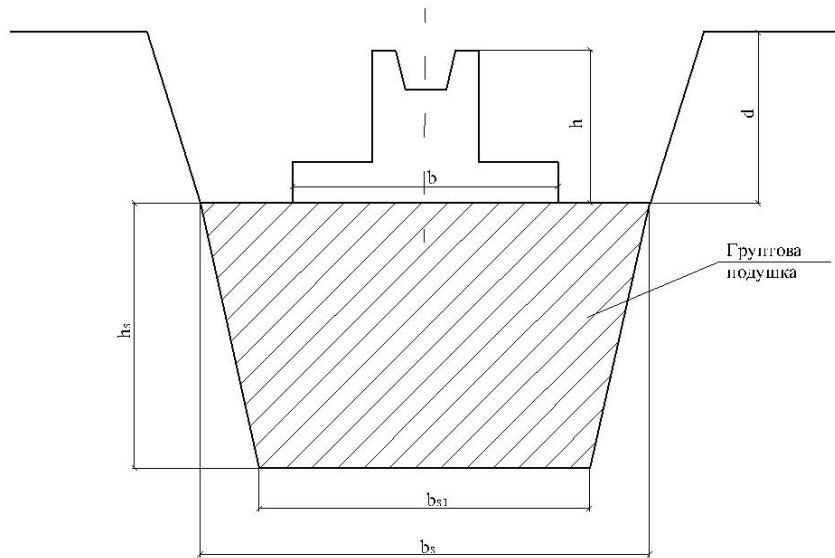


Figure 6.1 – Foundation on a soil cushion

Calculate the estimated resistance for the soil underlying the cushion, kPa

$$R_z = (M_\gamma \cdot k_z \cdot b_z \cdot \gamma_{II} + M_q \cdot (d + h_s) \cdot \gamma'_{II} + M_c \cdot c_{II}),$$

where γ_1, γ_2 and k are equal to 1.

Check the condition, kPa:

$$\sigma_{zp} + \sigma_{zg} < R_z,$$

where σ_{zg} – stress from the soil's own weight at a depth of $z = h_s$.

R_o is determined according to Table. 2.1.

For ϕ_{II} we find M_γ, M_q, M_c according to the table. 2.3.

Find the dimensions of the foundation sole by formula (2.1), m.

Determine R by formula (2.2).

Specify the values b_1 .

Find the value R_1 at b_1 .

We find R_2 i b_2 .

The discrepancy between b_2 and b_1 , R_2 and R_1 less 10%, then we finally accept it:

$$b_2 \text{ i } l_2 = 1,4b_2.$$

Perform edge stress checks using the formulas (2.4, 2.5).

After all the conditions are met, we finally construct the foundation.

PRACTICAL LESSON 8

CALCULATION AND DESIGN OF PILE FOUNDATIONS

Calculation of pile foundations based on bearing capacity. The calculation is performed in accordance with the requirements. When analyzing engineering-geological conditions, we identify a soil layer suitable for pile support. We select the type, kind, and method of pile driving – in this course project, driven prismatic reinforced concrete piles are used, and the method of driving is a diesel hammer. The depth of the pile cap base is 1.2...1.6 m. We determine the length of the pile, which depends on the engineering-geological and technological conditions, the method of connecting the pile

to the pile cap, and the depth of the pile into the bearing layer. In this case, we assume a rigid connection between the pile and the pile cap (300-350 mm), and the pile depth into the bearing layer is at least 1 m. Then the required pile length can be determined by the following relationship

$$L_{\pi} = (h_1 + h_2 + \dots + h_n) - d_p + 0,35 \text{ m} + 1 \text{ m}, \quad (8.1)$$

where h_1, h_2, h_n are the thicknesses of the layers through which the pile passes; d_p is the depth of the pile cap, m. After determining L_{π} , it is necessary to round this number to the nearest greater integer.

The cross-section of the piles should be 250 x 250, 300 x 300, or 350 x 350 mm. A single pile in the foundation and beyond its boundaries should be calculated based on the bearing capacity of the foundation soil, assuming that:

$$N \leq \frac{F_d}{\gamma_k}, \quad (8.2)$$

where N is the design load allowed on the pile when determining their number in the pile foundation;

F_d is the bearing capacity of the pile depending on the properties of the soil base;

γ_k is the safety factor (taken as 1.4 if the bearing capacity of the pile F_d is determined by calculation).

We determine the bearing capacity of a single friction pile by first dividing the soil thickness along the height of the pile into sections no more than 2.0 m long (Fig. 7.1):

$$F_d = \gamma_c(\gamma_{cr} \cdot R \cdot A + u \cdot \sum \gamma_{cf} \cdot f_i \cdot h_i), \quad (8.3)$$

where F_d – bearing capacity of a single friction pile in kN;

γ_c – coefficient of the pile working condition on the ground, taken equal to 1.0;

R – estimated soil resistance under the lower end of the pile, kPa, taken from Table H.2.1 [4] or Table 7.1;

Table 8.1 – Estimated resistance values under the lower end of driven piles and casing piles that are driven without soil removal

Immersion depth of the lower end of the pile, m	Estimated resistance under the lower end of driven piles and piles-shells, are immersed without soil excavation R, kPa						
	sandy soils of medium density						
	gravelly	large	-	Medium-sized	fine	dusty	-
	of clayey soils with a liquidity index of I_L						
	0	0,1	0,2	0,3	0,4	0,5	0,6
3	7500	6600 / 4000	3000	3100 / 2000	2000 / 1200	1100	600
4	8300	6800 / 5100	3800	3200 / 2500	2100 / 1600	1250	700
5	8800	7000 / 6200	4000	3400 / 2800	2200 / 2000	1300	800
7	9700	7300 / 6900	4300	3700 / 3300	2400 / 2200	1400	850
10	10500	7700 / 7300	5000	4000 / 3500	2600 / 2400	1500	900
15	11700	8200 / 7500	5600	4400 / 4000	2900	1650	100
20	12600	8500	6200	4800 / 4500	3200	1800	1100
25	13400	9000	6800	5200	3500	1950	1200
30	14200	9500	7400	5600	3800	2100	1300
35	15000	10000	8000	6000	4100	2250	1400

Notes:

1 Before the dash, the R values are for sandy soils, after the dash – for clayey soils.

2 For intermediate values of pile immersion depth and intermediate values of the I_L index, the R value is determined by interpolation.

A is the area of the pile bearing on the soil, m^2 , taken as the gross cross-sectional area of the pile or the net cross-sectional area of the pile shell;

U is the outer perimeter of the pile cross-section, m;

γ_{cR} and γ_{cf} – soil working condition coefficients under the lower end and along the side surface of the pile, respectively, which take into account the influence of the pile immersion method on the estimated soil resistance and are taken from Table 8.2.

f_i – estimated resistance of the i -th soil layer of the foundation along the side surface of the pile, kPa, determined by formula (7.4), for preliminary calculations it is permissible to take from Table 8.3;

Table 8.2 - Working conditions coefficients

Methods of immersion driven piles and shell piles that are sunk without excavation, and types of soils	Coefficients of soil conditions for calculating the bearing capacity of piles	
	under the lower end γ_{cR}	on the lateral surface γ_{cf}
1 Immersion of solid and hollow piles with a closed lower end by mechanical (suspended), steam-air and diesel hammers	1,0	1,0
2 Immersion by hammering and pressing into pre-drilled pilot wells with the pile ends being buried at least 1 m below the wellhead at its diameter:		
a) is equal to the side of a square pile;	1,0	0,5
b) 0.05 m less than the side of the square pile;	1,0	0,6
c) 0.15 m less than the side of a square or the diameter of a pile of round cross-section (for power transmission line supports).	1,0	1,0
3 Immersion with washing in sandy soils, provided that the piles are finished at the last stage of immersion without using washing for 1 m or more	1,0	0,9
4 vibration immersion of shell piles, vibration immersion and vibration pressing of piles into the soil:		
a) sands of medium density:		
- large and medium-sized;	1,2	1,0
- small;	1,1	1,0
- dusty;	1,0	1,0
b) dusty-clay with a liquidity index $I_L = 0.5$:		
- sandy loam;	0,9	0,9
- loam;	0,8	0,9
- clay;	0,7	0,9
c) dusty-clay with a liquidity index $I_L \leq 0$.	1,0	1,0
5 Hammer sinking of any structure of hollow reinforced concrete piles with an open lower end:		
a) when the diameter of the pile cavity is 0.4 m or less;	1,0	1,0
b) the same from 0.4 to 0.8 m.	0,7	1,0
6 Immersion by any method of hollow piles of round cross-section with a closed lower end to a depth of 10 m or more with subsequent installation at the lower end of piles of camouflaged expansion in sandy soils of medium density and in silty-clay soils with a liquidity index $I_L \leq 0.5$ with a diameter expansion equal to:		

a) 1.0 m regardless of the specified types of soil;	0,9	1,0
b) 1.5 m in sand and sandy loam;	0,8	1,0
c) 1.5 m in loams and clays;	0,7	1,0
7 Immersion by driving piles:		
a) sands of medium density, large, medium-sized and small;	1,1	1,0
b) in dusty sands;	1,1	0,8
c) in dusty-clay soils with a liquidity index $I_L < 0.5$;	1,1	1,0
d) the same, $I_L > 0.5$.	1,0	1,0

Table 8.3 – Values of estimated resistance on the side surface of driven piles and pile casings

Average depth of the soil layer, m	Estimated resistance on the lateral surface of driven piles and pile casings f , kPa								
	sandy coarse and medium-sized and clayey at $I_L=0,2$	sandy, fine clayey soils with $I_L=0,3$	sandy, dusty, and clayey soils $I_L=0,4$	clayey with a liquidity index I_L					
				0,5	0,6	0,7	0,8	0,9	1,0
1	35	23	15	12	8	4	4	3	2
2	42	30	21	17	12	7	5	4	4
3	48	35	25	20	14	8	7	6	5
4	53	38	27	22	16	9	8	7	5
5	56	40	29	24	17	10	8	7	6
6	58	42	31	25	18	10	8	7	6
8	62	44	33	26	19	10	8	7	6
10	65	46	34	27	19	10	8	7	6
15	72	51	38	28	20	11	8	7	6
20	79	56	41	30	20	12	8	7	6
25	86	61	44	32	20	12	8	7	6
30	93	66	47	34	21	12	9	8	7
35	100	70	50	36	22	13	9	8	7

Notes:

1 When determining the value of f , soil layers should be divided into homogeneous layers with a thickness of no more than 2 m.

2 The table is compiled for sandy soils of medium density.

3 The estimated resistance of dense sandy soils on the side surface of piles f should be increased by 30% compared to the values given in this table.

h_i – is the thickness of the i -th soil layer in contact with the side surface of the pile, m;

$$f_i = \sigma_{zg,i} \frac{v_i}{1 - v_i} \operatorname{tg} \varphi_{II,i} + c_{II,i}, \quad (8.4)$$

where $\sigma_{zg,i}$ – is the stress from the self-weight of the soil in the middle of the i -th design layer of the soil base;

v_i – Poisson's ratio of soil in the middle of the i -th design layer of the soil base, equal to 0.21 ... 0.29 for sands, 0.3 ... 0.37 for sandy loams and loams, and 0.4 for clays. When the soil is fully saturated with water, the design characteristics of the soil should be taken at the liquidity index determined by the formula:

$$I_L = \frac{0,9 \cdot e \cdot \frac{\gamma_w}{\gamma_s} - w_p}{w_L - w_p}, \quad (8.5)$$

where e is the porosity coefficient of soil of natural density,

γ_w – bulk density of water, $\gamma_w = 10 \text{ кН/м}^3$;

γ_s – bulk weight of soil particles, кН/м^3 ;

w_L, w_p – moisture content at the yield and rolling point, respectively.

If $I_L < 0,4$ should be taken $I_L = 0,4$.

If $I_L > 1$ resistance on the side surface is not taken into account.

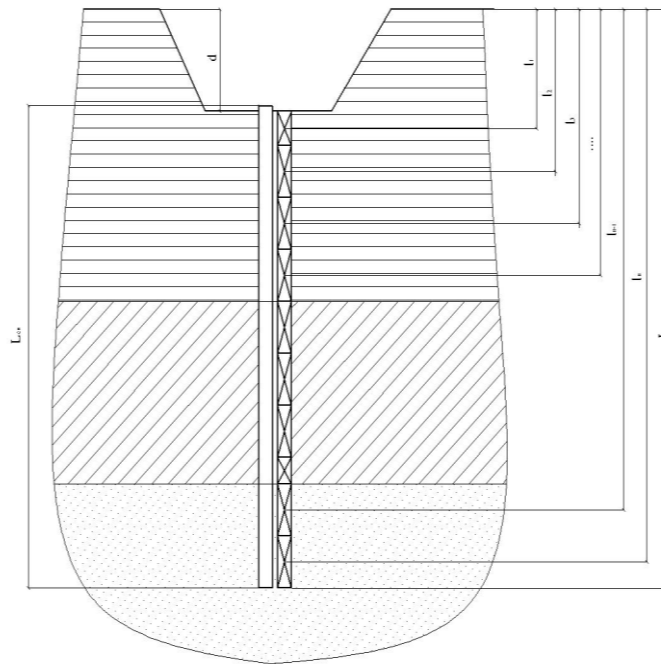


Fig. 8.1 - Design scheme of the pile

Piles based on the bearing capacity of the foundation soil in subsiding soils should be calculated based on the condition:

$$N \leq \left(\frac{F_d}{\gamma_k} \right) - \gamma_c \cdot P_n \quad (8.6)$$

where N is the design load, kN, per pile; F_d is the design bearing capacity of the soil at the base of a single pile.

γ_k - reliability factor (taken as 1.4 if the bearing capacity of the pile F_d is determined by calculation);

γ_c - coefficient of working conditions, depending on soil subsidence:

at $S_{sl} = 5 \text{ sm}$ - $\gamma_c = 0$, at $S_{sl} \geq 2 S_u$ - $\gamma_c = 0.8$ for intermediate values S_{sl} - γ_c is determined by interpolation;

P_n - loading friction force.

$$P_n = u \sum_0^{h_{sl}} \tau_i \cdot h_i \quad (8.7)$$

where u - pile perimeter, m;

h_{sl} - design depth, m, within which the forces of lateral friction of subsiding soil layers are summed up, equal to the depth of the sole of the subsidence thickness due to the action of its own weight;

τ_i - estimated soil resistance, kPa, determined to a depth of 6 m using the formula:

$$\tau_i = \xi \sigma_{zg,i} \tan \varphi_i + c_i \quad (8.8)$$

ξ - a lateral pressure coefficient equal to 0,7;

φ_i , c_i - calculated values of the angle of internal friction and specific soil cohesion, averaged over depth h_i ;

σ_{zg} - vertical stress from the weight of water-saturated soil, kPa:

h_i - thickness, m, of the soil layer that subsides during soaking and is tangent to the side surface of the pile.

At a depth of 6 m $< h < h_{sl}$ τ_i is assumed to be constant, equal to the value of τ_i at a depth of 6 m.

Determine the required number of piles in the bush:

$$n = \frac{F_v}{N}. \quad (8.9)$$

We design the pile cap, taking into account that the distance between the pile axes is taken as (3...6)d, and the distance from the edge of the pile cap to the axis of the outer pile is d, where d is the side of the pile. We assume the height of the pile cap to be within the range of 500...600 mm. Since not only longitudinal force but also bending moment acts on the pile foundation, it is necessary to check the load on the outer pile, taking into account the action of the bending moment. The value of this load is determined by the formula

$$N_{\max} = \frac{N_d}{n} \pm \frac{M_x y}{\sum y_i^2} \pm \frac{M_y x}{\sum x_i^2}; \quad (8.10)$$

where N_d – calculated compressive force, kN, which includes: vertical load from upper structures, weight of pile cap weight of soil on its edges, and weight of piles; n – number of piles in the foundation, pcs; x_i, y_i – distance from the main axes to the axis of each pile, m;

M_x, M_y – calculated bending moments, kNm, relative to the main central axes x and y , the plan of the piles in the plane of the foundation base;

x, y – distance from the main axes to the axis of each pile for which the calculated load is calculated, m.

Since the loads on the extreme piles are determined taking into account the action of the moment, the design load is taken to be equal to 1.2 N (the short-term nature of the moment is taken into account).

A check is performed:

$$N_{\max} \leq \frac{1,2F_d}{\gamma_k}. \quad (8.11)$$

Calculation of pile foundations based on deformations

This calculation is performed for a conditional foundation on natural soil. We determine the contours of the conditional foundation (Fig. 8.2).

The boundaries of the conditional foundation are:

- at the bottom – limited by plane AB;
- on the sides – by vertical planes AB and BC, distant from the outer edges of the extreme rows of vertical piles at a distance $htg\alpha$, but not more than $2d$ in cases where dusty-clayey soils with a fluidity index $I_L > 0.6$ lie under the lower ends of the piles;
- at the top – by the leveling surface BG.

The average value of the angle of internal friction of the soils penetrated by the pile is determined, degrees:

$$\varphi_{II,mt} = \frac{\sum_{i=1}^n \varphi_{II,i} \cdot h_i}{\sum h_i}, \quad (8.12)$$

where is h_i the thickness of the i -th layer in contact with the side surface of the pile.

Average value of the angle of internal friction of soils, deg:

$$\alpha = \frac{\varphi_{II,mt}}{4}. \quad (8.13)$$

Width of the conditional foundation, m

$$B_{y\phi} = y + z + 2 \cdot h \cdot tg\alpha, \quad (8.14)$$

where y is the distance between the extreme piles, z is the width of the pile.

The length of the conditional foundation:

$$L_{y\phi} = x + z + 2 \cdot h \cdot tg\alpha, \quad (8.15)$$

where x is the distance between the outermost piles.

Standard weight of piles:

$$G_{CB}^H = n(L_{II} \cdot G_{II,1} + G_B), \quad (8.16)$$

where n is the number of piles, pcs,

L_{Π} – pile length, m,

$G_{\Pi,1}$ – weight of 1 m of pile, kN,

G_B – is the weight of the pile tip.

Normative weight of soil in the volume of the ABWG, kN:

$$G_{rp,y\phi}^H = B_{y\phi} L_{y\phi} \sum \gamma_i h_i. \quad (8.17)$$

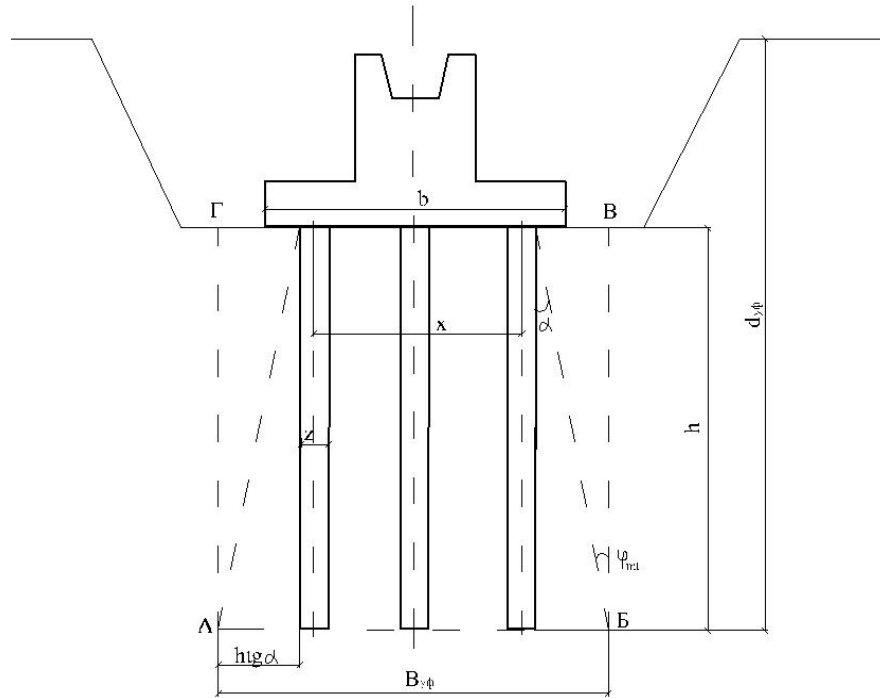


Figure 8.2 - Design scheme of a pile foundation by deformation

Average pressure under the sole of the conditional foundation:

$$P'' = \frac{N^H + G_p^H \cdot 1,1 + G_{\Pi}^H + G_{rp}^H}{B_{y\phi} \cdot L_{y\phi}}. \quad (8.18)$$

where N^H – standard vertical load on pile, kN,

G_p^H – standard weight of the pile cap, kN,

G_{Π}^H – standard weight of piles, kN,

G_{rp}^H – normative soil weight in the ABVG volume, kN.

Estimated soil resistance under the base of a conditional foundation, kPa.

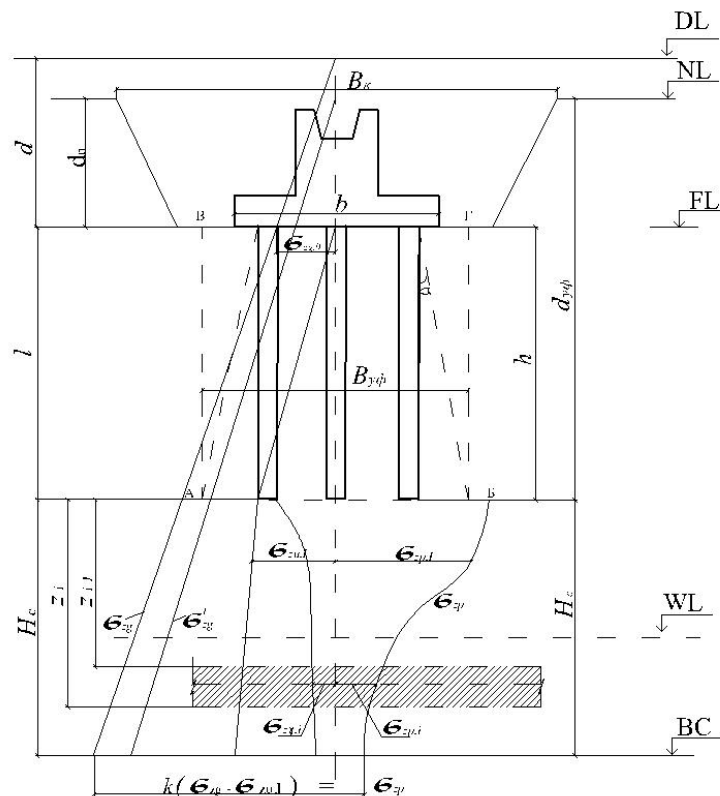
$$R = \frac{\gamma_{c1} \cdot \gamma_{c2}}{k} \cdot (M_{\gamma} \cdot B_{y\phi} \cdot \gamma_{\Pi} + M_q \cdot d_{y\phi} \cdot \gamma'_{\Pi} + M_c \cdot c_{\Pi}). \quad (8.19)$$

The basic condition is checked during the calculation of the pile foundation according to the second group of limit states.

$$p_{II} < R. \quad (8.20)$$

Calculation of the settlement of the pile foundation

The settlement of pile foundation is calculated using formula (3.1) in the same way as the settlement of foundation on natural base.



Buf – width (diameter) of the conditional foundation at the mark of lower end of piles with length l , formed at an angle $\varphi/4$ from the base of the pile cap; $\sigma_{z,ul}$ – stress from the soil's own weight at the mark of the conditional foundation, starting from the base of the pile cap; other designations according to Fig. 3.1

Figure 8.3 – Diagram of the distribution of vertical stresses at the base of the pile foundation according to the model of a linearly deformed half-space for calculation according to the conditional foundation diagram

LIST OF RECOMMENDED SOURCES

1. DSTU B A.1.1–25–94. Soils. Terms and definitions. – [Effective from 1994–10–01]. – Kyiv: Derzhbud Ukrainy, 2001. – 52 p.
2. DSTU B V.2.1–2–96 (GOST 25100–95). Bases and basements of buildings and structures. Soils. Classification. – [Effective from 1997-04-01]. – Kyiv: State Committee of Ukraine for Urban Development and Architecture, 1997. – 47 p.
3. DSTU B V.2.1-17:2009 Foundations and bases of buildings and structures. Soils. Methods for laboratory determination of physical properties – [Effective from 2010-10-01]. – Kyiv: State Committee of Ukraine for Urban Development and Architecture, 2010. – 36 p.
4. DBN V.2.1–10–2009. Amendment No. 1. Bases and foundations of structures. Basic design provisions. – [Effective from 2011–07–01]. – Kyiv: Ministry of Regional Development and Construction of Ukraine, 2011. – 55 p.
5. DBN V.2.1–10:2018. Bases and foundations of buildings and structures. Basic provisions. – [Effective from 2019–01–01]. – Kyiv: Ministry of Regional Development and Construction of Ukraine, 2018. – 40 p.
6. DSTU-N B V.1.1-27:2010 Protection against hazardous geological processes, harmful operational influences, and fire. Building climatology. – [Effective from 2011–01–01]. – Kyiv: Ministry of Regional Development and Construction of Ukraine, 2011. – 127 p.
7. DBN A.2.1–1–2008. Engineering surveys for construction. – [Effective from 2008–07–01]. – Kyiv: Ministry of Regional Development and Construction of Ukraine, 2008. – 74 p.
8. DBN V.2.6-98:2009 Structures of buildings and structures. Concrete and reinforced concrete structures. Basic provisions. – Kyiv, 2008. – 78 p.
9. DBN V.1.1-5-2000. Buildings and structures on subsiding territories and soils of subsidence. Part 1. Buildings on subsiding territories. Part 2. Buildings and structures on soils of subsidence / State Construction Committee of Ukraine. Effective from 1.07.2012

- Kyiv: State Committee for Construction, Architecture and Housing Policy of Ukraine, 1999. – Part 1 – 65 p., Part 2 – 84 p.
10. Bases, foundations, and underground structures: Designer's reference book / M. I. Gorbunov-Posadov, V. A. Ilyichev, V. I. Krutov, et al.; edited by E. A. Sorochan. – Moscow: Stroizdat, 1985. – 480 p.
11. Soil Mechanics, Bases and Foundations: Textbook / L. M. Shutenko, O. G. Rud, O. V. Kichaeva, O. V. Samorodov, O. V. Gavriilyuk; edited by L. M. Shutenko; translated from Russian. – Kharkiv: O. M. Beketov National University of Civil Engineering, 2017. – 563 p. – An electronic version is available. (Access mode: <https://eprints.kname.edu.ua/45175/>, free).
12. Zotsenko M.L. et al. Engineering geology, soil mechanics, bases and foundations. – Poltava, 2003. – 550 p.
13. Manual for the design of bases for buildings and structures (to SNiP 2.02.01-83). NIIOSP named after N.M. Gersevanov – Moscow, Stroizdat, 1986. – 415 p.
14. Construction in difficult engineering and geological conditions: lecture notes for full-time and part-time students of the second (master's) level of higher education in the specialty 192 – Construction and Civil Engineering / O. V. Kichaeva; O. M. Beketov National University of Urban Economy in Kharkiv. – Kharkiv: O. M. Beketov National University of Urban Economy, 2020. – 51 p. <https://eprints.kname.edu.ua/57677/>.
15. Soil Mechanics. Basics and Foundations: Textbook for Students / V. B. Shvets, I. P. Boiko, Yu. L. Vinnikov et al. – Dnipropetrovsk: Porohy, 2012. – 197 p.
16. Soil Mechanics, Bases and Foundations: Textbook for Students / S. B. Ukhov, V. V. Semenov, V. V. Znamensky, et al. – Moscow: ASV, 1994. – 527 p.
17. Khrapata, I., Lyapin, O. (2025). Investigation of processes occurring in swelling soils depending on natural moisture content. Bases and Foundations, (50), 316–322. <https://doi.org/10.32347/0475-1132.50.2025.316-322>.
18. O.I. Bondarenko, G.G. Strizhelchik, I.V. Khrapata, E.P. Vasilenko, O.E. Lyapin. Features of foundation construction in flooded and potentially flooded areas. Collection of scientific works of the Ukrainian State University of Water Transport. Issue 210, 2024.

<http://lib.kart.edu.ua/handle/123456789/28317>

19. Genadiy Strizhelchik, Irina Khrapatova, Oleksandr Liapin and Olexandr Bondarenko. Geotechnical risks in the reconstruction of buildings and structures affected by explosion. IOP Conference Series: Earth and Environmental Science, Volume 1499, International Conference on Urban Infrastructure Sustainable Development and Renovation (MistoBUD-2025) 30/01/2025 - 01/02/2025 Kharkiv, Ukraine. <https://doi.org/10.1088/1755-1315/1499/1/012019>.

20. Genadiy Strizhelchik, Viacheslav Iegupov, Irina Khrapatova Ensuring urban resilience construction in difficult engineering and geological conditions (geotechnical view) IOP Conference Series: Earth and Environmental Science, Volume 1499, International Conference on Urban Infrastructure Sustainable Development and Renovation (MistoBUD-2025) 30/01/2025 - 01/02/2025 Kharkiv, Ukraine. <https://doi.org/10.1088/1755-1315/1499/1/012048>.

21. Strizhelchik G., Khrapata I., Shokarev V., Lyapin O. On the issue of geotechnical risks in the restoration of buildings destroyed by explosions in the south-eastern and southern parts of Ukraine. – Collection of materials from the international scientific and practical conference “Safe, comfortable, and capable territorial community” (October 11-13, 2023), 2023. Pp. 43-44.

<https://science.nmu.org.ua/ua/conferences/Forum/Zbirnyk.pdf>

Electronic educational publication

Irina Khrapatova

Methodical recommendations
for practical classes, for making calculations
and graphic work
and organizing independent work
on an academic discipline

**«CONSTRUCTION IN DIFFICULT ENGINEERING-GEOLOGICAL
CONDITIONS»**

*(for full-time foreigner students
second (Master's) level of higher education
specialty G19 Building and Civil Engineering)*

Responsible for the issue I. Khrapatova
Editor
Computer layout I. Khrapatova

Plan 2025, pos.

Signed for printing. Format 60 × 84/16.
Page limit.

Publisher and manufacturer: O.M. Beketov
Kharkiv National University of Urban Economy,
17 Chornohlazivska Str., Kharkiv, 61002. E-mail
address: office@kname.edu.ua Certificate of a
publishing entity: DK No. 5328 dated 11.04.2017.