STATENS GEOTEKNISKA INSTITUT

SWEDISH GEOTECHNICAL INSTITUTE

No. 51

SÄRTRYCK OCH PRELIMINÄRA RAPPORTER

REPRINTS AND PRELIMINARY REPORTS

Supplement to the "Proceedings" and "Meddelanden" of the Institute

Skå-Edeby Test Field—Further Studies on Consolidation of Clay and Effects of Sand Drains

1. Soil Movements below a Test Embankment
   Robert Holtz & Göte Lindskog

2. Long-Term Loading Tests at Skå-Edeby, Sweden
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3. Excavation and Sampling around Some Sand Drains at Skå-Edeby, Sweden
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(1–2) Proc. Specialty Conf. on Performance of Earth and Earth–Supported Structures, Purdue University 1972, Vol. 1:1 and Vol. 3

(3) 6th Nordic Geotechnical Meeting, Trondheim 1972

PREFACE

One of the proposed sites in the 1950's for a new international airport for the Stockholm region was Skå-Edeby, located about 25 km west of the city. The soils at this site consist mainly of very soft, normally consolidated medium sensitive glacial and post-glacial clays with a thickness of 9 to 15 m. In order to study the expected settlements and the effect of vertical sand drains, it was decided in April 1957 that the Swedish Geotechnical Institute (SGI) should carry out large-scale field tests at Skå-Edeby. Four extensively instrumented circular gravel fills were constructed. The soil underlying three of the fills was provided with sand drains of varying spacings, while the fourth area was undrained. In addition to the measurement of pore pressures and settlements, the strength and deformation properties of the soils were determined by comprehensive laboratory investigations.

Preliminary results of the investigations were published in September 1957 in a report by a special committee which had been appointed to consider the airfield location question. No definite conclusions could be drawn at that time about the consolidation rates and the final settlement of the fills. The Skå-Edeby airport project was later abandoned for economic reasons. However, it was decided that the field tests should be continued because of their importance to similar future projects including highway construction.

In charge of the field tests were Dr Sven Hansbo and Mr Justus Osterman, Director of the Institute at that time. The results of the work were presented by Hansbo in 1960 in SGI Proceedings No. 18: "Consolidation of Clay, with Special Reference to Influence of Vertical Sand Drains. A Study Made in Connection with Full-Scale Investigations at Skå-Edeby". In this report, which incidentally constituted Hansbo's doctors thesis, a complete description of the test site and of the results obtained up to mid-1959 were presented, as well as a new consolidation theory for sand drains wherein non-Darcian flow was considered.

In 1961, one of the circular test fills at the Skå-Edeby test field was unloaded, and the surplus gravel fill was used to construct an additional loading test in the form of an embankment. The work was sponsored by the Swedish Road Board and the National Swedish Council for Building Research. Preliminary results from this test were presented at the 3rd European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden (SGI Reprints and Preliminary Reports No. 7, 1964) by J. Osterman and G. Lindskog.

The investigations at Skå-Edeby were continued in 1970 by Dr Robert D. Holtz, a visiting research engineer at the Institute. The study was supervised by Dr Bengt Broms and Mr Göte Lindskog, while Mr E. Schwab and Mr B.G. Holm assisted substantially with various phases of the work. The project was sponsored by the National Swedish Council for Building Research.

The study involved primarily a summary and analysis of the measurements complemented with other laboratory and field investigations. The results have been reported in three papers which are included in this publication. The first two papers were presented at the ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures at Purdue University in 1972 and the third at the 6th Nordic Geotechnical Meeting in Trondheim that same year.

In view of the history of the Skå-Edeby test field, with its favourable soil conditions and with its comprehensive instrumentation, this field may be regarded as unique. It has, in fact, provided a better understanding of the consolidation process, including the behaviour and effect of vertical sand drains. The Institute fully intends to continue the consolidation studies at the test field as well as to use it for other geotechnical research projects.

Stockholm, April 1973

SWEDISH GEOTECHNICAL INSTITUTE

Proceedings of the Specialty Conference on the Performance of Earth and Earth-Supported Structures

Purdue University/June 1972/ASCE

SOIL MOVEMENTS BELOW A TEST EMBANKMENT

Robert D. Holtz,[1] A.M. ASCE, and Göte Lindskog[2]

ABSTRACT

In 1961, the Swedish Geotechnical Institute started an additional loading test at the Skå-Edeby test field, 25 km west of Stockholm, Sweden. One of the original test fills was unloaded and the surplus gravel used to construct an embankment 40 m long, 1.5 m high, and with a crest width of 4 m. In addition to the usual settlement meters and piezometers, inclinometer tubes were installed near the edge of the fill to detect horizontal movements in the subsurface soils. The soils at the site are typical of the test field: soft, normally consolidated, medium sensitive post-glacial and glacial varved clays, about 15 m deep.

The settlement and pore pressure histories of the test are presented along with a record of the substantial horizontal movements that have occurred. It was found that the present rates of dissipation of pore pressure and settlements are occurring somewhat faster than predicted by one-dimensional consolidation theory with the c_v determined from the oedometer test. If the contribution of lateral movement to vertical settlement is considered, then the field c_v based on the observed settlements is about twice the laboratory value but about half the field settlement c_v found in the other undrained test area at Skå-Edeby.

A recent sampling and testing program showed that there has been essentially no change during the past 10 years in either the water content or the undrained shear strength of the clay below the embankment.

DESCRIPTION OF TEST EMBANKMENT

In 1961, one of the circular test fills at the Skå-Edeby test field, 25 km west of Stockholm, Sweden, was unloaded, and the surplus gravel fill was used to construct an additional loading test in the form of an embankment. Preliminary results of measurements of the test were reported by Osterman and Lindskog (1963)

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Information on the location of the test embankment and a general description of the geology of the site are given by Holtz and Broms (1972). The geotechnical profile for the embankment is shown in Fig. 1; results of routine laboratory and field investigations are presented for both the original conditions before loading and 10 years later in 1971.

The embankment is 40 m long, 1.5 m high and has a crest width of 4 m; side slopes are 1.5:1. It was constructed in three stages as indicated on Fig. 2, and the embankment was completed in a little over one month.

The stress applied by the fill was about 27 kN/m^2 and based on the original shear strengths of the clay (Fig. 1), the factor of safety against failure was about 1.5. Still, from elastic theory, the maximum shear stresses under the edge of the embankment were quite high and may have locally exceeded the shear strength. Consequently, the observed lateral movements, described later, have been rather significant.

Instrumentation was installed near the middle of the embankment at the locations shown in Fig. 2. The piezometers, except P8 which is an open pipe, were the hydraulic SGI type II (Kallstenius and Wallgren, 1956). Settlements were measured by two types of gages. Those placed on the ground surface under the fill were simply steel rods welded to plates with settlement readings made by precise leveling. The gages for measuring settlements at depth consisted of a rod to firm bottom within a pipe welded to an earth screw. Relative movements were determined by a dial indicator.

Horizontal movements at the sides of the embankment were measured in plastic tubes by the SGI inclinometer (Kallstenius and Bergau, 1961).

RESULTS OF MEASUREMENTS

Fig. 3 is a summary of the load, pore pressure, and settlement records for the past 10 years. The overload has decreased somewhat due to settlement of the fill into the rather high water table. The variation in piezometric head shown in Fig. 3 is typical of the long-term response of the SGI piezometers (Holtz and Broms, 1972). The short-term response of the piezometers was too slow for a meaningful analysis of the development of pore pressures during loading.

The distribution of settlements was quite consistent, and thus only averages for some meters at corresponding locations and depths are presented. For example, the average difference between the gages at the ground surface, S5, S6, and S7, was less than one cm. Gage S12:4.0 was installed over a year after start of loading and its initial settlement was assumed to be about 15 cm. Settlement measurements were also made at the ends of the embankment. They showed a maximum difference of 4.5 cm between the ends and the center of the embankment after 10 years.

Horizontal movements were determined from periodic inclinometer surveys. In order to get a clearer picture of

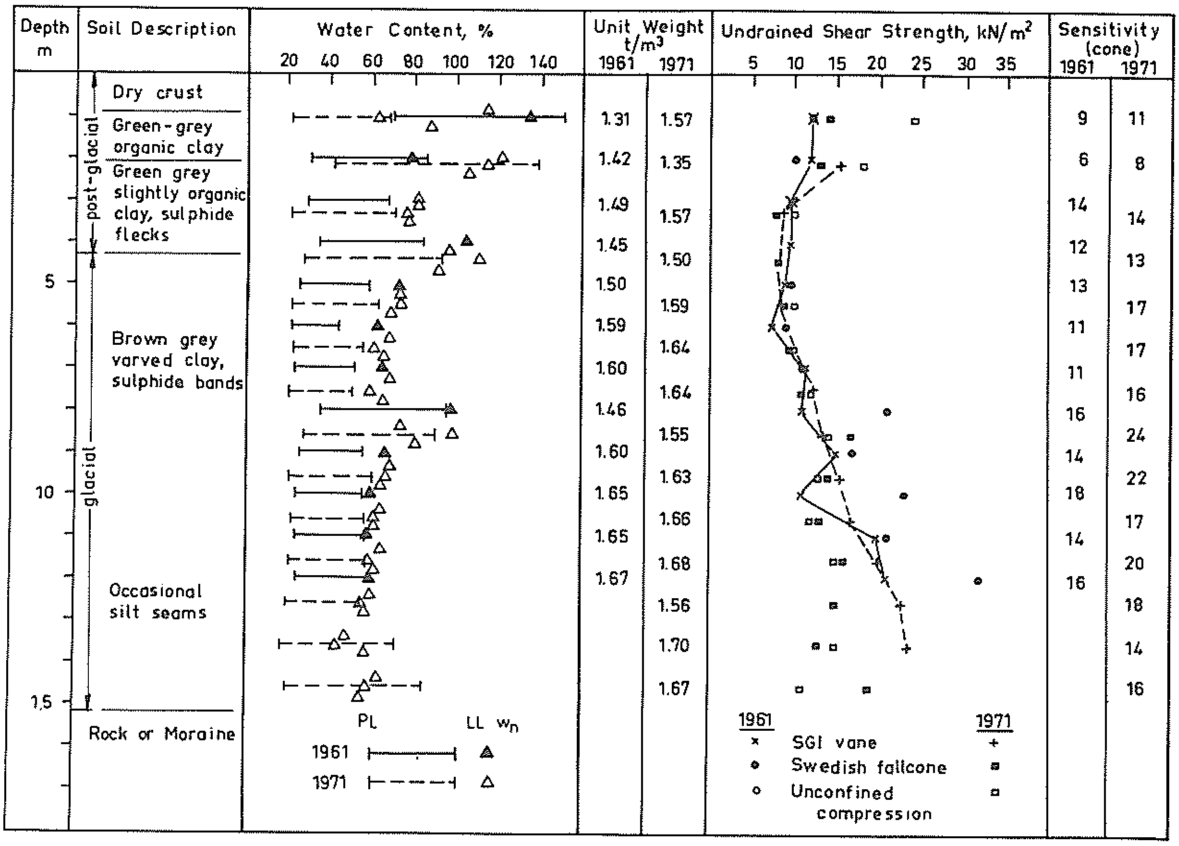

FIG. 1. GEOTECHNICAL PROFILE; 1971 TESTS RESTORED TO 1961 ELEVATIONS.

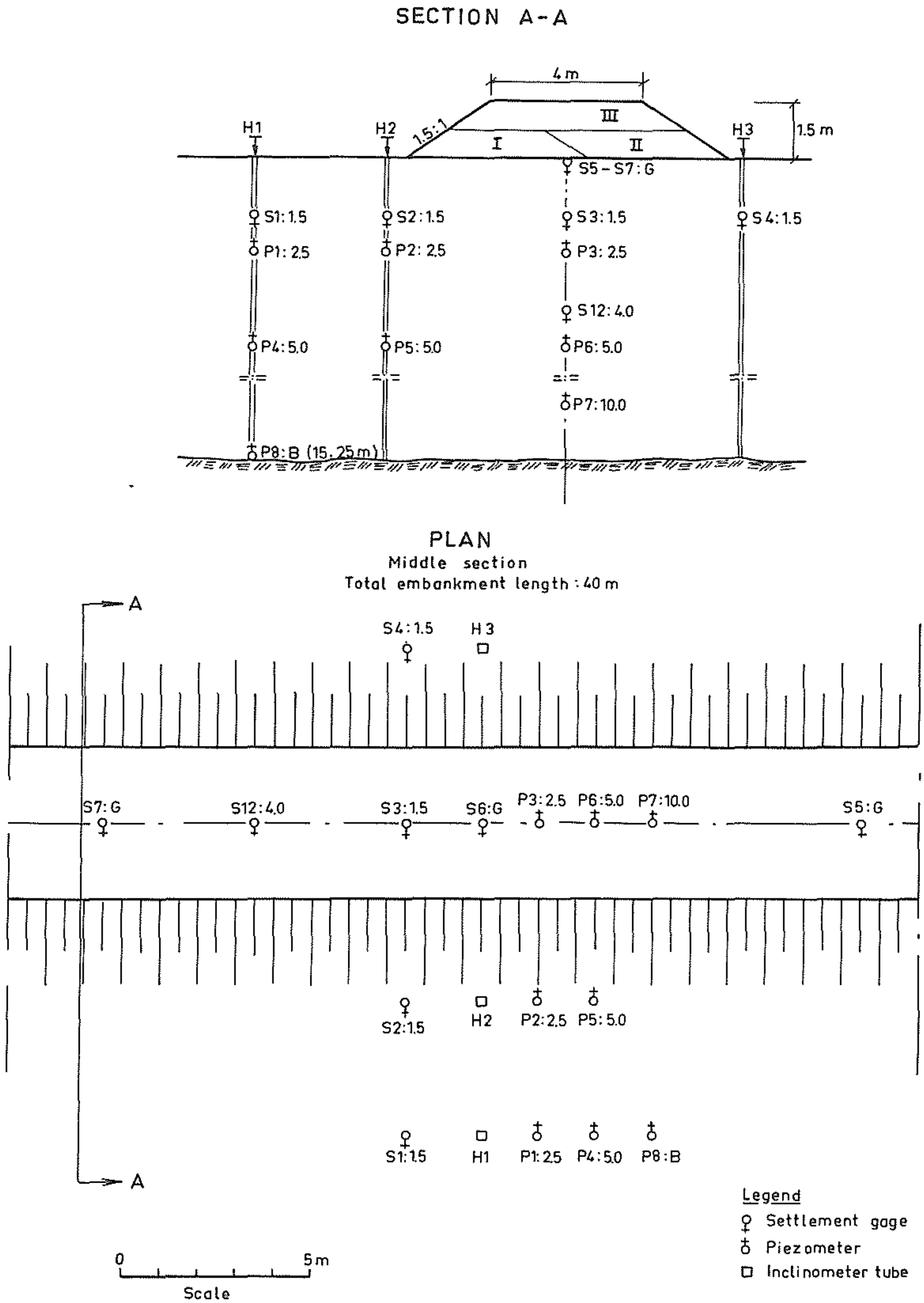

FIG. 2. PLAN AND SECTION OF EMBANKMENT SHOWING GAGE LOCATIONS AND DEPTHS BELOW GROUND SURFACE IN m.

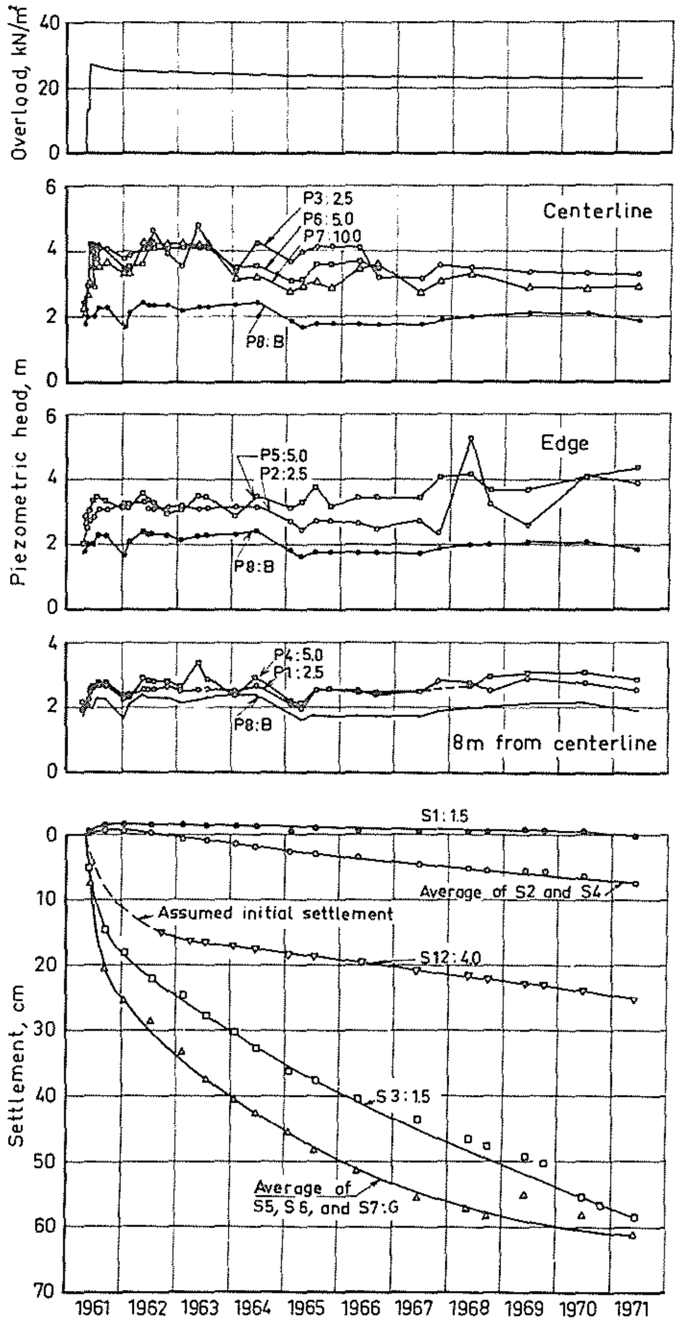

FIG. 3. OVERLOAD, PORE PRESSURE, AND SETTLEMENT HISTORY OF THE EMBANKMENT.

these movements, only some representative measurements are shown in Fig. 4. The initial line was determined a few days prior to start of construction and the embankment was completed on 1 June 1961.

From the settlement gages and horizontal movement records, the locus of a point 1.5 m below the surface and on the line of the inclinometer tubes can be plotted (Fig. 5). The loci shown in Fig. 5 are not exactly at the same points in space, because the settlement gages were located some 2 m from the inclinometer tubes (Fig. 2). Some error also can be attributed to the fact that the inclinometer surveys were not made at exactly the same time as the other gages were read.

The influence of the embankment load was felt some distance from the edge of the embankment. Horizontal movements were observed in inclinometer tube H1 (Fig. 4), while the piezometers both at the edge and 8 m from the centerline showed increases in pore pressure right after loading.

In 1969, most of the settlement gages showed a significant rise or heave relative to the previous readings. This was especially apparent in the surface gages at centerline. At the same time, a rather strong inward movement was observed in the horizontal movements. Since then, the settlements have tended to continue as before while the horizontal movements have reversed direction and have tended to move outward again. This anamolous behavior of 1969 shows especially well in Fig. 5 and the reason for it is not readily apparent. The movements in the clay suggest that the embankment load decreased as, for example, might occur if the ground water table rose significantly. There is no evidence that this occurred and it seems especially unlikely since 1969 was unusually dry in Sweden. The water table was apparently not appreciably affected either way, at least at the time the settlement and pore pressure measurements were made (2 June). The inclinometer surveys for that year were made almost exactly one month later on 3 July.

DISCUSSION OF RESULTS

Settlements.-- Based on oedometer tests conducted during the original soils investigation before construction of the test embankment, the ultimate primary compression was predicted to be about 1.5 m, if no consideration for load reduction was made, and about 1.2 m if load reduction was taken into account. This analysis followed conventional procedures wherein classical one-dimensional consolidation theory was used. Since the pore pressures (Fig. 3) under the embankment have not yet dissipated, no check can be presently made as to the validity of the prediction of the ultimate primary consolidation.

The rate of consolidation of the embankment was also predicted in 1961 using conventional settlement analysis procedures and the derived U-t relationship is shown in Fig. 6. This curve is based on an average c_v of 0.8×10^{-4}

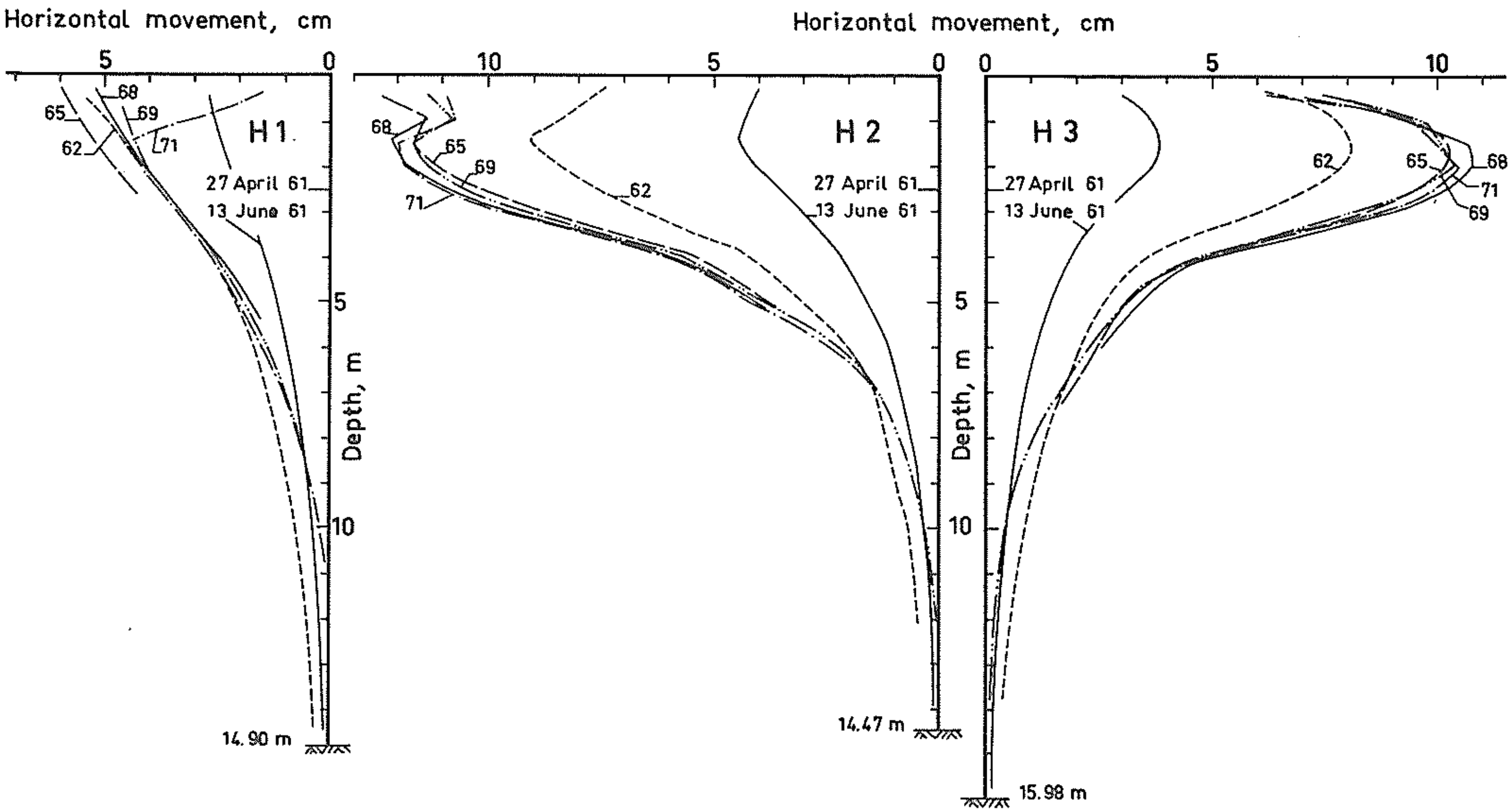

FIG. 4. HORIZONTAL MOVEMENTS SINCE BEGINNING OF LOADING AS DETERMINED BY THE INCLINOMETER TYPE SGI.

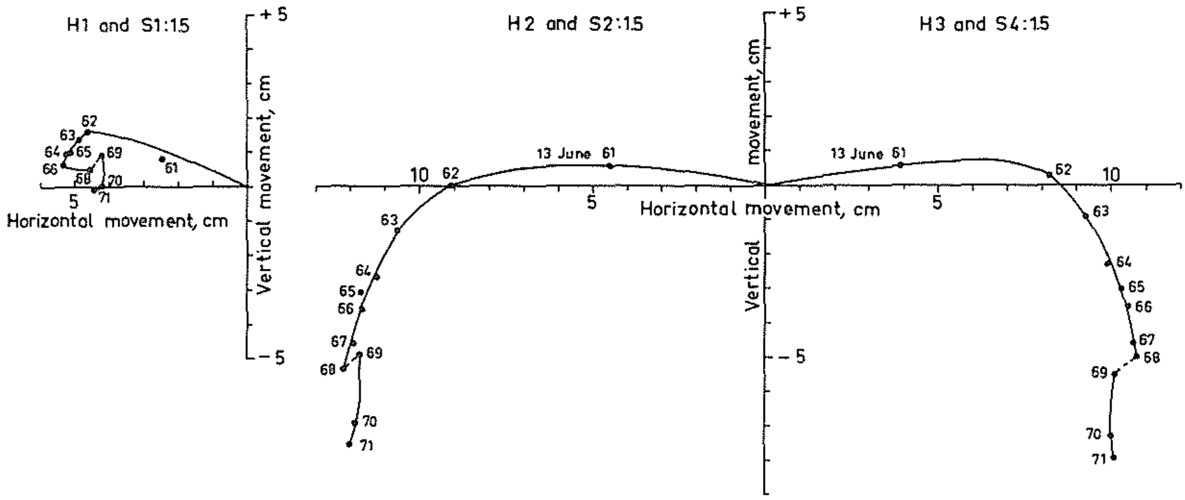

FIG. 5. LOCI OF HORIZONTAL AND VERTICAL MOVEMENTS AT INDICATED GAGE LOCATIONS.

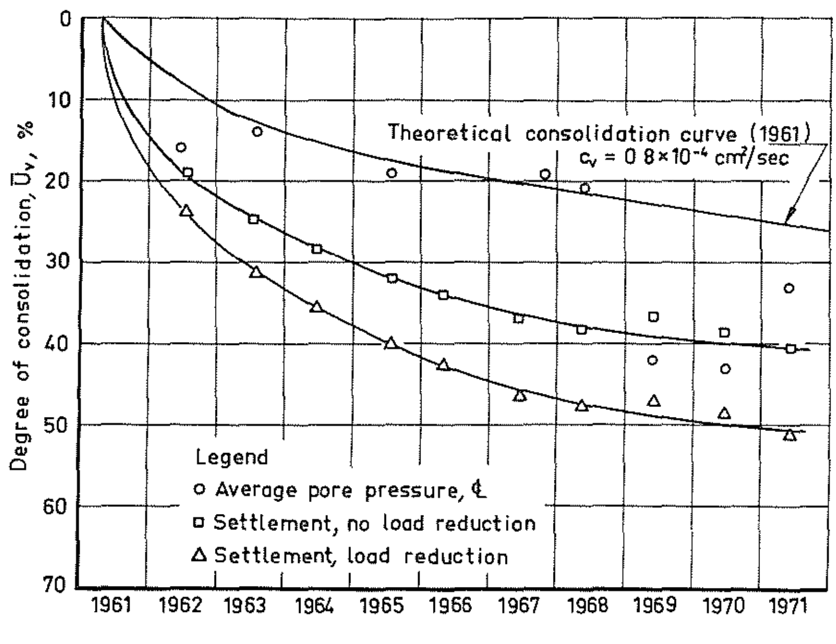

FIG. 6. DEGREE OF CONSOLIDATION AS DETERMINED FROM PORE PRESSURE AND SETTLEMENT OBSERVATIONS COMPARED WITH THEORETICAL (ONE-DIMENSIONAL) TIME-CONSOLIDATION CURVE (1961). LATERAL MOVEMENT NOT CONSIDERED.

cm^2/sec. Also shown in Fig. 6 is the degree of consolidation calculated from the average pore pressure dissipation at the centerline for the times indicated by the points. The agreement with the original theoretical prediction is reasonably good for the first 7 years of loading, but after that, it is poor even when average values are used and probably reflects inaccurate pore pressure measurements.

Finally, in Fig. 6, time-consolidation curves are given based on the observed rates of settlement and the predicted ultimate primary compressions given above. In these curves, no consideration is given to the contribution of lateral movement to vertical settlement. This contribution can be estimated, as was done by Osterman and Lindskog (1963), by calculating the volume of clay displaced from the observed lateral movements (Fig. 4) and by assuming a parabolic distribution of settlements under the embankment cross-section. Time-consolidation curves thus calculated would fall somewhat above but approximately parallel to those shown in Fig. 6. For example, in 1971 the average consolidation was about 12 to 15 percent less if lateral movement was considered.

From the average consolidation determined from settlement and pore pressure dissipation rates, a field coefficient of consolidation, c_v, can be calculated from classical theory (Table I). It is interesting to compare these results with those obtained from the other undrained test fill, Area IV, at Skå-Edeby (Holtz and Broms, 1972). For that area the rate of the pore pressure dissipation was predicted reasonably well by the oedometer c_v, and those values agree quite well with the laboratory value for the embankment. Depending on how the settlement rates were determined, the settlement c_v's range from slightly above these values to as high as four times greater. The most reasonable settlement value probably considers both lateral movement and load reduction, and this is about half the value from settlements found for Area IV (Table I).

If the excess pore pressure values measured under the embankment in 1970 and 1971 are correct, then both the settlements and pore pressures are currently following classical theory quite well with a field c_v about 2 to 4 times greater than would be predicted from the oedometer test. This is not unreasonable, considering the varved nature of the clay and the effect of sample disturbance on the oedometer test. However, it is not clear the piezometer values of the last two years are correct, and it would be fortuitous if one-dimensional theory would apply so accurately to such an obviously plane strain case. According to Schiffman, et.al. (1969), for some plane strain loadings, a coupled three-dimensional consolidation theory would predict that the rate of dissipation of the excess pore pressure would be less than for conventional one-dimensional theory for soft clay, and greater for stiff clay. Another factor which must be considered is the possibility of secondary compression going on during primary - a phenomenon observed by Chang (1972) at a test field north of Stockholm and possibly an explanation for the discrepancy between pore pressure dissipation and settlement rates observed in the undrained Area IV (Holtz and Broms, 1972).

TABLE I

COMPARISON OF LABORATORY AND FIELD COEFFICIENTS OF CONSOLIDATION

	c_v, $x10^{-4} cm^2/sec$
Embankment:	
Laboratory (1961)	0.8
Field	
a) from excess pore pressures	
1970	2.3
1971	1.2
b) from settlements (no lateral movement)	
with load reduction	3.3
no load reduction	2.0
c) from settlements (incl. lateral movement)	
with load reduction	1.6
no load reduction	1.0
Area IV (Holtz and Broms, 1972):	
Laboratory (1957)	0.4-0.5
Field	
a) from excess pore pressures (1971)	0.7
b) from settlements	3.0

Horizontal movements.-- As shown in Figs. 4 and 5, significant horizontal movements have occurred at the edge of the embankment. Maximum movement was at 1.5 m depth, and except for the reversal of 1969, the movements were quite consistent. There may have been some inward movement below 8 m after 1962, and the inclinometer at H1 indicated a similar reversal after 1965, possibly due to frost action.

The rate of lateral movement has obviously decreased significantly. During the early stages of the test, lateral movements were nearly 4 cm/yr, while presently, movement is about 0.2 cm/yr at 1.5 m depth. But they are still tending to go outward, even after 10 years. Osterman and Lindskog (1963) hypothesized that because of consolidation, the rate of lateral movement would eventually equal and/or become less than the rate of shrinkage, at which time, the horizontal movements would appear to reverse. From the present data, this point has apparently not been reached yet for this embankment.

Settlements calculated from changes in water content.-- Chang (1972) found that settlements at the center of the undrained test area at the Väsby test field could be predicted within about 15% from the change in water content which occurred at the centerline during the period of loading. A similar analysis (Holtz & Broms, 1972) for the undrained area at Skå-Edeby showed that virtually all the settlement at the centerline could be accounted for by changes in water content. These analyses were not so successful for the drained areas, probably due to the heterogeneity of the clay. Only a very crude averaging process can be attempted in any case, and perhaps close agreement is merely fortuitous. From Fig. 1, it is apparent that no significant change in water content has occurred during the past 10 years. Unfortunately, no detailed water content studies were made before loading, so this conclusion may not be exactly correct. In fact, if average water contents from four boreholes made outside but near the embankment in late 1961, are used, then a settlement of about 72 cm would be predicted for the centerline, and about 56 cm for a borehole made recently (1971) at the edge of the fill. Both these values would suggest that all the settlement of the embankment is due to classical consolidation. Still, we know that the clay did move out, especially in the top 5 m, and the contribution of this movement to vertical settlement cannot be discounted.

Change in soil properties due to consolidation.-- From Fig. 1, it is quite obvious that no significant change in the undrained shear strength has occurred during the past 10 years, and this tends to verify the observation that water content changes have been negligible, also no significant differences were observed in the classification properties between 1961 and 1971.

CONCLUSIONS

Based on the observations of movements in the clay under the test embankment at the Skå-Edeby test field, the following conclusions can be made:

1. In addition to large vertical settlements, substantial horizontal movements have occurred in the clays at the edge of the embankment, and the movement was observed at least as far as 8 m from the centerline. If the contribution of lateral movement to vertical settlement is considered, then the c_v from the present settlement rate is about half the corresponding c_v found for the other undrained area at Skå-Edeby and about twice the oedometer c_v.
2. The rates of consolidation based on pore pressure dissipation and settlement rates today are somewhat faster than predicted by one-dimensional consolidation theory and the c_v from the oedometer test. This is qualitatively contrary what might be expected from three-dimensional consolidation theory for soft clay.
3. The values of the coefficient of consolidation determined from observed pore pressure dissipation and settlement rates ranged from two to four times greater than the c_v determined from the oedometer test.
4. Even after over 10 years of settlement, no significant increase in the undrained shear strength or decrease

in the water content has occurred in the soft clays below the embankment.

ACKNOWLEDGEMENTS

The authors wish to acknowledge with thanks the contribution of the Swedish Road Board and the National Swedish Council for Building Research to the construction and instrumentation of the test embankment. The latter organization also generously supported the latest studies reported in this paper. The authors also wish to thank members of the SGI staff who assisted with various phases of the investigation: Most of the field measurements were made by E.G. Karlsson and E. Norén, while the recent field investigations were supervised by S.-E. Höök; A.-M. Körberg conducted the laboratory tests; the drawings were made by I. Danielsson and S. Johansson, and the manuscript was typed by E. Kjellström.

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SOIL MOVEMENTS BELOW A TEST EMBANKMENT

Robert D. Holtz[1], A.M. ASCE, and Göte Lindskog[2]

In the paper[3], we stated that the measurements of excess pore pressures remaining under the test embankment were questionable because of the rather wide scatter shown by the SGI type piezometers. The pore pressures were remeasured using NGI vibrating wire piezometers during the spring of 1972 and an excess pore pressure profile was obtained which was similar to those found under the other undrained test area at Skå-Edeby (Holtz and Broms, 1972, Fig. 15).

Based on the excess pore pressure remaining in the middle of the clay layer, the average degree of consolidation today is about 58 percent. This is equivalent to an one-dimensional c_v of about 3.8×10^{-4} cm^2/sec, or significantly greater than the c_v values found previously (Table I in the paper).

The effect of lateral movement on the degree of consolidation based on the settlement was discussed in the paper, but the curves were not shown on Fig. 6. A corrected version of that figure is reproduced below with the results from the 1972 measurements included.

It is felt that the most reasonable model for the degree of consolidation based on settlements is the one wherein both lateral movements and load reduction are included. For this case, the average degree of consolidation is about 39 %, or much less than that based on the excess pore pressure remaining under the fill as of 1972. Both these rates are, however, significantly greater than predicted by one-dimensional theory and the oedometer test, which is shown as the "theoretical" curve in Fig. 6. Contrary to what was stated in the paper, the difference is reasonable for plane strain, and work is currently in progress to develop the correct theoretical consolidation curve for this embankment.

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3 Vol. I, Part 1, pp 273-284.

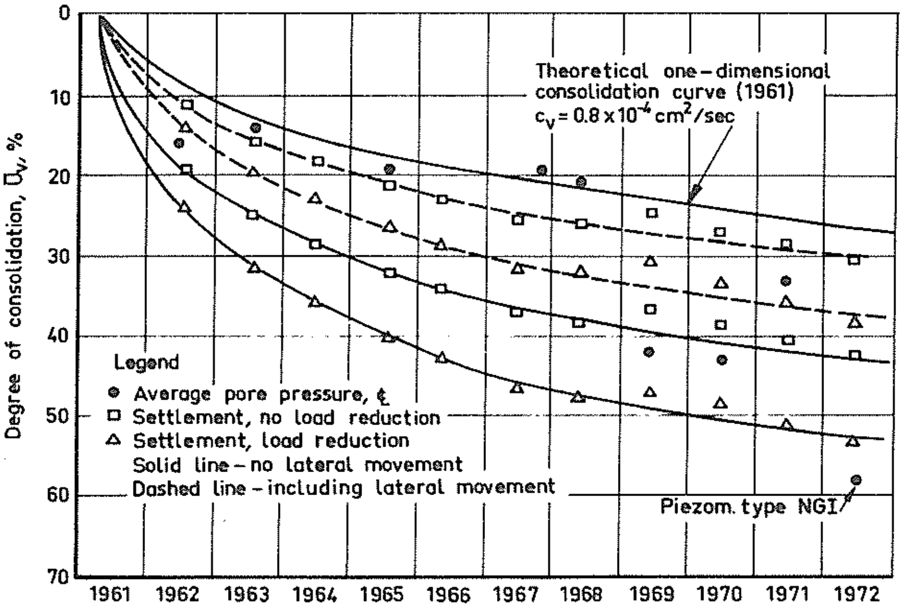

FIG. 6 DEGREE OF CONSOLIDATION AS DETERMINED FROM PORE PRESSURE AND SETTLEMENT OBSERVATIONS COMPARED WITH THEORETICAL (ONE-DIMENSIONAL) TIME-CONSOLIDATION CURVE (1961).

REFERENCE

Holtz, R.D. and Broms, B. (1972). "Long-Term Loading Tests at Skå-Edeby, Sweden", Proceedings of this Conference, Vol. I, Part 1, pp 435-464.

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Purdue University/June 1972/ASCE

LONG-TERM LOADING TESTS AT SKÅ-EDEBY, SWEDEN

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ABSTRACT

The paper presents a review of observations and test results from the full-scale loading tests at the Skå-Edeby test field, located about 25 km west of Stockholm, Sweden. The field, constructed in 1957 by the Swedish Geotechnical Institute, consists of four extensively instrumented circular test fills up to 70 m in diameter, three of which are provided with sand drains of varying spacings (Hansbo,1960). The soils are soft, normally consolidated, medium sensitive post-glacial and glacial varved clays, 9 to 15 m thick.

The summaries of 14 years of settlement and pore pressure observations are presented and analyzed. Hansbo's(1960) theory of consolidation for sand drains, wherein non-Darcian flow is considered, predicted the observed rates of settlement and pore pressure dissipation somewhat better than classical theory. Field values of c_h for the drained areas were about the same as the c_v-value from oedometer tests, probably due to remolding during driving of the drains.

For the undrained area, the oedometer c_v quite accurately predicted the dissipation of excess pore pressure, while c_v determined from the observed settlement rate was much larger and suggests that considerable secondary compression has gone on during the primary phase. Yet, all of the settlement at the center can be accounted for by observed changes in water content during consolidation.

Hansbo's predictions of final primary settlement based on the oedometer test were within 5 to 20 percent for the drained areas. No evidence was found that remolding due to driving of the drains affected the rates and amounts of primary and secondary compression.

Because of consolidation, the average water contents have decreased 5 to 20 percent, the average undrained shear strength has increased about 5 kN/m^2, and the s_u/σ'_{vo} ratio has decreased slightly.

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DESCRIPTION OF TEST FIELD

In 1957, in connection with the planning for a new airport, the Swedish Geotechnical Institute, in cooperation with the Swedish Road Board, designed and constructed a test field 25 km west of Stockholm, Sweden. The field consisted of four circular loaded areas, three of which were provided with 18 cm diameter sand drains at various spacings. Hansbo (1960) described in detail the installation of the drains, the construction of the test fills, and the instrumentation installed at the test site; thus, only a summary is given in this paper.

Geological Description and Geotechnical Properties.--The general geologic conditions at the site are rather typical of the very recent glacial and post glacial clay deposits of central Sweden. The site probably emerged from the sea only 400 to 500 years ago; today the ground surface is only about 2.5 m above the mean level of the Baltic sea. The deepest sediments are less than 10,000 years old and, as discussed in detail by Pusch (1970), both the glacial and the post-glacial sediments were formed in oxygen-poor environments and thus are rich in iron sulfide. A dry, weathered crust, typically about 1 m thick, overlies 3 to 5 m of post glacial, slightly organic and rather homogeneous clays.

The ground water table is usually located 0.5 to 1.0 m below the ground surface. The deeper glacial sediments are thinly varved and dipping near the top and become thicker (1 to 2 cm) and more horizontal with depth. Sulfide bands are common, and near the bottom occasional seams of sand and silt are found. Bedrock or dense glacial moraine is found typically at a depth of 9 to 15 m (Fig. 1). The glacial clay is usually 4 to 10 m thick. Salinity measurements show that some leaching has occurred because both the upper post glacial clays and the deeper glacial clays contain today about 0.5 percent salt in the pore water. Predominate cations are Fe, Na, and K in the post-glacial clays, while in the glacial clays, Na is the most abundant cation but significant amounts of Mg, K and Ca also are present. The organic content is about 0.4 percent for both clay types.

Geotechnical soil profiles are presented in Figs. 2 through 7. Shown are both the properties as determined in 1957 prior to loading and the results of recent (1971) sampling and testing beneath the loaded areas. Generalized soil descriptions based on a visual classification of the specimens are also given. The subsurface investigations are typical of those commonly conducted in Sweden and include field tests with the SGI vane as well as standard 50 mm Swedish piston sampling.

The water contents, especially in the upper layers just below the dry crust, are high, and often significantly above the liquid limit. The undrained shear strength as determined by the several methods is very low even by Swedish experience. Sensitivity is normal for such deposits in cen-

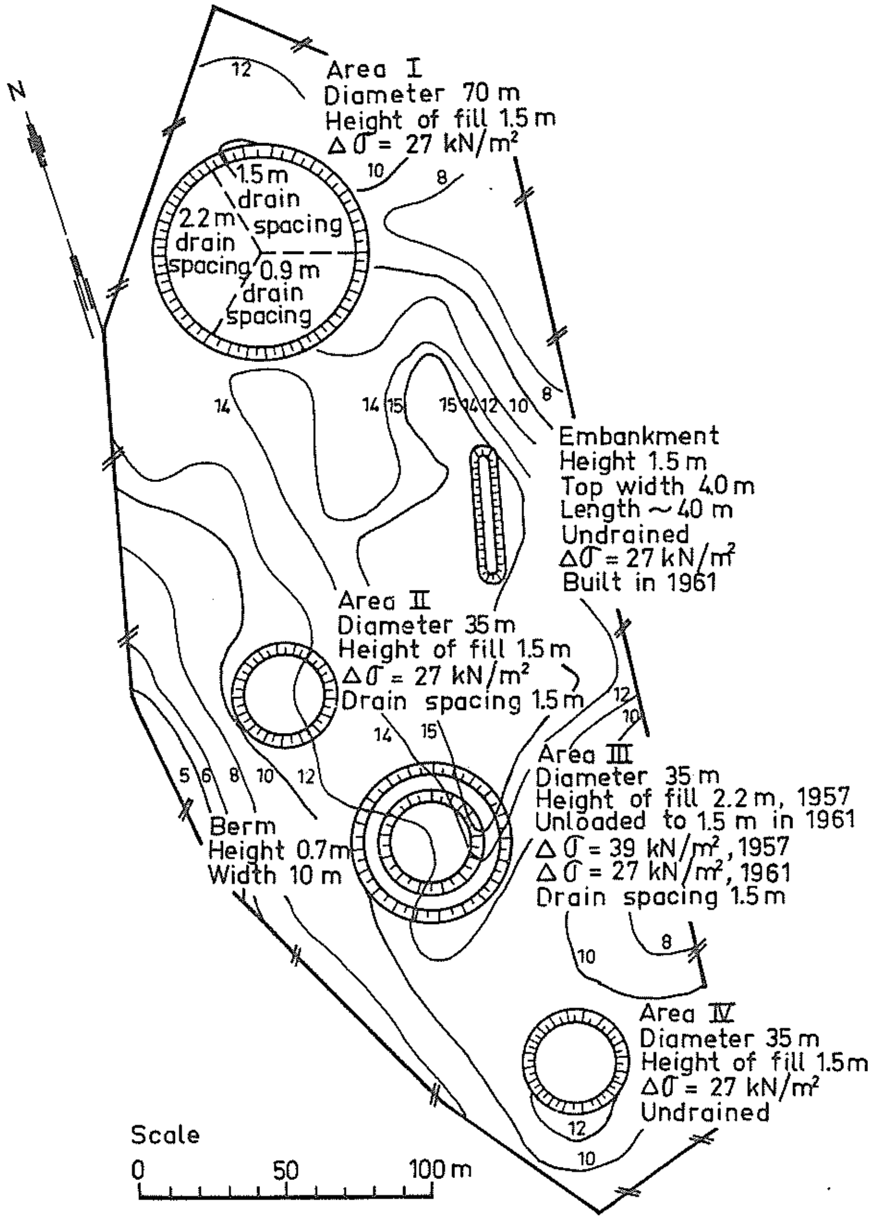

FIG. 1. PLAN OF THE SKÅ-EDEBY TEST FIELD. CONTOUR LINES INDICATE DEPTH TO FIRM BOTTOM, IN m.

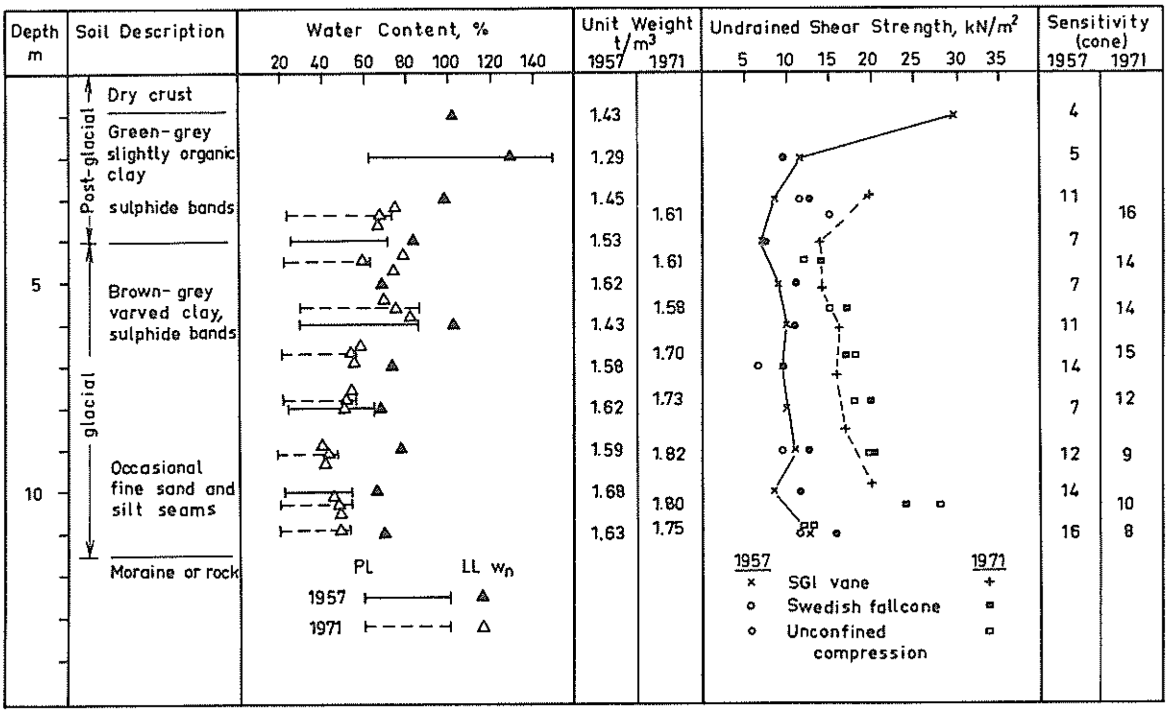

FIG. 2. GEOTECHNICAL PROFILES, AREA I, 0.9 m DRAIN SPACING; 1971 TESTS RESTORED TO 1957 ELEVATIONS.

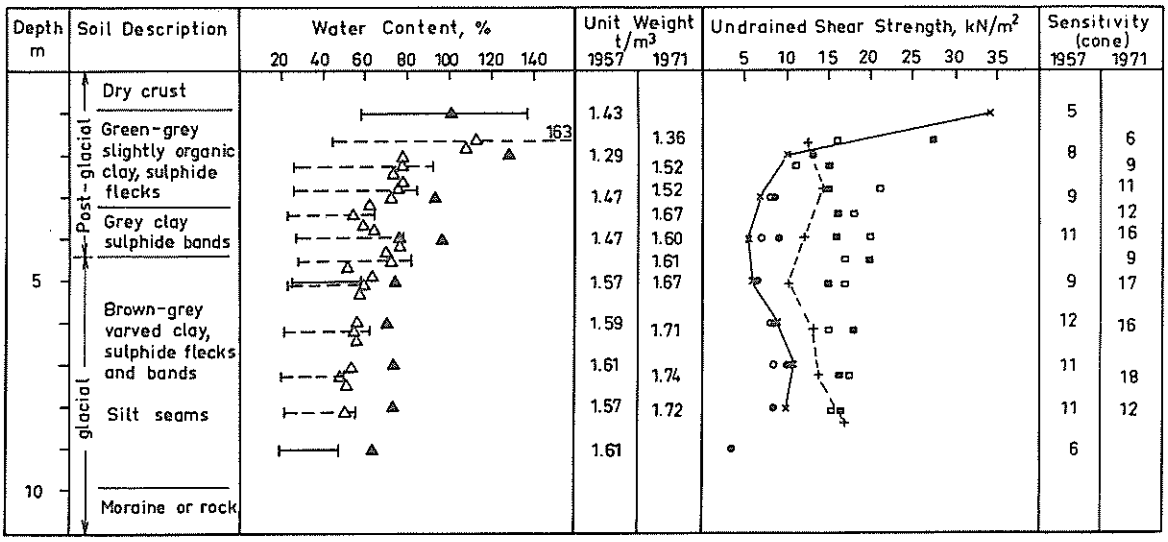

FIG. 3. GEOTECHNICAL PROFILES, AREA I, 1.5 m DRAIN SPACING; 1971 TESTS RESTORED TO 1957 ELEVATIONS. LEGEND ON FIG. 2.

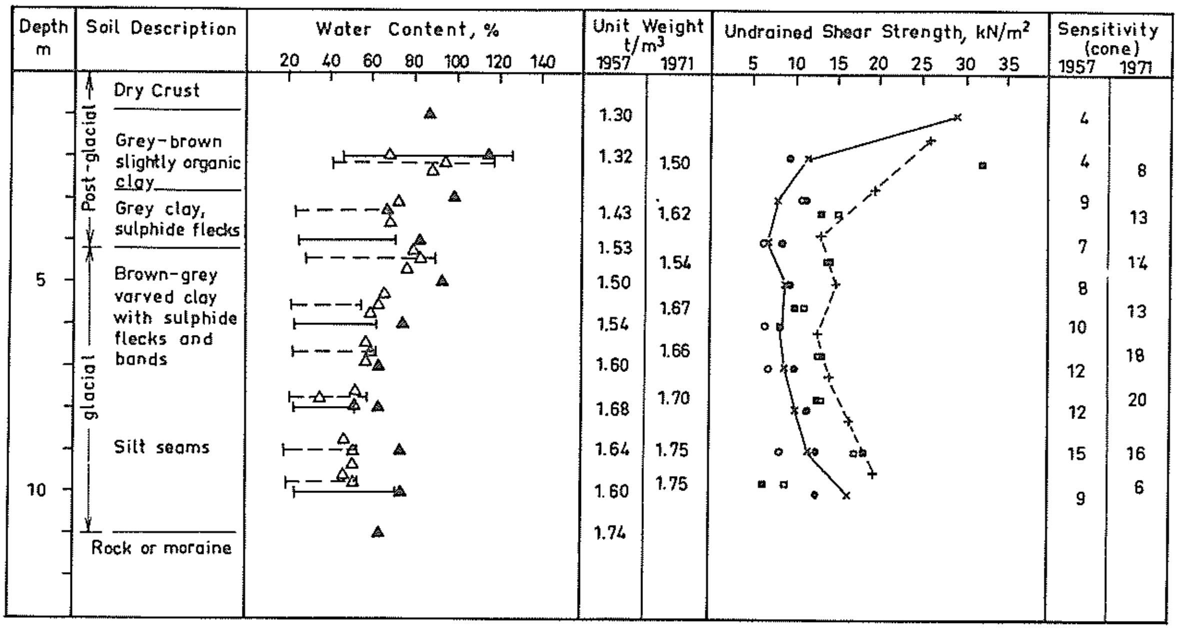

FIG. 4. GEOTECHNICAL PROFILES, AREA I, 2.2 m DRAIN SPACING; 1971 TESTS RESTORED TO 1957 ELEVATIONS. LEGEND ON FIG. 2.

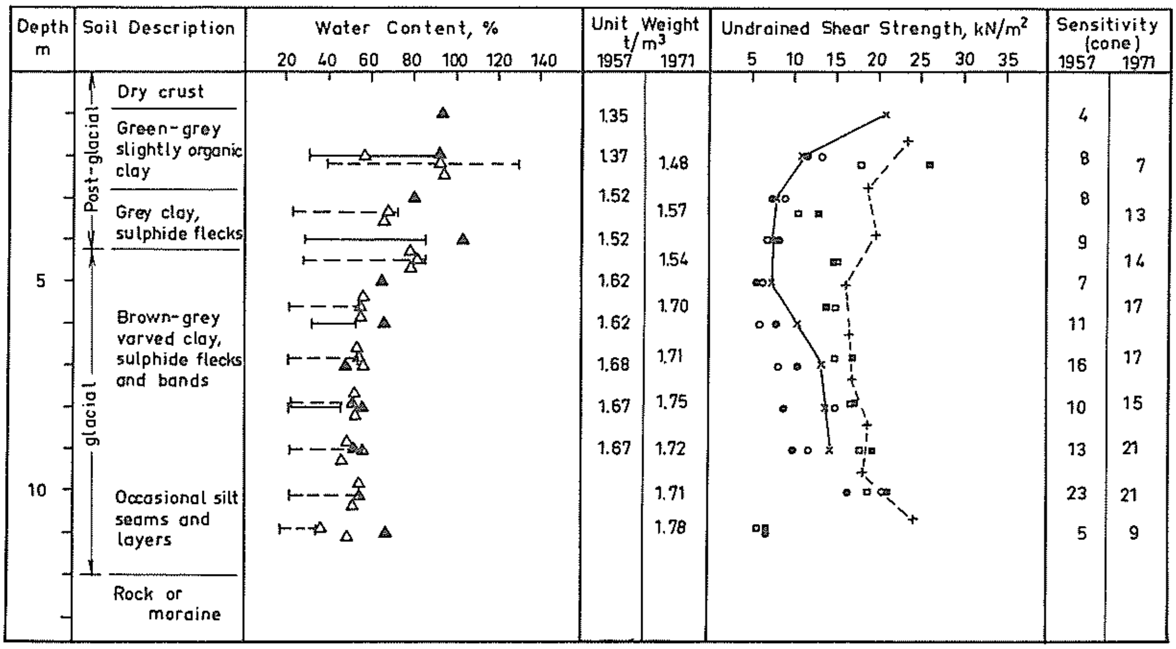

FIG. 5. GEOTECHNICAL PROFILES, AREA II; 1971 TESTS RESTORED TO 1957 ELEVATIONS. LEGEND ON FIG. 2.

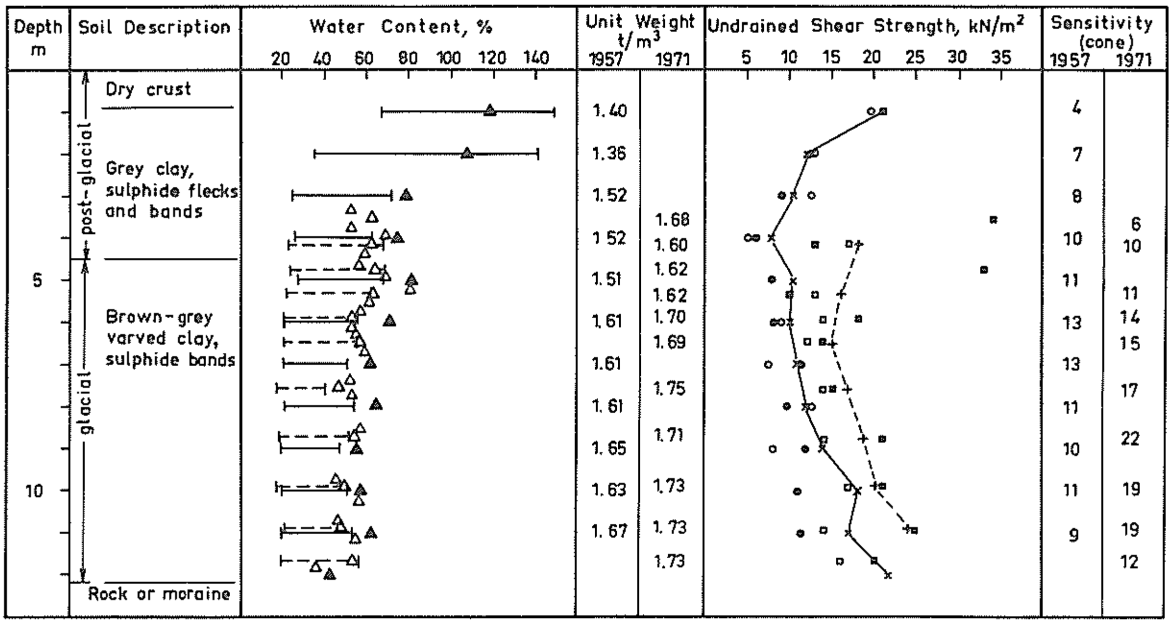

FIG. 6. GEOTECHNICAL PROFILES, AREA III; 1971 TESTS RESTORED TO 1957 ELEVATIONS. LEGEND ON FIG. 2.

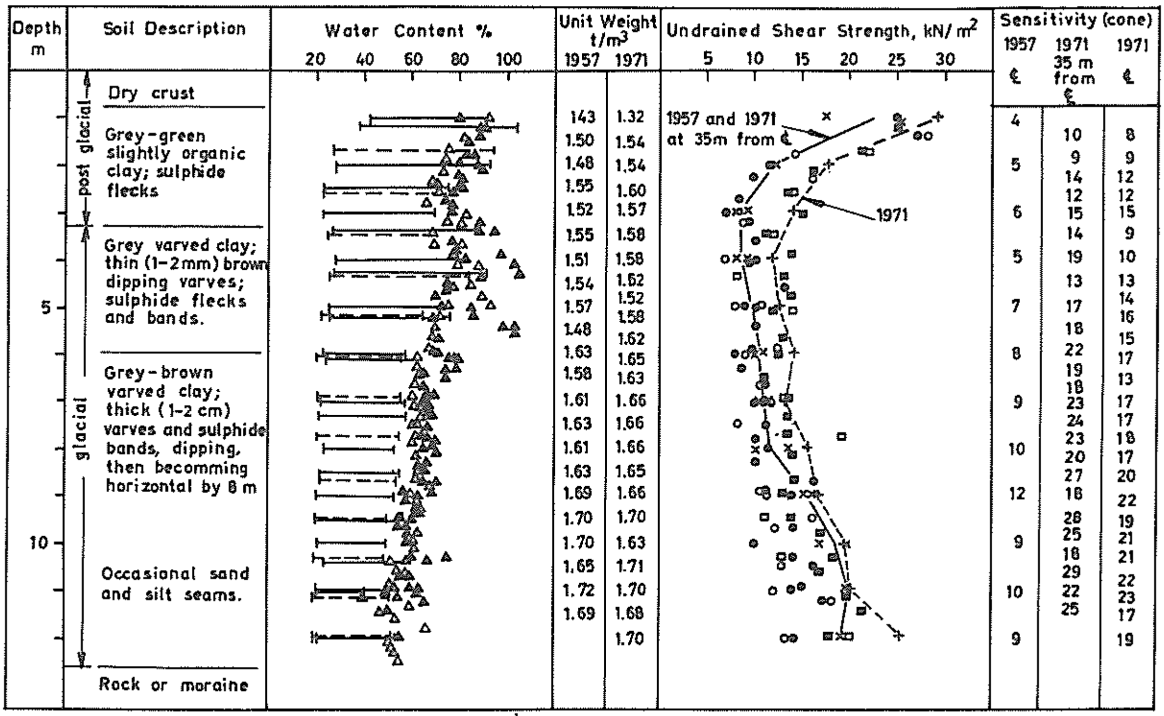

FIG. 7. GEOTECHNICAL PROFILES, AREA IV; 1971 TESTS RESTORED TO 1957 ELEVATIONS. LEGEND ON FIG. 2.

tral Sweden.

The clay is not perfectly homogeneous throughout the test field or below the test areas, and some generalizations are necessary in the interpretation of the test results. However average values from samples taken 35 m from the center of Area IV (Fig. 7) agree quite closely with the properties published by Hansbo (1960).

Average values from oedometer tests given by Hansbo (1960), Tables I and II, were verified in a limited testing program on samples taken outside Area IV. Because of sample disturbance, the determination of the preconsolidation pressure from the 1957 oedometer tests is difficult, but generally the results indicate a normally consolidated material. Oedometer tests on 50 mm standard piston samples from outside Area IV verified this conclusion, but scatter and the large increments used in the standard test made a precise determination difficult. An additional series of tests were conducted on samples taken with a 124 mm piston sampler between Areas I and II. Duplicate 50 mm and 80 mm diameter oedometer specimens were loaded with small increments, 2.5 to 5 kN/m^2, in order to accurately determine the preconsolidation pressure. The tests from the same depths agreed within 2 kN/m^2. Below about 5 m the clay was essentially normally consolidated assuming reasonable effective overburden stresses, and it did not exhibit any of the quasi-preconsolidation pressure found on some Norwegian marine clays (Bjerrum, 1967).

Test Areas.-- Originally, four circular tests fills were constructed, of which three were provided with sand drains. Fig. 1 shows the location of each area as well as pertinent dimensions, load magnitude and history, drain spacing, etc. Details of the installation of the sand drains are given by Hansbo (1960). Briefly, a 16 cm steel pipe, closed by an 18 cm diameter hinged lid, was driven to firm bottom by a pile driver. The pipe was filled with sand and then withdrawn. The drains were arranged in essentially a triangular pattern and more than 3,500 drains were installed with an average length of about 12 m.

Construction of the fills followed the following sequence. First, about 25 cm of the organic top soil was removed and a working platform of sand 50 to 70 cm thick was laid down. The drains were then installed, and two successive lifts of a clean sand-gravel fill were placed over each area. Final loads are shown in Fig. 1.

The factor of safety against bearing capacity failure was about 1.5 for the three areas loaded to 27 kN/m^2. For the heavily loaded area, Area III, the factor of safety was only slightly greater than one. Berms were thus used to prevent local slides (Figs. 1 and 10). Even in the areas with a high factor of safety, the theoretically calculated shear stresses below the edge of the loaded areas are quite high and are probably equal to or even slightly larger than the shear strength of the soil below the dry crust. Consequently, lateral yielding of the soil has undoubtedly occurred under the edges of the fills.

Instrumentation.-- Considerable instrumentation for the measurement of pore water pressures and settlements were installed under each test area as indicated in Figs. 8 through 11. In the drained areas, the piezometers were installed approximately halfway between the sand drains; also some settlement meters were placed in some drains below Area I (Fig. 8). The piezometers were the SGI hydraulic type (Kallstenius and Wallgren, 1956). The settlement gages were simple rods welded to earth screws (Kjellman, et.al., 1955). The settlement gages were read with a precise level from a bench mark on a nearby bedrock outcrop. Attempts were made to measure the horizontal movements both at the surface and at some depth, but after a few years, the instruments were abandoned because of obvious errors in the readings. Initial surface movements caused by the driving of the drains were, however, measured successfully (Hansbo, 1960).

RESULTS OF MEASUREMENTS OF PORE WATER PRESSURES AND VERTICAL SETTLEMENTS

The load, pore pressure, and settlement records are summarized in Figs. 12, 13, and 14 for the four test areas.

The net applied load decreased with time due to the settlement of the fill into the high water table. The indicated pore pressures were measured by piezometers located at the center of each area and at a depth of 5 m (Figs. 8 through 11). The indicated settlements correspond to the compression of the 5 m thick clay layer located between 2.5 m and 7.5 m below the ground surface. The behavior of this layer was analyzed by Hansbo (1960) because the clays above 2.5 m are undoubtedly overconsolidated due to dessication, and the depth to firm bottom varied throughout the test field from 9 m to almost 15 m (Fig. 1). Complete settlement and pore pressure records are available at the Swedish Geotechnical Institute for all the gages shown in Figs. 8 through 11.

The distribution of settlements under the fills was essentially consistent for all test areas. The settlements 10 m from the center were about the same as those at the center, while the gages located 2 to 3 m from the edge typically showed settlements about 30 to 40 percent of those at the center of the areas. The gages located outside the loaded areas at a distance of one diameter away from the center have settled at the most 3 to 4 cm.

The SGI piezometers showed considerable annual variation after a few years, and the scatter given in Figs. 12, 13, 14 for the piezometers located at 5 m depth are typical for almost all piezometers. Some gages are obviously malfunctioning today. However, it is interesting to note that the gages in Area IV, which showed a similar scatter (Fig. 14) on a year to year basis, all agreed within 0.2 m in piezo-

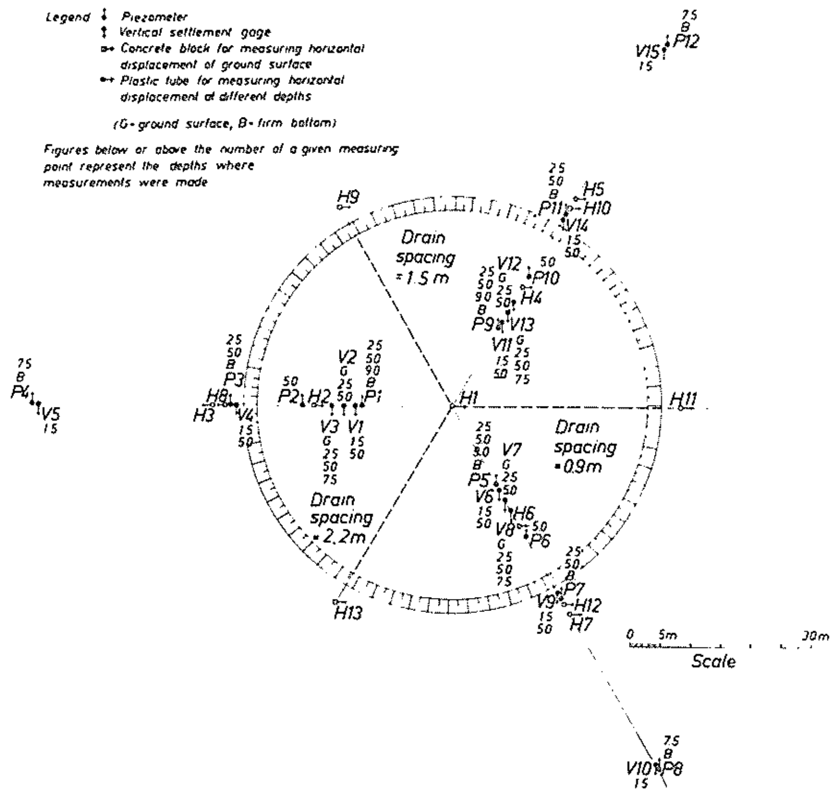

FIG. 8. AREA I: DRAIN SPACINGS, GAGE LOCATIONS AND DEPTHS BELOW GROUND SURFACE IN m. SETTLEMENT GAGES NOS. V2, V7, and V12 ARE PLACED IN SAND DRAINS. FROM HANSBO (1960).

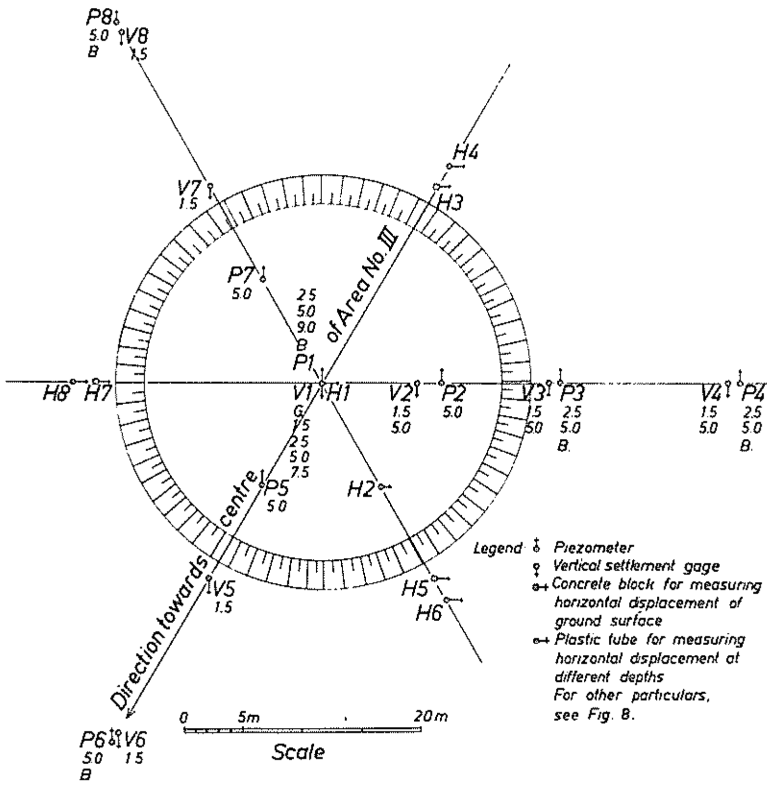

FIG. 9. AREA II: GAGE LOCATIONS AND DEPTHS BELOW GROUND SURFACE IN m. FROM HANSBO (1960).

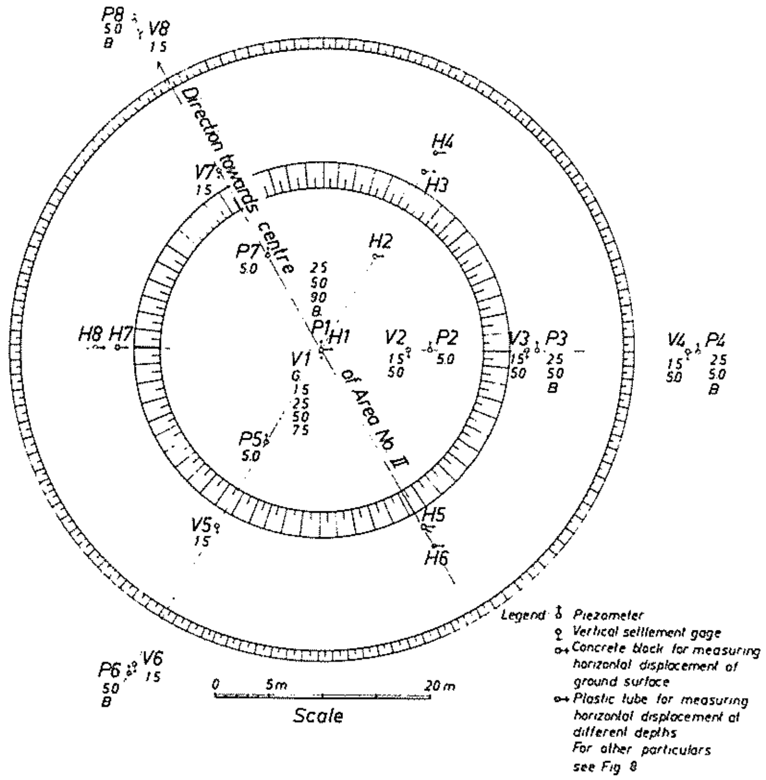

FIG. 10. AREA III: GAGE LOCATIONS AND DEPTHS BELOW GROUND SURFACE IN m. FROM HANSBO (1960).

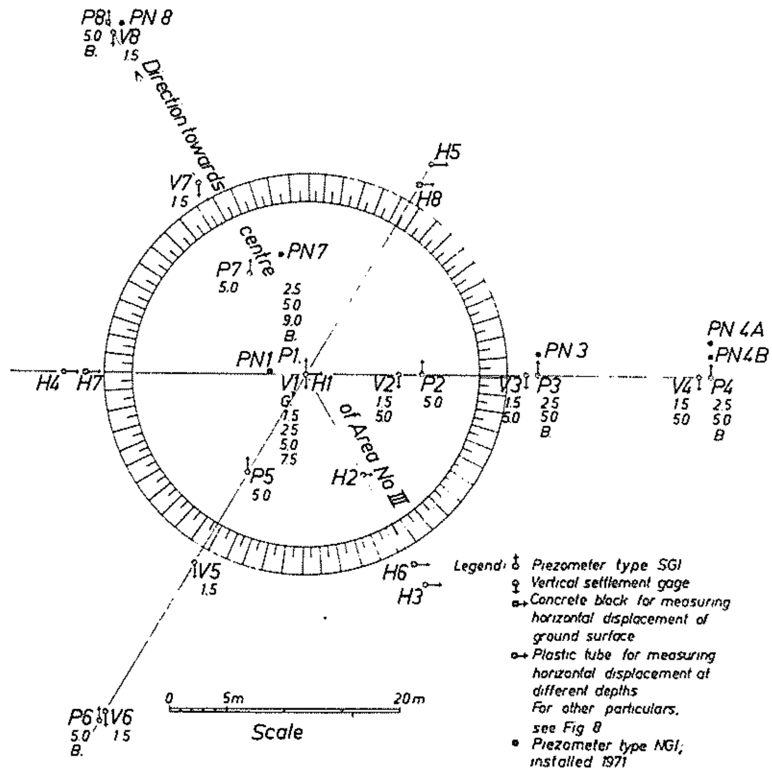

FIG. 11. AREA IV: GAGE LOCATIONS AND DEPTHS BELOW GROUND SURFACE IN m. FROM HANSBO (1960).

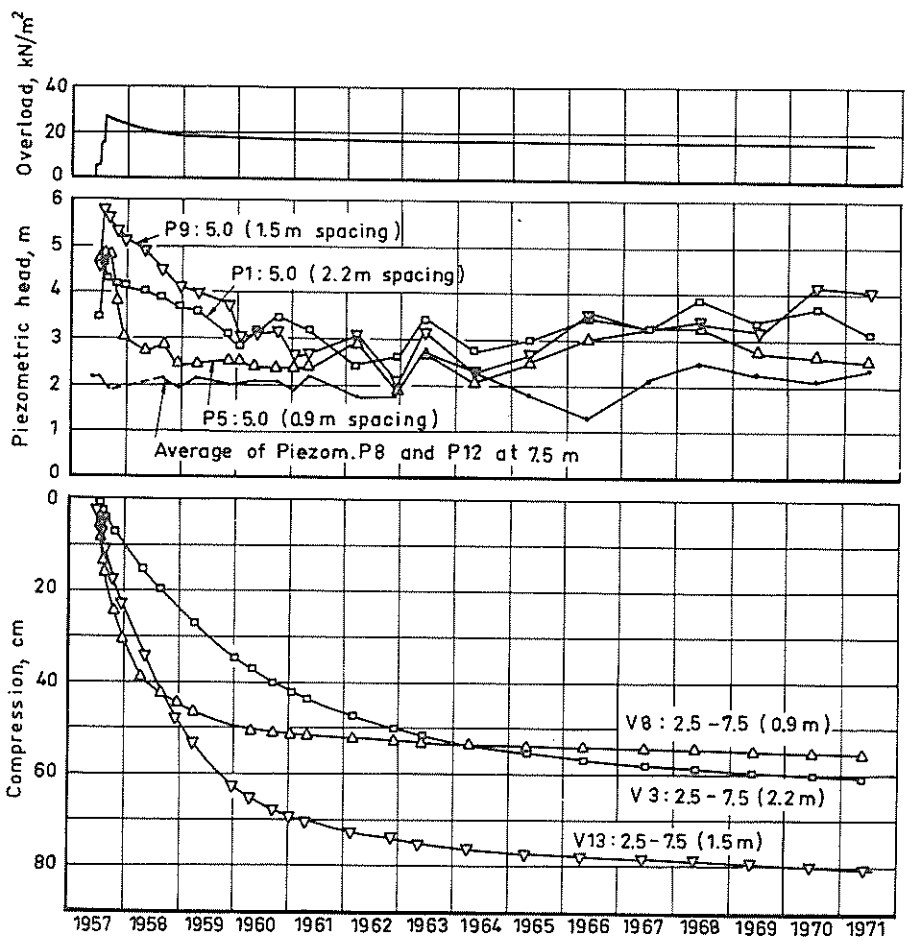

FIG. 12. OVERLOAD, PORE PRESSURE, AND SETTLEMENT HISTORY OF AREA I.

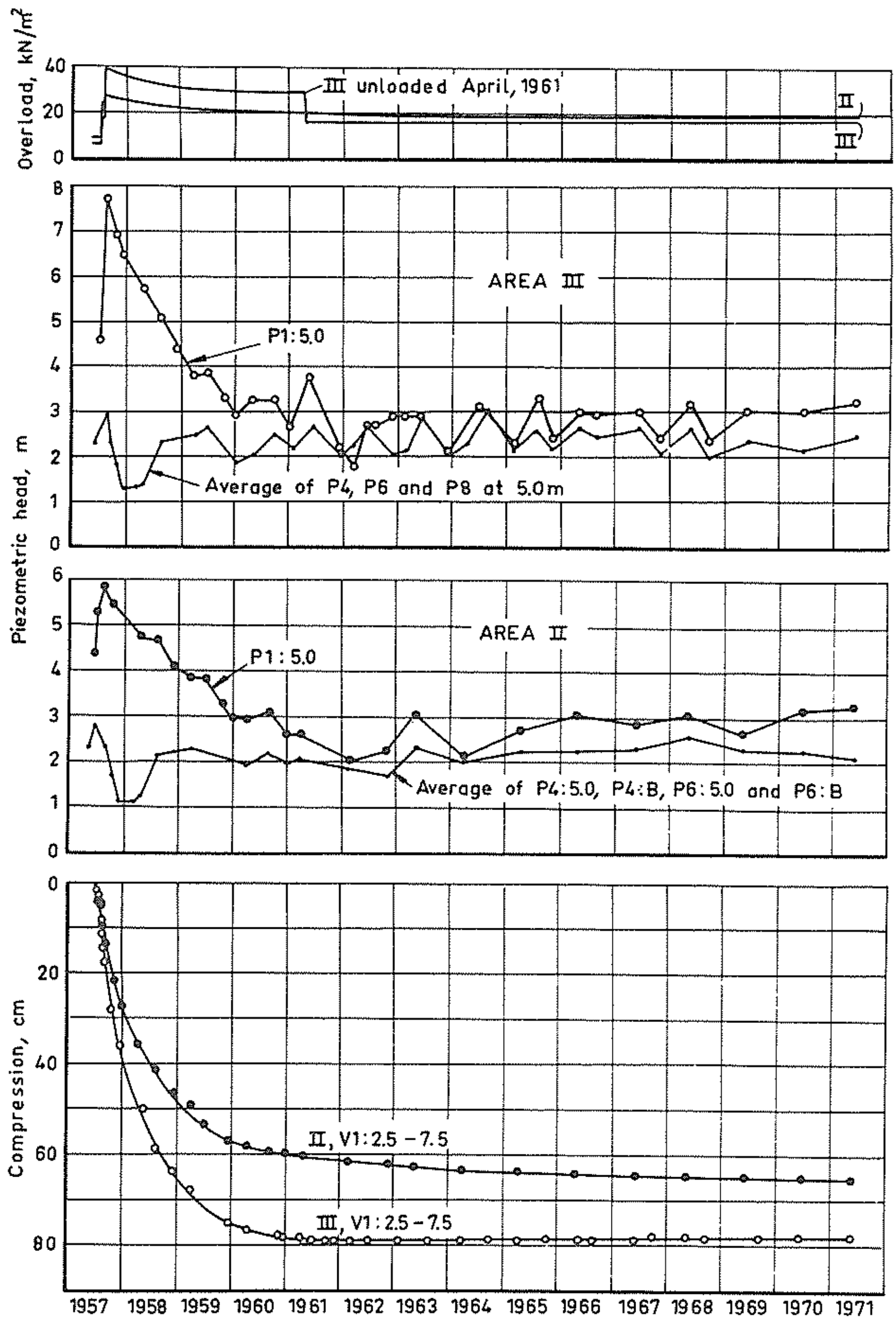

FIG. 13. OVERLOAD, PORE PRESSURE, AND SETTLEMENT HISTORIES OF AREAS II AND III.

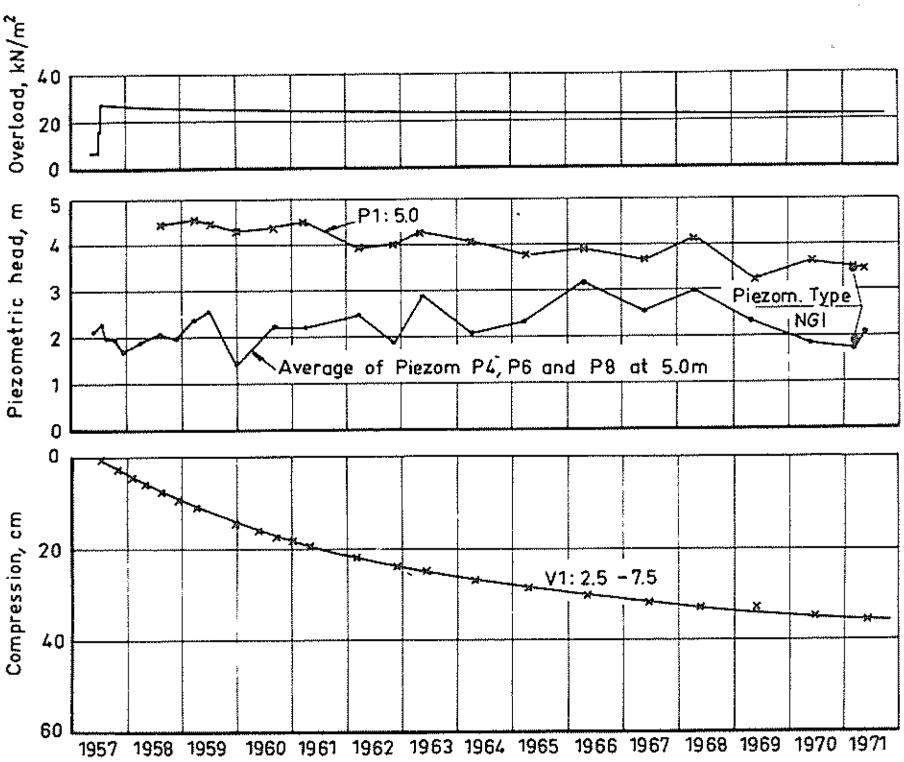

FIG. 14. OVERLOAD, PORE PRESSURE, AND SETTLEMENT HISTORY OF AREA IV.

metric head with measurements by the NGI vibrating wire piezometers in early 1971, as shown in Fig. 15. The water table in the natural ground as of March, 1971, was determined by three additional piezometers located 35 m from the center of the area (Fig. 11). The three agreed quite well with each other, and within 0.2 m of that level given by the nearby SGI piezometers. The observed difference of 0.2 m in head was less than the difference due to temperature sensitivity and atmospheric pressure changes between adjacent NGI piezometers which were left in place more than a few days.

DISCUSSION OF RESULTS

Ultimate Primary Settlement.-- It is interesting to compare the predictions of settlement made by Hansbo (1960) with observed settlements during the past 14 years. Table I is a summary of observed and calculated ultimate primary settlements for the investigated 5 m thick layer.

Hansbo based his calculations on an average value of the compression index as determined by oedometer tests on samples taken with an older model piston sampler. In addition, the analysis included an estimate of the settlement of the fills into the water table. The predictions were in error by about 10 percent as of 1971. It should be noted that the effective overburden stress, which depends on the elevation of the ground water table, has fluctuated considerably, both annually and throughout the years. Shown in Table I are recalculated values of ultimate primary compression using slightly different overburden pressures and location of the ground water table, as well as the actual settlement of the ground surface as of 1971.

Also included in Table I are predictions of ultimate primary settlement based on the actual dissipation of pore pressure as of the middle of 1959. Hansbo estimated the lateral movement of the clay from the pore pressure and settlement behavior as of 1959 of the undrained area and from his own measurements of surface movements subsequent to loading. The estimated movements and the dissipation rates were admittedly crude, but the calculated ultimate settlements agreed reasonably well with the values predicted from oedometer tests.

How well do these predictions check with the observed settlements at the end of primary consolidation phase? In general, the agreement is quite good. The only exception is the sector in Area I with the 0.9 m drain spacing where either the laboratory data are significantly in error or there is some stiffening effect of the drains, which tends to reduce the actual settlements. The 0.9 m sector of Area I is the only one where the modified settlement computations of Table I more closely predict the actual settlement.

It is interesting that the average settlement calculated by the two methods predict exactly the observed settlement

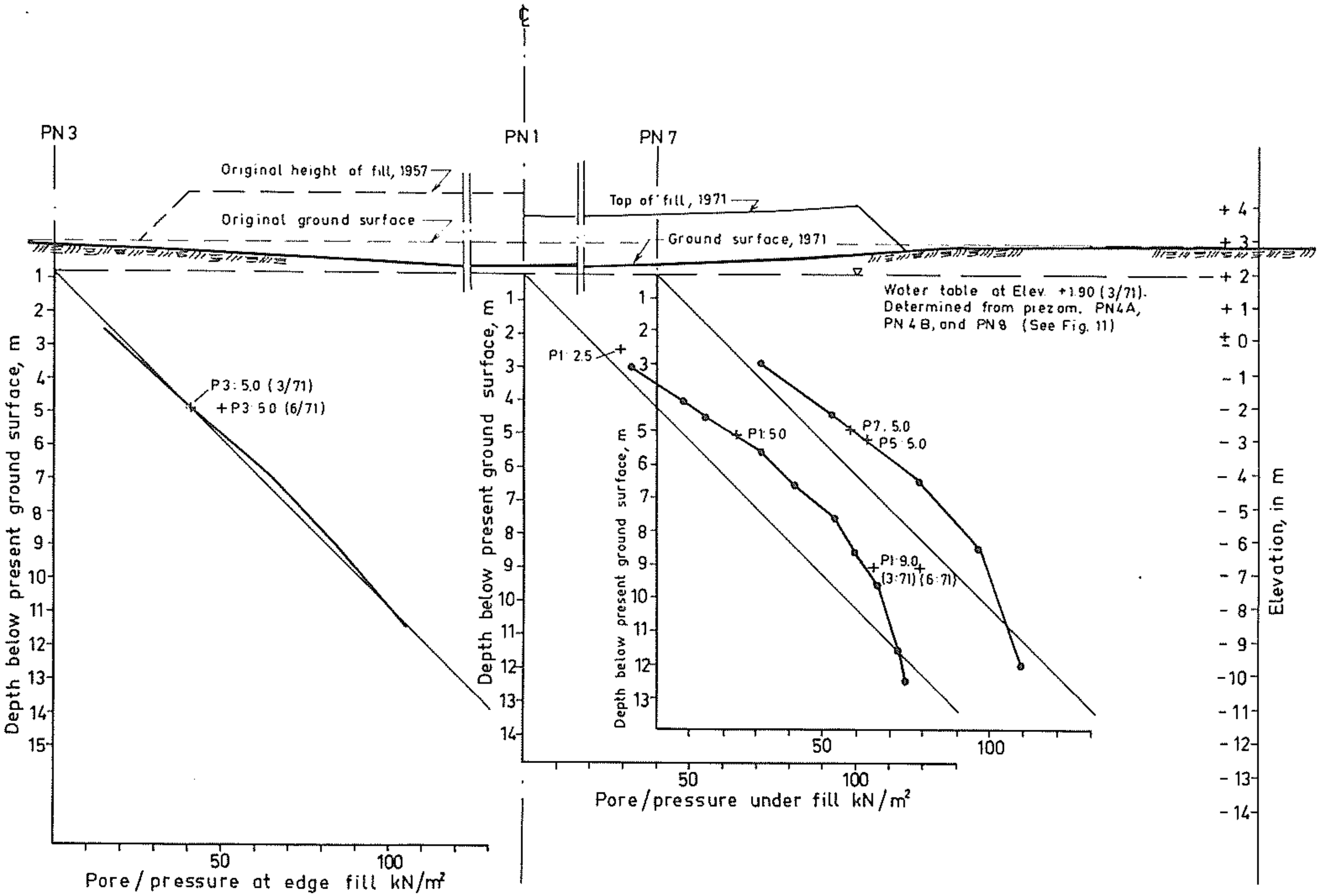

FIG. 15. PORE PRESSURE UNDER AREA IV FROM PIEZOMETERS TYPE NGI, FEB-MAR, 1971.

TABLE I

OBSERVED AND PREDICTED PRIMARY SETTLEMENT OF THE 5 m CLAY LAYER BETWEEN 2.5 AND 7.5 m

		Estimated average C_c from oedometer tests (Hansbo, 1960)	Predicted Settlement, cm				Observed settlement cm	
			From Oedometer tests		From Pore pressure Dissipation as of 1959			
Area	Drain Spacing	C_c	Hansbo[1] (1960)	Slightly[2] modified	No Outflow	With Outflow	End of primary Consolidation	Total after 14 Years
I	0.9 m	1.85	63	54	48	46	50(1960)	56
I	1.5	1.95	70	61	78	78	74(1962)	80
I	2.2	1.50	50	46	51	50	59(1969)[3]	61
II	1.5	1.45-1.55	54-57	50-53	63	60	61(1962)	65
III	1.5	1.25-1.45	65-75	58-67	81	75	78(1961- unloaded)	78
IV	no drains	1.30-1.50	47-54	41-48	-	-	Still in Primary	36

1 Using Hansbo's assumptions of ultimate settlement of ground surface, $\gamma_t = 1.5\ t/m^2$ at 5 m depth and an average location of the ground water table at 25 cm below the ground surface.

2 Using observed settlement of ground surface in 1971, $\gamma_t = 1.55\ t/m^2$, and an average location of the ground water table at 50 cm below ground surface.

3 Pore pressure data inconclusive for this sector (Fig. 12). End of primary consolidation assumed when the settlement rate became approximately equal to the rates of secondary for the other two sections of Area I.

of the 1.5 m sector of Area I. This is probably a coincidence, since the settlement measurements for the settlement gage located at 7.5 m depth are questionable. The gage indicated unusually small settlements in comparison with other gages placed at the same depth. Hansbo (1960) attributed this effect to a possible local rise of the bedrock surface of perhaps the presence of a glacial erratic block. Both conditions have been found at other localities in the test field. The small measured settlements resulted in a large compression of the 5 m thick layer (Fig. 12). Yet, had the compression of the 5 m layer been significantly less, then the predictions would have been way off.

The agreement for the 2.2 m sector of Area I is not so good, but the determination of the primary settlement is far from exact. If the highest predicted values from the oedometer test are used the differences are only 3 to 4 cm for Areas II and III, and the prediction based on dissipation of excess pore pressure is excellent.

It can be concluded that the original predictions by Hansbo (1960) are reasonably correct, within about 5 to 20 percent of measured values in most areas. The evidence herein suggests that the remolding of the clays due to the drain driving has not increased the total settlement. The computations of the ultimate primary settlement were made in the usual manner with no allowance for remolding of the clay. Also, the performance of the undrained area so far tends to verify this conclusion, as described later.

Rate of Consolidation from Dissipation of Excess Pore Pressure.-- Using the pore pressure observations in the drained areas (Figs. 12 and 13), the degree of consolidation, or consolidation ratio, U_h, at a given time and a given elevation can be calculated from

$$U_h = 100 \left(1 - \frac{u}{u_{io}}\right) \% \qquad (1)$$

where u_{io} is the initial excess pore pressure after completion of loading. The results, shown as points for the average of the piezometers at 5 m depths, are given in Fig. 16. Not all the points followed smooth curves because of the piezometer fluctuations discussed previously.

The first item of interest is the calculation of the parameter n, which Hansbo (1960) used to describe the deviation from Darcy's Law. He found experimentally for the Skå-Edeby clay, that n was approximately equal to 1.5. From his consolidation theory which considers the non-linearity of flow, he developed certain relationships which allow the computation of the parameter n from the field data. He found from the field pore pressure dissipation as of 1959 that the average value of n was about 1.5 to 1.6. A comparison was made of the times for equal degrees of consolidation between Areas II and III. These two areas had the same drain spacing but the initial applied load was different. Hansbo's theory predicted that the rate of consolidation should decrease with increasing applied load and thus with increasing initial excess pore pressures. When the same calculations for n were made from the data after 1959, it was found that n increased to about 1.8 when the degree of consolidation reached 80 to 90 percent. Calculations were made to determine the effect of a higher value of n on the theoretical time curves derived from Hansbo's theory, and it was found that the curves using a constant n = 1.5 appeared to fit the observed pore pressure dissipation rates somewhat better than if n = 1.7 or 1.8 was used for higher degrees of consolidation. Thus, in the following theoretical work a value of 1.5 is used.

From equations derived by Hansbo (1960), and from classical consolidation theory applied to vertical drain wells (Barron, 1948; Kjellman, 1948, 1949), the theoretical U-t relationships can be calculated for different values of the coefficients of consolidation. It should be noted that the degree of consolidation in this case is the average consolidation for the 5 m layer considered. These curves are shown in Fig. 16, and the table in the figure indicates the values of c_h and λ utilized in the calculations. These coefficients, which differ slightly from those originally given by Hansbo, correspond better to the long-term response of the clays under the loaded areas. The consolidation coefficients were determined from the observed pore pressure dissipation times at several values of $\bar{U}$. Then a reasonable average was chosen for the calculation of the curves shown in Fig. 16. For the λ-theory, a constant value of n = 1.5 was used for reasons described previously. The differences between the classical and Hansbo's theories are not large but the latter theory appears to fit the observed pore pressure dissipation rates somewhat better. Of course, additional corrections of c_h and λ could be made to achieve a slightly better fit, but such adjustments are not warrented considering the variations of the pore pressure measurements. Comparison will be made later between laboratory and field values of the consolidation coefficients.

Rates of Consolidation from Settlement Rates.-- The average rate of consolidation can also be determined from

$$\bar{U} = \frac{\delta}{\delta_p} \tag{2}$$

where δ is the settlement at a given time and δ_p is the ultimate primary consolidation as determined by either oedometer tests, or in this case, from observed settlements when the excess pore pressures in the field are completely dissipated (Table I). The results are given in Fig. 17. The curves are slightly different from those in Fig. 16 based on the pore pressure dissipation. For the 0.9 m sector in Area I the curves are almost identical; they also agree quite well in the 2.2 m sector of Area I until $\bar{U}$ = 65 percent. After this, the pore pressure measurements are unreliable. In the other areas, the rates from the settlement observations were always greater than the rates determined from the pore pressure dissipation. The difference was evident but not large. There are many assumptions involved in

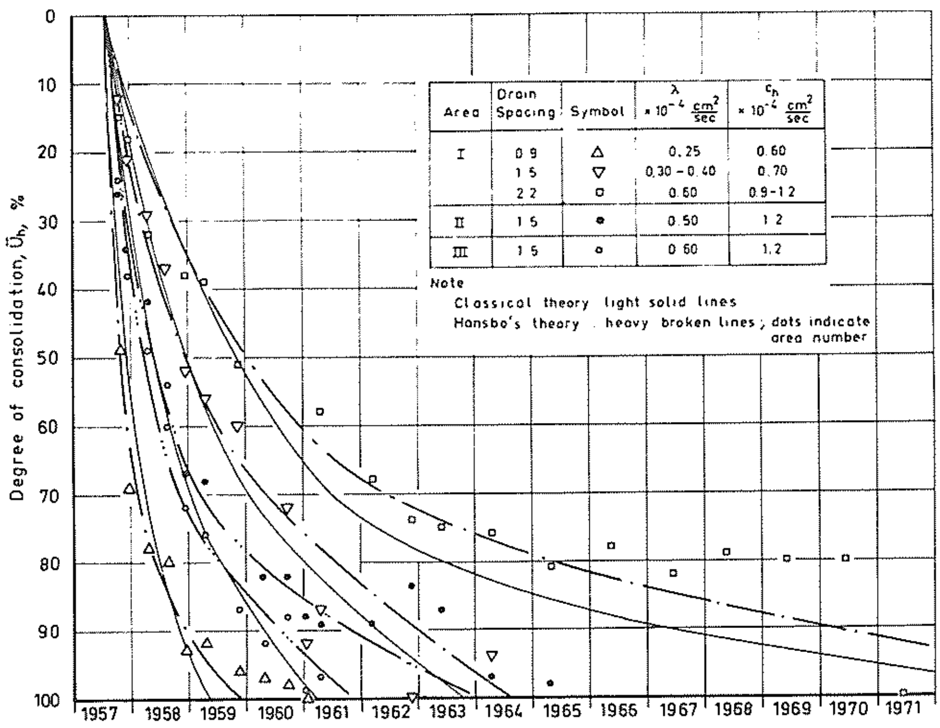

FIG. 16. THEORETICAL TIME-CONSOLIDATION CURVES COMPARED WITH OBSERVED DEGREES OF CONSOLIDATION BASED ON PORE PRESSURE OBSERVATIONS.

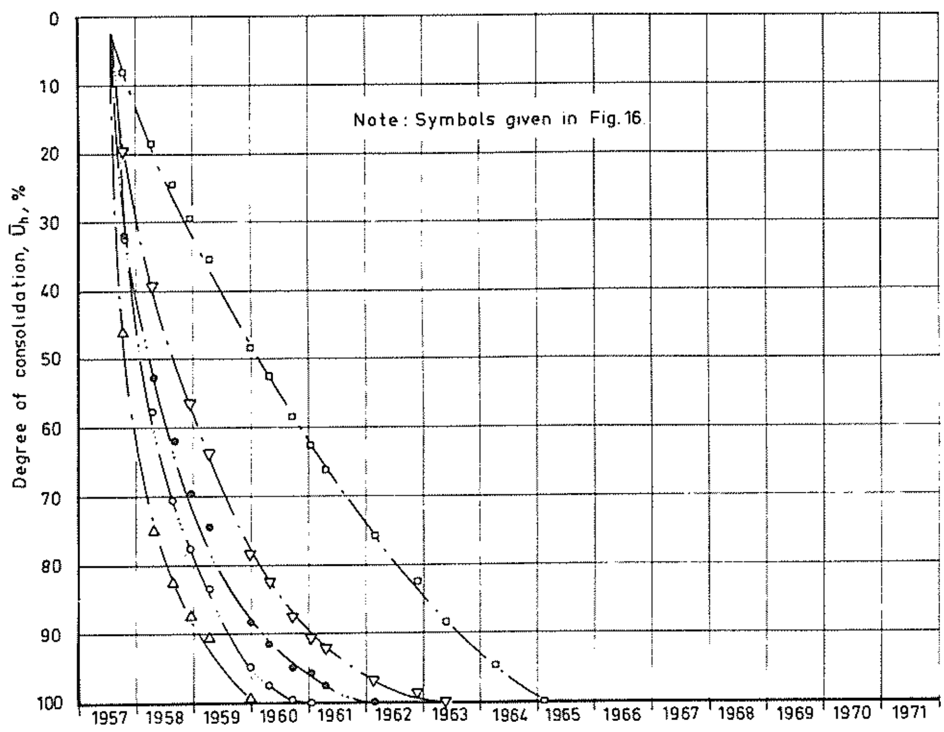

FIG. 17. TIME-CONSOLIDATION CURVES BASED ON OBSERVED RATES OF SETTLEMENT.

the settlement calculations that it may be fortuitous that the agreement is so good.

The calculation of the curves in Fig. 17 followed Hansbo (1960) wherein the total consolidation ratio or percent consolidation was obtained from the equation (Carrillo,1942)

$$\bar{U} = \bar{U}_h + \bar{U}_v - \bar{U}_h\bar{U}_v \qquad (3)$$

where the subscripts refer to the direction of consolidation. The rates of average vertical consolidation, $\bar{U}_v$, were determined from the settlement response of Area IV using Eq. 2 and the average ultimate primary consolidation predicted from the oedometer test (Table I). The settlement of this area was assumed to be due to vertical consolidation, and lateral displacement of the clay was not considered.

As shown by Hansbo (1960), the effect of this assumption on $\bar{U}_v$ was not large up to the middle of 1959, but after that time the effect was appreciable on the behavior of Area IV. Since all the areas except the 2.2m sector of Area I essentially were fully consolidated by 1962, the effect of lateral movement on the consolidation rates in Fig. 17 is not considered significant. Also, the water content changes, presented later, suggests that the effect of lateral displacement on the vertical settlement is not as large as originally thought.

Analyses of the settlement rates indicate that the area with the closest drain spacing had the highest settlement rate and thus achieved 100 percent primary consolidation the fastest. The second highest rates were observed in Area III, the area with an initial overload. In addition, these curves appear to agree somewhat better with the theoretical consolidation curves from the λ-theory (Fig. 16). This theory also predicted that the rate of consolidation increases with increasing load, observed in Area III in contrast to Area II in contradiction to the classical theory. At least, for the Skå-Edeby clays, the remolding due to the close drain spacing does not appear to have had a detrimental effect on the settlement rate. Average response of the 1.5 m sector in Area I and of Area II is about midway between the curves for the 0.9 m and 2.2 m spacing of the drains for Area I. Drains are commonly installed in Sweden with a spacing less than 1.5 m, and the results so far at Skå-Edeby support the generally favorable results achieved with sand drains. The effect of drain spacing on secondary compression rates will be discussed later.

Settlement Response of the Undrained Area.-- It is interesting to analyze the behavior of the undrained Area IV in terms of the two types of analyses just presented. The scatter in the piezometer readings, even when averages of several piezometers at the same depth were used, was so large that a presentation of results similar to Fig. 16 was considered meaningless. From the measurement program in March 1971 (Fig. 15), however, some conclusions can be drawn as to the degree of consolidation at that time. Using the initial excess pore pressures (Hansbo 1960), the degree of consolidation $\bar{U}_v$, from Eq. 2 was found to be only 35 percent at both 5 m and 9 m depth. If the settlements (Fig. 14) and Hansbo's predicted values of the ultimate primary consolidation (Table I) are used in Eq. 2, the calculated degree of consolidation is 67 to 77 percent or an average of 72 percent. This value is significantly higher than that indicated by the measured pore pressures.

This discrepancy is strange, especially considering the rather normal, almost classical behavior of the drained areas. Furthermore, as will be described later, the observed vertical settlement at the center of the area corresponds almost exactly to the settlement calculated from the changes in water content between 1957 and 1971. This suggests that the observed settlement at least at the center of the area should correspond to the classical case.

One explanation for this apparently anomalous behavior is that the so called "secondary" compression has been going on during the "primary" consolidation phase, as first was suggested by Bjerrum (1967). Chang (1972) has observed similar, but more extreme, behavior at the Väsby test field north of Stockholm. The division into a "primary" and a "secondary" phase is rather artificial, and the separation is merely a convenient explanation of the discrepancies from the classical consolidation theory. In most clays the amount of secondary consolidation which occurs during the primary phase is small and can safely be neglected. This is not the case at Skå-Edeby or at Väsby. There are, however, some additional aspects at Skå-Edeby with respect to the Bjerrum-Chang idea. Because of the ongoing secondary consolidation, an increase of the apparent preconsolidation stress to values larger than present or past overburden pressures will occur,according to the hypothesis proposed by Bjerrum. Such an increase has been observed for some Norwegian clays (Bjerrum, 1967). No such quasi-preconsolidation was found at Skå-Edeby even with careful sampling and testing. The material appeared to be essentially normally consolidated. One explanation could be the youth of the clays at Skå-Edeby. They emerged about 500 years ago above the sea level,as mentioned previously. The present rate of uplift is about 5 mm/year. Further research is necessary to clarify the importance of quasi-preconsolidation for these clays.

It was suggested earlier that the ultimate primary settlement was not increased by the remolding due to the driving of the drains, and it is interesting to compare the response of the drained areas with what may be the ultimate settlement of Area IV. The remaining pore pressure indicates that the ultimate primary settlement of Area IV will exceed 100 cm, or twice the settlement predicted from the oedometer test results (Table I). Thus, there is no evidence from either predictions from the oedometer test or from the measured excess pore pressure that driving of the drains has increased the ultimate primary settlement.

Laboratory and Field Determination of the Coefficients of Consolidation.-- Theoretical values of the coefficients of consolidation, λ and c_h, were calculated from the best fit curves to the pore pressure dissipation data (Fig. 16). From the classical theory, c_v can be calculated from the measured degree of consolidation for Area IV. The calculated values of the coefficients are compared with the results from oedometer tests (Hansbo, 1960) in Table II. First, it will be noted that the values of c_h are about four times larger than the c_v-values probably due to the sedimentary and varved nature of the clay. Yet, the field values of c_h are about half the laboratory c_h-values, and not much larger than the laboratory c_v-values, which verifies an observation by S. Johnson (Riordan, et.al., 1971). This difference must be due to remolding of the clays during the driving of the sand drains. The calculated c_v-values differed for the undrained area, because of the difference in degree of consolidation from the two approaches mentioned above. The value based on the settlement rates is not far from the laboratory c_h-value, which suggests that horizontal consolidation was predominant in Area IV. Yet, on the other hand, the value of c_v from the remaining excess pore pressure is very close to that determined by oedometer tests. Thus, the c_v-value determined from oedometer tests, quite accurately predicts the excess pore pressure which remained in the clay as of 1971. But the rates of settlement of the undrained area, as discussed previously, are apparently poorly predicted from oedometer tests, even though the test apparently is satisfactory for the drained areas.

Settlements in Drains vs. Settlements Near Drains.-- In order to study the distribution of settlements in and around the drains, settlement gages were installed at three depths within some drains in all the sectors of Area I (Fig. 8). Table III presents the total settlements as of 1971 of those gages and the adjacent gages in the same sector. It can be seen that there is no consistent pattern of the settlements which indicates that the drains essentially have settled along with the surrounding clay.

TABLE III

SETTLEMENTS, cm, IN AND AROUND THE DRAINS IN AREA I, AS OF 1971

Depth of gages, m	Drain Spacing, m								
	0.9			1.5			2.2		
	V6	V7[1]	V8	V11	V12[1]	V13	V1	V2[1]	V3
Ground surface		124	130		107	109		112	115
1.5	122			108			111		
2.5		125	117		97	94		103	103
5.0	82	82	80	54	60	62	72	71	76
7.5			61			13			43

[1] Gage located in drains

TABLE II

COMPARISON OF LABORATORY AND FIELD COEFFICIENTS OF CONSOLIDATION

$\times 10^{-4}$ cm^2/sec

Area	Drain Spacing	c_v Minimum value from oedometer test, Hansbo (1960)	c_h From special oedometer test, Hansbo (1960)	λ Fig. 16	c_h Fig. 16	c_v From excess pore pressure 3/71	c_v Average from settlement measurements
I	0.9 m	0.7	2.0-2.2	0.25	0.6		
I	1.5	0.5	(Sample	0.3-0.4	0.7	-	-
I	2.2	0.5	from between Areas	0.6	0.9-1.2		
II	1.5	0.4-0.7	I and II)	0.5	1.2	-	-
III	1.5	0.4-0.6		0.6	1.2	-	-
IV	No drains	0.4-0.5		-	-	0.7	3.0

Note: Laboratory c_v-values were determined by the $\sqrt{t}$-fitting method.

Secondary Compression.-- As indicated in Table I, the magnitude of the secondary compression in the drained areas has been small since the complete dissipation of the excess pore pressures. Table IV gives average rates of secondary compression for all settlement gages in the drained areas. Several important observations can be made.

From Table IVa it can be seen that there is a significant difference in rates of secondary compression between Areas I and II where the drain spacing is the same. The difference is likely due to the increased disturbance of Area I over Area II during the driving of the drains. The records from the driving indicate that the 1.5 m sector of Area I was the last to be completed after all drains had been installed in the other sectors. It is possible that the driving of drains in the 0.9 m sector could have affected the conditions in the adjacent 1.5 m sector. Measurements by Hansbo (1960) indicate that the heave outside Area II due to the drain driving was much less than for Area I, although the displaced volume of clay was less for Area II than for Area I. It is interesting to note, too, that for Area II, the rates of secondary observed for the gages located 10 m from the center agree better with rates from Area I for comparable depths. When settlement rates from Area III, which was unloaded, are compared with the other areas, it can be seen that the secondary compression was definitely reduced by the unloading. And further, the rates for this unloaded area have been relatively constant for the past seven years (1964-71).

From Table IVb, it can be seen that the rate of secondary compression is definitely less for the small drain spacing, a somewhat expected result. It was thought that the secondary compression will increase with increasing disturbance due to the decreasing drain spacing. This comparison may be questioned because the rate of secondary compression could not clearly be established for the 2.2 m spacing sector in Area I (rates in Table IVb are averages for the past three years), and the sector only recently has reached 100 percent primary consolidation. Further, if the rates of Area II with 1.5 m spacing of the drains are compared, then the sector with the closer drain spacing did indeed have the greater rates, probably due to disturbance.

The rather small amounts and rates of secondary compression in the drained areas suggest that the reduction of the excess pore pressures reduces also the secondary settlement, both the part that goes on during the primary (e.g. Area IV) and the part after completion of primary compression.

The field rates have also been compared with rates of secondary compression from conventional 24-hr oedometer tests. The laboratory rates were on the average about 25 times faster than the field rates at comparable depths. One, therefore, must be very careful about predicting field rates of secondary compression from conventional oedometer tests.

TABLE IV

AVERAGE RATES OF SECONDARY COMPRESSION IN mm/yr AFTER THE END OF PRIMARY CONSOLIDATION

a. Constant Drain Spacing (1.5 m)

Depth m of settlement gage	Same Load, 27 kN/m^2			Higher Initial Load, 39 kN/m^2; then unloaded to 27 kN/m^2 after 4 years
	Area I	Area II		
	Center	Center	10 m from Center	Area III after unloading
Ground surface	7.8	3.3		1.8, 3.0[2]
1.0	5.9[1]	3.2[1]		
1.5	7.9	3.3	6.5	0.8
2.5	6.0	2.8, 4.2[1]		4.0 (61-65) 0.0 (66-71)
5.0	3.3, 4.9	2.0		1.4
7.5	0.7	1.0, 0.1[1]	4.7	1.6, 1.0[1]
2.5-7.5	4.7	2.6	-	-0.8 (rebound)

[1] New settlement gages; one year's measurements.

[2] Gage installed in 1961 when area was unloaded; average rate has been constant for six years.

b. Variable Drain Spacing - Area I

Depth m of settlement gage	Drain Spacing		
	0.9 m	1.5 m	2.2 m
Ground surface	4.8	7.8	12.8
1.5	4.8	7.9	12.0
2.5	4.9	6.0	10.7
5.0 (average of two gages)	2.9	4.1	6.2
7.5	2.6	0.7	3.2
2.5-7.5	3.8	4.7	5.8

Change in soil properties due to consolidation.-- Shown on the geotechnical profiles (Figs. 2 to 7) are the results of a recent sampling and testing program wherein the soil properties were determined on samples taken from under the loaded areas. The test results indicate a marked decrease (5 to 20 percent) in water content and a significant increase in undrained shear strength (about 5 kN/m^2) for all areas. Changes of the Atterberg limits noted by Hansbo (1960) were not clearly established, although there is a tendency for the liquid limit to decrease in some areas after consolidation. Definite changes of the sensitivity as measured by the Swedish fall cone test could not be determined, because the sensitivity of samples taken from outside the loaded area (Fig. 7) was higher than that reported by Hansbo (1960), even though there were no significant differences in all the other classification properties. Consequently, the sensitivities reported by Hansbo and shown in Figs. 2 to 6 are subject to question. If the values from the 1971 testing program (Fig. 7) are compared, then there apparently is a slight decrease in sensitivity due to the consolidation. Chang (1972) observed a similar decrease in the sensitivities from vane tests at the Väsby test field near Stockholm.

Table V gives the changes of the ratio s_u/σ'_{vo} due to consolidation. The calculations are based on a water table located about 0.5 m below the ground surface, an average unit weight of the clay, and the average vane strengths of the clay from 5 to 10 m below the ground surface. Above 5 m, the influence of surface drying (dry crust) can be seen in the strength-depth relationship. Effective overburden pressures were used in the computations because the material is essentially normally consolidated as described previously. For the ratios calculated for 1971, the stress at depth was taken as the overload less the decrease in load due to settlement of the fill into the water table. For Area III, the second figure in Table V is based on the lower overload after unloading in 1961 (Fig. 13).

From Table V, the increase in strength relative to the increase in effective stress at depth was less than the initial s_u/σ'_{vo}-ratios, and thus the ratios in 1971 are generally somewhat less than the ratios before consolidation. This implies that further changes in water content, for example, due to secondary compression may cause a further increase of s_u/σ'_{vo}.

Vertical settlements and changes in water content.-- Previous attempts at Skå-Edeby to relate the observed changes in water content to vertical settlement have been somewhat unsuccessful due to scatter in the test data (Hansbo, 1960; Osterman and Lindskog, 1963). However, Chang (1972) showed that the vertical settlement at Väsby test field after 23 years of loading could be predicted within about 15 percent from the changes in water content. A similar study was thus undertaken at Skå-Edeby. The soil profile was divided into convenient layers of initial thickness H_o, and the change in thickness or settlement, ΔH, of

TABLE V

CHANGE IN s_u/σ'_{vo} DUE TO CONSOLIDATION FROM 1957 TO 1971

Area	Drain Spacing	Surface Load	s_u/σ'_{vo} 1957	$\Delta s_u/\Delta\sigma'_{vo}$ 1957-1971	s_u/σ'_{vo} 1971
I	0.9 m	27 kN/m^2	0.22	0.21	0.21
I	1.5	27	0.28	0.13	0.22
I	2.2	27	0.24	0.19	0.22
II	1.5	27	0.27	0.21	0.25
III	1.5	39/27	0.26	0.15/0.20	0.21/0.24
IV	-	27	0.28	0.13	0.23

each layer was calculated by

$$\Delta H = \frac{G_s \Delta w\ H_o}{1 + G_s w_o} \tag{4}$$

where G_s is the specific gravity of the solids, and w_o is the initial natural water content prior to loading. The average changes in water content, Δw, were estimated from Figs. 2 to 7. The calculated total surface settlements are given in Table VI. Also given for Area II is a calculated settlement which is within about 10 percent of the measured value based on water contents determined just after loading, but which were not given by Hansbo (1960) nor are shown in Fig. 5.

It can be seen that the water content changes overestimate the settlements by about 25 to 30 percent in all sectors of Area I, while they underestimate by about the same amount in Areas II and III. The reason for the discrepancy is probably the rather large natural variation in water content and the lack of detailed water content information in 1957. The good agreement between calculated and observed settlements for Area IV is because the comparison was made with a detailed water content profile made outside the area in 1971 (Fig. 7); these average values agreed reasonably well with those given by Hansbo (1960).

For the undrained Area IV, it was assumed that rather large lateral movements in the clay contributed to the vertical settlement, especially at the edge of the fill. This contribution can be determined by deducting the theoretical settlement due to the change in water content from the observed settlement at the edge of the fill. The changes in water content gave an unexpected settlement of 84 cm below the edge of the fill while the interpolated measured settlements were only about 50 cm. (There were no settlement gages at the edge of the fill). Apparently, lateral or ho-

TABLE VI

SURFACE SETTLEMENT OBSERVED AND CALCULATED FROM THE OBSERVED CHANGES IN WATER CONTENT

Area	Drain Spacing	Settlements, cm	
		Observed	Calculated
I	0.9 m	130	161
I	1.5	109	143
I	2.2	115	153
II	1.5	113	84-102[1]
III	1.5	152	123
IV	none	72	74

[1] See text.

rizontal consolidation has occurred below the edge of the fill, which has caused a significant reduction in water content. Initial measurements by Hansbo (1960) indicated signigicant lateral movements, and later measurements at Skå-Edeby on an embankment (Osterman and Lindskog, 1963; Holtz and Lindskog, 1972) verified that relatively large lateral movements indeed do occur under the edges of fills and embankments. Because of the large rotation of principal stresses which occurs at the edge of the fill, relatively high lateral stresses exist which cause lateral consolidation of the soil and a decrease of the water content.

CONCLUSIONS

Based on the long-term performance of the test fills at Skå-Edeby, the following conclusions are drawn:

1. Hansbo's (1960) predictions for final primary consolidation were within 5 to 20 percent of the observed final primary compression in the drained areas.

2. Remolding of the clay due to driving of the drains did not appreciably increase the ultimate primary compression or had any detrimental effects on the primary settlement rates. The value of the field coefficient of horizontal consolidation, c_h, was about the same as or only slightly greater than the c_v determined from conventional oedometer tests, apparently due to remolding.

3. The area with the closest drain spacing, 0.9 m, had the highest consolidation rate and achieved 100 percent primary consolidation first. The second highest rate was for the area with the heaviest load and with 1.5 m drain spacing.

4. Hansbo's (1960) theory of consolidation for sand drains wherein the deviation of flow from Darcy's Law is considered, predicted the observed settlement and pore

pressure dissipation rates somewhat better than the classical consolidation theory. The best value of n, which expresses the deviation of flow from Darcy's Law, was found to be 1.5 from the rates of consolidation determined from observed pore pressure dissipation rates.

5. In the undrained area, the average consolidation based on the dissipated pore pressure after 14 years is about half that based on the observed rates of settlement. Apparently, some of the settlement is due to secondary compression going on during the primary phase, since the rates of pore pressure dissipation follow closely those predicted by classical theory and valves of c_v determined from conventional oedometer tests. Rates of settlement are very poorly predicted from oedometer tests, even though the test values are apparently satisfactory for the drained areas.

6. Measurements from settlement gages located in the sand drains indicate that the drains have essentially settled along with the surrounding clay. No apparent stiffening effect was detected.

7. Rates of secondary compression in the drained areas were quite small, and it is not clearly established whether remolding due to driving of the drains increased the rate. The field rates were on the average about 1/25th the rates determined from conventional 24-hr oedometer tests. These low field rates, coupled with the anamolous behavior of the undrained area, suggest that reduction of the initial excess pore pressure also reduces the secondary settlement, especially that part which goes on during primary consolidation.

8. On the average, consolidation for 14 years has decreased the water content from 5 to 20 percent, increased the undrained shear strength by about 5 kN/m^2, and slightly decreased the s_u/σ'_{vo} ratio.

9. Vertical settlements at the center of the drained areas could be accounted for by changes in water content within about 25 to 30 percent; the same prediction for the center of the undrained area was almost exact. At the edge of the undrained area, the measured settlements were less than indicated by the changes in water content. Apparently, horizontal consolidation has occurred at the edge of the fill.

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LONG-TERM LOADING TESTS AT SKÅ-EDEBY, SWEDEN

Robert D. Holtz[1], A.M. ASCE, and Bengt Broms[2], M. ASCE

The theoretical degree of consolidation curves given in Fig. 16 in the paper[3] were in error for $\bar{U}_h$ greater than 90 percent. The corrected figure is presented below.

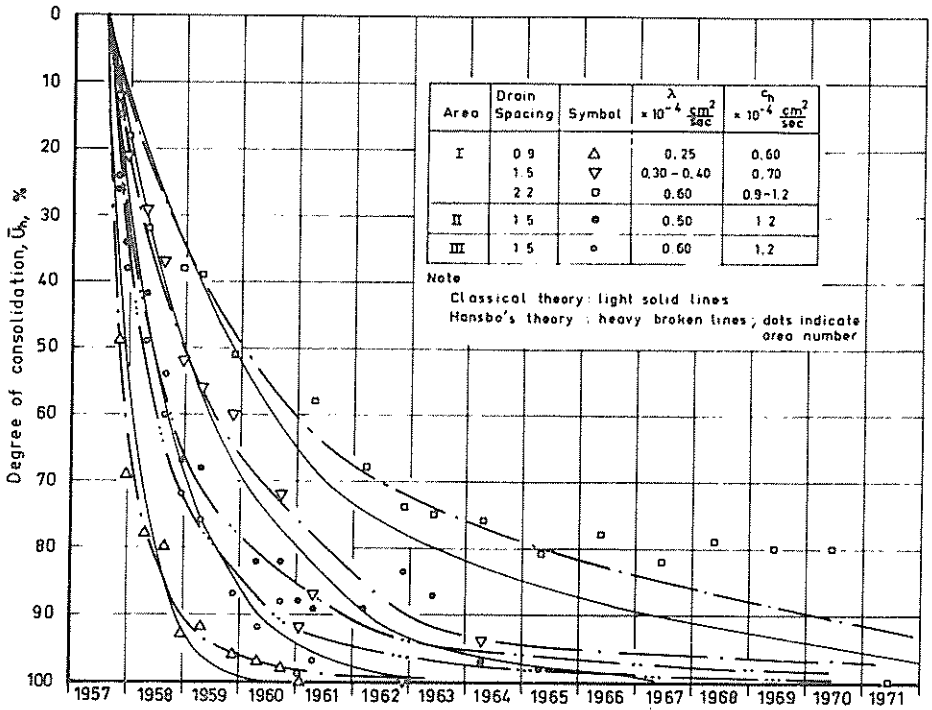

FIG. 16 THEORETICAL TIME-CONSOLIDATION CURVES COMPARED WITH OBSERVED DEGREES OF CONSOLIDATION BASED ON PORE PRESSURE OBSERVATIONS:

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[3] Vol. I, Part 1, pp 435-464.

Excavation and Sampling around some Sand Drains at Skå-Edeby, Sweden

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ABSTRACT

Recent investigations into the long-term performance of the test fills at the Skå-Edeby test field included a study of the effects of disturbance during driving of the sand drains and some of the changes in the clays that have taken place after almost 15 years of consolidation. Results of a piston sampling program around some of the sand drains are presented along with a detailed water content profile obtained from a test pit excavated to a depth of 2 m below the original ground surface. At that depth, the water content next to the drain was significantly less than in between the drains, while no such difference was observed at 4 and 6 m. The water content profile and other evidence presented in the paper suggests that the zone of disturbance was slightly less than one drain diameter thick. At 2 m depth, the drains were highly irregular in shape and contained pieces of clay within the sand cylinder. Still, the drains did function essentially as designed (Holtz and Broms, 1972). Settlements of the drains are compared with settlements of the surrounding clay and it was found that there was no significant stiffening effect due to the drains.

INTRODUCTION

It is generally believed that driving of displacement-type sand drains results in severe disturbance of the surrounding soils. Casagrande and Poulos (1969), for example, made an excellent case against the use of closed-end mandrel-driven drains in soft, sensitive, varved, and organic clays. They pointed out that the effect of the smear zone, or zone of severe remolding of the clays adjacent to the drain, could reduce any advantage of utilizing sand drains.

However, Johnson (1970) pointed out that there is very little direct evidence in the literature as to the effect of displacement-type sand drains on the important consolidation parameter, c_v. Most work on soil disturbance has come from studies of the changes in soil properties due to displacement-type pile driving (e.g., Orrje and Broms, 1967; Flaate, 1972), and it is clear that pile driving causes large increases in pore water pressures, some reduction in shear strength, and some changes in water content. Of course, increased pore pressures indicate only that the total stresses have increased and not necessarily that the clay has been disturbed. And unfortunately, as Johnson (1970) stated, changes in soil properties observed after pile driving cannot be correlated directly with the effect of disturbance on the c_v simply because increased loadings usually imposed as part of the sand drain design minimized the effects of disturbance on c_v. Johnson also summarized several cases where sand drains were definitely beneficial when included in a properly designed preloading situation. In Sweden, driven sand drains for highway construction have in general been satisfactory, and as noted by Holtz and Broms (1972), the drains apparently have performed very well indeed at the Skå-Edeby test field. Remolding due to driving did not appreciably increase the ultimate primary compression nor did it have any apparent detrimental effect on the primary or secondary settlement rates. However, the horizontal coefficient of consolidation, c_h, which is probably about four times greater than the vertical coefficient, c_v, was reduced by remolding to about the value of c_v as determined from oedometer tests and the performance of the undrained area.

With displacement-type drains, as with piles, there is no doubt that the clays are disturbed. A zone adjacent to the mandrel is probably completely remolded, and the degree of disturbance decreases as the distance from the drain surface increases. Casagrande and Poulos (1969) state that the zone of severe disturbance is equal to the displaced cross-sectional area of the mandrel. At Skå-Edeby, for example, the outside diameter of the sand drains was about 18 cm – then by Casagrande and Poulos' criterion, the thickness of the zone of remolding should have been 3.5 or 4 cm.

As a part of a recent review of the performance of the test areas at Skå-Edeby, and in view of the controversy

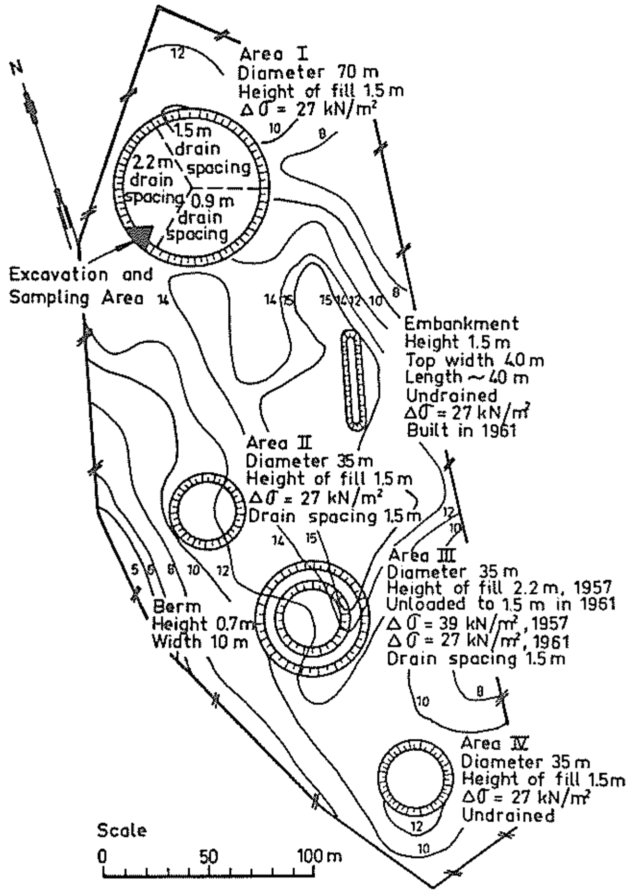

Fig. 1. Plan of the Skå-Edeby test field showing location of present work. Contour lines indicate depth to firm bottom in m.

surrounding sand drains with regard to disturbance due to driving, it was thought advisable to take a detailed look at the condition of the clays adjacent to the drains. It was realized that after more than 14 years, considerable changes in the clay have occurred, and thus it was certainly possible that no residual effects of driving disturbance might be found. Still, it would be of interest to examine the drains after so many years of consolidation and settlement.

TEST SITE

The test field at Skå-Edeby and the soil conditions there have been described by Hansbo (1960) and Holtz and Broms (1972). Fig. 1 is a plan of the test field with the site of the work described herein indicated. Since comparison of the disturbed soils would be with the "undisturbed" clays between the drains which had had the same stress history, the work was concentrated in the area with the widest drain spacing. Because of settlement, the original ground surface is now below the ground water table, so the site was chosen near the edge of the fill where the total settlement is less. Fig. 2 is a summary of the geotechnical properties in this area (Holtz and Broms, 1972).

The gravel fill was removed by an excavating machine, and after the water table in the area was lowered by pumping, the drains were located by careful hand excavation. Four of the drains so located are indicated in Fig. 3 by wooden stakes, and the second drain from the left

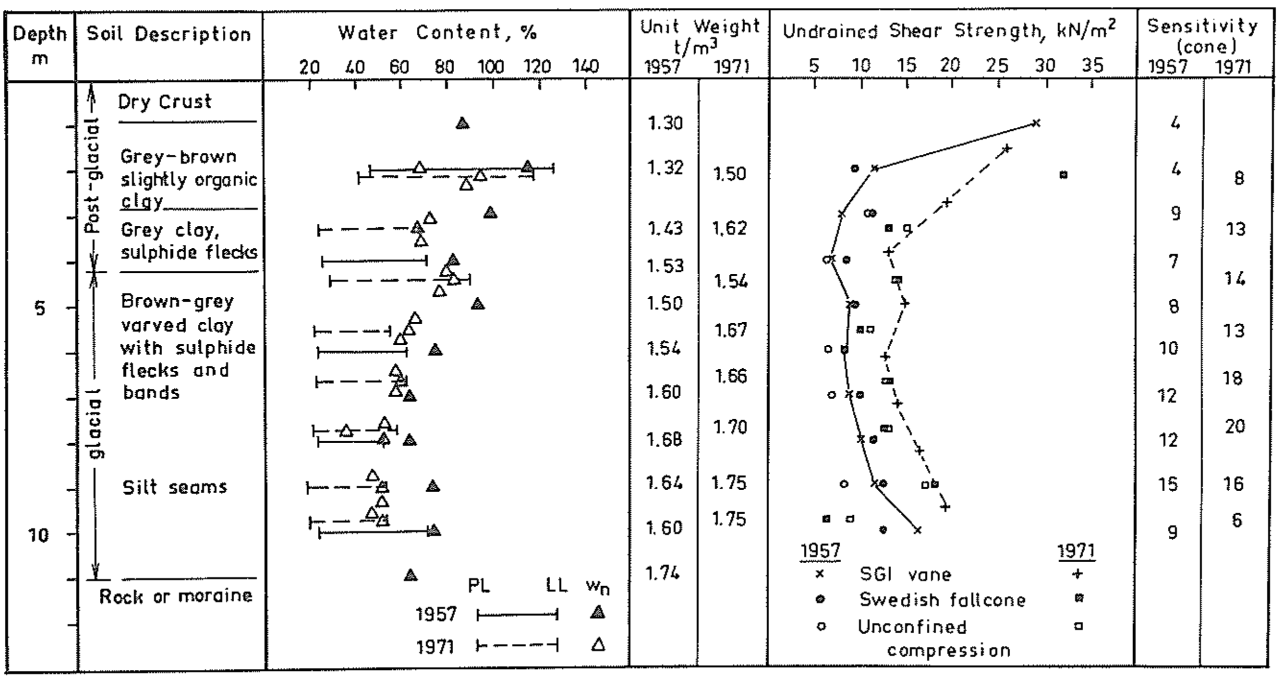

Fig. 2. Geotechnical profile of Area I, 2.2 m drain spacing. (After Holtz and Broms, 1972.)

Fig. 3. Location of the sand drains after removal of the gravel fill.

can be clearly seen in the photograph. Two methods of sampling were used: conventional piston sampling to a depth of 6 m, and a test pit to a depth of about 2 m. The test pit was excavated around the drain shown furtherest to the right in Fig. 3.

PISTON SAMPLING AND RESULTS

Standard piston sampling was employed to obtain specimens of the subsurface clays at 2, 4 and 6 m below the top of the dry crust. Sample holes were located at the edge of the drain, and at 10, 20, 50 and 100 cm from the drains. It was realized, of course, that the holes were probably too close together for good undisturbed samples, but at least an index of the classification properties could be obtained. Routine tests at each depth and location included water content, Swedish fall-cone and unconfined compression strengths, sensitivity, fineness number and the plastic limit. The fineness number has been found to be about equal to the liquid limit for the Skå-Edeby clays.

Variation of Properties with Distance from a Drain

The variation of water content with distance from the drain is shown in Fig. 4. The range of values observed is indicated by the length of the bar while the average value is shown by the point; depth of sampling is also indicated. The water content was definitely lower next to the drain at a depth of 2 m, while there was essentially no change at 4 and 6 m depth. Undrained shear strength was determined by both Swedish fall-cone and the unconfined compression tests, and the variation in these properties with distance is shown in Figs. 5 and 6. As might be expected from the variation in the water content, only samples at 2 m depth showed any significant decrease in shear strength as distance from the drain face increased. As far as the other properties were concerned, unit weights followed the pattern shown by the water content. No discernable difference was detected in the sensitivity as determined by the fall-cone test – the sensitivity was essentially the same next to the drains and away from them. Similarly, the plastic limit was about constant with distance. The fineness number, too, was almost constant for samples at 4 and 6 m, but tended to be lower next to the drain at 2 m depth. It will be recalled that the clay at 2 m is slightly organic (Fig. 2) and apparently water content changes due to consolidation also caused changes in the fineness number, a result noted previously at Skå-Edeby (Hansbo, 1960; Holtz and Broms, 1972). If the old and new Atterberg limits are plotted on the Casagrande LL–PI diagram, the changes are parallel to the A-line.

Change in Properties, 1957–1972

Holtz and Broms (1972) discussed the changes in the soil properties which have occurred because of more than 14 years of consolidation. The present sampling program was compared with that presented in Fig. 2, and in general the agreement was excellent. The differences that were observed could be attributed to differences in sampling depths, which were not precise, and to the areal variation in properties found at Skå-Edeby. In general, the water content decreased about 10 percent while the undrained shear strength increased about 5 kN/m^2. The fineness number tended to decrease slightly as noted above. Changes in the other properties from 1957 to the present were negligible or inconclusive.

TEST PIT

In order to be able to visually examine a sand drain in detail, a test pit around a drain was excavated by hand. The sides of the hole were supported by telescoping steel cylinders, 2 m in diameter, and water seepage was controlled by pumping. The section of the drain first exposed was in the dry crust and it was essentially a right circular cylinder as shown in Fig. 7. The excavation was continued to a total depth of over 2 m, or well below the dry crust and into the softer clays. At this depth, the obviously disturbed clays were removed by knives and a wire saw, and water content samples were carefully cut from fresh clay in two rows on the same line but at two elevations about 3 to 5 cm apart. Specimens averaging about 30 g wet weight were taken every 2 cm from the drain edge to 15 cm away, and then every 5 cm to 90 cm away from the drain face.

The water content variation is shown in Fig. 8. The top row variation may indicate some effects of disturbance due to the excavation procedure, but the variation is not unusual for Skå-Edeby. Interestingly enough, the profile strikingly resembles those given by Flaate (1972) for the variation in water content observed after driving timber piles into a silty Norwegian clay. The bottom row

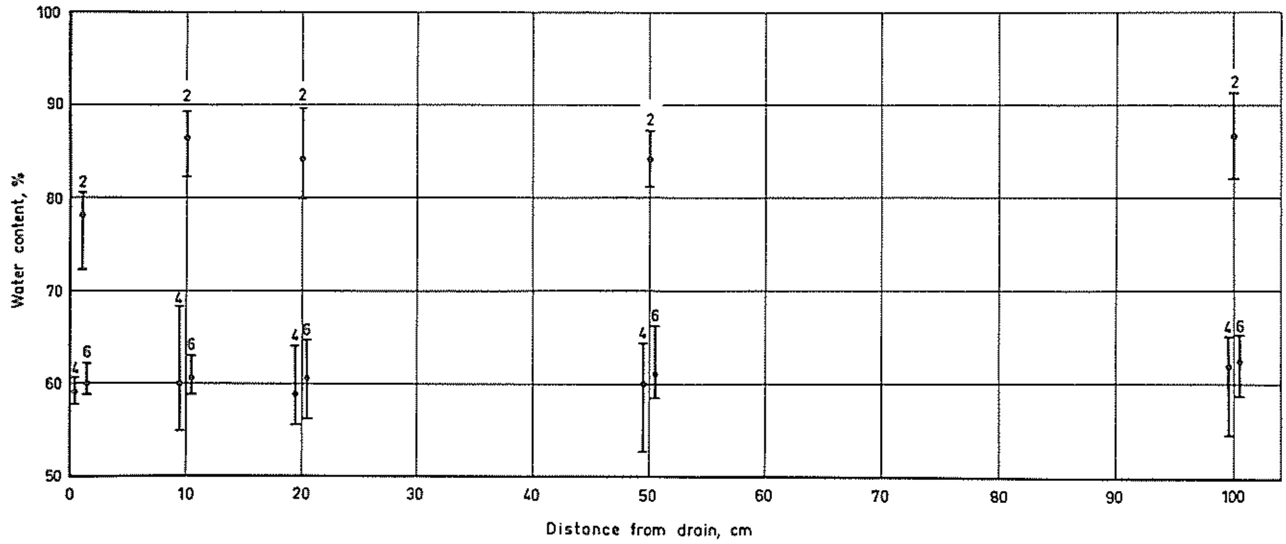

Fig. 4. Distribution of water content with depth and distance from the drains.

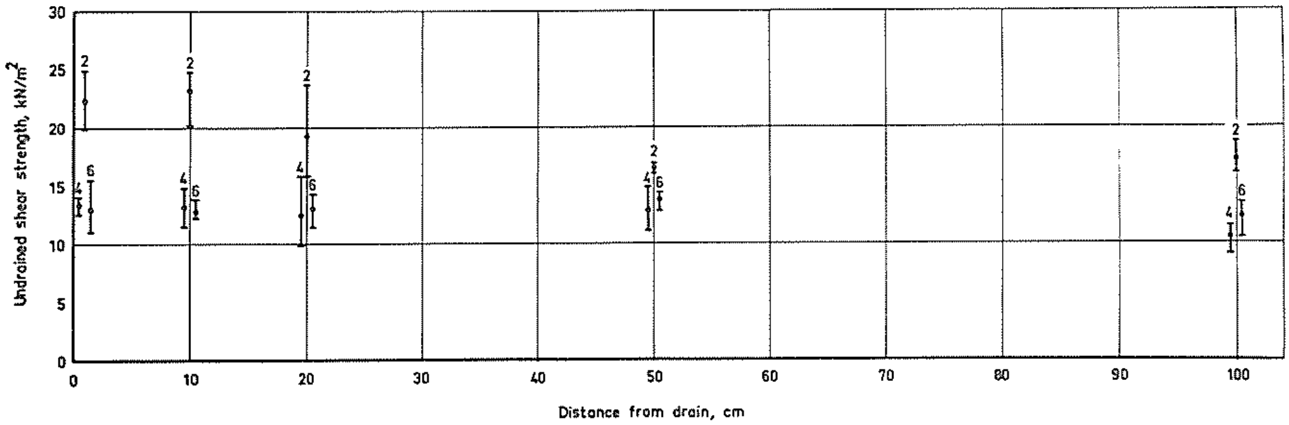

Fig. 5. Distribution of undrained shearing strength as determined by the Swedish fall-cone test with depth and distance from the drains.

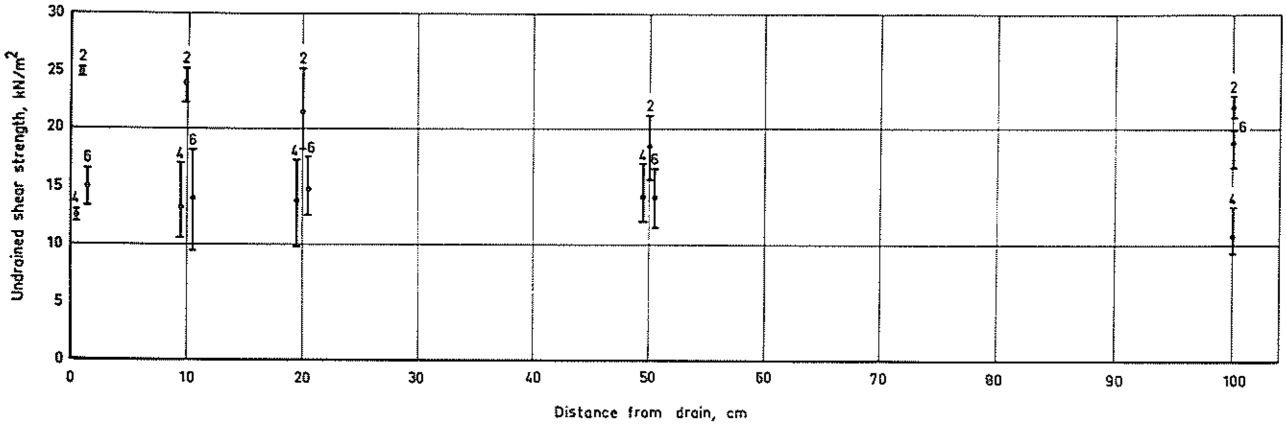

Fig. 6. Distribution of undrained shearing strength as determined by the unconfined compression test with depth and distance from the drains.

Fig. 7. Section of sand drain in the dry crust.

is much more uniform – the maximum range is only 2½ percent from 15 to 90 cm from the drain. Both profiles indicate a significant reduction in water content next to the drain, but the water content rapidly increases to the average value about 15 cm away from the drain face. The observed reduction in water content next to the drain face can be attributed to two sources: (1) the reduction that occurs when a remolded clay is reconsolidated, and (2) the large horizontal stresses which probably existed at the sand-clay interface due to the displacement occurring during installation and due to the settlement of the sand drains.

In order to expose a cross-section of the drain, the clay around the drain was carefully trimmed away by a wire saw. Very fine cracks were first encountered about 20 cm away from the drain face and they contained sand particles. Nearer the drain, the cracks became increasingly wider, and in Figs. 9 and 10 the cracks right next to the drain face are shown. In addition rather large pieces of clay were also found within the drain itself. These are shown quite clearly in Fig. 10 about the level of the 10 cm mark on the scale. In fact, the drain at 2 m depth is obviously quite irregular and is not at all a right circular cylinder as it should be.

It can be seen in Fig. 9 that the massive disturbance and cracking was confined to only one direction from the drain; the surrounding clays in the other directions were found to be completely intact and apparently homogeneous.

Several vertical sections approximately 15 by 25 cm and 5 mm thick were cut from the surrounding clays both

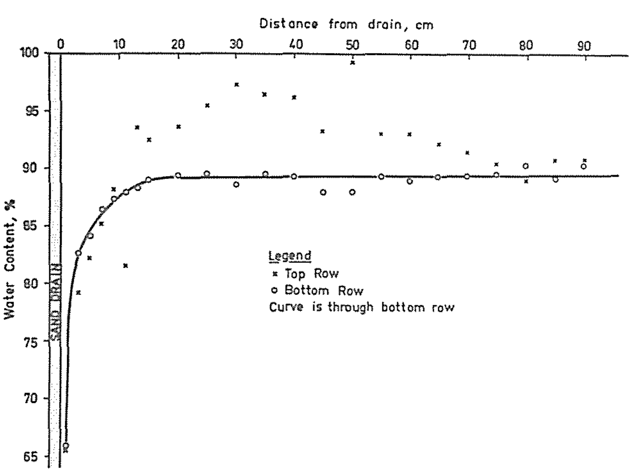

Fig. 8. Water content variation with distance from the drain at 2 m depth, as determined by hand-trimmed specimens taken in the test pit.

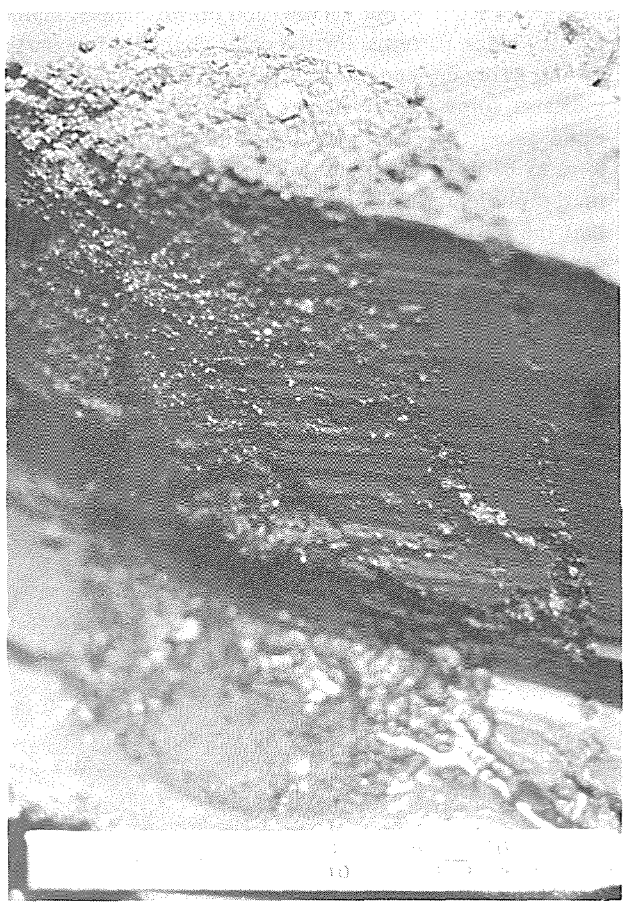

Fig. 9. Cross-section through the drain and the surrounding clay.

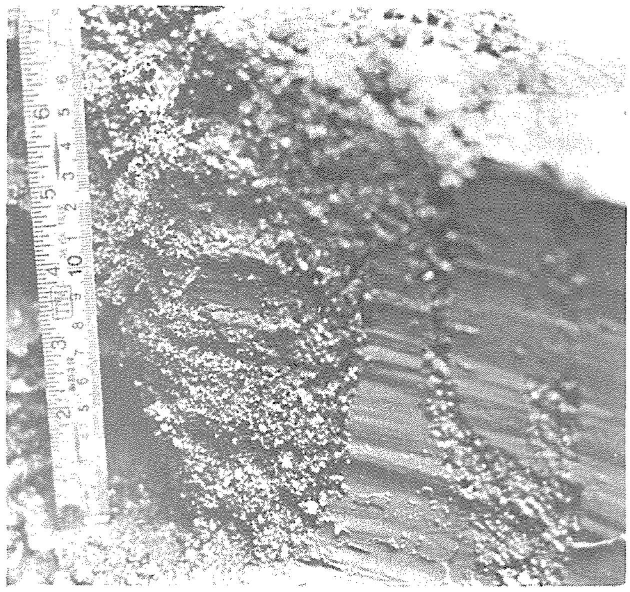

Fig. 10. Another view of cross-section of fig. 9.

in the intact regions and through the cracked zones. The slabs were allowed to dry slowly in the humid room to see if any differentiation in color could be observed after partial drying or if a significant pattern of cracks appeared after complete drying. The clays at 2 m are post-glacial (Fig. 2) and consequently rather homogeneous in appearance (i.e., not varved) and unfortunately, neither the dry crack patterns nor the color changes after partial drying were very conclusive. The sample shown in Fig. 11 indicates some lighter streaks which are vertical at the drain face (the edge with the sand grains) and which tend to become horizontal about 15 cm away. The scale of the photograph is about half size. It will be recalled that the water content variation of Fig. 8 was essentially confined to the zone 15 cm from the drain face. These facts suggest that the zone of disturbance due to driving was probably about 15 cm, or slightly less than one drain diameter, thick.

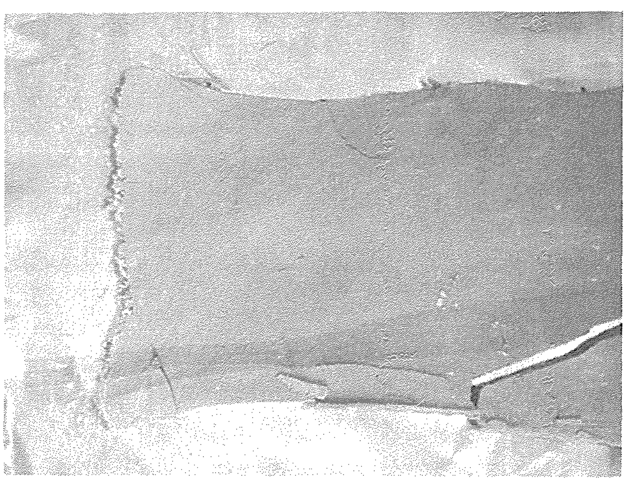

Fig. 11. Partially dried thin section of clay adjacent to the drain

STIFFENING EFFECTS OF THE DRAINS

Hansbo (1960) installed settlement gages in some sand drains in Area I to study the distribution of settlements in and around the drains. He concluded that a "column" effect, wherein the drains were significantly less compressible than the surrounding clays, was not observed. Fig. 12 gives the settlements of the gages at 5 m depth in all three sectors of Area I. Only in the 2.2 m spacing sector is there some suggestion that the settlement of the drain is less than the settlement of the surrounding clay, but the differences are rather small. The performance of the gages in the 1.5 m spacing sector are inconclusive, while in the sector with 0,9 m spacing, the drain appears to have settled slightly more than the surrounding clay. Again the differences are small, and as Holtz and Broms (1972) concluded, the drains have essentially settled along with the surrounding clay.

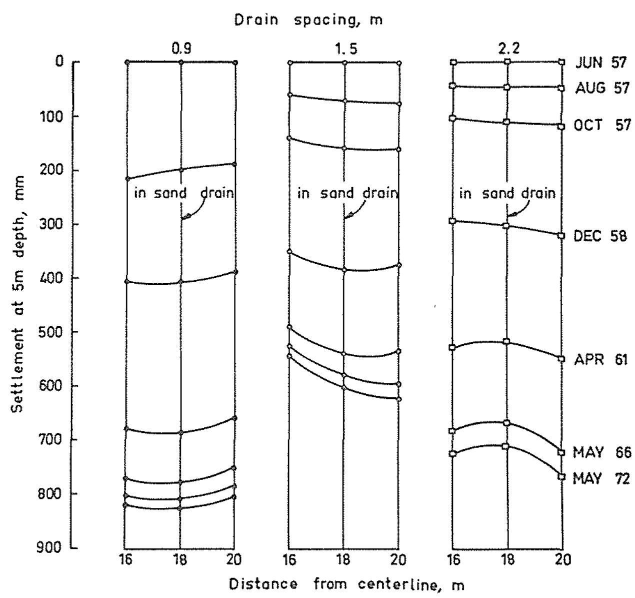

Fig. 12. Settlement of the sand drains compared with settlements of the surrounding clay.

CONCLUSIONS

1. From the piston sampling and the test pit, it was found that the water content was significantly less nearer the drain face than in between the drains at 2 m depth. However, a similar effect was not found at 4 and 6 m by piston sampling.
2. Unit weights, the fineness number, and the undrained shearing strength as determined by both the Swedish fall-cone test and the unconfined compression test, tended to follow the pattern of the water content variation. There was no significant variation of the other soil properties with distance from the drain.
3. After more than 14 years of consolidation, the water content decreased on the average about 10 percent while the shearing strength increased about 5 kN/m^2.

4. The shape of the drain at 2 m depth was highly irregular and showed evidence of rather severe disturbance. Cracks filled with sand were observed 20 cm away from the drain face and large pieces of clay were found within the sand cylinder.
5. The water content profile and the appearance of partially dried sections of clay from adjacent to the drain suggest that the zone of severe disturbance was slightly less than one drain diameter thick, or much greater than the displaced cross-sectional area of the mandrel.
6. No significant stiffening effect was found when settlements of the drains were compared with settlements of the surrounding clays.

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