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# Runway on "Geodrain"-ed Soft Clay in Turku, Finland

## La Piste de l'Aeroport de Turku sur l'Argile Molle Traitée par "Geodrains"

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SYNOPSIS The existing runway in Turku could be lengthened only towards up to 15 m deep soft clay deposits. A study of alternative foundation methods showed vertical drainage combined with stepwise loading to give the most economical one of technically practicable solutions. The basic design parameters were $c_h/c_v$ = 2.5, $c_v$ = 0.3 $m^2/a$, and the time available for achieving 100 % of primary consolidation was two years. In spite of the severe climatic conditions the installation of the 350 000 m of GEODRAINS was performed successfully. The specially designed drain stitcher could prepare a starting hole through frozen top layers and install simultaneously the prefabricated drain. The consolidation of the soft clay stratum proceeded as expected, although some indicators of the progress of consolidation did not support the conclusions drawn from settlement observations. The authors want to stress, that the level of uncertainty in connection with the interpretation of the consolidation rate is well acceptable in the light of the relatively scarce efforts involved in investigations and observations.

## INTRODUCTION

The existing runway in east-west direction at the Turku airport (SW-Finland) restricted the air traffic especially during the winter season because of its length of only 2 000 meters. The oldest part of this runway, the central part of a length of 1 500 m, has been founded in the 50s on a sand deposit, which unfortunately reached at both ends to up to 15 m deep clay basins. The first troubles at both soft ends had to be met in the early 60s, when the runway was lengthened to 2 000 m, using simple preloading technique. At these parts settlements and successive repairs had still to be faced, when, as a consequence of the technical development and increased safety requirements, either a new lengthening of the existing runway to a length of 2 500 m or alternatively a completely new 2 660 m long runway in cross direction was demanded. In cross direction no foundation problems would have arised, but the runway would have intersected the protective area of a ground-water pumping station. Due to the appr. three times higher construction costs required for a completely new runway it was decided to win additional 560 m of the west-end clay basin for aviation purposes.

## CHOICE OF FOUNDATION METHOD

The subsoil in the lengthening area consists of a clay stratum with the thickness varying from 4 to 14 m, the shear strength of the clay being $s_v$ = 8...20 kPa, partly covered with a sand layer up to 3 m thick or/and a thin top layer of peat. Thorough subsoil investigations have been performed. They consisted of weight soundings, field vane tests, soil sampling at two significant points and laboratory testing.

The demanded surface level of the runway lengthening (see Fig. 1) corresponds to a surcharge of up to 100 $kN/m^2$, which without subsoil improvement would have caused severe differential settlements over a period of many decades. The total settlements were estimated to be in a range of up to 1.5 m...2 m. Both the expected differential and the long term settlements were not acceptable for a safe runway.

After considering all foundation methods technically practicable, the decision was made in favour for vertical drainage using prefabricated draines of the type GEODRAIN.

Undoubtedly piling combined with a concrete slab would have been the securest, but also the most expensive foundation alternative, creating costs appr. six times those of the chosen solution.

Total replacement of the clay by cohesionless material had to be kept out of considerations. It would have been impossible to depose the waste masses (0.4 millions $m^3$) in the vicinity within an economical transport distance.

Simple preloading would not have had the desired efficiency within the construction period available mainly due to the fill height limitations set by air traffic regulations.

Embankment piling combined with single pile caps was abandoned for many reasons, as great variations in piling depth or fill height variations, which would have led to a both uneconomic and technically uncertain design. Especially at the top end of the runway lengthening a combination of short piles at great distance with small fill height, which consequently required high coverage by pile caps, would not have met the requirements set to the foundation.

The low bearing capacity and the depth variations of the clay stratum combined with fill height

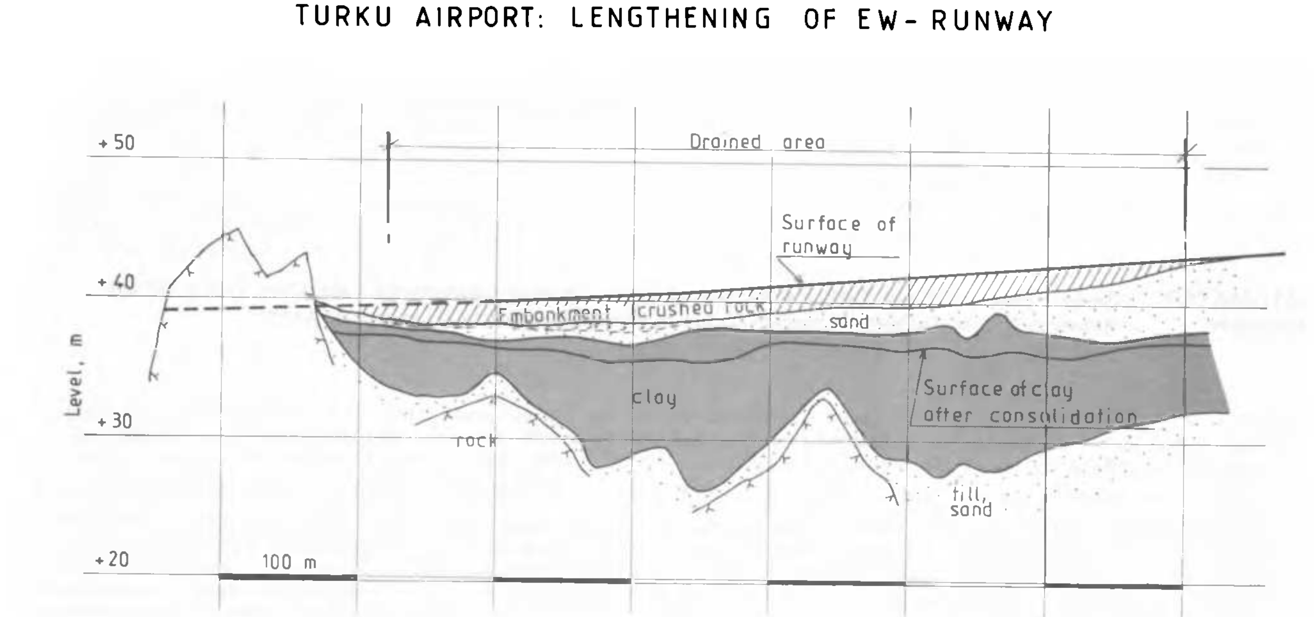


Fig. 1 Length section at the centre line of the runway lengthening.

variations led to the conclusion, that besides relatively big settlements also plastic horizontal deformations had to be expected. In the case of subsoil improvement by vertical drainage the suitable type of drain had to be flexible and guaranteed continuous, even if horizontal movements should occur, and above all secure, because a runway was considered to be unsuitable for experimenting.

Large diameter sand drains would have fulfilled these requirements, supposing they could have been installed with a non-displacement method. Displaced sand drains of e.g. 400 mm diameter were expected to create severe disturbance of the subsoil, jetting seemed inpracticable because of the winter conditions and replacement type drains would have created not desirable waste.

Prefabricated drains of the type GEODRAIN promised to be both secure and efficient, and since the early 70s they have gained sufficient reputation at installations all over the world.

## SUBSOIL

The decision to improve the soft clay stratum by means of vertical drainage made additional subsoil investigations necessary. The variations in the thickness of the clay layer and the depth of the firm bottom were thoroughly determined by weight and manual soundings in a dense network. The first stage soil explorations consisted in addition to field vane tests and weight soundings of soil sampling at two representative points and corresponding laboratory investigations in respect to consolidation parameters. Additional soil sampling and laboratory tests were performed in order to improve the reliability of the soil parameters gained earlier. No attempt was made to increase the number of thoroughly investigated test points. All consequent decisions were based on the design parameters evaluated from these two test points, which had to represent a total area of 40 000 $m^2$. At a late stage of design the homogeneity of the clay stratum and the in situ consolidation coefficient $c_v$ were checked by means of piezometer soundings. They confirmed the validity of the design parameters obtained from the earlier subsoil investigations. Typical sounding diagrams are presented in Fig. 2. A rough idea of the subsoil conditions might be obtained from the average soil parameters listed in Table I.

TABLE I

Soil characteristics

| | |
|---|---|
| Modulus number | $m_1 = 6.0$, $m_2 = 66$ |
| Stress exponent | $\beta_1 = -0.7$, $\beta_2 = 0$ |
| Consolidation coefficient | $c_v = 0.30\ m^2/a$<br>$c_h = 0.75\ m^2/a$ |
| Modulus of confined compression | M = 0.6...1.3 MPa |
| Shear strength (field vane test) | $s_v = 8...20$ kPa |
| Penetration resistance (weight sounding) | 100...150 kN. |

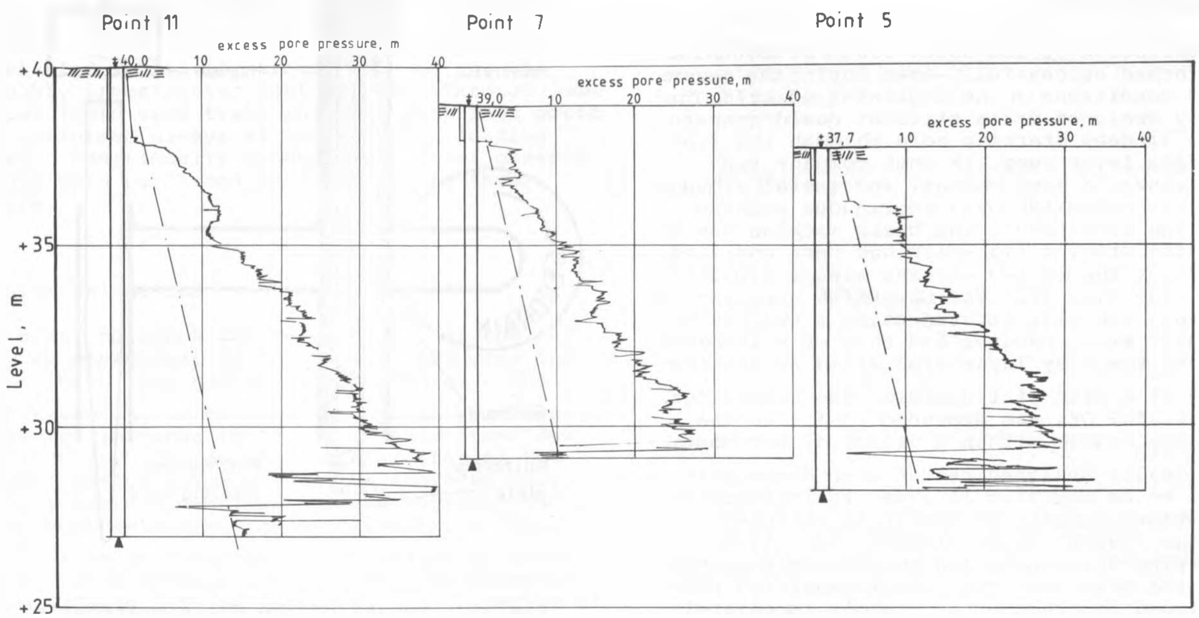


Fig. 2 Typical pore pressure sounding diagrams.

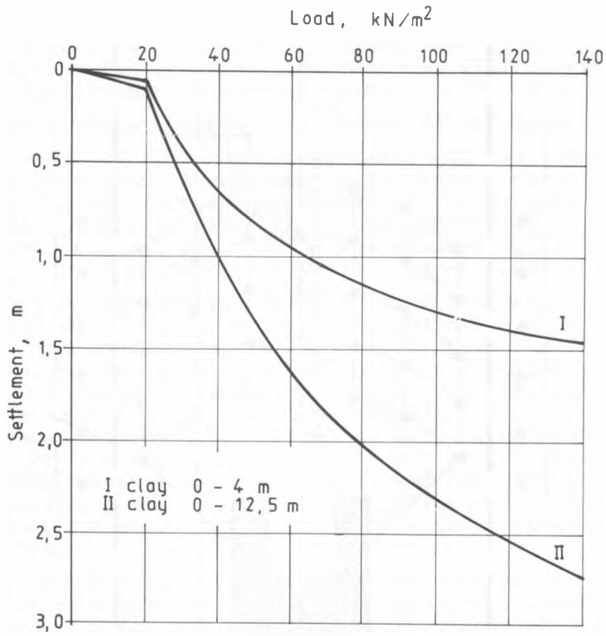


Fig. 3 Predicted total settlements vs. embankment load as function of clay thickness.

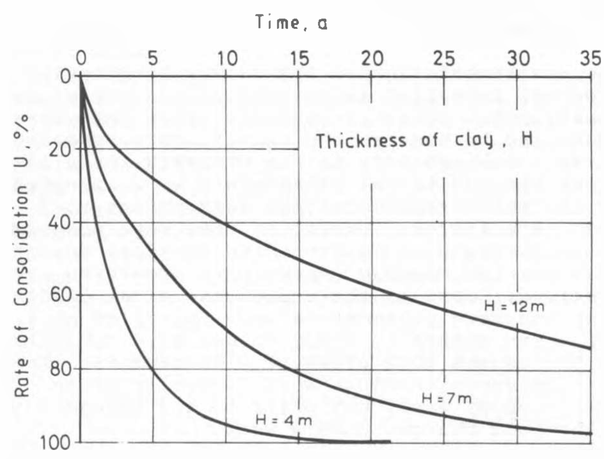


Fig. 4. Consolidation rate as function of clay thickness.

The correlation between total settlements and surcharge (embankment load) as function of the clay layer thickness is given in Fig. 3. The corresponding time vs. degree of consolidation graph is presented in Fig. 4.

Both figures show clearly the necessity of measures to accelerate the consolidation process of the soft clay deposit.

As a consequence of international runway design rules, which had to be observed with respect to the maximum gradient, the centre line of the runway had to be fixed to a level of appr. 1.5...2.9 m above the existing soil surface. Therefore it was impossible to keep the surcharge smaller in order to reduce the total settlements.

DRAIN INSTALLATION

The installation of the about 350 km of GEODRAINS was performed successfully even during the severe climatic conditions in the beginning of 1978. The specially designed drain stitcher could prepare an up to 2 m deep starting hole through the frozen surface layer resp. through compact top layers (runway's base course) and install simultaneously the GEODRAINS by a continuous pushing process (no vibrating). The basic machine was a 240 kN pile driving rig, which had been modified to carry out the a.m. two working stages simultaneously (see Fig. 5). The GEODRAINS were inserted into the clay stratum using a 60 x 120 $mm^2$ rectangular steel mandrel and they were anchored underneath the clay layer with aid of an anchorage plate of a size of 120x150 $mm^2$. The contractor, POHJAVAHVISTUS OY, was demanded to finish the drain installations within a period of six month.

The originally designed rectangular drain pattern had to be modified in order to improve the installation capacity of the drain stitcher. E.g. in the case of drain distance c/c = 1.0 m seven drains were installed successively only by turning the mast (see Fig. 6). This modified pattern allowed to prepare the prehole immediately before the inserting of the GEODRAIN without interruption of the ongoing drain installation. Only close to the existing runway, where the old fill thickness reached 6...8 m (an area of about 5000 $m^2$), the top 2 m of the base course had to be removed to make the drain installation possible. At a narrow zone preholes up to 4...6 m deep had to be prepared by an extra piling rig with a dummy pile.

INSTRUMENTATION

The instrumentation in the runway lengthening area was installed in several stages. Prior to construction three piezometers (type BAT) were installed to observe the in situ pore pressure heads. Subsequently to the GEODRAIN installations ten additional piezometers were inserted at the three representative test points to 3 resp. 4 different levels. As they were placed at equal distance to 4 surrounding vertical drains, in fact, the maximum excess pore pressures were observed. Close to the end of the final loading step ten more piezometers were installed at 4 different points in order to get more reliability on the excess pore pressure measurements. The pore pressure readings were generally taken every second week, but prior to and immediately after load changes once a week.

The settlements were observed by means of 16 settlement plates installed at the base of the runway embankment. Eight of these represented the part of the clay basin drained with one GEODRAIN / $m^2$ (the clay layer thickness varied at the measuring points from 5.1 to 10 m), two were placed in the area drained with an intensity of 0.6 / $m^2$ (clay thickness 5.2 resp. 6.6 m), two more in the undrained, only overloaded areas (clay thickness 2.2 m). Since July 1979 four more settlement rods were observed at the connection to the existing runway. All the settlement observations were done every second week.

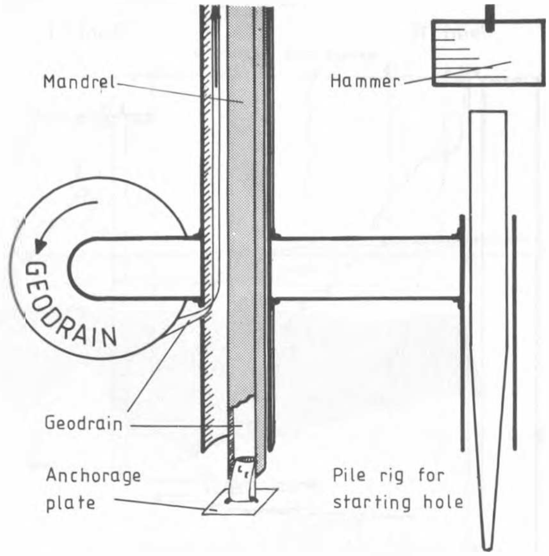


Fig. 5 Detail of the drain stitcher's prehole driving rig.

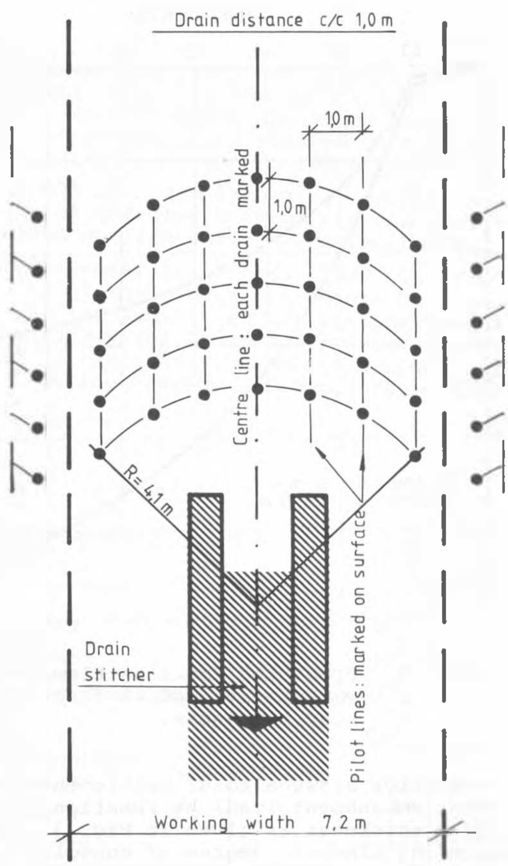


Fig. 6 The modified drain pattern in the case of c/c = 1.0 m

Prior to the construction of the embankments (the material of which was mainly unsorted blasted rock) six 250 mm concrete pipes were installed vertically around each of the thoroughly investigated test points. Through these pipes field vane tests and soil samplings could be performed always at the end of a loading step. Additionally ground water level observations were performed once a month in these pipes.

DESIGN PRINCIPLES

In order to reach the design level of the runway an embankment up to 3 m high, 80 m wide and over 500 m long had to be constructed. The corresponding surcharge of up to 65 kN/m$^2$ was expected to cause total settlements from 1000 to 1800 mm. The desired acceleration of the consolidation process could be achieved only with a certain overloading, and together with the compensation of the settlements, a total load of up to 100 kN/m$^2$ was found to be necessary. This corresponded to total settlements from 1300 to 2300 mm (see Fig. 3). A period of about two years (from summer 1978 to summer 1980) was available to achieve the desired degree of consolidation. To avoid stability problems the load had to be increased stepwise, in three to four intervals.

The vertical drains were dimensioned by aid of Barrons design principles (Barron, 1948), using a coefficient of consolidation $c_v = 0.3$ m$^2$/a and a ratio $c_h/c_v = 2.5$. These values were elaborated from laboratory tests and confirmed by piezometer soundings. With these parameters no "safety factors" were used in the design. The equivalent diameter of the prefabricated sheet drain type GEODRAIN was assumed to be $d_e = 0.05$ m (Hansbo, 1979)(Rathmayer, 1979).

Because each of the three to four designed loading steps could last only about 6 months and an average degree of consolidation U = 50 to 70 % should be reached, a drain distance of c/c = 1.0 m (retangular pattern) was chosen in the case of clay thickness exceeding 7 m. For areas with the clay thickness varying between 4 and 7 m a drain distance of c/c = 1.3 m was found reasonable. Further it was designed to improve a clay stratum with a thickness of less than 4 m simply by overloading without vertical drains.

An example of the designed loading steps and the corresponding settlement progress is shown in Fig. 7 for the case of clay thickness of 7 m.

The bearing capacity of the clay restricted the first loading step to 40...50 kN/m$^2$. With the next steps the load was increased by 15...25 kN/m$^2$. The designed degree of consolidation should be reached within 5 months, but the design marginal allowed time intervals varying between 4 and 8 months, if the consolidation process should proceed faster or slower than expected. A drain distance of c/c = 1.0 m was was also found necessary to accelerate the still ongoing consolidation of the clay layer underneath the existing stopway ( old fill ).

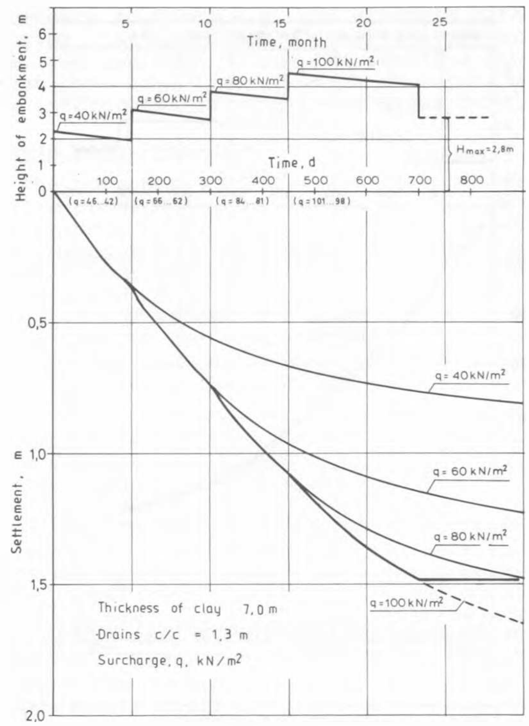


Fig. 7 Designed loading scheme vs. predicted settlements in the case of 7 m clay and drain distance c/c = 1.3 m.

COMPARISON OF MEASURED AND PREDICTED PREFORMANCE

Settlements

Typical examples of observed settlements versus time and embankment load are presented in the Fig. 8 a...d. In the very beginning of the loading process the observed settlements were somewhat smaller than expected. The first reactions which arose, were to doubt the efficiency of the prefabricated sheet drains. Another explanation, which was offered, concerned the in earlier and similar cases observed discrepancy between values of preconsolidation stress obtained in laboratory and the real preconsolidation stress values in situ.

In August 1979 the settlements at all observation points were carefully analysed. Taking advantage of the consolidation parameters obtained from laboratory tests and known load conditions from the field the analysis was performed by varying the equivalent diameter of the prefabricated sheet drains and the original stress conditions within reasonable limits. The analysis proved that the preconsolidation stress had been assumed about 10 kPa too small in the design calculations. The equivalent diameter of the GEODRAINS proved to be $d_e = 0.05$ m as reported from earlier applications (Leminen et al., 1978).
Based on these results of the analysis the theoretical time-settlements curves have been corrected and after that they fitted reasonably well in the observed values (Fig. 8 a...d).

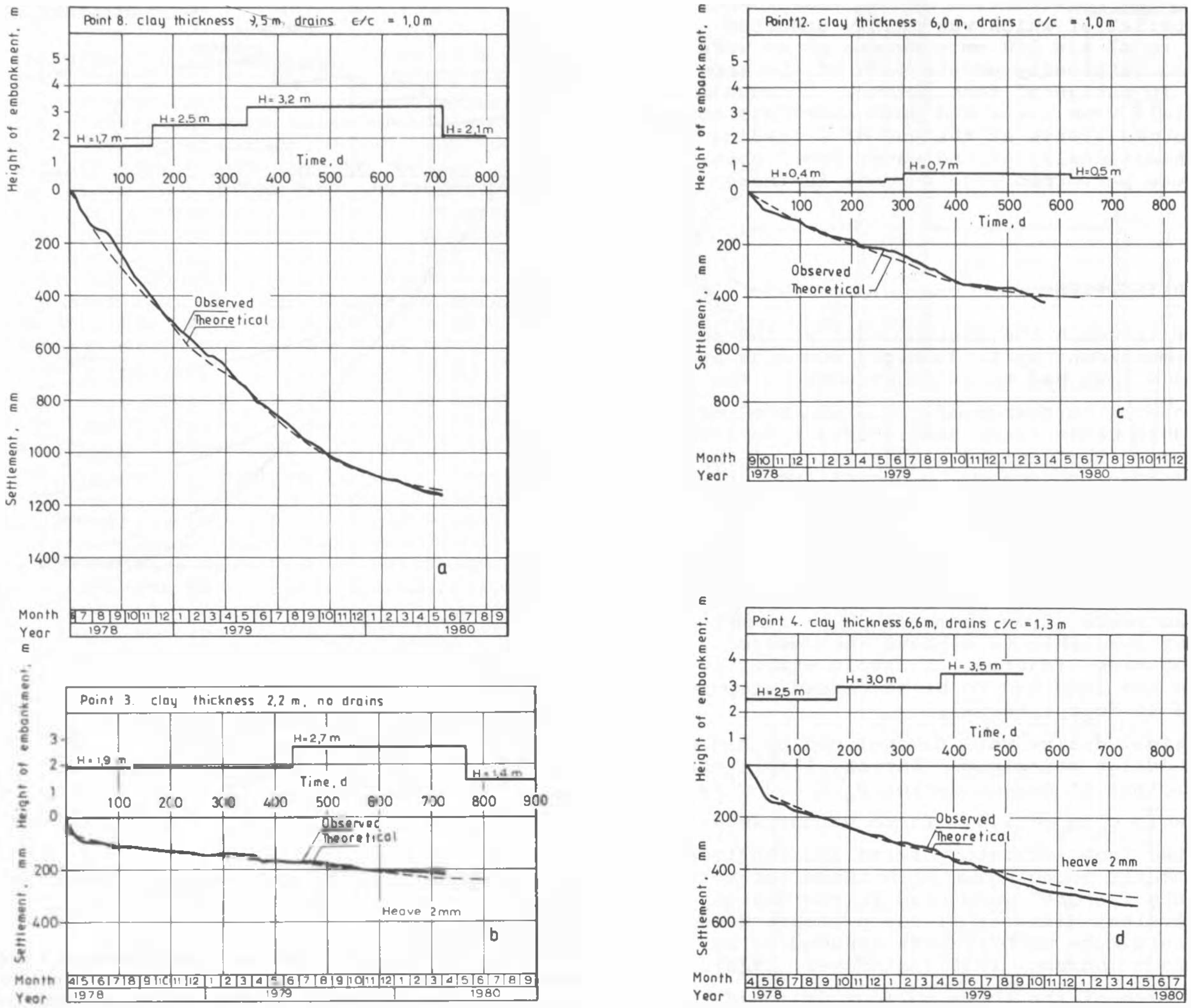


Fig. 8 a...d During different loading stages observed settlement rates c) partly consolidated clay layer beneath old fill (Stopway area).

TABLE II

Comparison of predicted and observed rate of settlements.

| point nr. | drains c/c, m | thickness of clay, m | At the end of loading (march 1980) | | | | After the replacement of overload (april 1980) | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | load, kN/m² | obs. settlement s, mm | calculated total settl. mm | rate of consolidation % | final load kN/m² | calculated total settl. $s_c$, mm | settlement left $s_c$-s, mm |
| 12 | 1,0 | 6,0 | 14 | 420 | 450 | 93 | 10 | 250 | - |
| 11 | 1,0 | 8,7 | 60 | 1095 | 1100 | 99 | 45 | 930 | - |
| 10 | 1,0 | 5,1 | 63 | 830 | 890 | 93 | 40 | 550 | - |
| 9 | 1,0 | 8,4 | 74 | 1260 | 1350 | 93 | 57 | 1170 | - |
| 8 | 1,0 | 9,5 | 57 | 1125 | 1170 | 96 | 40 | 900 | - |
| 7 | 1,0 | 7,2 | 72 | 965 | 1030 | 94 | 52 | 700 | - |
| 6 | 1,0 | 10,5 | 85 | 930 | 1020 | 91 | 62 | 720 | - |
| 5 | 1,0 | 8,0 | 55 | 650 | 680 | 96 | 32 | 170 | - |
| 4 | 1,3 | 6,6 | 67 | 525 | 580 | 91 | 40 | 200 | - |
| 3 | No drains | 2,2 | 58 | 215 | 300 | 72 | 36 | 225 | 10 |
| 2 | 1,3 | 5,2 | 52 | 255 | 320 | 80 | 27 | 50 | - |
| 1 | No drains | 2,3 | 23 | 190 | 220 | 86 | 5 | 50 | - |

The overload was replaced during April-May 1980. Since that the average degree of consolidation had reached a value of U=93...99 % with a drain intensity of 1.0 / $m^2$, about U=80...91 % with a drain intensity of 0.6 / $m^2$ and U=72...80 % in the areas without drains (see Table II). The amount of settlement exceeded at 11 from 12 points the total expected settlements for final surcharge, and only at one observation point settlements of 10 mm had still to be expected. So, according to the settlement observations, no meaningful consolidation settlements had to be expected after the replacement of the overload.

Changes in soil index parameters

The main results of laboratory tests before and after the consolidation period of nearly two years are shown in Fig. 9. Both the water contents and the void ratios have changed clearly. The water contents have decreased towards a value close to the liquid limit. Also the consolidation parameters have changed and resemble now typical values for overconsolidated clays ($\beta = 0$). Interpreting the changes in the void ratios and water contents with respect to the consolidation of the clay stratum an average degree of consolidation of U = 95 % could be evaluated (for February 1980). This result agrees well with the observed settlements (see Table II).

Concerning the preconsolidation stress no significant change could be evaluated from the samples tested. The preconsolidation stress remained even after the consolidation period in the same order of size as prior to loading. Probably this is due to the relatively long time necessary to change e.g. the strength parameters of clay. An unavoidable small disturbance of the clay specimen during the sampling process might have some effect, which therefore makes the interpretation of small changes difficult.

Excess pore pressures

As the pore pressure tips had been placed at the centre point of a square formed by four GEODRAINS, the maximum excess pore pressures had been the subject of the measurements performed. Consequently some discrepancy to the predicted average degrees of consolidation was obvious. Actually the excess pore pressures remained still at the end of the consolidation period at relatively high values. A typical distribution of observed pore pressures versus depth is shown in Fig. 10.

It has to be mentioned that there existed certain difficulties with the pore pressure measurements. A lot of measurements had to be rejected, because a tight connection between the tip's steel nozzle and the pressure sensor could not be achieved. The result of this fault has been, that the measured excess pore pressure values were significantly influenced by the water column in the protecting steel pipe. Also measurements during the cold winter periods were extremely laborious. At the late stage of the consolidation process the continuity of the pore pressure measurements could be guaranteed by the following measure. Plastic hoses were fitted to the steel nozzles of the BAT piezometer tips and consequently used as stand pipes for the pore pressure measurements.

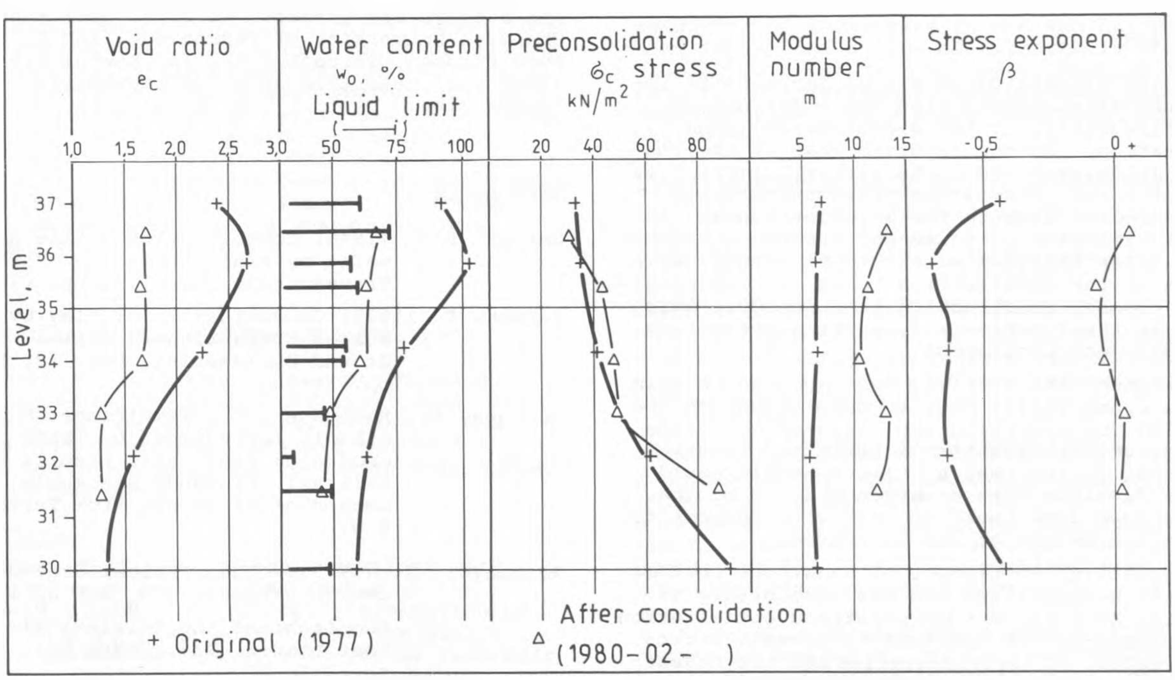


Fig. 9 Changes in some soil charecteristics due to the consolidation process.

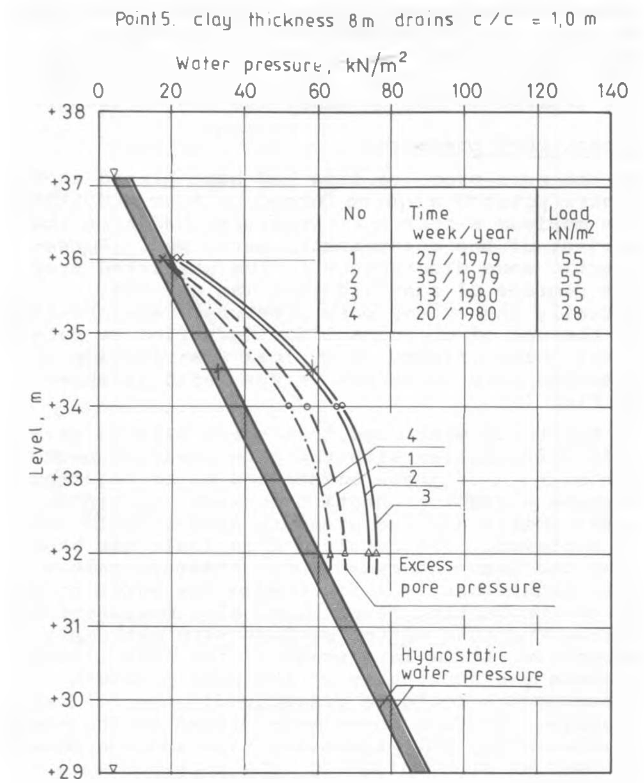


Fig. 10 Measured excess pore pressures at different loading stages

CONCLUSIONS

The choice of vertical drainage for subsoil improvement asked, concerning the construction time, flexibility of the executive building organisation. In the reported case of the runway lengthening at the Turku airport, the Finnish National Board of Aviation tried to optimize their temporal demands for a longer runway, as possible increase in passenger transport capacity or earlier realized safety improvements, with respect to the construction costs. It was possible to choose a construction time schedule which fitted in their nation-wide working activities and employment programmes.

The design of the vertical drainage can be said to be a conservative one, to some extent on the safe side for achieving the designed consolidation degree with reasonable certainty in time. Additionally, the loading time schedule could be kept flexible with a margin of 2...3 months. But no concessions could be made with respect to the limited height of the surcharge.

The flexibility in respect of construction time proved to be justified and was undoubtedly of financial benefit. But the practised optimization was applied also to the subsoil investigations, to the extent of instrumentation and the measuring resp. following up efforts involved.

The basic assumption, that the designed loading sequence will not cause essential horizontal deformations within the clay stratum, effaced inclinometers from the instrumentation list. Assuming additionally, the clay deposit to be homogeneous within acceptable limits, a more dense network of observation points and also deep settlement gauges were considered unnecessary. The finally installed intrumentation and the efforts involved in following up procedures had to serve the following purposes. A safe progress of the subsoil consolidation had to be ensured. In order to enable organizing of the necessary resources for load placement or removal a forecast of necessary measures should be made possible appr. 3 months in advance.

It has to be admitted that the decisions had finally to be based on settlement observations with some support from soil investigations (determination of soil index parameters as water content, void ratio, modulus number and modulus exponent,...). The measured excess pore pressures which showed, to some extent, delay of the consolidation progress, was not in good agreement with e.g. the settlement observations or soil investigations. In this connection it has to be pointed out clearly that a definite estimate of the reliability of the pore pressure measurements fails because of lack of supporting data, as inclinometer measurements or results from deep settlement gauges.

Judging finally, how successfully the subsoil was improved with respect to the demanded purposes, there remained no doubts that the vertical drainage by means of prefabricated GEODRAINS has been an optimum solution.

ACKNOWLEDGEMENTS

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