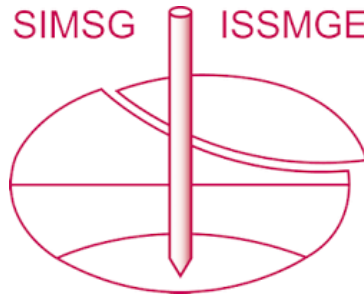


INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



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.

Geotextile reinforced land reclamation in the Bay of River Vantaa, Helsinki

La réclamation des terres du Baie de la Rivière Vantà, Helsinki, à l'aide de géotextiles

H. G. RATHMAYER, M.Sc. (Civ. Eng.), Senior Research Engineer, Technical Research Centre of Finland,
Geotechnical Laboratory, Espoo, Finland

O. E. KORHONEN, M.Sc. (Civ.Eng.), Division Manager, Geotechnical Department of the City of Helsinki, Helsinki, Finland

SYNOPSIS The City of Helsinki is reclaiming land for a recreation area in the bay of river Vantaa. On the 15 to 43 m thick mud and soft clay deposits of the shoreline single and twin embankments have to be erected. The consolidation process will last for 2 to 6 centuries. The overall stability of the shore is improved with woven polypropylene geotextile reinforcement. Short term tensile strength properties of 150 / 120 and 500 / 80 kN/m and in the long term a load take up reminder of 25% of failure load at 7% elongation are required. Design principles, results of in-situ and laboratory tests and problems encountered with their evaluation are presented. The chosen design and working methods are backed up by observations made at a 400 m long trial embankment and at earlier constructed reinforced embankments in the same area.

INTRODUCTION

The west shore of the bay of river Vantaa in Helsinki has for several decades been used as a dump. Owing to poor subsoil conditions (soft mud and clay deposits 15 ... 43 m thick), high filling levels of surplus material and lack of proper design the stability of this shore line is today insufficient. Slide planes have formed down into the soft deposits, some failures have occurred and the shore is in continuous motion towards the sea.

New plans call for the building of a recreation and harbour storage area. Over a distance of about 2200 metres a new shore line is to be built and it will run at most 200 m out from the present shore line. The depth of water in the future shore area will vary between 1 and 2,5 m.

The shore line is planned to be constructed mainly by means of floating reinforced embankments. The area behind them will be filled with masses of surplus material.

A first trial section of the reinforced floating embankment, 400 m long, was constructed in autumn 1983. At this section two different types of reinforcement and the proper installation technique were tested. New sections are planned to be built during the winter period 1984/85.

The design is backed up also by most valuable information gained from settlement observations of old and recent fills in the area, by inclinometer measurements and by observations made since summer 1982 at four test embankments reinforced in different ways.

SITE CONDITIONS

The subsoil at the west shore of the bay of river Vantaa consists of 15 to 43 m thick mud and soft postglacial clay layers. Heavy soil and ceramical waste masses, dumped during

several decades without proper design, rest on top of these layers. The water depth varies on the area under construction between 1,0 and 1,5 m.

The ongoing movements of these fills towards the bay are controlled with about 20 settlement rods and at the most critical area also by aid of inclinometer measurements. The settlement rates observed are up to 70 mm/year, the horizontal movement rates have reached values of up to 20 - 50 mm/month. At a depth of 20 m under the fills a failure zone has been mobilized. An impression of the instability of the area can be obtained from Figure 1 showing observed horizontal movements.

A design map of the shore is presented in Figure 2 showing the new recreation area and the harbour storage plant. The new shore line will have a length of about 2,2 kilometres behind which an area of approx. 200 000 m² has to be filled up.

The site investigations consisted besides of routine testing methods, as vane tests, Swedish weight sounding and sampling of undisturbed samples for further laboratory testing, also of more sophisticated in situ testing at the new

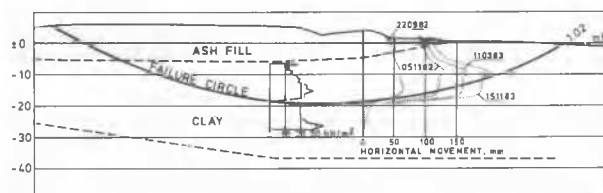


Fig. 1. Example of horizontal movements caused by uncontrolled fill. West shore of river Vantaa Bay, Helsinki.

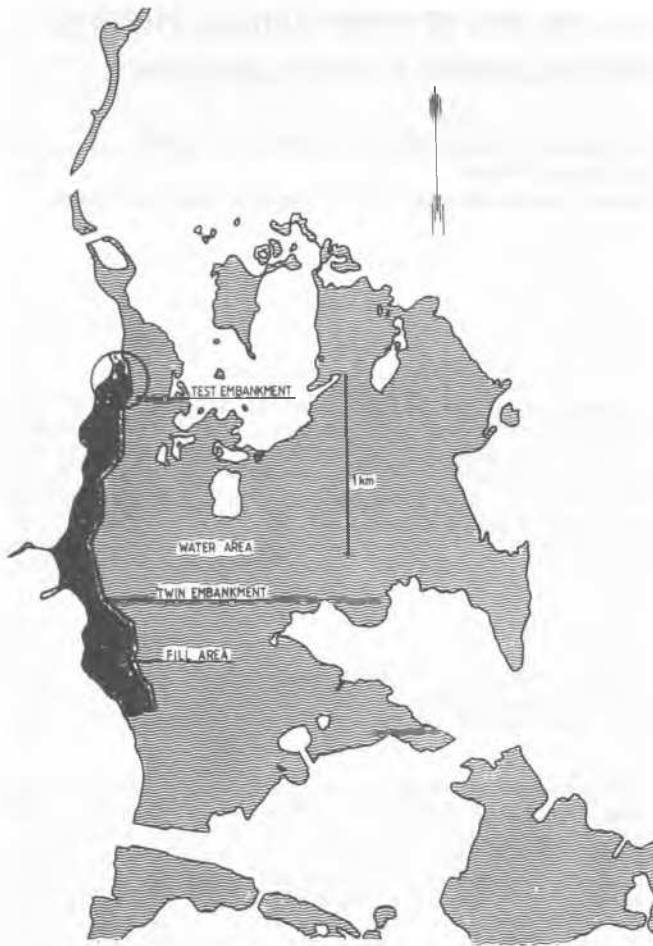


Fig. 2. Map of the new shore line.

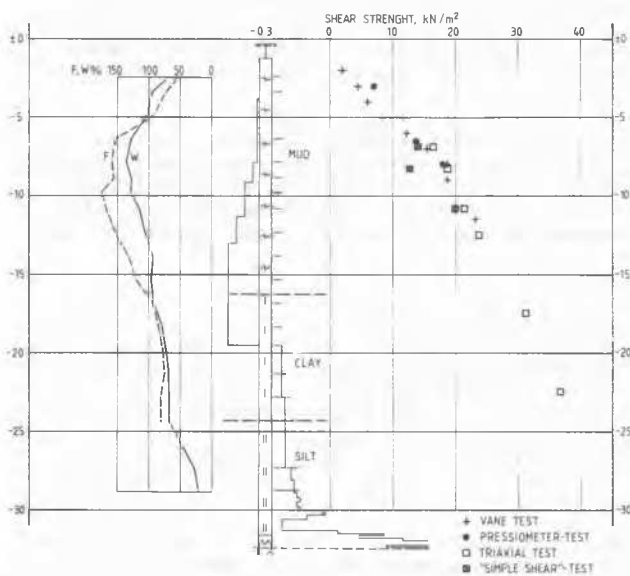


Fig. 3. Typical soil profile.

shore line. To clear up anisotropy and drained shear parameters of the soft soil layers pressuremeter tests and slowly rotated vane tests, using different size and shape of the vane blade, were performed.

In the slowly rotated vane tests the maximum shear stress was reached after 110 ... 160 minutes. The measured shear strength values were only about 60 - 70 % of those measured with routine procedure.

The strength and deformation properties were also determined from triaxial tests and simple shear tests. The triaxial tests were made as anisotropically consolidated undrained tests.

The shear parameters above level -12,0 N.N. varied in the limits of $c' = 1,8 - 7,9 \text{ kN/m}^2$ and $\rho' = 22,0^\circ - 32,0^\circ$. Below level -12,0 N.N. the limits were $c' = 1,4 - 3,8 \text{ kN/m}^2$ and $\rho' = 21,4^\circ - 24,2^\circ$ respectively. Also bearing plate tests were performed using different plate sizes. A typical soil profile is shown in Figure 3.

THE FLOATING EMBANKMENT SOLUTION

Single and twin embankment

The floating embankments are designed to be constructed by aid of geotextile reinforcement.

The reinforcement is required to act conjointly with the soil. The strength of the soil and of the reinforcement should be mobilized simultaneously. The reinforcement is located on top of the mud layer. A sand layer of a thickness of 0,3 - 0,5 m in thickness on top of the geotextile shall protect the reinforcement. The embankments are built of crushed rock.

At the northern end of the construction area the relatively better soil conditions allow the use of single floating embankments. Twin embankments are used where the thickness of the clay deposit is exceeding 25 meter. Typical sections of the embankments are illustrated in Figure 4.

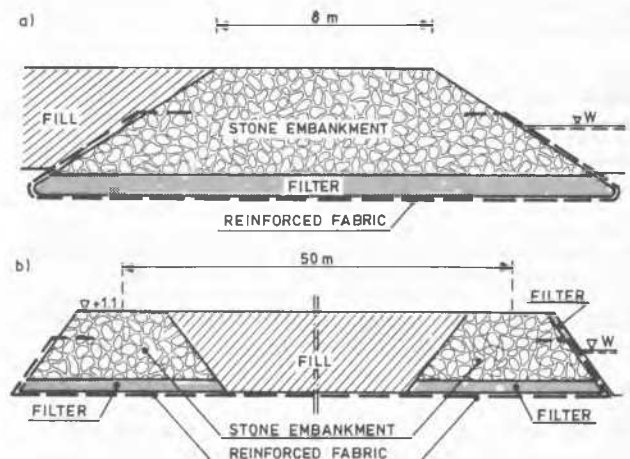


Fig. 4. Cross sections of the single and twin embankments.

Design principles

The design of the embankments is based on the strength and deformation properties of the soft clay deposit as well as on the thickness of it. For stability reasons the fill levels of the embankments may vary between +1,0 and +1,2 m above medium sea level, in some sections the fill height may rise up to +2,0 m.

The settlement predictions are based mainly on triaxial test results and also on oedometer test results.

The initial settlement of the embankment was estimated to be in the range of 450 - 600 mm (after construction the measured settlement was in the range of 300 - 700 mm).

The stability of the embankments is strongly dependent on the progress of settlements. The stability calculations were made by $\rho = 0$ and $c' - \rho'$ -methods for different construction stages and time periods. Especially three periods were controlled in the design. The initial stage for the period $t = 0-1$ years, the medium consolidated stage for $t = 20-30$ years and the final stage at $t = 100$ years. For different thicknesses of the clay deposits the required 90% consolidation times are given in Table 1.

Table 1. Time for 90 % consolidation as function of clay thickness.

Thickness of clay (m)	90 % consolidation time (years)
20	200
25	310
30	450
35	610

For the different design periods one has to consider the progress of settlements, the pore pressure development, the increase of the shear strength of the soil and the demanded corrections of the top level of the embankments.

The main conclusions of this design procedure were:

- Good knowledge of both short term and long term soil properties is essential for all design stages.
- The creep and relaxation behaviour of the reinforcing geotextile has to be taken into account.
- The deformation and strength properties of soil and geotextile reinforcement have to be adequately mobilized. For purpose of design the stress-strain properties of both soil and geotextile have to be known also as a function of time.

The principles of the combined action between soil and reinforcement is illustrated in Figure 5. Corresponding to the deformation behaviour of the clay the reinforcement strains may vary between 6 and 10 %. The principles of embankment design with reinforcement to improve the stability is illustrated in Figure 6.

As to be seen from Fig. 6 has the reinforcement also to be anchored.

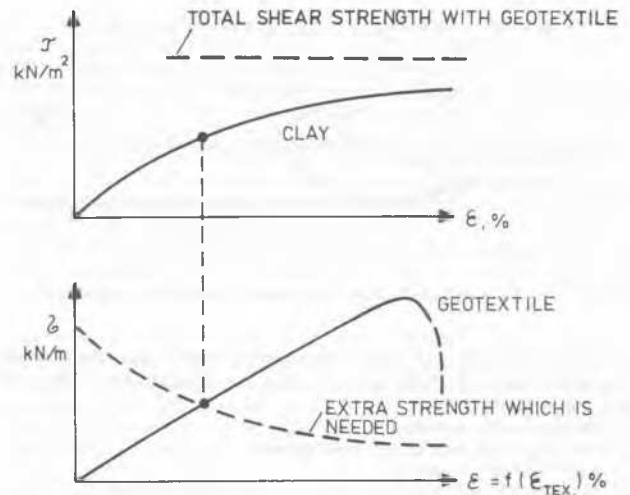


Fig. 5. Combined action of clay and reinforcement.

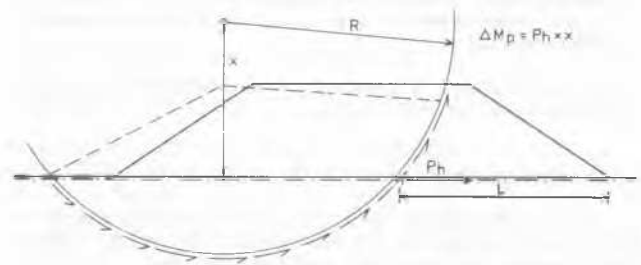


Fig. 6. Adopted design principles for reinforced embankment.

An example of long term stability calculations is given in Table 2. The anchor length L is calculated assuming that there are only cohesion forces acting on the reinforcement-clay-interface.

Table 2. Example of stability calculation.

M_{act} kN m/m	M_{STAB} kN m/m	ΔM_{STAB} kN m/m	R m	X m	P_h kN/m	L m	F -
6645	8446	1522	19	9	169	53	1,5

- M_{act} = active moment
- M_{STAB} = restoring moment
- ΔM_{STAB} = restoring moment activated by reinforcement
- R = radius of failure circle
- X = vertical distance between circle pole and reinforcement
- P_h = required force of reinforcement
- L = anchoring length
- F = factor of safety

The loading scheme and stability situation during the construction period is shown in Fig. 7.

CONSTRUCTION SEQUENCE

The reinforcement was placed on the mud layer with the help of a set of 7 - 10 pontoons. The largest dimensions of the fabric were 218 metres in width and 50 metres in length. Some parts of the fabric were installed with the help of light motor boats. The reinforcement was covered with a thin sand layer (0,3 - 0,5m) which had to act as a filter and to protect the geotextile. Sand and also crushed rock was spread out with a scoop of an excavator.

In the first stage the top level of the embankment was raised up to the sea level. Then the edges of the fabric were turned up on the embankment. Thereafter the embankment was raised to the designed height. The water area behind was filled with waste materials (moraine, silt, soft clay, etc.).

The fabric sheets were jointed together by sewing. Part of the sewing operation was done at the manufacturing plant, the rest of the joints were sewed on the pontoon. The joints were sewed with a portable stitcher and using an overlap of approximately 0,10 m in the seams. The handling of the reinforcement and some of the construction operations are shown in Figures 11 and 12.

REINFORCEMENT OF THE TEST SECTION

Material requirements

As the design calculations have shown, it is necessary to use reinforcement to achieve a sufficient factor of safety for the construction stage and during the time of consolidation. For a rotational failure the direction perpendicular



Figures 11 and 12. Embankment under construction

to the embankment axis is dominant. Assuming a 25 % level of long-term strength for polypropylene (PP) woven geotextiles the required short-term tensile strength level was assessed to 150 - 200 kN/m at break, 60 - 80 kN/m of which have to be mobilized at a strain level not exceeding 7 %. For ensuring safe installation progress the tensile strength requirements in the direction of the embankment axis (weft direction of the geotextile) were assessed to 65 % of the above mentioned requirements. Full strength mobilisation had to be guaranteed for the seams (joints), also.

A first selection of suitable woven geotextiles was based on out-of-plane strength testing with the pull-out cone test according to NORDTEST testing procedure NT Build 242 and on strip tensile tests (50 mm strip width) performed as CRS tests with strain rate of 100 mm/min.

The reinforcing geotextile installed was the Terram 85 K type. This fabric has an ultimate strength of about 150 kN/m in warp and 120 kN/m in weft direction. The corresponding strains at failure are roughly 30 % in warp and 20 % in weft direction. The strength mobilized at a strain level of 7 % was between 60 and 65 kN/m.

TESTING PROCEDURES AND RESULTS

Cone pull out tests

The tests were performed according to the NORDTEST testing procedure NT Build 242. The test gives an indication of the out-of-plane strength properties as a function of tensile strength in both warp and weft directions and of the weaving structure.

Permeability

The water permeability of the offered geotextiles was tested with constant head flow tests using pressure heads varying between $h = 0,5$ m and 1,56 m.

Fines retention

The ability of the woven geotextiles to retain fines was tested according to the BAW (Bundesanstalt für Wasserbau, Karlsruhe, F.R.Germany) testing procedure for soil type 4 in turbulent flow. The soil used in these propeller tests was a clayey silt taken from the site of installation.

PROBLEMS ENCOUNTERED WITH THE PREDICTION OF LONG TERM BEHAVIOUR OF REINFORCING GEOTEXTILES.

As far as testing of high strength woven geotextiles is concerned there are at present no generally accepted standardized testing procedures available for a fair comparison of materials manufactured from different raw materials.

Testing rate

According to existing standards for tensile strip testing the time to reach break may vary between 20 and 30 seconds. As some of the

materials are quite sensible to the rate of testing (strain rate) and knowing the differences in the elongation at break, fabrics made of polypropylene have to be tested at a much higher strain rate as e.g. polyester fabrics. The strain rates may vary between 100 and 200 %/min. On the other hand soil strength parameters which are determined at strain rates of 2 %/min (for granular soils) or essentially slower ones for cohesive soils (varying between 0.2 %/min to 0.0002 %/min), are used for the geotechnical design.

Testing temperature

Textile testing should according to DIN 50014 be performed at temperature and humidity controlled conditions, using either 20°C and 65 % R.H. or 23°C and 50 % R.H.. In the authors' opinion testing climate should reflect the in-soil conditions, which in Finland would mean soil temperatures of +8...+9°C and wet testing.

For many applications the knowledge of strength properties at frost temperatures is essential. Unfortunately only little or no information is available from the material suppliers on the influence of low temperatures on the strength properties of fabrics.

Clamping

For high strength technical fabrics the clamping technique is playing an essential role for achieving of repeatable failure strength values. Simple pressure clamping technique has proved to be unsufficient, when the tensile strength values exceed about 45 kN/m. Slippage in the clamps might occur, strain measurements get inaccurate.

Long-term properties

Long term loading tests with Terram 85 K samples were started in March 1984. Strips of 50 mm width from both warp and weft directions are loaded statically under controlled temperature conditions, using a temperature of +10°C. The

load levels were chosen between 60 and 95 % of failure load. The results of these tests are presented in Fig. 13. With a load level of 95 % of average failure load one sample failed after 2 months of loading. All the other samples are still intact after 6 months of loading. This information gives a good backfeed for stability considerations. The excess pore pressure at the site has exceeded its maximum and consolidation of the subsoil is in progress.

Stress relaxation

A test series to study stress relaxation of the fabric material at working loads and temperatures is intended to be started in autumn 1984. The strain level in these tests will be 7 %. These tests could provide the designer with the essential information, how long the loading capacity of 25 % of failure strength is available at strains less than 7 %.

REFERENCES

- BAW (1978). Vorläufige Richtlinien für die Prüfung und Dimensionierung von textilen Filtern im Verkehrswasserbau. Bundesanstalt für Wasserbau, Karlsruhe, F.R.G.
NT Build 242 (1984). Nonwoven Geotextiles. Plunger pull out test. NORDTEST, Helsinki, Finland.

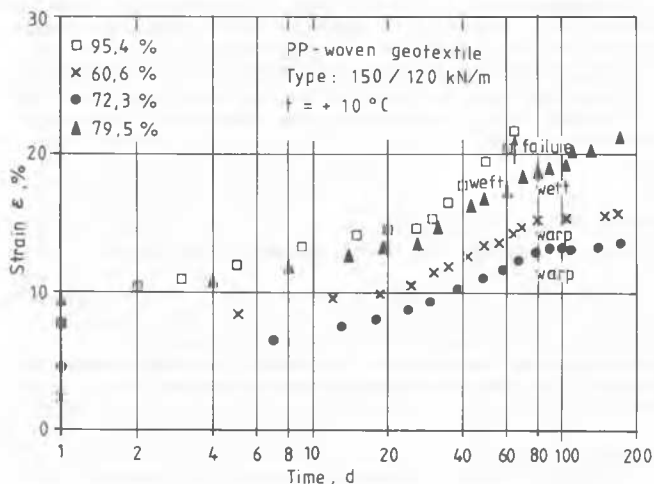


Fig. 13. Creep behaviour of PP woven geotextile at +10°C.