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QUICK CLAYS OF EASTERN CANADA

BY

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LES ARGILES SENSIBLES DE L'EST DU CANADA

SOMMAIRE

Le présent exposé décrit les résultats de recherches poursuivies au cours des quinze dernières années sur les argiles sensibles de l'est du Canada. La composition et la structure minérales de ces argiles sont semblables à celles d'autres argiles marines de la période glaciaire mais elles contiennent souvent dans leurs interstices une eau à teneur en sel exceptionnellement faible. L'existence de fortes liaisons rigides entre les particules de ces dépôts qui sont surtassés apparaît clairement. Les liaisons se rompent quand le terrain est chargé et ceci peut entraîner des forts changements de volume ou une diminution considérable de résistance dans le cas où l'égouttement ne peut se faire. L'auteur a mené à bien quelques études sur les caractéristiques physico-chimiques du sol et sur sa constitution mais la plupart des recherches menées jusqu'à ce jour ont consisté en observations, au laboratoire et sur le terrain, de la réaction de la structure du sol aux variations de contrainte. La forte influence de la vitesse de chargement a montré la nécessité d'interpréter soigneusement les essais de laboratoire pour les appliquer sur le terrain.

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RECENT QUICK-CLAY STUDIES, 6

QUICK CLAYS OF EASTERN CANADA

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SUMMARY

This paper describes the results of research carried out during the past fifteen years on the quick clays of eastern Canada. The mineral composition and structure of these clays is similar to that of other glacial marine clays but they often exist with an unusually low salt content in the pore water. There is evidence of strong rigid bonds between particles of those deposits that are overconsolidated. The bonds are broken when the soil is loaded and this may lead to large volume changes or to exceptional loss of strength if drainage is not permitted. Some studies have been made on the physico-chemical characteristics of the soil and on its fabric but most of the research to date has involved laboratory and field observations of the response of the soil structure to stress changes. The important influence of rate of loading has revealed the need of careful interpretation of laboratory tests for field application.

INTRODUCTION

"Quick clay" is a class of natural soils with unusual characteristics. The most striking feature of a quick clay is its tendency to transform from a relatively brittle material to a liquid mass when it is disturbed. The engineering implications of dealing with such a material are readily apparent. In Canada, quick clays occur commonly in the valleys of the St. Lawrence and Ottawa Rivers, and in recent years a considerable amount of geological and engineering research has been devoted to them.

GEOLOGY

Toward the close of the last glacial period, the northern part of North America was depressed several hundred feet. When the ice finally retreated, the present St. Lawrence River valley was invaded by marine waters. The extent of this arm of the ocean called the Champlain Sea is outlined on Fig.1.

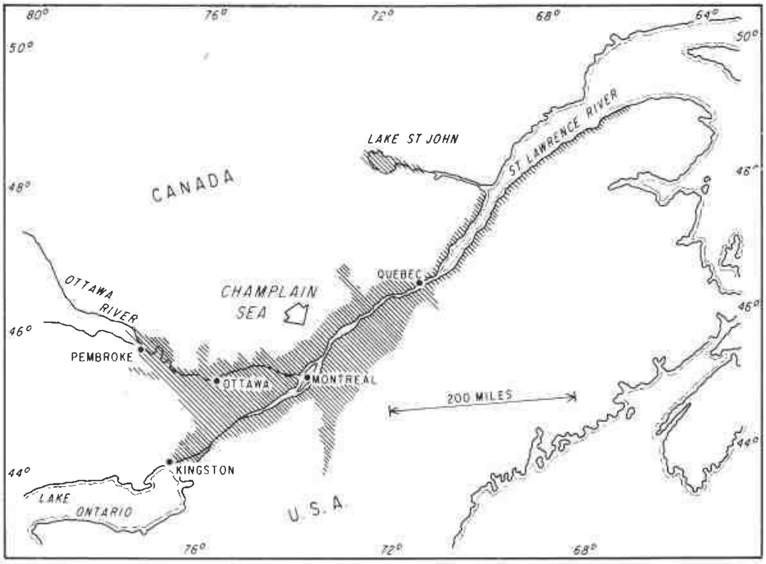


Fig.1. The Champlain Sea.

The Champlain Sea existed for about 4,000 years, from 12,000 years B.P. to 8,000 years B.P. (KARROW, 1961). During that period the land rose by more than 700 ft. (230 m) by isostatic adjustment and the eustatic rise in sea level was more than 100 ft. (33 m) (KENNEY, 1964). The highest recorded marine level is 690 ft. (226 m) above the present mean sea level. This is near Ottawa, where the maximum sea depth has been estimated at several hundred feet (JOHNSTON, 1917).

In a geological review of the Champlain Sea, KARROW (1961) described the origin of the sediments that accumulated in it. In the early stages these were derived primarily from stream erosion of land underlying or adjacent to the retreating ice. Deltas were built where the streams entered the sea leaving typical deposits of variable material, the finer sediments being carried farthest from shore. Underwater slumping created further variations in the final sedimentary deposits. During later stages, as the land level rose isostatically, some of the original marine deposits were eroded from the shorelines and redeposited in the sea.

Recent geological work in the Ottawa area suggests possible major redeposition of marine clays into fresh water (GADD, 1962). It is reasoned that the salinity of the water changed rapidly due to a sudden influx of fresh water (probably from the Great Lakes region) accompanied by major erosion. This opinion is based on the occurrence of large volumes of oxidized non-calcareous and non-fossiliferous clays lying unconformably on unoxidized, calcareous, and fossiliferous clays. This would explain the low salt content in the Ottawa clays (CRAWFORD and EDEN, 1965); however, if these upper clays were actually deposited in fresh water, some explanation must be found for their open structure and sensitivity.

The structure may be due to flocculation caused by residual cations remaining even after washing by the fresh water.

The sensitive fine-grained sediments of the Champlain Sea are called "Leda" clays in the engineering literature, a name said to have been first used by the early Canadian geologist Dr. J. W. Dawson.

COMPOSITION OF LEDA CLAY

Few samples of Leda clay have been subjected to mineral analysis. X-ray diffraction studies on a dozen samples from the St. Lawrence valley revealed chlorite, illite, amphibole, quartz, and feldspar in all samples (Karrow, 1961). Brydon and Patry (1961) carried out extensive tests, including mechanical, chemical, and mineralogical analyses, on 18 samples from four sites in the Ottawa area, and found no marked differences in mineralogy. Quartz, feldspars, amphiboles, micas, and chlorites were found in fractions of all sizes. Most of the samples were slightly calcareous, the coarser material containing the higher amount of carbonate. The origin of the clays was attributed to the igneous and metamorphic rocks of the Canadian Shield. In a study of three samples of marine clay, Allen and Johns (1960) reported hydrous micas, chlorite or vermiculite as the abundant clay minerals.

STRUCTURE

The "structure" of a clay soil is a complex space frame of solid particles held together by plastic and rigid bonds. The "fabric" refers to the size, shape and geometric arrangement of particles, ranging from a parallel to a random or card-house orientation. Quick clays have a structure that is "sensitive" to distortion and their structural properties change dramatically when they are strained.

The Leda clays, like other sensitive clays, are thought to have a card-house fabric because of their environment during deposition. Particle dimensions range from about 50m μ to about 50 μ, or by a factor of about 1000. The coarser fractions are probably inert materials of more or less spherical shape and the finer fractions are the elongated or plate-shaped clay minerals. The clays are generally "inactive" (Eden and Crawford, 1957).

Several undisturbed specimens of Leda clay have been viewed with a scanning electron microscope (Gillott, 1968). Water was removed from the specimens by freeze drying and the viewed surface was cleaved on a chilled metal block by a sharp blow from a chilled chisel while the specimen was frozen. Typical micrographs, magnified approximately 4500 times, are shown in Fig.2. Fig.2A shows the components of a clay soil (62% clay size) which has a natural water content of 58% and a liquid limit of 53%. It has a measured preconsolidation pressure of 2 tons/sq. ft. (kg/cm^2). Fig.2B shows a clay from the same general area but with a preconsolidation pressure of 5 tons/sq.ft. (kg/cm^2), a natural water content of 55%, and a liquid limit of 31%. Both micrographs are of horizontal planes and similar views on vertical planes suggest a random orientation of particles in Fig.2A and a more parallel arrangement in Fig.2B. This difference is anticipated from the differences in overconsolidation.

Efforts have been made to measure the change in fabric of Leda clay during anisotropic consolidation using an X-ray diffraction technique (QUIGLEY and THOMPSON, 1966). The technique is based on the principle that if the clay platelets are oriented along a particular plane, the intensity of basal X-ray reflections obtained by X-raying this plane will be greater than that on other planes. Undisturbed specimens were subjected to various vertical pressures up to 64 tons/sq. ft. (kg/cm^2), removed from the oedometer and impregnated with a polyethylene glycol. A smooth horizontal surface was ground and each specimen was subjected to X-ray analysis. The degree of parallelism was inferred from measurements of the amplitude of the illite peaks obtained on the smooth surfaces.

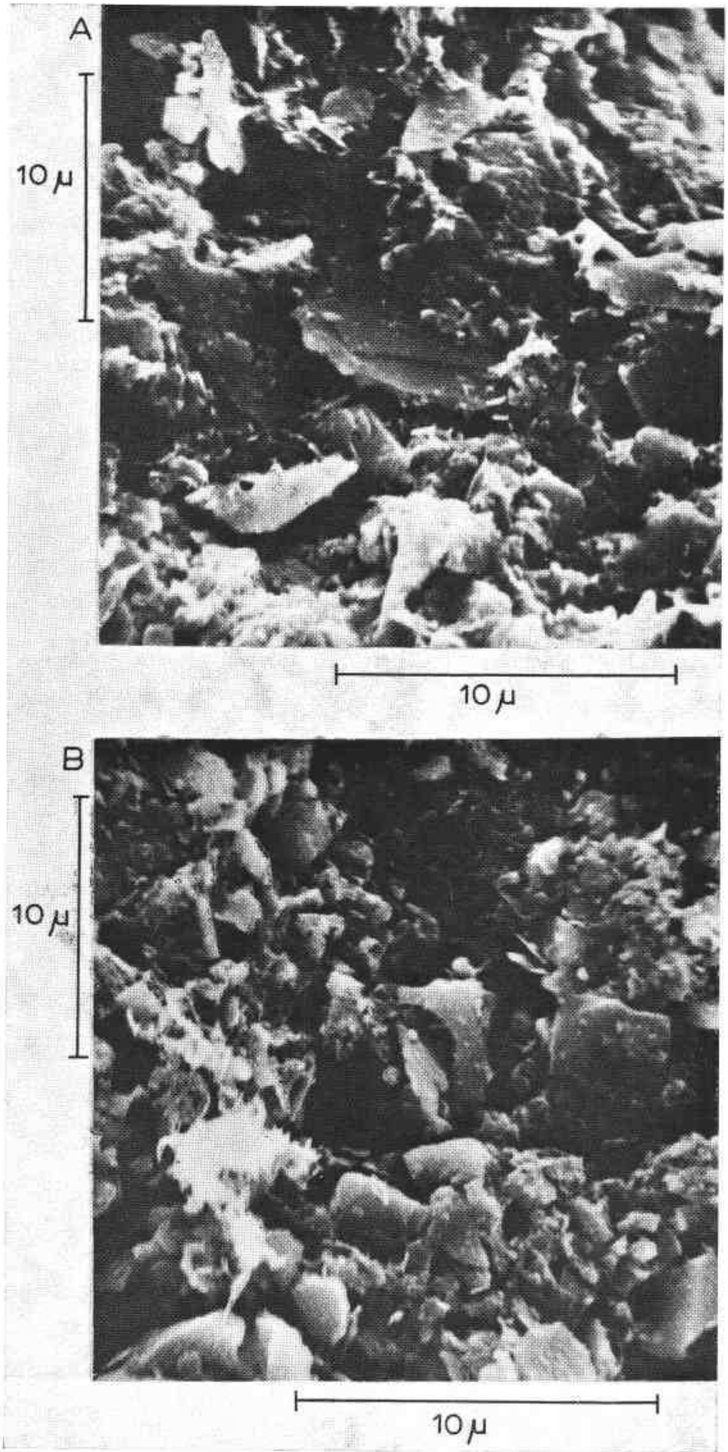
The results of these tests showed little or no particle reorientation at pressures up to the preconsolidation pressure. Beyond the preconsolidation pressure the structure begins to break down and there is an abrupt increase in particle parallelism. Similar tests on remoulded soil showed greater parallelism and lower void ratios at all pressures. QUIGLEY and THOMPSON (1966) suggest that with most of the bonds broken the particles are free to rotate and therefore assume lower void ratios and greater parallel orientation under a given pressure. The relationship between fabric and void ratio was the same for undisturbed and remoulded soil. This led to the conclusion that the final void ratio under a given load is a function of the degree of induced parallelism.

Fabric can also be determined by measuring the thermal conduction of a clay specimen on various axes because the conductivity of the clay minerals is highly anisotropic. If the mineral particles are parallel the conductivity in the direction of parallelism will be greater than it is in a perpendicular direction. Tests on natural Leda clay gave values of conductivity in the horizontal direction only about 10% higher than that in the vertical direction, compared with values up to 70% higher for a lacustrine clay, which is known to have a parallel fabric (PENNER, 1963b). Consolidated remoulded Leda clay showed greater parallelism than the natural soil. Penner also showed the differences in fabric of Leda clay and the lacustrine clay by optical viewing of thin sections between crossed polaroids.

Fig.2. Electron micrographs of Leda clay (×4,500).

A. Clay soil with a natural water content of 58%, a liquid limit of 53% and a preconsolidation pressure of 2 tons/sq.ft.

B. Clay soil with a natural water content of 55%, a liquid limit of 31% and a preconsolidation pressure of 5 tons/sq.ft.


A
10 μ
10 μ
B
10 μ
10 μ

Shrinkage characteristics of Leda clay and the same lacustrine clay described above have been studied by WARKENTIN and BOZOZUK (1961). The undisturbed Leda clay had about equal horizontal and vertical shrinkage, but for the lacustrine clay the vertical shrinkage was three times the horizontal shrinkage. This is attributed to the random orientation of Leda clay particles and the tendency to parallelism in the lacustrine clay.

It is obvious from the open structure of Leda clay and from its stress-deformation characteristics that there are often substantial intrinsic stresses or bonds holding the particles together. The exact nature of these bonds is not known, but they are considered to be of two types—rigid and plastic (CRAWFORD, 1963a). The rigid bonds appear to be especially important in the overconsolidated,

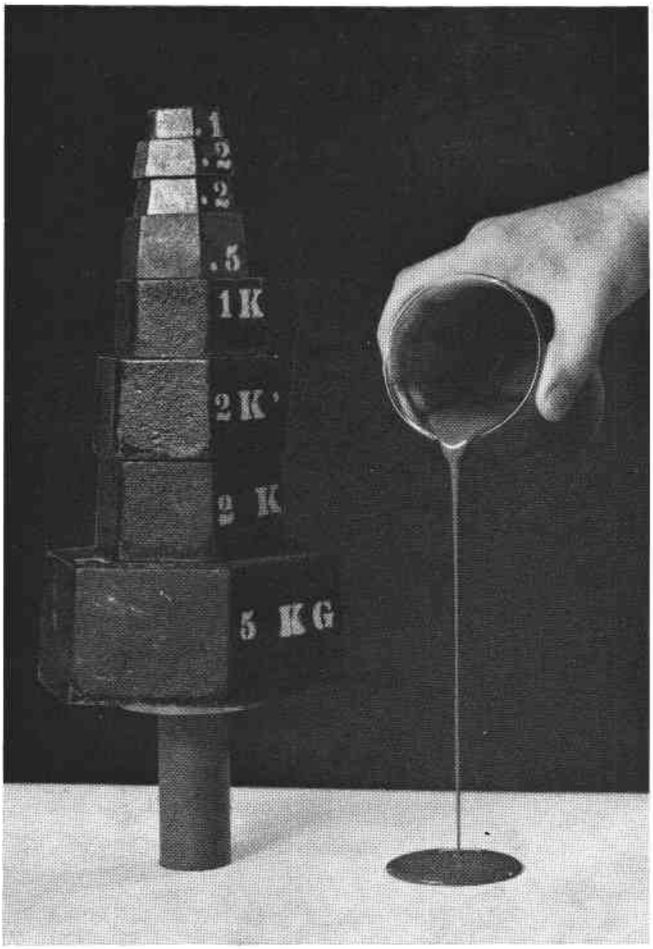


Fig.3. Demonstration of undisturbed and remoulded strength of Leda clay. (CRAWFORD, 1963a.)

high strength deposits. These may be due to recrystallization between particles or to cementation by such agents as iron oxide or carbonates. In the few chemical analyses available, up to 7% iron by weight (BURN, 1966) and more than 12% carbonate (BRYDON and PATRY, 1961) have been found.

SENSITIVITY

The "sensitivity" of a soil is the ratio of its undisturbed strength to its remoulded strength at the natural water content. Clays with a sensitivity greater than 32 were termed "very quick clays" by BJERRUM (1954). The Leda clays usually have a sensitivity greater than 20. The undisturbed strength is often several hundred times as great as the remoulded strength. Very high values of sensitivity are not reliable because of the difficulty of measuring the remoulded strength of very soft (semi-liquid) clay (EDEN and KUBOTA, 1961). The relative strengths of a typical sample of Leda clay are shown in Fig.3.

Early work in Norway showed a correlation between sensitivity and pore water salt concentration but this relationship is not found in the Leda clay. In the Ottawa area, for example, most of the clays have less than 2 g/l salt concentration, but the sensitivity varies widely (CRAWFORD and EDEN, 1965). In order to investigate this apparent anomaly, PENNER (1963a) studied the influence on shear strength of changing the electrolyte content of the pore water and found that a slight increase could greatly increase the sensitivity. In one test the addition of 1 g of sodium metaphosphate per 100 g of soil increased the sensitivity from 30 to about 2000. The effect was attributed to an increased repulsion between adjacent particles caused by an increase in the electrokinetic potential of the system, and this led to an experimental study of the electrokinetic behaviour of the soil (PENNER, 1965).

Specifically, the electrokinetic potential (i.e., the potential difference across the diffuse layer in a clay–water system) was computed from measured values of osmotic flow when an external electrical potential was applied across a plug of undisturbed soil. The variation of sensitivity with electrokinetic potential, shown in Fig.4, is a clear indication of their interrelationship. The two inconsistent specimens were of low plasticity and had low surface areas. Measurements of the specific conductivities of the pore fluid (i.e., salt concentration) showed that soils with high salt contents cannot achieve high sensitivities because of the flocculating effect, but that at low salt contents the sensitivity may be high or low.

Penner's work demonstrated that sensitivity is not necessarily a function of the pore water salt content but is related to the electrokinetic potential and hence is consistent with the theory of interparticle repulsion predicted by ROSENQVIST (1955). There is therefore no physico-chemical contradiction to the geological view that some of the Leda clays were deposited in fresh or brackish water.

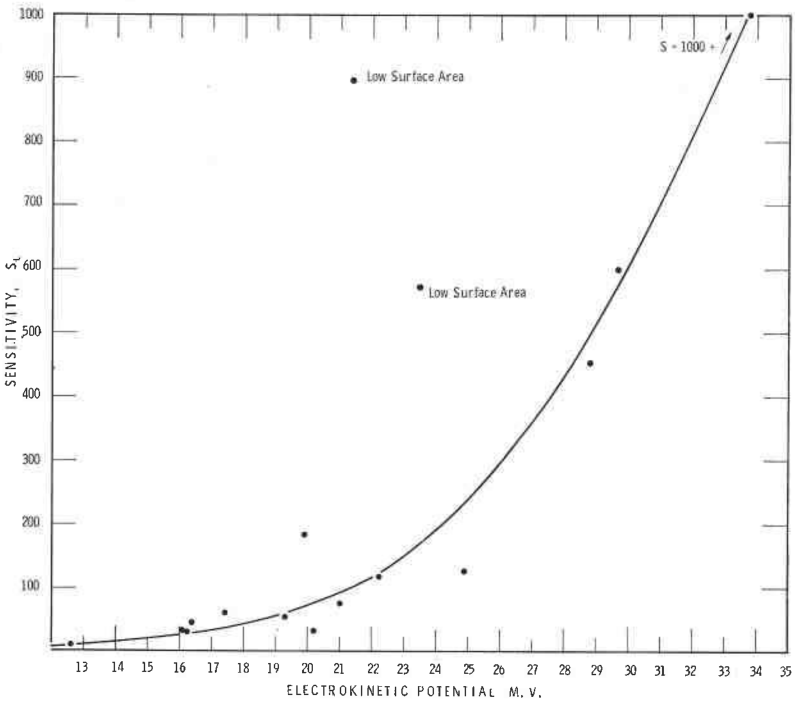


Fig.4. Electrokinetic potentials of undisturbed Leda clay as a function of sensitivity (PENNER, 1965).

COMPRESSIBILITY

Because of its rigid structure, Leda clay has a low index of recompression, but when the load-carrying capacity of the structure is exceeded the compression index is very high. This results in a sharply defined "preconsolidation pressure" in laboratory tests.

In some areas Leda clay is almost normally consolidated, but there are also widespread deposits of substantially overconsolidated clays that can support large loads without detrimental settlements. Reliable determination of the preconsolidation pressure from laboratory specimens is therefore of great importance and has prompted much research on the compressibility characteristics of Leda clay.

For ordinary consolidation testing, the total load applied to the specimen is doubled every day in order to get a reasonable spread of test points on a semi-logarithmic plot. In a highly sensitive compressible clay, the change in void-ratio between the last point on the recompression section and the first point on the virgin compression section of the curve may be quite large. This presents a special problem in interpretation of the preconsolidation pressure due to the steepness of the virgin curve.

In a study of this problem a series of tests was made in which the load increment ratio was varied between 1 and 1/10. It was found that with small load increments the pressure/void ratio curve could be defined more accurately than in normal testing (HAMILTON and CRAWFORD, 1959). It was noted that at pressures near the preconsolidation pressure, the rate of compression was not simply a function of pore pressure dissipation but appeared to be related to the breakdown of the soil structure. In these tests, therefore, the load increments were added when compression under the previous load had decreased to a rate equal to that at the end of primary compression in normal tests. As a result there was not much variation in the over-all rate of testing.

In a later series of tests, the rate of loading was found to have a substantial effect on the pressure/void ratio curve (CRAWFORD, 1964). Results of tests on three specimens are shown in Fig.5. The lower curve is for a test in which load increments were applied once a week. The middle curve shows the results for daily loading. The upper curve was obtained by adding loads at the end of primary consolidation under the preceding load. The first test lasted for 5 weeks, the second for 5 days, and the third for about 3 hours. All tests were repeated at least once; the three curves are typical of other similar tests. Three specimens were loaded under a controlled rate of strain at about the same average rate as that in the third test

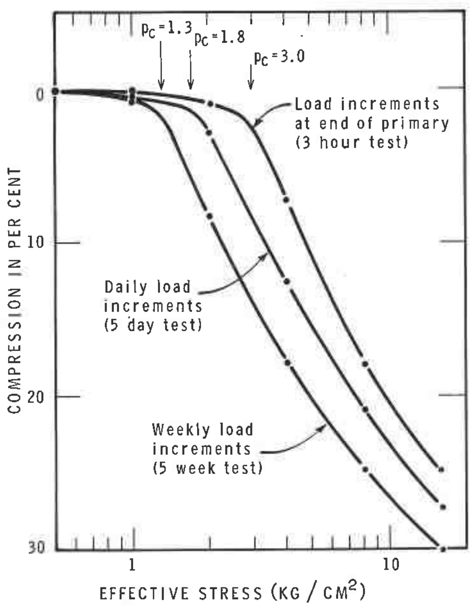


Fig.5. Compression-log pressure curves for various rates of incremental loading.

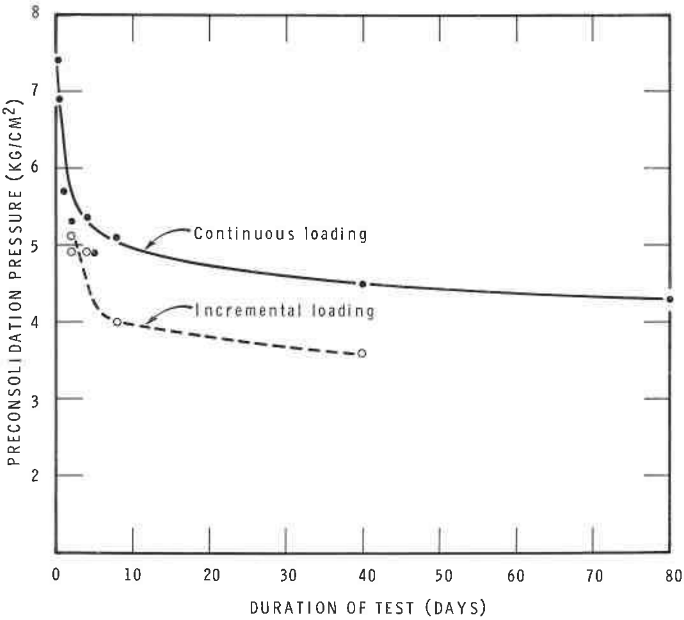


Fig.6. Influence of test duration on estimated preconsolidation pressure.

in Fig.5 and they had equivalent effective stress/void ratio curves. The results show that the preconsolidation pressures vary from 1.3 to 3.0 tons/sq.ft. (kg/cm^2) depending on the method of test.

A further series of tests was made on more overconsolidated specimens of Leda clay and preconsolidation pressures ranged from 4.9 to 7.4 tons/sq.ft. (kg/cm^2) (CRAWFORD, 1965).

Research was then extended to much slower rates of loading with tests lasting from 2 days to 3 months, and the preconsolidation pressures varied from 4.3 in the slow test to 5.3 tons/sq.ft. (kg/cm^2) in the fast test (JARRETT, 1967). These tests covered a wide enough range of rates to isolate a pressure zone in which the structural effects of the soil were controlling the rate of compression. In this zone, from approximately 4 to 7 tons/sq.ft. (kg/cm^2), about 4 times as much compression occurred under the slow loading as under rapid loading.

From all of these tests it may be concluded that the observed variations in the pressure/void ratio curves are functions of the test procedure. Specimens have to be loaded in large increments in order to assess the hydrodynamic effects needed to compute a coefficient of consolidation, but such loading of a thin specimen may cause it to compress a million times faster than it would under field loading. The tests have shown that even where hydrodynamic effects are negligible, the amount of compression is highly dependent on the rate of loading, and the classical concepts of consolidation as applied to highly sensitive clays require review.

In the Seventh Rankine Lecture BJERRUM (1967) assessed both the laboratory and the field behaviour of a very uniform normally-consolidated clay and concluded that after consolidation on a geological time scale "... a cohesive clay will develop increased strength and a reserve resistance against further compression". This "reserve resistance" allows the geological preconsolidation pressure to be exceeded, under relatively short term loads, without significant volume change. This can therefore account for the high values measured in the laboratory.

It is now possible to compare the influence of test duration (or rate of loading) on the measured preconsolidation pressure. This is done in Fig.6 based on tests reported by CRAWFORD (1965) and JARRETT (1967). The solid curve is for tests made with continuous loading and the "dashed" curve is for incremental loading in which the rate of compression varied greatly during each increment. The incremental loads have apparently caused structural damage and result in a conservative estimate of preconsolidation. In practice this may just compensate for the difference between laboratory and field rates of compression.

OVERCONSOLIDATION

The most heavily overconsolidated natural deposit of Leda clay reported in the literature is from the lower St. Lawrence River valley. It has a measured preconsolidation pressure of about 12 tons/sq. ft. (kg/cm^2), a pressure consistent with geological evidence of previous overburden (CONLON, 1966). Many other parts of the valley contain similar soils which are more or less normally consolidated under existing overburden.

It is often suggested that the measured preconsolidation pressure of these soils may be due to a weathering action and not to overburden that has been stripped off in geological time. In order to investigate this question, an assessment was made of all the available test results on carefully chosen samples in a limited area around Ottawa where the geological history is similar (CRAWFORD and EDEN, 1965).

Some of the samples were obtained in blocks from excavations and the others were taken with special thin-walled piston samplers. Consolidation tests were performed on specimens taken at elevations from 80 to 310 ft. above sea level and the results are shown as dashed lines on Fig.7. Each line is a statistical average of many tests on a single profile identified by a letter. Double letters *BB* and *KK* indicate tests on block samples. Profile *B* was a deep excavation in which it was possible to compare tests on block samples with tests on tube samples. It may be noted that the overconsolidation measured on block samples is about 1 ton/sq. ft. (kg/cm^2) higher than that for tube samples.

Recent observations of piezometric levels have revealed a marked downward flow of ground water in many areas. Seepage stresses may therefore account

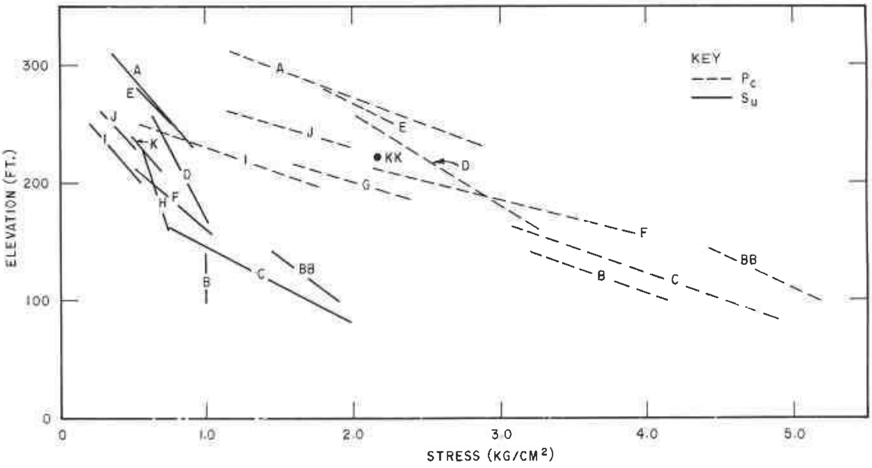


Fig.7. Preconsolidation pressure P_c and undrained shear strength S_u in relation to elevation. Each line is a statistical average of many tests on a single profile identified by a letter; double letters indicate tests on block samples (CRAWFORD and EDEN, 1965).

for part of the measured overconsolidation. Nevertheless, the observed relationships between preconsolidation pressure and elevation are unquestionably related, in general, to an inferred original sediment surface. The inferred surface is probably too high as it is above the present surface at all locations. This may be due to superimposed seepage stresses, chemical action and to influences of compression rate. From a consideration of field and laboratory evidence the writer has concluded that the fabric of the clay was established under low stresses and that weak electrical links and chemical cementation between particles began immediately to form a bonded structure. As the overburden increased, the structure was compressed unidimensionally while still retaining its essentially card-house orientation. Under these greater loads, the particles would be pushed closer together and additional contacts would be established. The chemical bonds between particles in intimate contact would undoubtedly be much stronger than those developed under small loads. This mechanistic picture appears to satisfy both the external loading and chemical bonding concepts in relation to the response of the present soil structure to stress changes.

OBSERVED SETTLEMENTS

Field observations of full-scale structures are essential for the reliable assessment of the compressibility of natural soils, especially of highly compressible, sensitive soils. For this reason one of the major projects of the Division of Building

Research of the National Research Council of Canada is the field measurement of settlements.

The National Museum in Ottawa, built in 1910, is a classical settlement failure. A differential settlement of nearly 2 ft. (60 cm) has left the structure badly distorted. The building is a massive structure on spread footings with bearing pressures ranging from 1 to 4 tons/sq. ft. (kg/cm^2). Computed settlements were generally much greater than the actual settlements (CRAWFORD, 1953). This can be partly explained by the lack of satisfactory sampling and testing equipment when the study was made about 15 years ago. A second factor influencing the accuracy of the settlement computation is the unknown degree of load transfer within the structure.

Settlement observations on three apartment blocks in Ottawa were begun in 1949. Each of the buildings rests on a reinforced concrete mat and the bearing pressures range from 1.0 to 1.2 tons/sq. ft. (kg/cm^2). Due to excavation the net loading on the subgrade is only about $^1/_2$ the bearing pressure applied by the mat (i.e., averaging 0.6 ton/sq. ft.). The overconsolidation of the subsoil is about 1.6 tons/sq. ft. (kg/cm^2) and the observed settlement averaged about 0.5 inch (1.3 cm) (LEGGET et al., 1961). Computed elastic settlement could account for about $^1/_2$ of the measured settlement. Settlements computed from recompression curves obtained in the consolidometer greatly overestimated the movements.

Large earth embankments provide ideal loading for research because the increase in stresses in the subsoil can be computed with reasonable accuracy. Three large embankments have been used for this purpose in the Ottawa area. In two of the three, the applied stresses were in the recompression range.

Queensway, Green Creek

In 1957 an embankment 23 ft. high was placed on a thick deposit of moderately overconsolidated Leda clay. The applied stress of more than 1 ton/sq. ft. (kg/cm^2) is equal to about half of the overconsolidation pressure. Settlement gauges installed at various depths beneath the fill (BURN, 1959) showed that most of the compression occurred in the upper 50 ft. The total settlement after 10 years is approximately 5 inches (13 cm), half of which occurred immediately when the load was applied (BURN and HAMILTON, 1968).

Queensway, Main Street

A second embankment, which also caused stress increases in the recompression range, was built in 1963. Fill 22 ft. high was placed over 90 ft. of soil. Four years after construction, the settlement at the base of the fill was about 4.5 inches (11 cm), of which nearly 3 inches (7 cm) occurred in the upper 30 ft. of subsoil.

Kars Bridge Fill

The third embankment is located over only slightly overconsolidated clay. The first 20 ft. of fill was placed in November 1959. In June 1961 the height was increased to 26 ft. Settlement gauges placed at the base of the fill and at various depths in the subsoil have provided a good record of the amount and rate of compression of the natural ground. Piezometers under and adjacent to the loaded area have revealed the build-up and dissipation of pore water pressures (EDEN and POOROOSHASB, 1968).

In April 1967 the base of the embankment had settled 20 inches (50 cm) at the centre-line. The total thickness of soil is about 50 ft. but half of the compression occurred in a soft clay layer extending from 20 to 35 ft. beneath the original surface. When the first 20-ft. fill was completed (loading time of 15 days), the piezometric excess pressure in the centre of this layer had increased to 27 ft. of water. A month after the loading was completed the excess head had decreased to 17 ft. of water. After 2 years the excess head was 11 ft., and during the past 4 years (to 1967) it has remained almost constant at 5 or 6 ft. It is of special interest that very little pressure gradient has been observed between the centre of the layer and its upper and lower boundaries. These observations lead to the conclusion that the pore water pressures are being created by a breakdown of the soil structure and that the rate of consolidation is therefore not dependent only on the permeability of the soil.

In order to predict the settlement of this embankment two types of consolidation test were carried out. One set of samples was loaded by the normal incremental procedure and a second set was loaded first to the overburden pressure, then to the pressure of a 20-ft. fill, and finally to a pressure equivalent to a 26-ft. fill. Both sets of tests indicated an ultimate settlement of about 3 ft. (90 cm) but the latter type of test gave a predicted rate much closer to the actual rate. Observations to date indicate that the ultimate settlement will be almost 3 ft.

A record of settlement has also been obtained on a light single-storey building resting on a gravel mat over about 40 ft. of almost normally consolidated clay at Gloucester, near Ottawa. The building is located a few miles from the Kars embankment described earlier. Although the total loading is small, it caused stresses equal to the measured preconsolidation pressure in the subsoil and a settlement of several inches has occurred.

A summary of settlements for the five case records is given in Table I. In each case the tabulated values represent settlement after seven years. In cases 2, 3 and 5 settlements of individual layers of the subsoil were measured so that layers of maximum compression during the period of observation could be selected. In cases 1 and 4 the regions of maximum compression were estimated on the basis of soil tests. These "most compiessible layers" were then treated as full scale consolidation tests in order to assess the practical reliability of the preconsolidation

TABLE I

SETTLEMENT OF COMPRESSIBLE LAYERS

Case no.	Location	Soil depth		Total sett.[1] (cm)	Compressible layer			Average effective stresses in compressible layer (kg/cm^2)			P_f/P_c
					depth	compression					
		(m)	(ft.)		(m)	(cm)	(%)	initial P_o	final P_f	preconsol. P_c	
1	Apartment buildings	15	50	1.3	9	1	0.1	0.7	1.3	2.3	0.56
2	Green Creek	37	120	11	15	7	0.5	0.7	1.7	3.2	0.53
3	Main Street	27	90	15	9	9	1.0	0.5	1.8	2.5	0.72
4	Gloucester	20	66	17	12	15	1.2	0.55	0.8	0.8	1.0
5	Kars	15	50	50	4.6	26	5.7	0.7	2.2	1.4	1.6

[1] All settlements are quoted for a 7-year period.
In case no. 3 they are extrapolated from a 4-year record.

pressures measured in the laboratory. For this purpose the initial average effective stress in the layer and the change in effective stress due to the applied loading were calculated. The average preconsolidation pressure in the layer was estimated from routine laboratory tests carried out usually in 5–7 days. The ratio of the final effective stress to the average preconsolidation stress is used to illustrate relative stress levels. Values up to 1 are in the recompression range and values greater than 1 are in the virgin compression range.

For convenience, average values for the three apartment buildings are quoted because the subsoils were similar and the loading and settlements were about the same in each case. The settlement measurements on the apartment buildings were begun only after the slabs and basement columns were in place, and the initial movements were, therefore, not observed.

The percentage settlement in the compressible layers is plotted with respect to time after loading in Fig.8. The ratio of the average final effective stress to the average preconsolidation stress is shown on each curve. It can be seen from these curves that significant long term settlements occur at stresses well below the measured preconsolidation pressure. This may be considered to be due to secondary or delayed consolidation and the reasoning developed by BJERRUM (1967) appears to be appropriate. The test results appear to be more realistic in practice when the test is conducted slowly enough to incorporate an appreciable amount of secondary consolidation.

An interesting record of the settlements measured at a large industrial plant at Varrennes, Quebec has been published by CASAGRANDE et al. (1965). The

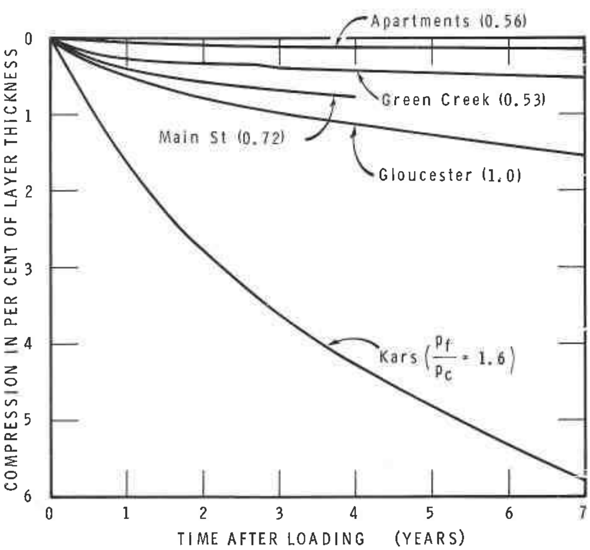


Fig.8. Time-settlement curves under field loading.

measured overconsolidation of the subsoil at the site was about 0.75 ton/sq. ft. (kg/cm^2). The applied load was sufficient to stress the subsoil up to the preconsolidation pressure. Measured settlements actually ranged from 1 to 6.5 inches (2.5–16 cm). The authors attributed much of the settlement to virgin compression caused by increased stresses due to lowering of the ground water table at the site. In a discussion of the paper LEONARDS (1966) suggested that much of the measured settlement might have been due to elastic type compression, consolidation in the recompression range and secondary consolidation. It may be concluded that computations of settlement for Leda clay are open to a variety of errors when the maximum stresses are close to the measured preconsolidation pressure of the soil.

SHRINKAGE

Leda clay contains limited amounts of swelling clay minerals. At normal water contents, however, its open structure will collapse and shrink substantially when dried. The influence of clay concentration on shrinkage was studied by mixing remoulded Leda clay with varying proportions of glass beads (DE JONG and WARKENTIN, 1965). It was observed that both the amount of shrinkage and the water content at the "shrinkage limit" decreased linearly with decreasing percentage of clay as long as the glass beads were present as inclusions in the clay. At clay concentrations of less than 30%, the glass beads were in contact with one another and very little shrinkage occurred.

The differential shrinkage of thin wafers of undisturbed Leda clay was measured by WARKENTIN and BOZOZUK (1961). Total vertical and horizontal shrinkage was found to be the same and this was attributed to initial random particle orientation. The shrinkage limit decreased from 27 to 20% when the soil was remoulded, indicating a probable breakdown of the edge-to-face particle arrangement.

Samples did not regain this original water content when rewetted but the amount of regain increased as the amount of drying increased. The authors concluded that shrinking increased the degree of parallel orientation and decreased the interparticle spacing, causing the diffuse layer of exchangeable ions to overlap and to exert an osmotic swelling force between the approaching flat surfaces. As the amount of drying increases, this osmotic swelling becomes more prominent than the surface tension forces in the voids and is thought to control the moisture regain.

Surface drying and wetting occurs seasonally and causes some volume change in the natural clay. The greatest volume changes result from moisture absorption by the roots of vegetation as they penetrate the high water content clays at lower depths. Vertical ground movements were measured at various depths and distances from a row of mature elm trees during a particularly hot and dry summer at Ottawa in 1955 (BOZOZUK and BURN, 1960). Although the trees had stood at the site for about 40 years, the combination of weather factors increased their transpiration and decreased the available soil moisture to such an extent that the ground surface near the trees settled 3 inches (8 cm) in 4 months. Even at a depth of 6 ft. the vertical movement was 2 inches (5 cm). The influence of the trees extended laterally for a distance almost equal to their height of 55 ft. (17 m). Most of this settlement is not reversible. It was shown by these studies that vertical ground movements can be related to "soil moisture depletion", a quantity that can be estimated from weather records.

Shrinkage of the subsoil causes much damage to shallow foundations in Leda clay. A survey of 574 structures in central Ottawa revealed the nature of movements in relation to soil and vegetation conditions (BOZOZUK, 1962). Differential settlements up to 14 inches (35 cm) were observed in buildings 60–80 years old. Little variation in soil properties was found but large movements were always associated with the occurrence of large elms and other broad-leafed trees. In most cases, the greatest settlement had occurred on the street side where rows of trees are common and much of the precipitation is collected by the storm sewers.

The distortion of a house resting on spread footings 5 ft. deep is shown in Fig.9. At the left side of the photograph there is a line of mature trees and on the right side is an open grassed area. Three curves in Fig.10 show the water content profiles between the trees and the house, under the house, and under the grassed area. The natural water content in the clay averages about 80%. Within the region of the trees' root system the water content has been reduced to less than 40% to a depth of 10 ft. (3 m). Under the house a sufficient reduction in water content has occurred to cause the damage illustrated in the previous figure.

It has been concluded in general that if the upper 10 ft. of Leda clay has a water content exceeding 40% it is susceptible to shrinkage and special efforts should be made to prevent differential drying (CRAWFORD, 1961a).

SHEARING RESISTANCE

The strength of Leda clay has attracted wide engineering interest because of the unusual tendency of this material to liquefy and fail dramatically under certain conditions. In its undisturbed state it varies from a very soft material with undrained shear strengths as low as 0.15 ton/sq. ft. (kg/cm^2) (BURKE and DAVIS, 1957) to a very hard material with shear strengths as great as 4 tons/sq. ft. (kg/cm^2). (CONLON, 1966). When disturbed it may have the consistency of a viscous liquid. The undisturbed, undrained shear strength is, on the average, equal to about 30% of the preconsolidation pressure (CRAWFORD and EDEN, 1965). The undrained shear strength is compared in Fig.7 with the preconsolidation pressure at several locations.

The undisturbed clay has a rigid structure. Good specimens will fail in undrained compression at about 0.5% strain. Elastic moduli as high as 1,500 tons/sq. ft. (kg/cm^2) have been measured in the laboratory (CONLON, 1966). A modulus of 720 tons/sq. ft. was computed from rebound measurements during an excavation 31 ft. deep and this value agreed with laboratory tests on specimens from the site (BOZOZUK, 1963).

It is common practice to measure the undrained strength of Leda clay with a field vane device. EDEN and HAMILTON (1957) found that strengths measured by the field vane equaled or exceeded the maximum strengths obtained from laboratory

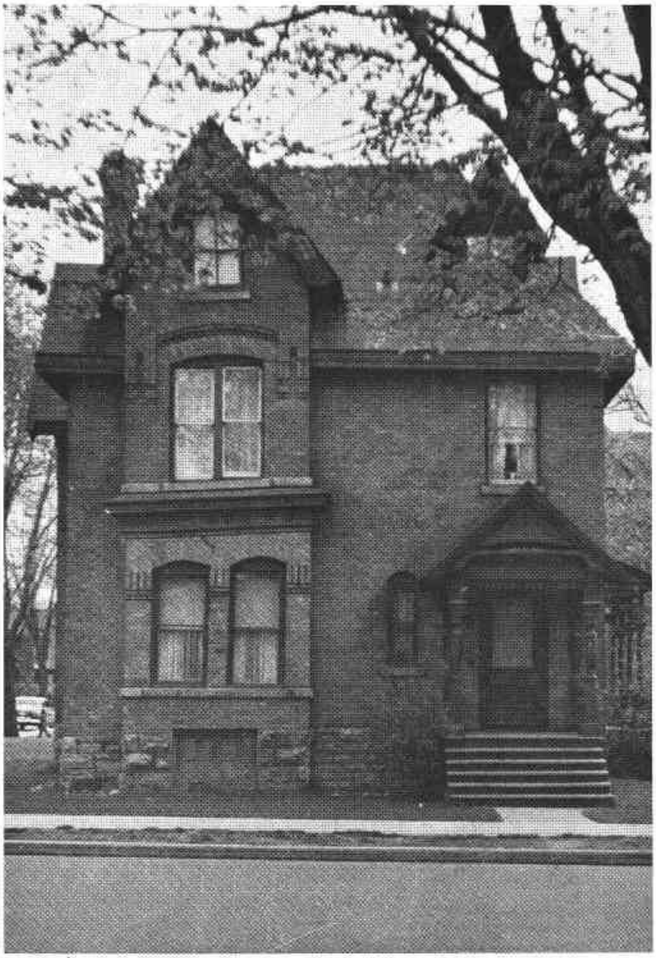


Fig.9. Building distortion due to shrinkage of the subsoil.

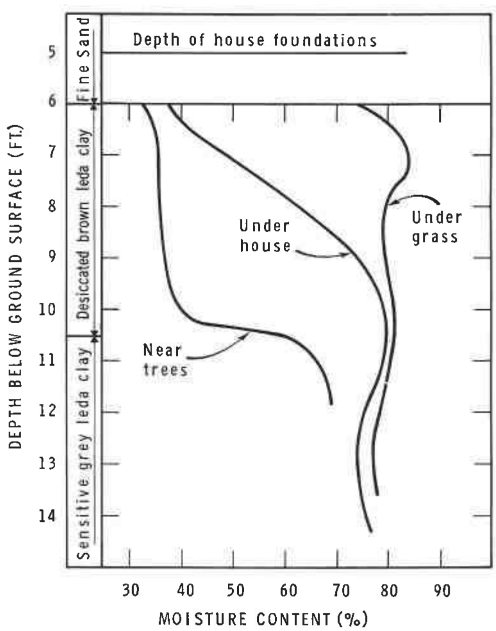


Fig.10. Variation in the moisture content of the subsoil.

compression tests. Vane test results were found to be more consistent than laboratory results. Some evidence was presented to indicate that stability analyses based on vane tests were satisfactory.

Bazett et al. (1961) compared laboratory compression tests with field vane tests in the study of a full-scale test trench failure. They found that the laboratory tests agreed with the undrained failure analysis but use of the vane tests resulted in a serious overestimate of the safety factor. The authors concluded that the vane tests were accurate and attributed the discrepancy with field behaviour to loss of strength due to lateral expansion of the material at the trench wall.

From a review of tests at nine sites near Ottawa, Eden (1966) reaffirmed that field vane tests are more consistent and yield higher strength values than laboratory compression tests on soft to medium strength clays. For shear strengths greater than 1 ton/sq. ft. (kg/cm^2) the field vane test was not considered to be reliable. This conclusion was based on a comparison with compression tests on specimens trimmed from block samples of strong, brittle clay. In this case it was thought that the structure was partly destroyed by the insertion of the vane.

A series of experiments has been carried out in order to investigate the influence of strain rate and stress path on the effective stresses and ultimate shearing resistance of specimens of Leda clay (Crawford, 1959, 1961b, 1963a). It has been shown that for overconsolidated specimens the maximum undrained compressive strength decreases substantially within decreasing rate of strain. For each 10-fold increase in the time to failure the ultimate strength decreases about 10%. The slowest test recorded failed after 8 weeks at about 60% of the strength measured in a rapid test. Pore pressures with specimens under sustained constant stress will increase due to breakdown of the sensitive soil structure, but if the ambient stress is very low this does not appear to occur (COATES et al., 1963).

The lower strength measured in slow tests cannot therefore be attributed to higher pore pressures, but must be due to a time-dependent reduction in true cohesion. Measurement of pore water pressures at the ends and within soil specimens during axial loading revealed higher pore pressures at the ends, and this was attributed to disturbance caused by end restraint (CRAWFORD, 1963b). For reliable pore pressure measurements it is necessary to use a back pressure system and to conduct the test at less than 2% axial strain/hour.

One set of unconfined compression tests was conducted on specimens immersed in water in order to reduce the build-up of pore pressures during shear. The average compressive strength at failure for three specimens was 2.4 tons/sq. ft. (kg/cm^2). They all failed at about 0.5% strain after loading for more than 2 hours. Almost all of the shearing resistance under these test conditions is due to cohesion or interparticle bonds in the soil.

Research on the strength properties of Leda clay has been confined in general to the evaluation and interpretation of friction and cohesion components of the undisturbed soil. Specimens cut from block and tube samples have been subjected to various stress paths and rates of strain while the response of the soil skeleton to the effective stress changes was observed. A summary of the writer's views (CRAWFORD, 1963a) led to the conclusion that most of the shearing resistance of moderately overconsolidated Leda clay could be attributed to cohesion caused by intrinsic stresses and interparticle bonds. When failure occurred at stresses lower than preconsolidation pressure it was suggested that the use of the Mohr-Coulomb theory might be inappropriate.

Recent work by KENNEY (1966) offers an alternative approach in which shearing stress is related simply to the effective stress on the failure plane. This value can be obtained from a variety of test methods and applied directly to a practical problem. He has used this approach very successfully in analysing actual landslides in quick clays in which cementation is a minor component. Kenney defines friction as the strength component that is nearly directly proportional to the applied effective stress and is mobilized through the mutual sliding of soil particles irrespective of change in void ratio. By this definition the term may include a substantial amount of cohesion according to the HVORSLEV (1960) concept. Kenney also recognizes a second component due to cementation bonds, which may be strain- and time-dependent and which he found to be of great significance in Canadian clays from Labrador.

CONLON (1966) apparently used a similar concept when he divided the observed shearing resistance of clay from the lower St. Lawrence valley into two components—cementation bond and friction—and arrived at a drained friction angle of 36 °. This angle coupled with the cementation component appears to give an unrealistic evaluation of the shear strength of the soil.

STABILITY PROBLEMS

Stability problems can be solved only by field research. Much research has been done in recent years in an effort to correlate stress-deformation properties of Leda clay with full-scale failure conditions. The bearing capacity of footings on Leda clay is not in general a serious stability problem because excessive settlements usually occur before shear failure. Settlements of several feet can occur under buildings and earth embankments without bearing failure. There have been some bearing failures of rapidly constructed fills, but they are not common.

The failure of an oil storage tank founded on soft, sensitive Leda clay has been described by BROWN and PATERSON (1964). The tank, 44 ft. high and 70 ft. in diameter, was placed on a 12-ft. sand mat over 35 ft. of clay. The tank sank vertically 6–10 ft. within 24 hours of filling, although it had been partially filled for several months. When full, the increase in vertical stress at the base of the sand mat was estimated to be about 1.5 tons/sq. ft. (kg/cm^2). The undrained shear strength of the clay was about 0.25 ton/sq. ft. (kg/cm^2), giving a computed factor of safety of 1.2. The subsoil at the site was overconsolidated by only about 0.25 ton/sq. ft. (kg/cm^2) and had an initial void ratio of about 2. It is evident, therefore, that if the foundation had not failed due to shear it would have experienced several feet of settlement.

The most serious stability problem is with natural slopes. Sometimes a small bank failure will develop into an earth flow extending over a large area. In some sections of the Ottawa–St. Lawrence valley the topography is dominated by landslide scars such as shown in Fig.11. A map showing major landslides in the region has been published (HURTUBISE et al., 1957) and some of them have been described (CRAWFORD, 1961a).

Recent investigations of two flow slides near Ottawa have indicated that an effective stress analysis using the method of slices for first failure gives a reasonable approximation of the stability (CRAWFORD and EDEN, 1967). The influence of field pore pressures has been found to be of the greatest importance in assessing the stability. A similar investigation of a major landslide in the lower St. Lawrence valley showed that it could not have been predicted by ordinary analyses (CONLON, 1966). In this case the clay was so strongly cemented that the landslide could be explained only on the basis of a progressive failure initiated by dynamite blasting. It was also noted that the landslide occurred in the springtime after an unusually heavy rainy season.

Ground water is undoubtedly an important factor in these failures even if they do not develop into flow slides. During the spring of 1967 after unusually wet weather, there were many bank failures along the river banks at Ottawa.

Fig.11. Landslide scars in Leda clay (R.C.A.F. photograph).

Inspection soon after failure invariably revealed water running out of sandy or silty layers in the fresh face of the slide.

A failure in a cut bank along the right-of-way for the Quebec North Shore and Labrador railway was described by Woods et al. (1959). The failure of the 70-ft. high bank occurred more than 2 years after it was cut on a $1^1/_2$–1 slope. The slide took place just 3 days after a 4-inch rainfall. A similar failure occurred in a road cut near Ottawa in 1965. The cut, approximately 40 ft. deep, was made 5 years before the failure, which occurred after a night of heavy rains. A second failure occurred 2 weeks later after another heavy rainstorm. After each failure great quantities of water were released from the failure surface.

Analyses of these failures have not generally been satisfactory. Undrained analyses usually result in a substantial overestimate of the safety factor. Effective stress analyses are subject to error because of the unknown water conditions and uncertainty of the failure mechanism. Even the case records referred to must be treated with suspicion because the precise method of failure is unknown. Tension cracks are not always taken into account and the initial failure surface does not appear to agee with the computed critical surface.

CONCLUSIONS

The mineralogy and structure of the Canadian quick clays are similar to those found in other parts of the world. Extensive deposits of the Leda clay have a low salt content pore water and their sensitivity does not correlate with salt content. Deposits which have been overconsolidated are strongly cemented.

Highly plastic deposits of Leda clay are very compressible. Although well-defined pressure-void ratio curves can be obtained in laboratory tests the absolute value of void ratio at any pressure is greatly dependent on the test procedure. In normal tests (lasting 5 days) a considerable amount of secondary consolidation is permitted. Field observations show that much secondary consolidation occurs in nature. The available evidence suggests that a 5-day test will overestimate the preconsolidation pressure of a good sample but that this may be compensated by disturbance of tube samples. A significant amount of compression has been observed in the field at pressures well below the preconsolidation pressure in the ground. Field observations indicate that pore pressures are created by breakdown of the soil structure as it compresses under load and this may compensate for pressure decreases by drainage.

Large settlements occur due to shrinkage in the upper 10 or 15 ft. of Leda clay. The shrinkage is caused primarily by the withdrawal of soil water by vegetation, especially by trees. Shrinkage potential is related to the natural water content. The problem can be avoided by restricting tree growth near shallow foundations. Unless the soil is dried to a very low water content it has little swelling potential.

The strength of Leda clay varies from very soft to very stiff. The measured undrained strength depends on the rate of testing. Most of the shearing resistance under low stresses has been attributed to cohesion. The most difficult strength problems are encountered in attempting to assess the stability of slopes in over-consolidated deposits. Much field research will be required to solve this problem satisfactorily.

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