INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

SIMSG ISSMGE

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Successful Highway Construction on Very Soft Soils

Un Ouvrage Routier Efficace sur Sols Compressibles

S. VIDMAR Professor of Soil Mechanics, University in Ljubljana, Yugoslavia
A. GABERC Assistant of Soil Mechanics, University in Ljubljana, Yugoslavia
E. KARDELJ University in Ljubljana, Yugoslavia

SYNOPSIS The highway Ljubljana - Postojna crosses the northern boundary part of the Ljubljana Marsh. Due to high compressibility of the upper peaty, silty and clayey layers of the subsoil with total thickness of about 12 m settlements up to 160 cm had to be foreseen. In order to reduce the post-construction settlements to tolerable values, vertical wick drains, preloading, light fill material and partially the excavation of superficial peat were used. The applicability of simplified consolidation and stability analyses was checked by observations on a test embankment, and the efficacy of polypropilene covers of wick drains by laboratory and field tests. Settlements observed during the construction of the highway and two years afterwards agree satisfactorily with forecasted values.

INTRODUCTION

The tectonic depression of the Ljubljana Marsh is filled up with pleistocene fluvial and marsh sediments and with holocene lake deposits. The new highway crosses in total length of about 4000 m the boundary part of the Marsh where the thickness of very soft upper layers is moderate varying between 4 and 20 m; they consist of peat and organic soil, of a very soft holocene clayey silt, called "snail clay" owing to the content of snail shells, and the upper soft part of pleistocene clayey sediments. In order to keep the embankment elevation on the designed level of 80 to 180 cm, settlements of up to 160 cm had to be foreseen. Taking into account also the secondary time effects the elevations have been required to overpass the designed values by up to 25 cm at the end of construction in order to coincide with the designed values twenty years afterwards. Owing to this requirement, to different embankment heights and to different thickness of compressible layers different modes of construction had to be applied and combined, namely by using for embankments either heavy gravely or crushed stone material, or a sandwich fill composed of 30 cm thick layers of light fly ash and 10 cm thick layers of sand; by excavating or not the peat and organic soils; by installing or not the vertical wick drains in a net of equilateral triangles with sides a and the preloading. The modes of construction used in single sections and a typical cross-section of the highway are shown in Fig. 1.

SOUNDING DATA

In the line of the highway the boreholes were drilled 100 m apart in average; most of them reached the bedrock of Triassic dolomite or Carboniferous shale, but no one was deeper than 40 m. The obtained stratigraphic profile is shown in Fig. 1. The thickness of peat and organic soil varies between o.8o and 2 m (except the section from km 11.700 to 12.900 where the ground cover consists of a clayey silt of intermediate or stiff-plastic consistency), the thickness of the snail clay is between 0.80 and 7 m, and the underlying soft clay of liquid or weak-plastic consistency is up to 12 m thick; the deeper layers consist of clays and silts of intermediate to stiff-plastic consistency, with intercalations of uniform fine sands.

Along the line of the highway the groundwater level has been found in depths between 0.5o and 3 m.

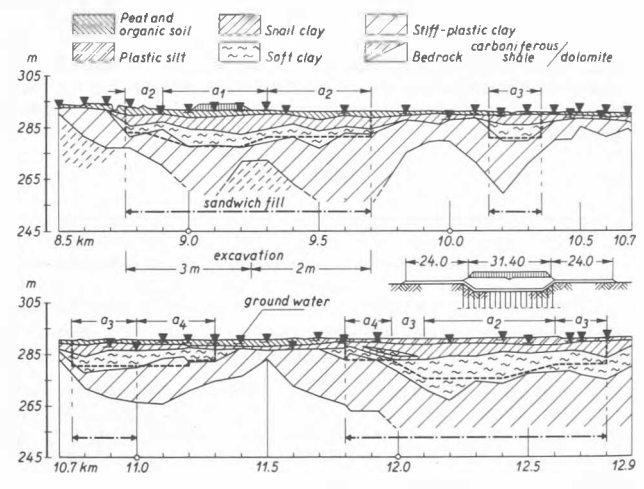


Fig. 1 Longitudinal Profile along the Highway Axis

a_1 = 1.25 m a_2 = 1.50 m
a_3 = 1.75 m a_4 = 2.00 m

TABLE I

Layer	AC - Classification	Grain Size <0.002	Grain Size 0.002/0.06	Grain Size 0.06/0.2	Liquid Limit w_L %	Plastic Limit w_P	Water Content w %	Unit Weight γ kN/m^3
I	Pt						40 - 200 [6]	10 - 15
II	MH	10	85	5	60 [1]	35 [1]	80 - 125 [2] 40 - 100 [3]	13 - 17
III	CI - CH - MH	20	75	5	40 - 65	22 - 30	40 - 80 [4] 30 - 50 [5]	16 - 19
IV	MH - CH - MI - - CI - CL				30 - 75	18 - 30	25 - 50	17 - 21

[1] average values
[2] over the bedrock synclinals
[3] between the synclinal sections
[4] in the upper part of the layer
[5] in the lower part of the layer
[6] depending on the groundwater level

SOIL PROPERTIES

The geotechnical properties of soils were investigated by laboratory tests on undisturbed samples taken in boreholes, and by field vane tests.

Water content, grain size, consistency and unit weight

The data are given for four typical layers (layer I = peat and organic soil, layer II = snail clay, layer III = the upper soft clay and layer IV = the lower stiffer silty and clayey deposits) in Table I.

In series II and III the water content does not decrease appreciable with depth.

Deformability and permeability

The tests were performed mostly in oedometers on samples of 7 cm in diameter and 2 cm in height.

For the peat and organic soil (layer I) the following representative effective pressure versus void ratio relationship has been deduced from a good few of tests:

TABLE II

σ' kPa	0	10	20	50	100
e	4.08	3.96	3.75	3.34	2.95

For the expected stress increase in the depth of the layer the coefficient of volume change m_v between 0.0025 and 0.0033 (kPa)$^{-1}$ has been obtained from the above relationship. The corresponding average value of the coefficient of permeability to be considered in the consolidation analysis, has been found as $k = 10^{-9}$ m/s.

For the snail clay (layer II) the oedometer test results were grouped for single sections, and for each group the representative effective stress versus void ratio curve and coefficient of permeability versus void ratio plot was constructed. The resulting diagrams have been presented in Figs. 2 and 3.

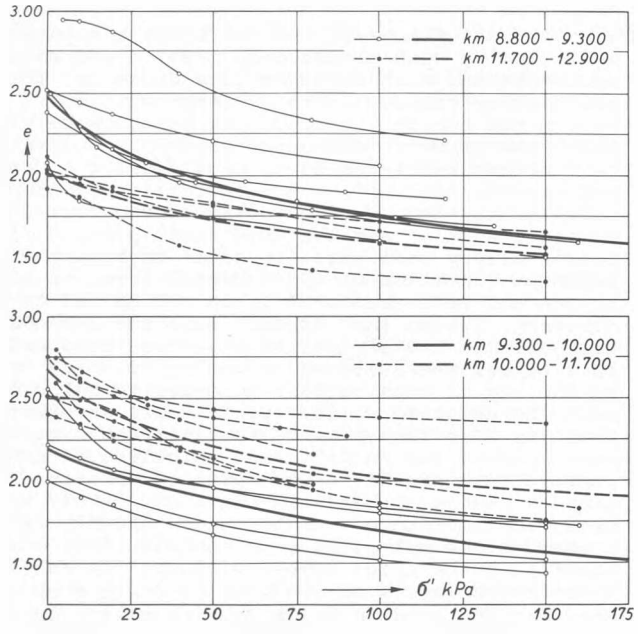


Fig. 2 Oedometer Curves for the Snail Clay

In order to determine the moduli of shear deformation corresponding to the negligible volume change at the very beginning of the consolidation process, undrained triaxial tests were performed on two undisturbed samples of the snail clay, 10 cm in diameter and 20 cm in height. The samples were taken in km 9.100 in the depths of 2 and 3 m respectively; their natural water content was 83 and 86 % respectively. After consolidation at their effective overburden pressures, the coefficient of the earth pressure at rest having been assumed to be K_o = 0.8, the undrained sam-

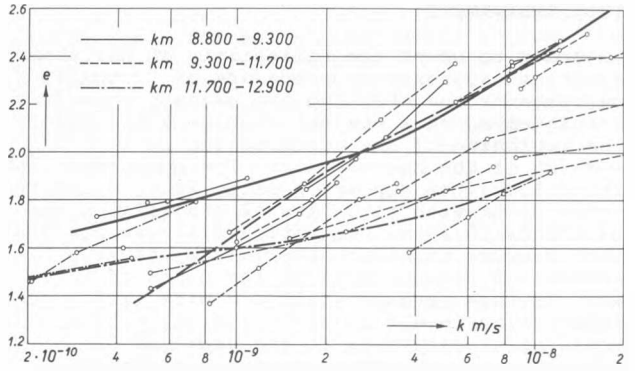


Fig. 3 Coefficient of Permeability - Void Ratio Plots for the Snail Clay

ples were subjected to successive total load increase by intervals $\Delta\sigma_1$ = 20 and $\Delta\sigma_3$ = 12.5 kPa simulating the expected stress increase in the depth of 2.5 m under the embankment load increment of 20 kPa. The resulting shear moduli G are presented in Table III. For stress states as obtained by elastic analyses, in the middle of snail clay layer in the central part of the highway cross-section, values G close to the moduli, appearing in Table III for similar stress ratios, can be obtained from oedometer test data by using the well known relation

$$G = \frac{(1 - 2\nu)}{2(1 - \nu)} E_{oed} \qquad (1)$$

with the Poisson's ratios between 0.25 and 0.30.

TABLE III

Moduli of shear deformation G kPa

σ_1/σ_3	25/20	45/32.5	65/45	85/57.5	105/70	125/82.5	145/95
Sample 1	drained	1950	810	380	200	110	50
Sample 2	drained	1030	730	510	310	150	60

In the domain of stress intervals in the depths along the layer III, the oedometric coefficient of volume change varies between 0.00125 $(kPa)^{-1}$ and 0.00060 $(kPa)^{-1}$, and the corresponding coefficient of permeability between 5 and 1.10^{-10} m/s. The test results were not uniform enough to allow the construction of representative charts for single sections. The charts obtained on samples from the nearest borehole were considered for the prediction of settlements along the highway line.

The layer IV is less compressible, the average coefficient of the volume change being about 0.00033 $(kPa)^{-1}$. On the whole it contributes to total settlements by about 10 to 20 cm, this is some ten per cent only.

Shearing resistance

The results of the undrained shearing resistance as obtained by field vane tests are presented in Fig. 4. Owing to the great dispersion of test results, the values at the lower boundary of the corresponding domains were considered as allowable shear stresses, i.e. by taking, in stability analyses, the safety factor F = 1.

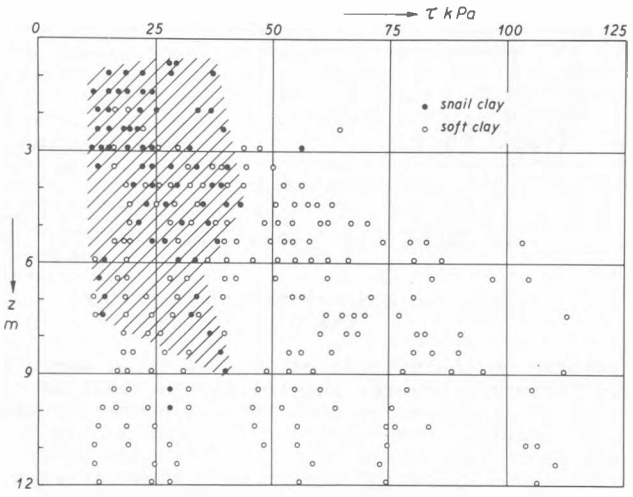


Fig. 4 Vane Test Results

CONTROL EXPERIMENTS UPON WICK DRAINS

In order to force the settlements to develop mainly during the construction of the highway, preloading was used and vertical wick drains of Kjellman's type have been driven into the soil in all sections where the forecasted settlements had overpassed 60 cm. In drains the plastic flat ribbed wick is protected by filtering covers made of a syntetic substance called polypropilene. The permeability and durability of filters were checked by laboratory tests of long duration. Cylindrical soil samples of 10 cm in diameter and 20 cm in height were surrounded with a ring of wick drains, covered by a rubber membrane. In the axis of the sample a sand drain of 1.7 cm in diameter and 12 cm in height was installed and connected with a water container. The flow of water occurred under a constant pressure of 55 kPa which corresponds to the average pore pressure appearing in the soil in the early consolidation phase. From the sandy core, water seeped through the sample in radial direction towards the wick drains and was collected in a calibrated vessel. In a triaxial cell the sample was subjected to the all around pressure of 60 kPa.

Two kinds of polypropilene covers were tested: the non-woven polypropilene cover (sample No 2) and the woven polypropilene cover (sample No 3), both on snail clay samples with the initial water content of 93 % and 81 % respectively. In order to compare test results with the efficacy of a known wick drain system which had been successfuly used in numerous sites, sample No 1 was equipped with the Terrafigo drains protected by cardboard covers; the sample consists of the same snail clay as the sample No 2 (water content 93%). Terrafigo drains were installed also on the periphery of the sample No 4 consisting of the soft silty clay from layer III (water content 45 %). Test results are presented in Fig. 5 by the rate of discharge q versus time plots whereby q is

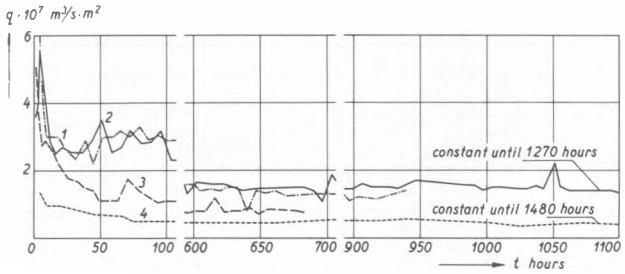


Fig. 5 Laboratory Test Results Upon the Wick Drains

related to the unit (1 m²) of the inner surface of filtering covers. The initial decrease of the rate of discharge up to about ten hours for snail clay samples, something more for the sample No 4, has to be attributed mainly to the pore water squeezed out, during the primary consolidation, with decreasing velocity, and partly to the infiltration of small particles into the filter which governs also the subsequent discharge decrease by diminishing the permeability of the filter wrapper. After about 500 hours this effect too despired in all four tests, further seepage occurring with constant speed.

Since the efficacy of the non-woven polypropilene covers had been found close to the efficacy of the Terrafigo cardboard wrapper, this filter was used for all vertical drains along the highway.

At the end of the tests the state of filters was checked. The polypropilene covers could easily be separated from the soil and were proved to be durable. The cardboard wrappers were found to be sticked to the soil on about 85 % of the total contact surface; the ribs of the plastic core penetrated somewhat into the soil, leaving enough free profile for the efficacious flowing of the pore water.

PROCEDURE OF FORECASTING SETTLEMENTS

Consolidation settlements have been calculated with the assumption that the strain state is linear. Oedometric moduli were used as stress and (owing to secondary effects) time dependent values, and the dependence of the permeability on the porosity was taken into consideration. Instantaneous settlements have been computed as pure deviatoric settlements appearing at the very beginning of the consolidation process (zero volume strains, Poisson's ratio $\nu = 0.5$); shear moduli governing these settlements were substituted by the drained values as obtained from oedometer tests according to equation (1) by assuming the Poisson's ratio $\nu = 0.4$. The details of the procedure have been presented in a previous paper (Gaberc & Vidic, 1977).

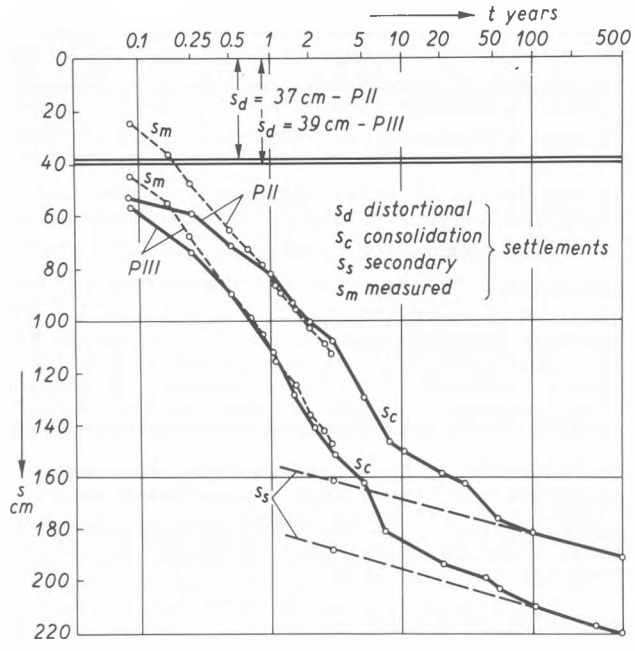
In the drained sections of the highway a preloading of 15 kPa was applied for a period of at least one year. In undrained sections the preloading was somewhat lower. The development of settlements had been predicted for stress states during preloading as well as afterwards, and the expected consolidation line was constructed by joining both curves starting at the time of removal of preloading.

TEST EMBANKMENT

In order to check the reliability of the above described approximate procedure of forecasting settlements and checking the bearing capacity, a test embankment was built three years before the beginning of the construction in km 9.040 - - 9.200 of the highway line. The embankment was 160 m long, the top width being 26.40 m and the total width with both lateral stabilizing embankments 65.40 m. The entire fill was divided into four sections in intervals by 40 m: P I with removal of organic soil to the depth of 2.1 m, with fly ash sandwich fill and with 13.5 m deep sand drains in distances of 2.31 m; P II with removal of organic soil to the depth of 3.1 m, with fly ash sandwich fill and with sand drains in distances of 4.62 m; P III with the common heavy fill placed on the organic soil, and with sand drains in distances of 4.62 m; and P IV as under P III, but without drains.

Pore pressures, settlements, horizontal deformations and total pressures have been measured. The results of two years observations have been presented in a previous paper (Gaberc & Vidic, 1977). In Fig. 6 the settlement versus time plots for two central points in the test sections P II and P III are reproduced and supplemented with further measurements up to November 1977. In the same figure the forecasted settlement versus time plots are shown as obtained by the previously described procedure; the primary consolidation analysis was performed in several time intervals in order to take into account, in each interval, the coefficient of permeability $k = k(e)$ corresponding to the void ratio obtained in the previous time interval.

The initial deviatoric settlements have been calculated by using moduli of shear deformation obtained from equation (1) with an average Pois-



Fif. 6 Settlement Versus Time Plots for the Test Embankment

son's ratio value $\nu = 0.3$ for all layers, neglecting the different stress levels in each layer. The results of the described undrained triaxial tests on snail clay show that more accurate values of deviatoric settlements can be obtained by choosing Poisson's ratio ν less than 0.3 in the domain of stress levels at the surface of the underground, but higher than 0.3 in greater deepths (layers III and IV). For that reason, the deviatoric settlements along the highway line have been calculated by using an average value of Poisson's ratio $\nu = 0.4$ for all layers.

The satisfactory agreement between the computed and measured settlements has justified the application of the approximate computation procedure. The pore pressure measurements have supported such a justification although in a less convincible way. The suitability of the applied computation has been corroborated also by the simultaneous comparison of measured settlements with the results of more rigorous analyses of the consolidation process either for sections without (Gaberc & Kovačič, 1975) or with vertical drains (Šuklje & Kovačič, 1979).

The experience collected by observations on the test embankment required a denser system of vertical drains than they were built in under the test embankment, as well as the combination of the effect of drains with an overload during a period of at least one year. Since the horizontal displacements, measured along vertical boreholes driven under the lateral embankments, continued to increase with more or less constant speed for a long time after the end of filling, a rather conservative stability control has been carried out.

COMPARISON OF MEASURED AND COMPUTED SETTLEMENTS

For the drained cross-section in km 9.500 with 2 m deep excavation the combined forecasted consolidation plot has been compared, in Fig. 7, with the measured settlements that overpass the predicted values by about 17 cm, i.e. by 12 %. In order to point out the effect of drainage, the consolidation lines predicted for undrained underground are shown in the same figure.

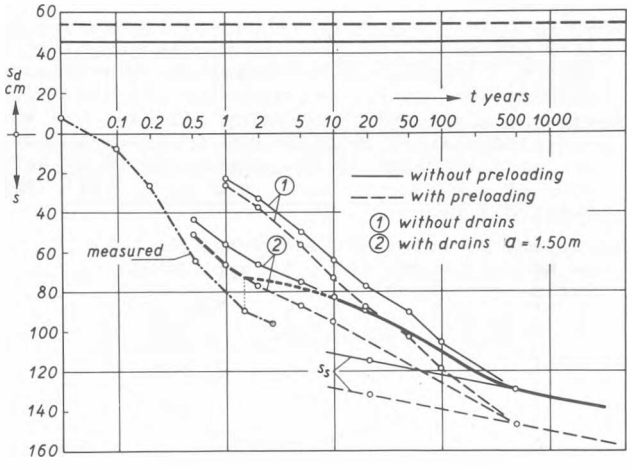


Fig. 7 Consolidation Plots in km 9.500

The longitudinal profiles of settlements for entire line between km 8.500 and 12.800 are presented in Fig. 8. In the lower part of the figure, the thick full line represents the settlements in the axis of the highway, measured at the time of the end of construction. The dotted line connects computed settlements corresponding to the time t = 22 years, i.e. 20 years after the end of construction, while the forecasted settlements immediately after removal of the preloading have been indicated, in a few cross-sections, by small circles. In the upper part of the figure, Δs line indicates the post-construction settlements observed in May 1980, i.e. about nine months after the end of construction. The overlying profile shows total pressure onto the ground surface, including the preloading.

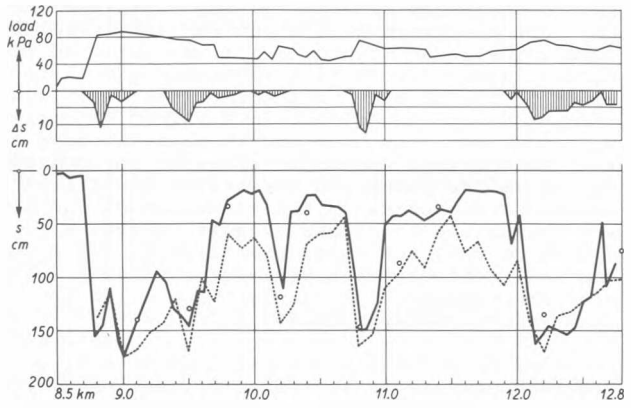


Fig. 8 Longitudinal Profile of Settlements in the Highway Axis

FIELD CONTROL OF THE EFFICACY OF WICK DRAINS

In km 10.200, the pore water squeezed out through two wick drains, 12 m distant from the highway axis, has been separately collected and measured during a period of about one year starting with the beginning of filling. Simultaneously, settlements s were observed in the same points. The s versus t (time) plots have been opposed to the corresponding H versus t plots in Fig. 9, H being

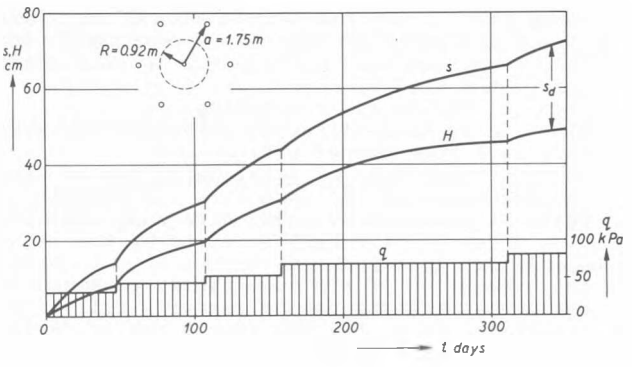


Fig. 9 Field Tests of the Efficacy of Drains

the height of the cylinder whose radius equals the effective drain radius R, and whose volume is equal to the quantity of the pore water squeezed out till time t. The distance between the corresponding plots increases with time up to the value of 22 cm. It may be attributed to additional deviatoric settlements increasing with increasing embankment load q; the forecasted initial (constant volume) settlements had been evaluated as high as 29 cm whereby the load of the basic embankment layer filled before the beginning of measurements had not been taken into account.

BEARING CAPACITY

The bearing capacity was checked by the common stability analysis applying circular potential failure surfaces and the $\phi' = 0$ method. Cohesion varying with depth was taken into account according to vane test data in the way explained in the last paragraph of the section "Soil Properties". The safety factor F > 1 was obtained providing lateral protecting embankments on both sides of the highway embankment (see Fig. 1) with varying width and height. This rather expensive safety measure has been proved satisfactory; no sliding occurred due to overpassing the bearing capacity. The width and the height of lateral embankments depends on the elevation of the highway and on the thickness of soft layers; e.g. in a section with the final elevation of 3.2 m above the soil surface and a preloading of 15 kPa, lateral embankments 21 m in width and 1.1 m in height have been required.

In order to check the need of such extensive lateral embankments in further construction of highways in the Marsh domain, two test embankments have been recently deposited in km 9.100 outside the highway, one on drained and second on undrained soil. Owing to the small thickness of the snail clay and to stiff lenses in the underlying soft clay, the location of these test embankments is not representative for the soil composition along the highway line. Nevertheless, the comparison of the observed failure loads with the results of the stability analyses used for the highway design has given useful information concerning favourable drainage effects on the bearing capacity of soft soils.

CONCLUSIONS

- Large and non-uniform settlements of very soft clayey and silty deposits as appearing in the Ljubljana Marsh can efficiently be accelerated during the highway construction by applying vertical drains and preloading.
- Wick drains with filtering polypropilene wrappers have been proved efficacious.
- The non-uniformity of settlements due to varying thickness of soft deposits can successfully be overcome by using different construction modes in different sections of the highway line.
- The excavation of the superficial peat and organic soil and the use of sandwich fill with prevailing light fly ash layers are suitable means for reducing the total amount of settlements
- The first fill layer is recommended to be put on a continuous carpet of porous polypropilene or similar plastic material. In sections where peat or organic soil need not to be excavated, this carpet is preferably to be put directly onto the grass cover.
- The settlements and consolidation process may reliably enough be predicted by simplified computation procedures provided that the forecasted values were favourably checked by field measurements made on an appropriate test embankment. Furthermore soil parameters evaluated from simple tests are recommended to be verified by few perfect tests (e.g. shear moduli by undrained triaxial tests) using advanced procedures, and the results of simplified computations to be compared with the results of more rigorous analyses.
- The bearing capacity control taking into account initial values of the vane test results and neglecting the increasing shearing resistance with increasing degree of consolidation seems to be conservative. Special bearing capacity field tests on drained soil would be useful.

ACKNOWLEDGEMENTS

Sounding, field tests, measurements of settlements and some of laboratory tests described and used in this paper were performed by the Geological Institute Ljubljana under the supervision of Mr. F. Vidic, and field tests of wick drains efficacy made by the Hydrotechnical Laboratory in Ljubljana. Polypropilene covers of wick drains were suggested by Mr. J. Žmavc and Mr. E. Pavlin, Road Community of Slovenia. The authors wish to express their gratitude to prof. L. Šuklje for his helpful suggestions in preparing the paper.

REFERENCES

Gaberc, A & Kovačič, I (1975).
Konsolidacija slojevitega polprostora (Consolidation of the layered half-space, in Slovenian). XIII savjetovanje i skupština JDMTF, 161 - 167, Budva.

Gaberc, A & Vidic, F (1977).
Test embankment on soft soils. Proc. 5th Danube European Conference, (II), 55 - 66, Bratislava.

Šuklje, L & Kovačič, I (1979).
Consolidation of drained multilayer viscous soils. Int. Conf. on Evaluation and prediction of subsidence, ASCE, New York, 373 - 388, Pensacola Beach.

Verdeyen, J, Roisin, V & Nuyens, J (1975).
La Mécanique des Sols, 490 pp. Dunod, Paris.