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Laboratory measurement of pore water pressure induced by a model friction pile

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LABORATORY MEASUREMENT OF PORE WATER PRESSURE INDUCED BY A MODEL FRICTION PILE

BY

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ABSTRACT

This research program has been undertaken to develope a laboratory testing procedure for the measurement of pore water pressure induced by the static loading of a model friction pile. Most of the investigation involved the design, construction, and testing of equipment to be used during the tests which would produce satisfactory and reliable results. It was also desired to simulate field conditions during the tests.

A series of tests were performed varying the consolidation pressure on the soil around the model pile. The effects of changing the location of the pore pressure measurements was also investigated. Results of the tests when compared with field observations indicated: 1) An appropriate method for the measurement of induced pore water pressures around model piles in the laboratory has been developed and 2) Laboratory tests as described in this thesis can reasonably estimate pile behavior in the field.

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a^2	radial dimension of region of local shear failure.
K_0	the coefficient of earth pressure at rest.
K_1	permeability of region of local shear failure.
K_2	permeability of region of general shear failure.
k	dimensional constant
k_1	diffusivity of region of local shear failure.
Q	strength of instantaneous surface source; excess pore water pressure in region of local shear failure.
$\bar{t}$	time interval
U_1	excess pore water pressure in soil adjacent to the pile surface.
u	excess hydrostatic pressure; micron.
ΔU_a	the change in pore pressure due to change in ambient pressure.
ΔU_m	the total pore pressure.
ΔU_s	the change in pore pressure due to shearing strain.
$(\Delta U/p)_{max}$	the change in pore pressure divided by the consolidation pressure obtained from a conventional consolidated-undrained test.
σ'_{1i}	vertical effective stress.
ϕ_ρ	friction angle determined from pile test.

$\overline{\Phi}_p$	effective friction angle determined from pile test.
ϕ_{cu}	friction angle determined from consolidated-undrained test.
$\overline{\Phi}_{cu}$	effective friction angle determined from consolidated-undrained test.
$W_{i-b}\%$	the difference in water content of soil adjacent to the pile and the boundary soil.

I. INTRODUCTION

A. General

Many field and laboratory tests have been made to analyze the amount of disturbance induced when a pile moves through the soil. One source of disturbance occurring in the field is an increase in pore water pressure due to the pile remolding the adjacent soil. Although pore pressures have been measured many times in the field, little attempt has been made to measure pore water pressures that occur around a miniature pile under laboratory conditions.

It is the purpose of this investigation to set up an initial testing procedure in the laboratory to measure pore water pressures around a miniature friction pile and to compare the results with data that has been obtained from the field.

B. Research Program

The research program for this thesis followed the steps listed below:

1. A review of literature was made to determine all information pertaining to the research program.
2. An appropriate soil was designed and prepared for the testing program.
3. The physical properties of the soil were evaluated.
4. Samples of fully saturated soil were prepared in a sedimentation device.

5. Procedures for measuring pore water pressures at designated distances from the pile were tried until an appropriate method was found.
6. Tests were made and results were correlated with data obtained in the field.
7. From the correlations and comparisons, conclusions were drawn and recommendations were given.

II. REVIEW OF LITERATURE

A. Pore Water Pressures vs. Pile-Soil Interaction

In early investigations of piles driven into clay soils, it was noted that the soil was remolded during the driving operations and the bearing capacity of the pile increased with time. The attribution and correlation of this increased load carrying capacity to changes in hydrostatic pressures were not published until the early 1950's.

Zeevart (1950) and Krynine (1950) noted that an increase in pore water pressure could cause a flow of moisture away from the pile and thus cause an increase in shear strength due to consolidation of soil around the pile. No attempt was made, however, to measure or calculate the extent and influence of this phenomena.

Reese and Seed (1955) used Terzaghi's (1943) applications of heat diffusion for one dimensional consolidation, and applied the principle to dissipation of pore water pressure from the surface of a pile. Using the solution of Carslaw and Jaeger (1947) the equation was presented as:

$$u = Q / 4 k t \pi$$

where:

u = excess hydrostatic pressure

Q = strength of instantaneous surface source

k = dimensional constant

t = time interval

The above equation applies to pore pressures occurring close to the pile during the latter part of the dissipation process, and is independent of the dimension of the pile and the location of the instantaneous source.

Seed and Reese (1955) and (1957) drove an instrumented pile into San Francisco "Bay Mud" and measurements of different pore water pressures around the pile were taken. They concluded that there were two causes of pressure changes: first, there was an increase in pore pressures due to remolding of the soil, and second, the pore pressure increased due to the pile displacing the soil during driving. It was proposed that the rate of increase in bearing capacity of the pile was directly related to the dissipation of pore pressures in the soil near the pile and could be a measure of the time required for the pile to obtain its maximum bearing capacity.

Seed and Reese also noted that the pile gained more than five times its initial bearing capacity with time. During the driving operation, organic, silty clay lost 70% of the strength differential that would have occurred had the soil been completely remolded. After driving was completed the soil reconsolidated and had a measured shear strength 60% higher than in the undisturbed state. They noted, however, that not every clay soil would behave in exactly the same manner, even if the same type of pile were used.

Bjerrum and Johannessen (1960) made field measurements of pore water pressures which developed when two bridge abutments were placed in a soft, marine clay, and they discussed the effect that the pore pressures had on the stability of a nearby slope. It was stated:

> "If piles are driven for an abutment close to a slope, the temporary rise in pore pressure is of course accompanied by a corresponding reduction in effective stress and thus by a reduction in shear strength. During the process of dissipation of the pore pressure...there will be a critical time...when the safety factor reaches a minimum value...The minimum value of the safety factor cannot be predicted, due to the pressent lack of knowledge on pore pressures and their redistribution."

It was observed from the field measurements that very high pore pressures, which were equal to or exceeding the total overburden pressure, were set up by driving a pile into the soil. These high pressures dissipated rapidly at first then proceeded at a slower rate, and 60% of the total excess pressure was dissipated in one year. It was observed that increases in pore pressures at distances farther than fifteen times the pile diameter were small. It was again stated that the type of clay and dimension of the pile had an effect on the amount of disturbance caused by pile driving.

Soderberg (1962) followed Terzaghi's (1943) theory of consolidation to derive dissipation analysis curves in accordance with two assumptions on the behavior of soil. When driving commenced he assumed that soil acted as an elastic-plastic material, while at the completion of driving the pile, it acted as a viscous liquid which would not support tension. The curves developed were determined by the dimension of the pile and the coefficient of consolidation of the soil. See Figure (1).

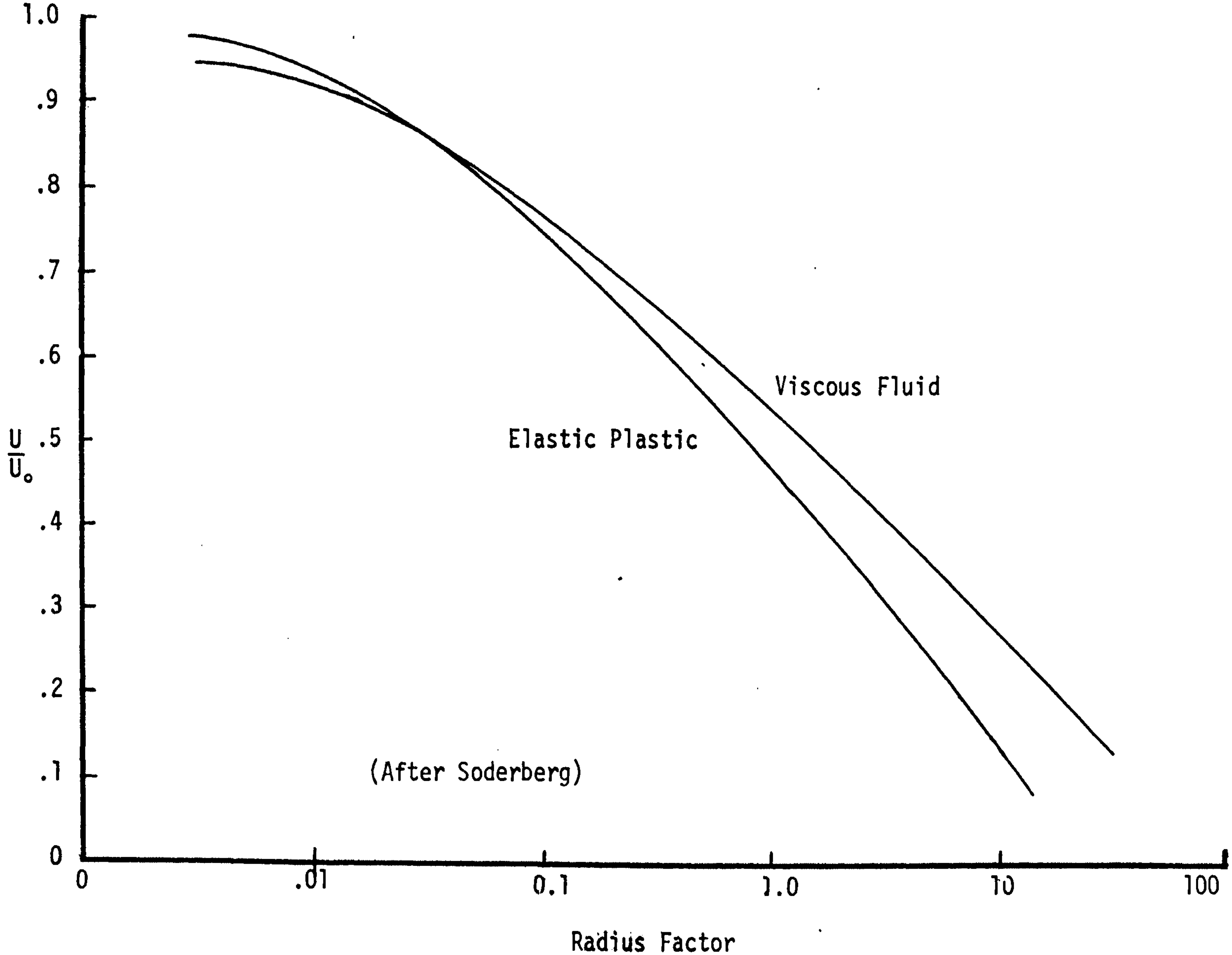

Figure (1). Initial Pore Pressure Patterns

Lambe and Horne (1965) conducted field measurements of the normally consolidated "Boston blue clay" into which concrete piles were driven. It was reported that very high pore pressures developed due to piles being driven into the soil below pre-augered holes. Measurements taken in the pre-augered zone gave pressures of about one third of those in the augered zone. Lambe and Horne observed that the rate of pore pressure build up and dissipation proceeded very rapidly. Although the highest pressures were recorded close to the piles, significant pore pressures were measured as far as 100 feet away from the pile driving operation.

Lo and Stermac (1965) presented a theoretical equation for estimating the maximum pore pressures developed during the driving of a pile. They submitted that the induced pore pressure was composed of two parts; one resulting from the change in *ambient pressure and the other induced by shearing strain. The two equations take the following form:

$$\Delta U_a = (1-K_o)\ \sigma'_{1i}$$

$$\Delta U_s = (\Delta U/p)_{max}\ \sigma'_{1i}$$

where:

ΔU_a = the change in pore pressure due to change in ambient pressure.

ΔU_s = the change in pore pressure due to shearing strain.

K_o = the coefficient of earth pressure at rest.

σ'_{1i} = vertical effective stress.

* Ambient pressure is defined as the initial stresses in the ground.

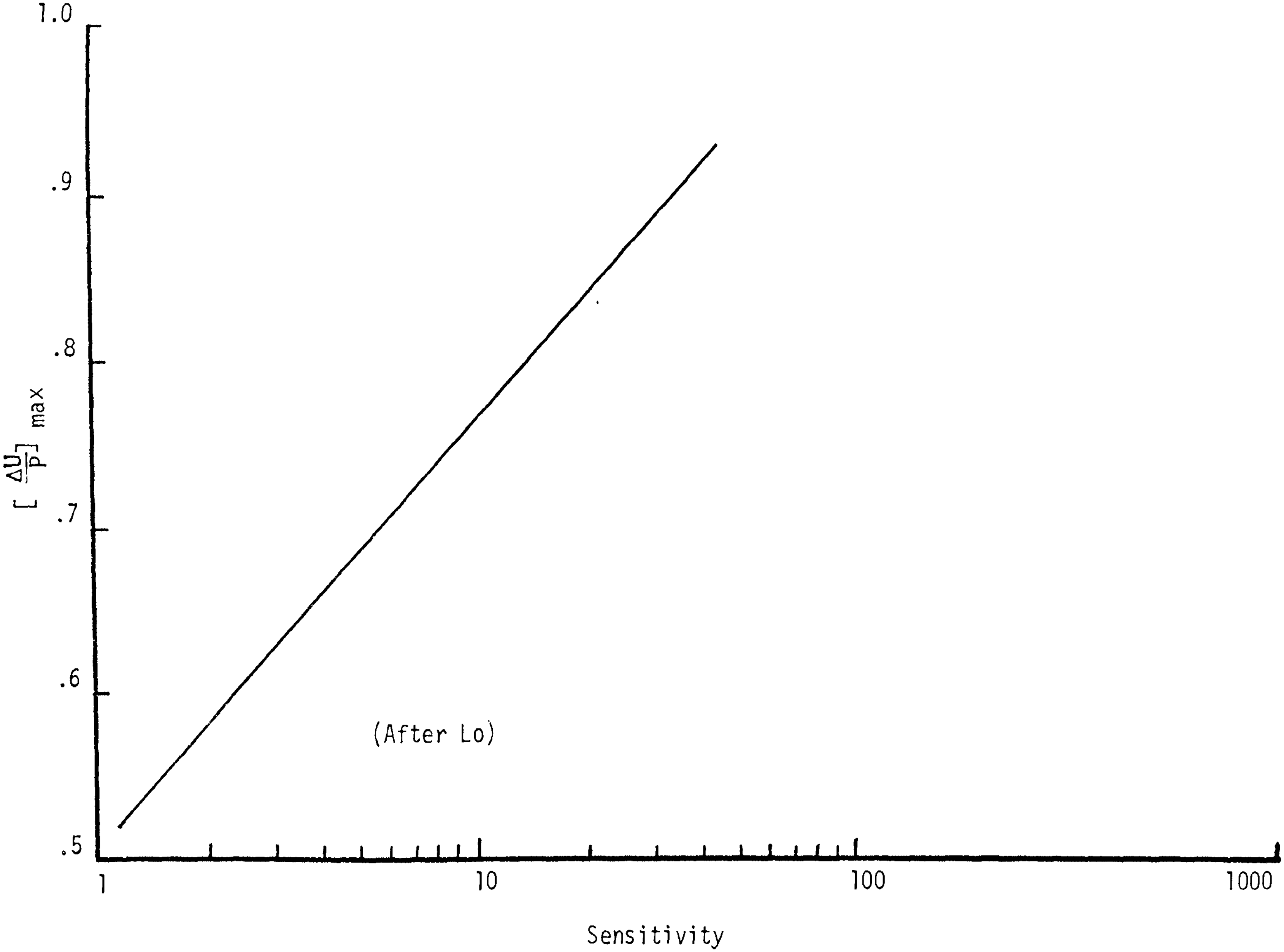

Figure (2). Relation of Maximum Pore Pressure Ratio to Sensitivity

$(\Delta U/p)max$ = change in pore pressure divided by the consolidation pressure obtained from a conventional consolidated-undrained test.

The total pore pressure, ΔU_m, was the summation of ΔU_a and ΔU_s or:

$$\Delta U_m = [(1-K_o)+(\Delta U/p)_{max}]\sigma'_{1i}$$

The proposed equation is dependent on the stress history and pore pressure characteristics of the clay and independent of the dimensions of the pile. Lo (1968) also submitted that the pore pressure ratio, $(\Delta U/p)_{max}$, was directly proportional to the logarithm of the sensitivity of a normally consolidated clay. See Figure (2).

In the analysis of data obtained from field measurements of normally consolidated clays and silts, Lo and Stermac (1965) observed that induced pore pressures were maximum and constant within the failure zone close to the pile and increased with depth. At a distance of approximately sixteen pile diameters from the pile, the change in pressure was considered negligible. It was stated that driving of adjacent piles slightly increased the induced pressures within the failure zone, while those measured outside the failure zone were the direct summation of pressures caused by the driving of adjacent piles until a maximum value, equivalent to that in the failure zone, was reached.

Orrje and Broms (1967) taking field measurements of sensitive, normally consolidated clays in Sweden, observed that pile driving induced pore pressures that exceeded existing total overburden pressures by as much as 20%. In the cases sited, 65% to 85% of the induced pressure dissipated within 24 hours after the piles were driven. The remaining percentage of excess pressure dissipated rather slowly and was attributed to the reconsolidation and lowering of water content of the soil mass. It was again noted that the induced excess pressure increased with depth.

Airhart, Hirch and Coyle (1967) made extensive tests with a full scale instrumented pile in the field. Pore water pressures were measured during static loading tests as well as during the pile driving operation. Airhart, et al. (1967) stated that during static loading tests, pore water pressures should increase by a small amount, (compared to those induced by driving the pile), due to the elastic strains in the soil adjacent to the pile. When the ultimate bearing capacity of the pile is reached, the downward movement of the pile would produce a less dense arrangement of the soil particles, thus causing the pore pressures to reduce. When the load is removed, the soil particles settle into a more dense arrangement than could be achieved during the pile movement causing a rise in pore pressures that could be higher than those prior to loading.

The achievement of a more dense arrangement, thus an increase in shear strength of the soil tends to verify the observations of Dubose (1957) who noted an increase in load carrying capacity with successive retests of the same pile.

Airhart, et al. (1967) proposed an equation for the excess pore water pressure involving both the properties of the soil and the pile. The equation is given as:

$$U_1 = \frac{Q}{4}\frac{K_1 a^2}{K_2 k_1 t}$$

where:

Q = excess pore water pressure in region of local shear failure.

U_1 = excess pore water pressure in soil adjacent to the pile surface.

K_1 = permeability of region of local shear failure.

K_2 = permeability of region of general shear failure.

k_1 = diffusivity of region of local shear failure.

t = time.

a^2 = radial dimension of region of local shear failure.

In summary, investigators have illustrated that very high pore pressures can be developed by a pile being driven into soil. These pressures have been directly related to the changes in shear strength of the soil and the increase in bearing capacity of the pile with time and equations have been theorized to calculate these pressures. Excess pore pressures, due to pile driving, have been observed to exceed the total overburden pressure.

Investigators have stated that the amount of pore pressure induced is dependent on the physical properties of the soil, the location at which the pressure is measured, and the type and dimension of the pile.

III. THE RESEARCH PROGRAM

A. Research Procedure

The main objective of the research program was to consolidate a homogeneous, saturated sample of soil, 4 inches in diameter, around a model friction pile 3/4 inch in diameter and measure pore water pressures that developed due to loading the pile to failure. Measurements were taken at different distances from the pile surface and using different consolidation pressures.

Since the measurement of pore water pressures around a model friction pile had not been undertaken in the laboratory before the time of this research program, many methods and procedures were used on a trial and error basis before adequate results were obtained.

B. Soil Preparation

Since the measurement of pore water pressures was the primary concern in this investigation, it was desired to use a sensitive soil with a high silt content so that high pore water pressures would develope with shear strain. In preliminary testing, a mixture of 20% clay and 80%silt was used, but it was found to be so sensitive that even with the most careful of handling considerable disturbance was noticeable. Further testing showed that a mixture of 30% clay and 70% silt was stiff enough to be workable, but still had sensitivity high enough to create large pore pressures.

The clay material was a refractory kaolinite that was obtained from a kaolin mine located in Kentucky. X-ray defraction of the material indicated that it was primarily kaolinite with a small amount of montmorillinite clay minerals. A hydrometer analysis was performed on the material, and the results obtained showed the gradation to be approximately 80% clay size and 20% silt size particles with 100% of the material passing the No. 200 sieve. The grain size distribution of the clay material is given in Figure (3). The liquid limit was found to be about 64%; the plastic limit, 33%; and its specific gravity was 2.59.

The silt portion of the research soil was obtained from wind blown deposits on the east bluffs of the Mississippi River in St. Clair County, Illinois. It is classified as Roxana II loess series which was deposited during the previous glacial period. A grain size analysis of the material showed it to be about 10% clay, 87% silt, and 3% retained on the No. 200 sieve.

In order to obtain the silt portion of the loessial material, a sedimentation tank process was used to separate the silt sized particles from the clay sized particles.

To do this a 95 gallon stock watering tank was used as a basin to mix approximately 90 pounds of the loess with enough water to fill the tank 17 inches deep. Approximately 500 grams of sodium hexametaphosphate were added to the mixture so that the clay particles would be dispersed and stay in suspension. The mixture was agitated by a water jet and small shovel while the tank was filling. The rate at which the soil particles fall through the water, according to Stoke's Law, is related to the size of the particles. After the soil and water were mix-

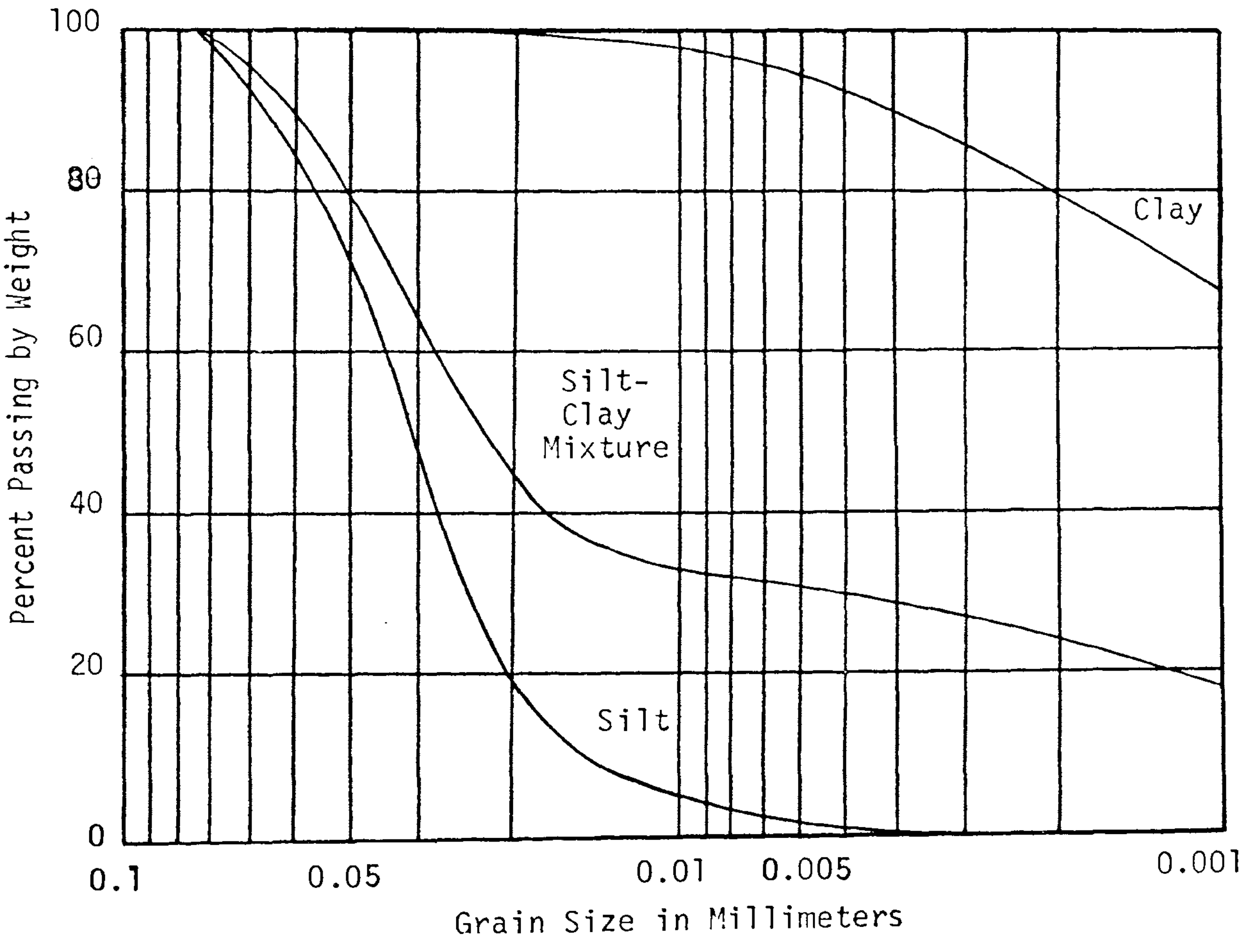

Figure (3). Gradation Curve for the Research Soils.

ed thoroughly, a computed time was allowed for the silt particles to fall below a specified point. The water containing the clay size particles was siphoned out of the tank and the washing process was then repeated. During each washing process, a sample of the mixture was taken with a 1000 milliliter graduated cylinder so that the progress of the clay removal could be visually observed. A hydrometer was placed into the graduated cylinder and, after approximately 10 washing and siphoning cycles, the hydrometer readings were relatively constant. The process was then continued for about 5 more cycles without adding the sodium hexametaphosphate so that it could be removed from the mixture. The material was then oven dried and passed through a No. 200 sieve to remove the sand.

A hydrometer analysis was made on the silt and the gradation curve is shown in Figure (3). Atterberg Limits were not run on the silt since it was cohesionless. The specific gravity was found to be 2.70.

C. Sample Preparation

For the type of research to be performed it was desirable to have a homogeneous soil with a B *coefficient of 1. Subsequently, samples were made by a sedimentation unit which was designed to produce a saturated sample by the one dimensional consolidation of a soil slurry.

The sedimentation unit consisted of a 4 inch I. D. lucite tube

* B coefficient is the ratio of the change in pore water pressure to the change in confining pressure. Skempton (1954)

that fit into a base plate containing a porous stone and a drainage system. The tube was secured by a top plate which was connected to the base by two 3/8 inch threaded steel rods. A piston, which fit inside the tube, had drainage holes connected to a porous stone and was used to apply the consolidation pressure. A load of 300 pounds, applied by a lever system, was transferred to the piston by a 1/2 inch steel rod. The rod was maintained steady by a guide cap which fit in the top plate. Filter paper was placed on the porous stones to prevent clogging of the stones with clay size particles. Schematics of the unit are shown in Figures (4) and (5).

The air dry mixture of 30% clay and 70% silt was placed in a metal container where it was mixed thoroughly. 1900 grams of the material produced the desired height of consolidated soil and was mixed with approximately 800 milliliters of distilled, deaired water. The resulting slurry was thin enough so that it would pour easily into the sedimentation tube. An o-ring, slightly larger than the inside diameter of the tube fit between the piston and the slurry so that no soil could extrude between the piston and the tube. The piston was pushed through the tube until it was in contact with the o-ring and soil. The load was then applied, and the sample allowed to consolidate for three days. Water contents taken at the top, middle and bottom showed a variance of 1% which was tolerable, Olson (1962). Samples were also checked for segregation by hydrometer analysis, and it was found to be negligible.

The liquid limit of the silt-clay mixture was 22%; the plastic limit was 15%; and the specific gravity was 2.67.

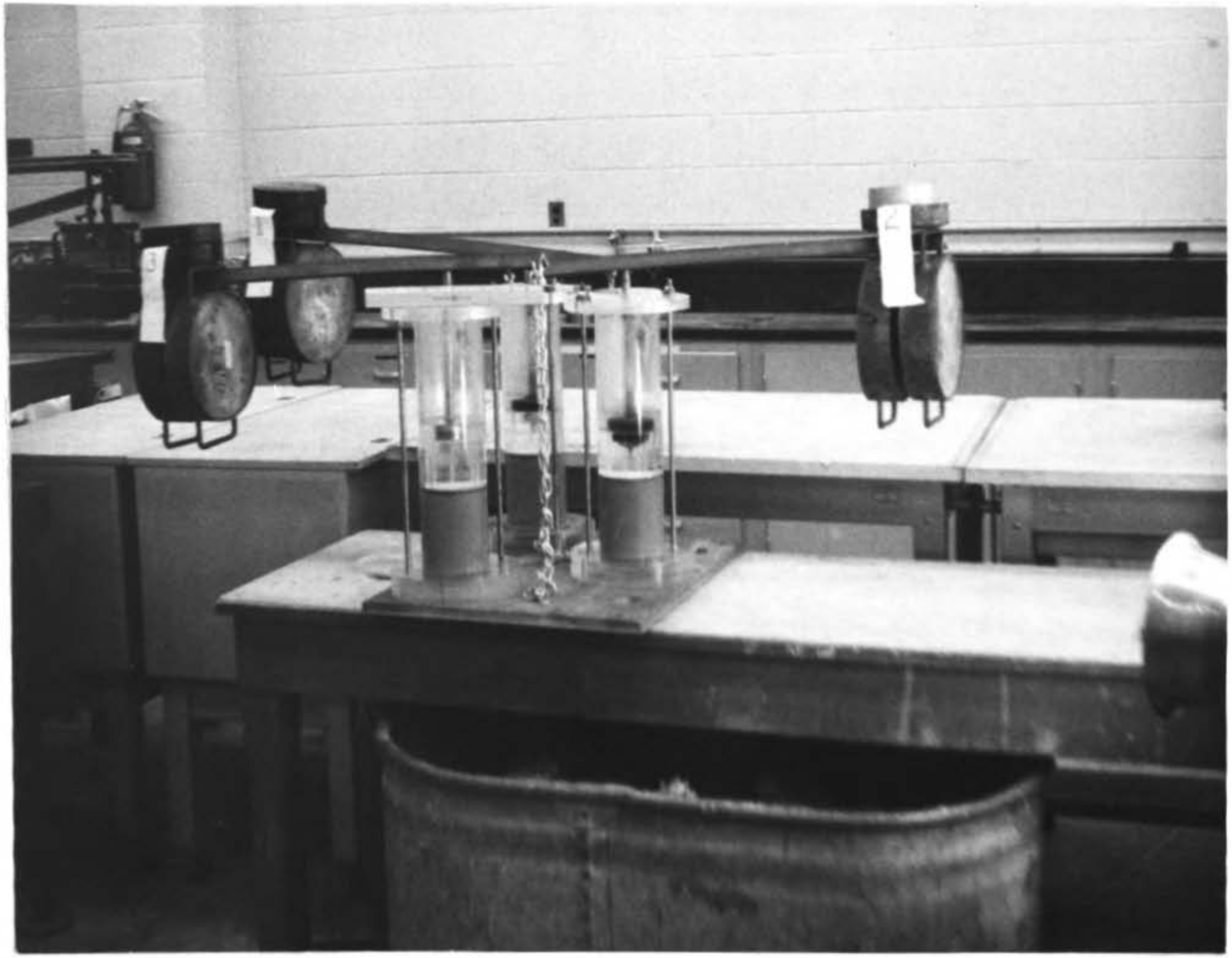

Figure (4). Assembled Sedimentation Unit.

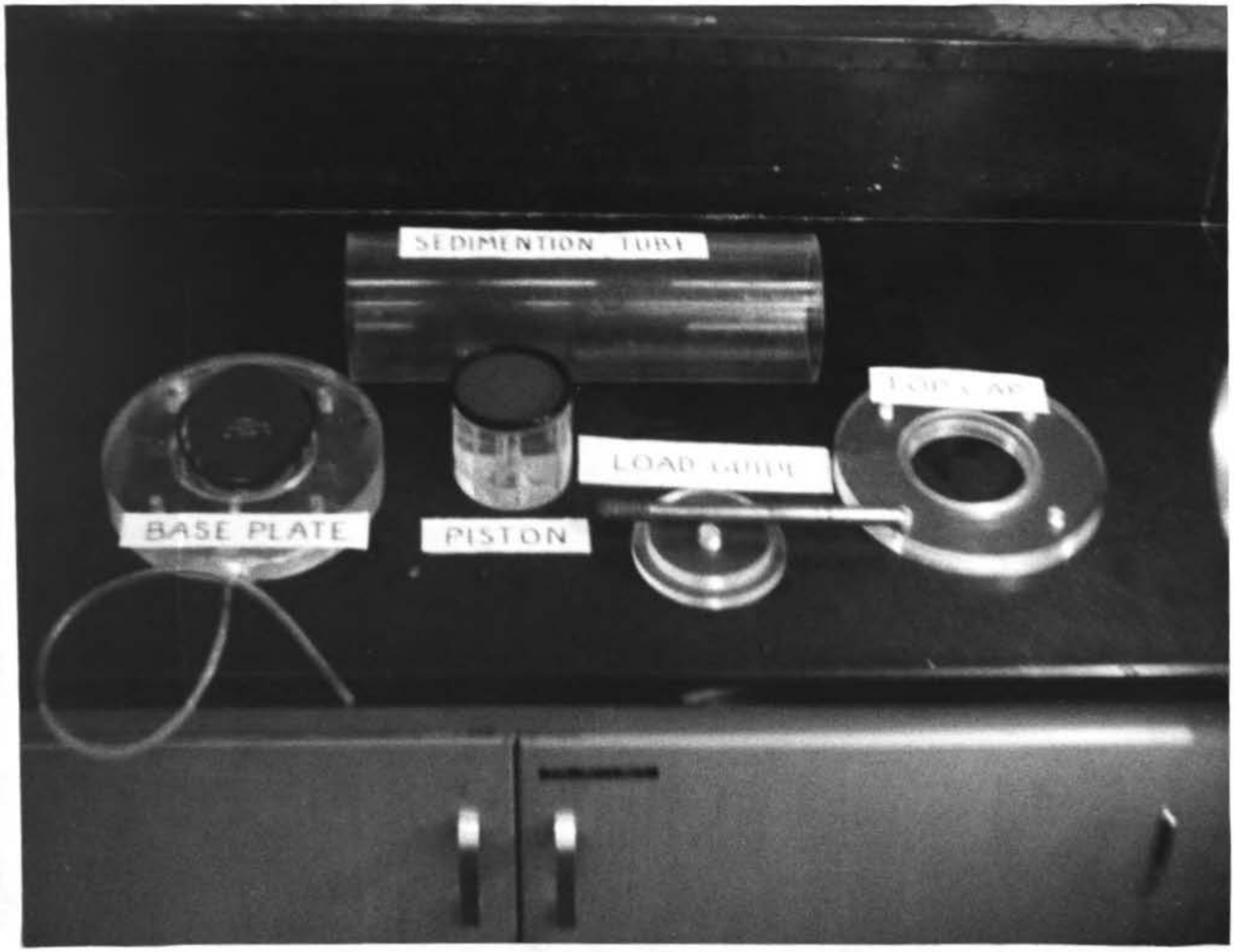

Figure (5). Disassembled Sedimentation Unit.

Table I. shows the physical properties of the soil.

D. Testing Equipment

The most important part of this research was the developement of equipment that would accurately measure small changes in pore water pressure at any point inside a soil sample. A pressure transducer, C.E.C. No. 4-312-0001, was available for this research and was used to measure the pore water pressure.

A hypodermic needle, 1/16 of an inch in diameter was used as a probe which, when saturated with deaired water, would transfer water pressure from any point inside the sample to the transducer. Since it was extremely difficult to completely deair the needle, soil migration into the needle due to the change in volume of air bubbles under pressure occurred. This migration would cause clogging of the needle and consequently a large time lag for response of the transducer to differential pore pressures. Different methods for eliminating the clogging were examined. In all cases, a pilot hole was made using a similar needle with a sharp point. In one attempt a porous stone was ground to fit into the end of the needle, saturated and inserted into the pilot hole. This proved to be unsatisfactory since a "skin" of clay would form on the stone and produce a large response time.

A rubber membrane was placed over the end of the needle and allowed to deflect with changes of pressure. This method was inadequate, though, because volume changes that occurred would break the membrane. Also it was very difficult to attach the membrane to the needle so that

TABLE I.

Physical Properties of the Research Soil

Liquid Limit	22%
Plastic Limit	15%
Plasticity Index	7%
Specific Gravity	2.67
% Clay Fraction Less Than 2u . . .	24%
A.A.S.H.O. Classification	A-4(8)
Unified Classification.	CL-ML

there would be no leaks. The method that was finally adopted and proved to be quite successful was the use of a No. 200 mesh screen placed in the end of the needle. The system was saturated with deaired water, frozen, and inserted into the sample. No clogging would occur since a structure of silt would form on the outside of the screen while clay particles passing through into the needle would stay in suspension during the testing.

Since the sample was consolidated and tested inside a triaxial testing chamber, transferring the pore water pressure from the needle, inside the sample, to the transducer on the outside of the cell was also a problem. The probe was soldered to a copper elbow which in turn was connected to the base of the cell by a 1/8 inch Saran tube. The transducer housing was in turn connected to the base of the cell opposite the needle and separated by a no volume change Klinger valve.

In order to make measurements of pore water pressures developed by the skin friction of the pile, the end of the pile could not be in the sample. Since the soil was to be consolidated around the pile, special equipment had to be developed for this purpose. Two plastic caps, 1 1/2 inches thick and 4 inches in diameter were constructed. Both caps had holes drilled through their centers which were slightly larger than the diameter of the pile. The center hole of the top cap was rounded on one side. Six 1/8 inch drainage holes were drilled 60 degrees apart and approximately 1 inch from the center in the bottom cap, to allow drainage of water from the sample during consolidation.

The model pile used was a 7 3/4 inch aluminum rod that was roughened over its entire length. One end was tapered while the other end was flat with a 3/16 inch drilled and tapped hole. In order to seal the hole in the top cap, a thick rubber membrane, 6 inches in diameter, was connected to the top of the pile by rubber cement and strong rubber bands.

The triaxial cell with a capacity to test 4 inch diameter samples was obtained from the Wykeham-Farrance Company, England. The loading piston was modified for this research by attaching a 3/16 inch thread rod to the lower end so that the pile and the piston could be joined.

Load was applied by a Farnell Testing Machine. Since it was a constant strain rate device, a proving ring was used to measure loads. Deflection measurements were made by attaching SR-4 strain gages to the sides of the proving ring, connecting them into a Wheatstone Bridge circuit and applying the circuit to a strain indicator.

E. Testing Procedure

The testing procedure outlined below was found to be the best sequence, and was the one followed in this research.

1. The probe was saturated with water by inverting the base plate, forcing deaired water through the probe and tightening it to the base plate. A saturated piece of plastic tubing was then connected to the end of the needle to keep it saturated while other work was being done. It should be noted that the space from the probe to the transducer had been saturated prior to this time.

2. The sedimentation unit was disassembled and the piston was extracted and placed into water so that the porous stone would remain saturated.
3. The filter papers were removed from both ends and the bottom of the sample was trimmed flat to the end of the tube.
4. The top cap and then the bottom cap were placed inside the tube on top of the sample. The caps were used as a guide for the sharpened thin walled steel pipe which was used to make the pilot hole in the soil.
5. The bottom cap was removed from the tube and placed on a saturated porous stone on the pedestal of the triaxial cell.
6. The pile was pushed through the pilot hole in the sample.
7. The sample was partially extruded by inverting the sedimentation tube and pushing on the top cap with a short piece of drill rod which fit around the pile and rubber membrane. The sample was then trimmed so that the remaining sample length was 4 1/2 inches.
8. Two pieces of filter paper, 4 inches in diameter, with a 3/4 of an inch diameter hole cut in the center of each one, were placed on the bottom of the sample around the projection of the pile.
9. The sample was extruded fully from the tube and placed on the bottom cap with the projection of the pile in the center hole.
10. A saturated paper towel was wrapped around the sample to accelerate the consolidation process.
11. A rubber membrane was placed around the sample, and secured by o-rings placed around the base plate pedestal and at the lower

part of the plastic top cap.

12. A vacuum was applied to the sample to hold it in place until a confining pressure could be applied to it.

13. The rubber membrane which was connected to the pile was stretched over the top of the top cap and secured to it by an o-ring.

14. The height at which the probe was to be inserted into the sample was measured and a small pilot hole was made with a hypodermic needle to the desired distance from the pile.

15. The piece of saturated plastic tubing was removed from the probe and dry ice was placed against the probe to freeze the water inside.

16. The probe was pushed into the pilot hole and secured in place by cementing the rubber membrane of the sample with silicone sealant.

17. Centering of the sample and the pile was accomplished by putting the top part of the triaxial cell in place over the sample and sighting through the hole for the piston at the top of the pile. The sample was moved at the bottom cap until centered.

18. The top of the triaxial cell was clamped to the base plate and modified piston was pushed through the opening in the top of the cell and fastened into the top of the pile while the cell was being filled with water.

19. An appropriate cell pressure and a 25 p.s.i. back pressure were applied to the sample and kept constant by an oil filled dashpot system.

20. The sample was allowed to consolidate for 24 hours or when the pore water pressure, which could be measured by the probe was equal to the back pressure.
21. The triaxial cell with the consolidated sample was set on the loading machine and all electrical and pressure connections were completed. A dial gage was set up to measure the deflection of the pile.
22. The loading machine was started with a constant strain rate of .002 inches per minute. This slow rate was chosen to simulate static loading of the pile.
23. Measurements were made of time, load, deflection of the pile and pore water pressures. Readings were taken at constant time increments.
24. Termination of the test was made when the load would start to decrease appreciably or when the test had run two hours.
25. After each test, water contents were taken at various places in the sample and the distance that the end of the probe was away from the pile was measured.

Fifteen tests were run using three different consolidation pressures and five distances that the end of the probe was from the side of the pile.

Consolidation pressures of 25, 41 and 55 p.s.i. were used to represent depths of 30, 45 and 60 feet of soil respectively.

The probe was inserted 0, 1/2, 1, 2 and 3 centimeters from the pile.

A skematic of the equipment set up and ready for testing is shown in Figure (6).

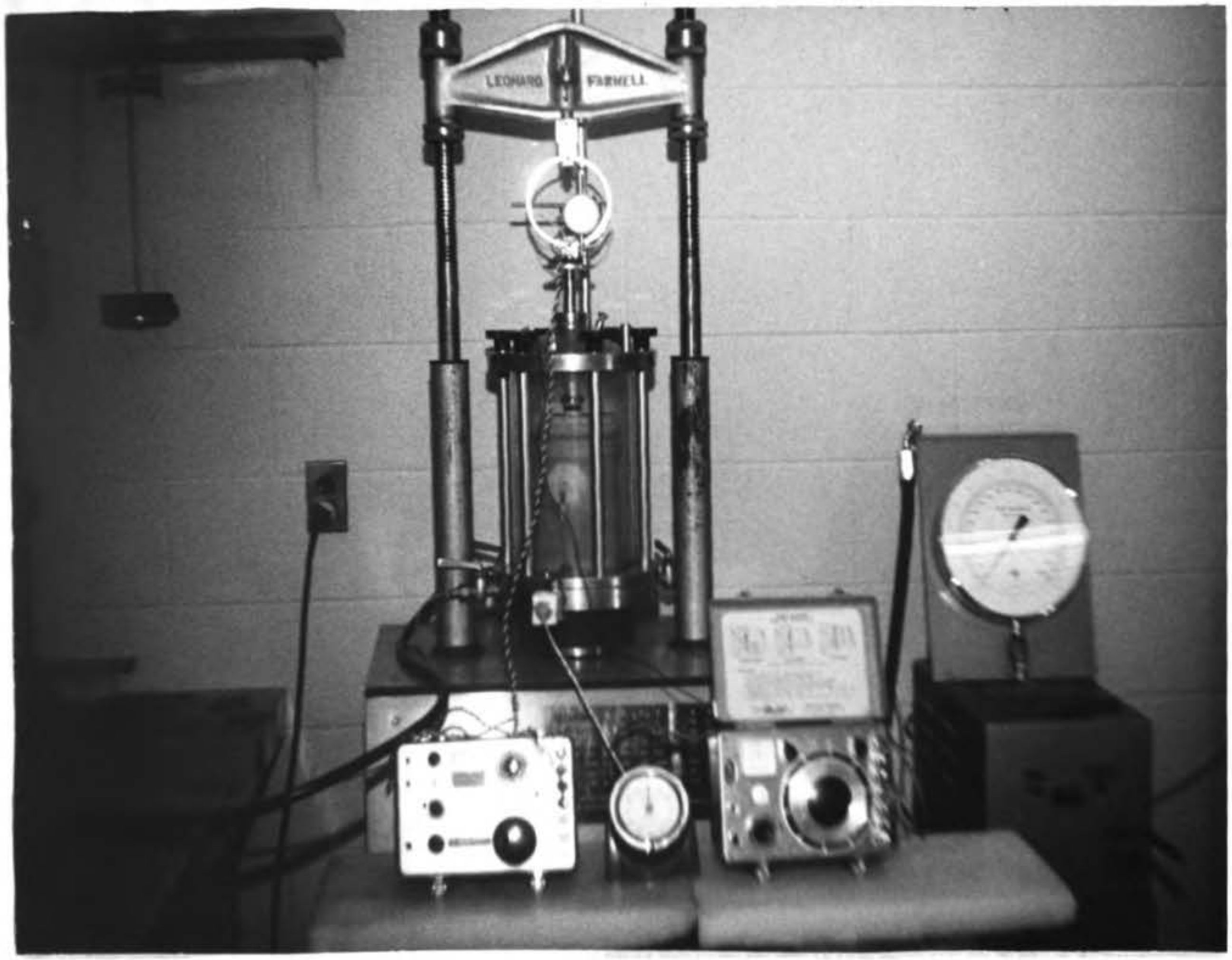

Figure (6). Equipment and Sample Ready for Testing.

IV. DISCUSSION OF RESULTS

A. Pore Pressure Changes

Recall Airhart, et al. presented data obtained from measurements of induced pore pressure during the static loading of an instrumented pile in a soft clay stratum. The characteristics of the induced pressures measured and the applied load versus time is shown in Figure (7).

Airhart indicated that the pore pressures increased until the ultimate load carrying capacity of the pile was reached. As the pile then plunged downward through the soil, the pore pressure decreased rapidly and ultimately became negative. When the load was released from the top of the pile, the pore pressure increased again to a value higher than that achieved during loading.

Figures (8) through (22) show the results obtained from laboratory tests where the load, induced pore pressure and deflection of the top of the pile are plotted versus time.

The pore pressures increased with load until failure occurred. Failure was considered to have taken place when the deflection curve reached a constant slope of .002 inches/minute, indicating that the pile was moving at a constant rate through the soil. As the failure transpired, the pore pressures first decreased rapidly and then proceeded more slowly to a constant value. In almost all cases failure resulted in a pore pressure decrease. When the load was removed it was observed that the pore pressure increased rapidly for a short time and then dissipated. No measurement of this phenomenon was made, but

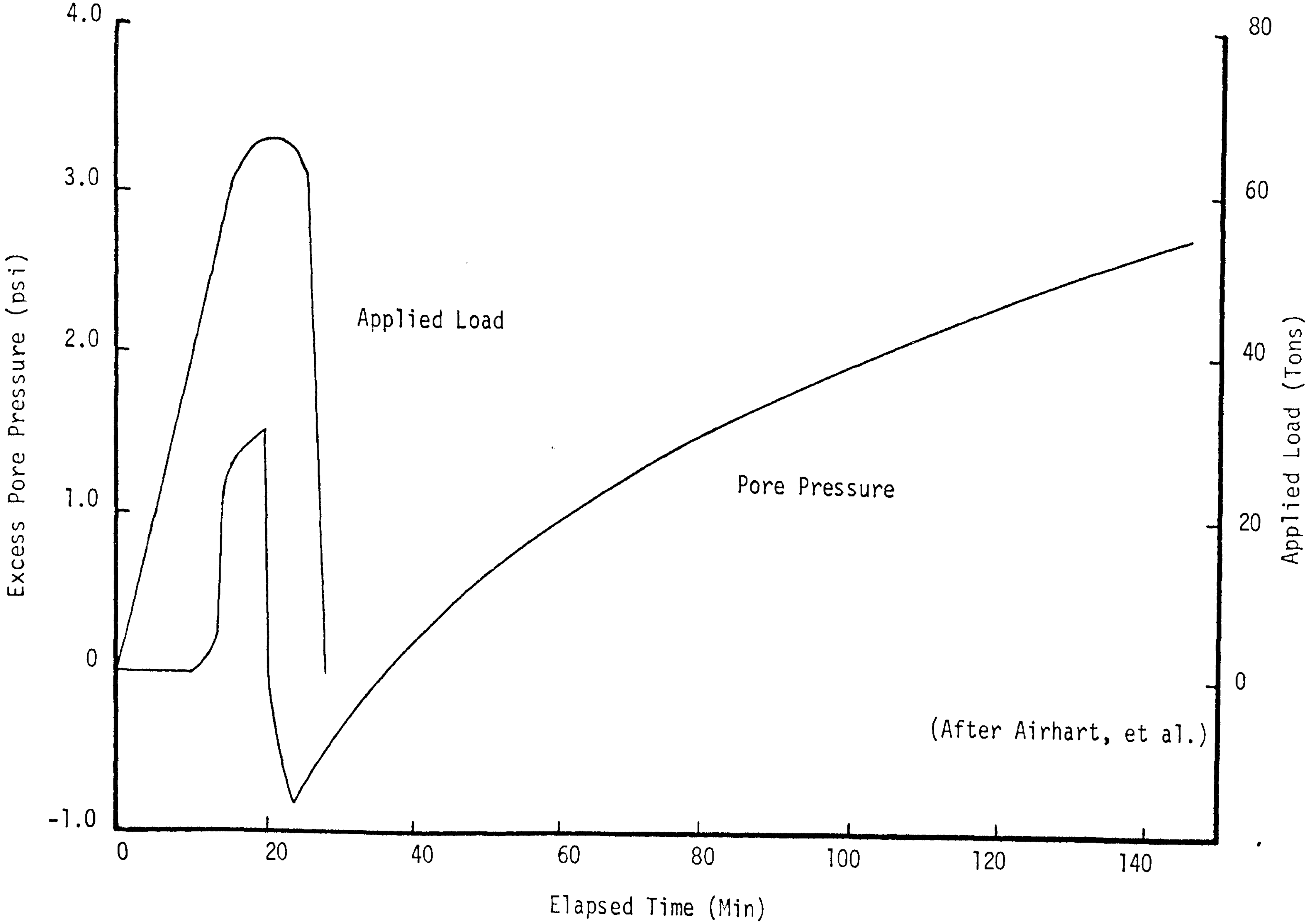

Figure (7). Excess Pore Pressure Due to Static Load Test

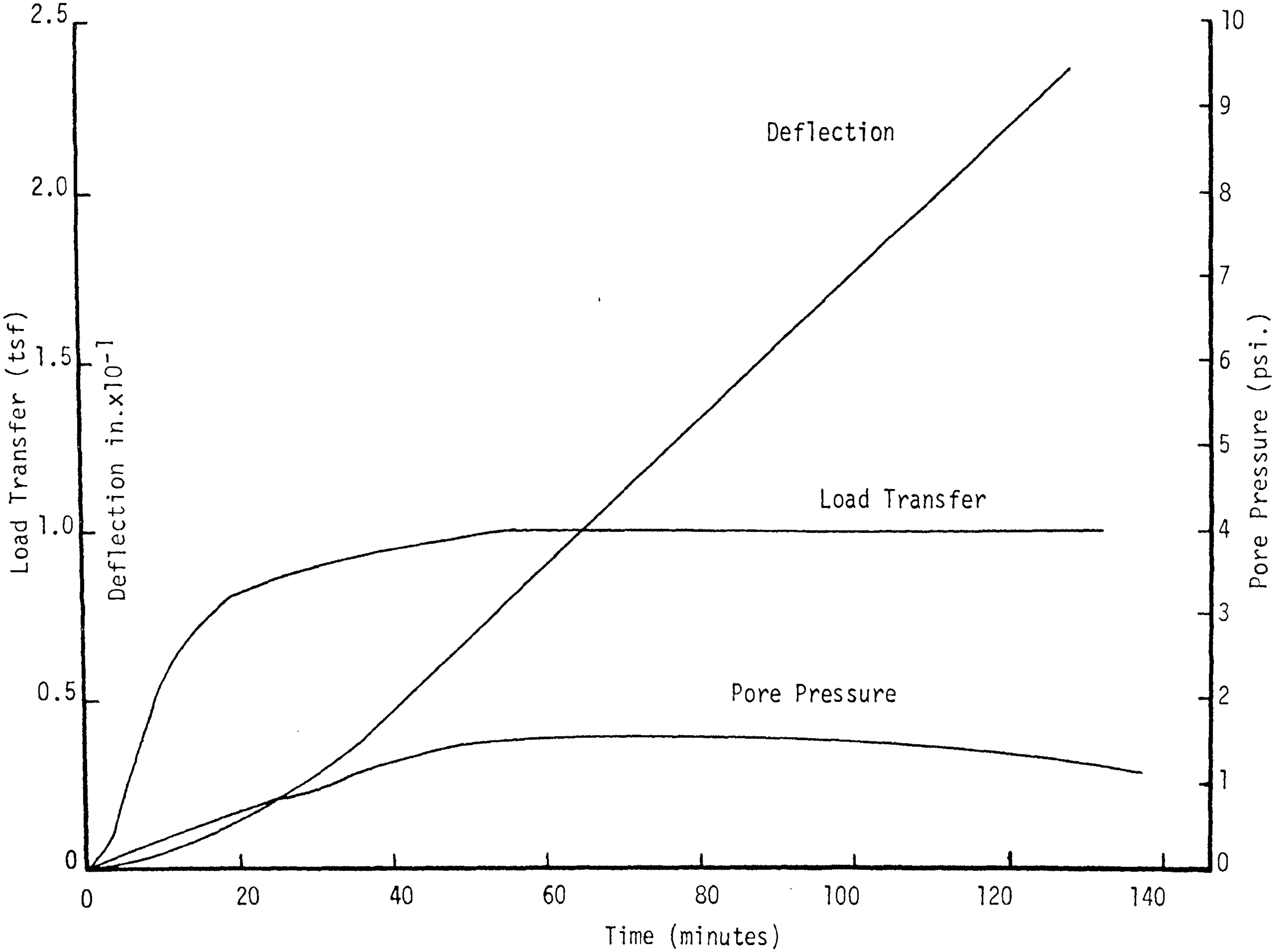

Figure (8). Test No. 1 The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 25 p.s.i. Consolidation Pressure with a Probe Distance of 0 cm.

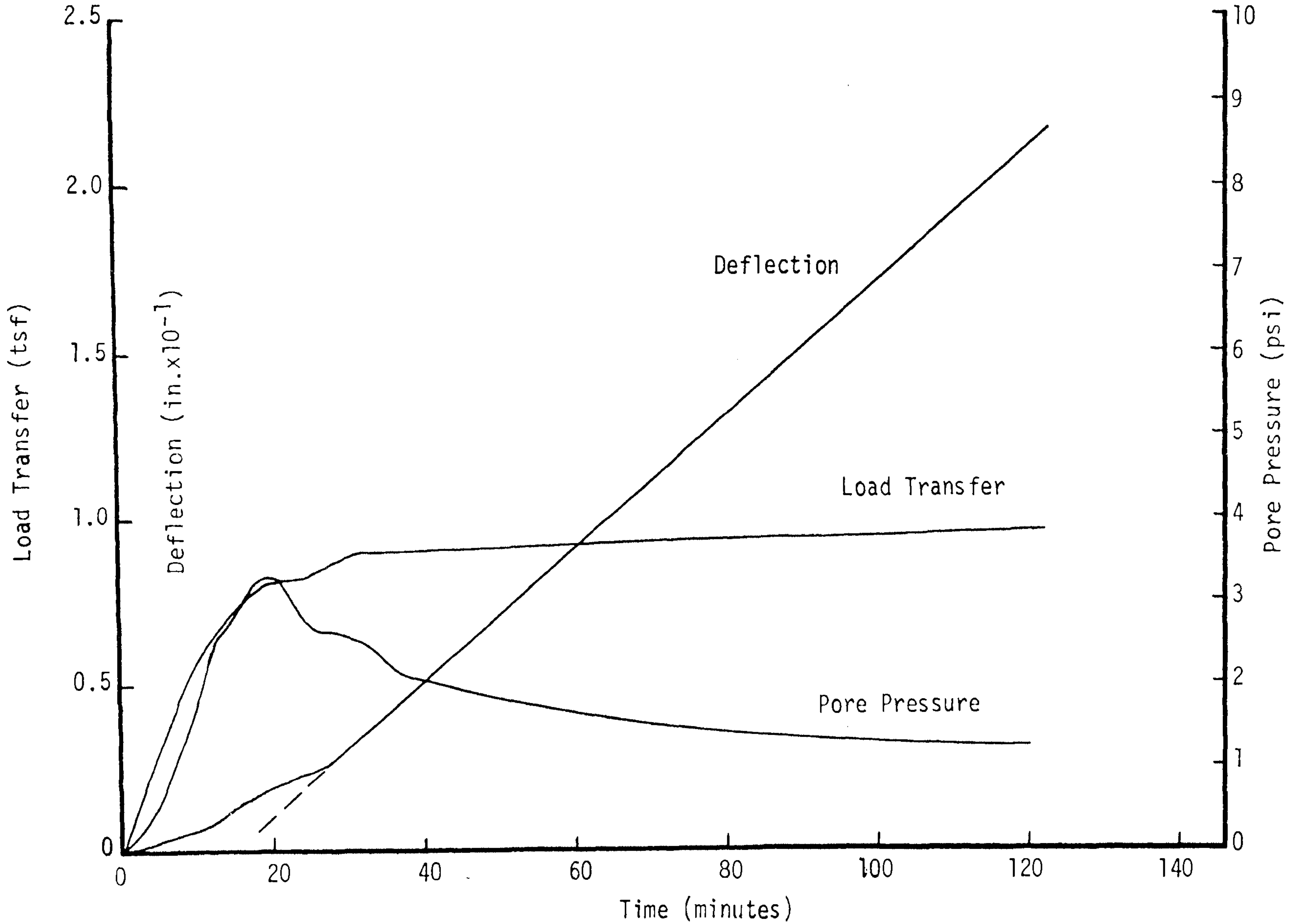

Figure (9). Test No. 2. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 25 p.s.i. Consolidation Pressure with a Probe Distance of 1/2 cm.

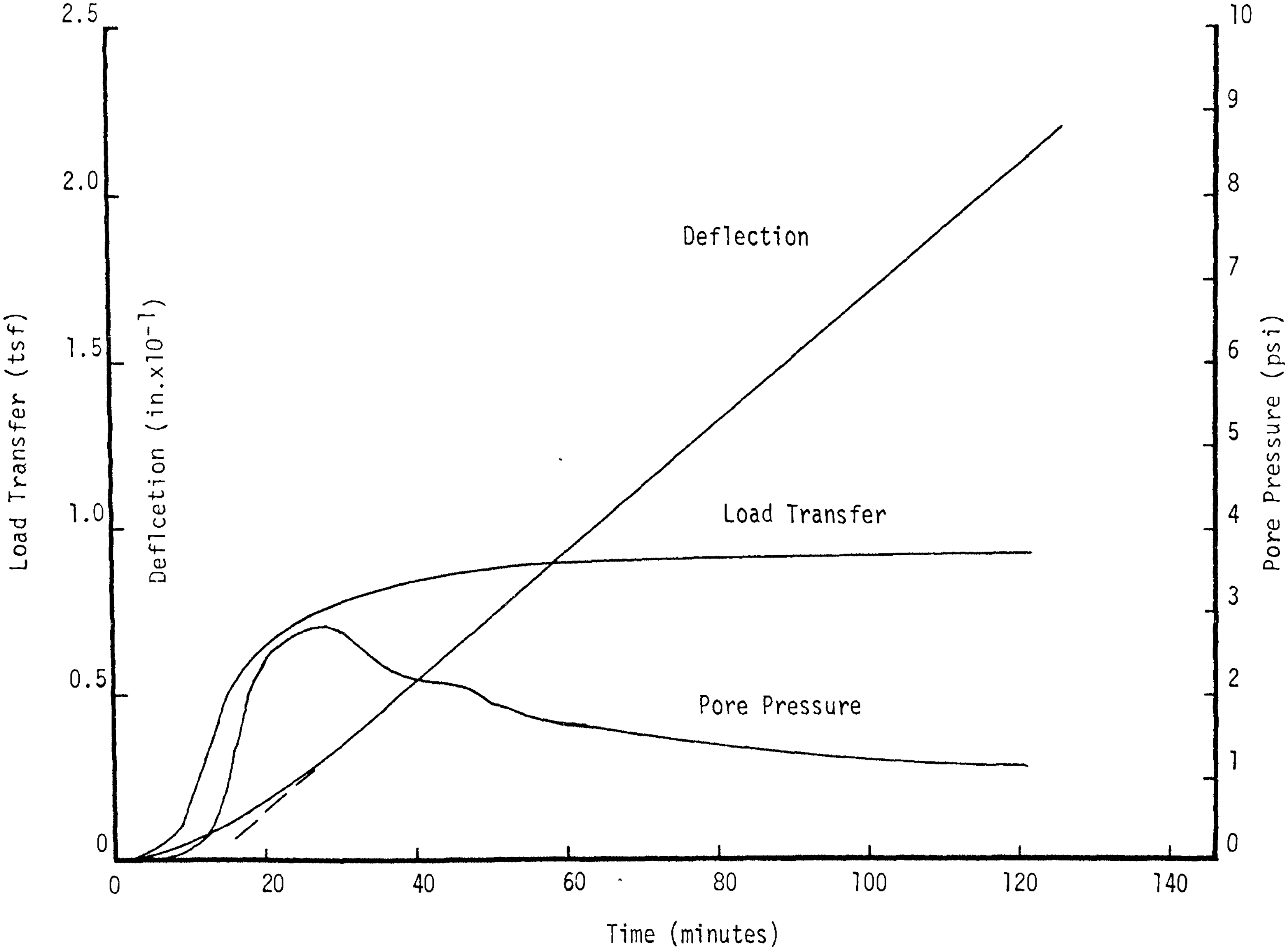

Figure (10). Test No. 3. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 25 p.s.i. Consolidation Pressure with a Probe Distance of 1 cm.

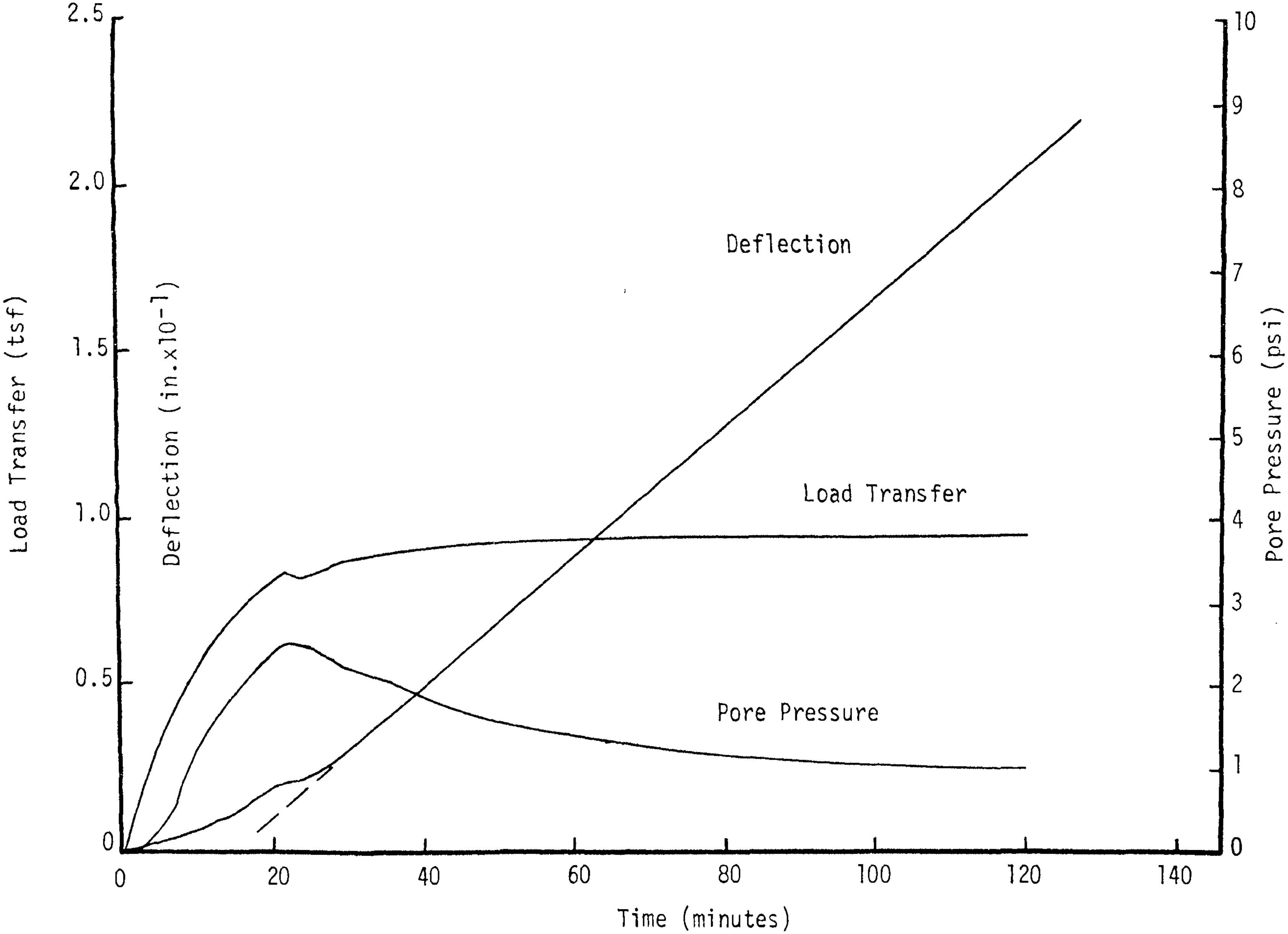

Figure (11). Test No. 4. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 25 p.s.i. Consolidation Pressure with a Probe Distance of 2 cm.

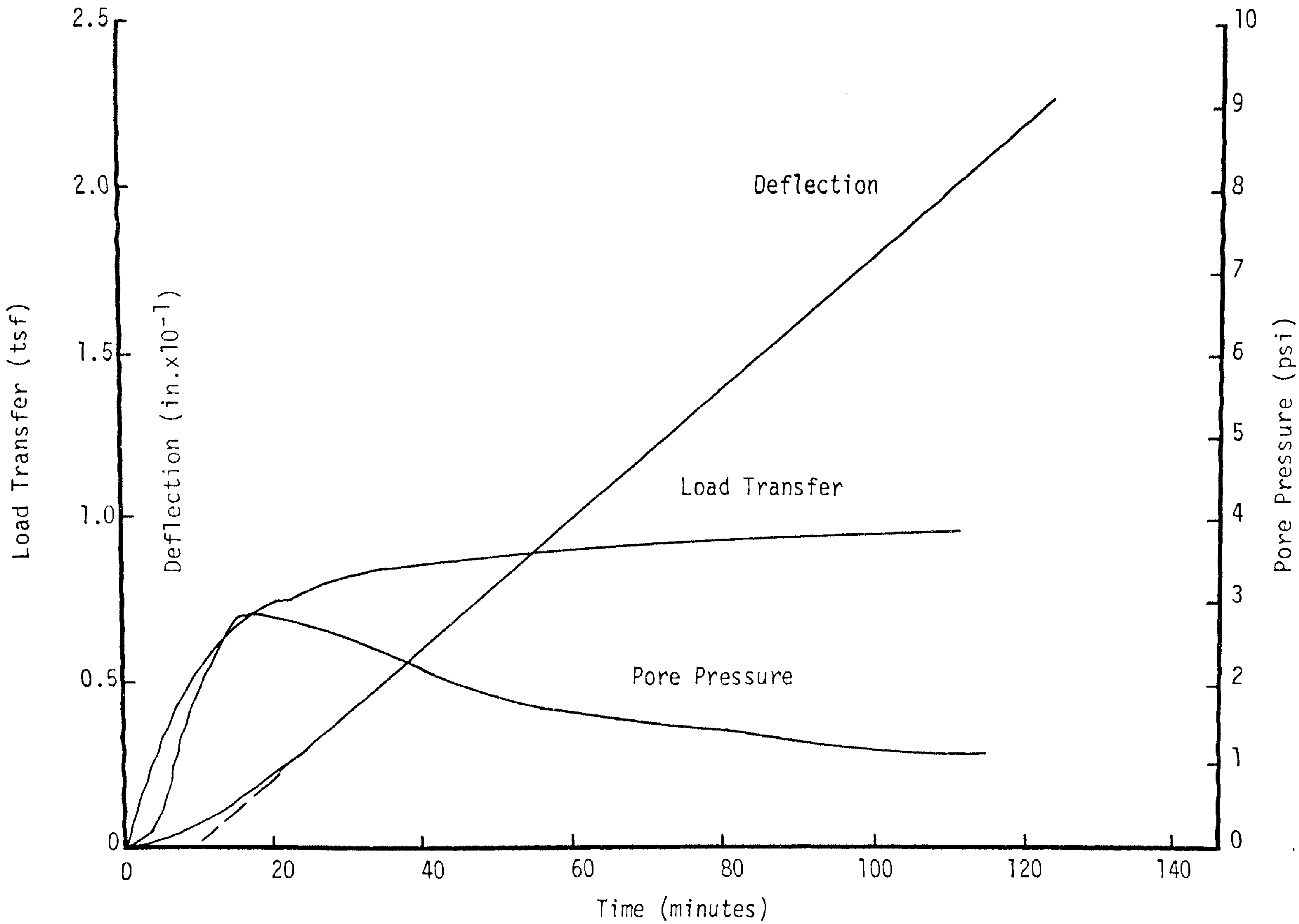

Figure (12). Test No. 5. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 25 p.s.i. Consolidation Pressure with a Probe Distance of 3 cm.

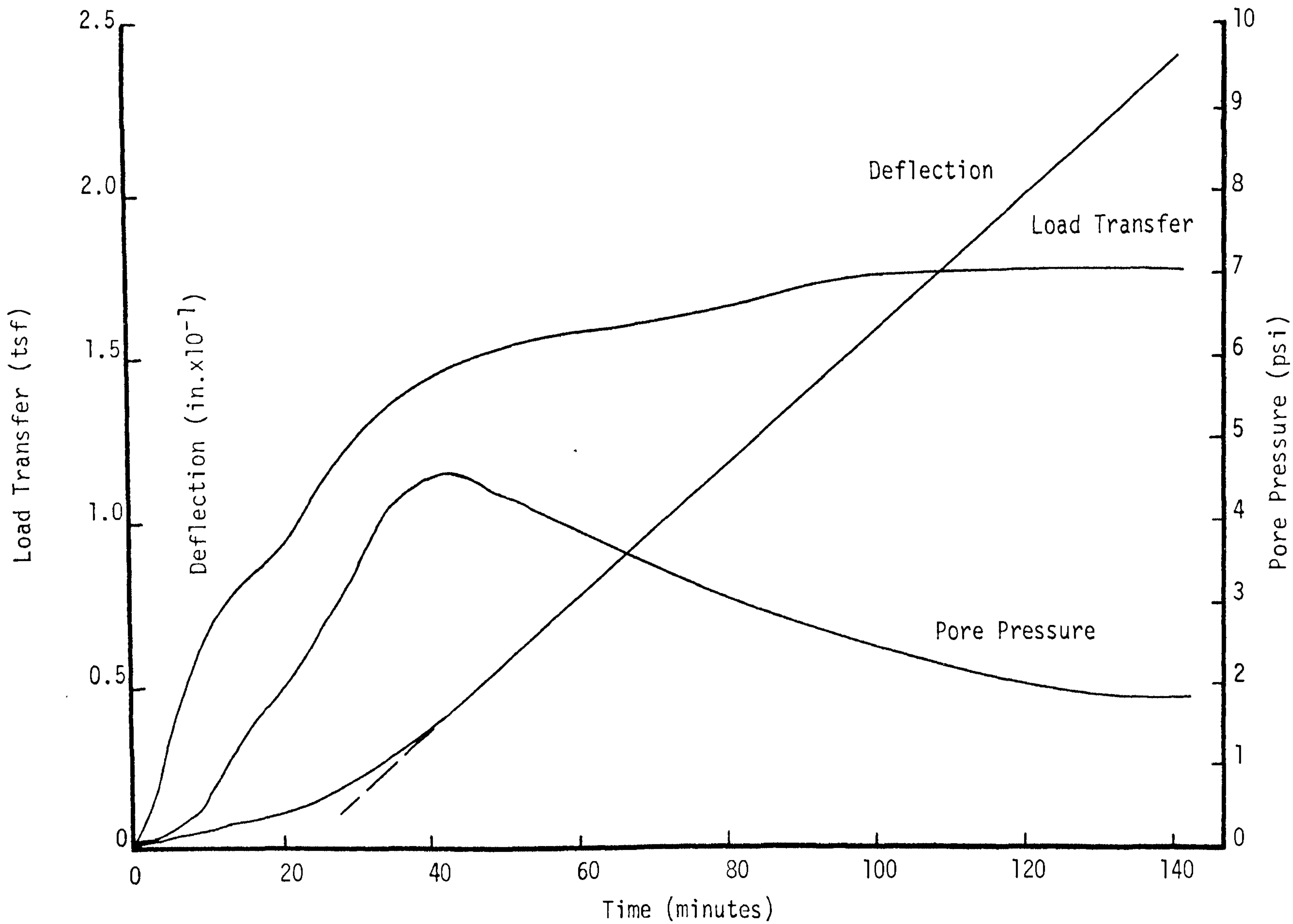

Figure (13). Test No. 6. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 41 p.s.i. Consolidation Pressure with a Probe Distance of 0 cm.

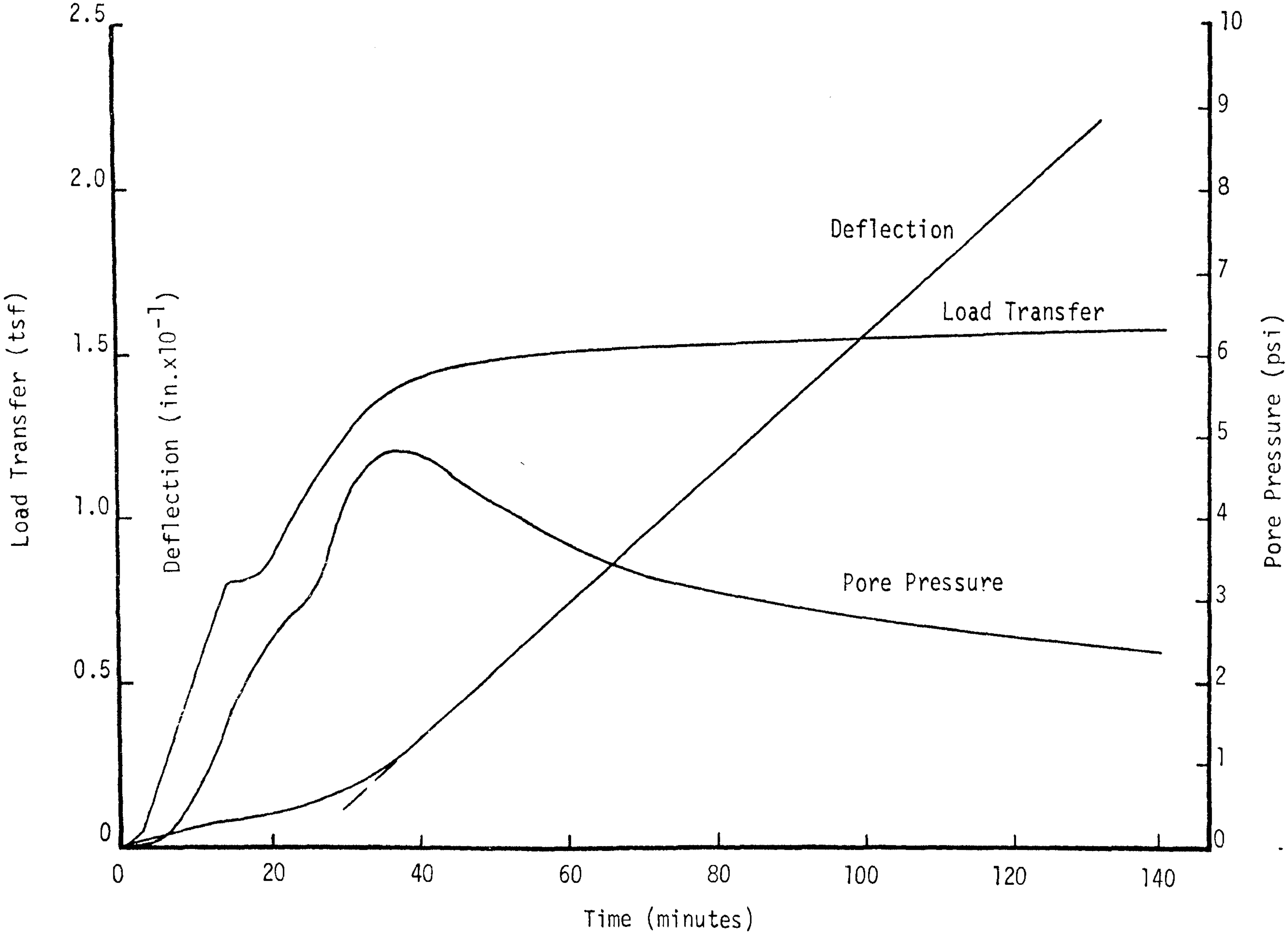

Figure (14). Test No. 7. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 41 p.s.i. Consolidation Pressure with a Probe Distance of 1/2 cm.

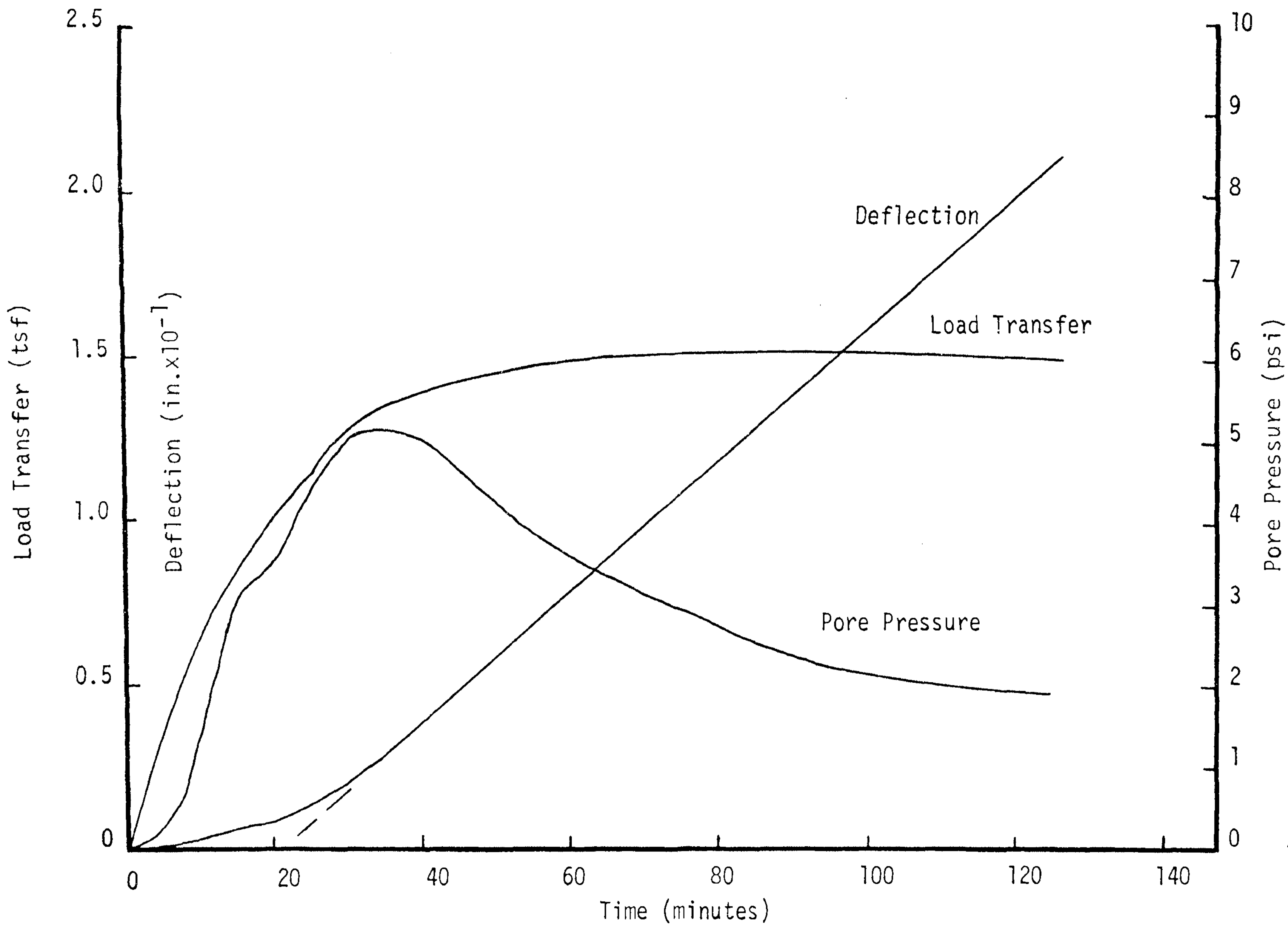

Figure (15). Test No. 8. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 41 p.s.i. Consolidation Pressure with a Probe Distance of 1 cm.

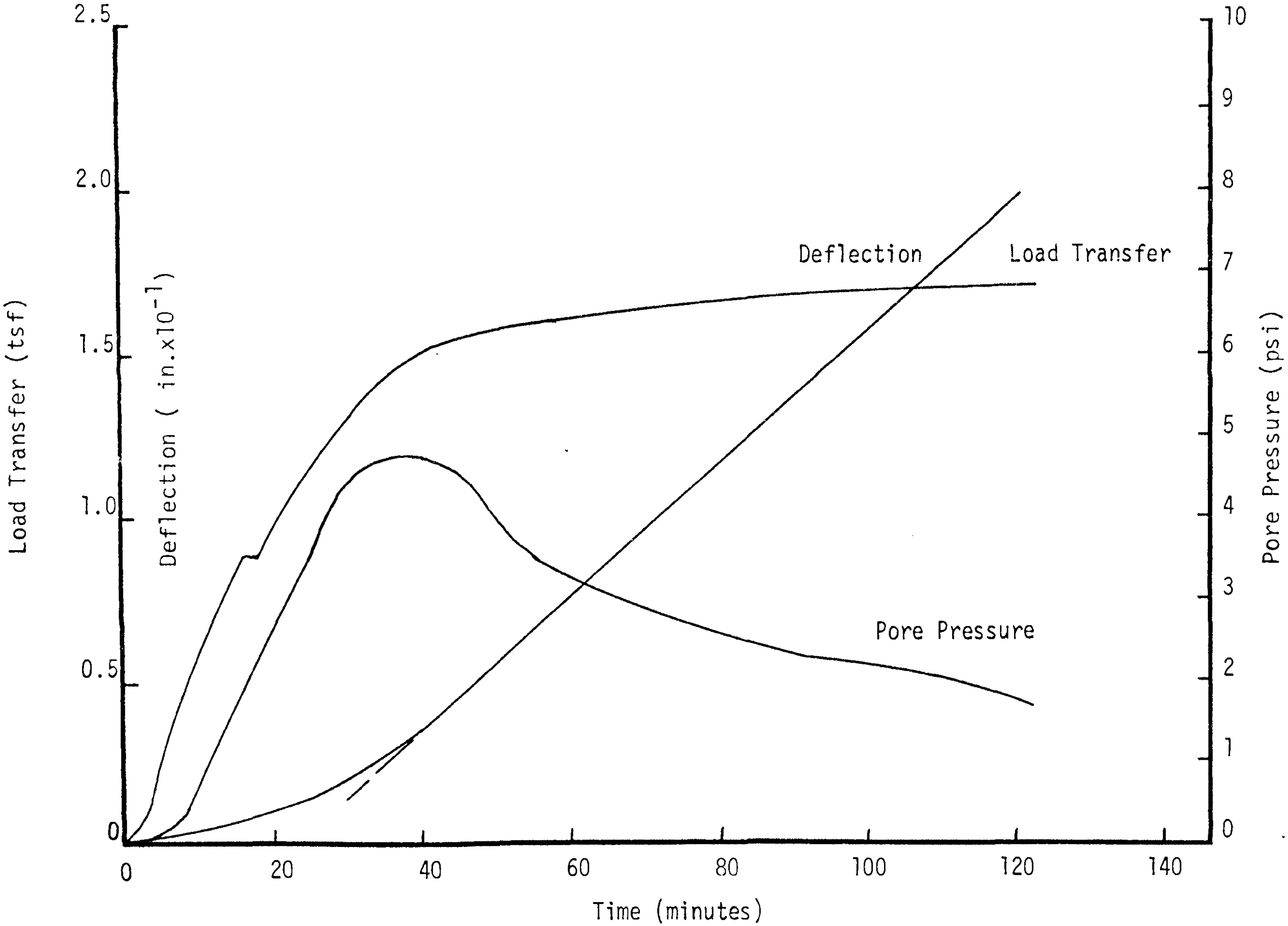

Figure (16). Test No. 9. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 41 p.s.i. Consolidation Pressure with a Probe Distance of 2 cm.

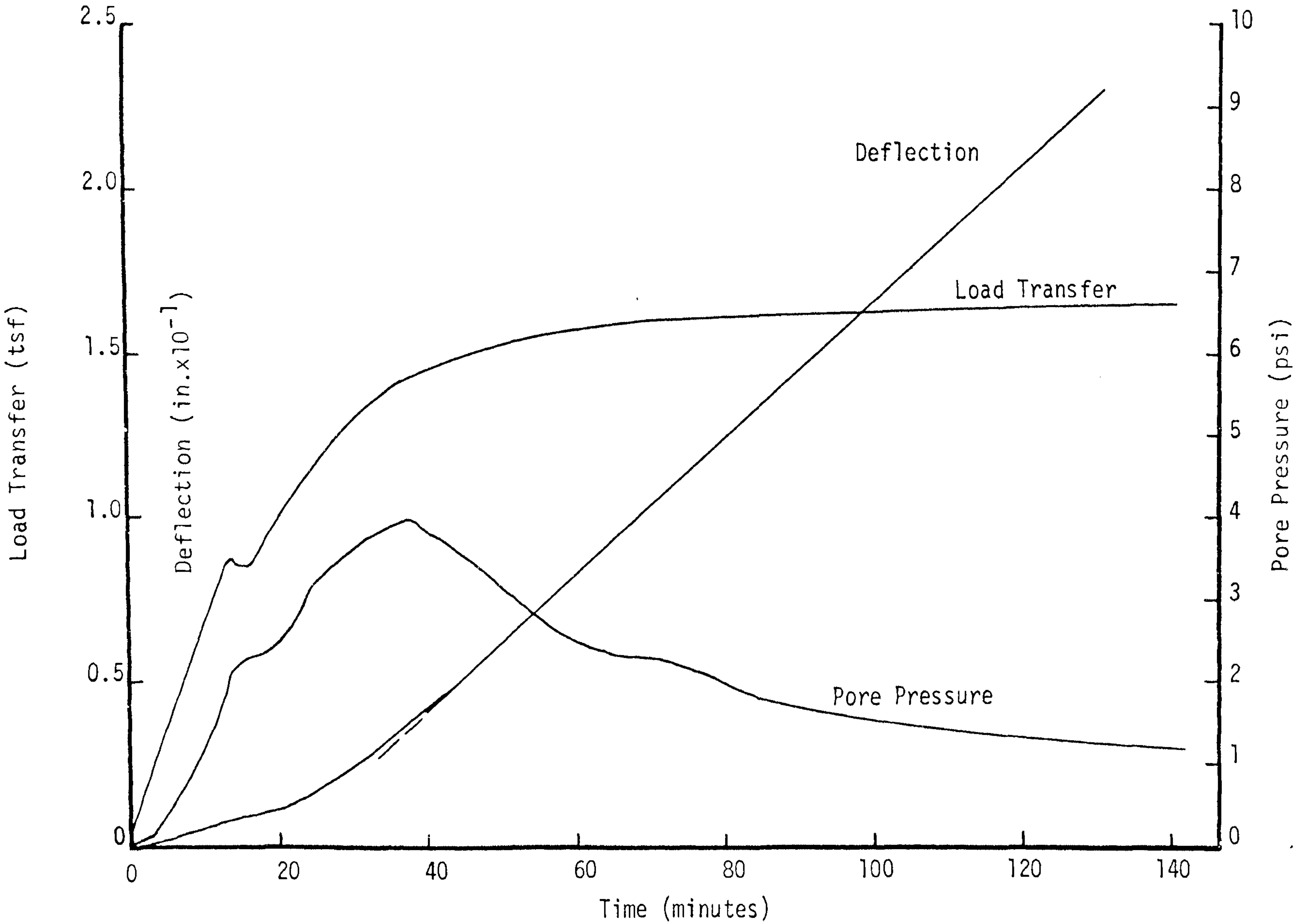

Figure (17). Test No. 10. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 41 p.s.i. Consolidation Pressure with a Probe Distance of 3 cm.

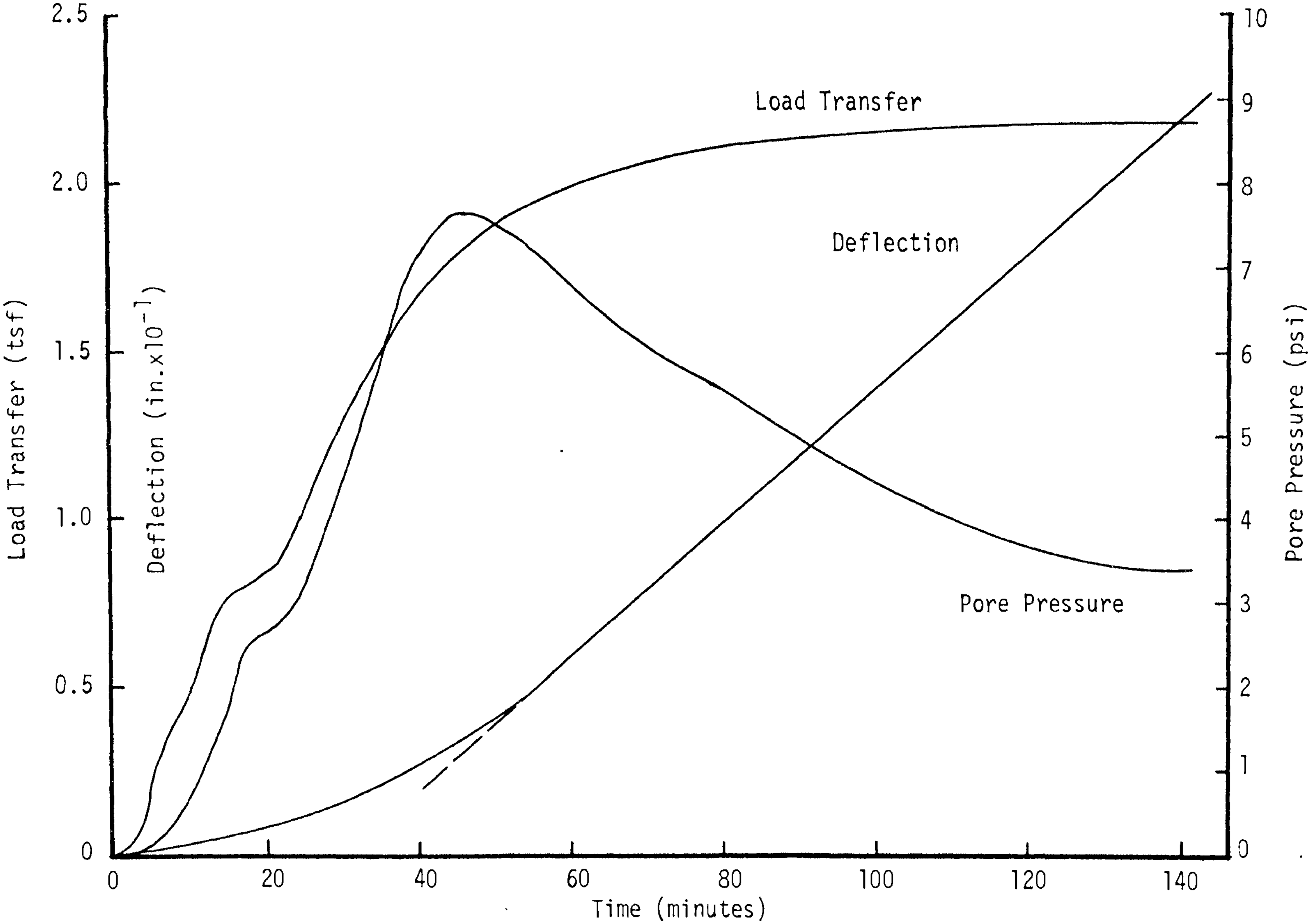

Figure (18). Test No. 11. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 55 p.s.i. Consolidation Pressure with a Probe Distance of 0 cm.

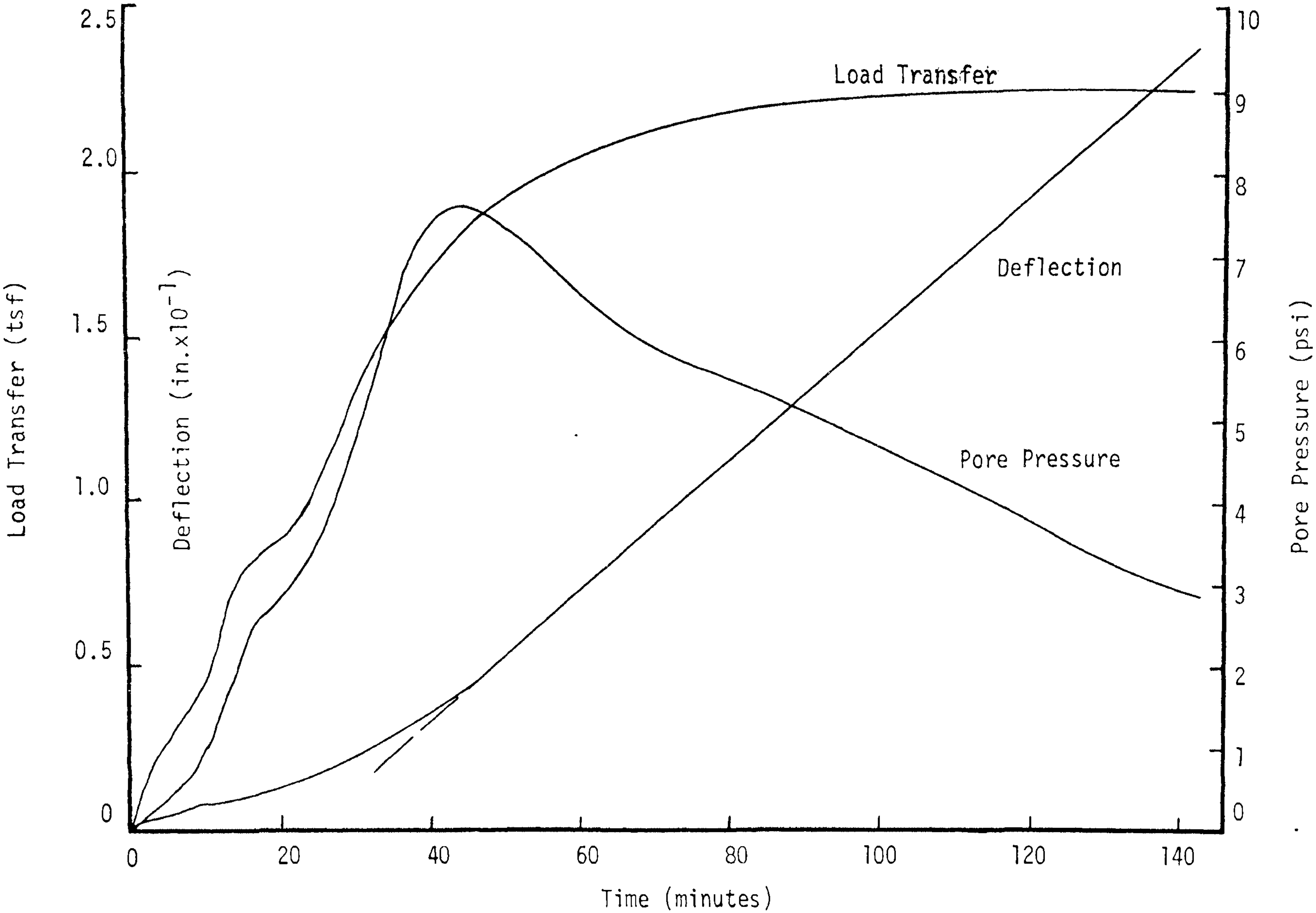

Figure (19). Test No. 12. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 55 p.s.i. Consolidation Pressure with a Probe Distance of 1/2 cm.

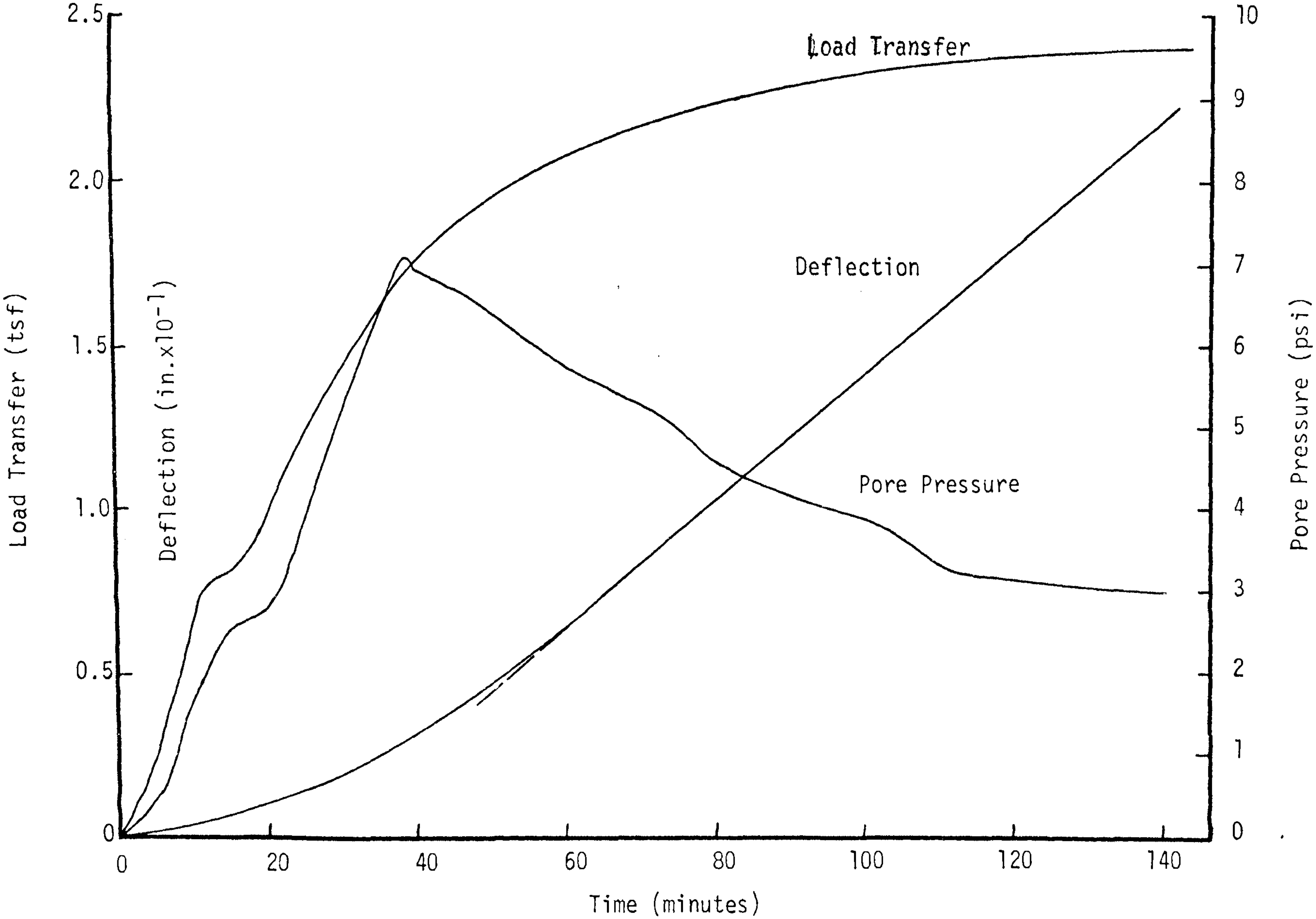

Figure (20). Test No. 13. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 55 p.s.i. Consolidation Pressure with a Probe Distance of 1 cm.

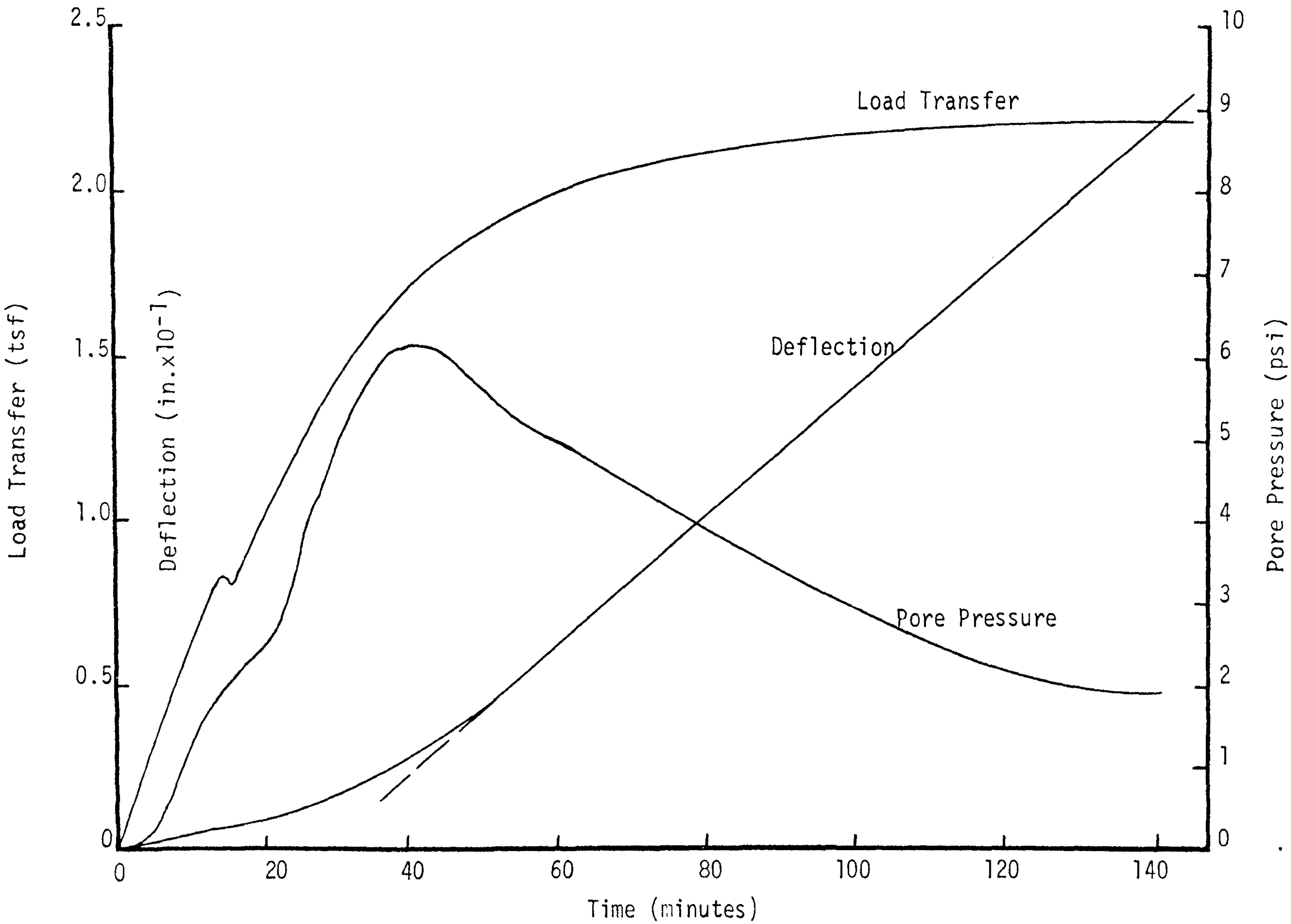

Figure (21). Test No. 14. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 55 p.s.i. Consolidation Pressure with a Probe Distance of 2 cm.

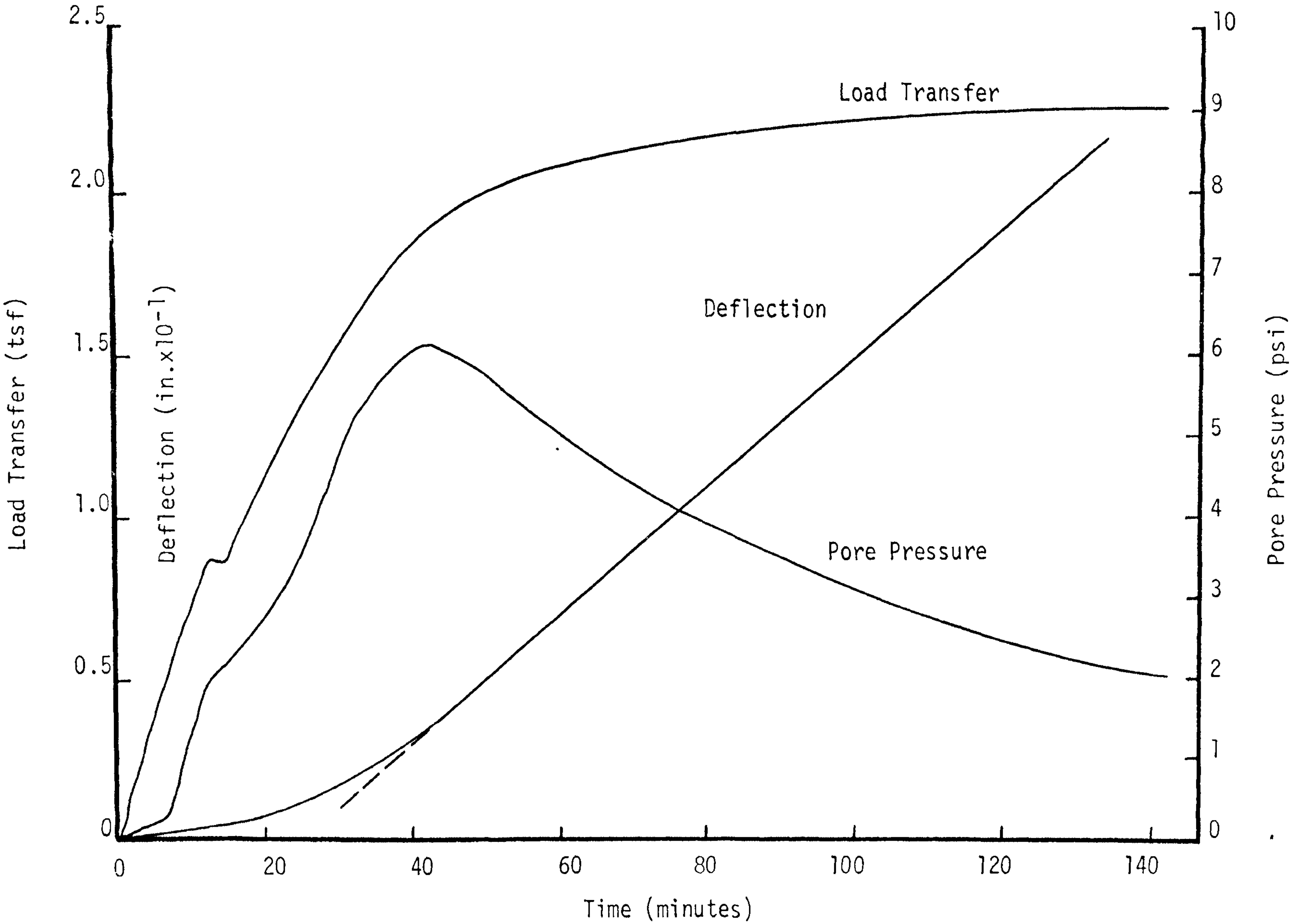

Figure (22). Test No. 15. The Relationship between Pore Pressure, Load Transfer, and Deflection versus Time at 55 p.s.i. Consolidation Pressure with a Probe Distance of 3 cm.

it was noted in all tests performed.

The differences in characteristics of the reduction in pore pressures determined by Airhart, et al. (1967) and that found in this research is attributed to the difference in the mode of failure. Since the pile did not plunge rapidly through the soil as it did in the field, the density of the soil was not affected as much as it was in the field, and the pore pressure dropped at a slower rate. The load did not decrease as rapidly as Airhart, et al. (1967) showed, but increased slowly after failure. A small amount of remolding was taking place, but the pore pressure decreased continually throughout the rest of the test. The rate of pore pressure decrease slowed down considerably toward the end of the test.

At each successive consolidation pressure, the respective maximum induced pore pressures increased by the same percent as the increase in consolidation pressure. The basic characteristics of the pore pressure curves, however, did not change.

It should be noted that the pore pressure curve plotted in Figure (8) does not have the same characteristics as those of the other tests. Erratic behavior of the transducer during the test indicated that the results obtained were not correct.

The relationship of the maximum induced pore pressure to the distance that the end of the probe was located from the pile for each consolidation pressure is shown in Figure (23). The pressures decreased slightly with increased distance from the pile. This is in agreement with many observations made in the field.

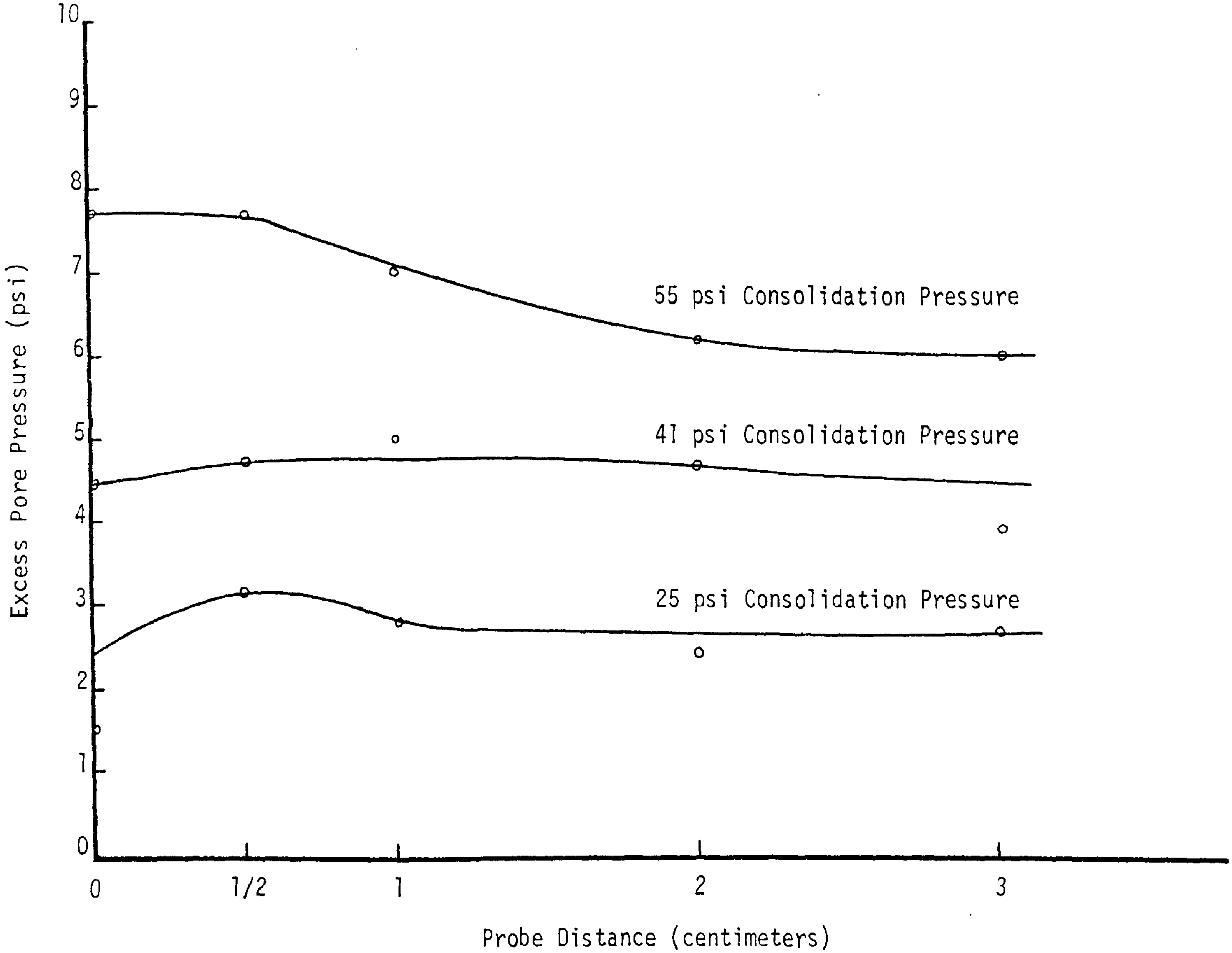

Figure (23). Relationship of Excess Pore Pressure to Probe Distance for Different Consolidation Pressures.

In Figures (8) to (22) it is noted that at the beginning of the test there was a small lag in time before a rapid increase in pore pressure occurred. Yong and Warkentin (1966) have explained in their discussion of pore pressure probes and transducers that a time lag will occur in activation of pore pressure due to the movement of water within the sample. Since the transducer has a small activation volume, a short time is needed for water to move through the soil before pressure is registered. The time lag, therefore, is dependent on the permeability of the soil and the activation volume of the transducer. In this investigation the time needed for activation was approximately the same for each test.

B. Load Response

Coyle and Reese (1966) made load tests on model friction piles with the soil consolidated in a modified triaxial chamber. It was noted that the load transfer increased to a maximum and then decreased to a lower constant value. See Figure (24). Broms and Hellman (1968) and Van Weele (1957) further noted that the friction resistance of a pile will increase to a maximum value and then slowly decrease as shown in Figure (25).

Load transfer versus settlement curves shown in Figures (26) to (28) show that the load did not decrease from a maximum value but increased continually to a constant value. This characteristic is explained by the type of soil used. The high permeability of the clay silt mixture allowed the pore pressures to dissipate rapidly from the remolded area, and a gain in shear strength resulted from the drainage

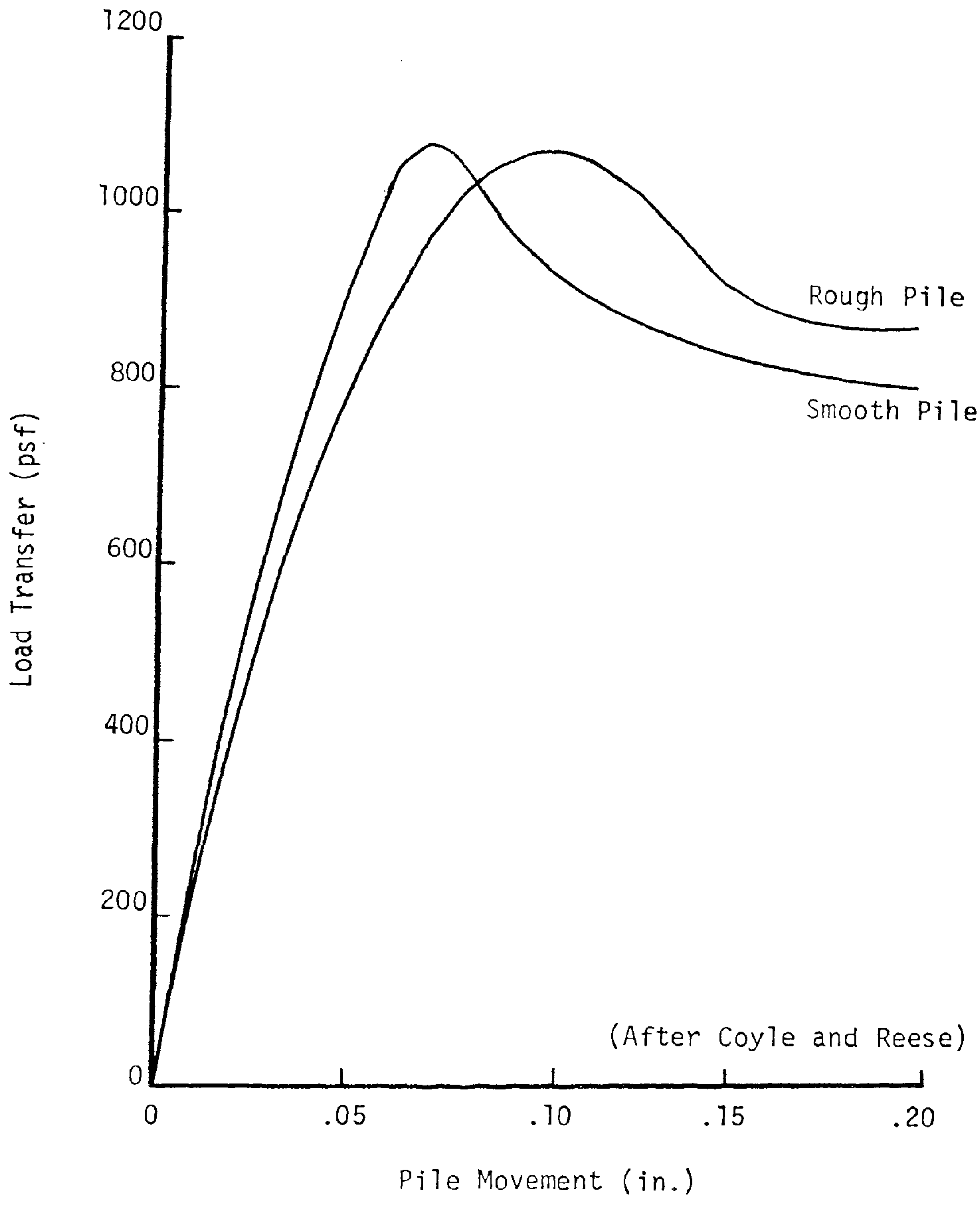

Figure (24). Load Transfer versus Pile Movement Curves for 1/2-in. Smooth and Rough Piles.

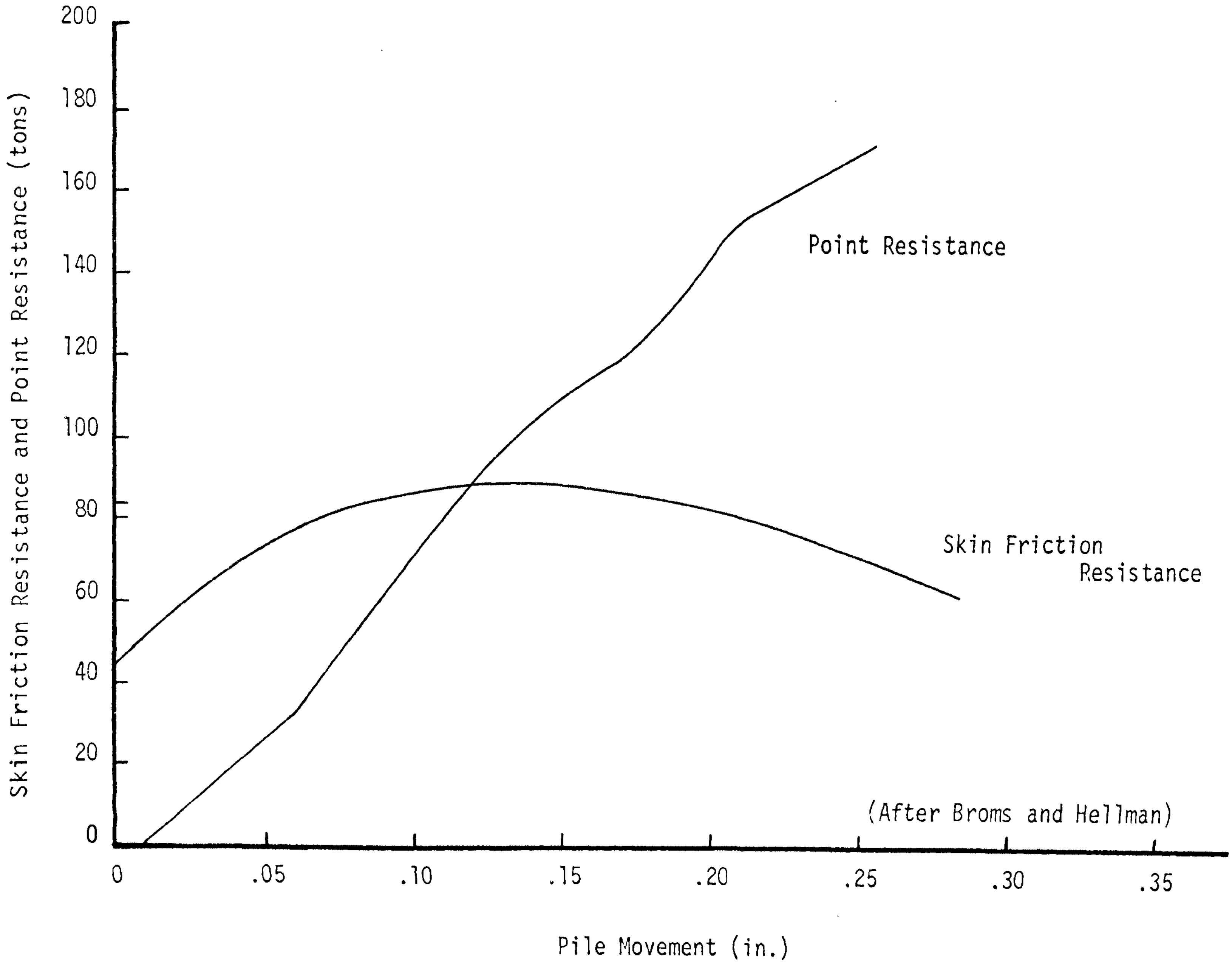

Figure (25). Relation Between Point Resistance and Displacement of Pile Point.

of moisture from the soil adjacent to the pile. Moisture content determinations taken after each test showed an average of a one percent decrease in moisture content in the soil adjacent to the pile compared to that of the outlying soil. Owing to the low plastic index of this soil, a small change in moisture content can increase the shear strength appreciably. Also, visual observation showed that the soil adjacent to the pile was more dense than the outlying soil.

From the load vs settlement curves, Figures (26) through (28), it can be seen that the load increased rapidly at small pile deflections until failure was reached. Approximately 80% of the ultimate load carried by the pile at the end of the test was transferred to the soil at failure. The figures also show that the load carrying capacity of the pile increased with increasing consolidation pressure.

It should be noted from Figures (8) through (22) that, at the point of failure, there was a small drop in pore pressure and load. It is suggested that this small drop in load is the point where the pile started its initial movement through the sample, and the deflections registered before this time were the elastic movements of the pile and the soil. When the pile began its initial movement through the soil, the elastic energy stored in the loading ring caused it to deflect downward a small amount which resulted in a sudden drop in load. The pore pressure then leveled off for a short time. When the loading ring completed its elastic movement, transfer of load from the pile to the soil again developed and the load and pore pressure increased at the same rate as before the small failure occurred.

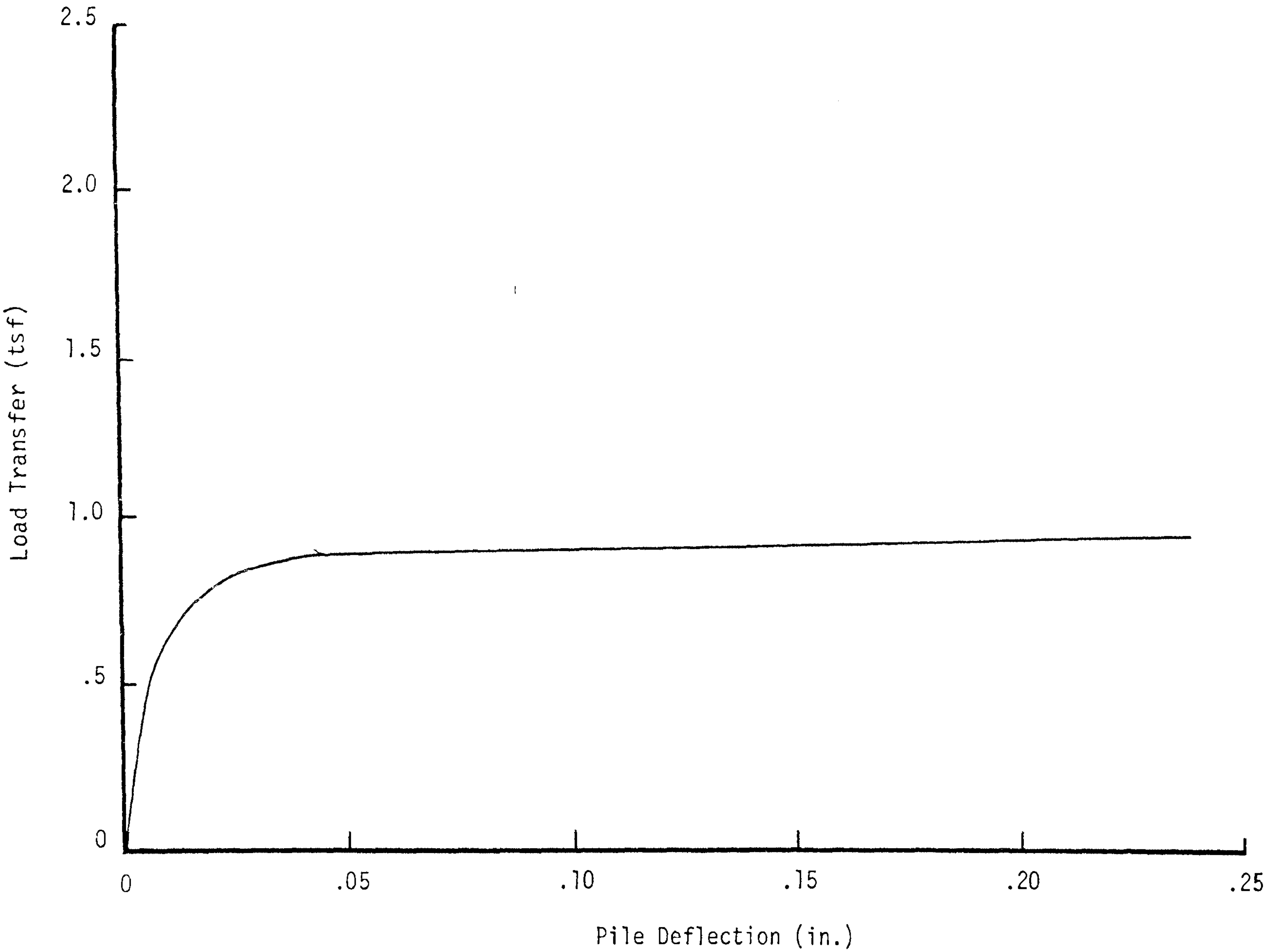

Figure (26). Average Load Transfer versus Pile Deflection at 25 p.s.i. Consolidation Pressure.

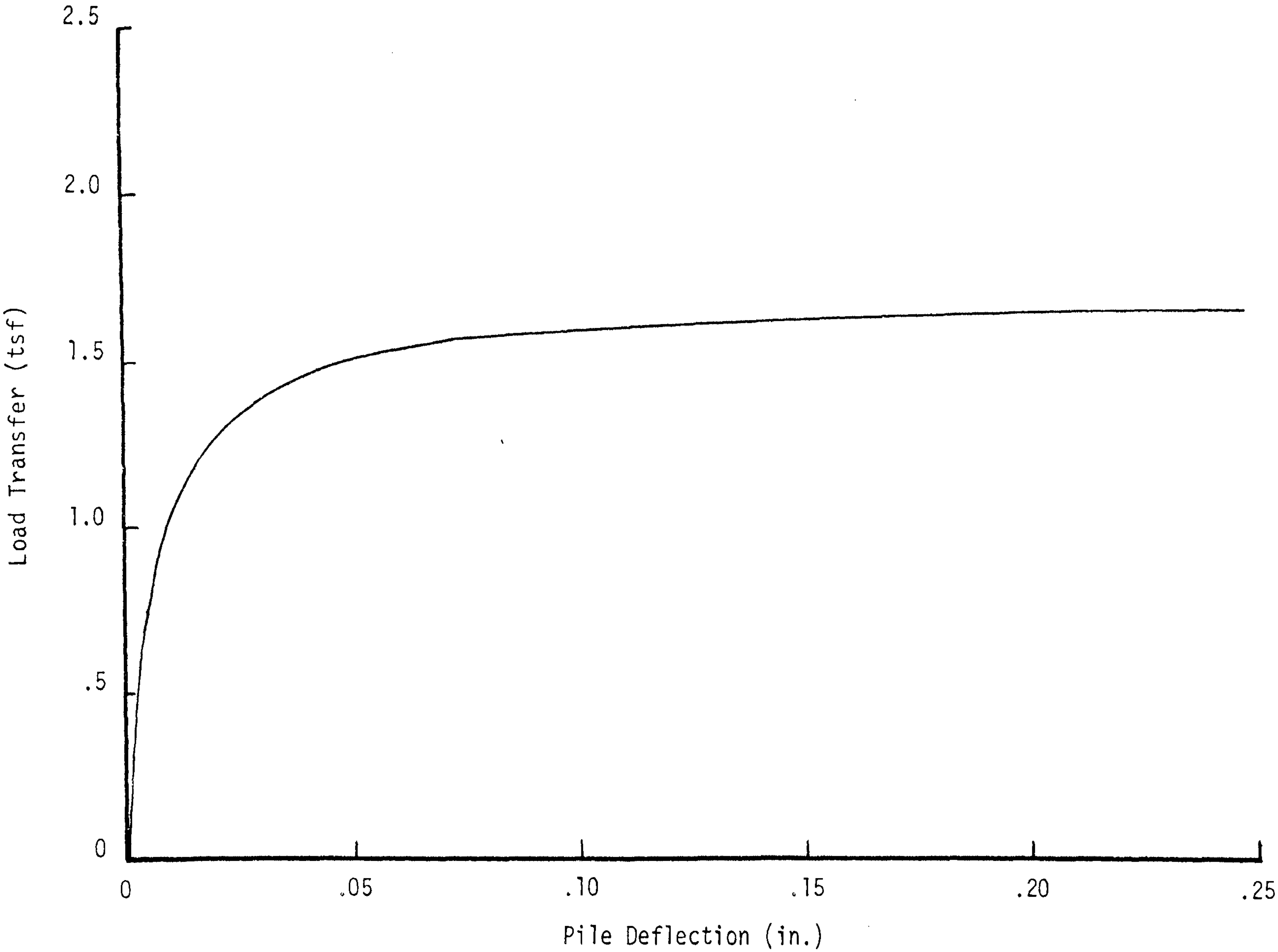

Figure (27). Average Load Transfer versus Pile Deflection at 41 p.s.i. Consolidation Pressure.

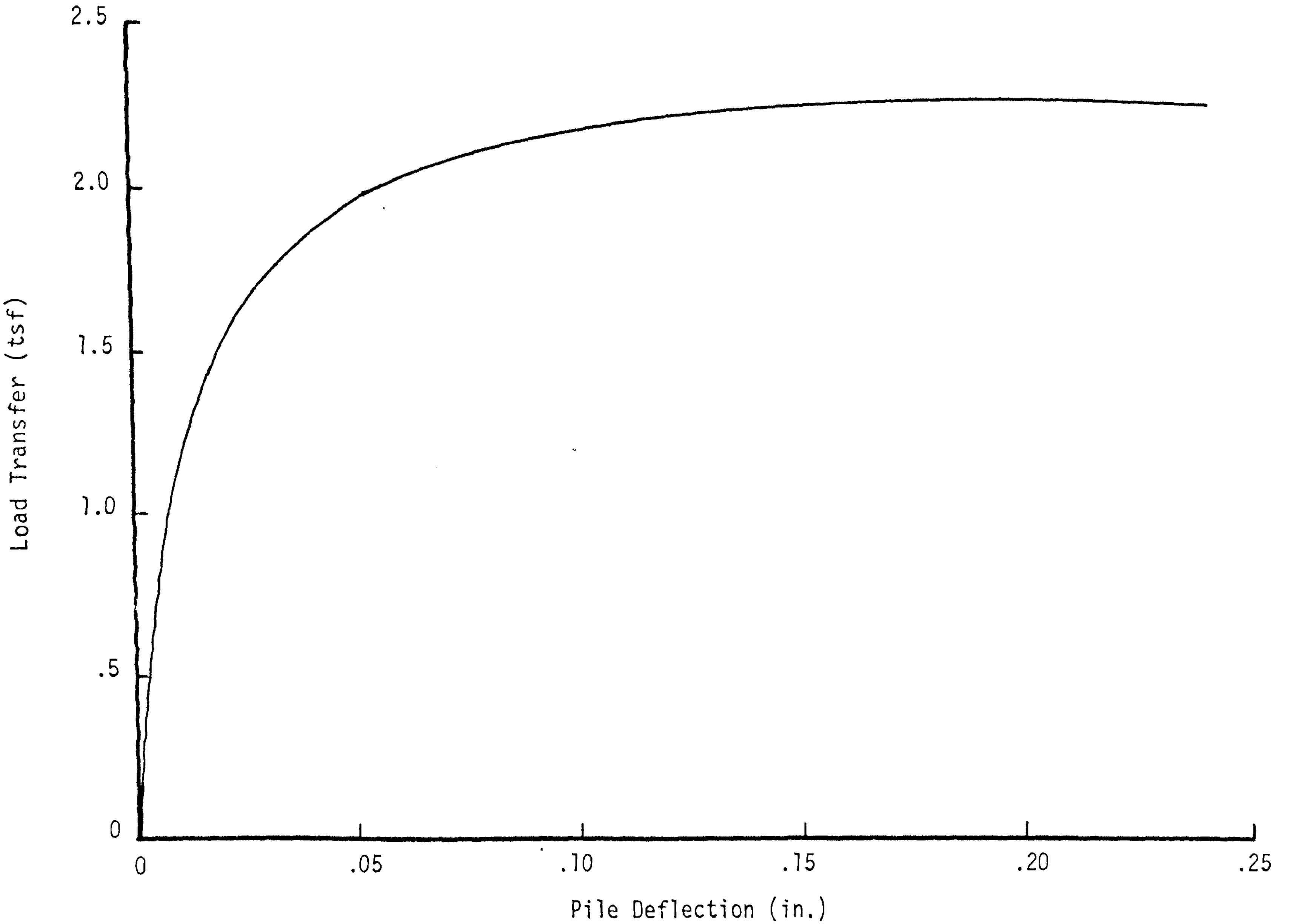

Figure (28). Average Load Transfer versus Pile Deflection at 55 p.s.i. Consolidation Pressure.

Coyle and Reese (1966) showed that during model pile tests in saturated clay, the ratio of load transfer to shear strength was equal to one for a roughened pile. Examination of the pile after testing showed a dense film of soil adhering to the pile indicating a soil to soil failure which would utilize the full shear strength of the soil in the area of the pile.

In Figure (29) Mohr's rupture envelopes have been drawn comparing the total and effective angles of friction obtained from consolidated-undrained tests with those calculated from the pile tests. The triaxial tests gave total and effective angles of friction of 13 degrees and 22 degrees, respectively while the friction angles calculated from the pile tests were 23 degrees and 28 degrees, respectively. It must be noted that calculations of "B" coefficients during the triaxial tests indicated that the soil was unsaturated for the tests of the two lowest consolidation pressures. For this reason only one Mohr's circle was used to calculate the effective angle of friction. Analysis of this data indicated that the shear strength of soil at the failure of the pile was higher than its initial value. During the pile tests, drainage of the sample was allowed in order to simulate drainage that would occur in the field. It is submitted that some consolidation of the soil occurred prior to failure of the pile due to movement of moisture from the soil adjacent to the pile thus causing an apparent increase in shear strength. Also, drainage of excess pore pressures which where induced during the insertion of the pile prior to testing, may have caused the structure to break down around the pile giving a more dense arrangement and more strength to the soil.

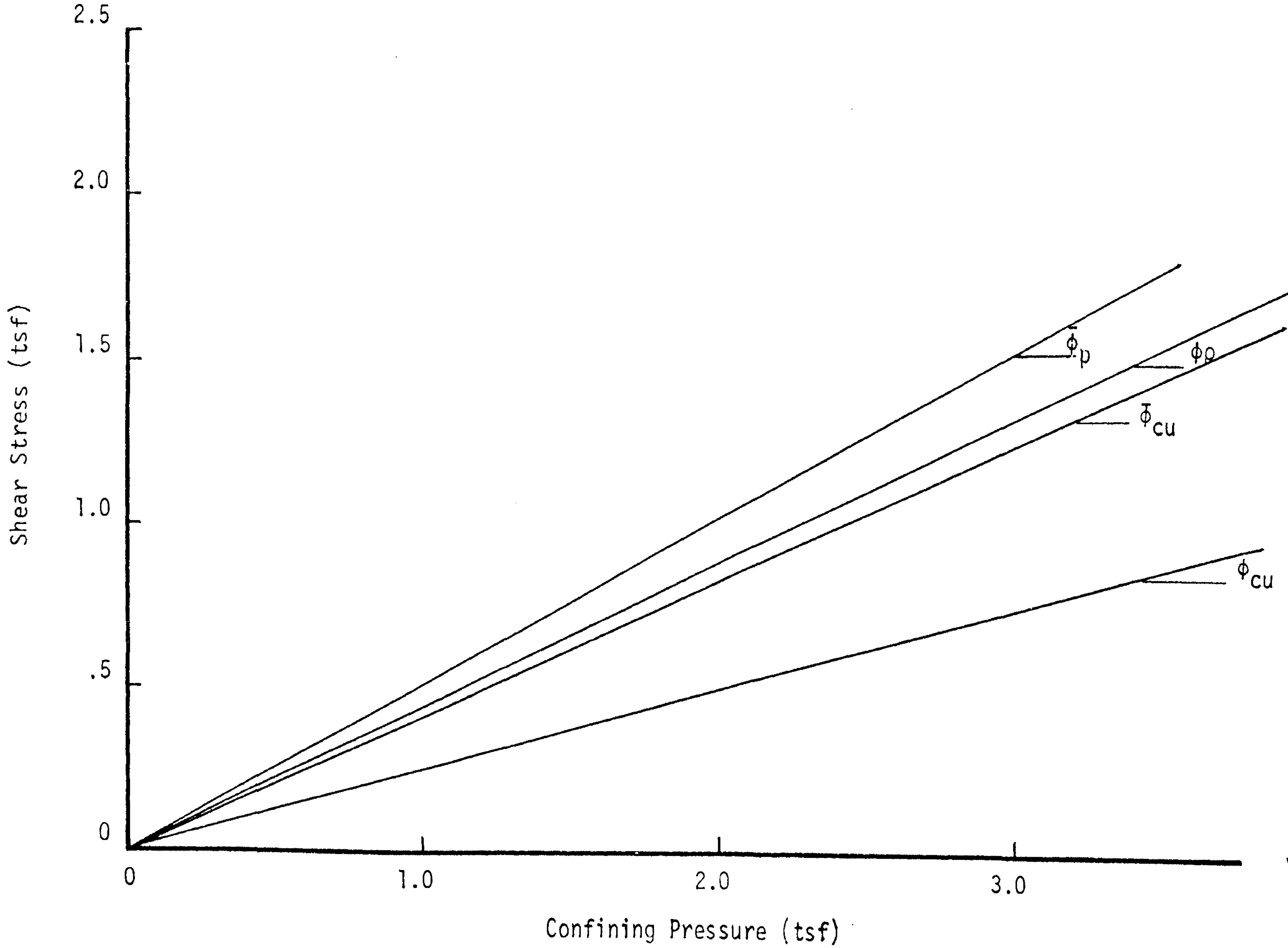

Figure (29). Comparison of Rupture Envelopes of CU-Test and Pile Test.

V. SUMMARY AND CONCLUSIONS

The purpose of this program of research was to develope an initial laboratory testing procedure for the measurement of pore water pressures induced by the static loading of a model friction pile and compare load transfer and pore pressure characteristics with field observations.

Equipment was designed and used to prepare a relatively homogeneous sample of research soil, consolidate it around a model pile inside a triaxial compression chamber and measure pore pressures during the test. A mixture of 30% clay and 70% silt was chosen as the research soil since it is a sensitive soil and permeable enough to allow dissipation of pore pressure without excessive time delay.

Loading tests were conducted using the continuous loading method as described by Cook (1961) on samples which were consolidated under different cell pressures. Measurements of load transfer, pile movement, time, and induced pore pressure at a known radial distance from the pile were recorded for each test.

From the results obtained, the following conclusions were drawn.

1. An appropriate method for the laboratory measurement of pore water pressures induced by a model pile during static loading has been developed in this research program.
2. Tests performed on model piles in the laboratory using the procedures outlined gave results similar to those reported in the field.

3. Induced pore pressures increased rapidly until pile failure occurred. After failure, the excess pressure dissipated rapidly for a short time and then decreased slowly to a constant value.
4. Load transfer increased rapidly to failure and approximately 80% of the ultimate load carried by the pile at the end of the test was transferred to the soil at failure. After failure occurred, load transfer progressed slowly to a constant value.
5. Shear strength of the soil adjacent to the pile increased by a small amount due to consolidation of the soil by the dissipation of excess pore pressures.

VI. RECOMMENDATIONS

Since this research program was the first to attempt to measure pore water pressure adjacent to a model pile in the laboratory, more time was spent in the design of equipment and developement of procedures than performing tests. It is evident, therefore, that much more testing and research of this subject should be considered. The following is a list of recommendations for further research of this subject:

1. Changing the type of soil from a plastic clay to a coarse sand would determine the extent at which the change in type of soil would influence the results.
2. Varying the rate and type of loading on the pile would be helpful in observing the changes in ultimate bearing capacity and shear strength of the soil due to different modes of failure.
3. Tests conducted on samples where the vertical position of the probe is varied would help to determine the amount of dissipation of pore water pressure that occurs during the test.
4. Results of field tests and laboratory tests conducted with undisturbed field samples would give an insight to the reproducibility of field conditions in the laboratory.

VII. APPENDIX A

SUMMARY OF RESULTS

RESULTS OF TEST NO. 1

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-B}%
0	0	0	0	--
3	.076	.002	.038	
5	.228	.003	.057	
7	.362	.004	.095	
9	.522	.005	.152	
11	.587	.007	.190	
13	.673	.008	.228	
15	.738	.011	.332	
17	.789	.013	.418	
19	.791	.014	.475	
24	.859	.018	.760	
31	.900	.028	.911	
35	.930	.037	1.120	
45	.960	.057	1.350	
51	.980	.079	1.460	
61	.995	.090	1.540	
75	1.000	.122	1.540	
90	1.015	.150	1.480	
107	1.015	.199	1.410	
125	1.015	.228	1.330	

RESULTS OF TEST NO. 2

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b} %
0	0	0	0	1.00
3	.168	.0015	.133	
6	.370	.0035	.701	
9	.531	.0055	1.500	
12	.646	.0090	2.470	
15	.723	.0125	2.850	
18	.781	.0165	3.220	
20	.808	.0195	3.220	
25	.820	.0215	2.670	
31	.880	.0330	2.470	
35	.890	.0395	2.220	
40	.898	.0500	1.990	
45	.906	.0595	1.820	
50	.912	.0695	1.800	
55	.920	.0790	1.710	
60	.925	.0890	1.670	
75	.935	.1190	1.480	
80	.937	.1295	1.430	
90	.942	.1500	1.270	
100	.950	.1700	1.230	
110	.952	.1915	1.220	
120	.949	.2115	1.210	

RESULTS OF TEST NO. 3

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	1.20
3	.017	.0020	.038	
6	.037	.0050	.095	
9	.111	.0060	.152	
12	.316	.0092	.247	
15	.472	.0115	1.050	
18	.577	.0155	2.080	
20	.628	.0182	2.430	
25	.719	.0264	2.750	
30	.775	.0340	2.750	
35	.810	.0430	2.370	
40	.839	.0545	2.180	
45	.855	.0625	2.090	
50	.870	.0725	1.900	
55	.880	.0795	1.710	
60	.885	.0900	1.610	
70	.891	.1100	1.520	
80	.910	.1295	1.420	
90	.920	.1505	1.230	
100	.924	.1772	1.190	
110	.928	.1940	1.140	
120	.925	.2139	1.100	

RESULTS OF TEST NO. 4

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	1.10
3	.189	.0010	.019	
6	.372	.0035	.323	
9	.520	.0050	.950	
12	.634	.0078	1.481	
15	.708	.0115	1.900	
18	.766	.0155	2.180	
20	.796	.0185	2.335	
25	.832	.0220	2.430	
30	.875	.0295	2.180	
35	.900	.0390	2.090	
40	.911	.0495	1.900	
45	.922	.0585	1.710	
50	.926	.0680	1.520	
56	.931	.0790	1.425	
60	.935	.0800	1.385	
70	.940	.1075	1.290	
80	.945	.1275	1.195	
93	.949	.1425	1.140	
100	.953	.1660	1.100	
110	.961	.1860	1.040	
120	.965	.2060	.950	

RESULTS OF TEST NO. 5

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	0.5
3	.222	.0015	.330	
6	.392	.0030	.664	
9	.514	.0055	1.710	
12	.603	.0090	2.280	
15	.665	.0135	2.750	
18	.715	.0185	2.800	
20	.740	.0220	2.780	
25	.787	.0310	2.660	
30	.826	.0405	2.560	
35	.850	.0500	2.320	
40	.868	.0500	2.180	
45	.885	.0655	2.000	
50	.897	.0755	1.800	
55	.911	.0855	1.710	
60	.920	.0955	1.615	
70	.934	.1165	1.520	
80	.945	.1370	1.425	
90	.950	.1585	1.235	
100	.954	.1795	1.178	
110	.953	.2005	1.140	

RESULTS OF TEST NO. 6

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{1-b}%
0	0	0	0	1.20
3	.147	.0020	.095	
6	.374	.0030	.190	
9	.581	.0042	.380	
12	.755	.0060	.853	
15	.810	.0075	1.370	
18	.844	.0080	1.710	
20	.907	.0090	1.880	
25	1.110	.0135	2.730	
30	1.257	.0200	3.220	
35	1.360	.0270	4.180	
40	1.430	.0650	4.460	
45	1.480	.0455	4.460	
50	1.520	.0550	4.270	
57	1.550	.0070	3.190	
60	1.570	.0750	3.800	
70	1.610	.0955	3.420	
80	1.650	.1156	3.040	
90	1.675	.1360	2.660	
100	1.690	.1565	2.370	
110	1.705	.1760	2.300	
120	1.720	.2010	2.000	
130	1.720	.2180	1.880	
140	1.721	.2390	1.800	

RESULTS OF TEST NO. 7

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	1.0
3	.213	.0010	.095	
6	.421	.0010	.380	
9	.605	.0025	1.140	
12	.760	.0040	2.180	
15	.861	.0060	3.130	
18	.924	.0070	3.320	
20	1.000	.0085	3.560	
25	1.165	.0140	4.740	
30	1.280	.0125	5.410	
35	1.345	.0295	5.120	
40	1.393	.0390	4.940	
45	1.430	.0485	4.550	
50	1.450	.0590	4.170	
55	1.470	.0690	3.800	
60	1.490	.0790	3.510	
70	1.505	.0990	3.040	
80	1.515	.1185	2.660	
90	1.520	.1390	2.310	
100	1.520	.1600	2.140	
110	1.518	.1805	2.050	
120	1.511	.2005	1.860	

RESULTS OF TEST NO. 8

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	0.8
3	.031	.0035	.038	
6	.254	.0045	.133	
9	.472	.0055	.493	
12	.664	.0068	1.005	
15	.811	.0085	1.710	
18	.805	.0090	2.090	
20	.861	.0095	2.540	
25	1.080	.0120	3.000	
30	1.240	.0175	4.100	
35	1.370	.0240	4.750	
40	1.425	.0325	4.750	
45	1.455	.0425	4.500	
50	1.470	.0525	4.160	
55	1.490	.0625	3.930	
60	1.505	.0729	3.660	
70	1.510	.0935	3.280	
80	1.540	.1135	3.100	
90	1.550	.1350	2.940	
100	1.560	.1560	2.820	
115	1.570	.1875	2.620	
120	1.575	.1980	2.520	
130	1.580	.2190	2.430	
140	1.585	.2400	2.330	

RESULTS OF TEST NO. 9

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	--
3	.102	.0015	.095	
6	.331	.0025	.190	
9	.535	.0040	.494	
12	.700	.0052	1.235	
15	.845	.0068	1.800	
18	.865	.0075	2.240	
21	.990	.0090	2.620	
25	1.160	.0130	3.610	
30	1.310	.0195	4.440	
35	1.425	.0270	4.700	
40	1.505	.0355	4.730	
45	1.550	.0445	4.540	
50	1.575	.0540	3.990	
55	1.595	.0640	3.480	
60	1.610	.7350	3.270	
70	1.635	.0940	2.900	
81	1.660	.1150	2.560	
90	1.670	.1355	2.280	
100	1.683	.1565	2.180	
110	1.705	.1770	1.990	
120	1.710	.1975	1.750	

RESULTS OF TEST NO. 10

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	--
3	.223	.0005	.095	
6	.432	.0028	.436	
9	.624	.0040	1.061	
12	.795	.0065	1.613	
15	.850	.0080	2.280	
18	.940	.0090	2.340	
20	1.011	.0105	2.410	
25	1.180	.0160	3.230	
30	1.313	.0235	3.610	
35	1.400	.0325	3.900	
40	1.460	.0420	3.800	
45	1.505	.0515	3.610	
50	1.530	.0620	3.140	
55	1.555	.0730	2.750	
60	1.575	.0832	2.410	
70	1.605	.1040	2.280	
80	1.620	.1250	2.000	
92	1.621	.1530	1.610	
100	1.625	.1690	1.520	
110	1.635	.1890	1.425	
120	1.650	.2180	1.270	
130	1.660	.2285	1.235	
140	1.665	.2520	1.175	

RESULTS OF TEST NO. 11

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	1.25
3	.045	.0005	.057	
6	.264	.0020	.190	
9	.401	.0050	.610	
12	.611	.0060	1.120	
15	.775	.0073	1.750	
18	.811	.0080	2.560	
20	.826	.0083	2.620	
25	1.000	.0105	3.190	
30	1.280	.0165	4.500	
36	1.505	.0125	6.110	
40	1.660	.0268	7.080	
45	1.781	.0340	7.650	
50	1.870	.0423	7.550	
55	1.930	.0510	7.260	
60	1.990	.0595	6.780	
70	2.060	.0785	6.010	
80	2.120	.0980	5.750	
91	2.150	.1205	5.000	
100	2.160	.1375	4.420	
110	2.180	.1600	3.950	
121	2.185	.1825	3.740	
130	2.190	.2020	3.470	
140	2.188	.2210	3.190	

RESULTS OF TEST NO. 12

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	--
3	.043	.0020	.190	
6	.286	.0030	.391	
9	.390	.0065	.720	
12	.603	.0075	1.348	
15	.782	.0090	2.050	
18	.845	.0105	2.600	
20	.960	.0115	2.770	
25	1.040	.0125	3.440	
30	1.330	.0170	4.630	
35	1.540	.0215	6.180	
40	1.700	.0275	7.320	
45	1.840	.0350	7.570	
50	1.920	.0430	7.350	
55	1.990	.0519	7.930	
60	2.040	.0625	6.520	
70	2.126	.0825	5.910	
80	2.180	.1125	5.500	
90	2.210	.1325	5.050	
100	2.235	.1525	4.610	
110	2.249	.1725	4.170	
120	2.250	.1925	3.790	
130	2.251	.2125	3.260	
140	2.251	.2330	2.920	

RESULTS OF TEST NO. 13

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	1.0
3	.115	.0030	.228	
6	.342	.0040	.228	
9	.558	.0053	.588	
12	.756	.0070	1.420	
15	.811	.0080	2.070	
18	.874	.0085	2.560	
20	.986	.0100	2.660	
25	1.225	.0135	2.750	
30	1.435	.0185	3.910	
35	1.605	.0245	5.500	
40	1.750	.0315	6.930	
45	1.860	.0390	6.690	
50	1.945	.0470	6.410	
55	2.010	.0555	6.060	
60	2.080	.0645	5.700	
70	2.180	.0830	5.280	
80	2.240	.1002	4.550	
85	2.270	.1102	4.360	
90	2.290	.1205	4.170	
100	2.330	.1400	3.890	
110	2.360	.1600	3.320	

RESULTS OF TEST NO. 14

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	1.0
3	.190	.0005	.095	
6	.384	.0020	.380	
9	.593	.0035	1.140	
12	.750	.0050	1.615	
15	.863	.0065	2.020	
18	.940	.0070	2.340	
20	1.040	.0080	2.520	
25	1.260	.0122	3.800	
30	1.450	.0172	5.030	
35	1.600	.0228	5.900	
40	1.730	.0290	6.170	
45	1.830	.0365	5.990	
50	1.900	.0450	5.510	
55	1.960	.0540	5.120	
60	2.000	.0630	4.940	
70	2.070	.0820	4.310	
80	2.120	.1015	3.800	
90	2.160	.1215	3.140	
100	2.190	.1420	2.950	
110	2.210	.1620	2.560	
121	2.220	.1820	2.180	
130	2.230	.2030	2.000	

RESULTS OF TEST NO. 15

Time min.	Load Transfer tsf	Deflection in.	Pore Pressure psi	W_{i-b}%
0	0	0	0	1.0
3	.224	.0005	.095	
6	.448	.0010	.211	
9	.650	.0020	1.140	
12	.843	.0030	1.900	
15	.874	.0045	2.180	
18	1.025	.0055	2.460	
20	1.090	.0060	2.660	
25	1.335	.0098	3.650	
30	1.535	.0165	4.850	
35	1.705	.0235	5.600	
40	1.845	.0315	6.020	
45	1.935	.0410	5.960	
50	1.990	.0505	5.700	
55	2.040	.0600	5.310	
61	2.080	.0720	4.840	
70	2.130	.0900	4.460	
80	2.160	.1100	3.890	
90	2.200	.1300	3.510	
100	2.220	.1505	3.170	
110	2.240	.1705	2.940	
120	2.250	.1905	2.520	

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IX. VITA

Charles Alfred Miller was born in St. Louis, Missouri on September 26, 1945. He recieved his primary and secondary education from the Affton Public School System, Affton, Missouri, which is a suburb of St. Louis.

He attended the University of Missouri at Rolla where he recieved the degree of Bachelor of Science in Civil Engineering in January, 1968. He entered graduate school at the University of Missouri-Rolla in January, 1968 and was appointed a graduate teaching assistant.

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