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USE OF MICROPHOTOGRAPHY AND DENSITY-TESTING METHOD AT RESEARCHES ON DISTRIBUTION OF TENSION-AND DEFORMATION-CONDITION OF SANDY AND TIED SOIL.

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The theoretical researches on the distribution of tension of sandy and tied soil are connected with names like Boussinesq, Strohschneider, Kögler, Scheidig, Hugi, Gerber, Jansen, Engesser, Caquot and Dörr; the practical researches with names like Marsten, Anders, Schlick, Clemmer, Spangler, Masen, Winfrey, Jansen, Talbot, Braune, Vain, Janda, Crum, Yamaguti and Armes.

Because all these works are well-known I do not want to enter into full particulars. The methods used at all these researches are the following: the deformation and stretching condition were taken and made visible with the aid of a mechanical apparatus. Exceptions are the works by Hugi, Gerber and myself, by whom the deformation conditions of sand material were observed by means of Röntgen-rays.

At my present researches I used new methods which not only may clarify and simplify the control of the existing methods of calculation but may serve also as solution of foundation problems (general as well as pile works).

The methods used by myself are:

1) the microphotographical (used on sand, sandlike material and soil of colloidal mixture)
2) the density-testing method (tried on different proof material).

<u>ad 1)</u> The sand was sieved and scale - and needle - shaped substances were eliminated. In such a way the proof material consisted of almost equally big grains with more or less three equal expansions. In that way the researches were simplified and the result clearer. The pores which resulted in this sandy substance were of greater uniformity (so to speak of homogeneous condition) and the observation of the movement of the isolated grains was easier because of the uniformity of motion.

Part of the sandy material was mixed up with some cement into beton and tied. Before the act of amalgamation this material was tested:

gruff - with the Vikat-apparatus (the change of irruption with a standardized needle which penetrated into the testing material)

fine - through delicate thermo-elements (increasing of temperature by amalgamation).

At the first stage of amalgamation when the Vikat-apparatus does not offer any resistance and the thermo-elements don't demonstrate any Peltier-current, consistency testings (according to Atterberg's system) were applied to the cement-tied material. Because of no difference between the two proof materials they are to be considered at this stage as soil mechanically identic.

At this first stage the cement-tied material was loaded in series;

the first series with a weight,

the second series with a testing pile which was inserted into the testing material.

As the act of amalgamation took place the material grow hard. In this way the movements of the grains which were caused through loading became stable for further examination.

The hard grown beton testing material was cut, that is

the loaded substance in main meridial level

the piled substance vertical to the pile axis.

The transversal sections were treated by methods known in the metallography and petrography and afterwards at a striking polarized light studied and photographed.

<u>Fig. 1, 2 and 3</u> are pictures of the beton substance three times enlarged. At this slight enlargement the change of porosity through shifting cannot easily be observed, also the diversity of refraction of light makes a clear observation difficult.

<u>Fig. 5, 6, 7</u> are pictures 50 times enlarged. Here the change of the pores is already clearly discernable.

Fig. 5 is near the loaded spot,
Fig. 6 " 2 cm from the loaded spot,
Fig. 7 " 5 " " " " "

The black stains are pores, the white and grey ones the sand respectively cement-tied grains. It is plainly visible that the pores become greater when farther away from the loaded spot and that at a greater distance from the loaded spot no more shifting can be

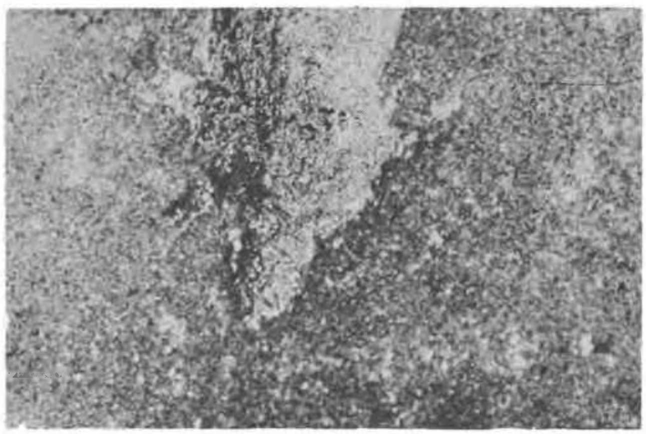

FIG.1

FIG.2

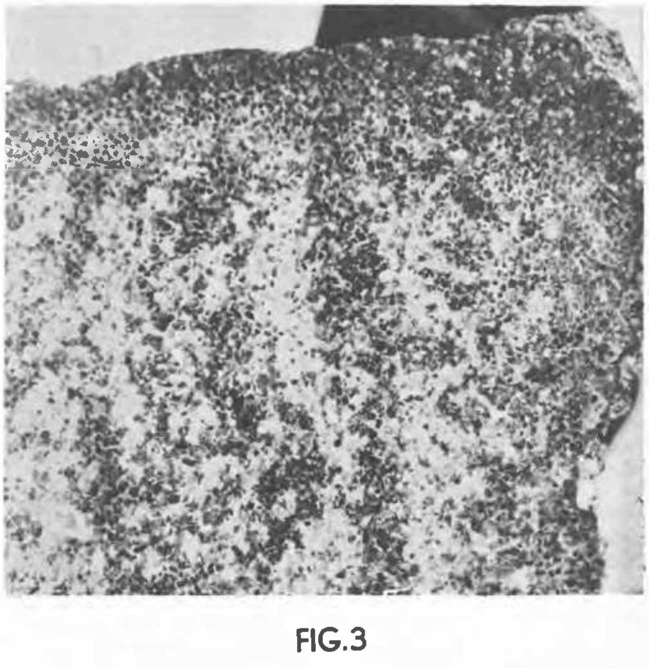

FIG.3

FIG.5

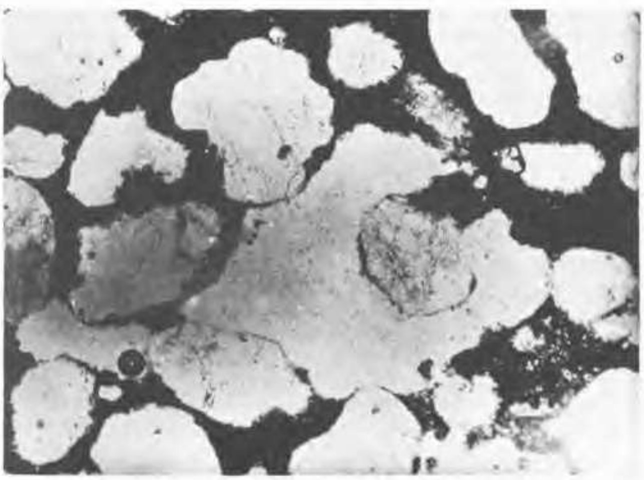

FIG.6

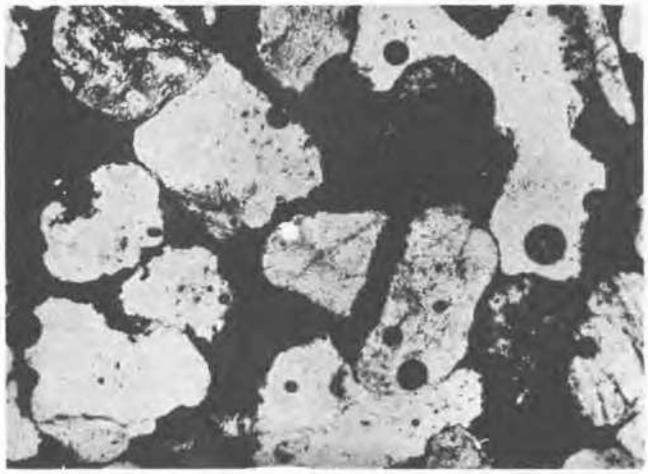

FIG.7

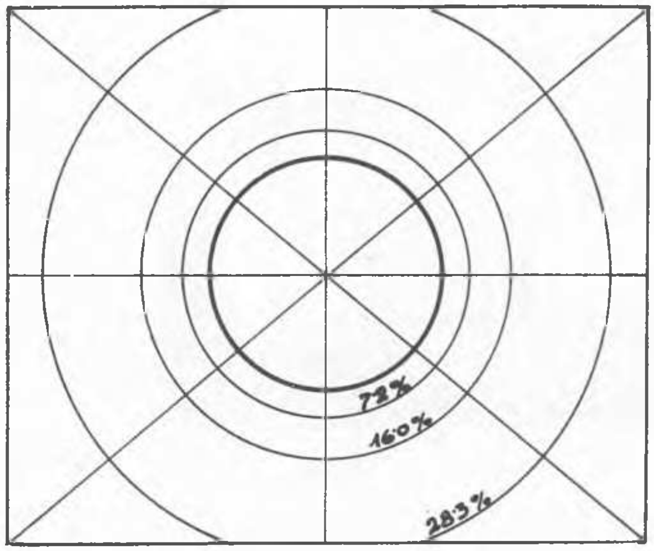

FIG.8

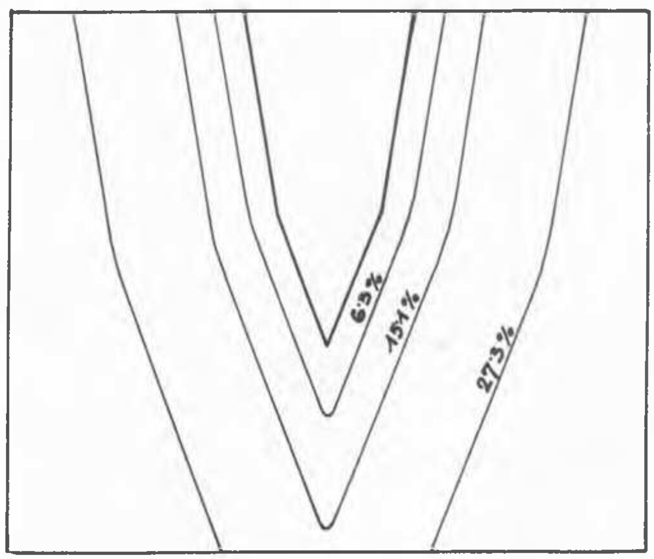

FIG.9

observed. There the pores are similar to those of the same substance but unloaded testing material. The porosity was established controled and planimetrical proved after Rossival's known method. (on a line net measured cuttings). From the connection between the black and the white planes (resp. the corresponding linear planes) the proportion of porosity of the loaded soil material was numerical determinable at a given distance from the loaded spot.

Fig. 8, 9 demonstrate on the pressed sand material on the meridial surface, on the pile vertical to the axis the "Isobaren" that is the lines which unite the spots of equal motion-size respectively the equal proportion of porosity and the equal specific pressure. The pictures of the loaded (pressed material are nearly the same as those from the Boussinesq equation derived pictures of function. Through the driving of a pile there result in the ground concentric shapes equal to the pile-transverse-section (circular form, ellipsis, multangular forms). It is easily comprehensible that in their sphere of action the tensions are functions of the original porosity, the moisture-contents and the weight of loading. But till now it was impossible to prove a simple connection between these dimensions.

ad 2) experienced on
a) hardened beton material
b) paraffin testing material
c) loam (clay) soil.

Concerning the three testing materials the proving method is the same.

Fig. 10 shows Brinnel's apparatus with which small steel bullets differently loaded with 10 to 100 kg were pressed into the testing substance. The depth of penetration is measured with the aid of a Zeisswatch for measuring to an exactness of one micron. On the cementtied substance which was loaded to 100 kg the resonance of the penetrating could exactly be measured. The loading of the surface (problem of the loaded semi-space (Halbraum)) through reducing the porosity, creates a density of the substances which fades away at an inverted connexion to the distance.
Similar phenomena can be observed from the loading with a pile.

Fig. 11 shows the "isobars" (similar depths of penetration of the bullets, similar Brinnel's densions from the same loading) on paraffin testing material.

Fig. 12 shows the building of a pile-foundation in the proximity of one already existing. The sphere of tension of the new foundation should be resolved and established in what manner the condition of tension of the new

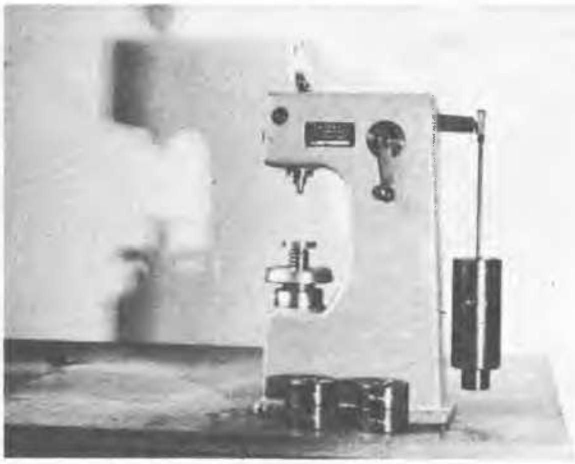

FIG.10

FIG.11

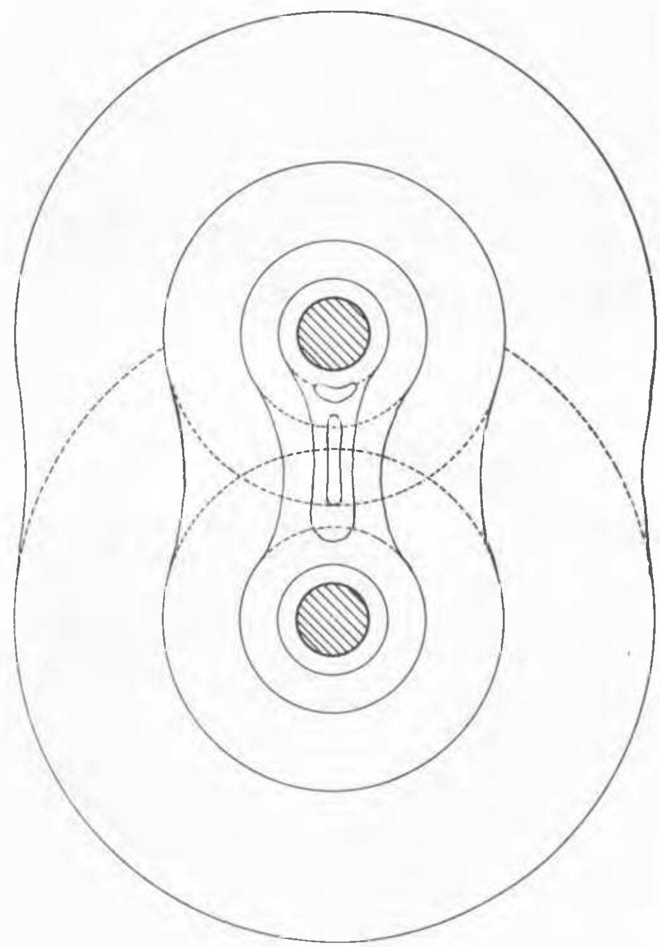

FIG.12

foundation upsets the old one's, resp. at what distance the new foundation has to be built without evoking dangerous tensions in the sphere of the old one. We know from practical experiences that piles too nearly driven cause density of soil and increase the carrying capacity. If there were piles driven again in the same sphere of density the resistence of the ground increases against this irruption. Through experiments on moulds the distance of the piles at which no influence of tension exists, can be fixed. Because of no "overlaping" (Uebergreifungs) tension a high degree of stability can be obtained. However the distance of piles can also be fixed, utilizing the density of soil which is caused by the "overlaping" tension. Because of the increased carrving capacity of ground material-expences can be reduced through retrenchment of the transverse section or through diminishing the number of piles.

Comparing the pictures of the two methods they nearly show the same results.

For the definition of the distribution of tension in grounds the two methods can equally be indicated as fit for use.

The use of these methods is of special value both at definition of foundation in groups and building erected on piles.

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VISCOUS FLOW TUBE MODEL

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SUMMARY

A model method is presented whereby variations of excess pore water pressures in a consolidating soil mass may be obtained in cases that include both expansion and compaction of soil mass. The various physical relationships between the prototype and model are discussed and a comparison of results obtained from a model and by formal mathematics are made. The basic idea of the model may also be used for the solution of frost penetration into the ground.

The viscous flow tube model is a hydraulic device which indicates approximately the variation of hydrostatic excess head of the pore water in a consolidating soil mass. The model is well suited for use, not only for illustrating such variations of head for the simple case of compression with one-dimensional flow, but also it may be used to obtain reasonable approximate solutions that are very difficult of solution by formal mathematical means. Various approximate mathematical solutions have been proposed 1) 2) but to the writer's knowledge a viscous flow tube model has never been used before for this purpose. Electrical models have been used for somewhat similar problems, such as heat flow and flow of compressible fluid flow, but the viscous flow tube model has a superiority over such electrical models because it can be adapted without difficulty to the cases where the same soil mass is subjected to expansion and then compression during the process of consolidation.

The upper sketch of figure 1 indicates a prototype foundation of width 2(H + L) centrally loaded for a width 2 H by an instantaneously applied uniform strip load. The compressible soil is assumed to be so highly stratified that all excess water forced out of the foundation by the load drains laterally. The prototype foundation is divided into equal segments, which are represented in the model by the vertical reservoir tubes. The amount of water forced out of the soil per unit load is simulated in the model by the amount of water drained out of the compression reservoir tubes per unit loss of head. The same conception holds for the soil expansion and the expansion reservoir tube. Thus the cross-sectional area of each tube represents the compressibility or expandibility of each segment in the prototype.

The water conductivity of the prototype soil per unit gradient is the permeability coefficient multiplied by the area normal to the direction of flow. In the model, the conductivity is simulated by the carrying capacity of each resistance tube per unit gradient. The hydrostatic excess head in the pore water of the prototype is represented in the model by its hydraulic grade line above the zero datum of the model.

The basic partial differential equation for consolidation at any given point in the prototype is 2):

$$\frac{k\,A\,\delta^2 u}{\gamma\,\delta x^2} = \frac{A\,a_v\,\delta u}{(1+e)\,\delta t} \qquad (1)$$

(net outflow rate of water) = (net rate of volume change of the soil mass)

where

k = coefficient of soil permeability in the horizontal direction,

A = unit cross-sectional area of prototype normal to the direction of flow,

γ = unit weight of water,

u = hydrostatic excess pore water pressure,

x = horizontal distance from the centre line of the prototype,

a_v = coefficient of soil compressibility

a_e^v = coefficient of soil expandibility

e = void ratio of soil,

t = time from application of load.

If the prototype soil is expanding because of