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Forrest, J. B.

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FIELD STUDIES OF THE CONSOLIDATION RESPONSE OF PEAT

by

J. B. Forrest

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PREFACE

This report is concerned with a field program investigating the load-deformation characteristics of peat. The site of the research project was the Mer Bleue Peat Bog near Ottawa, about seven miles from the Building Research Centre.

The author of this report is Professor J.B. Forrest of the Faculty of Engineering, Carleton University, Ottawa, who spent the summer vacation of 1966 as a guest worker with the Division.

The study is part of a continuing investigation into the engineering characteristics of peat or "muskeg", being pursued by the Soil Mechanics Section, Division of Building Research, under the direction of I.C. MacFarlane.

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June 1967

R.F. Legget
Director

TABLE OF CONTENTS

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J. B. Forrest

The consolidation characteristics of peat have generally been identified within the framework of current theories applied to inorganic compressible soils. Consolidation of clays is often arbitrarily separated into a primary consolidation phase, as originally developed by Terzaghi (Taylor, 1948), and a secondary consolidation, or creep, phase.

In the first or primary stage, the rate of volume change is considered to be largely controlled by the permeability of the soil or the resistance offered to the outflow of pore water. Following the dissipation of excess pore pressure, further volumetric deformation is considered to be a function of the resistance of the soil structure to plastic deformations or to structural reorientation of the particles.

Procedures for estimating settlement by breaking down strains into volumetric and deviatoric or shear components have been summarized by Lambe (1964); these methods are based, however, upon extensive laboratory work and, as such, will not be used in this report. Some writers (Evgen'ev, 1961; Adams, 1964), in considering consolidation of peat, believe that secondary compression (that consolidation taking place under conditions of very small pore pressures) involves the expulsion of free water from the solid matter constituting the peat structure. This would, in effect, be a primary consolidation theory applied at the sub-macroscopic level to individual "solid" particles making up the peat.

This explanation does not suggest a clearly defined point at which the two different phases of consolidation can be separated. Wilson and Lo (1966) point out that, since the degree of consolidation in a consolidating soil layer is very much a function of position with respect to the drained boundaries, the arbitrary division of consolidation into a primary and secondary range is impossible unless one considers only very limited regions of the stressed material. Experimental work on peat generally shows no distinct change in the consolidation behaviour occurring at the end of the period of excess pore water pressures. By plotting different functions of settlement versus functions of time, regions characterized by apparent differences in consolidation behaviour may be noted. An example of this type of plot is given by Wilson et al. (1965) where the log of rate of consolidation is plotted versus the log of time. The time of apparent change in consolidation characteristics in this type of plot does not, however, coincide with the end of the period of excess pore water pressures.

In this paper any reference to the secondary consolidation will imply the settlement taking place at a point following dissipation of measurable excess pore water pressure at that point. Many writers (e.g. Hamilton and Crawford, 1959; Taylor, 1942) agree that the consolidation characteristics of soils, as determined by experimental techniques in the laboratory, are functions of variations in the testing procedures. Leonards and Girault (1961) emphasize the importance of load increment ratio, and suggest that any field predictions of settlement should be based upon analogous load ratios in the field. Leonards and Girault noted that different ranges of the applied load increment, particularly with increments in the vicinity of the preconsolidation load, result in distinctly different settlement-log time curves.

As an example of the effects of different consolidation loads on the interpretation of the classic consolidation theory, calculation of the Terzaghi coefficients of consolidation has been carried out using values of vertical permeability reported by MacFarlane (1965b). The coefficient of consolidation in logarithmic scale is plotted versus consolidation pressure in Figure 1. It may be observed from this figure that for applied loads above the order of 1.5 kilograms per square centimeter the coefficient of consolidation remained relatively constant, as is often the case with other compressible soils, notably some inorganic clays. For small loads such as those to which the peat may be subjected under actual field conditions, however, the coefficient of consolidation varies drastically with change in consolidation pressure.

For this reason, in a laboratory investigation conducted in conjunction with a field investigation, an attempt should be made to maintain applied loading conditions similar to those of the field.

This report is primarily concerned with a limited field investigation of the consolidation characteristics of peat in situ. Due to the time element and to the limited physical resources available at the time of the investigation, it was not expected that an exhaustive treatment of the problem or any far-reaching conclusions could be provided. The investigation was necessarily of a pilot nature, designed primarily to test the feasibility of the approach.

The rates and magnitudes of settlement occurring in a peat layer subjected to a circular plate loading were observed at the surface and at two depths below the centre of the plate. Pore water pressure measurements were recorded at three elevations below the centre of the plate and at two elevations beneath a point on the circumference of the plate.

The results of several consolidation tests conducted on samples of peat secured from the test area are also presented herein as a basis for comparison between field and laboratory behaviour.

SITE

Terrain

The sites chosen for the field investigations were in a "confined" muskeg area of approximately 8 square miles, about 9.5 miles east of Ottawa. This area is known as the "Mer Bleue Peat Bog," and is described by MacFarlane (1964). The actual test sites were just north of the end of the Dolman Ridge Road, the northernmost of two relatively narrow ridges extending eastward into the peat bog from its western extremity (Figure 2 (a, b)).

The general muskeg classification here is EI - FI, according to the Radforth System (MacFarlane, 1958). At the test sites, the muskeg has a depth of about 10 feet and overlies a horizontal layer of blue clay, which appears to have been deposited in the depressions of a former river channel. The elevation, based on M.S.L., of the edge of the Mer Bleue is about 225 feet. It is a so-called "highmoor" type of bog, the centre being slightly higher than the edges.

The actual test sites are characterized by low woody shrubs (up to 2 feet high) intermixed with patches of hummocky mosses and short grasses. The sites selected for placing the loading apparatus are shown, approximately, in Figure 2b and were all on grassy areas.

Material

The plate loading tests were conducted on grassy FI muskeg. Many of the characteristics of the subsurface peat are reported by MacFarlane (1965b). The specific gravity of solids varies within the range of 1.40 to 1.44 grams per cubic centimeter, with an average of about 1.41. Organic contents are in the range 87 to 93 per cent. Natural water content in this area varied from about 600 to 1200 per cent. Water content is the ratio, given as a percentage, of the weight of water driven off by drying the peat samples for two days at 105°C to the dry weight of material remaining.

Plots of water content versus depth below the ground surface are shown in Figures 3a and b.

SAMPLING PROCEDURES

The letters L and F are used to refer to laboratory and field tests, respectively. Samples for laboratory consolidation tests were obtained by two means. A 2.8-inch diameter thin-wall piston sampler was used to secure specimens for tests L1 to L4. These samples were secured from depths of 2 to 3 feet in the vicinity of field sites F1 and F2 (Figure 2b). A $1\frac{1}{2}$-cubic foot block sample, obtained from a depth of about 2 feet in the vicinity of field tests F3 and F4, was used to provide specimens for laboratory consolidation tests L6 and L7.

A bucket auger was used to secure samples for water content determinations. Water contents were also run on specimens secured from both the piston samples and the undisturbed block sample.

PROCEDURES

Laboratory Tests

Several standard consolidation tests, with and without pore pressure measurement, were conducted in the laboratory.

Pore pressures were recorded in tests L1 through L4 and were measured at the base of the sample while drainage was permitted at the top. No pore pressure measurements were made for tests L5 - L7.

Two methods of measuring pore water pressure were used. The first method utilized a null-indicator, as explained by Bishop and Henkel (1962), to prevent outflow of water from the base of the sample by balancing the pore pressure against a mercury manometer. This method was used for tests L1, L2, and L4. The second method was utilized in Test L3, where an electronic pressure transducer was connected to a pen recorder to give a continuous recording of pore pressure. For tests L1 to L6 the increments of loading were 0.037, 0.25 and 0.50 kilograms per square centimeter. The first two increments correspond to the two values of average contact stress experienced by the muskeg surface in the field.

For consolidation test L7, the specimen was consolidated using significantly smaller load increments, with the total loading range overlapping that of tests L1-L4 and L6. This test was conducted in order to obtain some indication of the behaviour of the peat under intermediate load increments.

In all tests, each increment of loading was allowed to remain on the specimen for at least 24 hours. In Test L4 the load increment corresponding to the maximum surface load increment in the field was allowed to remain on for several weeks. The specimens tested were about 2.8 inches in diameter and from 0.785 to 1.0 inch in thickness.

A triaxial consolidation test was conducted on a sample 2.8 inches in diameter and 5.6 inches in length. This test attempted, by use of the proper values of confining or cell pressure, to prevent lateral strain of the sample during consolidation under a vertical load. Although time did not permit this approach to be developed further, it suggests worthwhile possibilities. It is the aim of the author to initiate the carrying out of one-dimensional (Ko) triaxial tests as well as regular consolidated undrained tests to provide ultimately for a comparison between actual field settlements and those predicted by methods proposed by Skempton and Bjerrum (1957) and Lambe (1964).

Field Tests

(a) Loading

Some method of applying load to the muskeg surface was required, but due to conservation considerations and to the poor trafficability of the terrain the methods available for this purpose were severely limited. For these reasons only small-scale field tests to determine compressibility were considered practical at this time. In view of its accessability, portability and disposability, water was used for loading, but due to the great quantities of water required the loading area had to be very small. Even with the small loading area used in these tests, the stresses applied to cause consolidation are considered to be below desirable levels. The small loading area had two additional disadvantages:

(i) applied stress could be expected to decrease fairly rapidly with depth, with the result that a relatively small region of the muskeg experienced a significant stress increase, and;

(ii) the complexities of finishing with a two-dimensional problem as opposed to a simpler one-dimensional one increase the interpretation difficulties.

A 36-inch diameter wooden plate was used to transmit the applied load to the muskeg surface (Figure 4(a, b)). The loading platform consisted of three tiers of 4- by 4-inch timbers (the first two layers consisting of timbers about 4 feet in length) that rested directly upon the loading plate and supported a 3/4-inch thick plywood platform. The plywood platform was used to support the water containers, in this case eight 45-gallon drums. The shorter lengths of 4- by 4-inch timbers were used as fillers to raise the top row of longer timbers directly under the plywood platform and to keep them from coming into contact with the muskeg surface after some settlement had occurred. Even with this precaution the longer 4 by 4's did come into contact with the top of the moss mat, but this caused a negligible reduction of the stresses applied through the plate.

Two loading increments were applied to the plate. The first was caused by the weight of the platform and barrels and increased the stress on the muskeg surface by an average value of 0.037 kg/cm^2 (76 psf). After about three days, when settlement under the platform load had virtually ceased, the second load increment was applied by pumping the barrels full of water. This second load increment increased the average contact pressure under the plate by about 0.254 kg/cm^2 (520 psf) to a total load of approximately 0.291 kg/cm^2 (596 psf).

For the different loading tests water had to be pumped from different sources; this caused loading time for the second increment to vary from 20 minutes to 1 hour, and added an additional unknown factor to the test procedure: i.e. the rate of pore pressure dissipation occurring during actual load application was variable. The eight non-interconnected barrels used for loading required careful observation of the platform during the loading procedure to prevent tilting. Despite extreme care some tilting did occur, with possible kneading of the underlying peat strata as a result.

For the first two tests the loading plate was placed directly on the upper moss mat. As upward drainage was possible through a hole in the middle of the plate (which permitted installation of the settlement gauges) and because of the high compressibility of the moss, it was decided that more realistic results could be obtained by removing the living moss mat directly beneath the plate and by placing a very thin blanket of fine sand. This procedure was followed for tests F3 and F4.

(b) Deformation Measurements

Settlement was measured at four elevations: at the ground surface and at depths of approximately 2, 4, and 6 feet directly below the centre of the plate. This measurement of settlements at depth was achieved by using three telescoping pipes with helical plates welded at their ends (Figure 4a).

During test F1, there was some indication that the rod extending to the deepest reference elevation was binding. In subsequent tests the shafts of the settlement gauges were greased throughout their lengths to minimize soil friction along the sides of the pipes and any friction between them.

Elevations were read periodically at the tops of the settlement rods using a precise level. Elevations were also taken on the loading platform. Large diameter head steel pins 4 inches long were placed flush with the top of the mat in the vicinity of the loading plate. Elevations were taken periodically on the tops of these pins. In addition, regular observations were made during the load period of any tilting of several 3-foot long steel "tilt" pins inserted 2 feet into the mat at distances of 4 and 6 feet from the centre of the loading plate (see Figures 4b and c).

(c) Pore Pressure Measurements

Pore pressures were measured at various locations beneath the plate, both during and after the load applications. "Geonor" piezometers (Figure 5), shortened to one-third their initial effective lengths, were pushed into the muskeg from outside the periphery of the plate. These piezometers were connected by $\frac{1}{4}$-inch nylon tubing to Bourdon gauges at the muskeg surface, thus providing an essentially constant volume system. The piezometers were modified because with a relatively confined stress area the full-length "Geonor" piezometers would have too much of an averaging effect upon the recorded pore pressures.

A second problem in the piezometer installation was the presence of the "E" rods used to advance the piezometers into the soil. These rods, if left connected to the piezometers, would provide a system of reinforcing to the peat foundation and would, therefore, severely modify the stress conditions. Removing the "E" rods not only made retrieval of the piezometers difficult, but introduced the danger that this would provide a seepage path along the plastic tubing. This would accelerate dissipation of pore pressures and impair the significance of observed pore pressure response.

To check the effect of removing the "E" rods, preliminary loading tests (F1 and F2) were conducted, using two piezometers symmetrically placed below the loading plate, but with the "E" rod removed from one of them. In these trial cases similar values of pore pressure were recorded and the response times for the gauge readings were in complete accord. On the basis of these tests, the "E" rods were removed from the piezometers during field tests F3 and F4.

The problem of retrieving the piezometers after loading was solved by attaching a flexible wire to the end of the piezometer before installation. This wire was used to pull the piezometer out of the soil following completion of testing. The tubes connecting the piezometers to the Bourdon gauges were kept completely buried in the peat and led to a waterproofed wooden box that housed the Bourdon gauges (Figure 6).

The box enabled the gauges to be situated below the water table, even during the dry summer period. This prevented them from being subjected to negative gauge pressures that might have brought any dissolved air in the pressure system out of solution. An additional benefit of placing the gauges and gauge lines well below the ground surface was the insulation provided, which would minimize temperature effects. To improve the accuracy of the readings the Bourdon gauges were calibrated over the appropriate range by immersing the piezometers in a container of water to which measured hydrostatic heads were applied.

Piezometer readings were observed only during and following the application of the major load increment. The initial platform load, due to its very small stress increases, was not sufficient to generate measurable pore pressures.

OBSERVATIONS AND RESULTS

Laboratory Tests

Plots of laboratory consolidation, in terms of the percentage of the original volume versus logarithm of pressure, are shown in Figure 7 for consolidation tests L1 to L4, L6 and L7. Void ratio versus logarithm of pressure for these tests is shown in Figure 8. The difference between Figures 7 and 8 with regard to the relative positioning of the various test plots is due to the fact that the samples were at different initial void ratios. From Figure 7 it may be noted that the slope of the settlement-log pressure curve, as determined from consolidation test L7 using small load increments, is a function of pressure. The slopes, as determined from the consolidation tests performed using load increments corresponding to those in the field, are noted to be in some accord with the average slope for the semi-log plot for consolidation test L7. This is particularly so for test L6, which was conducted on a sample obtained from almost the same location as that of L7. Samples for tests L1 to L4 were obtained using a thin-wall piston sampler at field sites 1 and 2 (Figure 2b), whereas samples L6 and L7 were taken from the block sample adjacent to field sites 3 and 4.

In Figure 8 agreement between consolidation tests L6 and L7 on specimens from the same block sample is reasonably good. Specimens 1 to 4, however, have considerably lower void ratios initially (possibly due to sample disturbance associated with the thin-wall sampler) and, therefore, the void ratio-log pressure curves are much flatter in this case.

Settlement versus logarithm of time following application of the major load increment is plotted for consolidation specimens 1 to 4 in Figure 9. Samples 1 and 2, tested by means of a compressed air loading device, experienced some variations in applied loading and for this reason long-term settlements are not plotted for these tests.

Consolidation test L4 was continued for more than 60,000 minutes. A plot of settlement versus logarithm of time is shown in Figure 10. As this test was conducted during mid-summer without provision for environmental control, irregularities from a smooth curve may be partially due to variations in temperature and humidity.

From Figure 9 it may be seen that although the slopes of the settlement-log time plots decrease noticeably during the first 10 to 15 minutes of loading, they show no tendency to decrease further at longer times. The rate of settlement, as would be observed on a settlement-log time plot, is continually decreasing with time. Consolidation test L3 experienced an accidental preloading of an unknown fraction of the load increment. It is probably because of this that its settlement plot, shown in Figure 9, has a somewhat different shape from that of tests L1, L2, and L4.

Figure 11 shows pore water pressures plotted against log time for consolidation tests L1 to L4. All tests indicated that the excess pore water pressures were largely dissipated within 10 minutes.

Field Tests

(a) General

A comparison of water contents of peat samples secured from similar elevations in adjacent holes is shown in Figure 3b. It may be seen that the piston samples have water contents consistently higher than those determined from the bucket auger.

Figure 3a illustrates a plot of water content versus depth based on samples obtained with the bucket auger in the immediate vicinity of test sites 3 and 4. Also on Figure 3a is shown the average of water contents determined from the block sample. This indicates that the bucket auger causes some squeezing out of the pore water during sampling, resulting in water content values, w, about 50 units lower than those of the soil *in situ* (i. e. w = 930 per cent versus w = 980 per cent for the block sample). On the other hand, from Figure 3b, the specimens from the thin-wall piston sampler appear to give water contents higher than those of the bucket auger. On the basis of this very scant information the piston samples might be expected to show water contents ranging from values of the *in situ* material to values about 20 per cent too high.

This unusual phenomenon can be explained as follows. In obtaining a peat sample, using a thin-wall tube, the fibres tend to resist penetration of the cutting edge. This compresses the peat immediately below the cutting face, whereas rebound of the peat occurs inside the sampler where the fibres have been severed. The inflow of ground water into the tube due to the excess pressure generated by pushing the tube into the soil provides a ready supply of moisture for sample expansion within the tube.

(b) Deformations

Total settlements for field tests F1 to F4 versus time, following the commencement of major load application, have been plotted in Figures 12 to 15, respectively. Figure 12 (Test F1) shows settlement at three elevations: surface, 2-foot depth, and 4-foot depth. It includes both a loading phase and a reloading phase, following a period of two days during which the major load increment was removed to permit rebound. For some reason the 6-foot deep reference point did not settle; consequently no plots are shown for this point. The basic difference between tests F1 and F2, as compared with tests F3 and F4, was that in the latter two loading was applied directly to the peat through a sand cushion; whereas field tests F1 and F2 were conducted on the moss mat, which was initially about 10 inches thick.

Some irregularities occur in these curves, particularly in the settlement of the deeper reference points. In these cases the settlement readings seem to alternate, particularly at the later time stages. This would appear to be due to the fact that the settlements are less than the errors in levelling. In order to minimize levelling errors in the consolidation results, deformations in the peat at a given time were calculated by subtracting the elevations determined for the different reference points at that time. This avoided compounding any errors made in calculating the height of instrument during level surveys. A plot of settlement, as a percentage of the original layer thickness, versus time, following commencement of final increment loading, is shown in Figure 16.

The average values of vertical stress for each of the consolidating layers of tests F3 and F4 were approximated using the theory for a uniformly distributed load on the surface of an elastic half-space. The average stress in each layer was assumed to be that calculated at the mid-depth of the soil layer directly below the centre of the circular loading plate.

Obviously the plate used in the field tests reported herein behaves rigidly, but since the points where the stresses are of interest are somewhat below the plate surface, the error involved should not be too great.

The calculated values of stress in tests F3 and F4 are plotted in Figure 17 versus deformations of the three layers between the reference points. This is done for both the initial and major load increments. Figure 17 results from a grouping of the strain responses for different soil layers at sites 3 and 4.

Although Figure 17 combines the response of soil layers having variations in initial moisture content, preconsolidation pressure, etc., it serves as a very rough indication of the settlement-pressure relation existing in the field. Figure 18 consists of plots of surface elevation at different times versus distance from the centreline of the loading plate for test F2.

Table I shows the change in inclination of the two steel tilt pins placed 1 foot and 3 feet from the edge of the plate. From Figure 18 it may be seen that the surface of the muskeg in the vicinity of the plate depresses under the effect of the plate loading, with this effect decreasing outward from the plate. This behaviour is further demonstrated by the behaviour of the pins (Table I), which lean inward during loading but show a tendency to straighten towards their original position during rebound.

(c) Pore Pressures

Pore pressures for field tests F3 and F4 are shown in Figures 19 to 22 for piezometers placed approximately 2, 4, and 6 feet beneath the centre of the loading plate, and 2 and 4 feet below the edge of the loading plate.

The pore pressure readings determined during field tests F1 and F2 were used primarily to check the performance of the piezometers. As such, they were not placed in positions designed to give the most useful soil response data; consequently, these results are not reported. Also, the drainage conditions of tests F1 and F2 are even less well defined than those of tests 3 and 4, for which the plate was placed upon a very thin layer of fine sand over raw peat. As all piezometer readings reached their maximum values at the same time, i. e. immediately following the completion of loading, piezometer readings at this time are used in order to get a picture of developed excess pore water pressure throughout the peat. Contours based upon these values are shown in Figures 23 and 24. Loading times are shown on Figures 19 to 22. It is noted that for field test F4 a shorter loading period than for test F3 resulted in higher pore pressures.

PERFORMANCE OF FIELD EQUIPMENT

The field equipment appeared to perform reasonably well, despite the relatively small stresses that could be applied in the tests. The large surface area of the piezometers, as compared with the small volume changes required to exert pressures on the gauges, resulted in no observable lags in gauge response. Pore pressures tended to start building up immediately upon commencement of pumping, and started to taper off after loading was completed. Maximum pore pressures were recorded within a couple of minutes of completion of loading.

Although the pressure measuring system appeared to respond satisfactorily, difficulty was experienced in reading the Bourdon gauges accurately, because the maximum values of the recorded pore pressure were only a few pounds per square inch, whereas the Bourdon gauges had pounds per square inch as their finest division. Difficulties were experienced, particularly in the readings of settlement at the 6-foot level. Movements were so slight following the loading period that settlements were of the same order as the tolerances in reading the level.

DISCUSSION AND CONCLUSIONS

Using laboratory loading increments equivalent to the average contact stresses to be applied in the field results in only two points for most of the load-settlement plots of Figures 7 and 8. This loading procedure is questionable, however, since in the field only the surface of the muskeg layer experiences loads of this magnitude. Figure 8 shows some differences in the slopes of the void ratio-log pressure relation, but these are primarily due to differences in initial void ratios. The points for consolidation tests L6 and L7, taken from the same block sample, show relatively good agreement despite the different load increment used. The convergence of the different test plots with increasing load would indicate that differences in sample history (degree of disturbance, etc.) are obliterated by the greater loads. Such factors as the component of stress carried by side friction in the consolidometer obviously exert some influence on the plots of Figures 7 and 8, but this is not considered to be significant and is ignored in this report.

The use of the envelope to the plotted points in Figure 8 as a reasonable value of the compression index C_c, to be used in predicting total settlement for field tests F1 and F4, is justified by the relative agreement in the e-log P plots for consolidation tests L6 and L7, the samples of which were from a block sample taken from the immediate vicinity of field sites 3 and 4. The location of e-log pressure points representing consolidation tests L1 to L4 well below this envelope may be due to both the lower initial water contents and disturbance associated with the piston sampler. Using the envelope of Figure 8, therefore, a C_c value of about 6.2 is obtained that is within the range of values reported by MacFarlane (1965).

Figure 17 shows plots of deformation of the individual soil layers located between the points of observed settlement in the field. Settlement values are plotted against values of the average vertical stress as calculated using the theory of elasticity. A plot of the elastic vertical stress distribution at various positions under a flexible 3-foot diameter plate is shown in Figure 25. The stresses plotted in Figure 17 were those at the mid-depth of the individual layers, directly below the centre of the plate.

Disadvantages of this approach include such factors as:

(a) The plate remains rigid, thus preventing uniform load distribution as assumed for the calculations.

(b) The peat does not exhibit purely elastic or even delayed elastic characteristics.

(c) The real situation is actually a layered system, with a stiffer stratum (clay) at the 10-foot depth.

In spite of these shortcomings, the complexities of a more realistic treatment of the above stresses cannot be justified at this stage. In addition, the geometry of the situation may be expected to minimize the effects of points (a) and (c), whereas point (b), to some degree at least, must always be a problem. Taking a straight-line envelope to the plotted points of Figure 17 and using an initial water content of 950 per cent, a specific gravity of solids of 1.40 and a degree of saturation of 95 per cent, the equivalent C_c is found to be 3.51 or approximately one half of that determined from the laboratory consolidation tests. As pointed out previously, the plots of Figure 17 represent different layers and, therefore, materials subjected to different preconsolidation pressures. As the buoyant weight of the peat in situ is only about one pound per cubic foot, however, differences in preconsolidation pressure are considered negligible, at least below the level of seasonal moisture variations. The similarity in water contents at various depths noted in Figure 3a and b supports this. On the basis of this investigation, therefore, it may be expected that prediction of field settlements based upon laboratory results might be double those actually occurring in the field.

Obviously the time the load is permitted to remain on the soil prior to taking deformation readings is a critical factor. The deformations in Figures 8 and 17 were those taken following loading periods of two days and two weeks for the laboratory test and the field tests, respectively. The total settlements observed during both the field and laboratory tests were about 135 per cent of the settlements observed during the periods of dominant excess pore pressure dissipation.

The plots of laboratory settlement versus log time in Figure 9 show a trend towards slight S-shaped curvature over the initial 100 to 200 minutes. The irregular shapes of these plots make it rather difficult to attempt to separate settlement into a primary and a secondary consolidation phase merely on the basis of their shapes.

From Figure 11 it may be observed that pore pressure dissipation during the laboratory consolidation tests is largely completed within 10 to 20 minutes of application of the load increment. No notable changes in the rates of settlement observed in Figure 9 occur at times corresponding to completion of excess pore pressure dissipation in Figure 11. This lack of agreement between changes of slope of the settlement-log time plot for peat and effective pore pressure dissipation has been noted by other researchers (Lo, 1964).

Figures 12 to 16, illustrating field observations, show rather irregular settlement plots not unlike those of Figure 9 for laboratory consolidation. The data in Figures 12 to 15 are plotted in a slightly different form in Figure 16. The time-settlement plots for the peat at different levels below the loaded area show similar response characteristics. The effect of depth is to reduce the magnitude of the settlement. As with the lab tests, the plots of field settlement do not suggest, by their shapes, an obvious division of settlement into primary and secondary phases.

Figures 19 to 22 show pore pressure dissipation during field testing to be concentrated over a period of about 3,000 minutes following commencement of loading. The pore pressures show a build-up during the loading period, reach their maximum value at completion of loading, and then decrease. The response curves of Figures 19 to 22 are very similar to those reported by Lake (1960).

Figures 19 to 22 indicate that the shapes of the pore pressure-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-log time plots for the different peat layers. Also, no significant changes in the slopes of the settlement-log time plots of Figures 12 to 16 occur at times corresponding to completion of excess pore pressure dissipation. Nevertheless, it may be observed by comparing Figures 9 and 10 with Figure 16 that the rates of consolidation occurring in the laboratory following completion of the excess pore pressure phase are greater than those observed at corresponding times in the field. There are three possible causes:

(1) Structural disturbance in the laboratory specimens, due to sampling, results in larger structural readjustments than occur due to loading the peat in situ.

(2) The higher rates of applied loading in the laboratory, as compared with those in the field, result in greater disturbance and therefore greater structural readjustments.

(3) The shorter drainage paths in the laboratory result in the excess pore pressure phase being completed at an earlier stage in the period of structural readjustment that follows the loading of peat.

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 exaggerated flattening of the peaks of excess pressure in field test No. F3 was probably caused by sticking of the Bourdon gauges. In test F4 each gauge was tapped sharply prior to reading. Observations of the behaviour of this type of gauge indicated that the influence of any "hanging up" of the gauge in test F3 would cause a delay in the response only during the period of decreasing pore pressures, i.e. the maximum recorded pore pressure would not be affected.

The lack of any tendency for pore pressure readings 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 Figures 12 to 16 do not lend themselves to choosing an apparent division point between classic secondary and primary phases cannot be explained on the basis of different degrees of consolidation occurring throughout the compressible layer at any given time.

Comparing the times of pore pressure dissipation in the field with those determined in the one-dimensional consolidation tests in the laboratory indicates the unworkability, in this case, of the so-called square law used to relate time of consolidation in the field to time in the laboratory by means of the ratios of the squares of the lengths of drainage paths. Actually, some workers have indicated that if a relation of this nature can be established, then the exponent is more of the order of 1.5 (Lea and Brawner, 1963). This might agree more closely with the observations in this study, but the fact that no retardation of response is experienced in pore pressure dissipation with depth invalidates any "scaling laws."

The preceding discussion postulates a predominantly horizontal drainage. This postulation is strengthened when one considers that the permeability of peat in the horizontal direction is generally much greater than that in the vertical direction (Colley, 1950; Miyakawa, 1960).

Figures 23 and 24 are based on measured pore pressures recorded at the time of completion of load application. 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 serve as a reasonably good indication of pore pressure distribution, however. The contours plotted in Figures 23 and 24, in spite of the different rates of load application and therefore magnitude of pore pressure generated, are similar in shape.

In Figure 26 are plotted contours of constant vertical stress distribution beneath a uniformly loaded circular foundation resting upon an elastic half-space. The basis for Figure 26 is the same as that for Figure 25. 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.

The pore pressure parameter A (Skempton, 1954) 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 A (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, prior to any drainage, to equal the vertical stress at that point. Thus, assuming undrained conditions, the shapes of the contours of constant vertical stress in Figure 26 would be expected to correspond somewhat to those of the excess pore pressure contours of Figures 23 and 24.

A comparison of the shapes of the contours of Figures 23 and 24 with those of Figure 26 indicates that the contours of the latter have much flatter inclinations. This is possibly a result of faster dissipation of excess pore water pressure near the surface than at depth. As Figures 19 to 22, however, indicate similar times of pore pressure dissipation throughout the depth of the peat layer (at least to a depth of 6 feet), a complete explanation on the basis of vertical drainage of the steepness of the pore pressure contours would appear unfounded.

When compared with the measured excess pore pressures, the calculated vertical stress increases (using Figures 25 or 26) occurring under the loading plate, due to loading, 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 depths of 6 feet. This might indicate that in this material the stress applied at the surface extends to a much greater depth than that indicated by the elastic theory. The effect of the stiffer clay layer at a depth of 10 feet is not considered sufficient to cause such a large discrepancy. A possible explanation might be Terzaghi's local shear hypothesis (Terzaghi and Peck, 1948). The term "local shear" is used to explain failure caused by excessive settlement directly below a footing, etc., but it is not associated with complete rupture of the supporting material. Local shear occurs near the edges of the footing and large compressions occur in the material immediately below the footing base. This would result in a situation somewhere between that of one-dimensional 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 this latter case the stresses decrease rapidly with depth, as is shown in Figures 25 and 26, 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 of the above might be that the A values for this peat are greater than 1, approaching the value of 2 at the 6-foot depth. This possibility can be investigated by further laboratory work.

In conclusion, the following points are salient:

(a) For both laboratory and field tests about one quarter of the total observed consolidation took place following dissipation of measured excess pore pressure.

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

(c) Any possibility of dividing settlements into primary and secondary consolidation phases solely on the basis of standard void ratio-log pressure plots did not appear feasible.

(d) 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.

(e) The essentially horizontal drainage occurring in the field prohibits the use of scaling laws to predict the times of equal percentages of consolidation in the field from observation of laboratory behaviour.

(f) Due to radial drainage, the degree of consolidation appeared to be relatively uniform throughout the depth of the consolidating stratum in the field.

(g) Greater structural reorientation as exhibited by consolidation took place in the laboratory than in the field during the period following excess pore pressure dissipation. This might be partly explained by sampling disturbance and partly by the differences in the lengths of drainage paths, which caused earlier dissipation of pore pressures in the laboratory.

(h) 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 two things:

(i) Shearing strains around the perimeter of the plate cause vertical stresses below the plate to be larger than those calculated using the elastic theory; i. e. local shear caused the material below the plate to approach a condition of one-dimensional stress

(ii) Stresses applied to the peat result in developed pore pressures greater than the increase in vertical stresses; i. e., "A" values greater than 1.

APPRAISAL AND RECOMMENDATIONS

This project, although limited by time and resources, satisfied the initial goal of the investigation. A basis for the evaluation of the particular approach was established, and the conclusions reached will contribute towards the efficient planning of future and more extensive investigations. The project also suggests pertinent laboratory procedures to be initiated to support a field investigation of this nature.

The extent of the occurrence of horizontal drainage in the field suggests that both larger loaded areas and different sizes of loaded areas should be used in the field. The importance of rate of loading on the generation of pore pressures suggests that this step of the test procedure should be provided with a greater degree of control. The apparent departure from elastic behaviour of peat under load suggests that the use of transducers to measure the total pressures at various locations throughout the peat would be valuable.

The procedure used to apply loading to the peat in situ in this investigation was not altogether satisfactory and would be improved by increased loading capacity and better control of loading rate. The quantity of water required to provide a substantial load militates against this loading method for any realistic loads. It is suggested that anchored piles be investigated as a means of loading, used in association with an A-frame.

The differences between measured pore pressures in the field and the calculated vertical pressures, based upon elastic theory, suggest the possibility that the pore pressure parameter A for this peat is greater than 1. The value of the A parameter should be measured in laboratory tests to get an indication of its significance in the behaviour demonstrated by the peat in the field.

Triaxial consolidation tests conducted in the laboratory with no lateral strains (K_o tests) could be used to suggest the changes in lateral stresses set up in the in situ peat under surface loads having a very confined area. Supporting tests conducted by consolidating peat specimens in a direction perpendicular to that on which they are laid down could then give some idea of the degrees of lateral deformation occurring under field loading.

Laboratory tests in which the excess pore pressures are controlled to maintain the rate of dissipation observed in the field could be used to determine the effects of the inconsistency existing between drainage rates in the laboratory and those in the field. Laboratory tests could be conducted to determine the values of the coefficients of permeability and the coefficients of consolidation under the different conditions of vertical and radial drainage and at different consolidation pressures. These values would permit analytical correlations with the experimental results.

REFERENCES

Adams, J. I., 1964. A Comparison of Field and Laboratory Consolidation Measurements on Peat. Proc. Ninth Muskeg Research Conference. N. R. C., Assoc. Committee on Soil and Snow Mechanics, Technical Memo. 61, Ottawa, p. 117-135.

Bishop, A. W. and D. J. Henkel, 1962. The Measurement of Soil Properties in the Triaxial Test. Edward Arnold Publishers, London. 2nd Ed.

Colley, B. E., 1950. Construction of Highways over Peat and Muck Areas. American Highways, Vol. XXIX, No. 1, p. 3-6.

Evgen'ev, I. E., 1961. The Accelerated Method of Obtaining Peat Soil Compression Characteristics. (In Russian). Torfyanaya Promyshlennost, No. 1, p. 25-6.

Hamilton, J. J. and C. B. Crawford, 1959. Improved Determinations of Consolidation Pressure of a Sensitive Clay. ASTM Special Technical Pub. No. 254, p. 254-71.

Lake, J.R., 1960. Pore Pressure and Settlement Measurements During Small-Scale and Laboratory Experiments to Determine the Effectiveness of Vertical Sand Drains in Peat. Proc. Conf. on Pore Pressure and Suction in Soils, Butterworths, London, p. 103-107.

Lambe, T. W., 1964. Methods of Estimating Settlement, Proc. Am. Soc. Civil Engineers, 90, SM 5, p. 43-67.

Lea, N. D. and C. O. Brawner, 1963. Highway Design and Construction over Peat Deposits in the Lower Mainland Region of British Columbia. Highway Research Board, Research Record No. 7, Washington, D. C., p. 406-24.

Leonards, G. and P. Girault, 1961. A Study of the One-Dimensional Consolidation Test. Proc. Fifth International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, p. 213-18.

Lo, M. B., 1964. Analysis of the Rates of Consolidation for Peat. Unpublished M. Eng. Thesis, Faculty of Graduate Studies, McMaster University.

MacFarlane, I. C., 1958. Guide to a Field Description of Muskeg. N. R. C., A. C. S. S. M. Technical Memorandum 44 (Revised) (NRC 4214).

MacFarlane, I. C., 1964. Muskeg Field Program - Mer Bleue Peat Bog. DBR Internal Report No. 304, DBR-NRC.

MacFarlane, I. C., 1965a. The Consolidation of Peat, A Literature Review. DBR Technical Paper No. 195, NRC No. 8393.

MacFarlane, I. C., 1965b. Some Preliminary Consolidation Tests on Peat. DBR Internal Report No. 318.

Miyakawa, I., 1960. Some Aspects of Road Construction in Peaty or Marshy Areas in Hokkaido, with Particular Reference to Filling Methods. Civil Eng. Res. Inst., Hokkaido Development Bureau, Sapporo, Japan.

Scott, R. F., 1963. Principles of Soil Mechanics. Addison-Wesley Publ. Co., Inc., Reading, Mass. 550 pp.

Skempton, A. W., 1954. The Pore Pressure Coefficients A and B. Geotechnique, Vol. 4, No. 4, p. 143-7.

Skempton, A. W. and L. Bjerrum, 1957. A Contribution to the Settlement Analysis of Foundations on Clay, Geotechnique, Vol. 7, No. 4, p. 168.

Taylor, D. W., 1948. Fundamentals of Soil Mechanics. John Wiley and Sons, Inc., New York.

Taylor, D. W., 1942. Research on Consolidation of Clays. Serial No. 92, Massachusetts Institute of Technology, Dept. of Civil and Sanitary Engineering.

Terzaghi, K. and R. B. Peck, 1948. Soil Mechanics in Engineering Practice. John Wiley and Sons Inc., New York.

Wilson, N. E. and M. B. Lo, 1966. The Progress of Consolidation in an Organic Soil. Proc. Eleventh Muskeg Res. Conf. N. R. C., Associate Committee on Geotechnical Research, Technical Memorandum No. 87, Ottawa, p. 1-9.

Wilson, N. E., N. W. Radforth, I. C. MacFarlane, and M. B. Lo, 1965. Rates of Consolidation for Peat. Proc., Sixth International Conference on Soil Mechanics and Foundation Engineering, Montreal. Vol. 1, p. 407-411.

TABLE I

INCLINATION OF VERTICAL TILT PINS

Location from Centreline of Loading Plate		Time
2-1/2 ft off-centre	4-1/2 ft off-centre	
0°	0°	Platform in place
3°	1°	Barrels filled
6°	2°	One day after loading
7-1/2°	2°	Four days after loading
8°	2°	Eleven days after loading
4°	1°	One day after loading

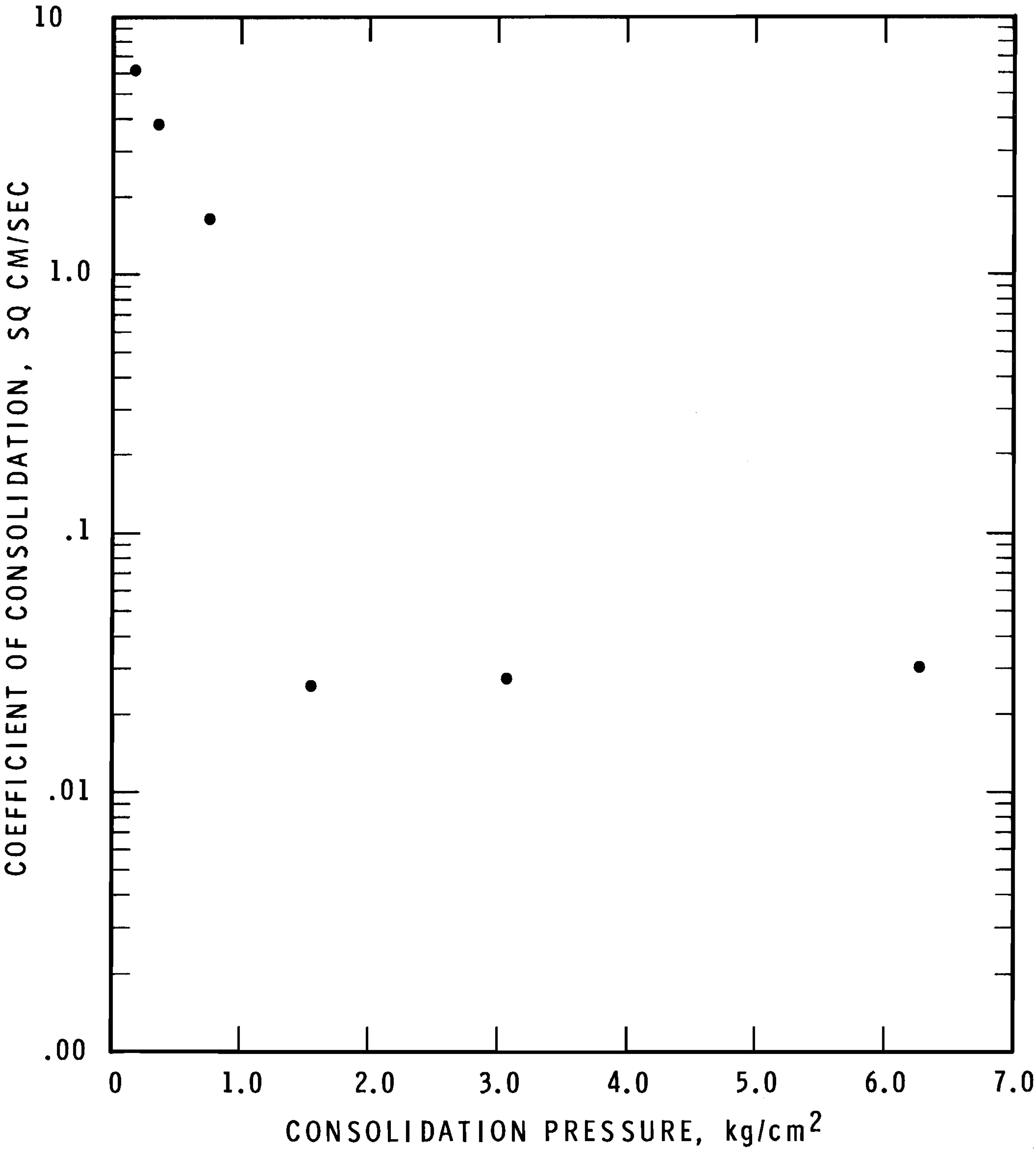

FIGURE 1

COEFFICIENT OF CONSOLIDATION VERSUS CONSOLIDATION PRESSURE (Sample 63-2-3, after MacFarlane 1965)

BR 3942-1

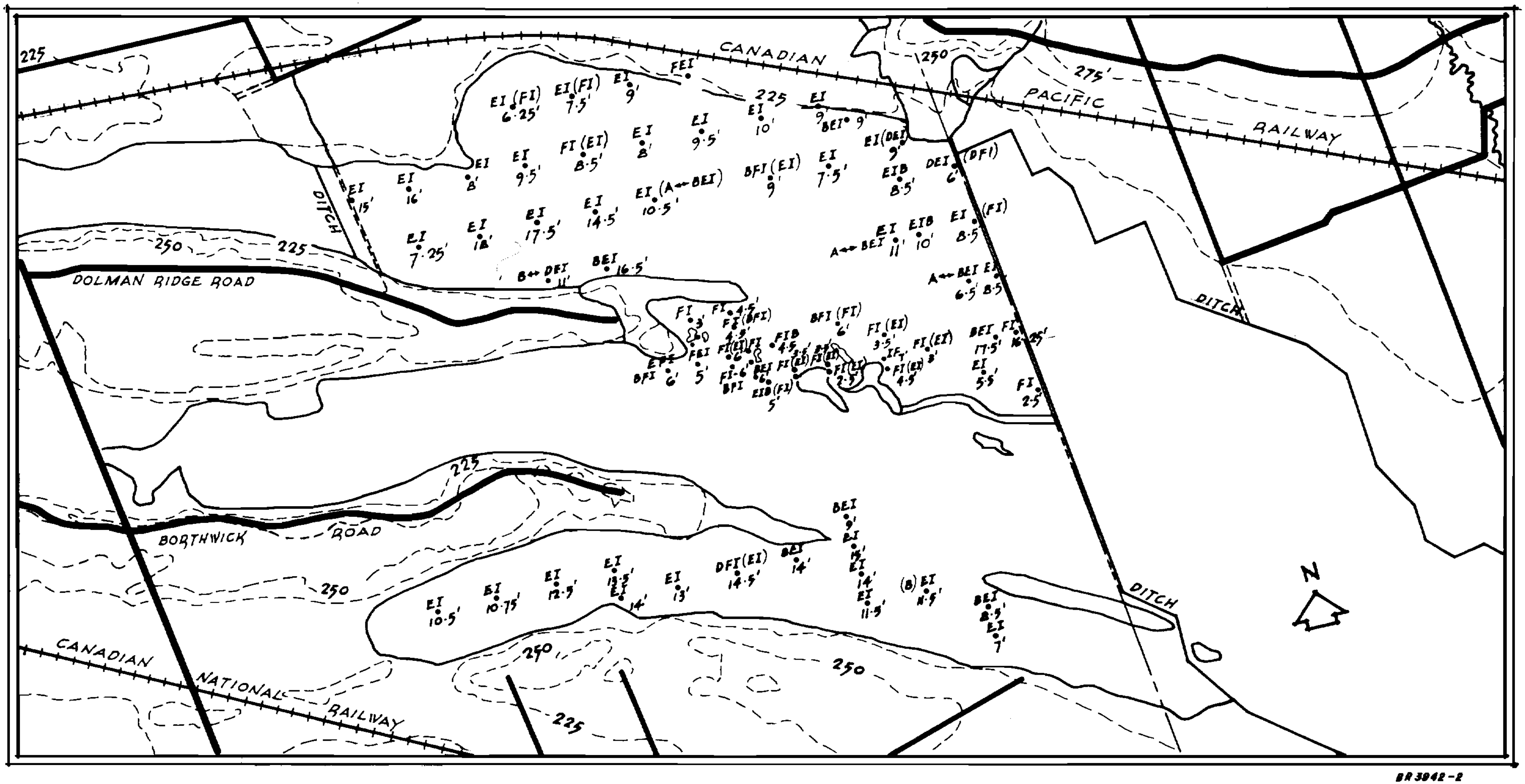

FIGURE 2a

MER BLEUE PEAT BOG, OTTAWA

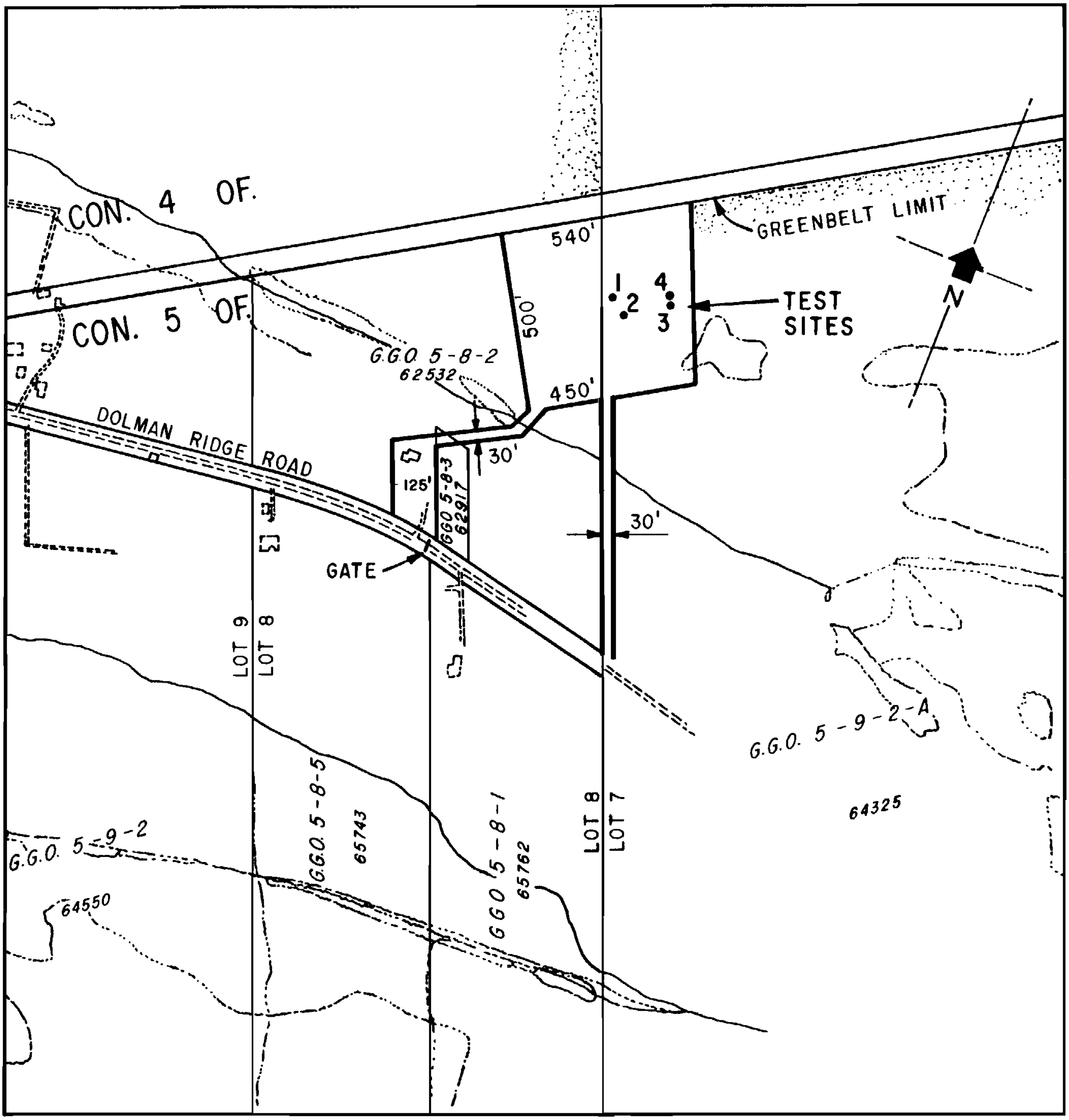

FIGURE 2b

SITE OF MUSKEG LOADING TESTS AT MER BLEUE

BR 3942-3

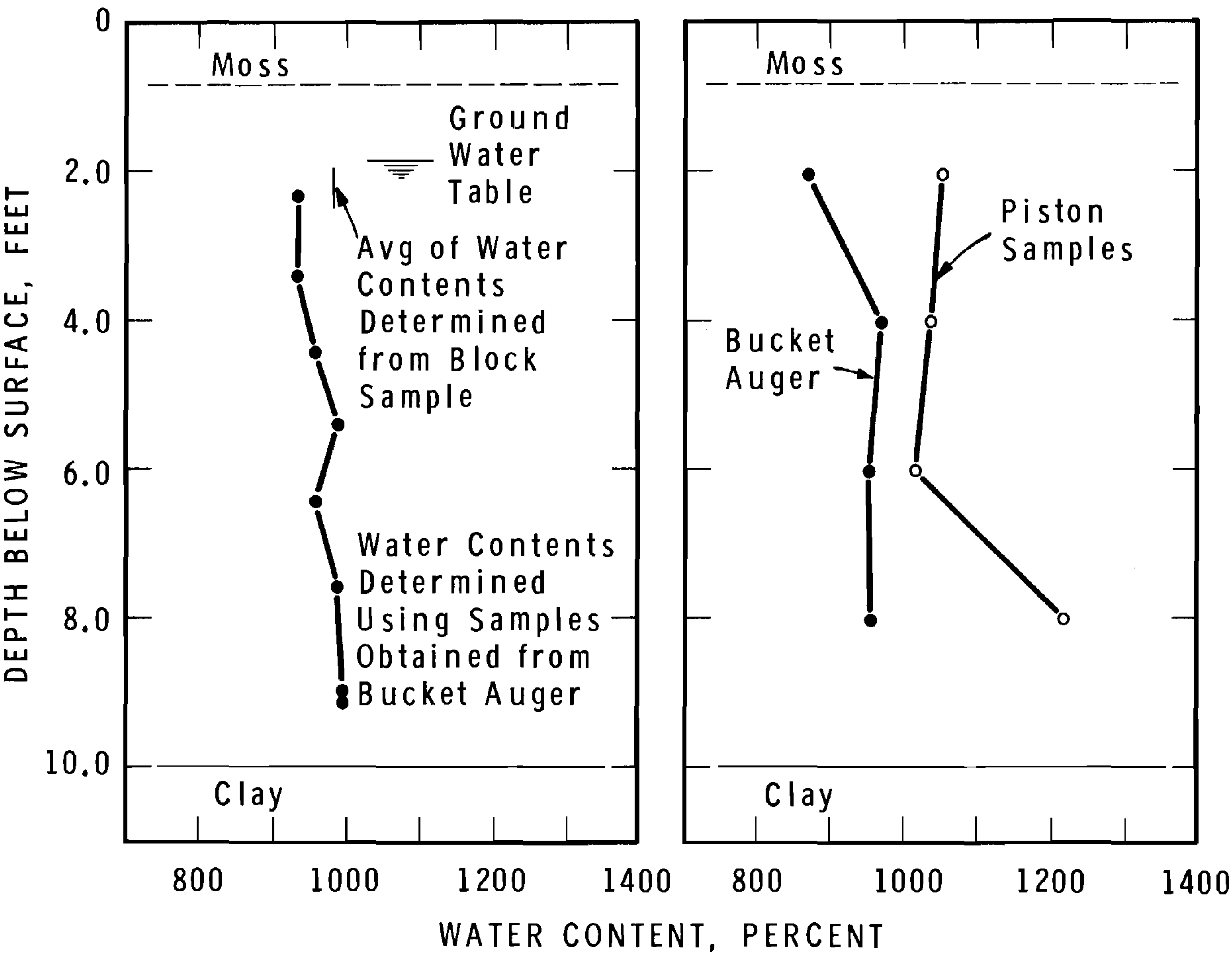

FIGURE 3a

WATER CONTENT VS DEPTH NEAR SITES 3 AND 4

FIGURE 3b

WATER CONTENT VS DEPTH FROM TWO SAMPLING PROCEDURES

BR 3942-4

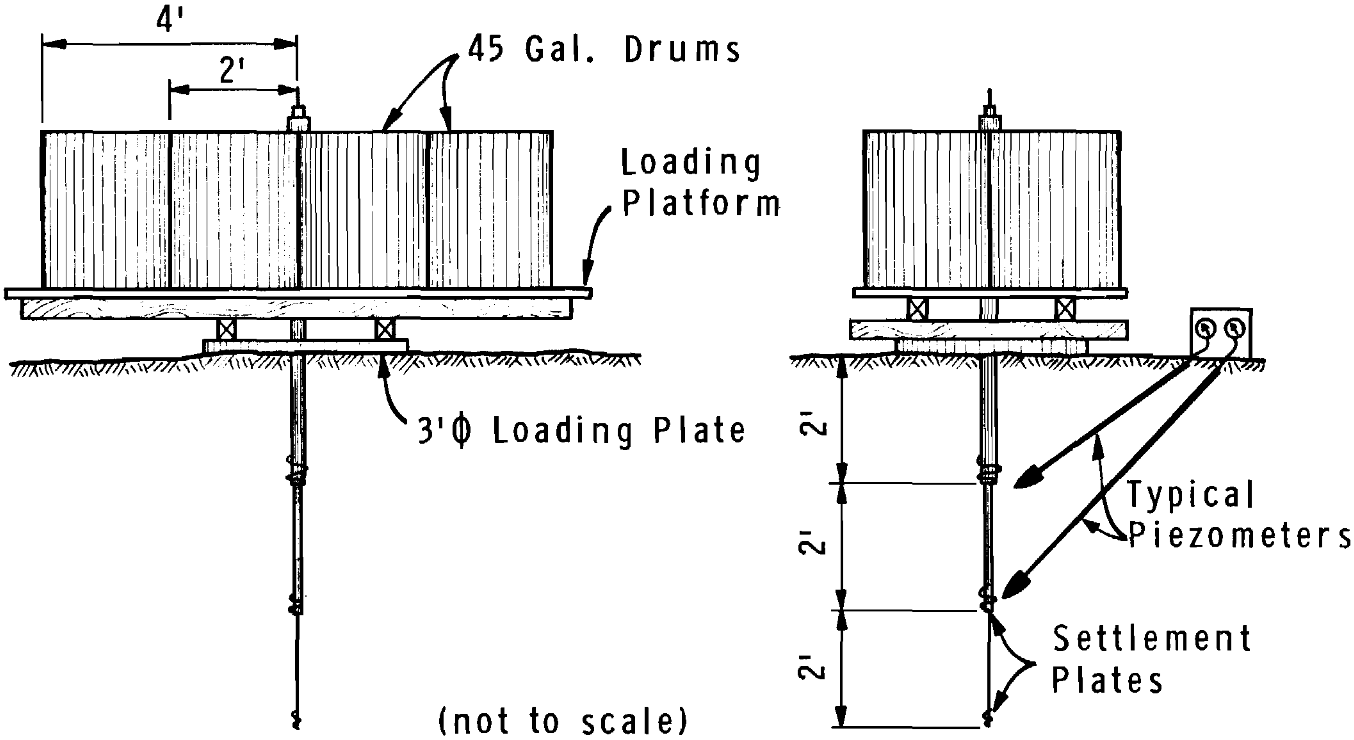

FIGURE 4a

LOADING ARRANGEMENT AND INSTRUMENTATION

BR 3942-5

Figure 4b. Loading plate, installed settlement rods and vertical pins

Figure 4c. Field loading set-up

Figure 5. Modified Geonor Piezometer

Figure 6. Wooden box holding Bourdon gauges

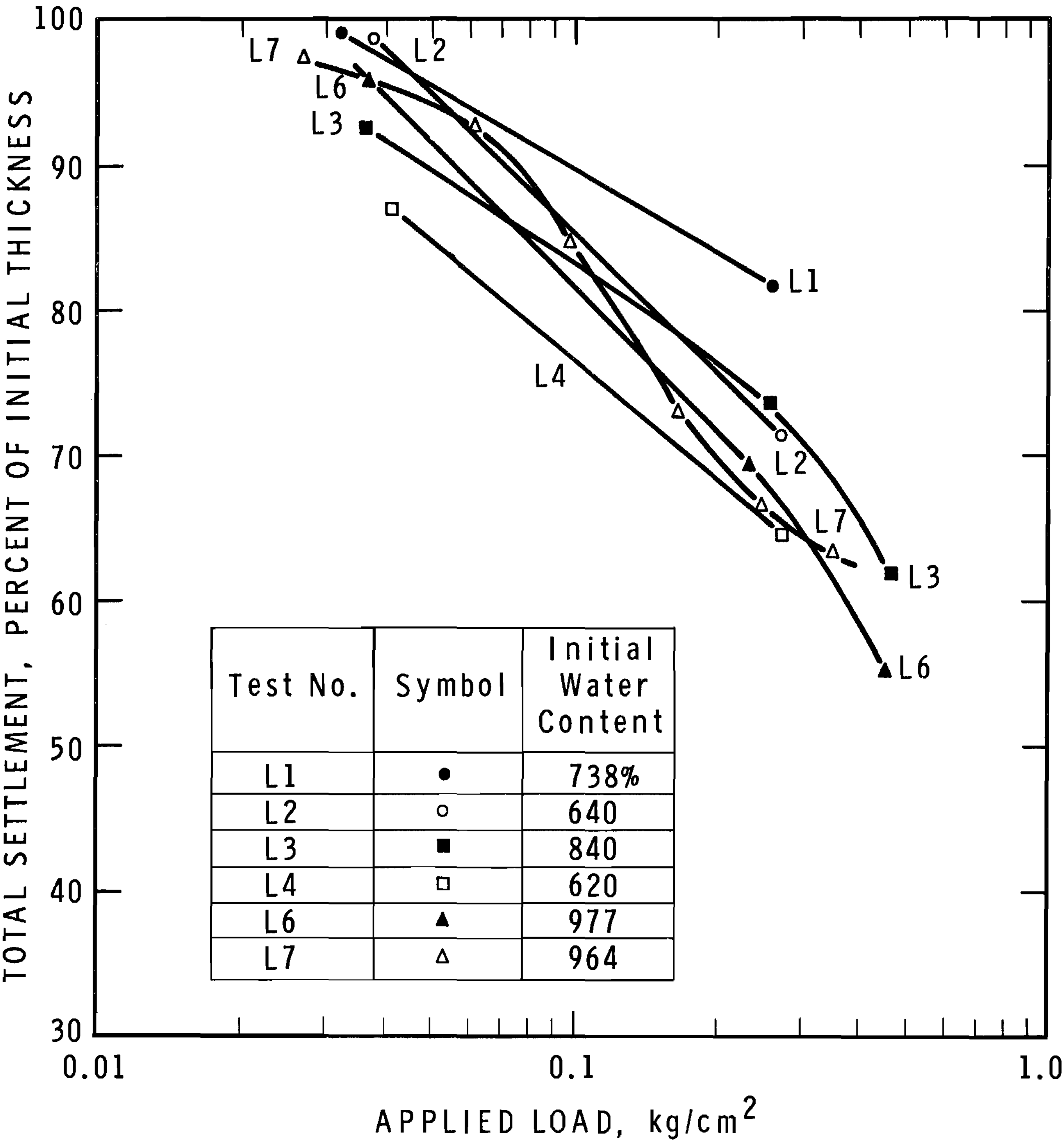

FIGURE 7

SETTLEMENT - LOG PRESSURE RELATIONSHIP FOR LABORATORY CONSOLIDATION TESTS

BR 3942-6

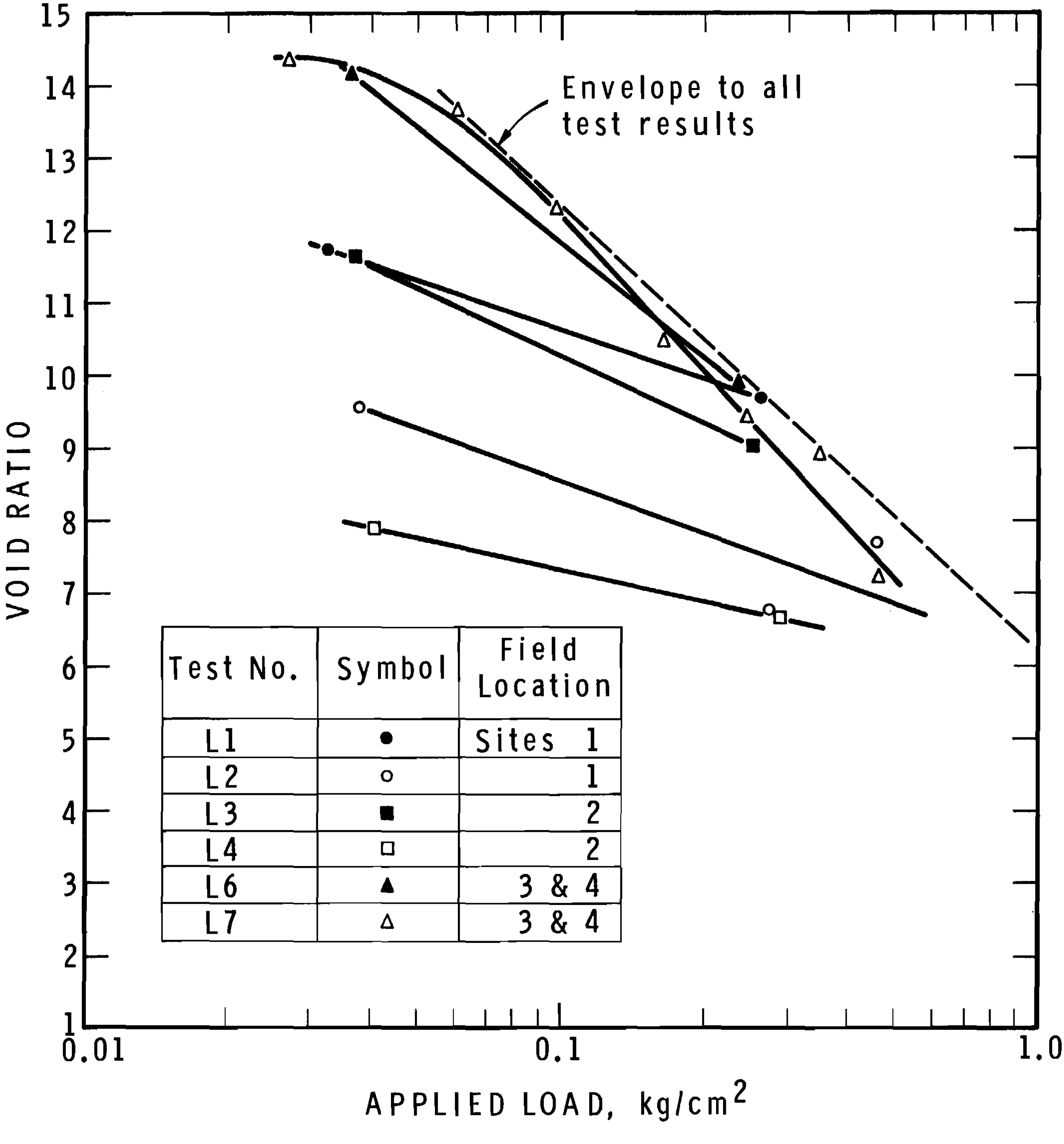

FIGURE 8

VOID RATIO - LOG PRESSURE RELATIONSHIPS FOR LABORATORY CONSOLIDATION TESTS

BR 3942-7

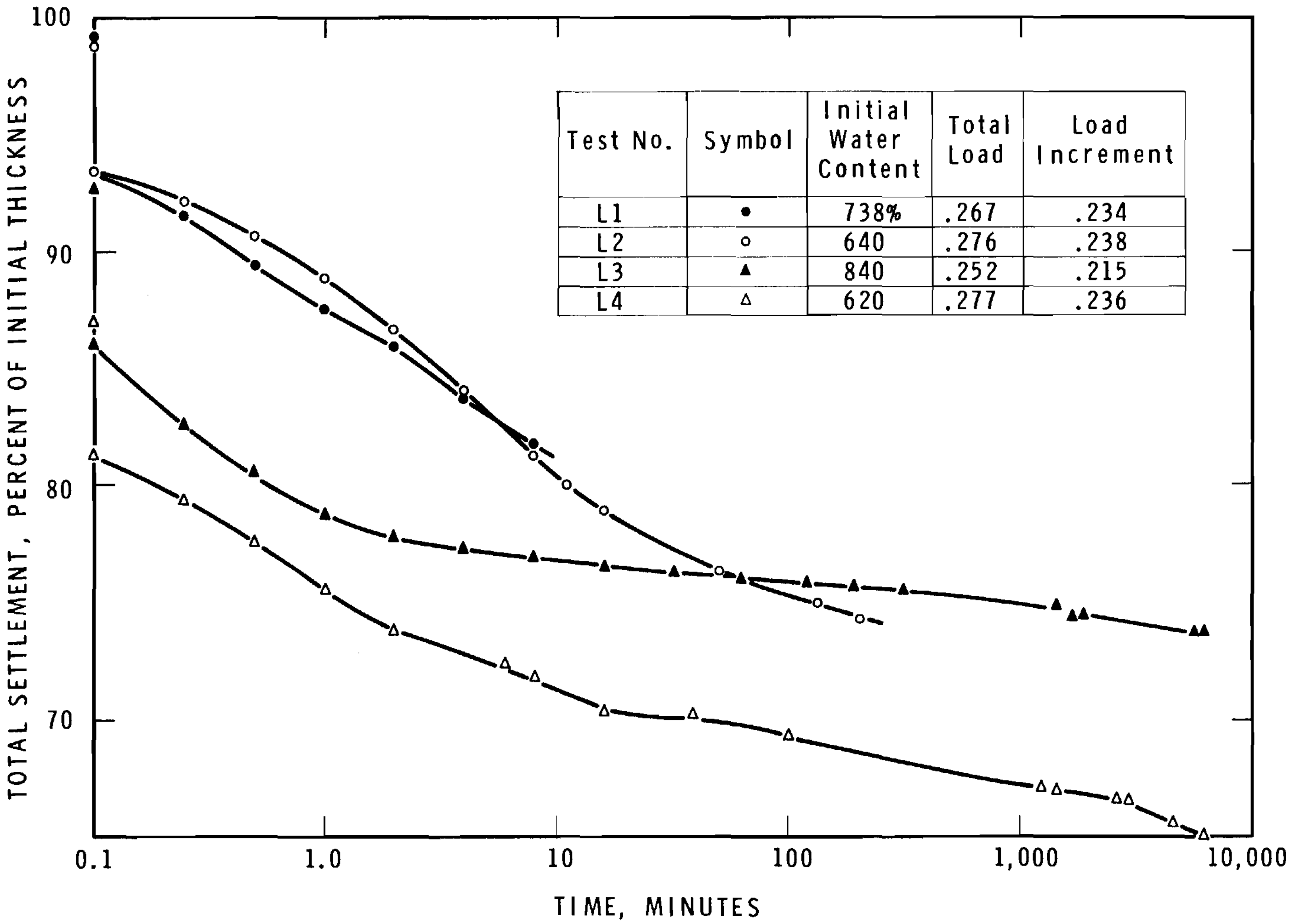

FIGURE 9

SETTLEMENT - LOG TIME FOR LABORATORY CONSOLIDATION TESTS

BR 3942 - 8

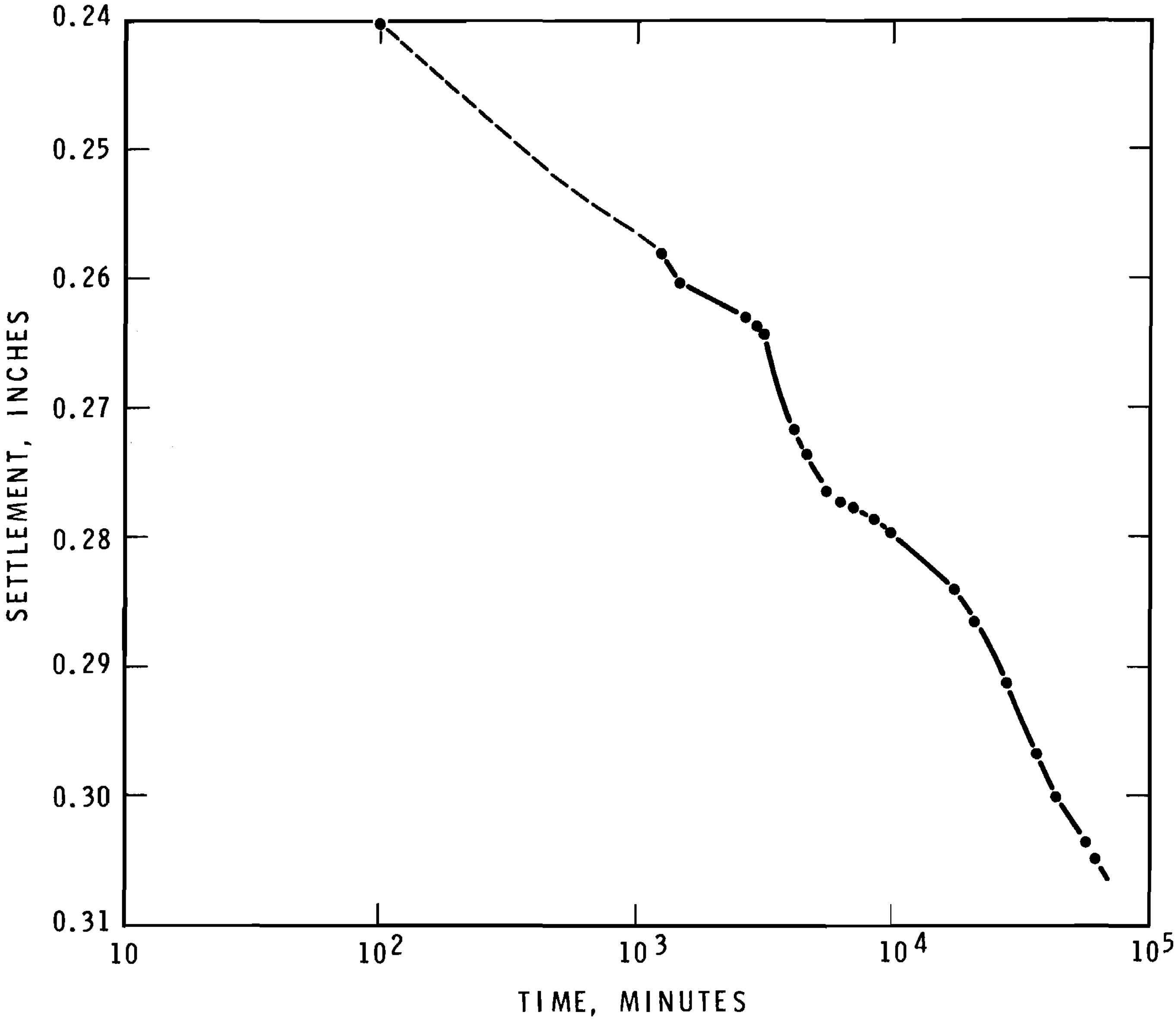

FIGURE 10

SETTLEMENT - LOG TIME FOR LABORATORY CONSOLIDATION TEST NO. L4

BR 3942-9

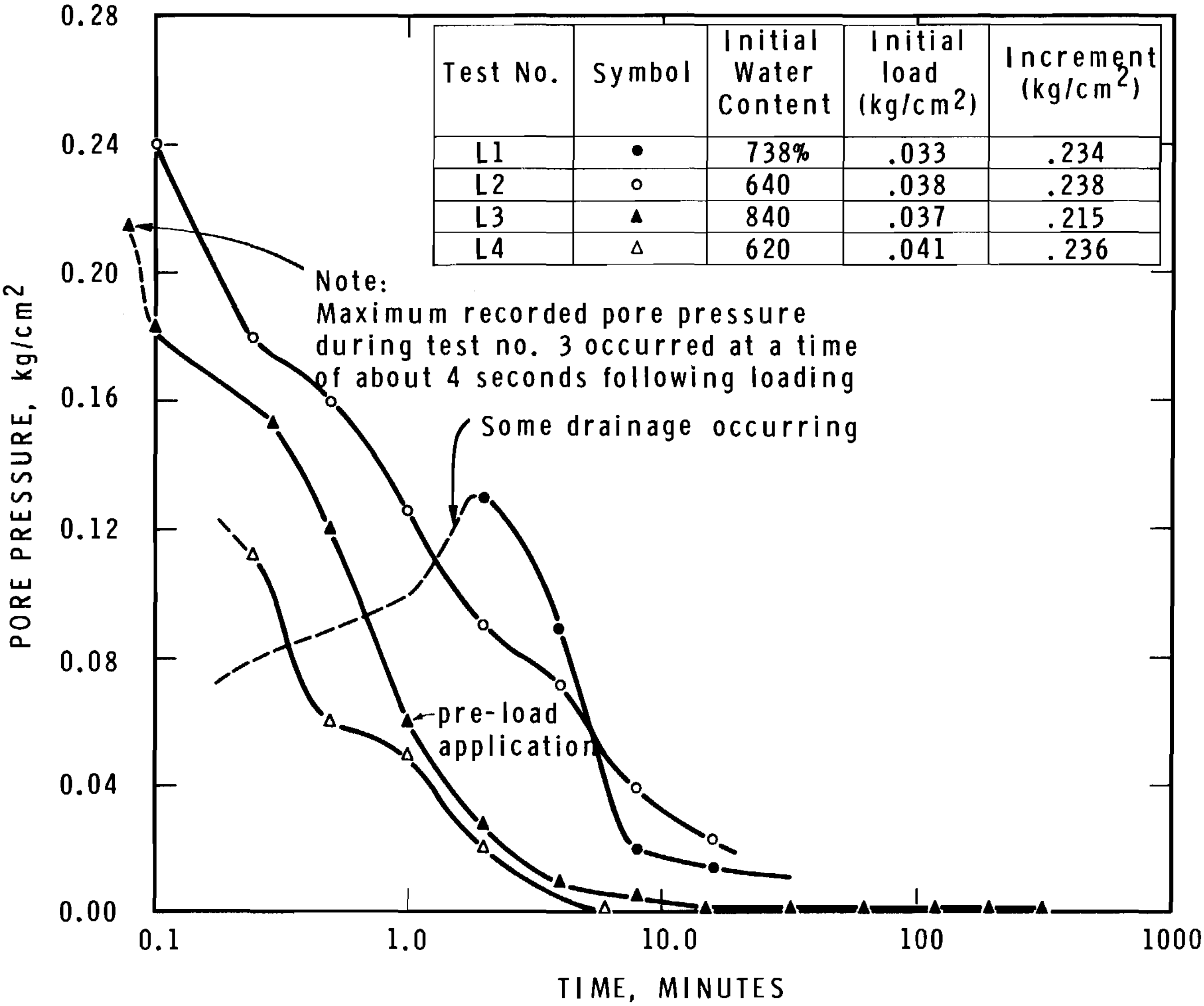

FIGURE 11

PORE PRESSURE DISSIPATION FOR LABORATORY CONSOLIDATION TESTS

BR 3942-10

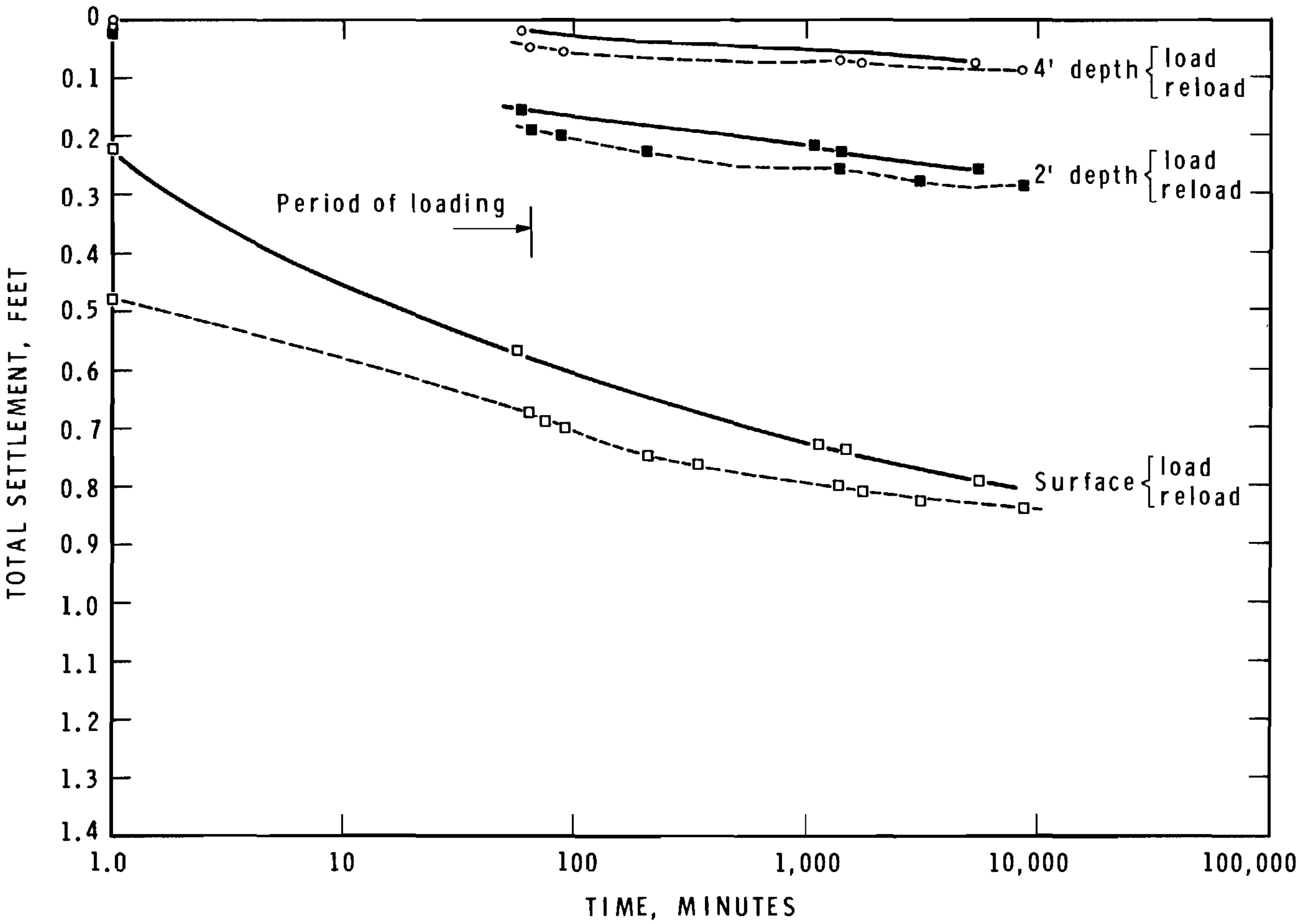

FIGURE 12

TIME-SETTLEMENT CURVES FOR FIELD TEST F-1

BR 3942-11

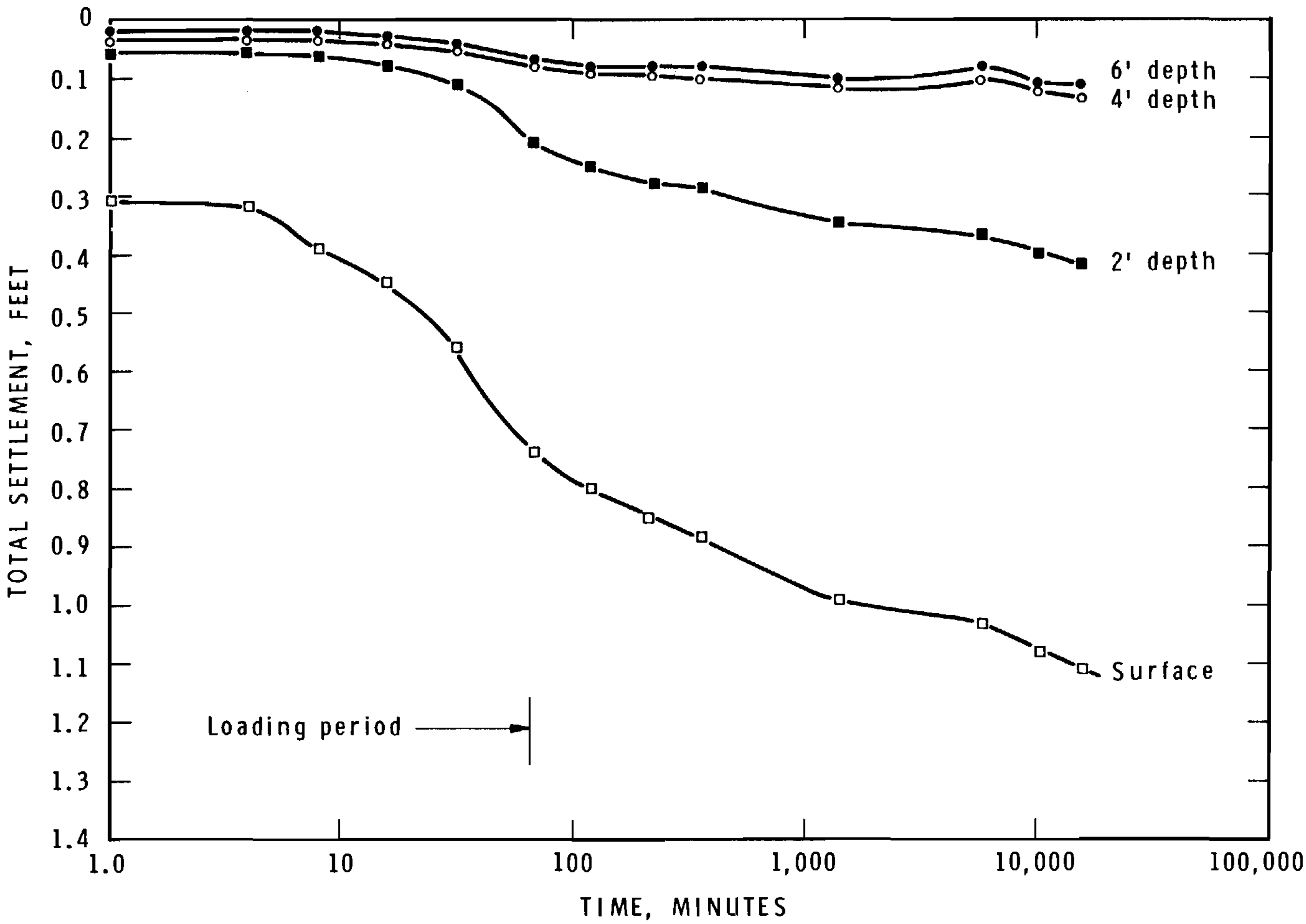

FIGURE 13

TIME - SETTLEMENT CURVES FOR FIELD TEST F-2

BR 3942 - 12

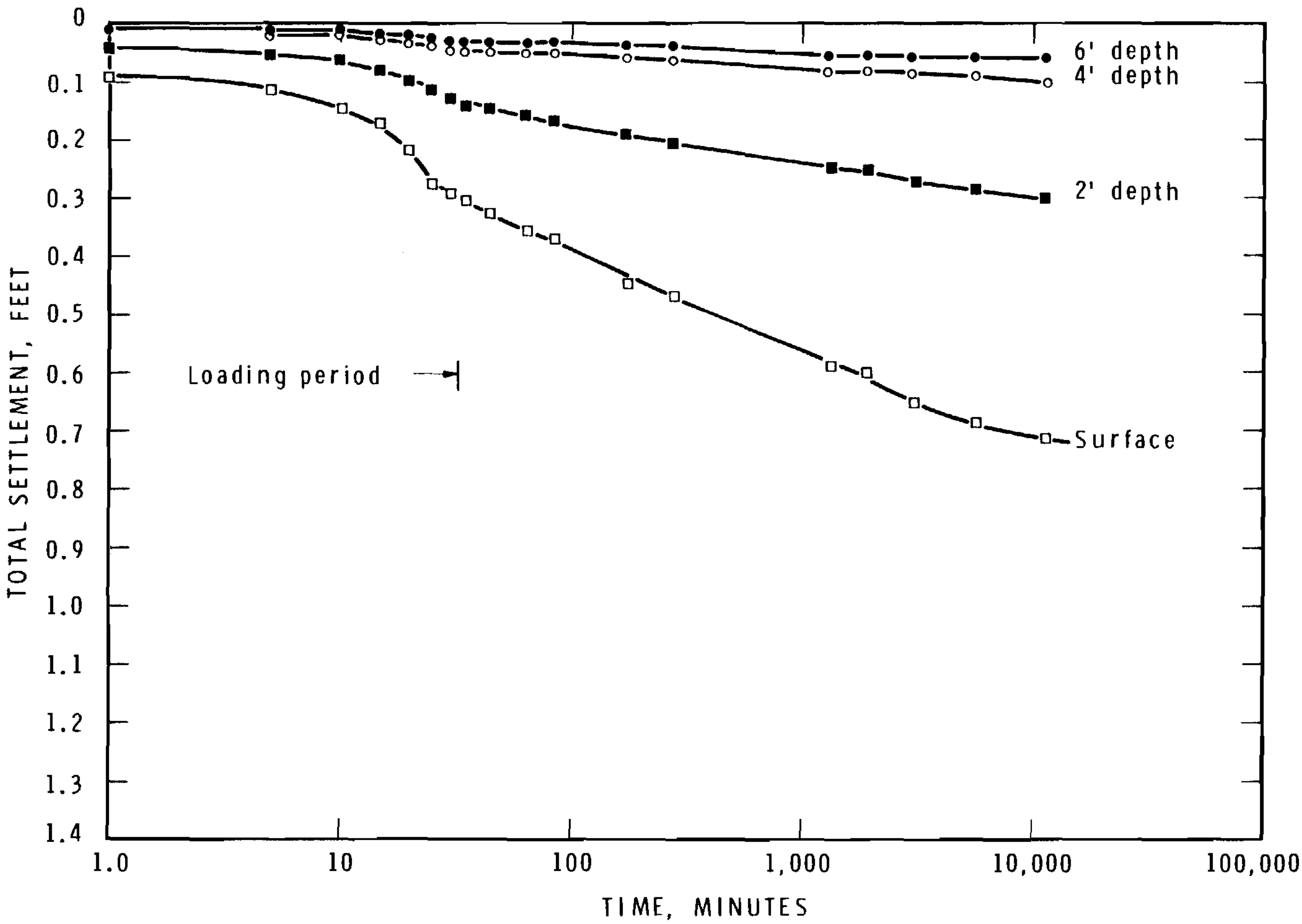

FIGURE 14

TIME - SETTLEMENT CURVES FOR FIELD TEST F-3

BR 3942-13

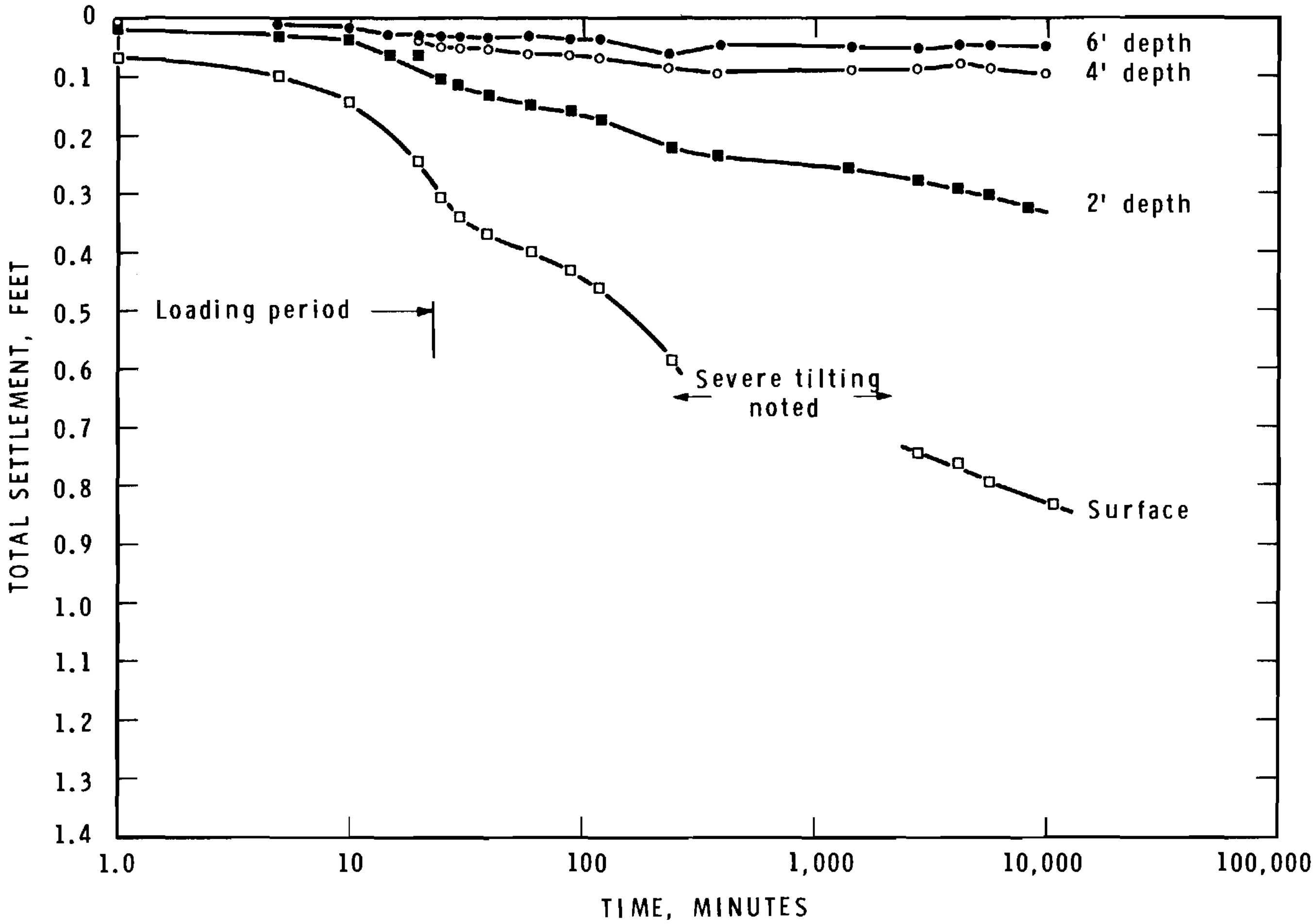

FIGURE 15

TIME - SETTLEMENT CURVES FOR FIELD TEST F-4

BR 3942-14

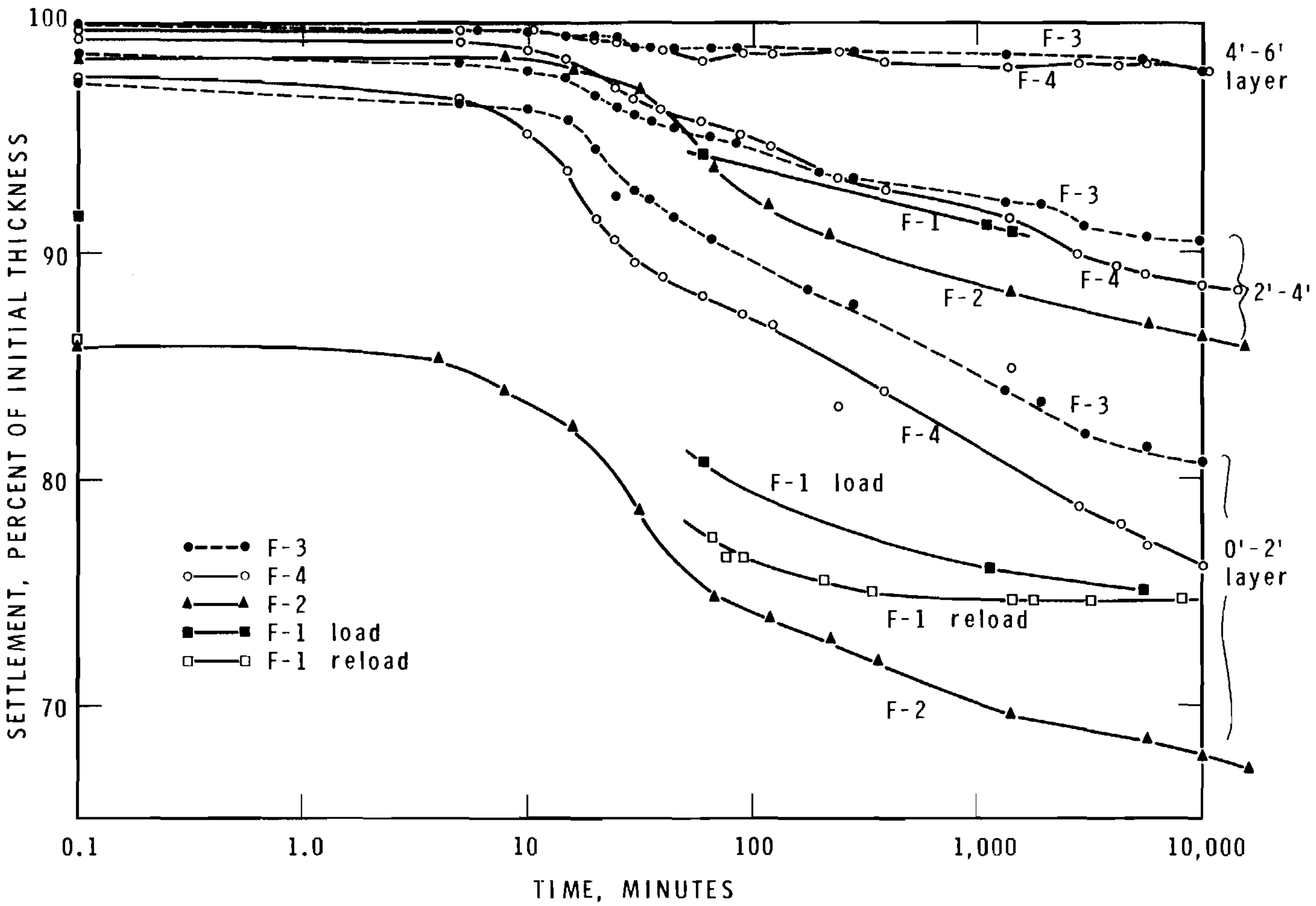

FIGURE 16

TIME - SETTLEMENT CURVES FOR DIFFERENT PEAT LAYERS FOR FIELD TESTS F-1 TO F-4

BR 3942-15

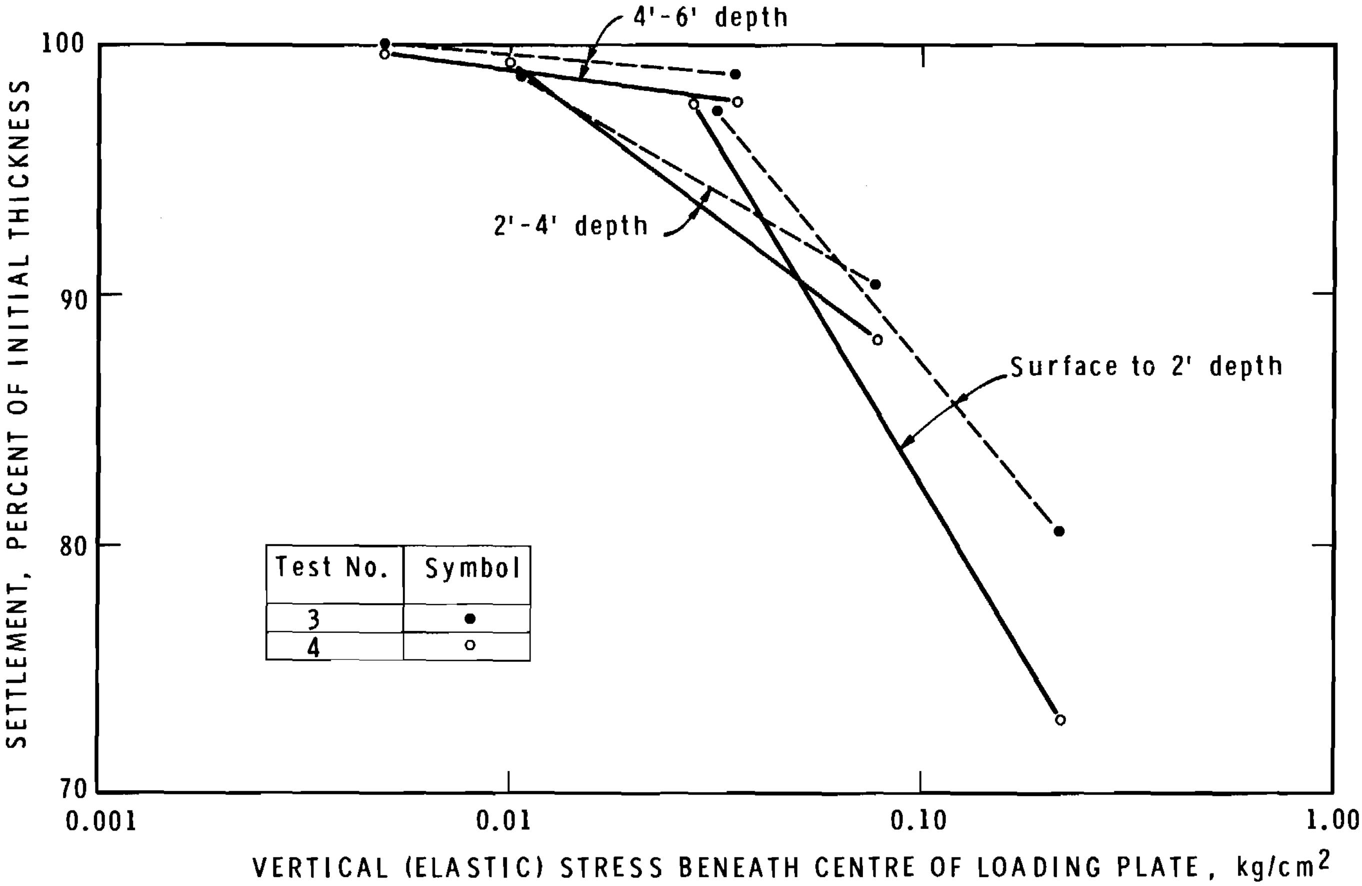

FIGURE 17

SETTLEMENT VERSUS CALCULATED AVERAGE VERTICAL STRESS FOR INDIVIDUAL SOIL LAYERS UNDER FIELD TESTS F-3 AND F-4

BR 3942-16

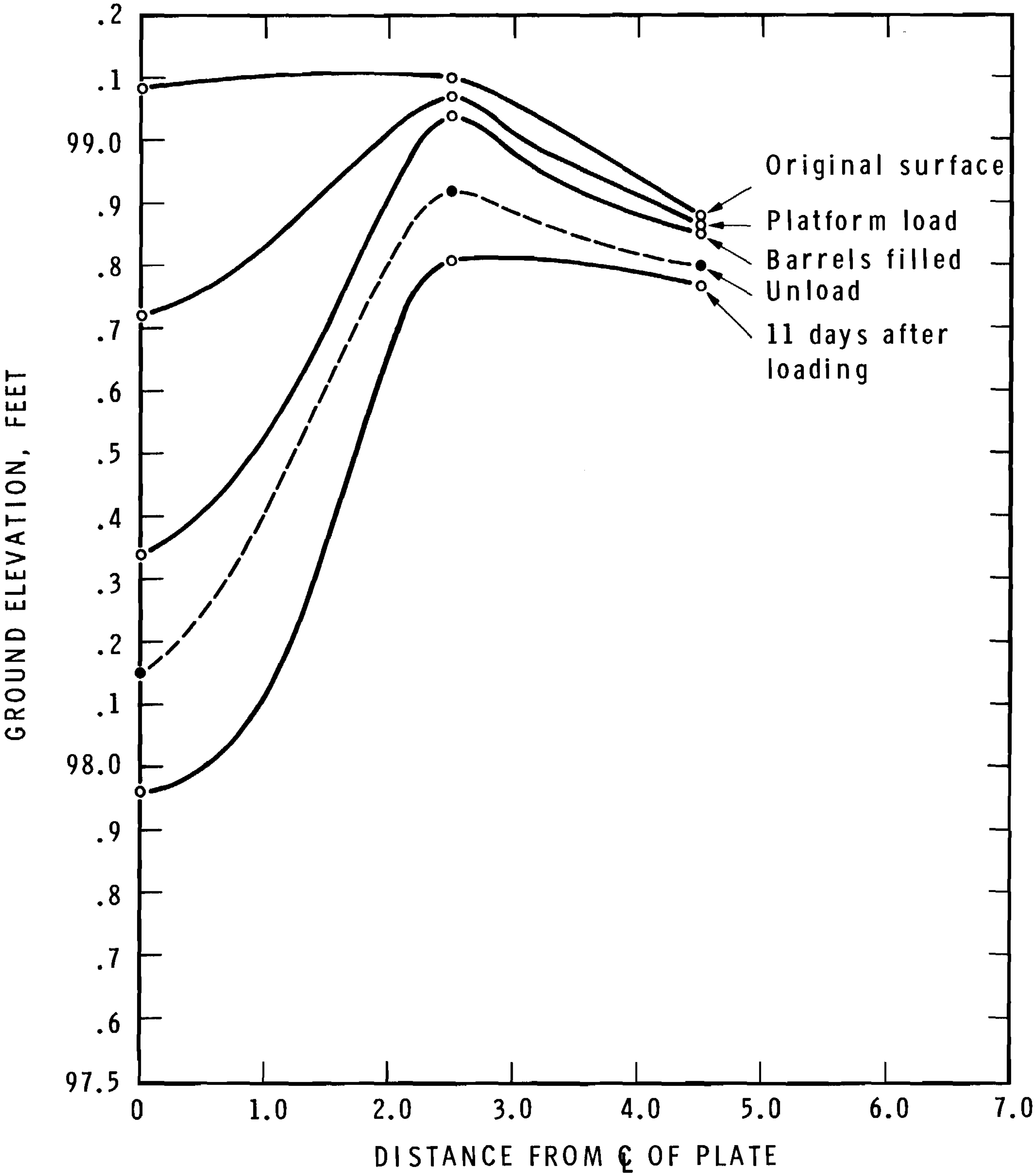

FIGURE 18

VARIATION IN GROUND ELEVATION DUE TO APPLIED LOADS
TESTS F-2

BR 3942-17

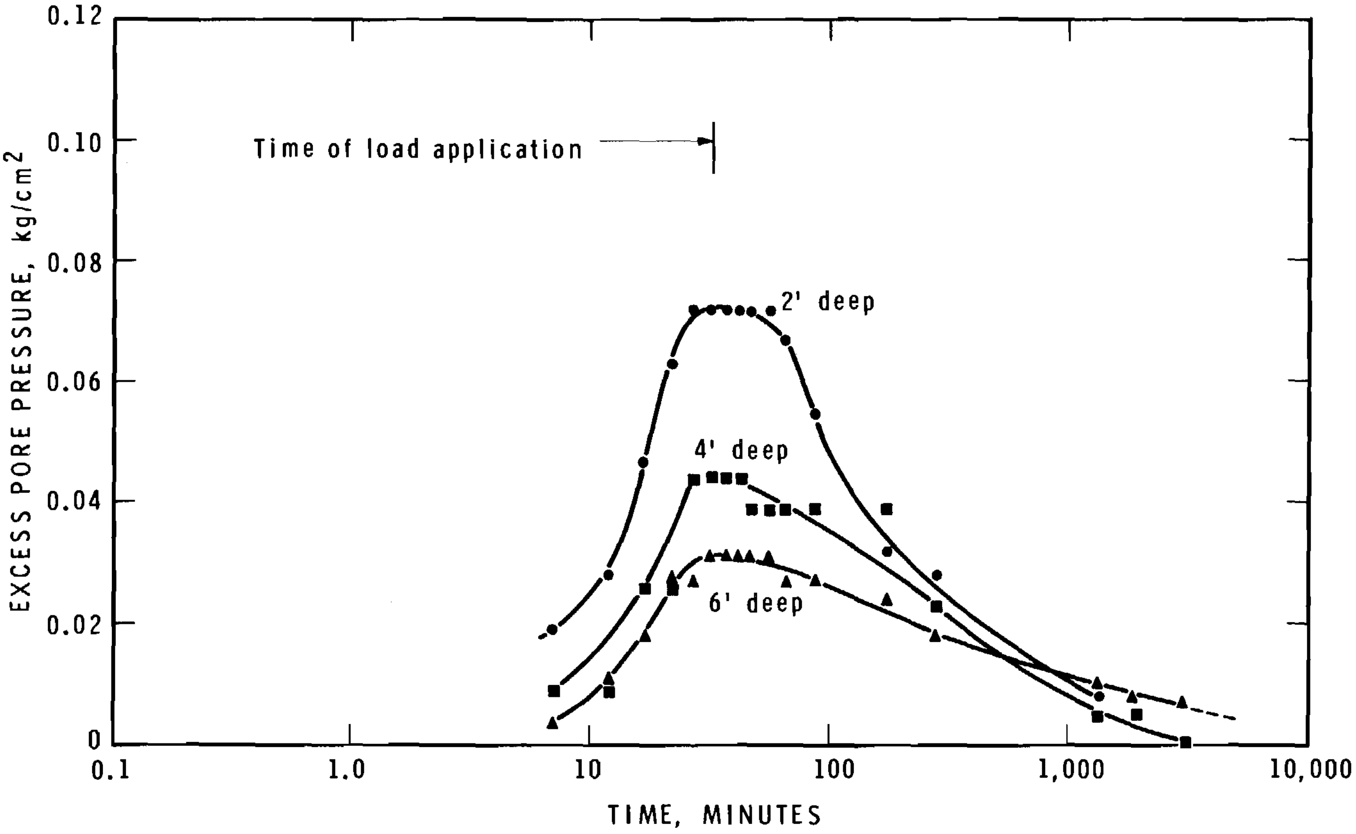

FIGURE 19

PORE PRESSURE DISSIPATION AT POINTS UNDER THE CENTRE OF THE PLATE DURING FIELD TEST F-3

BR 3942-18

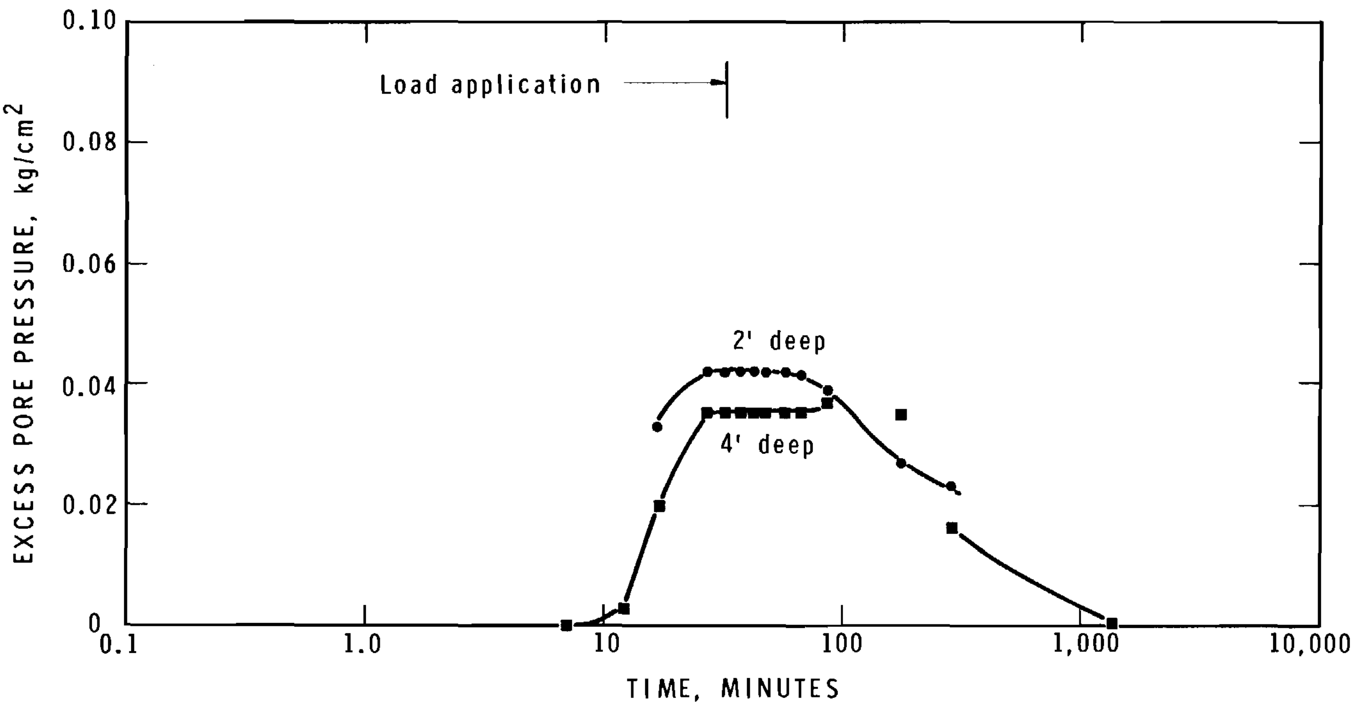

FIGURE 20

PORE PRESSURE DISSIPATION AT POINTS UNDER THE EDGE OF THE PLATE DURING FIELD TEST F-3

BR 3942-19

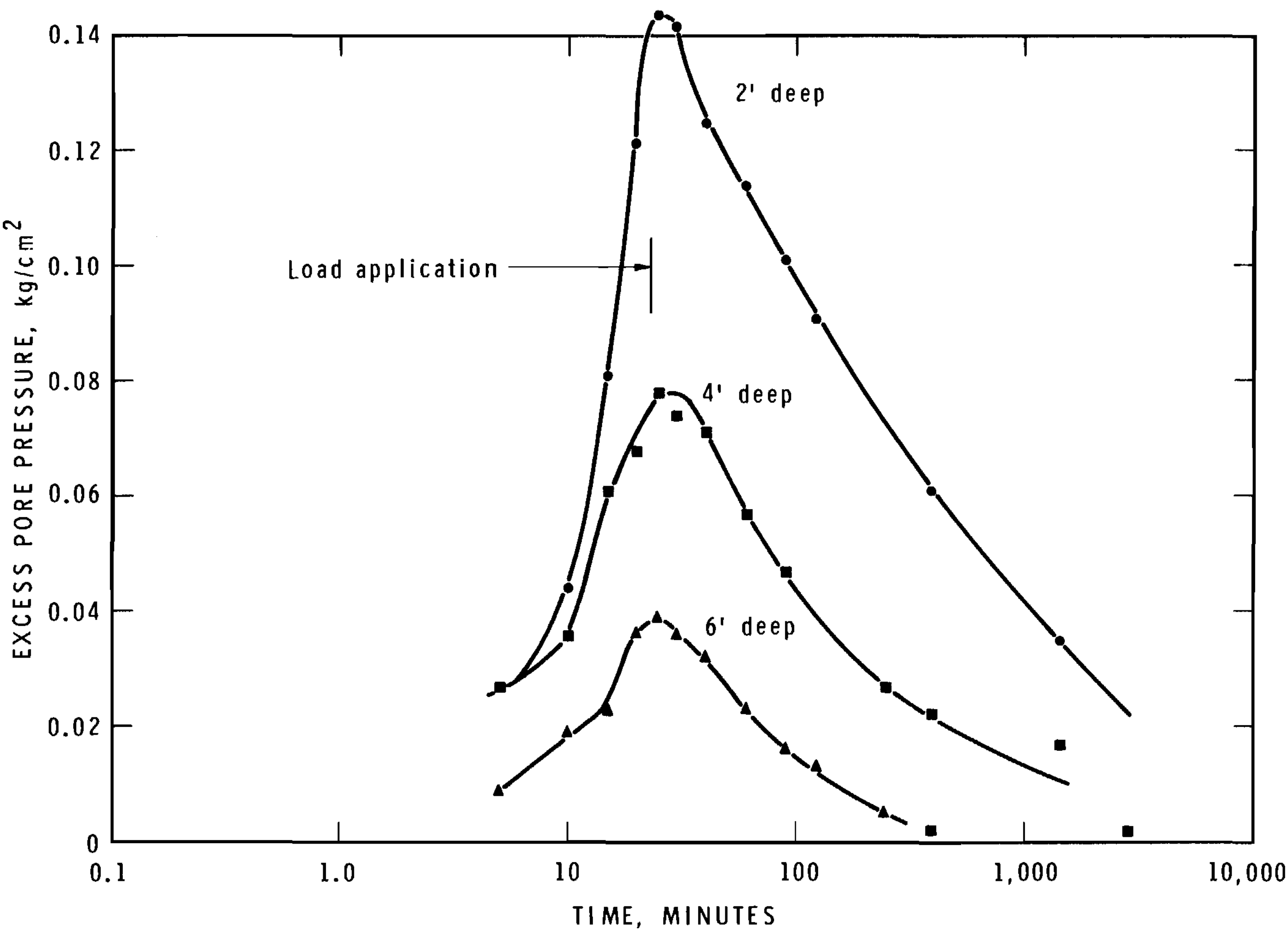

FIGURE 21

PORE PRESSURE DISSIPATION AT POINTS UNDER THE CENTRE OF THE PLATE DURING FIELD TEST F-4

BR 3942-20

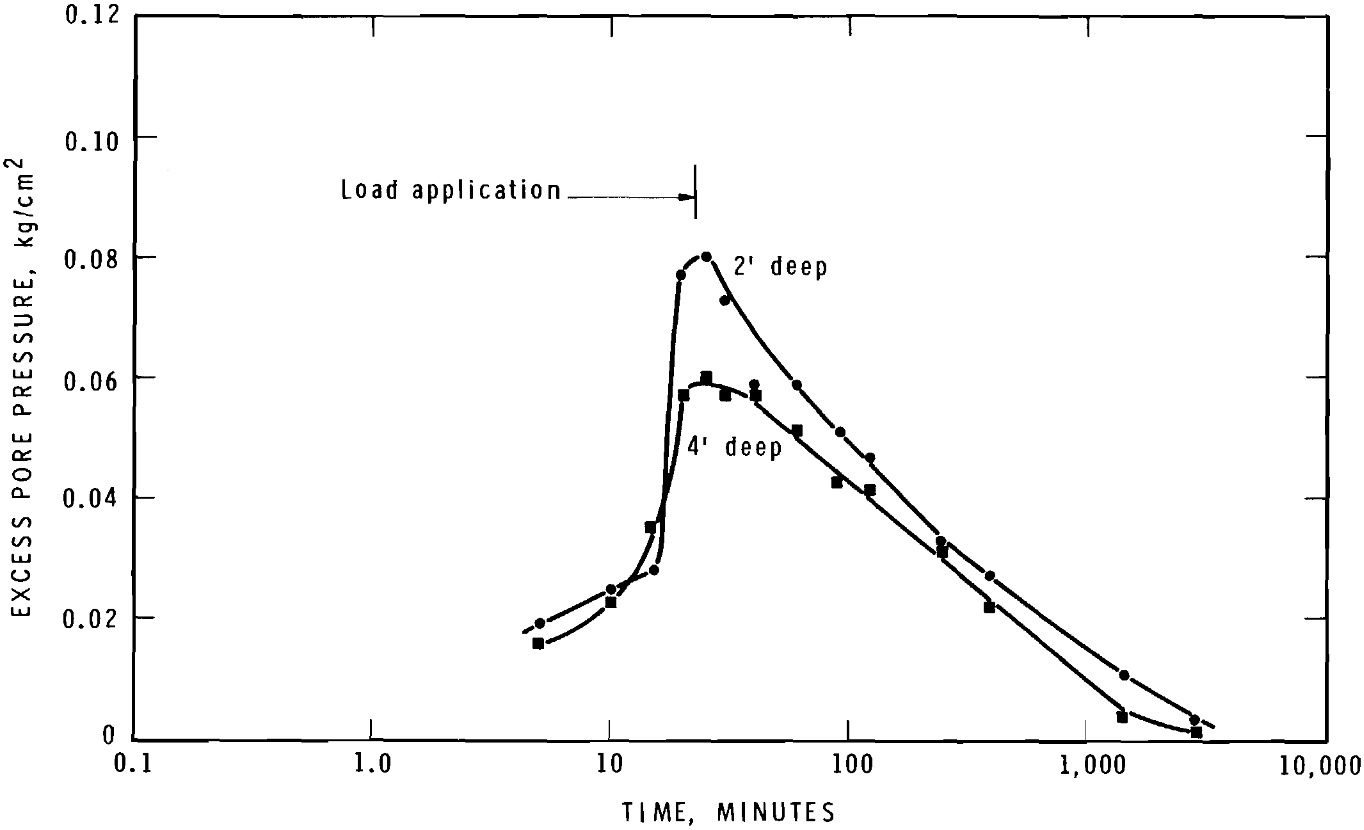

FIGURE 22

PORE PRESSURE DISSIPATION AT POINTS UNDER THE EDGE OF THE PLATE DURING FIELD TEST F-4

BR 3942-21

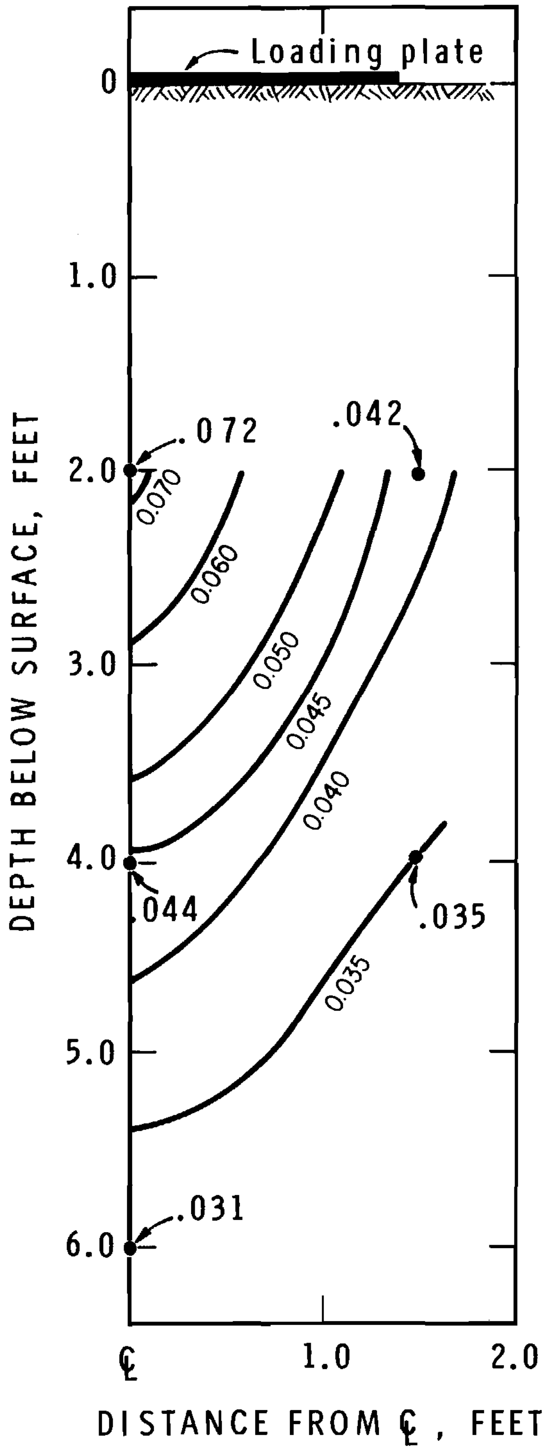

FIGURE 23

CONTOURS OF EXCESS PORE WATER PRESSURE (kg/cm^2) DURING TEST F-3 (at completion of loading)

BR 3942-22

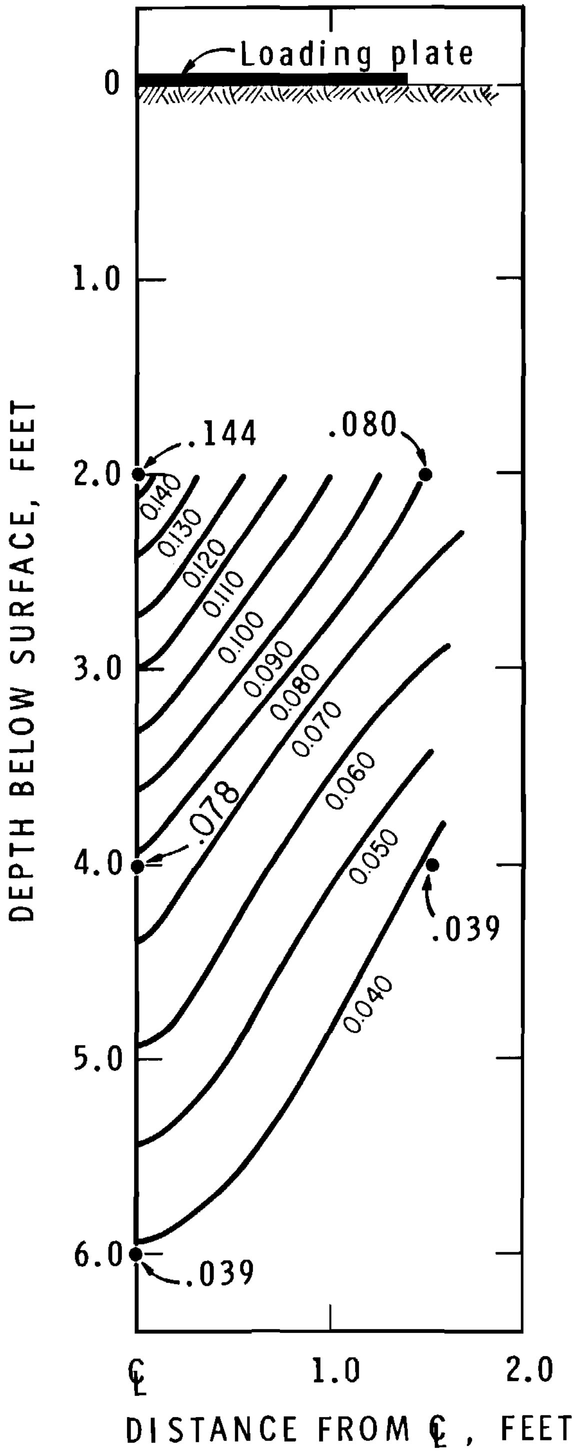

FIGURE 24

CONTOURS OF EXCESS PORE WATER PRESSURE (kg/cm^2) DURING TEST F-4 (at completion of loading)

BR 3942-23

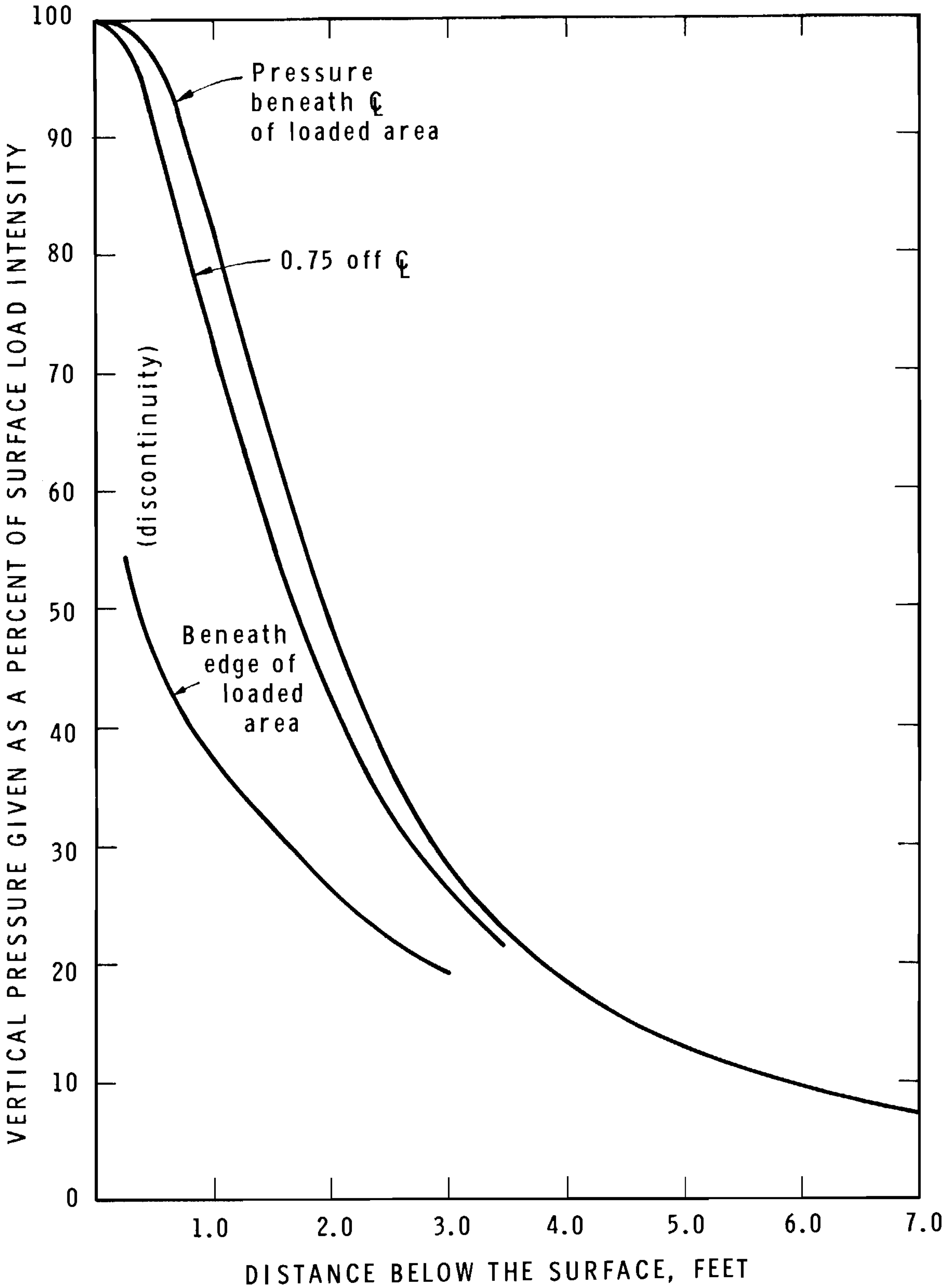

FIGURE 25

VERTICAL STRESS DISTRIBUTION IN A HOMOGENEOUS ISOTROPIC ELASTIC SEMI-INFINITE MEDIUM WITH A 3' DIAMETER UNIFORMLY DISTRIBUTED LOAD ACTING UPON THE SURFACE
(from Scott, 1963, Appendix B)

BR 3942-24

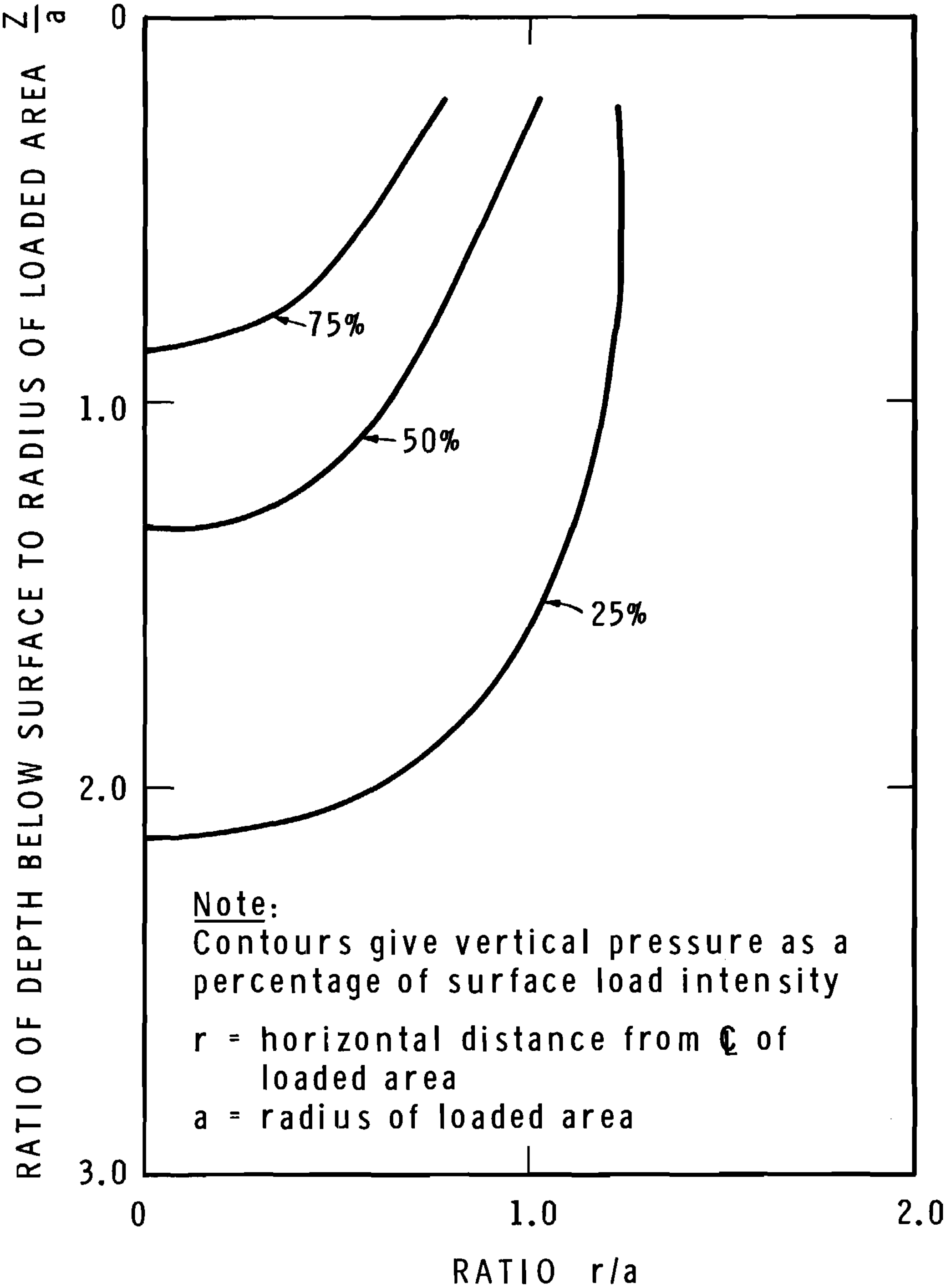

FIGURE 26

CONTOURS OF CONSTANT VERTICAL STRESS IN A HOMOGENEOUS ISOTROPIC ELASTIC SEMI-INFINITE MEDIUM WITH A UNIFORMLY DISTRIBUTED LOAD OF DIAMETER 2a ACTING UPON THE SURFACE (from Scott, 1963, Appendix B)

BR 3942 -25