Consolidation Properties of an Organic Clay Determined from Field Observations

T. J. SCHMIDT, Delaware State Highway Department, and
J. P. GOULD, Mueser, Rutledge, Wentworth, and Johnston, New York, N.Y.

A program of observations of settlement and pore water pressures was undertaken in a soft organic silty clay for control of construction of a section of the Christina River Interchange of Delaware Interstate Routes I-95, I-495, and I-295 near Wilmington, Delaware. The roadway embankment was stabilized by use of vertical sand drains plus surcharge. Values of compression index and coefficient of consolidation computed from field observations were compared with laboratory test data. Undisturbed samples obtained at a later time when primary consolidation was largely complete were utilized to evaluate the gain in strength of the organic clay stratum. Conditions disclosed by field observations agreed reasonably well with those assumed for design.

•SOFT and compressible organic clay adjacent to the original course of the Christina River was stabilized by use of vertical sand drains plus surcharge in construction of the Delaware Interstate Highway routes south of Wilmington, Delaware. Numerous piezometers and settlement plates were installed early in the filling operations for control of construction and to determine when the surcharge loading had served its intended purpose. The observations permitted an evaluation of in situ properties of the organic clay, which could be compared to laboratory test values and design assumptions. The design, construction, observation program, and the results obtained are summarized herein. The stabilization was planned under contract with the Delaware State Highway Department by Moran, Proctor, Mueser, and Rutledge (now Mueser, Rutledge, Wentworth, and Johnston) of New York, engaged in a joint venture with J. E. Greiner Company of Baltimore for design of the Christina River Interchange.

SUBSOIL CHARACTERISTICS

The subsoil to be stabilized is a soft-to-medium dark gray organic silty clay with scattered vegetal matter. It is classified as OH material in the Unified Soil System with liquid limits between 65 and 100 and plasticity index about 8 points below the A-line. It is fairly typical of estuarine deposits formed in the lower reaches of slow-flowing streams on the Atlantic coastal plain. It was probably laid down in post-Pleistocene times as a result of the drowning of the stream valley by a 25- to 35-ft rise in sea level from a postglacial low water stage about 6,000 years ago. Before construction the surface of the deposit was several feet above sea level, and the material appears to have been overconsolidated by about 750 lb/sq ft in excess of overburden pressure. This was probably caused by a temporary low sea level stage at some time since deposition. There is no evidence of the existence of a greater height of overburden in the past. The surface material is not notably stiffened by desiccation but contains a concentration of vegetal matter in some locations. It is underlain by compact to very compact Pleistocene sands, followed by cretaceous coastal plain sediments.

The boring exploration program for design included recovery of a large number of 3-in. undisturbed samples obtained with a fixed-piston device, plus use of the Swedish

Paper sponsored by Committee on Soil and Rock Properties and presented at the 47th Annual Meeting.

foil sampler and vane shear equipment. The average initial properties of the organic silty clay, determined from field and laboratory tests, are listed in Table 1. Consolidation tests on the best quality undisturbed samples showed a drastic decrease in c_V values in a ratio of at least 5 to 1 between the recompression and virgin compression ranges of pressures. Shear strengths generally increased linearly with depth. The division of strength values at 7 ft depth in Table 1 is merely a convenience for analysis.

DESIGN

Stabilization was planned for sand drain plus surcharge in three separate sections, totaling 2 mi in length, where the highway embankment was to overlie a considerable thickness of organic soils. The two sections with the greatest thickness of organics, averaging 25.5 ft, controlled the overall stabilization requirements. Sand drains were 16-in. diameter, spaced at 7 ft center to center in triangular array, driven by displacement methods through 3 ft of sand drainage blanket and 1-ft working mat to the underlying Pleistocene sands. The average total height of fill required was 14 to 15 ft, measured from original ground surface to the bottom of the pavement base course. The average nominal height of fill plus surcharge chosen for design was 24.5 ft, including about 10 ft of surcharge without considering settlement.

A total surcharge loading period of not less than 400 days was computed as necessary at the least favorable cross section to eliminate primary consolidation plus one cycle of secondary compression under the final roadway embankment plus pavement. This required completion of an average of about 90 percent of primary consolidation under full surcharge, expressed in terms of effective stress transfer, or about 95 percent in terms of void ratio decrease or settlement. A vertical coefficient of consolidation of 0.03 sq ft/day was selected as a reasonably conservative value for design from the results of 21 consolidation tests, performed in the conventional manner with vertical double drainage. The selection disregarded the much higher coefficients exhibited in the recompression range of loading, but was somewhat larger than the lowest coefficients obtained at the highest applied pressures. This number was utilized in design without considering a possible higher horizontal coefficient or the effect of disturbance and smear due to driving drains, factors which were considered to be roughly balancing in their effect.

Based on an average initial shear strength of 350 lb/sq ft, stabilizing berms were designed to provide a safety factor of not less than 1.25 against shear failure during construction, including a conservatively selected gain in strength during placing of the fill. The berms ranged from about 120 to 200 ft in width and 9 to 18 ft in thickness. The rate of filling in sand drain sections was limited in the construction specifications to a maximum of 1 ft of height of fill placed per week in order to conform to the rate of gain of shear strength required for stability.

TABLE 1

IN SITU PRECONSTRUCTION PROPERTIES OF ORGANIC SILTY CLAY STRATUM

Index Properties
Unified soil classification: OH
Natural water content = 84%
Liquid limit = 84
Plastic limit = 46
Plasticity index = 32
Specific gravity = 2.61
Void ratio = 2.2
Dry unit weight = 51 pcf
Saturated unit weight = 94 pcf
Initial degree of saturation range = 97 to 100%
Consolidation Characteristics
Average effective overburden pressure for complete submergence = 400 psf
Probable average preconsolidation = 1150 psf
Overconsolidation = 750 psf in excess of submerged overburden pressure
Average virgin compression index, C_c = 0.95
Typical coefficient of consolidation in virgin compression range, c_V = 0.03 sq ft/day = 3×10^{-4} cm^2/sec
Typical coefficient of consolidation in recompression range = 0.2 sq ft/day = 2×10^{-3} cm^2/sec
Shear Strength Characteristics
In situ shear strength = 250 psf in upper 7 ft, 400 psf below 7 ft depth
Average shear strength for design = 350 psf
Angle of shearing resistance from drained shear tests = 27°
Angle of shearing resistance for consolidated undrained shear above the preconsolidation stress = 16°
Sensitivity = 3 to 4

CONSTRUCTION

Construction operations began with placement of sand blanket and temporary settlement plates. Sand drainage blanket material was pumped at an average rate of 350 cu yd/hr by a stationary dredge from a collection sump pit which was filled by borrow excavated in the dry. The 11 percent passing the No. 200 sieve in the dry borrow was reduced to an average of 1.5 percent in the hydraulic placing operations. The contractor elected to make the 1-ft working mat of the same material as the sand drainage blanket. Vertical sand drains were installed from the working mat using a 16-in. outside diameter closed-end mandrel driven by a Vulcan OR hammer. A total of 24,300 separate vertical drains were placed amounting to 700,000 lineal ft of drains, averaging 29 ft per drain. As driving of sand drains progressed, collector pipes were laid and drainage windrows were installed within the blanket at right angles to the collector pipes. Fill material for the roadway embankment ranged from a coarse-to-fine sand with some gravel to a sandy silt, placed at an average field moisture content of 10.7 percent in 8-in. lifts and compacted by vibratory sheepsfoot rollers to an average dry density of 120.4 lb/cu ft.

Certain stabilizing berms were placed hydraulically and consisted of disposal material obtained by dredging unsuitable soils in other embankment areas. Elsewhere, berms were placed in the dry by truck dumping. During the first winter of construction, 1962-63, filling was suspended when it had reached an average of about two-thirds of the final total height. The construction sequence is summarized as follows:

1. Sand blanket placed—February, May, and June 1962;
2. Sand drains driven—April to September 1962;
3. Embankment fill placement started—July to October 1962;
4. Winter suspension—December 1962 to April 1963;
5. Fill complete to the top of surcharge—May to August 1963;
6. Surcharge removal commenced—February 1965.

OBSERVATION PROGRAM

A total of 43 Casagrande-type porous-stone piezometers were installed at 17 locations during the initial stages of construction with 97 settlement plates of various types. Piezometers were placed in boreholes made from the working mat after driving sand drains. Piezometers were positioned within the compressible stratum on a vertical line midway between drains in groups of three to a boring, located at center and quarter points of the thickness of layer. Stones were sealed from each other by bentonite. Observations of pore water pressures did not commence until at least several weeks following driving of sand drains in the immediate vicinity. Certain individual piezometers or observation wells were added in the upper sand blanket material and in the underlying compact Pleistocene sands.

Some settlement plates were placed on top of the sand blanket and attached to the riser pipe that carried piezometer plastic tubing through the fill. Other settlement plates were installed at or near the base of the sand blanket so that generally a complete record of movement is available from the beginning of filling operations.

Piezometers serve the dual purpose of controlling the rate at which height of fill increased and of determining progress of consolidation prior to removing surcharge. Although the piezometers functioned satisfactorily in the first 8 months, many of the lower and middle piezometers were rendered inoperative when the surface settlement exceeded about 2 ft and the differential settlement between surface and piezometer stone was about 1 ft. In planning for instrumentation on future contracts, it is hoped to avoid this condition by placing one piezometer per borehole, leaving casing around the plastic tubing lines, and by employing a double-tubing Casagrande piezometer of the type recently used successfully by the Port of New York Authority in similar compressible soils.

During May and June 1964, 11 replacement piezometers were installed at six locations at the center of the organic stratum as substitutes for those damaged at critical locations, placed midway between drains and one space removed from the original piezometers. Undisturbed samples were obtained in the borings for the replacement piezometers and particular attention was paid to recovering samples precisely at the piezometer elevation.

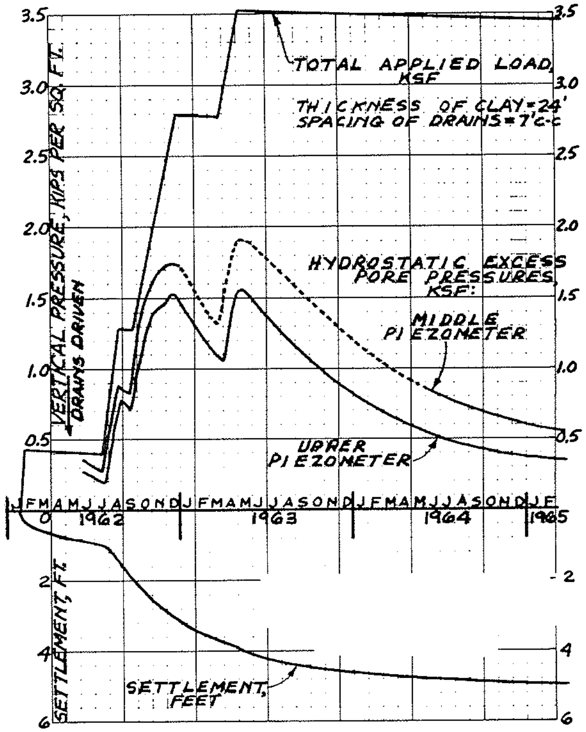

Figure 1. Loads, pore pressures, and settlements—piezometers 2 and 2A.

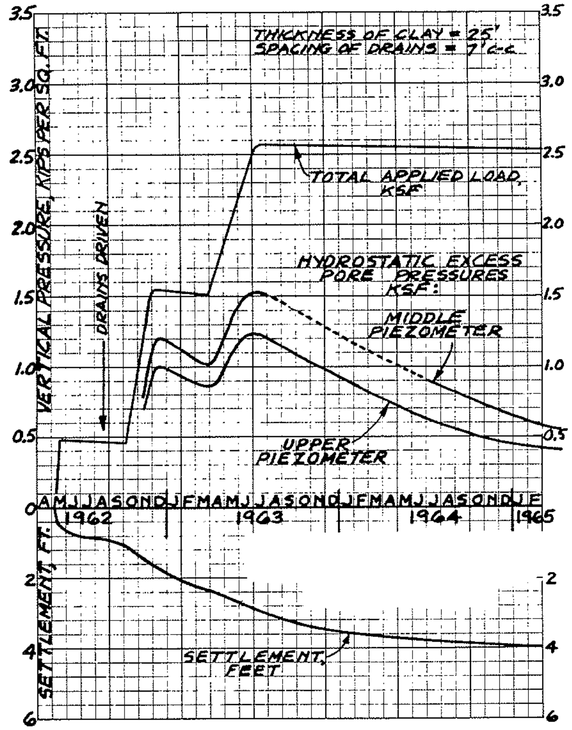

Figure 2. Loads, pore pressures, and settlements—piezometers 11 and 11A.

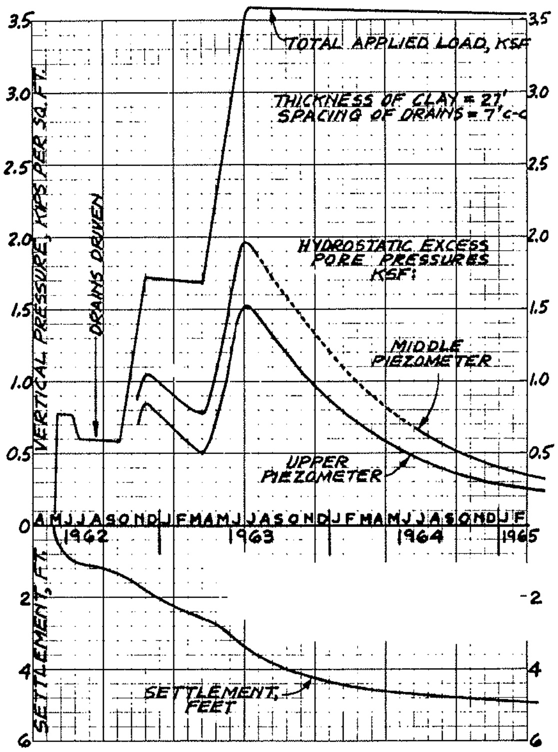

Figure 3. Loads, pore pressures, and settlements—piezometers 13 and 13A.

SUMMARY OF FIELD OBSERVATIONS

Three sets of piezometer and settlement observations taken at locations of thick organic clay and relatively high pore pressures are shown in Figures 1, 2, and 3. Piezometer readings are plotted as kips per square foot of hydrostatic excess pore water pressures, compared to the total applied pressure of fill including sand blanket, permanent fill, and surcharge. The dashed section of the pore pressure record is that interval when middle and lower piezometers were inoperative and the record is interpolated to June or July 1964 when the replacement piezometers were under observation. The small decrease in applied load in periods following placing of fill is due to settlement of the base of fill below the upper water table.

The progress of primary consolidation taken at various dates in the stabilization program, in terms of settlement, effective stress increase, and void ratio decrease, is listed in Table 2. While the history of pore

TABLE 2

PROGRESS OF PRIMARY CONSOLIDATION DETERMINED FROM FIELD OBSERVATIONS

Property	Initial Conditions Jan. 1962	Completion of Fill May-July 1963	Final Sampling June 1964	Surcharge Removal Feb. 1965	100% Consolidation Under Surcharge
Thickness of clay stratum, ft	25.5	21.8	21.1	20.85	20.7
Measured settlement, ft	—	3.7	4.4	4.65	4.8 (Theoretical)
Decrease in void ratio	—	0.46	0.55	0.58	0.60
Void ratio	2.20	1.74	1.65	1.62	1.60
Total applied load, ksf	—	3.4	3.4±	3.4±	3.4±
Observed maximum hydrostatic excess, ksf	—	1.9	0.8	0.5	0.0
Average hydrostatic excess, ksf	—	1.1	0.4	0.25	0.0
Average percent primary consolidation, stress basis	—	68	88	93	100
Average percent primary consolidation, settlement basis	—	77	92	97	100

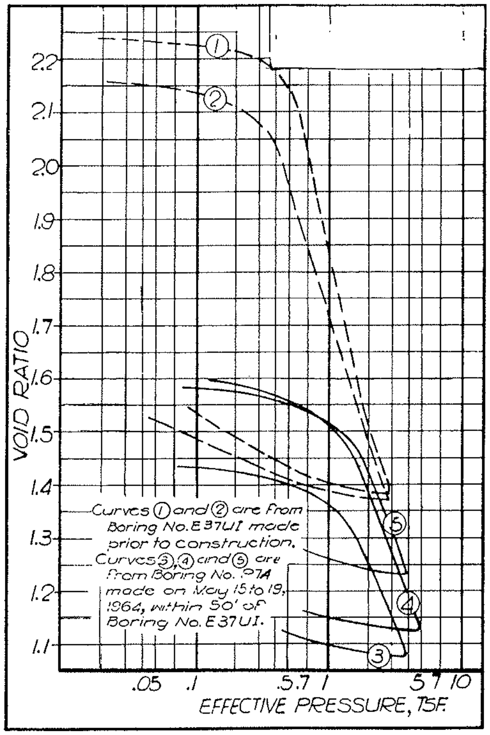

Figure 4. Consolidation curves, before construction and during stabilization.

pressure buildup varied with the sequence of load application, the maximum hydrostatic excess midway between drains and at midheight of the thick clay layer was fairly consistent at 55 to 60 percent of the total applied load at the completion of fill in May to July 1963. The average hydrostatic excess at this time was about 32 percent of total applied load, and completed primary consolidation averaged 68 percent of the ultimate under full surcharge, in terms of effective stress transfer. In a number of locations the pore pressures at the middle of the layer and at the lower quarter point were roughly equal, whereas pore pressures at the upper quarter point averaged about 0.3 kip/sq ft less than those measured lower in the layer. Pore pressures at the lower quarter points are not plotted in Figures 1, 2, and 3.

At the end of the 400-day minimum surcharge period in September 1964, the maximum hydrostatic excess averaged 0.7 kip/sq ft, about 21 percent of total applied pressure. The overall primary consolidation averaged about 89 percent of ultimate, in terms of effective stress transfer. In February 1965 the maximum hydrostatic excess was 0.5 kip/sq ft, about 16 percent of total applied pressure, and consolidation averaged about 93 percent of the ultimate. The amount of load to be removed in excavation of the surcharge after settlement averaged about 0.9 kip/sq ft, and it was determined that surcharge could be removed at any time after November 1964. Surcharge removal commenced in February 1965 and subsequent measured settlements have been insignificant. The numerous alignment stakes indicated that horizontal movements during construction did not exceed several tenths of a foot and were everywhere within tolerable limits.

No complete information was obtained as to the maximum pore water pressures developed by displacement driving of sand drains. Readings taken at certain piezometers several weeks after driving of drains indicated that the pore pressures built up by displacement stresses amounted to no more than about 6 to 8 ft of water or 400 to 500 lb/sq ft, midway between drains, and that the excess pressures had dissipated in about one month after driving. These relatively low pressures could be explained by the presence of free gas in the organic soils. The gas content is indicated by the large immediate settlement after placing the sand blanket. In any case, the average weight of overburden and fill in excess of original groundwater pressures was about 0.8 kip/sq ft at the time of drain driving and the maximum pore pressures caused by displacement probably were limited by this weight.

FINAL LABORATORY TESTING PROGRAM

Thin-tube undisturbed samples were obtained by fixed-piston sampler in each of the borings made for the six replacement piezometers installed in June 1964. Twelve consolidation tests and 51 unconfined compression tests were performed on these samples to compare with tests for the design investigation and with field performance data. A comparison of typical consolidation tests performed on samples before construction and on samples of June 1964 is shown in Figure 4. It can be seen that the tests on samples

TABLE 3

INCREASE IN SHEAR STRENGTH DURING SAND DRAIN STABILIZATION

Condition	Effective Vertical Stress, psf	Average Test Value of Shear Strength, psf	Ratio of Strength to Effective Vertical Stress, c/p
Initial conditions, Jan. 1962	400	350	Not Valid
Preconsolidation conditions, Jan. 1962	1150	350	0.30
Final sampling, June 1964	3200	920	0.29
Increase over preconsolidation condition, Jan. 1962 to June 1964	2050	570	0.28

of June 1964 exhibit void ratio-pressure curves which continue the trend established in the range of virgin compression by the preconstruction testing. The original test curves were concave upward in the manner expected of good quality undisturbed samples.

The average moisture content of the June 1964 samples equaled 62 percent, compared with 84 percent for the preconstruction test samples. This decrease in moisture content is consistent with a decrease in void ratio of 0.55, computed from the average settlement of 4.4 ft observed in June 1964 (Table 2).

Shear strengths obtained from the 51 unconfined compression tests of June 1964 ranged from 750 to 1250 lb/sq ft and averaged 920. This meant that consolidation to June 1964 had provided a strength increase of 570 lb/sq ft beyond the preconstruction strength of 350. The average effective stress at the strength test sample locations in June 1964 was interpreted as 3200 lb/sq ft from the pore pressure observations. This amounted to an increase of 280 lb/sq ft over the initial overburden pressure, or 2050 lb/sq ft above the preconsolidation stress. The original ratio of strength to vertical pressure (c/p ratio) and the final and incremental ratios are given in Table 3. All values equal approximately 0.3. By contrast, the c/p ratio obtained by analysis of slides in similar Atlantic Coast estuarine materials, when the ground spreads and horizontal pressures are lower than at-rest values, is typically about 0.25. The combined effect of disturbance and high pore pressures ordinarily produced by displacement driving would be expected to greatly decrease shear strength around the drains, at least in the early stages of the consolidation process. However, the damage to strength appears to be compensated by compression at high effective stress.

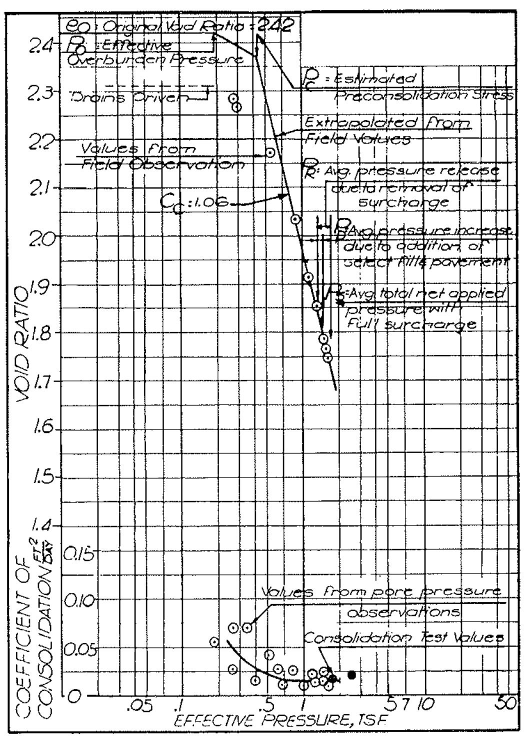

Figure 5. Comparison of field and laboratory data, piezometers P2 and P2A.

CONSOLIDATION CHARACTERISTICS

Figures 5 through 10 compare the laboratory consolidation tests of June 1964 at the six replacement piezometer locations with consolidation properties determined from field observations at the same locations.

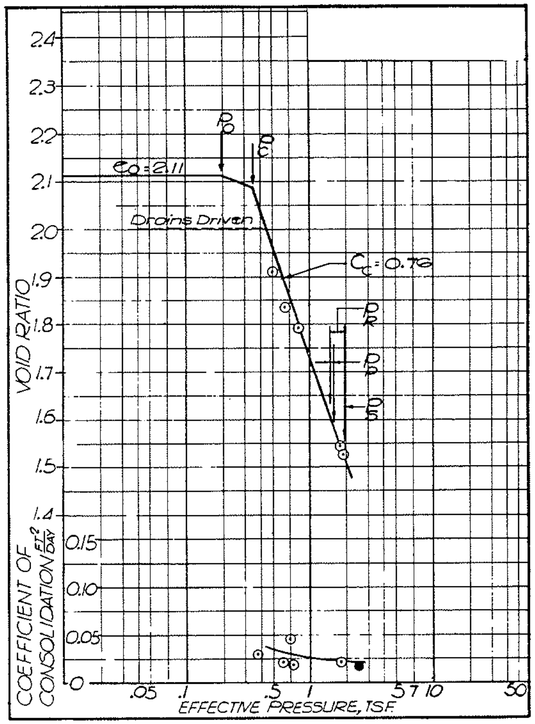

Figure 6. Comparison of field and laboratory data, piezometers P5 and P5A.

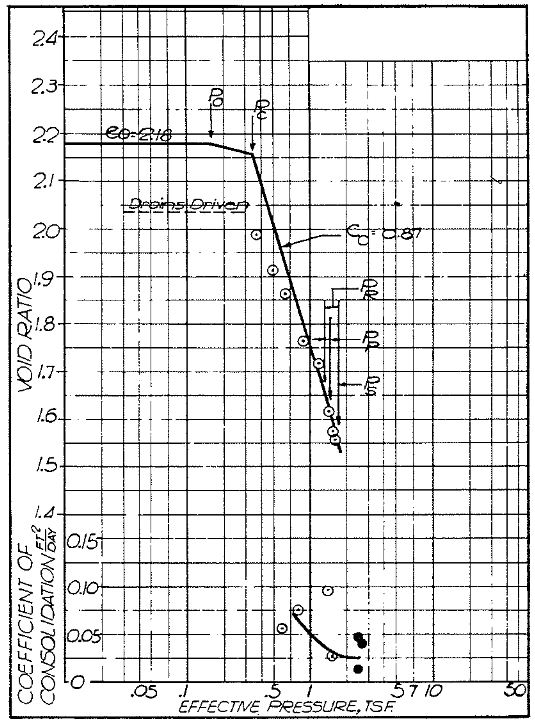

Figure 7. Comparison of field and laboratory data, piezometers P7 and P7A.

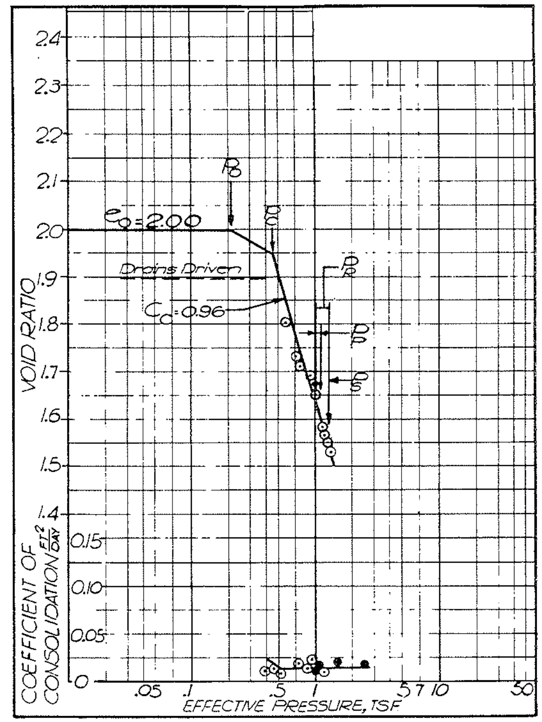

Figure 8. Comparison of field and laboratory data, piezometers P11 and P11A.

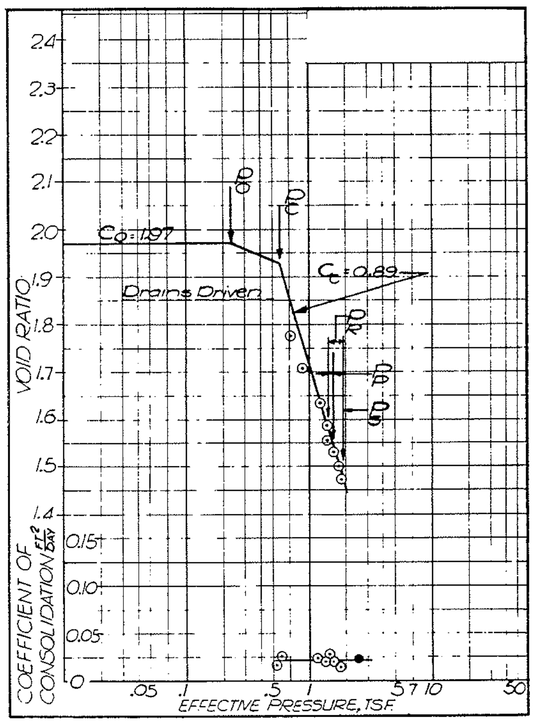

Figure 9. Comparison of field and laboratory data, piezometers P13 and P13A.

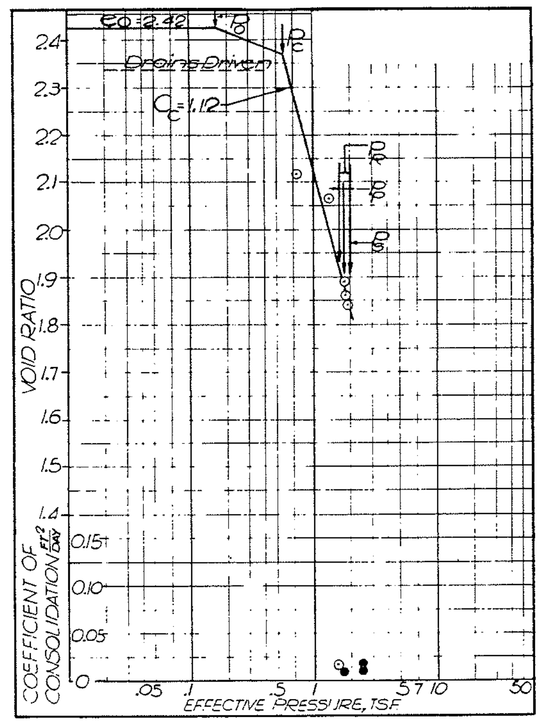

Figure 10. Comparison of field and laboratory data, piezometers P15 and P15A.

In the lower panel of each Figure the coefficient of consolidation computed from observed pore pressure dissipation is plotted with values computed from the June 1964 tests in the higher pressure ranges. Field values, indicated by open circles, are point values computed for individual piezometers from the decrease in excess pore pressure during periods of constant load, assuming radial drainage to sand drains and ignoring smear and disturbance. The laboratory test values, indicated by solid circles, were obtained from conventional tests with vertical double drainage.

In certain locations, where pore pressures could be evaluated in the recompression range of loading, field values show a marked reduction of consolidation coefficient from the recompression to the virgin compression range. However, this difference is not as great as in the original laboratory tests, where the ratio of c_v values was at least 5 to 1. At the higher pressure range, field c_v and laboratory test values correspond fairly well, varying from 0.013 to 0.025 sq ft/day and averaging 0.017 sq ft/day. These are to be compared with the c_v of 0.03 sq ft/day assumed for design as an average throughout the entire pressure range. The difference is undoubtedly due in part to disturbance caused by displacement driving.

The upper panels in Figures 5 through 10 represent a reconstruction of the field compression curve, derived in the following manner: The observed hydrostatic excess pressures were plotted in profile on the vertical line at the piezometer locations for a series of dates during the observation program. The average pore pressure on this line midway between drains was corrected to reflect lower pore pressures near the drains. The average effective stress at this date was obtained by subtracting the average pore pressure from overburden plus applied load. The void ratio at corresponding dates was obtained by subtracting the void ratio decrement, computed from observed settlements, from the original average test value of void ratio at this location. The corresponding void ratio vs effective pressure points are plotted as open circles. The straight line for virgin compression is extrapolated backward from the circled values at higher pressures, ignoring the concave upward shape of the laboratory test curve. The straight line segment in the recompression range is estimated from the laboratory tests, extending from initial overburden pressure to intersect the virgin slope at the apparent preconsolidation stress.

As a further check the estimated ultimate primary settlement was obtained by extrapolating the field log time vs settlement curves and verified by the square root of time method. The corresponding void ratio when plotted against the average total net applied pressure with full surcharge load gives a point that is in very close agreement with the field compression curve.

It can be seen that the circled field values fall below the extrapolated virgin compression straight line at lower pressures in the manner of disturbed laboratory consolidation test curves. The reconstructed C_c values range from 0.76 to 1.12 and the average slope of the straight-line virgin compression line equals 0.94, compared to the design value of 0.95. Such a close agreement must be considered fortuitous, based on the chance similarity of materials utilized in the comparison. It appears that although disturbance in sand drain driving lowers the equilibrium void ratio for the initial loading, it does not significantly alter the void ratio reached at much higher pressures.

Various stress conditions are indicated on the reconstructed field compression curves in Figures 5 through 10, as follows:

P_o = initial effective overburden;

P_c = preconsolidation stress;

P_s = load under full surcharge;

P_R = load decrement on removal of surcharge; and

P_p = load increment on addition of base course and pavement.

CONCLUSIONS

This study is intended to provide a comparison between laboratory design values and soil properties determined from field observations relative to primary consolidation of a soft organic clay. The raw measurements from field observation programs, which are presented merely as time curves of settlement or pore pressures, are only of passing interest in that form. Utility for future designs requires an analysis of observations to derive from them parameters of significance to the design problem.

1. The evidence indicates that the effect of displacement driving of sand drains in this particular case is less than has frequently been reported, probably because of the cushioning effect of free gas within this organic soil.
2. The value of the c/p ratio deduced from the strength increase during stabilization equals about 0.3 compared to the ratio of 0.25 which has been evidenced in shear failures of similar estuarine deposits. Evidently the inevitable damage to strength by displacement driving has been compensated by compression at high effective stress.
3. The final total settlement can be computed with reasonable accuracy from conventional tests on undisturbed samples in the situation where loading extends far into the virgin compression range. Displacement driving of drains appears to have increased the amount of settlement in the early stages of load transfer without altering the final equilibrium void ratio under high loads. The increased early settlements may result from a combination of driving disturbance, gas contained in the organic soil, and a higher c_v value in the recompression range of loading. Whatever the cause, a c_v value computed from this rapid early settlement record cannot be utilized alone to extrapolate rates of consolidation and times for surcharge removal.
4. Disturbance by displacement driving appears to reduce substantially the high c_v values in the recompression range determined from laboratory tests. However, the c_v values under larger loads well within the virgin compression range appear to be little altered by this field condition.
5. At the present state of knowledge, a precise sand drain design employing a carefully determined ratio of vertical and horizontal coefficients of consolidation with an allowance for smear or disturbance is difficult to justify for organic soils of the type tested here and for conventional methods of installation. The procedure utilized in this design, wherein an average vertical coefficient of consolidation is utilized with no allowance for smear, appears appropriate. However, the c_v value must be selected conservatively, generally disregarding in the selection the higher coefficients given in the recompression range by better quality laboratory tests.