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A study on load transfer of model friction piles

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ABSTRACT

Previous investigations concerned with disturbance of soft clay by driving displacement piles are summarized. The initiation of excess pore water pressures due to pile driving, and mechanisms of load transfer are also described, along with the applicapability of model pile testing in analysis of the various phenomena.

Load tests were conducted on assorted sizes of model friction piles embedded in sedimented soil samples which consisted of a silt and clay mixture. Pilot holes of various diameters were cut in the samples.

A theoretical load distribution curve was used to calculate the load transferred to the soil as a function of pile embedment. Laboratory vane shear test results were compared to the load transferred to the soil by the pile.

Results of the research program indicate that: 1) The ratio of the load transferred to the soil to the undrained strength of the soil changes with depth in the sample, and with the ratio of the pilot hole diameter to the pile diameter, 2) an optimum pilot hole size exists for each pile which offers a balance between low soil disturbance and high load carrying capacity, 3) the soil sample size should be four to five times the pile size to achieve valid load test results, and 4) the ultimate load that a friction pile can support increases with time after pile insertion and with increased rates of penetration.

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ml.	milliliter
cm.	centimeter
mm.	millimeter
μ	micron
psf	pounds per square foot
R_d	ratio of pilot hole diameter to pile diameter
T-S	ratio of the load transferred to the soil by the pile to the undrained shear strength of the soil

I. INTRODUCTION

A. <u>General</u>

As programs of large scale construction continue and expand, suitable foundation sites at strategic locations become less available. Where large, deep deposits of soft silts and clays are encountered, economics and practical design considerations dictate the use of "floating" pile foundations. These structures gain support from the skin friction developed between the soil and pile wall, and do not bear directly on firm strata below. By their nature they are subject to excessive settlements and occasional bearing capacity failures and are, therefore, subject for concern.

Although considerable amount of information has been accumulated on soil-pile interaction by the driving of full-scale instrumented piles, the cost and equipment involved is usually restrictive. The complexity of most natural soil deposits renders generalizations based on the results of such tests questionable, and damage to sensitive instruments during driving may mean a small return of usable data gained from the investment.

In recent years many investigators have turned to the use of model piles because they cost less and have greater flexibility with respect to the foundation geometry and soil conditions. To date many and varied test procedures have been chosen to achieve the desired compatibility

between laboratory and field test results.

The purpose of this research program was to interpret results of load tests on model friction piles. Series of tests were conducted such that the effects of different values of selected variables could be studied.

B.   Outline of Research

The research program was carried out in six general steps:

1.   A review of existing literature was performed to become familiar with soil-pile interaction in general, and to act as a guide in the choice of suitable procedures to achieve valid results.

2.   Two basically pure soil types were obtained and tests were carried out to determine the physical properties of each.

3.   A mixture of the two soils was used in preparing two sizes of sedimented samples of the desired consistency.

4.   Functional and sensitive equipment commensurate with the desired degree of accuracy was developed.

5.   Tests to study the effects of pilot hole size, pile size and sample size were conducted, along with vane shear and unconfined compression to correlate results.

6.   The results were analyzed, conclusions drawn and recommendations given.

II. REVIEW OF LITERATURE

A. Soil-Pile Interaction Studies with Full-Scale Piles

   1. The disturbance of cohesive soils by pile driving

As early as 1915, Karl Zimmerman observed and reported the influence of pile size and shape on disturbance to the surrounding soil. He noted that in varved clays the distortion was easily seen, and in some cases disturbed zones occurred which extended at least one pile diameter away from the pile.

In 1932 Casagrande put forth the first significant theory concerning the action of plastic clay around driven displacement piles. The clay samples were from the Laurentian Valley in Canada, an area known for its sensitive clays. From confined and unconfined compression tests run on undisturbed and remolded samples taken at varying distances radially from the pile, it was discovered that some clays near the pile experienced an increase in their coefficient of compressibility. In some instances this increase was so great that the soil compressed under its own weight. Casagrande suggested that the clay immediately surrounding the pile would be remolded[1] to a distance of one half of the pile diameter, and to a distance of 1.5 pile diameters the clay would be disturbed sufficiently to cause a rather large increase in compressibility.

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[1]Remolding refers to the destruction of the arrangement of the molecules in the absorbed layers of the clay particles, and injury to the clay structure that developed during sedimentation.

Using similar experimental analyses, Zeevaert (1950) studied the effects of driving wood piles into the soft sensitive clays underlying Mexico City. He concluded that there are three zones of soil with differing properties in the form of annular rings around the newly driven pile. (See Fig. 1). Zone I represents completely remolded clay which is in constant movement during the driving process. This zone of remolded soil has a thickness equal to 0.2 times the pile diameter, and completely surrounds the pile.

Zone II is in the shape of an annular ring surrounding Zone I and consists of soil that is quite disturbed. According to theory, the soil moves during the driving process, but very little, and probably extends to one diameter from the pile wall. In Zone III the clay is assumed to remain unaltered during driving, except for a temporary upward elastic deformation due to pressures caused by the displaced volume of remolded clay during driving.

Although the exact values for the boundaries of the zones do not correspond between the Casagrande and Zeevaert theories, they agree in the basic idea that there are zones of soil undergoing local and general shear while piles are driven into the strata. Guided by these theories, engineers came to believe that driving piles into soft clay might cause sufficient remolding or disturbance to produce appreciable settlement of the surface from the weight of the soil itself, and that the presence of piles may prove detrimental instead of beneficial.
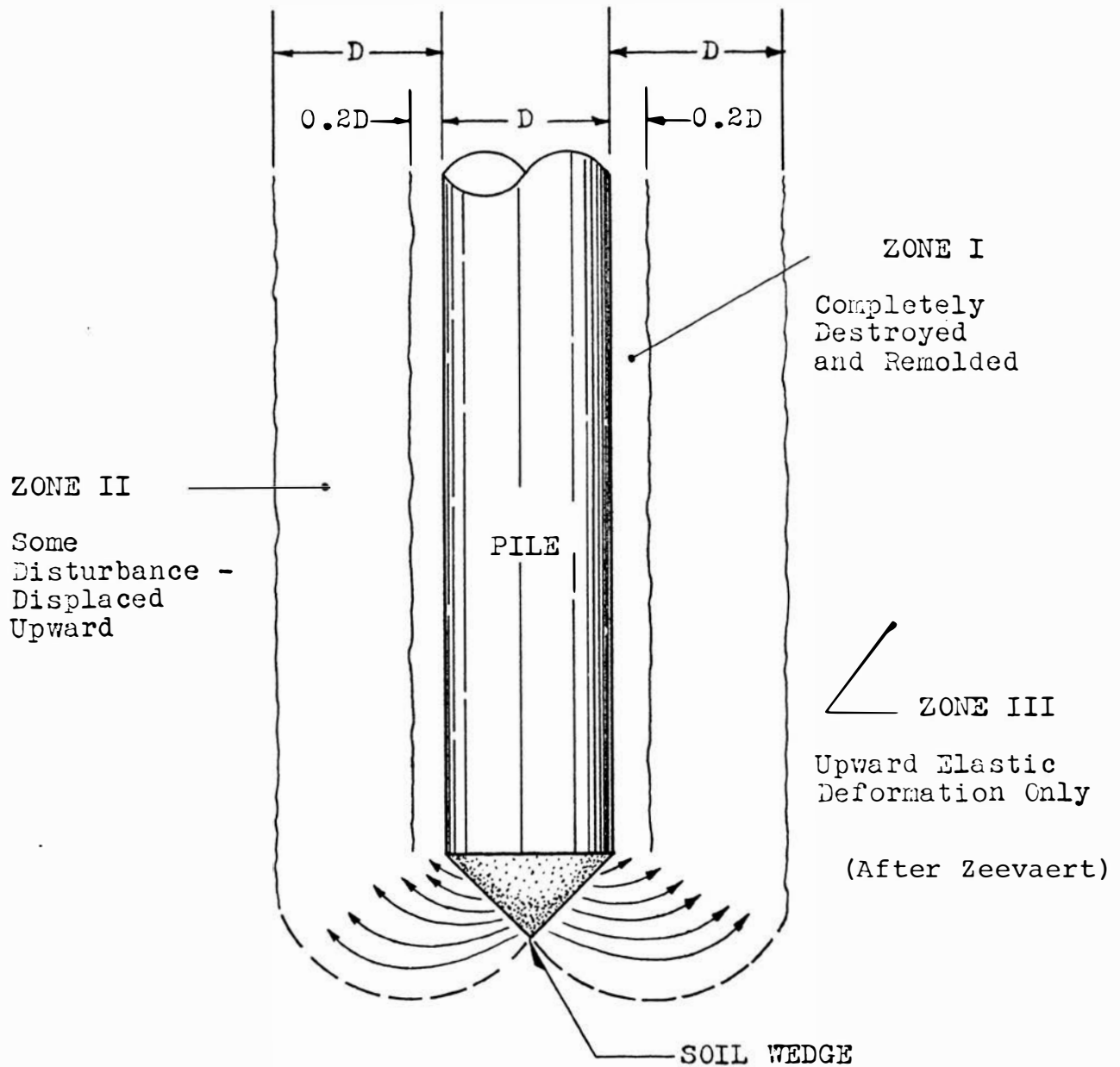
Fig. 1. Zeevaert's concept of soil failure adjacent to pile.

In a study by Cummings, Kerkhoff and Peck (1950) con-
clusions conflicting to those above were drawn. Undisturbed
samples taken around wooden piles driven in a strata of
Detroit clay indicated that the region of disturbance is limit-
ed to a zone within a few inches of the pile, and that the
main volume of the soil mass is undisturbed. The Detroit clay
consists of a layer of stratified or varved clay, and a layer
of relatively homogenous clay. It was discovered that after
a short time the strength of the homogeneous clay close to
the pile showed an increase in strength, and the varved clay
showed a slight increase in shear strength with the passage
of time. Moisture content determinations made prior to and
following pile driving showed no indication of a migration of
moisture from the clay adjacent to the pile. From these
findings the authors concluded that it was nearly impossible
to sustain a settlement of the surface near the driven piles.

In the study above the first samples were taken one month
after driving. This time lapse was used as an argument against
the validity of the conclusions on the basis that "set-up", or
thixotropy, could have a sizable effect within that period of
time. The practical concept of set-up is that water is
released by the sudden disturbance of the clay soil during
driving, and this water tends to lubricate the pile as driving
is continued. Readjustment of the disturbed clay may occur
within the first one or two days after driving, and partially
explains the usual difficulty met if redriving is necessary
(Legget,1950). It is interesting to note that the water

content of the clay studies by Cummings, et. al. lay almost midway between the plastic and liquid limit of the soil.

Legget (1950) indicated that in many other cases cited, the soils under consideration were very near, at, or above their liquid limit, and that in the final analysis the observed disturbance may depend on the sensitivity and liquidity index of the soil.

Conclusions similar to those expressed by Cummings, et. al. were drawn from a study by Holtz and Lowitz (1965). Their tests in a lean clay with low sensitivity showed no significant loss in shear strength of the soil adjacent to the pile after driving. They concluded that pile driving did not produce shear strength and compressibility changes that would cause detrimental settlement to the structure. The vane shear tests performed immediately after driving were three feet from the pile, and possibly outside any zone of disturbance. Subsequent tests 1 1/2 feet from the pile were conducted six weeks later, possibly after a thixotropic regain of strength.

In contrast, a recent investigation by Orrje and Broms (1967) with displacement piles in a Swedish quick clay gave results in agreement with Casagrande's theory. The sensitivity ratios of the clay ranged from 20 to 50 and the liquidity index was near one. The vane shear strength decreased within a region extending outward from the pile wall a distance 1.5 times the pile diameter. A similar decrease in strength occurred within a pile group with a spacing between piles less than four pile diameters. The opposing results of these latter

two case histories further suggest that the sensitivity of the soil has a definite bearing on the amount and extent of disturbance created by pile driving.

In a study by Rutledge (1948), not directly concerned with piles, it was found that a semi-logarithmic plot of the compressive strength versus water content relation is a curve parallel to the virgin portion of the consolidated clays. Based on this relation, Rutledge plotted results of water content determinations and shear strength tests from the study by Cummings, et. al., and showed that the soil near the pile was in a condition intermediate between complete remolding and an undisturbed state. In short, he implied that Casagrande's theory may be a bit excessive,and Cummings' a bit conservative, with the true situation lying somewhere between, depending on the physical properties of the clay.

The latest and possibly the most comprehensive studies on soil-pile interaction were conducted by Airhart (1967). He performed static and dynamic loading tests on fully instrumented piles equipped with pressure transducers, strain gages and accelerometers. The relative extent of the failure mechanisms was established through both direct and indirect means. Borings were made at the site prior to and following the driving operation at different distances radially from the pile. Evaluations of the changes in shear strength, void ratio, water content and unit weight of the soil were made from those borings. In addition, X-ray absorption techniques were used to indirectly observe changes in the soil structure around the

pile by taking a "side-view" of the location after the driving process.

In Airhart's final analysis, the annular ring of disturbed soil around the pile was divided into two distinct zones of local and general shear. (See Fig. 2). The zone of local shear represents that region closest to the pile, and the soil within that zone is brought to a complete plastic failure with its structure being destroyed. He observed this zone to extend outward to a distance 3.5 times the radius of the pile from the pile wall. He further divided the first zone to include a dynamic flow zone in which the soil is carried downward by the pile. This sub-zone extends to roughly three fourths of the pile radius from the pile face. The general shear region, or that extending beyond the local shear zone, is an area in which the soil is stressed wholly within the elastic range of the soil. Comparison of Figs. 1 and 2 indicates some similarity between the theories of Airhart and Zeevaert.

Airhart's methods of direct and indirect observation offer excellent possibilities for continued research on the action of clays near newly driven piles. There is no doubt that in some soils disturbance initiated by pile driving may be excessive to the point of being detrimental to the service of the foundation and structure.

2. Pore water pressures induced by pile driving

A phenomenom which has been observed many times during the loading of saturated cohesive soils is the build-up of

The page shows a figure with concentric circles labeled with zones and radii measurements.
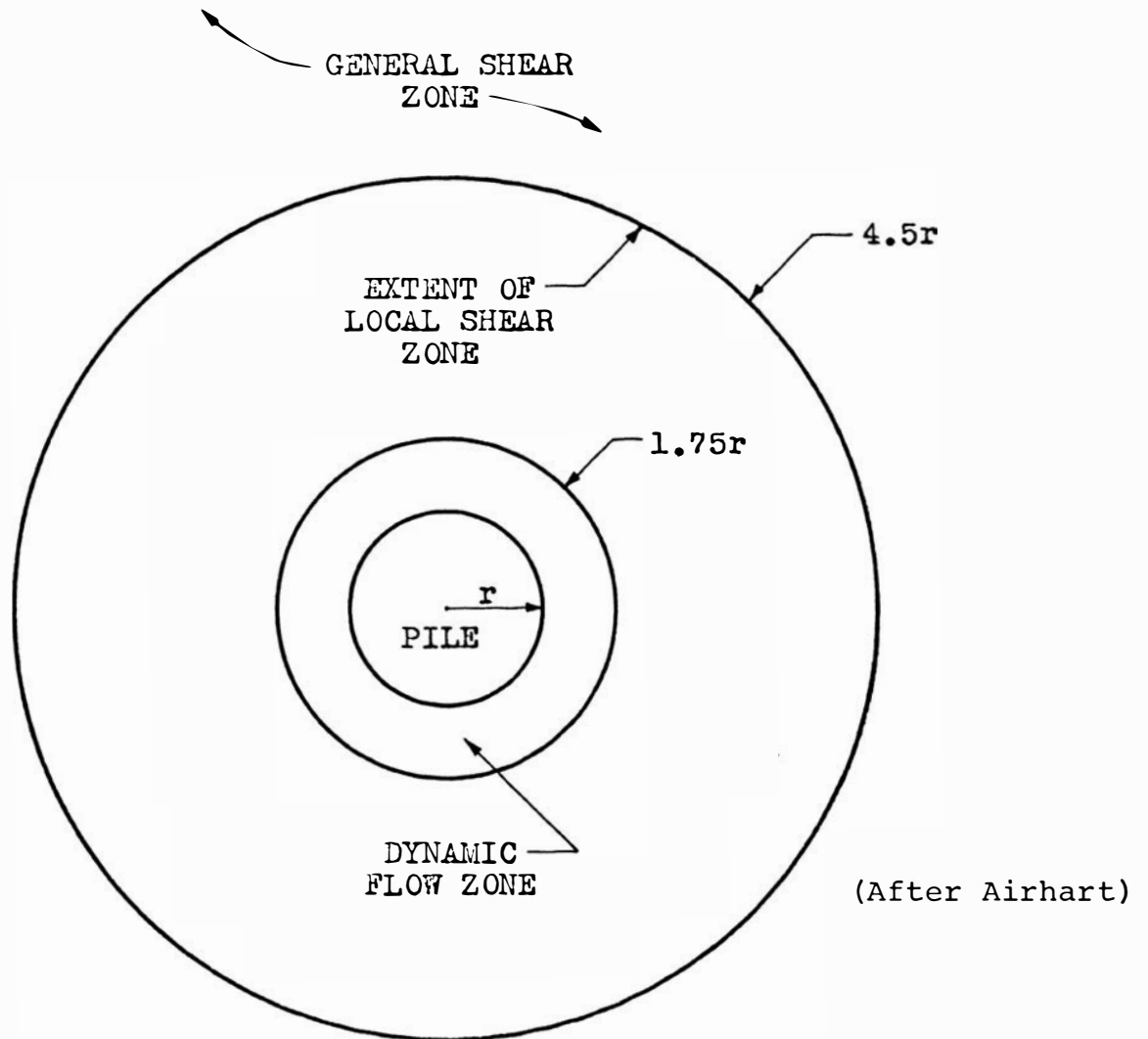
Fig. 2.   Airhart's concept of soil failure
adjacent to pile

pressure in the pore water of the system. It has been shown that this pore water pressure is a function of load application, time, soil structure, sensitivity and stress history. This phenomenon is also noted with soil loading as a result of pile driving. The disturbance of the saturated soil structure around the surface of the pile results in a change in void ratio and subsequent change in pore water pressure. These changes in pressure further affect the complicated mechanisms of load response in a pile-soil system. After loading, the difference between the pore water pressure in the immediate vicinity of the pile and in the neighboring soil mass is reduced by a process called dissipation[2]. It is believed that the magnitude and duration of this pressure difference is a function of the extent of soil disturbances, and the permeability and diffusivity of the soil (Airhart et. al., 1967)

Seed and Reese (1957), Soderberg (1962), and Airhart (1967) have investigated the excess pore water pressures developed during the pile driving process, and have sought to correlate this dissipation with the load-carrying capacity of the pile.

Seed and Reese (1957), using full scale hollow steel piles instrumented with pore water pressure transducers, observed excellent agreement between the rate of increase in shear

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[2]Pore pressure dissipation refers to the reduction in the value of pore water pressure in a given volume of soil by the migration of water.

strength of the soil and the rate of decrease of excess pore pressures. They reported that the rate of increase of the bearing capacity of the pile lagged the processes above, but suggested there was a time lag in the development of bond between the soil and the pile. To the practicing engineer, these theories imply that an accurate method of predicting excess pore pressure dissipation will give the magnitude of the time interval between pile driving and the time when the full supporting capacity of the pile is developed. More valid load tests can be performed after this time interval.

Seed and Reese (1957) extended the concept of shear diffusion to the radial dissipation of excess hydrostatic pressure from the surface of a pile. Assuming an instantaneous surface source of strength, Q, acting over the surface, and that the soil into which the pile is driven behaves as a viscous fluid, they derived an expression for the pore water pressure as a function of time as follows:

$$u = \frac{Q}{4\pi Kt}$$

> where Q = strength of instantaneous surface
> source at t = 0,
>
> K = diffusivity constant,
>
> t = time from driving to the time in
> question, and
>
> u = excess pore water pressure at time t.

An inspection of the equation indicates that the solution is independent of the radius of the pile. In addition, the assumptions used in the derivation imply that the material close to the pile has uniform diffusion characteristics, and

that the source of excess pore water pressure exists in the immediate area of the pile surface. It was suggested in the previous section that the soil exterior to the pile differs in character with radial distance from the pile-soil interface. It follows that an area of remolding, however small, does exist and that this area acts as a source of excess pressure.

In their comparison of pore pressure readings and results of the equation cited before, Seed and Reese report that their equation gives somewhat erroneous answers for short times, but that the correlation increases slightly with increases in time, provided their transducer measurements were accurate.

Soderberg (1962) made a significant step in realizing the physical situation of pore pressures around piles. He restricted his solution to one assumed initial excess pore pressure source, but he expected the dissipation curves to follow either of two basic assumptions regarding the behavior of soil as an engineering material. The first assumption is that the soil acts as a perfectly elastic-plastic material, and the second is that, at the instant of driving, the soil acts as a viscous liquid. His solutions are too lengthy for reproduction in this paper, but it may be well to note that the solutions based on each assumption plot very close on a graph by the coefficient of consolidation as the abscissa. The viscous fluid assumption gives slightly higher values for the percentage dissipation. Most important is the fact that in Soderberg's final analysis, the lateral dimension of the pile and the coefficient of consolidation of the soil

determines the time required for pore water dissipation. It does not, however, recognize the effect of the disruption of the existing soil structure in the region immediately surrounding the pile which is similar to the drawback associated with Seed and Reese's work.

In a study for the Texas Transportation Institute, Airhart, Hirsch and Coyle (1967) do not average the soil properties of the various zones of disturbance over the extent of influence, as done in the works cited previously. Instead, they continued from the study by Airhart (1967) and assumed the types and extents of disturbance as put forth in the previous section, i.e., zones of local and general shear. Their derivation is based on a mathematical model having two source areas of heat diffusion with differing intensities, similar to the zones of disturbance. Their final equation is expressed as follows:

$$Q = \frac{4u_1 K_2 k_1 t}{K_1 a^2}$$

where Q = excess pore water pressure in the region of local shear failure,

u_1 = excess pore 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 in question, and

a = radial dimension of region of local shear failure (4.5 r from center of the pile)

An examination of this expression shows that it involves the permeability and diffusion properties of the soil in the regions of disturbance, and the radial extent of local shear failure, the zone with which researchers are generally most concerned. The latter is a function of both the lateral dimensions of the pile and the plastic strength of the clay soil, as discussed previously.

While the expressions provided by Soderberg, and Reese and Seed work well for long term pressure dissipations, the equation derived by Airhart, et. al. seems to accurately predict the dissipation process for a much wider time interval. Soderberg's results, when plotted on a semi-logarithmic scale, show rapid initial dissipation with the process slowing down gradually with time. For time periods closely following pile driving, Airhart's mathematical model indicates a linear dissipation process, which is slower compared to Soderberg's results. Airhart, et. al. believe their model more closely represents the physical situation, where driving produces a zone of local shear and thus an area of remolded soil which acts to slow down the dissipation process shortly after driving, instead of increased dissipation during the very early stages, as other theories suggest.

3. Load transfer by a single friction pile

Load transfer is, as the term implies, the transfer of the concentrated load at the top of the pile to the soil surrounding the pile, and to the soil located beneath the pile

tip. A large amount of study has been devoted to the sub-
ject in recent years, and many of the results have caused
ideas that were once considered basic truths about pile load
transfer to be discarded.

In this section load transfer at the tip will also be
considered since field load tests necessarily include some
tip loading, however small, which interacts with the side
friction in supporting the pile load.

The most successful method of determining load transfer
to date is the incorporation of strain gages mounted along
the length of the instrumented pile to be drived. The strain
gages measure the strain, and thus the load, in the pile at
various depths. That load is subtracted from the imposed load
at the pile butt, and the resulting load thus represents the
load that has been transferred to the soil above the depth
in question (D'Appolonia and Hribar, 1963).

Johannessen and Bjerrum (1965) caution that a single
report of a set of poor measurements can give rise to a whole
new theory of load transfer. For this reason the instrumen-
tation for test piles should be investigated as much as
possible before the tests are considered reliable. Heavy duty
SR-4 strain gages are considered the best devices for measur-
ing strains in steel pile walls at present (Crandall, 1948).

The two basic types of situations in which piles are
used are those where piles transmit the imposed butt load
through a compressible strata to a layer of material suited to
carry the load, or the case where piles transmit the imposed

load to the surrounding soil through skin friction, with the pile tip not resting on hard strata (Chellis, 1951). In the context of this thesis, skin friction is considered a stress resulting from the mutual effect of soils and structures in the transmission of forces from one to the other across a contact surface.

Johannessen and Bjerrum (1965) give simple mechanical concepts to the situations cited above. In the first case, the point bearing pile, as the pile is loaded the maximum movement of the pile with respect to the soil occurs at the top, or butt. Therefore the load transfer should lead to the ultimate skin friction at the upper portion of the pile, with the transfer spreading downward as the load is increased. When sufficient movement occurs along the pile to mobilize almost all the skin friction, the added load is transferred to the tip. In the latter instance, the friction pile, all of the above concepts apply, but the tip can move more easily and deformations can be greater. All of these concepts can be modified by the soil profile, the stiffness of the pile or tip effects in dilatant soils; therefore load transfer may vary from zero to 100%, and even be negative, as in the case of the soil "dragging" the pile downward.

Sowers and Martin (1961) give a different explanation for the unequal load distribution along a friction pile. From the theory of elasticity, vertical displacement decreases with depth for a vertically applied point load on a homogeneous semi-infinite elastic solid. They reason that with a rigid

pile in contact with the soil the full pile length, a constant
vertical strain from top to bottom will necessitate a greater
load transfer at the bottom. In short, they suggest that in
order to keep the strain constant with depth, more load is
transferred at the bottom, with the failure progressing up-
ward and the distribution becoming more uniform as the ulti-
mate load is approached.

A completely rigid pile is practically non-existant and
elastic compression of the pile allows load transfer near the
top. In a study by Broms and Hellman (1968) with an instru-
mented pile, more than 50% of the total skin friction developed
before any movement resulted at the tip. This indicates the
theory proposed by Sowers and Martin may not fully apply to a
soil-pile system.

Van Weele (1957) proposes that the settlement of the pile
butt is due to 1) elastic compression in the pile, 2) elastic
compression of the soil layers below the pile tip, 3) residual
settlement of the subsoil, and 4) any small bending that may
take place in the pile. In addition, he theorized that the
total skin friction remains essentially the same after a cer-
tain depth, and that the value of the coefficient of sub-
grade reaction, k, depends on the dimensions of the pile point.

Thurman and D'Appolonia (1965) found that a reduction in
k reduces the total friction force, and thus the ultimate
load of the pile. They noted an increase of butt movement
even with low loads if k is small, because a larger load is
transmitted along the full length of the pile. Their equations

predicting load transfer at the pile tip show a decreasing difference between ultimate loads of friction and end-bearing piles as overburden becomes a dominant factor. As an example, they computed the ultimate load of a 90 foot pile in end-bearing as 50,000 psi, which is greater than the pile strength. This suggests that placing piles to deep bedrock may not be justifiable in some instances.

Mansur and Kaufman (1958) conducted load tests with tapered pipe piles driven from 45 feet to 80 feet through silt onto sand. They reported very low load transfer near the tip. Seed and Reese (1957) measured load transfer right down to the pile point in their tests with piles driven into San Francisco "Bay Mud". This implies that tip transfer might be dependent upon the soil on which the tip rests. In line with this implication, Davisson (1956) speculates that low load transfer at the tip occurs in soils that dilate with shearing, such as granular soils, stiff clay, or silt.

Mohan, Jain and Kumar (1963) carried out load tests with piles in medium sand, silt and clay, and found the skin friction to be maximum at mid-depth, although the soils at the bottom of the pile possessed greater shear strength, where greater load transfer might be expected. Schlitt (1952) has reported similar results.

Peck and Davisson (1963) concluded that load transfer is also a function of the pile material. Load tests on single piles showed that a hollow steel pipe pile developed 35% of the vane shear strength as skin friction whereas a wooden pile

developed 70% of the vane shear. While these percentage figures
may be unique for each test site, Peck believes this deviation
is due to differences in the roughness and elastic moduli of
the piles, and the ability of pore water to migrate into the
wood grain, thus aiding excess pore water pressure dissipation.
By nature, wooden piles have a slight taper which might in-
crease load transfer at the pile sides by the added vertical
component of support. Tomlinson (1957) found that pile taper
greatly increases the ultimate pile load in stiffer clays,
but the effects are reduced as the consistency of the clay
becomes softer.

It is evident that pile load transfer is a function of
the shear strength, dilatancy aspects, and coefficient of
subgrade reaction of the soil, along with the particular depth
under consideration. Load transfer is also dependent on the
roughness, taper, elastic compression and possibly other
physical characteristics of the pile.

These case histories serve to show that the combination
of full scale instrumented piles and trained, experienced
observers has proved invaluable in contributing to the field
of knowledge concerning soil-pile interaction.

B. Studies with Model Piles in Cohesive Soils

Testing with model piles in relatively inexpensive, can
be conducted in less time, and produces results quickly with
greater control over soil conditions and equipment. In the
context of this discussion, model piles will be considered
generally one inch or less in diameter with length to diameter

ratios equivalent to commercial piling used in construction.

An ultimate problem encountered when using model piles is how well the data obtained will apply to the full-scale physical situation. Several notable studies with single model piles have been conducted to answer this question and to investigate other aspects. Kondner (1962) took a dimensional analysis approach to the solution of the problem. Using the Buckingham Pi theorem (Murphy, 1950) and certain parameters of the model system, such as pile length, soil shear strength, and ultimate load, he derived equations containing dimensionless functions which were considered to mathematically describe the action of pile foundations. In the single model pile tests it was assumed that the piles would develop skin friction equal to the shear strength of the soil as determined by the unconfined compression test. Kondner noted that the pile "plunged" at an ultimate load equal to the soil shear strength times the submerged surface area of the pile, one of his derived equations. No other definition of failure, besides plunging, was given. It may have been possible to experience excessive settlement, another form of failure, under equilibrium conditions at smaller loads than those that made the pile plunge.

In Kondner's (1962) tests the soil was a compacted clay with a maximum shear strength of 650 pounds per square foot. The model foundations were one-eighth inch diameter brass piles. The piles were tested singularly and in groups of varying configurations to evaluate group efficiency. Peck and

Davisson (1963) applied Kondner's group efficiency equations to several tests on model pile groups reported by Ghanem (1953) and concluded that the expressions were over-conservative by a factor of 2 or more. Peck and Davisson also stated that in field tests with single steel piles, as little as 37% of the soil shear strength was mobilized as skin friction. These discrepancies infer that Kondner has neglected such factors as soil disturbance and thixotropy. It appears that dimensional analyses and purely mathematical models have yet to be developed which will fully describe the action of piles in clay.

Cooke and Whitaker (1961) conducted a model study in brown London clay using 3/4 inch pile shafts with differing lengths and enlarged bases of 2,3, and 4 times the shaft diameter, one of the few tests of this type. Their tests showed that the resistance of the shaft was mobilized at very small penetration movements (about 0.5% of shaft diameter), while penetrations of 10% to 15% of the enlarged base diameter were necessary to develop ultimate bearing capacities for the base. These results suggest that the shaft resistance for single piles is more a function of soil-pile adhesion, requiring very small failure strains on the order of 0.004 inch. On the other hand, base failure is a function of the soil shear strength characteristics, and requires larger failure strains. These trends may also suggest that failure strains are dependent on pile shape.

Another, very significant test procedure has been investigated by Whitaker and Cooke (1961). They compared differing modes of load application to determine which was most consistent and gave the highest correlation with field tests in clay. They used maintained load increments (the standard load test), a constantly increasing load applied by jack, and a large dead load causing the pile to plunge to refusal. The last test was conducted by measuring the force necessary to maintain a constant penetration rate for the pile. The latter method was found to give the best correlation between field load tests with square concrete piles and model tests with 1/4 inch brass piles. This correlation was judged by the similarity between the load-settlement curves and reproducibility of results. Their subsequent laboratory tests showed that for a wide variation in loading rates, the constant rate of penetration test (hereafter denoted as the CRP test) gave ultimate loads varying only 4 per cent. In the field test, two identical piles were driven at 0.064 and 0.031 inch per minute and both failed at 132 tons with 0.35 inch of penetration.

Whitaker and Cooke explain that in the maintained load test (ML), the first mentioned above, the loads are supposed to be applied slowly to represent equilibrium conditions. In the field, the load test is conducted too fast, generally for economic reasons, and the end point for settlement is difficult to determine. In the CRP test the ultimate load on the jack can be determined easily and quickly, and the design load can be chosen. The settlement from the design load can then

be determined by the ML test.

Halcrow and Sharman (1961) were very enthusiastic with the interchange capabilities of the CRP and ML forms of testing, and believed that the CRP method could be written into many construction specifications dealing with friction piles in clay.

Holtz and Lowitz (1965) studied the changes in compressibility of lean clay due to the driving of 1/2 inch diameter wooden dowels (model piles) into Shelby tube samples of the clay. The samples with the piles showed little overall change in compression index, but a totally remolded sample showed a similar small change. They observed a 100% increase in driving resistance one day after initial driving, and at times this increase took place in only one hour. These observations indicate that model piles create soil disturbance and experience gains in ultimate load capacity just as do full-scale piles in the field.

A recent series of tests with three eighths inch and one half inch smooth and rough steel model piles has been carried out by Coyle and Reese (1966). The piles were placed in 2 1/2 inch diameter samples which were then consolidated for about four days under various confining pressures in a large triaxial cell to simulate depth. Using a penetration rate of 0.06 inch per minute in the CRP test method, it was found there was a straight-line relationship between load transfer and settlement for small penetrations.

Coyle and Reese discovered that the maximum load transfer is a function of confining pressure, within certain limits. Above confining pressures of 45 psi there was little difference between the ultimate loads developed by the rough and smooth piles. It is believed the higher confining pressures forced a soil-to-soil failure away from the pile wall, and the load was a function of the soil's shear strength. At lower confining pressures the piles developed less than the vane shear strength of the soil. In this instance failure took place at the soil-pile interface and the maximum load transferred was a function of the adhesion between the pile and soil. At confining pressures less than 45 psi, the rough piles developed slightly higher adhesion than the smooth piles. Families of curves were developed which show the skin friction mobilized increasing as confining pressure is increased. This compares with Thurman and D'Appolonias' (1965) theories. The results from two smooth piles tested at low confining pressures showed that the smaller pile developed more adhesion.

Ghanem's investigation (1953) produced similar results. In order to evaluate pile group efficiency, it was necessary to compare the ultimate load of a single pile with the average load a similar pile carries as a member of a particular group. It was observed that the single piles developed less than the soil's shear strength as skin friction, but the piles in group action developed the full shear strength of the soil at the group's perimeter and base.

Potyondy (1961) conducted an investigation concerning skin friction between soils and various construction materials. Results from tests with clay and steel plates of varying roughness, simulating steel piles, showed that the skin friction increased with normal load to a limiting point. After this there was no further gain in the skin friction and it remained somewhat lower than the shear strength of the clay. Below this limiting normal stress there was a variation in the density of the clay, due to changes in saturation, pore water pressures, and expulsion of air near the contact area. When the soil was subjected to a 48 hour period of preloading, the limiting normal force decreased slightly and the skin friction closely approached the clay's shear strength as an upper limit. In no case did the initial adhesion equal the shear strength or cohesion of the clay, regardless of normal loading.

Potyondy theorized that skin friction is dependent on the physical state of the clay at the contact area, whether day, moist, semi-fluid, or lubricated. Concerning the construction material, the skin friction depends on the cleanliness, humidity, atmospheric dust, oxides or films present, surface finish, velocity of sliding, contact pressure, temperature, grain size, direction of grain and any static loads or vibrations present. The relative influence of each is unknown at the present time.

Recent static loading tests with model piles conducted by Raba (1968) were of interest because of the newly devised equipment and procedures used. Extruded 2.80 inch diameter

remolded clay samples were encased in a steel jacket to prevent bulging at the bottom while a 1.00 inch diameter smooth hollow aluminum pile, instrumented with three strain gage rosettes, was pushed completely through the sample. No set time interval between pile insertion and load testing was indicated. The static tests were conducted by the CRP method using a penetration rate of 0.05 inch per minute. A circular pilot hole was cut in the sample to guard against radial cracking of the sample while the pile was initially inserted.

In Raba's research program a preliminary study was carried out to determine if an optimum pilot hole diameter exists. The pilot holes were cut with hollow aluminum tubes aligned by a renovated triaxial cell frame. It was concluded that differing pilot hole diameters below 0.75 inches had little or no effect on the ultimate static load. Between pilot hole diameters of 0.75 and 1.00 inch, the maximum load decreased greatly, with the lowest load occurring with a pilot diameter equivalent to the pile diameter, or one inch. The results and procedures of those tests gave rise to several questions. There was some doubt: 1) whether there may be a particular optimum pilot hole diameter, 2) if the ratio of optimum pilot hole size to pile diameter changes with pile size, and 3) if sample size plays a particular role.

It is clear that research incorporating model piles not only reproduces the action of full-scale piles in the field, but adds significant knowledge to the subject of soil-pile interaction through increased flexibility in the evaluation of all variables.

III. THE RESEARCH PROGRAM

A. Introduction

The testing program was selected to study the effects of certain parameters, such as pile size, sample size and pilot hole size, on the behavior of unconfined model friction pile tests. Sedimented samples 1.4 and 2.5 inches in a diameter were prepared with a soil consisting of 40 per cent clay and 60 per cent silt. Assorted sizes of pilot holes were cut in the soil samples after they were encased in lucite jackets. Brass and hollow steel piles 1/4, 1/2, and 3/4 inch in diameter were inserted in the samples using the pilot holes and two guide frames to align the piles vertically.

Results of CRP (constant rate of penetration) tests on the model piles were produced in the form of load versus penetration curves giving the ultimate load transferred from the pile to the soil by skin friction. Vane shear strength values from tests performed next to the piles at definite depths were used as a basis of comparison between the findings of the various load tests. The vane shear tests also offered a means of investigating the variation of shear strength in each sample.

The change in ultimate load carried by the pile with various time intervals between pile insertion and load testing was also noted. In addition, several tests were conducted to measure the effects of different penetration rates in the CRP test.

B. The Soil Used in Testing

Floating pile foundations are normally constructed on softer soils, therefore, the soil used in model testing should be of similar consistency. To simulate field conditions soil with a shear strength of 500 psf or less was desired for testing.

Although the laboratory is equipped with a Vac-Aire extrusion machine capable of producing homogeneous saturated clay samples, the machine had only one size of extrusion template, and two sizes of samples were needed. Homogeneous saturated samples could be made with existing sediment tubes of 2.5 and 1.4 inch diameters. The decision to sediment samples, therefore, dictated that a mixture of silt and clay be used. The testing schedule would not allow adequate time for complete consolidation of a pure clay slurry, whereas, the addition of silt-sized particles would decrease the time required to consolidate samples (Spencer, 1968).

A kaolinitic clay was obtained from a deep mining excavation in Kentucky. Atterberg limits were determined according to ASTM standards (1961), and these values, along with other physical properties of the clay, are reported in Table I. Because the liquid limit and plasticity index were considered high for a normal kaolinite, and the specific gravity somewhat low, an x-ray analysis of the clay was performed for the purpose of further classification. The analysis showed dominant keolinite peaks, indicating a high percentage of this mineral

TABLE I. Physical Properties of the
Soils used in Research

	Clay	Silt and Clay Mixture
Liquid Limit, %	64	28
Plastic Limit, %	31	17
Plasticity Index, %	33	11
Specific Gravity	2.59	2.65
Per cent finer than 2μ	87	33
AASHO Classification	A-7-5(20)	A-6 (8)
Unified Classification	CH	CL

present. A secondary montmorillonite peak occurred which indicated a small amount (probably 7 to 10 per cent) of this mineral was also present, enough to account for the physical properties observed (Grim, 1953).

A hydrometer analysis of the clay indicated that nearly 80 per cent by weight was finer and 2μ. (See Fig. 3). The position of the clay on Casagrande's modified plasticity chart is indicated in Fig. 4.

Silt was obtained by fractional sedimentation of coarse-grained, primary loess taken from the Altonian soil formation in a loess bluff adjacent to the Mississippi River flood plain. The bluff is near French Village in St. Clair County, Illinois, and the loess was a Roxana Type II deposited during Wisconsin glaciation. A hydrometer analysis showed that 10 per cent of the loess was finer than 2μ and 3 per cent was retained on a No. 200 sieve. The remaining 87 per cent was silt-sized.

A series of hydrometer analyses with tap water indicated that calgon solutions of more than 1 per cent concentration would allow flocculation of the silt and clay particles of the loess. After the loess was air dried and pulverized, approximately 90 pounds were placed in a small watering tank with a solution of tap water and 1 per cent calgon. The tank was filled to a depth of 36 cm, corresponding to the height of the water column for hydrometer analyses. The loess and water was stirred briskly with a shovel for five minutes for complete particle separation and agitation. Immediately after stirring
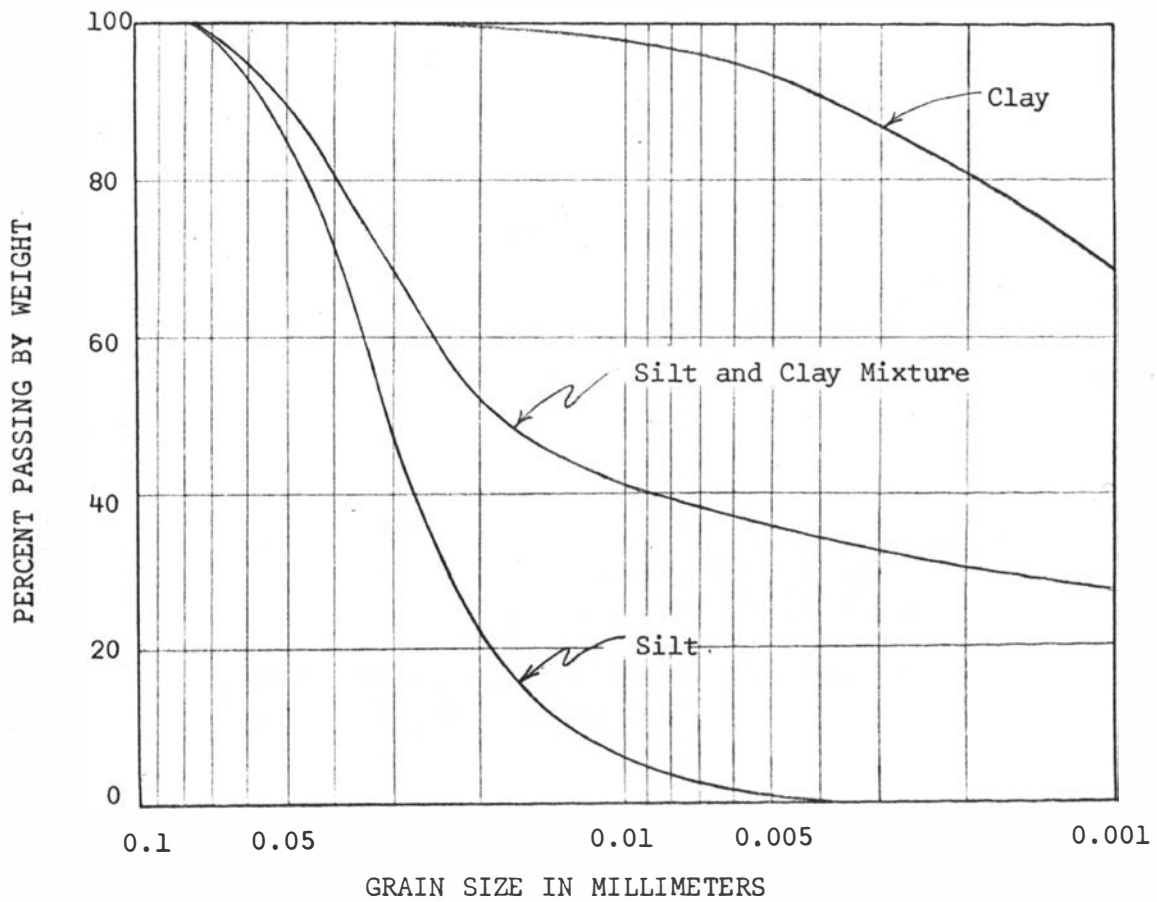
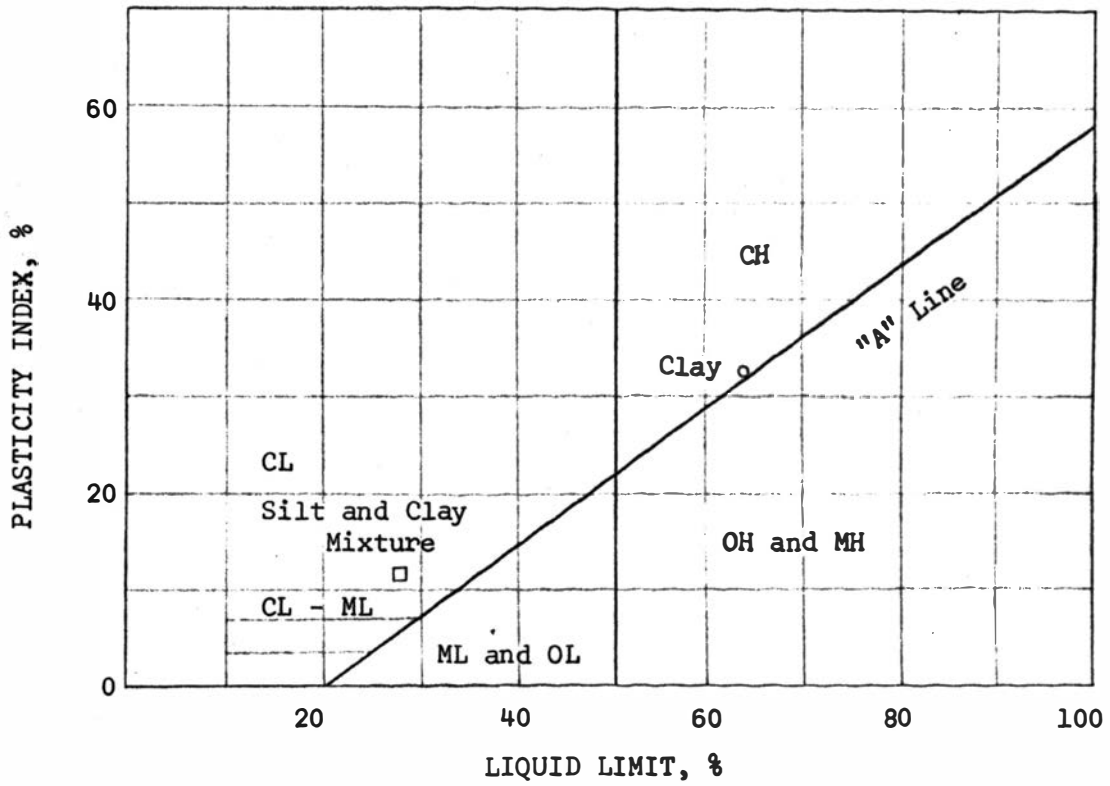Fig. 3.   Grain size distribution curves for
the soils used in the research

Fig. 4. Positions of soils used in research on plasticity chart

a 1000 ml. cylinder was dipped into the tank, and with one sweeping motion from bottom to top, filled to the 36 cm. mark; the purpose being to conduct an analysis to monitor the prog- ress of sedimentation in the tank. The gray color of the silt and brown color of the clay gave evidence when most of the silt settled out, and the clay and water were drawn off the top with a siphon.

The tank was again filled to the mark with tap water and 1 per cent calgon, the sediment stirred from the tank bottom, and the process repeated until the sedimentation analysis indicated less than 1 per cent clay-size particles remaining. The sediment was then retrieved, air-dried, pulverized, and sieved through a No. 200 sieve, and the portion retained on this sieve was rejected. The grain-size distribution of the resulting silt is also indicated in Fig. 3. The specific gravity of the silt particles is 2.70.

C. Soil Sample Preparation

Jackson (1968) has shown that a soil mixture comprised of 60 per cent silt and 40 per cent clay will consolidate rather quickly but exhibit no increase in volume during shearing under undrained conditions. The grain-size distribution of the component soils used by Jackson was actually 72 per cent silt-size particles and 28 per cent clay-size particles based on the MIT Soils Classification (Terzaghi and Peck, 1967).

A mixture of 60 per cent silt and 40 per cent clay was mixed according to the values obtained from the mathematical expression:

$$\text{Per cent silt} = \frac{\text{weight of silt}}{\text{weight of silt + weight of clay}} \times 100 \text{ per cent}$$

A grain-size distribution chart shows that 67 per cent of the resulting mixture was silt-size and 33 per cent was finer than 2μ. (See Fig. 3). The higher percentage of clay-size particles served to prevent dilatancy and possible build-up of negative pore water pressure which would result in too-high shear strength values of the soil. The plasticity characteristics and certain physical properties of the mixture appear in Table I and Fig. 4.

A measured amount of distilled, de-aired water was slowly added to 220 grams of the soil mixture and blended with a metal spatula until a thin slurry ($I_L=5$) was obtained. This slurry was whipped with a commercial food mixer for a minimum of two minutes and transferred to 1.4 inch diameter (I.D.) lucite sediment tanks where the slurry was de-aired with a vacuum line for at least five minutes. (See Photograph 1.) Preliminary testing showed that complete consolidation of the measured amount of slurry occurred in 36 hours under a loading of 2 kg/cm^2. The product was a five inch high sample with an average vane shear strength of 300 psf.

With the strength requirements being met, twenty similar samples were subsequently prepared. The samples were wrapped in clear saran plastic, sealed, and placed in large plastic
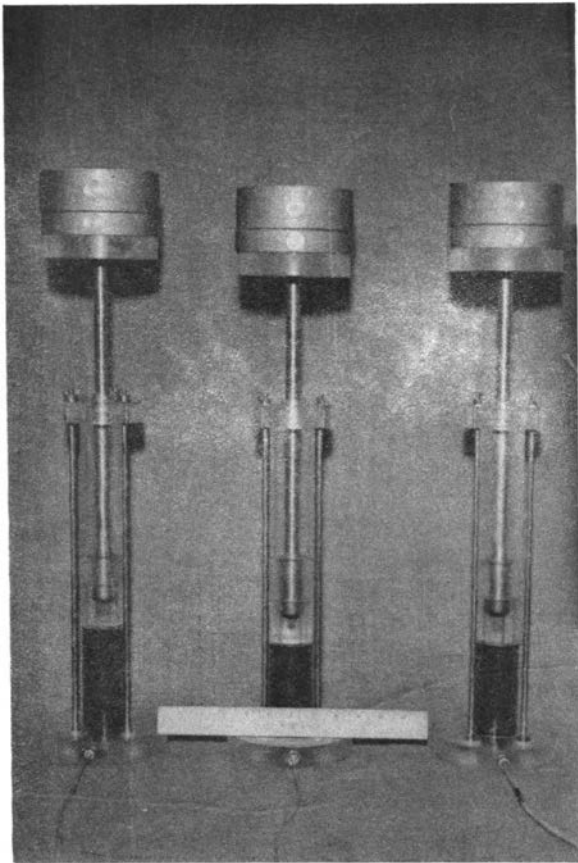
Photograph 1. 1.4 Inch diameter
lucite sedimentation tubes.
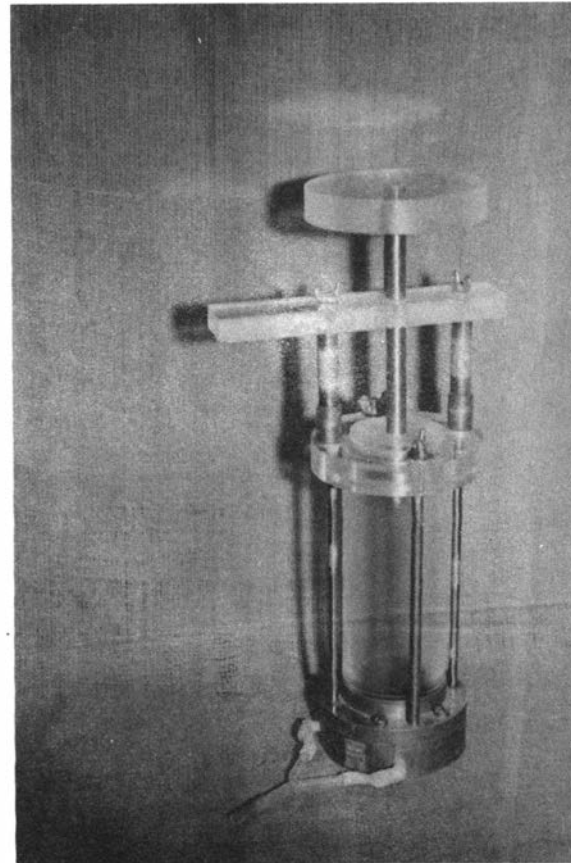
Photograph 2. Soiltest model C-252
consolidation cell with 2.5 inch
diameter lucite sedimentation tube.

bags according to the chronological order in which they were prepared. The samples were stored in a moist room maintained at 70^0F. and near 100 per cent humidity until testing time. Each sample was rotated every few days to minimize moisture migration.

Samples of similar consistency were prepared in a renovated Soiltest model C-252 consolidation cell. It was equipped with an 8 inch high, 2.5 inch diameter (I.D.) lucite sediment tube with a corresponding-sized piston and loading platform. (See Photograph 2.) In order to achieve a consolidation load of 2 kg/cm^2, nearly 140 pounds were placed on the circular loading platform. Photograph 2 also shows the extra steel rods, cast iron collars, and plexiglass alignment bracket which enabled the loading platform and steel piston rod to support the weight without canting to one side, causing the piston to bind against the cylinder wall.

The low height of the 2 1/2 inch sediment tube imposed a restriction on the volume of slurry that could be placed in the tube for consolidation. To obtain samples with a final height of 4 1/2 inches the slurry was prepared to a thicker consistency (I_L=3) than that used in the 1.4 inch samples. The methods of mixing and de-airing the slurry were similar to those discussed previously and the resultant samples were wrapped, stored, and rotated with the smaller samples under similar conditions until needed.

Hydrometer analyses conducted by Jackson showed that no particle segregation occurred during sedimentation of slurries

with liquidity indices equivalent to those indicated above.

D. Equipment and Instrumentation

1. Model piles

Testing the effects of model pile size required: (1) the use of several convenient sizes of piles, (2) that the range of pile diameters be relatively large, and (3) the piles be of sufficient durability to resist buckling and deterioration during the testing program. These criteria were best ful-fulled using piles 1/4, 1/2, and 3/4 inch in diameter, all constructed from metal stock. (See Photograph 3.)

The 1/4 inch pile was obtained in the form of a solid brass welding rod. The rod was cut to an 18 inch section, rounded at the driving end, and beveled 30° to the vertical for one sixteenth inch on the opposite end. The pile was then roughened with coarse garnet paper by hand.

The 1/2 and 3/4 inch piles were made from hollow steel alloy conduit cut into 18 inch long sections. The 3/4 inch pile was actually 0.705 inch in diameter but will be referred to as 3/4 inch for sake of expediency. Each pile was chucked into a metal lathe and polished with coarse emory cloth and steel wool to remove surface dirt and film collected during storage. They were then roughened over the bottom 8 inch section of their length with the edge of a metal file while rotated slowly by the lathe. The depth of striation was about 0.01 inch for the steel piles. The roughened brass pile was
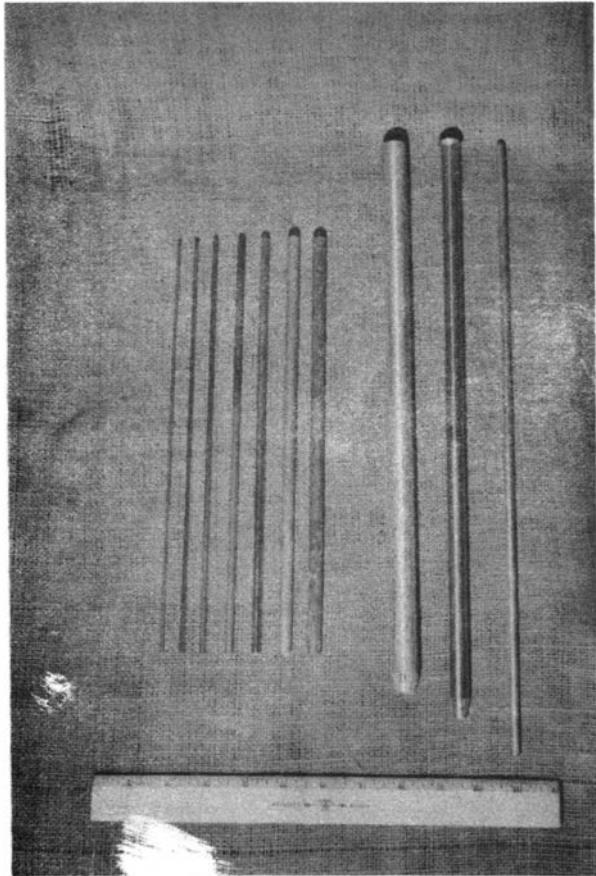
Photograph 3. Assorted brass pilot
hole cutters (left) and model piles
(right).
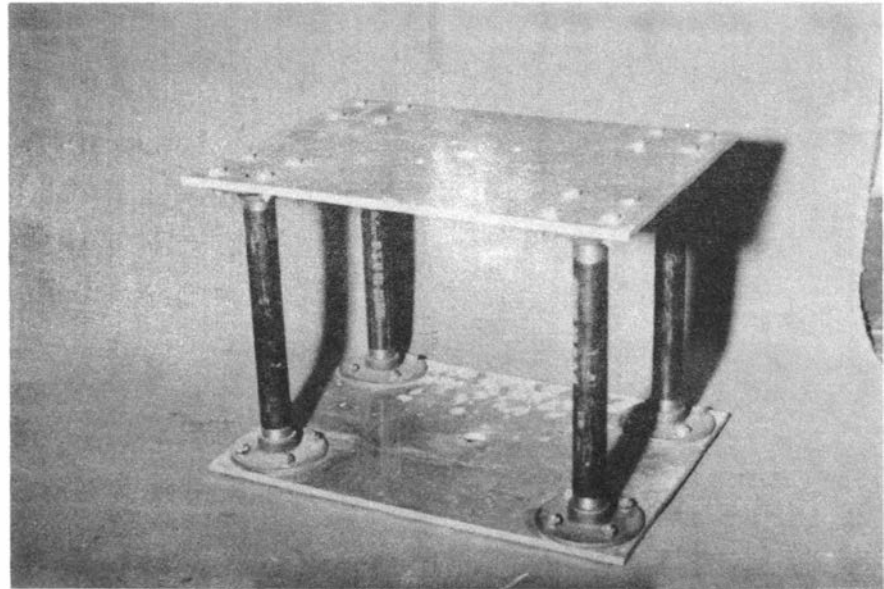
Photograph 4. Heavy-duty aluminum frame.

smoother to the touch. All piles were subjected to air and moisture simultaneously with wet paper toweling for over five days, but no rust or surface film developed on the piles.

Numerous longitudinal slits were cut in the bottom one half inch of the hollow-steel piles with a metal saw and the remaining nibs were hammered inward until they touched, forming a taper about 30° to the vertical. This taper was created to aid in centering the pile in the pilot hole, and eliminate the sharp open end of the hollow piles. An open-ended pile would cut a new pilot hole equal to the pile diameter during insertion and erase the effects of any smaller pilot hole formed previously. Photograph 3 also shows assorted drawn-brass tubing used in cutting pilot holes in the soil samples. The outside diameters and ratios of the brass tube diameters to the three pile diameters for all the tubes are enumerated in Table II.

2. Alignment frames

Initial testing proved that human error involved in both cutting the pilot holes and inserting the piles by hand without a means of mechanical alignment produced canted pilot holes and piles. Misalignment of the pile with respect to the pilot hole would give non-symmetrical zones of soil disturbance. If the piles were not driven in a vertical direction, bearing capacity resistance to driving unrelated to skin friction would be produced, as with a battered pile.

TABLE II.   Sizes of Pilot Hole Cutters Used and Ratios to
Pile Diameters

|		$R_d^\dagger$ ratio for 1/4 inch pile	R_d ratio for 1/2 inch pile
Diameter of brass pilot hole cutter, in inches	1/8	0.50	**
	3/16	0.75	**
	7/32	0.875	**
	1/4	1.00	0.50
	3/8	*	0.75
	7/16	*	0.875
	9/20	*	0.90
	1/2	*	1.00

† R_d = Ratio of pilot hole diameter to pile diameter

* No R_d ratio possible - pilot hole larger than pile

** R_d not used in pilot hole study

A heavy-duty frame measuring 10 inches wide, 15 inches long and 11 inches high was constructed of quarter-inch aluminum plates separated by four 10 inch by 1 inch diameter cast iron pipes and pipe flange fittings. (See Photograph 4.) Three-quarter inch holes were drilled in the center of the plates, but it became evident that this limiting-size hole would not contribute additional alignment to the smaller piles and brass tubing to the extent desired.

A second frame, shown in Photograph 5, was designed to provide adequate alignment for each size of pile and pilot hole cutter. It was connected to the underside of the top plate of the larger frame with four one-quarter inch bolts and wing nuts. This second frame consisted of two three-quarter inch thick plexiglass plates separated by 1 1/2 inch wood blocks. Alignment holes measuring one sixty-fourth inch larger than each of the pilot hole cutters and piles were drilled through both the top and bottom plexiglass plates of the smaller frame. Each alignment hole had its respective four mounting holes such that it could be centered under the larger hole in the metal frame, as needed. The plexiglass frame allowed only a 0.05 inch deviation from the vertical as the pilot hole cutters and piles were pushed through the alignment holes and into the soil samples.
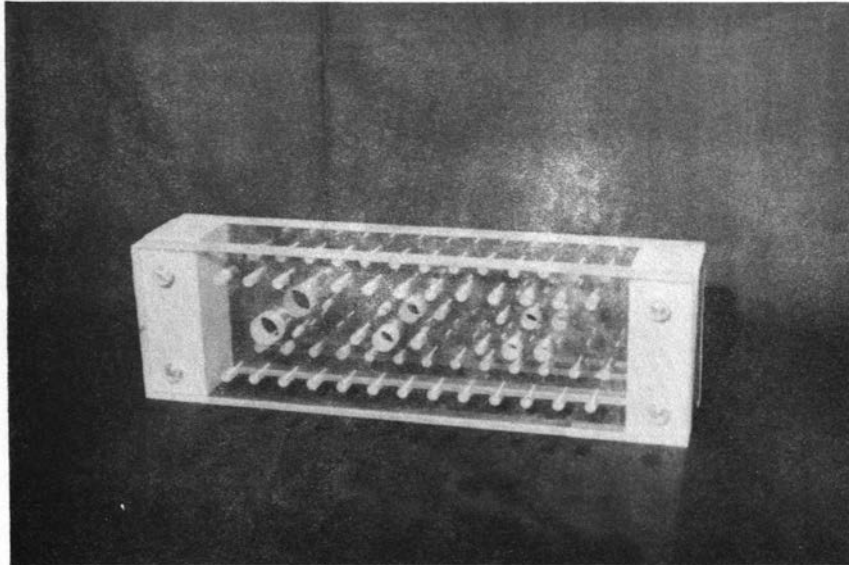
Photograph 5. Plexiglass alignment frame
for model piles and pilot hole cutters.
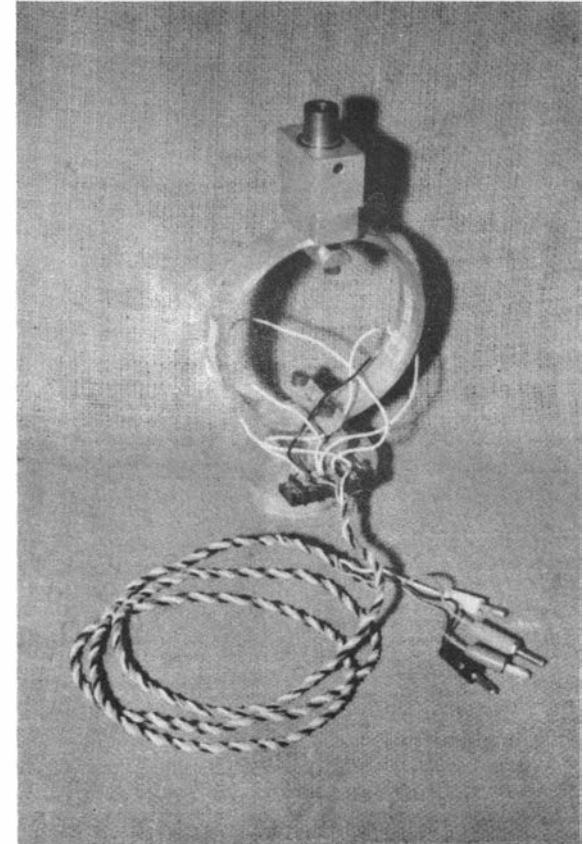
Photograph 6. Steel loading ring
with strain gage arrangement.

3. Rigid soil sample jackets

Two 5 inch long sections of cylindrical plexiglass
tubing, one 2.5 inches in diameter and the other 1.4 inches
in diameter, were used to encase the respective sized soil
samples during the cutting of the pilot hole, pile insertion,
and pile loading. These plexiglass jackets applied a con-
fining pressure to the sample only when or if the sample tend-
ed to expand radically, hence the terminology "unconfined"
model pile test. In short, the jackets served three purposes:
(1) they stopped the soil samples from bulging radially and
cracking during pile insertion, (2) they prevented damage of
the samples subjected to handling through the various testing
procedures, and (3) they helped guard against moisture loss of
the sample during the time interval between pile insertion and
load testing.

4. Load and penetration measuring instruments

Preparatory tests conducted to examine the performance of
the newly developed equipment indicated that the 1/4 inch brass
pile would fail under a load of roughly 2 pounds at a settle-
ment of near 0.004 inch. With the measuring instruments in-
volved only two accurate reading could be recorded before
failure, too few to adequately define the load-settlement
characteristics of the pile. Pile penetration was measured
with a gage sensitive to 0.001 inch,and a 4 1/2 inch diameter
steel loading ring equipped with a 0.0001 inch division

deformation gage indicated only about a pound per division. This sensitivity was considered too low.

An Ames deformation gage with a sensitivity of 0.0001 inch was used thereafter to measure penetration thus increasing ten-fold the accuracy of these readings. The loading ring was dismantled and cleaned thoroughly with steel wool and scouring powder. The exact mid-height of the ring was located on both crescents, both inside and out, and marked. Four BLH A-5-1 bonded wire strain gages were mounted to the ring at those positions with DuPong Grip. They were held in position with C-clamps and hard rubber cushions for 24 hours, enabling the glue to dry completely with the wire gages in intimate contact with the ring. Later, the gages were sprayed with Gage Cote, a clear waterproofing material. (See Photograph 6.)

The gages were connected in a wheatstone bridge arrangement so that the change in electrical resistance of the two gages due to shortening of the inner fibers of the ring, dependent on the compression of the ring under load, would be averaged. The same wiring procedure was followed for the gages in tension on the outer fibers of the ring, which were elongated. This arrangement provided automatic compensation for any possible eccentric loading on the ring. The bridge leads were connected to a Budd Model HW-1 digital strain indicator with a deformation sensitivity of 1 micro-inch per inch.

Calibration of the ring with the strain indicator yielded a constant load factor of exactly 25 micro-inches per inch per

kilogram, or a maximum equivalent load sensitivity of 0.088 pound. Changes of temperature in the laboratory caused the zero-load strain reading from the gages to shift considerably as a result of thermal expansion of the loading ring. A total of ten calibration checks were made during the testing program, and although the zero-load strain varied, the calibration curves remained perfectly parallel. This system of instrumentation allowed the recording of six to eight accurate load-penetration readings before the ultimate load of the piles was reached.

5. Vane shear device

A Farnell model 280 laboratory vane shear device was used to measure the shear strength of the soil. These values of shear strength were related to the skin friction resistance that was mobilized by the soil and measured with a load ring. A calibrated spring which applies 3 inch-pounds of torque with an angular movement of 180° was chosen for its applicability in softer soils. The height, diameter, and thickness of the four-winged vane used was 13.0 mm, 12.8 mm, and 0.9mm, respectively.

E. Laboratory Test Procedures

The first phase of the testing program was devoted to studying the effects of different sized pilot holes bored for each pile while investigating the load transfer aspects of each pile-soil system.

1. <u>Sample encasement and pile insertion</u>

Soil samples were chosen for testing from the curing room in the chronological order of preparation and storage to minimize time-dependent thixotropic effects. After the saran wrapping was removed from the sample the plexiglass jackets were slipped downward over the samples. The 2.5 inch jacket shaved a bit of the sample from the sides during encasement and fit tightly. Damp paper toweling was wrapped around the 1.4 inch samples to fill a small annular space between the smaller jacket and sample.

The jacketed samples were centered over the hole in the bottom plate of the metal frame and the plexiglass frame was adjusted with the desired pilot cutter guide hole centered under the corresponding hole in the upper plate. The pilot holes were cut by pushing the chosen brass tubes through the soil sample while being guided by the plexiglass frame. The tubes were pulled on through the sample from underneath the frame so as not to reverse the direction of remolding. In several tests no pilot hole was used. The plexiglass frame was then adjusted to guide the particular pile being used and the pile was inserted into the sample with a slow constant push by hand until about one inch of the pile projected below the sample. The height of the sample was considered the depth of pile embedment. Saran was wrapped around the sample top and pile to avoid moisture loss and system was allowed to set for 90 minutes. The literature suggested no precedence

for this waiting period, which was chosen arbitrarily to allow dissipation of pore pressure created by pile insertion. After the waiting period the metal frame was centered in the Farnell compression machine equipped with the loading ring described in the previous section. A two inch spacer was situated below the frame to allow free vertical movement of the pile tip. (See Photograph 7.)

2. Pile load testing

Load tests were conducted using the CRP test method, accomplished by measuring the load necessary to maintain a constant rate of penetration imposed on the pile. This method of testing has been used with success by Whitaker and Cooke (1961), Coyle and Reese (1966), and Raba (1968).

Failure was defined as the point at which the load on the pile failed to increase or began to decrease. Prior testing showed that failure took place between 0.0015 and 0.0075 inch, therefore a constant penetration rate of 0.002 inch per minute was chosen, with failure taking place in about three minutes.

3. Shear strength testing

After the pile load test the pile was extracted and vane shear tests were conducted throughout the entire sample to investigate changes in shear strength with depth. The vane spring was rotated 10° per minute, within the limits suggested by Goughnour and Sallberg (1964) for tests in soils of low
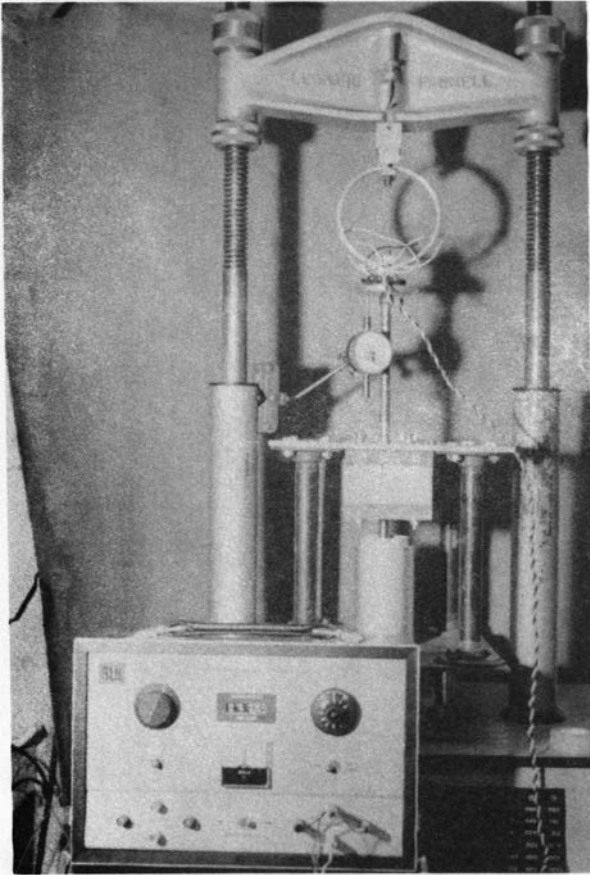
Photograph 7. Pile loading system and instrumentation.
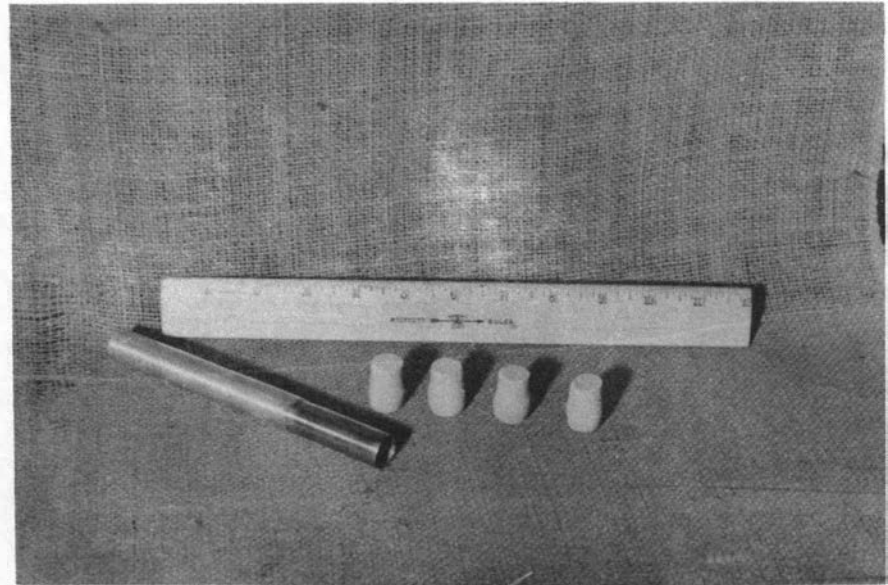
Photograph 8. Miniature "Shelby" tube and several failed samples from unconfined compression tests.

plasticity. Six inch long "dummy" piles of each diameter were available for insertion into the sample where the original pile had been to prevent soil from bulging into a vacant space during the vane shear tests.

In the 2.5 inch samples, there was ample room next to all sizes of dummy piles to conduct two vane shear tests at each of four depths. Vane tests were also carried out at each of eight depths in some samples to obtain shear strength versus depth relationships. (See Fig. 5.) A moisture content determination was made at each position of a vane shear test.

Before the 3/4 inch pile was tested in the 1.4 inch samples, the vane shear tests were performed in the middle of the sample. The remolded areas were then cut from the sample as a 1/2 inch diameter pilot hole was formed by the usual procedure. For the 1/4 and 1/2 inch piles in the 1.4 inch samples, there was too little space to conduct vane shear tests between the dummy pile and the jacket wall. Any vane tests performed at mid-sample before the pile test would have remolded an area outside the pile perimeter. Instead, the load test was conducted as usual, whereupon the sample moisture contents at selected depths were correlated with vane shear - water content relationships derived from 1.4 inch samples.

4. Correlation of vane shear to unconfined compressive strength

In most soil-pile adhesion studies the shear strength of the soil is expressed in terms of the unconfined compression
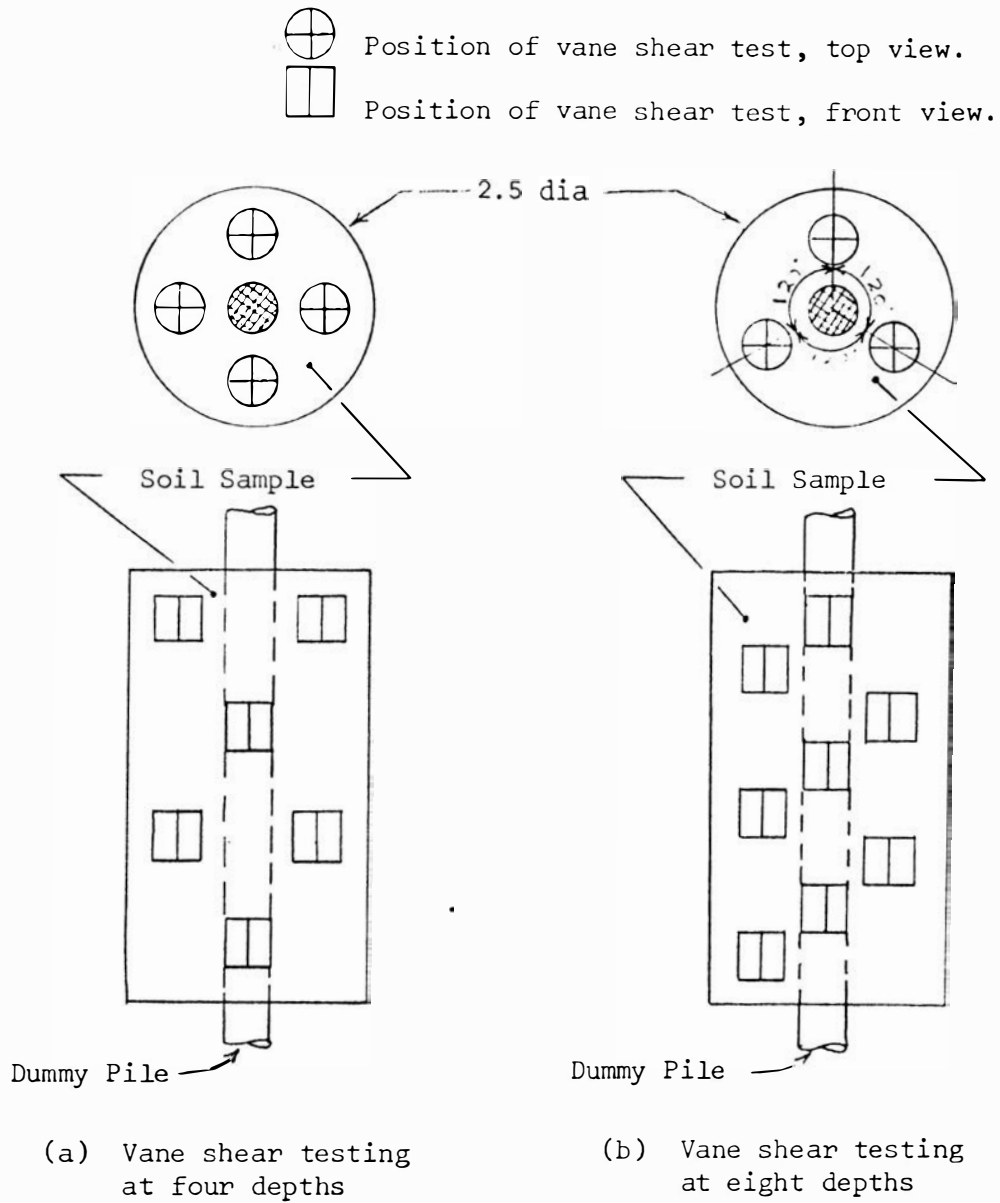
⊕ Position of vane shear test, top view.

⊓ Position of vane shear test, front view.

─── 2.5 dia ───

Soil Sample   Soil Sample

Dummy Pile ───   Dummy Pile ───

(a) Vane shear testing   (b) Vane shear testing
    at four depths   at eight depths

Fig. 5.   Location of vane shear tests for the
           determination of shear strength dis-
           tributions of 2.5 inch samples

strength. An attempt was made to correlate the numerous vane shear values determined during this research with the unconfined compressive strength of the test soil. Since the shear strength of the soil decreased with sample depth, no accurate correlation was possible by performing an unconfined compression test on a large sample and comparing the results with several vane tests made along the sample depth. Failure in the unconfined sample would take place at the weakest section, i.e. the sample bottom, while failure by the vane is imposed on a vertical plane surrounding the vane. Another difficulty involved is the wide variation between individual samples.

To overcome the problem, a small steel "Shelby" tube was tooled and polished on a lathe to inside and outside diameters of 0.625 and 0.675 inch, respectively. The tube was pushed into a 2.5 inch sample to the desired depth and held in position. The vane was then inserted close to the tube to a similar depth and the vane shear test was conducted. Figure 6 is a schematic diagram of the relative positions of the tube and vane during testing. After the tube was extracted, the soil core was retrieved and trimmed to the proper height for a length to diameter ratio of 2.0 and placed in the Farnell compression machine. Unconfined compression tests were carried out with the small samples using a strain rate of 1.5 per cent per minute. The small steel tube and several failed samples are pictured in Photograph 8.
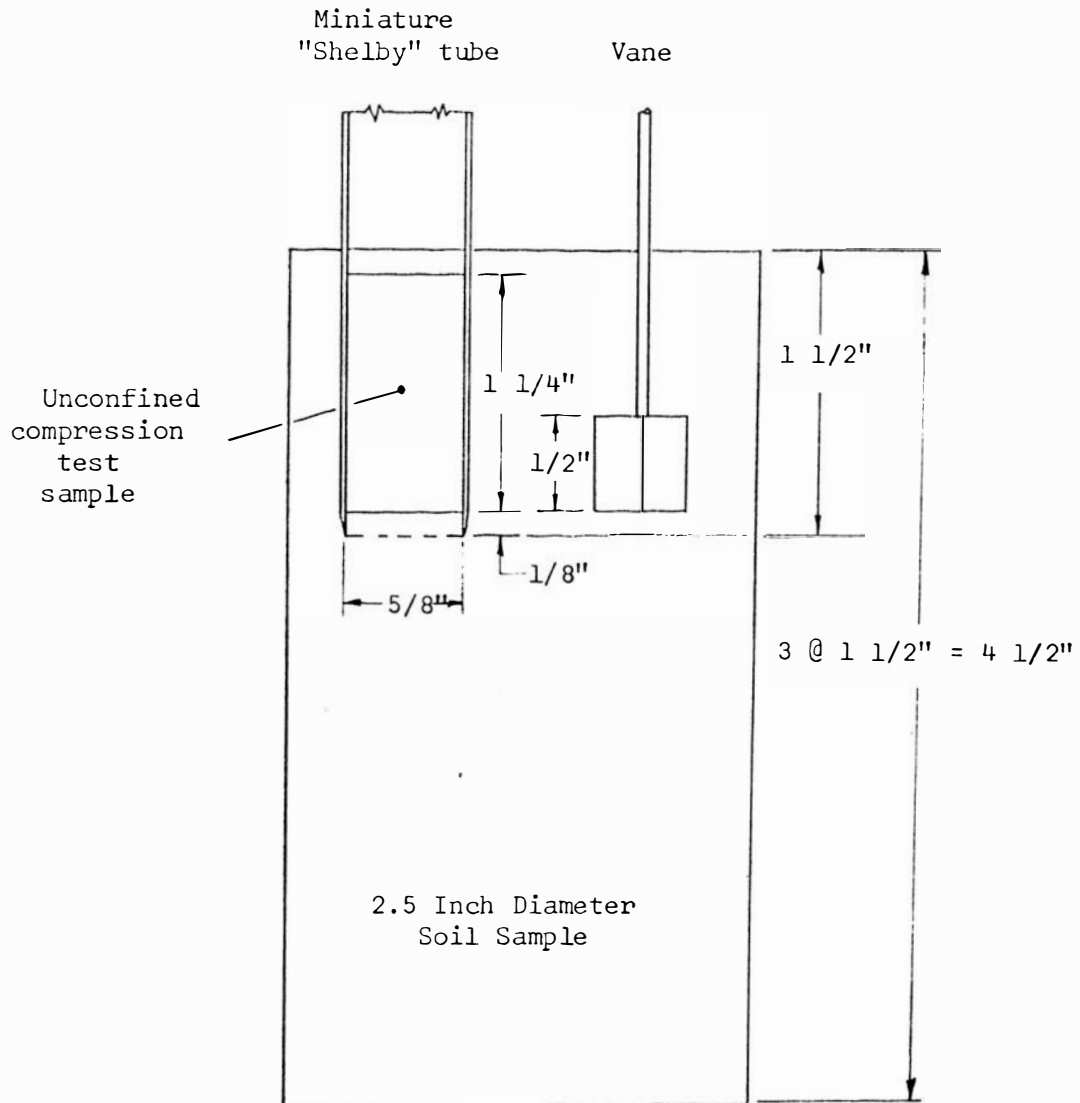
Fig. 6.   Location of tests to correlate vane shear
           strength to unconfined compressive strength
           of soil

5. Testing for effects of elapsed time

A series of tests was executed to measure the gain in
the ultimate load on the pile with time after pile embedment.
A 1/2 inch pile was inserted into a jacketed 2.5 inch sample
with no pilot hole. The ultimate load supported by the pile
was measured two minutes after insertion, and subsequent
ultimate load tests were conducted at 5, 10, 15, 50, 60, 90,
170, 315, 610, 1260, 3400, and 7200 minute intervals after
each previous test. These time intervals were chosen so that
their distribution on a logarithmic scale would be evenly
spaced. The soil-pile system was wrapped with saran and
placed in the moist room during the intervals that were longer
than 90 minutes.

6. Testing for effects of penetration rates

The effects of differing penetration rates on the ulti-
mate pile load were also investigated since penetration rates
used in this research were much slower than those cited in the
literature. Ultimate loads on a 1/2 inch pile in a 2.5 inch
sample with no pilot hole were measured at ten minute inter-
vals using penetration rates of 0.002, 0.004, 0.008, 0.016,
and 0.032 inch per minute.

IV.   TEST RESULTS AND ANALYSIS

A.   Physical Properties of the Soil Samples

1.   Shear strength distributions

Prior to investigating the load transfer between the soil and model piles, it was necessary to know the strength properties of all the soil samples used. Preliminary testing indicated the vane shear strength and moisture content varied with depth in the sedimented samples. This section is devoted to describing the nature and magnitude of the variance of these properties.

As described in the preceding chapter, vane shear tests were conducted next to the piles at pre-determined depths in the 2.5 inch samples. In the first series of tests with the 1/4 inch pile in 1.4 inch soil samples, the general procedures were modified due to lack of space between the pile and soil jacket for vane shear testing. Each of the six plotted points on the graph of Fig. 7 represents the results of one vane shear-moisture content determination in a one inch long section of soil cut from the top or bottom of each of six samples. All of the samples used in this series were approximately the same initial height and age. Rutledge (1948) found that the moisture content versus log shear strength relationship is linear for a normally consolidated clay, thus a straight line was fitted to the six data points by the method of least squares. Moisture contents were taken in the
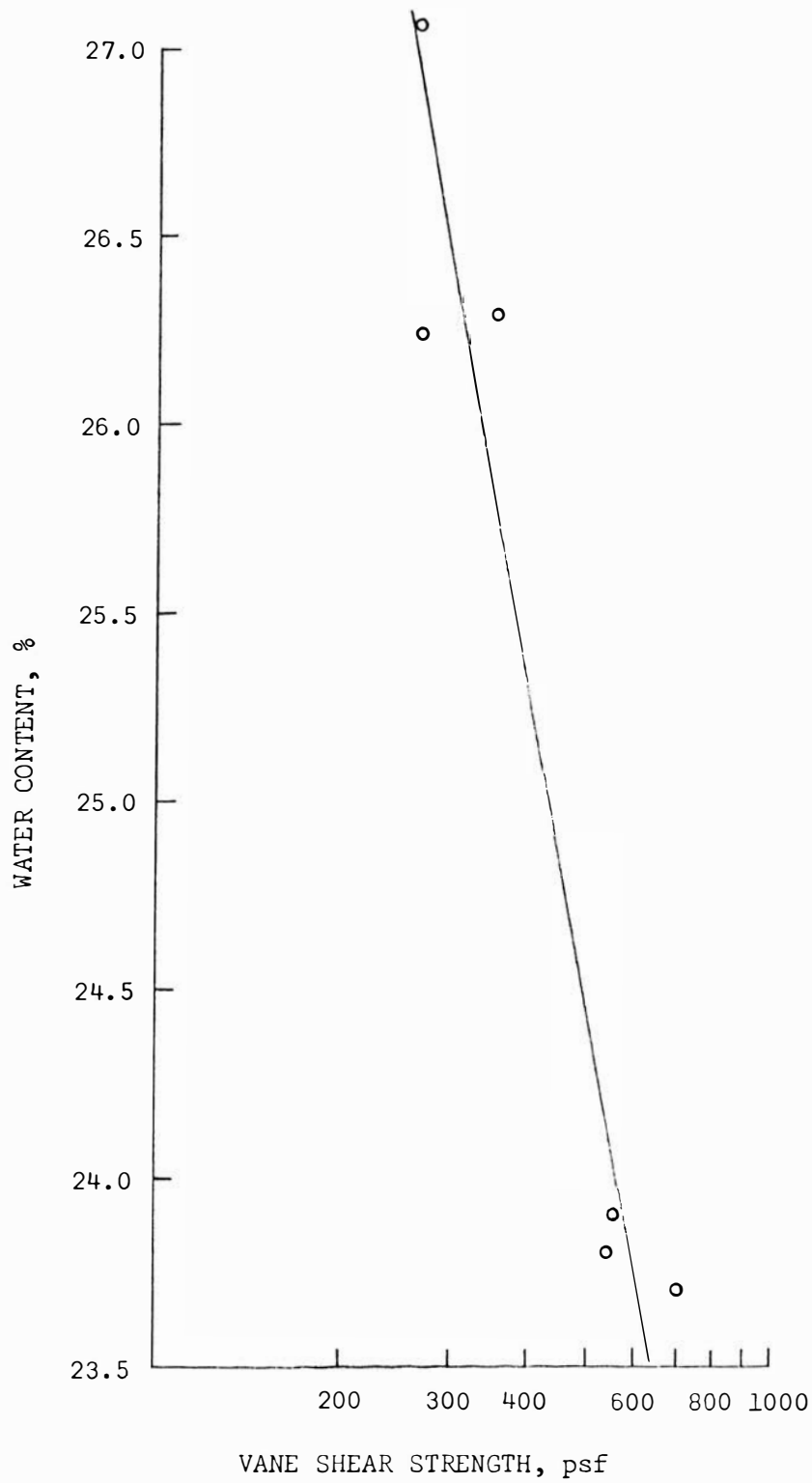
Figure 7 . Vane shear-moisture content relation for 1.4 inch samples
used with 1/4 inch pile.

remaining portion of each sample which was used for a pile
load test. With these moisture content values the respec-
tive vane shear strengths were obtained from the best-fit
line of Fig. 7 and plotted versus the depths in each sample
from which the moisture contents were taken. (See Fig. 8.)
Although the points plotted for each sample do not determine
a straight line in all cases, the shear strength distributions
were assumed linear for the sake of uniformity and simplicity.
The slight non-linear trend is discussed later. Each line,
which was fitted by the method of least squares, represents
the vane shear strength distribution with depth in one par-
ticular sample.

All samples were given a code number which would identify
its size and use. Referring to the legend of Fig. 8, sample
#1-1.4-¼-0 denotes the sample number, the sample diameter,
the diameter of the pile, and the diameter of the pilot hole,
respectively. All measurements are in inches.

It is evident from an inspection of the curves of Fig. 8
that the vane shear strength of the soil decreased with depth
in the samples. The magnitude of this decrease from the top
to the bottom of the sample is approximately 50 per cent of
the maximum vane shear strength. The reason for this decrease
in shear strength was the dissipation of effective stress with
depth in the soil sample during the sedimentation process. It
is thought this dissipation of stress was partly due to wall
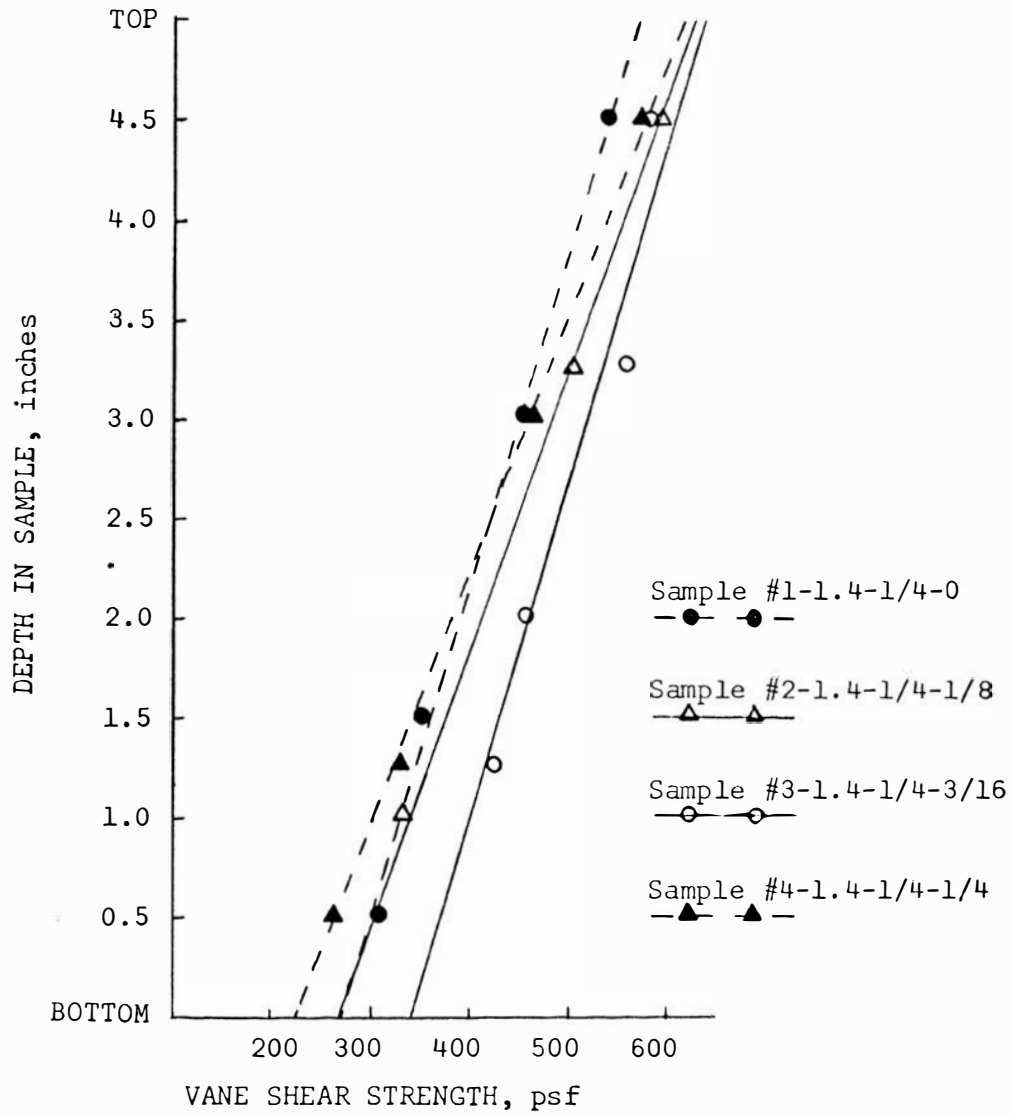friction between the soil particles and the sediment tube.

Fig. 8.   Vane shear strength distributions in 1.4
inch samples used with 1/4 pile

This would cause the bottom of the sample to be less con-
solidated than the top.

The results of vane shear tests in 2.5 inch samples used
with the 1/4 inch pile are shown in Fig. 9. The shear
strength distributions of these samples are presented on two
graphs for ease of interpretation. The vane shear strengths
were plotted versus depths at which they were performed, and
no moisture content-shear strength correlation as described
before was necessary. A close inspection of the positions of
the plotted points of Fig. 9 indicates there was a slight
trend for the shear strength to decrease more near the top
of the sample than through the middle or bottom. This trend
is realistic considering the greater portion of stress dis-
sipated immediately under the load, as theorized by Boussinesq
(Terzaghi and Peck, 1967). The trend was not evident nor
equally prominent in all samples, therefore a linear decrease
of strength with depth was assumed. Each straight line showing
the shear strength distributions was fit by the method of
least squares.

Little explanation can be offered as to why the shear
strength distributions for similarly prepared samples are not
parallel nor coinciding. In some instances the load on the
sedimenting samples was maintained for an additional 12 to 72
hours, and the extra time spent in secondary consolidation
could have resulted in slightly stronger samples. Some of the
samples were sedimented during the daylight hours while others
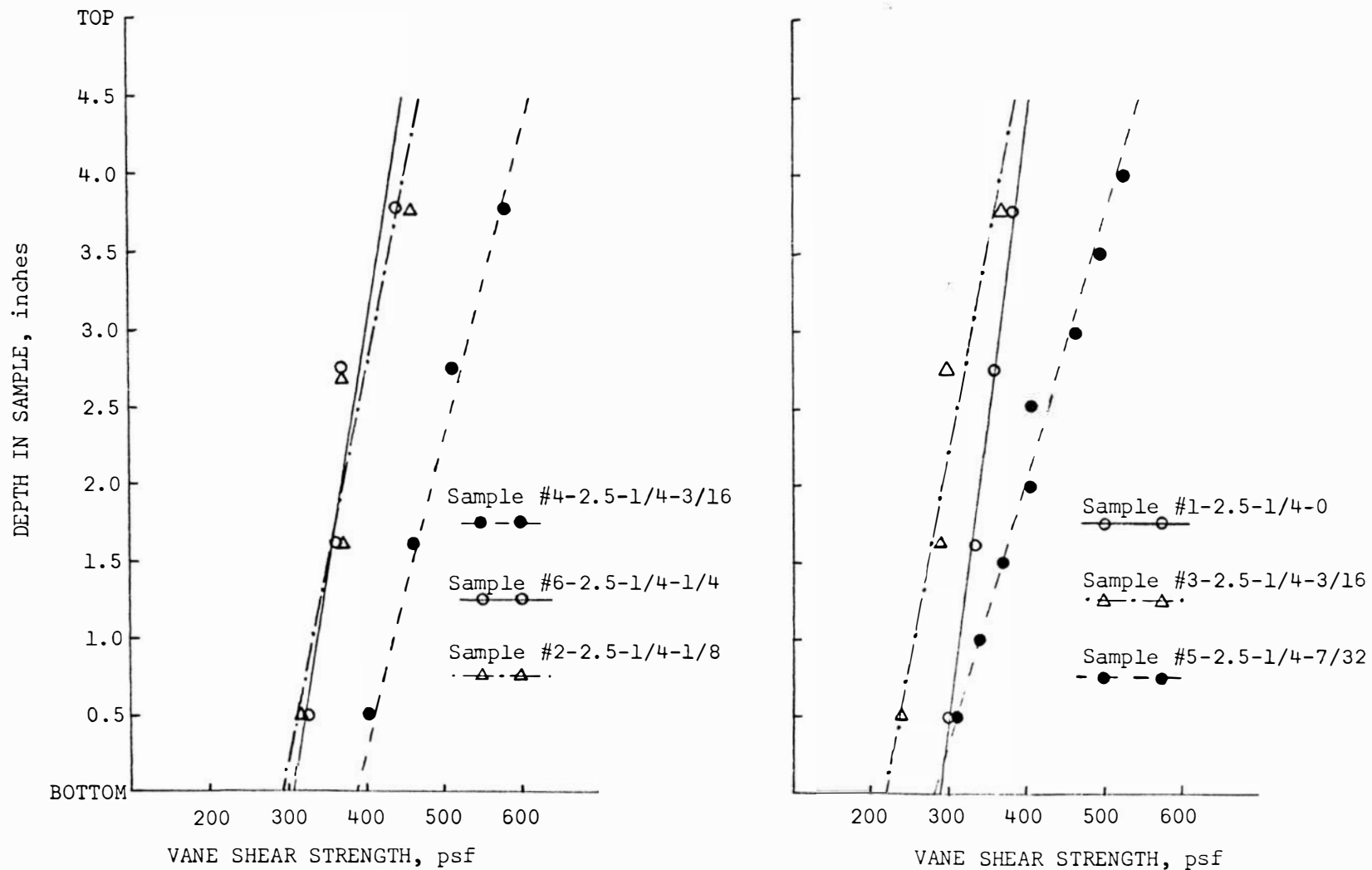
Fig. 9.   Vane shear strength distributions in 2.5 inch
           samples used with 1/4 inch pile

underwent primary consolidation during the night when temperatures in the laboratory could drop substantially during the colder months. This temperature difference could have affected the consolidation process and thus the strength properties of the samples.

After the 1/2 inch pile was tested in the 1.4 inch samples, insufficient space was available to conduct the vane shear tests. The vane shear-moisture content relationship represented in Fig. 7 could not be used because of an age difference which might have meant a strength difference between the two groups of samples. Three 1.4 inch samples known to be of the same age were chosen, and one sample was used for a vane shear-moisture content relationship. (See Fig. 10.) Very little scatter of the points about the best-fit line occurs, which is contrary to results shown in Fig. 7. The moisture content versus log shear strength relationship gained from a single sample was quite linear. From a comparison of results shown in Figs. 7 and 10, it appears the relationship is unique for each sample tested. This could be due to the fact that each sample was consolidated under one particular set of conditions.

Moisture contents were taken in the other two 1.4 inch samples after the pile load test, and the corresponding vane shear at each depth was taken from the curve of Fig. 10 and plotted in the manner previously described. The resulting shear strength distributions for these and all remaining samples are shown in the figures of Appendix A.
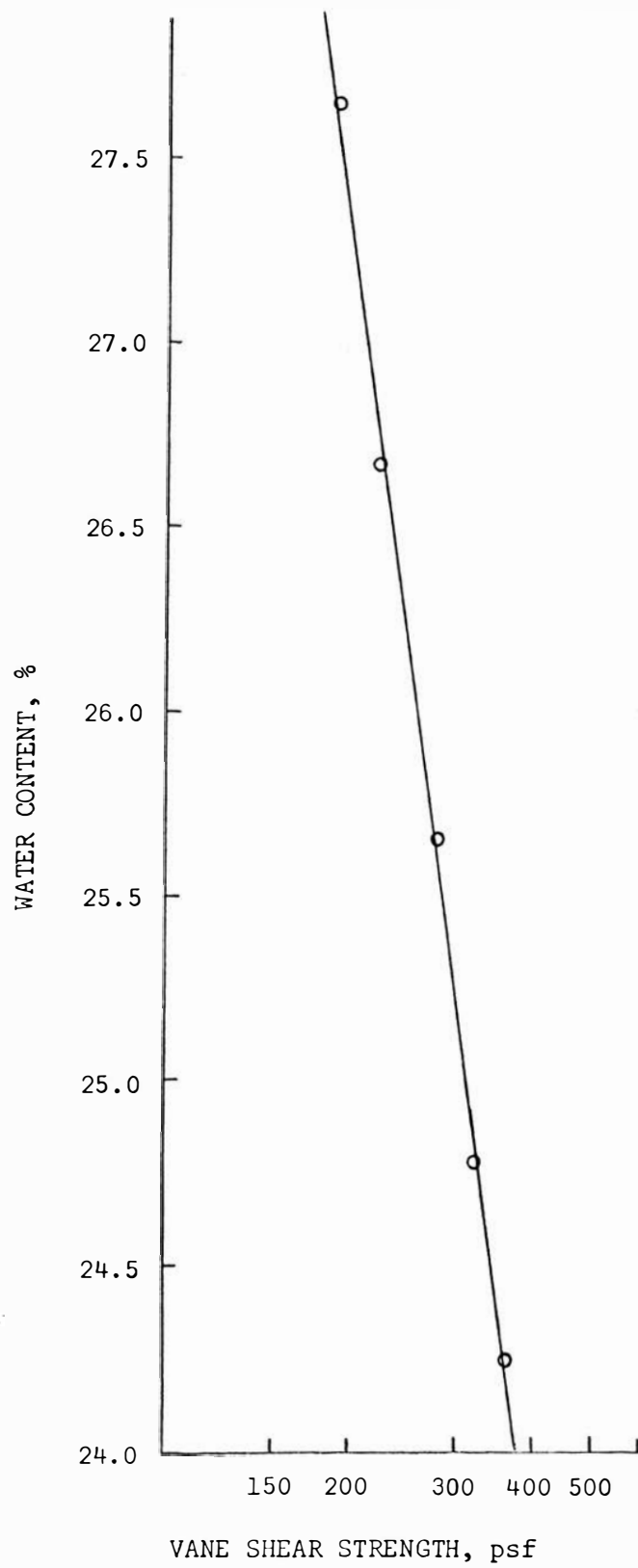
Fig. 10.   Vane shear - moisture content relation for
             1.4 inch samples used with 1/2 inch pile

The method of vane shear-moisture content correlation to obtain shear strength distributions produced as little scatter of the plotted points about the best-fit lines as that obtained from direct vane shear testing at each depth. This lack of excessive scatter, assuming lack of scatter as a valid criterion for judgement, infers that both methods used to obtain shear strength distributions in the soil were equally valid.

On the other hand, the direct method, vane shear testing next to the pile, did not involve an assumption regarding the validity of one shear strength-moisture content relation for several samples. In addition, the non-linear shear strength distributions, which seems more realistic, was most evident in the test results from samples tested directly with the vane device. This leads to the conclusion that the direct method of vane testing was more accurate, and certainly more favorable from the standpoint of speed and ease in interpretation.

2. Variation of moisture content

The variation of moisture content with depth in the soil samples does not appear to be linear. Moisture content distributions of two representative samples are shown in Fig. 11. These samples were chosen on the basis that their shear strength distributions and log vane shear moisture content relationships showed a high degree of linearity. Moisture
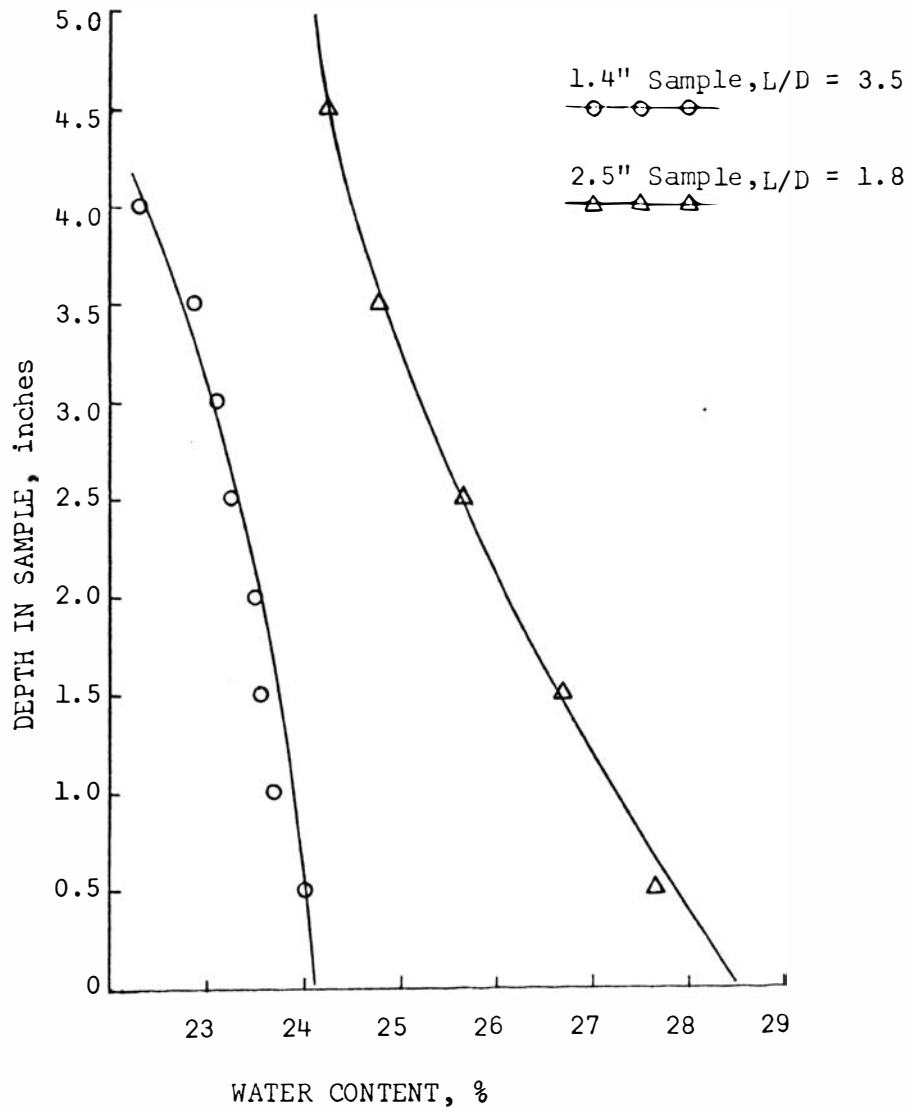
Fig. 11.   Moisture content distributions in two
             representative soil samples

contents were taken from the same 1.4 inch sample used for the vane shear-moisture content correlation shown in Fig. 10. The other moisture content values are from sample #4-2.5-½-½. (See Fig. 31, Appendix A).

A significant point regarding these curves is that the range in water content, and thus the range in shear strength, from the top to the bottom of the sample is considerably greater in the 1.4 inch sample. Most 1.4 inch samples had a length-to-diameter (L/D) ratio greater than 3.0. This L/D ratio, as opposed to one less than 2.0 for the 2.5 inch samples, leads to more frictional drag between soil particles and the sediment tube wall per unit volume of soil sample. Increased frictional resistance gives rise to greater dissipation of effective stress. Lower effective stress near the sample bottom resulted in a less dense structure, higher void ratios, and higher moisture contents.

3. Correlation of vane shear to unconfined compressive strength

The procedures used to obtain test values to correlate the vane shear strength and the shear strength determined by the unconfined compression test were outlined in detail in the previous chapter. The graphical representation of this correlation is illustrated in Fig. 12. The curves which were fitted to the data by the least-squares method, intersect at a water content value near 27 per cent, which is slightly below the liquid limit of the soil. The ratio of vane shear
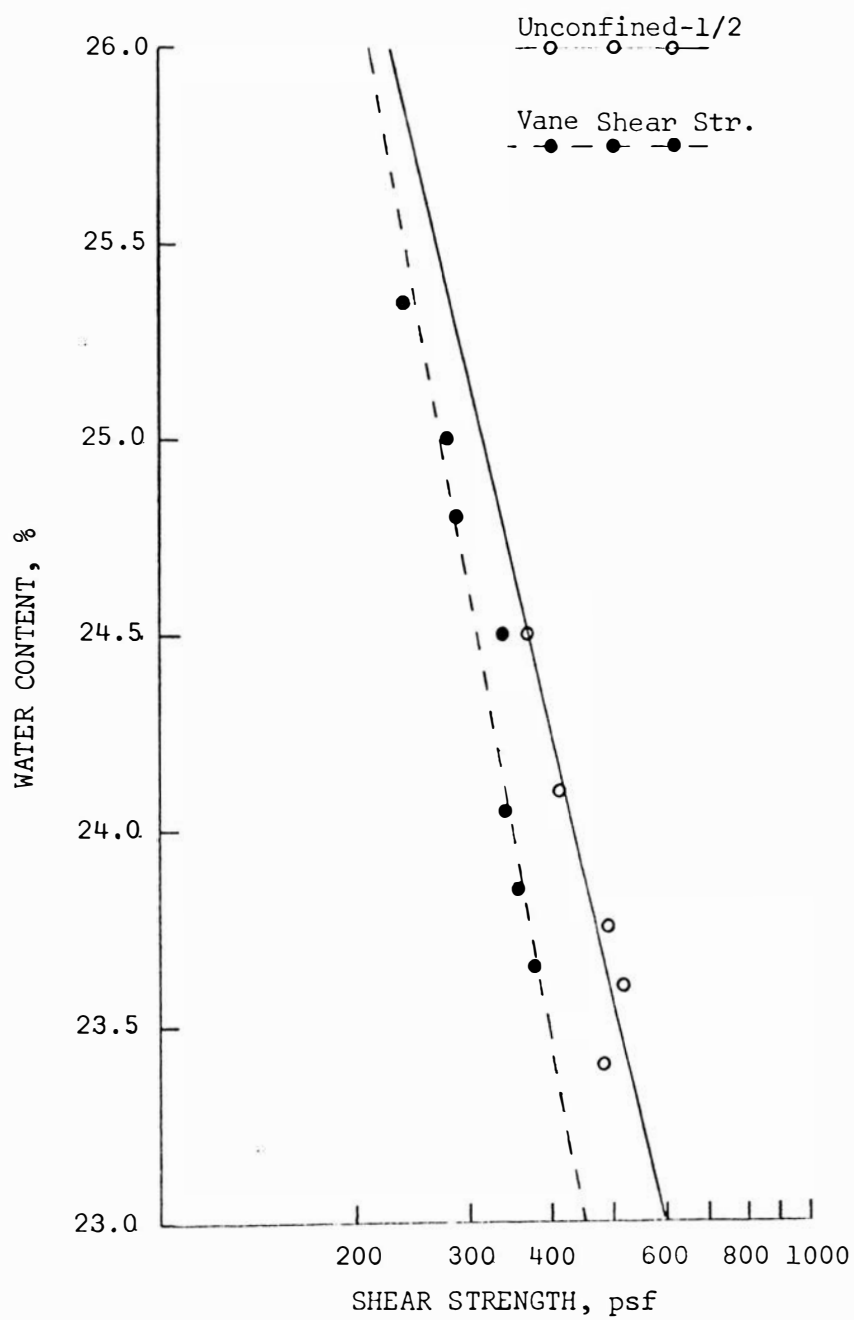
Fig. 12.   Variation of vane shear and unconfined
compressive strength with water content

strength to one half of the unconfined compressive strength changes with the water content, or void ratio, of the soil, and thus changes with depth in each sample. The ratio varied from 0.93 to 0.75 within the range of water contents found in any one sedimented sample.

Goughnor and Sallberg (1964) found that soils with a plasticity index (PI) less than 14 per cent had vane shear-to-unconfined ratios less than 1.0. One explanation is that a soil having a low P.I. has an intergranular friction force that is mobilized in the unconfined compression test but possibly not in the vane shear test, thus the low ratios. Near the sample top where the soil structure is assumed more dense because the moisture content is lower, a resulting higher intergranular friction might explain the higher unconfined compressive strengths relative to the vane shear strengths.

Another possibility is that since the samples were anisotropically consolidated, the values of shear strength determined from failure on different planes would disagree.

The variance of shear strength found in each sample was much greater than was expected at the beginning of this research. This variance of physical properties was primarily due to the large length-to-diameter ratios of the sedimented samples. Although the existence of a shear strength distributed with sample depth simulates a natural soil deposit, it eliminated the possibility of choosing with experimental soil-

pile adhesion values. An alternative method for comparing soil shear strength to skin friction is introduced in the following section.

B.   Load Transfer of Model Piles as a Function of Pilot Hole, Pile, and Sample Size

   1.   Theoretical considerations

According to the theories of Zeevaert (1950) and Airhart, et. al (1967), a pile driven into soft clay displaces the soil beneath the tip by the action of a soil wedge created at the tip. As this soil is displaced to the side of the pile, its structure is completely destroyed. Cutting a pilot hole in a sample to be tested, extracts soil that would otherwise be located below the pile tip, and reduces the amount of soil that is displaced, remolded, and forced to the side. This reduction in the volume of remolded soil beneath the pile tip allows a decrease in the radial extent and degree of disturbance of the soil near the pile.

   Mathes (1968) has shown that thixotropic effects are time-dependent. By measuring soil-pile adhesion at a definite interval of time after pile insertion, 90 minutes in this research, the magnitude of the strength gain within that interval will be constant for each test. If the disturbance created by pile insertion is small, then the magnitude of the regained strength will result in a higher adhesion value than if the initial disturbance is quite large. If the

assumptions stated above are correct the relative disturbance due to different ratios of pilot hole size to pile diameter can be determined by measuring soil-pile adhesion.

One measure of soil-pile adhesion is the magnitude of the load that can be transferred from the pile to the soil. The maximum soil-pile adhesion would occur if the load transferred to the soil equalled the shear strength of the soil; therefore a relative measure of adhesion is the ratio of the load transfer to the soil shear strength. The selection of one particular shear strength value from the entire sample for comparison with the average load transferred would not be valid because the shear strength changes with depth in each soil sample. Although the shear strength distributions are fairly linear, and the vane shear strength at mid-depth represents the average vane shear in the sample, the pile load transferred to the soil is neither constant with depth, nor totally dependent on the soil's shear strength. Coyle and Reese (1966) have shown that the ratio of load transfer to soil shear strength changes with pile penetration and depth.

At present, the only valid method of determining load distribution with depth is to instrument steel-walled piles with strain gages, and, knowing the elastic properties of the pile, calculate the load remaining in the pile at chosen depths. The model piles used in this research were not so instrumented, therefore a theorized load distribution curve that would give reasonable values for load transfer with depth in the soil samples was required.

A typical load distribution curve in clay developed by Reese and Seed (1955) was first considered for use. Preliminary calculations based on the geometry of the curve produced load transfer values exceeding the shear strength of the soil at the lower depths. A total regaining of shear strength of the lean clay soil only 90 minutes after remolding seemed quite unlikely, therefore both the transfer values and the curve were assumed incorrect. The curve did not fully apply because the test soil in Reese and Seed's research exhibited fairly constant values of shear strength with depth, unlike the soil samples used in this research.

Airhart, et. al. (1967) have given several load distribution curves obtained from two full-scale instrumented hollow steel piles driven into a soil deposit consisting mainly of silty clay. Because the soil type in that research resembled the soil used in this study, and the shear strength distribution for both the soils were similar, i.e. decreasing with depth, the load distribution curves were considered applicable. The load distribution curve illustrated in Fig. 13(B) is considered to represent the general properties of the curves from the tests by Airhart, et. al., and is not an exact copy of any one particular curve. The typical curve of Fig. 13(B) differs from the one developed by Seed and Reese (1955) in that it distributes the largest percentage of the load to the soil from the pile's mid-sections. This difference arises from dissimilar shear strength distributions of the soils of the two studies.
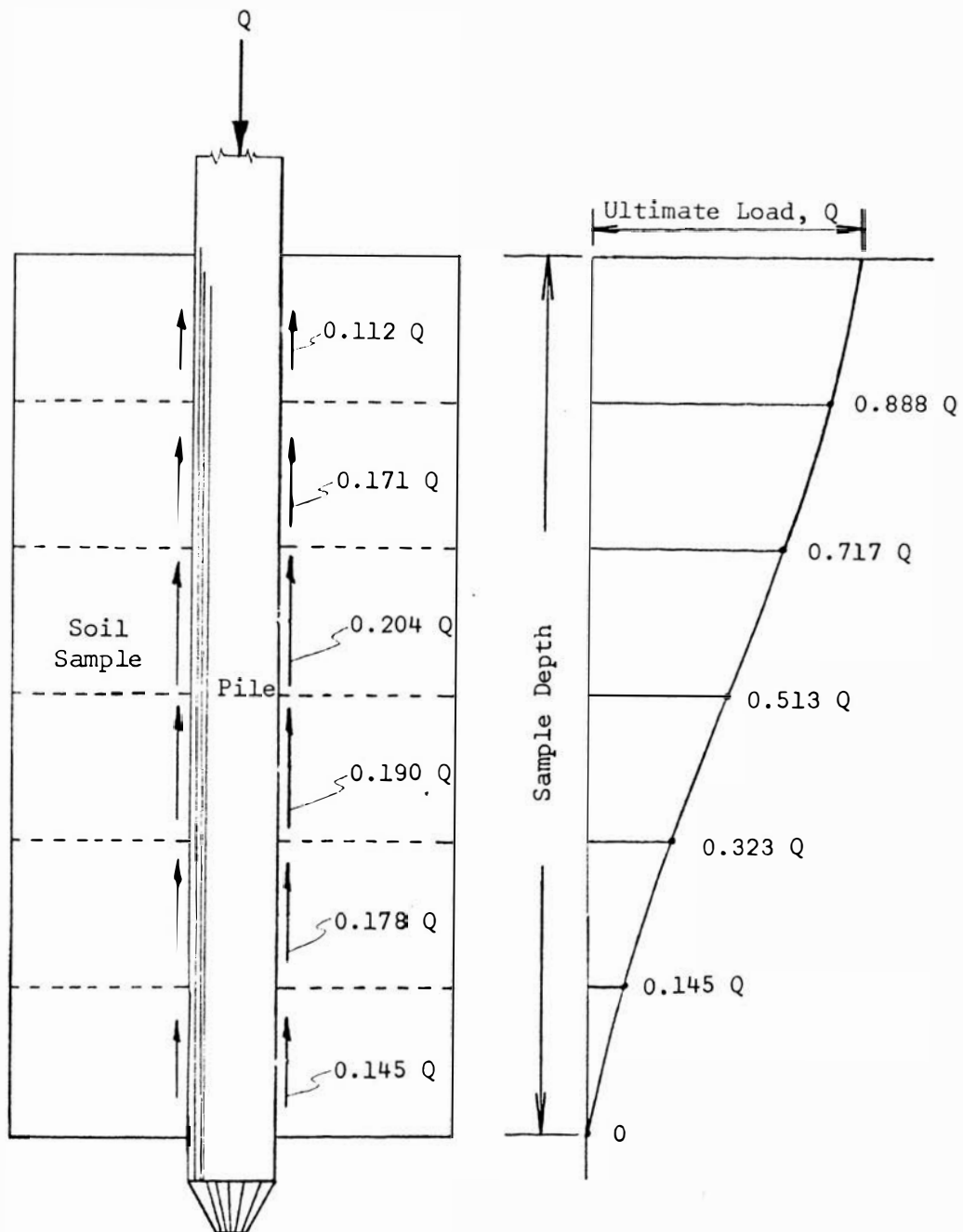
(A) Load Transferred to Soil   (B) Load in Pile

Fig. 13.   Load distribution versus depth for model pile-soil system

The failure mechanisms of the field piles of Airhart, et. al. (1967) and the model piles of this study differ. After a field pile has been driven and allowed to set for several days, the soil that was remolded adjacent to the pile assumes a more dense arrangement after dissipation to excess pore water pressure and consolidation. With a steel pile, clay particles make intimate contact with the pile wall by means of a mutual oxide coating and a rough surface of contact. During load tests the soil particles within a zone slightly away from the pile move relative to each other and produce a less dense arrangement wherein pore water can concentrate and result in weakened particle bonds. These weaker bonds give rise to a progressive soil-to-soil failure away from the pile wall. This explains the thin layer of clay that is occasionally seen adhering to the field pile as it is pulled.

In the model pile tests of this study the time allowed for consolidation was shorter than for the field pile and the consolidation pressures much smaller. The fact that the model piles developed no rust or other oxide film after one week of soaking in water indicated that water did not bind electro-chemically with the model pile surface as with the steel field pile. The lower bond at the pile wall allowed failure at the water-lubricated steel surface. This explains why only a slight dull sheen on the pile, due to soil and water caught in the striations, was observed after extraction of the model pile. Failure at the soil-pile interface infers that load transfer in the model piles is a function of soil-pile adhesion,

which depends partly on the shear strength of the soil.

Although the failure mechanisms of the field and model pile are not exactly the same, the load distribution curve of Fig. 13(B) is considered reasonable for the model piles. The theory is that the sensitivity of the soil and the amount of remolding allows the load distribution curve to be valid based either on soil-pile adhesion or the shear strength of the soil.

The sensitivity of the soil to remolding increased with depth in the soil sample because the water content also increased with sample depth. In the upper region of the sample, where the moisture content and sensitivity to remolding was relatively low, the remolding energy provided by pile insertion produced the lowest amount of disturbance in the soil adjacent to the pile. The overburden at the sample top was so low that the disturbed or remolded soil was only slightly forced against the pile. This means that soil-pile adhesion was low. Along the middle of the sample the moisture content and sensitivity was intermediate between that at the top and bottom, and the remolding energy produced a higher degree of disturbance than at the top. The overburden pressure at mid-section was slightly higher than at the top and the disturbed soil next to the pile, acted upon by a higher normal force, caused higher adhesion.

Near the sample bottom the water content and sensitivity to remolding was highest. The given amount of remolding energy produced a high degree of disturbance, or remolding, at a water content near the liquid limit which allowed a release of pore water that lubricated the soil-pile interface.

Although the overburden at the bottom was greatest, the higher lubrication at the pile wall prevented the adhesion being greater than the adhesion near the middle. The load that was transferred to the soil was assumed dependent on the soil-pile adhesion therefore most of the load transfer occurred near the mid-section of the model pile.

Referring again to Fig. 13(B) the load distribution curve gives values, in decimal form, representing the portion of the ultimate load, Q, remaining in the pile at that particular depth. The decimal values given in Fig. 13(A) represent the portion of Q transferred to the soil in that segment, and are the differences between the loads in the pile at the top and bottom of each respective segment. The curve of Fig. 13(A) represents the soil reaction, or the distribution of load carrying capacity of the soil. The percentage of ultimate load transferred in each segment is considered a point load applied at the middle of each segment rather than a distributed load, which is the physical situation. This allowed the load to be more easily compared with the vane shear strength at the middle of each segment. The exact method of comparison is explained in detail later.

2. Load tests with 1/4 inch pile

Results of load tests using the 1/4 inch pile in 1.4 inch samples with various sizes of pilot holes are shown in Fig. 14.
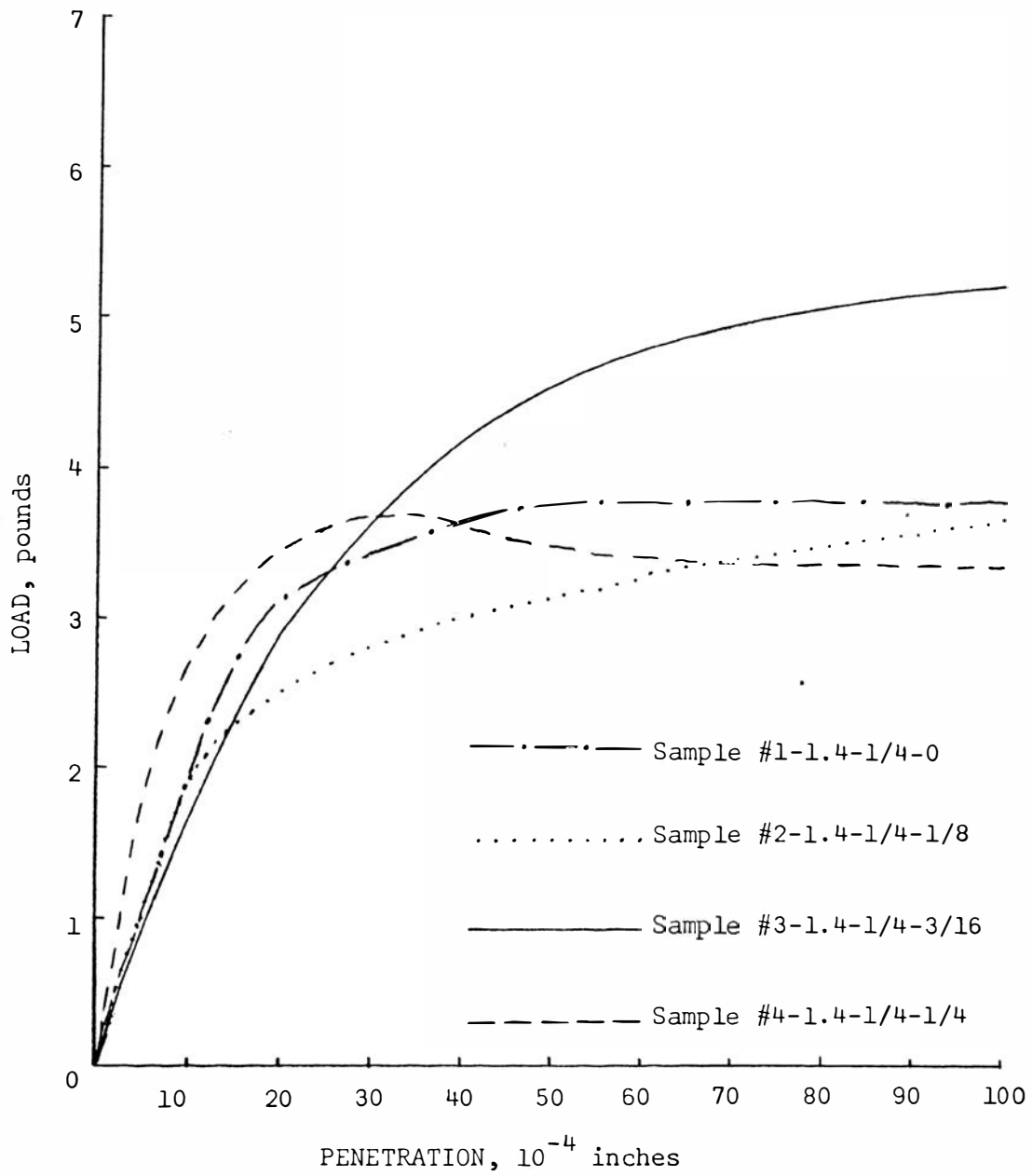
Fig. 14.   Load versus penetration for 1/4 inch pile
            in 1.4 inch samples

The shear strength of the soil was not considered in the construction of these curves, therefore the sample which gave the highest ultimate load did not necessarily produce the highest ratio of load transfer[3] with respect to the shear strength of the soil. The ultimate load was considered the highest point on the load-penetration curve. If the highest point did not occur before a penetration of 0.01 inch, it was taken at that penetration, although loading did not cease until a total pile movement of 0.15 inch was reached.

Although the relative positions of the curves in Fig. 14 indicate little, the geometry of the curves show that there is a linear relationship between load and penetration for very small pile movements, as noted by Coyle and Reese (1966).

The load distribution curve in Fig. 13(A) was used to calculate the ratio of the unit load transferred to the soil by the pile to the shear strength of the soil, hereafter referred to as the T-S ratio. The determination of the T-S ratios in each sample was carried out in four steps as follows:

1. Each soil sample was divided into six imaginary horizontal segments of equal height, usually 3/4 inch.

2. The average vane shear strength of each segment, taken from the vane shear versus depth curve, was multiplied by the surface area of the model pile in that segment. These values, in pounds,

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[3]Load transfer refers to the pile load divided by the surface area of pile embedment and has units of stress, such as pounds per square foot.

represented the maximum load that the soil segments
could have carried if the soil-pile adhesion was
equal to the shear strength of the soil.

3. The ultimate load determined in the pile load
test, in pounds, was multiplied by the load
distribution factor for each segment. (See
Fig. 13(A).) These values represented the loads
that were assumed transferred to each respective
soil segment.

4. For each segment, the load calculated in step 3
was divided by the load determined in step 2,
giving the T-S ratio.

The final products of the calculations enumerated above
were six T-S ratios for each pile test. In the context of
the discussion that follows, the ratio of the pilot hole diam-
eter to the pile diameter will be referred to as the R_d
ratio.

In Fig. 15 the T-S ratios for the 1/4 inch pile in the
1.4 inch samples are shown with various R_d ratios. The T-S
ratios were plotted versus the mid-height of each respective
imaginary soil segment. The curves connecting the T-S ratios
at the selected depths indicate that the T-S ratio is a func-
tion of the depth in the sample, as suggested by Coyle and
Reese (1966). The curves also show the T-S ratio to be fairly
constant throughout the lower half of the sample due to the
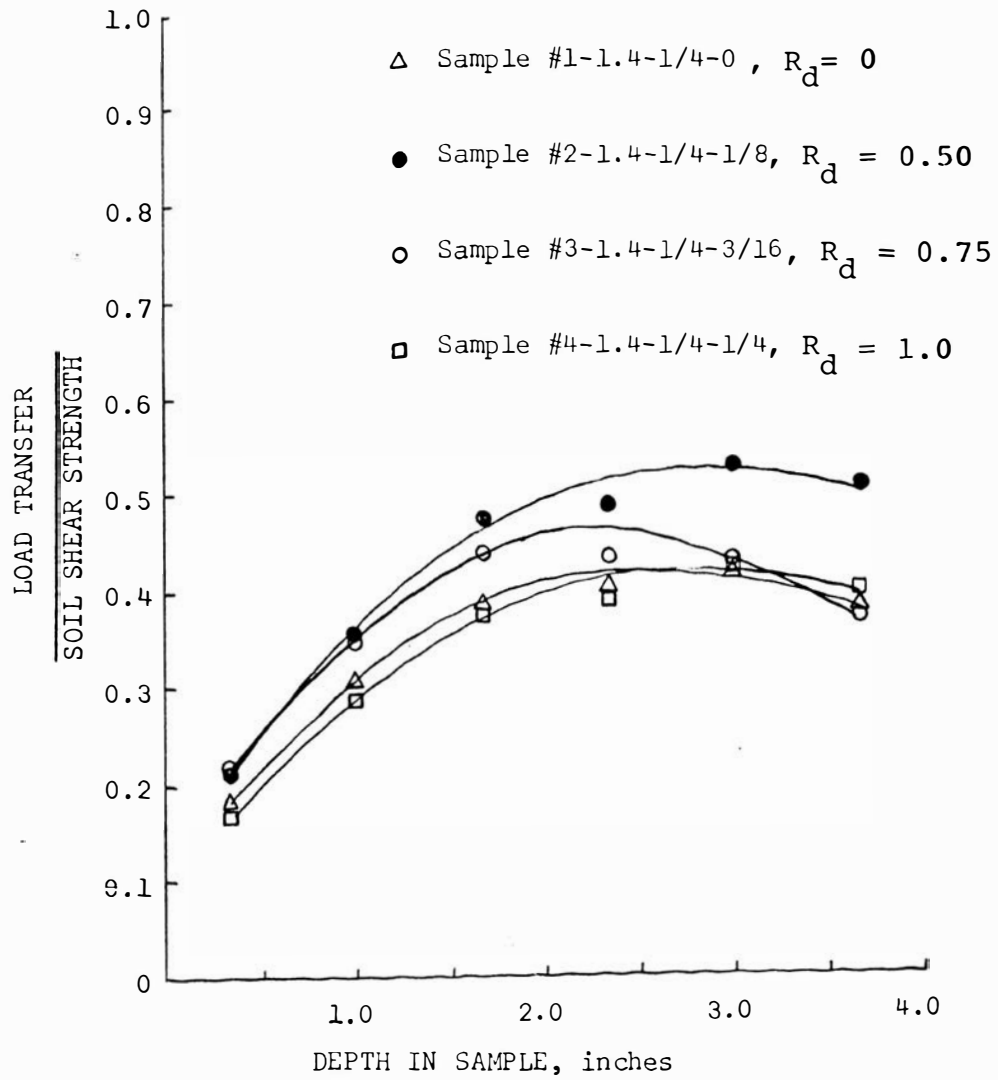combination of changing shear strength of the soil with depth,

Fig. 15.   Variation of T-S ratio with depth for
various R_d ratios - 1/4 inch pile,
1.4 inch samples

and the properties of the load distribution curve.

The highest T-S ratio occurred in the sample with the 1/8 inch pilot hole, or $R_d = 0.5$, and implies that a pilot hole one half the diameter of the pile was optimum for the conditions in the testing procedure. The lowest curve in the figure, $R_d = 1.0$, shows there is a possibility that less than full pile-to-soil contact resulted with a pilot hole equal to the pile diameter. The low curve for the sample with no pilot hole, $R_d = 0$, indicated that the soil disturbance adjacent to the pile was higher than when pilot holes smaller than the pile diameter were used, thus the relatively low adhesion values at the time of testing.

The lower curve of Fig. 18 shows more clearly the relation of the maximum T-S ratio to the respective R_d ratio in each sample. The optimum T-S ratio of the curve infers that an optimum pilot hole size does exist which gives a proper balance between low soil disturbance during pile insertion and high soil-pile adhesion for the particular soil sample. Basically, the pile developed roughly one half of the soil's shear strength as skin friction. Only four tests were conducted for the pilot hole study above because the 1.4 inch soil samples were found soft and sensitive to the touch, as well as generally undesirable for the testing procedure because of the involved moisture content-vane shear relationships required.

A second pilot hole study for the 1/4 inch pile was carried out in 2.5 inch samples to check the validity of the first study, and determine whether sample size can cause any change in results. The load versus penetration curves derived from the tests on the 1/4 inch pile in 2.5 inch samples appear in Fig. 16. The same general rules that applied to the similar curves for the previous study also apply to these curves i.e., the sample and pilot hole that produced the highest ultimate load did not necessarily develop the highest T-S ratio because the load transferred was not compared to the shear strength of the soil.

Referring to the curves of Fig. 16, the same linear relationship between load and penetration for initial pile movements, as witnessed with the 1.4 inch samples, occurred in the 2.5 inch samples. However, failure in all the 2.5 inch samples took place at relatively low penetration values, about 0.003 inch, regardless of pilot hole size. The fact that the 2.5 inch soil samples are generally stiffer than the 1.4 inch samples is offered as a possible explanation for this phenomenon.

The T-S ratios for the 1/4 inch pile in the 2.5 inch samples were calculated as previously outlined, and appear in Fig. 17. There are notable differences in the results of this second pilot hole study. A sample with an R_d ratio of 0.75 developed the highest T-S ratio at all depths. The validity of that particular test was checked by conducting another similar test, and the results agreed within 6 per cent. As an
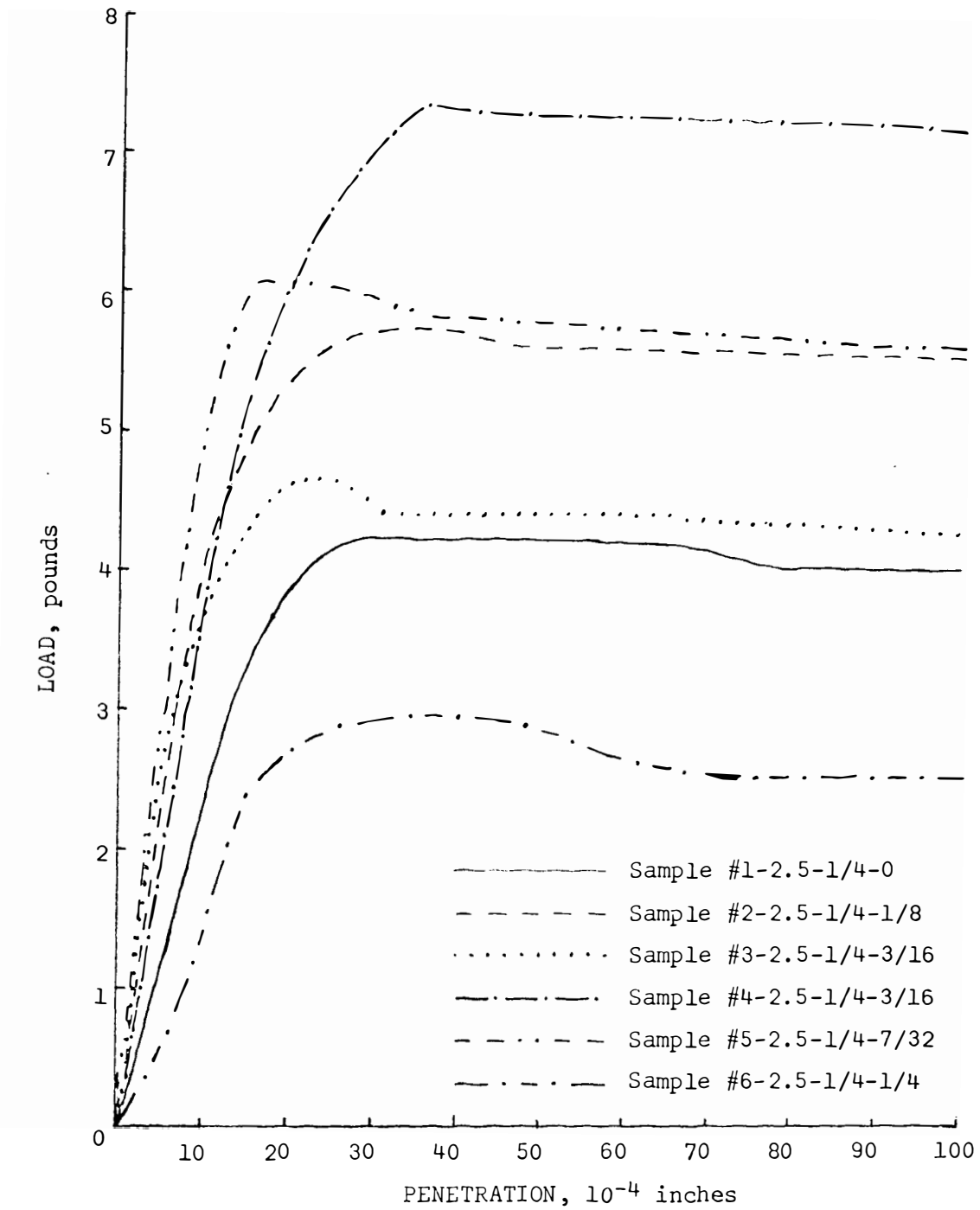
Fig. 16.   Load versus penetration for 1/4 inch pile
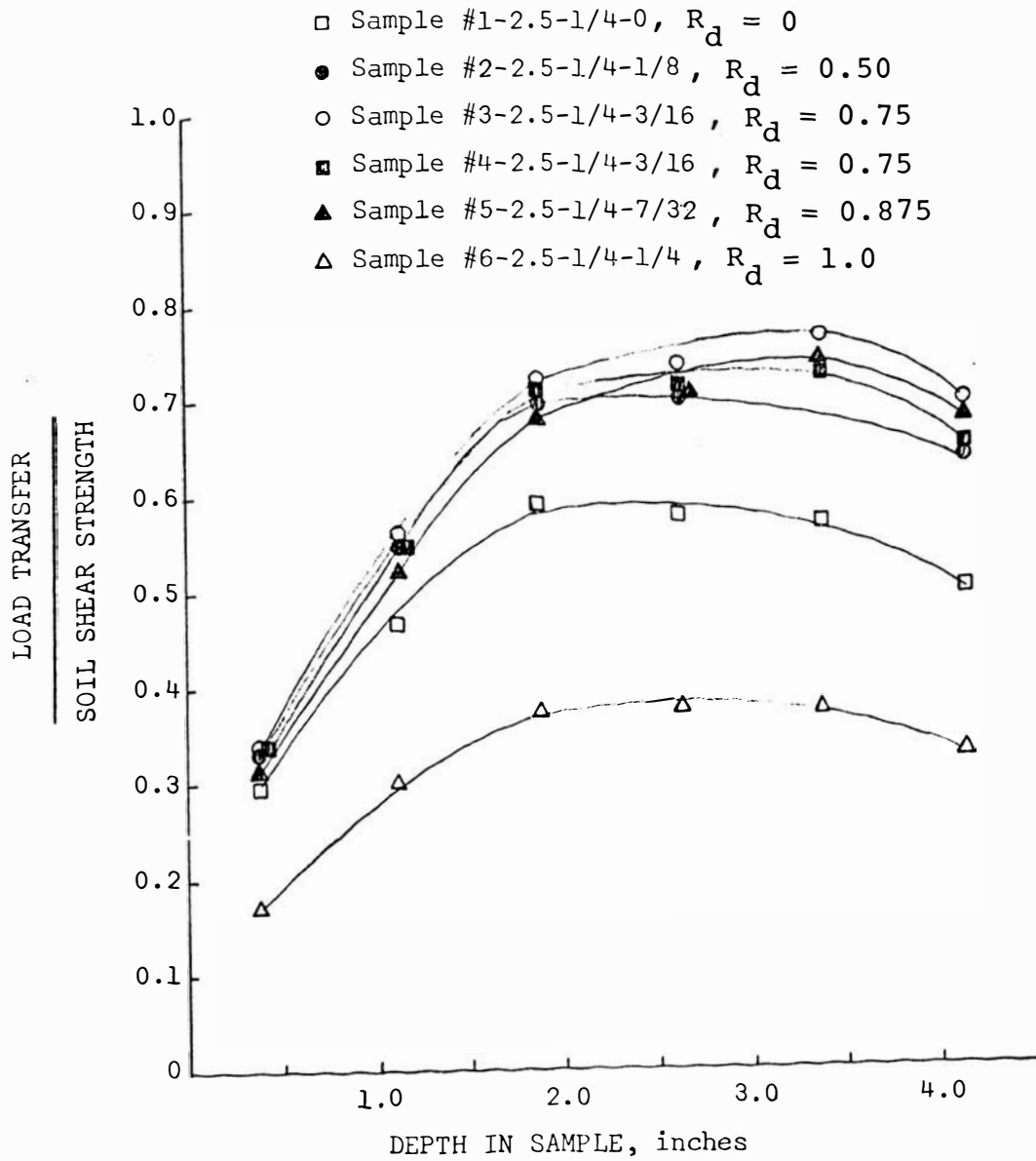   in 2.5 inch samples

Fig. 17.   Variation of T-S ratio with depth for various
R_d ratios - 1/4 inch pile, 2.5 inch samples

additional check, a sample with a 7/32 inch pilot hole
(R_d = 0.875) was tested and the resulting T-S ratios were
nearly as high as those obtained in the samples with R_d
ratios equal to 0.75. The 2.5 inch sample with an R_d ratio
of 1.0 developed very low T-S ratios with respect to the
other tests. This trend did not occur in the 1.4 inch
samples. Since the 2.5 inch samples were stiffer in consist-
ency than the smaller samples, the soil was unable to close
in around the pile under the available overburden pressure
and less soil-pile adhesion developed.

Referring to the upper curve of Fig. 18, the maximum
T-S ratios in the 2.5 inch samples indicate that the optimum
R_d ratio for the 1/4 inch pile is near 0.75. The maximum T-S
ratios compared in Fig. 18 occur at depths from 2.0 to 3.5
inches in the samples. No specific depth was chosen for
comparison of the T-S ratios, but Fig. 17 shows that the
order of the ratios may change with depth, although not
significantly.

After the first test with the 1.4 inch samples the
striations on the 1/4 inch brass pile became clogged with soil
particles. This clogging was though to have negligible effects
on the roughness of the pile at the time, and was initially
ignored. The smoothness of the 1/4 inch pile when tested in
the 1/4 inch samples may have accounted for the dissimilar
results of the two test series. In the second set of tests
using the 2.5 inch samples the pile was cleaned and roughened
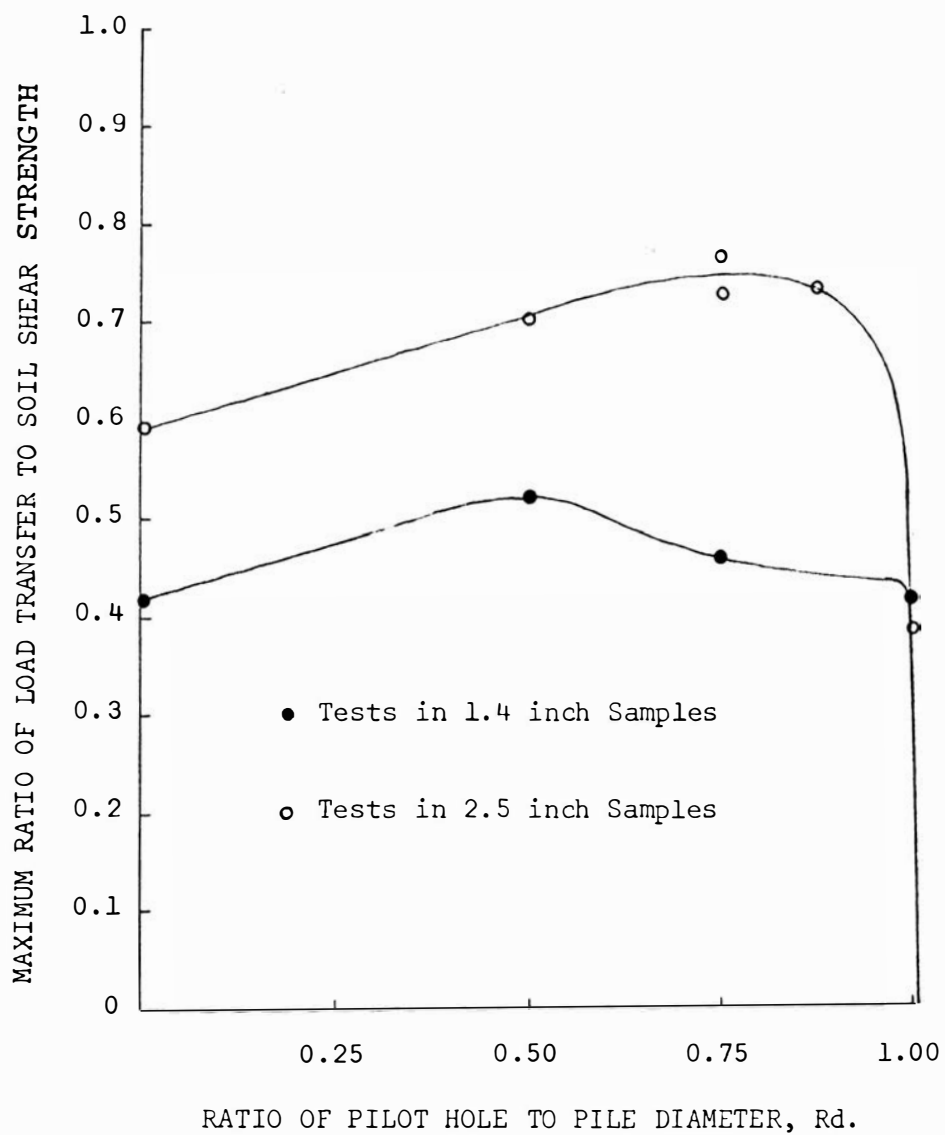before each test.

Fig. 18.   Results of pilot hole study to determine
                optimum R_d ratio - 1/4 inch pile

Another possible explanation for the different optimum R_d ratio and higher T-S ratios in the 2.5 inch samples was the existence of added confining pressure. As noted previously, the 2.5 inch lucite jacket fit the sample snugly, while a small annular area, filled with paper toweling, occurred between the 1.4 inch samples and their respective jacket. Pile insertion created radial forces that tended to expand or crack the samples. In the 2.5 inch samples those forces may have been dissipated by elastic deformation of the soil or rebounded from the rigid jacket wall. Either the elastic rebound of the soil after dissipation of excess pore water pressures or the reflection of radial forces from the rigid jacket could have produced a confining pressure on the pile. If pile loading is a partially drained process than an increase in confining pressure could have caused the increased load bearing capacity of the pile.

The 1.4 inch samples had a small diameter and lacked the rigid wall in intimate contact with the soil, so that radial forces were not dissipated or reflected, and permanent expansion of the sample resulted with little or no confining pressure on the pile. A lack of confining pressure may have required a smaller pilot hole ratio to achieve the highest soil-pile adhesion. The existence of higher confining pressures, assumed possible in the 2.5 inch samples, would have forced the soil against the pile and produced higher adhesion values with slightly larger pilot holes than in the 1.4 inch samples. The added confining pressure, or the prac-

tice of roughening the pile before each test, or a combination
of these is thought to have accounted for the consistently
higher adhesion in the 2.5 inch samples.

In the final analysis, it appears that an optimum pilot
hole diameter equal to about 0.75 times the pile diameter
existed for the 1/4 inch pile. The pilot hole study in the
2.5 inch samples was considered more valid because more tests
of better quality were involved. The fact that similar pilot
hole studies in two sample sizes yielded slightly different
results implies that some conditions of testing, such as con-
fining pressure and pile roughness, were not the same for
both test batteries. Attempts were made to control or eliminate
such variables but, apparently, the mechanisms of soil-pile
adhesion in model testing are much more complicated and dif-
ficult to isolate than first assumed.

3.   Load tests with 1/2 inch pile

A third pilot hole study was conducted to determine if
the optimum R_d ratio changes with pile size. The study was
made with 2.5 inch samples because more were available, they
could be handled easily, and vane shear testing in the larger
samples was a simple procedure.

The load versus penetration curves for the 1/2 inch pile
tested in 2.5 inch samples appear in Fig. 19. The curves are
all generally similar in that linear portions are evident at
the lower penetration values, and failure occurred between
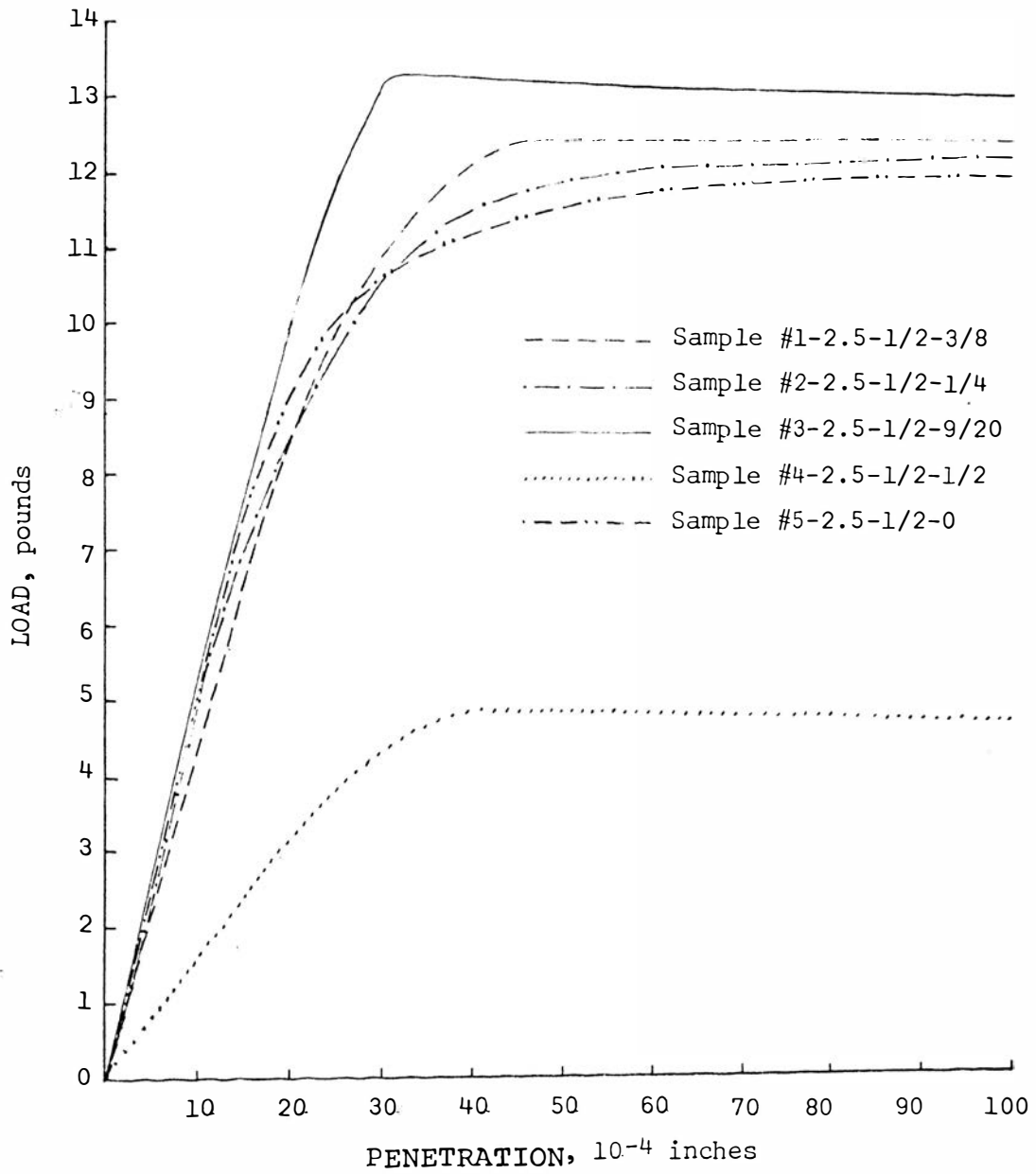o.003 and 0.005 inch. In all tests the loads reached an ulti-

Fig. 19.   Load versus penetration for 1/2 inch pile
in 2.5 inch samples

mate value and thereafter failed to decrease perceptively.

A steel ball was placed between the hollow butt of the 1/2 inch pile and the loading ring to minimize eccentric loading. Seating of the steel ball produced initial load readings with very little pile movement. After the ball was properly seated the load increase became linear, therefore the linear portion of the curves were adjusted to zero penetration. In the test with R_d - 1.0 the load necessary to cause failure was not large enough to seat the ball completely, and the penetration of the pile and movement of the ball alone during seating were difficult to separate and plot. Little or no soil-to-pile contact at certain locations along the pile might account for this very low ultimate load.

The variance of the T-S ratios with sample depth and pilot hole size, or R_d ratio, is quite similar to the trends noted using the 2.5 inch samples with the 1/4 inch pile. (See Fig. 20.) The sample with the 3/8 inch pilot hole, R_d = 0.75 yielded the highest T-S ratio. The sample with no pilot hole, R_d = 0, produced T-S ratios less than those from samples with pilot holes smaller than the pile diameter. This suggested that insertion of the model pile without a pilot hole disturbed the soil to a greater extent than if a pilot hole was used.

The low T-S ratios for the sample with R_d = 1.0 indicates that the confining pressures available in the 2.5 inch sample were not of sufficient magnitude to cause an elastic deformation of the soil against the pile wall, and low soil-pile
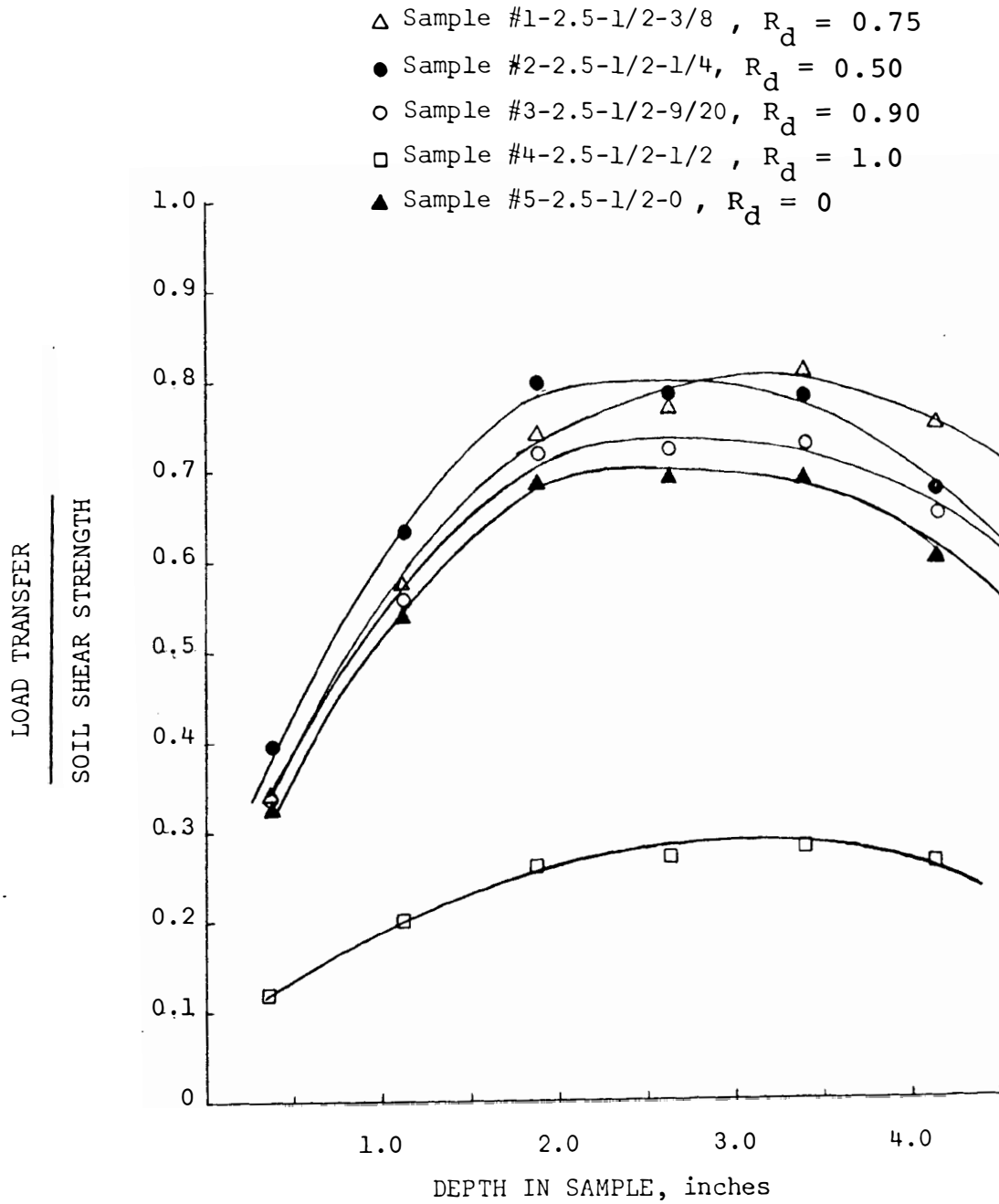
Fig. 20.   Variation of T-S ratio with depth for various
             R_d ratios - 1/2 inch pile, 2.5 inch samples

adhesion was developed. As suggested previously, there may have been sections of the pile that did not make contact with the soil when R_d = 1.0, for after a few such tests very little soil was seen trapped in the pile striations. However, in calculating the T-S ratios in samples with R_d = 1.0, the soil-to-pile contact was assumed uniform as in all other samples, even though the physical situation may have differed.

The maximum T-S ratio as a function of the R_d ratio in this study is shown more clearly in Fig. 21. The highest point on the curve indicates that a pilot hole equal to 0.75 times the 1/2 inch pile diameter is optimum in the 2.5 inch samples. Attention is called to the high degree of similarity between this curve and the upper curve of Fig. 18, which also applies to tests in 2.5 inch samples. The absolute values of the T-S ratios in both the curves referred to differ by less than 7 per cent. The fact that the 1/2 inch pile was rougher than the 1/4 inch pile easily accounts for the slightly higher adhesion values in this last pilot hole study. The maximum T-S ratio which occurred in the bottom half of the samples were used for comparison in Figs. 18 and 21. The order of the ratios may have changed at shallow depths where the T-S ratio values were closely grouped.

It appears that the optimum R_d ratio does not change within the limits of the pile sizes studied, provided such factors as sample size and consistency, confining pressure and pile roughness do not change perceptively from test to test.
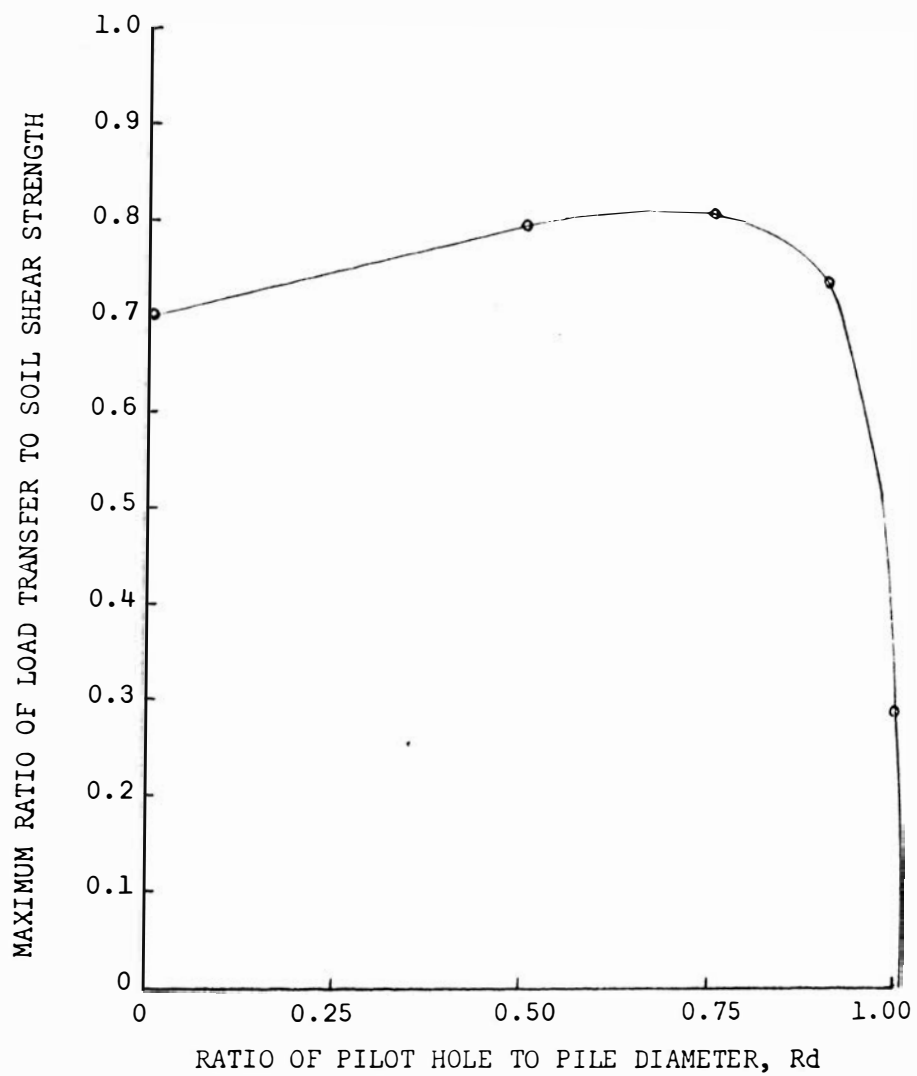
Fig. 21.   Results of pilot hole study to determine optimum R_d ratio - 1/2 inch pile

Load tests with the 1/2 inch pile were also conducted using two 1/4 inch samples to study any effect on load transfer from a change in sample size. No pilot hole study was carried out for the 1/2 inch pile in 1.4 inch samples due to the shortage and undesirable characteristics of the smaller samples. For this reason the optimum pilot hole size determined in the study on the 1/2 inch pile above, 3/8 inch, was used in the smaller samples.

The load versus penetration curves for the two tests are given in Fig. 22. It is believed the relative softness of the smaller samples caused the linear portions of the two curves to be less steep than in the curves from tests in 2.5 inch samples. Although the ultimate load for sample #2 was the lower of the two, the vane shear strength distribution for that sample is generally higher. (See Fig. 30, Appendix A.) Since the pilot hole size was the same for both tests, the opposite trend seemed reasonable.

The T-S ratios of sample #1 are nearly equivalent to those in the 2.5 inch sample with the 3/8 inch pilot hole. (See Fig. 23.) Unlike the 1/4 inch pile in the smaller samples, the 1/2 inch pile could have forced enough soil against the jacket to generate higher confining pressures, and thus the relatively high adhesion, or T-S ratios. The low T-S ratios for sample #2 are not considered within the assumed range of normal error (5 to 10 per cent) associated with tests conducted under similar conditions.
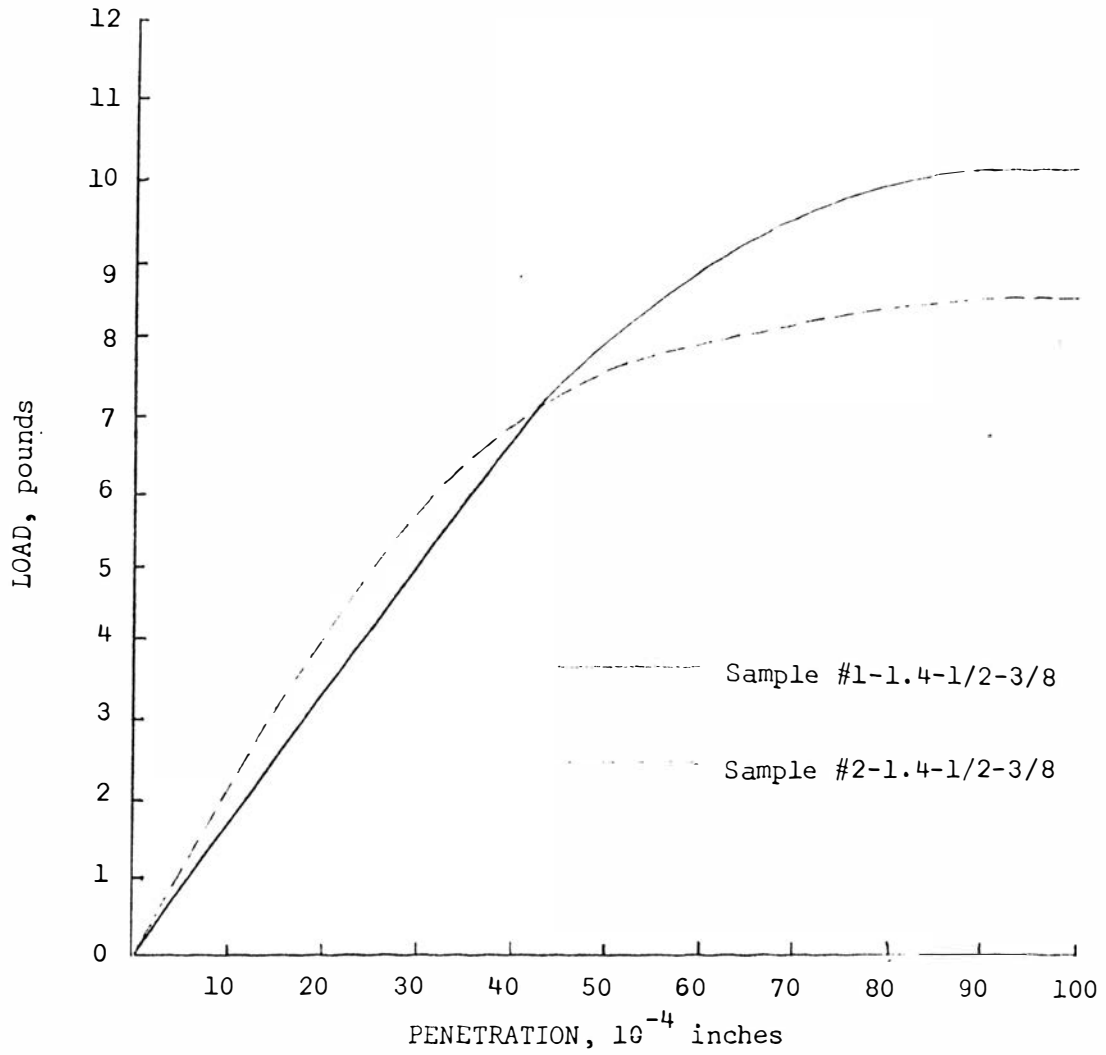
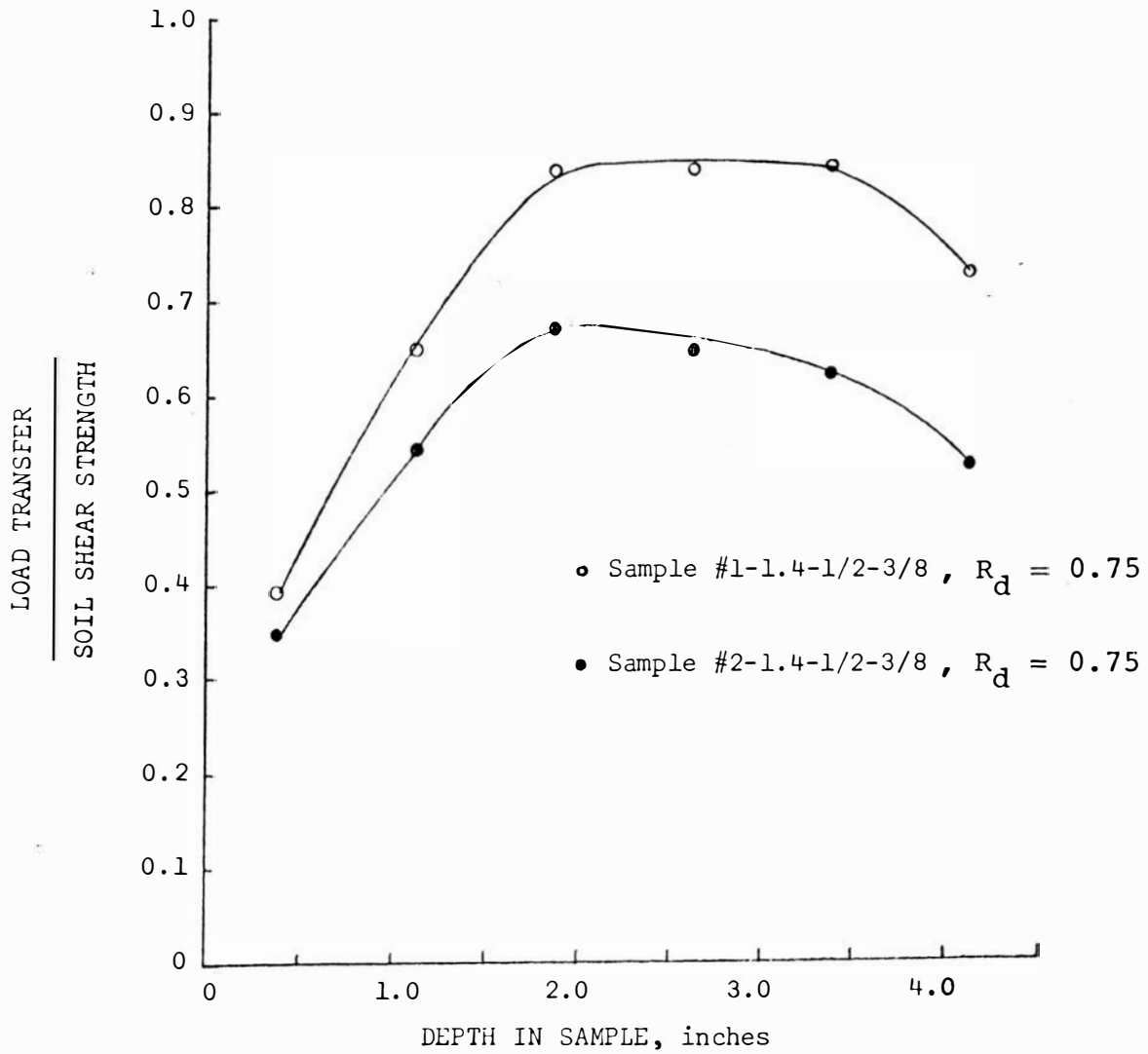Fig. 22.   Load versus penetration for 1/2 inch pile
in 1.4 inch samples

Fig. 23.   Variation of T-S ratio with depth for
1/2 inch pile in 1.4 inch samples

4. Load tests with 3/4 inch pile

Load tests with the 3/4 inch pile in both sizes of samples were conducted to study the load transfer aspects of a model pile that is relatively large compared to the sample size. No pilot hole study was carried out for the 3/4 inch pile due to the lack of assorted pilot hole cutters large enough to provide a wide and useful range of R_d ratios. A 1/2 inch pilot hole cutter was the largest available, and it was used in all tests with the 3/4 inch pile, giving an R_d ratio equal to 0.66.

The load versus penetration curves for the four tests are shown in Fig. 24. The same steel ball used with the 1/2 inch pile was seated in the top of the hollow 3/4 inch pile, therefore the same small initial non-linear portions of the curves were evident, and the steep linear portions were adjusted to zero penetration. The two upper curves, for tests using the 2.5 inch samples, have well defined linear portions with failure occurring at 0.006 inch. Sample #2-2.5 developed a high ultimate load relative to sample #1-2.5 considering its shear strength distribution is only slightly greater. (See Fig. 32 , Appendix A.) As mentioned previously, the so-called 3/4 inch pile was actually 0.705 inch in diameter, and the nearest-sized guide hole in the plexiglass frame that would maintain proper alignment, 23/32 inch, provided only 0.013 inch clearance for the pile. Any slight shift of the metal frame in the loading machine would have caused the pile to
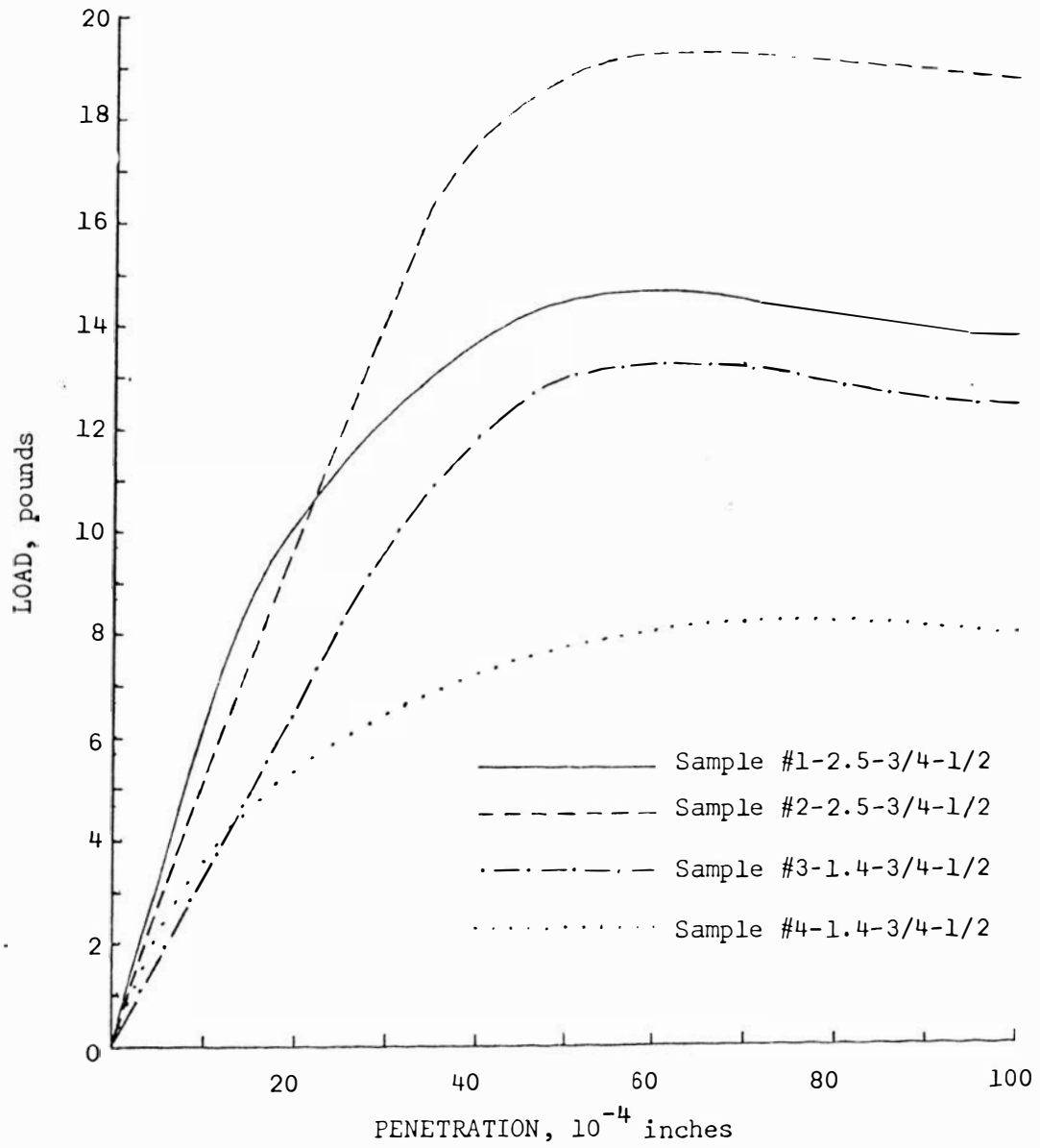
Fig. 24.   Load versus penetration for 3/4 inch pile
            in 1.4 and 2.5 inch samples

bind against the plexiglass frame and produce a frictional force independent of soil-pile adhesion. Although reasonable care was taken in all the tests, this friction would have accounted for unusually high ultimate loads.

The ultimate load for sample #2-1.4, shown in Fig. 24, is hardly more than one half the ultimate load for the other 1.4 inch sample although their shear strength distributions are very similar. (See Fig. 32, Appendix A.) It is thought that base failure may have been the cause for such a low ultimate load, compared to the results of the other tests. Base failure occurred when the load on the pile, due to soil-pile adhesion built up during the test, was sufficient to cause vertical deformation in the bottom of the sample where the soil consistency is softest.

During base failure the pile movement was not completely relative to the soil sample, and the pile and sample, acting as a unit, moved downward as the base of the sample was compressed. Because the pile was moving through the sample very little, lower adhesion was developed before a total pile movement of 0.01 inch was reached. The concept of base failure seems particularily applicable where the model pile is large and the sample small and relatively soft at the base. It could have also caused the erratic results of tests on the 1/2 inch pile in 1/4 inch samples.

The variance of the maximum T-S ratios from sample to sample showed little definite trends. (See Fig. 25). The respective T-S values of the two curves for the 2.5 inch
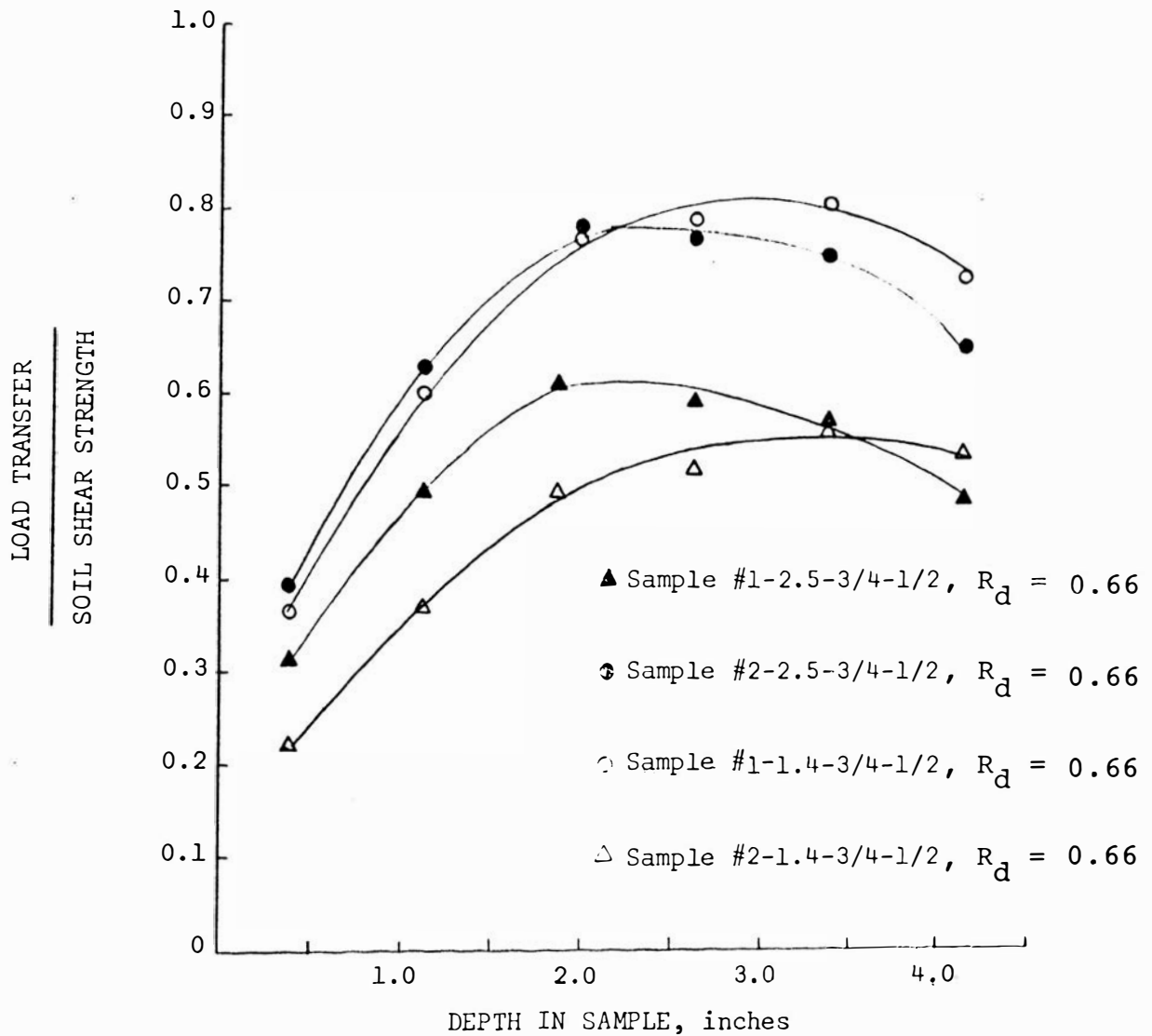
Fig. 25.   Variation of T-S ratio with depth for 3/4 inch pile in 1.4 and 2.5 inch samples

samples differed by nearly 30 per cent, and the curves for the smaller samples disagree by a factor of 40 per cent or more. A deviation of results of this magnitude between tests conducted under similar conditions was not expected.

Apparently the insertion of such a large model pile, compared to the sample sizes, disrupted, and possibly damaged, the samples to such an extent as to impose testing conditions that varied from test to test. Whatever factors contributed to the inconsistent results of the 3/4 inch pile test series, it is evident that in model pile tests of this nature the sample diameter should be at least four to five times the diameter of the pile in order to achieve reproducible results.

C.   Load Transfer as a Function of Time

A review of literature established the fact that the ultimate load that a pile can support increases with time after it is driven. A 1/2 inch pile was inserted into a 2.5 inch sample with no pilot hole and tested at set time intervals to determine whether the same phenomenon would occur with the model pile-soil system of this research. Knowledge was also desired concerning how the ultimate load at 90 minutes, the usual time the pile was allowed to set before testing, compared with the ultimate load determined immediately after, and several days after insertion of the pile. The results of those tests appear in Fig. 26.
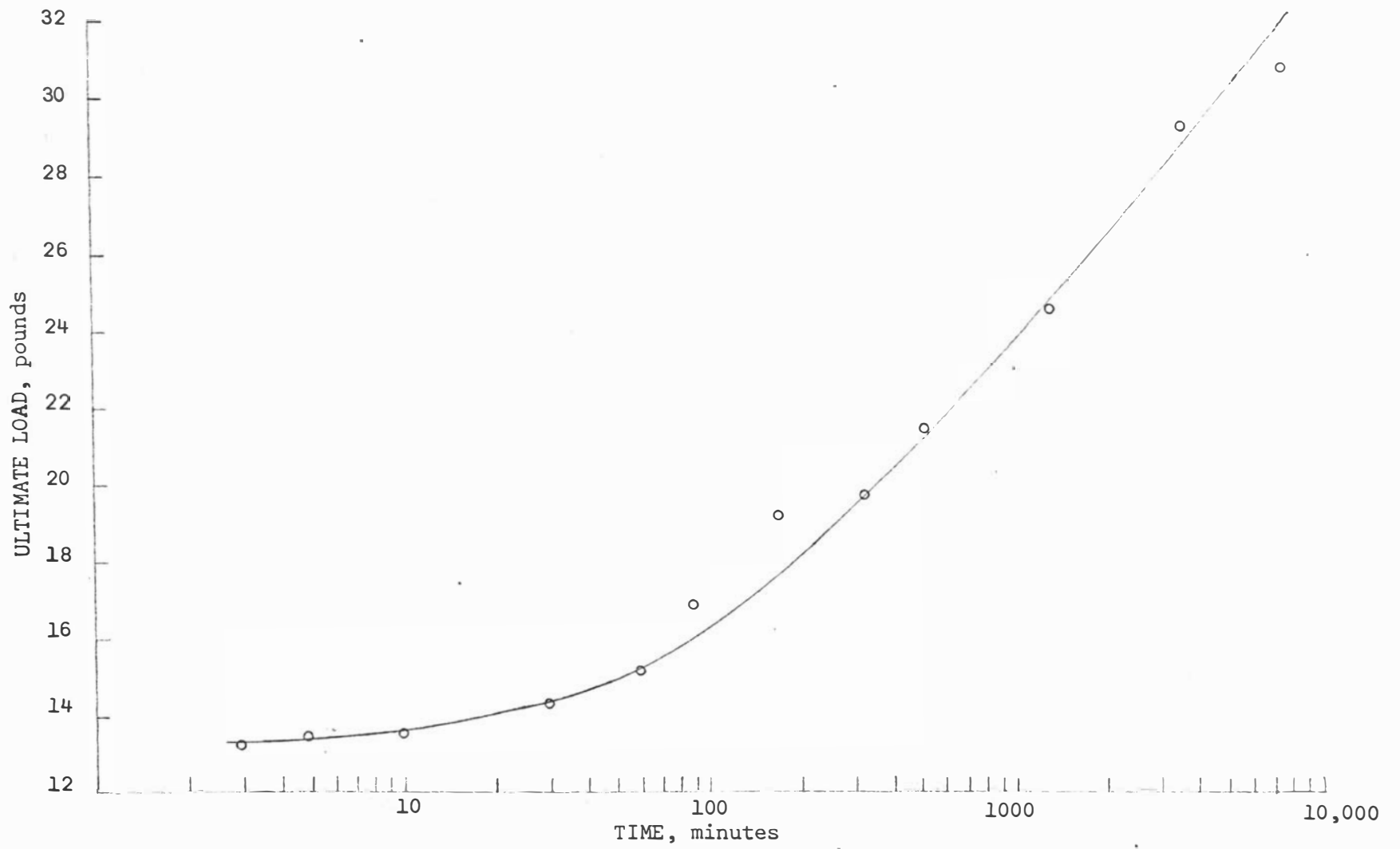
Fig. 26.   Increase in ultimate load with time after pile insertion

The ultimate load which the pile supported increased with time similar to the compression of a confined soil layer under constant load, and implies that similar mechanisms may apply to both cases. There is a slight dip in the curve near the 150 minute interval that deviates from the smooth consolidation-type curve drawn. No explanation could be given for this deviation with the available data. As the curve of Fig. 26 shows, five days after insertion the ultimate load had doubled and there was no tendency for the rate of increase of ultimate load to level off.

The curves of Fig. 27 show how the T-S ratios within the sample increased with time. The curve for the 90 minute test gave T-S values higher than those from similar tests reported previously. Due to the method of testing, the pile was loaded 90 minutes after a previous test, and not strictly 90 minutes after insertion, therefore a slight difference in results.

Unlike other T-S ratio curves, Fig. 27 shows the T-S ratios increasing above 1.0. It must be kept in mind that the vane shear values used to calculate the T-S ratios were obtained from tests performed at least 1/4 inch away from the pile surface. The fact that T-S ratios greater than 1.0 occurred does not suggest the development of adhesion greater than the shear strength of the soil, but simply that the shear strength of the soil next to the pile increased, and thus the adhesion increased. This increase was due to disturbance (or remolding), densification, consolidation and thixotropic hardening of the soil immediately surrounding the pile. The
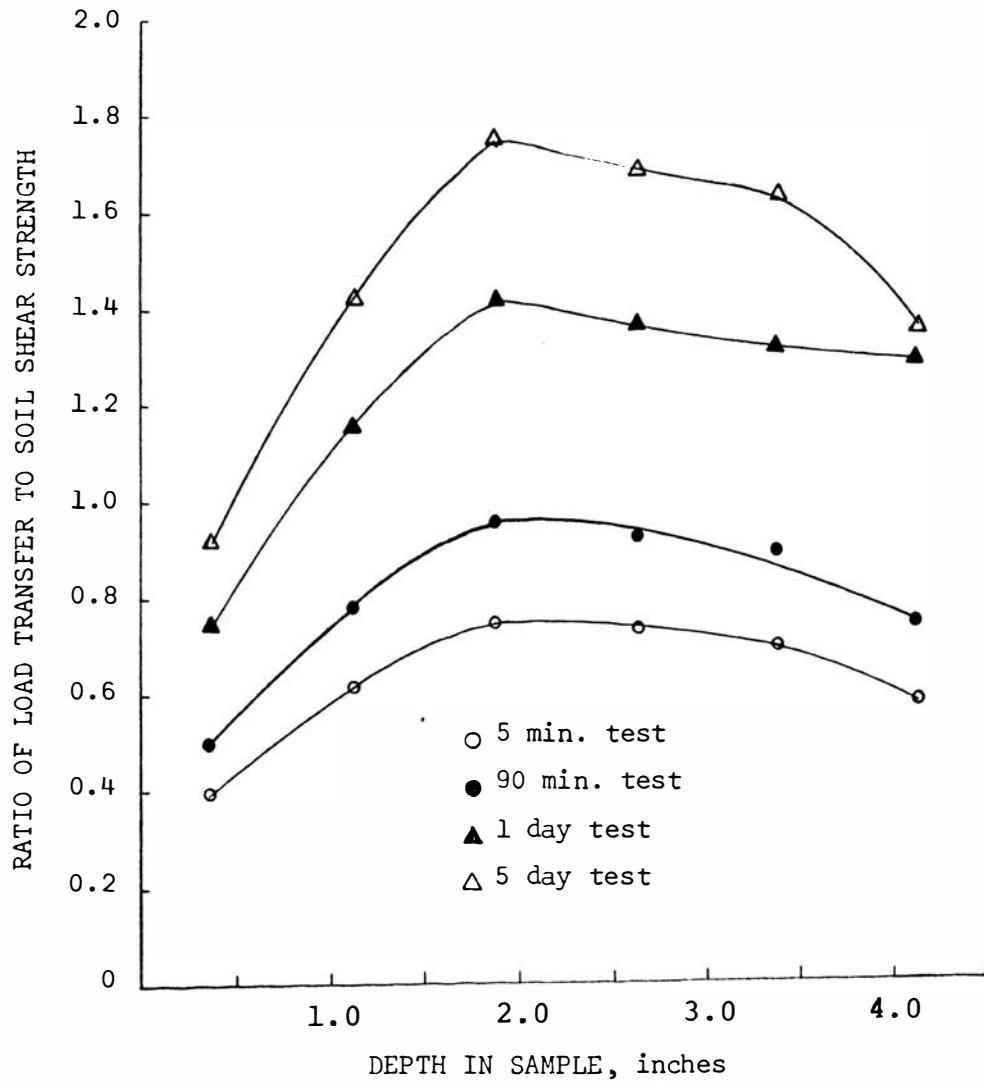
Fig. 27.   Increase in T-S ratios with time after pile insertion

T-S ratios greater than 1.0 resulted from comparing the adhesion between the pile and the newly developed soil structure with the shear strength of the soil before disturbance and gain in strength. Cummings, et. al. (1950) have noted similar results.

A greater amount of soil was seen adhering to the pile as it was extracted after the elapsed time test than in any prior tests. The more dense structure and greater adhesion that was developed at the pile-soil interface with the passage of time could have easily accounted for this.

The results of this test indicated that insertion of the pile caused disturbance that resulted in a subsequent gain in strength above the original strength of the soil. Also, it appears that the load transfer between the pile and soil became less dependent on soil-pile adhesion, and more a function of the soil's shear strength with the passage of time.

D.   Load Transfer as a Function of Penetration Rate

Load tests with various penetration rates were carried out on the 1/2 inch pile in a 2.5 inch sample with no pilot hole, and the change in ultimate load with the change in penetration rate was noted. The results of those tests are shown in the graph of Fig. 28.

Irrespective of the rates of penetration used, attainment of the ultimate loads took place at a pile movement of 0.004 to 0.005 inch. A CRP (constant rate of penetration) of 0.032
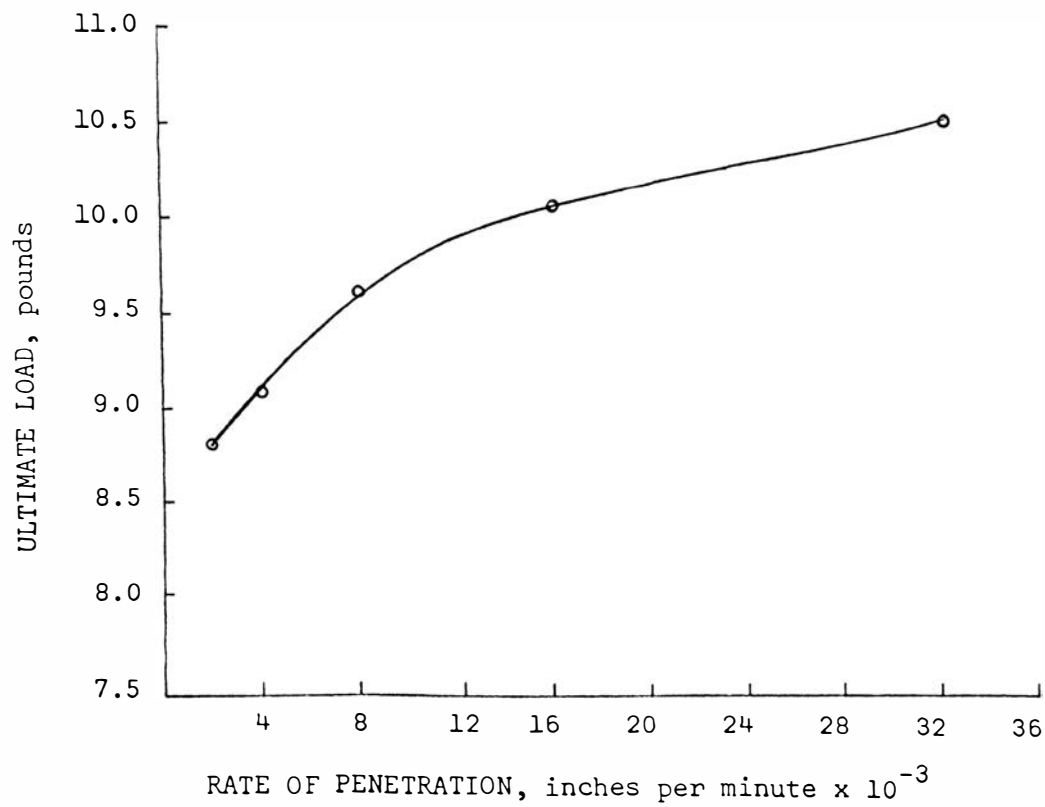
Fig. 28. Ultimate load versus penetration rate

inches per minute, sixteen times the normal CRP used, yielded a 20 per cent increase in the ultimate load over that obtained with a rate of 0.002 inches per minute. It appears that with rapid loading from the high CRP's bridging occurred in the soil structure and lower pore water pressures were developed. Lower pore pressures allowed higher effective stress and the soil structure supported more load from the pile. Richardson and Whitman (1963) reported the increase in load capacity of soil with high strain rates was most prominent at low strains, as in the load tests of this research. Failure occurred in only 12 seconds at the CRP of 0.032 inches per minute, thus no tests at higher rates were conducted due to the inability to monitor the measuring instruments at a faster rate.

Figure 29, based on the same data as Fig. 28, shows there was a near-linear relationship between the increase of ultimate load and the logarithm of the rate of penetration for the range of CRP's tested. Only a more extensive and detailed study of the same nature would indicate if this type of relationship is valid in all soil-pile systems and ranges of penetration rate.

Whitaker and Cooke (1961) observed only a 4 per cent increase in ultimate load with the highest multiple (8) of their standard CRP. Failure in their tests occurred in three to ten minutes, although the standard CRP was 0.06 inches per minute, thirty times the standard penetration rate in this research.
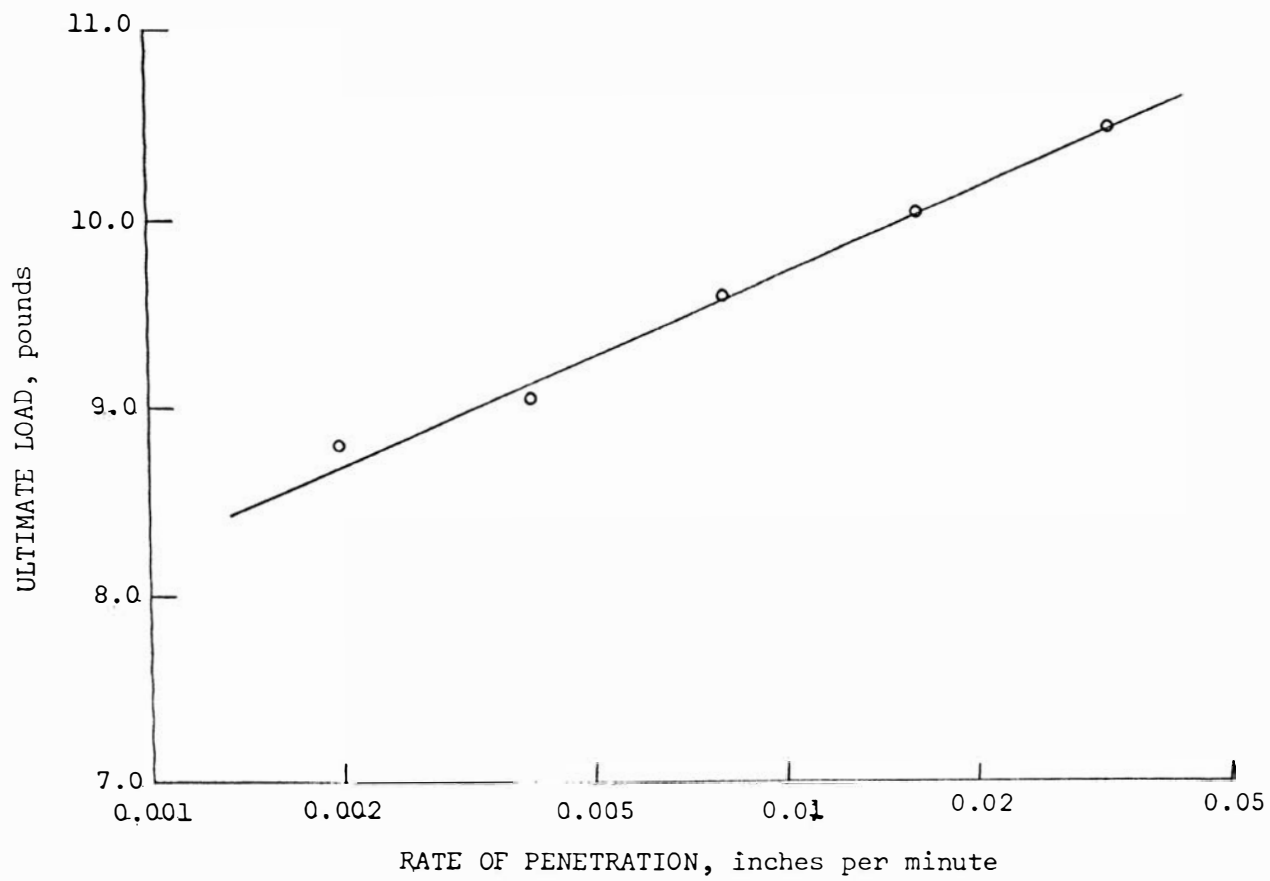
Fig. 29. Ultimate load versus log penetration
rate

The necessity of such a relatively slow CRP, and failure at the very low penetration values, also indicates that failure was a function of soil-pile adhesion in this research. It is thought that soil-to-soil failure, noted in other model pile studies, would have occurred at much greater pile movements compared to the penetrations at failure in this research.

V. SUMMARY AND CONCLUSIONS

A review of literature on soil-pile interaction studies with full-scale instrumented piles shows that displacement piles driven into soft clay cause disturbance to the soil structure and an increase in the pore water pressure of the system. The degree of disturbance is dependent on the liquidity index and sensitivity of the soil. Many programs of research have shown that model piles reproduce the action of full-scale piles and offer the advantage of increased flexibility with respect to the foundation geometry and soil conditions.

In this research load tests on model friction piles were conducted by measuring the load required to maintain a constant rate of penetration (CRP) of the pile. Three sizes of metal piles were inserted into two sizes of encased sedimented soil samples consisting of a definite proportion of silt and clay. Assorted sizes of pilot holes were cut in the samples before pile insertion and specially-made alignment frames and load measuring instruments were incorporated in the test procedures. The purpose of these tests was to study the effects of varying pile, sample and pilot hole size on the ultimate load capacity of the piles.

Vane shear tests were carried out in order to obtain shear strength distribution curves for each soil sample, and to correlate vane shear strength to the unconfined compressive strength of the soil. A theoretical load distribution curve

for the pile was developed whereby the load transmitted to the soil could be compared to the vane shear strength of the soil at six depths in the samples.

The conclusions from this study are:

1. The vane shear strength of the soil decreased with sample depth whereas the moisture content and void ratio increased with depth.

2. The ratio of the vane shear strength to one half the unconfined compressive strength was always less than 1.0 and varied with the moisture content of the sedimented soil.

3. The load on the pile varied linearily with penetration for small pile movements before failure.

4. The value of penetration at failure was constant with time after pile insertion and was not generally a function of pile diameter, pilot hole size of sample size.

4. No model friction pile developed the vane shear strength of the soil as skin friction within 90 minutes after pile insertion.

6. The very low penetration at failure and lack of soil adhering to the pile inferred that failure was a function of soil-pile adhesion in the standard 90 minute test.

7. The ratio of the load transferred to the soil
    to the shear strength of the soil changed with
    sample depth and was highest near the sample
    bottom.

8. The T-S ratios in the soil sample were a function
    of the ratio of pilot hole diameter to the pile
    diameter, or R_d ratio.

9. There was an optimum R_d ratio, usually 0.75, that
    offered a balance between low soil disturbance
    with pile insertion and high load carrying capac-
    ity of the pile. This optimum value did not
    change with pile size, but did change with sample
    size due to uncontrollable confining pressures
    initiated by pile insertion.

10. When the size of sample used was less than 4 to 5
    times the pile diameter erratic results and pos-
    sibly base failure occurred.

Load tests were also conducted at various time intervals
after pile insertion to study the effects of time on the
ultimate load capacity of the pile. The conclusions from this
study are:

1. The ultimate load that the pile could support
    increased with time after pile insertion.

2. The T-S ratios increased with time and eventually
    became greater than 1.0 when compared to the vane

shear strength of the undisturbed soil. It is
assumed that pile insertion caused remolding and
consolidation that resulted in a more dense soil
structure and higher shear strength adjacent to
the pile wall.

3. It is believed the failure mechanism became less
a function of soil-pile adhesion and more dependent
on the shear strength of the soil with the passage
of time.

Load tests using various penetration rates in the CRP test
showed that the ultimate load capacity of the pile increased
with an increase in penetration rate. Failure in each test
occurred at the same penetration value, and the increase in
load capacity varied almost linearily with the logarithm of
the penetration rate.

VI. RECOMMENDATIONS

Much of the time devoted to this research was consumed in the development and building of equipment, thus the testing program was limited. Many more tests are needed to determine the limits of the variables for which the conclusions previously drawn are valid. The results of this research serve as a basis for the recommendations regarding future study on model testing of this nature.

1. The use of a model pile instrumented with strain gages would produce a more accurate load distribution curve for use in the analysis of results.

2. If, after pile insertion, confining pressures of known magnitude were imposed on sedimented soil samples with L/D ratios of 2.0 or less, then more homogeneous samples would result. The problem of unknown confining pressures affecting results would also be partially solved.

3. A longer time period between pile insertion and load testing, 3 to 5 days, would allow the dissipation of pore water pressures and more complete consolidation of the soil around the pile. With the added time the mechanisms of failure would be more similar to that of a field pile.

4. Results of soil shear strength tests conducted under varied conditions of drainage in a triaxial

cell could be correlated with pile load trans-
fer values in an attempt to determine the exact
soil-pile failure mechanism.

VII.   APPENDIX A

VANE SHEAR STRENGTH DISTRIBUTIONS

FOR SOIL SAMPLES

TOP

DEPTH IN SAMPLE, inches

4.5

4.0

3.5

3.0

2.5

2.0

1.5

1.0

0.5

BOTTOM

200   300   400   500   600

VANE SHEAR STRENGTH, psf

Sample #1-1.4-1/2-3/8
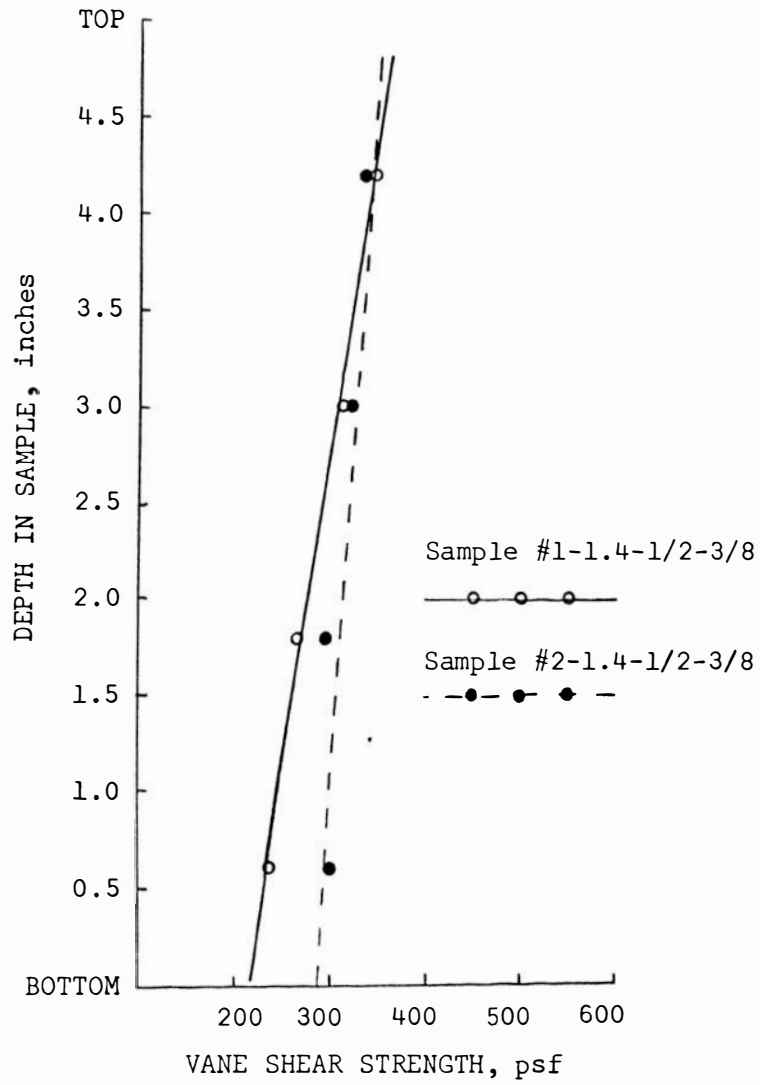
Sample #2-1.4-1/2-3/8

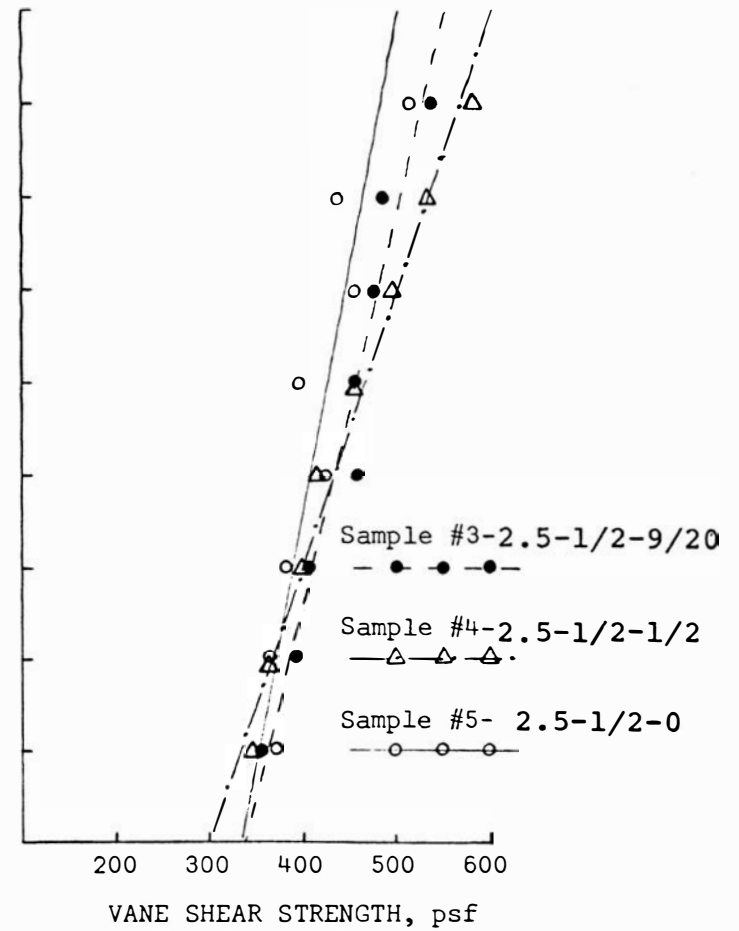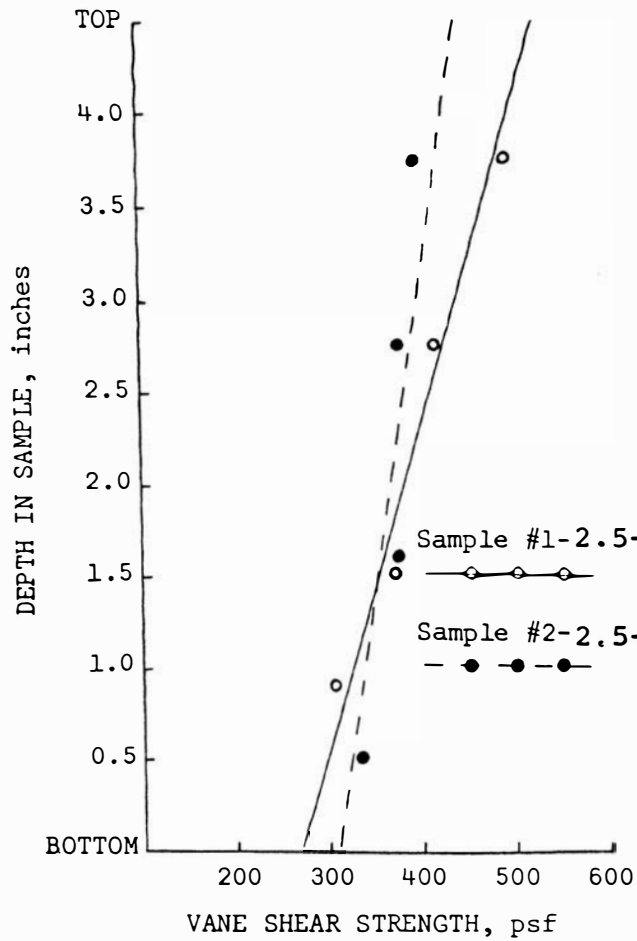Fig. 30.   Vane shear strength distributions for 1.4 inch samples used with 1/2 inch pile
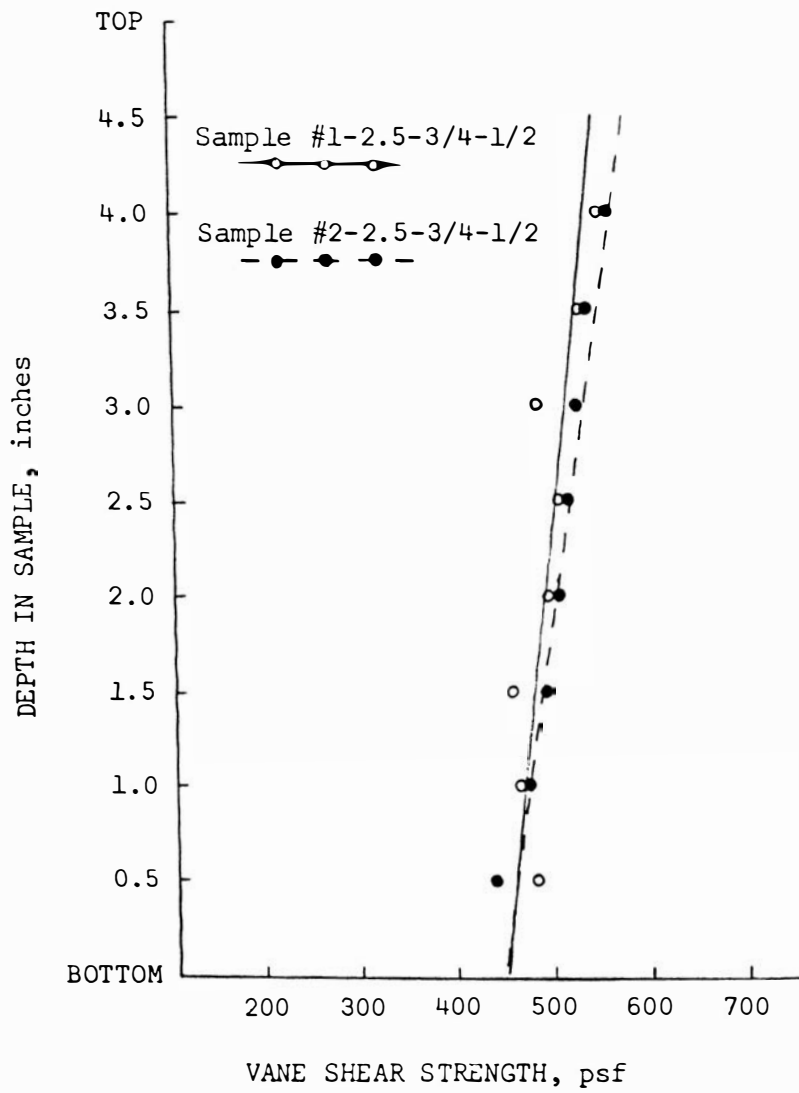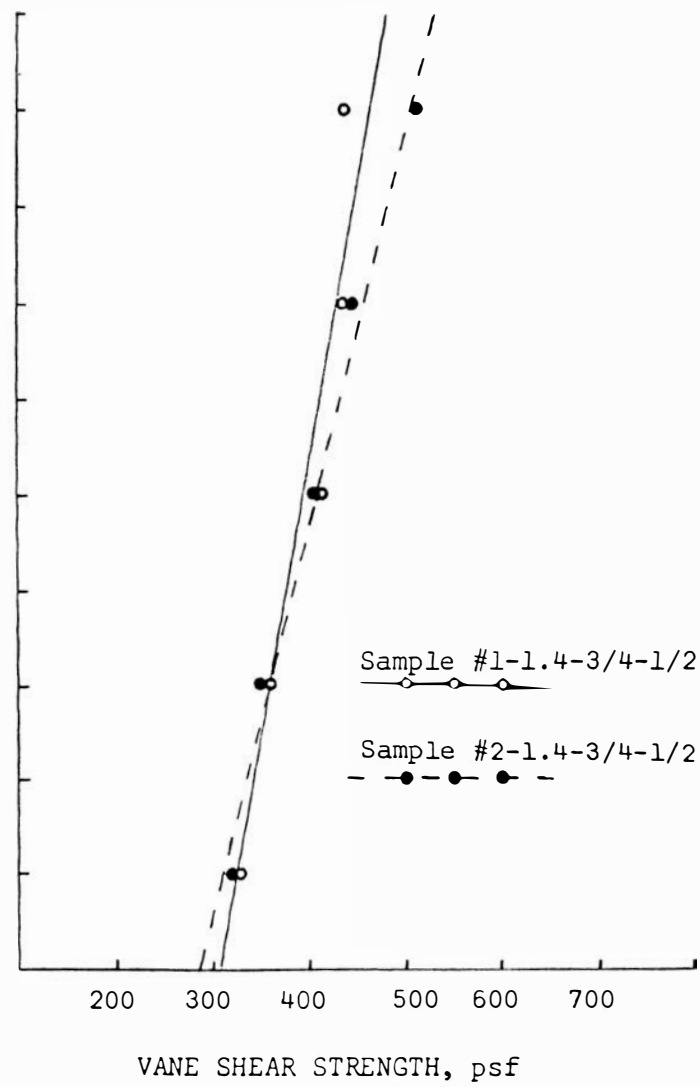
Fig. 31.   Vane shear strength distributions for 2.5 inch samples used with 1/2 inch pile

Fig. 32.   Vane shear strength distributions for 1.4 and
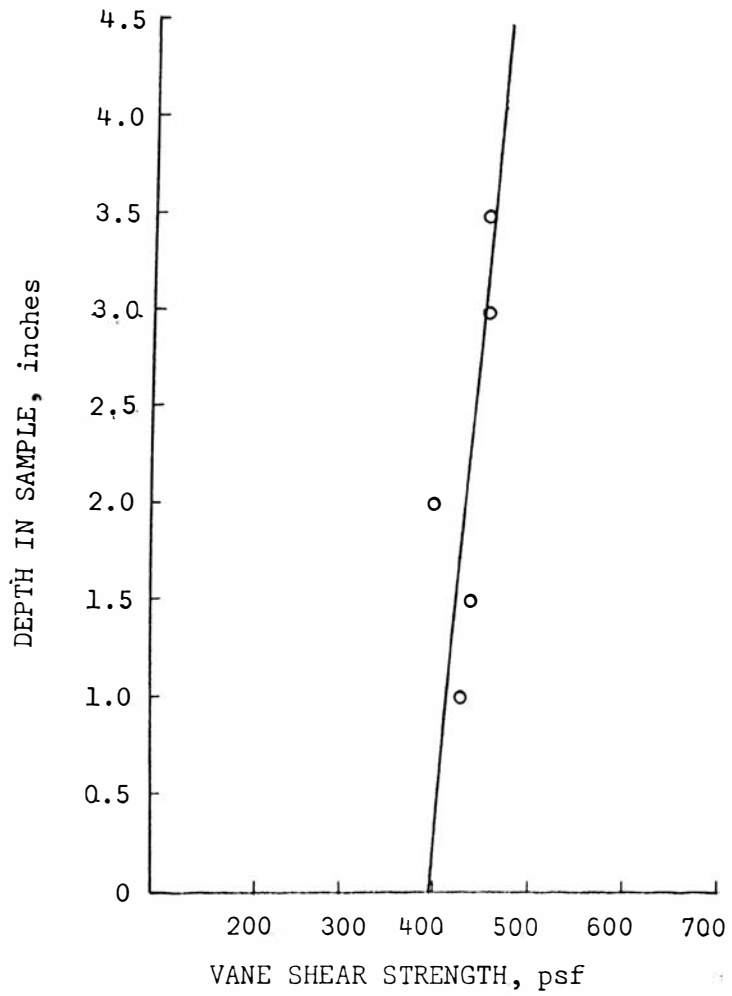2.5 inch samples used with 3/4 inch pile

Fig. 33.   Vane shear strength distribution for
            sample from elapsed time test

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IX. VITA

Richard Milton Franke was born on July 5, 1943 in Centralia, Illinois. He received his primary and secondary education in the Centralia, Illinois Public School System.

In June, 1963 he was graduated from the Centralia Junior College with the degree of Associate in Arts. After studying electrical engineering at the University of Illinois, Urbana, Illinois, he was entered at the University of Missouri- Rolla in February, 1965. A Union Oil Scholarship was awarded him during his undergraduate work and he was graduated with the degree of Bachelor of Science in Civil Engineering in June of 1967. In August, 1967 he was certified an "Engineer in Training" by the Missouri State Board of Architects and Engineers. In September of 1967 he entered graduate school at the University of Missouri-Rolla and was awarded a National Science Foundation traineeship.

Mr. Franke is married to the former Miss Carolyn Vinette Mach of Kansas City, Missouri. He is a member of Chi Epsilon, Tau Beta Pi, and Phi Kappa Phi honorary fraternities.