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Settlement prediction based on pressuremeter and oedometer test results
La prévision des tassements basée sur les résultats des essais pressiométriques et oedométriques

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SYNOPSIS: The paper is referred to the experimental investigation of the pressuremeter method and the conventional oedometer parameters for settlement prediction of superficial foundations on granular soils. In rigid footing loading tests, the deformations are broken down into deformations due to volume change of soil and into deformations resulting from shear phenomena. The analysis leads to critical consideration of pressuremeter method and explains the discrepancy that is observed between measured and calculated, with oedometer modulus, settlements. The possibility of using the pressuremeter parameters in classical methods is examined and the design parameters resulted from this investigation are given.

1 INTRODUCTION

Two of the most adaptable methods of settlement prediction are the use of oedometer parameters in the classical methods and the pressuremeter method of Ménard. The last one is used in France almost exclusively. The opinions, concerning the reliability of these two methods, diverge. In the disputable simulation of plane superficial footing to a rigid sphere, sunk into the soil in the pressuremeter method, the considerable discrepancies between measurements and predictions to which the use of oedometer parameters in classical methods leads, are placed against. This paper intends to the investigation and the comparative consideration of these two methods, based on experimental results.

2 THE PRESSUREMETER METHOD

In the pressuremeter method, a clear distinction of the half space, below a footing, into two different regions, is made. A region close to the footing, where a spherical stress field is accepted that predominates and the rest deeper region, where it is assumed that stress conditions of deviatoric character principaly predominate. In Figure 1, these two regions, in case of a rigid circular footing, are illustrated. The footing settlement will result from the compression of soil half-sphere C and from the shear deformations of region D. Therefore, for the calculation of the two settlement components, the elastic solution which gives the reduction of the ray of a sphere subjected to external pressure and the elastic solution of Josselin de Jong (1957) for the settlement of a rigid sphere, sunk into an infinite medium, are applied respectively. The different stress conditions which predominate over these two regions, impose the use of two different deformation moduli; a shear modulus for region D which will be determined by net shear tests and a compression modulus for region C which can be determined either by spherical character tests or only by empirical correlations between these two mo-

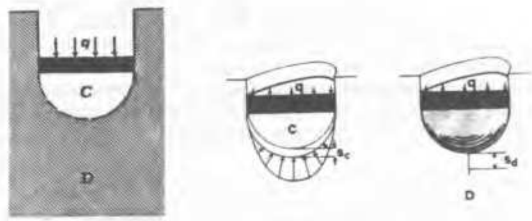


Figure 1. The pressuremeter method.

duli. According to Ménard and Rousseau (1962), the stress state which is developed during the pressuremeter test, represents the deviatoric character of region D.

Hence, the two settlement components can be written:

$$s_c = \frac{1}{9K_M} \cdot q \cdot B \cdot \lambda_c \text{ ....settlement due to volume change of region C}$$

$$s_d = \frac{1}{12G_M} \cdot q \cdot B_o \left(\lambda_d \frac{B}{B_o}\right)^{\alpha} \text{ ...settlement due to shear distortion of region D}$$

where q=the net average bearing stress, B=the width of the footing, B_o=a reference width, usually equal to 0,6 m, λ_d, λ_c=shape factors, α=an empirical factor, $\alpha<1$, depending on soil type (for linear elastic medium $\alpha=1$), G_M=the pressuremeter shear modulus, K_M=the compression modulus; it is estimated through the semi-empirical relation: $K_M=2{,}66\ G_M/\alpha$.

3 TEST DESCRIPTION

The tests consisted of four loading tests on a natural scale model. In an experimental container filled by dry sand, successive loads on a rigid strip footing embedded in the sand, were applied and final strip footing settlements and sand deformations were measured. Appropriately located vertical and horizontal arrangements of thirty electric deformation recorders inside the sand, allowed the breakdown of vertical de-

a) *b)* *c)*

Figure 2. Model tests: a) Strip footing and hydraulic loading installation. b) Vertical and lateral deformation gauges inside the sand. c) Cross section ⊢┬...deformation gauges, ⊕ ...pressuremeter test location.

formations into deformations due to volume change of soil and into deformations resulting from lateral sand deformation. The recorders were a type of variable inductance gauges with measurement accuracy equal to 0,001 mm. The experimental arrangement is illustrated in Figure 2. The loading of the reinforced strip footing, with 40 cm in width, was applied by nine hydraulic jacks. The sand used for the tests was a uniform medium-grain to coarse-grain sand with U=2,0. The pressuremeter characteristics were determined by a GB pressuremeter. The mechanical soil characteristics had been determined by a serie of oedometer, direct shear and triaxial tests.

4 TEST RESULTS AND SETTLEMENT CALCULATION. CRITICAL CONSIDERATION OF THE METHODS

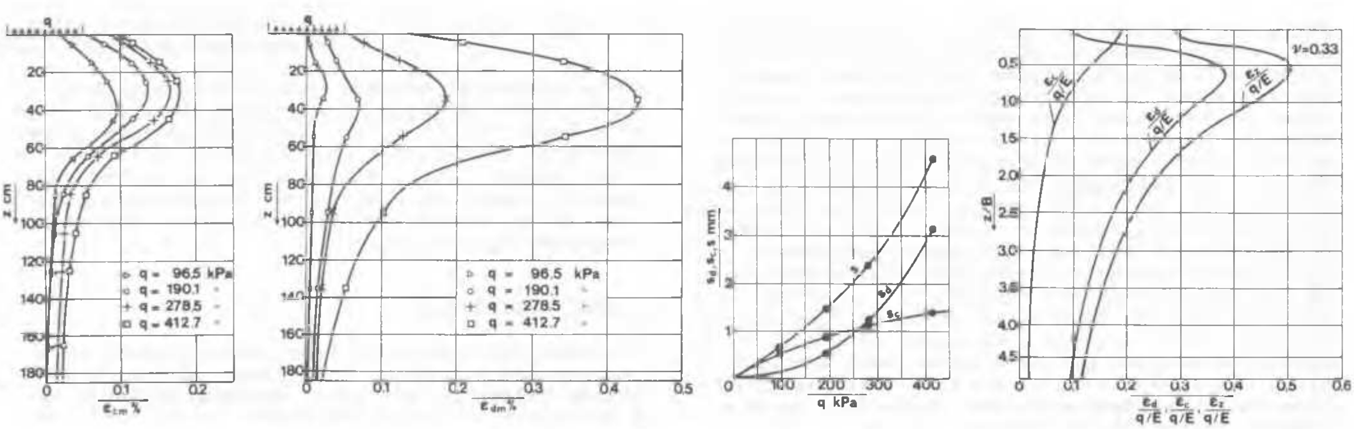
The final deformation components were determined for four different load sizes in each density.

The test results for density Dn=0,89 are illustrated in Figures 3a, 3b. In Figure 3a the distribution with depth of the mean -on the footing width- vertical strain ε_{cm} due to volume change and vertical strain ε_{dm} due to shear distortion, was plotted for each load. Their maximum values appear to be in a distance from the footing almost equal to 0,75 to 1,0 B. For relative small loads, the volume change strains predominate; with load increase, oppositely, the shear strains predominate.

The above observations do not agree with the considerations made in pressuremeter method. Over 80% of the volume change settlements is realized in the region where Ménard and Rousseau accept zero values of volume change strains, whereas considerable shear strains are observed close to the footing. For the experimental data (α=0,33, B≃B_o, λ_d=2,70, λ_c=1,50), the volume change settlements owed to be, according to pressuremeter method equations, equal to about one sixth of shear settlements for all the four examined load sizes.

For the investigation of Boussinesq elastic solution, the total vertical strain ε_z was broken down into volume change strain ε_c (spherical strain) and into vertical shear strain ε_d (deviatoric strain). In Figure 3c, this breakdown for the linear elastic medium with ν=0,33, was plotted in terms of q/E for the case of rigid strip footing. The maximum values of the two strain components appear in depth z equal to z=0,65B for ε_d and z=0 for ε_c.

Comparing the measured and the theoretical



a) *b)* *c)*

Figure 3. a) Distribution with depth of the measured mean values of volume change strains ε_{cm} and of shear strains ε_{dm} in density 0,89. b) Breakdown of total settlement into volume settlement s_c and into shear settlement s_d. c) Theoretical distributions of strain components along the vertical axis below a rigid strip footing in terms of q/E (elastic medium, ν=0,33).

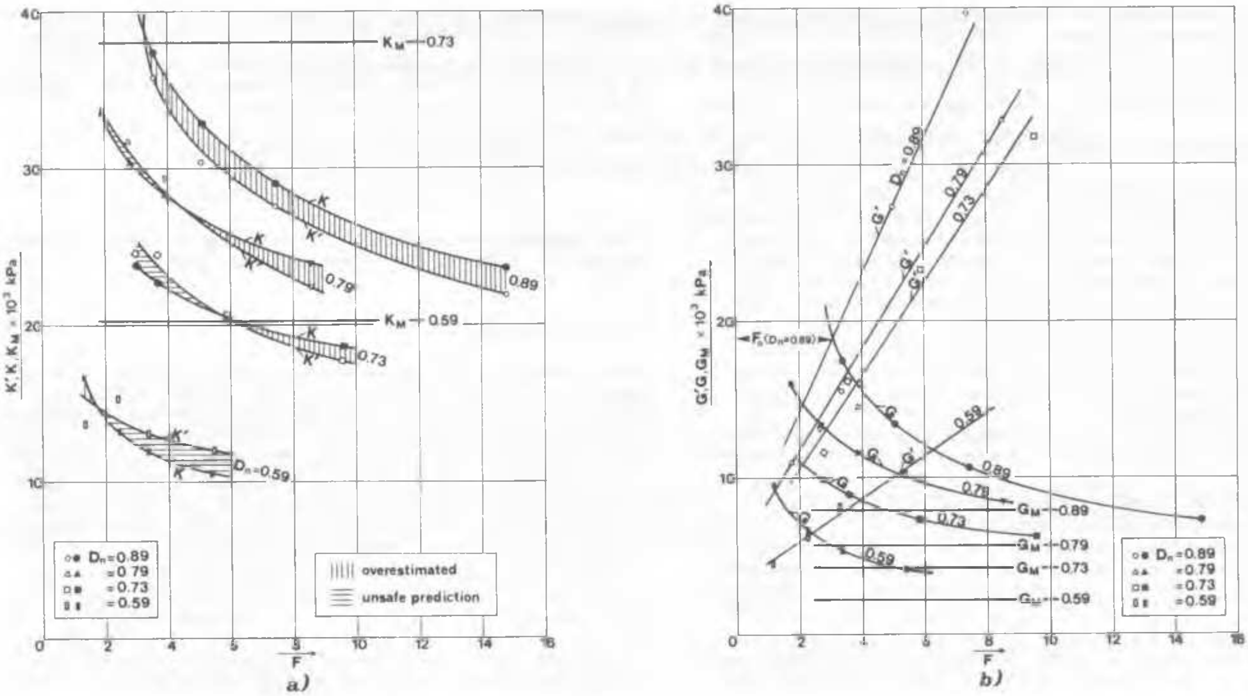


Figure 4. Comparison of backcalculated moduli, K', G' and moduli measured in oedometer, in terms of compression modulus K and shear modulus G, and pressuremeter moduli G_M, K_M. a) Case of volume change settlement. b) Case of shear settlement. ○△□▫...measured, ●▲■▪...backcalculated.

strain distributions, according to Boussinesq, it is found out that their principal differences lie in the fact that the maximum values of measured and theoretical volume change strains ε_c appear to be in different depths and that the presence of the deviatoric strains, during the tests is considerably reduced, for small loads. For the experimental data (strip footing, ν=0,33, z=4,6B), the application of Boussinesq's solution would lead, regardless of load size, to shear settlements about three and half times greater than volume change ones (Valalas, Marangos, 1986).

This last discrepancy should be attributed mainly on the wellknown different behavior which real soil presents in deviatoric and spherical stress field. The soil resistance increases considerably with the increase of spherical stress tensor, whereas it reduces with the increase of deviatoric stress tensor (Figure 3b); the increase speeds of the two strain components with load size are different (Figures 3a, 3b).

Hence, the consideration of Mênard and Rousseau to use two different moduli, a volume change modulus for spherical stress field and a shear modulus for deviatoric stress field, is judged as right. From the experimental results, it is concluded that, this way can lead to the improvement of classical methods.

The breakdown of total vertical strain into its components, in Boussinesq solution, allows to investigate the usefulness of moduli, determined experimentaly, for the prediction each of the settlement component separately.

For the investigation usefulness of oedometer modulus in this direction, in Figures 4a, 4b, the moduli K', G', which lead to the measured settlement components, as well as the one dimensional compression moduli E_s, measured in oedometer, in terms* of volume change modulus K and shear modulus G, are plotted for each density, in dependance on safety factor against failure F (according to DIN 4017). The moduli E_s^* were determined in the depth of the maximum measured strains: z=1,0B, for each load size. In those figures, it is observed that in order to have identification of predictions to measurements for different values of F, different values of moduli, for the same density every time, should be introduced.

In case of volume change settlement s_c, the backcalculated moduli K' approach sufficiently the moduli measured in oedometer, in all the examined cases (Figure 4a).

In case of shear settlement s_d, the dependance of G' from F was approached linearly (Figure 4b). In this figure, the decreasing function of G' from F and the increasing function of modulus, measured in oedometer, from F, are illustrated clearly. This divergence has as consequence the prediction accuracy to depend mainly on safety factor value. There will be a value F_o where predictions will be identified to measurements. This value was ranged between 2,2 and

* $K=E_s\frac{1+\nu}{3(1-\nu)}$, $G=E_s\frac{1-2\nu}{2(1-\nu)}$, $E_s=\upsilon\cdot\left(\frac{p}{p_1}\right)^w$, w...experimental soil parameter: $0\leq w\geq 1,0$, $\upsilon=E_s$ for p=100kPa, p_1=100kPa, p...vertical stress at depth z=1,0·B: $p=\sqrt{p_o\cdot(p_o+\sigma_v)}$ ...the geometrical mean value in this depth of vertical pressure p_o of the overburden and vertical stress σ_v resulting from footing bearing pressure.

3,0 for the examined densities. For $F>F_o$, s_d will be overestimated, whereas for $F<F_o$ the predictions will be unsafe. The overestimation of s_d for high values F is due to the reduced values of shear modulus, for small loads, to which the theoretical relation $G=E_s\cdot\frac{1-2\nu}{2(1-\nu)}$ leads, as results also from the comparison of triaxial and oedometer test results, where the initial tangent moduli, determined by triaxial tests, are much too high. The contrary happens for big loads.

All the above explain also the discrepancy between predictions and measurements which is observed in classical methods. Especialy in case of dense deposits, where the applicated bearing pressure is usually much smaller than bearing capacity, settlement overestimations will be expected.

Hence, it is concluded that the oedometer modulus is appropriate for settlement prediction due to volume change of soil. However, this modulus can lead to considerable inaccurate predictions of shear settlements.

The use of oedometer modulus in Boussinesq's solution led to ratio values, between measured and predicted volume change settlement, ranged between 0,84 and 1,17 with a mean value of 1,0 and standard deviation SD=0,1. For shear and total settlement, these values were respectively equal to 0,08-1,21, 0,51, SD=0,31 and 0,29-1,20, 0,62, SD=0,24.

The ascertainment, during these tests, of sufficient prediction of volume change settlement by oedometer method (assumption of confining lateral deformation) is of special practical importance. In this method, the knowledge of value ν is not necessary because the spherical component is calculated directly from E from the relation $\varepsilon_c=\sigma_v/3E_s$. The dispersion values in this case, were found equal to 0,88-1,39, 1,05, SD=0,14.

The investigation usefulness of pressuremeter parameters for settlement prediction by classical methods, demonstrated that pressuremeter modulus G_M is determined very small (Figure 4b). Contrarily, it was found out that the empirical relation $K_M=2{,}66\ G_M/\alpha$ leads to high values of compression modulus (Figure 4a). The solution of rigid sphere*, the empirical coefficient and the assumption that the volume change strains are limitated in region C, are called up to compensate the small values of G_M.

The experimental investigation of pressuremeter method led to ratio values, between measured and predicted settlement equal to 0,25-1,39, 0,83, SD=0,37 and 2,13-5,61, 3,18, SD=0,94 for s_d and s_c respectively. However the ratio values between measured and calculated total settlement were found equal to 0,86-1,65 with a mean value of 1,2 and SD=0,25.

Hence, it is resulted that the concentrated experience in pressuremeter techniques is so valuable that the pressuremeter method can be classified in the category of the usefull empirical methods, regardless of the theoretical unsufficiencies which we have seen till now.

The found out dependance of prediction accuracy from F, during the tests, puts the condition of using variable -with load size imposed on the footing- deformation moduli $G_M(F)$, $K_M(F)$. A statistical investigation based on test results led to the following relations:

$$G_M(F)=G_M\cdot(0{,}2421\cdot F+0{,}2454)$$

$$K_M(F)=K_M\cdot(0{,}4346-0{,}0193\cdot F)$$

The dispersion values, in this case, were found equal to 0,82-1,38, 1,01 and SD=0,17.

* It is demonstrated that in case of rigid strip footing, for linear elastic medium, the total settlement coefficient of Boussinesq solution, is about two times greater than that of pressuremeter method.

5 CONCLUSIONS

The classical methods as they are applied nowadays with the use of only one deformation modulus, can lead only accidentaly to satisfactory settlement predictions. The sufficient problem approach presupposes the use of two different deformation moduli. A spherical modulus for settlement prediction due to volume change of soil and a deviatoric modulus for settlement prediction due to shear deformations.

The oedometer modulus is appropriate for the prediction with Boussinesq's solution of the portion of settlement due to volume change of soil. For the deviatoric portion of settlement it seems that Boussinesq's solution can lead to useful results using an appropriate shear modulus. The investigation has to be directed in search of a variable -with deviatoric stress size- shear modulus, determined by constant volume shear tests.

The use of pressuremeter modulus G_M, in classical methods, is not indicated. The theoretical base of pressuremeter method is unsufficient. However, the experience of many years demonstrated that this method can be considered as a useful empirical method.

ACKNOWLEDGEMENTS

The tests have been carried out at the Laboratory of the State Institute of Soil Mechanics of Nurnberg. The author wishes to express his gratitude to Professor M. Kany and Dipl. Ing. S. Jänke, who have assisted in several ways.

REFERENCES

Josselin de Jong, G., (1957), "Application of Stress Functions to Consolidation Problems", Proceedings of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, August, Vol. 1, pp. 320-323.

Ménard, L., Rousseau, J., (1962), "L'évaluation des tassements-Tendances nouvelles", Sols-Soils, Vol. I, No. 1, Juin, pp. 13-29.

Valalas, D., Marangos, Ch., (1986), "Propositions pour l'amelioration des methodes de prevision des tassements des fondations superficielles", Annales de Travaux Publics de Belgique, No. 6, pp. 527-542.