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A COMPARISON OF LABORATORY RESULTS WITH IN-SITU PROPERTIES OF LEDA CLAY

BY

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ANALYZED

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

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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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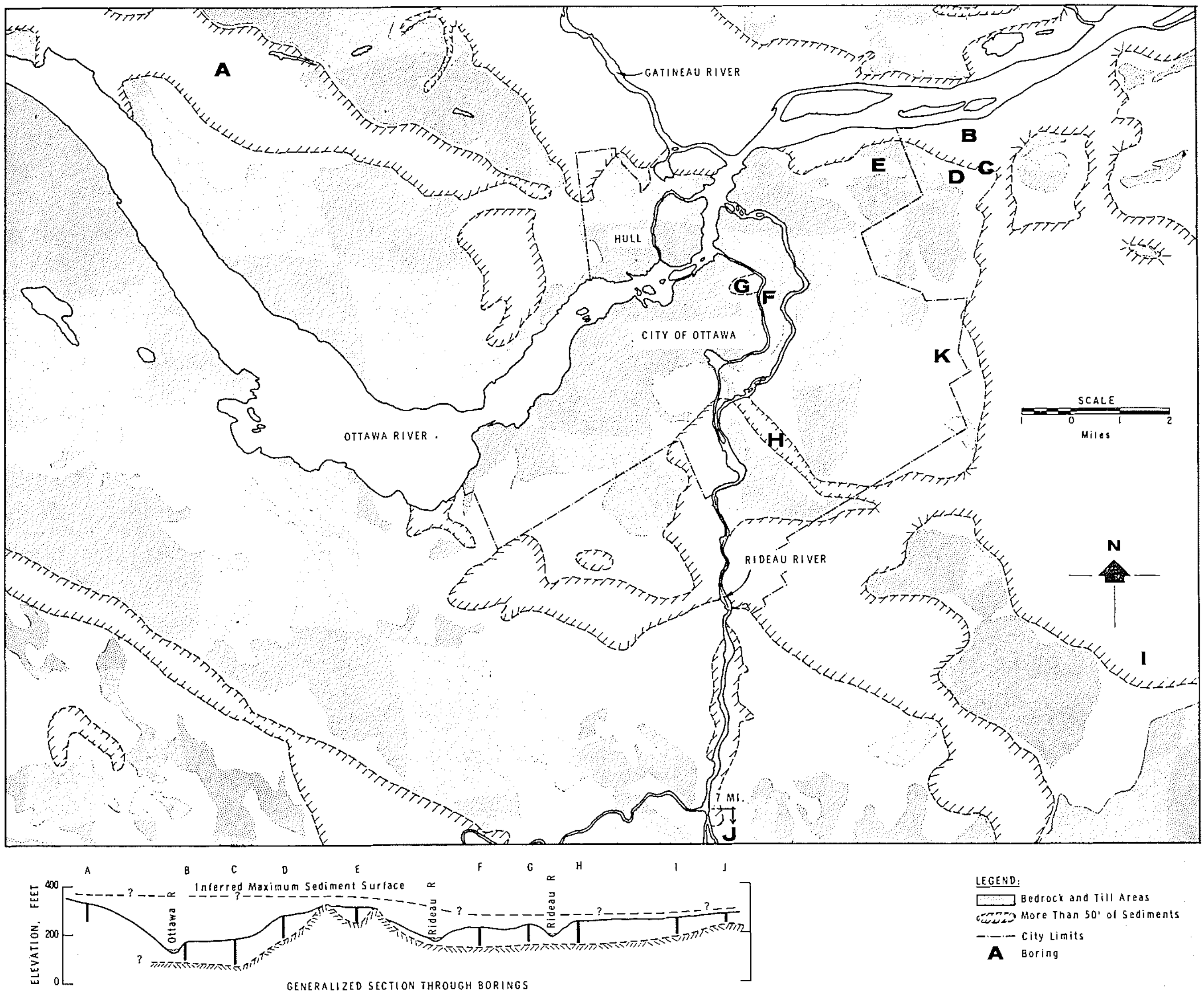

FIG. 1. Map of Ottawa region (geology after Gadd (1963) and Brandon (1961)).

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, %	Averages: I_P %	Averages: I_L	Averages: -2μ, %	Averages: γ, lb/cu.ft.	Std. error, kg/sq.cm.: p_n	Std. error, kg/sq.cm.: S_u	Remarks
A	Brecken-	330	20	.67	.36	1.2	.30	20–	0.4–	79	36	1.4	80	96	±.24	±.07	Uniform clay, low carbonates. Drying crust extends to 20 ft. Extensive clay deposit to great depths.
	ridge		100	2.14	.91	2.9	.31	150	3.4								
B	Sewage	171	20	.38	1.00	3.0	.33	25–	.1–	55	19	2.0	66	105	—	±.15	Highly plastic clay to 50 ft, medium sensitivity. Below 50 ft, somewhat coarser, extremely sensitive.
	plant		70	1.44	1.00	4.1	.25	500+	2.1								
BB	Sewage	171	30	.56	1.47	4.5	.33								—	—	
	plant		70	1.44	1.87	5.2	.37										
	Green		20	.68	.75	3.1	.24	5–	.6–								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.
C	Creek	182	100	2.38	1.98	4.9	.40	25	13.7	66	40	0.9	70	104	±.44	±.20	
	Fill																
	Green		20	.69	.64	2.0	.32	20–	.1–								Fine clay throughout. Drying crust to 20 ft. Extremely sensitive at greater depths. Higher terrace than B.
D	Creek	277	90	2.02	1.02	3.0	.34	500	.5	69	34	1.7	77	101	±.34	±.12	
	Slide																
E	National	310	30	.83	.55	1.8	.30	15–	.1–	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.
	Research		60	1.40	.77	2.3	.33	500+	1.5								
F	Main St.	222	15	.52	.57	2.3	.25	15–	.1–	51	22	1.7	50	112	±.49	±.15	Highly plastic clay at surface becoming siltier with depth. Extremely sensitive at depth. Isolated clay pocket.
			65	1.74	1.03	3.9	.26	500+	.6								
G	National	235	20	.86		1.6		10–	.1–	68	40	2.2	56	105	—	—	Same as for F.
	Museum		50	1.45		2.4		150	1.8								
H	Heron Rd.	250	20	.70	.58			20–	.3–	49	14	2.4	51	108	—	±.14	Highly plastic clay becoming siltier and extremely sensitive with depth.
			90	2.20	.75			70	.5								
I	Gloucester	260	10	.16	.21	0.6	.35	30–	.5–	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.
			60	.96	.56	1.7	.33	500+	2.1								
J	Kars	280	20	.38	.29	1.2	.24	25–	.2–	60	20	2.0	60	101	±.22	±.07	Extremely sensitive clay layer from 20 to 50 ft. Alluvium and drying crust to 20 ft.
			50	.95	.52	2.0	.26	500+	.7								
K	Walkley Rd.	255	15		.50										—	±.07	
			45		.70									-			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

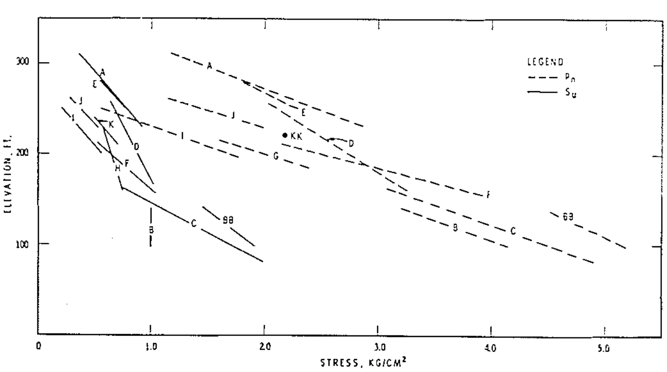

FIG. 2. p_n and S_u in relation to elevation.

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.

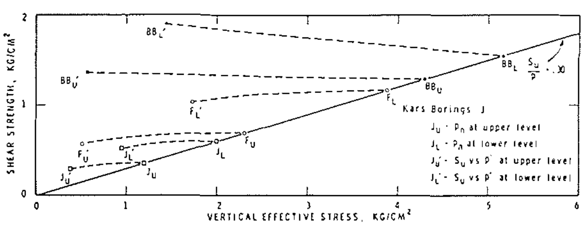

FIG. 3. Relation between preconsolidation pressure, strength, and existing effective stresses for three locations.

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. $\tan^{-1} S_u/p$ 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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