EVALUATION OF THE ACTIVE ZONE OF SETTLEMENT

DETERMINATION DE LA ZONE ACTIVE DU TASSEMENT

ОПРЕДЕЛЕНИЕ СЖИМАЕМОЙ ЗОНЫ

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SUMMARY. Deep settlement measurements show that the settlement is damped out at a certain depth under the foundation base and does not spread ad infinitum as is assumed in the theory of the elastic isotropic semi-stratum. Investigations show that a satisfactory coincidence between calculated and measured settlement is obtained when it is computed as a settlement of a layer of limited depth. It is known that during soil loading plastic deformations occur only after exceeding its structural strength p_{str} of the soil. This means that stresses in the soil σ_z, lower than the structural strength provoke only a negligible (elastic) settlement, i.e. the active zone ends where $\sigma_z = p_{str}$. It is proved in the paper that all existing tables and diagrams for the distribution of the stresses in depth under the loaded area can be used for calculating H_a. It is sufficient to replace z by H_a and σ_z by p_{str}.

THE CALCULATION of the anticipated settlement is an indispensable part of designing buildings and structures. In evaluation of the basement area according to the limit state of deformations should be fulfilled the condition

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$$S \leqslant S_{lim}, \tag{1}$$

where S stands for calculated deformations, S_{lim} - for maximum admissible deformations for a given construction, pointed out, for example, in Tables 2 and 4 of POLSHIN and TOKAR (1957).

It is known that the calculated settlement in some cases is close to the actual one and in other cases differs significantly. The differences are due mainly to two causes:

1. Imperfect determination of the modulus of compressibility,
2. The unknown thickness of the settling layer, i.e. the depth under the foundation up to which the settlement is manifested - the active zone of settlement.

Both causes are investigated repeatedly. While the first cause is eliminated to a large extent by multiplying the modulus of compressibility, obtained in the laboratory or in situ, by a factor which for stiff soils is bigger than unity, the determination of the active zone of settlement has not yet attained satisfactory practical solution.

Deep settlement measurements show that the settlement is damped out at a certain depth under the foundation base and does not spread ad infinitum as it is assumed in the theory of the elastic isotropic semi-stratum.

The investigations show that good conformity is obtained between the settlement calculated and measured when it is computed as settlement of a layer with limited depth. Owing to this the depth of the active zone H_a in homogenous layer is determined after the equation

$$\sigma_{z=H_a} = \alpha \gamma H, \tag{2}$$

where $\sigma_{z=H_a}$ is the vertical stress under the foundation in the extreme depth of the active zone,

γH - is the layer proper load at the same depth (γ is the unit weight and H is the thickness of the settling layer),

α is the coefficient which in the opinion of different authors and in different norms varies in a rather wide scale - from 0.1 to 0.5, for example, according to SNiP II - Be.1-62 (1962) is $\alpha = 0{,}2$.

Hence the problem comes to determining the depth at which

$$z = H = H_a \tag{3}$$

It is proved that in soft soils H_a is considerably greater than in stiff soils (TSYTOVICH, 1968). According to the same author that $H_a = 2h_e$, where h_e is the equivalent height in this method. In soils with structural strength the active zone is still smaller.

It is known that at soil loading the plastic deformations occur only after overcoming the resistance which the soil structural bonds exert on the action of the tangential stresses which occur in it. The resistance at which plastic deformations begin is the structural strength of the soil $p_{str.}$ This indicates that stresses in the soil σ_z lower than the structural strength provoke only negligible (elastic) settlements, i.e. the active zone depth in this case is determined by the condition

$$\sigma_{z=H_a} = p_{str.} \qquad (4)$$

From equations (3) and (4) it follows that all existing tables and diagrams for the distribution of stresses in depth under loaded areas may be used for calculating H_a. It is sufficient to substitute z for H_a and σ_z for $p_{str.}$ i.e. in the place of the ratio $\frac{z}{B}$ is placed $\frac{H_a}{B}$ and in the place of $\frac{\sigma_z}{p_o}$ - $\frac{p_{str.}}{p_o}$ for given ratios $\frac{p_{str.}}{p_o}$ and $\frac{L}{B}$ the ratio $\frac{H_a}{B}$ is obtained on the ordinate from which H_a may be calculated. The thus modified diagram of Steinbrenner has the advantage that it shows illustratively and directly the effect of the structural strength on the depth of the active zone since

$$\frac{H_a}{B} = f\left(\frac{p_{str.}}{p_o}, \frac{L}{B}\right). \qquad (5)$$

For example, for square foundation (L/B = 1) of particular interest are the values shown in Table 1.

Table 1. Influence of the structural strength on the depth of the active zone of settlement at square foundation.

$p_{str.}$	H_a
0,1 p_o	2,1 B
0,2 p_o	1,4 B
0,34 p_o	B
0,5 p_o	0,75 B
0,7 p_o	0,5 B

Analogically, for example, table 9 of TSYTOVICH (1968) for the stresses under circular, rectangular and strip foundations, as well as other similar tables including the influent chart of NEWMARK (1935) for stresses under arbitrarily loaded area may be used. When the influent chart is applied the foundation base, after repeated testing, should be drawn on such a scale as to meet the condition

$$\frac{n}{m} = \frac{p_{str.}}{p_o} \qquad (6)$$

where n is the number of meshes covered by

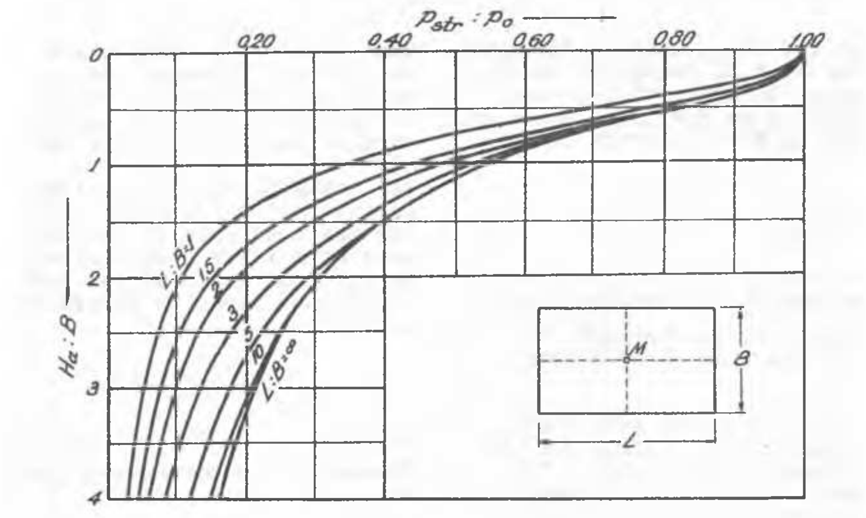

Fig.1. Modified diagram of STEINBRENNER for determining the active zone of settlement.

As an example, Fig.1 shows the well-known diagram of STEINBRENNER (1934) respectively modified for this purpose. By doubling in advance the semi-width 2b=B and quadrupling the ratio $4\frac{\sigma_z}{p_o} = \frac{\sigma_{z,M}}{p_o}$ the diagram for determining the stress under the angular point of a rectangular loaded area is transformed into diagram for the stress $\sigma_{z,M}$ under the middle point M. The use of the diagram is clear:

the foundation base and m is the number of all meshes of the influent chart. By means of the thus established scale is determined $z = H_a$. The method of SALAS (1948) may analogically be applied in an expedient manner. After this method the foundation base is drawn only once and the scale 1/ is determined experimentally, in which

$$\sum_{i=1}^{i=n} I_i^z \frac{A_i}{100} = \frac{p_{str.}}{p_o}, \qquad (7)$$

where $I_i^z = I_{R+1,T}^z$ is the value of the i^{th} ring at a ration R/λ obtained from the SALAS table,
R+1 and R are radiuses of circles which encircle the i^{th} ring,
A_i is the percentage part of the area occupied by the foundation base in the i^{th} ring.
After determining the ratio z/λ for which condition (7) is fulfilled $H_a = \lambda/z$.

When the ground is stratified the check after equation (4) should be carried out separately for each layer.

The method suggested for determining the depth of the active zone proceeds from the assumption that the structural strength of the respective soil is known. The structural strength of the soil, as we know, is created by different factors, the main of which are: overconsolidation, the soil shrinkage and the physico-chemical processes. They all exert effect on the compression curve of the soil. This requires the compression tests to be carried out carefully so that the structural strength of the soil may be determined each time. It may be determined more precisely when in the range of the anticipated value (usually at the beginning of the compression test) the loading is carried out at small consecurive stages. The value of the soil structural strength is determined easily when the compression curve is plotted in semi-logarithmic scale. Use may be made of the empiric method of CASAGRANDE (1936).

Certain soils in Bulgaria have exhibited structural strength up to 2 kg/cm^2 (DINGOSOV, 1969). If soil loading is applied, for example, $p_o = 3\ kg/cm^2$, then $p_{str} \approx 0{,}7\ p_o$ or at square foundation after Table 1, $H_a = 0.5B$.

The settlement computed with such H_a is significantly smaller than that usually calculated. It may be seen from this example of what great significance is the determination of the soil structural strength for the correct calculation of the settlement.

For normally consolidated soils without structural strength the method suggested for determining H_a may be applied, if p_{str} from equations (5) and (7) is substituted with

$$p_{\gamma T} = \gamma T, \qquad (8)$$

where p_T stands for the load applied on the foundation base,
- is the average unit weight from the terrain surface to the depth of the foundation.

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