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Performance of an Embankment on Peat

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SYNOPSIS A widely held view is that rates of compression of peats are controlled by secondary effects and thus cannot be analyzed using primary consolidation theory. Data are presented here for the time rates of settlement of an embankment on peat. Theoretical analyses based on laboratory vertical and radial flow consolidation tests, and utilizing a finite difference scheme, indicated that the soil had undergone a degree of mass flow and was disturbed during or before jetting of sand drains, but that the field settlements could be predicted rationally using primary consolidation theory.

INTRODUCTION

There is a widely held view that the time rate of consolidation of peats is controlled by secondary effects and that the classical theory of consolidation has little, if any, application in prediction of time rates of settlement. Laboratory and field data in support of this view were presented by Lake (1961) who found that rates of compression for peats at two sites were unaffected by drainage distance. Lewis (1963) drew similar conclusions for a field test at another site and Casagrande and Poulos (1969, p. 325) concluded, from a number of case histories, that "sand drains are of no value in highly organic materials."

In some of the field and laboratory cases where the shape of the time-settlement curves gave little evidence of primary consolidation, the problem may actually have been some combination of time dependent loading, partial saturation, mass flow, non-homogeneous soils, large reductions in hydraulic conductivity with effective stress, non-linear ε-$\bar{\sigma}$ curves, large strains, disturbance, and other such effects. Some of the laboratory and field data appear to have been influenced by effects of overconsolidation due to secondary consolidation. Even inorganic clays behave in a non-Terzaghian manner when overconsolidated.

An opportunity developed to become involved in the analysis of an embankment on peat during the design phase. After construction, additional samples were taken and tested in the laboratory; and analyses, reported here, were performed to try to determine if primary effects had occurred.

FIELD DATA

In 1974, the Eastern New Hampshire Turnpike, I95, was widened. At the crossing of the Taylor River, approximately 800 feet of roadway was constructed on an embankment on a tidal marsh underlain by peat, organic silts, and clays. As a result of large expected settlements, and questionable stability, the subsoil was provided with sand drains and was instrumented to obtain data on settlements and lateral movements. The area of interest here is at Station 194+50 where a settlement platform (SP37) was placed 41.4 feet right of the center line (R41.4') and first read on June 22, 1974. Times will be measured relative to June 1, 1974 so the first reading was on construction day (CD) 22. A section through the embankment in that vicinity is shown in Fig. 1. An inclinometer was placed at Sta. 194+50 at R95.5' and an

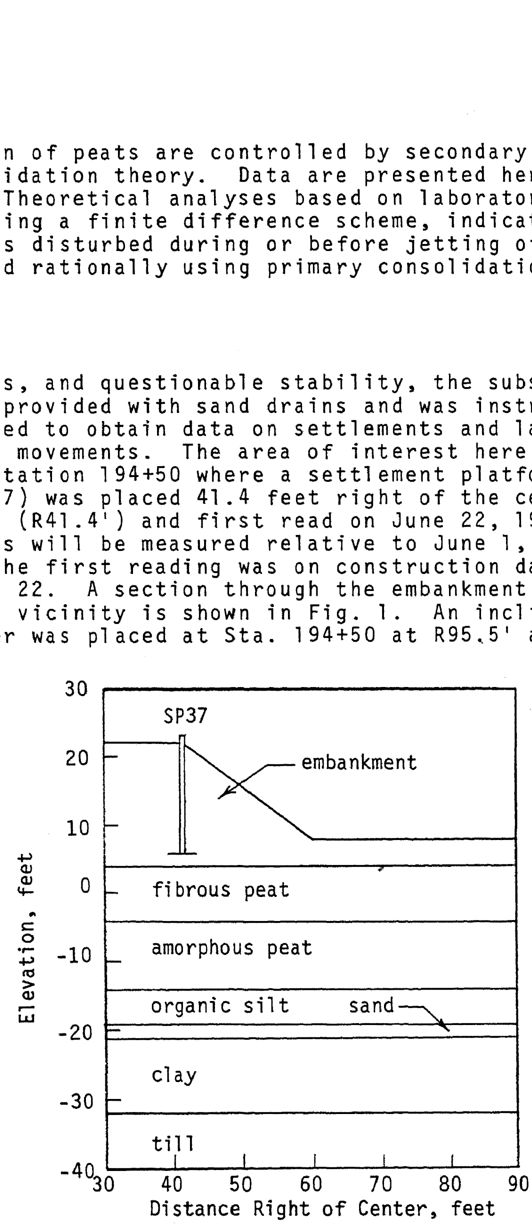

Fig. 1. Idealized Cross Section Through the Site in the Vicinity of Station 194+50

alignment stake at Sta. 194+50 at R160'. Representative measurements of fill elevation and settlement at SP37, and horizontal movements of the top of the inclinometer casings and the alignment stake are presented in Table 1.

TABLE I. Representative Measured Movements and Locations

TL days	EFL feet
0	4.0
22	*
25	7.0
35	*
40	8.0
72	*
80	6.0
85	7.5
160	22.0
340	*
340	19.0

TA days	DHA feet
20	0.0
30	0.8
40	5.3
54	6.8
80	7.2
110	7.9
180	

TM days	SM feet	TIN days	DHIN feet
22	0.0	89	0.0
25	0.6	98	0.1
32	1.9	105	0.2
40	2.1	109	0.3
46	2.2	111	0.4
50	2.5	118	0.5
53	3.9	122	0.6
56	4.6	129	0.7
60	4.9	133	0.8
67	5.3	143	0.9
73	5.6	153	1.0
80	5.6	157	1.1
87	6.0	161	1.2
97	6.7	172	1.3
113	7.9	190	1.4
127	8.6	211	1.5
148	9.4		
160	9.8		
171	10.0		
210	10.6		
235	10.7		
280	10.7		
340	10.8		

TL = construction day when a fill elevation was defined
EFL = fill elevation, a * denotes the beginning of a construction period
TM = construction day when a settlement was measured
SM = measured settlement
TIN = construction day when the inclinometer was read
DHIN = horizontal movement of top of inclinometer casing
TA = construction day when location of alignment stake was measured
DHA = horizontal movement of alignment stake

The approximate stratigraphy is shown in Fig. 1. Conditions varied across the site and terms used in field and laboratory descriptions sometimes conflicted, e.g., in classifying a sample as fibrous peat or amorphous peat, so the borders between layers were indistinct.

Twelve-inch diameter sand drains were installed in a square pattern, four feet on center, by jetting, to a depth of 27 feet (into the sand), on day 50. The water table was located essentially at the elevation of the original ground surface, which was 4.0 feet.

LABORATORY DATA

The results of a large number of index tests are summarized in Table 2. Although the water contents and Atterberg limits are high, they are not as high as for the peats encountered in some studies referenced earlier.

TABLE II. Soil Properties

Soil Type	Total Unit Weight pcf	Natural Water Content, %	Liquid Limit %	Plastic Limit, %
Fibrous Peat	59-72	200-875	325-650	100-375
Amorphous Peat	65-77	125-375	150-300	50-150
Organic Silt	84-100	45-175	45-135	30-60

Twenty-one conventional, incremental, one-dimensional consolidation tests were performed by the designer (Haley and Aldrich) on 0.75 to 1.00 inch thick by 2.5-inch diameter specimens obtained using 3-inch shelby tube samplers. After completion of the project, an additional boring was made, 3-inch shelby tube samples obtained, and three radial inflow and two vertical flow tests were performed at the University of Texas. Radial coefficients of consolidation were determined using the free strain theory (Trautwein et al., 1981).

Laboratory curves of settlement versus log time often yielded indistinct breaks between primary and secondary consolidation and were not found useful. When standard 24-hour loading procedures were used, the root plots were often continuously curved or had other shapes indicating non-Terzaghian consolidation. One radial flow and one vertical-flow consolidation test were performed using a quick-loading procedure in which successive loads were applied as soon as primary consolidation was completed under the present load. For these tests there was a substantial linear portion on the root plots and coefficients of consolidation were reasonably well defined. Coefficients of consolidation for the tests performed at the University of Texas on samples of the fibrous peat are presented in Fig. 2. The radial coefficients of consolidation (c_r) were about ten times the vertical values (c_v). This c_r/c_v ratio is surprisingly high for a soil that had no visible stratification.

Values of c_v were selected for the other layers in the upper range of the scatter bands. Selected values are shown if Fig. 3 for the fibrous peat, amorphous peat, organic silt, and clay. The c_r/c_v ratio was assumed to be ten.

Stress-strain curves for the peat, silt, and clay layers are shown in Fig. 4. The curves for the organic silt indicate that all samples were badly disturbed.

ANALYSES

Analyses were performed using a two-dimensional finite-difference scheme, using a program called SD4, which is a further development of the programs used in earlier studies (Olson et al., 1974; Pelletier et al., 1979). The program allows the soil to be horizontally stratified, and included sand layers may be assumed to be freely draining. Each layer is assumed to be instantaneously homogeneous but the coefficients of consolidation are any desired function of mean vertical effective stress. The stress-strain relationship may utilize either void

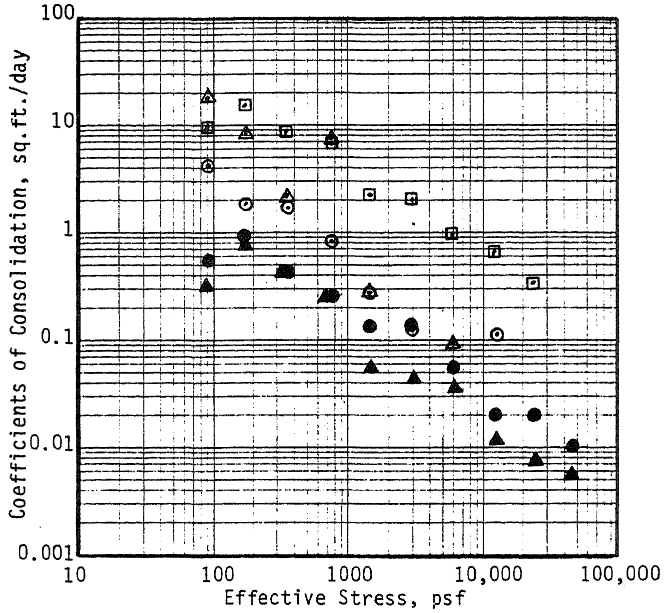

Fig. 2. Coefficients of Vertical (solid symbols and Radial (open symbols) Consolidation for Fibrous Peat

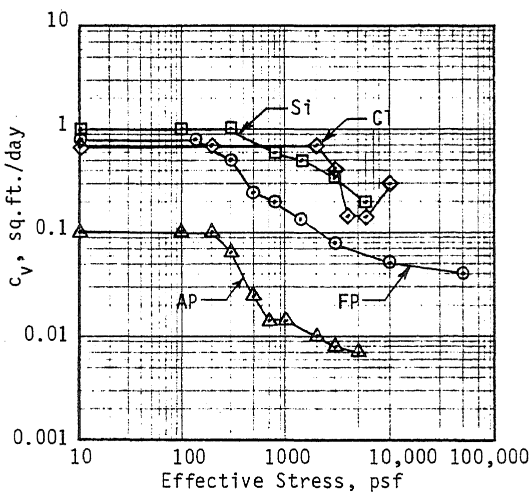

Fig. 3. Average Vertical Coefficients of Consolidation for "Undisturbed" Samples of Fibrous Peat (FP), Amorphous Peat (AP), Organic Silt (Si), and Clay (Cl)

ratio or strain and may be non-linear. The effects of large strain are approximated by updating the thickness of each layer after each time step. The applied load is fill with the elevation of the fill specified during construction periods. Effects of stress distribution are taken into account using influence factors which are depth dependent and may

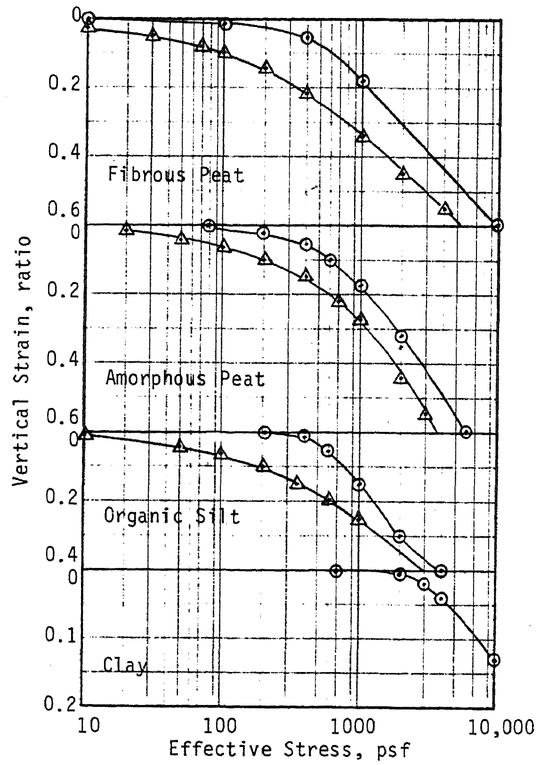

Fig. 4. One-Dimensional Stress-Strain Curve for "Undisturbed" (open circles) and Partially Disturbed (open triangles) Soils

differ for each construction stage to account for stress distribution by previously applied fill. Drains may be installed at any time with one-dimensional flow before drain installation and two-dimensional flow thereafter. The drains may penetrate only partially through the compressible layers. The effects of disturbance are approximated by assigning a new set of soil properties at the time of drain installation. Because disturbance results in a reduction in effective stress at constant void ratio, there is an automatic incremental change in excess pore water pressure.

The layering assumed for the analyses is shown in Table III.

TABLE III. Initial Condition

Layer No.	Soil Type	Thickness ft.	Submerged Weight, pcf
1	Fibrous Peat	4.0	4.0
2	Fibrous Peat	4.0	4.0
3	Amorphous Peat	10.0	8.0
4	Organic Silt	5.0	34.0
5	Sand	2.0	70.0
6	Clay	11.0	48.0

RESULTS OF ANALYSES

The first two analyses were performed assuming no drains, and using drains as they were installed, but assuming no disturbance from jetting. The predicted and measured time-settlement curves are compared in Fig. 5. According

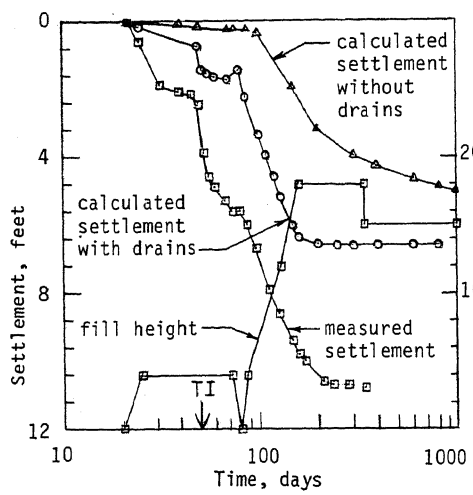

Fig. 5. Curves of Settlement and Fill Height versus Time. Analyses Utilized "undisturbed" soil properties.

to the analyses, the drains should have reduced the consolidation times greatly. However, measured settlements are much larger, and occur more rapidly, than the predicted settlements. The major discrepancy occurs when the first fill is applied, when stresses are so low that no significant volume change would occur. Based on field observations it is believed that this first large settlement was due to local mass movements as fill was pushed onto the site by bulldozers.

This movement was too local to be related to lateral movements in the slope indicator or at the alignment stake. With no means available for estimating the amount of such movement, and in keeping with the interest in studying only consolidation movements, the "measured" settlements were adjusted to match the calculated settlements up to the time of drain installation. The error associated with this adjustment should be quite small because the soils are relatively incompressible under these stresses (Fig. 4) and because of the short times involved.

However, even after correcting the measured settlements, the data still indicate that settlement occurs much more rapidly than is predicted by theory, a surprising result considering that the values of c_r are higher than one would expect. Several explanations seem available. One is that the settlement is due, in part, to large scale mass movement. A second is that jetting the holes resulted in an inwards movement of the side walls due to the overburden pressure of the fill, thus causing instantaneous settlement. A third is that the soil is disturbed, thus lowering the ε-log $\bar{\sigma}$ curves (also reducing c_v and c_r). A fourth is that the field c_v values are much greater than the lab values.

Measurements of horizontal deflections are shown in Table 1. The major discrepancy between the corrected measured and computed time-settlement curves (plot not included) develops before the major fill construction which began on about CD80. During the time period between CD50 and CD80 there was less than 0.2 feet of horizontal movement of the slope indicator casing but 3.1 feet of settlement. With the fill extending completely over the area of SP37 at this time there was little chance of large local lateral displacement and thus large scale mass movement is an unlikely explanation.

Undrained shearing strengths used for stability analyses were in the range of 300 to 600 psf. Considering that the holes were full of water, the developed shearing stresses should have been significantly less than the shearing strengths and large scale inward movements around the drain holes seem unlikely. In any case, there was no field evidence of such movements visible to the author.

The single analysis for vertical consolidation only (Fig. 5) indicated that an underestimate of c_v was unlikely to be an important cause of the discrepancy because of the dominence of horizontal flow. The horizontal coefficients of consolidation are already so large that it seems unlikely that they underestimate the field values by a large amount.

The primary cause of the discrepancy is thus believed to be disturbance, partially disturbance due to mud waves during placement of the first layer of fill and partially disturbance during jetting. Analyses were performed assuming the disturbance occurred when the drains were installed.

As a first trial the stress-strain curves were rounded, as shown in Fig. 4, to represent disturbance. Then a polynomial was used to relate hydraulic conductivity, k, to vertical strain, ε:

$$k = 10^{**}(a+b\varepsilon+c\varepsilon^2) \qquad (1)$$

where ** denotes exponentiation. The k-ε relationship for disturbed soil was assumed to be the same as for undisturbed doil. The vertical coefficients of consolidation were then calculated as:

$$c_v = k/m_v\gamma_w \qquad (2)$$

The computed and measured time-settlement curves are compared in Fig. 6. The calculated settlements are now a little too large and occur faster than was measured in the field. The degree of disturbance was probably overestimated for one or more of the layers and the c_r/c_v ratio could be reduced to a more typical range. Such adjustments could be made to bring the calculated and measured curves into conformance but

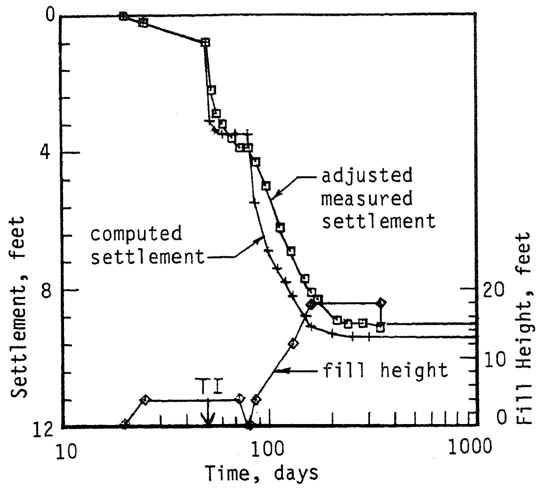

Fig. 6. Comparison of the Adjusted Measured Settlements and the Settlements Computed Assuming the Soils are all Slightly Disturbed.

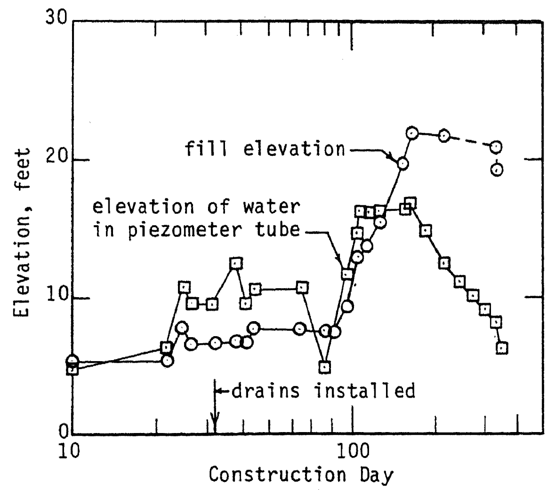

Fig. 7. Variation of Piezometeric Level in One Piezometer as a Function of Time and Fill Elevation

such refinements seem unnecessary here considering that radial coefficients of consolidation were only measured for one layer, that the method used to account for stress distribution is questionable, and that mass movements probably influenced the "measured" settlements in Fig. 6 after drains were installed. The similarity between the measured and computed settlements provides limited evidence that the soils were undergoing primary consolidation.

Further evidence is given by the piezometeric observations. More than 30 piezometers were installed and read regularly. The data are difficult to use in direct comparisons with the analyses for some of the following reasons: (1) With drains on only four-foot centers, it is not possible to define the radial position of the piezometers with respect to adjacent drains with enough precision to account for effects of large radial gradients in pore pressure. (2) The piezometers were of the open tube type and thus had slower reaction times than other types. (3) Some of the riser pipes leaked or overflowed. (4) Some piezometers failed. (5) Piezometers probably reacted to fill applied in nearby areas as well as fill applied (and reported) above the piezometer.

Nevertheless, all of the piezometers showed increases in pore pressure when fill was applied and a time dependent decrease, thereafter. An example is shown in Fig. 7 for a piezometer at station 193+97, R42.6 feet, with its tip in the fibrous peat at depth 7.0 feet. The data show an increase in pore pressure at the time the drains were jetted, increases in pore water pressure during construction stages, rapid dissipation of pore water pressure during early stages when c_v and c_r were high, gradual dissipation of pore water pressures as settlement occurred under surcharge, and rapid decrease in pore pressure at the end when surcharge was removed.

SUMMARY AND CONCLUSIONS

An embankment, rising 18 feet above the ground surface, was constructed above a soil profile containing about twenty feet of peat. Sand drains were used to accelerate consolidation. Laboratory tests indicated that the vertical coefficients of consolidation were high and that horizontal values were about ten times the vertical values. Nevertheless, settlement occurred much more rapidly in the field than predicted. The explanation seems to be that a significant amount of mass movement occurred and that the soil was disturbed during filling and/or drain installation operations. The soil appears to have undergone primary consolidation but with large changes in soil properties, large strains, a non-linear stress-strain curve, and other effects that deviate from classical theory. Field observations did not extend out onto a secondary settlement curve.

The most useful laboratory data came from tests with rapid loading and root-time curve fitting.

ACKNOWLEDGEMENTS

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