Settlements of an Embankment on Organic Soil

Tassements d'un Remblai sur Sol Organique

D. SARAČ Professor of Soil Mechanics, Institute for Geotechnics and Foundation Engineering, Civil Engineering Faculty, Sarajevo, Yugoslavia

M. POPOVIĆ Lecturer of Soil Mechanics, Institute for Geotechnics and Foundation Engineering, Civil Engineering Faculty, Sarajevo, Yugoslavia

SYNOPSIS . Analysis of the settlements and pore pressure dissipation is presented for an embankment constructed on the layer , about 10 m . thick,composed of peats and organic clays . The embankment , with maximum height 5.15 m , is widened by berms so that entire width of the profile is 75 m . Characteristic variation of soil properties was obtained , where void ratio , water content and organic matter content increase with depth . The case is considered as one-dimensional consolidation with soil parameters obtained by oedometer tests . Influence of non-linear stress - strain relationship is analysed , as well as variation of the coefficient of permeability , soil heterogeneity and thickness decrease of the compressible layer . Results of the calculations are compared with field observations of pore pressures and settlements . Rather good agreement between prediction and performance is obtained , even with simplified calculation assumptions .

INTRODUCTION

Analysis of the settlements and pore pressure dissipation is presented for an embankment constructed on the layer , about 10 m. thick , composed of peats and organic clays . The embankment , 1,7 km long , with maximum height 5.15 m is designed for storage purposes . By reason of very low shear strength of soil , the embankment is widened by berms , so that entire width of the embankment profile is 75 m. Even with such profile, it was necessary the rate of the construction to be limited in order to improve shear characteristics by partial consolidation of the subsoil . The stability conditions and accepted solution of the embankment are given in a previous paper of authors (Sarač, Popović , 1980).

A characteristic cross - section of the embankment is shown in Fig. 1. The embankment construction was divided to two basic phases . The partial subsoil consolidation under the load of the phase 1. should have improved the stability conditions at construction of the phase 2. Water drainage at the contact surface between the embankment and subsoil is enabled by a special drainage system . A sand layer is situated under compressible soil, so that pore water flow during the consolidation occurs in two directions .

In this paper ,problem of settlements and pore pressure dissipation is considered for the phase 1. of constructi -

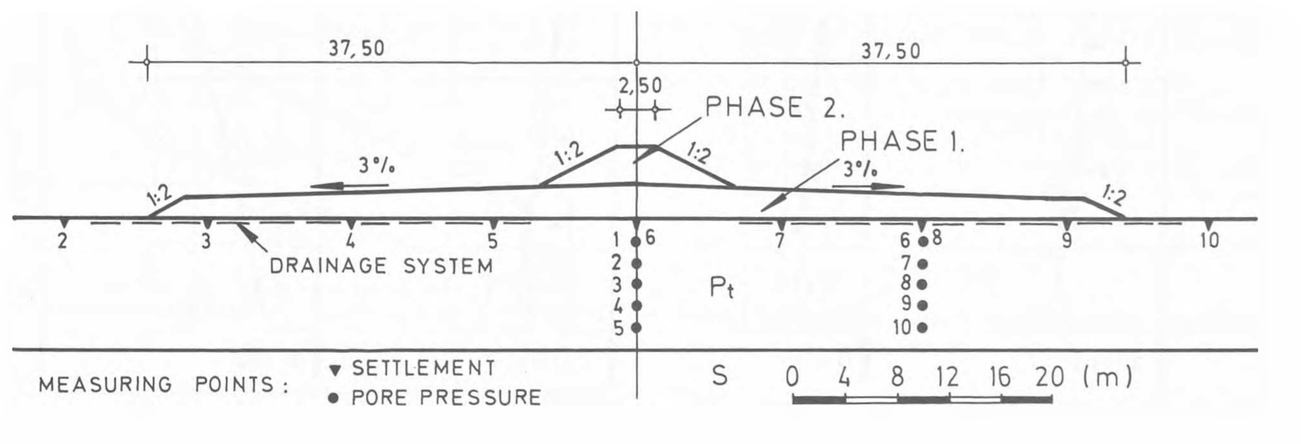

Fig.1. Cross-section of the embankment

on , in initial period , lasting about two years . The analysis of the consolidation in this period has given data on expected final settlements as well as on possibilities of subsequent construction , depending on attained effective stresses .

Comprehensive field and laboratory soil testings were carried out during the design elaboration , while the settlements and pore pressures were measured during the construction , with the purpose of control and eventual correction of accepted calculation parameters .

SOIL CONDITIONS

The upper layer of soil , mainly 10 m thick , consists of peaty soil , which is locally , immediately to the soil surface , transformed to organic clays . A layer of medium dense sand with relatively good mechanical properties is situated under the peaty soil . Ground water level is very high , so that the lowest levels during the year in the middle part of the embankment are about 1 m. below soil surface . Besides , the area is flooded in rainy periods .

As the embankment material , anorganic clays of medium plasticity (Cl) were used,from an adjacent borrow area.

Basic results of laboratory testing in peaty layer for a characteristic profile of the embankment - MS1 - are given in Fig.2. According to the classification , suggested by Amaryan et al. (1973), this peat belongs to category IV. - peaty soil . It is evident from diagrams that void ratio (e) and water content (w) considerably increase with depth . This is consequence of the fact that organic matter content also increases with depth in rather wide interval of 20 - 40 % . Void ratio varies in limits from about 3.0 to 6.0 and water content from 100 to 250. Effective unit weight (γ_{ef}) decreases with depth from 2.5 to 1.0 kN/m^3 . Consequently , increase of effective stresses with depth is very small . It was shown by statistical evaluation of the results that variations of above mentioned properties nearly correspond to linear relations , with parameters given in Fig . 2.

On the other hand , results of field vane tests and static cone tests have given relatively small variation with depth , except in upper zone , 1-2 m. thick , where slightly better results were obtained . Undrained shear strength , determined by vane tests, amounts in average 15 kPa and cone resistance about 250 kPa . The ratio q_c / τ_{fv} = 16.7 nearly corresponds to the results, obtained by Begemann (1965) in very soft clays (q_c/τ_{fv} 15).

SPT values N=1-2 blows were obtained for peaty layer .

Properties of sands , underlying peaty layer , are not important for considered problem , because of relatively small compressibility and high permeability .

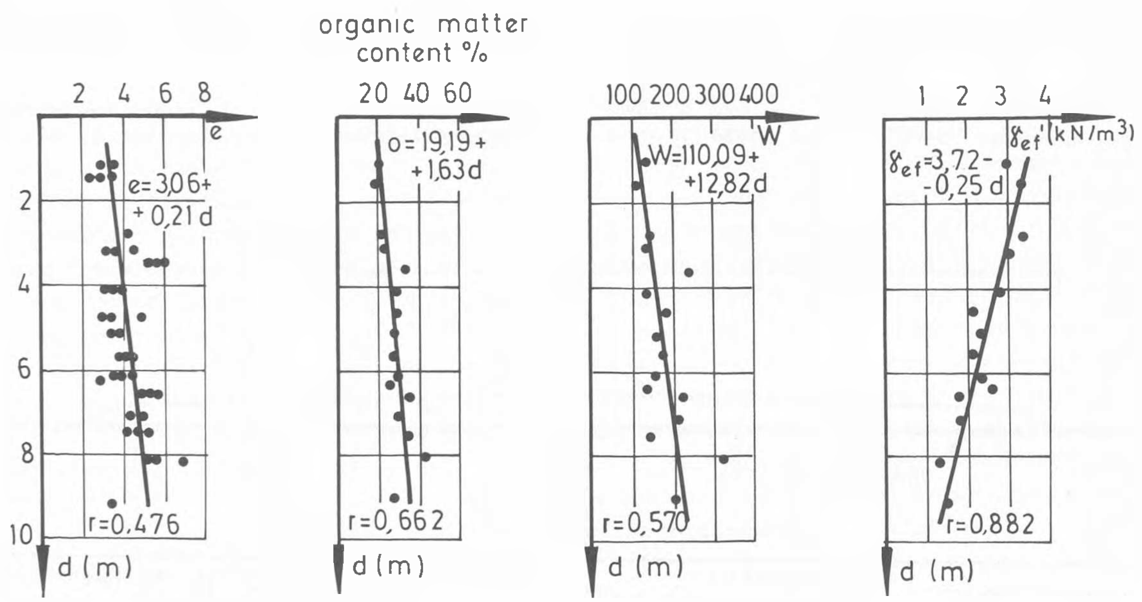

Fig. 2. Basic properties of peaty soil - profile MS1

CALCULATION SUPPOSITIONS PARAMETERS

As the ratio of the thickness of compressible layer to the embankment width is small, for the settlements in the embankment axis , case of one - dimensional consolidation is considered , with soil parameters obtained by oedometer tests . With regard to properties of compressible layer and construction conditions , at calculation of the consolidation following influences were involved : layer heterogeneity, nonlinear stress - strain relationship , variation of permeability coefficient with effective stresses and decrease of layer thickness during consolidation . Besides , the calculation is performed for time - irregular loading , including ground water level variations .

The calculation of primary consolidation was done by finite differences method, applied to one-dimensional case of Biot's differential equation (Šuklje , 1969):

$$\frac{\partial \sigma'}{\partial t} = \frac{1+e}{\gamma_w \cdot \frac{\partial e}{\partial \sigma'}} \cdot \frac{\partial}{\partial z}\left(k \cdot \frac{\partial u}{\partial z}\right) \qquad (1)$$

Layer heterogeneity was approximately taken into account by introduction initial void ratio e according to linear relation from Fig. 2.

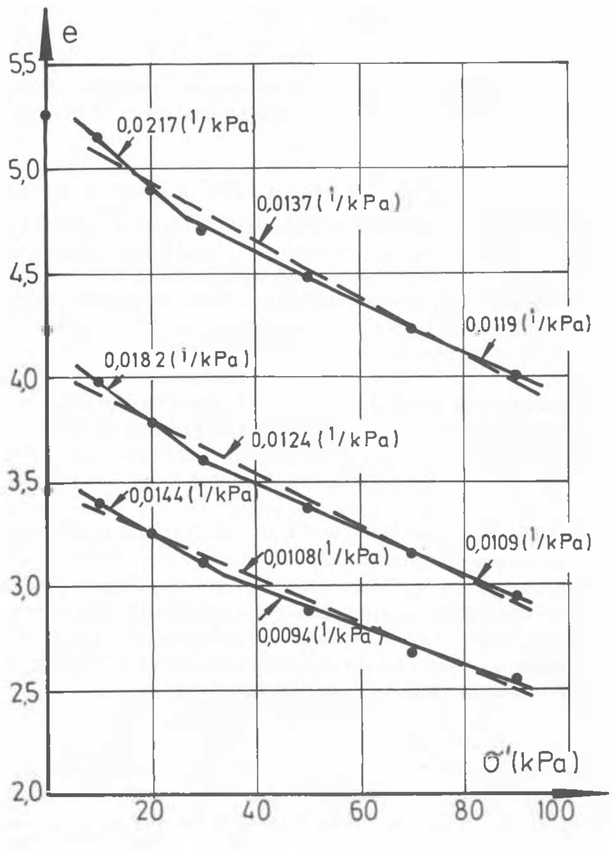

Fig. 3. Dependance e -σ'

Diagrams of dependance of void ratio on effective stresses (e - σ′) , obtained in oedometer tests , for samples from different depths , have shown considerable mutual differences , being logical consequence of layer heterogeneity . Three characteristic results are shown in Fig.3.

For calculation of the consolidation , nonlinear relationship e -σ′ is approximated with two straight segments , corresponding to the ranges of effective stresses 10-30 kPa and 30-90 kPa . Besides a linear relationship is applied , which corresponds to entire considered stress range 10-90 kPa , for the full load of phase 1. of the embankment . All relationships are obtained by statistical evaluation of corresponding data , with very high coefficents of linear correlation (over 0.90). It should be noted that the points below effective overburden pressure of natural soil are not taken into account .

It was stated that compressibility coefficients ($a = \partial e / \partial \sigma'$) for separate stress ranges vary with depth in similar manner as other properties of the compressible layer . This variation is also approximated with linear correlations , shown in Fig. 4. Somewhat lower correlation coefficients are obtained in this case , ranging nearly from 0.60 to 0.90 .

Permeability coefficients are tested in oedometer on 47 samples , for different effective stresses . Testing results for the same effective stress have shown considerable differences . Nevertheless , it was possible to establish that mean values of permeability coefficients indicate characteristic dependance on effective stress. This dependance is represented with piece - wise linear function (Fig.5).

At calculation of primary consolidation , influence of non-linear relationship e - σ′ is analysed , as well as influence of k- σ′ relationship , layer heterogeneity and decrease of layer thickness during consolidation. Consequently,following cases of calculation were performed:

(1) heterogeneous layer ; nonlinear (bilinear) e - σ′ relationship ; permeability coefficient according to Fig.5 ; thickness decrease of the compressible layer during consolidation .
(2) the same as (1), with linear relationship e -σ′ .
(3) the same as (1), with constant k, corresponding to to the average of experimental results .
(4) the same as (1) , for homogeneous layer with properties , corresponding to the middle of the heterogeneous layer .
(5) the same as (1) , with neglected thickness decrease during consolidation .

At all calculations, influence of own weight of compressible layer was neglected due to small effective stresses .

Initial undrained settlements are analysed for the plane - -strain case. Solution for finite elastic layer with strip uniform load is applied (Winterkorn,Fang,1975).Undrained moduli were determined on the basis of CU triaxial te-

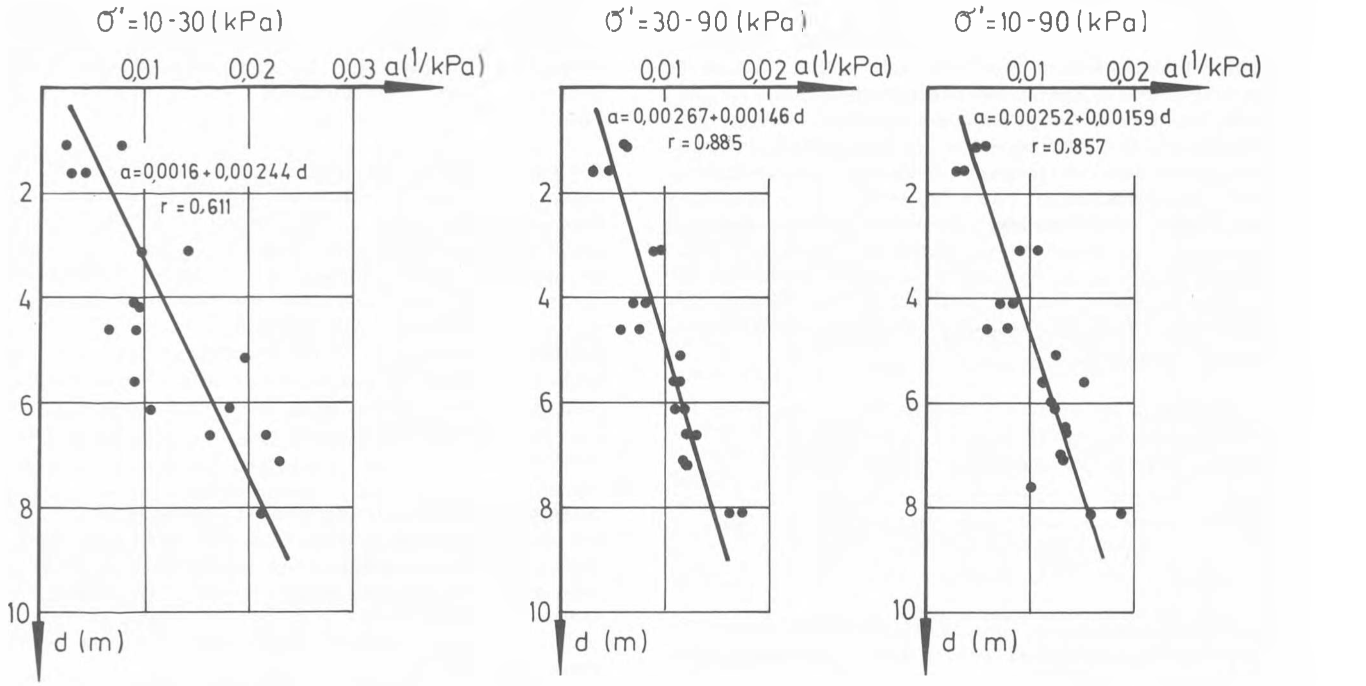

Fig.4. Variation of compressibility coefficients with depth

sts following nearly stress paths in embankment subsoil. Variations of undrained moduli for the considered range of streses were small and an average value E_u = 3700 kPa was accepted for the calculation .

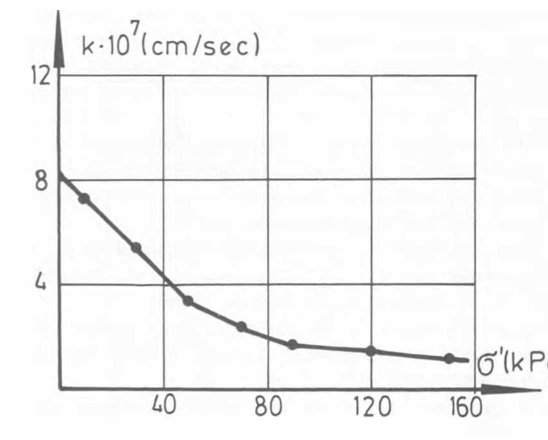

Fig.5. Dependance k -σ'

RESULTS OF CALCULATION AND FIELD MEASUREMENTS

Equation (1) is solved by explicit method of finite differences . A computer program is developed , which enables fast obtaining of solutions for any combination of the parameters.

Total height of the layer of 10.m. was divided to 20 equal parts, while time intervals amounted 0.30 days .

In order to compare calculation and measured values of settlements and pore pressures, calculations with design parameters were performed for real time - load sequence in considered construction period .

Results of comparative calculations are given in Fig. 6 and in Table I. Only consolidation settlements are given in Table I. , while in diagrams in Fig. 6 , initial settlements are added, amounting for the total load 9.8 cm .

Figures at separate lines in Fig.6.correspond to above mentioned cases of calculation, while measured settlements of the embankment are indicated by points. Settlements and maximum pore pressures for considered period are given in Table I. as well as final settlements . The values for separate cases are compared with general case (1)and corresponding ratio is given in brackets .

The highest differences in relation to the case (1)are obtained for the case (2),where linear relationship e -σ' for the range 10-90 kPa was introduced . Settlement differences for these two cases amount about 13% . Such differences are partly consequence of the fact that linearisation was performed for the range,corresponding to total load of the phase 1. of the embankment . However , maximum

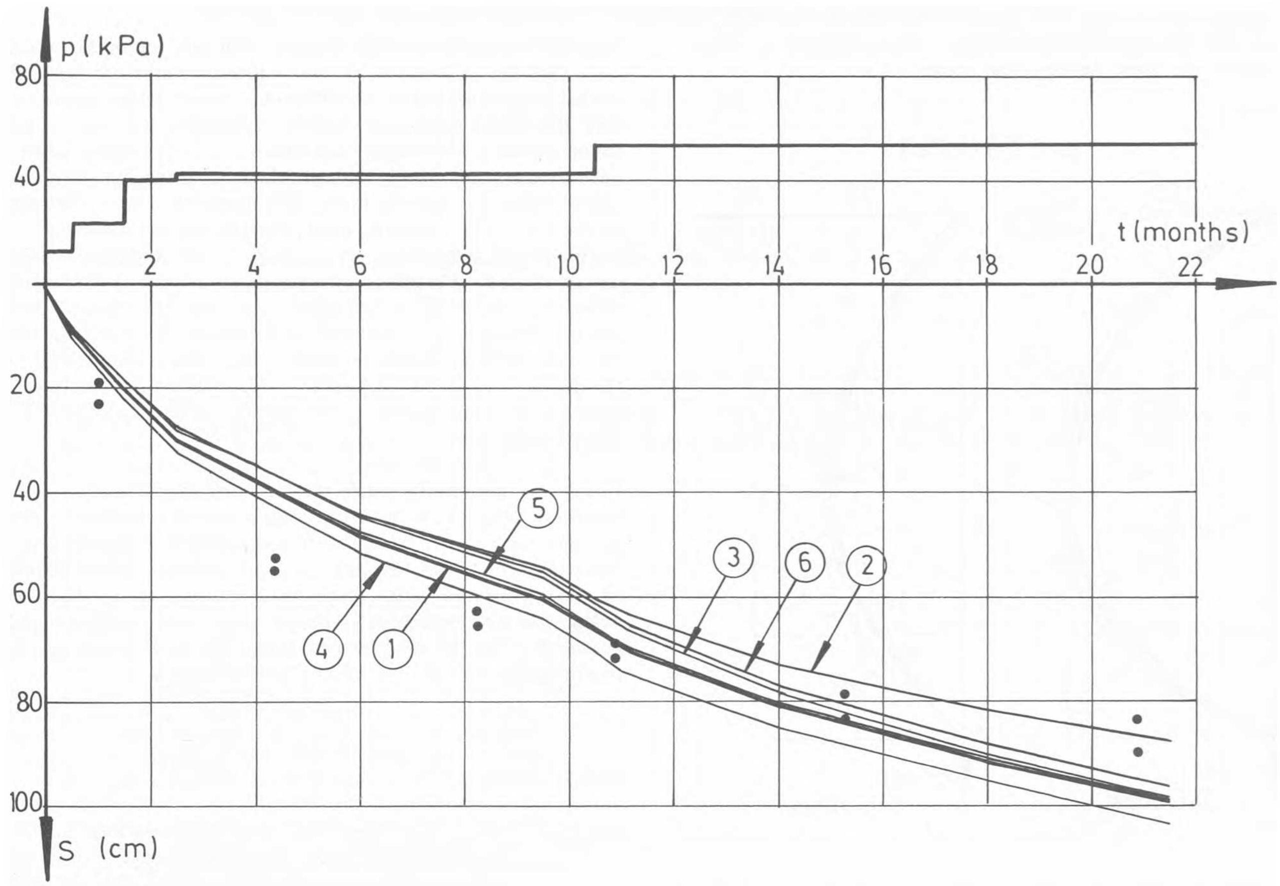

Fig.6. Time - settlement diagrams

effective stresses for considered period were 53.5 kPa and a calculation was also performed for the case of linearisation in range 10-50 kPa . In this manner ,settlement difference in relation to the case (1) has decreased to about 6% . This case is indicated in Table I. as (2′) and is not given in Fig. 6.

Case (3), (k = const.), exhibits certain consolidation lagging , comparing with the case (1). However , settlement differences for the end of considered period are negligible , while maximum pore pressures are somewhat higher , due to different shape of pore pressure distributions . (Fig.7).

For the case (4) , (homogeneous layer) , settlements about 4% higher than in the case (1) are obtained , while in the case (5) , with neglected thickness decrease , final settlements are 8% higher than in the case (1) . There are not differences in settlements for the cases (1) and (5) in considered period .

Considering relatively small differences for above mentioned five cases, the calculation with simplifications from all these cases is performed, i.e. for homogeneous layer with neglected thickness decrease during consolidation , constant permeability coefficient and linear

TABLE I
Results of comparative calculations

Case	t = 645 days		t = ∞
	s (cm)	u_{max} (kPa)	s (cm)
1	89.4 (1.00)	24.8 (1.00)	121.2 (1.00)
2	78.1 (0.87)	20.3 (0.82)	105.5 (0.87)
2′	82.1 (0.92)	21.9 (0.88)	114.4 (0.94)
3	89.0 (0.99)	26.5 (1.07)	121.2 (1.00)
4	94.1 (1.05)	25.0 (1.01)	126.4 (1.04)
5	89.0 (0.99)	27.6 (1.11)	130.3 (1.08)
6	86.8 (0.97)	26.1 (1.05)	127.9 (1.06)

relationship e - σ´ as in the case (2´) . This case (6) has given settlements 6% higher than in the case (1), while , for the considered period , consolidation in some measure lags behind the case (1)

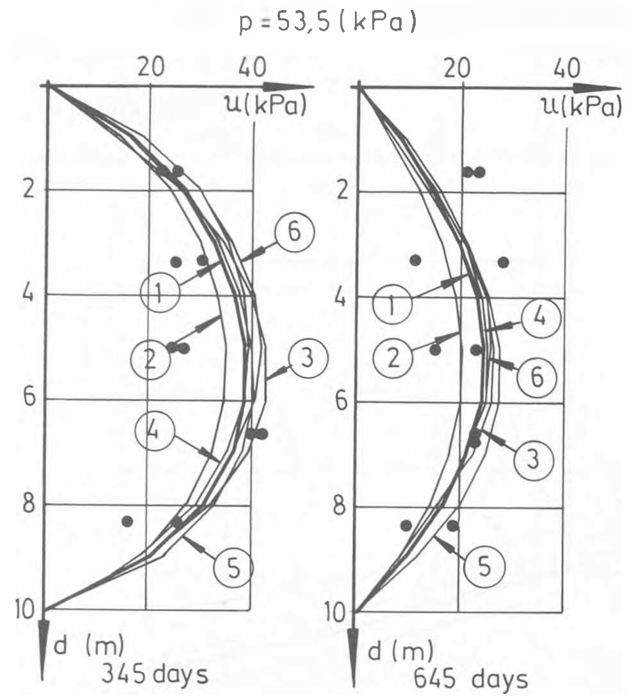

Fig. 7. Pore pressure distributions

Pore pressure distributions for two characteristic time - points are given in Fig. 7. All six calculation cases are presented and measured values are indicated by points . It is possible to state that the differences in pore pressure distributions for separate cases are relatively small .

It can generally pointed out that analysed influences have hot given mutual differences which would be of practical importance . Comparison of calculation and measurement results give higher differences than various calculation cases . Measured settlements in initial period are somewhat higher than calculated , while later they become equal or even smaller . It is probable that in initial period of consolidation real permeability coefficients were higher than determined ones on small samples in laboratory , with more expressed dependance on effective normal stress .

On the other hand , it is evident from Fig.7. that mutual differences of measured pore pressures are of the same order as differences between measured and calculated values . This is probably consequence of local irregularities in soil composition as well as of imperfection of measuring devices .

CONCLUSION

Calculation analyses have shown , for the considered case , that the influences of non - linear relationship e-σ´, variable permeability coefficient , layer heterogeneity and thickness decrease during consolidation are relatively small . Although variations of properties were rather considerable , the calculations have not given high differences in comparison with general case . Except in the case (2) , where linearisation of relationship e -σ´ is not performed in proper stress range , maximum obtained difference amounts 8% . Such differences have no practical importance for the considered problem . However , measured setlements in initial period of consolidation differ considerably from calculated settlements , what is probably consequence of the fact that real relationship k -σ´differs from laboratory determined one .

It can be generally said that rather good agreement between prediction and performance is obtained , even for the case with simplified calculation assumptions . However , it is neccessary to test rather large number of soil samples with statistical evaluation of the results ,as the individual values show considerable differences . The highest uncertainty is in laboratory determination of permeability coefficients .

REFERENCES

Amaryan L.S. , Sorokina G.V. and Ostroumova L.V. Consolidation laws and mechanical - structural properties of peaty solis. Proc. 8th ICSMFE, (2.2) , 1-6 , Moscow .

Sarač Dž., Popović M. (1980) . Stability conditions of an embankment on very soft organic soil . Proc. 6th Danube - Europ. Conf. on Soil Mech. and Found. Eng.,Varna (in preparation).

Šuklje L. (1969). Rheological aspects of soil mechanics. 1st edition , 571 pp. Wiley , London.

Winterkorn H.F. and Fang H.Y. (1975) . Foundation engineering handbook . 1st edition , 751 pp.Van Nostrand Reinhold , New York .