ON THE ENGINEERING METHOD OF PREDICTING FOUNDATION SETTLEMENT AND ITS APPLICATION

SUR LE DEVELOPEMENT DE LA METHODE TECHNIQUE POUR LA PREDICTION DES TASSEMENTS DES FONDATIONS ET SON APPLICATION

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Summary

I. The new developments in the engineering method of predicting foundation settlement, proposed in this paper, making use of a procedure of reducing a complex three-dimensional space problem of the theory of filtration consolidation and soil creep to an equivalent one-dimensional problem, enables, not only the dimensions, rigidity and shape of the foundation to be taken into account, but also the limited condition for lateral soil expansion, and the deformability of all components making up the soil.

Determinations of foundation settlement and their rate in time are carried out by means of very simple equations, derived in the closed form by the equivalent layer method.

II. Foundation settlements depend upon four components: the settlement due to bed soil consolidation; soil settlement and soil swelling as a result of decreased load during the period of pit excavation; settlement due to soil flow to the sides from under the edges of the foundation; and the settlement due to the loss of structure under the action of meteorological factors, ground water, dynamic effects and errors made by the builders.

Averaging the characteristics of soil deformation in the foundation base, according to the equivalent layer method, enables a simple method to be devised for taking into account the influence of the loading of adjacent foundations in investigating the deformation within the boundaries of a limited strata subject to compaction, i.e. the first component of settlement

Introduction.

Of great importance when structures are built on a strata of highly compressible soils are the prediction of foundation settlement and constructive measures that can be resorted to equalize the irregularity of the settlement so as to reduce its effect on the deformation of the bearing structures.

These problems are considered in the two parts of this paper.

The first part (by N.A.Tsytovich) is devoted to the development of the engineering method of predicting the settlement of structures on clayey-soils by calculating the compaction deformations of an e q u i v a l e n t l a y e r of soil. This permits the complicated space problem of the consolidation and creep theory of clayey soils to be reduced to an equivalent one-dimensional problem.

The second part (by B.I.Dalmatov) considers the reasons for irregular foundation settlement calculation of the settlement irregularity, taking into account the influence of adjacent foundations and the limits, of the layer of the compressed soils, and constructive measures for minimizing irregular settlement.

I. T h e d e v e l o p m e n t o f t h e e q u i v a l e n t l a y e r m e t h o d o f p r e d i c t i n g s e t t l e m e n t s.

The most important properties of clayey and silty soils - those that determine the magnitude and the rate of their settlements in time - are the overall deformability of the soils (as quasi-single-phase soils in the case of undisturbed rigid structural bonds, and as multicomponent compression-compacted bodies in the case when bonds have been disturbed under load) and, for a water-saturated state, the filtrating capacity and the deformability of all the components making up the soils. These components are: hard mineral particles, pore water and gaseous components (vapours and gases).

The general deformability of the soil, as is known, is evaluated by compression tests. Here it should be noted that it is essential to determine not only the factor of relative (volumetric) compressibility ($m_v = \Delta e/(1+e)\Delta q$, where e is the void ratio of the soil and q is the magnitude of external pressure), but also the so-called structural compressive strength (p_{str}), which is used in subsequent calculations. The structural compressive strength can be determined from the compression curve, if the soil is first loaded in very small steps - about 0.02 to 0.10 kg per sq.cm. Then, as the corresponding tests show, the deformation is very small up to a certain magnitude of external pressure (Fig.1), completely

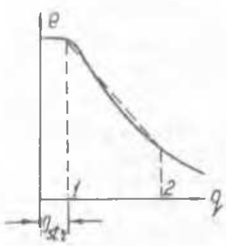

Fig.1.Determining structural strength by the compression curve (p_{str})

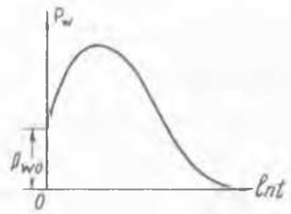

Fig.2.Determining the coefficient of initial pore pressure (β_o)

reversible and is due to the deformability of the rigid structural bonds. At an external pressure larger than a certain definite value for a given soil, that shall be called the structural compressive strength

A very important characteristic of the compaction of fully saturated clayey soils is the so-calles coefficient of the initial pore pressure (β_o), indicating what part of the external pressure (q) is taken up by the pore water at the moment of loading.

It is equal to $\beta_o = P_{wo}/q$, (1)

where P_{wo} is the initial pore pressure value.

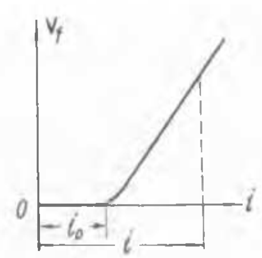

Fig.3. Determining the initial head gradient (i_o).

Figure 3 shows the relationship between filtration velocity and the head gradient for ductile (sticky) precompacted clayey and silty soils. As is evident from this curve, there will be practically no filtration and, therefore, no filtration consolidation of the soil, up to a certain value of the head gradient, which we shall call the i n i t i a l v a l u e (i_o)

In accordance with numerous experimental data (Mesohyan,1967; Tsytovich, Zaretsky, et al, 1967) the stressed-deformative state of the s o i l s k e l e t o n is well represented by the Boltzmann-Volterra linear theory of hereditary creep.

For the relative compressive strain (λ) we can write:

$$\lambda = \frac{1}{E}\left[\sigma(t) + \int_0^t \bar{K}(t,\tau)\sigma(\tau)d\tau\right] ; \qquad (2)$$

where E = instantaneous deformation modulus;
$K(t,\tau)$ = the creep nucleus and
$K(t,\tau) = E \cdot \bar{K}(t,\tau)$;
σ = stress.

In the case a t t e n u a t i n g soil skeleton c r e e p, the creep nucleus becomes

$$K(t,\tau) = \delta \cdot e^{-\delta_1 t} ; \qquad (3)$$

where δ and δ_1 = experimentally determined creep parameters.

Under natural conditions, the pore water in clayey and silty soils, occuring even below the level of the subsoil water, always contains a certain amount of air and vapour bubbles and always has a compressibility, characterized by the factor of relative compressibility (m_w)

It can be shown (Ter-Martirosyan, Tsytovich 1965) that the value m_w is determined by a very simple formula, namely:

$$m_w = (1 - J_w)/P_a ; \qquad (4)$$

where P_a = atmospheric pressure;
J_w = coefficient (degree) of water saturation.

As concerns the deformability of the pore air bubbles themselves, the modulus of their compressibility can be neglected, e.i. $1/m_a \approx 0$

As has been previously shown by the author (Tsytovich, 1961, 1963), the complex three-dimensional problem, encountered in determining the base settlement of structures, can be reduced to the calculation of the settlement of a certain equivalent soil layer, whose settlement (in the case of one-dimensional compression), will exactly equal the settlement of a foundation of the given dimensions.

The total stabilized settlement of a linearly deformable half-space, upon the action of a localized load, is determined, according to Boussinesq-Schleicher, by the following exact equation

$$S = \omega b (1 - \mu_o^2)\, q / E_o \qquad (5)$$

where ω = coefficient of the shape and the rigidity of the foundation
b = width of the rectangular area of the footing
E_o, μ_o = modulus of overall defomability and the coefficient of relative lateral soil expansion, analogous to Poisson's ratio.

Considering that the modulus of overall deformability upon compression without lateral expansion is equal to

$E_o = \beta / m_v$ where $\beta = [1 - 2\mu_o^2/(1 - \mu_o)]$

and substituting in equation (5) we obtain:

$$S = [A\omega b]\, m_v \cdot q \qquad (6)$$

$$A=(1-\mu_o)^2/(1-2\mu_o) \quad (6)$$

where $A=(1-\mu_o)^2/(1-2\mu_o)$.

The expression inside the square brackets on the right-hand side of equation (6) comprises a certain linear value, the so-called e q u i v a l e n t s o i l l a y e r, whose settlement is exactly equal to the settlement of a foundation of given dimensions and shape, taking into account the whole stressed zone under the foundation. Denoting the thickness of the equivalent soil layer by h_s, that is:

$$h_s=A\omega b \; ; \quad (7)$$

then the complete stabilized foundation settlement is equal to:

$$S_{\infty}=h_s \cdot m_v \cdot q \; ; \quad (8)$$

The larger the area of the foundation footing and its width, the larger is the equivalent soil layer and the magnitude of the active zone of soil compression affecting the settlement of a foundation of the given dimensions.

Detailed auxiliary tables have been worked out (Tsytovich, 1963,1968), for determining the magnitude of the equivalent soil layer and, in turn, the active compression zone. One of these tables, in an abbreviated form, is given below for soils, that are characterized by the coefficient of relative lateral expansion $\mu_o = 0.3$.

For other values of μ_o the given values ($A\omega$) should be multiplied by a constant factor: by Q=0.83, when μ_o =0.1 and by Q = 1.46 when μ_o=0.4.

Table 1

Values of $A\omega$ at μ_o= 0.3

Equivalent soil layer factor	Ratio of the sides of the footing area					
	1	1.5	2	3	5	10
$A\omega_m$	1.17	1.40	1.60	1.89	2.25	2.77
$A\omega_c$	0.687	0.832	0.938	1.092	1.289	1.558
$A\omega_h$	0.83	1.00	1.13	1.29	1.44	1.59

Table 1 lists the equivalent soil layer factors for the following cases:

- $A\omega_m$ - equivalent soil layer factor for a mean settlement of rigid (massive) foundations on a h o m o g e n e o u s h a l f-s p a c e
- $A\omega_c$ - equivalent soil layer factor for corner points of a rectangular loading area, used in determining the settlement of any point of a half-space of the "c o r n e r p o i n t m e t h o d",
- $A\omega_h$ - equivalent soil layer factor for rigid foundations upon the occurrence of incompressible rock at a depth equal to the thickness of the active compressive zone (according to B.I.Dalmatov).

The maximum value of the active compressive zone (Tsytovich, 1951,1963,1968) in engineering design can be taken equal to twice the thickness of the equivalent soil layer, i.e.,

$$max\, h_a = 2h_s \quad (9)$$

Indeed, if the mean pressure over the whole active compressive zone is roughly taken equal to $q_{aver.} \approx q_o/2$ the settlement of a layer of soil $2h_s$ in thickness will be determined by the previous equation (8).

As has been previously shown by the author (Tsytovich, 1967,1968) for homogeneous soils with a structural strength (p_{str}) and an initial head gradient (i_o), the magnitude of the active compressive zone (on the basis of simple geometrical ratios, ensuring from the construction on Fig.4) in the general case will be (see formule 10).

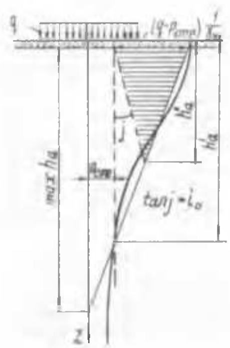

Fig.4. Determining the depth of the active compressive zone (ha) according to the equivalent layer method

$$h_a \approx 2h_s\left(1-\frac{i_o}{i_o+q/2h_s\gamma_w}\right)\frac{q_o}{q} \; , \quad (10)$$

where γ_w= is the unit weight of water and $q = q_o - p_{str}$

In calculating the settlement rate in time (determining) the degree of consolidation), in all the main cases of action of a local load, the outline of the generalized (equivalent) diagram of the compressing pressures can be taken as a triangle with its base at the footing of the foundations equal to q_o and its height equal to the depth of the active compressive zone h_a.

Of course, this diagram will just approximately represent the pressure variations with the depth, but is not contradictory to the essence of the matter, and greatly simplifies foundation settlement calculations without introducing impermissible errors.

The equivalent (triangular) diagram of compressive pressure can also be used for determining the mean value of the factor of relative compressibility of a lamellar soil strata (m_{va}) over the whole depth of the active compressive zone (h_a).

Accepting the notation in Fig.5, and taking into account the fact, that it may be assumed that $\sigma_z \approx q_o z_i/h_a$ without large error for the pressure values affecting the factor of relative compressibility (m_{va}) of various soils layers. Here,

z_i is the distance from the middle of the investigated layer to the depth of the active compressive zone (Fig.5).

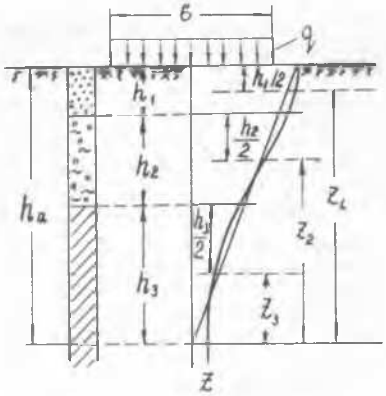

Fig.5. Concerning the determination of the compressibility factor of a lamellar strata of soil

Equating the settlement of the whole lamellar strata $S = h_a m_{va} q_o/2$ to the sum of separate layer settlements, up to the depth of the active compressive zone, we have

$$m_{va} = \frac{2}{h_a^2} \sum_1^{h_a} h_i m_{vi} z_i \,. \tag{11}$$

The mean filtration factor for a lamellar soil strata (k) is determined by Dachlers formula:

$$K = \frac{h_a}{\sum h_i/K_i} \tag{12}$$

where h_1 and k_1 = height and filtration factor of a separate soil layer.

Using formulas (11) and (12) we reduce the lamellar soil strata (up to the depth h_a) to a quasi-homogeneous soil strata, whose settlement equals:

$$S_\infty = h_s \, m_{va} \cdot q_o \quad , \tag{8'}$$

or in a more general case

$$S_\infty \approx h_a \cdot m_{va} \cdot q_o/2 \tag{8''}$$

These formulas can be used for determining foundation settlement on lamellar soil strata.

The equivalent diagram of compacting pressures enables the degree of consolidation U of the whole compressed soil zone under a foundation of given dimensions to be readily determined in closed form. Then it is used to determine the settlement of the foundation for any time interval (t) from the beginning of loading (i.e., S_t).

Concerning compacting pressures, decreasing with the depth according to the triangular diagram in the case of upward one-sided filtration (Tsytovich, Zaretsky et al, 1967), we have

$$U = 1 - B\frac{16}{\pi^2} \sum_{n=1,3}^{\infty} \left(1 - \frac{2}{\pi n} \sin\frac{\pi n}{2}\right) \frac{1}{n^2} e^{-n^2 M t} \,. \tag{13}$$

Then, expanding the expression after the summation sign into a series, and taking into account that (where S_∞ the full stabilized settlement determined from formula 8), we obtain for the settlement S_t:

$$S_t = \frac{1}{2} h_a m_v q_o \left\{ 1 - B\frac{16}{\pi^2}\left[\left(1 - \frac{2}{\pi}\right)e^{-Mt} + \frac{1}{9}\left(1 + \frac{2}{3\pi}\right)e^{-9Mt}\right]\right\} \tag{14}$$

where $B = \dfrac{1}{1 + m_w n \beta_o / m_v}$; $M = \dfrac{\pi^2 c_v}{4 h_a^2}$;

$c_v = \dfrac{K \cdot \beta_o}{\gamma_w m_v} \cdot B$ = soil consolidation coefficient, in the general case considered here

n = soil porosity.

If in the given equation for the factor (B), m_w is taken equal to zero, occurring only if the pore liquid is entirely free of vapours and gases, then B=1. This leads to equations that describe a pure filtration consolidation process, agreeing with the so-called "soil mass" (Terzaghi-Gersevanov), i.e., with a fully water-saturated soil having an uncompressible pore liquid. If the structural strength of the soil is equal to zero ($p_{str}=0$), then the initial pore pressure coefficient will equal $\beta_o = 1$. This complies with the so-called "soil paste", i.e., completely "structureless" soil.

In the general case, if, upon consolidation, account is taken of the soil skeleton deformation rate in time, according to the theory of hereditory creep; the compressibility of gas-containing pore water and the influence of the structural strength of the soil and initial head gradient on the active compressive zone, then, as has been shown in the paper cited above (Tsytovich, Zaretsky, et al, 1967), the foundation settlement on clayey and silty soils for any time (t) is determined by the equation:

$$S_t \approx \frac{1}{2} h_a m_v q_o \, \psi(t) \quad , \tag{15}$$

where

$$\psi(t) \approx \frac{1 - \frac{16}{\pi^2}B\left(1 - \frac{2}{\pi}\right)e^{-Mt} + \frac{\delta}{\delta_1}B\left\{\left(1 - e^{-\delta_1 t}\right) - \frac{16}{\pi^2}\left(1 - \frac{2}{\pi}\right)\left[\frac{e^{-Mt} - e^{-\delta_1 t}}{1 - M/\delta_1}\right]\right\}}{1 + \delta/\delta_1}$$

as symbols as previously denoted.

In the following, we give comparisons of calculated settlement values (according to the method set forth here) with those observed directly.

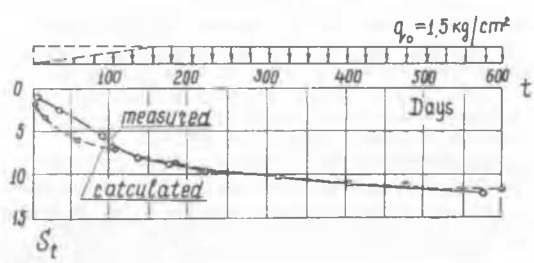

Fig.6. Comparison of designed and measured foundation settlements of a school house.

Table 2 lists, and Fig.6 shows a comparison between foundation settlements calculated by the author and those actually observed for a standard school building (footing size 1.3 x 34.5 m), built on a three-layer weak soil footing (with a layer of sandy loam h = 5.3 m, loam h = 1.9 m and varved clay h = 2.8 m).

Values of calculation factors were previously published in the author's book "Soil Mechanics," in 1940.

Table 2

Foundation settlements of a school house

Time	$S_{cal.}$	S_{actual}	Notes
25 days	4.0 cm	2.2 cm	nut fully loaded
106 days	6.7 cm	6.0 cm	-"-
305 days	9.5 cm	9.7 cm	at calculated load on soil
600 days	11.5 cm	11.5 cm	-"-
2.1 years	12.1 cm	12.1 cm	-"-

Table 3 lists data of other authors (Prof.B.I.Dalmatov and Assistant - Eng. Sotnikov) published in the "Materials of the All-Union Conference on Construction on Weak Clayey Soils" (Tallin, 1965).

Table 3

Foundation settlements of large buildings

Object	Settlements			Observation time
	according to the elementary suming method	according to the equivalent method	actual	
Administration building.	20.8 cm	42 cm	38.8 cm	26 years
Hotel "Rossiya"	15 cm	43 cm	45.3 cm	4 years
A 12-apartment building.	31 cm	51 cm	35.0 cm	1 year

As has been shown, the briefly presented engineering method of predicting foundation settlement by the calculation of an equivalent soil layer, in its up-to-date stage of development, enables, for most practical cases, not only the full stabilized foundation settlement to the determined, in accordance with its shape quickly, using simple formulas, but also enables its settlement rate in time to be calculated, taking into account: the filtration-consolidation of the compressed soil zone underneath the foundation, as well as its structural strength, compressibility of the pore water, initial head gradient value, initial pore pressure and the creep of the soil skeleton.

II. Causes of the development of differential settlements.

When great settlement in the base of a building occurs as the result of soil deformation, there may be observed simultaneous and great differential settlements of some foundations. In general the foundation settlement depends upon four main addends:

$$S = S_c + S_s + S_\beta + S_u \qquad (16)$$

The causes of the development of differential consolidation settlements are shown in Fig.7.

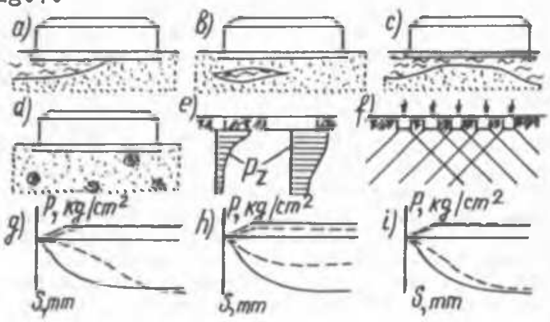

Fig.7. The causes of the development of differential consolidation settlements
a - thinning out of layers; b - lens-like bedding; c - different thickness of layers; d - presence of inclusion; e - non-uniformity of active zone; f - the influence of loading adjacent foundations; g,h - non-simultaneous and incomplete loading of foundations; i - non-simultaneous base soil consolidation

S_c - the settlement as the result of soil consolidation

S_s - swelling settlement of the bed in digging the trench depends upon the dimensions of the trench, soil stratification, presence of ground water, duration of the period of unloading the base and some other factors.

S_β - the settlement of soil flow is caused by the fact that high strasses arise under the foundation edges, and under their action there develop zones of plastic deformation resulting in redistribution of contact pressure (Florin, V.A., 1937).

S_u - the de-structuring settlement is caused by the destruction of natural soil structure by meteorological factors, by the action of ground water and gas, by dynamic actions and by blunders on the part of the builders.

The development of plastic deformation causes soil consolidation on the sides of the zones, and also the increase of the resistance to displacement in horizontal direction which may rise because of active pressure to the value of passive earth pressure (Dalmatov B.I.,1951). With the purpose of decreasing the valuse (S_B) the mean intensity of pressure on soil is limited in accordance with the development of plastic deformation zones.

It is common practice to use in work the building methods which bring S_B and S_u to minimum. In digging a trench, of small depth when the weight of the underground part of the building is not less than that of soil displaced by it, the settlements S_S develop mainly during the construction of foundations and making backward soil filling, and therefore they almost do not affect the deformation of the overground constructions. Thus in designing foundations the main consideration to be taken into account is the consolidation settlements – S_c.

D e t e r m i n a t i o n   o f   d i f f e r e n t i a l   s e t t l e m e n t s taking into account the loading of the adjacent areas and limitation of the soil strata under compression is of great importance.

As early as in 1934 prof.Tsytovich N.A. (1963) reduced the formula by Boussinesq-Schleicher (5) to formulae (7) and (8). For a two-play bed K.E.Yegorov (1958) obtained the expression which is analogous to that of (5), in which the settlement factor is additionally the function of the ratio of the compressed strata to the foundation width. This made it possible (B.I.Dalmatov, 1968) to express the determination of foundation settlement with limited compressed soil strata, using Tsytovich's methods – as:

$$S_\infty = H_e \cdot m_{va} \cdot q \qquad (17)$$

$$H_e = A \omega_h \, B \qquad (18)$$

where ω_h – is the settlement factor depending upon the ratio of the designed compressed strata to the width of the foundation base, the form of the foundation base and foundation rigidity. The meaning of ω_h is determined according to Table 1.

The designed compressed strata may be determined experimentally or by means of calculation. And its maximum meaning is equal to the double value of the equivalent layer (Tsytovich, 1963). Since the meaning of the coefficient of volume compressibility of soil (m_{va}), which is made average is included in (17), a simple method of determining foundation settlement may be proposed with the loading of the adjacent areas ((of foundations) taken into consideration. For this purpose it is quite enough to find the corresponding meaning of H which must depend additionally on the form of adjacent foundations and take into consideration the intensity of pressure distributed over their bases.

The foundation settlement A with the influence of loading the circular area B taken into consideration, the centre of which coincides with the centre of gravity of the foundation base A (Fig.8).

$$S_\infty = S_F + S_{R0} - S_{Ri} \qquad (19)$$

where S_F – is the settlement caused by loading the foundation A;

S_{R0} – is the settlement of the central point o in loading the circular area with radius R_0; S_{Ri} – the same radius R_i.

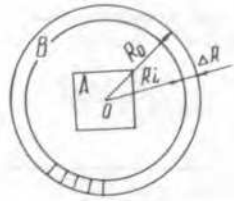

Fig.8. The plan of foundation with ring loading area.[Similar to Newmark, 1942]

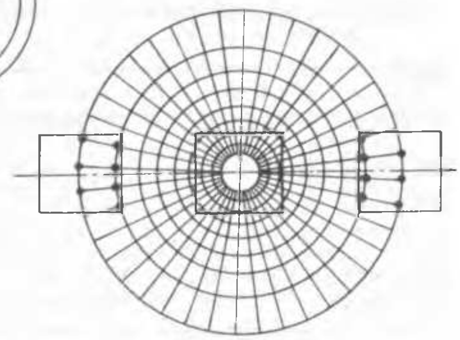

Fig.9.Circular chart for compressed thickness of 7 m, made in superposition with the plan of the foundation base.

If we substitute the meanings of the addends in (19) taking into consideration the expression in (17), then we obtain

$$S_\infty = [H_{eF} + (H_{e0} - H_{ei}) K_L] m_{va} \, q \,, \qquad (20)$$

where H_{eF} – is the conditional equivalent layer dependent on the dimensions of the foundation A;

H_{e0} and H_{ei} – are the same – from the radii R_i and R_0;

K_L – is the ratio of loading, distributed over the circle B, to q.

The values R_0 and R_i may be chosen in such a way as to obtain

$$H_{e0} - H_{ei} = N \qquad (21)$$

where N – is some definite value.

Taking $R_{i1} = 0$ and having the predetermined meaning of N, we can easily determine R_{01} to content with the expression (21). Then, taking $R_{12} = R_{01}$ we may find R_{02} with which $H_{e02} - H_{e12} = N$. By way of making successively such operations it is possible to obtain a system of concentric circles the loading of each with the load , i.e. with K_L = 1, will lead to the increase of the conditional equivalent layer for the value of N.

Let us divide the concentric rings by radii into n_0 of equal parts. The loading of each part of any ring will lead to the increase of the conditional equivalent layer for the value

$$\Delta H_e = \frac{N}{n_0} \qquad (22)$$

Since the value of the conditional equivalent layer depends on the ratio of the value of the compressed strata to the diameter of the loading area, the meaning R_0 is to be computed in accordance with the definite value of the compressed strata. The computations have been made for the strata of 5,7,10,15,20 and 30 meters. With obtained meaning R_0 it is possible to make circular charts by analogy with Newmark's charts (1942).

However in applying such charts on tracing paper to the foundation plan drawn on the same scale the parts of the rings falling on the contours of foundations would always be divided into two parts: falling on the adjacent foundation and being beyond its contour. With the purpose of facilitating the task it is reasonable to find the centres of gravity of the parts of rings and make circular charts of the position of those centres (Fig.9). In such a case the value, for which the conditional equivalent layer increases in loading the adjacent foundation, will be equal to

$$H_{en} = n \cdot \Delta H_e \cdot K_L \quad , \tag{23}$$

where n – is the number of points of intersection of the corresponding circular chart, falling on the adjacent foundation, the influence of which is under consideration.

If there are some adjacent, differently loaded foundations, the calculation will be only counting the points of intersection of the charts within the boundaries of the contour of the base of each foundation and multiplying it by the factor of load ratio K_{Li}. Then the conditional equivalent layer, with the consideration of loading the adjacent foundations, will be

$$H_e = H_{eF} + K_\mu \, \Delta H_e \sum_{i=1,\ldots}^{j} K_{Li} n_i \ . \tag{24}$$

In the expression (24) there was introduced the factor K_μ, which is the function μ_0, because it is reasonable to make circular charts for one meaning of μ_0. If the thickness of the active zone H_a does not coincide with the meanings of H, for which the circular charts are made, then n_i is determined by interpolation. By way of using circular charts it is easy to determine the foundation settlement, taking into consideration the loaded adjacent areas. This is especially important when designing foundations which transfer unequal loads to soil.

C o n s t r u c t i o n   m e a s u r e s taken for decreasing the injurious effect of differential settlements.

Such measures are – the construction rise which is given to the building or to some parts of it; the increase of flexibility of the building; the additional strength given the constructions; the use of foundations which level the differential settlements; the application of special constructive measures.

When great settlements are expected to take place, buildings or their parts are given some construction rise, i.e. their structures are positioned somewhat higher in accordance with probable settlements. If the settlements develop slowly, the value of the construction rise is determined from the expression

$$S_{ef} \approx \frac{1}{2}(S_\infty - S_{bld}) \ , \tag{25}$$

where S_∞ – is the final settlement;
S_{bld} – is the settlement developed during the construction period.

As flexible structures follow the bed settlement, additional stresses do not arise in their constructions. But buildings usually possess a final rigidity. Walls, framework etc. give this rigidity. An effective measure of decreasing the rigidity of a building is cutting it into parts by settlement joints. In the place of the joint the rigidity of a building is equal to 0. The minimum width of the clearance of the settlement joint is determined by the formula

$$\alpha = Kh(\tan\theta_R - \tan\theta_L) \ , \tag{26}$$

where h – is the distance from the foundation base to the hight for which the clearance width is determined;
$\tan\theta_R$ – is the inclination of the base of the right part of the building;
is the same for the left part;"
K – is the factor 1,3 – 1,5 taking into account the heterogeneity of the bed soil. If the inclination is developed in the direction to each other, then $\tan\theta_L$, is taken with minus.

With differential bed settlements there may appear additional tensile stresses, compressive stresses and shearing stresses in walls and other constructions, which are connected with their resistance to bending. Generally wall masonry has high resistance to compression and low resistance to tension. For the purpose of making walls more resistant to tensile stresses, arising from differential settlement, prof.B.D.Vasiliev (1952) as early as in 1936 began to apply the reinforcing of brick walls with reinforced-concrete chords. Reinforcing chords are placed in foundations and in each or in every 2nd floor in walls. The section of reinforcement in chords is determined with consideration of the simultaneous work of bearing structures of the construction and of bed soil, their differential compressibility being taken into account, Such kind of approximate calculations were considered by B.I.Dalmatov (Vasiliev, 1952) and by some other authors.

When high shearing forces in walls act so as to put them askew, there appear askew cracks above the openings and in the partitions. For increasing wall resistance to such stresses the chords are placed closer, the dimensions of openings are decreased, the width of partitions is increased and partition masonry is reinforced with mesh-reinforcement. Sometimes in the places of probable development of askew deformation a rigid monolithic reinforced concrete foundation is constructed.

In redistributing pressure over the foundation base of buildings in some places stresses are concentrated along the wall length, and they are not taken into consideration in usual calculations. The stress concentration is sometimes great, if there is some slightly compressible soil; therefore masonry is strengthened in lower floors.

When differential settlement proves to be inadmissible, it is possible to obtain the levelling of settlements at the expense of increasing the width of the foundation base. Such a calculation is easily made, if we have the condition of having ultimate permissible differential settlement (Dalmatov, 1968). Sometimes the levelling of settlements is obtained by making the foundation as a continuous slab, and by making shells and cross strips.

The weakest point with highly compressible soil is the place of adjoining low buildings to multi-storied constructions. Even with making settlement joints, the additional buildings get distorted or askew to wards the heavy part of the building. In such cases, if there is no possibility to give off pressure to denser subsoil either the base of the additional building is cut by sheet piling, driven through all the thickness of soft soil (Fig.10), or the

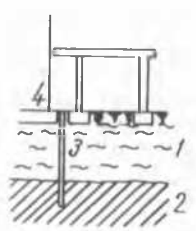

Fig.10. The chart of adjoining the additional building to the multi-storied part of the building. 1 - highly compressed soil; 2 - dense soil; 3 - sheet piling; 4 - the multi-storied part of the building.

additional building is erected on the cantilevers projecting from the foundation of the main part of the building. The construction of 12-storied brick buildings with one-storey additional shop buildings in Leningrad is an example of using sheet piling. According to the observation data the additional building gave settlement of 30 mm and the high part of the building - of 270 mm. In constructing additional buildings on cantilevers some air gap is left under them with the hight of expected settlement, because otherwise the reactive soil pressure during the process of settlement may be higher than the weight of the additional building.

Besides, it is necessary to consider the probable settlements of buildings and of the surrounding territory with regard to the deformation of inlets and underground communications.

In Leningrad and many other cities of the U.S.S.R. numerous buildings with a settlement of 40 cm and more have been erected with due consideration of calculations stated above.

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