Deformation below a Stiff Foundation

Déformations sous une Fondation Rigide

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SYNOPSIS Large-scale tests were carried out with a stiff concrete foundation, 115 x 115 x 30 cm, on a moderately compacted coarse-grained sand, taking into account the influence of the initial state of stress of the subgrade. The experiment was carried out on a stand for testing foundation and for analysing the stress-strain state of the subgrade; the stand represents a reinforced concrete box, 10 x 10 x 6 m. The deformations of the sand subgrade in the various sections (vertical and horizontal) under the foundation and at a distance of up to 1.5 times the width of the foundation are measured. The results allowed a picture to be obtained of the field of deformation of the sand subgrade in case of load increase and of multiple loading. The analysis of the test and calculation results indicates that no precise evaluation of the deformation state of the sand subgrade can be given by way of calculation based on the existing conventional calculated soil characteristics and methods, and that the multiple loading influences the deformation state of the sand subgrade only during the first 3 to 4 cycles.

The calculation of problems by taking into account the non-linear deformation properties of the structural materials is solved in principle by the methods of mechanics of continuous medium (Ilyushin, 1948; Birger, 1965). Their introduction in soil mechanics for the calculation of concrete problems from the stress-strain state of the subgrade became possible with the development of computer techniques during the last decades. However a number of mathematical difficulties emerge due to the peculiarities of the deformation properties of soils (Kryzhanovsky et al., 1977), such as: the non-linear relation of space deformation both to spherical stress tensor and to stress deviator; the non-linear relation of shape deformation both to stress deviator and to spherical tensor; the relation of space deformation and shape deformation to the load path and to the type of space state of stress; the non-similarity between stress and strain state in some cases of loading or deformation of the soil. Therefore it is appropriate to conduct an experimental study of the stress and strain state of the soil half-space. This study is all the more necessary since no accurate evaluation can be made of the initial equations used in the calculation methods taking into account some of the above mentioned peculiarities of the deformation properties of soils. Particular complications set it when the deformation state of the subgrade is studied under multiple loading. All this necessitated the performance of tests for the study of the stress-strain state of a sand subgrade below a stiff foundation within the framework of a large experimental programme conducted at the Department of Soil Mechanics, Foundation Engineering and Engineering Geology at the Higher Institute of Architecture and Civil Engineering. This paper gives the results of the study of the deformation state of a sand subgrade loaded with a stiff square foundation, 115 x 115 cm, under multiple loading.

The experiments were carried out on the stand for testing foundations and for analyzing the stress-strain state of the subgrade (Fig. 1) at the Department of Soil Mechanics, Foundation Engineering and Engineering Geology.

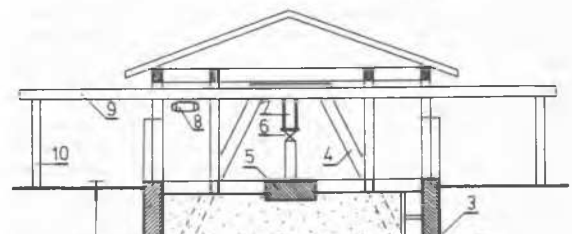

Fig. 1. Cross section of the stand

1 - artificial subgrade, 2 - retaining walls,
3 - groove for horizontal marks,
4 - steel structure, 5 - test foundation,
6 - hydraulic jack, 7 - movable steel support,
8 - telpher with lifting capacity 80 kN,
9 - monorail, 10 - supports.

The stand represents a box, 10 x 10 m in plan and 6 m deep, with retaining walls, enclosing the artificial subgrade. A vertical force of 6500 kN can be applied on the test foundation with the help of the steel structure, the movable steel support and the hydraulic jack. The artificial subgrade in the test consists of uniform coarse sand with the following grain size distribution: 20 to 5 mm - 5%; 5 to 2 mm - 16 %; 2 to 0.5 mm - 42 %; 0.5 to 0.25 mm - 25 %; 0.25 to 0.1 mm - 9 %; 0.1 to 0.01 mm - 3 %; dry density $\rho_{d\ min}$ = 1.51 g/cm^3 and $\rho_{d\ max}$ = 1.65 g/cm^3; void ratio e_{max} = 0.757 and e_{min} = 0.605; water content W_n = 3 to 4 %; angle of internal friction φ = 34°. The displacements of the sand subgrade points are measured with cord marks, representing metal disks, dia. 30 mm and 1 mm thick, mounted on a metal cord dia. 0.25 mm, which in turn is placed in a metal tube of external diameter 6 mm and internal - 4 mm. In order to avoid the influence of the tube on the movement of the metal disk during the deformation of the subgrade, the latter is located at some distance from the tube, which is greater than the expected displacement established in preliminary tests. Precautions are also taken to prevent the entering of sand grains in the tube and the jamming of the metal cord by making a packing of oil-soaked hemp. The effectiveness of these measures is established after the test when the marks are dismantled. The displacements of the marks are registered by dial indicators of an accuracy of 0.01 mm, fixed to the free end of the cord. In order to ensure a constant tensioning of the cord and to avoid the effect of friction between cord and tube, the cord is strained by a weight fixed at the indicator. The indicators are mounted on a metal base, which is not influenced by the loading of the foundation and by the deformation of the sand subgrade. The deformation of the sand subgrade surface was evaluated by surface marks, consisting of a metal disk dia. 100 mm, fixed to one end of the metal tube dia. 6 mm. The needle of the indicator, which records the displacement of the mark (Fig. 2), touches the other end of the tube. All parts of the marks described are made of metal having a negligible linear temperature elongation. In spite of this the tests were carried out under constant temperature. The contact stresses and the stresses in depth were also measured during the tests after the method described in (Stefanoff, Jellev, 1979; Jellev, 1978). Conditions close to the actual ones, under which the subgrade and the foundation operate, were

Fig. 2. General view of one of the tests.

provided during the tests. Thus for example the body of the square foundation was loaded with a reinforced concrete column 25 x 25 x 150 cm, fixed in it. This required the precise installing of the foundation and the loading jack, so as to obtain uniform settlement of the subgrade, which was followed by needle marks, mounted on all four angles of the foundation and by a vernier calliper of an accuracy of 0.01 mm. The disposition of the marks for measuring the vertical displacements is given in Fig. 3.

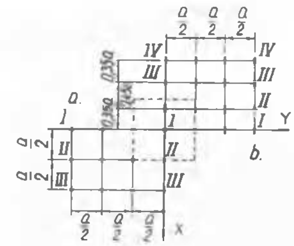

Fig. 3. Disposition of the marks along the verticals.

a - for measuring vertical displacements,
b - for measuring horizontal displacements.

In order to check the measurements some verticals are duplicated and the displacements are additionally measured with electromagnetic marks.

Loading was conducted in stages of 36 kPa up to 637 kPa, expressed as an average specific loading in the foundation base. At each loading stage records were taken immediately after the loading and after waiting for the damping of the settlement under an adopted stabilization of 0.1 mm/h. It should be pointed out that the stabilization of settlement thus adopted does not ensure the damping of the displacement in all points of the subgrade, which is particularly important when studying the deformed state of cohesive soil subgrade. After reaching the maximum loading q = 634 kPa the foundation was unloaded in stages in the reverse sequence. The tests were conducted with fourfold loading and unloading (Fig. 4).

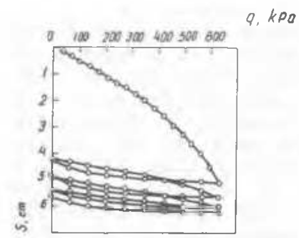

Fig. 4. Settlement diagramme s = f (q).

The results obtained for the vertical displacements w and for the horizontal displacements v, u are processed, giving relations which allow the fuller analysis of the deformed state of the sand subgrade.

Hereinafter the results are given of the analysis of the deformed state of the sand subgrade consisting of average compacted sand

(D = 0.467, e = 0.686) of a density ρ_n = 1.62 g/cm^3 and water content w = 2.6 %. The effect of the initial stress state of the artificial sand subgrade on its stress-strain state under loading was taken into account when conducting the test (Dovnarovich et al., 1977). Further on only some of the results are presented for lack of space. Fig. 5 shows the isolines, connecting points of equal vertical displacements w (in mm) under loads q = 139 kPa (Fig. 5a), q = 497 kPa (Fig. 5b), and q = 637 kPa (Fig. 5c) in section I-I (Fig. 3) for the first loading.

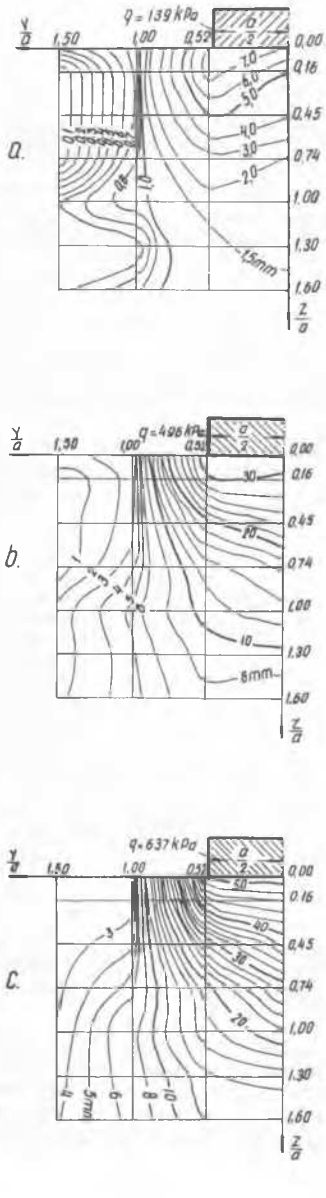

Fig. 5. Isolines of vertical displacements w in section I-I for first loading
a. q = 139 kPa; b. q = 497 kPa
c. q = 637 kPa

The analysis of the vertical displacements obtained for section I-I shows that for small loads (q = 139 kPa) the vertical displacements of the points located on the vertical y/a = 0.52 (under the edge of the foundation) are greater than those of the points of the vertical y/a = 0 (under the centre of the foundation) at a depth of up to z/a = 1.00. With the increase of the load, the displacements of the points under the foundation centre (y/a = 0) gradually increase (after loading with q = 498 kPa), and for load q = 637 kPa they are considerably greater for the points located at a depth z/a > 0.45. In case of unloading and reloading (Fig. 6) considerable changes of the isolines are observed in depth for small loads. With the increase of loading these changes decrease and when the maximum load q = 637 kPa is reached, they can be noticed only for the points in the zone of the foundation at a depth z/a < 0.45. At the third and fourth loading the character and distribution of the vertical stresses is practically preserved in size.

Fig. 7 gives the isolines of the vertical displacements (in mm) for section II-II (under the foundation edge) for loads q = 139 kPa and q = 637 kPa. It shows that the displacements of the points located on the vertical y/a = 0 are considerably greater than the remaining ones with the exception of the points in the zone z/a = 1 to 1.30 for a load q = 139 kPa.

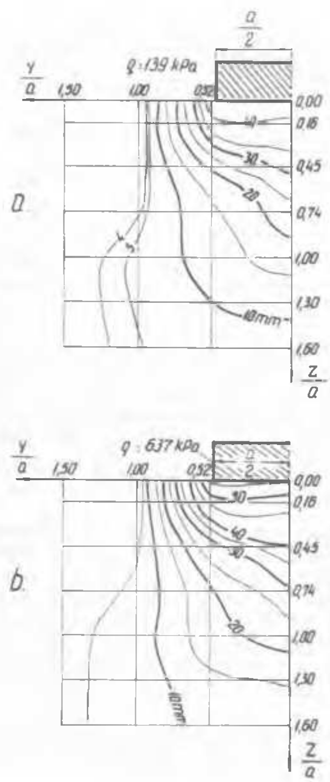

Fig. 6. Isolines of vertical displacements w for section I-I for second loading
a. q = 139 kPa b. q = 639 kPa

In case of second loading (Fig. 8) there are considerable differences in the character and size of the vertical displacements compared to those of the first loading only in the small stages of loading. With the increase of the loading, they decrease and for load q = 637 kPa they are noticed only in the middle of the foundation edge (y/a = 0) to a depth of $z/a < 0.45$.

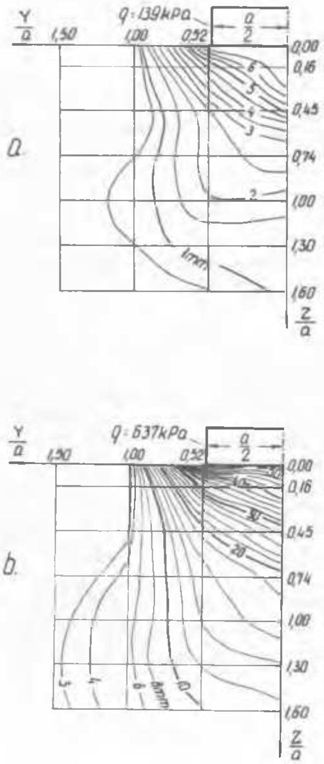

Fig. 7. Isolines of vertical displacements w for section II-II for first loading
a. q = 139 kPa b. q = 637 kPa

The vertical displacements of the points in section III-III are insignificant compared to those of sections I-I and II-II. With the increase of the loading, they increase, whereby for load q = 637 kPa (Fig. 9) they are greatest in the points of the zone z/a = 0.74 to 1.60 and y/a = 0.0 to 0.52 (Fig. 9).

In the second loading with the increase of the load, an increase is observed of the displacements of the points in the zone z/a = 1 and y/a = 1 (Fig. 10) which is due to the sideward pushing out of the soil (increase of horizontal displacements). Under a third loading the pushing out decreases and under a fourth loading it practically disappears.

The horizontal displacements of the points of the sand subgrade are negligible compared to the vertical displacements. Fig. 11 shows the isolines of points of equal horizontal displacements v (in mm) for section I-I. Under small loads q = 36 to 318 kPa, the horizontal displacements are of the order of 0.5 to 1.5 mm and are concentrated in the vertical

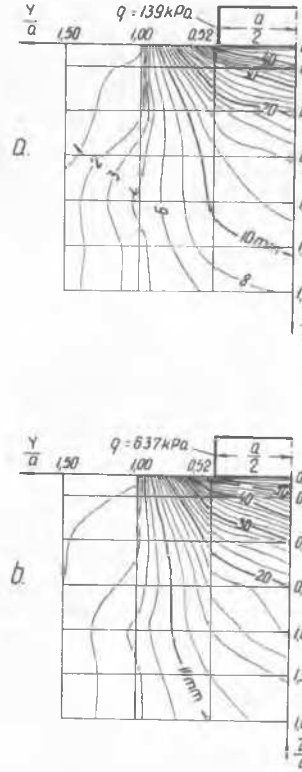

Fig. 8. Isolines of vertical displacements w for section II-II for second loading
a. q = 139 kPa b. q = 637 kPa

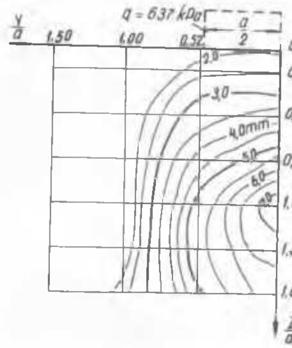

Fig. 9. Isolines of vertical displacements, for section III-III, first loading, q = 637 kPa.

y/a = 0.5 (under the foundation edge) (Fig.11a). With the increase of the load ($q > 497$ kPa) the displacements are increased, whereby the intensive horizontal displacements are extended sidewards (y/a = 1 to 1.5) and downwards (z/a = 1)to 1.5) (Fig. 11b), which means that, for a load q = 497 kPa, an elastic core is formed (Fig. 5c and 7b), which begins to push out the soil slightly towards the sides.

After unloading and reloading the elastic core is preserved, only its height is increased (Fig. 6a, 6b, 8a, 8b) and as a sequence the zone of the intensive horizontal displacements is changed in depth (Fig. 12). Under a third and fourth loading the horizontal displacements are practically stabilized.

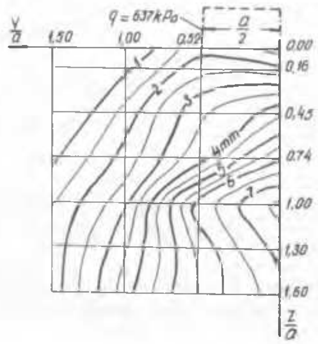

Fig.10. Isolines of vertical displacements w, section III-III, second loading, q = 637 kPa.

The patterns of the horizontal displacements in section II-II (Fig. 13) and in section III-III is similar.

The displacements were also calculated after the Finit elements method and after Boussinesq taking into account the actual distribution of the contact stresses measured during the test, as shown on Fig. 14.

The calculation after the Finit elements method was carried out taking into account the non-linear behaviour of the soil. The results obtained are unsatisfactory and are therefore not given here.

Fig. 15 shows the isolines of the vertical displacements w for section I-I, calculated after Boussinesq for a modulus of deformation M = 20 MPa and Poisson's ratio $\mu = 0.32$.

Their comparison with those measured during the test (Fig. 5c) shows that the calculated displacements are smaller.

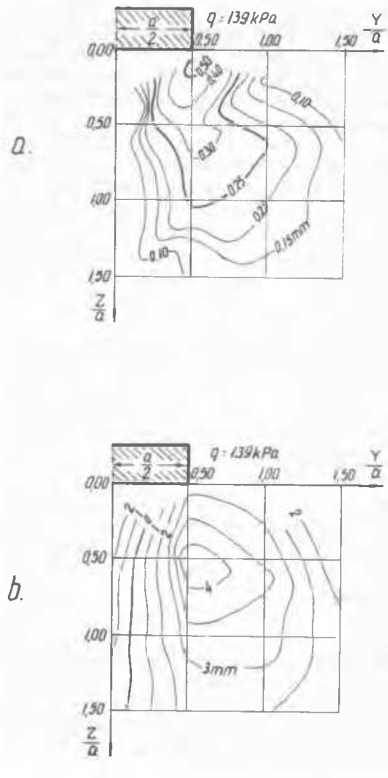

Fig. 11. Isolines of horizontal displacements v for section I-I for first loading
a. q = 139 kPa b. q = 637 kPa

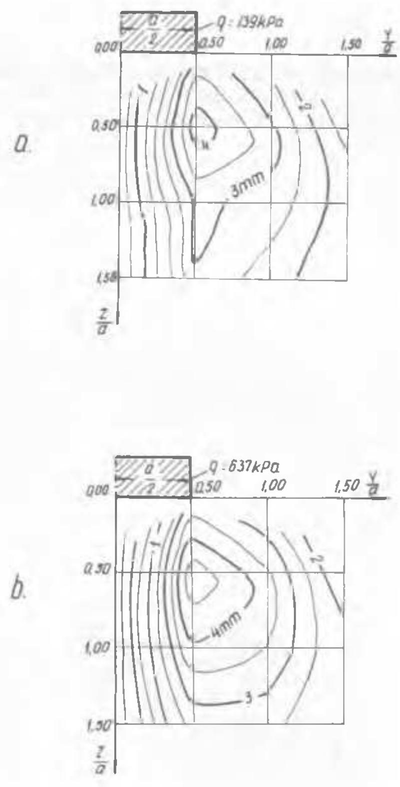

Fig. 12. Isolines of horizontal displacements, for section I-I, second loading
a. q = 136 kPa b. q = 637 kPa.

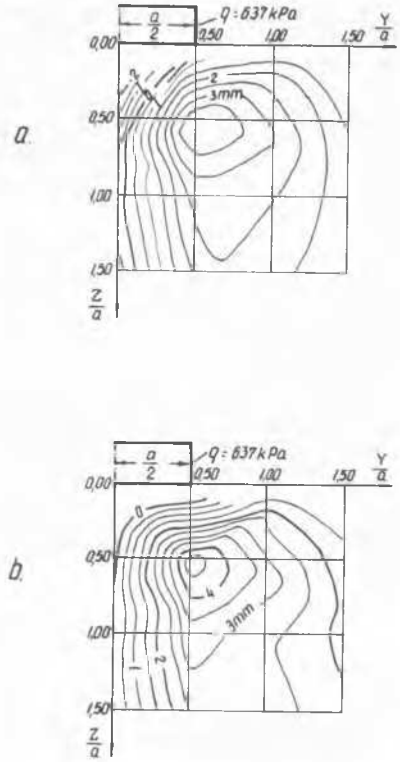

Fig.13. Isolines of horizontal displacements v for section II-II with load q = 637 kPa:
a - first loading; b - second loading.

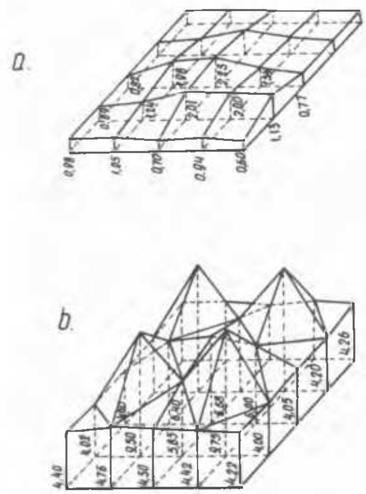

Fig. 14. Measured contact stresses:
a - q = 139 kPa; b - q = 637 kPa.

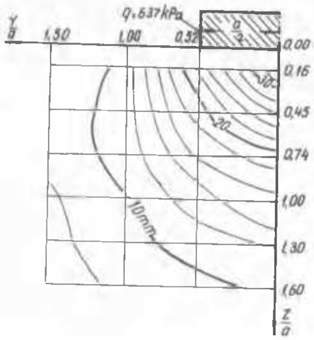

Fig.15. Isolines of vertical displacements w for section I-I, calculated after Boussinesq.

The study of the stress-strain state continues applying calculation and experimental methods.

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