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J. B. FORREST AND I. C. MACFARLANE

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ETUDES SUR PLACE DE LA REPONSE
DE LA TOURBE CONSOLIDEE

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Les auteurs présentent les résultats des recherches menées sur place sur les caractéristiques de déformation d'une tourbe molle dans le voisinage d'une solution de continuité du chargement. Ils comparent les différences de tassement sur place et au laboratoire, ainsi que les vitesses de chargement, les tassements observés et la diminution des pressions interstitielles. Les auteurs étudient ensuite brièvement les limitations de la théorie linéaire de l'élasticité.

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Proceedings of the American Society of Civil Engineers

FIELD STUDIES OF RESPONSE OF PEAT TO PLATE LOADING[a]

By J. B. Forrest,[1] and I. C. MacFarlane[2]

Peats are among the worst kinds of foundation material that may be encountered. Because of their extremely compressible nature in their natural state and their low shear strengths, they are often unsuitable for supporting structures of any kind. Methods of dealing with construction over peat include such techniques as: (1) Replacing the peat with inorganic materials; (2) carrying the foundation supports down to a better stratum; or (3) some form of stabilization or improvement of the peat properties in situ, such as preloading.

As the last-named method is often the least expensive, it has received much attention of late, particularly with regard to highway projects where very large areas of peat or organic soils are encountered. The preloading technique consists essentially of subjecting the in situ peat to a load in excess of that to be imposed by the final structure. In this way, settlements equal to the expected magnitude under the final loading, are secured relatively quickly; the excess load is then removed and the structure completed.

Efficient use of this technique requires the ability to predict in advance the behavior of peat with particular reference to: (1) The final settlements to be expected under different loads; (2) the rates at which such settlements will occur; and (3) the strength characteristics of the soil, as these control the allowable loading. The first two considerations are controlled primarily by the consolidation characteristics of the peat. Consolidation and increase in shear strength are related through rate of pore pressure dissipation.

Consolidation of a saturated soil is a time-dependent volume reduction involving a decrease in the water content of the soil. Any soil is a system of two or three spatially co-existent phases: a solid phase; a liquid phase; and

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sometimes (particularly for peat), a gas phase. When there is an increase of pressure on such a system in equilibrium, there is a volume change with an excape of fluid from the system. This process of volume reduction (consolidation) involves a time lag.

Consolidation of inorganic soils is thought to be divided into two stages: the primary consolidation stage (described by the classical concept of Terzaghi); and the secondary consolidation (or compression) stage. The time lag in the primary consolidation stage is associated with the dissipation of excess pore water pressures and results from the resistance to volume change offered by the escaping water. The time lag in the secondary compression stage is assoicated with plastic flow or creep, and, in effect, is due to resistance offered by the "solid" phase to volume change in the system.

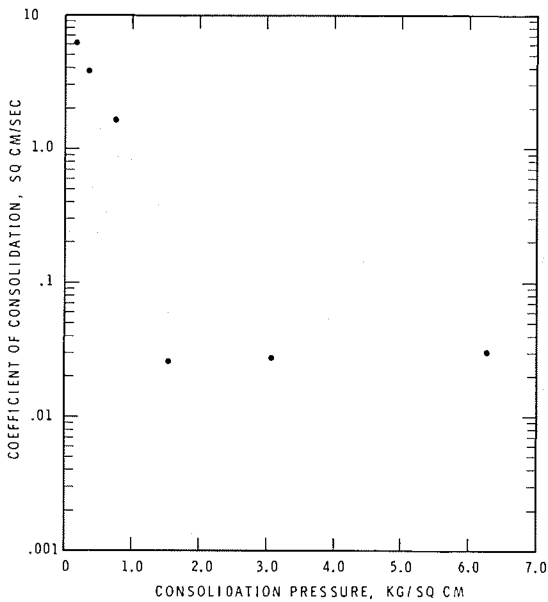

FIG. 1.—COEFFICIENT OF CONSOLIDATION VERSUS CONSOLIDATION PRESSURE

The approach to the consolidation process of peat has been generally similar to that for clays that exhibit exceptionally large secondary compression effects. Most investigators have been preoccupied with the need to obtain immediate results and have relied on this concept. A literature review (13)[3] clearly indicates that the two-stage concept of consolidation, one terminating at a clearly defined point and the other continuing for a long period of time, leaves something to be desired with reference to peats. The point at which primary consolidation ends and secondary compression begins is obscure in most cases.

Because application of the classical (Terzaghi) consolidation theory utilizes

[3] Numerals in parentheses refer to corresponding items in the Appendix I.—References.

curve fitting techniques (22), which require determination of the point of 100% primary consolidation, the standard consolidation test used for inorganic soils is much more difficult to interpret when used for peat. The shortcomings of this theory when applied to organic soils are emphasized in Fig. 1, where values of the (Terzaghi) coefficient of consolidation, c_v (calculated using measured values of vertical permeability), have been plotted against consolidation pressure for a sample of fibrous peat. The extreme variation of c_v with applied pressure below 1.5 kg per sq cm is primarily due to the drastic changes in coefficient of permeability, k, with consolidation. It is within this range of stresses (below 1.5 kg per sq cm) that most loadings applied to peat may be expected to fall. Similar results have been reported by Lea (10).

Although some success has been reported (6,20) in predicting the magnitudes of peat settlement from laboratory results, other writers (14,16,17,24) have not been able to support this claim. The prediction of rates of settlement on the basis of laboratory work has proved an even more difficult problem (1, 4,6,7,11,15,20).

The complexity of the consolidation rate phenomena is illustrated by the work of Lake (8,9), Root (18) and Brawner (2), all of whom found that sand drains did not significantly affect the rate of settlement of peat although they did increase the rate of excess pore water pressure dissipation. Consolidation and pore pressure dissipation are directly related, however, in the classical theory of consolidation.

It would appear, therefore, that the techniques available within the present framework of soil mechanics are not completely satisfactory when applied to peats; the stress-strain-hydrodynamic relations for peats require further investigation. This should deal not only with the soil behavior under usual conditions, such as unidimensional and hydrostatic compression, but should also examine response interrelationships under the conditions occurring in the vicinities of loading boundaries. The pore pressure distributions set up near applied loads and their dissipation patterns should also be observed.

This paper represents the results of a small-scale field investigation carried out on a 10-ft thick stratum of peat. Rates and magnitudes of settlement occurring at different depths beneath a confined surface loading were measured. Pore pressures were recorded at various points throughout the loaded peat stratum, and rates of pore pressure dissipation were compared with rates of consolidation and loading. Field results were compared with results obtained from a limited number of laboratory consolidation tests.

SOIL CONDITIONS

The sites chosen for the field tests were near Ottawa, in a confined muskeg area of general muskeg classification EI-FI, according to the Radforth System (12). The soil profile is shown in Fig. 2. The actual test sites were characterized by low woody shrubs intermixed with patches of hummocky mosses and short grasses. The loading tests were conducted on grassy FI muskeg, underlayed by nonwoody fine fibrous peat with specific gravity of solids of about 1.41 and organic content of from 87% to 93%. Variation of water content with depth, for different locations throughout the test area, is shown in Fig. 2. Water content is the ratio, given as a percentage, of the weight of water driven

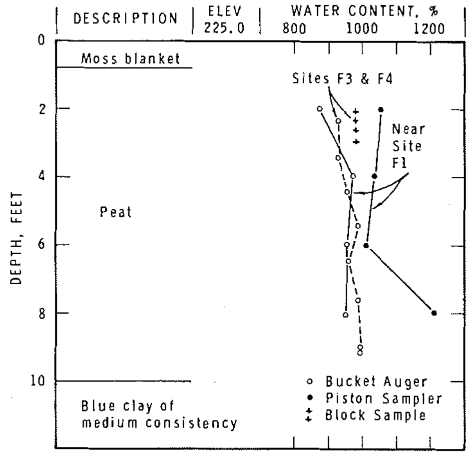

FIG. 2.—SOIL PROFILE IN THE VICINITY OF FIELD TESTS

off when the peat samples were dried at 105° C for 2 days, to the dry weight of material remaining.

TEST EQUIPMENT AND INSTRUMENTATION

Loading.—Loads were applied to the muskeg surface through a 36-in. diam circular wooden plate which supported a platform carrying eight 45-gal drums (Fig. 3). The load was applied in two increments. The first increment, consisting of the weight of the plate, which supported the platform and 45-gal drums, amounted to approximately 0.037 kg per sq cm (76 psf). The second increment of loading was applied by pumping the barrels full of water, thus increasing the average contact pressure by about 0.254 kg per sq cm (520 psf), for a total load of approximately 0.291 kg per sq cm (596 psf).

Deformation Measurements.—Settlement was measured at four elevations: at the ground surface; and at depths of approximately 2 ft, 4 ft, and 6 ft directly below the center of the plate. The gages (Fig. 3) were made up of three telescoping pipes with helical plates 2 in., 3 in., and 4-1/2 in. in diameter welded on their ends. Elevations were read periodically with a precise level at the tops of the settlement rods and on the platform.

To determine the effects of loading on the muskeg surface surrounding the plate, two types of reference pins were used. Large-diameter-head steel pins, 4 in. long, were placed flush with the top of the mat in the vicinity of the loading plate and elevations were taken periodically on the tops of these pins. During loading, regular observations were also made of the tilting of several 3-ft long steel "tilt" pins inserted 2 ft into the mat, 2.5 ft and 4.5 ft from the center of the loading plate.

Pore Pressure Measurements.—Pore pressures were measured at various locations beneath the plate, both during and after load application. "Geonor" piezometers, shortened to one-third their initial effective lengths, were pushed into the muskeg from outside the periphery of the plate. The spacing of piezometers was a follows: one each at depths of 2 ft, 4 ft, and 6 ft under the center of the plate; and one each at depths of 2 ft and 4 ft under the perimeter of the plate. The piezometers were connected by 1/4-in. nylon tubing to Bourdon gages at the muskeg surface, thus providing an essentially constant volume system. These nylon tubes were kept completely buried in the peat, up to the place where they entered a waterproofed wooden box which housed the gages.

This box enabled the gages to be situated below the water table, preventing them from being subjected to a negative gage pressure, which might have brought any dissolved air in the pressure system out of solution. An additional benefit of placing the gages and gage lines below the groundwater table was the provision of a degree of insulation which would minimize temperature effects. Piezometer readings were observed only in conjunction with the second load

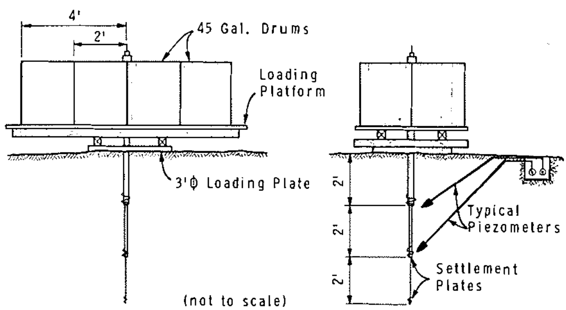

FIG. 3.—LOADING ARRANGEMENT AND INSTRUMENTATION

increment, as the initial platform load was insufficient to generate measurable pore pressures.

TEST PROCEDURES

Four field tests, numbered F1 to F4, were carried out at intervals of 3 weeks to 2 months. The procedure was as follows: the plate was placed on the muskeg surface; the settlement gages and piezometers placed in their desired locations with reference to the plate; and the initial readings taken.

For loading tests F1 and F2, the loading plate was placed directly on the upper moss mat. To exclude the compression of this material from that of the peat proper, in tests F3 and F4 the living moss mat was removed and the loading plate placed on a 2-in. thick cushion of sand laid over the raw peat.

The platform and 45-gal drums were then placed on the plate. After about 3 days, when settlement under the platform load had virtually ceased, the second load increment was applied by pumping the barrels full of water. Because it was necessary to pump water from different sources for the different load tests, loading times for the second increment of loading varied considerably. Care had to be taken during the loading procedure to avoid unbalancing the platform, which could result in tilting, and eccentric load distribution.

While the 45-gal drums were being filled, settlement readings and pore pressure gage readings were taken every 5 min. After pumping was completed, readings were taken at increasingly longer intervals, depending upon the rate of variation of the quantities being observed. Only field test F1 included an unloading and reloading cycle.

RESULTS

Deformations.—Typical plots of settlement against time, after the start of major load application, are shown in Fig. 4. The curves in this figure denote the settlments occurring during test F3, at four locations initially at depths of 0 ft, 2 ft, 4 ft, and 6 ft beneath the center of the plate. The settlements do not start from zero at very small times because the settlement that occurred during the initial platform loading period is included, whereas the time scale represents only the length of time following the start of the application of the second load. The period of loading is also shown in Fig. 4. The inclusion of the initial settlement with the settlement that occurred under the major load increment is followed throughout this paper.

In Fig. 5, the percentage settlement in the 0-ft to 2-ft layer, during all four field tests, is plotted against the time after major increment of load application began. Settlement here is given as a percentage of layer thickness. The marked differences between the curves for tests F1 and F2 as compared with tests F3 and F4 are due to the inclusion of the moss mat in the former. Results for the 2-ft to 4-ft and the 4-ft to 6-ft deep layers show less difference and so have been omitted from Fig. 5. A reloading cycle carried out during test F1 is recorded in Fig. 5.

The average values of vertical stress under each load increment for each of the consolidating layers, i.e., 0 ft to 2 ft, 2 ft to 4 ft, and 4 ft to 6 ft, under the center of the plate, have been approximated by the linear theory of elasticity. The average stress occurring in each layer, assumed to be the calculated vertical stress at mid-depth, has been plotted in Fig. 6 against the final deformation of that layer for tests F3 and F4.

Fig. 7 consists of plots, for test F2, of surface settlement at different times versus distance from the centerline of the loading plate. Table 1 shows the change in inclination of the two steel tilt pins placed 1 ft and 3 ft from the edge of the plate. This figure shows that the surface of the muskeg, in the vicinity of the plate, depresses under the effect of the plate loading, and that this effect decreases outward from the plate. This behavior is further demonstrated by that of the pins (Table 1), which lean inward during loading but tend to regain their original straight position during rebound. These results indicate primarily delayed elastic, as opposed to plastic, deformation of the surface of the peat in the vicinity of the plate. The membrane effect that the moss mat might be expected to exhibit may account largely for this.

Pore Pressures.—Pore pressures for field test F4 are shown in Fig. 8 for piezometers placed approximately 2 ft, 4 ft and 6 ft beneath the center of the loading plate, and 2 ft and 4 ft below its edge. Loading time for this test is also indicated on the figure. Fig. 9 shows pore pressure results, in tests F3 and F4, for positions under the center of the loading plate. For field test F4, a shorter loading period than for test F3, resulted in higher pore pressures. As all piezometer readings reached their maximum values at the same time, i.e., immediately following the completion of loading, these readings

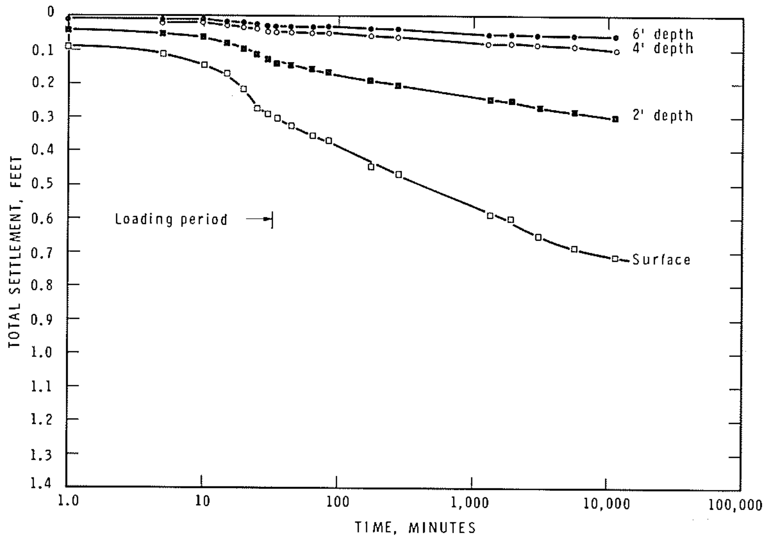

FIG. 4.—TIME-SETTLEMENT CURVES FOR FIELD TEST F3

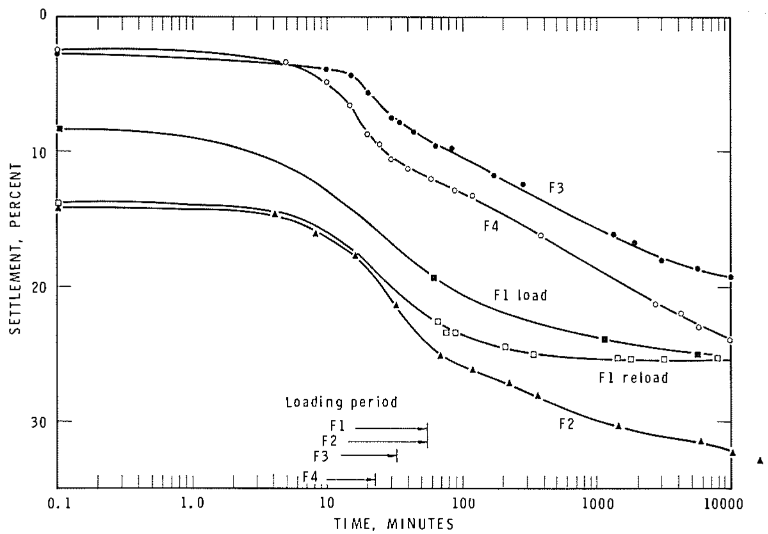

FIG. 5.—TIME-SETTLEMENT CURVES FOR 0-FT-2-FT DEEP PEAT LAYER FOR FIELD TESTS F1 TO F4

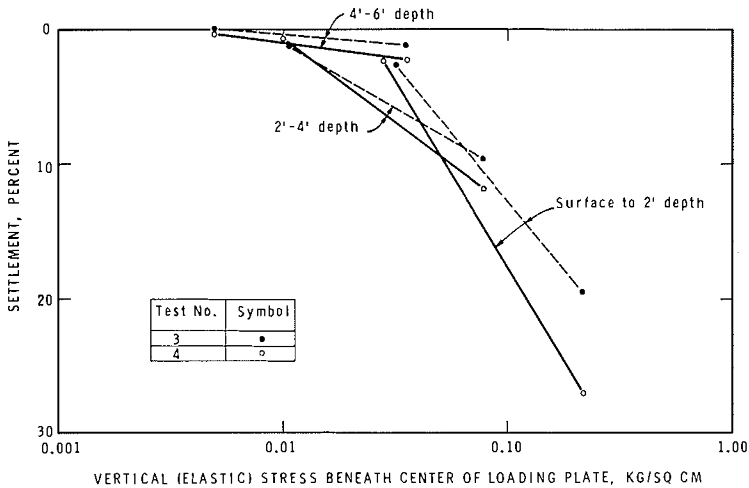

FIG. 6.—SETTLEMENT VERSUS CALCULATED AVERAGE VERTICAL STRESS FOR INDIVIDUAL SOIL LAYERS UNDER FIELD TESTS F3 AND F4

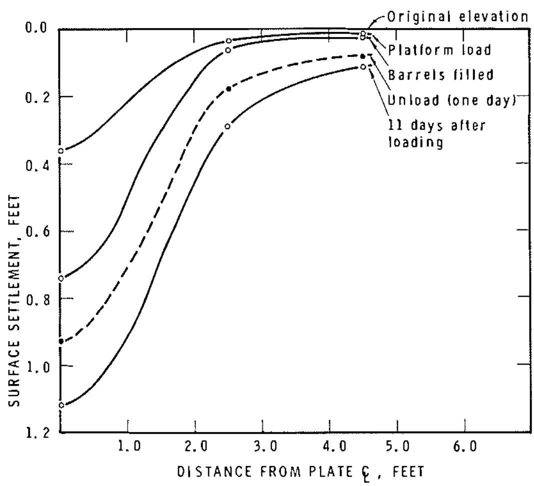

FIG. 7.—SETTLEMENT OF GROUND SURFACE DUE TO APPLIED LOADS TESTS F2

were used to examine the excess pore water pressure that developed throughout the peat. Contours for test F4, based upon these values, are shown in Fig. 10.

Laboratory Tests.—Seven oedometer-type consolidation tests were conducted for comparison with the field tests. The consolidation specimens were 2.8 in. in diameter and about 1 in. in thickness. Typical laboratory results are given in Figs. 11, 12 and 13. Fig. 11 shows a typical settlement-time curve for a peat sample subjected to a load increment corresponding to that of the major load increment applied in the field. Fig. 12 indicates the dissipation of pore water pressure with time under the load increment of Fig. 11. The peat sample for this test was taken with a piston sampler for 3 ft below ground surface in the vicinity of field test F1.

In Fig. 13, the void ratio is plotted against applied vertical pressure for two specimens taken from a block sample secured in the immediate vicinity of test sites F3 and F4. Pore pressures were not measured during these tests. One consolidation test, which will be denoted L1, was conducted using load increments similar to those applied to the muskeg surface in the field;

TABLE 1.—INWARD INCLINATION OF VERTICAL TILT PINS, IN DEGREES

Location From Center Line Of Loading Plate		Time
2-1/2 ft off center (1)	4-1/2 ft off center (2)	(3)
0	0	Platform in place
3	1	Barrels filled
6	2	1 day after loading
7-1/2	2	4 days after loading
8	2	11 days after loading
4	1	1 day after unloading

another test, L2, was carried out using very small load increments, in an attempt to discern if any influence resulted from different rates of loading.

The best straight line fit to the plotted points of test L2 represents a fairly good approximation for the region of the virgin consolidation curve. The slope of this line compares favorably with the slope of a line drawn through the two points of test L1. The effect of sample disturbance is to move these lines to the left; consequently, it is appropriate to draw a straight line envelope to the plotted points of Fig. 13. The slope of this envelope represents the best approximation of the compression index, C_c, as determined by laboratory tests for the peat at sites F3 and F4.

EXAMINATION OF PROGRAM

This investigation was a pilot program designed to study the compression properties of peat by small-scale field loading, and to evaluate instrumentation techniques. The limitations imposed by this approach are: (1) Only a small region of the peat layer was stressed; (2) the assumption of simple undimensional compression, even under the center of the plate, is not strictly true; and (3) the rigidity of the plate presents an inconsistency with respect

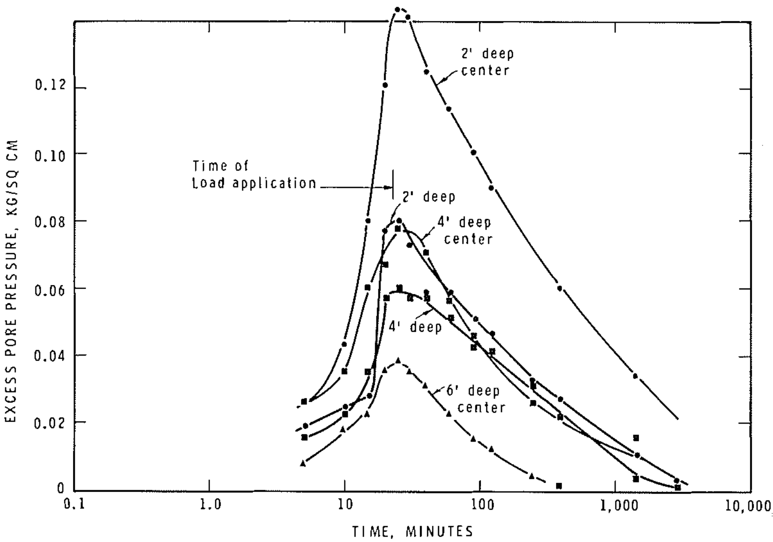

FIG. 8.—PORE PRESSURE DISSIPATION AT POINTS UNDER CENTER AND EDGE OF PLATE DURING FIELD TEST F4

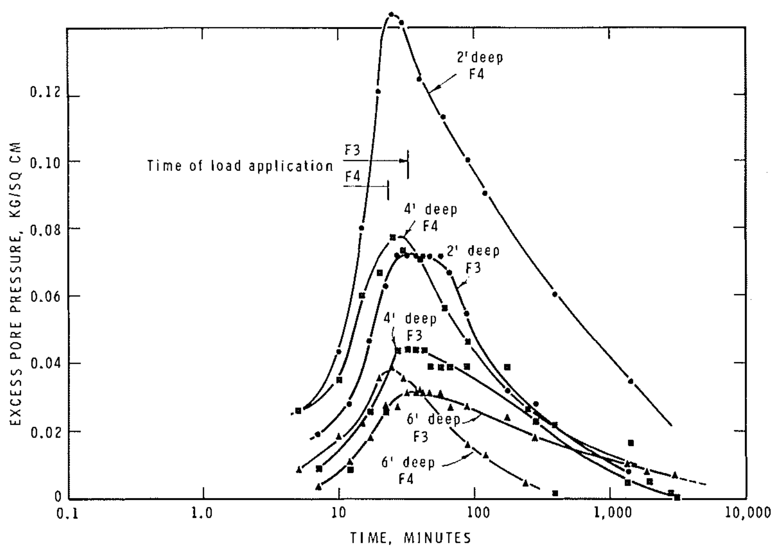

FIG. 9.—PORE PRESSURE DISSIPATION AT POINTS UNDER CENTER OF PLATE DURING FIELD TESTS F3 AND F4

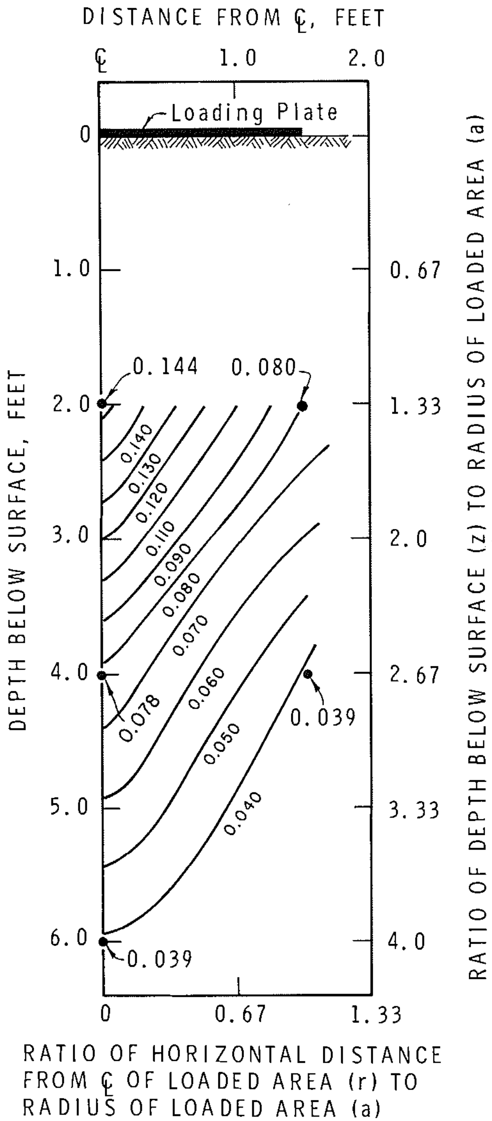

FIG. 10.—CONTOURS OF EXCESS PORE WATER PRESSURE, IN KILOGRAMS PER SQUARE CENTIMETER, DURING TEST F4 (AT COMPLETION OF LOADING)

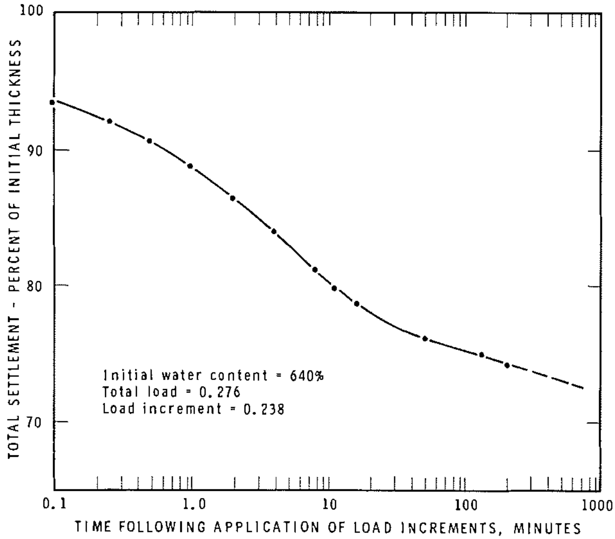

FIG. 11.—TYPICAL SETTLEMENT—LOG TIME CURVE FOR LABORATORY CONSOLIDATION TESTS

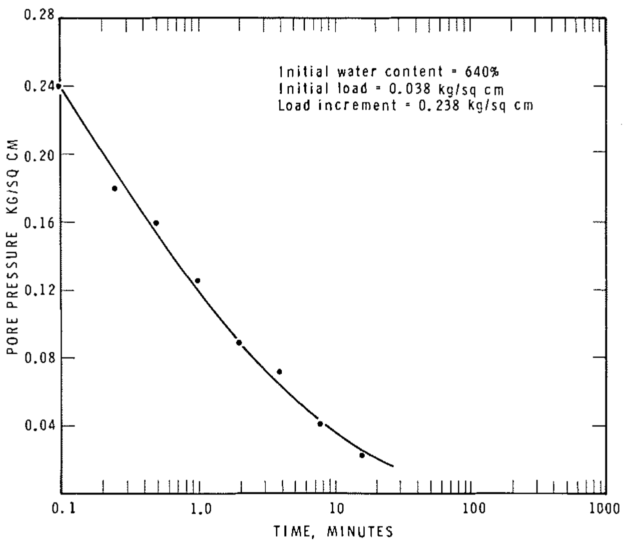

FIG. 12.—PORE PRESSURE DISSIPATION FOR TYPICAL LABORATORY CONSOLIDATION TESTS

to the stress calculations, which were based upon a uniformly distributed surface load.

On the positive side, however, the limited extent of the loaded surface: (1) Permitted a more detailed treatment of the behavior of the peat-matrix in the boundary areas of plate loading; (2) emphasized the effects of field loading rates on pore pressure dissipation; and (3) permitted, because of the fast system response, more information to be secured in a short period of time.

The settlement gages and the pore pressure measuring system appeared to perform quite satisfactorily. The large surface areas of the piezometers (as compared with the negligible volume changes required to obtain readings on the Bourdon gages) resulted in satisfactory gage response. The only difficulty experienced with the settlement gage occurred at the 6-ft depth, at later stages of settlement, when the movements were very small. A slight tendency

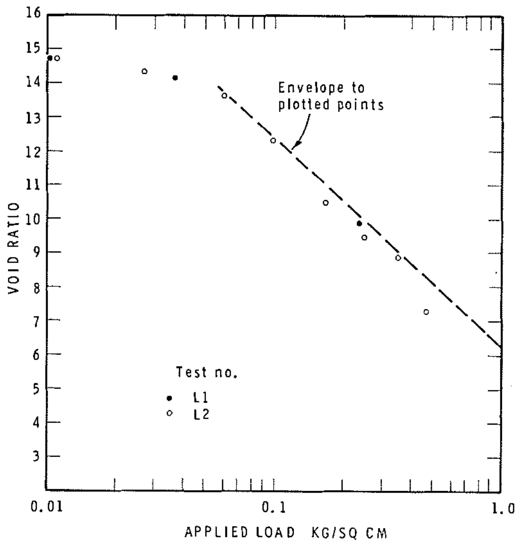

FIG. 13.—VOID RATIO-LOG PRESSURE RELATIONSHIP FOR LABORATORY CONSOLIDATION TESTS

for the deepest reference section to "hang up" was overcome by maintaining a layer of grease between the telescoping gage sections. The rapidity with which the field pore pressures were dissipated, indicated that closer control of the field loading rate would be desirable, as this presents an additional variable.

The combined response of soil layers that have variations in initial moisture content, preconsolidation pressure, and possible other incongruencies are shown in Fig. 6. Although this figure contains errors associated with an elastic approximation of stress, it indicates (roughly) the relation between settlement and load that exists in the field. If one takes a smooth envelope to the plotted points of Fig. 6 (a slope of approximately 24% settlement per log cycle of stress, and if typical values are used, e.g., initial water content of

950%, a specific gravity of solids of 1.40, and a degree of saturation of 95%), the equivalent compression index, C_c, as defined in soil mechanics literature, is found to be approximately 3.5. On the other hand, the laboratory value for the C_c, as determined from the envelope to the plotted points of Fig. 13, is about 6.2. This latter value falls within the range of values determined during an extensive but as yet unreported laboratory program carried out at the Division of Building Research, National Research Council of Canada by the second writer. The laboratory specimens used to produce the curves of Fig. 13 were secured in the immediate vicinity of the field tests which supplied the data for Fig. 6. This would indicate that, for this particular peat, total settlements predicted on the basis of laboratory work might be about double those to be expected in the field. Similar results were reported by Brawner (3).

The time the load is permitted to remain on the soil before deformation readings are taken is obviously a critical factor. The deformations in Figs. 6 and 13 were observed following loading periods of 2 days for the laboratory tests and 2 weeks for the field tests. The total settlements observed during both the field and laboratory tests were about 135% of the settlements observed during the periods of dominant excess pore pressure dissipation.

The plots in Figs. 4 and 5 are somewhat irregular, but not significantly different from the typical laboratory curve in Fig. 11. These plots show similar response characteristics for the peat at different levels below the loaded surface; the only effect of depth is to reduce the magnitudes of response. Neither most of the laboratory curves nor the field plots of Figs. 4 and 5 suggest an obvious division of settlement into primary and secondary phases. The differences in rates of settlement, as denoted in Fig. 5, can be partly explained by differences in applied loading rates; however, the slight variations in character of settlement for the different loads may be explained largely on the basis of the incongruities of the in situ peat material.

If the log-time curve-fitting method (22) were applied to the curves of Fig. 5, times of 100% primary consolidation would be indicated, coinciding roughly with time of field loading completion, which actually represents time of maximum pore pressures. The unloading and reloading cycle carried out in test F1 indicates that rebound of about two-thirds the settlement that occurred under the major load increment is recoverable. (The period of unloading was equal in time to the loading period.) The magnitude of settlement upon reloading approaches very nearly that experienced under the initial loading cycle. Thus, the behavior of the peat has some of the characteristics of delayed elasticity. The significance of this very limited experiment is to suggest that the most efficient use of the preloading technique would minimize the time between removal of the preload and completion of the final structure.

Figs. 8 and 9 show pore pressure dissipation during field testing to be concentrated over a period of about 3,000 min after the commencement of loading. The pore pressures build up during the loading period, reach their maximum value at completion of loading, and then decrease. The response curves of these figures are similar in shape to those reported by Lake (8).

Figs. 8 and 9 indicate that the shapes of the pore pressure and log-time plots are similar throughout the peat depth, their main difference being a decrease in pore pressure magnitude with depth. This agrees with the similarity noted between the settlement and log-time plots for the different peat layers. No significant changes in the slopes of the settlement and log-time plots of Figs. 4 and 5 occur at times corresponding to completion of excess pore pressure dissipation.

The differences between the magnitudes of the pore pressures generated in field tests F3 and F4 appear to be due to the difference in time of load application. The excess pore pressures generated in field test F4 are approximately double those generated during field test F3, and show a much more pronounced "peaking" effect.

The lack of any discernible tendency for pore pressure response from piezometers at depth to lag behind those nearer the surface, would indicate that drainage under the particular conditions of these tests is primarily horizontal. This suggests that the peat at a given time is essentially at the same degree of consolidation throughout its depth. Thus, the fact that the field settlement curves of Figs. 4 and 5 do not lend themselves to choosing an apparent division point between classic primary and secondary phases cannot be explained on the basis of different degrees of consolidation occurring throughout the compressible layer at any given time.

This also indicates how infrequently cases will occur wherein ratios such as those derived from the so-called "square law" can be employed to relate time of consolidation of peat in the field to time in any one-dimensional laboratory test. Incidentally, some writers have indicated that if a relationship of this nature can be established, then the exponent is closer to 1.5 (11). Therein the fact that no retardation of response in pore pressure dissipation was experienced with depth, prevented of course, any scaling between the field results and the laboratory tests completed to date.

The preceding explanation postulates a predominantly horizontal drainage. This probability is strengthened by the observation that the permeability of peat in the horizontal direction is generally much greater than that in the vertical direction (5,15).

Fig. 10 is based on measured pore pressures recorded when load application for test F4 was completed. The contours are necessarily interpolated between points of measured pore pressure and therefore do not exactly represent the actual pore pressures between points of measurement. They do serve, however, as a reasonably good indication of pore pressure distribution.

In Fig. 14 contours of constant vertical stress distribution, beneath a uniformly loaded flexible circular foundation, resting upon an elastic half-space, are plotted. Although this does not precisely fit the field loading situation, the complexities of determining the stresses at some depth under a rigid plate make a more accurate treatment unjustified within the limits of this investigation.

Pore pressure parameter $\overline{B}$ (21), relating increase in pore pressure to increase in applied vertical pressure, was not known for this particular peat at the time of the test project. Assuming a value of 1 for $\overline{B}$ (which would be considered reasonable for normally consolidated clays), the excess pore pressures generated in the peat due to the vertical pressure increase would be expected (before any drainage occurred) to equal the vertical stress at that point. This assumes that the major principal stress and vertical stress coincide. This theoretically occurs only under the center of the plate, but considering this inconsistency, only further emphasize the conclusions drawn. Thus, assuming undrained conditions, the shapes of the contours of constant vertical stress in Fig. 14 would be expected to correspond somewhat to those of the excess pore pressure contours in Fig. 10.

Comparison of the shapes of the contours shown in Fig. 10 with those of Fig. 14 reveals that the contours of the latter have much flatter inclinations. This is possibly a result of faster dissipation of excess pore water pressure occurring near the surface than at depth. As Figs. 8 and 9 indicate similar times of pore pressure dissipation throughout the depth of the peat layer, an explanation, purely on the basis of vertical drainage, of the steepness of the pore pressure contours would appear unjustified.

The calculated vertical stress increases (using Fig. 14) that occur under the loading plate due to loading, when compared with the measured excess pore pressures, show the generated pore pressure to be in excess of the vertical stresses calculated by the elastic theory. This trend becomes more apparent with depth, with the ratio of measured excess pore pressure to calculated vertical stress increase approaching 2, at a depth of 6 ft. This might

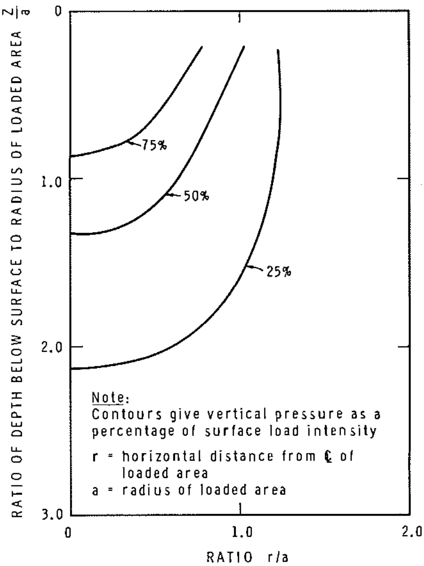

FIG. 14.—CONTOURS OF CONSTANT VERTICAL STRESS IN HOMOGENEOUS ISOTROPIC ELASTIC SEMI-INFINITE MEDIUM WITH UNIFORMLY DISTRIBUTED LOAD OF DIAMETER $2a$ ACTING UPON SURFACE [FROM SCOTT (19)]

indicate that in this material the stress applied at the surface extends to a much greater depth than that indicted by the elastic theory. The effect of the stiffer clay layer at a depth of 10 ft is not considered sufficient to cause such a large discrepancy.

A possible explanation for this might be Terzaghi's local shear hypothesis (23). This term, "local shear" is used to explain such failures as, for example, that caused by excessive settlement directly below a footing, but is not associated with complete rupture of the supporting material. Regions of confined (local) shear occur near the edges of the footing and large compressions occur in the material immediately beneath the footing base. This would result in a situation somewhere between that of unidimensional compression, where the vertical stress extends throughout the depth of the medium, and that of a body of limited extent resting upon an ideally elastic half-space. In the latter case, the stresses decrease rapidly with depth as shown in Fig. 14, whereas in the actual situation, the distribution of stress following that of the elastic analogy is prevented by local shear failure in the vicinity of the plate edges. Another explanation might be that the $\overline{B}$ values for this peat are greater than 1, approaching the value of 2 for the peat at the 6-ft depth. This possibility can be investigated by further laboratory work.

CONCLUSIONS

1. For both the laboratory and the field tests about one-quarter of the total observed settlement took place following dissipation of measured excess pore pressures, but the excess pore pressure phase in the laboratory consisted of a smaller percentage of total test time than in the field. Greater structural reorientation as exhibited by consolidation takes place in the laboratory than in the field during the period following excess pore pressure dissipation. This can be explained partly by sampling disturbance and partly by the differences in the lengths of drainage paths which cause earlier dissipation of pore pressures in the laboratory.

2. The compression index or the slope of the void ratio and log effective pressure curves for laboratory consolidation was about twice as steep as that for the field case.

3. Any possibility of dividing settlements into primary and secondary consolidation phases solely on the basis of standard settlement and log-time plots did not appear feasible.

4. Pore pressure response, as indicated by the build-up and dissipation of excess pore pressures, was relatively independent of depth below ground surface, indicating primarily horizontal drainage.

5. The essentially horizontal drainage that occurred in the field prohibited the use of scaling laws to predict field consolidation from observation of oedometer test behavior, at least for loads of limited extent. The degree of consolidation in the field was relatively uniform throughout the depth of the consolidating stratum, which was apparently due to radial drainage.

6. Pore pressures measured at depth were in excess of the magnitudes of vertical stress calculated using the elastic theory. This means at least one of three things: (a) Yielding of the peat around the perimeter of the plate causes vertical stresses below the plate to be larger than those calculated using the elastic theory, i.e., zones of confined plastic flow made the theory of elasticity inapplicable; (b) stresses applied to the peat result in developed pore pressures greater than the increase in vertical stresses, i.e., $\overline{B} > 1$; or (c) the occurence of large strains, with their nonlinear effects, invalidates the linear theory of elasticity used to calculate stresses in organic soils.

The results of this project provide a basis for the planning of further detailed investigations. The importance of horizontal drainage under field loads of limited extent was shown, as was the importance of rate of field loading on pore pressure generation and dissipation. The departure of the peat—from the point of view of stress analysis—from elastic behavior, suggests the advantages of using field pressure transducers in precise field examinations. Further examination of the consolidation process and even of application of the effective stress concept to highly organic soils is warranted.

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APPENDIX I.—REFERENCES

1. Anderson, K. O., and Hemstock, R. A., "Relating Some Engineering Properties of Muskeg to Some Problems of Field Construction," *Proceedings*, 5th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 61, Ottawa, 1959, pp. 16-25.
2. Brawner, C. O., "The Muskeg Problem in B.C. Highway Construction," *Proceedings*, 4th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 54, Ottawa, 1958, pp. 45-53.
3. Brawner, C. O., "The Principle of Preconsolidation in Highway Construction over Muskeg," *Roads and Engineering Construction*, Vol. 97, No. 9, 1959, pp. 99-100, 102, 104.
4. Buisman, A. S. K., "Results of Long-duration Settlement Tests," *Proceedings*, 1st International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1936, pp. 103-106.
5. Colley, B. E., "Construction of Highways over Peat and Muck Areas," *American Highways*, Vol. 29, No. 1, 1950, pp. 3-6.
6. Goodman, L. J., and Lee, C. N., "Laboratory and Field Data on Engineering Characteristics of Some Peat Soils," *Proceedings*, 8th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 74, Ottawa, 1962, pp. 107-129.
7. Hillis, S. F., and Brawner, C. O., "The Compressibility of Peat with Reference to Major Highway Construction in British Columbia," *Proceedings*, 7th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 71, Ottawa, 1961, pp. 204-227.
8. Lake, J. R., "Pore Pressure and Settlement Measurements During Small-Scale and Laboratory Experiments to Determine the Effectiveness of Vertical Sand Drains in Peat," *Proceedings*, Conference on Pore Pressure and Suction in Soils, Butterworths, London, 1960, pp. 103-107.
9. Lake, J. R., "Investigations of the Problem of Constructing Roads on Peat in Scotland," *Proceedings*, 7th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 71, Ottawa, 1961, pp. 133-168.
10. Lea, N. D., "Notes on the Mechanical Properties of Peat," *Proceedings* 4th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 54, Ottawa, 1958, pp. 53-57.
11. Lea, N. D., and Brawner, C. O., "Highway Design and Construction over Peat Deposits in the Lower Mainland Region of British Columbia," *Research Record No. 7*, Highway Research Board, Washington, D.C., 1963, pp. 406-24.
12. MacFarlane, I. C., "Guide to a Field Description of Muskeg," ACSSM Technical Memo 44 (revised), National Research Council of Canada, Ottawa, 1958.
13. MacFarlane, I. C., "The Consolidation of Peat: A Literature Review," *Division of Building Research Technical Paper No. 195*, National Research Council of Canada, Division of Building Research, Ottawa, 1965. (National Research Council of Canada 8393).
14. Mickleborough, B. W., "Embankment Construction in Muskeg at Prince Albert," *Proceedings*, 7th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 71, Ottawa 1961, pp. 164-185.

15. Miyakawa, I., "Some Aspects of Road Construction in Peaty or Marshy Areas in Hokkaido, with Particular Reference to Filling Methods," Civil Engineering Research Institute, Hokkaido Development Bureau, Sapporo, Japan, 1960.
16. Ringeling, J. C. N., "Measuring Groundwater Pressures in a Layer of Peat Caused by an Imposed Load," *Proceedings*, 1st International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Mass., Vol. 1, 1936, pp. 106–111.
17. Ripley, C. F., and Leonoff, C. E., "Embankment Settlement Behavior on Deep Peat," *Proceedings*, 7th Muskeg Research Conference, National Research Council of Canada, ACSSM Technical Memo 71, Ottawa, 1961, pp. 185–204.
18. Root, A. W., "California Experience in Construction of Highways Across Marsh Deposits," *Bulletin 173*, Highway Research Board, Washington, D.C., 1958, pp. 46–66.
19. Scott, R. F., *Principles of Soil Mechanics*, Addison-Wesley Publishing Co., Reading, Mass., 1963.
20. Shea, P. H., "Unusual Foundation Conditions in the Everglades," *Transactions*, ASCE, Vol. 120, 1955, pp. 92–102.
21. Skempton, A. W., "The Pore Pressure Coefficients A and B," *Geotechnique*, Vol. 4, No. 4, 1954, pp. 143–147.
22. Taylor, D. W., *Fundamentals of Soil Mechanics*, John Wiley and Sons, Inc., New York, 1948.
23. Terzaghi, K., and Peck, R. B., *Soil Mechanics in Engineering Practice*, John Wiley and Sons Inc., New York, 1948.
24. Ward, W. H., "A Slip in a Flood Defence Band Constructed on a Peat Bog," *Proceedings*, 2nd International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 2, 1948, pp. 19–23.

APPENDIX II.—NOTATION

The following symbols are used in this paper:

a = radius of loaded area;
$\overline{B}$ = pore pressure coefficient;
C_c = compression index;
c_v = coefficient of consolidation;
k = coefficient of permeability;
r = horizontal distance from center line of loaded area; and
Z = depth below surface of loaded area.

6652 STUDIES OF RESPONSE OF PEAT TO PLATE LOADING

KEY WORDS: consolidation; consolidation coefficient; elastic deformation; elasticity; elastic theory; field data; field tests; loading rate; organic soils; peat; permeability; plate load tests; pore water pressures; primary consolidation; secondary compression; settlement; soil mechanics; time settlement relationship

ABSTRACT: This paper presents the results of a series of small-scale field tests carried out to investigate the in situ deformational characteristics of a soft peat, particularly in the vicinity of a load discontinuity. A 3-ft diam plate was used to apply loads to the surface of a 10-ft thick stratum of nonwoody fine fibrous peat with a moisture content of about 950%. Rates and magnitudes of settlement, and pore pressures at five positions were measured and compared. Field results were supplemented by data obtained from a limited number of oedometer-type laboratory consolidation tests, conducted with and without pore pressure measurements. The geometry of the settlement-logarithmic time plots is described. Emphasis is placed on the prevalence of horizontal drainage and its significance. The departure of soft organic materials from the predicted behavior as based upon the linear theory of elasticity is described, and possible reasons for lack of agreement are presented.

REFERENCE: Forrest, J. B., and MacFarlane, I. C., "Field Studies of Response of Peat to Plate Loading," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM4, Proc. Paper 6652, July, 1969, pp. 949-967.