Canadian Papers Presented at the Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal, September 1965

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ASSOCIATE COMMITTEE ON GEOTECHNICAL RESEARCH

CANADIAN PAPERS PRESENTED AT THE SIXTH INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING, MONTREAL, SEPTEMBER 1965

TECHNICAL MEMORANDUM NO. 88

OTTAWA
DECEMBER 1965

PREFACE

The Sixth International Conference on Soil Mechanics and Foundation Engineering was held in Montreal, Canada, from 8 to 15 September 1965. This was the second such conference held in North America, the first being at Cambridge, Massachusetts, in 1936. The Second, Third, Fourth and Fifth Conferences were held in Europe at Rotterdam in 1947, Zurich in 1953, London in 1957, and Paris in 1961.

This Technical Memorandum contains the reprints of the seven Canadian papers appearing in the Proceedings of the Sixth International Conference. This is in keeping with the practice followed by the Associate Committee for the past Conferences.

The International Society of Soil Mechanics and Foundation Engineering is composed of more than forty National Sections. The executive body for the Canadian Section is the Associate Committee on Geotechnical Research (formerly the Associate Committee on Soil and Snow Mechanics) of the National Research Council. The principal function of the Canadian Section is to assist in the further development and application of soil mechanics in Canada. Enquiries with regard to the Canadian Section will be welcome; they may be addressed to the Secretary, Associate Committee on Geotechnical Research, c/o Division of Building Research, National Research Council, Ottawa 2, Canada.

Robert F. Legget,
Chairman.

Ottawa
December 1965

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The Engineering Behaviour of a Canadian Muskeg

Le Comportement technique d'une tourbière canadienne

J. I. ADAMS, *Supervising Engineer, Soils Section, Research Division, Hydro-Electric Power Commission of Ontario, Toronto, Canada*

SUMMARY

The engineering behaviour of a Canadian muskeg was studied both in the laboratory and in the field. It was shown that the strength of the peat was essentially frictional and that its permeability varied widely during consolidation. Two distinct stages of consolidation were observed, a short-term stage which is believed to be the expulsion of free pore water in the peat, and a long-term stage which is believed to be the compression of the solid peat matter. Long-term pore pressures were observed which are believed to be associated with the compression of the solids.

SOMMAIRE

Le comportement technique d'une tourbière canadienne a fait l'objet d'études tant en laboratoire que sur le terrain. De ces études, il ressort que la résistance de la tourbe est essentiellement une résistance de frottement et que sa perméabilité varie considérablement durant la consolidation. Deux étapes distinctes de consolidation ont été observées, l'une à court terme qui, croit-on, est l'expulsion de l'eau capillaire libre, et l'autre à long terme qui, croit-on, est la compression de la tourbe solide. On a observé des pressions capillaires à long terme et l'on pense qu'elles sont associées à la compression des solides.

"MUSKEG" IS ORGANIC TERRAIN which has resulted from the incomplete decomposition of surface vegetation. It consists of dead and fossilized organic matter known as peat supporting a surface layer of living vegetal matter. The living layer ranges in consistency and stature from grasses and mosses to relatively high bushes and trees. Although peat is by nature fluid and compressible, its properties are variable and are largely determined by the living matter from which it originates. The correlation of the living layer with the co-existing peat layer was observed by Radforth (1952) and formed the basis of his engineering classification of muskeg. Although muskeg is found in many parts of the world, its presence in the northern hemisphere is most noticeable in northern Europe, the U.S.S.R., and Canada. In Canada, as will be seen in Fig. 1, muskeg covers more than 50 per cent of the land surface south of the tree line, and its presence has presented formidable obstacles to the development of natural resources in the north. In the development of hydraulic power sites in northern Ontario, problems associated with muskeg determine to a large extent the economic feasibility of many potential power projects.

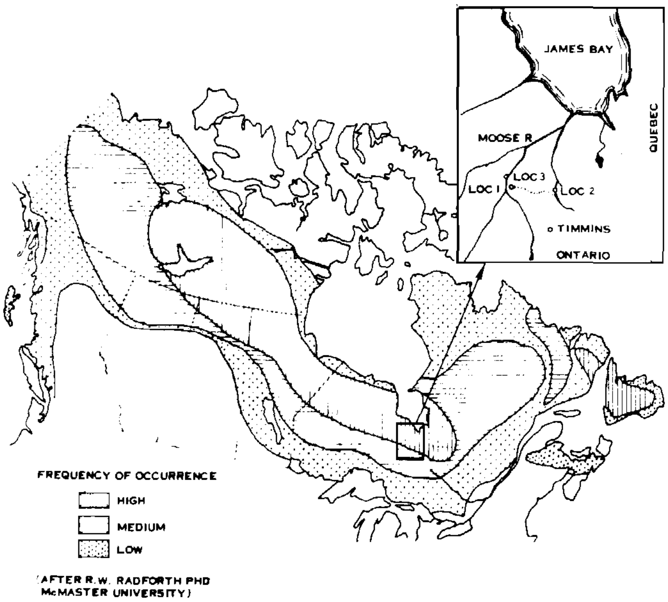

FIG. 1. Map of Canada showing areas of muskeg occurrence.

Studies of the engineering properties of peat were undertaken by the Ontario Hydro Research Division in 1960. Initial laboratory tests included those of consolidation, permeability, and triaxial compression. Instrumentation of peat underlying several embankments has been carried out, and an attempt has been made to relate field to laboratory behaviour. Although much of this work has been presented earlier (Adams, 1961, 1963), the results of all of the work, including some recent laboratory studies, are now presented and reviewed. The behaviour of peat with respect to strength and compressibility is discussed, and general concepts are suggested which may be helpful in predicting the engineering behaviour of muskeg.

LOCATION AND CLASSIFICATION OF MUSKEG

Studies of muskeg were made at three locations in the Moose River basin, as shown in Fig. 1. The muskeg-cover classification and the results of physical tests on the peat at the three locations are shown in Table I. At each of the locations shallow embankments were constructed on muskeg for the purposes indicated in the table. Instrumentation of the peat foundations was carried out and measurements of settlement and pore-pressure development were made during and subsequent to construction. "Undisturbed" bulk samples of the peat were obtained from each location and used in laboratory studies.

LABORATORY STUDIES

The literature reveals uncertainty as to whether peat behaviour is frictional or cohesive. Hanrahan (1954), for instance, held that the strength of peat was essentially cohesive. Others considered the possibility that the fibre tensile strength might influence peat behaviour. With respect

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TABLE I. PHYSICAL PROPERTIES AND CLASSIFICATION OF PEAT

Location	Muskeg* cover	Peat*	w per cent dry wt.	G	pH	Ash per cent dry wt.
1 Little Long Rapids GS (access road	ADI/BEI	No. 9-11	200–600	1.62	4.8–6.3	12.2 to 22.5
2 Abitibi Canyon GS (sewage lagoon embankment)	F/I	No. 2	355–425	1.73	6.7	15.88
3 Harmon GS (block dam)	DFI/B	No. 6–4	330–375	1.65	6.2	12.27

*Radforth classification (1952).

to the consolidation of peat, the observed behaviour was similar in most instances, but the interpretation of results varied from the opinion that the consolidation of peat was essentially "primary," to one that it was essentially "secondary."* All appeared to agree that settlement was continuous. A good example of the long-term behaviour of peat was given by Buisman (1936), who cited embankments on peat (Holland) in which continuous settlement, linear with the logarithm of time, was recorded for more than eighty years. Another consistent observation was that of the marked change in the permeability of peat with change in volume (see, for example Miyakawa, 1960). The testing described below was carried out with the hope of resolving some of the uncertainties.

Permeability Tests

Permeability tests were carried out on remoulded peat in an 8-in.-diameter settlement-permeability device. The peat was consolidated under successive increments of vertical compression. At the completion of each loading period the permeability of the peat was determined by a falling head test. Fig. 2 shows that the peat was initially quite pervious

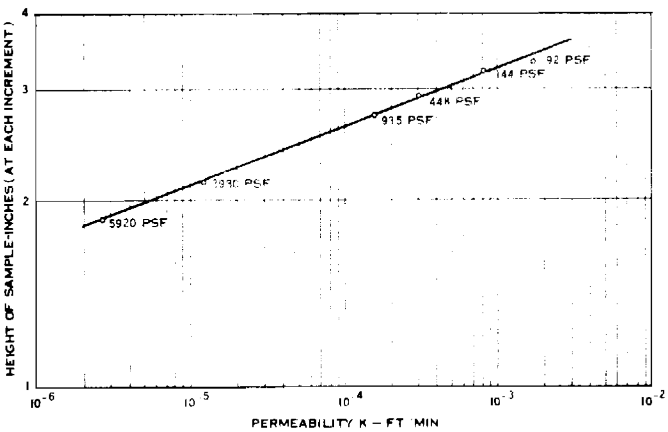

FIG. 2. Permeability *versus* sample height.

but became relatively impervious under high compressive loads. Of significant interest is the linear variation of the logarithm of k, the coefficient of permeability, with the logarithm of H, the thickness of sample. Similar findings were reported by Miyakawa (1960).

Strength Tests

Samples of peat were prepared from "undisturbed" bulk samples obtained at location 2. Undrained triaxial tests with pore-water–pressure measurements were carried out on samples which were saturated by back pressure and con-

*"Primary" consolidation is considered here as that portion occurring mainly under an excess hydrostatic pressure, and "secondary," that portion occurring mainly under zero, or negligible hydrostatic pressure.

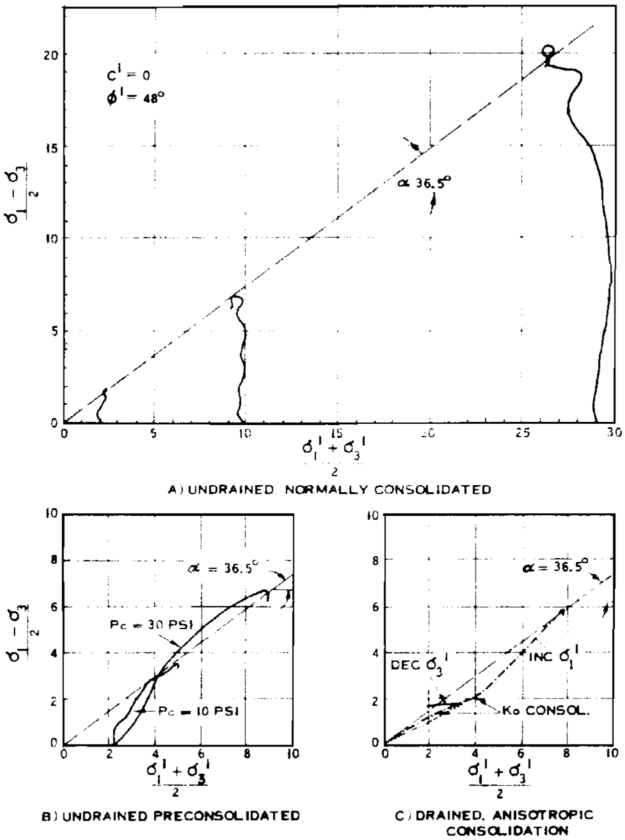

FIG. 3. Triaxial stress plots.

solidated isotropically under 2, 10, and 30 psi confining pressures. Two samples were unloaded after isotropic consolidation under 10 and 30 psi and tested at preconsolidation ratios of 5 and 15 respectively. Drained triaxial tests were made on two samples of peat which were consolidated anistropically (zero lateral strain) and then failed by increasing σ'_1 or decreasing σ'_3. Stress plots of all the triaxial tests are given in Fig. 3. It will be seen that in all tests the stress plots at failure (max σ'_1/σ'_3) fall generally on a single line through the origin, and that the angle of shearing resistance in terms of effective stress indicated from these tests is high ($\phi' = 48°$). Correction for rate of volume change was applied to the drained tests but the correction was found to be insignificant. The K_0 value of the peat for the two drained tests was calculated to be of the order of 0.3.* The preconsolidated samples showed a slight prestress effect but it would appear that this effect is minor for even highly preconsolidated material.

Consolidation Tests

Consolidation tests were carried out on samples from the three locations shown on Fig. 1. The thicknesses of

*Where $K_0 = \sigma'_3/\sigma'_1$ for zero lateral strain.

the samples are shown in Fig. 5. On the 8-in.- and 4.5-in.-diameter samples the excess hydrostatic water pressure was measured during the tests by noting the rise in water level in fine-bore plastic tubing connected to the base plates of the samples, drainage being allowed to the top surface.* The consolidation loads were varied from 30 to 2,000 psf, and were applied in single increments, as well as in multiple increments ($\Delta P/P = 1$).

The results of a typical test on peat are shown in Fig. 4 in which settlement and pore pressure are plotted to different time scales. In each time plot (arithmetic, square root, and logarithm) it will be noted that there is an initial settlement, S_0, occurring in a relatively short period of time ($t_0 = 5$ min). The settlement continues but at

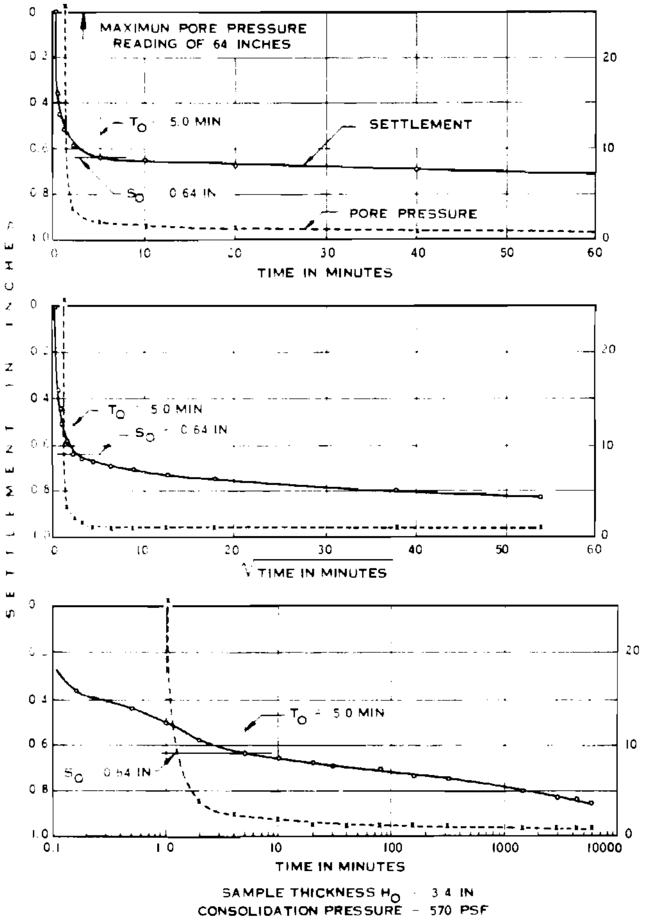

FIG. 4. Typical consolidation curve.

a much slower rate which is approximately linear with the logarithm of time. These results are reasonably typical of all tests carried out in the present study. It may be noted that the excess pore pressure was almost entirely dissipated during the initial consolidation period; a residual pressure of low magnitude (1 inch of water) remained, which decreased slightly with time.

The initial settlement S_0 was calculated for each test and expressed as the ratio $(\Delta H/H_0)$ (change in height over initial height). This value was plotted to the logarithm of the applied load as shown in Fig. 5. Although the points are scattered, they fall within a band indicating a general

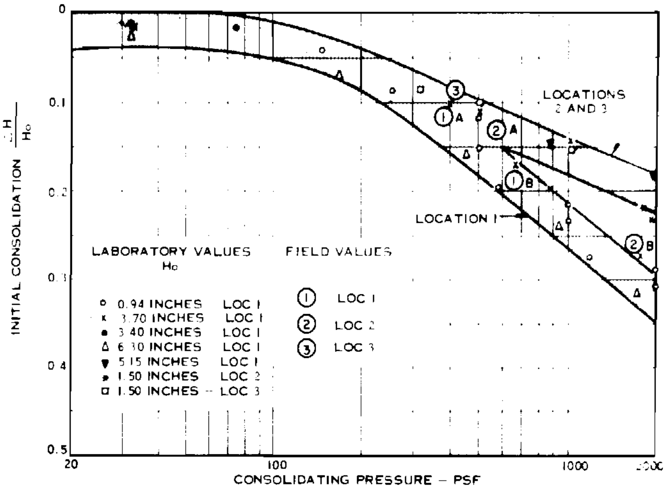

FIG. 5. Initial consolidation.

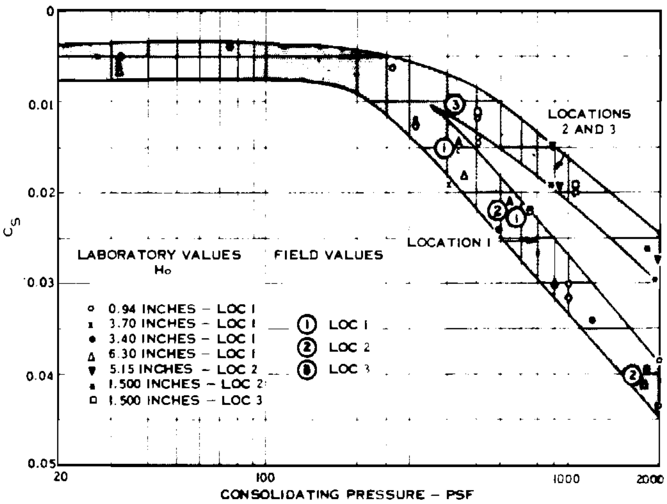

FIG. 6. Long-term consolidation.

relationship between the initial compression of the peat and the magnitude of the applied load. It will also be seen that at higher pressures the relationship for the peat from location 1 differs from those for locations 2 and 3.

It has been shown that the long-term consolidation of peat is generally linear with the logarithm of time. As the slope of this plot appeared to vary in a logical manner with both applied load and sample thickness, it was thought to be significant for relating laboratory and field behaviour. The concept was originally suggested by Buisman (1936) and was mentioned more recently by Miyakawa (1960). For all tests carried out, the slope of the long-term log plot was calculated and the value $\bar{C}_s$ was determined by dividing the slope value by the thickness of the peat $(H_0 - S_0)$ at the start of the long-term compression. The $\bar{C}_s$ values plotted against the logarithm of the applied load are shown in Fig. 6. It will be seen that an approximate grouping was obtained indicating a general relationship for the rate of long-term consolidation with the magnitude of the applied load. Also, the relationship for the peat from location 1 differs from those for locations 2 and 3.

*The measurements are obviously in error during rapid volume change but are considered accurate when the rate of volume change is relatively low.

COMPARISON OF LABORATORY AND FIELD BEHAVIOUR

In each of the field embankments at the three locations indicated on Fig. 1, instruments were installed in the peat foundation prior to construction, and measurement of foundation settlement and pore-pressure development were recorded both during and subsequent to placing the embankment material. Descriptions of the sections instrumented are given in Table II.

TABLE II. DESCRIPTION OF FIELD SECTIONS INSTRUMENTED

Location	Depth of fill (ft)	Depth of peat (ft)	Muskeg classification	Description of underlying mineral soil
1A*	6.0	7.0	ADI	dense till
1B*	3.5	14.0	BEI/EI	dense till
2A	7.0	5.6	F/I	soft marl
2B	15.0	4.7	F/I	soft marl
3	4.4	8.1	DFI/B	dense till

*Corduroy mat used on top of muskeg.

During construction of each embankment a large settlement was recorded immediately on first application of load. By the end of the construction period the consolidation occurred at a much lower rate. Although the settlement was irregular it was believed to be essentially linear with the logarithm of time. The pore-pressure development was appreciable and in the case of the deepest peat (15 ft at location 1) approached the vertical unit weight of the embankment. In all cases the pore pressures appeared to be continuous. The settlement and pore-pressure measurements from the three locations were plotted to the logarithm of time (Fig. 7).

To compare the field consolidation with the laboratory consolidation, the immediate settlement expressed as $\Delta H/H_0$ was calculated from each of the embankments instrumented as well as the coefficient of secondary consolidation $\bar{C}_s$. These values were plotted on Figs. 5 and 6 to compare with the laboratory relationships. The $\bar{C}_s$ values shown cover a comparatively short period, the maximum period being about 1½ years at location 1. It will be seen that the field values of $\Delta H/H$ and $\bar{C}_s$, for locations 1 and 3, plot within the laboratory range. The field values from location 2 plot below the laboratory range. At this location, however, compressible marl was found underlying the muskeg, and the field values were calculated on the basis of the peat thickness only.

DISCUSSION

The consolidation of peat was observed in the laboratory and in the field to occur in two distinct stages. In an initial stage relatively large-magnitude compression occurs in a short period. The duration of the initial stage is in terms of minutes in the laboratory and in terms of days or weeks in the field, for the cases observed. A long-term stage follows in which the rate of settlement is much less and essentially linear with the logarithm of time. It was shown that the magnitude of the initial settlement was directly related to peat thickness and applied load, and that both field and laboratory measurements confirmed this general relationship. Further, it was shown that the rate of long-term consolidation could be related to peat thickness and applied load. This relationship also was confirmed by laboratory and field measurements although the latter cover a comparatively short period.

It has been shown that the permeability of peat decreases in a predictable manner with reduction in volume, and this behaviour is believed to be a significant characteristic of peat. Although not shown in the testing, it is believed that the solid constituents of peat contain a high percentage of water and are compressible. The following concept of the consolidation of peat is suggested and is based primarily on the above two assertions. On application of load, the free pore water in the peat is expelled under excess hydrostatic pressure. Since the peat is initially quite pervious and the percentage of pore water is high, the magnitude of consolidation is large and this period of consolidation is short. As the peat undergoes a large volume change the permeability is significantly reduced. During this period the effective consolidating pressure is transferred from the pore water to solid peat fabric in a manner similar to the primary consolidation of clay. Unlike mineral soil the solid peat is compressible and will sustain only a certain proportion of the

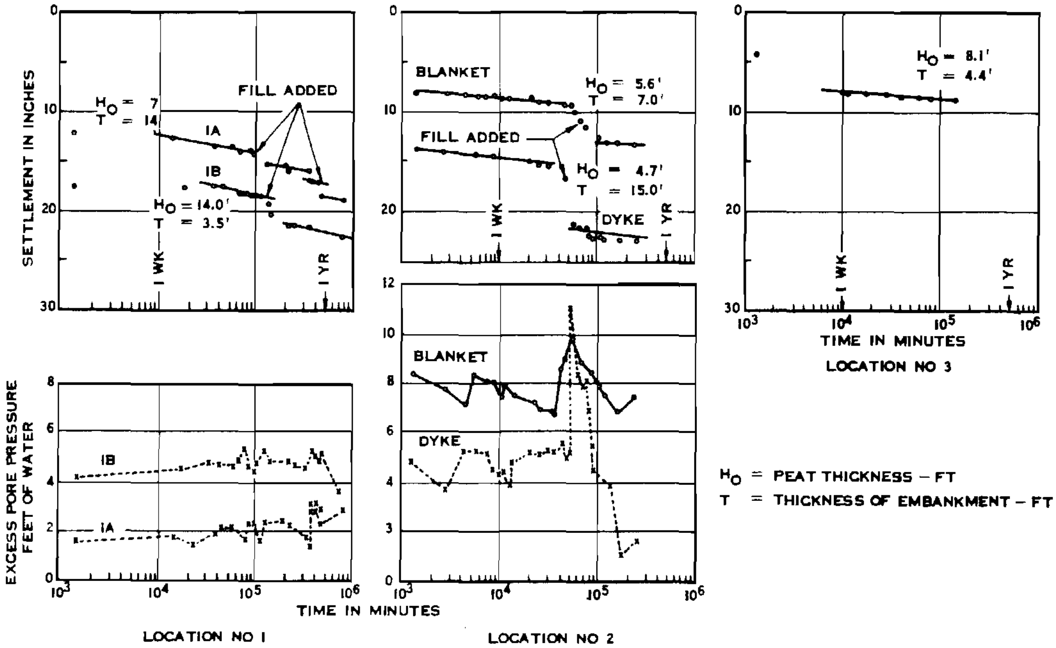

FIG. 7. Results of field instrumentation.

total effective stress, depending on the thickness and permeability of the peat mass. An equilibrium condition is eventually achieved when the rate of compression of the solid peat is the same as the rate of compression of the peat mass, at which time the pore pressure, or more probably the hydraulic gradient, becomes constant.

If the flow of water through peat is according to Darcy's law, i.e., $dH/dt = iK$, where the hydraulic gradient i is a constant, and the coefficient of permeability K varies with thickness according to the relationship shown in Fig. 2, i.e., $\log K/K_0 = C \log H/H_0$, it can be shown that H, the peat thickness, will vary approximately with the logarithm of time. The field and laboratory observations tend to support this concept.

CONCLUSIONS

The strength of the peat is shown to be essentially frictional and in accordance with the principle of effective stress. Although the behaviour of the peat is similar to that of granular material, it is only slightly dilatant even when highly preconsolidated. A somewhat unique characteristic of peat is an unusually low K_0 value. Since this value implies that appreciable shear stresses occur during normal consolidation, the magnitude of construction pore-water pressures is particularly significant in determining the stability of peat.

The consolidation of peat is shown to occur in two distinct stages: an initial stage which for most cases can be considered immediate and a long-term stage which continues indefinitely at a slow rate. General relationships for the magnitude of the initial consolidation and the rate of long-term consolidation were developed from laboratory data with which the field measurements show reasonable agreement. It is suggested that the initial consolidation is the result of expulsion of the free water in the peat mass and that the long-term consolidation is the result of expulsion of water contained in the solid peat matter.

ACKNOWLEDGMENTS

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A Comparison of Laboratory Results with *In-Situ* Properties of Leda Clay

Comparaison des essais de laboratoire avec les propriétés *in situ* de l'argile Leda

C. B. CRAWFORD, *Head, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada*

W. J. EDEN, *Research Officer, Soil Mechanics Section, Division of Building Research, National Research Council, Ottawa, Canada*

SUMMARY

Test results on Leda clay from eleven locations at Ottawa show that the undrained strength and preconsolidation pressure increase with decreasing elevation in general accordance with classical theory. In those locations where a substantial geological unloading has occurred the strength has been maintained, revealing the importance of true cohesion as a component of shear strength. This field evidence is particularly useful in assessing and interpreting laboratory shear tests on this soil.

SOMMAIRE

Des essais sur de l'argile Leda obtenue en onze endroits différents dans la région d'Ottawa indiquent, en accord avec la théorie classique, qu'en général la résistance au cisaillement en teneur d'eau constante et les pressions de préconsolidation augmentent avec une élévation décroissante. Les sols, aux endroits qui ont subi un déchargement géologique substantiel, ont cependant maintenu leur résistance originale. Ceci révèle l'importance de la cohésion vraie comme composante de la résistance au cisaillement. Cette preuve *in situ* est tout particulièrement utile dans l'évaluation et l'interprétation des essais de résistance au cisaillement en laboratoire.

THE LEDA CLAY OF EASTERN CANADA is an unusual soil, quite similar to the clays of Scandinavia (Bjerrum, 1954; Kallstenius, 1963). It is very compressible under load, shrinks substantially on drying, and is composed of inert particles and relatively non-swelling clay minerals (Brydon and Patry, 1961). The brittle, sensitive structure of the clay, when disturbed, transforms into a liquid mass and it is this characteristic that limits confidence in the ability to sample and to interpret laboratory tests for the stress-deformation properties of the natural soil. This paper is a correlation of laboratory and field observations within a limited area which results in some fundamental and practical concepts of more general interest.

GEOLOGY

For many years Leda clay has been considered a marine deposit of the Champlain Sea which invaded the Ottawa and St. Lawrence River lowlands toward the end of the glacial period. It is recognized that there must have been a substantial fresh water influence in the Ottawa area because it was several hundred miles from the open sea. Recent geological work (Gadd, 1963) has suggested that these lowlands carried the substantial drainage of the upper Great Lakes after the semi-marine period and that much of the marine clay was eroded and redeposited. This would explain the generally low pore water salt concentrations (Table I). The open, flocculated structure of the redeposited clay can be attributed to residual cations from the original deposit. Further evidence of redeposition is the general observation that marine shells occur in layers as half-shells and without the orientation expected in natural deposition. Also the clays are generally low in carbonates whereas the reverse is a feature of marine deposits.

Fig. 1, a map of the Ottawa area, is based on reports by Brandon (1961) and Gadd (1963) and shows rock outcrops or till (shaded), deep deposits of postglacial sediments (enclosed in hatching) with shallow deposits between. The major clay deposits occur in what appear to be previous channels of the ancient Ottawa river. Eleven borings (A to K) ranging in surface elevation from 171 feet to 330 feet are shown on the map. Average properties of the soil at each boring (below the drying crust) are given in Table I. At the Sewage Plant and Walkley Road sites (locations B and K, Fig. 1) block samples were obtained at depth and some results of tests on these are listed under numbers BB and KK (Table I).

Most of the soils are highly plastic ($I_p = 30–40$) except for a middle layer usually occurring between elevation 170 and 200 feet. The upper values of sensitivity are of little significance because they depend very greatly on the method of testing (Eden and Kubota, 1961). Salt content of the pore water is generally less than 2 grams/liter except in boring C where it increases to 13.7 grams/liter at great depth. This may account for the greater than normal increase in strength with depth and for the higher than average plasticity at boring C.

CONSOLIDATION PROPERTIES

The more plastic specimens of Leda clay have a low coefficient of recompression, a characteristically sharp break at the preconsolidation pressure, and a high coefficient of virgin compression. The clays of low plasticity are more difficult to interpret and usually indicate a lower than average preconsolidation pressure. A limited investigation showed the load increment ratio to have little if any effect on the pressure-void ratio curve (Hamilton and Crawford,

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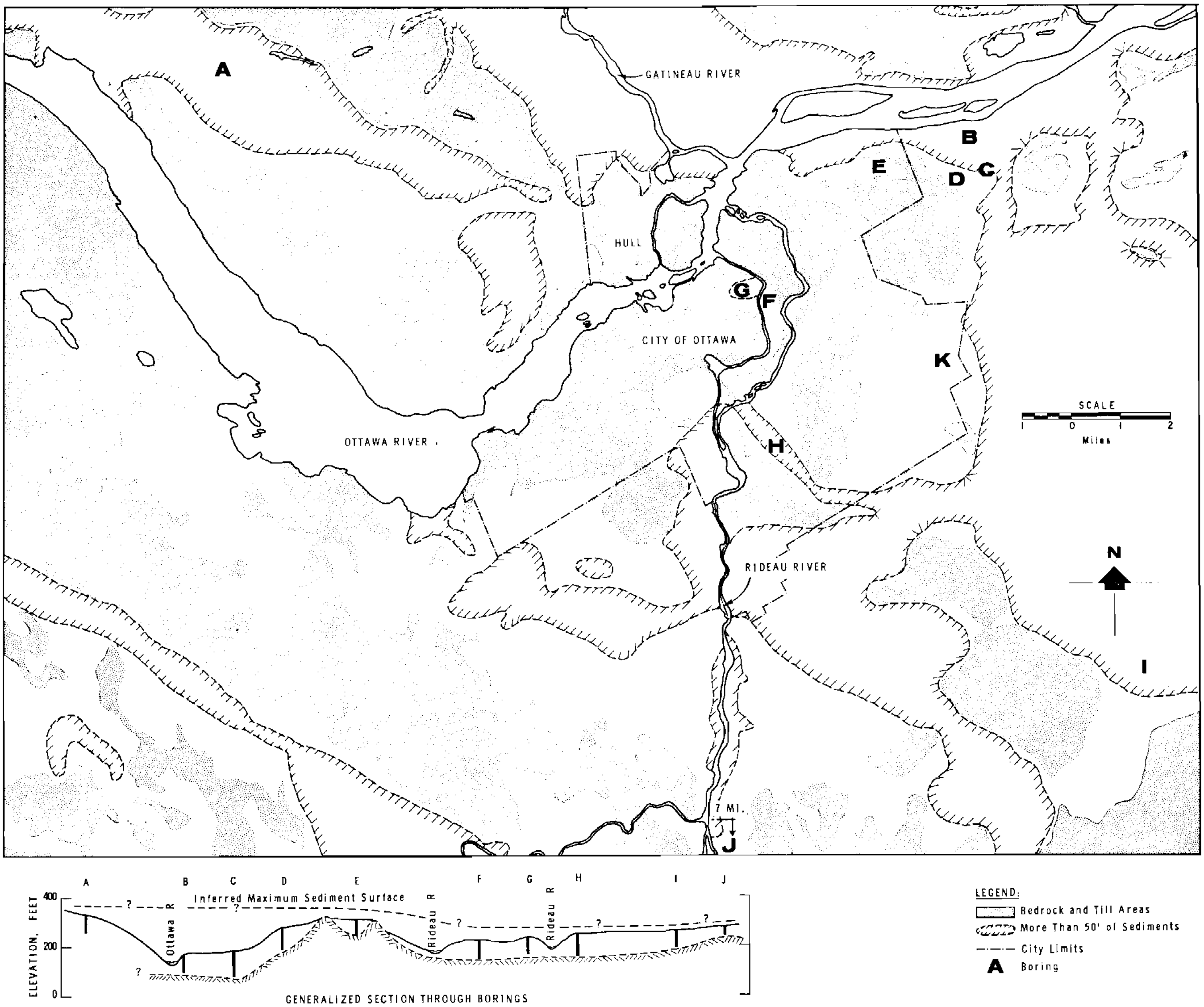

FIG. 1. Map of Ottawa region (geology after Gadd (1963) and Brandon (1961)).

1959) but it can also be shown that the measured preconsolidation pressure can be doubled by rapid increases in effective stress as compared with slow increases (Crawford, 1964).

A positive check on the laboratory compression parameters is possible only by field observations. Measurements on a number of structures and earth fills have confirmed approximately the recompression characteristics but only one well-documented case of virgin compression is under study (Eden, 1961). A general relationship between preconsolidation pressure and elevation had been shown previously (Eden and Crawford, 1957; Crawford, 1961) and is developed further in this paper.

UNDRAINED SHEAR STRENGTH

The most consistent and reproducible undrained shear tests of Leda clay are made in the field with a vane apparatus. Unconsolidated compression tests on specimens from thin-walled piston samplers usually yield values from one-half to the full field vane strength. The lower values are attributed to sample disturbance and to the reduced effective stress level in the laboratory test. Increasing the time to failure in the laboratory appears to decrease the compressive strength by about 10 per cent per log cycle. Because the soil is so brittle (often failing in unconfined compression at less than 1 per cent strain) the usual undrained test in the laboratory or the field reaches failure in less than 5 minutes.

CONSOLIDATED, UNDRAINED SHEAR STRENGTH

When undisturbed specimens of the brittle, sensitive clay are consolidated in a triaxial compression device, it is apparent that the natural, rather rigid structure is partially destroyed. A considerable amount of consolidation must occur to compensate for the loss of structural strength (Crawford, 1963) and such a test does not represent many actual problems. A study of triaxial test results, however, and the effective stress paths followed in the specimens reveals some useful general characteristics.

A common feature of most triaxial tests on Leda clay from the Ottawa area is that once the natural preconsolidation pressure has been exceeded by the ambient pressure, the undrained shear strength is equal to about 30 per cent of the

TABLE I. SUMMARY OF SOIL PROPERTIES

	Location	Surface elev., ft	Depth, ft	p'	S_u	p_n	S_u/p_n	S_t	SC	Averages: w, %	I_P %	I_L	-2μ, %	γ, lb/cu.ft.	Std. error, kg/sq.cm.: p_n	S_u	Remarks
A	Brecken-ridge	330	20 100	.67 2.14	.36 .91	1.2 2.9	.30 .31	20 150	0.4– 3.4	79	36	1.4	80	96	±.24	±.07	Uniform clay, low carbonates. Drying crust extends to 20 ft. Extensive clay deposit to great depths.
B	Sewage plant	171	20 70	.38 1.44	1.00 1.00	3.0 4.1	.33 .25	25– 500+	.1– 2.1	55	19	2.0	66	105	—	±.15	Highly plastic clay to 50 ft, medium sensitivity. Below 50 ft, somewhat coarser, extremely sensitive.
BB	Sewage plant	171	30 70	.56 1.44	1.47 1.87	1.5 5.2	.33 .37								—	—	
C	Green Creek Fill	182	20 100	.68 2.38	.75 1.98	3.1 4.9	.24 .40	5– 25	.6– 13.7	66	40	0.9	70	104	±.41	±.20	Medium to highly plastic clay throughout. Salt content increases linearly from 0.6 grams/liter to 13.7 grams/liter. High carbonates at depth—same clay terrace as B.
D	Green Creek Slide	277	20 90	.69 2.02	.61 1.02	2.0 3.0	.32 .34	20– 500	.1– .5	69	34	1.7	77	101	±.34	±.12	Fine clay throughout. Drying crust to 20 ft. Extremely sensitive at greater depths. Higher terrace than B.
E	National Research	310	30 60	.83 1.40	.55 .77	1.8 2.3	.30 .33	15 500+	.1– 1.5	68	20	2.5	66	98	±.37	±.11	Clay throughout becoming siltier with depth. Drying crust extends to 30 ft. Extremely sensitive at depth. Isolated clay valley.
F	Main St.	222	15 65	.52 1.74	.57 1.03	2.3 3.9	.25 .26	15– 500+	.1– .6	51	22	1.7	50	112	±.49	±.15	Highly plastic clay at surface becoming siltier with depth. Extremely sensitive at depth. Isolated clay pocket.
G	National Museum	235	20 50	.86 1.45		1.6 2.4		10– 150	.1– 1.8	68	40	2.2	56	105	—	—	Same as for F.
H	Heron Rd.	250	20 90	.70 2.20	.58 .75			20– 70	.3– .5	49	14	2.4	51	108	—	±.14	Highly plastic clay becoming siltier and extremely sensitive with depth.
I	Gloucester	260	10 60	.16 .96	.21 .56	0.6 1.7	.35 .33	30– 500+	.5– 2.1	72	23	2.0	71	99	±.15	±.06	Fine clay throughout. High water content, low plasticity, high sensitivity. Carbonates high. Extensive level clay plain. Drying crust to about 10 ft.
J	Kars	280	20 50	.38 .95	.29 .52	1.2 2.0	.24 .26	25– 500+	.2– .7	60	20	2.0	60	101	±.22	±.07	Extremely sensitive clay layer from 20 to 50 ft. Alluvium and drying crust to 20 ft.
K	Walkley Rd.	255	15 45		.50 .70										—	±.07	Highly plastic clay—bedrock less than 50 ft.
KK	Walkley Rd.	255	33		.62	2.2	.28	50	1.7	58	28		62				

p' = effective overburden stress, kg/sq.cm.; S_u = undrained shear strength, kg/sq.cm.; p_n – preconsolidation pressure, kg/sq.cm.; SC = salt content grams/liter.

consolidation pressure, or $S_u/p = 0.30$. If the effective stress path during such a test is examined, it will be seen that the specimen is substantially overconsolidated at failure and it has been contended that the strength is maintained at the reduced effective stress by intrinsic stresses or cohesion. It is further argued that if this effect is reduced by assessing the shearing resistance at stresses less than failure, a maximum friction angle for the material is of the order of 17°. Although this has been referred to as a true angle of friction, it would be greater than the true angle if the soil had a significant intrinsic stress effect (cohesion) under conditions of normal consolidation (Crawford, 1963).

FIELD COMPARISONS

The significant soil properties from eleven borings are collected in Table I. The surface elevation is given for each location and the vertical effective stress (p'), the undrained shear strength (S_u), and the laboratory measured preconsolidation pressure (p_n) are given at two depths. The upper depth is at the bottom of the drying crust and the lower depth is at the bottom of the boring. Each figure was obtained from a statistical evaluation of many tests (standard errors are recorded in Table I) and the resulting straight lines showing p_n and S_u in relation to elevation are plotted in Fig. 2. A generalized cross-section is included with Fig. 1 to illustrate the inferred original surface elevation of the clay deposit.

All strength tests were made with a field vane apparatus and consolidation tests were performed on specimens obtained with a thin-walled piston sampler except for a series of tests on block samples from sites B and K. At site B the usual tests were made before construction and then block

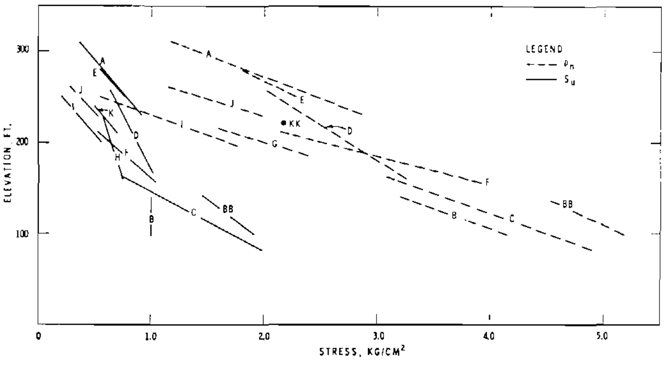

FIG. 2. p_n and S_u in relation to elevation.

samples were obtained at intervals during excavation to a depth of 70 feet. Consolidation and compression tests were performed on these samples and they are noted on Fig. 2 as BB.

The preconsolidation pressure measured on the block samples from site B is more than 1 kg/sq.cm. higher than that measured on the tube samples. Similarly the strengths measured on block samples are nearly double those obtained with the field vane. The authors are of the opinion that the field vane may be inappropriate for testing such stiff overconsolidated clays and this could account for the failure to show an increase in strength with depth. Previous tests on block samples from higher elevations compared more favourably with tube samples.

DEDUCTIONS FROM LABORATORY AND FIELD OBSERVATIONS

Laboratory and field observations must be correlated in order to establish confidence in dealing with these sensitive clays. On the basis of the previous discussion an attempt is made therefore to review in a geological sense the stress history of typical sections of the soil deposit and to relate this to present strength and effective overburden stresses.

To begin, an assumption was made that the average S_u/p ratio of the original normally consolidated clay was equal to 0.30. This assumption is based on the logical concept (proposed by Skempton, 1948) that the shearing resistance (S_u) developed in a normally consolidated clay deposit is related to the overburden pressure (p). Further studies by Skempton, by Bjerrum (1954), and by Osterman (1960) led to a limited correlation between the S_u/p ratio and plasticity. The logic of this relationship is not so obvious and there is a substantial discrepancy between field and laboratory values, especially for soils of low plasticity (Osterman, 1960; Bjerrum and Simons, 1960). There may be several reasons for this discrepancy including, in particular, variations in isotropy. From published values for other clays and from observed relationships between undrained strength and preconsolidation pressure (Table I) on Ottawa clays and from triaxial tests the value of $S_u/p = 0.30$ was chosen.

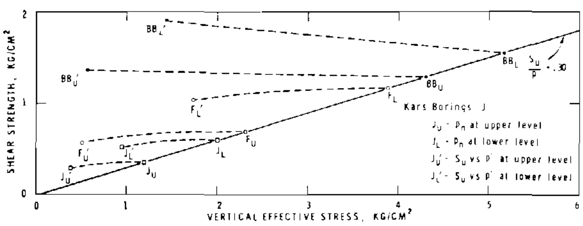

FIG. 3. Relation between preconsolidation pressure, strength, and existing effective stresses for three locations.

In Fig. 3 the average preconsolidation pressure at upper and lower levels for three locations (Table I) is plotted on the average S_u/p line. Other borings fit into the same pattern. It is reasoned that if the undrained strength could have been measured when the entire thickness of soil was normally consolidated under its own weight the strength would have been equal to that indicated. Furthermore, if the strength test was carried out without drainage, then the vertical effective stress at failure would be equal to the consolidation pressure and $\tan^{-1} S_u/p$ would represent the maximum possible angle of shearing resistance in terms of vertical effective stresses. If part of the resistance was due to true cohesion, then the true angle of friction would be somewhat lower. $\text{Tan}^{-1} S_u/p$ is approximately equal to 17° and the interpretation is therefore compatible with previous deductions based on the triaxial test (Crawford, 1961). The usual interpretation of triaxial tests, in the authors' opinion, gives much too great an effective angle of friction due to the overconsolidation at failure. A similar conclusion has been reached by Osterman (1962).

Joined to each of the points that represent strength and effective stress under normally consolidated conditions are points representing existing undrained strength at the present vertical effective stress. The effective stresses on the *in-situ* undrained shear plane are assumed to remain unchanged during test due to equal increases in pore pressure and total stresses. The dashed lines infer the possible effective stress path during geological unloading. Of primary interest is the fact that the strength has been maintained after a substantial reduction of effective stresses in the ground. This is attributed to a form of metamorphosis in which bonds develop between the soil particles under sustained pressure and prevent swelling and loss of strength when the overburden stresses are relieved geologically. The retained strength under reduced effective stresses must be attributed to cohesion rather than to increased friction angle.

In those locations, such as BB, in which the strength at present is higher than the geologically inferred strength, this may be accounted for by one or more of the following reasons: the assumed S_u/p may be too low, as suggested by observed S_u/p_n for the borings; the measured preconsolidation pressure may be too low particularly at great depths when the sensitivity is high; the strength may have increased by cementation; the measured strength may be too high when compared with consolidation properties which are measured at much slower rates of strain. The last-noted reason may be the most important one when the influence of strain rate on structural deformation is recalled. The determination of the proper rate of testing for a particular problem is probably the most important gap in present knowledge.

CONCLUSIONS

1. Engineering evidence is compatible with geological evidence of the complex history of the Ottawa clays. Current thinking suggests two major types of clay—one a reworked product of the other.
2. The good correlation between laboratory and field tests when related to absolute elevation provides a sound basis for evaluating geological history and increases confidence in the test results.
3. This clay has been relieved of effective stresses in nature without substantial loss of strength. The residual strength is attributed to true cohesion.
4. When effective stresses are relieved by the build-up of pore pressure in a triaxial test, the shearing resistance has often been attributed to the mobilization of a rather large friction angle. It follows from these observations of stress release in the field that this is in fact a true cohesion resistance and the usual interpretation of the triaxial test on such clays is strongly questioned.
5. Because rate of loading has a great influence on the performance of most clays, it is a factor that must be taken into account when comparing shear and consolidation properties. This is emphasized by the higher than expected undrained strength at some of the locations described.
6. Sample disturbance is an important factor as shown by comparison of block and tube samples. It is probably of most significance on samples which have experienced the greatest amount of stress release.

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Triaxial Shear Characteristics of a Compacted Glacial Till under Unusually High Confining Pressures

Caractéristiques de cisaillement triaxial d'une moraine glaciaire compactée sous des pressions latérales très élevées

A. E. INSLEY, *Partner, R. C. Thurber and Associates Ltd., Victoria, Canada*
S. F. HILLIS, *Senior Soils Engineer, CASECO Consultants Ltd., Vancouver, Canada*

SUMMARY

The paper describes the equipment and techniques used and results obtained from a series of triaxial shear tests carried out on 6-inch diameter specimens of glacial till compacted at three different moisture contents. Since the till will be used as the core material for an exceptionally high rockfill dam, the tests were carried out at unusually high confining pressures, up to 450 psi. In addition to standard drained and consolidated-undrained tests, special drained tests, which involved measuring the load inside the cell by means of strain gauges, were performed to evaluate the magnitude of piston friction in the apparatus.

It is concluded that the high confining pressures have little effect on the shear-strength parameters of the till in terms of effective stresses, but a considerable effect on the pore-pressure parameter $\bar{A}_f$ which is increased from the usual low value for this type of material as the confining pressure is increased.

SOMMAIRE

L'article décrit l'équipement, les techniques utilisées et les résultats obtenus après une série d'essais triaxiaux faits sur des échantillons d'argile rocailleuse d'origine glaciaire de diamètre de 6 pouces compactés à trois différentes teneurs en eau. Comme ce sol devait être utilisé pour la partie centrale d'un barrage en enrochement exceptionnellement haut, les essais furent exécutés à des contraintes ambiantes peu communes, allant jusqu'à 450 livres par pouce carré.

En plus des essais standards drainés et consolidés non-drainés, on a effectué des essais drainés spéciaux pour mesurer la friction du piston dans la cellule. On conclut que les contraintes ambiantes très grandes ont peu d'effets sur les paramètres de résistance au cisaillement exprimés en fonction des contraintes effectives. Elles ont cependant un effet considérable sur le paramètre de pression interstitielle $\bar{A}_f$ qui augmente avec la contrainte ambiante.

THE TESTS DESCRIBED are part of a laboratory testing programme being carried out on the core material for the proposed Mica Creek dam. This dam will be on completion one of the highest earth or rockfill dams in the world, and the major power structure of the Columbia River Development in Canada.

Since the dam will rise 650 feet above the river bed and some 790 feet above the lowest point in the underlying bedrock channel, the normal stresses which will exist near the base of the structure will be much higher than those usually encountered in soils engineering practice. In order to simulate those stresses, the triaxial shear tests were performed at effective confining pressures varying from 50 psi to the unusually high value of 450 psi.

Furthermore, in order to include most of the particle sizes found in the natural soil, the test specimens were made 6 inches in diameter.

DESCRIPTION OF MATERIAL

The material tested is a well-graded, slightly plastic glacial till containing 17 per cent gravel to 1½ in., 41 per cent sand, 32 per cent silt, and 10 per cent clay. The liquid and plastic limits are 21 per cent and 17 per cent respectively. The standard Proctor dry density, corrected to include particle sizes up to 1½ in. is 136 pcf and the optimum moisture content is 9.8 per cent. The specific gravity of the material varies from 2.75 to 2.80. The gravel sizes are mainly granitic gneiss and quartz with some friable schist. Mica flakes are present in the fines.

TESTS PERFORMED

In order to determine the effective stress parameters, a total of 18 standard drained triaxial tests were carried out on specimens compacted at optimum moisture content, and at 2 per cent below and 2 per cent above optimum.

As a check on the parameters obtained from the drained tests, and to obtain some measure of the pore-pressure behaviour, five consolidated-undrained triaxial tests with pore-pressure measurements were performed on specimens compacted at optimum moisture content.

As will be noted, the triaxial cell used has a large-diameter piston which slides in a fixed bronze bushing. There is thus the possibility of piston friction influencing the loads as measured. It was decided, therefore, to modify the apparatus so that the load could be measured inside the cell (which of course eliminates any concern in this regard), and to carry out special drained tests using the modified equipment.

This has been done, but only three results of long-term tests are available yet. These results, obtained from the special drained triaxial tests performed at both 2 per cent dry and 2 per cent wet of optimum, are considered of interest and are reported herein. With these latter tests it was also decided to check that the times to failure were sufficient to ensure that no significant pore pressures developed during shear. This was done by inserting a probe into the specimen at mid-height to check that there was no difference in pore pressure between the middle and the back pressure applied at the ends.

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TESTING EQUIPMENT

The apparatus and layout is similar in design to that developed at Imperial College and, despite the high confining pressures and large-diameter test specimens required, almost all of the equipment can be obtained readily from commercial suppliers. The triaxial cell and loading system were supplied by Clockhouse Engineering Ltd. and the pressure supply and pore-pressure units by Wykeham Farrance Engineering Ltd. The load-sensitive studs used for the special drained tests were manufactured by the Strainsert Company.

The triaxial cell can accept specimens 6 inches diameter and 12 inches high, and since it is made of steel a maximum confining pressure of 1500 psi can be used. The piston through which the axial load is applied is 3¼ inches in diameter and is of precision-honed stainless steel running in a fixed bronze bushing. Leakage past the piston during a test is inhibited in a conventional manner by floating a thin layer of castor oil on top of the cell water.

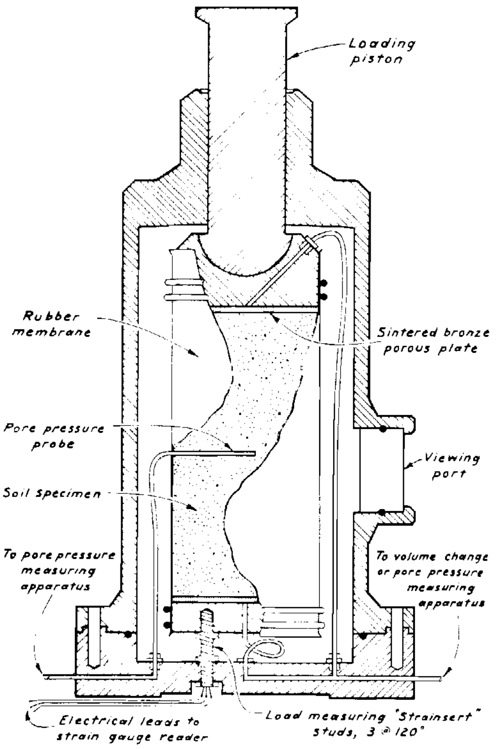

FIG. 1. Section through triaxial cell.

For the special drained tests, it was necessary to design and assemble a new cell base as shown on Fig. 1. With this base, the axial load may be measured inside the cell by the three load-sensitive studs which support the pedestal upon which the soil specimen is mounted. These studs were specially designed for this purpose. Each contains two electric strain gauges so orientated that they are sensitive to axial load only and are unaffected by the cell water. The studs are more sensitive to load than a proving ring of an equivalent range. In order that a mid-height pore-pressure probe may be fitted into the specimen, provision is made in the new base for an additional pressure connection. To eliminate joints, the probe is made of one continuous length of flexible nylon tubing. For all the tests described, the porous end plates placed against the ends of the test specimens were of sintered bronze to prevent breakage under high compression stresses. The latex rubber membranes used to seal the specimens had a wall thickness of 0.02 or 0.03 inches.

The pressure supply system, which is capable of applying pressures up to 500 psi, is the mercury pot type (Bishop and Henkel, 1962). The standard system has been modified to supply this increased pressure, and to fit the laboratory layout. Nylon tubing and high pressure couplings are used throughout. The pore-water-pressure measuring unit is the conventional null indicator, and the early steel type (Bishop and Henkel, 1962) was found to be adequate without modification. The loading frame used has a maximum capacity of 100,000 pounds and the machine is gear-driven to provide a constant rate of strain which can be varied from 0.16 to 0.0001 inches per minute.

PROCEDURES

The soil was pre-mixed to the desired moisture content and allowed to "cure" for at least 24 hours prior to compaction. The compaction was carried out in a 3-part mould of 6 inches inside diameter, and was achieved by 56 blows of a standard drop hammer on 1½-inch-thick layers. Immediately after compaction, the specimens were removed from the mould, measured, and weighed. Except in the case of the special drained tests dry of optimum, wool wicks were placed around the perimeter of the specimens to accelerate uniform pore-pressure distribution. For all tests, the specimen was enclosed in a single rubber membrane coated with silicone grease.

For both standard and special drained tests, the material was saturated before testing. This was accomplished by first placing the specimens on a pedestal separate from the cell and subjecting them for a period of about 7 days to a small flow developed by applying a suction of -2 psi at the top and -0.5 psi at the bottom. Records of the air and water flows were kept throughout. The specimens were then placed in the cell and increasing increments of ambient pressures applied and the resulting pore pressures observed. Pore-pressure parameter B values (Skempton, 1954) of 0.95 or greater were accepted as an indication of full saturation. Back pressures of 40 to 50 psi were also applied during the consolidation and shear stages to further ensure saturation.

For the consolidated-undrained tests, the specimens were tested at the compaction moisture content without a back pressure. Owing to the high initial degree of saturation obtained, both as compacted and after application of the high cell pressures, and the high values of pore pressure which result from these cell pressures, it was considered that satisfactory and applicable readings could be obtained without a back pressure and with the coarse porous end plates.

Testing speeds for both types of test were chosen using the consolidation behaviour as a guide (Bishop and Henkel, 1962).

For the special drained tests, a pore-pressure probe was also installed into a pre-drilled hole at the mid-height of the specimen. In addition, in order to actually measure the piston friction, the deviator stress applied to the specimen was measured simultaneously inside and outside the triaxial cell.

RESULTS

Standard Drained Tests

The stress-strain-volume change relationships are shown on Fig. 2 and a plot of $(\sigma'_1 - \sigma'_3)/2$ *versus* $(\sigma'_1 + \sigma'_3)/2$ at failure is shown on Fig. 3. Apart from the numerical

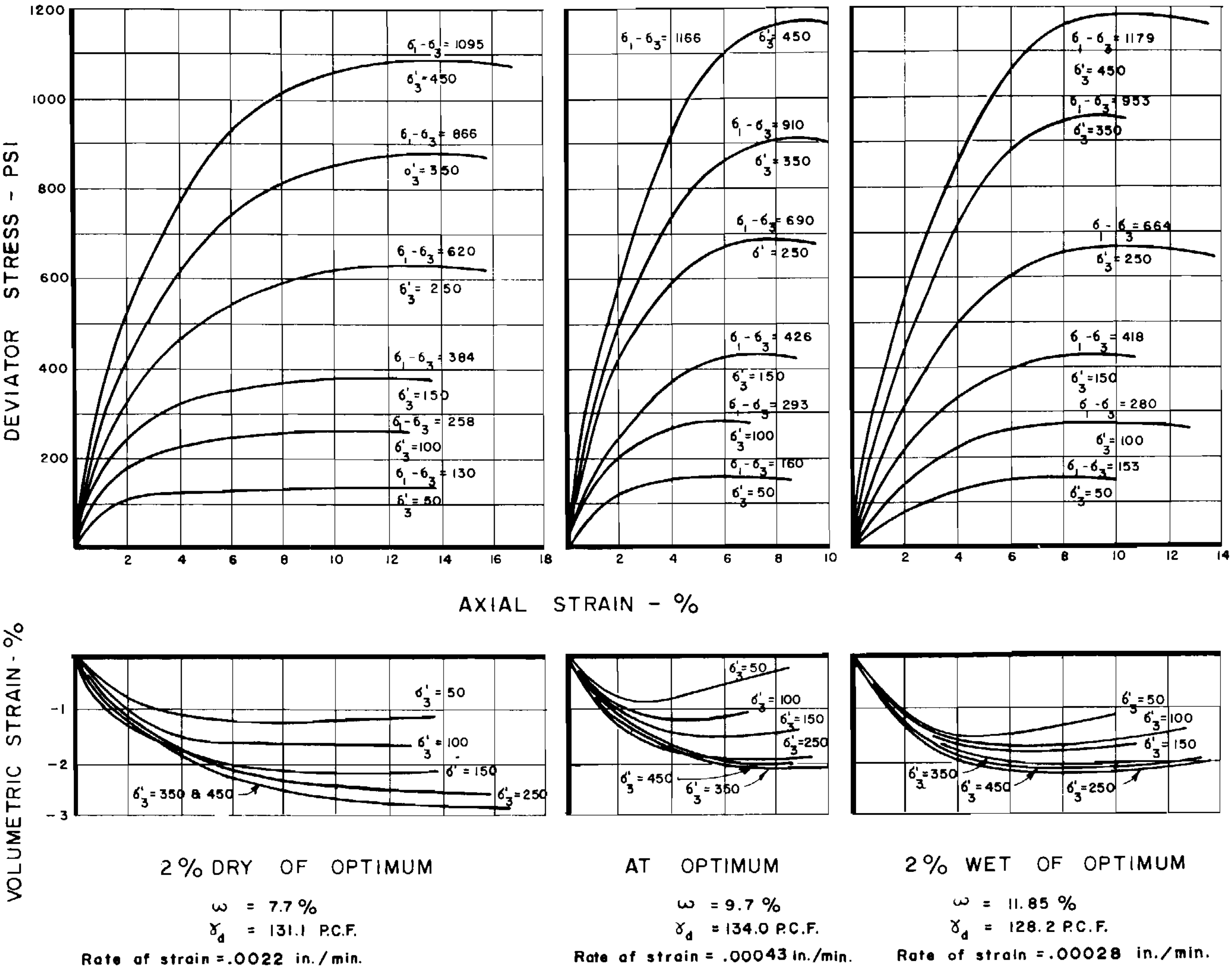

FIG. 2. Standard drained triaxial test results.

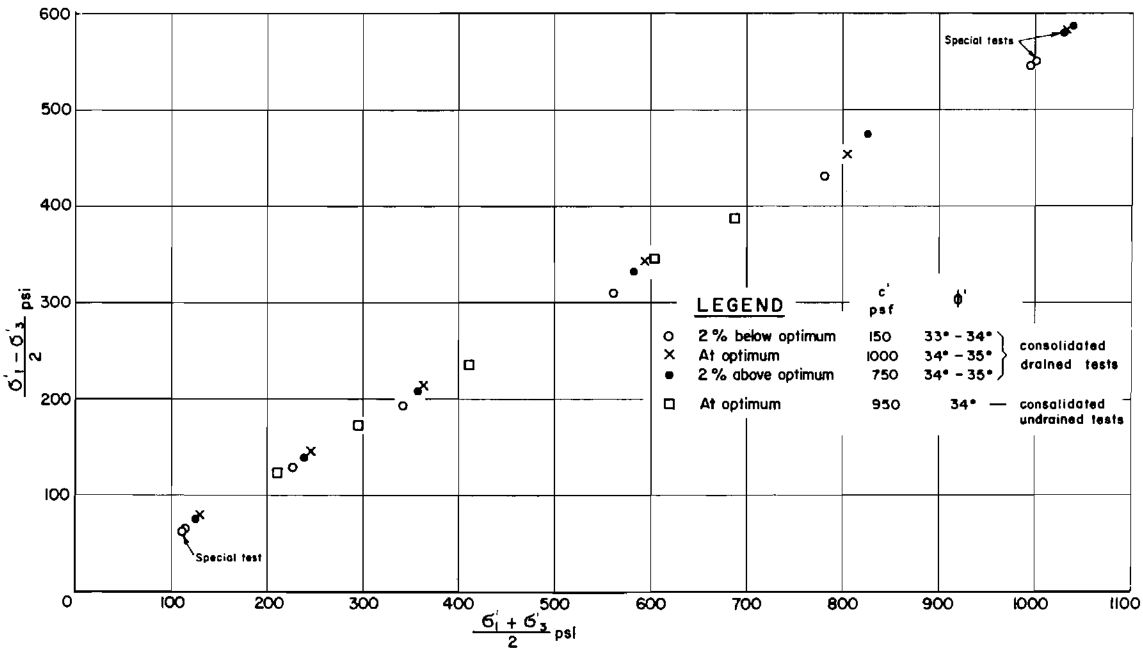

FIG. 3. Drained and consolidated-undrained triaxial tests: $(\sigma'_1 - \sigma'_3)/2$ *versus* $(\sigma'_1 + \sigma'_3)/2$.

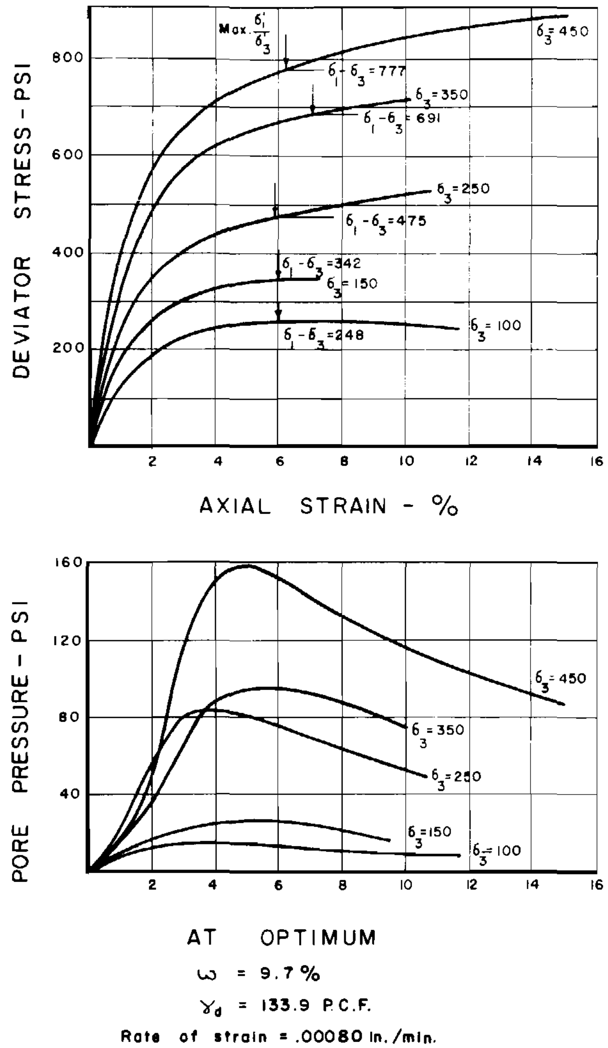

FIG. 4. Consolidated-undrained triaxial test results.

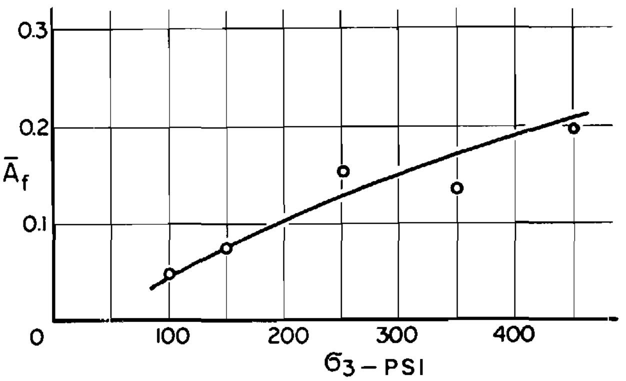

FIG. 5. Consolidated-undrained triaxial test results, $\bar{A}_f$ *versus* σ_3.

values of shear-strength parameters (Fig. 3), the principal points of interest from the test results are as follows:

1. There is little difference in the measured shear strength of the material compacted at the three different moisture contents. The slightly lower strength of the material compacted dry of optimum can be accounted for by reference to the volume change curves shown on Fig. 2, where it will be noted that, unlike specimens compacted at the other moisture contents, throughout the entire stress range the dry of optimum specimens showed no tendency to dilate. This, and the higher failure strains, may be due to the drier material developing a more "flocculated" structure during compaction (Seed, *et al.*, 1960).

2. For all three compaction moisture contents, there is only a very small decrease in the slope of the shear strength envelope with increase in confining pressure. Again, this can be related to the volume change behaviour shown on Fig. 2 where it is apparent that, for a given compaction moisture content, the slopes at failure of the axial strain *versus* volumetric strain curves do not vary substantially within the range of confining pressures used. This is in contrast to the behaviour of sands, or sands and gravels (Hall and Gordon, 1963; Hirschfeld and Poulos, 1963; U.S. Army Corps of Engineers, 1963, 1964; R. A. Spence Ltd., 1963), and can be attributed to the presence of fines and the well-graded nature of the till.

3. The testing time to failure required to ensure a fully drained condition during shear increased as the compaction moisture increased. This again can probably be related to the structure developed during compaction.

Consolidated-Undrained Tests with Pore-Pressure Measurements

The stress-strain-volume change relationships are shown on Fig. 4, and a plot of $(\sigma'_1 - \sigma'_3)/2$ *versus* $(\sigma'_1 + \sigma'_3)/2$ at maximum effective stress ratio is shown with the drained test results on Fig. 3. A plot of $\bar{A}_f$ (Skempton, 1954) *versus* effective confining pressure, also at maximum effective principal stress ratio, is given on Fig. 5. A study of these results reveals two points of interest:

1. The shear-strength characteristics in terms of effective stress are almost identical to those obtained from the standard drained tests.

2. The pore-pressure parameter $\bar{A}_f$ increases considerably from the usual low value for this type of material as the confining pressure increases.

Special Drained Tests

A summary of the results of these tests is given in Table I, and for comparison with the other tests, a plot of

TABLE I. RESULTS OF SPECIAL DRAINED TESTS

Compaction moisture content (per cent)	Effective confining pressure (psi)	Rate of strain (min.)	Failure strain per cent	Maximum deviator stress measured: Outside cell (psi)	Maximum deviator stress measured: Inside cell (psi)	Mid-height residual pore pressures at failure (psi) (in excess of back pressures)
7.4	50	0.0022	12	124	124	0
7.9	450	0.0022	14	1104	1100	+7.0
11.4	450	0.00028	11	1160	1151	0

$(\sigma'_1 - \sigma'_3)/2$ *versus* $(\sigma'_1 + \sigma'_3)/2$ at failure is shown on Fig. 3. Since only three tests have yet been performed, it would be unwise to assume that the results are completely representative. However for these tests, three interesting facts are apparent.

1. The shear-strength values are almost identical to those obtained from the standard drained tests.

2. The amount of piston friction in the apparatus was negligible. This fact is particularly significant considering that the specimens contained gravel sizes, were strained to at least 14 per cent, and, in the case of the specimen compacted wet of optimum, the testing time was about 3½ days. It should be pointed out, however, that the specimens failed by uniform bulging with almost no tilting of the top cap.

3. Although there were some small pore pressures developed in the centre of the specimen during the initial stages of shearing when the rate of volume change is greatest, these soon dissipated and the pore pressures at failure were zero or negligible. This is of interest considering that the time to failure for the specimens compacted dry of optimum was only about 15 hours.

CONCLUSIONS

It is considered that the following conclusions can be drawn from the series of triaxial tests carried out on this compacted glacial till.

1. The effective angle of shearing resistance is about 34° and the measured apparent cohesion varies from about 150 psf to 1000 psf, depending on the compaction moisture content.

2. Increasing confining pressure to 450 psi has little effect on the effective shear-strength parameters. This fact is attributed to the well-graded nature of the material and to the high percentage of fines which inhibit any substantial alteration of the volume change, or dilatancy, characteristics throughout the range of confining pressure used.

3. In contrast to conclusion (2), the ratio of pore pressure to deviator stress at failure, $\bar{A}_f$, is increased considerably as the confining pressure is increased.

4. On the basis of a limited number of tests, the amount of piston friction which occurs with the cell used would appear to be small.

5. The method of measuring the load inside the cell by 3 load-sensitive bolts is successful and straightforward.

6. Even at the highest confining pressures, no difficulties were experienced with single rubber membranes or with the porous end plates.

7. Commercially available equipment and conventional testing techniques can be adequately adapted to suit a testing programme on fairly large-diameter specimens of glacial till at high effective confining pressures.

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The Effect of Pore Pressures on the Undrained Strength of a Varved Clay

Influence de la pression interstitielle sur la résistance, à teneur en eau constante, d'une argile stratifiée

D. L. TOWNSEND, *Queen's University, Kingston, Canada*
G. T. HUGHES, *Royal Military College, Kingston, Canada*
J. A. CRUICKSHANK, *Queen's University, Kingston, Canada*

SUMMARY

Triaxial shear tests, using lateral filters to facilitate pore-pressure equalization between the layers, were carried out on consolidated-undrained samples of a natural varved clay obtained from a borehole at New Liskeard, Ontario. During triaxial shear, a higher pore pressure developed within the more clayey layer than within the more silty layer of the varved soil. These pore pressures tended to equalize with time so that the more clayey layer consolidated while the coarse silty layer expanded. These considerations agree with the experimental observation that there was a linear decrease in strength in terms of effective stress as the percentage clay layer material increased, and a probable curvilinear decrease in strength in terms of total stress as the per cent clay layer increases. A simplified theory is given to support the experimental observations.

SOMMAIRE

Des essais triaxiaux ont été faits sur des échantillons consolidés, à teneur en eau constante, d'une argile stratifiée provement d'un forage à New Liskeard, Ontario. On s'est servi de filtres latéraux afin de faciliter l'égalisation des pressions interstitielles entre les couches de sol. Durant le cisaillement triaxial, on a constaté qu'une pression interstitielle plus élevée s'est développée dans les couches très argileuses que dans les couches très limoneuses du sol stratifié. Les différentes pressions avaient tendance à s'égaliser avec le temps, de sorte que les couches argileuses se consolidaient pendant que les couches limoneuses gonflaient. Ces considérations confirment les observations expérimentales selon lesquelles il y a une décroissance linéaire de la résistance en terme de contraintes effectives et une diminution curviligne probable de résistance en terme de contraintes totales quand la proportion des couches argileuses augmente. Une théorie, plus simplifiée, est présentée à l'appui des observations expérimentales.

VARVED CLAYS ARE LAYERED SOILS deposited in a glacio-lacustrine environment with layers usually alternating in composition between silty clays and clayey silts. These soils are often normally consolidated, with low activities, and liquidities in the order of 1.0. An extensive review of engineering case records has been presented by Milligan, Soderman, and Rutka (1962), and some indication of the range in properties has been presented by Metcalf and Townsend (1960).

The layered nature of these soils and their sensitive structure have created difficulties in assessing their geotechnical properties. Although the layers are not often distinct physical units, it is usually convenient to describe them in this manner. DeLory (1960), and Bazett and Brodie (1961) have presented detailed test results which show that such properties as the water content, Atterberg limits, and grain size distribution vary within the individual layers in widely differing patterns.

In connection with a highway embankment which was to be constructed across a deposit of varved clay at New Liskeard, Ontario, the authors obtained a series of samples from 10-foot intervals which could be used for an investigation of shear strength. Since the primary purpose of the investigation was to assist in the settlement analysis of the embankment, a majority of the samples had to be used for that purpose, leaving a relatively small quantity for strength tests. In view of the limited amount of material, it was decided to use a multi-stage consolidated-undrained triaxial testing programme (Kenney and Watson, 1961) with pore-pressure measurements. Although both normal and slow rates of strain were used in the test programme, this report is restricted to a discussion of the results based upon the normal rate of strain.

PREVIOUS RESEARCH

Using probes, Hughes (1962) found that higher pore pressures were developed in the fine clayey layer of a varved soil than in the coarse layer during undrained triaxial shear of two natural soils, as well as for an artificially layered soil. As pore-pressure equalization developed between the two layers, this observation suggested that the fine layer would drain and consolidate, while the coarser silty layer would gain water and tend to rebound. In undrained triaxial shear, the change in void volume of the two layers of the varved soil must be equal. Hence

$$\Delta V_{vf} = \Delta V_{vc}$$

or

$$\Delta e_f \cdot V_{sf} = V_{sc} \cdot \Delta e_c \tag{1}$$

where V_s = the volume of solids, and f and c refer to the fine and coarse layers respectively. Considering that the fine layer consolidates while the coarse layer expands, and noting that $(u_f - u_a)$ and $(u_a - u_c)$ represent the changes in the effective stresses due to pore-water equalization between the layers, the usual volume change relationships when combined with Equation 1 yield

$$v_{sf} \cdot C_{cf} \log\left[\frac{\sigma_3 + (u_f - u_a)}{\sigma_3}\right] = v_{sc} \cdot C_{rc} \log\left[\frac{\sigma_3}{\sigma_3 - (u_a - u_c)}\right] \tag{2}$$

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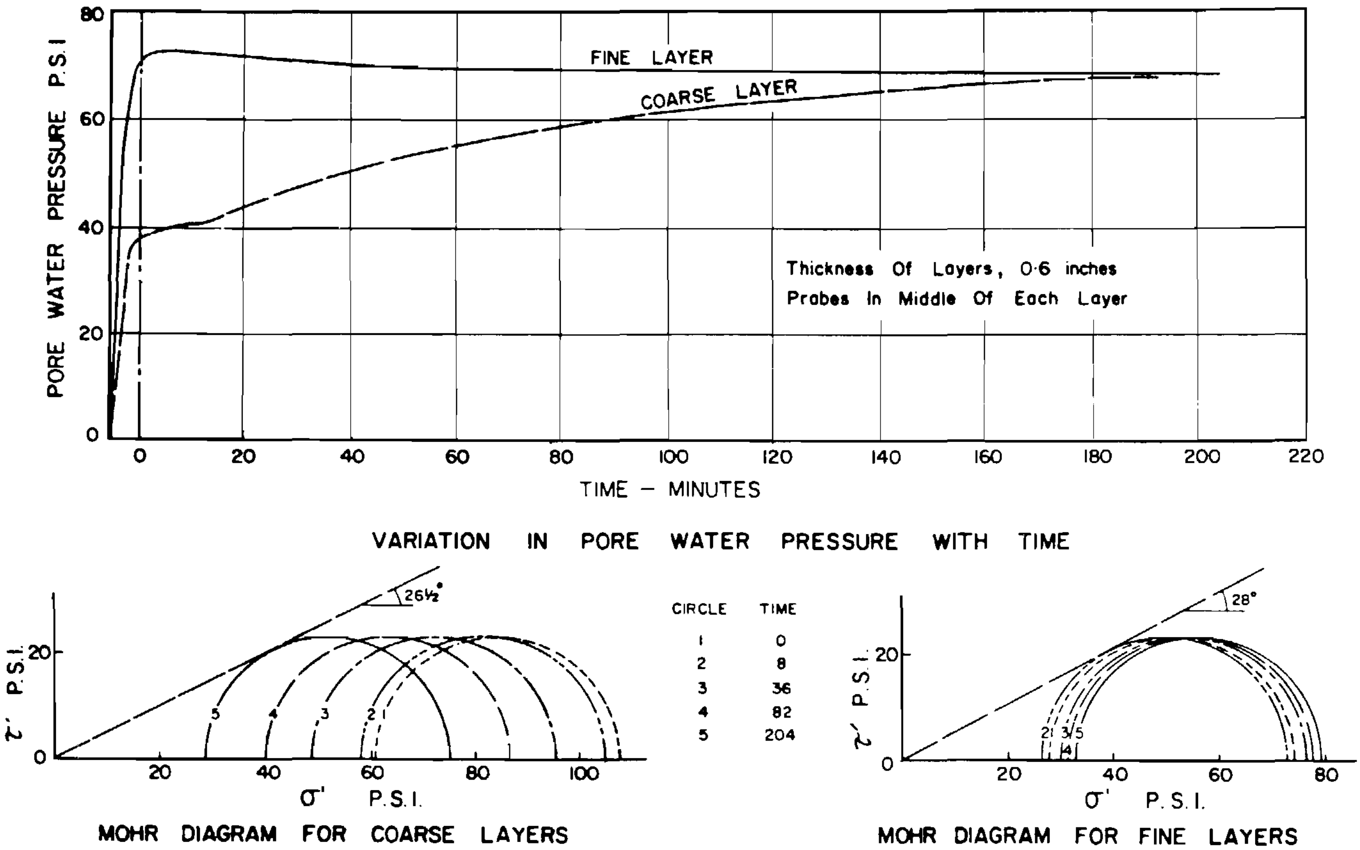

FIG. 1. Mohr circles and pore-pressure variation with time at mid-layer position for natural Matabitchuan varved clay.

where C_{cf}, C_{rc} = compression index of the fine layer, or the rebound index of the coarse layer; u_f, u_c = pore pressures developed in the fine or coarse layer, assuming each acts independently; u_a = final equilibrium or average pore pressure; p = consolidation pressure of the sample.

Considering that $\sigma_3 = p$ for the consolidated-undrained triaxial tests, the above equation may be rearranged and simplified to:

$$\left[1+\left(\frac{u_f}{p}-\frac{u_a}{p}\right)\right]^{\frac{C_{cf}\cdot r_{sf}}{C_{rc}\cdot r_{sc}}}=\left[\frac{1}{1-\left(\frac{u_a}{p}-\frac{u_c}{p}\right)}\right] \quad (3)$$

Fig. 1 shows the results of a test on natural varved clay from Matabitchuan during which the applied principal stress difference was held constant at 93 per cent of the expected failure conditions. The final pore pressure, using Equation 3 was within 2 per cent of the final observed pore pressure. The effective stress Mohr circles are also given for various times during the test.

TESTING PROGRAMME

Hughes showed that equalization of the pore pressures within both layers of the varved sample occurred between 90 and 240 minutes for samples with 0.6-in. layers when triaxially tested without filter strips. Since soils in the field are seldom loaded to within failure conditions in shorter times, it was decided to test the samples under approximately equalized pore-pressure conditions. Lateral filter strips were used with the samples to facilitate the moisture migration during the consolidated undrained tests.

Oedometer tests indicated a mean value of $c_v = 2.4 \times 10^{-4}$ sq.in./sec, and this was used in conjunction with the considerations presented by Bishop, *et al.* (1960) for pore-pressure equalization between the centre and the base to determine the fastest rate of loading for the 3.0 × 1.5-in. diam triaxial samples. Although some silty layers of a varved clay may fail at strains in the order of 1 per cent, a typical composite strain to failure was taken as 3.5 per cent. This suggested that the rate of strain should be in the order of 0.4 per cent per min. Due to limitations of the multi-stage testing technique, failure was assumed to occur at the maximum principal stress difference.

In addition to the strength testing, routine identification tests were conducted. There was little difference between the layers of similar material, and the typical properties are indicated in Table I. Measured layer thicknesses ranged from 0.02 to 1.0 in. for the fine clayey layers, and from 0.02 to

TABLE I. TYPICAL PROPERTIES OF INDIVIDUAL LAYERS

Layer	w (per cent)	w_L (per cent)	I_P	I_L	Clay size (per cent)	Activity
Coarse (silty)	26	28	6	0.7	20–30	0.25
Fine (clayey)	62	62	36	1.0	80–88	0.43

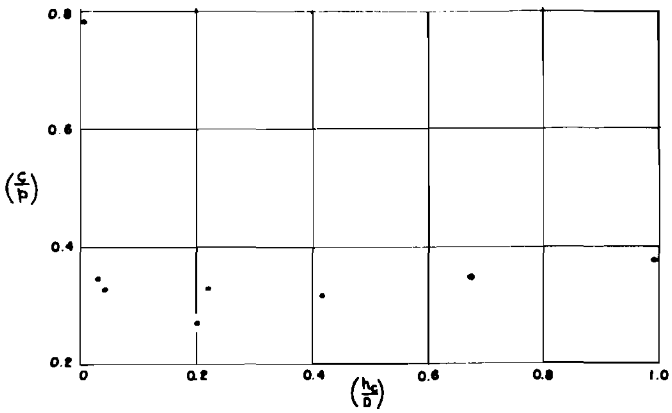

FIG. 2. Relationship between (c/p) and (h_c/D).

0.75 in. for the coarse silty layers. One 3.0-in. sample was obtained of the homogeneous clayey silt (the coarse layer) with apparently no major laminations, which enabled the properties of this constituent to be reasonably determined. The I_p of the single coarse sample was 11, while the other tests indicated $I_p = 6$ for the coarse material. It is thus felt that there was some clay in the bulk sample, and the experimental results have been tempered accordingly. The separate layers were seldom equal in the 20 samples tested.

GENERAL FAILURE CONDITIONS

Although failure may be dependent upon the instability of an individual layer, it is not immediately apparent if this instability will initiate a complete failure plane intersecting adjacent layers, or whether localized bulging may develop solely within weak similar layers by a squeezing action (as suggested by Milligan, *et al.*, 1962). Due to the nature of the multi-stage testing programme, detailed direct examination of each failed specimen was precluded. However, if the failure was essentially by squeezing of a weak layer, a general relationship might exist between the observed total strength (c/p) and the ratio of the clay layer height (h_c) to the sample diameter (D). The stronger layer could restrict the failure plane development similar to the way that concrete specimens are influenced by the height-diameter ratio. The total strength results for the tests are shown in Fig. 2, and it may be noticed that the total strength is nearly independent of the layer thickness.

It appeared that the failure plane had developed through adjacent layers under the conditions of pore-pressure equalization. The theoretical inclination of this plane would be $(45° + \phi'/2)$ where ϕ' is the effective angle of internal friction for the appropriate layer. Neglecting the ellipsoidal shape of the failure plane, it was found to a first approximation that a straight failure plane could be used to replace the composite surface, and would intersect each layer at the same inclination. For these assumptions, the apparent bulk strength of the varved material could be expressed as

$$\tau_{avg}(\Sigma l_f + \Sigma l_c) = \tau_f \, \Sigma l_f + \tau_c \, \Sigma l_c \qquad (4)$$

where τ_{avg} = apparent bulk strength; τ_f, τ_c = shear strength of the fine (f), coarse (c) layer; l_f, l_c = length of the failure plane intersecting the fine or coarse layers. Because of the assumed constant angle of inclination, the equation may be rearranged using the thickness of the individual layers to be:

$$\tau_{avg} = \tau_f \frac{\sum h_f}{\sum h_f + \sum h_c} + \tau_c \frac{\sum h_c}{\sum h_f + \sum h_c} \qquad (5)$$

where h_f, h_c = total thickness of the fine (coarse) layers, measured along the axis of the sample between the ends of the failure plane.

A further simplification may be developed using the percentage fine or coarse layers within the total sample, changing (5) to be:

$$\begin{aligned} \tau_{avg} &= \tau_f \,(\%\text{ fine layer}) + \tau_c \,(\%\text{ coarse layer}) \\ &= \tau_c - (\tau_c - \tau_f)(\%\text{ fine layer}). \end{aligned} \qquad (6)$$

In terms of *effective stresses* and for normally consolidating soils, the shear strength may be given by the usual expression

$$\tau = (\sigma_n - u) \tan \phi'. \qquad (7)$$

However, after pore-pressure equalization within the layers has occurred, the term $(\sigma_n - u)$ will be a constant, and equation (6) reduces to:

$$\tan \phi'_a = \tan \phi'_c - (\tan \phi'_c - \tan \phi'_f)(\%\text{ fine layer}) \qquad (8)$$

where ϕ'_f, ϕ'_c, ϕ'_a, represent the internal friction angles for the fine, coarse, and average conditions. The results of the effective stress tests are shown in Fig. 3, which shows

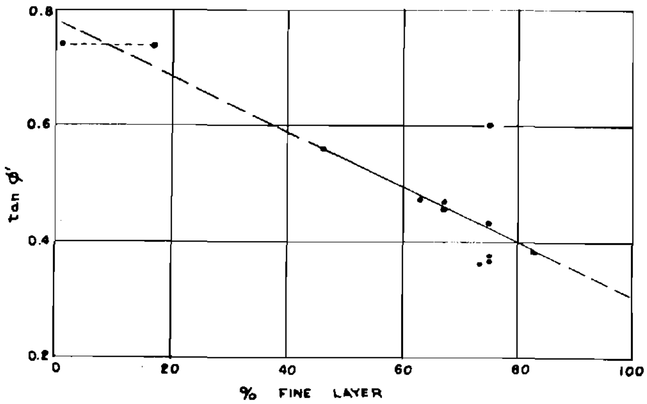

FIG. 3. Relationship between observed value of tan ϕ' and the amount of clay layer material in the sample.

that tan ϕ'_a varies linearly with the per cent fine layer material in the varved soil, as would be suggested by (8).

It has usually been in the cases where the *undrained shear strength* of the soil is applicable that difficulties have been encountered in assessing the strength of a varved clay. In total stress terms (7) may be rearranged to:

$$\left(\frac{c_a}{p}\right) = \frac{c_c}{p} - \left(\frac{c_c}{p} - \frac{c_f}{p}\right)\%\text{ fine layer} \qquad (9)$$

where $c = s_u$ = the undrained shear strength at consolidation pressure, p; and the subscripts a, c, f, denote the shear strength of the bulk or average soil, the coarse layer, and the fine layer respectively. If there was no pore-pressure migration between the layers, Eq 9 would suggest a straight-line relationship similar to that for effective stresses. However, Fig. 4 suggests a curvilinear pattern. A majority of the results were obtained from samples with between 40 per cent and 80 per cent fine layer material in the total sample, which limits the above observation. The single almost homogeneous coarse layer result is based upon the one sample of clayey silt, adjusted in position for the small increase in the plasticity index.

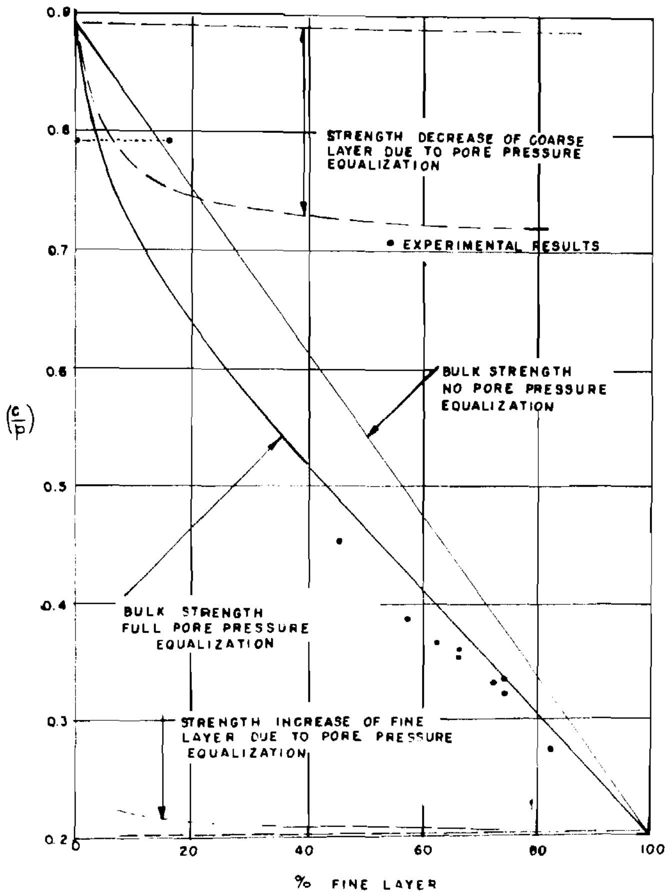

FIG. 4. Relationship between (c/p) and the amount of clay layer material.

THEORETICAL CONSIDERATIONS

An explanation for a curvilinear total stress relationship may be developed using the pore-pressure equalization concepts of Hughes (1962). With the values indicated in Table II, the exponent term in Eq 3 may be rearranged as follows:

$$N = \frac{C_c \cdot V_{sf}}{C_r \cdot V_{sc}} = \frac{C_c \cdot \gamma_d (\% \text{ fine layer})}{C_r \cdot \gamma_d (\% \text{ coarse layer})}$$

$$= \frac{0.90}{0.03} \cdot \frac{60.8}{90.0} \frac{(\% \text{ fine layer})}{(\% \text{ coarse layer})}$$

$$= 20 \frac{(\% \text{ fine layer})}{(100 - \% \text{ fine layer})}$$

Considering a normally consolidated homogeneous soil, analysis of the Mohr circle for the failure conditions indicates that the failure may be written as

$$\left(\frac{c}{p}\right) = \left(1 - \frac{u}{p}\right) \frac{\sin \phi'}{1 - \sin \phi'} \tag{10}$$

Considering first the fine or clayey layer, the geometrical relationships expressed by the Mohr circles at failure are given by

$$\sin \phi'_f = \frac{(\sigma_1 - \sigma_3)_h}{(\sigma_1 + \sigma_3)_h - 2u_f} = \frac{(\sigma_1 - \sigma_3)_v}{(\sigma_1 + \sigma_3)_v - 2u_a} \tag{11}$$

where the subscript h denotes conditions developing in a homogeneous sample and the subscript v denotes conditions in the fine layer of the varved soil. Based upon test observations that the fine layer developed the higher pore pressure, it was assumed that this pressure would be lower than in the same material acting as a homogeneous unit—due to the drainage and equalization in the layered case.

Rearrangement of Eq 11, and subtraction, yields

$$(\sigma_1 - \sigma_3)_v - (\sigma_1 - \sigma_3)_h = \sin \phi'[(\sigma_1 + \sigma_3)_v - 2u_a - (\sigma_1 + \sigma_3)_h + 2u_f] \tag{12}$$

and using the general relationships that: $\sigma_{3v} = \sigma_{3h} = p$, and $(\sigma_1 - \sigma_3)/2p = c/p$, Eq 12 may be rearranged as follows

$$\left(\frac{c_f}{p}\right)_v - \left(\frac{c_f}{p}\right)_h = \left(\frac{u_f}{p} - \frac{u_a}{p}\right) \frac{\sin \phi'_f}{1 - \sin \phi'_f} \tag{13}$$

A similar analysis of the effect of the varved structure on the strength of the coarse layer may be developed. Here it is assumed that the pore pressure in the coarse layer after equalization will be higher than the pore pressure in a homogeneous sample of the coarse soil due to migration of the water from the fine layer. The resulting equation is

$$\left(\frac{c_c}{p}\right)_v - \left(\frac{c_c}{p}\right)_h = \left(\frac{u_c}{p} - \frac{u_a}{p}\right) \frac{\sin \phi'_c}{1 - \sin \phi_c} \tag{14}$$

If the individual properties of each of the constituent layers of a varved soil are known, from Equations 3, 9, 10, 13, 14, it is possible to assess the effect of the pore-pressure equalization between the constituent layers of a varved clay of varying layer thickness upon the strength of the sample in terms of total stress.

TABLE II. MECHANICAL PROPERTIES OF THE SEPARATE LAYERS

Property	Fine layer	Coarse layer	Remarks
γ_d (lb/cu. ft.)	60.8	90.0	From oedometer
w (per cent)	62	26	
C_c	0.90	—	From oedometer
C_r	—	0.03	From oedometer
ϕ'	18°	38°	Extrapolation of tests
u/p	0.55	0.44	Observed
c/p	0.20	0.89	Computed

The estimated soil properties for the individual layers of this varved soil are given in Table II. Equation 10 relates three soil properties which may be measured directly if a homogeneous soil sample were available. However, detailed analysis of the individual properties could not be made, and extrapolation to homogeneous conditions was necessary for the fine layer of soil. In view of the theoretical linear extrapolation suggested for ϕ' and the relative insensitivity of the u/p ratio to changes in layer thickness, these values have been used to compute a value for c/p, rather than taking an extrapolated result. Close agreement between the theoretical and experimental aspects is not to be demanded but the results as shown in Fig. 4 do provide a reasonable agreement.

DISCUSSION

Examination of the results shows two facts, regardless of whether the failure plane does or does not intersect adjacent layers of the sample: (*a*) the strength of the coarse layer is decreased significantly by equalization of the pore pressures in the layers—this strength decrease being in the order of 25 per cent when the amount of fine layer exceeds 25 per cent; (*b*) the strength of the fine layer is increased only slightly and tends to become insignificant when the fine layer exceeds 25 per cent. Thus the predominant effect of the pore-pressure equalization appears to be a weakening of the layer with the lower initial pore pressure, rather than an appreciable strengthening through drainage of the layer with the higher initial pore pressure.

The experimental results are seen to lie close to the calculated graphs for bulk strength. While the result is fortuitous, it still should be noted that the patterns are similar. The assumption that a failure plane developed and intersected layers of the varved soil appears to be reasonable. The net effect of the equalization is seen to be a decrease in the bulk strength above that which might be considered from linear considerations. A small layer of fine material in the sample causes a large decrease in the strength, and as the thickness of the fine layer is increased, the strength becomes progressively smaller. The assumption that the fine (clayey) layer developed the higher initial pore pressure is consistent with the observations by Hughes (1962) that the layer with the higher liquid limit, the higher plasticity index, and the higher liquidity index tended to develop the higher induced pore pressure during the test.

It should be noted that most of the varved clays had low activities and were obtained from depths below 50 feet. Thus there would be some disturbance due to sampling upon the soil structure as suggested by Skempton (1953). In addition, reconsolidation in the laboratory tends to modify the soil structure. The resulting values of total strength, expressed by the c/p values, are higher than normally encountered in the field with vane testing. A value of $c/p = 0.27$ is given by Milligan, *et al.* (1962) for a similar deposit of varved clay with approximately equal silt and clay layers, while Eden and Bozozuk (1962) reported a $c/p = 0.14$ for a varved clay nearby, but at a higher elevation. Hence, the observed total strength values should not be considered typical, although the pattern of strength reduction due to pore pressure equalization is still valid.

CONCLUSIONS

A simplified theory has been presented for the change in strength, in terms of effective and total stresses, of a varved soil due to varying layer thickness and assuming that a failure plane occurs along a plane intersecting adjacent layers.

The strength in terms of effective stress decreases linearly with the increase in the amount of fine material. The bulk strength of a varved clay appears to decrease in a curvilinear pattern as the amount of fine layered material increases, with the predominant effect being that due to a decrease in the strength of the coarser soil.

ACKNOWLEDGMENTS

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The Rates of Consolidation for Peat

Vitesse de consolidation de la tourbe

N. E. WILSON, *Organic and Associated Terrain Research Unit, McMaster University, Hamilton, Canada*
N. W. RADFORTH, *Organic and Associated Terrain Research Unit, McMaster University, Hamilton, Canada*
I. C. MACFARLANE,* *Organic and Associated Terrain Research Unit, McMaster University, Hamilton, Canada*
M. B. LO, *Organic and Associated Terrain Research Unit, McMaster University, Hamilton, Canada*

SUMMARY

This investigation is concerned with the behaviour under load of a characteristic peat type. An examination is made of the mechanics of consolidation to determine whether the dynamics involved constitute a simple process or a multiple one with events which can be isolated. The complex botanical structure is investigated with respect to: (*a*) the way water is held within the peat and (*b*) the arrangement and size of constituents, and the influence these factors have on the rates of consolidation. In establishing a fundamental approach, tests involved only amorphous granular peat, although it is recognized that other structural types of peat might have been chosen.

When the peat samples are loaded, there is a significant change in the rates of consolidation after some time has elapsed. The fundamental e–log t graph is used as a basis for analysis, and when converted to a log (de/dt)–log t graph, the data produce curves in the form of two tangents to a short arc. These curves are parallel for variations of sample height, stress, or initial void ratio. Three series of tests were conducted where the sample height, stress, and initial void ratio were individually varied. For these tests, straight-line envelopes can be drawn tangentially to the derived curves, and can be described by three equations. This graphic representation makes it possible to construct a nomograph which can be used to predict the behaviour of this peat under load. It is indicated that consolidation is not entirely a function of the rate at which free water can be expelled from the peat but that other relationships within the peat, evidently structural, exert considerable influence.

SOMMAIRE

Cette étude se rapporte au comportement sous charge d'un type caractéristique de tourbe. On y examine la mécanique de consolidation pour déterminer si la dynamique en jeu constitue un procédé simple ou un procédé multiple composé d'évènements qui peuvent être isolés. Nous avons étudié la structure botanique complexe par rapport (*a*) à la façon dont l'eau est retenue dans la tourbe et (*b*) à l'arrangement et aux dimensions des éléments, et l'influence que ces facteurs ont sur les vitesses de consolidation. Pour établir une approche fondamentale du problème, les essais ont été faits sur de la tourbe granulaire amorphe bien qu'on reconnaisse que d'autres types structuraux de tourbe aient pu être choisis.

Quand les échantillons de tourbe sont chargés, il se produit un changement significatif dans les vitesses de consolidation après quelque temps. On emploie le graphique fondamental e–log t comme base pour l'analyse, et une fois converties en un graphique log (de/dt)–log t, les données produisent des courbes en forme de deux tangentes à un arc court. Ces courbes sont parallèles pour des variations de hauteurs des échantillons, de contraintes, et de rapports des vides initiaux. On a fait trois séries d'essais où on a fait varier individuellement la hauteur de l'échantillon, l'effort, et le rapport des vides initiaux. Pour ces essais, on peut tracer des droites enveloppes tangentes aux courbes théoriques, qui peuvent être décrites par trois équations. Cette représentation graphique rend possible la construction d'un nomogramme qui peut être employé pour prédire le comportement de cette tourbe chargée. Il semble que la consolidation ne soit pas uniquement en fonction de la vitesse à laquelle l'eau libre peut être rejetée de la tourbe, mais que d'autres relations évidemment structurales à l'intérieur de la tourbe exercent une influence marquée.

AN EXAMINATION is made of the mechanics of consolidation for a selected peat to determine the dynamics involved. The possibility of complexity arises because of the characteristic stucture of the peat which consists of macroscopic, microsopic, and submicroscopic elements, together with the interrelationship of these various elements with themselves and with the water inherent in the peat. The structure of peat, as contrasted to that of mineral soil, suggests that an approach to consolidation other than the classical should be made. This proposition is the objective of the work.

All peat is an accumulation of partially decomposed and disintegrated plant remains which have been fossilized under conditions of incomplete aeration and high water content. Physicochemical and biochemical processes cause this organic material to remain in a state of preservation over a long period of time.

*Seconded from National Research Council, Canada.

The peat for this investigation was procured from a confined muskeg, as contrasted with continuous or blanket terrain, near Parry Sound, Ontario. It has an FI cover and is classified as an "amorphous granular" peat (Radforth, 1952, 1955). The letter symbols refer to a sedge-grass–moss cover common on several continents. In North America, this cover is found as far north as fifty feet from the permanent ice cap and south to the Gulf of Mexico. Unless secondary disturbance has occurred it signifies beneath it the presence of a non-woody peat, highly colloidal in its organic constitution. Predominantly the peat particles though granular lack special form, hence the amorphous granular designation.

The peat deposit was 4 feet deep and the homogeneous sample was obtained from approximately the 3-foot depth and adjacent to open water. In order to achieve reproducibility of results, the peat was remoulded by mixing. It has a natural water content in excess of 600 per cent of dry

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FIG. 1. Amorphous granular peat (magnification, 200 ×).

weight and a specific gravity of 2.0. Although the loss on ignition is only 25 per cent by weight, by volume the soil is primarily organic. Due to an infusion of mineral matter in the form of silt and fine sand, it approaches an organic silt. Under microscopic examination, the organic constituent is a mixture of commonly occurring rod-shaped diatoms (microscopic unicellular algae with silicified cell walls—see Fig. 1) together with pollen grains, spores, and microparticles in an organic colloidal matrix. The diatoms, with silicified cell walls contributing to the mineral content in the loss on ignition test, partly account for the low organic content. A small quantity of minute non-woody axes (fibres) also was observed. The constitution of the peat indicates that it accumulated within a body of water by the deposition over many centuries of enormous quantities of diatoms, airborne pollen and spores, and the somatic remains of aquatic and emergent plants and aquatic invertebrates. Concurrent with this deposition, the inwashing of mineral matter from erosion of the surrounding soil occurred. As the lake filled up, submerged anchored aquatic vegetation (e.g., water lilies and subsequent plants) obtained a footing and contributed their remains to the peat mass. Eventually the lake was filled and covered by a mat such as the present moss and sedge-grass vegetation.

In establishing a fundamental approach and an experimental technique, tests involved only remoulded amorphous granular peat although it is recognized that other structural types of peat might have been chosen.

CONSOLIDATION TESTS

The three parameters, sample height (H), stress (σ), and initial void ratio (e_0), are the most important factors affecting the characteristics of peat consolidation. Other factors such as temperature and atmospheric pressure have an effect but a lesser one. In order to evaluate the individual influence of sample height (H), stress (σ), and initial void ratio (e_0) on the consolidation characteristics, three series of tests were conducted with each of these three parameters individually varied. In these test series the sample height (H) was varied from 0.27 to 6.45 inches (0.69 to 16.38 cm), the applied stress (σ) from 0.45 to 7.03 lb/sq.in. (0.03 to 0.49 kg/sq.cm.), and the initial void ratio (e_0) from 8.6 to 12.4.

Consolidation tests were conducted in a 6-inch (15.24-cm) diameter cylinder, 12 inches (30.48 cm) high, with drainage to the top only; provision was made for the measurement of pore water pressures at the base. A porous stone (Norton # P2120) was fitted into the piston which was sealed to the cylinder walls by two O-rings. Loads were applied by dead weights. In all the tests, only one load was applied to each sample.

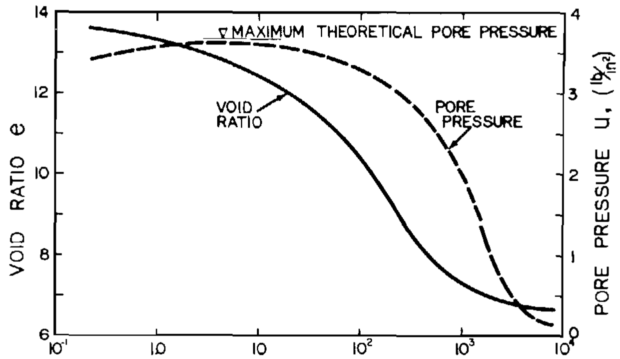

FIG. 2. Typical e–log t graph.

The data, plotted on the regular e–log t graph, are shown on Fig. 2. It indicates that the peat exhibits considerable "secondary" settlement; this is believed to be due to structural modification. On this typical e–log t graph, it is noted that the pore water pressures had unusual characteristics. The maximum pressure does not occur until some time has elapsed and does not reach the full theoretical value. This phenomenon also was observed by others (Taylor, 1942) who attributed it to the plastic resistance associated with "secondary" settlement. A further indication of "secondary" settlement is that the peat continues to settle after the dissipation of excess pore water pressures. These pore water

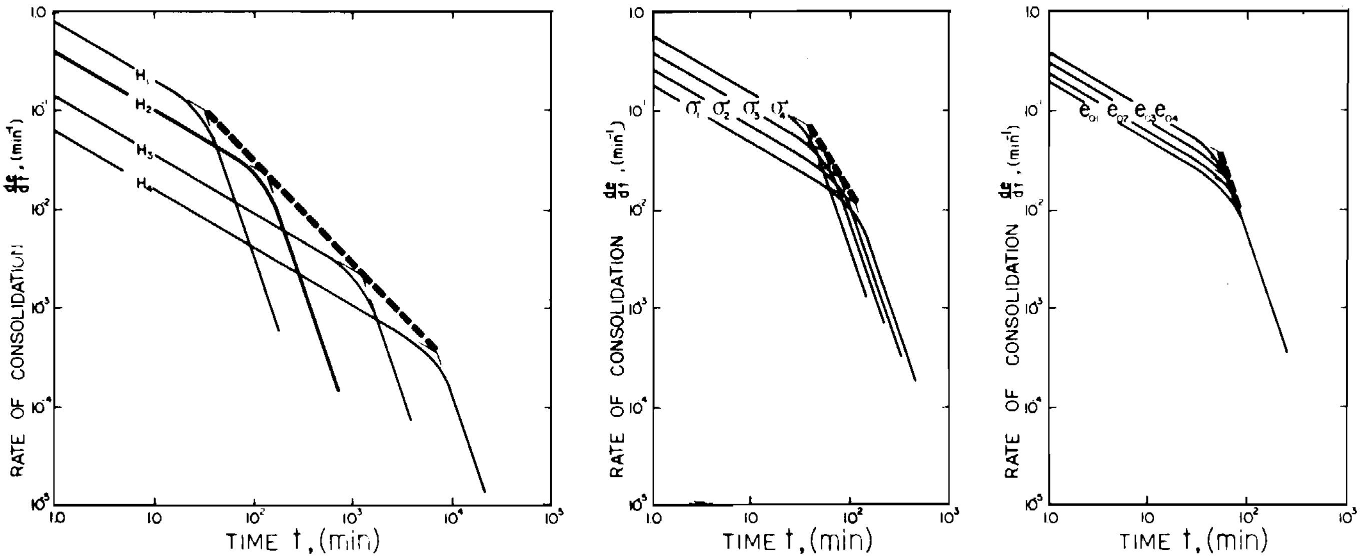

FIG. 3. Log (de/dt)–log t graphs for individual variations of H, σ, and e_0.

pressures do not reduce to zero but have a very low value as volume changes continue and, consequently, a small hydraulic gradient exists.

ANALYSIS OF RATES

The fundamental e-log t graph is used as a basis for analysis; the data are converted to rates of consolidation (de/dt) and plotted as a log (de/dt)–log t graph. Two tangents to a short arc are produced (Fig. 3). This shows that, after some time (t_t) has elapsed following loading, a significant change in the rates of consolidation occurs. The initial tangent, or "early" stage of consolidation, is analogous to "primary" consolidation; the final tangent, or "late" stage of consolidation, is analogous to "secondary" consolidation. The time at maximum curvature (t_t) is analogus to $U =$ 100 per cent, although it occurs slightly earlier than the comparable time as found by the Casagrande construction and much earlier than the time when the pore water pressure approaches zero. The rates of consolidation (de/dt) and the elapsed times (t_t) are dependent upon three parameters: sample height (H), stress (σ), and initial void ratio (e_0). On the log (de/dt)–log t graph, the curves are parallel for variations of H, σ, or e_0 (Fig. 3).

Fig. 3 shows that the sample height (H) and the stress (σ) affect the rates of consolidation in both the "early" and the "late" stages, whereas the initial void ratio (e_0) affects the rates of consolidation to some extent in the "early" stage but has no effect on the "late" stage.

The tangent portions of the curves are described by two equations, one for the "early" stage of consolidation and one for the "late" stage of consolidation. These equations have the general form:

$$\log (de/dt) = \log t^k + \log C$$

or

$$de/dt = C.t^k.$$

The two stages are expressed by:

$$\log (de/dt) = \log t^{k_1} + \log C_1 \text{ — "early" stage}$$

$$\log (de/dt) = \log t^{k_2} + \log C_2 \text{ — "late" stage,}$$

where k_1 and k_2 are the negative slopes of the lines in the "early" and "late" stages respectively; C_1 and C_2 are the intercepts on the (de/dt) axis, when $t = 1$, for the "early" and "late" stages respectively.

For variations in sample height (H) the slopes of the tangents, k_1 and k_2, are fixed for a particular sample of peat, when stress (σ) and initial void ratio (e_0) are kept constant. The initial rate of consolidation (C_1) is a function of $(H, \sigma,$ and $e_0)$. As k_1 and k_2 are obtainable from consolidation tests, $C_1 = f_1\ (H, \sigma, e_0)$, and $t_t = F\ (H, \sigma, e_0)$, it is possible to determine the void ratio of peat under load at any time during the test.

To investigate the influence of the varied parameter $(H, \sigma,$ or $e_0)$ on the log (de/dt)–log t graph, the tangents

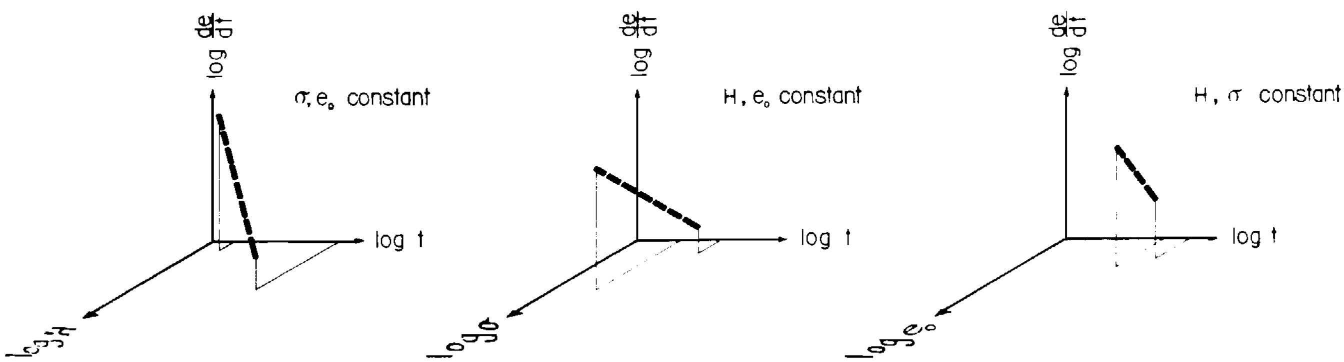

FIG. 4. Three-dimensional log-log-log graphs for individual variations of H, σ, and e_0.

for the "early" and "late" stages are produced to the intersection point (t_t). A straight line is drawn through these intersection points for each test series. This line relates the elapsed time (t_t) to the significant change in rate of consolidation both to the rate of consolidation and to the varied parameter (H, σ, or e_0). For each varied parameter, this line can be plotted on a three-dimensional log-log-log graph and again a straight line relationship exists (Fig. 4). The data for the three tests series, with individual variations of H, σ, and e_0, were checked graphically and were within the range of experimental error.

Using these graphs which indicate that the consolidation characteristics can be described in analytical terms or by graphic means, it is possible to construct a nomograph to determine the void ratio of the peat during the tests.

The void ratio at any time (t_1) during "early" stage of consolidation is determined from the equation $de/dt = C_1 t^{k_1}$. The change in void ratio is obtained:

$$\Delta e_1 = \int_1^{t_1} \frac{de}{dt}\,dt$$

$$= \int_1^{t_1} C_1 \cdot t^{k_1}\,dt$$

$$= \left[\frac{C_1}{k_1+1} t_1^{k_1+1} - 1\right].$$

As $(t_t) = F\ (H, \sigma, e_0)$, the end of "early" stage consolidation and the beginning of "late" stage consolidation can be obtained. The void ratio at any time during the "late" stage can be obtained from the equation

$$de/dt = C_2 . t^{k_2}$$

where $C_2 = f_2\ (H, \sigma, e_0)$. The change in void ratio at any time (t_2) during the "late" stage from the void ratio at the end of the "early" stage is:

$$\Delta e_2 = \int_{t_t}^{t_2} \frac{de}{dt}\,dt = \frac{C_2}{k_2+1}\,[t_2^{k_2+1} - t_t^{k_2+1}]$$

when $t_t < t_2 < \infty$.

If the tangent for the "late" stage is a straight line for an infinite time, the maximum value of Δe_2 is found by letting $t_2 \to \infty$. For this peat $(k_2 + 1)$ is negative. Thus

$$\Delta e_{2(t_2 \to \infty)} = \frac{C_2}{k_2+1}\,[-t_t^{k_2+1}].$$

The total change in void ratio for any time interval is the sum of the void ratio changes in both the "early" and "late" stages; thus

$$\Delta e_{\text{total}} = \Delta e_1 + \Delta e_2 + \Delta\delta,$$

where $\Delta\delta$ is the change in void ratio during the first unit of time.

GRAPHICAL PRESENTATION

A nomograph (Fig. 5) is constructed on the log (de/dt)–log t axes in a manner similar to Fig. 3. As the slopes of the tangents for the "early" and "late" stages (k_1 and k_2) are fixed for variations of one of the parameters (H, σ, or e_0), two series of lines are drawn to represent these tangents.

For a sample of a particular height, the values of C_1 (the rate when t = 1) and t_t (the time to the significant change in the rates of consolidation) can be obtained. Entering these values into the nomograph, it is possible to plot the line relating the rates of consolidation to the elapsed time from loading. The dotted line shown on the nomograph is an example of this procedure and corresponds to line H_2 on Fig. 3a. This typical nomograph has been drawn to show the influence of sample height (H) on rates of consolidation (de/dt) with values of stress (σ) and initial void ratio (e_0) held constant. It is possible to draw three nomographs similar to the development of Fig. 3.

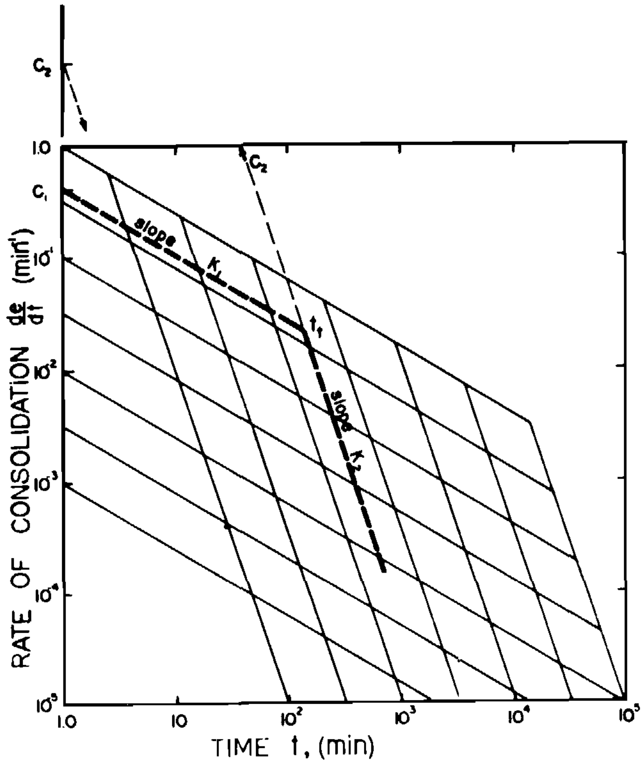

FIG. 5. Nomograph for settlement predictions.

Consequently, the change in void ratios, for any elapsed time during the consolidation process, can be calculated from the values of rates of consolidation (de/dt) and time (t).

CONCLUSIONS

A microscopic examination assists in the interpretation of the unusual consolidation characteristics of the peat, both in terms of rates of settlement and pore-water-pressure dissipation. This examination also may explain the occurrence of plastic resistance as found by Taylor. The physical structure and arrangement of the particles greatly affect the size and continuity of the pores, and hence affect the permeability. The wide range of microstructure found in different peat types is exhibited by Figs. 1 and 6. They represent the amorphous granular peat under discussion and a non-woody fine fibrous peat, respectively.

When a comparison is made of the characteristics of the curves on the log (de/dt)–log t graph, for samples under identical test conditions, the initial rate of consolidation of an amorphous granular peat is lower than that of a fine fibrous peat due to the lower permeability of the former. Furthermore, the differences in permeability result in a greater elapsed time to the significant change in the rates of consolidation for amorphous granular peat. Based on this comparison, a hypothesis can be made of the behaviour

FIG. 6. Fine fibrous peat (magnification, 200 ×).

of these peats under load. In the "early" stage of consolidation, the slope of the line for the fine fibrous peat is steeper than that for the amorphous granular due to the more rapid change in permeability.

The amorphous granular peat is a highly complex structure of descending particle size from fine sand and silt through diatoms, spores, microparticles, to colloids. The fibrous peat is an intricate and complicated cellular structure, with the cells of the hydrophilic plants of which it consists capable of holding immense quantities of water. Consequently, peat under compression may exhibit not only "primary" and "secondary" consolidation characteristics but also tertiary, quaternary, etc., characteristics. Investigation of these phenomena is continuing.

Colloidal phenomena relative to compression characteristics are not yet fully understood. It has been demonstrated previously, however, that amorphous granular peat under compression acts as a quasi-plastic material (Schroeder and Wilson, 1962). Concurrent with the expulsion of water under excess hydrostatic pressure, the particles rearrange their positions and deform. In particlar, floccules of colloids tend to plug the pores and interstices. Although the initial permeability may be high, it rapidly becomes greatly reduced, even under constant load. The "secondary" effects predominate when the rate of plastic deformation of the organic matrix becomes slower than the rate of expulsion of water from the decreasing volume of voids within the matrix. A subsequent effect is the collapse of cell walls of the constituents of the organic matrix.

The complex behaviour of the peat can be explained by the complexity of its structure which varies widely for different peat types (MacFarlane, 1957; Adams, 1963). Nevertheless, it has been demonstrated that one peat type can be analysed mathematically and graphically to permit the prediction of the rates of consolidation at any time during the loading process. On the results obtained herewith, it is proposed that this approach may be valid for other peat types. Research is continuing to verify this and enable the field conditions to be predicted.

ACKNOWLEDGMENTS

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Bearing Capacity of Pile Groups under Eccentric Loads in Sand

Capacité portante d'un groupe de pieux soumis à des charges excentrées dans le sable

H. KISHIDA, DR.ENG., *Post-Doctoral Research Fellow of National Research Council of Canada, Nova Scotia Technical College, Halifax, Canada*

G. G. MEYERHOF, PH.D., D.SC., *Dean, and Head of Department of Civil Engineering, Nova Scotia Technical College, Halifax, Canada*

SUMMARY

Model tests on free-standing pile groups and piled foundations with varying numbers and spacings of piles under central and eccentric loads in loose and dense sands are analysed by an extended theory of bearing capacity of foundations.

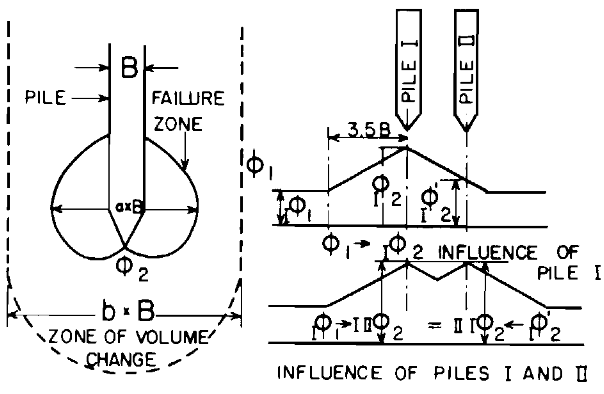
SOMMAIRE

Des essais sur modèle de groupes de pieux isolés et de fondations sur pieux en modèles avec variation du nombre et de l'espacement des pieux sous charges centrées et excentrées dans les sables meubles et denses sont analysés par une extension de la theorie de la capacité portante des fondations.

PREVIOUS RESEARCH on the bearing capacity of pile groups in sands is limited to free-standing groups (pile caps above ground surface) under central loads (Cambefort, 1953; Kezdi, 1957, 1960; Stuart, *et al.*, 1960; Hanna, 1963). A series of model tests has therefore been made on such groups and on piled foundations (pile caps resting on soil) under central and eccentric loads in sands, and the results are compared with an extended theory of the ultimate bearing capacity of pile groups (Meyerhof, 1959, 1960).

THEORY

When a pile is driven into loose sand, its relative density is increased, and the horizontal extent of the compacted zone along the shaft has a width of about 6 to 8 times the pile diameter (Meyerhof, 1959). However, in dense sand, pile driving decreases the relative density because of the dilatancy of the sand, and the loosened zone along the shaft has a width of about 5 times the pile diameter (Kérisel, 1961). From these observations the authors made the assumption that the angle of internal friction ϕ of sand changes linearly with the distance from the pile (where $\phi = \phi_2$), to a radius of about 3.5 times the pile diameter (where $\phi = \phi_1$), as shown in Fig. 1.



(a) (b)

FIG. 1. Characteristic zones near piles in sand: (*a*) zones of failure and volume change near single pile; (*b*) influence zones of change in angle of internal friction near piles in group.

Based on the above-mentioned field data the relationship between ϕ_1 and ϕ_2 in sands may be written

$$\phi_2 = \tfrac{1}{2}(\phi_1 + 40°). \qquad (1)$$

An angle of $\phi_1 = \phi_2 = 40°$ from Eq 1 means no change of relative density due to pile driving; this critical value was confirmed by shear-box tests in which the applied vertical load was 20 per cent of the unit point resistance of single piles. The distribution of ϕ_2 due to pile groups can be obtained approximately by superposition of Eq 1 in the sequence of pile driving (Fig. 1b).

The bearing capacity of a single pile can be estimated by the bearing capacity theory (Meyerhof, 1959) in which an average value of ϕ within the failure zone of an over-all width of about 4 times the pile diameter is used (Fig. 1a). The total bearing capacity of free-standing pile groups is governed either by individual pile failure or by pier failure, whichever gives the lower value (Meyerhof, 1960). For individual pile failure the total bearing capacity can be estimated as the sum of that of the individual piles, which differs from the sum of that of single piles. The ratio between these two values was checked by tests and was theoretically related to the prestressing and change of principal stresses in the sand caused by the adjacent piles. For pier failure the total bearing capacity can be estimated by taking the equivalent pier area limited by the centre of the perimeter piles as outside surface and using an average ϕ within a failure zone of an over-all width of 4 times the equivalent pier width.

The total bearing capacity of piled foundations is estimated from the bearing capacity of free-standing pile groups by allowing for the influence of the pile cap. This influence consists of the bearing capacity of the pile cap and its surcharge effect on the point resistance of the piles of the group, using the whole pile cap for individual pile failure (Fig. 2b) and using the portion of the pile cap outside the equivalent pier area for pier failure (Fig. 2a).

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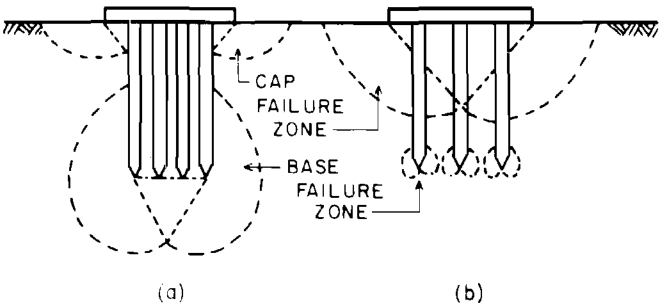

FIG. 2. Failure zones at piled foundation: (*a*) pier failure, (*b*) individual pile failure.

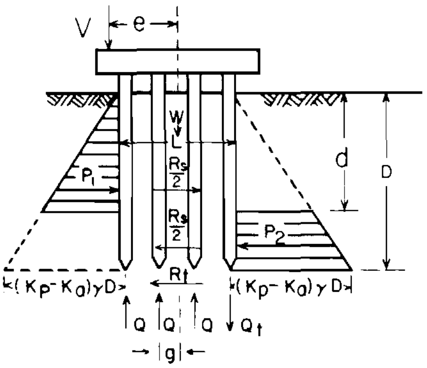

FIG. 3. Free-standing pile group under eccentric load at failure.

The bearing capacity of pile groups under eccentric loads can be estimated by including the lateral forces on the sides of the groups and, for individual pile failure, also the uplift resistance of piles (Meyerhof, 1960). For free-standing pile groups carrying a vertical load V with an eccentricity e (Fig. 3), the equilibrium equations of moments for the rotation, vertical, and horizontal forces are, respectively,

$$Ve = P_1 \frac{(D-2d)}{3} - P_2\left[\frac{(D-d)(D+2d)}{3(D+d)}\right] + R_s\frac{D}{3} + \Sigma Qg + Q_t\frac{L}{2} \qquad (2)$$

$$V = \Sigma Q - Q_t - W \qquad (3)$$

$$P_1 = P_2 + R_t \qquad (4)$$

where D = embedded length of piles, d = depth of rotation of group, L = width of group, P_1 and P_2 = total net horizontal earth pressure on end piles, Q and Q_t = bearing capacity and uplift resistance of individual pile, R_s and R_t = horizontal resistance on side and toe of group, and W = weight of group. The depth of the centre of rotation was estimated at about 0.7D from Eq 4 assuming $P_1 = P_2$.

The moment caused by eccentric load, $V \times e$, is balanced by the moment due to lateral forces on the sides of the pile group until it reaches the maximum value corresponding to the coefficient of passive earth pressure. Within this limit, the eccentricity of load has no effect on the point resistance, while the skin friction on the end piles increases

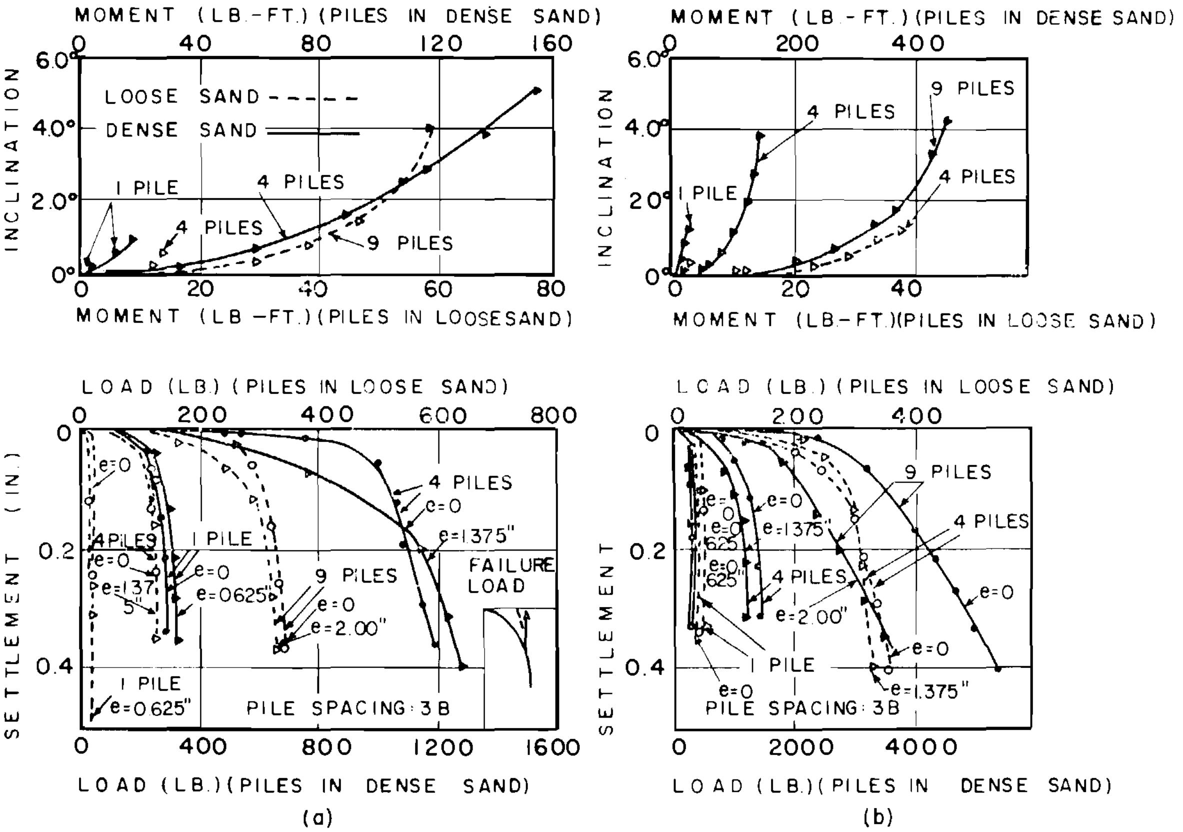

FIG. 4. Typical results of loading tests on model pile groups in sand: (*a*) free-standing pile groups, (*b*) piled foundations.

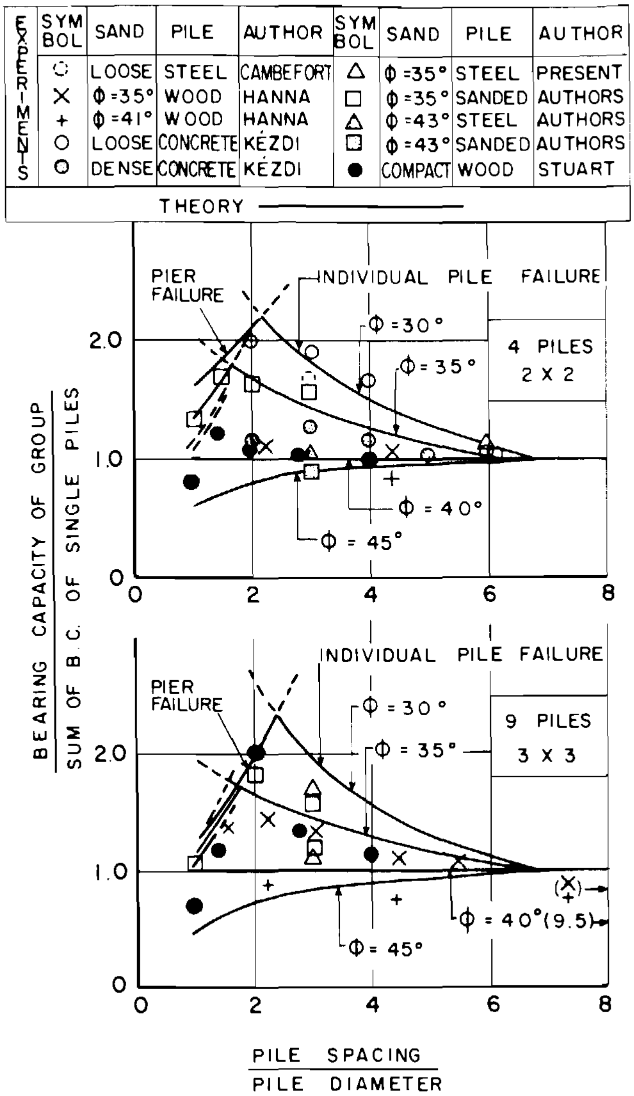

FIG. 5. Bearing capacity of free-standing pile groups under central load in sand.

with the mobilization of the earth pressure, and the total bearing capacity is thus expected to increase slightly. When the moment $V \times e$ is greater than the ultimate moments as a result of side resistance, the difference between these two moments must be balanced by an eccentric point resistance and, for individual pile failure, by any uplift resistance of piles (Fig. 3). This resistance acts on the effective contact width of the foundation as for shallow foundations (Meyerhof, 1953) and the total bearing capacity decreases with an increase of eccentricity.

On this basis the total bearing capacity of pile groups under eccentric load can be estimated from Eqs 2 and 3 on the assumption of a load V for given eccentricity e using a trial and error method. For the calculation of the side resistances, the equivalent width can be assumed as the pile spacing or 3 times the pile diameter whichever is less for individual pile failure, or 3 times the equivalent pier width for pier failure. The suggested theoretical calculations can readily be extended to pile groups, carrying a load V with double eccentricities e_x and e_y on the major axes as for shallow foundations.

ANALYSIS OF TESTS

In order to assess the validity of the theory the present experiments and those of previous investigations will be analysed below. The present model pile groups consisted of steel piles, ½ in. in diameter and 12 in. long, with 60° tips and steel caps, and they were either smooth or rough (sanded). Single piles and square groups of 4 to 9 piles with a spacing up to 6 times the pile diameter were pushed into dry well-graded sand, which was either loose (density of 94 lb/cu.ft. and friction angle of 35°) or dense (density of 117 lb/cu.ft. and friction angle of 43°) and contained in a large steel box. Free-standing pile groups with an embedded length of 11 in. and piled foundations with an embedded length of 12 in. were loaded to failure under different eccentricities of the load applied to the pile cap. The failure load was defined from a load-settlement curve, in which settlements were measured at the point of load application. Typical load-settlement and moment-inclination curves of the foundations are shown in Fig. 4.

For free-standing pile groups under central load the

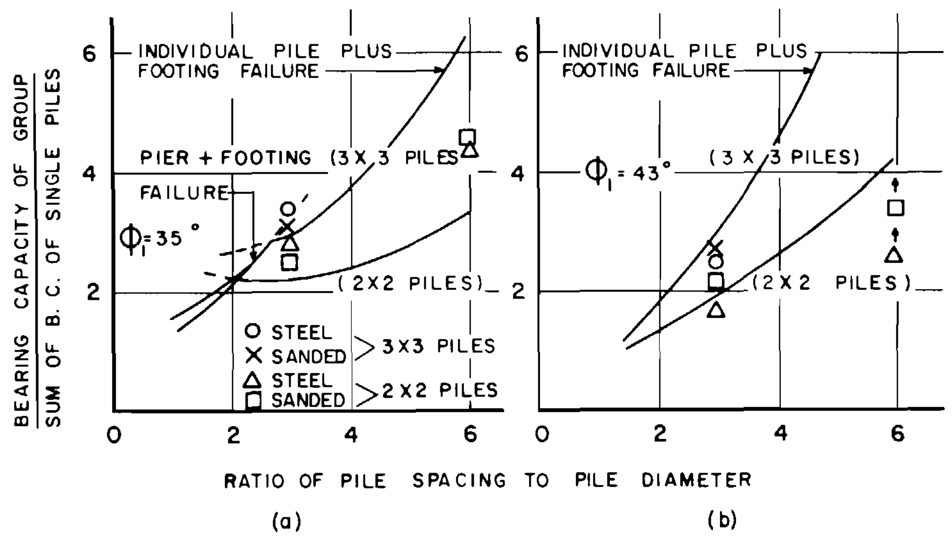

FIG. 6. Bearing capacity of piled foundations under central load in sand: (*a*) loose sand, (*b*) dense sand.

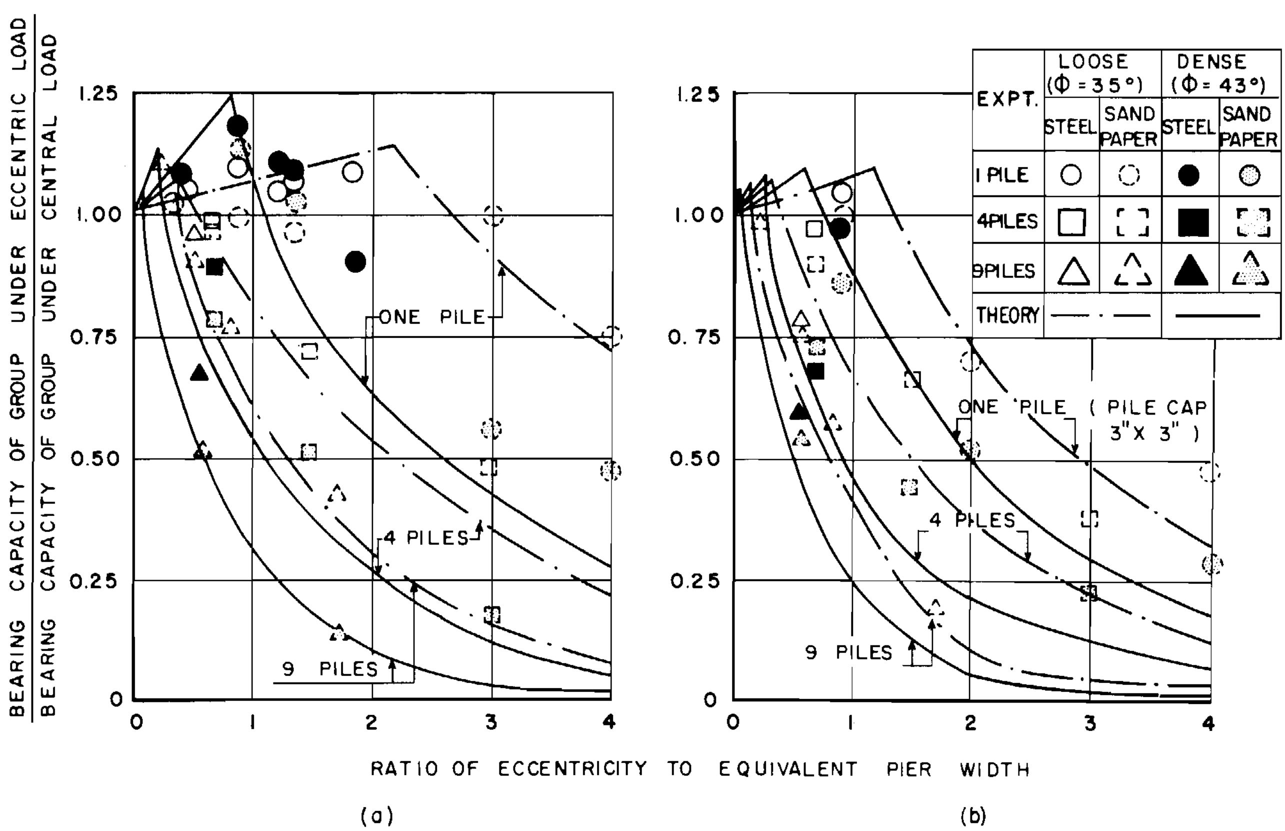

FIG. 7. Bearing capacity of pile groups under eccentric load in sand: (*a*) free-standing pile groups, (*b*) piled foundations.

theoretical and experimental results compare fairly well (Fig. 5). The total bearing capacity of a given number of piles in loose sand increases with smaller pile spacing from about 7 times the pile diameter to a maximum value at about twice the pile diameter. At about this pile spacing the failure criterion changes from individual pile failure to pier failure, when the bearing capacity decreases with smaller pile spacing. In dense sand the total bearing capacity decreases as the pile spacing decreases, and pier failure does not occur because of the effect of dilatancy. The critical value of the angle of internal friction, in which pile spacing has no significant effect on the total bearing capacity, was found to be about 40°. For a given pile spacing, the change of bearing capacity by group action increases with the number of piles, as would be expected.

For piled foundations under central load (Fig. 6) the total bearing capacity in loose sand increases with greater pile spacing and exhibits pier failure up to a pile spacing of about 3 times the pile diameter. For a greater pile spacing the total bearing capacity increases at a smaller rate and tends to support the estimated values. In dense sand pier failure does not occur, and the total bearing capacity is somewhat less than estimated.

For free-standing pile groups and piled foundations under eccentric load in both loose and dense sands the theoretical and experimental results are in fair agreement (Fig. 7). The total bearing capacity of a given number of piles increases somewhat with a small eccentricity of the load to a maximum value, after which the bearing capacity decreases rapidly with greater eccentricities. For a given ratio of eccentricity to equivalent pier width, the total bearing capacity decreases with a greater number of piles of a given length and approaches that of a shallow foundation, as would be expected.

CONCLUSIONS

The total bearing capacity of free-standing pile groups can be estimated from an extended bearing capacity theory of piles, which shows that in loose sand the bearing capacity is greater than that of the sum of single piles because of the compaction of the sand; the reverse holds true for dense sand because of dilatancy of the material. The total bearing capacity of piled foundations can be estimated from the bearing capacity of free-standing pile groups by allowing for the influence of the pile cap using simple methods of analysis.

Small eccentricities of the load have no significant influence on the bearing capacity of free-standing pile groups and piled foundations because the applied moment is mainly resisted by the earth pressure moment on the sides of the group. At larger eccentricities the total bearing capacity decreases rapidly because of smaller point resistance of the group by a reduction of the effective base area, as would be expected theoretically.

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Induced Pore Pressures during Pile-Driving Operations

Pressions interstitielles produites durant le battage de pieux

K. Y. LO, *Supervising Foundation Engineer, Department of Highways, Ontario, Canada*
A. G. STERMAC, *Principal Foundation Engineer, Department of Highways, Ontario, Canada*

SUMMARY

A theory for estimating the maximum pore pressures induced by pile driving is presented. Theoretical results are compared with measurements carried out at three sites, and some published data. The maximum pore pressures may be estimated with reasonable accuracy by the method proposed. Its magnitude depends on the stress history and pore-pressure characteristic of the clay, but is independent of pile dimensions. Within a radial failure zone of the soil surrounding the pile, the pore pressures are constant; outside this zone they decrease rapidly with distance from the pile. The pore pressure generally increases with depth resulting from increase in effective overburden stress. There is no direct summation of pore pressures within the failure zone of a pile owing to the driving of adjacent piles.

SOMMAIRE

On présente une théorie pour évaluer les pressions interstitielles maximales produites durant le battage de pieux. Les résultats théoriques sont comparés aux valeurs expérimentales mesurées en trois sites différents. La théorie proposée permet d'évaluer de façon assez précise la pression interstitielle maxima dont la valeur est fonction des caractéristiques de l'argile et est indépendante des dimensions du pieu. A l'intérieur de la zone radiale de rupture, la pression interstitielle est constante tandis qu'à l'extérieur de cette zone, elle décroit rapidement en fonction de la distance du pieu. En général, la pression interstitielle augmente avec la profondeur à cause d'une augmentation de la contrainte effective. Enfin, on n'a observé aucune augmentation de la pression interstitielle, dans la zone de rupture, due au battage de pieux adjacents.

THE PHENOMENON OF PORE WATER PRESSURES being set up when piles are driven into the ground has long been recognized. It is known that the dissipation of these pore pressures is a governing factor in the increase of bearing capacity of piles. It is also possible that the induced pore pressures may affect the stability of slopes into which piles are driven. In order to explore these problems, a knowledge of the magnitude and distribution of the induced pore pressures with respect to both depth and distance from the pile is essential.

Information on pore pressures set up because of pile driving is rather scanty. Some field measurements have been reported by Bjerrum, *et al.* (1958), Bjerrum and Johannessen (1960), and recently, by Milligan, *et al.* (1962). Theoretical treatments of the problem have been attempted by Soderberg (1962) and Ladanyi (1963). The inadequacy of some theoretical approaches used has been discussed (Lo and Stermac, 1963). In this paper, the behaviour of pore pressures caused by driving a single pile and a group of piles is described. A tentative method of estimating the maximum induced pore pressure is proposed, based on the pore pressure-strain theory (Lo, 1961). Results of field measurements at three sites are compared with the predictions of the theory. The theoretical framework is based on the behaviour of normally consolidated clays, however. The application of the theory to overconsolidated clays therefore requires modification and is presently under consideration. It is also obvious that pore pressures close to the pile tip cannot be predicted by the theory because of end effects.

INDUCED PORE PRESSURE DUE TO SOIL DISPLACEMENT

When a pile is being driven into the ground, the soil is displaced radially by the pile body. An element of soil close to the pile will be deformed as shown in Fig. 1. Within a distance r_0 from the centre of the pile, shearing strain will be so large that complete failure of the soil within this radial zone will occur.

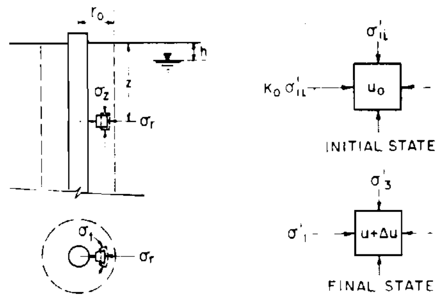

FIG. 1. Stresses in the failure zone around a pile.

Before the pile is driven, the initial stresses in the ground are the vertical effective stress σ'_{1i} and the horizontal effective stress σ'_{3i}, together with the pore pressure u_0. The vertical and horizontal stresses are also the principal stresses. Since the horizontal strain in a natural deposit is zero, σ'_{3i} is equal to $K_0\,\sigma'_{1i}$ where K_0 is the coefficient of earth pressure at rest.

When the pile is driven, the direction of maximum displacement is radial. It follows that within the failure zone, the radial stress becomes the major principal stress.* The resulting maximum excess pore pressure Δu_m will then be composed of two parts, one resulting from the change in total ambient pressure $\Delta\sigma_3$ and the other from shearing as follows:

$$\Delta u_a = (1 - K_0)\sigma'_{1i}, \tag{1}$$

$$\Delta u_s = (\Delta u/p)_m \sigma'_{1i}, \tag{2}$$

*It may be shown that the tangential stress $\sigma_t > \sigma_z$, the vertical stress by using the plastic solution of a thick-wall sylinder subjected to both radial and axial flow under internal and external stresses.

where $(\Delta u/p)_m$ is the maximum pore pressure ratio and p is the consolidation pressure. Hence,

$$\Delta u_m = \Delta u_a + \Delta u_s$$
$$= [(1 - K_0) + (\Delta u/p)_m]\sigma'_{1i}. \tag{3}$$

It is seen, therefore, that the estimation of the pore pressure set up as a result of pile driving involves the determination of the coefficient of earth pressure at rest and the maximum pore pressure ratio.

The pore pressure ratio $\Delta u/p$ is measured in a conventional consolidated-undrained triaxial test with pore-pressure measurements. In this test, the pore-pressure ratio $\Delta u/p$ increases with the applied stress difference, then reaches its maximum $(\Delta u/p)_m$, and remains constant after a certain strain is attained. It has been shown that the ratio $\Delta u/p$ is independent of the direction of the stress path, time of sustained loading, and the consolidation pressure for normally consolidated clays (Lo, 1961; Bjerrum and Lo, 1963). Experimental evidence also suggests that $(\Delta u/p)_m$ is independent of the stress system employed (Lo, 1963). The advantage of using the pore-pressure ratio $(\Delta u/p)_m$ is therefore apparent. In this paper, the prediction of excess pore pressure due to pile driving will be limited to its maximum value as already described.

CASE 1, WALLACEBURG

A comprehensive instrumentation programme was carried out at a site 3.5 miles west of Wallaceburg at the southwestern tip of Ontario. The subsoil stratigraphy, together with the results of field and laboratory tests on 3-in. piston samples from a typical borehole, are shown in Fig. 2. The points for laboratory tests represent the average result of at least two experiments. *In-situ* shear tests were performed by vanes manufactured both by Geonor and by the Ontario Department of Highways. The line plotted represents the statistical line for a large number of tests at close intervals with the Norwegian vane.

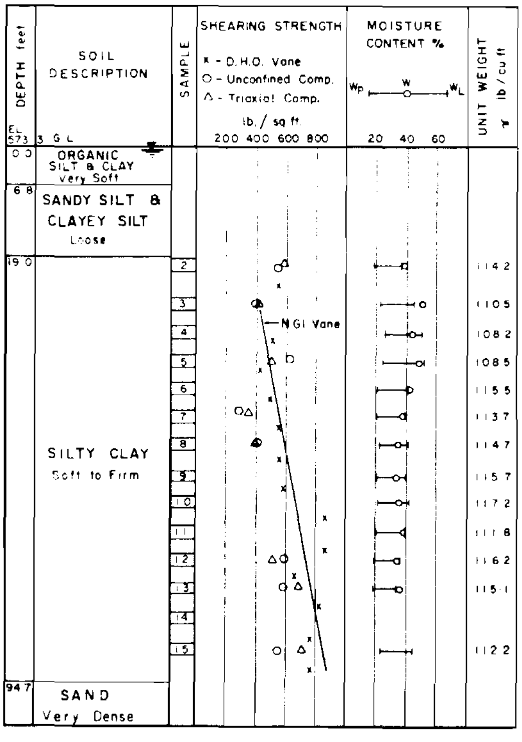

FIG. 2. Results of a typical borehole, Wallaceburg.

The sensitivity of the clay varies from 4 to 6. Consolidation tests showed that the clay is normally consolidated.

Four series of triaxial tests with pore-pressure measurements were performed on samples isotropically consolidated, and one series of tests was carried out on samples cut horizontally. The pore-pressure ratios $(\Delta u/p)_m$ are plotted in Fig. 3 against consolidation pressure for both vertical and

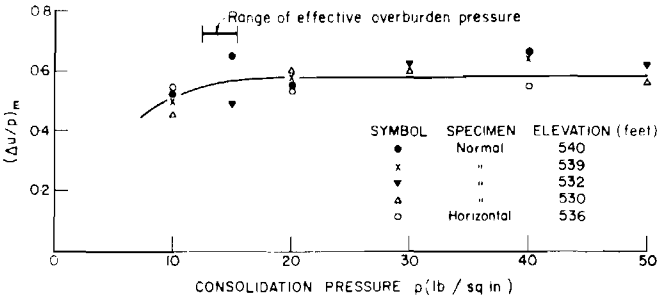

FIG. 3. Relation of $(\Delta u/p)_m$ and p for Wallaceburg clay.

horizontal samples. No significant difference in the pore-pressure behaviour was found between the two types of tests. A value for $(\Delta u/p)_m$ of 0.57 is representative in the normally consolidated range. The coefficient of earth pressure at rest K_0 from five tests was found to be 0.5.

Instrumentation

Two model steel piles, 3.5 in. in diameter, with closed ends were driven by a drop hammer weighing 350 lb falling 30 in. Eight piezometers were installed at different distances and depths from the piles as shown in plan in Fig. 4a and Table I. The first pile (A) was driven to 45 ft and the second (B) to 48 ft. The two piles were spaced as close

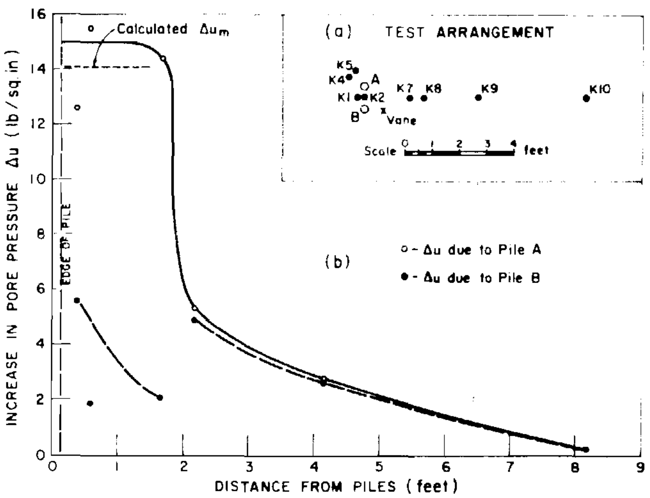

FIG. 4. (a) locations of piezometers; (b) distribution of induced pore pressures, Wallaceburg.

together as possible (6.5 in. edge to edge) with one group of piezometers in between so as to study the effect of the second pile on the pore pressures set up by driving the first pile. Three piezometers were installed 20 ft away from the test site to determine the groundwater conditions and possible fluctuations during the test.

Results of Field Measurements

The results of pore-pressure measurements are plotted in Fig. 5. The pore pressures due to the driving of each pile are quite easily discernible. Observations show that the pore pressure starts to increase when the tip of the pile is about

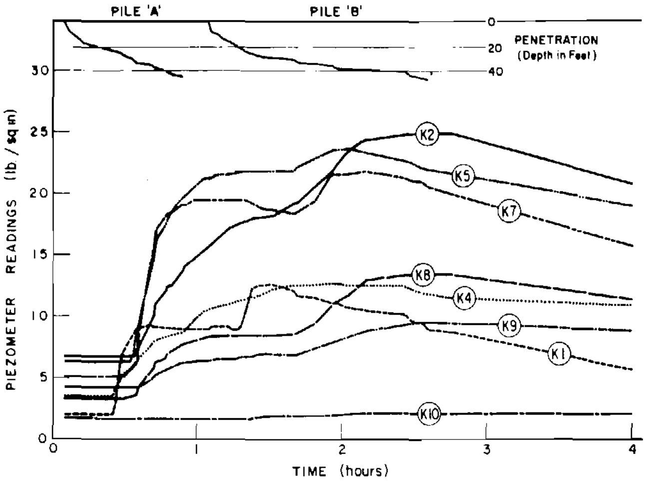

FIG. 5. Results of pore-pressure measurements, Wallaceburg.

3 ft from the piezometer tip, then increases very rapidly when the pile passes by the tip. The maximum pore pressure for piezometers close to the pile is registered some 20 min after the pile reaches the piezometer tip. This delay is probably due to the time response of the piezometers. For piezometers at farther distances from the pile, some small effect of the redistribution of pore pressures from regions of high pressures to regions of low pressures is present.

The pore pressures at different depths and distances induced by driving each pile are presented in Table I. In Fig. 4b, the results for piezometers 35 ft deep are plotted against distance from the piles. The sharp drop in pore pressures between 1.5 and 2.0 ft from the pile is evident, suggesting the existence of a failure zone of high pore pressures around the pile as postulated in the theory. The pore pressures set up by the second pile are only about 30 per cent of those due to the first pile within the failure region. However, outside this zone the summation of pore pressures is 100 per cent. These results are of considerable theoretical

TABLE I. RESULTS OF PORE-PRESSURE MEASUREMENTS, WALLACEBURG

Piezometers				Δu (lb/sq. in)			Δu_m
		Distance (in.)		Due to driving			
No.	Depth (ft)	Pile A	Pile B	Pile A	Pile B	Total	Calculated
K-1	25	6.0	6.0	7.2	3.5	10.7	9.9
K-4	25	7.5	14.0	7.5	1.6	9.1	
K-2	35	5.0	5.0	12.6	5.6	18.2	14.0
K-5	35	7.5	17.5	15.5	1.8	17.3	
K-7	35	20.0	20.0	14.4	2.1	16.5	
K-8	35	26.0	26.0	5.3	4.9	10.2	
K-9	35	50.0	50.0	2.7	2.6	5.3	
K-10	35	98.0	98.0	0.2	0.2	0.4	

and practical significance. It immediately suggests that when a cluster of piles is driven, the pore pressures due to each individual pile will not sum up directly, but will remain constant at a maximum value. This fact will be illustrated in another case record subsequently.

The calculated maximum pore pressures at 25 and 35 ft are respectively, 9.9 and 14.0 lb/sq.ft. and are shown in Table I. The calculated values agree reasonably with the measured pore pressures induced by the first pile.

CASE 2, GHOST RIVER

The site at which measurements were carried out was approximately 20 miles east of Matheson in Northern Ontario. The soil stratigraphy is shown in Fig. 6, and some relevant soil properties may be found in Fig. 7. Artesian pressures were measured in all strata. A series of triaxial tests was carried out on samples taken from the varved clay stratum. The pore pressure ratio $(\Delta u/p)_m$ was found to be 0.65. The varved clay is slightly overconsolidated.

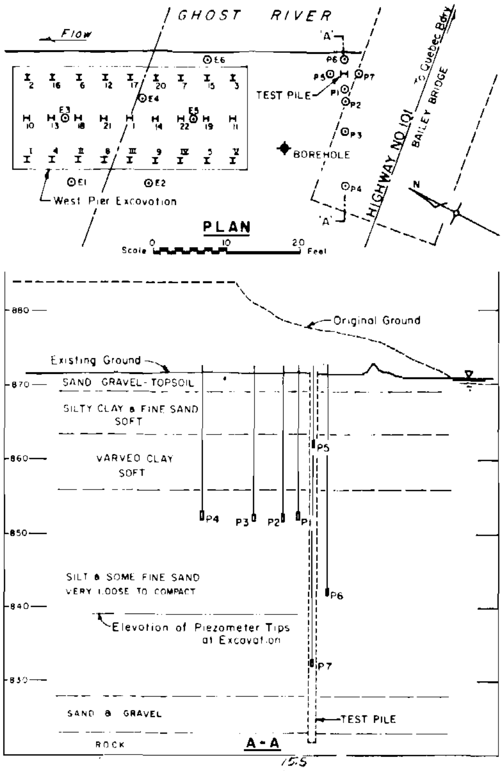

FIG. 6. Locations of piezometers and pile-driving sequence, Ghost River.

Attempts to recover undisturbed samples from the silt stratum were unsuccessful. Consequently, no triaxial tests were performed. However, both the standard and cone penetration tests showed that the silt is very loose except near the bottom of the stratum. The behaviour of the silt during shear is probably similar therefore to that of the silt laminae in the varved clay stratum. Since the triaxial samples of varved clay are composed of more than 70 per cent of silt, it is not unreasonable to assume that the ratio $(\Delta u/p)_m$ of the silt is of the same order of magnitude.* The coefficient of earth pressure at rest is taken as 0.4.

Instrumentation

Piezometers were installed at a test pile outside the excavation and at the excavation to observe the pore pressures resulting from driving a cluster of piles. The locations of the piezometers are shown in Fig. 6.

At the test pile (10 BP 42 steel H-pile), seven piezometers were installed. Piezometers P-1, P-2, P-3, and P-4, were placed at a depth of 20 ft at distances of 2, 4, 8 and 15 ft from the centre of the pile. P-5, P-1, P-6 and P-7 were put down at 10, 20, 30, and 40 ft depths at equal distances of 2 ft from the pile. At the pier excavation, six piezometers were placed at Elevation 839, 25 ft below the bottom of the excavation.

*Tests carried out on a fine very loose sand by Bjerrum, Kringstad, and Kummeneze (1961) showed similarly high pore pressure ratios.

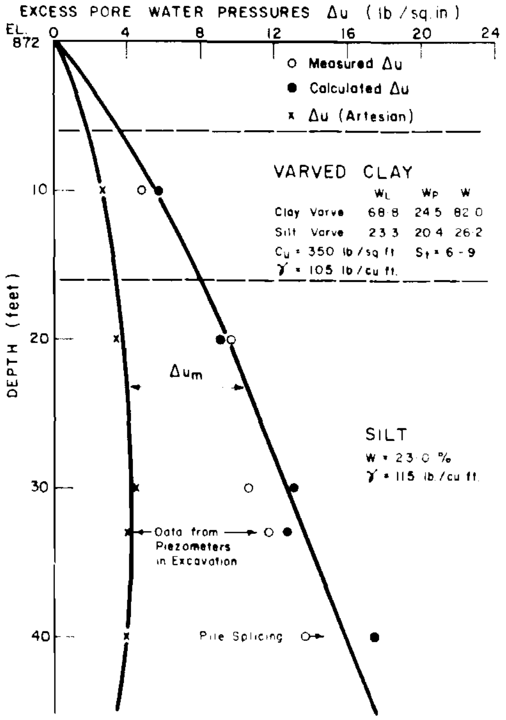

FIG. 7. Comparison of calculated and measured excess pore water pressure, Ghost River.

Results of Field Measurements

The results of measurements for the test pile are shown in Fig. 7. It is evident that the magnitude of the induced pore pressures increases with depth. Induced pore pressures at 20-ft depth and at 2, 4, 8, and 15 ft away from the pile were 6.2, 5.1, 2.3, and 0.8 lb/sq.in., decreasing with distance from the pile. It is of interest to note that the maximum pore pressure is registered almost instantaneously when the pile tip passes by the piezometer tips for the piezometers which are closest to the pile, probably as a result of the relatively high permeability of the silt. When the pile reached bedrock, it was given 100 blows by a Delmag diesel hammer with a rated energy of 22,500 ft-lb, but no increase in pore pressure was observed. These observations illustrate further that the pore pressures are predominantly strain-controlled.

The predicted and measured induced pore pressures at different depths are shown in Fig. 7 for piezometers 2 ft away from the centre of the pile. Data from measurements in the pier excavation are also plotted in the same figure. The calculated and measured values are in reasonable agreement. The measured pore pressure at 40-ft depth is too low because of the elapsed time required for splicing of the pile at 36 ft. Some dissipation of pore pressure must have occurred during this period. The artesian pressures existing at the site are also plotted in the same figure.

Fig. 8 shows typical results recorded in piezometers E-3, E-4, and E-6 at the excavation during the driving of a cluster of piles. The numerals indicate the sequence of pile driving corresponding to those shown in Fig. 6. The piles were driven at a fairly rapid rate, but the figure shows that a certain value of pore pressure was not exceeded. In fact the maximum value attained is approximately the same as that found in the driving of a single pile as shown in Fig. 7. It is clear, therefore, that within the failure zone of an individual pile there is no direct summation of pore pressures due to adjacent piles in a piled foundation.

CASE 3, WABI RIVER, NEW LISKEARD

The layout of the foundation piles (14 BP 73 steel H-piles) together with the piezometers, is shown in Fig. 9a.

The soil conditions were determined by extensive borings and in one borehole, continuous 3-in. piston samples were taken to a depth of 41 ft. Briefly, the soil profile consists of 9 ft of mottled grey and brown silty clay, followed by a laminated clay to a depth of 23 ft. A thick deposit of varved clay follows and extends to bedrock at 150 ft.

In the varved clay stratum, the thickness of the silt layers is 0.5 in. while that of the clay layers varies from 0.5 to 1 in. The liquid and plastic limits of the silt laminae average 28 and 20 per cent and those of the clay laminae 65 and 26 per cent. The moisture contents of the silt and clay layers are respectively 27 and 68 per cent. The undrained shear strength of the bulk sample is 700 lb/sq.ft. and the sensitivity is approximately 10. The unit weight averages 110 lb/cu.ft.

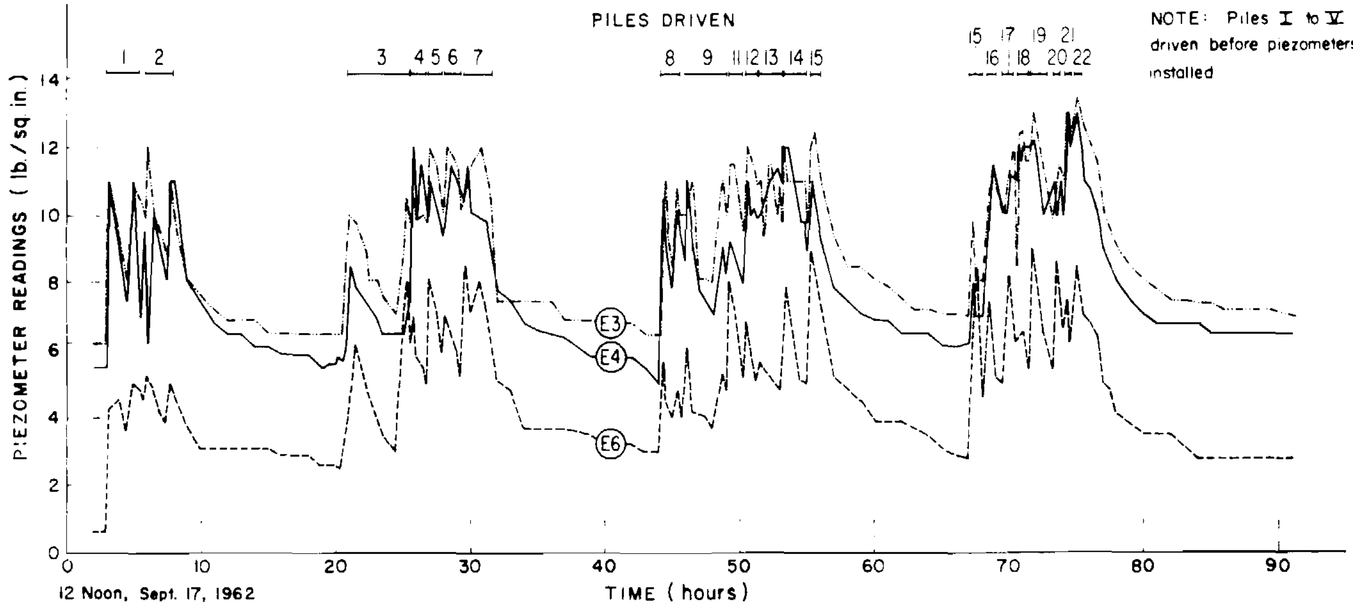

FIG. 8. Typical results of measured pore pressure in a piled foundation, Ghost River.

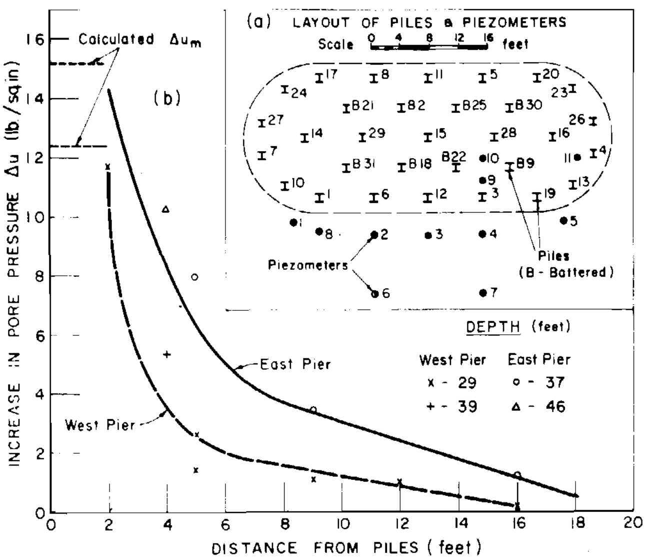

FIG. 9. (a) layout of piles and piezometers; (b) distribution of pore pressures, Wabi River.

The varved clay is slightly overconsolidated. The ratio $(\Delta u/p)_m$ determined from specimens cut horizontally from tube samples is 0.42. The coefficient of earth pressure at rest was found to be 0.5. Details of results of triaxial tests will be reported elsewhere (Lo, 1964).

Results of Field Measurements

The increase in pore pressure due to pile driving is plotted against distance from the centre of the pile in Fig. 9b. It is clear from the figure that the induced pore pressures increase with depth at the same radial distance from the pile. The short dotted lines indicate the estimated maximum induced pore pressures at depths of 29 ft and 37 ft. The agreement between calculated and observed values is not unreasonable.

Similar results as those shown in Fig. 8 at the Ghost River site were obtained when the foundation piles were driven. These results are therefore not presented herein.

SOME PUBLISHED CASE RECORDS

1. Bjerrum, *et al.* (1958) observed that the induced pore pressure within the foundation of 35-cm-square concrete piles in a soft clay was as high as the effective overburden. The soft clay has a sensitivity of approximately 3. $(\Delta u/p)_m$ may therefore be assumed to be 0.6 and K_0 is probably 0.5. From Eq 3, the estimated Δu_m is 1.1 times the effective overburden.

2. Bjerrum and Johannessen (1960) recorded induced pore pressures of 5 tons/sq.m. at depths of 7.5 m and 10 m depth in a foundation of 20-cm-square, hollow-box steel piles. The sensitivity of the soft clay is 6. The $(\Delta u/p)_m$ ratio may be taken as 0.7 and K_0 assumed to be 0.6. The estimated values of Δu_m are 7.5 and 9.6 tons/sq.m., respectively. Since the closest piezometers were 1 m away from the pile, the maximum induced pore pressures were not measured. However, the observed values lie below the predicted pore pressures.

CONCLUSIONS

A theory based on the pore pressure-strain relationship for estimating the maximum induced pore pressure due to pile driving has been developed. The predictions of the theory are compared with field measurements in three case records, and some available information in the literature. The results of this study lead to the following conclusions, applicable to normally consolidated and very slightly overconsolidated clays or loose silts.

1. The maximum induced pore pressure by driving a single pile may be estimated with reasonable accuracy by the method described.

2. The magnitude of the maximum induced pore pressure depends on the pore pressure ratio $(\Delta u/p)_m$, and the initial state of stress in the ground, but is independent of the dimension or type of pile.

3. As a result of the increase in effective overburden pressure, the induced pore pressures increase with depth at the same radial distance from the pile. For the case records studied herein, the maximum induced pore pressure varies from 1.0 to 1.3 times the initial effective overburden stress in the ground.

4. Within the failure zone of the soil surrounding the pile, the induced pore pressures are maximum and equal. Driving of adjacent piles only increases the pore pressure slightly. Outside this zone, the induced pore pressures by driving adjacent piles sum up directly until the maximum value is attained. Therefore, the maximum pore pressures induced by driving a number of piles in a pile foundation may be predicted.

5. Outside the failure zone of a pile, the pore pressure decreases rapidly with distance. At a distance approximately 16 times the diameter of the pile, the pore pressure is practically negligible. This result has been reported by Bjerrum and Johannessen (1960), and Milligan, *et al.* (1962).

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