Active Zone beneath Foundations

La Zone Active au-dessous des Fondations

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SYNOPSIS Vertical displacements at different levels beneath the foundation base of a loading plate and of a grain elevator were measured. The loading plate was placed on the surface of four types of sandy to clayey foundation soil, the grain elevator rested on a sandy and clayey granitic residual soil. The depth where the vertical displacement practically disappears is called depth of active zone. According to the results of the measurements it equals the foundation width (diameter) ±50%. The existence of an active zone may be explained by a non-linear stress-strain behaviour of the foundation soil and/or its non-homogeneity. In the latter case the subsoil non-homogeneity was defined by a deformation modulus calculated from the vertical strain increments measured at different depths. Assuming that the settlement of such a non-homogeneous subsoil corresponds to that of a homogeneous subsoil layer of finite thickness equal to the depth of active zone, one may calculate active zone depth. There was reasonable agreement between the calculated and measured values.

INTRODUCTION

Differences between calculated and observed settlements are often recorded. One possible explanation of this discrepancy may consist in that the vertical subsoil deformations do not extend beyond a depth called depth of active zone. If the thickness of this zone is known, the settlement could be calculated reliably.

There are many empirical methods of the determination of the depth of active zone, which are based either on the foundation dimensions or the vertical stress increment/geostatic stress ratio, the former being sometimes combined with an absolute value and the nature of the foundation soil is often also taken into account. A rational method does not exist.

In the paper the results of four plate loading tests are presented together with those of the measurement of vertical displacement beneath a grain elevator. The measured value of the active zone depth is analysed theoretically.

PLATE LOADING TESTS

As a part of an extensive, State financed research program focused on the improvement of the calculation of structures settlement, static loading tests with a 138 cm dia circular loading plate are being carried out on various subsoil types.

The aim of the tests is to compare the moduli of deformation determined by different laboratory and field tests, and to evaluate the active zone depth. For this purpose underground settlement marks were installed below the loading plate to record the depth of zero vertical displacement.

The underground settlement marks were placed in 17 boreholes of 3 cm dia drilled into the subsoil through the loading plate and also located in its vicinity (Fig. 1). There were 7 marks in each borehole fixed at 0.5 m intervals. The displacement of the marks was recorded by the modified method of J.B. Burland et al. (1972).

Below, the results of the active zone depth measurements are presented, obtained from four in-situ loading tests using a plate resting on the ground surface. Further tests made at depth as well as tests using strip footings are in progress.

For the tests, soil types of high, medium or low compressibility were chosen, in each case the subsoil being homogeneous. Their geotechnical parameters are given in Table I. The fine silty sand of high compressibility was tested twice - first during a rainy, weather, at saturation degree 1.0 (test No.1), and during a dry weather at saturation degree 0.6 (test No.2). The water content variations influenced remarkably the compressibility,

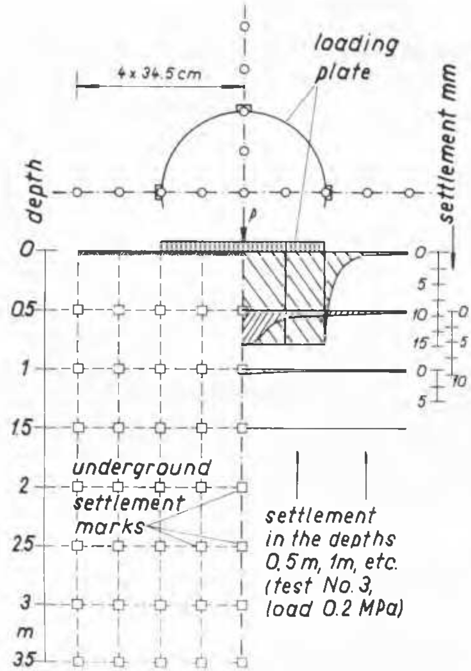

Fig. 1 Arrangement of underground settlement marks. Above: the lay-out, left: the cross section, right: dependence of the vertical displacement of the marks on their distance along the vertical axis

mostly due to the negative pore pressure in the unsaturated state.

The modulus of deformation in the last column of Table I has been calculated for a load range of 0.1-0.2 MPa of the respective loading test.

The load-settlement diagrams of the four tests are shown in Fig. 2. The duration of each test was about two months, as every loading stage was kept until deformations practically ceased. In Fig. 2 the loading stages are indicated by circles.

The depth of the active zone may be derived from the curves showing settlement at different depth levels (Figs. 3 and 4). The settlement indicated at each level is the mean value of the vertical displacements of

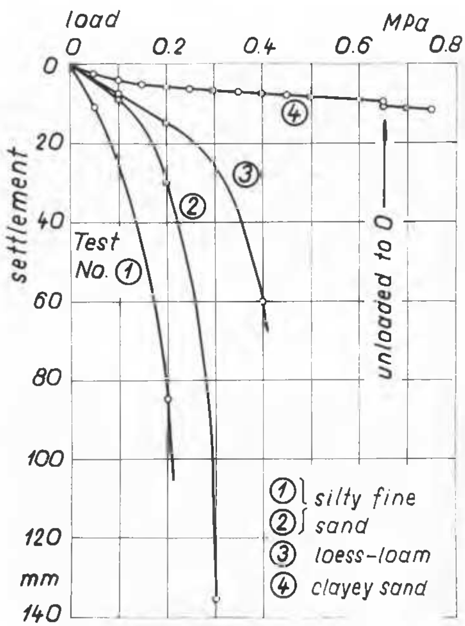

Fig. 2 Load-settlement curves of four plate loading tests

Table I

Load test No.	Soil	Soil type	Grain size distribution % clay 0.002 mm	silt	sand 0.074 mm	Atterberg limits % w_L	w_P	I_P	Dry unit weight g/cm^3	Water content %	Degree of saturation	E MPa
1	Uniform silty fine sand	ML	8	55	37	33	26	7	1.40	35	1.0	1.6
2										21	0.6	4.5
3	Loess loam	CL	10	65	25	38	20	18	1.67	18	0.8	12.3
4	Very compact clayey sand	SC	10	15	75	26	13	13	1.65	9	0.35	53.0

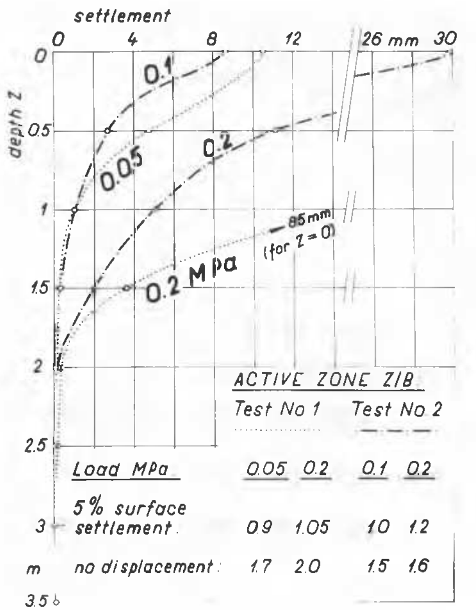

Fig. 3 Loading tests No. 1 and No. 2 on silty fine sand, ML, of low density. No. 1 - degree of saturation Sr = 1.0; No. 2 - S_r = 0.6

four symmetrical underground settlement marks which are at a distance of B/4 from the vertical axis (B=138 cm is the loading plate diameter). With increasing distance from the plate axis, the settlement diminishes, but from a certain depth downwards this is no more the case (cf. Fig. 1 - to the right).

From the practical point of view, the active zone depth may be regarded as the depth where the settlement equals 5% of the plate settlement. In Figs. 3 and 4 the active zone depth for different loads is indicated, on the one hand, by the Z/B ratio (Z=depth) which has been calculated according to the above criterion, and, on the other hand, it is considered as corresponding to the depths of non-measurable displacements. Since the settlement-depth curve joins the zero vertical axis asymptotically, the first criterion is preferred. As shown in Figs. 3 and 4, if the "5%" criterion is applied, the active zone range is about 0.5-1.5 B.

In Fig. 4 two settlement-depth curves are compared (for two load stages) with the vertical stress increment calculated according to the theory of the elastic half-space. It is clear that the assumption of a linear stress-strain relation is unrealistic.

In the tests referred to, the depth of active zone depends on the load increasing with the load increase. For example, in test No. 4, the load increasing from 0.05 to 0.75 MPa, the depth of the active zone became more than double.

Contrary to other authors (e.g. P.A. Konovalov, 1971), no increase of the active zone depth with decreasing deformation modulus of the subsoil was recorded (compare the Z/B values of all four tests at the same load, e.g. 0.2 MPa).

SETTLEMENT OF A GRAIN ELEVATOR

The grain elevator at Bor near Tachov is a reinforced concrete, 45 m high structure based on a 20x61 m reinforced concrete foundation mat at a depth of 4.7 m under the ground surface. The average contact pressure due to the dead load and maximum mean contact pressure are 0.13 and 0.35 MPa respectively.

The foundation soil consists of highly weathered granite. Within the foundation base and further 10 m downward, the granite is decomposed into a stiff to hard coarse-grained green-white sandy clay. With increasing depth the decomposition of the granite decreases. The ground water level was 1.5 m under the surface.

After excavating the foundation pit down to a depth of 6 m, its bottom was covered with a 1.3 m thick layer of sandy gravel. This layer was connected with the drainage system used for the lowering of the ground water level under the foundation mat. An insulation was put on its surface, and the foundation mat concreted.

After nearly completing the structure and before its first filling, five 133 mm dia boreholes were bored at a distance of 1.8 m from the edge of the foundation mat and in its transversal axis. Concrete was poured on the bottom of these boreholes up to a height of 60 cm, and a 63 mm dia steel pipe was inserted in it. Its bottom and upper end were provided with a steel plate and a bench mark. The casing was then pulled upwards for about 60 cm. The position of the internal pipe was secured by guides. The top of the protective casing was provided with a lockable cover. Deep bench marks were installed at different depths downward to 26 m below the foundation mat. The highest mark was situated at the level of the base of the sandy gravel layer under the foundation. The arrangement of the bench marks is shown in Fig. 5.

Deep bench marks were installed before the first filling of the grain elevator, and levelled. During the filling of the elevator with grain, their vertical displacement was recorded until full stabilization, from August 1972 to September 1975. Table II and

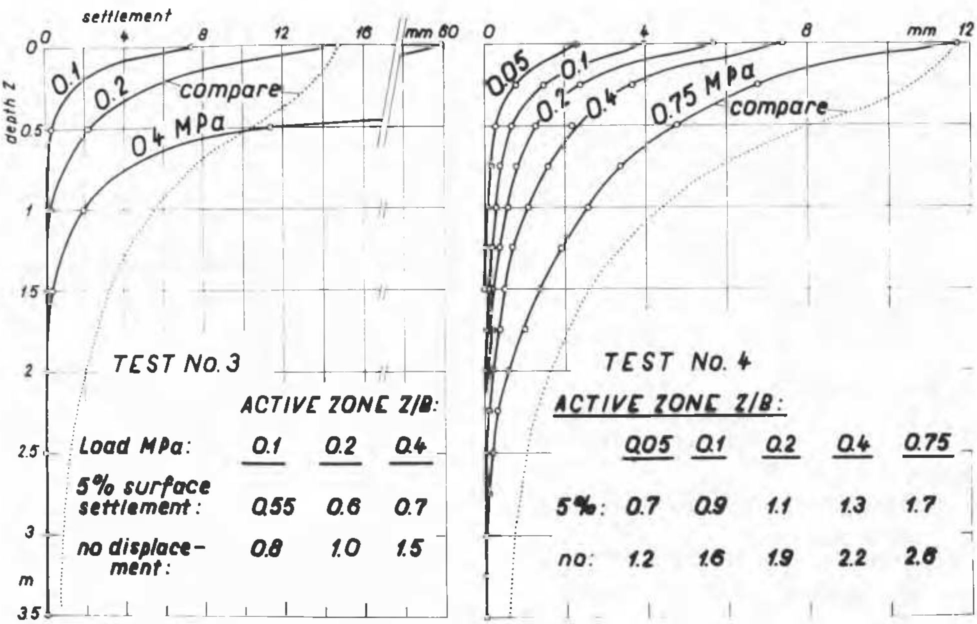

Fig. 4 Loading tests No. 3 on loess loam, CL, and No. 4 on very dense clayey sand, SC. Compare the lines of vertical deformation with the dotted lines of vertical stress increments

Fig. 5 shcw the measured maximum vertical displacements of the marks, due to the increase of the mean contact pressure in the foundation base from 0.13 to 0.32 MPa, and the calculated modulus of deformation E.

Table II

Point No.	Depth below surface m	Measured settlements mm	E MPa
1	6.3	17.7	55
2	12.4	9.4	80
3	20.4	1.8	404
4	26.0	1.0	367
5	31.0	0.2	

The recorded vertical displacements indicate the depth of the active zone to be about the width of the foundation mat under its base.

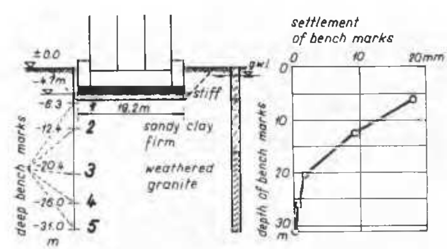

Fig. 5 Settlement of the subsoil of a grain elevator

ANALYSIS OF RESULTS

Fig. 6 presents a dimensionless plot of all the results. Curve a and b indicate the displacements of the grain elevator if its foundation base is transformed into an equivalent circular area (its diameter B_e = 39.4 m) and with the original dimensions, i.e. B = 20m, respectively. The plate loading tests conform with the settlement of the grain elevator although the axis of the depth-settlement profile is not identical in both cases.

Compared with the theory of elasticity, the vertical strain decreases with depth much more rapidly (cf. Fig. 4). This may be physically explained assuming the vertical strain results from the sliding of the soil

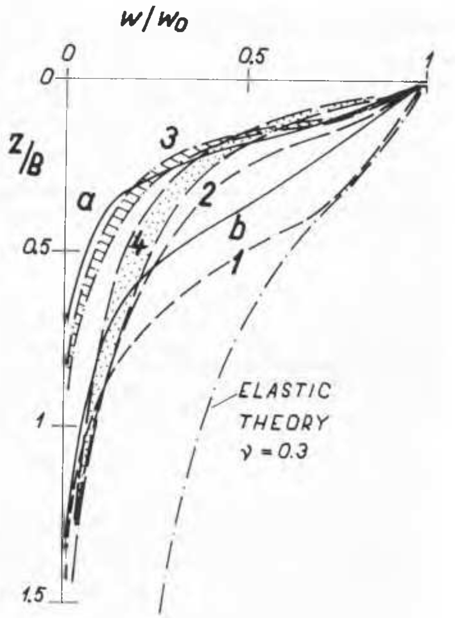

Fig. 6 The effect of depth on the vertical displacements in the loading range 0.2-0.4 MPa; 1,2,3,4 - loading tests (cf. Fig. 2); a,b - grain elevator (cf. Fig. 5); Z-depth below the foundation base, w-settlement at the depth Z, $w_o = w$ at $Z = 0$

particles or grains over each other. The position of contact forces K (Fig. 7) in an undisturbed soil is quite random. With increasing load the contact forces gradually move into their limit position A, inclined to the normal of the contact plane at the angle of intergranular (material) friction ϕ_μ (the contact strength of two particles can be additionally increased by their cementation). Only if statistically significant number of particles start to slide (i.e. $K \rightarrow A$), the soil will be considerably deformed.

Beneath the footing vertical stress increments decrease downwards and so does, in the same way, the number of contact forces that have reached their limit position. This is manifested by non-linear behaviour. At the same time, owing to the increase of the geostatic pressure, the number of contacts in a soil unit increases, the soil then becoming more deformation-resistant in the downward

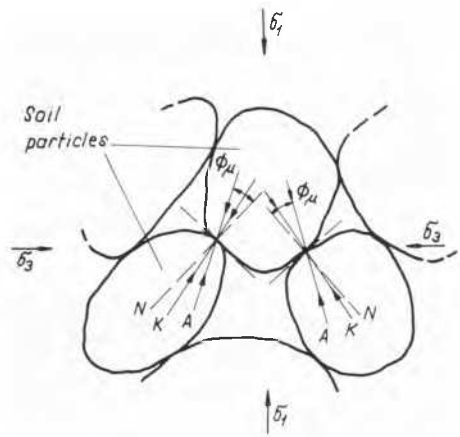

Fig. 7 Contact forces between soil particles: K - in the initial random position, A - in the limit position, N - normal

direction (non-homogeneity). Both effects, that of non-linearity and non-homogeneity, result phenomenologically in a gradual decrease of the deformation modulus of the foundation soil with depth.

The second model, that of a linear non-homogeneous half-space, being more comfortable in consequence of the validity of the principle of superposition, is taken as the basis of a theoretical analysis of the measured depths of the active zone D_a. It is assumed that the settlement of a foundation on a layer (linear homogeneous isotropic) of finite thickness D_a equals the settlement of the same foundation on a non-homogeneous half-space.

In case of linear non-homogeneity when

$$E = E_o + k\frac{Z}{B}$$

(E - deformation modulus at depth Z, $E_o = E$ at the foundation base, B - width or diameter of the foundation) and using the results of Yegorov (K.E. Yegorov and A.A. Nichiporovich, 1961) and W.D. Carrier and J.T. Christian (1973) one may obtain a graph (Fig. 8) correlating the depth of the active zone of a circular footing with the parameters of non-homogeneity E_o and k (for Poisson's ratio $\nu = 0.3$ common for soils). The usual range of $D_a/B = 1/3$ to 2 corresponds to $E_o/k = 0.2$ to 3.

To calculate the magnitude of D_a/B for the loading plate and the grain elevator requires to dispose of a set of independently measured

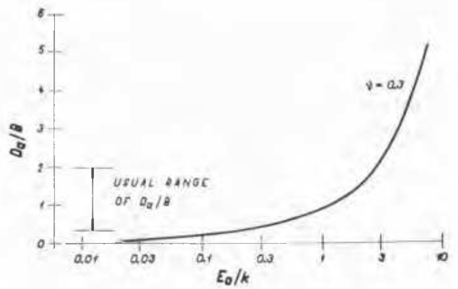

Fig. 8 The relationships between the subsoil non-homogeneity parameters E_o, k and the depth of the active zone D_a

deformation moduli E. Such measurements being rather unrealistic in the cases under consideration, the evaluation was based on the values of E calculated from the in-situ measured vertical strain and stress increments calculated according to the theory of elasticity. Two sets of such typical values are shown in Fig. 9. If extremely high values of E are neglected, straight lines a and 4 can be interpolated. Using Fig. 8, then for the grain elevator (B_e should be used) D_a/B_e = 0.45, for the loading plate D_a/B = 1.35. If compared with Fig.6, there is good agreement in the measured and calculated values of D_a/B in the latter case, while in

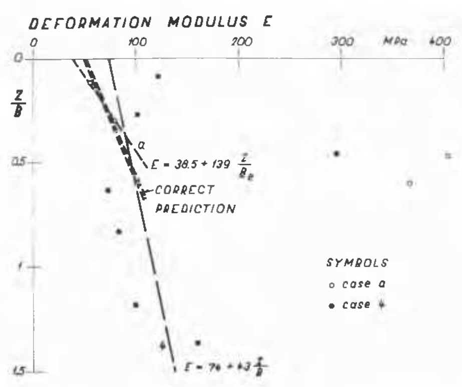

Fig. 9 Deformation modulus as computed from the vertical strain and stress increments (plate loading test No.4 - loading range 0.2 to 0.4 MPa; a - grain elevator, 0.1 to 0.3 MPa)

the first case the actual value of $D_a/B_e = 0.6$ requires a corrected law of non-homogeneity (cf. Fig. 9). Such a correction reasonably fits the experimental data of E.

For the determination of the active zone depth a rational procedure may be therefore recommended, namely relating it to the subsoil non-homogeneity. Actually, a mathematical model of the subsoil constructed as a compressible layer of the thickness D_a underlain by an incompressible layer (i.e. the assumption of a discontinuously non-homogeneous half-space) is a rough substitute for a physically mostly better substantiated model of a continuously non-homogeneous subsoil.

CONCLUSIONS

The instrumented plate loading tests and grain elevator measurements proved that the vertical compression of the foundation soil reaches to the limited depth under the foundation base. This depth, called the depth of active zone, was equal to the diameter (width) of the foundation ±50%. The linear (elastic) theory cannot account for such a rapid attenuation of vertical strains.

The plate loading tests indicated the depth of the active zone to increase with load. On the other hand, it appeared to be not directly dependent on the modulus of deformation of the foundation soil.

Locally confined spreading of the deformations in the foundation soil beneath the loaded areas can be explained and physically substantiated by the non-linearity and/or non-homogeneity of the subsoil. Assuming linearly increasing deformation modulus with depth, the depth of the active zone can be coupled with the non-homogenity parameters of the subsoil and thus a rational basis can be given for its calculation.

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