AN ABSTRACT OF THE THESIS OF

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Title: A DETERMINATION OF ULTIMATE PILE CAPACITY IN CLAYEY SILT

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Predetermining the ultimate capacity of piles driven into purely cohesive soils and purely cohesionless soils involves, respectively, the consideration of soil shear strength in a completely undrained and a completely drained condition. For the intermediate cases concerning soils that possess both cohesion and internal friction (silts, clayey silts, sandy clays, etc.) the capacity prediction may assume either condition of drainage depending upon the characteristics of the soil, its stress history, and the duration of load. The actual conditions of failure for piles driven into these soils are uncertain unless test piles can be observed under a series of loads to failure.

This study involves the determination of the capacity of an individual pile driven into a clayey silt. Capacity predictions based upon static formulas by Terzaghi and Peck, Meyerhof and William Moore are calculated for both drained and undrained conditions around the pile. By the performance of a series of load tests is is found that

in the soil encountered the ultimate load supported by the pile is best described by the Meyerhof determination, assuming a state of complete drainage.

A Determination of Ultimate Pile Capacity in Clayey Silt

by

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

A DETERMINATION OF ULTIMATE PILE CAPACITY IN CLAYEY SILT

INTRODUCTION

The capacities of pile foundations have historically been determined by one of three basic methods: The dynamic formulas based in principle upon the conservation of energy; load testing wherein test piles are actually proof loaded or loaded to failure; and the static formulas combining bearing and frictional resistances determined from soil properties. Accepted mainly for their simple calculations, the dynamic formulas, based upon the concept that energy-in equals work-out, vary extensively in their scopes and complexities from the basic and widely used Engineering News formula to the empirically-reduced Hiley formula. Characteristically, these equations can vary drastically in their results, are considered unreliable when driving piles in cohesive soils or in fine sands below the level of ground water, and afford no indication as to the actual degree of adequacy developed in a foundation. Also, the resulting substructure is not determined until the piles are actually driven, thus discounting the concept of an engineered foundation.

Load testing effects the most reliable and consistent means of pile evaluation. Properly conducted pile tests enable the engineer to correlate actual capacities with the soils encountered in adjacent borings, to observe the effects of cyclic loadings, and to evaluate the influence of load duration and pile settlement. However, these tests, which must be performed prior to design of the foundation, involve several weeks of effort and great expense in labor, equipment and materials. As a result, only the larger projects can economically justify their adoption.

The static formulas predetermine ultimate pile capacities through the employment of soils data from standard laboratory tests. This allows the engineer to separate and analyze the soil strata indicated in borings, to review with reasonable assurance the nature of a pile's supporting capabilities (i.e., end bearing and/or friction), and to select the degree of safety with which the foundation can be expected to perform. Within the boundaries of data normally available in a soils investigation and with the temperance of good judgment, these provide a most rational and convenient method for design of pile foundations.

Many researchers have investigated the bearing capacity of piles in purely cohesive soils, clays, and purely cohesionless soils, sands and gravels, and have postulated static formulas that closely correspond to their particular cases. However, little research has been devoted to the capacity of piles driven into soils that lie in the area between these two extremes; silts, clayey silts, sandy silts, sandy clays.

The purpose of this research is to study pile capacity predictions in clayey silts. In particular, it involves the application of a theoretically-based, semi-empirical, static equation presented by Terzaghi and Peck, a theoretical approach by Meyerhof and a basic form equation patented and proffered by William Moore. The resulting capacity predictions are tested for accuracy by performance of load tests.

DESCRIPTION OF SITE AND PROPOSED CONSTRUCTION

The test site is a 6.3 acre plot in the Hillsdale area of southwest Portland, Oregon. Topographically, it is an open end basin with the north, south and west portions sloping toward a central low area, which in turn slopes to the east. The central and eastern low areas are covered by marsh with ponds forming during the wet months and surface drainage occurring all year. The discharge of subsurface water into the basin is also evident in various locations along the slopes, just above the line of marsh grass.

An apartment complex consisting of nine separate structures is to be erected on this site. The proposed buildings embody two and three story sections, terraced into the ground slopes in two, one story steps. They are 30 feet by 127 feet in plan dimension and are structurally composed of block masonry bearing walls and reinforced concrete floors. To accommodate the buildings earth fills varying in depth from 18 inches to 17 feet are required over the site.

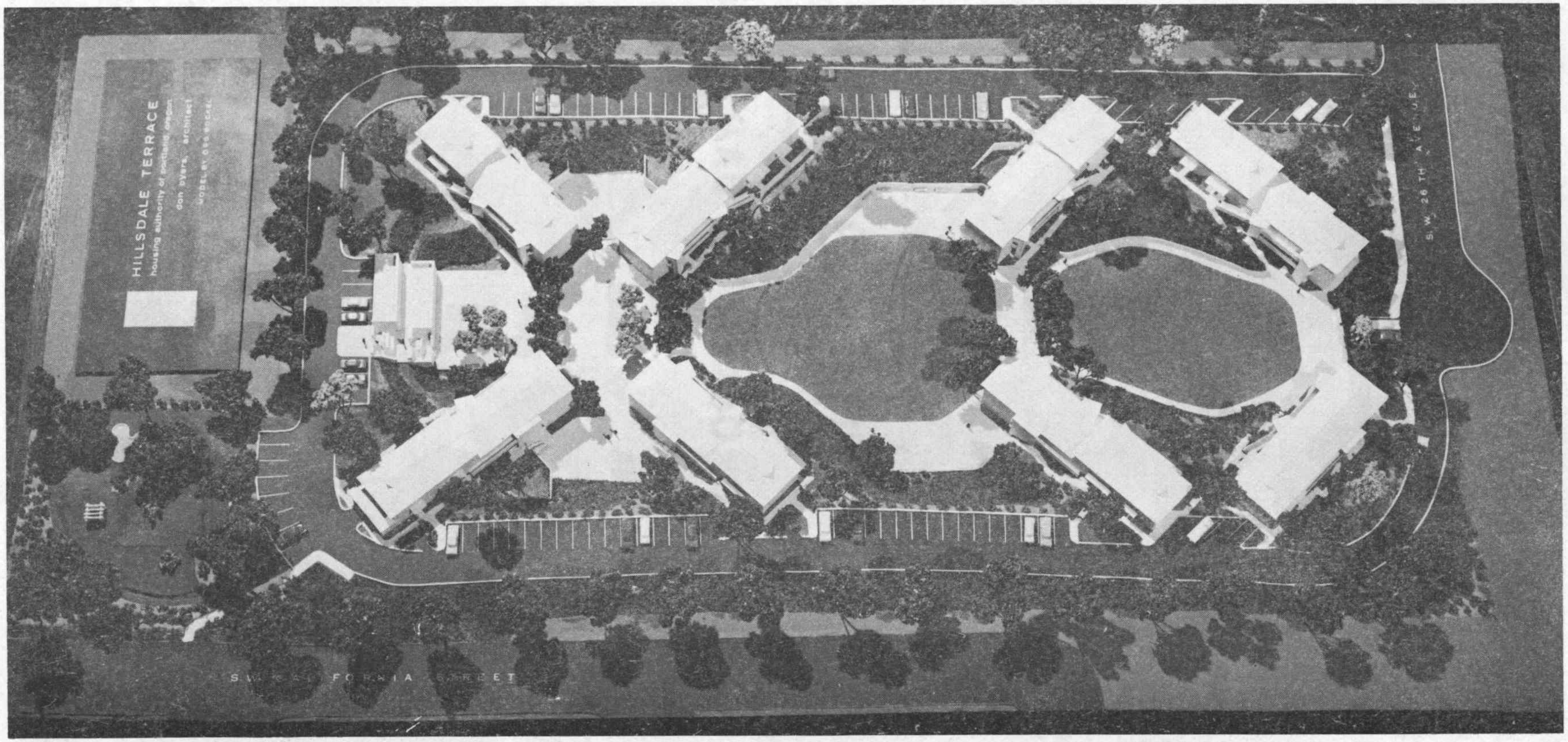


Figure 1. Model of Test Site and Proposed Structures.

SOILS TESTS AND DATA

Direct Shear Data

As part of an original soils investigation conducted in conjunction with the proposed site improvements, Dames and Moore, consulting soils engineers, explored subsurface conditions with 14 borings. These explorations were drilled by a rotary drilling rig with an auger attachment and extended from various beginning elevations to depths ranging from 13 to 15 feet. Soils in the borings were classified in the field by visual and textural examination and undisturbed samples of the various strata were extracted with a Dames and Moore sampler. A log of these borings is presented in Figure 2.

Standard laboratory procedures (25) were employed on the various soils to determine moisture contents, dry densities, plastic limits and liquid limits. These quantities assist in comparing and correlating the different strata as to location and physical characteristics and provide a general image of the soil upon which the engineer can qualitatively predict its reaction under different conditions of service. The results of these tests are given to the left of the boring logs.

Direct shear tests were also performed on the undisturbed samples taken from the borings. This testing was accomplished by the use of a direct shear testing and recording apparatus in which a

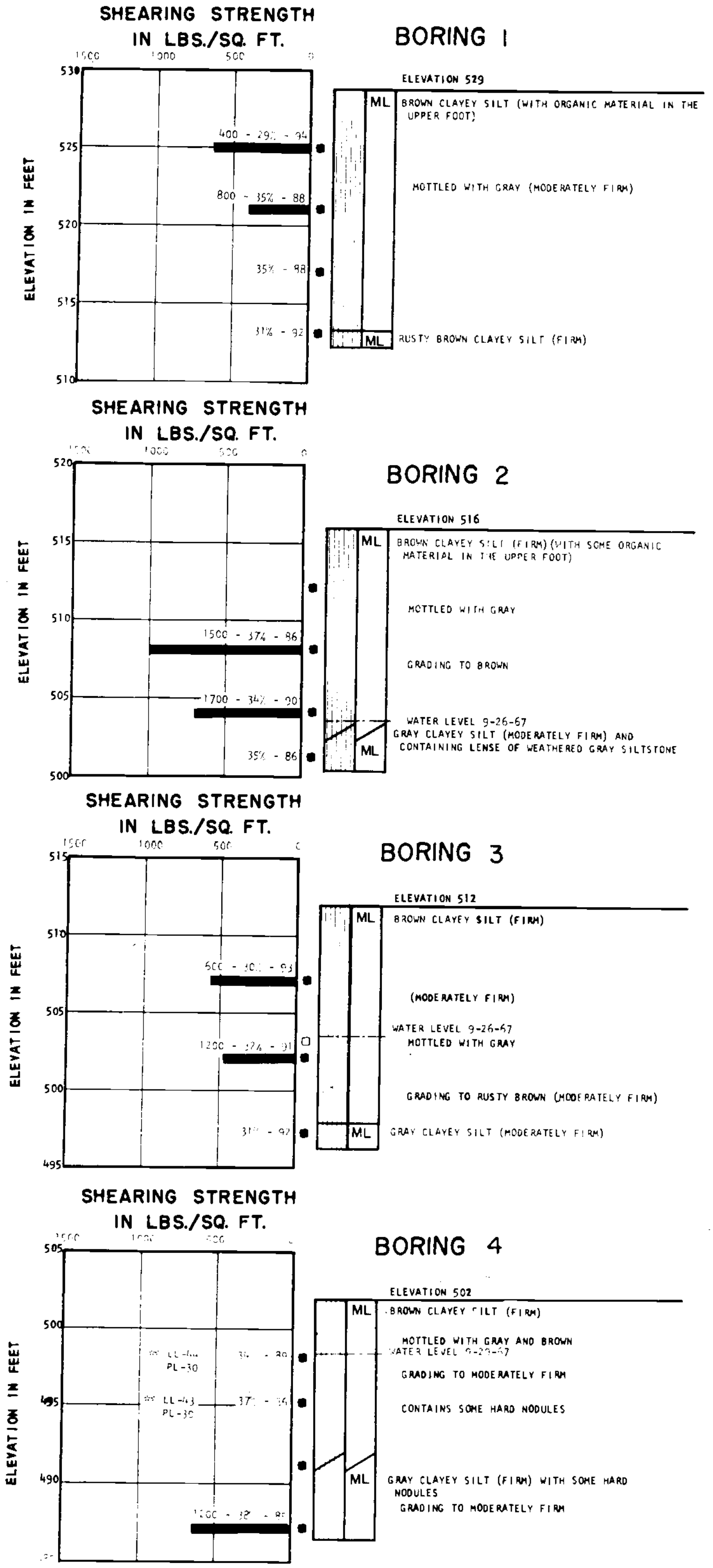
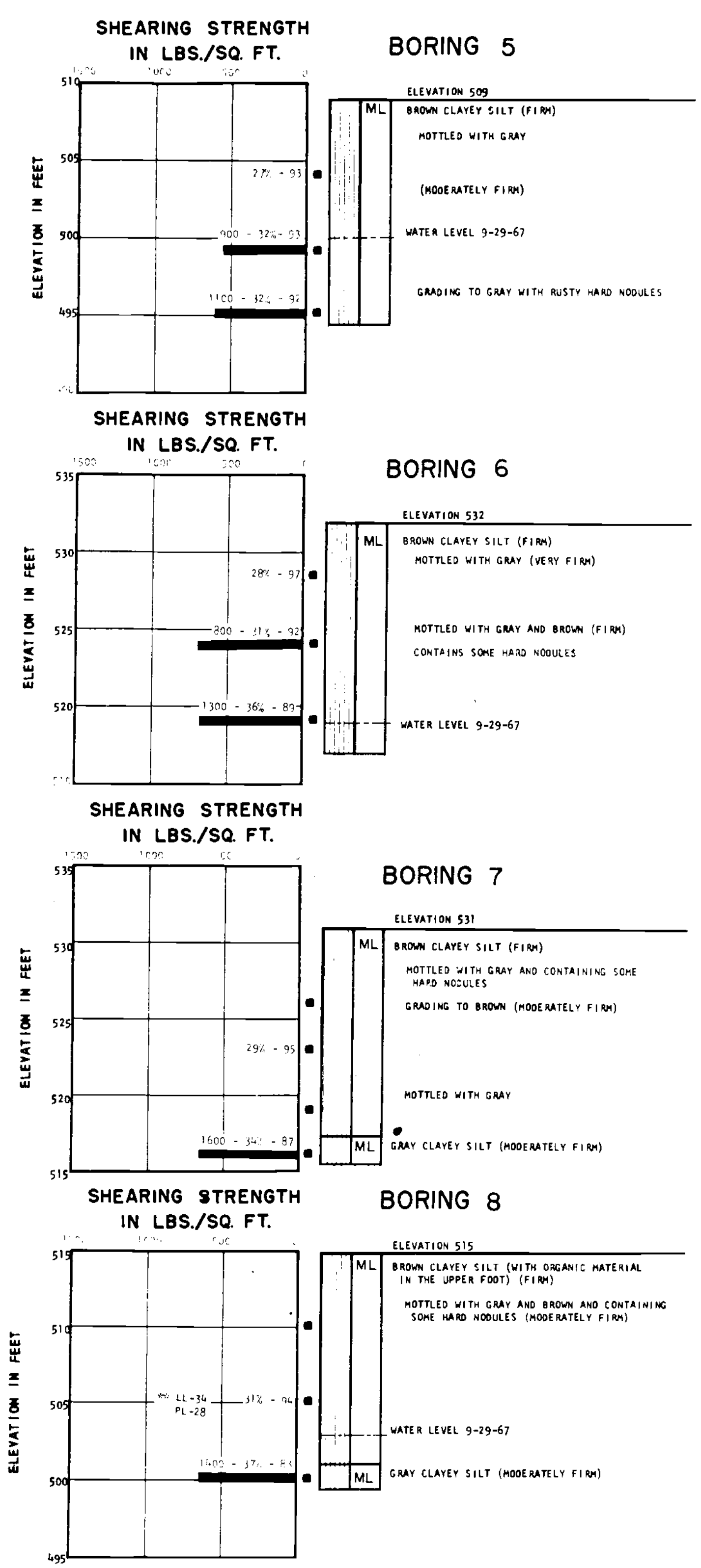


Figure 2. Log of Borings and Soils Data from Direct Shear Tests.



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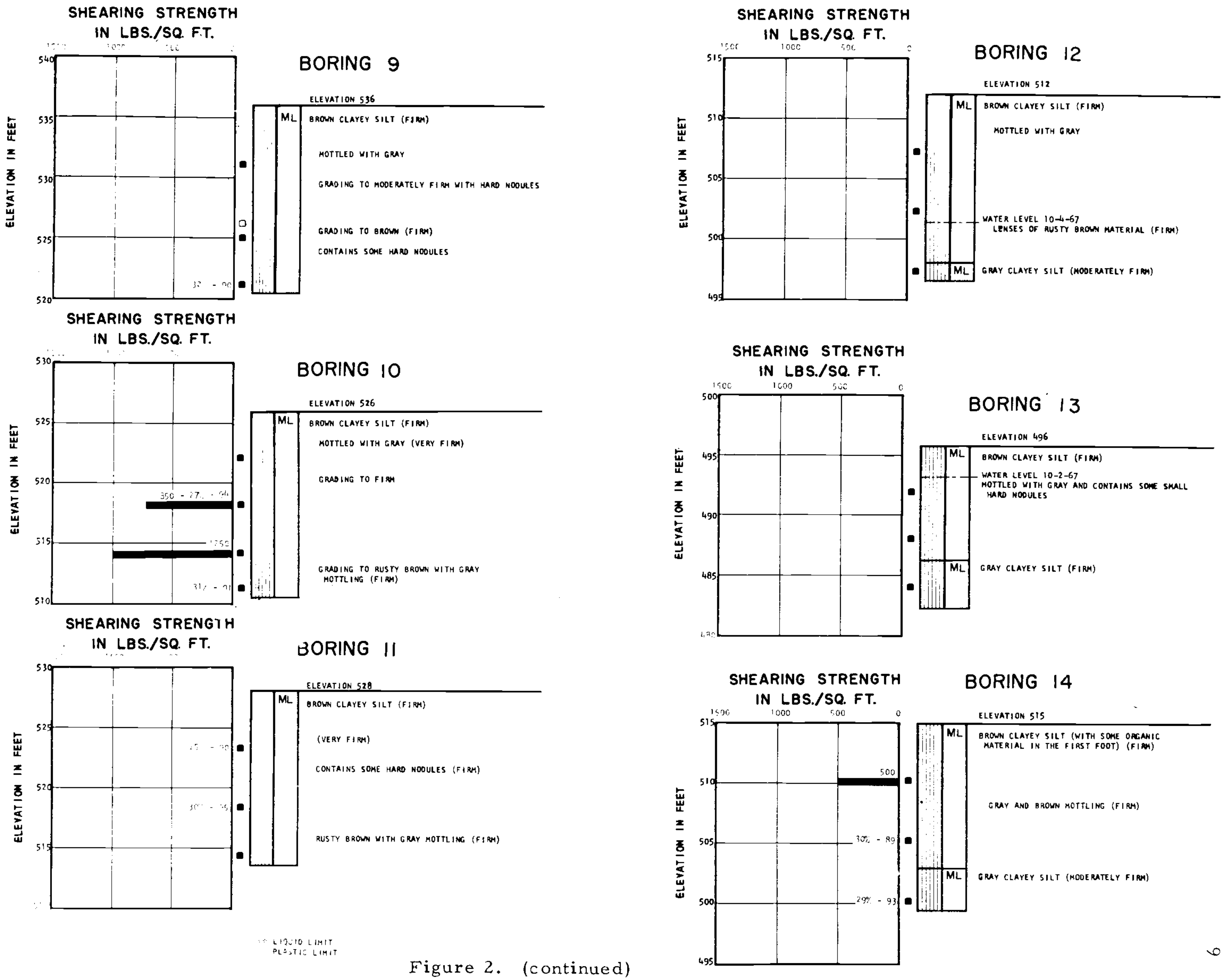


Figure 2. (continued)

three inch long soil sample encased in three brass rings, two and one half inches in diameter and one inch in length, is subjected to direct double shear. During the test a constant confining pressure approximating the in situ condition is applied normal to the ends of the sample through porous stones. The shearing failure is caused by moving the center ring at a constant rate of deflection in a direction perpendicular to the axis of the sample. The samples are free to drain during shearing with the degree of drainage depending upon the rate of shearing and the permeability of the soil. The resulting shearing data is presented in Figure 2.

A plot of confining pressure versus shearing strength is shown in Figure 3. The values of cohesion and angle of internal friction shown on the graph are average values determined by the method of least squares. These represent the soil over the site as a single, uniform material.

Triaxial Shear Tests

A hand auger was used to produce two borings in the immediate test area. After two unsuccessful attempts with a Shelby tube, relatively undisturbed samples of the various strata were obtained with a Dames and Moore sampler. The boring record and subsequently determined soil properties are listed in Table 1.

The performance of a series of triaxial shear tests (2) on these

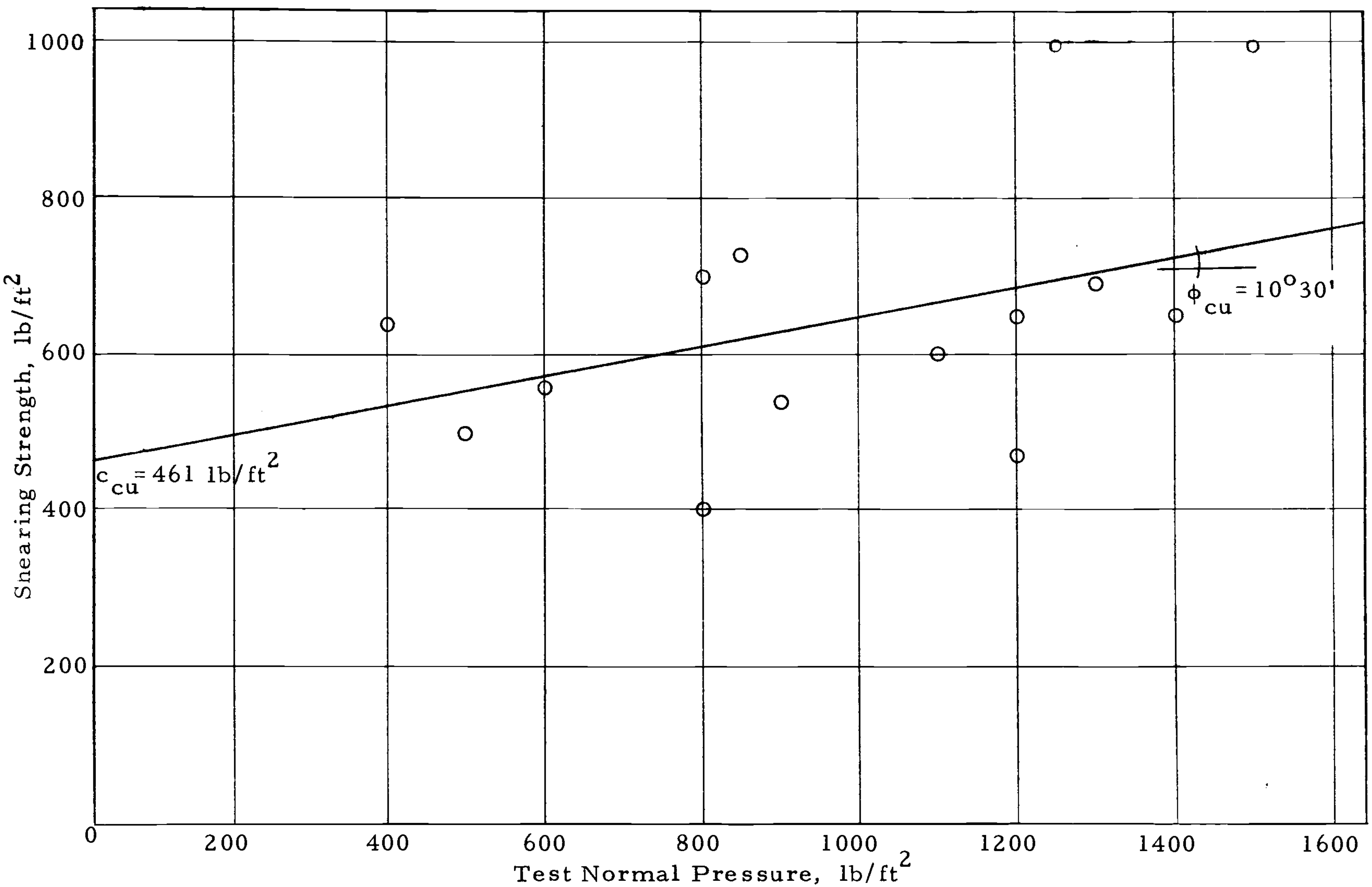


Figure 3. Average Strength Envelope from Direct Shear Tests.

Table 1. Soil data from hand auger borings.

SAMPLE	DEPTH	WATER CONTENT	SOIL DESCRIPTION
	0'		Boring started ten feet east of test pile. Elevation: 518.0
J-1-1	1 1/2'	37.3	Gray brown clayey silt
T-1-1	4'	44.6	Light brown clayey silt.
J-2-1	4'	35.9	Grey brown clayey silt. Sample from second boring two feet south of first boring.
D-2-1	4'	34.6	Grey brown clayey silt. Sample from second boring two feet south of first boring.
J-1-2	6 1/2'	40.8	Light brown clayey silt.
T-1-2	9'	33.6	Light brown clayey silt.
D-1-2	10'	34.2	Light brown clayey silt.
D-1-3	13'	33.4	Grey clayey silt. Brown to grey contact zone @ 13 feet.
J-1-3	13'	30.3	Brown to grey clayey silt--contact zone.

samples was chosen as a means of refining the available soils information. After trimming, the undisturbed samples, 2.8 inches in length and 1.4 inches in diameter, were consolidated under confining pressures simulating those of the soil in place. They were then failed in an undrained condition under a constant rate of strain of 4.1×10^{-3} inches per minute. Readings were made at increments of 0.01 inches of strain.

Following the first failure, two of the samples were allowed to reconsolidate under a confining pressure approximately four times that of the original test. Failure was again induced and recorded under the conditions of drainage and strain previously described.

The resulting Mohr diagrams, illustrating both the total and effective stress circles (total stress σ equals effective stress σ' plus pore pressure u) are shown in Figures 4a and 4b with the corresponding values of cohesion and angle of internal friction.

For comparison purposes the results of the triaxial and direct shear tests are plotted in Figure 4c. The average strength envelope from the direct shear tests is definitely lower than those from the triaxial tests. The low range of parameters indicates that shearing during the direct tests was rapid enough to effect an undrained condition.

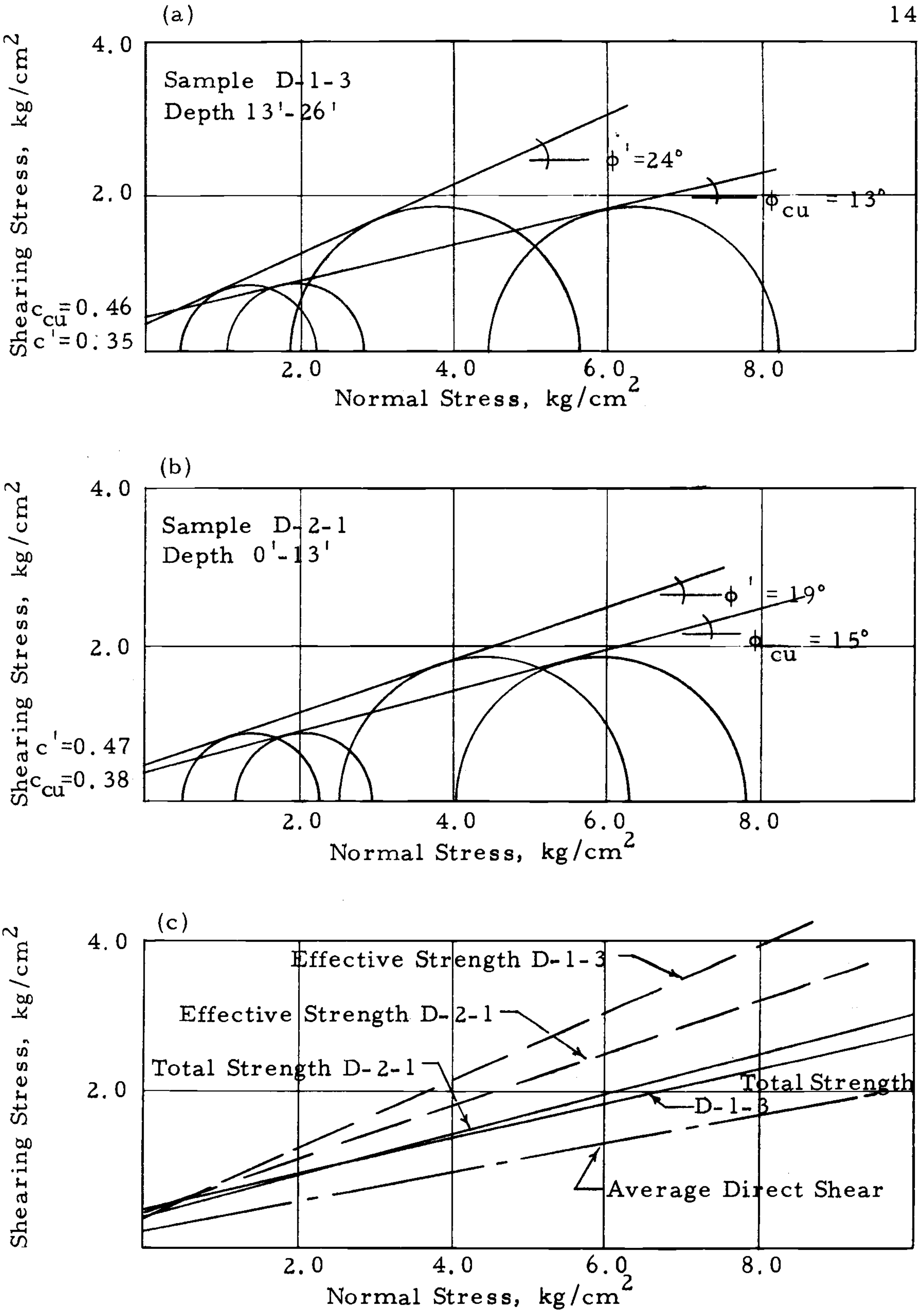


Figure 4. Shear Strength Envelopes from Triaxial Shear Tests.

DETERMINATION OF PILE CAPACITY

General Conditions

In calculating the pile capacities, information derived from the hand auger borings and the triaxial shear tests is selected as best representing the soil near the test pile. The nature of the soil below the depth of the boring is assumed to have properties similar to the soil last encountered. Due to the effects of driving the coefficient of lateral earth pressure (K) is assumed to be 1.0 (13). Soil at the tip of the pile is considered firm and the influence of ground water begins three feet below the ground surface.

A creosote-treated, class B, timber pile is the subject of the determination. It has an embedded length of 26 feet, its tip diameter is nine and three quarters inches and its butt diameter at ground level is twelve and one quarter inches. The sides of the pile taper uniformly at the approximate rate of one inch in 20 feet. It is assumed that the pile is rigid and relatively rough. The test conditions are diagrammed in Figure 5.

Terzaghi and Peck Formula

The basic form of the pile formula proposed by Terzaghi and Peck (23) expresses the static load resistance, Q_u, of a pile to penetration into the ground as the sum of point resistance, Q_{pr}, and skin

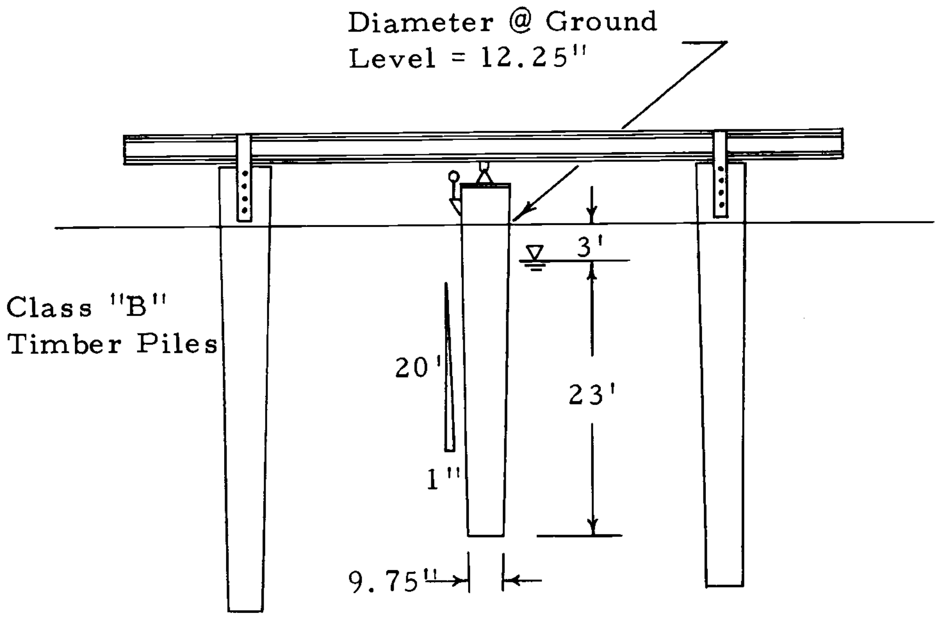


Figure 5. General Conditions for Capacity Determinations.

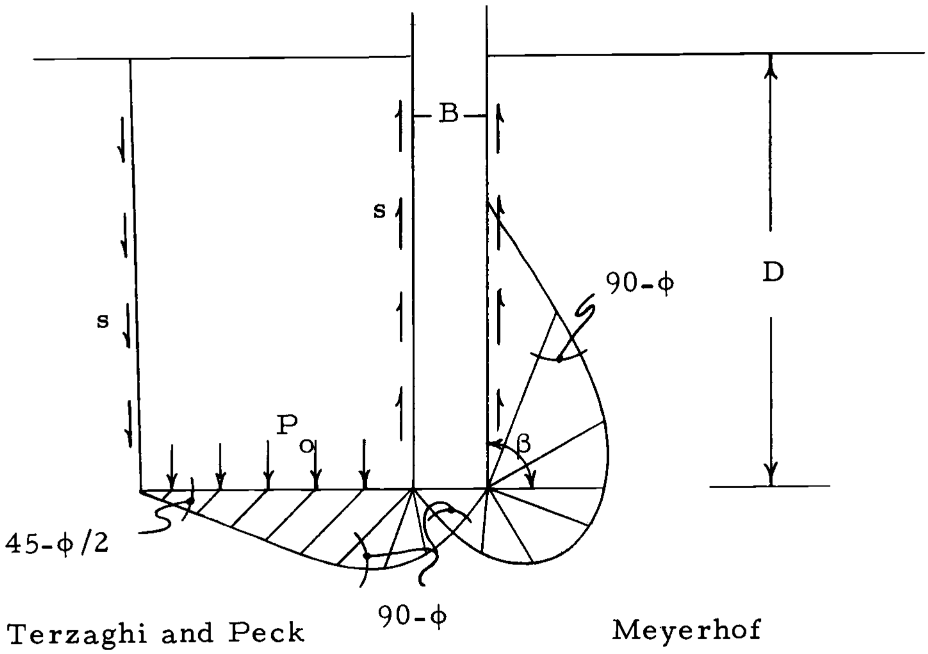


Figure 6. Assumed Failure Conditions in Theoretical Formulas.

friction, $2\pi r\, D_f s$:

$$Q_u = Q_{pr} + 2\pi r\, D_f s$$

where r = average radius of the pile, feet

D_f = friction length in the load carrying strata, feet

s = average ultimate skin friction and adhesion between the pile and the soil, lb/ft^2

By assuming a plastic state of equilibrium beneath the tip of the pile and on the basis of experimental data, the point resistance, Q_{pr}, of a circular footing in firm soil was determined by Terzaghi and Peck to be

$$Q_{pr} = \pi r_p^2 (1.3cN_c + \gamma\, DN_q + 0.6\, \gamma\, r_p N_\gamma)$$

where r_p = tip radius, feet

c = soil cohesion, lb/ft^2

γ = average density of overburden, lb/ft^3

D = depth to the tip, feet

and N_c, N_q, N_γ = dimensionless bearing capacity factors dependent upon the soil's angle of internal friction, ϕ, and the assumed existence of roughness at the base of the pile. Combining this with the skin friction gives the total expression:

$$Q_u = \pi r_p^2 (1.3cN_c + \gamma\, DN_q + 0.6\, \gamma\, r_p N_\gamma) + 2\pi r\, D_f s.$$

Utilizing the triaxial shear test data and calculating the appropriate bearing capacity factors, the Terzaghi and Peck equation predicts an ultimate capacity for the pile of 48.0 tons under drained conditions and 45.1 tons in the undrained state.

Meyerhof Formula

The Meyerhof formula (14) is a theoretical extension of the work by Terzaghi and Peck. This equation is based on the assumptions that the zones of plastic equilibrium increase with foundation depth and will vary with the shape of the foundation (Figure 5). For a circular pile the general form of this equation is

$$Q_u = \pi r_p^2 (cN_{cr} + \gamma DN_{qr} + \gamma\ rN_{\gamma r}) + 2\pi r\ D_f s$$

where all of the terms are defined as previously listed except that N_{cr}, N_{qr} and $N_{\gamma r}$ are general bearing capacity factors which depend on the depth and shape of the foundation as well as the angle of internal friction and the roughness of the base.

Approximate values of these bearing capacity factors have been derived and plotted by Meyerhof. By using these values corresponding to β equal to 90° (see Figure 6) and D/B equal to 32 and by inserting the data from the triaxial shear tests, the Meyerhof prediction of ultimate capacity is 50.9 tons using effective strength parameters and 48 tons using total strength parameters.

Moore Method

The method of predetermination derived from the practical experience of William Moore (16) expresses ultimate pile capacity as the sum of end bearing and frictional resistances, similar to the two theoretical approaches. However, the Moore equation limits end bearing to $\pi s\,(\pi r_p^2)$ and restricts frictional resistance to $(2\pi\, rD_f)\, P_l \tan\alpha$, or $(2\pi r\, D_f)\, s$, whichever is less; where

s = the shearing strength of the soil, lb/ft^2

r_p = the radius of the pile tip, feet

r = the average radius of the pile, feet

D_f = the pile length, feet

P_l = the lateral pressure on the pile, lb/ft^2

α = the friction angle between the pile and soil.

Moore suggests that the friction developed by lateral pressures, $P_l \tan\alpha$, should be multiplied by the term $\frac{\pi s + P}{P}$, where P is the surcharge pressure, to take into consideration the increase in lateral pressure due to displacement of soil during driving. For most silts, the shearing strength (s) will be less than this modified friction thus reducing the Moore equation to

$$Q_u = \pi s\,(\pi r_p^2) + (2\pi r\, D_f)\, s.$$

The resulting predicted capacity is 43.8 tons assuming complete drainage and 40.9 tons assuming no drainage.

LOAD TESTING

Test Equipment

The main test assembly chosen for the load test consisted of a single test pile flanked on either side by anchor piles. To minimize interference during loading, the anchor piles were spaced a distance of five feet center to center from the test pile (25). Spanning between the two anchor piles and directly over the test pile, a steel beam (20 I 65.4) was erected. It was secured to each anchor pile by a four inch by one half inch thick, U-shaped steel strap and four one and one eighth inch bolts passing through the heads of the anchor piles (see Figure 7). A 12 inch square by one inch thick bearing plate was provided to assure a uniform pressure distribution over the head of the test pile.

The test loads were applied and maintained manually by means of a hand pump, a four and one half inch, Ashcroft #1056 pressure gauge, and a 60 ton, Simplex hydraulic jack. The pressure gauge was calibrated in pounds per square inch which, when multiplied by the area of the ram in the jack, gave the load on the pile in pounds.

The measurement of pile settlements was achieved by the use of a micrometer dial indicator with calibrations to one thousandth of an inch. The indicator was fastened to a steel hanger which, in turn, was clamped to a horizontal wood frame running perpendicular

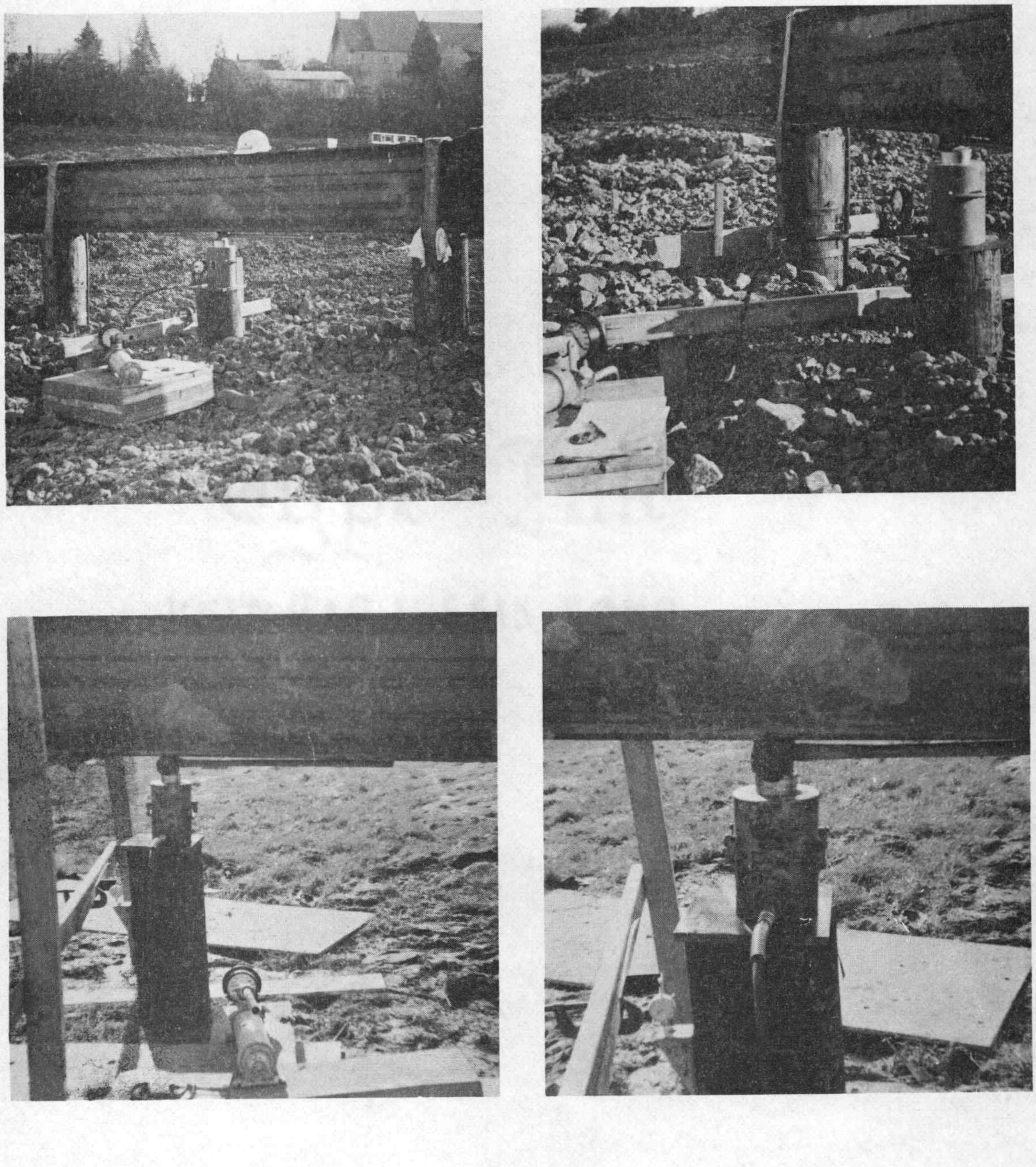


Figure 7. Test Assembly and Instruments.

to the line of the piles. The stakes supporting the wood frame were set approximately four feet outside of the test pile as a precaution against movement of the ground around the pile. To act as a zero reference and a smooth surface on which the micrometer point could be placed, a steel angle bench mark was lagged into the side of the test pile about 12 inches below the cut-off elevation.

Movement of the anchor piles was observed but not recorded. This was accomplished by driving a nail into each anchor pile just above a taut piano wire stretched parallel, and immediately adjacent, to the piles. Elongation of the anchor piles was recorded subjectively as the relative motion between the nails and the wire.

Due to an error in cutting off the test pile, a four inch, steel shim was required between the hydraulic jack and the resistance beam. As a means of expediency, this was effected by using the head of a sledge hammer. No errors or inaccuracies should have developed from this improvisation.

Driving the Test Pile

The selected test pile and two anchor piles were driven on February 20, 1969, using a Vulcan #1, single-acting, steam hammer rated at 15,000 foot-pounds of energy per blow. The test pile, originally 36 feet long, was driven to a depth of 26 feet and was cut off approximately three feet above the ground surface. The anchor piles, 34 and 36 feet long, were driven to 30 and 32 foot depths, respectively. The tops of the anchor piles were trimmed only enough to provide a level bearing for the resistance beam.

Prior to driving, all of the piles were marked at one foot intervals along their lengths to provide a reference for recording resistance during driving. The resulting driving records are presented in Figure 8.

An anchor pile was driven first with the test pile and second anchor pile following in that order. The influence of the disturbance created during driving is apparent in the reduced driving resistance of each succeeding pile. The increased resistance of the second anchor pile after it passes the tip of the test pile indicates that the remoulding effect caused by the two previous piles is absent. No heaving of the ground around the piles or rebound of previously driven piles was noted.

Test Procedure

The method of applying load to the test pile is fashioned after ASTM designation: D1143-61T. It involves the application of a series of equal load increments at one hour intervals up to a maximum of anticipated ultimate capacity. Settlement readings for each increment of load are taken at elapsed times of 2, 10, 20, 40 and 60 minutes. At projected ultimate load, if the pile has not failed as

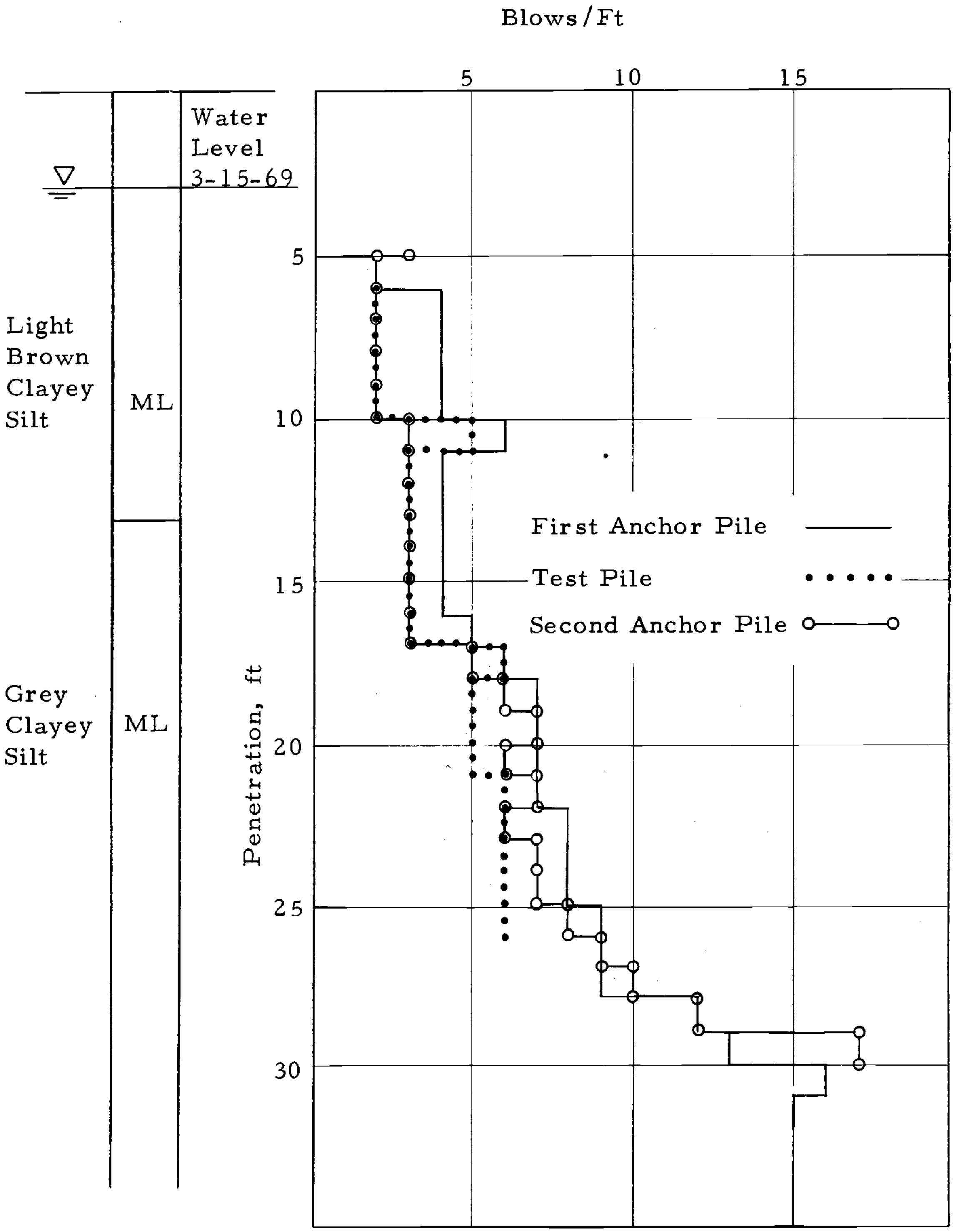


Figure 8. Pile Driving Log.

indicated by progressive settlement under a constant load, that load is maintained on the pile with readings made during each succeeding hour at 20, 40, and 60 minute intervals until settlement ceases. At this time the application of load again proceeds until failure is achieved.

Following failure, the pile is allowed to come to rest with the final, at-rest load recorded. From this point the load is decreased at one hour intervals to 50 percent, 25 percent and zero percent of the maximum sustained load. Rebound measurements are made at 1, 10, 20,40 and 60 minutes during each decrement of load.

Load Tests

The first load test was performed a week after the test pile had been driven, assuming that this would allow sufficient time for dissipation of pore pressures due to driving. Since the triaxial shear tests had not originally been contemplated, the projected ultimate capacity was calculated using the lower, direct shear strength data in the Terzaghi and Peck formula. After holding the calculated capacity for five hours with no more than two-thousandths of an inch per hour settlement, the pile was loaded to failure.

A week after the first test, a second set of loads was applied. The assumed capacity for this test was 90 percent of the failure load from test number one; which allowed not only a full scale of loads to be applied but permitted an investigation into the settlement caused by a sustained load over a longer period of time. This assumed capacity was maintained for 24 hours before additional load, up to the capacity of the jack, was imposed.

On the third week after driving, the test pile was subjected to a third cycle of loads using a 100 ton, Simplex Re-Mo-Trol hydraulic jack. Instead of following the previous procedure of loading to an anticipated capacity, this test was intended to proceed directly to failure at a loading rate of eight tons per hour. However, before this could be attained, the test assembly failed at an attempted load of 72 tons, when the strap bolts began moving through the head of the anchor piles.

Load-settlement curves for the three tests are plotted progressively in Figure 9, without the effects of sustained loadings. The ultimate capacities indicated on the first two curves are the elastic limits as described by Housel (8). These limits are designated by the points on the load-settlement curves which correspond to a settlement equal to the total amount of rebound.

In Figure 7, it can be seen that the test pile is out of vertical alignment. To compensate for this condition the hydraulic jack was placed off center on the pile head during the tests. The effect of this adjustment was investigated in the fourth load test where load was applied with the jack centered on the pile. Data for this last test is not

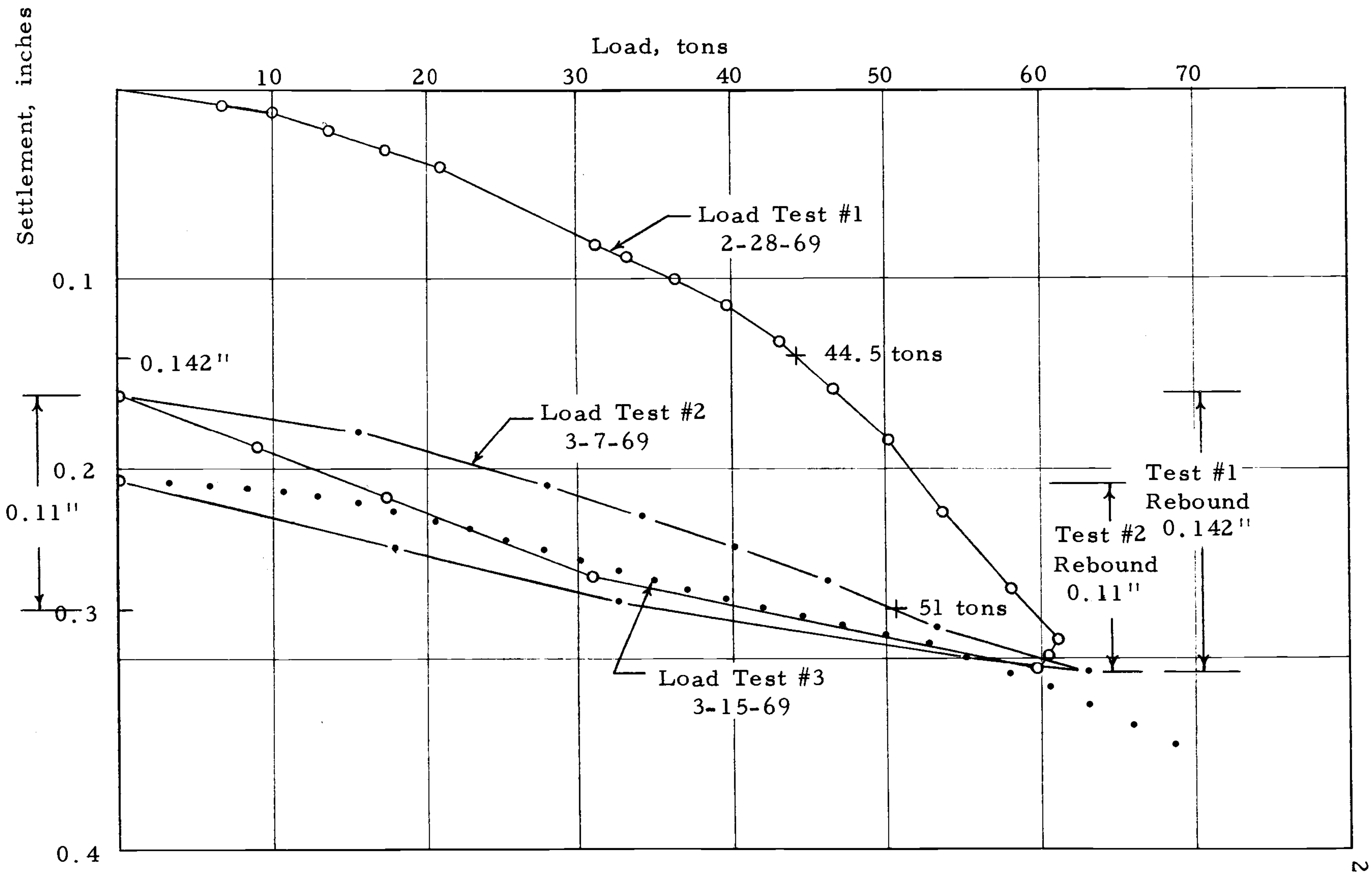


Figure 9. Load-Settlement Curves for Test Pile.

presented because it does not contribute to the study at hand. It is sufficient to state that at a load of 60 tons the pile head was horizontally displaced one-half of an inch. This movement was not noticeable in any of the tests where the jack was eccentric on the pile.

Also notable in portions of Figure 7 is the presence of approximately 12 inches of rock fill in the test area. This fill was placed three days prior to the third test as part of the construction operation. Since sufficient time had passed to allow dissipation of created pore pressures and since the resulting surcharge pressure is relatively minor, it can be assumed that the influence of the fill is insignificant.

DISCUSSION

A review of the driving records reveals that a fairly constant resistance to penetration occurs between the elevation of the hand auger boring (approximately 16 feet of penetration) and the tip of the test pile. This uniformity serves to verify the assumption in the pile capacity determinations that the soil below the boring is similar to that last encountered.

The driving logs for the deeper anchor piles also indicates a substantial increase in driving resistance beginning about two feet below the test pile. This firmer material ratifies the second assumption of the theoretical predictions that the soil at the pile tip is firm. The effect of the firm material in increasing the bearing resistance of the test pile should not significantly influence the results of this investigation.

The load tests were originally considered to reach failure when the test pile settled progressively under a constant load. However, since only the first test fulfilled this criterion, a method defining failure as the limit of recoverable settlement was employed, as illustrated in Figure 9.

Plotting load versus time for points of equal settlement, Figure 10, indicates a definite increase in the carrying capacity of the test pile with time. This increase in strength is mainly attributable

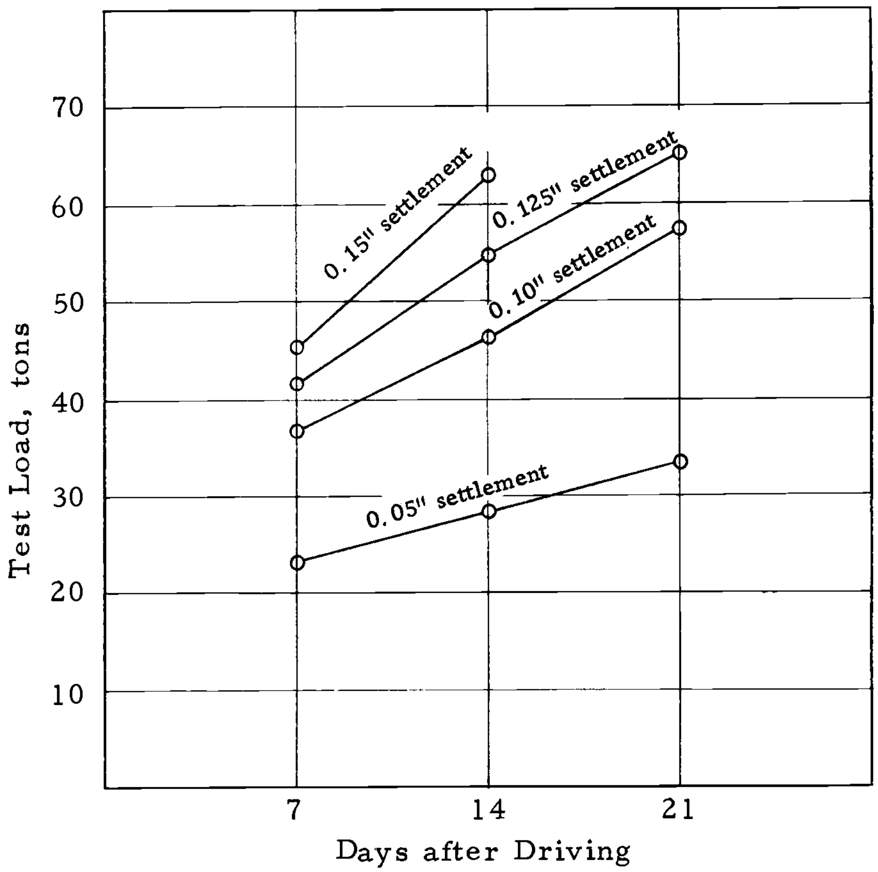


Figure 10. Progressive Test Loads at Equal Settlements.

to the gradual reconsolidation around the pile of the zone of soil that was disturbed during driving. The limits of this disturbed material can be expected to extend beyond the pile three or four times its diameter (11, 15), with the degree of disturbance and reconsolidation occurring inversely to the distance from the pile. Logically associated with this consolidation will occur a radial increase in density and lateral pressure effected by the assimilation of the displaced soil.

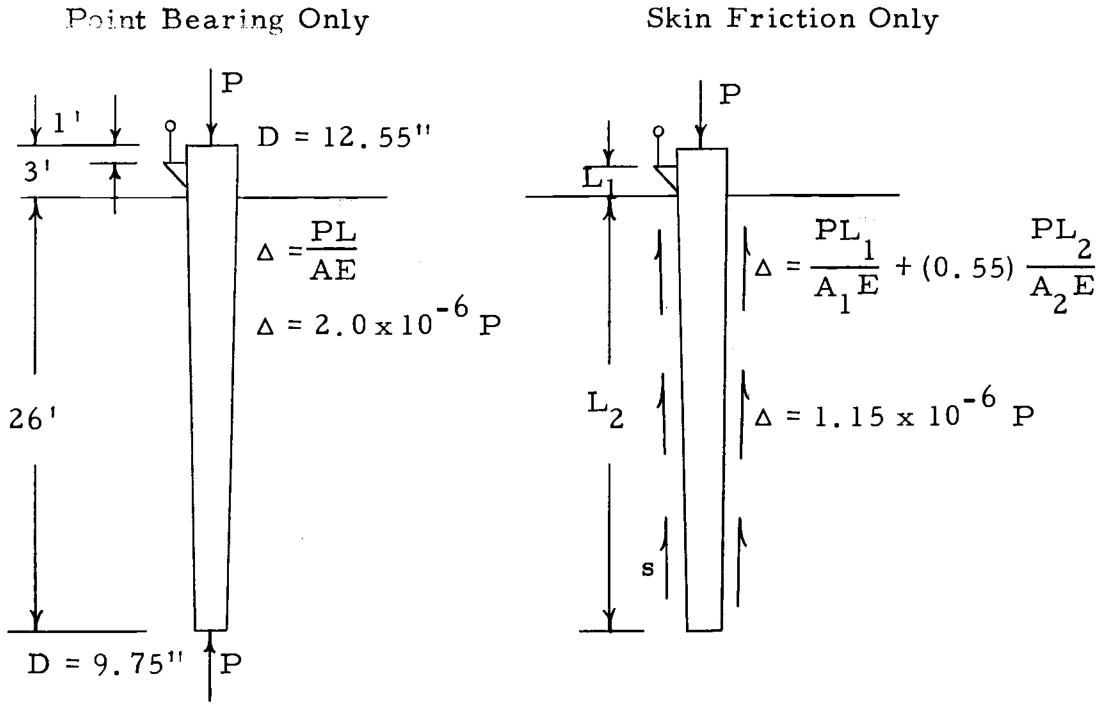
In Table 2, a comparison of the calculated capacities and the elastic limits from Figure 9 reveals that neither the drained nor undrained calculated capacities vary more than about 14% from the actual pile resistance in the first test. The Moore formula underestimates the ultimate capacity in both cases and is comparatively the most conservative due to the restriction on point bearing as an unvarying function of shear strength only.

The Terzaghi and Peck prediction for the undrained state closely approximates the first test value and within the accuracy of the test and theory exactly describes this failure condition as one of fully developed bearing resistance and skin friction under a lateral pressure equal to the overburden pressure. Assuming that this description is valid and evaluating the elastic shortening of the pile, described in Figure 11, leads to an anticipated strain of 0.11 inches as compared to 0.14 inches from the load test.

Using the higher tip resistance from the Meyerhof formula and

Table 2. Comparison of calculated and test capacities

	CALCULATED CAPACITIES AND PILE COMPRESSIONS						ELASTIC LIMITS AND PILE COMPRESSION FROM LOAD TESTS		
	UNDRAINED			DRAINED					
	Terzaghi & Peck	Meyerhof	Moore	Terzaghi & Peck	Meyerhof	Moore	Test #1	Test #2	Test #3
Point Bearing	5.1 tons	8.0 tons	0.9 tons	5.1 tons	8.0 tons	0.9 tons	44.5 ton	51 ton	maximum test load 69 ton
Skin Friction	40 tons	40 tons	40 tons	42.9 tons	42.9 tons	42.9 tons			
Elastic Compression	0.11"	0.12"	0.09"	0.12"	0.13"	0.10"	0.14"	0.11"	0.15"
$Q_u/Q_{test\ \#1}$	1.01	1.08	0.91	1.08	1.14	0.98	1.0	1.15	1.55
$Q_u/Q_{test\ \#2}$	0.88	0.94	0.80	0.94	1.0	0.86	0.87	1.0	1.35
Q_u/Q_{max}	0.65	0.69	0.59	0.69	0.74	0.63	0.64	0.74	1.0



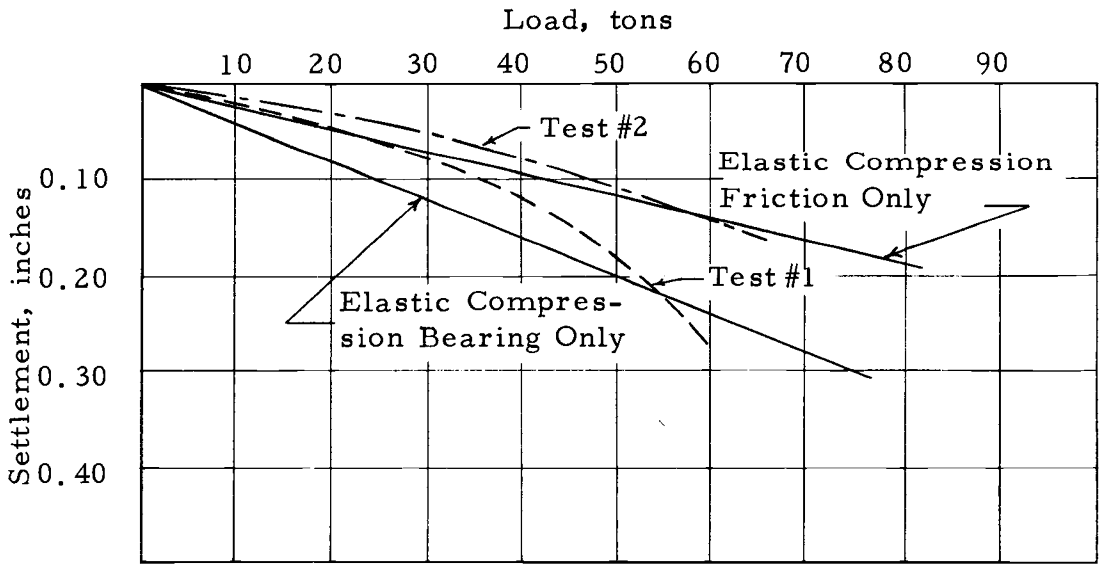


Figure 11. Assumed Elastic Compression of the Test Pile.

adjusting the skin friction to simulate the ultimate capacity in the first test results in a calculated, elastic compression of the pile equal to 0.116 inches. This too is insufficient to explain the test results and indicates that more of the ultimate resistance is being generated at the pile tip and less along its surface than either of the theories predicts. In order to effect this condition the bearing capacity factors would have to be increased and the coefficient of lateral pressure reduced. Determined from the strain at the ultimate load, assuming that the expressions in Figure 11 are accurate, the point bearing would be 23.3 tons, the frictional resistance would be 21.2 tons. Since it is very unlikely that the tip resistance could develop to this magnitude and since only two-thirds of the total cohesion would be developed it must be concluded that, during the first load test, the skin friction is not uniformly distributed along the pile shaft but rather varies from a maximum resistance near the tip to some lower value near the ground surface.

Upon examining the results of the second load test, it is evident that the best correlation between actual and calculated capacities occurs in the drained condition. As shown in Table 2, the Terzaghi and Peck and the Moore formulas underestimate the capacity, whereas the Meyerhof equation exactly simulates the ultimate test load.

The estimated shortening of the pile under the projected Meyerhof load also corresponds to the test results. This would tend to suggest that the frictional resistance along the pile had developed to a more uniform distribution during the week that had elapsed since the first test. The fact that the recorded ultimate compression is less than the approximated value may indicate a distortion of this distribution of skin friction, possibly parabolic as reported by Vesić (28) for piles in sand. However, within the scope of this study an attempt to refine the correspondence between the measured and calculated ultimate conditions in the second test would not be significant.

The third load test tends to indicate that the eventual capacity of the test pile will exceed all of the explored predictions. Realizing a limitation on the magnitude of the tip resistance, it is reasonable to assume that any additional load beyond the magnitude of that in the second test will be carried by frictional resistance. To develop this additional surface force it is necessary that the lateral pressure around the pile increase. Using the maximum sustained load of 69 tons and assuming Meyerhof's description of tip resistance and a uniform distribution of skin friction, the average coefficient of lateral pressure at the end of the third test would be 2.18. This value falls within the range of results of an investigation by Ireland (9) concerning pulling tests on piles in sand.

The calculated elastic compression corresponding to these assumed components of the maximum load is 0.17". This again is higher than the observed test compression and acts to substantiate the previously cited parabolic distribution of friction reported by Vesic' for short piles in sand.

Using the elastic compression as a criterion, it is apparent that the nature of the pile's supporting capacity varies with time. During the first test reconsolidation around the pile is incomplete, while the relatively undisturbed zone beneath and adjacent to the tip is intact. This results in the development of full bearing resistance and higher frictional forces near the pile tip; thus effecting the increased settlement.

In the second test sufficient time has passed to allow dissipation of enough of the excess pore pressure to approximate the original soil condition with a lateral pressure coefficient of one. The curtailment of settlement is explained by the fact that, since the surface cohesion is mobilized first (3, 8, 26) and since a definite amount of settlement must occur before the point resistance is fully activated (3, 23), the increased, uniformly distributed surface friction retards transmission of load to the tip.

The third test indicates a tendency toward further consolidation around the pile with a resulting increase in lateral effective pressure. This consolidation appears to be approaching a condition of passive earth pressures, although data is insufficient to derive any specific relationship or final values.

RECOMMENDATIONS AND CONCLUSIONS

Recommendations

In general, the method of testing and the testing devices performed reasonably well. However, the presence of certain, inherent deficiencies and inaccuracies limited the scope of testing and the versatility of the test data. These inadequacies may be avoided by the following provisions:

1. The test assembly and loading equipment should be designated to carry twice the calculated ultimate capacity.
2. Certified pressure gauges should be used on both the hydraulic pump and jack so that two independent load readings can be made.
3. Dial indicators should be employed to measure both the vertical and horizontal displacement of the test pile and should be provided at each anchor pile as an auxiliary check for load and relative, vertical motion.

Conclusions

Noting the various limiting conditions of the test apparatus and the methods of interpreting the test results, the following general conclusions are applicable to the clayey silts encountered in this investigation:

1. Within practical limits the capacity predictions give only slightly conservative estimates of pile resistance. However, the assumed lateral pressure coefficient heavily influences these calculated capacities.
2. The nature of the skin friction on piles in clayey silts varies with time but can be computed using effective stresses for long term loadings.
3. The lateral pressure on the surface of a driven pile in clayey silts can be expected to equal or exceed the overburden pressure.

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