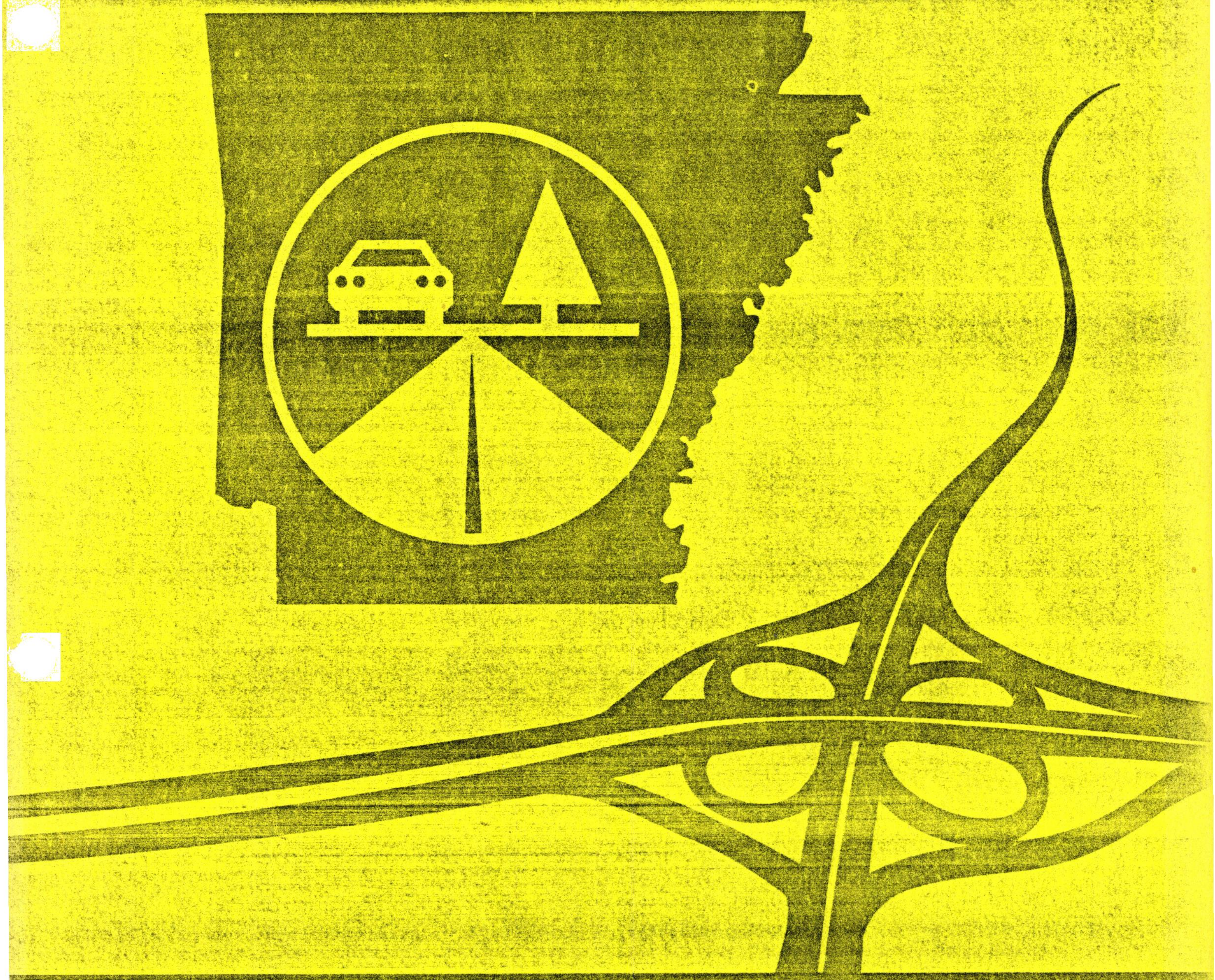

An Evaluation of Rapid Pile Load Tests

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1978

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.

4. Title and Subtitle

Evaluation of Rapid Pile Load Tests

5. Report Date

June 1978

6. Performing Organization Code

7. Author(s)

Robert C. Welch

8. Performing Organization Report No.

9. Performing Organization Name and Address

Department of Civil Engineering
University of Arkansas
Fayetteville, Arkansas 72701

10. Work Unit No.

11. Contract or Grant No.

HRC-36

12. Sponsoring Agency Name and Address

Arkansas State Highway and Transportation Department
P.O. Box 2261
Little Rock, Arkansas 72203

13. Type of Report and Period Covered

Final Report

14. Sponsoring Agency Code

15. Supplementary Notes

This study was conducted in cooperation with the Arkansas State Highway and Transportation Department and the U.S. Department of Transportation, Federal Highway Administration.

16. Abstract

Eight piles were tested using the maintained load (ML) test, Texas Quick (TQ) test and the constant rate of penetration (CRP) test. The results were compared on the basis of maximum load, load-settlement relationships, and load transfer behavior. Three piles were driven in sand, three piles were driven in clay, and two piles were driven in stratified deposits. One of the piles in sand and one of the piles in clay were instrumented steel pipe piles.

No significant difference in maximum load was observed for the three types of load tests. The load-settlement curves were essentially the same up to about 60 percent of the maximum load. Above 60 percent of the maximum load, greater deformations at the same load were observed for the TQ test than for the CRP test. Even greater deformations were observed for the ML test. The center of load in the pile was at the shallowest depth for the CRP test and was observed to be successively lower for the TQ test and the ML test.

17. Key Words

Piles, pile load tests

18. Distribution Statement

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price

Form DOT F 1700.7 (8-69)

FINDINGS AND CONCLUSIONS

The findings and conclusions resulting from this research are:

1. There is no significant difference in failure load produced by the maintained load test, the Texas quick test, and the constant rate of penetration test.
2. The load-settlement relationship is essentially the same up to about 60% of the failure load for all three test procedures used. This covers the normal working load range.
3. The load transfer behavior is essentially the same up to about 60% of the failure load for all three test procedures used.
4. The Texas quick test and the constant rate of penetration test will yield results that are essentially equivalent to the maintained load test.
5. The Engineering News formula currently used in the Standard Specifications did not accurately predict the capacity of the test piles on this project.

IMPLEMENTATION

The rapid pile load tests used in this research (Texas quick test and constant rate of penetration test) have yielded results essentially equivalent to those obtained from the maintained load test. Rapid tests require less time to perform and cause less construction delay than maintained load tests and are therefore more economical and convenient to perform. Of the two rapid load tests used in this research, the Texas quick test is preferred because less expensive equipment is required (the same equipment used for the maintained load test may be used), and the data observation is slightly easier than for the constant rate of penetration test. It is recommended that the Arkansas Highway Department adopt a rapid pile load test procedure, specifically the Texas quick test. An Implementation Package for the Texas Quick Test (IP 77-8) is available from FHWA. A sample specification, adapted from Arkansas Standard Specifications, Texas specifications, and the ASTM procedure, is given in the Appendix.

The inherent deficiencies of the Engineering News pile formula are well known and have been demonstrated by this project. The Hiley formula and the wave equation produced the most consistently reliable results on this project. It is recommended that the Arkansas Highway Department adopt the wave equation and/or a comprehensive pile-driving formula such as the Hiley formula. Two Implementation Packages for the Wave Equation (IP 76-13 and IP 76-14) are available from FHWA.

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CHAPTER I

INTRODUCTION

One of the first problems facing a bridge foundation designer is the choice of foundation type. Piles are frequently chosen because of their ability to transmit loads to a deep, relatively incompressible stratum of high strength. Other conditions such as inclined ground surface, lateral loading, scour, etc., also favor the use of pile foundations. If piles are the designer's choice, he must assess the ultimate capacity of the piles and, if possible, the load deformation behavior of the pile foundation. Several methods are available for predicting pile capacity and the most common are: predictions based upon measured or inferred soil properties, predictions based upon driving resistance when the pile is installed, and static load tests on typical piles. Static load tests are the most reliable measure of pile capacity and are often used to verify the capacity predicted by other methods.

The objective of this research is to compare several methods of performing static pile load tests on the basis of failure load, load-settlement behavior of the top of the pile, and load transfer in typical Arkansas soils. The tests selected for comparison in this research are the maintained load test as specified by the Arkansas Highway Department (AHD, 1972), the Texas Quick Test (Fuller and Hoy 1970), and the constant rate of penetration test (Whitaker and Cooke, 1961). If the results obtained by the quick test procedures are equivalent or comparable to the results of the maintained load test, then a quick test could replace the specified maintained load test. A quick test would be more economical as well as more convenient to perform, and would reduce delays in construction due to pile load tests.

Subsequent chapters will present brief discussions of the various predictive methods and a detailed comparison of the results of a series of pile load tests. Some of these tests were performed on piles instrumented to measure load transfer behavior.

A suggested quick test procedure and method of interpreting the results are included in the Appendix.

CHAPTER II

PREDICTIVE METHODS

Prediction of pile capacity is accomplished by (1) using measured or inferred soil properties and relationships based upon assumed failure modes, or by (2) using the dynamic driving resistance and equating the kinetic energy furnished by the hammer to the energy expended in advancing the pile and the energy losses in the hammer-pile-soil system. Each of these prediction methods will be discussed briefly in this chapter.

Predictions Based Upon Soil Properties

Predictive methods based upon soil properties usually fall into two categories: (1) limit equilibrium methods and (2) load-deformation methods. The methods most commonly used are the limit equilibrium methods.

Limit Equilibrium Analysis. In limit equilibrium analysis, a rigid-plastic deformation condition is assumed. The pile is considered incompressible and skin friction and end bearing reach their maximum values simultaneously. It is also assumed that loads transferred to the soil through friction or bearing do not influence the existing lateral or vertical earth pressures.

The ultimate capacity of a pile, Q_{ult}, can be determined by summing the total frictional resistance, Q_{SF}, and the maximum end bearing resistance, Q_{EB}.

$$Q_{ult} = Q_{SF} + Q_{EB} \tag{2.1}$$

The frictional resistance is the average friction or adhesion multiplied by the surface area of the pile.

$$Q_{SF} = f_{avg} PL \tag{2.2}$$

where:

f_{avg} = average unit skin friction or adhesion

P = perimeter of the pile

L = embedded length of the pile

The adhesion developed in clays is usually less than the shear strength or cohesion. Tomlinson (1969) has examined the relationship between skin friction in clays and the undisturbed shear strength. The ratio of skin friction to undisturbed shear strength is called the adhesion factor, α. A plot of α as a function of shear strength is shown in Figure 2.1. The skin friction of piles in clay can be determined by using Figure 2.1 and the following expression.

$$f = c\alpha \tag{2.3}$$

where:

c = undisturbed shear strength or cohesion

α = adhesion factor

The frictional resistance in sands is dependent upon the effective lateral earth pressure acting upon the pile surface and the coefficient of friction between the soil and the pile material. Above some critical depth, z_c, both vertical and horizontal effective stresses increase linearly with depth, but are essentially constant below the critical depth (Vesic, 1967). This critical depth is a function of relative density, D_r, and has been observed as follows:

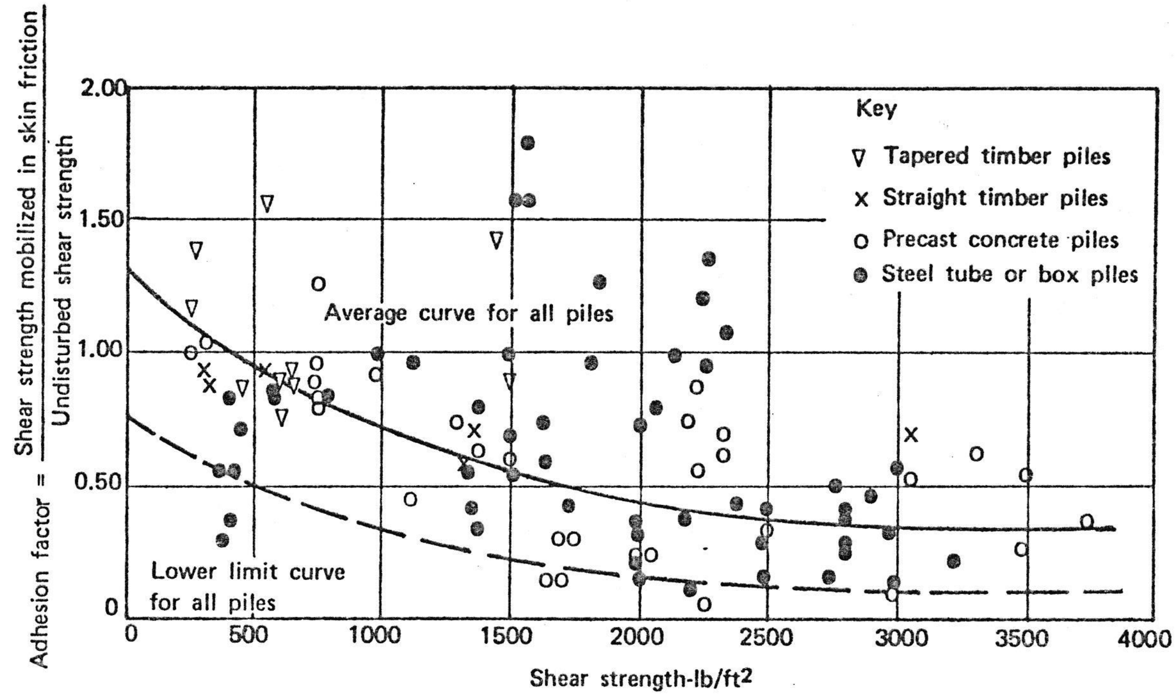


Figure 2.1 The Adhesion Factor as a Function of Shear Strength (after Tomlinson)

$$\text{For } D_r < 30\%,\ z_c = 10D \tag{2.4}$$

$$\text{For } D_r > 70\%,\ z_c = 20D \tag{2.5}$$

where:

z_c = critical depth

D = pile diameter or width

For values of D_r between 30% and 70%, linear interpolation may be used. The effective vertical stress in the vicinity of the pile can be determined as follows:

$$\text{For } z < z_c,\ \bar{p}_v = \bar{\gamma} z \tag{2.6}$$

$$\text{For } z \geq z_c,\ \bar{p}_v = \bar{\gamma} z_c \tag{2.7}$$

where:

$\bar{p}_v$ = effective vertical stress

$\bar{\gamma}$ = effective soil unit weight

z = depth below ground surface

The effective horizontal stress may be expressed as a function of the effective vertical stress.

$$\bar{p}_h = K_s\ \bar{p}_v \tag{2.8}$$

where:

$\bar{p}_h$ = effective horizontal stress

K_s = lateral pressure coefficient

The construction procedure has a significant influence on the lateral earth pressure and K_s. Values of K_s for various installation procedures (Sowers and Sowers, 1970) are given in Table 2.1.

The frictional resistance of soil against pile, best described as a skin friction angle, δ, depends upon soil type, pile material, and surface texture. Potyondi (1961) has examined the frictional resistance

TABLE 2.1

LATERAL EARTH PRESSURE

COEFFICIENT IN COHESIONLESS SOILS

Soil	Displacement Condition	K_s
Loose Sand	Jetted Pile	0.5 to 0.75
(D_r < 30%)	Drilled Pile	0.75 to 1.5
	Driven Pile	2 to 3
Dense Sand	Jetted Pile	0.5 to 1
(D_r > 70%)	Drilled Pile	1 to 2
	Driven Pile	3 to 5

of several pile-soil combinations and his values of δ are given in Table 2.2. The skin friction of piles in sand can be determined as follows:

$$f = \bar{p}_h \tan \delta \tag{2.9}$$

or

$$f = K_s \bar{p}_v \tan \delta \tag{2.10}$$

For depths less than the critical depth,

$$f = K_s \bar{\gamma} z \tan \delta \tag{2.11}$$

and for depths equal to or greater than critical

$$f = K_s \bar{\gamma} z_c \tan \delta \tag{2.12}$$

The end bearing component of pile capacity, Q_{EB}, can be determined by the general bearing capacity equation, using factors appropriate for deep foundations.

$$Q_{EB} = q_{ult} A_t = (cN_{cp} + \bar{p}_v N_{qp} + \tfrac{1}{2}\gamma D N_{\gamma p}) A_t \tag{2.13}$$

where:

q_{ult} = ultimate tip bearing capacity

TABLE 2.2

Proposed coefficients of skin friction between soils and construction materials

[$f\phi = \delta/\phi$, $fc = \frac{c_a}{c}$, $fc_{max} = \frac{c_a max}{c_{max}}$; without factor of safety]

Construction material			Sand		Cohesionless silt			Cohesive granular soil		Clay		
			$0.06 < D <$ 2·0 mm		$0.002 < D < 0.06$			50% Clay + 50% Sand		$D \leqq 0.06$ mm		
	Surface finish of construction material		Dry	Sat.	Dry	Sat.		Consist. I. = 1·0–0·5		Consist. Index: 1·0–0·73		
			Dense		Dense	Loose	Dense					
			$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	fc	$f\phi$	fc	fc_{max}
Steel	Smooth	Polished	0·54	0·64	0·79	0·40	0·68	0·40	—	0·50	0·25	0·50
	Rough	Rusted	0·76	0·80	0·95	0·48	0·75	0·65	0·35	0·50	0·50	0·80
Wood	Parallel to grain		0·76	0·85	0·92	0·55	0·87	0·80	0·20	0·60	0·4	0·85
	At right angles to grain		0·88	0·89	0·98	0·63	0·95	0·90	0·40	0·70	0·50	0·85
Concrete	Smooth	Made in iron form	0·76	0·80	0·92	0·50	0·87	0·84	0·42	0·68	0·40	1·00
	Grained	Made in wood form	0·88	0·88	0·98	0·62	0·96	0·90	0·58	0·80	0·50	1·00
	Rough	Made on adjusted ground	0·98	0·90	1·00	0·79	1·00	0·95	0·80	0·95	0·60	1·00

A_t = area of pile tip

c = cohesion in the vicinity of the tip

γ = effective unit soil weight in the vicinity of the tip

D = pile diameter or width

N_{cp}, N_{qp}, $N_{\gamma p}$ = deep foundation bearing capacity factors (See Figure 2.2)

Since D is usually small, the $N_{\gamma p}$ term is often neglected. For piles in cohesionless soils (c = 0), the end bearing may be determined by the following expression:

$$Q_{EB} = \overline{p}_v \, N_{qp} \, A_t \tag{2.14}$$

For cohesive soils (ϕ = 0, N_{qp} = 1), the end bearing becomes:

$$Q_{EB} = (c \, N_{cp} + \overline{p}_v) \, A_t \tag{2.15}$$

The concept of critical depth should be applied in determining $\overline{p}_v$ for cohesionless soils but should not be applied in the case of cohesive soils.

Soil properties required by the analysis described above may be measured by laboratory tests on undisturbed samples or may be inferred from the results of field tests such as the quasi-static cone penetration test, or the vane shear test.

Load Deformation Analysis. Analysis of the load-deformation behavior of piles may be accomplished by using a load transfer function approach or by using an axisymmetric finite element analysis. In certain cases, an elastic solid analysis based on the Mindlin equations could be used. Only the load transfer function method will be discussed in this report.

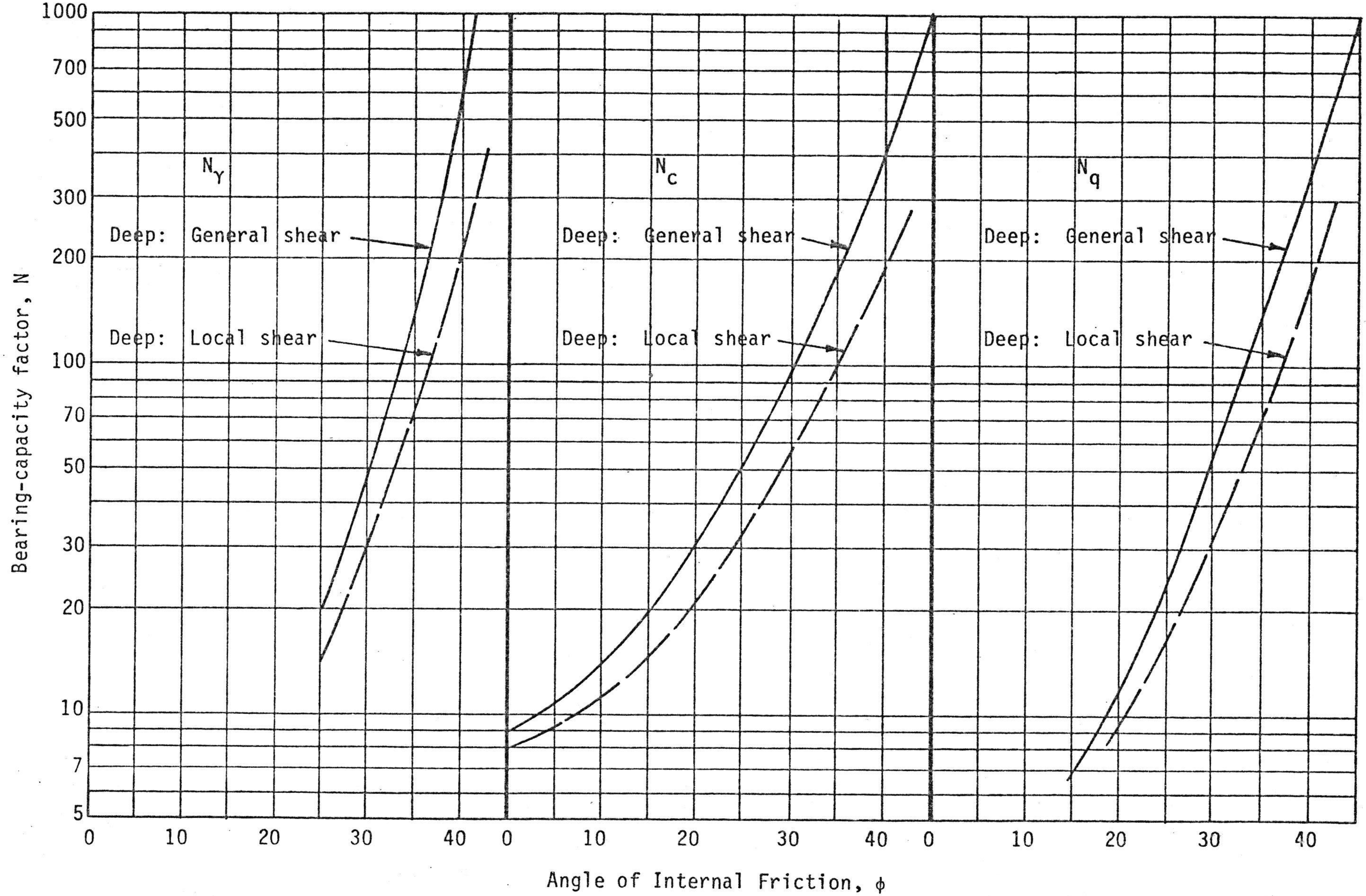


Figure 2.2 Bearing capacity factors for deep square or cylindrical foundations (Sowers and Sowers)

In the load transfer function analysis, the pile is treated as a deformable member, the stress-displacement relationships for skin friction and end bearing are considered and may exhibit non-linear behavior, and the peak values of skin friction and end bearing are not required to occur simultaneously. It is assumed that loads transferred to the soil do not affect existing lateral or vertical stresses.

This method of analysis requires that the pile be divided into segments and a load transfer curve showing developed skin friction vs. displacement be developed for each segment. (See Figure 2.3.) A tip load vs. tip displacement curve is also required. To compute the load-settlement curve for the top of the pile, the solution proceeds through the following steps (Coyle and Reese, 1966):

1. Assume a small tip movement.
2. Determine the tip load corresponding to the assumed tip movement.
3. Estimate the midpoint movement of the bottom segment.
4. From the appropriate load transfer curve, determine the load transferred to the soil through skin friction.
5. The load at the top of the bottom segment is equal to the tip load plus the skin friction load.
6. Use the average load in the pile segment and compute the elastic deformation at the midpoint of the segment.
7. Compute a value for movement of the midpoint of the segment by adding the elastic deformation at the midpoint to the movement of the bottom of the segment (the tip, in this case).
8. If the computed movement does not agree with the assumed movement within a specified tolerance, repeat steps 4 through

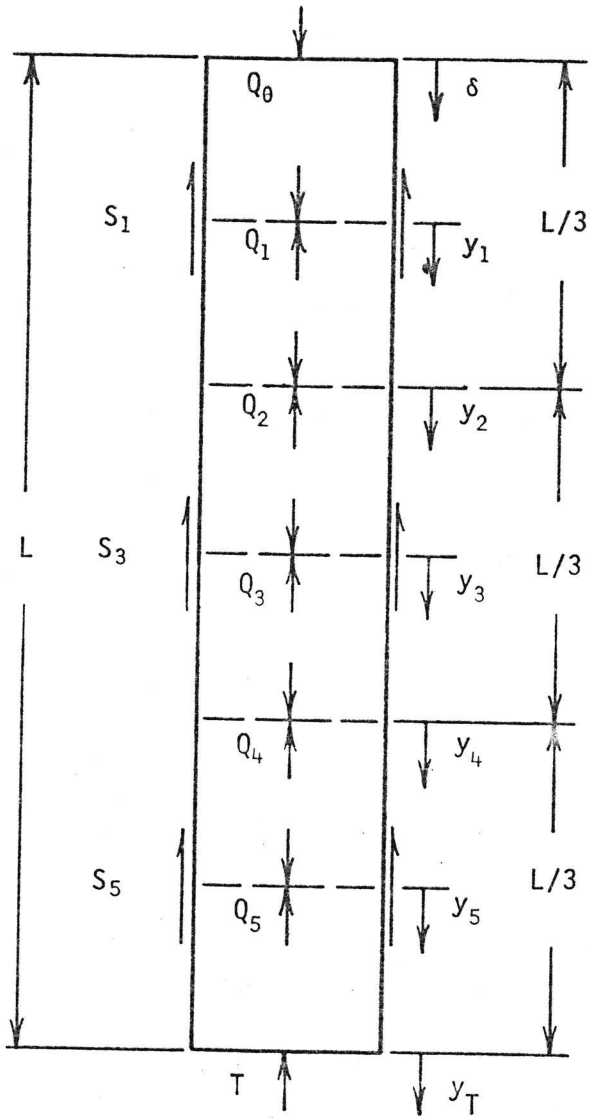


Figure 2.3a Axially Loaded Pile Divided into Three Segments

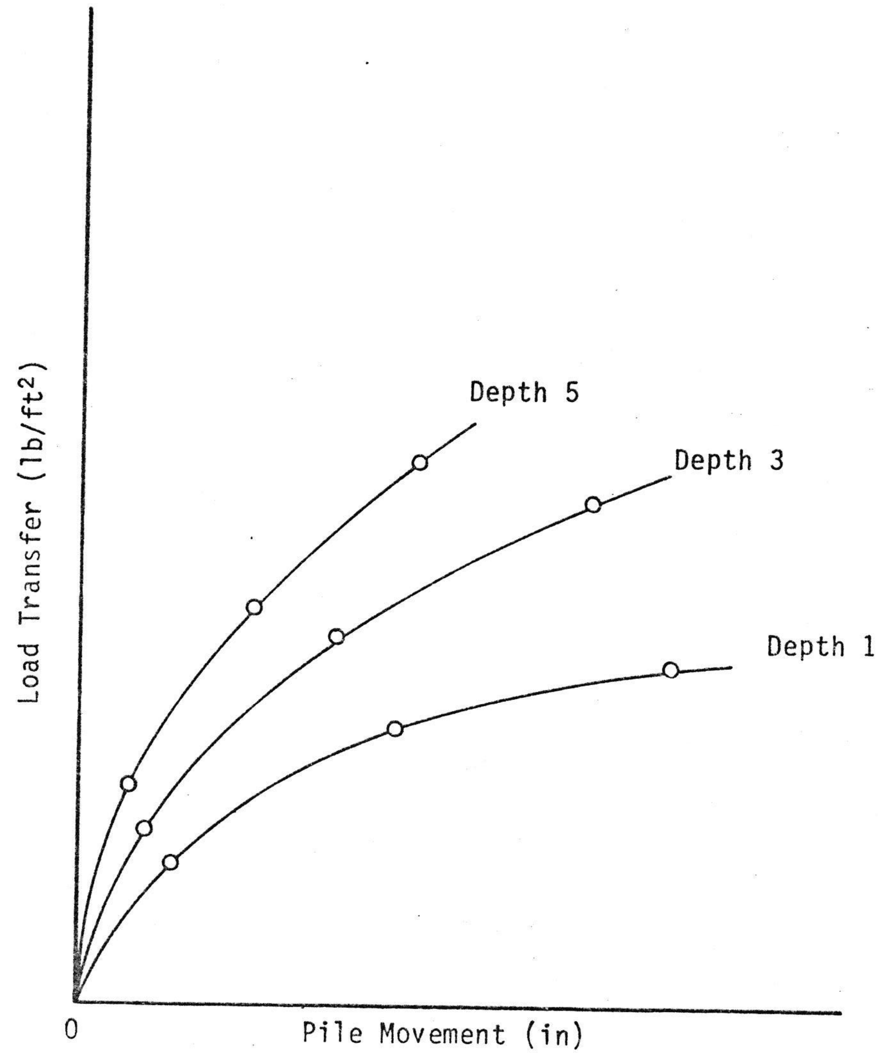


Figure 2.3b Typical Curve Showing Load Transfer versus Pile Movement

7 until convergence is achieved.

9. Go to the next segment above and repeat the process until the top load and displacement have been determined.

10. Repeat this procedure using different assumed tip movements until enough points have been determined to adequately define the load-settlement curve.

Load transfer curves for clay, described by Coyle and Reese (1966), are shown in Figure 2.4. The curves for sand shown in Figure 2.5 are suggested by Coyle and Sulaiman (1967). The soil shear strength used in Figure 2.5 is based upon the assumptions that the lateral pressure coefficient is constant with depth and is equal to one.

The tip load vs. tip movement curves for piles bearing in clay are based upon work done by Skempton (1951). The relationship can be estimated from the following equation.

$$\frac{\rho}{B} = \frac{4}{E/c} \cdot \frac{q}{q_{ult}} \qquad (2.16)$$

where:

ρ = tip settlement

B = tip width or diameter

E = secant modulus of the clay at a ratio of applied stress to ultimate stress of q/q_{ult}

c = cohesion

q = tip bearing pressure

q_{ult} = ultimate bearing capacity of the tip

This can be related to compression test results by the equation

$$\frac{\rho}{B} = 2\,\varepsilon \qquad (2.17)$$

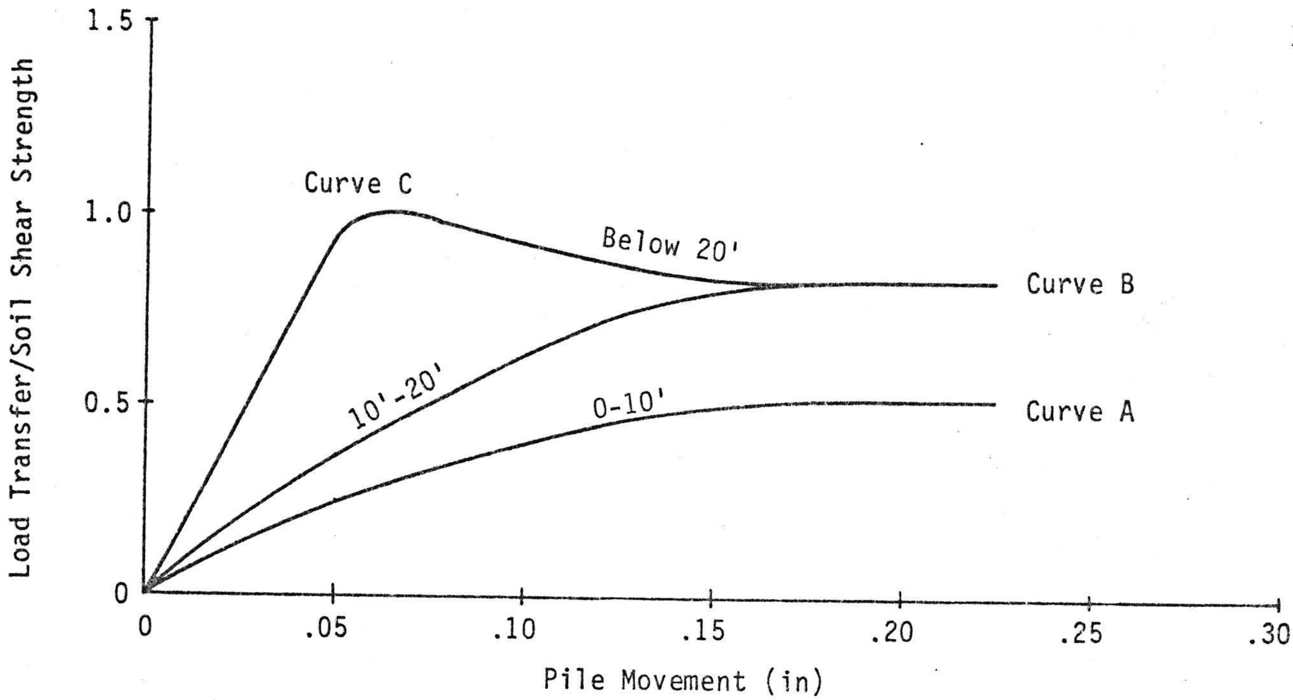


Figure 2.4 Load Transfer Curves for Clay

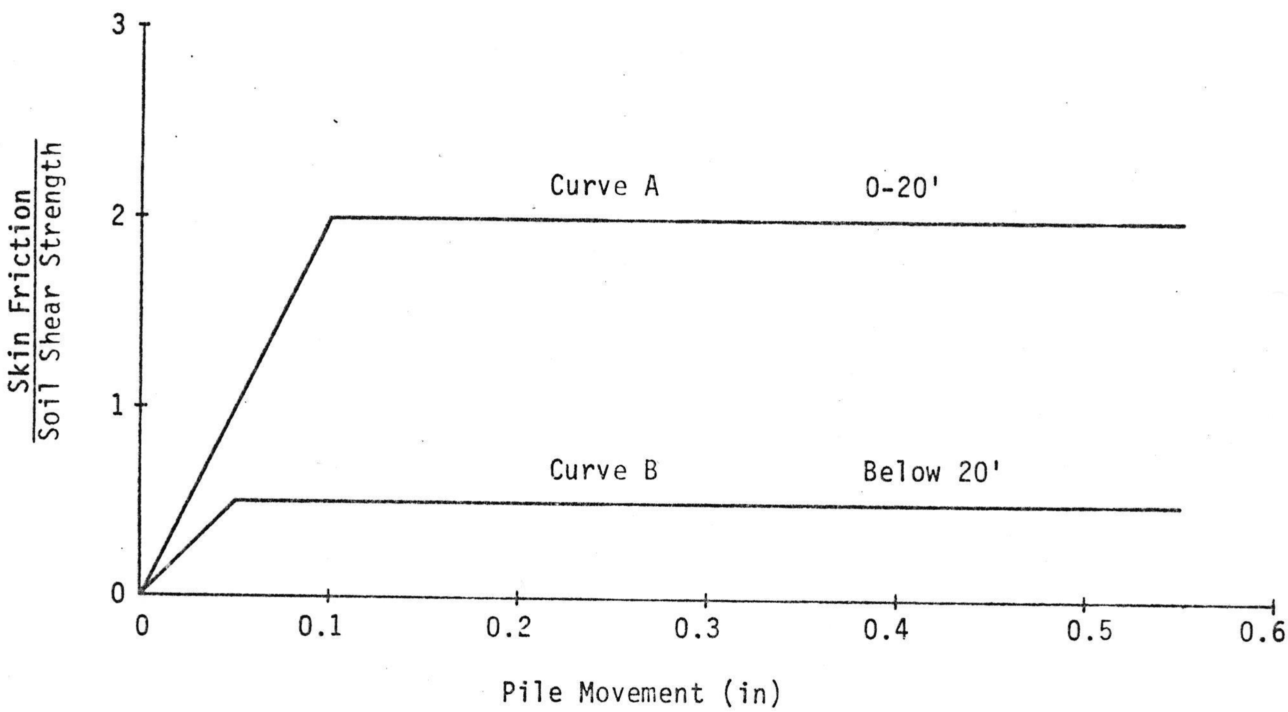


Figure 2.5 Load Transfer Curves for Sand

where:

ε = strain in compression test at a ratio of applied stress to ultimate stress of q/q_{ult}

The load-deformation behavior of piles bearing in sand is difficult to predict. Some typical values of ultimate tip resistance and tip resistance vs. tip displacement given by Reese (1978) for drilled shafts bearing in sand are given in Figures 2.6 and 2.7.

A computer program, PX4C3, developed at the University of Texas and based upon the load transfer function analysis described, was used to predict the load-deformation curves for some of the test piles in this research project.

Predictions Based Upon Driving Resistance

Predictive methods based upon driving resistance will usually fall into two categories: (1) methods based upon dynamic formulas equating the kinetic energy produced by the pile-driving hammer to the work done in advancing the pile plus the energy losses in the hammer-pile-soil system, and (2) methods based upon the one-dimensional wave equation describing the effects produced when a long slender rod is struck on its end.

Dynamic Formulas. The simplest dynamic formula is based upon the assumptions that the pile is perfectly rigid and that no energy is lost during driving.

$$Wh = R_u s \tag{2.18}$$

where:

W = weight of hammer

h = height of drop

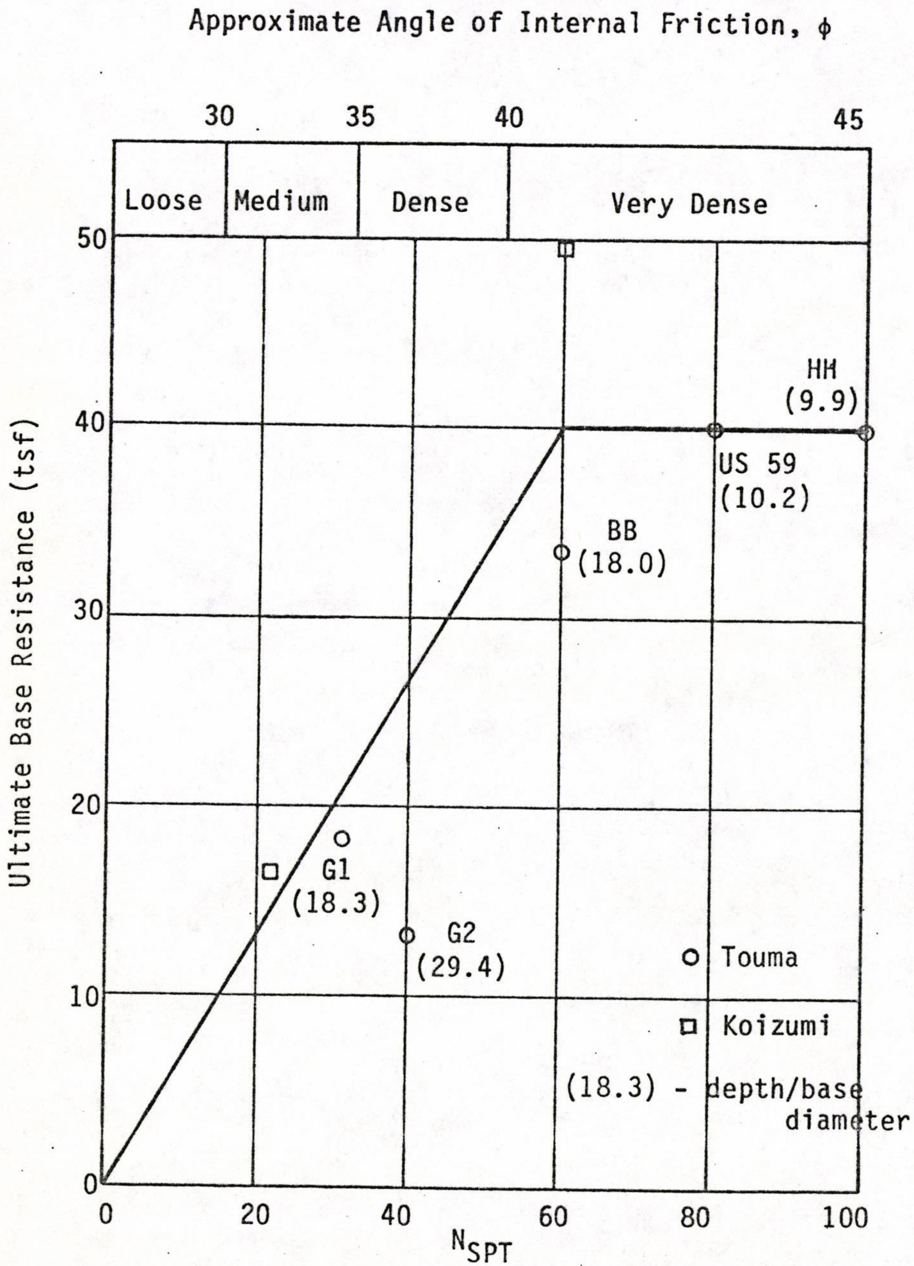


Figure 2.6 Ultimate Base Resistance in Sand versus N_{SPT}

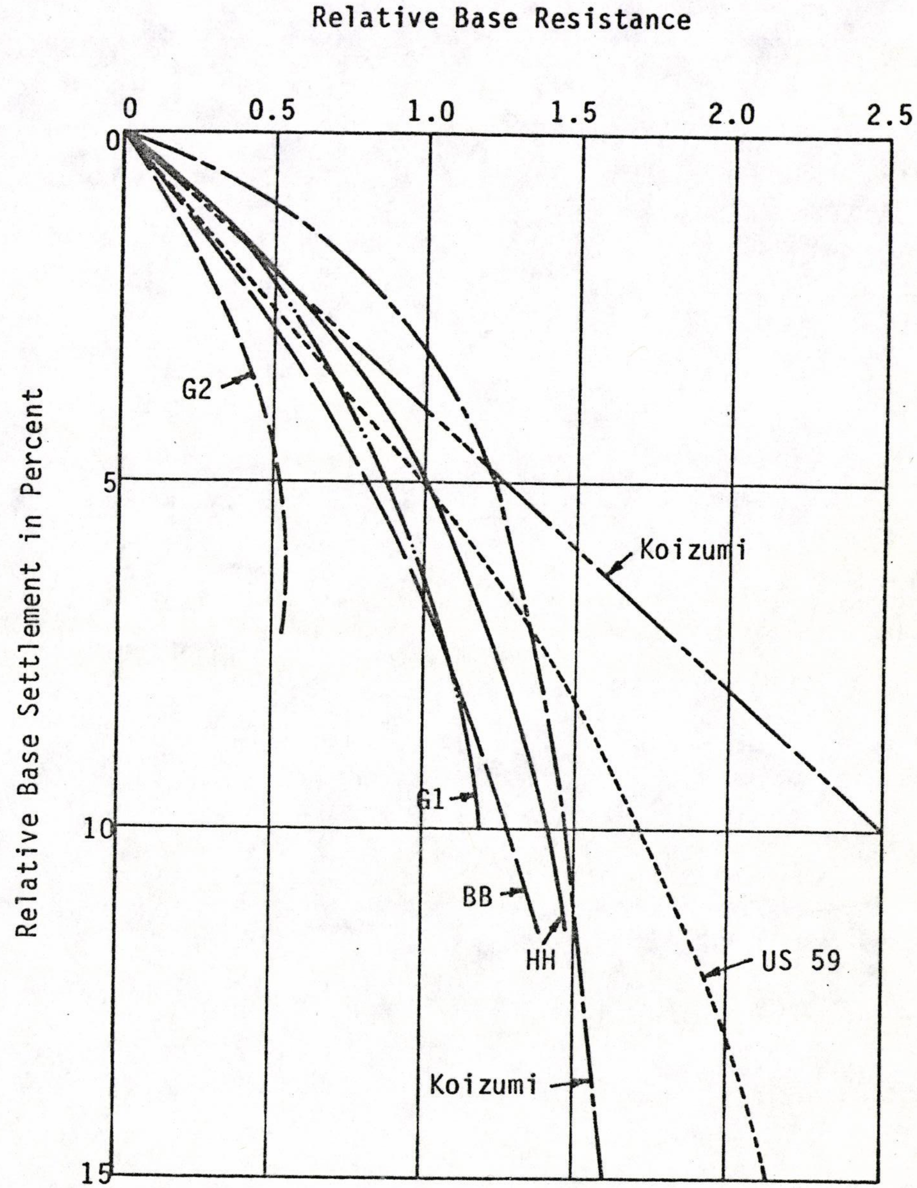


Figure 2.7 Relative Base Resistance in Sand versus Relative Base Settlement

R_u = ultimate pile capacity

s = set or penetration of the pile under the last blow

The weight, drop, and set can be measured and R_u can be determined. This equation (2.18) does not give reasonable values of R_u because there are significant energy losses in the hammer-pile-soil system. Energy is lost through friction in the hammer parts, impact, and elastic compression of the pile cap, pile, and soil. The primary difference between the various pile driving equations is the manner in which these losses are considered. For example, in the Engineering News formula,

$$R_u = \frac{Wh}{s + c} \qquad (2.19)$$

where:

c = elastic compression of hammer-pile-soil system

the energy loss, $R_u c$ is dependent only upon the type of hammer used to drive the pile. For all types of piles and soils, c is assumed to be 1.0 inch for drop hammers and 0.1 inch for single-acting steam hammers. The Hiley formula is based upon a more realistic appraisal of energy losses. This formula is considered a comprehensive formula and is expressed as

$$R_u = \frac{e\, W_r h}{s + 0.5(c_1+c_2+c_3)} \cdot \frac{W_r + n^2 W_p}{W_r + W_p} \qquad (2.20)$$

where:

e = efficiency of pile hammer (ratio of energy output to energy rating)

$W_r h$ = energy rating of hammer (W_r = wt. of hammer, h = ht. of drop

W_p = weight of pile

n = coefficient of restitution

c_1 = elastic compression of pile head and cap

c_2 = elastic compression of pile

c_3 = elastic compression of soil

The term $(W_r + n^2 W_p)/(W_r + W_p)$ is a treatment of energy loss during impact. The values of c_1, c_2, and c_3 may be estimated by using Tables 2.3, 2.4, and 2.5, or c_1 and c_2 may be computed by the following expression:

$$c = \frac{R_u \ell}{AE} \tag{2.21}$$

where:

c = elastic compression of cap (c_1) or pile (c_2)

ℓ = length or thickness of pile cap and packing for computing c_1 or effective length of pile for computing c_2

A = cross-sectional area

E = modulus of elasticity

A more reliable procedure for determining $c_2 + c_3$ is to attach a sheet of paper to the side of the pile and, as it is being driven, draw a pencil along a stationary horizontal support marking the paper. A sketch of the arrangement is shown in Figure 2.8a and a typical trace is shown in Figure 2.8b. From the trace, the set, s, and the elastic compression of pile and soil, $c_2 + c_3$, may be determined. If it is assumed that the energy loss is due only to compression of the pile, then, the Danish formula is obtained, with

$$R_u = \frac{W_r h}{s + 0.5 s_e} \tag{2.22}$$

Temporary Compression Allowance C_1 for Pile Head and Cap[a]

Material to which blow is applied	Easy driving, $p_1 = 500$ psi on cushion or pile butt if no cushion, in.	Medium driving, $p_1 = 1{,}000$ psi on head or cap, in.	Hard driving, $p_1 = 1{,}500$ psi on head or cap, in.	Very hard driving, $p_1 = 2{,}000$ psi on head or cap, in.
Head of timber pile....	0.05	0.10	0.15	0.20
3-4-in. packing inside cap on head of precast concrete pile....	0.05 + 0.07[b]	0.10 + 0.15[b]	0.15 + 0.22[b]	0.20 + 0.30[b]
½-1-in. mat pad only on head of precast concrete pile........	0.025	0.05	0.075	0.10
Steel-covered cap, containing wood packing, for steel piling or pipe...............	0.04	0.08	0.12	0.16
3/16-in. red electrical-fiber disk between two ⅜-in. steel plates, for use with severe driving on Monotube pile......	0.02	0.04	0.06	0.08
Head of steel piling or pipe...............	0	0	0	0

[a] Largely from A. Hiley, "Pile Driving Calculations with Notes on Driving Force and Ground Resistance," *The Structural Engineer*, vol. 8, July and August, 1930.[7] For a fuller discussion of the means of obtaining these values see this reference. For purpose of this article values represent average conditions and may be used.

[b] The first figure represents the compression of the cap and wood dolly or packing above the cap, whereas the second figure represents the compression of the wood packing between the cap and the pile head.

Note: Superior numbers (with or without letters) refer to the Bibliography, pp. 641*ff*., in which the material is organized by subject.

TABLE 2.3 (after Chellis)

TEMPORARY COMPRESSION VALUES OF C_2 FOR PILES

Type of pile	Easy driving, p_2 = 500 psi for wood or concrete piles, 7,500 psi for steel, net section, in.	Medium driving, p_2 = 1,000 psi for wood or concrete piles, 15,000 psi for steel, net section, in.	Hard driving, p_2 = 1,500 psi for wood or concrete piles, 22,500 psi for steel, net section, in.	Very hard driving, p_2 = 2,000 psi for wood or concrete piles, 30,000 psi for steel, net section, in.
Timber pile, based on value of E = 1,500,000. Proportion for other values of E given in Table VI[a]	$0.004 \times L^b$	$0.008 \times L^b$	$0.012 \times L^b$	$0.016 \times L^b$
Precast concrete pile (E = 3,000,000[a,c])	$0.002 \times L$	$0.004 \times L$	$0.006 \times L$	$0.008 \times L$
Steel sheet piling, Simplex tube, pipe pile, Monotube shell, Raymond steel mandrel[d] (E = 30,000,000)	$0.003 \times L$	$0.006 \times L$	$0.009 \times L$	$0.012 \times L$

[a] All other values in direct proportion to p_2 and inverse proportion to E.
[b] L should be considered as length to center of driving resistance, not necessarily full length of pile.
[c] May reach 6,000,000 for exceptionally good mix.
[d] When computing p_2 for a Raymond steel mandrel, it is suggested that the weight of the mandrel be divided by 3.4 × the effective length of pile in feet to obtain the average area.

TABLE 2.4 (after Chellis)

TEMPORARY COMPRESSION OR QUAKE OF GROUND ALLOWANCE C_3[a]

All values of p_3 to be taken on projected area of pile tips or driving points for end-bearing piles and piles of constant cross section; on gross area of pile at ground surface in case of tapered friction piles; and on bounding area under H piles

	Easy driving, p_3 = 500 psi, in.	Medium driving, p_3 = 1,000 psi, in.	Hard driving, p_3 = 1,500 psi, in.	Very hard driving, p_3 = 2,000 psi, in.
For piles of constant cross section[b,c]	0 to 0.10	0.10	0.10	0.10

[a] Largely from A. Hiley, "Pile Driving Calculations with Notes on Driving Force and Ground Resistance," *The Structural Engineer*, vol. 8, July and August, 1930.[7] For a fuller discussion of the means of obtaining these values see this reference. For purpose of this article values represent average conditions and may be used.
[b] It is recognized that these values should probably be increased in the case of piles with battered faces, but insufficient test data are available at present time to cover this condition.
[c] If the strata immediately underlying the pile tips are very soft, it is possible that these values might be increased to as much as double those shown.

TABLE 2.5 (after Chellis)

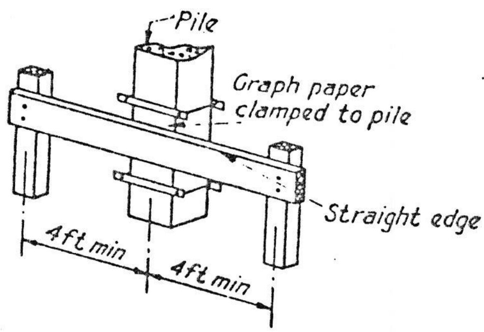


Figure 2.8a Apparatus for Taking Readings on Pile

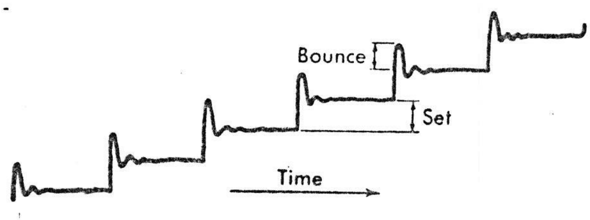


Figure 2.8b Diagram of Set and Temporary Compression

where:

s_e = elastic compression of the pile

and

$$s_e = \sqrt{\frac{2W_r\ h\ L}{AE}} \tag{2.23}$$

where:

L, A, and E = length, area, and modulus of elasticity of the pile

<u>Wave Equation Methods</u>. The wave equation describes the movement of stress waves in a long slender rod when it is struck on one end. This analysis was first applied to pile driving in the 1930's, but the tedious computations required inhibited its use. The development of high-speed digital computers and Smith's (1960) numerical solution of the wave equation have led to a fairly widespread use of this method of analysis. Two implementation packages presenting computer codes and documentation for application of the wave equation to pile driving are currently available (FHWA-IP-76-13, FHWA-IP-76-14). A different approach to the wave equation was taken by Goble and Rausche (1970). Transducers are attached to the pile near the top to measure the force and acceleration of the pile under a hammer blow. A small dedicated computer is used to determine the pile capacity from the transducer outputs.

In Smith's numerical solution of the wave equation, the hammer, pile and soil system are represented by a series of weights and springs (Figure 2.9). The cap block and anvil may also be depicted by weights and springs. The driving action is divided into small time elements of about .25 milliseconds and the pile is divided into segments of approxi-

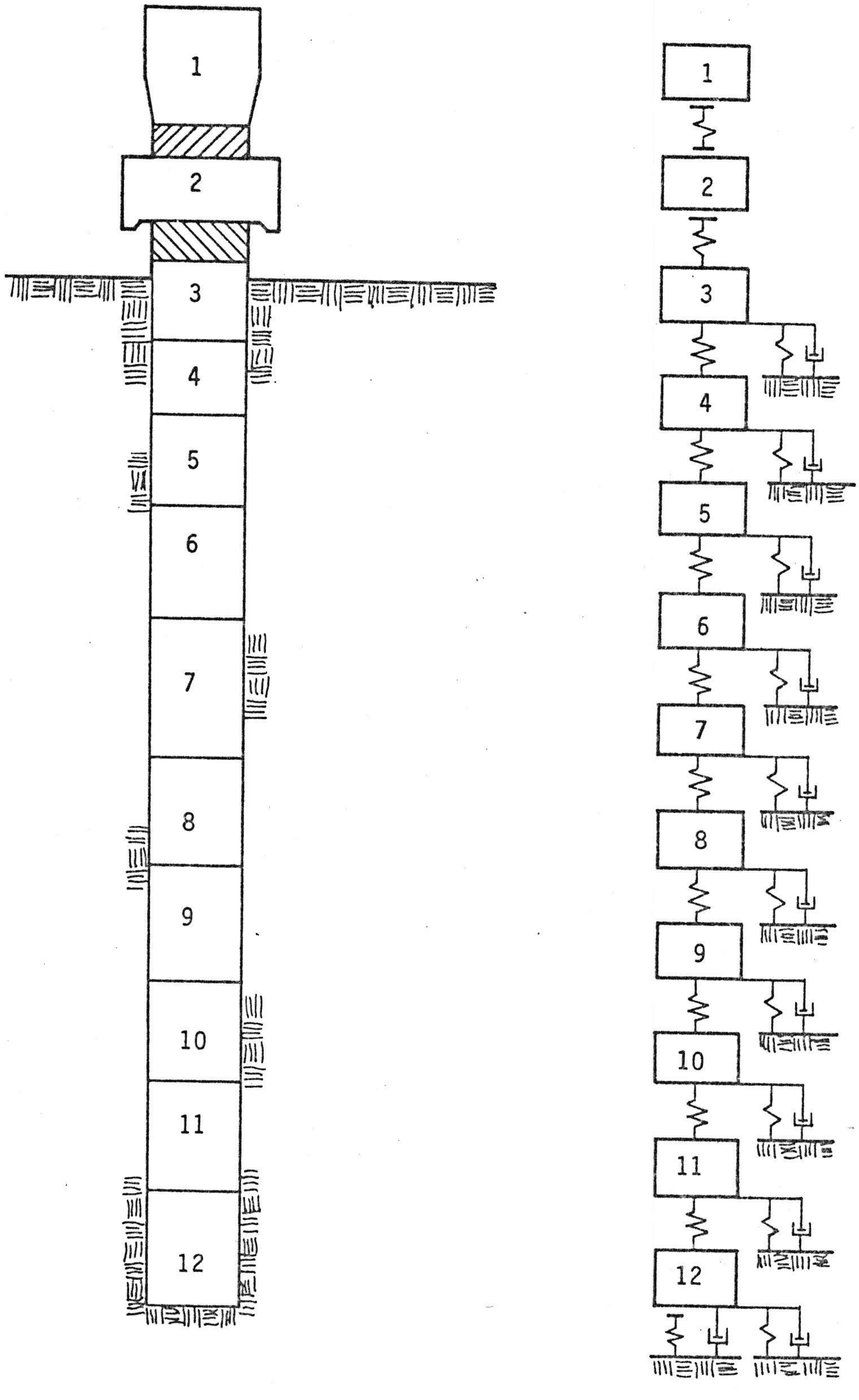


Figure 2.9 Idealization of a pile for purpose of analysis. Pile is divided into uniform concentrated weights and springs.

mately 5 to 10 feet. In this manner, a reasonably accurate determination of pile stresses and penetration may be made for any particular system. The spring constants, K, are found for elastic material such as the pile and cap from the formula:

$$K = \frac{AE}{L} \tag{2.24}$$

where:

A = cross sectional area

E = modulus of elasticity

L = segment length

Soil resistance is found for skin friction as well as point bearing. The soil is treated as an elastic-plastic material with stress-strain relationship as shown in Figure 2.10. The ultimate elastic movement of the soil is termed the quake (Q).

As the pile moves a distance A, it develops the ultimate resistance R_u. Further movement does not increase resistance and the point will continue to B on Figure 2.10a. Elastic unloading then occurs following line BC until all forces are zero. The permanent set of the pile is then the distance OC = AB.

Side resistance is calculated identically as point bearing except there are separate values of quake and ultimate resistance for each segment. The side friction may be distributed over the side of the pile by varying the stress-strain relationships of the individual segments.

These values of soil resistance have not included the time effects as yet. The ground will offer more resistance to rapid motion than to slow motion. To account for this, Smith (1960) represented "viscous damping." The evaluation of the wave equation gives a velocity, v_p.

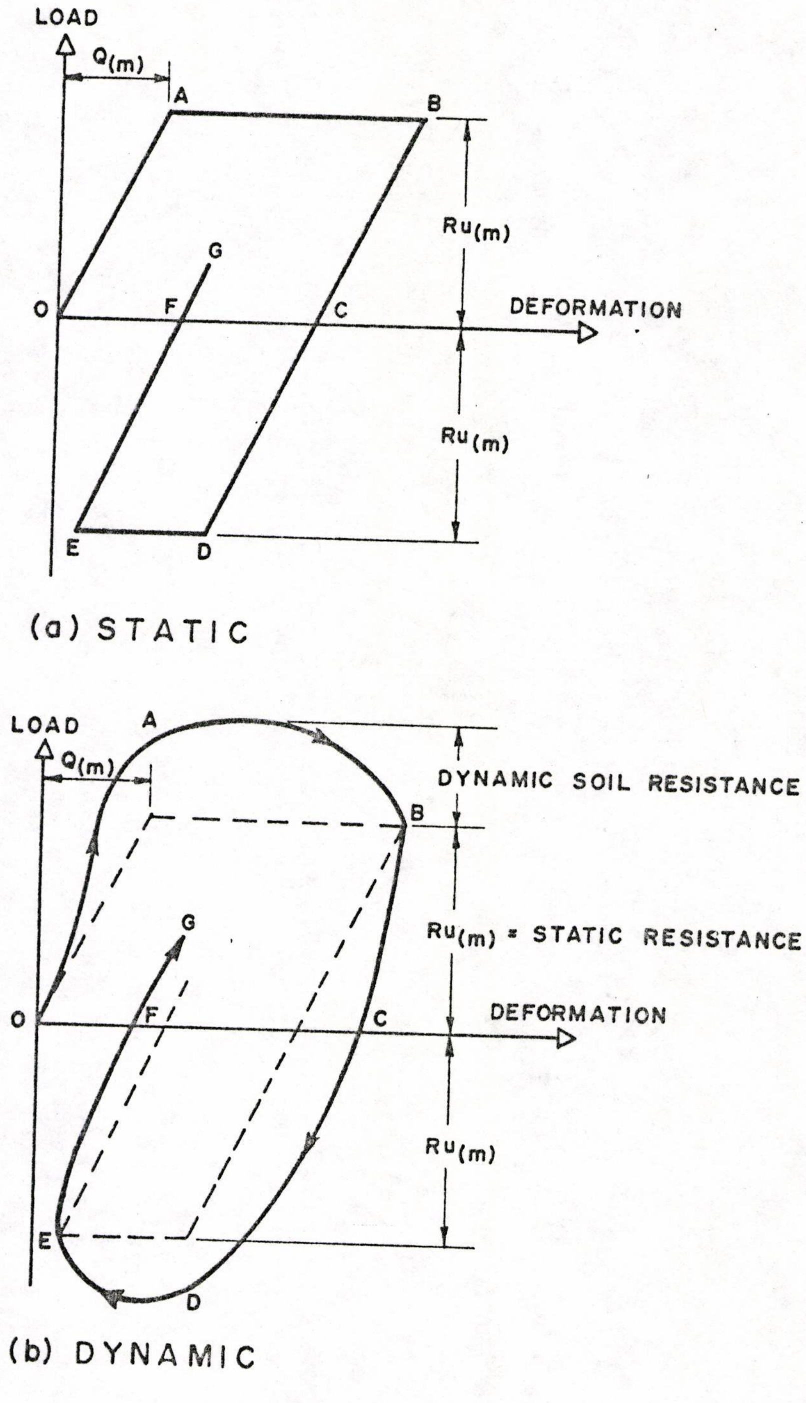


Figure 2.10 Soil load-deformation characteristics.

By applying a damping constant, J_p, to the velocity, the product $J_p\ v_p$ increases ground resistance to account for damping. At any point X on the curve of Figure 2.10b, the instantaneous damping resistance is $J_p\ v_p\ R_x$. The total resistance of the pile to penetration is the static resistance plus the damping resistance.

The Case Western Reserve device uses a simple force balance method to relate dynamic measurements to a static capacity. The pile is assumed to be a rigid body struck by a time-varying hammer force (Goble and Rausche, 1970). Motion of the pile is resisted by a force, R, given by the expression:

$$R(t) = R_0 + R_1\ V + R_2\ V^2 + R_3\ V^3 + \ldots \qquad (2.25)$$

where:

V = the velocity of the pile

R_0 = static capacity

R_1, R_2, R_3 = constants

Using Newton's Second Law at the instant of zero velocity, the resistance is found to be:

$$R_0 = F\ (t_0) - m\ \ a(t_0) \qquad (2.26)$$

where:

m = the mass of the pile

$a(t_0)$ = the acceleration at time t_0 when the velocity is zero

$F(t_0)$ = the force at the top of the pile at the same time

A force transducer and an accelerometer are attached to the pile near the top to monitor force and acceleration for each blow of the pile hammer. A small field computer unit receives, records, and analyzes the signals from the transducers and prints the computed pile capacity for each blow.

CHAPTER III

PILE LOAD TESTS

A pile load test measures the ultimate capacity of a single pile at the time of loading. Deformations observed during the test will give an indication of the behavior of the pile under short-term loading. No other method can provide this information with equal accuracy.

The capacity and behavior of pile groups cannot be determined from tests on single piles, nor can long-term deformations be determined from short-term tests. Another factor which must be considered is the possibility of downdrag or negative skin friction developing when a pile penetrates a compressible clay layer. It may also be possible for piles driven through very loose sands to lose some skin friction due to a stress relaxation in the sand. The possibility of negative skin friction or stress relaxation developing makes it desirable to separate the skin friction and end bearing components of pile capacity during pile load tests. At present, only two acceptable methods are available for this purpose. Load tests of piles which are instrumented to measure load distribution along the pile can separate skin friction and end bearing as can pulling tests performed after compressive loading tests. Load tests were performed on two instrumented piles as a part of this research.

There are many procedures for load-testing piles. The load test procedures used in this project include the maintained load test as specified by the Arkansas Highway Department, the Texas Quick Test, and the constant rate of penetration test. These procedures are described in the following paragraphs.

Maintained Load Test

Load tests using the maintained load (ML) test procedure may be either proof tests to verify pile capacity or failure tests to determine the ultimate capacity of the pile. Failure tests will allow the designer to work to a selected factor of safety and optimize his design. The actual factor of safety cannot be determined from proof tests and may be considerably higher than is required for a conservative but economical design.

In the ML test procedure, loads are applied in increments, and each increment is maintained for a specified time or until the rate of settlement is less than a specified value. After the maximum load has been reached and maintained for the required time, the load is removed decrementally at specified intervals. Movement of the top of the pile is recorded immediately before and after loading or unloading and at intervals while the load is maintained constant.

The ML test procedure required by Arkansas Highway Department Standard Specifications (1972) calls for loading the test pile to 200 percent of the design load in increments of 25 percent of the design load. Increments are added at 30 minute intervals with settlement readings taken immediately before and after the addition of each load increment and three times between load increments. The unloading of the pile is accomplished by three decrements of 25 percent of the applied load, a decrement of 15 percent of the applied load and a final decrement of 10 percent of the applied load. The decrements are removed at 30 minute intervals with rebound readings taken before and after each decrement. A final rebound reading is taken 12 hours after the entire test load has been removed. The Arkansas ML test procedure was used in this research, but loading was carried to failure whenever the capacity of the loading system was adequate. Details of the load and movement measuring system and the method of determining the failure load will be discussed in subsequent paragraphs.

Texas Quick Test

The Texas quick (TQ) test was described by Fuller and Hoy (1970) and has been adopted by the Texas Highway Department. For the Texas quick test procedure used in the research reported herein, the load increments were the same as for the ML test but were applied at intervals of two and one-half minutes. Settlement readings were taken immediately before and after each load increment. When the ultimate load was reached, loading was stopped and the load and settlement were allowed to stabilize. Load and settlement readings were taken at two and one-half and five minutes after loading was stopped. The entire load was then removed and rebound readings were taken immediately, and at two and one-half and five minutes after removal of the load.

Constant Rate of Penetration Test

The constant rate of penetration (CRP) test procedure was proposed by Whitaker and Cooke (1961). In this test, load is applied to the pile in a manner to achieve a constant rate of penetration of the pile into the soil. The rates of penetration recommended by Whitaker and Cooke (1961) are 0.03 inches per minute for cohesive soils and 0.06 inches per minute for cohesionless soils although they report that rates may vary from half to twice these values without significantly affecting the results. Simultaneous readings of load and settlement or rebound are taken during loading and unloading.

Equipment and Instrumentation

The equipment required to perform a pile load test includes the loading system and the reaction system. Instrumentation is required to measure the load and the movement of the top of the pile.

Loading System. The system for applying loads to the top of the pile may employ either gravity loads, such as a ballast platform, or loads produced by hydraulic rams or jacks. The ballast platform would rest directly on the head of the pile and known weights would be carefully stacked on the platform. Tilting of the platform is controlled by spacers at each corner to limit vertical movement. This loading method is inexpensive but is difficult to implement satisfactorily in the field. It can be used only with the maintained load test. Hydraulic rams present the most easily controlled loading system. High capacities may be achieved by the use of multiple rams. Fluid pressure to actuate the rams may be produced by hand pumping, electric-powered pumps or by air-operated pumps. Pressure-compensated flow control valves can give the constant volume of flow needed for the constant rate of penetration test and other available valves make load control for all types of tests easily achieved.

Reaction System. When hydraulic rams are used, a reaction system must be provided. In soils where little skin friction is available, a ballast platform supported at the edges and loaded with a weight in excess of the maximum load to be applied to the pile will provide a satisfactory reaction. Where skin friction can be developed, a reaction frame is usually the choice. Piles are driven at least five pile diameters or seven feet, whichever is greater, from the test pile and a reaction beam is securely fastened to the reaction piles. The hydraulic ram is placed on top of the test pile and acts against the reaction beam to push the pile into the soil.

Measuring Systems. It is necessary to measure the load applied to the top of the pile and the movement of the pile under the applied load. Load may be measured by a load cell or by determining the hydraulic pressure in the loading ram. There are many types of load cells but most employ electrical resistance strain gages mounted on an elastic member and are both accurate and precise. Other types use a LVDT to sense the movement of the elastic member and some use a sealed hydraulic capsule and pressure gage. Load cells provide greater accuracy than measurements based upon pressure in the hydraulic ram. Friction is present in the working parts of the ram and is accentuated by eccentric loading. In some cases friction may be as much as 10 to 15 percent of the applied load. The use of a swivel head between the jack and the reaction beam will probably reduce friction to less than 5 percent.

The movement of the top of the pile may be measured by dial gages, wire and scale, engineer's level, or displacement transducers such as LVDTs or linear potentiometers. Any support for a beam holding dial gages or transducers should be at least eight feet from the test pile and as far from the reaction piles as is practical.

Dial gages should have a range of two to three inches and read to the nearest 0.001 inch. The wire and scale consists of small diameter music wire strung horizontally in fromt of a machinists scale mounted vertically on the pile. The wire is anchored to the support on one end and passes through a pulley on the other. A weight attached to the wire maintains constant tension. A mirror mounted behind the scale is used to eliminate parallax when reading the position of the wire on the scale. The scale should be six inches long with divisions of 0.01 inch. Displacement transducers usually have infinite resolution and should be read to the nearest 0.001 inch. Level readings should be taken to the nearest 0.001 ft. A redundant system is highly desirable with dial gages or displacement transducers as the primary system and wire and scale or engineer's level as the back-up system.

Interpretation of Results

After the load-settlement relationship is determined, the failure load must be established. There is no universally accepted criterion for establishing failure, but it is generally accepted that both load and settlement should be considered. Chellis (1961) has summarized 17 different criteria as follows:

1. The test load shall be twice the contemplated design load and shall be maintained constant for at least 24 hr and until settlement or rebound does not exceed 0.22 in. in 24 hr. The

design load shall not exceed one-half the maximum applied load provided the load-settlement curve shows no signs of failure and the permanent settlement of the top of the pile, after completion of the test, does not exceed ½ in. (Boston Building Code).

2. Observe the point at which, no settlement having occurred for 24 hr, the total settlement including elastic deformation of the pile is not over 0.01 in. per ton of test load, and divide by a factor of safety of 2 (Department of Public Works, State of California).
3. The safe allowable load shall be considered as 50 percent of that load which, after a continuous application for 48 hr, produces a permanent settlement not greater than ¼ in. measured at the top of the pile. This maximum settlement shall not be increased by continuous application of the test load for 60 hr or longer (AASHO).
4. Observe the point at which the plastic curve breaks sharply, and divide by a factor of safety of 1.5.
5. Tests shall be made with 200 percent of the proposed load, and considered unsatisfactory if, after standing 24 hr, the total net settlement after rebound is more than 0.01 in. per ton test load (building laws of the City of New York).
6. Observe the point at which the gross settlement begins to exceed 0.03 in. per ton of additional load, and divide by a factor of safety of 2 for static loads or 3 for vibratory loads (W.H. Rabe, Design Engineer, Bureau of Bridges, State

of Ohio).

7. Draw tangent lines to the general slopes of the upper and lower portions of the curve, observe the load at their intersection, and divide by a factor of safety of 1.5 or 2.
8. Observe the point at which the slope of the curve of gross settlement is four times the slope of the graph of elastic deformation of the pile, and divide by a suitable factor of safety.
9. The allowable axial load on an isolated pile shall not exceed: (a) 50 percent of the yield point under test load. The yield point shall be defined as the point at which an increase of load produces a disproportionate increase in settlement; or (b) one-half of the load which causes a net settlement, after deducting rebound, of 0.01 in. per ton of test load, which has been applied for a period of at least 24 hr; or (c) one-half of that load under which, during a 40-hr period of continuous load application, no additional settlement takes place (optional rules of International Conference of Building Officials Uniform Building Code).
10. Take two-thirds of the maximum test load in a case where settlement is not excessive and where load and settlement were proportionate and the curve remained a straight line. Where the test load was carried to failure, take two-thirds of the greatest load at which settlement was not excessive and at which loads and settlements were proportionate (United States Steel Co.).

11. With several consistent tests over the area of the structure, take from one-half to two-thirds of the failure load, considered as somewhere in the vicinity of the break in the curve showing increased settlement per unit of load added (Bethlehem Steel Co.).
12. The safe allowable load shall be considered as 50 percent of that load which, after a 48-hr application, causes a permanent settlement of not more than ¼ in. (New York State Department of Public Works).
13. One-half of the test load shall be allowed for the carrying load, if the test shows no settlement for 24 hr and the total settlement does not exceed 0.01 in. multiplied by the test load in tons (Chicago Building Code).
14. Observe the load at which is produced an increase in settlement disproportionate to the increase in load, and apply a factor of safety of 2 (Los Angeles Building Code).
15. Observe the load carried without exceeding a total permanent settlement of ¼ in. in 48 hr and divide by a factor of safety of 2 (Louisiana Department of Highways).
16. For important permanent structures, take the safe load on well-driven timber and concrete piles, with a final set of, say, ten blows to 1 in. at one-half to two-thirds of the test load which produces a final settlement gradually of ½ in. after a period of 10 days' rest. For well-placed undriven concrete piles, tested to twice their estimated bearing capacity, the safe bearing load has been taken in practice

at one-half the test load which gives a settlement of 3/8 in. after a period of rest of 10 days (W. Simpson, "Foundations," Constable & Co., Ltd., London, 1928).

17. Observe the point at which the gross settlement begins to exceed 0.05 in. per ton of additional load, or at which the plastic settlement begins to exceed 0.03 in. per ton of additional load, and divide by a factor of safety of 2 for static loads or 3 for vibratory loads (Dr. R. L. Norlund, Raymond Concrete Pile Company).

The Federal Highway Administration recommends the Davisson criterion for quick load methods. This criterion requires that the gross pile head movement at 200 percent of the design load shall be less than the calculated elastic compression at that load (assuming that the load in the pile is uniform from head to tip) plus 0.15 inches plus 1/120 of the pile diameter.

The Texas Highway Department uses a combination of rules 7 and 17 for interpretation of the results of the Texas quick test. Details of the interpretation procedure are given below and in Figure 3.1.

1. Plot a graph of load versus gross settlement using any convenient scale.
2. Draw one line originating at the point of zero load and settlement and tangent to the initial flat portion of the gross settlement curve. (The slope of this line will be approximately the same as the slope of the recovery line.)
3. Draw a second line tangent to the steep portion of the gross settlement curve with a slope 0.05-in. of settlement per ton.

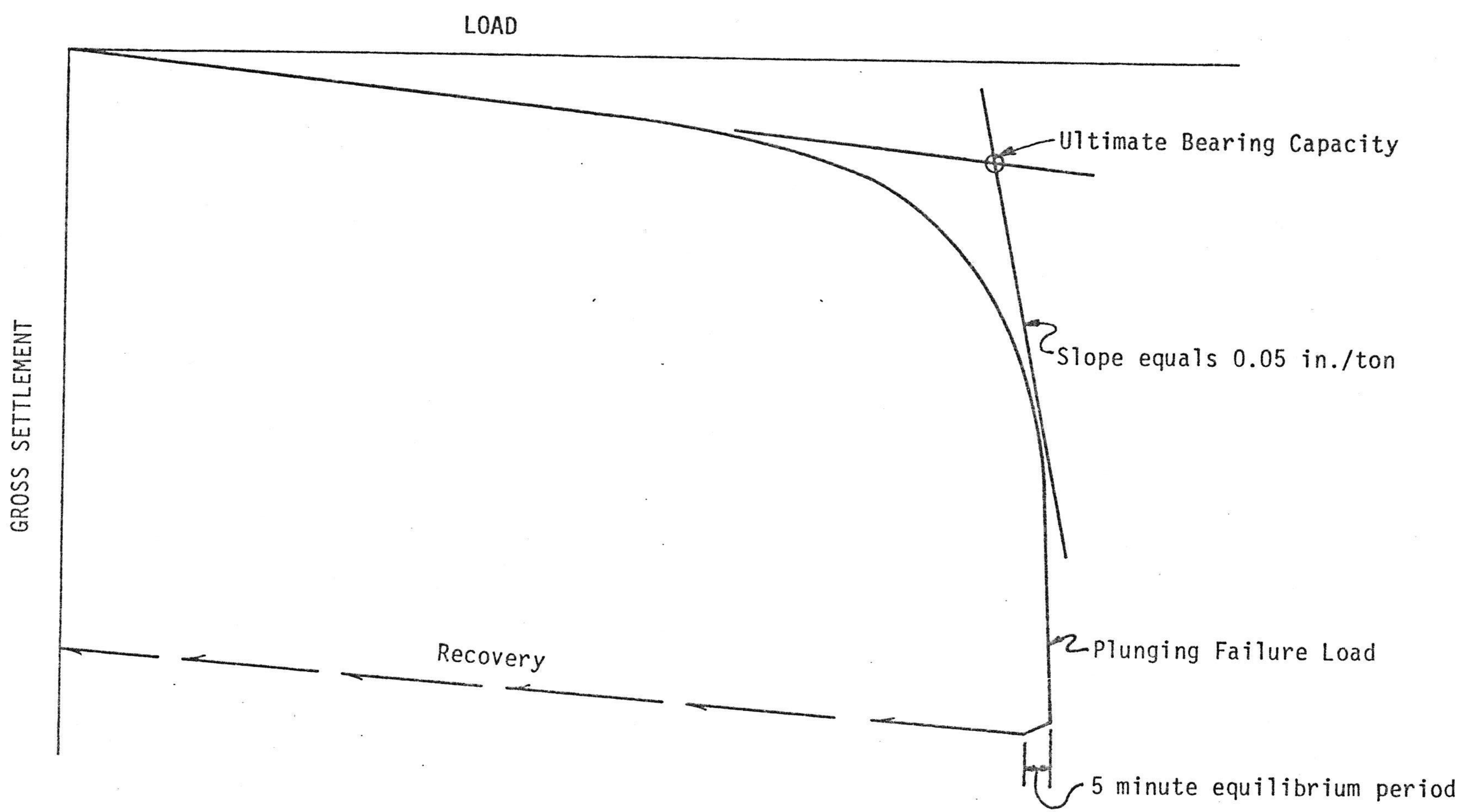


Figure 3.1 Typical Load Settlement Graph

4. The load at the intersection of the two tangents drawn in steps 2 and 3 is defined as the ultimate bearing capacity of the pile and will be used to establish a proven "maximum safe static" load.
5. The proven maximum safe static load for piling is defined as one-half of the ultimate bearing capacity obtained in step 4.

This method was used in the research reported herein to determine the failure load.

CHAPTER IV

INSTRUMENTATION FOR LOAD TRANSFER

Load is transferred from pile to soil through skin-friction and end bearing. If the load in the pile as a function of depth is known, then it is possible to determine the portions of load carried by skin friction and end bearing and to determine the distribution of skin friction with depth. If load distribution is measured at intervals during the load testing of a pile, then load transfer as a function of displacement can be determined. The purpose of the instrumented tests performed as a part of this research was to measure load transfer behavior and to compare this behavior for the various load test procedures used.

Virtually all of the pile instrumentation systems use strain or displacement measuring devices. The simplest system uses strain rods or "telltales" to measure deformation at intervals along the length of the pile. Strain rods encased in protective sheathing or tubing are anchored at various positions along the length of the pile and extend to the top of the pile. The rod is free to move without friction in its sheath and the movement of the top of the rod is monitored by a dial gage. The pile is an elastic member and thus the movements can be translated into loads by computing average strains between anchor points.

A more precise system would employ strain gages or strain transducers attached or embedded at various points along the length of the pile. Since the gage length of the strain gages is usually less than an inch and the gage length of most strain transducers is only a few inches, this system essentially measures load at a point. A recoverable strain transducer system was designed and constructed for use on this project. The elements and operation of this system are described in the following paragraphs.

The elements of the recoverable strain transducer system are the strain transducers, the positioning apparatus, and the data acquisition system. A sketch of a portion of the system is shown in Figure 4.1.

Strain Transducers

The strain transducers have a gage length of six inches and use 120 ohm electrical resistance strain gages as the sensing elements. Gages are mounted on both tension and compression faces of a steel bar one-eighth inch thick and one-half inch wide. The bar is bent into a semicircle with a radius of one inch. Steel bars,one-half inch square, are fastened to the bent bar at its diameter. The square bars contain sharp, hardened points spaced six inches apart. The hardened points are forced into the inside wall of a pipe pile and, as the pile is loaded, the bent bar which has the strain gages mounted on it is deformed. A sketch of the elements of the strain transducer is shown in Figure 4.2 Each transducer is individually calibrated and the strain gages may be read individually (quarter bridge), in pairs (half bridge) or the pair of transducers at a depth may be connected to read the average strain at that depth (full bridge). Connection as a full bridge gives the highest level of output but three lead wires from each transducer allow the gages to be read individually in case of

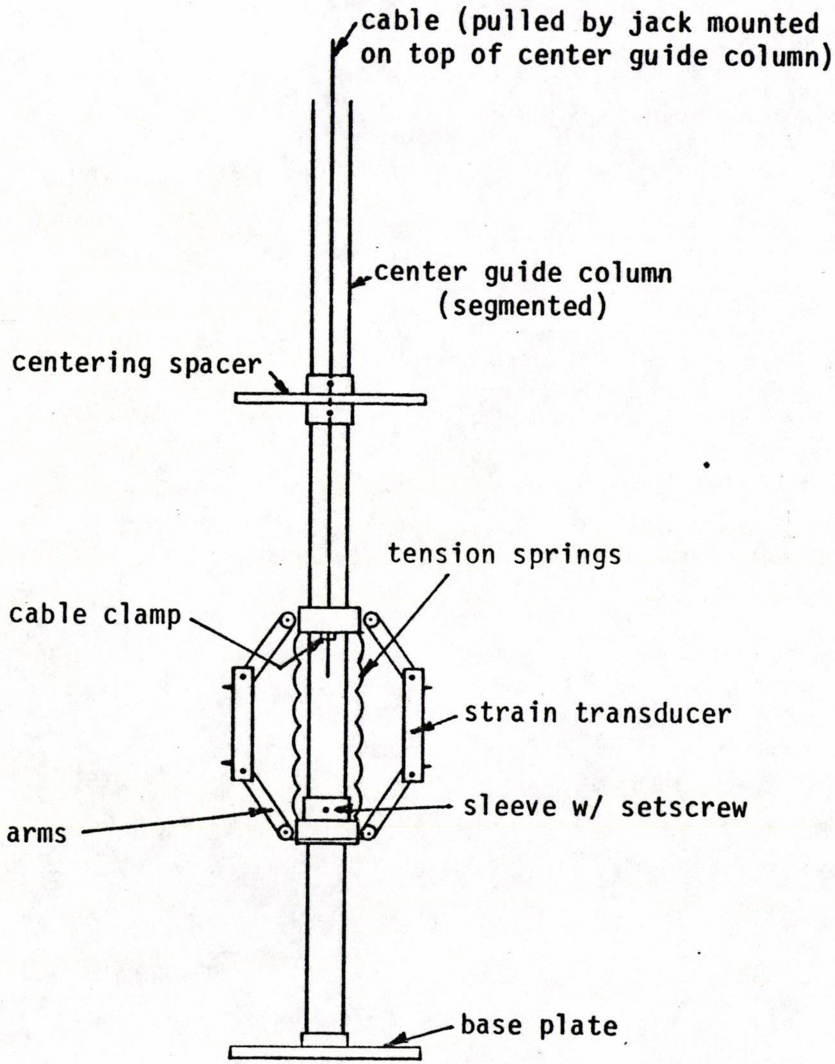


Figure 4.1 Sketch of Instrumentation System

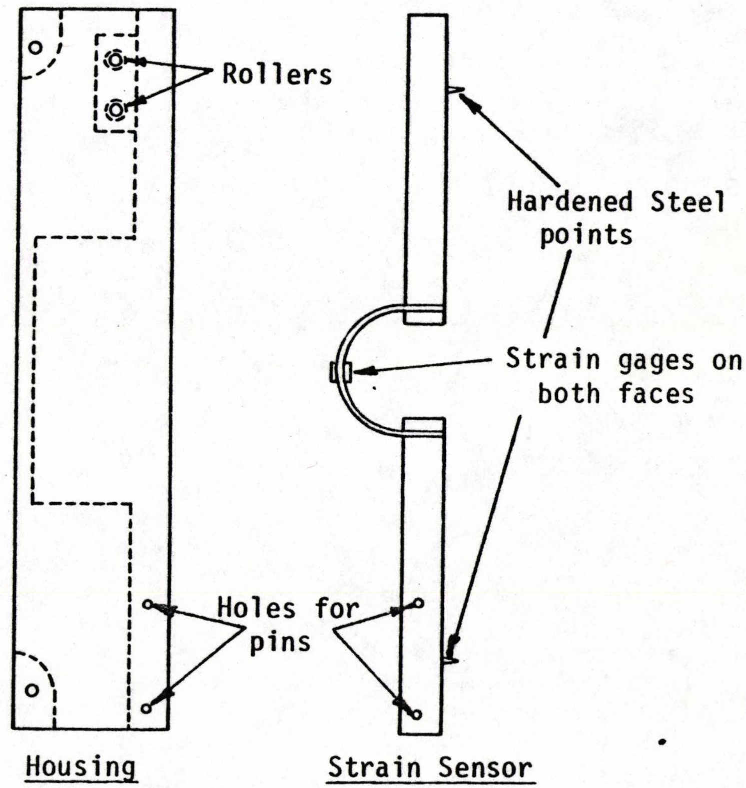


Figure 4.2 Elements of Strain Transducer

a malfunction.

Positioning Apparatus

The transducers are positioned along a central column within a steel pipe pile and tension springs push the points on the transducers into the pile wall. A base plate slightly smaller than the inside of the pile is fastened to the bottom of the central column and spacers to center the column are placed at intervals of about 20 feet. A sleeve with a setscrew is used to position each set of transducers along the central column. The column is segmented and screws together for ease in handling and installation. A cable attached to each set of transducers is pulled by a jack resting on top of the central column to retract the strain transducers for installation. Another cable attached to the base plate is used for handling the assembled system.

Data Acquisition System

The data from the strain transducers are taken with a 20-channel strain gage system consisting of a digital indicator, 20 channels with individual balance and gain controls, a scan controller, and a digital printer. Each channel may be individually set for quarter, half, or full bridge circuits. The scan control allows manual or automatic selection of the channel to be monitored. In the automatic mode, either one cycle through all channels in sequence or continuous scanning of all channels in sequence may be selected. The sampling rate is about one channel per second. As the channels are scanned, the reading is displayed on the digital indicator and printed on paper tape along with channel identification.

Calibration

Several modes of calibration are used to determine the calibration of the strain transducers. The displacement over the six-inch gage length may be measured with a micrometer and plotted versus gage output. Because of the small displacements involved, this approach does not provide adequate sensitivity. The alternate modes involve the installation of the transducers in a section of pipe pile, loading the section in compression in the laboratory and observing the output for quarter, half, and full bridge connections. To observe true strain, strain gages were mounted on the outside of the pipe pile section at the location of the strain transducers. If the same pile section is used for calibration as is driven in the field, the calibration curve of load versus strain reading may be plotted directly from the laboratory data.

CHAPTER V

TEST RESULTS

Load tests were performed at three job sites representing different soil conditions. These sites were in or near Newport, Smackover, and Redfield, Arkansas. The soil conditions were stratified sand, silt, and clay at Newport, sand at Smackover, and clay at Redfield. The load test procedures used at each site were maintained load (ML) test, Texas quick (TQ) test, and constant rate of penetration (CRP) test. Piles instrumented for load transfer measurements were tested at the Smackover and Redfield sites.

The same equipment was used for all piles tested. A hydraulic ram with a rated capacity of 150 tons was used and was actuated by an air-operated hydraulic pump. Load was determined by a Bourdon pressure gage and a pressure transducer reading hydraulic pressure on the ram. Settlement was measured to the nearest 0.001 inch with two dial gages placed on opposite sides of the pile. The redundant system was a wire and machinist's scale reading to the nearest 0.01 inch.

Newport Tests

Load tests were performed on two 16-inch octagonal concrete piles in connection with a railroad grade separation on State Highway 69 in Newport, Arkansas. The test piles have a design load of 44 tons and are incorporated into the structure. The load tests were performed six days after the piles were driven.

Test pile No. 1 was 45 feet long and was driven to a penetration of 43 feet. A seven foot deep excavation was made before driving the test pile and reaction piles. The tip of the test pile is at a depth of 50 feet below the original ground surface. Some difficulty was encountered in installing Test Pile No. 2. The first two piles (35 and 45 feet in length) did not provide sufficient bearing according to the dynamic formula (EN) used. These were subsequently pulled and a third pile 60 feet in length was driven. Eighteen additional feet were added to this pile and it was broken during driving at a depth of 74 feet. The 45 foot test pile was then driven approximately three feet from the broken pile. The disturbance of the soil around this pile undoubtedly caused a reduction in pile capacity but would not affect a comparison of quick tests to conventional tests.

Soil Conditions. The test site is located in an old floodway of the White River in north central Arkansas. The soil is generally composed of recent alluvial deposits of clay, sand and gravel. Five soil strata were found to be fairly uniform throughout the site. Logs of borings adjacent to each test pile are shown in Figures 5.1 and 5.2. Borings showed the depth to the water table to be from 6.5 to 10 feet below the original ground surface but subsequent excavations for footings showed the water table to be approximately 3 feet below ground surface. The presence of a city lake near the test site is probably responsible for the shallow ground water table.

Load-Settlement Curves. The load-settlement curves for the two Newport site test piles are given in Figures 5.3 and 5.4. The sequence of tests was maintained load test, constant rate of penetration test, and Texas quick test. A cumulative plot of load versus movement for Test Pile 1 is shown in Figure 5.5. The differences in ultimate loads carried by the test piles due to test procedures were less than 2 percent.

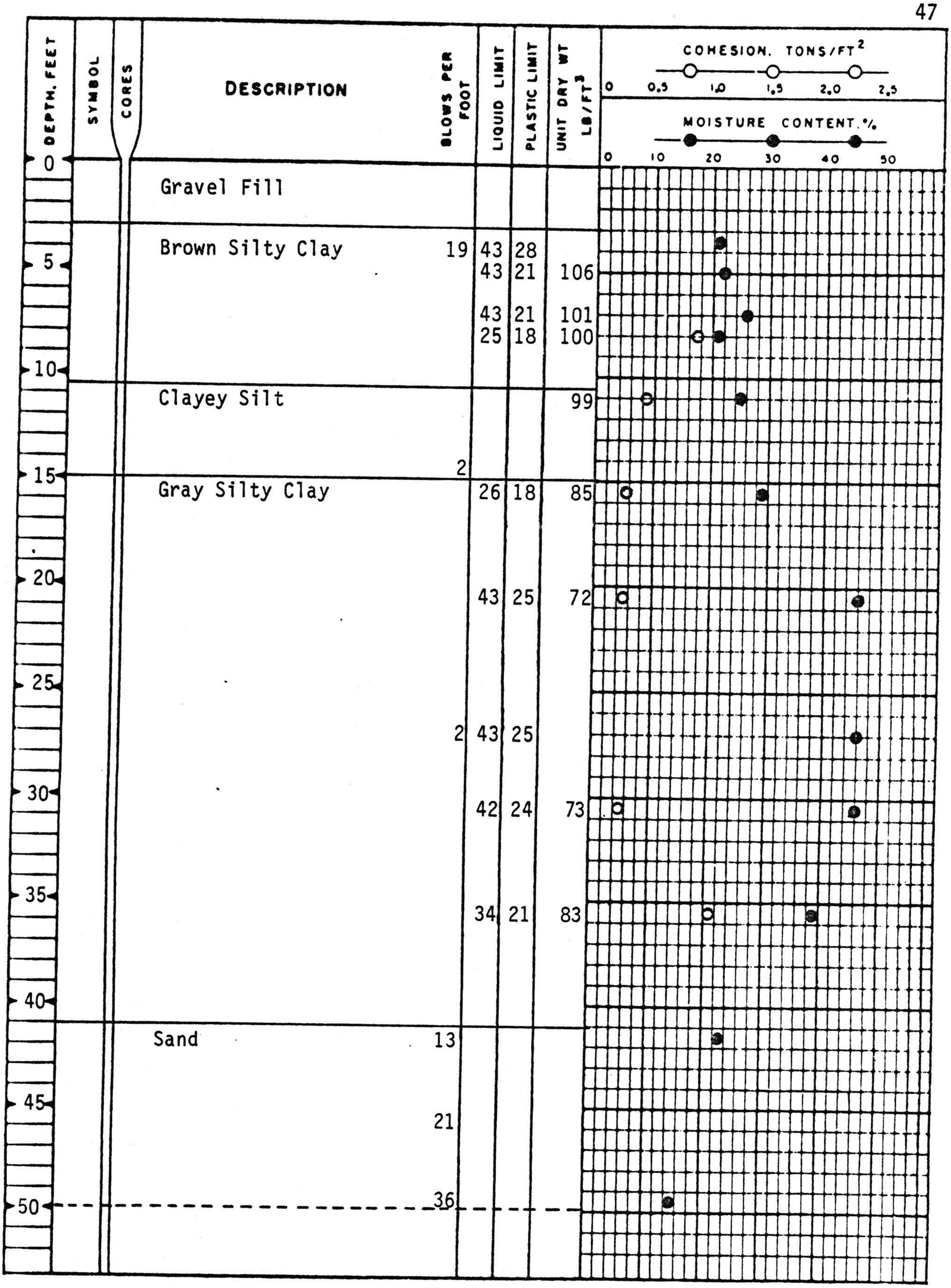


Figure 5.1 Boring Log for Newport Pile No. 1

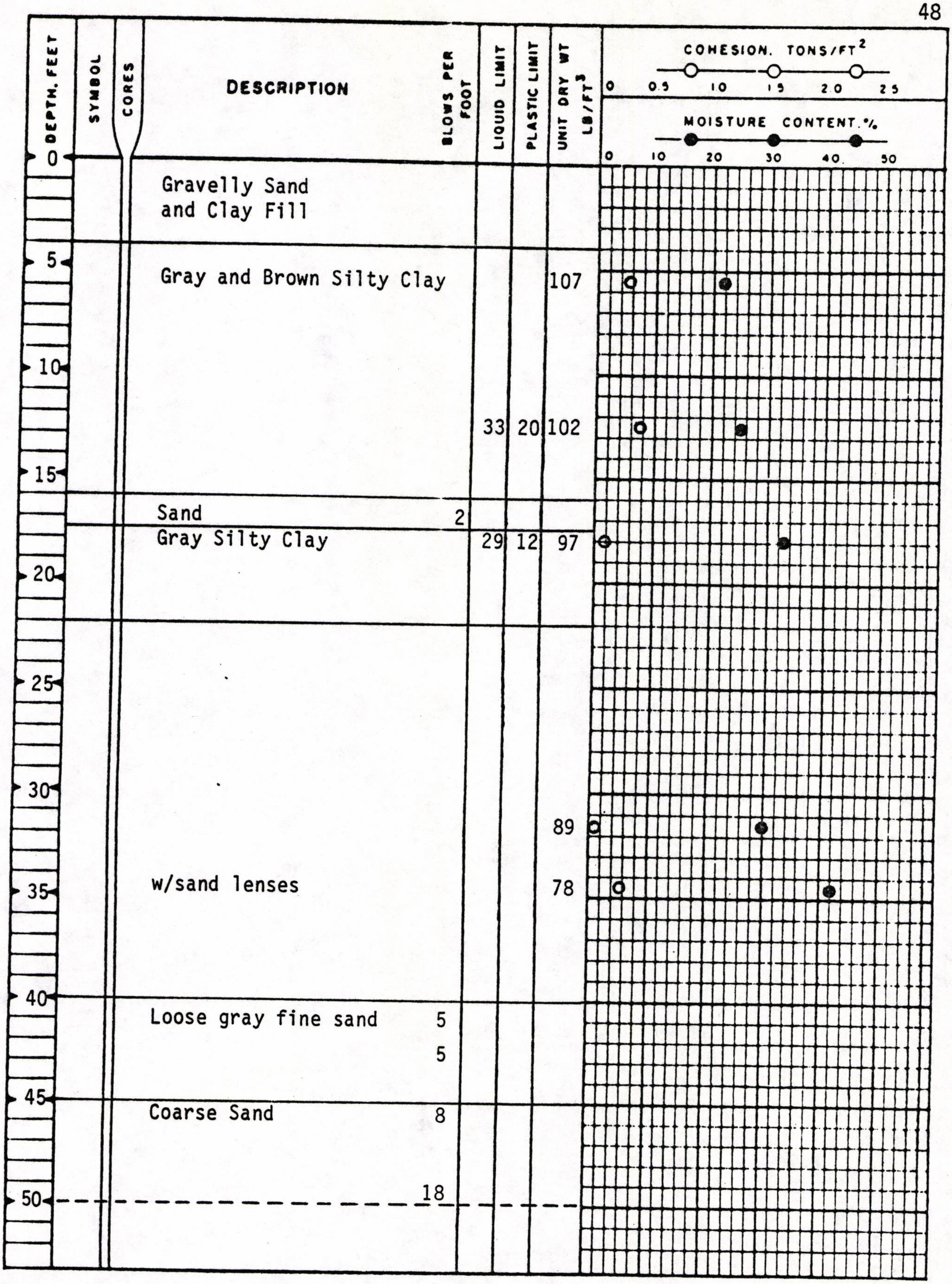


Figure 5.2 Boring Log for Newport Pile No. 2

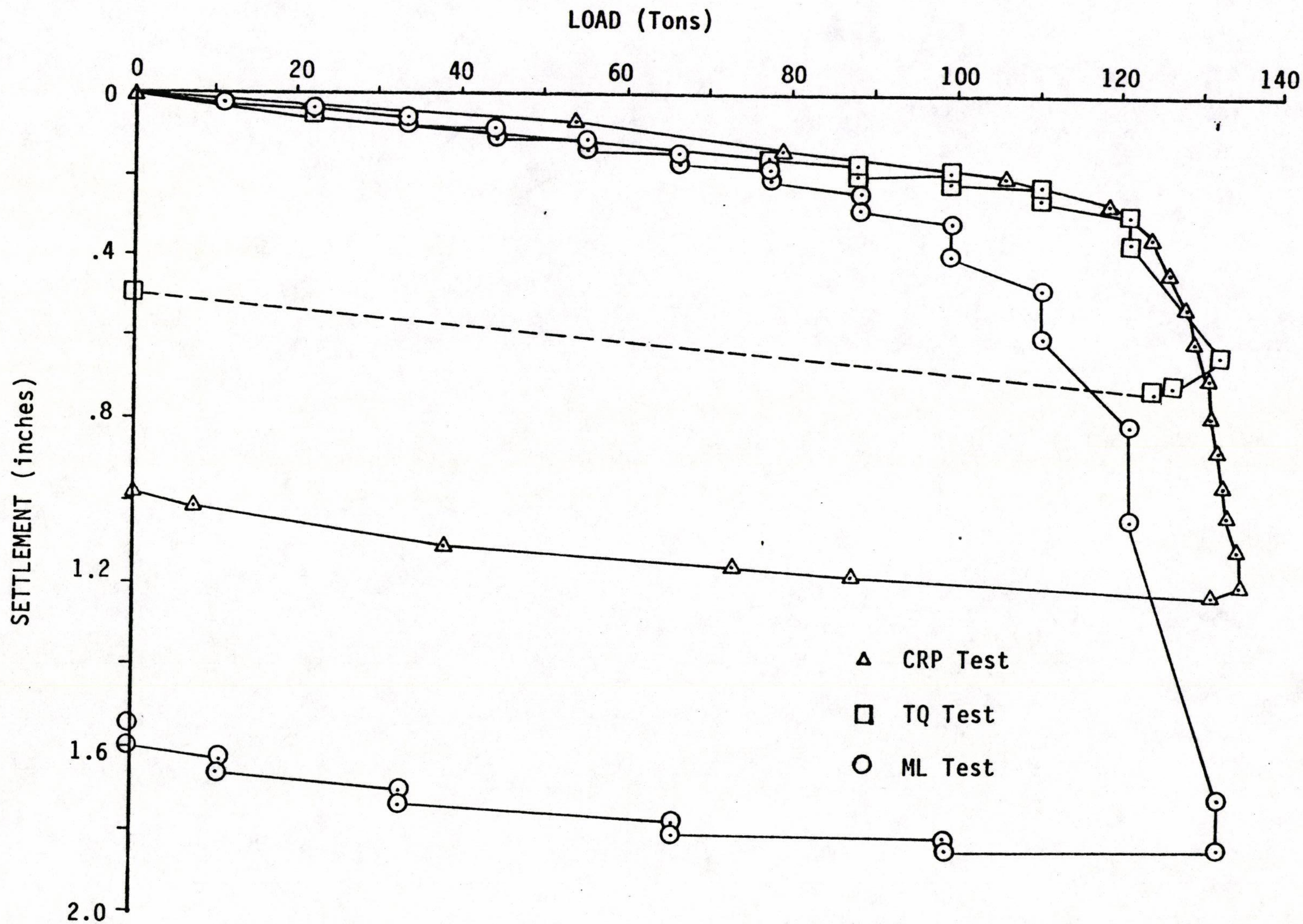


Figure 5.3 Load Settlement Curves for Various Test Methods (Pile Number 1)

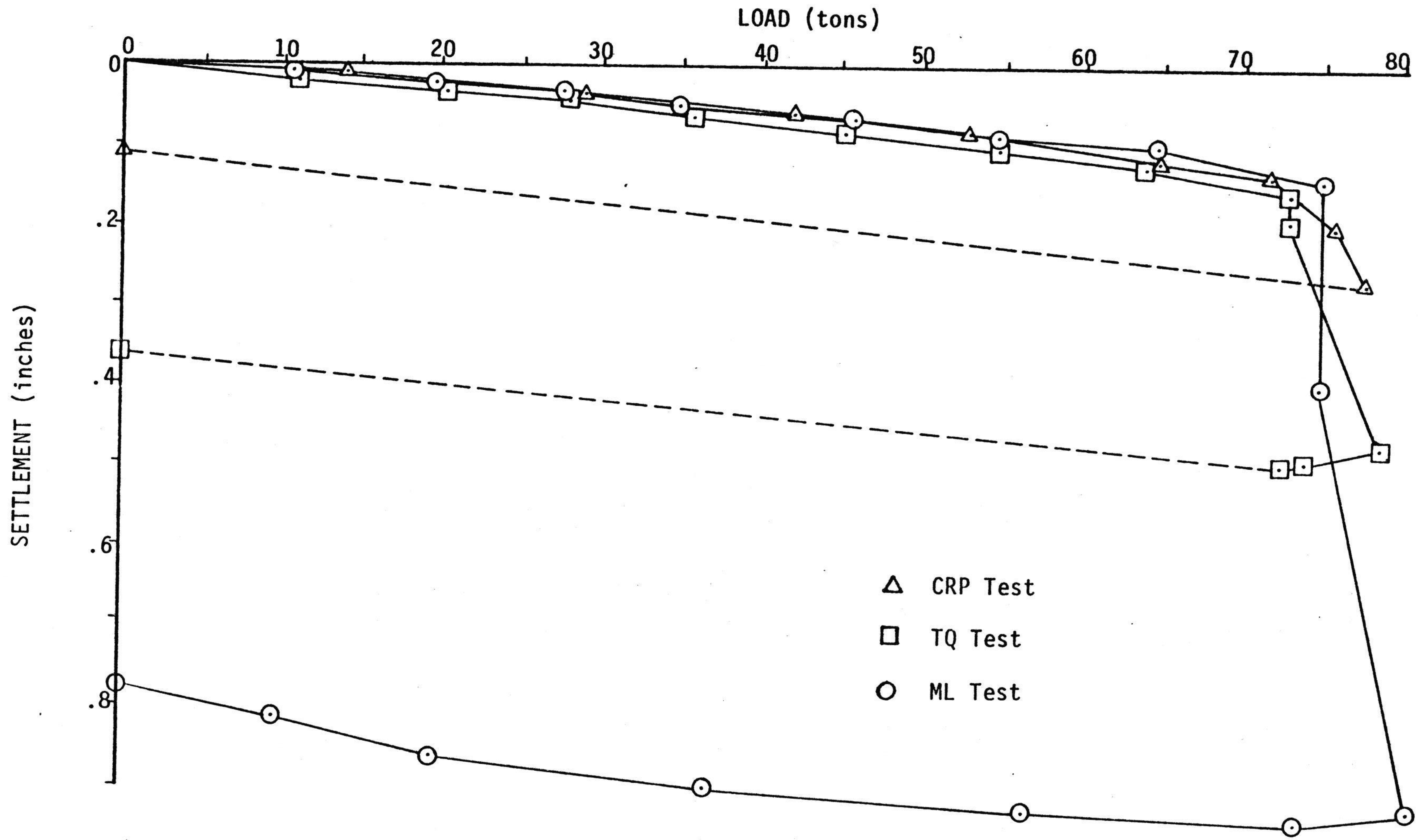


Figure 5.4 Load Settlement Curves for Various Test Methods (Pile Number 2)

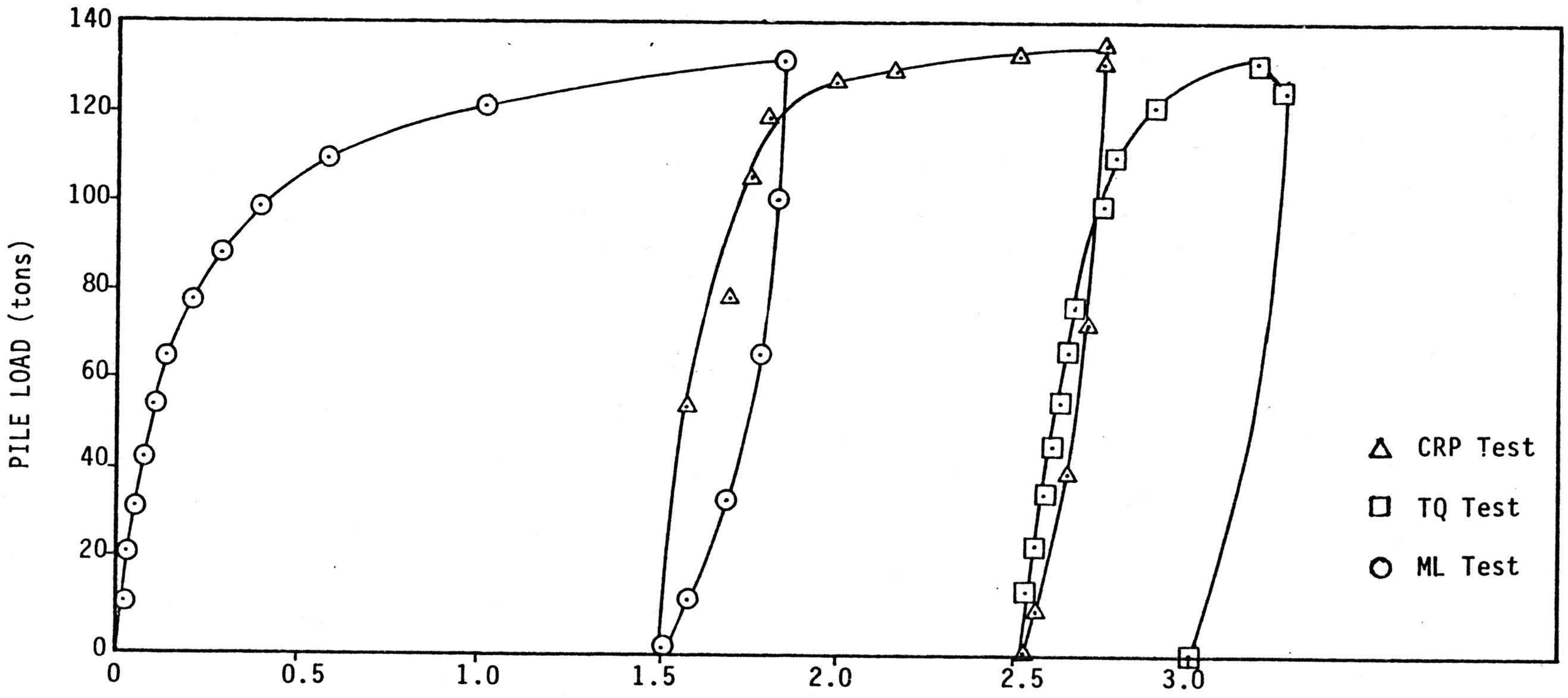


Figure 5.5 Series of All Load Tests as They Were Performed on Pile Number 1.

In the working load range (up to 50% of ultimate), there are no significant differences in the load-settlement curves. As the maximum load was approached for Test Pile 1, the CRP and TQ tests gave essentially the same results but the settlements observed in the ML test were larger. All three test procedures yielded almost identical results for Test Pile No. 2.

Pile Capacity Predictions. The ultimate load carrying capacity determined from the load tests was 126 tons for Test Pile No. 1 and 75 tons for Test Pile No. 2. These values are compared with several methods of predicting the ultimate capacity.

The static load carrying capacity of each test pile was calculated using the shear strength parameters obtained from standard penetration tests and laboratory tests on undisturbed samples. The contribution of skin friction in the sand immediately above the pile tip was not included due to arching as described by Vesic (1970). The predicted pile capacity is 123 tons with 71 tons end bearing and 52 tons due to skin friction for Test Pile No. 1. The static capacity of Test Pile No. 2 was predicted to be 83 tons, with 40 tons skin friction and 43 tons end bearing. This value is somewhat higher than the test load results but the effect of soil disturbance was not considered in the analysis.

Table 5.1 shows comparisons between the test results and several dynamic formulae plus the value obtained by use of the wave equation computer analysis. Any dynamic formula must necessarily predict the capacity of the pile at the time of driving. Since the pile was driven through a considerable layer of cohesive material, some set-up with time will result. Both piles were tested six days after driving, so the ultimate pile capacity determined from the load tests should be somewhat

TABLE 5.1

Predicted and Measured Pile

Capacities for Newport Test Piles

METHOD USED	ULTIMATE PILE CAPACITY (tons)	
	TEST PILE NO. 1	TEST PILE NO. 2
Load Tests		
ML	124	75
TQ	126	75
CRP	126	75
Engineering News Formula	281	123
Danish Formula	178	98
Hiley Formula	115	52
Wave Equation	100	61
Limit Equilibrium Analysis (Based on soil properties)	123	83

greater than the values predicted by the dynamic formulae. As may be seen in Table 5.1, the Danish formula and the Engineering News formula predicted values for ultimate load which are excessively high. The wave equation and the Hiley formula showed good agreement for these tests.

Smackover Tests

Load tests were conducted on three piles at new bridge locations over Holmes Creek and Smackover Creek on Arkansas State Highway No. 7 in Union County, Arkansas. The test site locations are shown on the vicinity map (Figure 5.6).

Test Pile 1 was driven at Holmes Creek and Test Piles 2 and 3 were driven at Smackover Creek. Test Piles 1 and 3 were 16-inch octagonal precast prestressed concrete piles and were also to be used as part of the bridge substructure. Test Piles 1 and 3 are to carry a design load of 60 tons and a minimum pile capacity of 120 tons. Test Pile 2 was an instrumented steel pipe pile 10-3/4 inches outside diameter with 3/8 inch wall thickness to be used only for research purposes. Test Piles 1 and 2 were driven to a penetration of 35 feet and Test Pile 3 was driven to a penetration of 27 feet.

A set-up time of 4 days was between driving and testing to allow pore pressures that developed during driving to dissipate.

Soil Conditions. The underlying soil strata generally consist of alluvial deposits of silt, sand, gravel and some clay. One soil boring was made at each test site as close as possible to the test piles. The soil strata encountered at each test site were different.

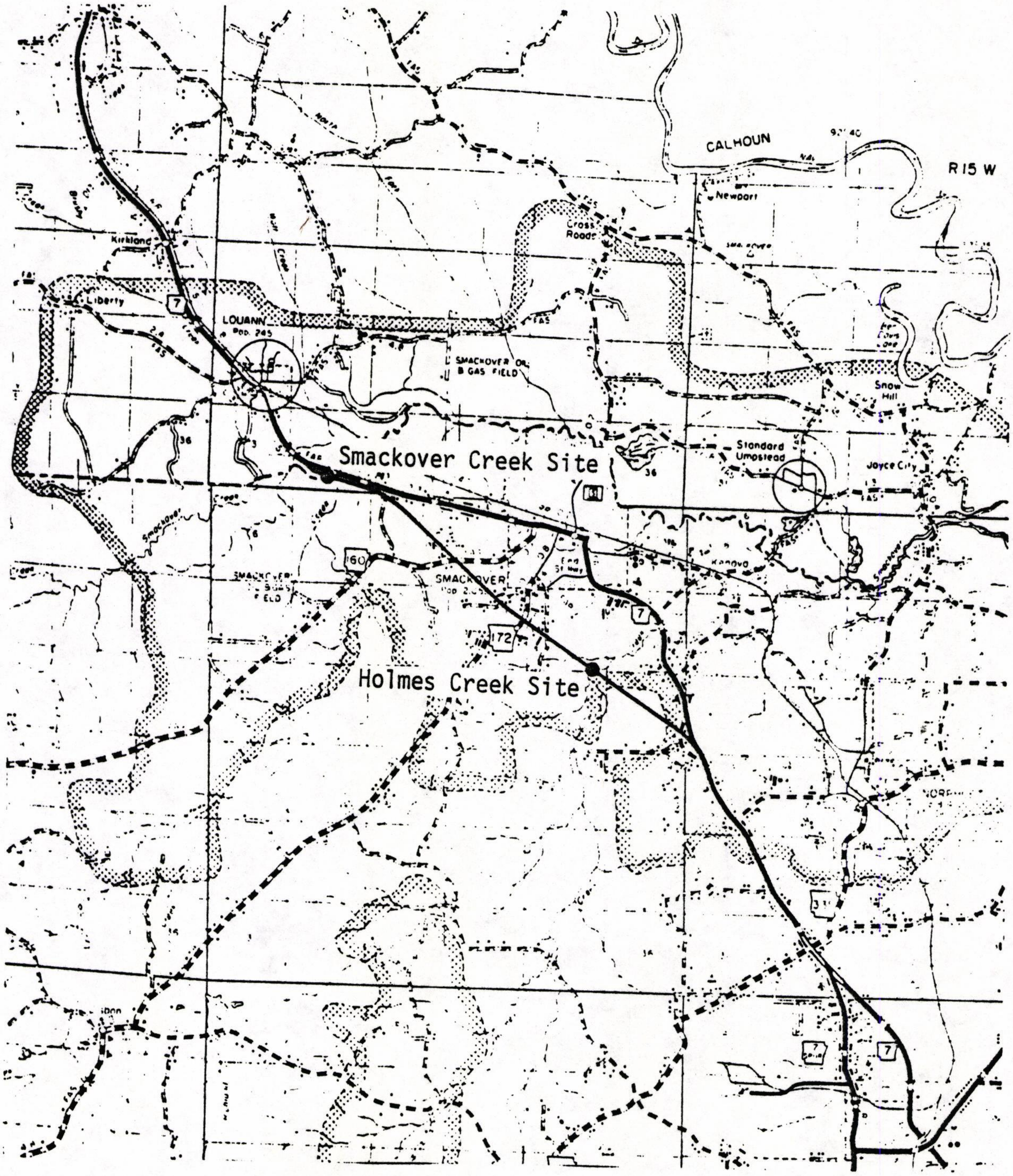


Figure 5.6 - Test Sites at Smackover, Arkansas

The soil boring at the Holmes Creek test site indicated about four feet of loose sandy silt and the remaining depth to be a medium dense to dense sand with the bottom several feet containing gravel. A boring depth of only 25 feet was obtained because the gravel stratum caved into the hole. The boring log for the Holmes Creek site (Test Pile 1) is shown in Figure 5.7.

The soil boring at the Smackover Creek Test site indicated about a four-foot stratum of brown silt, about ten feet of loose gray sand, about six feet of medium dense gray sand and gravel, about twelve feet of dense gray fine sand and the remaining depth is a dense gray clayey sandy silt. The boring log for the Smackover Creek site (Test Piles 2 and 3) is given in Figure 5.8.

Standard penetration tests were performed and disturbed samples were taken with the 2-in. split-spoon sampler. Several attempts were made to recover undisturbed samples by use of a 3-in.Shelby tube sampler but the samples could not be retained in the tube.

Laboratory tests were performed in order to determine the moisture content of all recovered samples. Additional laboratory tests were performed on a sand sample taken at a depth of 18 feet from the boring at the Smackover Creek site. A repeated direct shear test was conducted for each of the following conditions: to determine the angle of internal friction for the sand, to determine the angle of friction between the sand and a steel plate with approximately the same surface texture as that of the pipe pile, and to determine the angle of friction between the sand and the surface of a piece of the concrete pile. A rate of strain of .024 inches per minute was

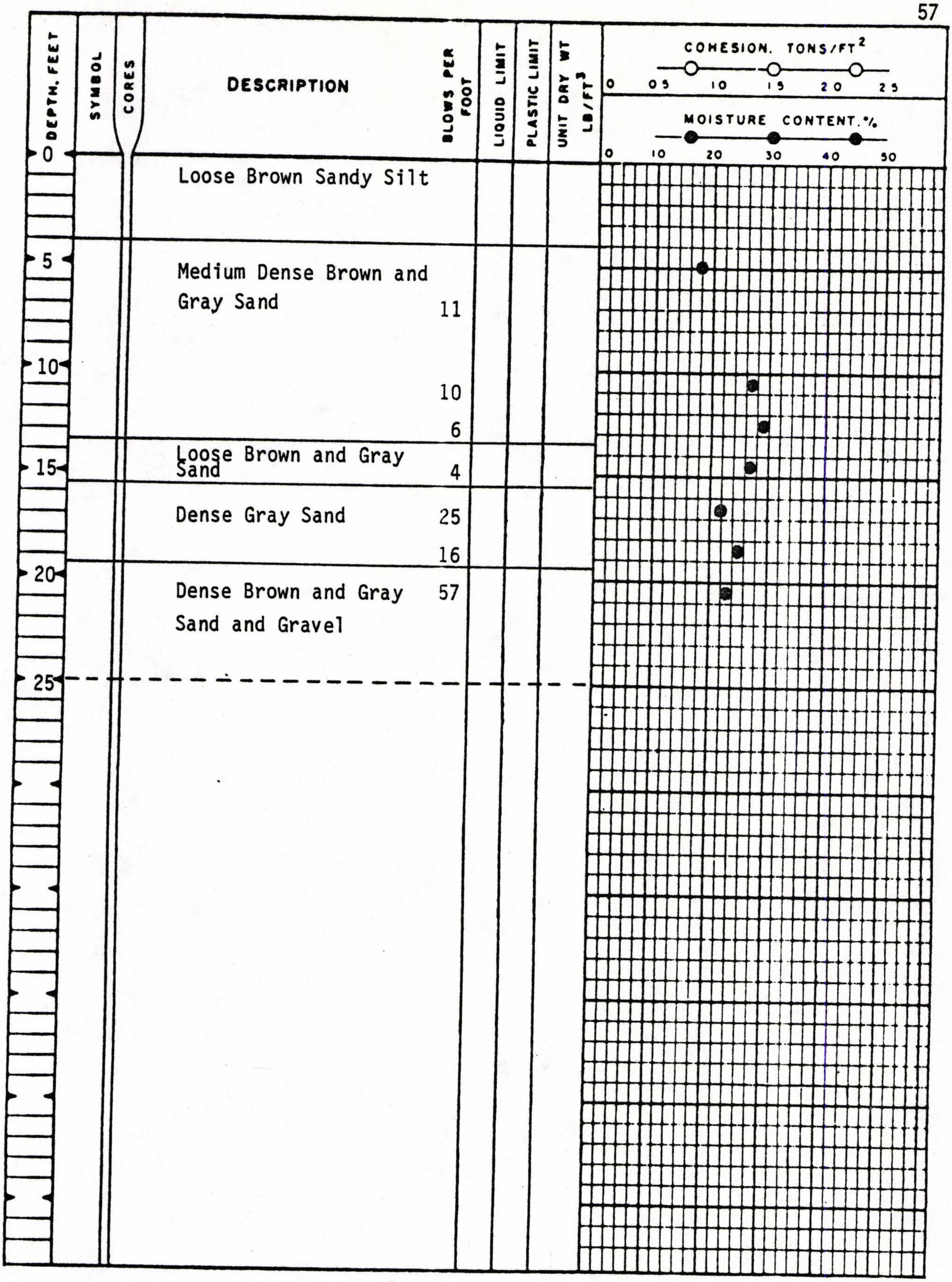


Figure 5.7 Boring Log at Holmes Creek Site

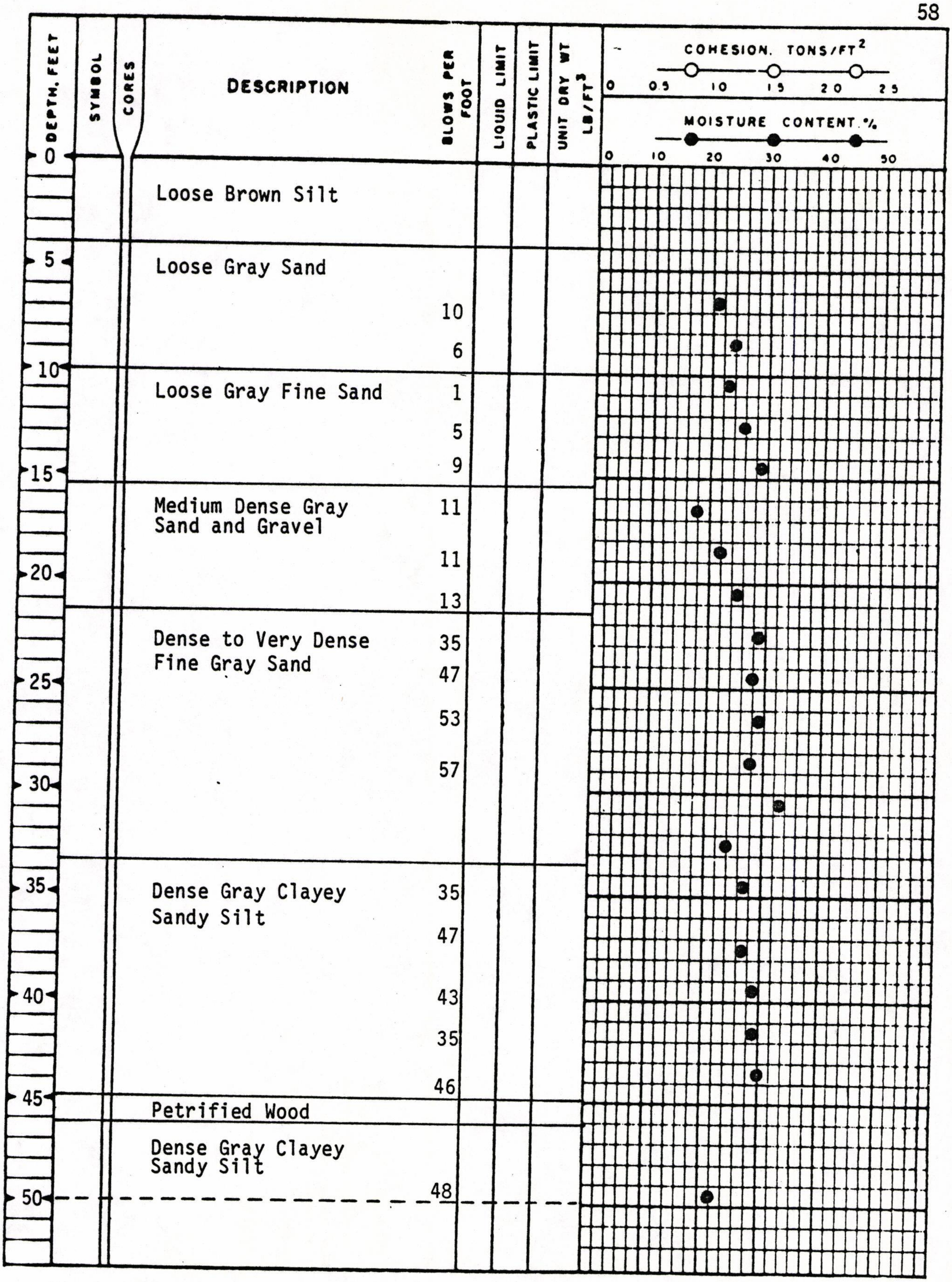
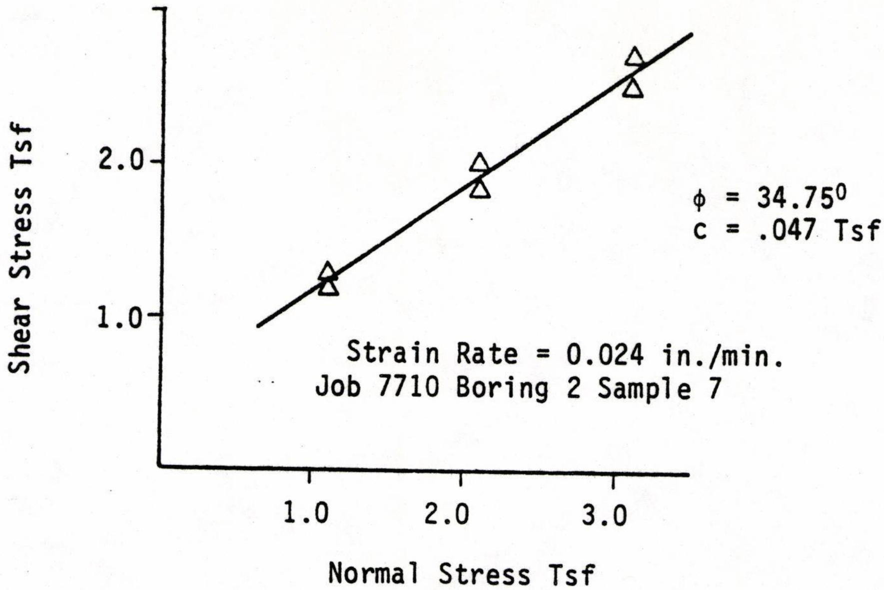


Figure 5.8 Boring Log at Smackover Creek Site

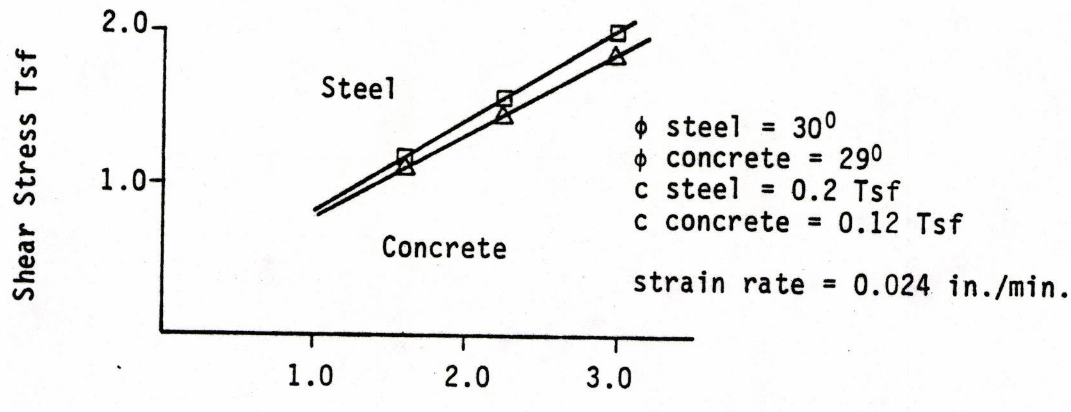
used for each repeated direct shear test. Large strains were obtained during each cycle of the repeated direct shear test resulting in reorientation of the sand particles in the shear zone. These tests indicated an angle of internal friction of 35^0 for the sand, an angle of friction of 30^0 between both the sand and the steel surface and the sand and the concrete surface. Plots of shear stress versus normal stress for these tests are given in Figure 5.9. Shear stress versus displacement curves are given for each of the above tests in Figure 5.10. These curves may be compared to the unit skin friction versus pile displacement curves determined from load tests on the instrumented steel pipe pile.

Load-Settlement Curves. The load-settlement curves for Test Piles 1, 2, and 3 are given in Figures 5.11, 5.12, and 5.13 respectively. The sequence of test procedures used for these piles was CRP, TQ, and ML. The 150 ton rated capacity of the hydraulic ram did not allow testing to failure of any of these piles. For some of the tests, the applied load was taken beyond the rated capacity of the jack by 25 percent to the maximum pressure output for the hydraulic pump in an attempt to apply a failure load to the test piles. This maximum load was used for the load tests which required only a short interval at this load and for the Maintained Load Test on Test Pile 1.

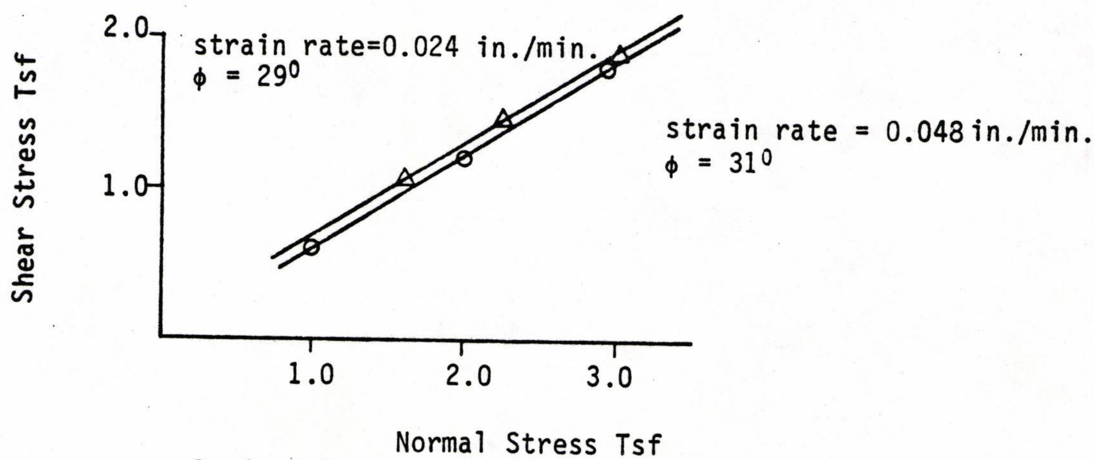
The load settlement curves for the CRP test on all test piles indicated that ultimate pile capacity was approached but not obtained. The load settlement curves for the TQ and ML tests agree closely for each test pile. These curves are approximately straight lines which indicates that ultimate load was not being approached. The CRP test



Shear Test of Sand

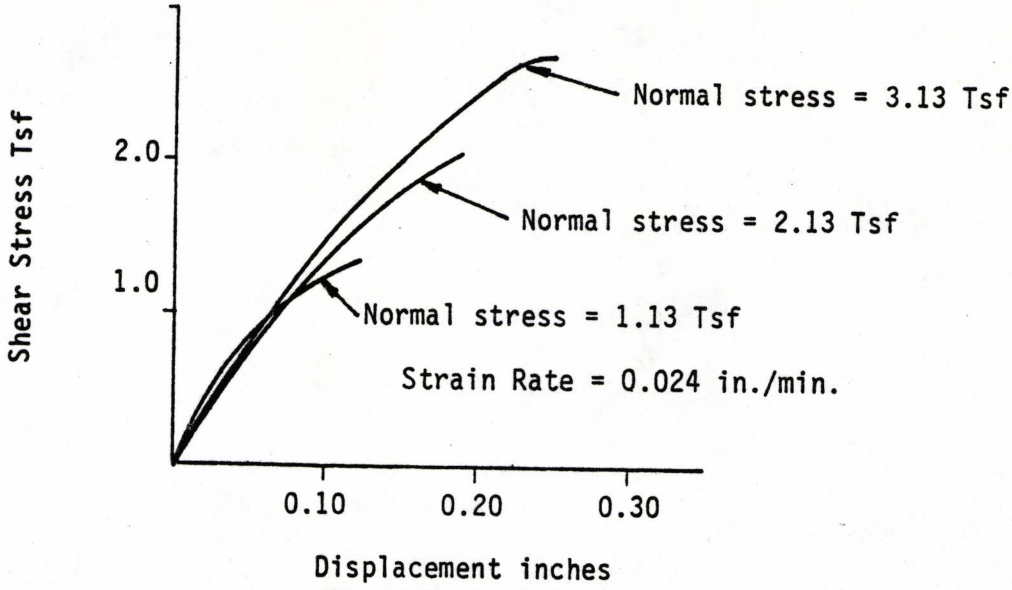


Sand against steel and concrete pile surface

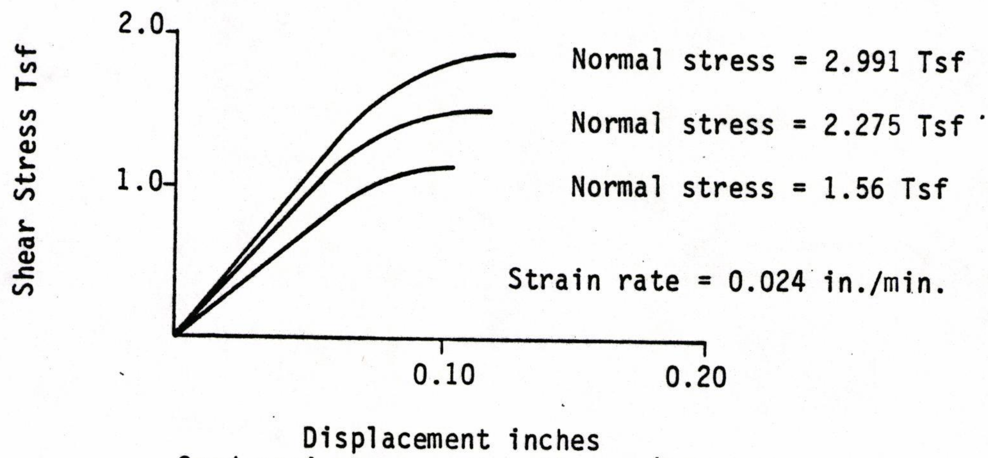


Sand against concrete pile surface

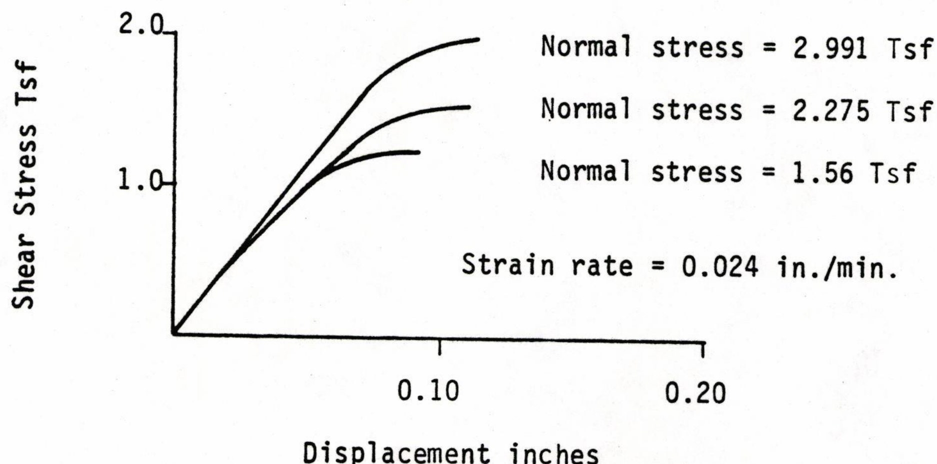
Figure 5.9 Angles of Internal Friction and Skin Friction



Shear Test of Sand



Sand against concrete pile surface



Sand against steel pipe pile surface

Figure 5.10 Shear Stress vs. Displacement Curves

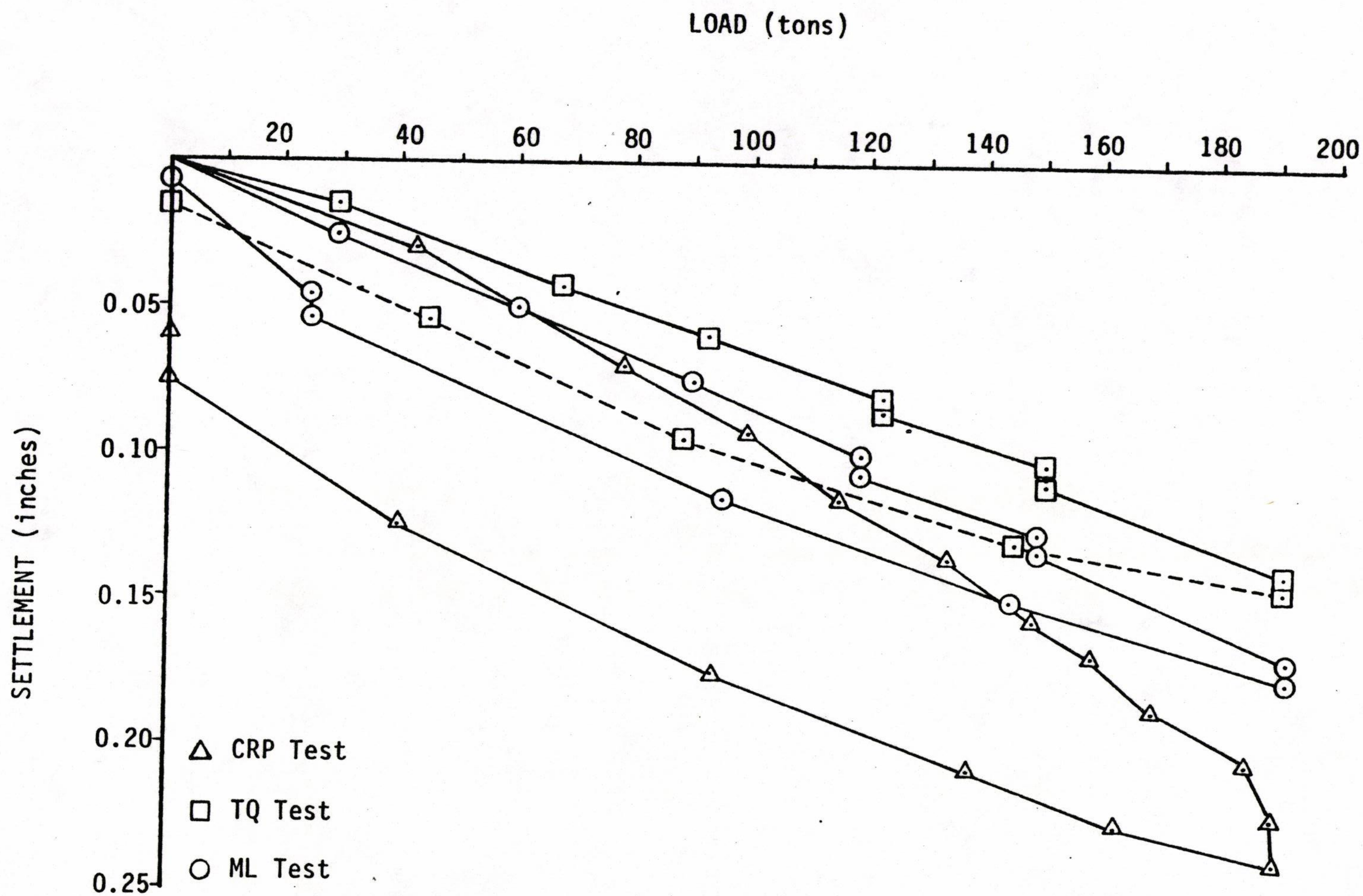


Figure 5.11 Load Settlement Curves for Test Pile Number 1

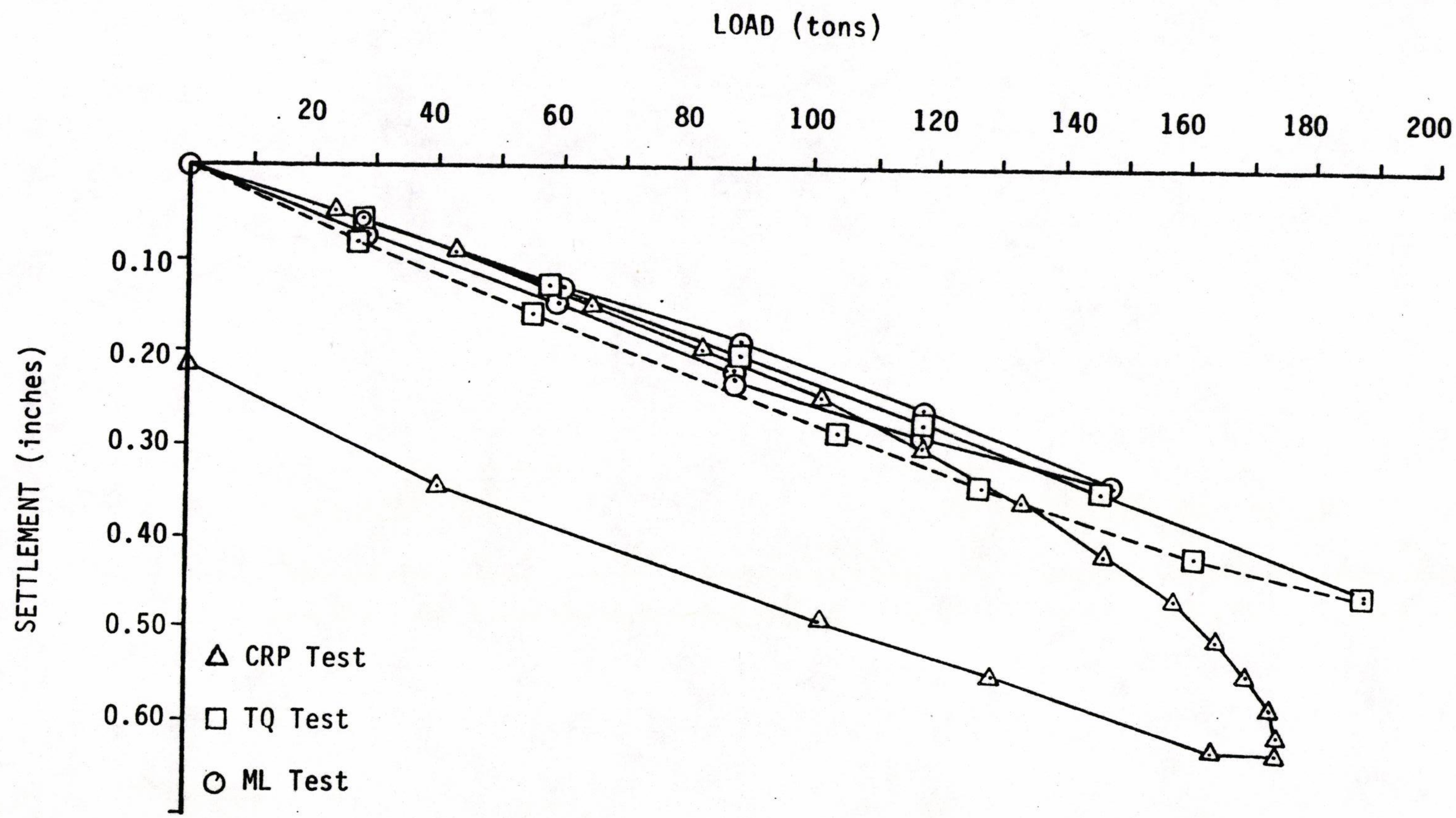


Figure 5.12 Load Settlement Curves for Test Pile Number 2

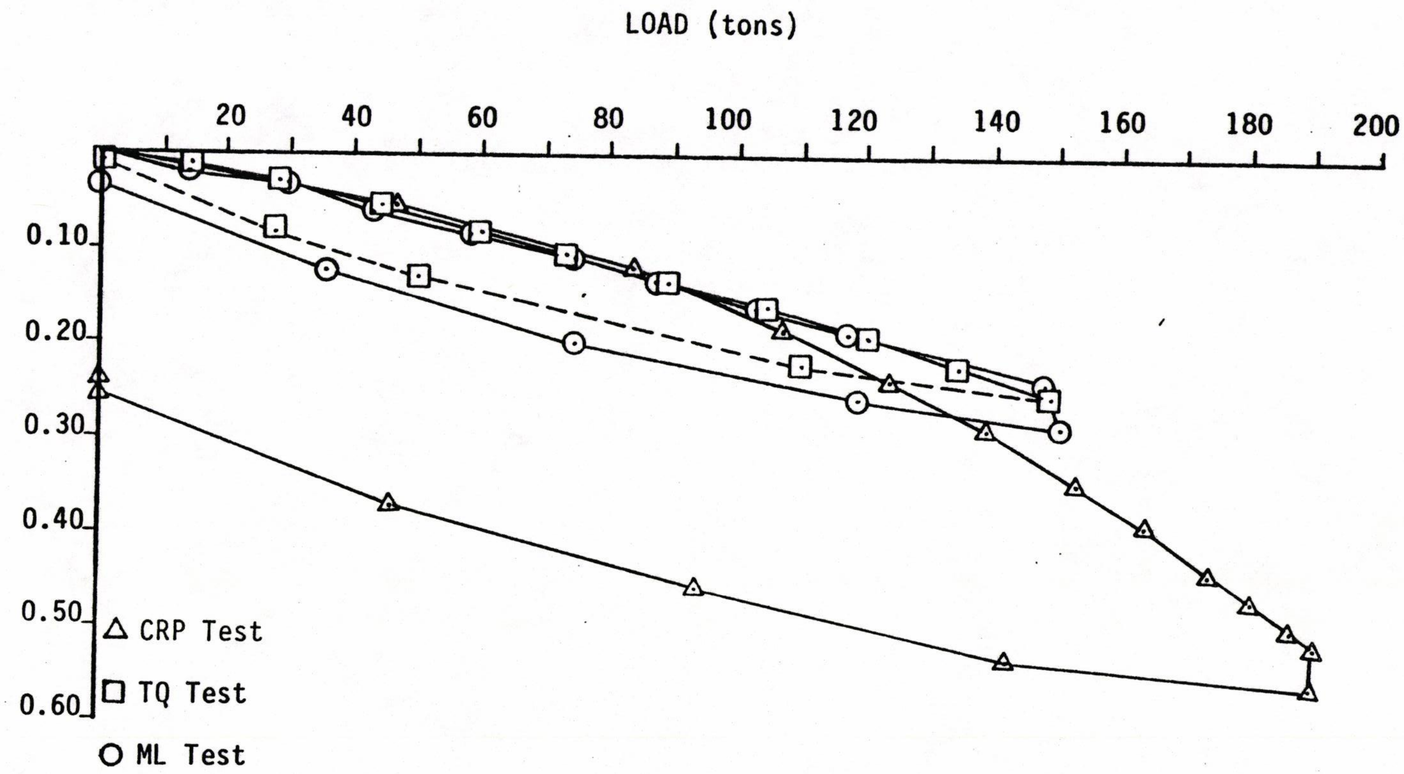


Figure 5.13 Load Settlement Curve fot Test Pile Number 3

produced a greater gross and net settlement for all test piles than the settlements produce by the TQ and ML tests. The magnitude of the settlements produced by the CRP test may be because the CRP test was the first load test performed on all test piles. The reaction piles were driven following the test piles, thus producing some uplift of the test piles and a reseating of the test piles occurred during the CRP test resulting in the large net settlement as compared to little or no net settlement for the TQ and ML tests. A second CRP test was performed on Test Pile 2 following the TQ and ML tests and the load-settlement curve was essentially the same as those of the TQ and ML tests.

Pile Capacity Predictions. The ultimate load carrying capacity of the piles at the Smackover test sites exceeded the capacity of the hydraulic ram used, so a comparison of actual capacity to predicted capacity is not possible. A tabulation of pile capacity predicted by several methods and the maximum load applied to the piles is given in Table 5.2. The dynamic formula values given for Test Pile 2 are based upon data taken during a restrike of the pile after the load tests were completed.

Load Transfer Behavior. Strain transducers were installed in Test Pile 2 after it was driven. Ten transducer pairs were spaced at intervals of four feet with the bottom set located one foot above the pile tip. The top set was two feet above the ground surface. A broken wire in the set placed nine feet above the pile tip resulted in no data from that location.

TABLE 5.2 Predicted and Measured Pile Capacities for Smackover Test Piles

Method Used	Ultimate Pile Capacity (tons) Test Pile 1	Test Pile 2
Load Tests*		
ML	190+	150+
TQ	190+	190+
CRP	190+	190+
Engineering News Formula	667	674
Danish Formula	330	238
Hiley Formula	135	189
Wave Equation	220	156
CWR Device	-	216
Limit Equilibrium (Based on Soil properties)	307	197

* The load capacity of the hydraulic ram was not sufficient to cause failure of either pile. Loads shown are the maximum loads applied.

The strain transducer output readings were used to determine the loads in the test pile at the transducer locations. The top strain transducer, placed approximately two feet above the ground surface, provided a check on the load indicated by pressure measurements in the hydraulic ram. The difference in load in the pile at any two points is the load transferred to the soil by skin friction between those two points. Since the surface area of the pile is known, the average value of skin friction may be computed. If the distribution of skin friction is linear with depth for this interval, then the average skin friction is the value of skin friction at the middle of the interval. The deformation of the pile between transducer levels may be computed by using the average strain in the pile for this interval. The displacement of any point on the pile may be determined by subtracting the cumulative deformation of the pile above that point from the measured displacement of the top of the pile. By computing skin friction and deformation values for several loads, a plot of skin friction versus deformation may be made for a point on the pile. The load distribution in Test Pile 2 for the CRP, TQ, and ML tests is shown in Figures 5.14, 5.15, and 5.16. The variation of skin friction with depth for the three test procedures is shown in Figures 5.17, 5.18, and 5.19. Figures 5.20, 5.21, and 5.22 show the development of skin friction as a function of pile displacement.

Redfield Tests

Load tests were conducted on three piles at grade separation structures on US 65 near Redfield in Jefferson County, Arkansas.

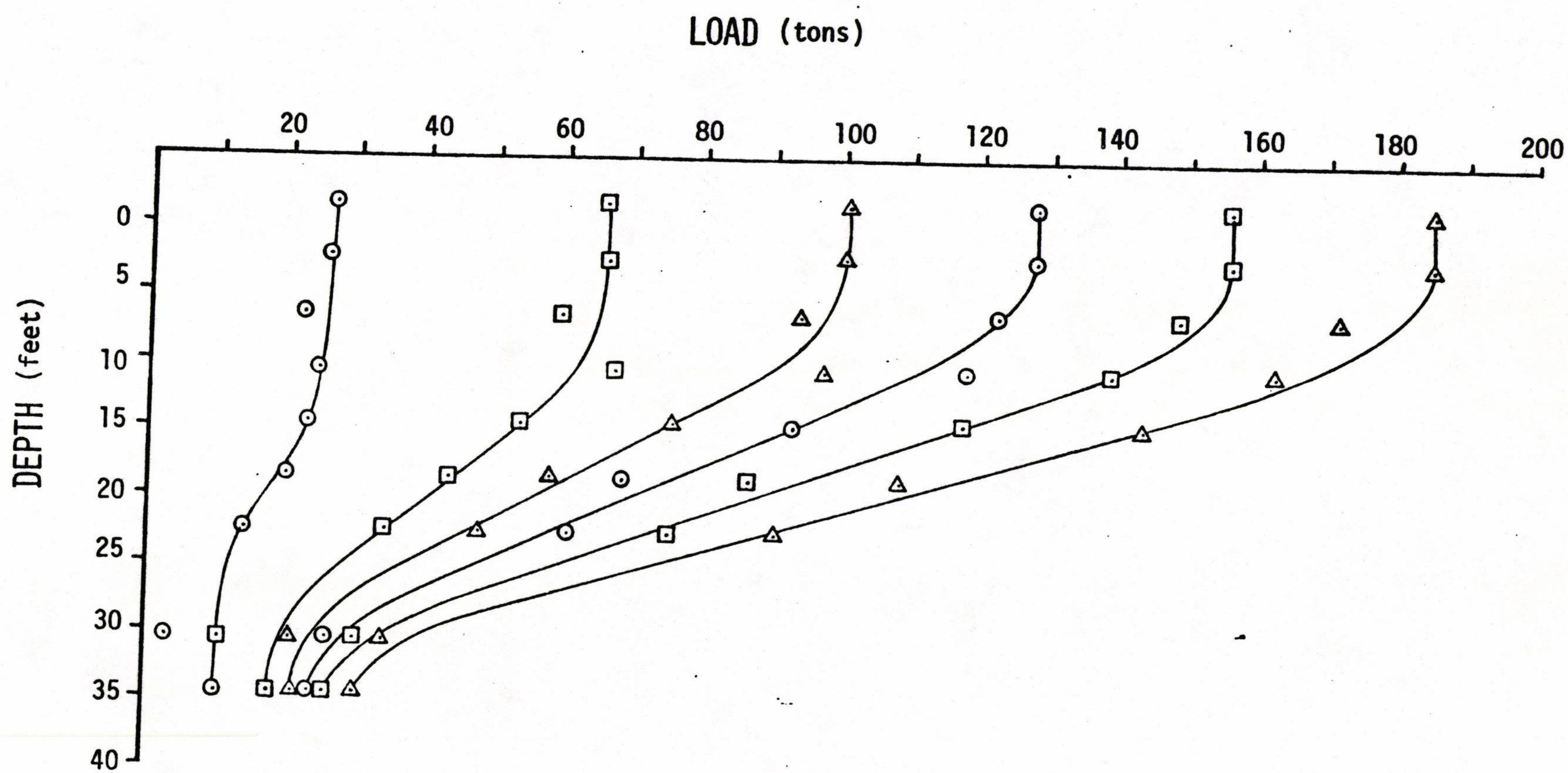


Figure 5.14 - Measured load in Test Pile 2 at various depths for CRP test

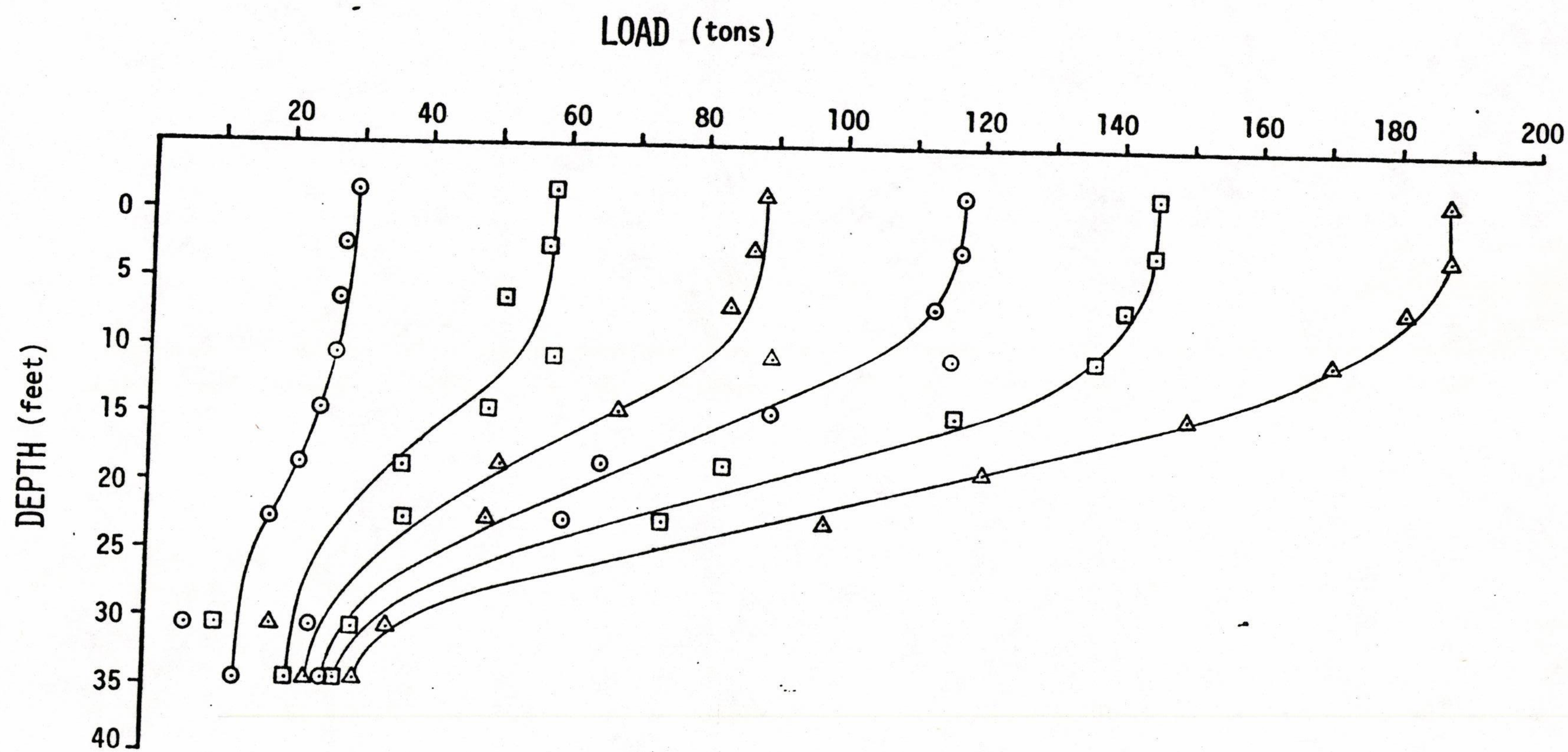


Figure 5.15 - Measured load in Test Pile 2 at various depths for Quick Test

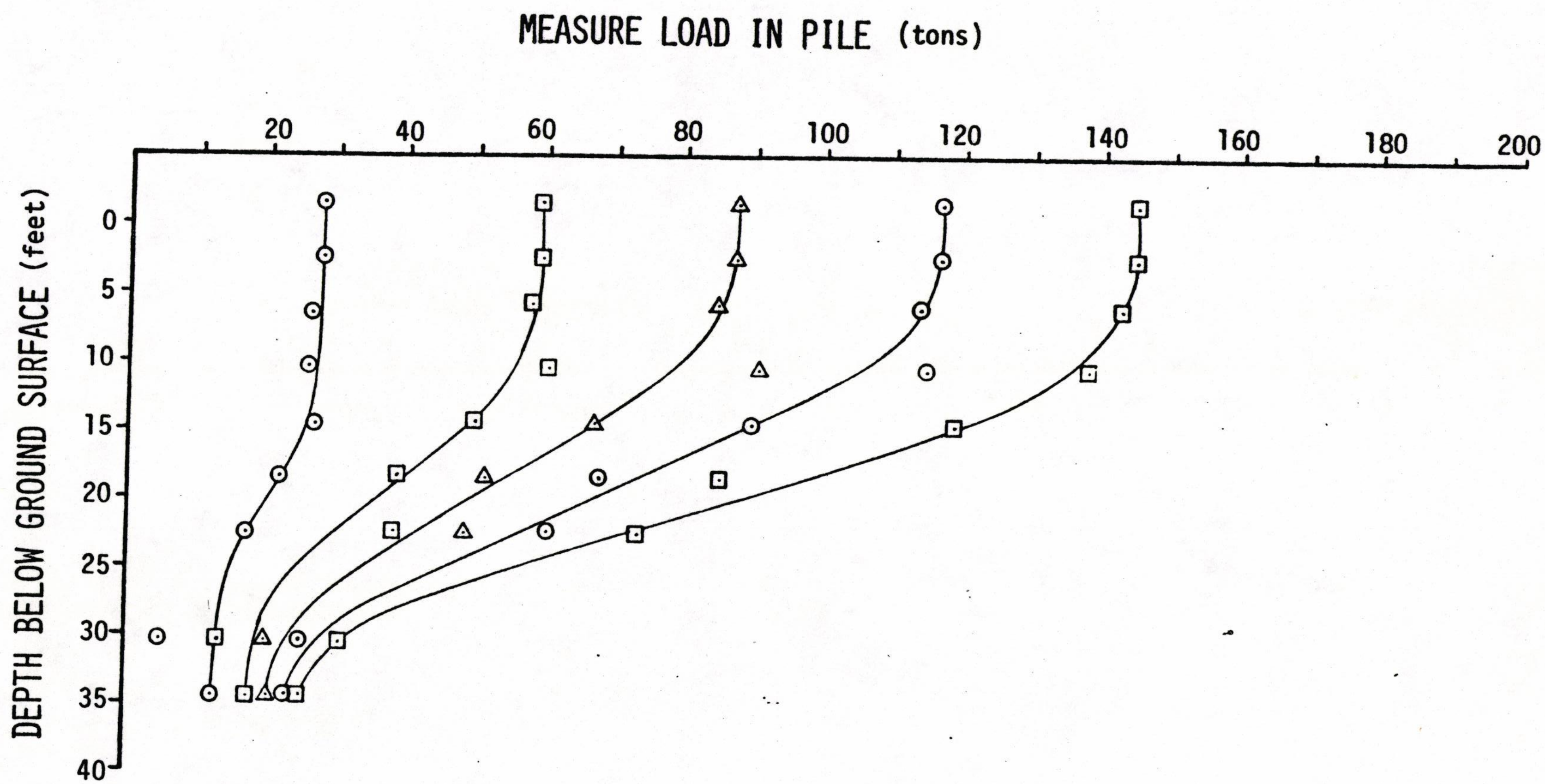


Figure 5.16 - Measured load in Test Pile 2 at various depths for Maintained Load Test

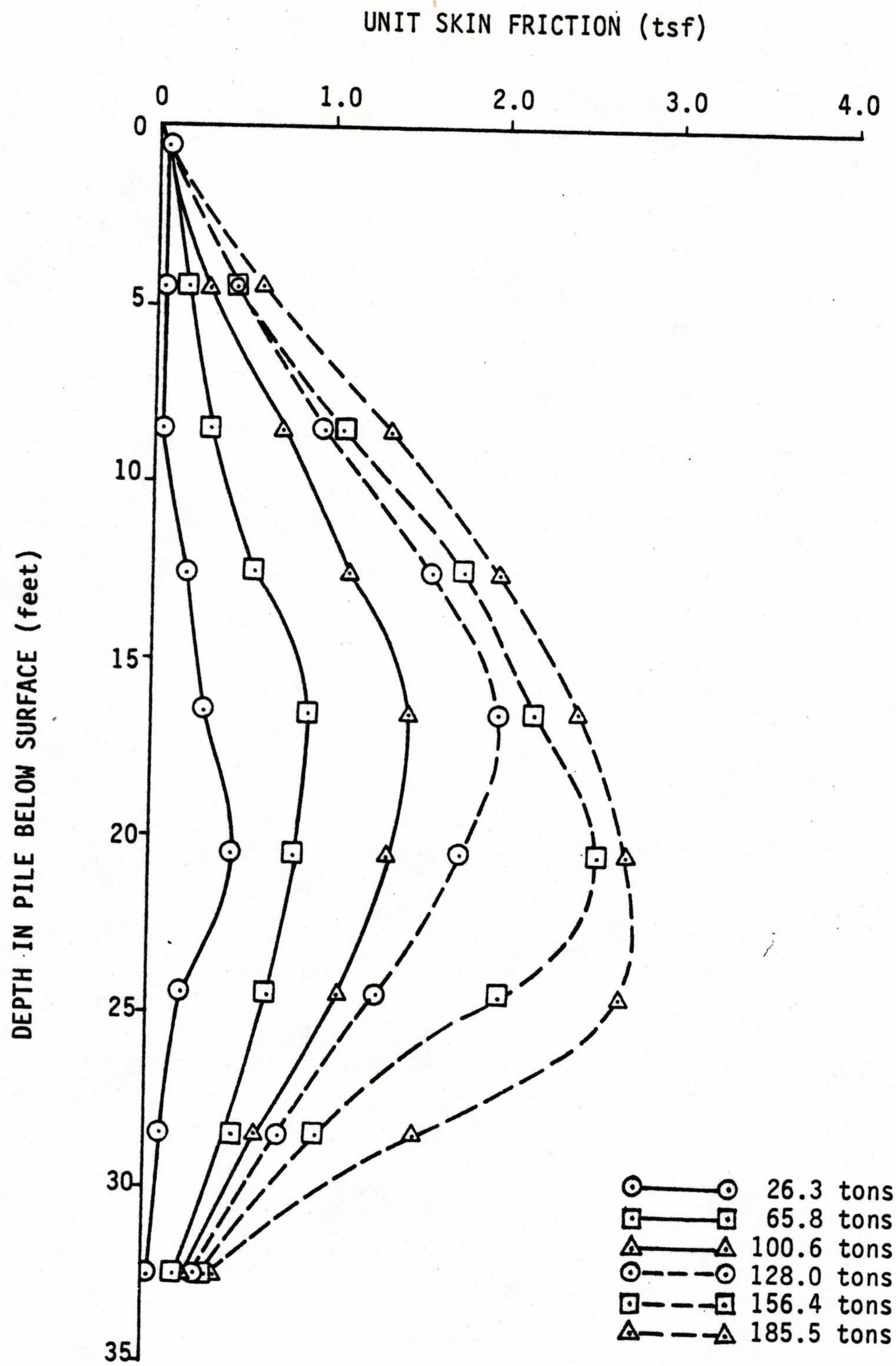


Figure 5.17 Unit Skin Friction versus Depth Curves for Various Loadings during the CRP Load Test on Test Pile 2

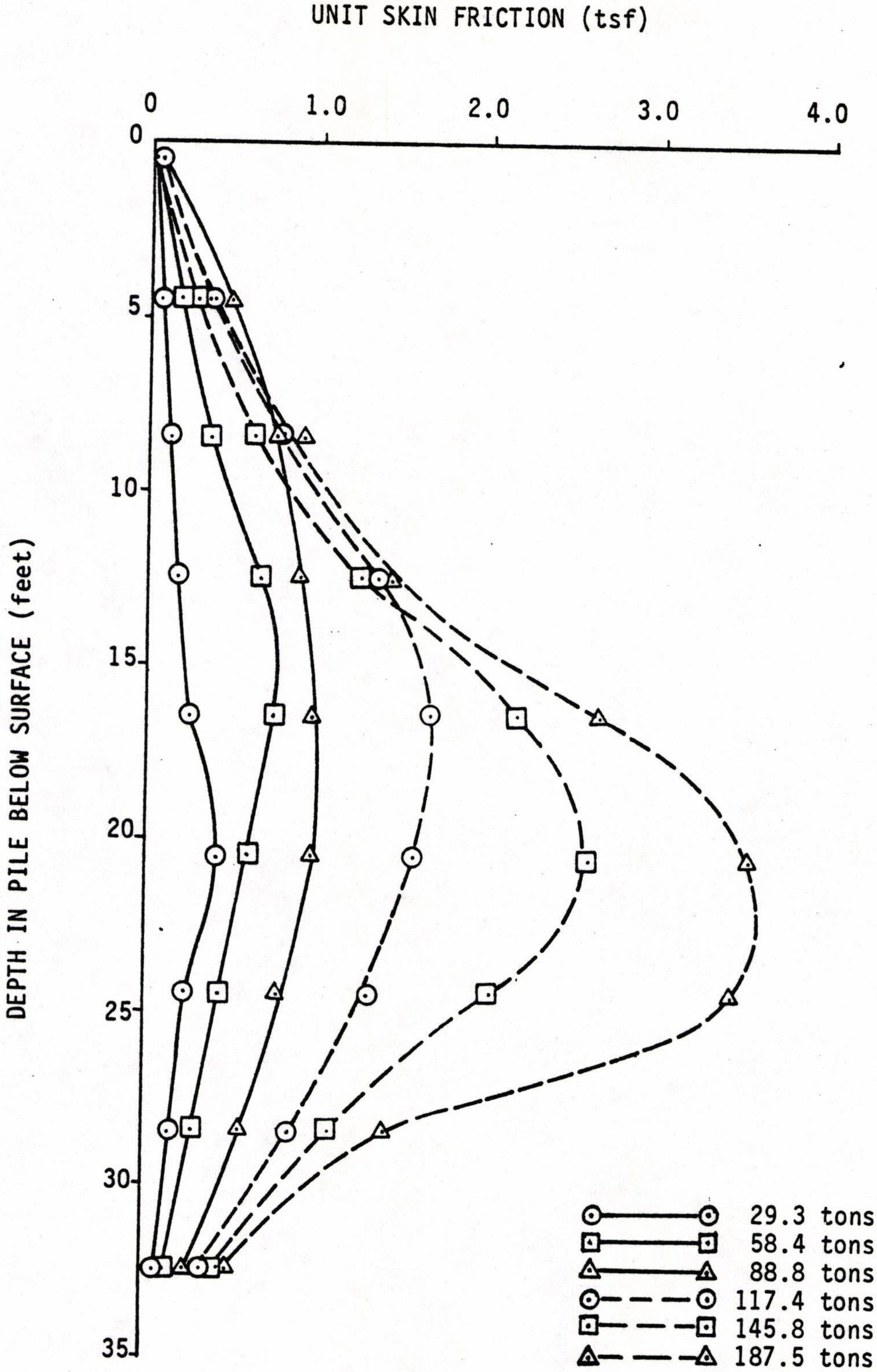


Figure 5.18 Unit Skin Friction versus Depth Curves for Various Loadings during the Quick Load Test on Test Pile 2.

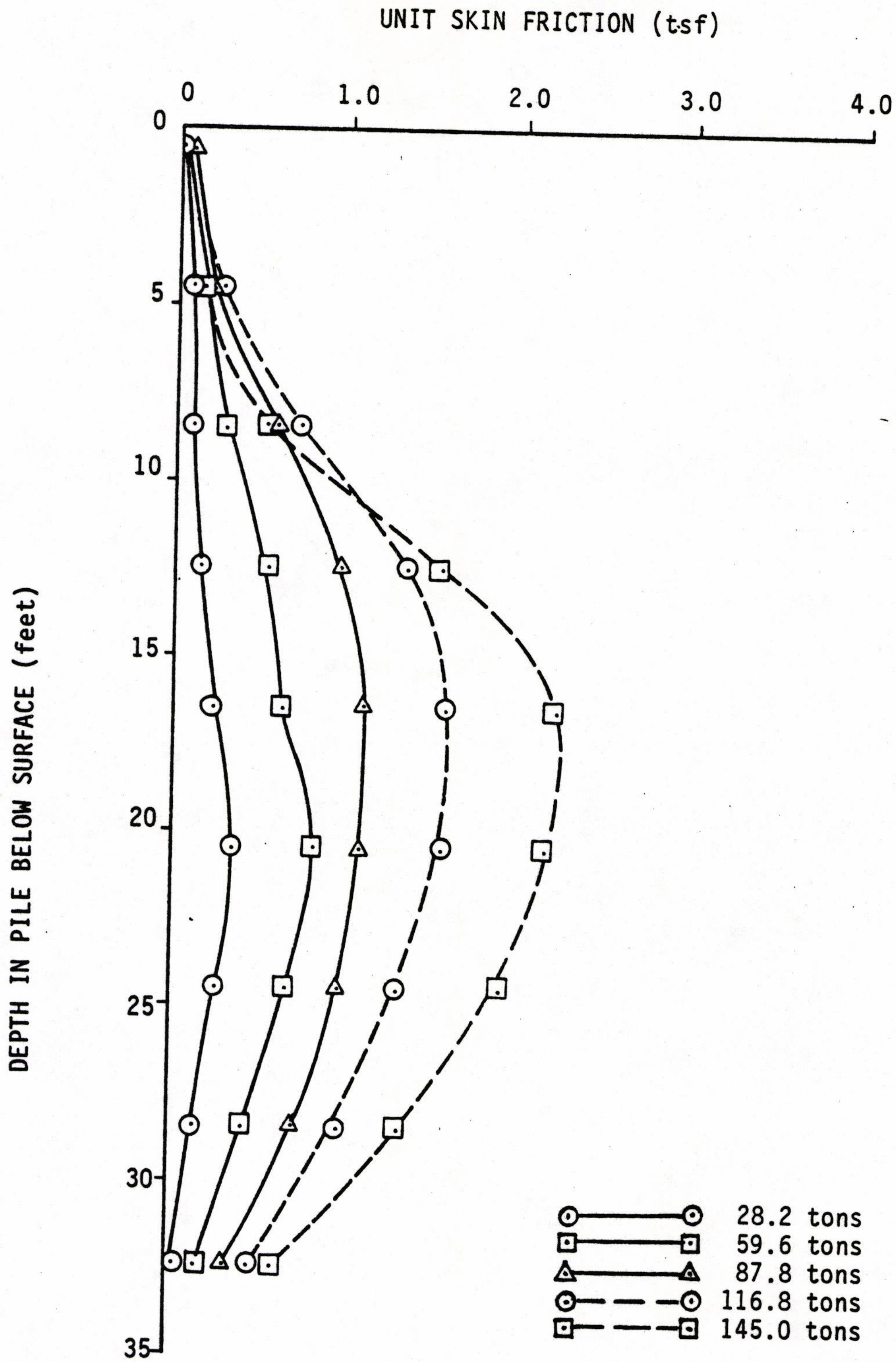


Figure 5.19 Unit Skin Friction versus Depth for Loads during the Maintained Load Test on Test Pile 2.

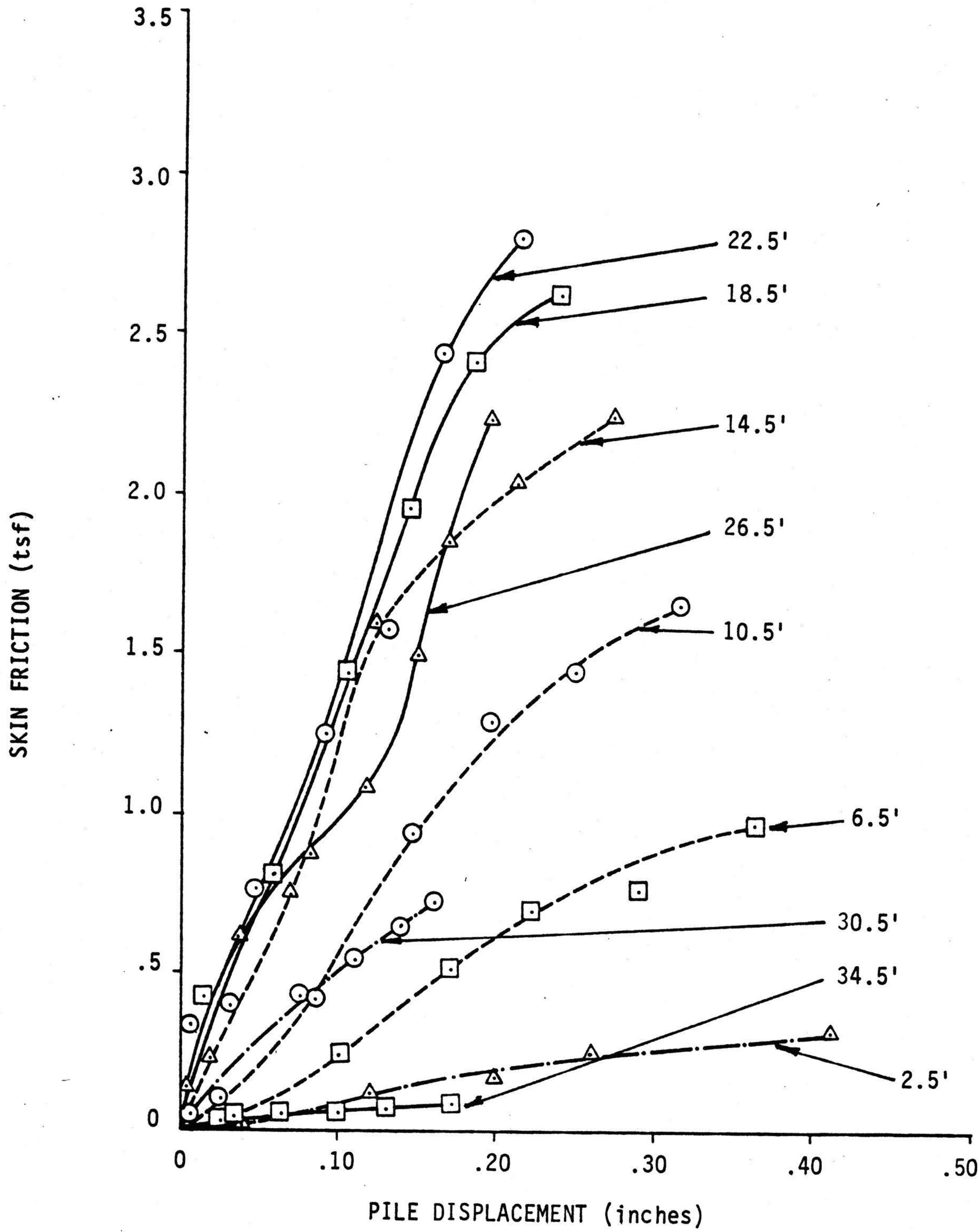


Figure 5.20 Skin Friction versus Pile Displacement at Various Depths during CRP Load Test on Test Pile 2.

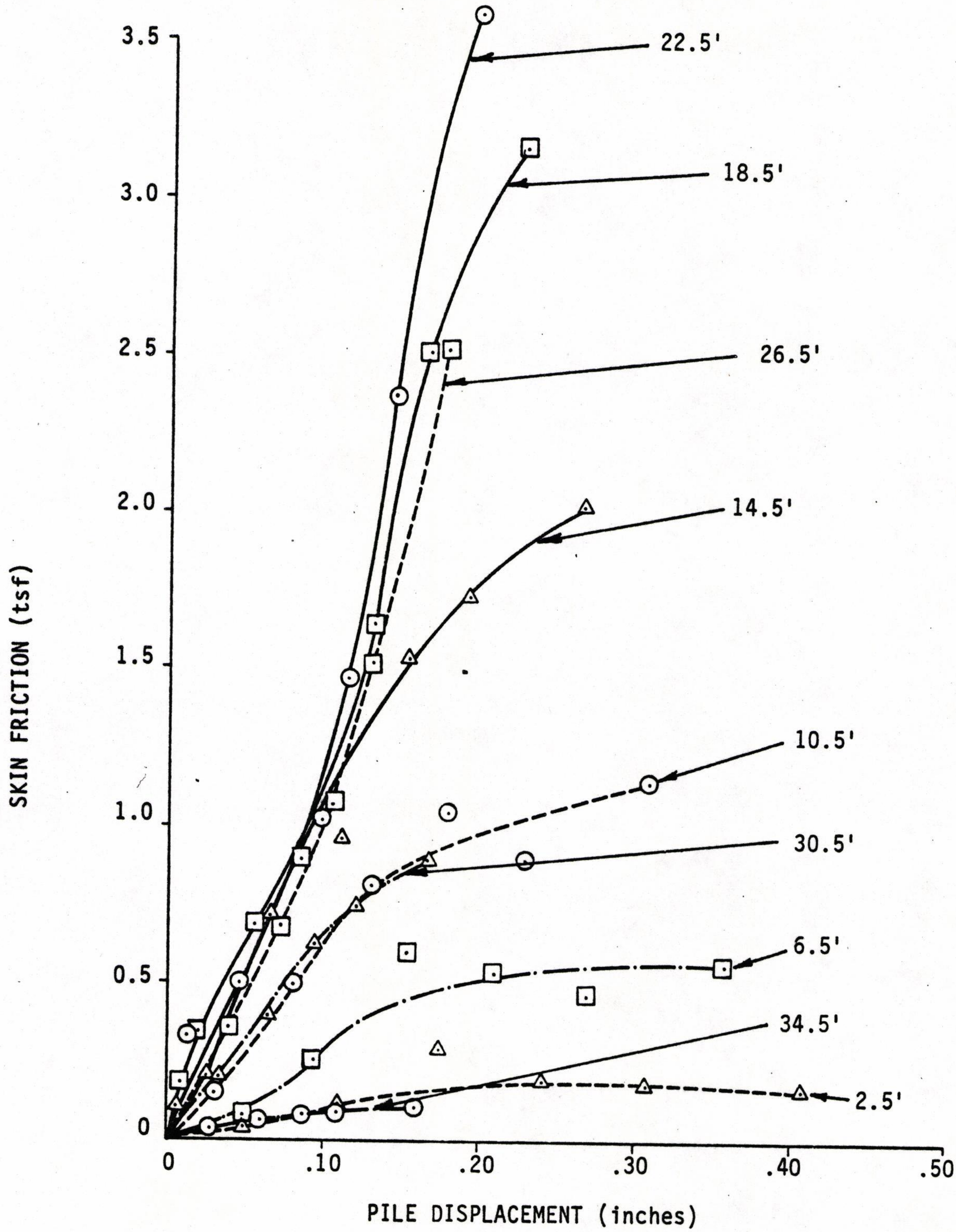


Figure 5.21 Skin Friction versus Pile Displacement at Various Depths during Quick Load Test on Test Pile 2.

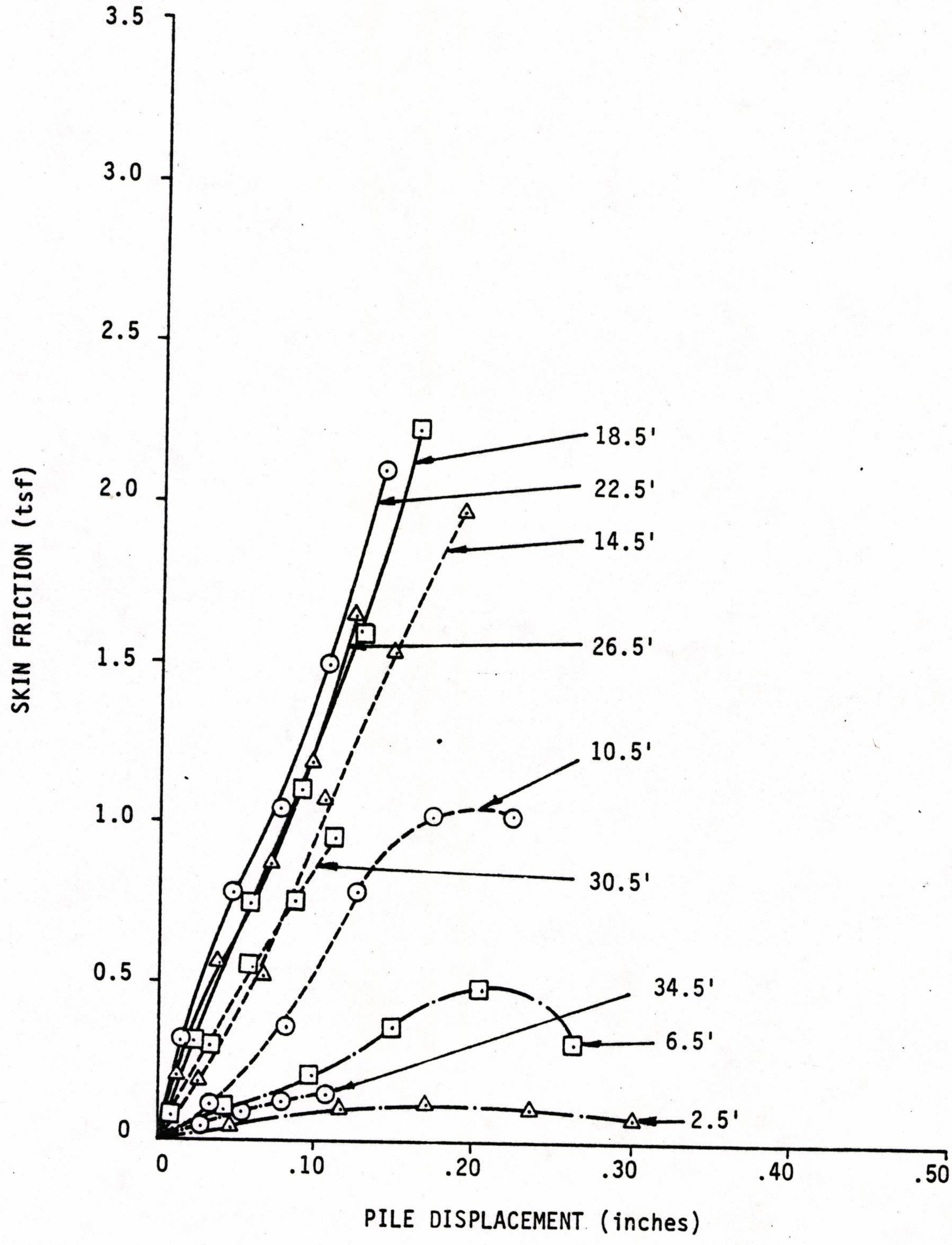


Figure 5.22 Skin Friction versus Pile Displacement at Various Depths during Maintained Load Test on Test Pile 2.

Test Piles 1 and 2, a 16 inch octagonal prestressed concrete pile and a 10-3/4 inch O.D. closed end pipe pile were driven at Redfield Interchange. Test Pile 3, also a 16 inch octagonal prestressed concrete pile, was driven at the Gravel Pit Road Underpass. The test procedures used were the CRP, TQ, and ML. The steel pile pile was instrumented so that measurements of load transfer could be made. The piles were 30 feet long and driven to a penetration of 18.5 feet.

Soil Conditions. Both sites are located in the Arkansas River valley and the soils present at both sites were predominantly stiff to very stiff clays. The borings were made by the Arkansas Highway Department and, in accordance with their standard procedure, standard split-spoon penetration tests were performed. Logs of the borings are given in Figures 5.23 and 5.24.

Load-Settlement Curves. The load-settlement curves for Test Piles 1, 2, and 3 are given in Figures 5.25, 5.26, and 5.27 respectively. The sequence of tests for these piles was CRP, TQ, and ML.

For the Redfield concrete pile, the ML test had the highest failure load of 69.5 tons while the TQ test gave the lowest of 63.5 tons. The variation from the average of the three types of load tests is ±5 percent.

The ultimate loads on the Redfield pipe pile were within 0.3 percent. The CRP test and the ML test gave an ultimate load 0.15 tons higher than the value given by the TQ test.

The results of the loading tests on the concrete pile at Gravel Pit Road were within 3 percent of each other with the TQ test

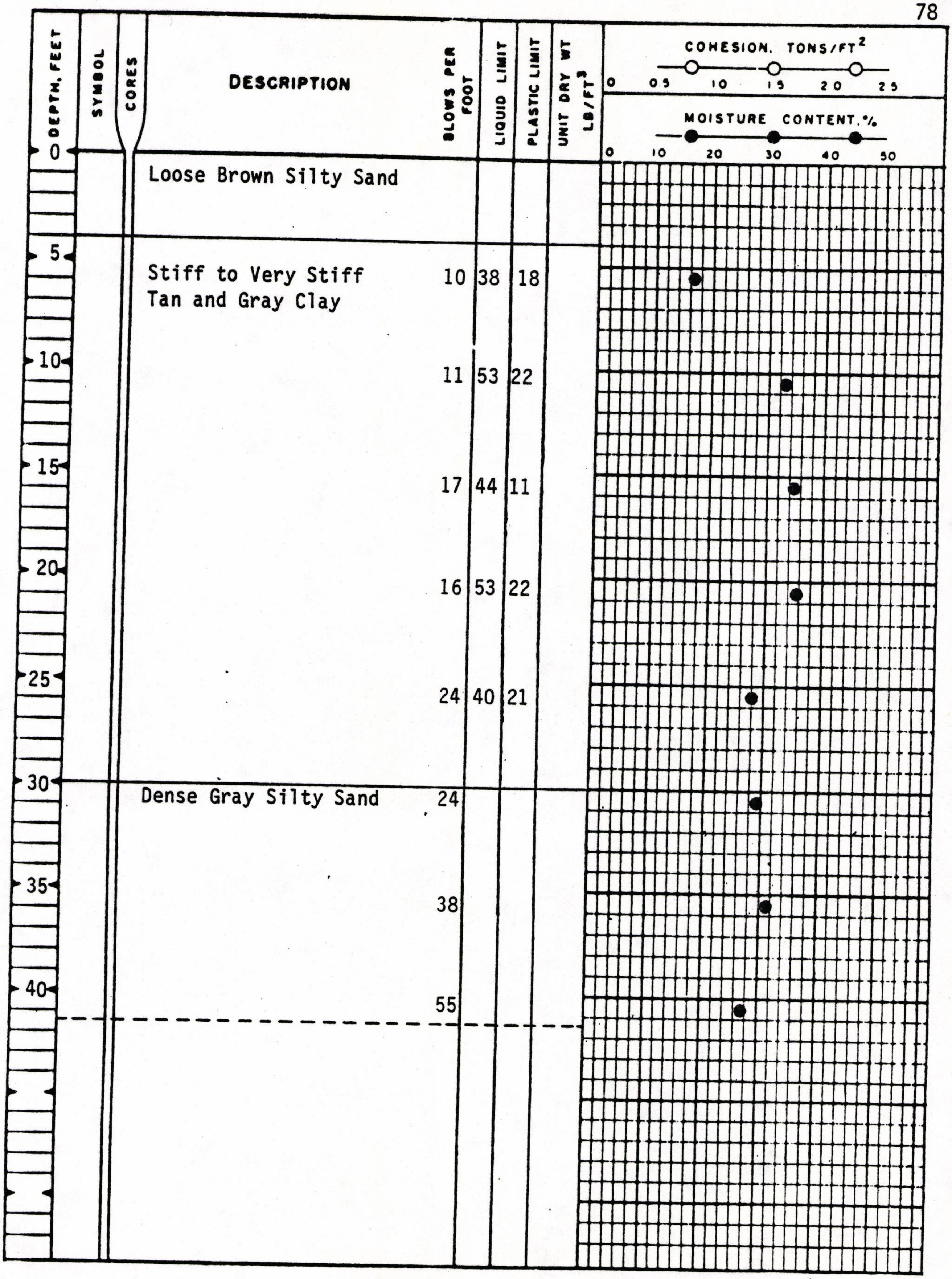


Figure 5.23 Soil Profile - Redfield Site

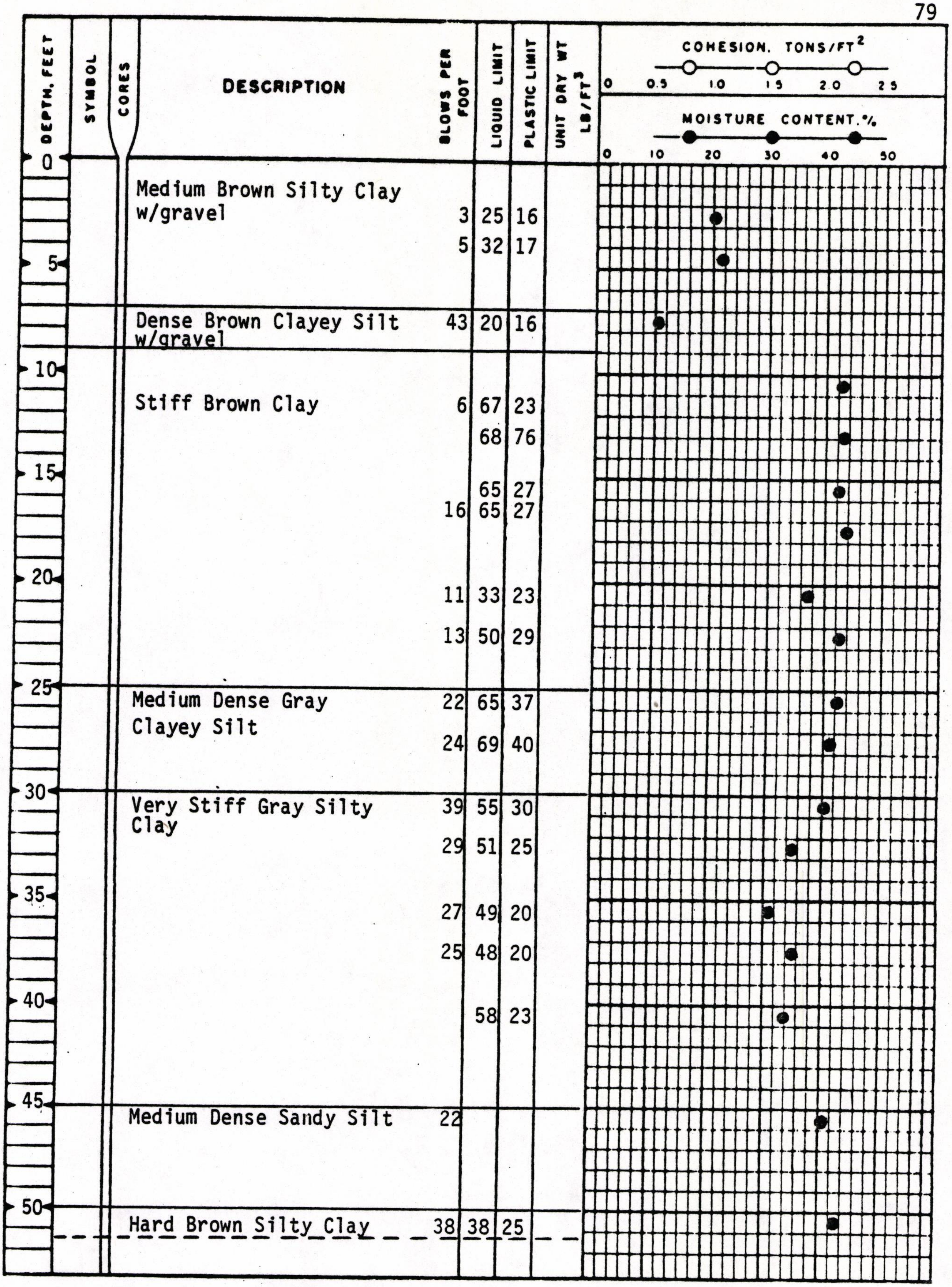


Figure 5.24 Soil Profile - Gravel Pit Road Site

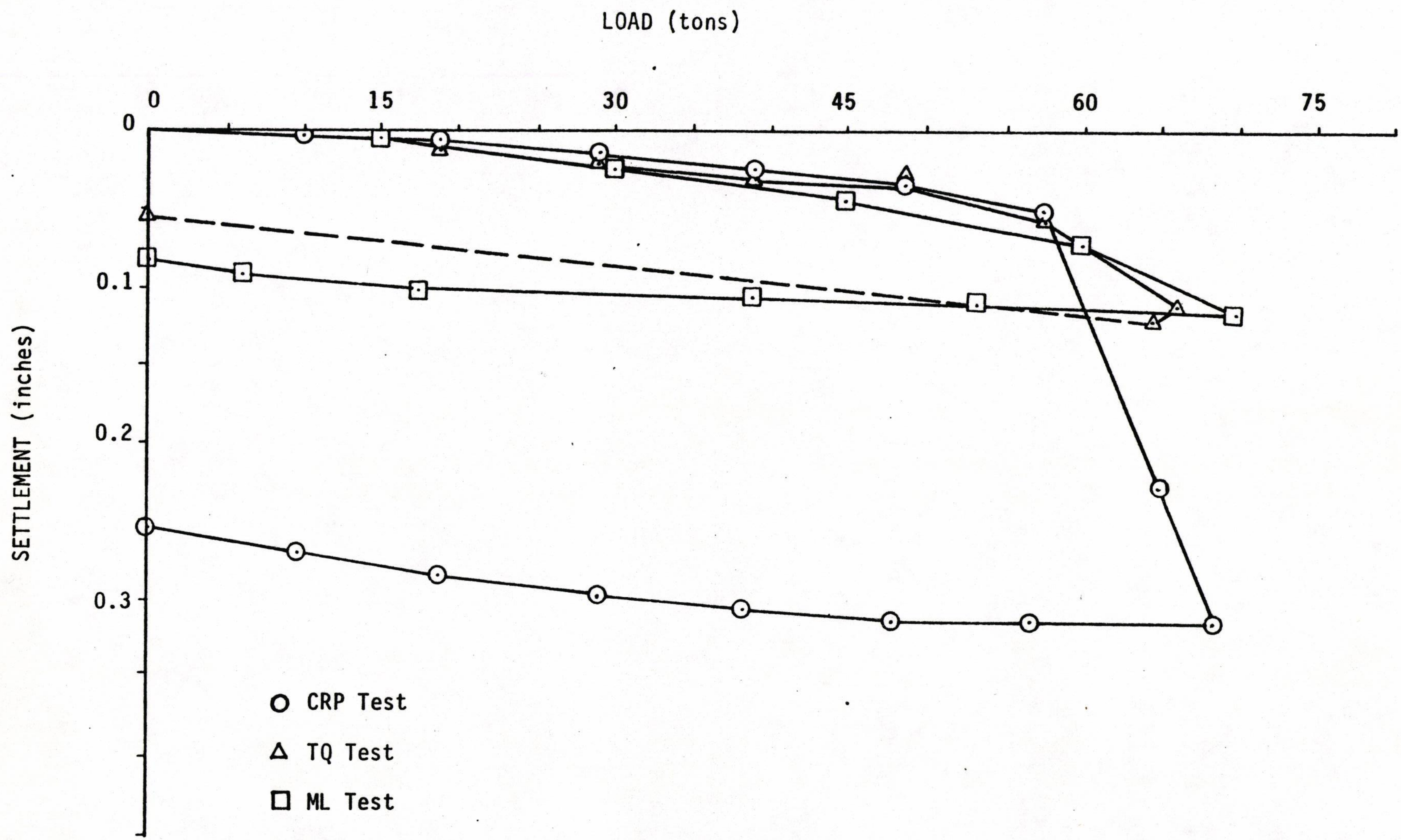


Figure 5.25 Load - Settlement Relationship of Redfield Concrete Pile

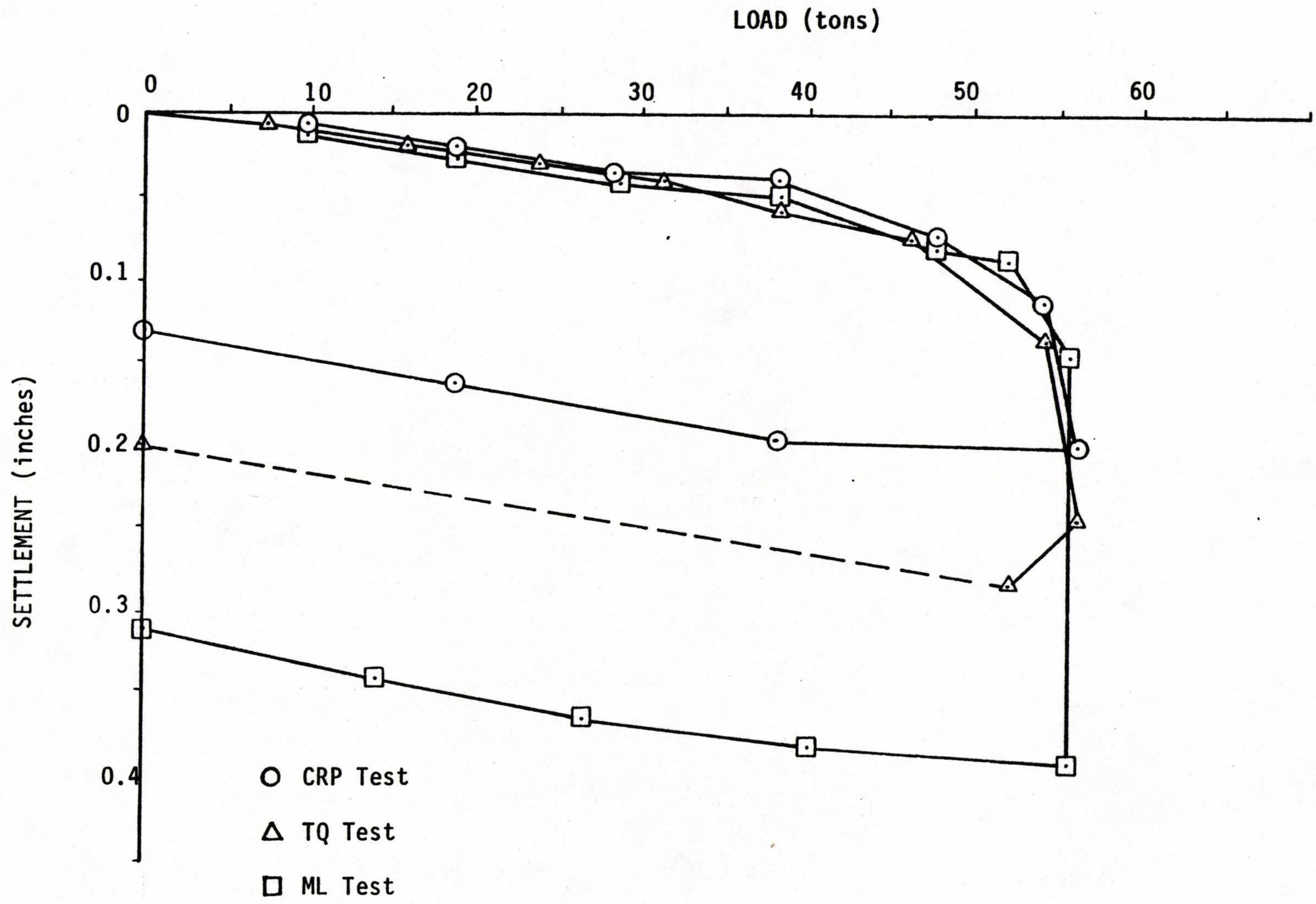


Figure 5.26 Load - Settlement Relationship of Redfield Pipe Pile

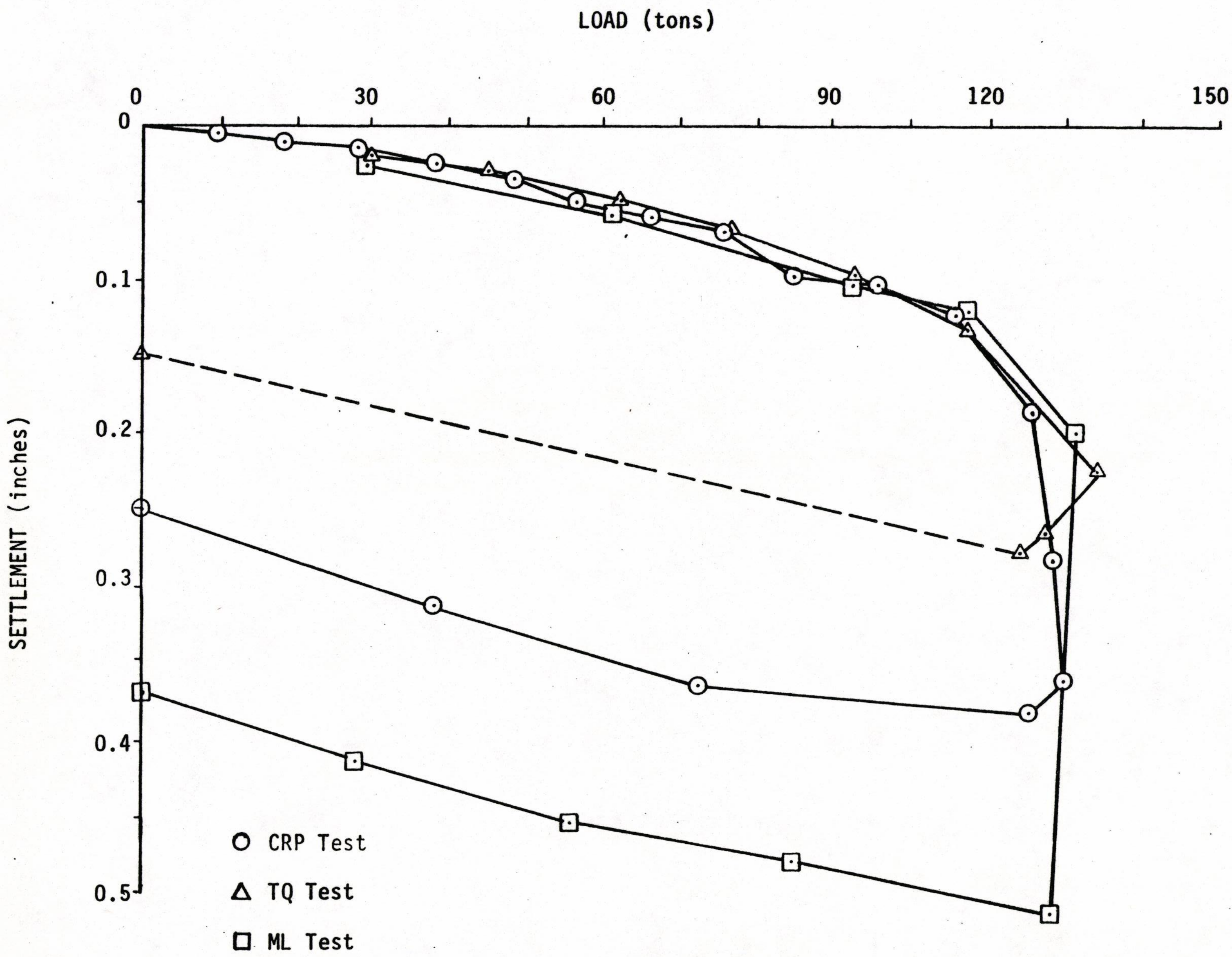


Figure 5.27 Load - Settlement Relationship of Gravel Pit Road Concrete Pile

giving the highest value and the CRP test, the lowest. The load-settlement curves for the 3 pile load tests approximate each other. The top settlements are within .02 of an inch of each other until failure. The variation of the results is small and not significant.

Pile Capacity Predictions. The ultimate load carrying capacity of the test piles as predicted by several methods is compared to the measured values in Table 5.3. The lack of reliable data on the shear strength of the soils was a handicap in applying the limit equilibrium method. The only data available for the Redfield and Gravel Pit Road sites were standard penetration test results and these were used to estimate shear strength. Correlation of shear strength of cohesive soils with standard penetration resistance is often unreliable and better estimates of pile capacity would have been obtained if shear tests on undisturbed samples had been performed.

Load Transfer Behavior. Strain transducer pairs were installed in Test Pile 2 after it was driven. Ten transducer pairs were spaced at intervals of two feet with the bottom pair located one foot above the pile tip. The top pair was two and one-half feet above the ground surface. A broken wire in one of the transducers at the third level above the tip required that the remaining transducer be connected as a half bridge. The load distribution in Test Pile 2 for the CRO, TQ, and ML tests is shown in Figures 5.28, 5.29, and 5.30. A comparison of load distribution as a function of test procedure is shown in Figure 5.31. The variation of skin friction with depth for the three test procedures is shown in Figures 5.32, 5.33, and 5.34. Figures 5.35, 5.36, and 5.37 show the development of skin friction as a function of displacement.

TABLE 5.3 Predicted and Measured Pile Capacities for Redfield Test Piles

Method Used	Ultimate Pile Capacity (tons)		
	Test Pile 1	Test Pile 2	Test Pile 3
Load Tests			
ML	69.5	55.8	121.3
TQ	63.5	55.6	123.2
CRP	65.5	55.8	119.4
Engineering News Formula	616.	300.	496.
Danish Formula	361.	104.	355.
Hiley Formula	68	52.	74.
Wave Equation	70	61.	100.
CWR Device	69.6	60.9	-
Limit Equilibrium (Based on soil properties)	63.0	39.5	75.5

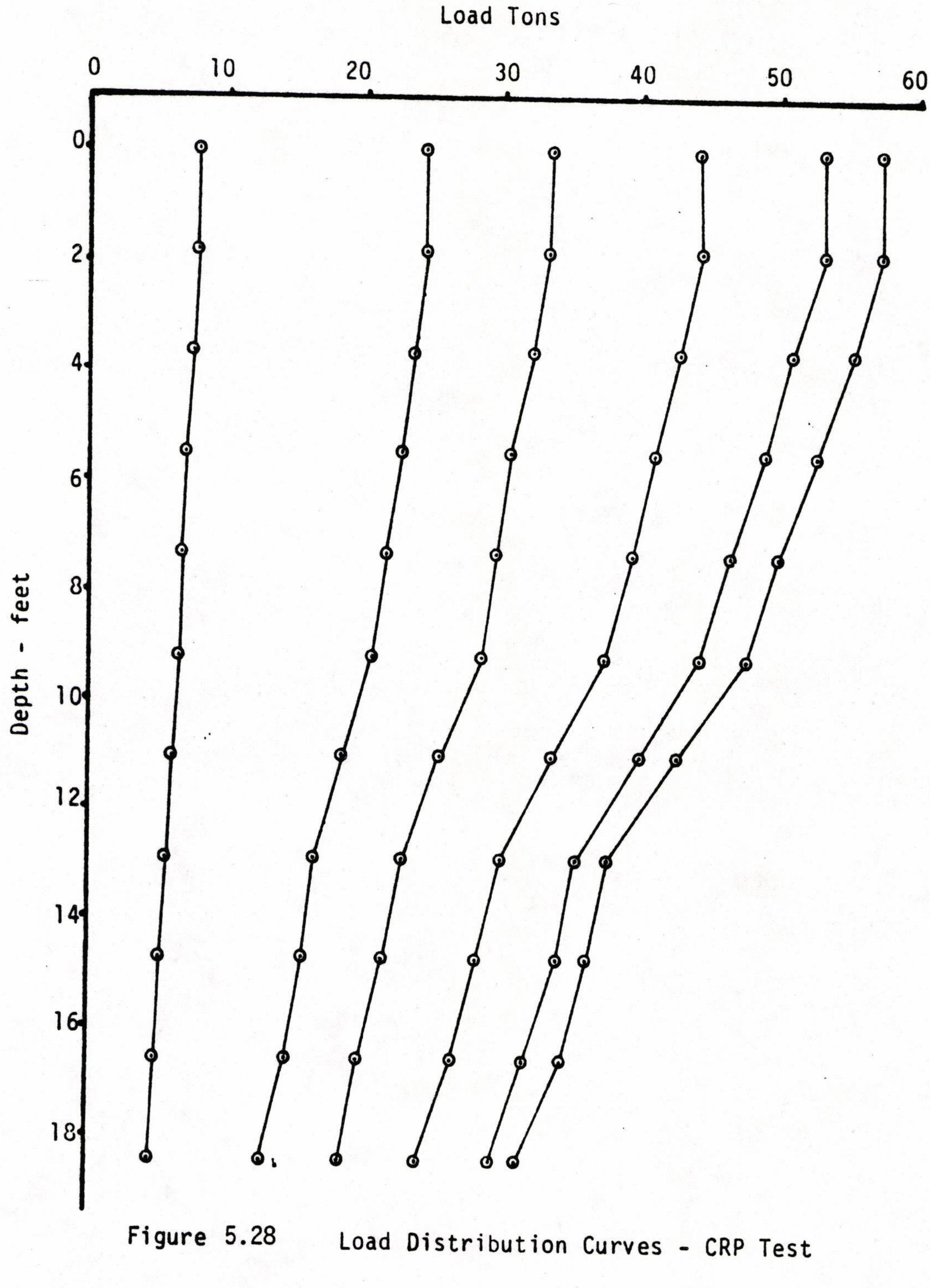


Figure 5.28 Load Distribution Curves - CRP Test

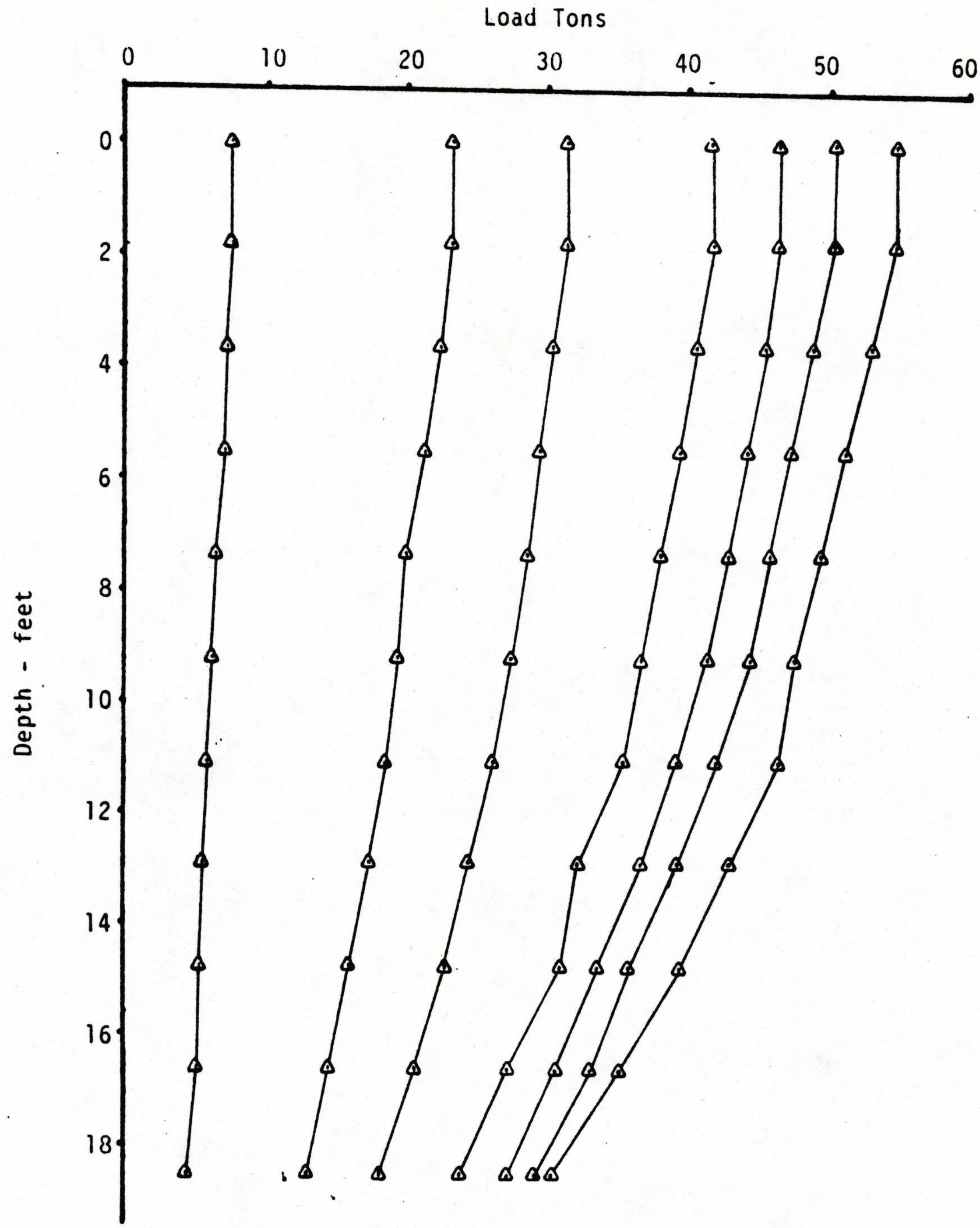


Figure 5.29 Load Distribution Curves-TQ Test

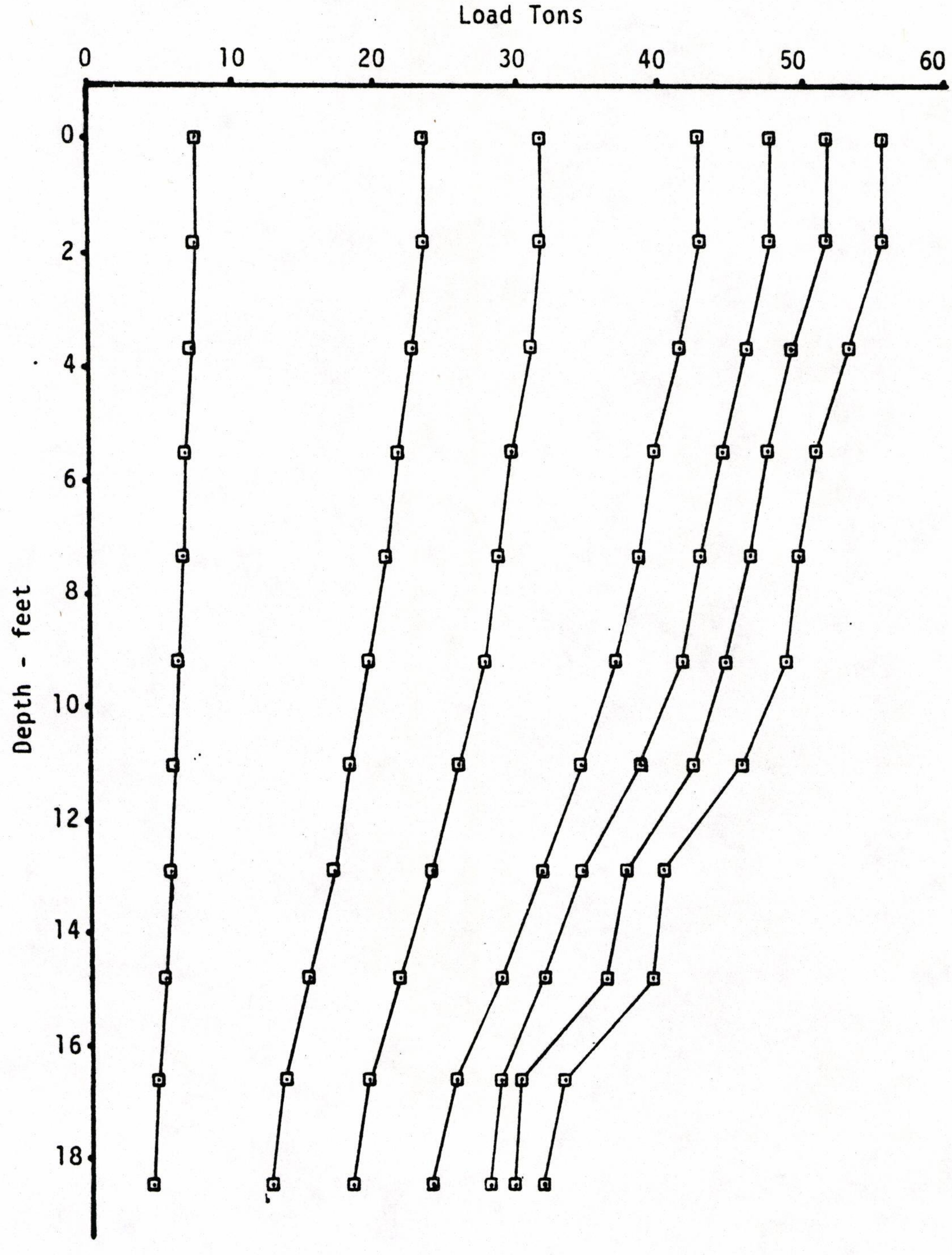


Figure 5.30 Load Distribution Curves - ML Test

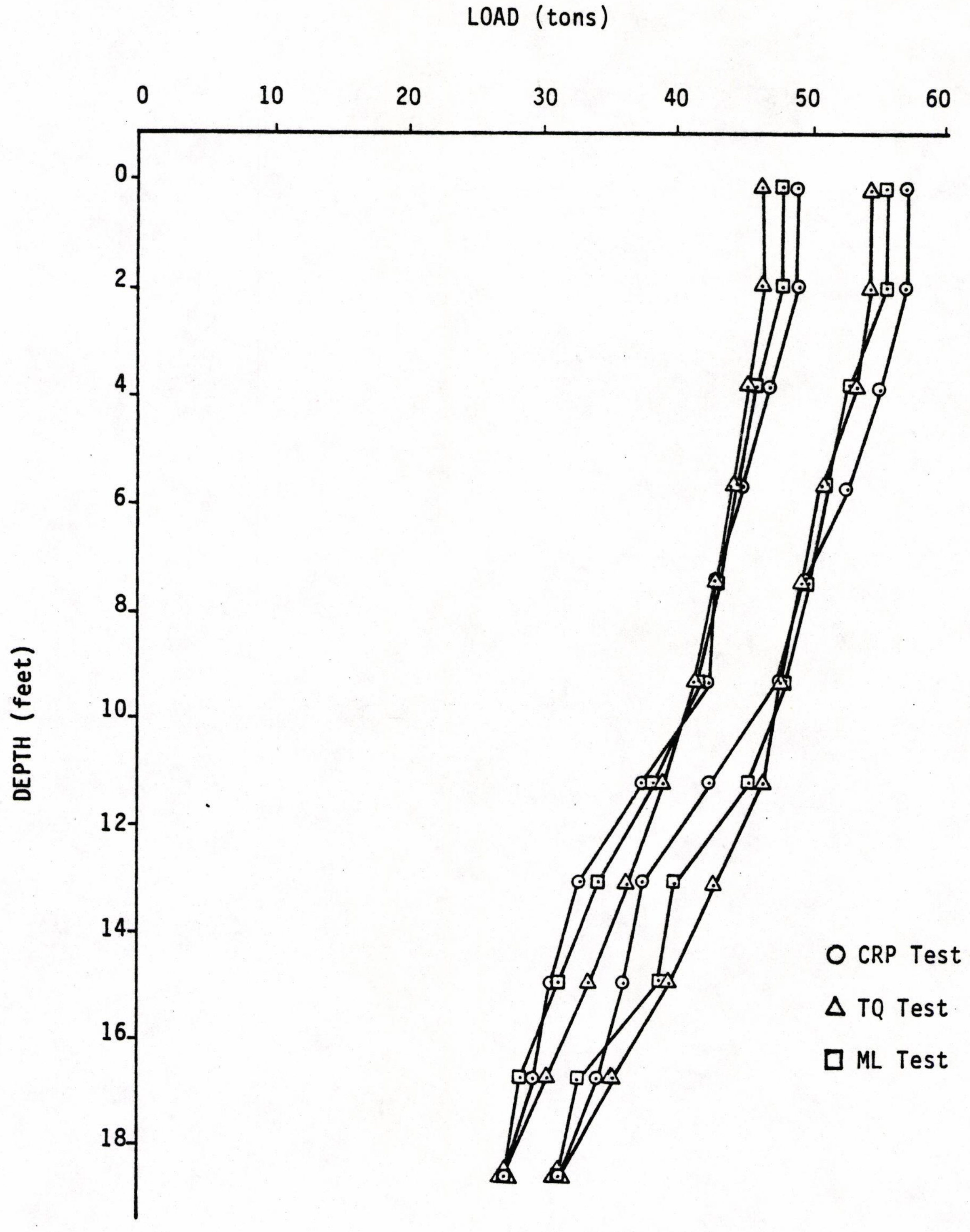


Figure 5.31 Comparison of Load in the Pile vs. Depth for the Different Tests.

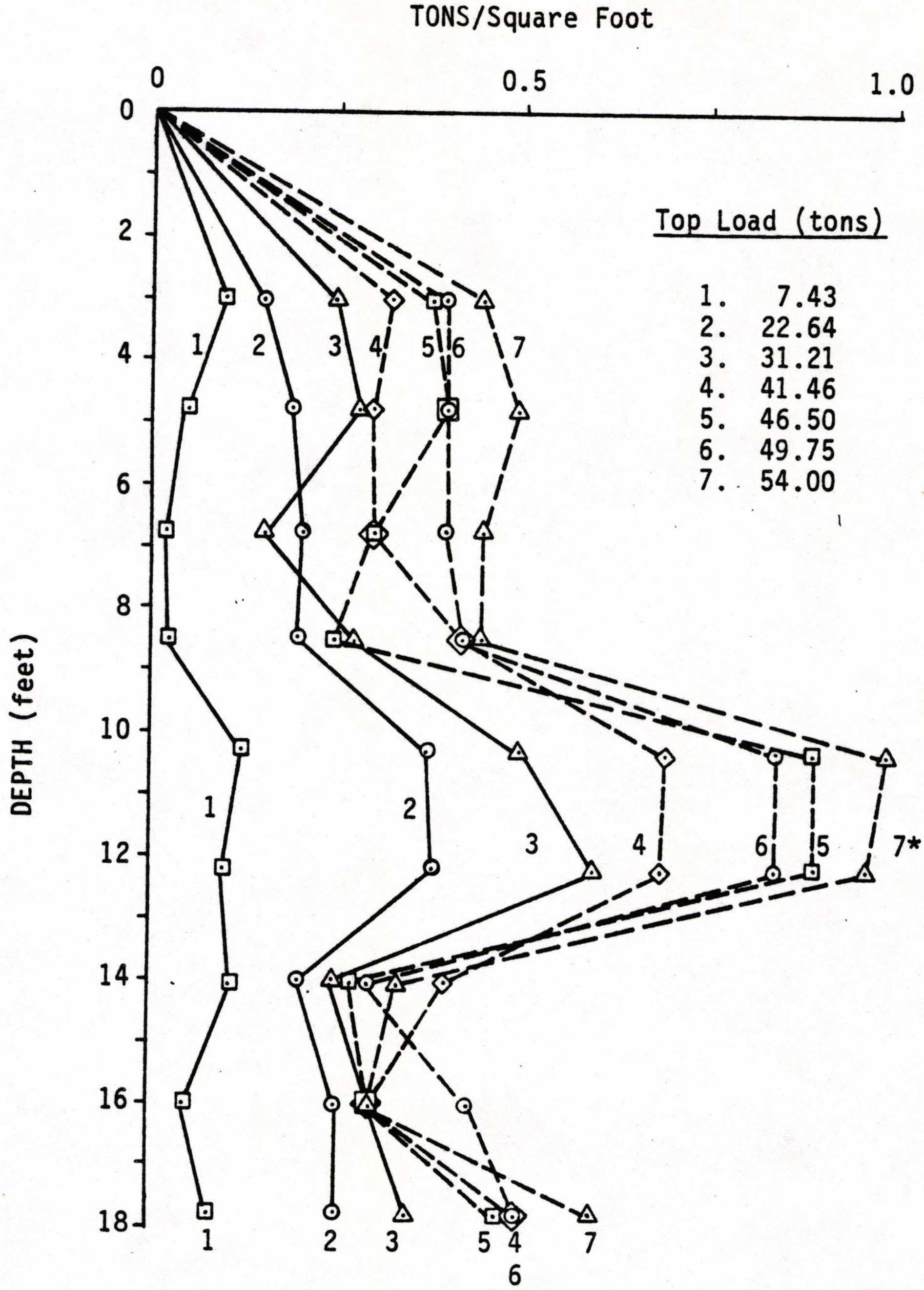


Figure 5.32 Skin Friction vs. Depth Curves from CRP Test

* Numbers refer to corresponding top loads.

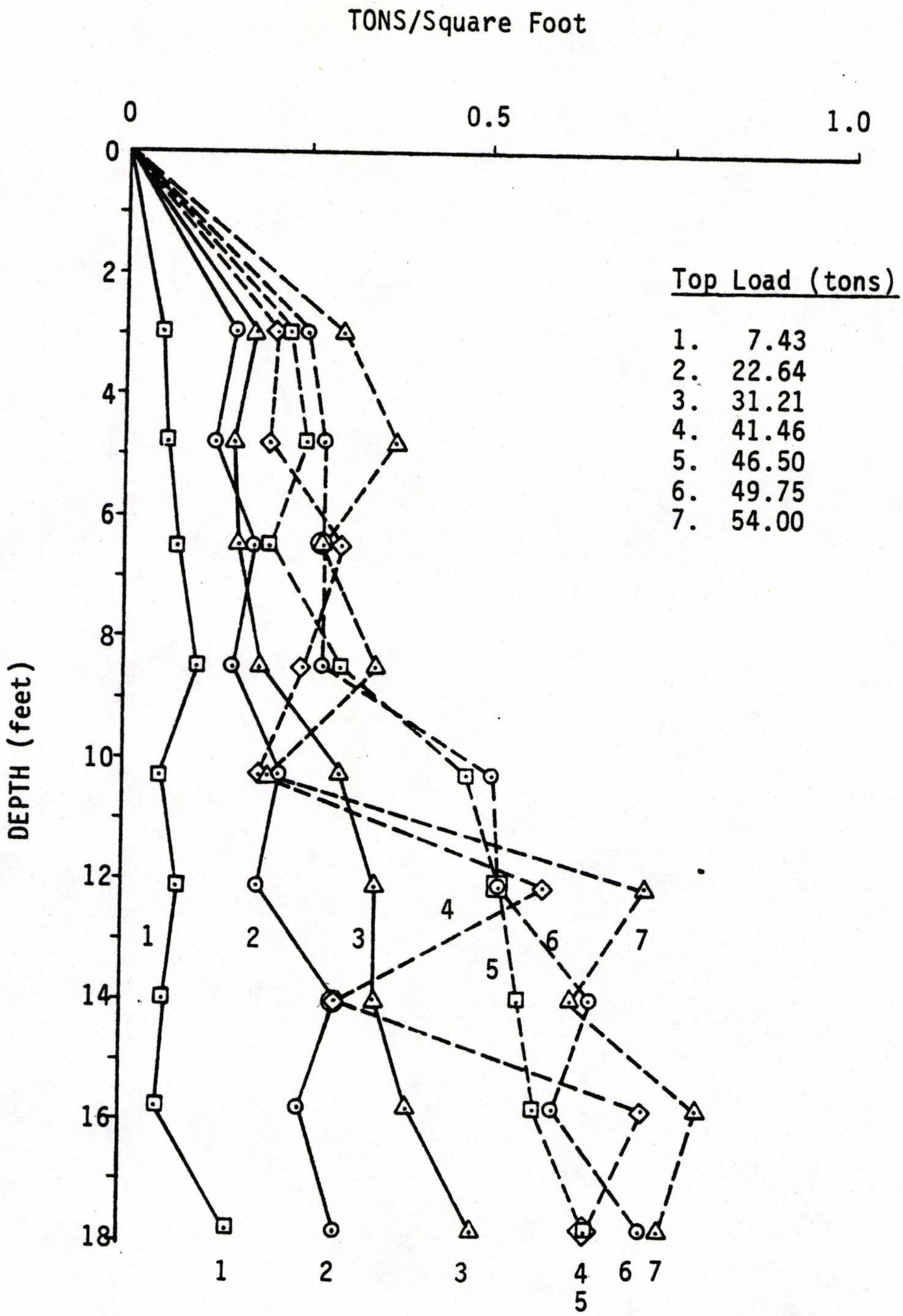


Figure 5.33 Skin Friction vs. Depth Curves from TQ Tests

* Numbers refer to corresponding top loads.

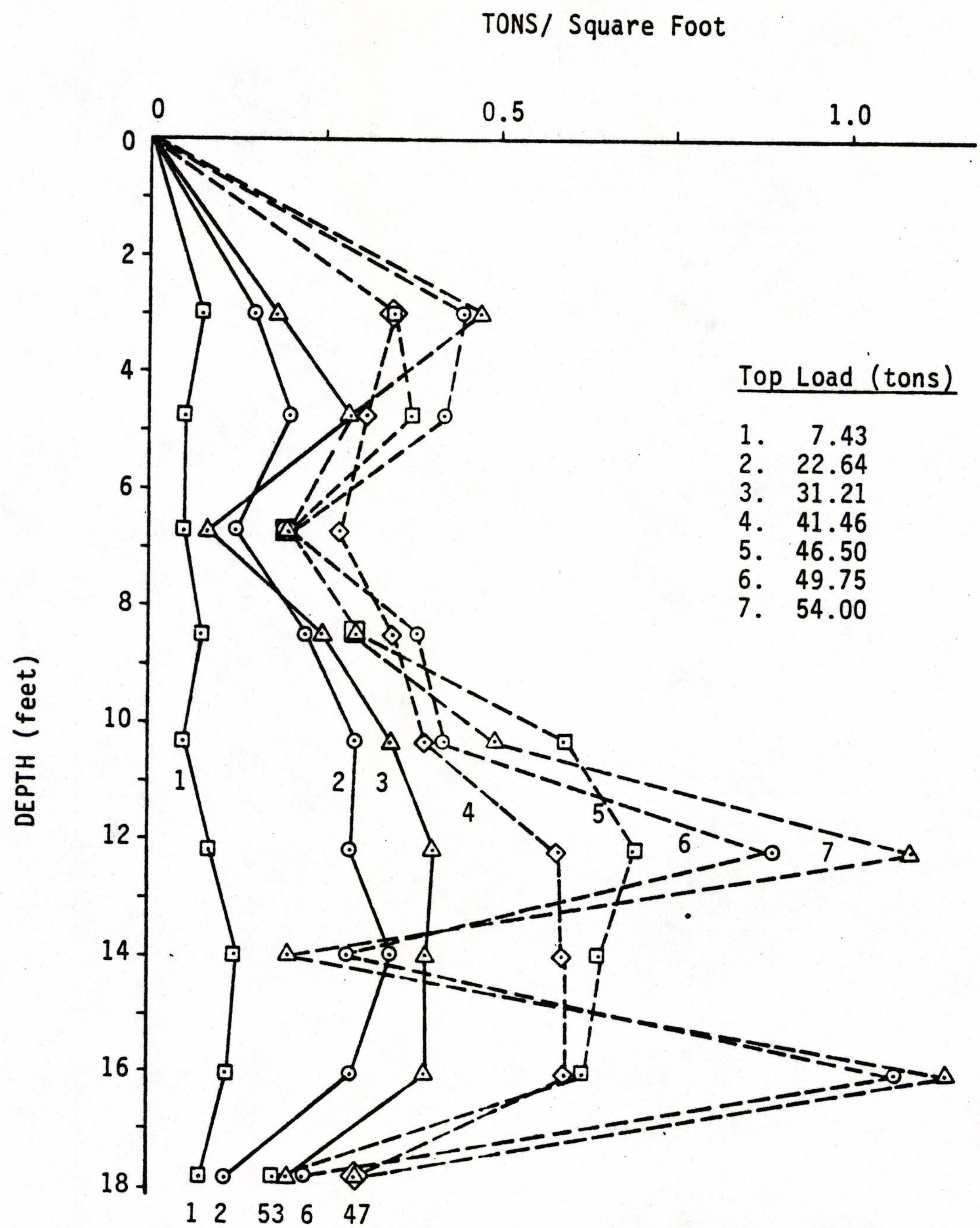


Figure 5.34 Skin Friction vs. Depth Curves from ML Test

* Numbers refer to corresponding top loads.

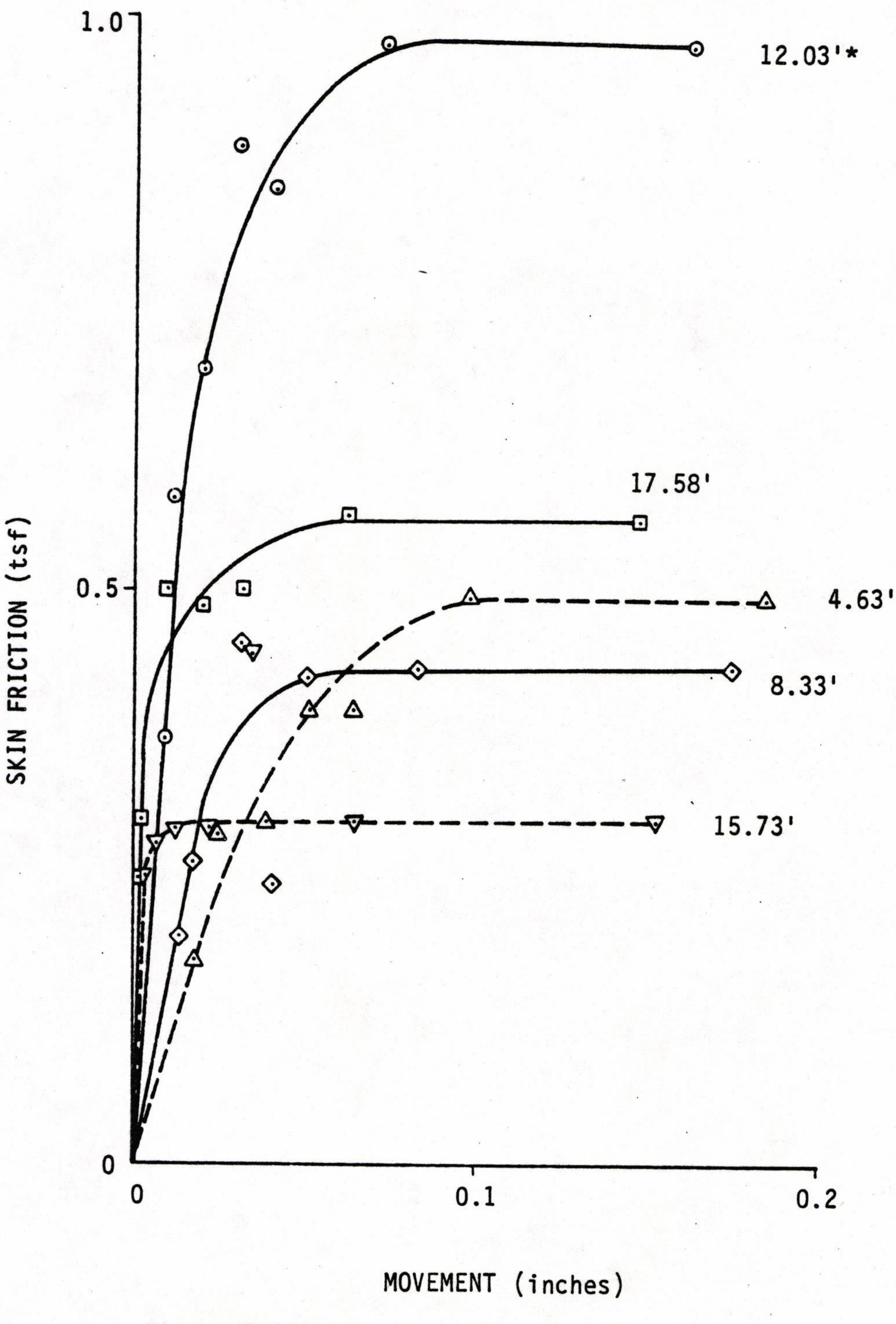


Figure 5.35 Skin Friction vs. Movement: CRP Test

* Numbers refer to depths in feet.

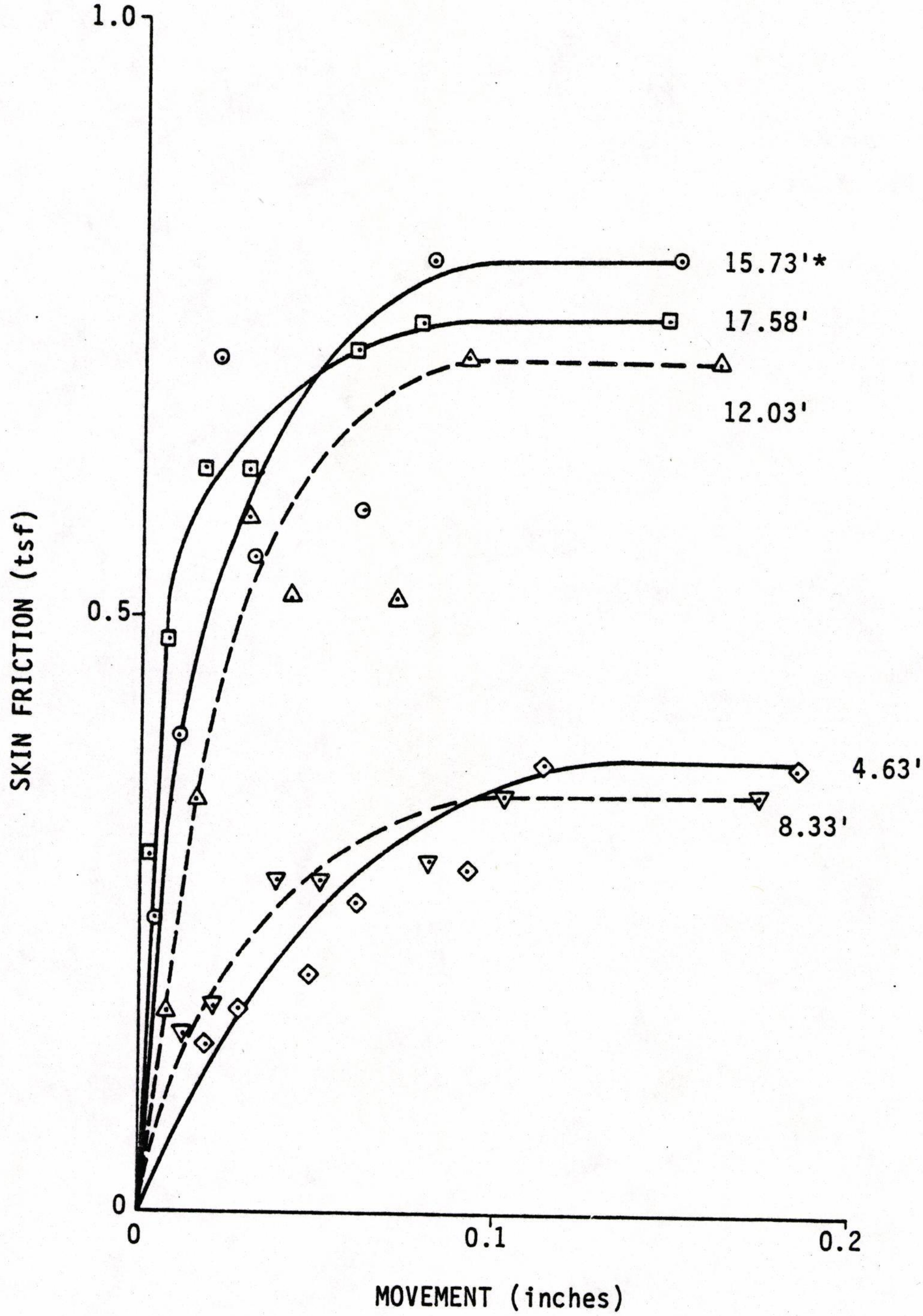


Figure 5.36 Skin Friction vs. Movement: TQ Test

* Numbers refer to depths in feet

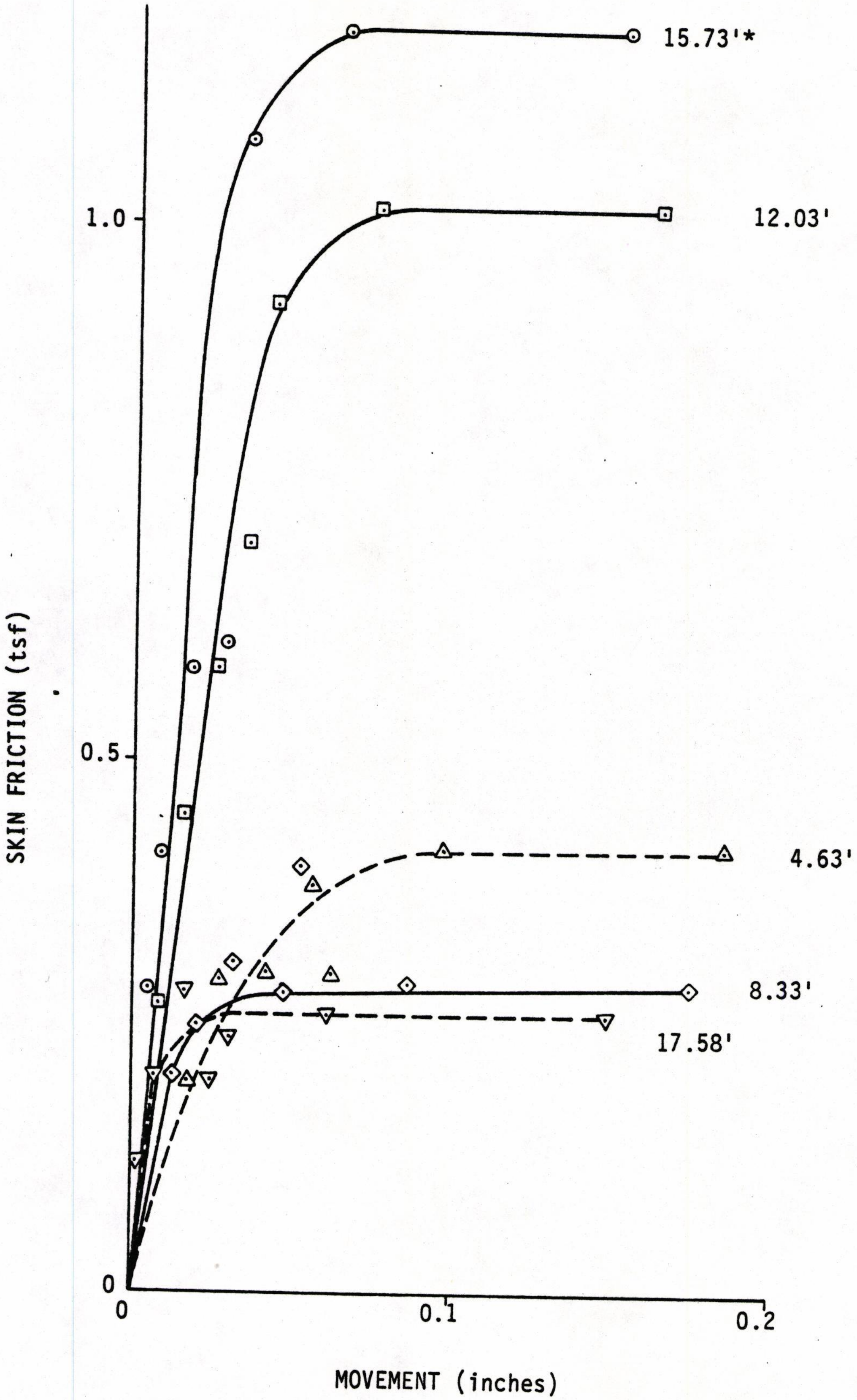


Figure 5.37 Skin Friction vs. Movement: ML Test

* Numbers refer to depths in feet.

CHAPTER VI

DISCUSSION OF RESULTS

If load test procedures give essentially equal results, then the choice of procedure should be on the basis of economy and convenience. The significant results from uninstrumented tests are the failure load and the load-settlement relationship. Additional information such as load distribution in the pile, developed skin friction and end bearing, and the development of skin friction and end bearing with pile movement can be obtained from piles instrumented to measure load transfer behavior. In this section, these items will be compared for the three test procedures used.

Failure Load

The failure loads for all the test piles did not vary significantly with test procedure. The maximum variation occurred at the Redfield site where a variation of 5% from the average was observed. The failure loads are given in Table 6.1 along with the failure loads predicted by several dynamic formulae (including the wave equation), the CWR device, and the limit equilibrium method of analysis. It can be seen that the Engineering News and Danish formulae do a poor job of predicting capacity while reasonable results are obtained by the other predictive methods. Where adequate soil data was available (Newport site) the limit equilibrium method accurately predicted the ultimate pile capacity. The properties of the clay at the Redfield site were based upon standard penetration values and the accuracy of pile capacity predictions at this site by the limit equilibrium method was marginal.

Method Used	Ultimate Pile Capacity (tons)						
	Newport No. 1	Newport No. 2	Smackover No. 1*	Smackover No. 2*	Redfield No. 1	Redfield No. 2	Redfield No. 3
Load Tests							
ML	124	75	190+	150+	69.5	55.8	121.3
TQ	126	75	190+	190+	63.5	55.6	123.2
CRP	126	75	190+	190+	65.5	55.8	119.4
Engineering News Record	281	123	667	674	616.	300.	496.
Danish Formula	178	98	330	238	361.	104.	355.
Hiley Formula	115	52	135	189	68.	52.	74.
Wave Equation	100	61	220	156	70	61.	100.
CWR Device	-	-	-	216	69.6	60.9	-
Limit Equilibrium Analysis (Based on soil properties)	123	83	307	197	63.0	39.5	75.5

* The load capacity of the hydraulic ram was not sufficient to cause failure of these piles. Loads shown are the maximum loads applied.

TABLE 6.1 Predicted and Measured Pile Capacities

Load-Settlement Relationship

The load-settlement curves for the test piles are independent of the test procedure for loads up to about 60% of the failure load. As failure is approached, the ML test gives the greatest settlements and the TQ test gives the next greatest settlements. This is probably due to creep under the high shear stresses (skin friction) existing on the sides of the pile. The load-settlement curves are also dependent upon the sequence in which the tests are performed. It appears that the first test performed will show more deformation than it would if it were performed later in the sequence. The sequence in which the piles are driven will also affect the load-settlement curves. The reaction piles should be driven first and the test pile driven last in order to avoid uplift of the test pile and large settlements when the pile is loaded.

Load Transfer Behavior

The load transfer behavior of the test piles was independent of the test procedure for loads up to about 60% of the failure load. As failure is approached, the ML test and to a lesser extent, the TQ test show less skin friction in the upper portion of the pile and a transfer of the load to skin friction at greater depths and a slight increase in end bearing. The deflection of the top of the pile increased as the load shifted lower in the pile. The piles tested in this project showed a stiffer response than was predicted by the Coyle and Reese (1966) and the Coyle and Sulaiman (1967) criteria. The skin-friction vs. deformation relationship is independent of test procedure for loads up to about 60% of the failure load but is test-dependent as failure is approached.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

Pile load tests measure the ultimate capacity and the short-term load-settlement behavior of a single pile. The capacity and behavior of pile groups, or long-term deformations cannot be determined from short-term tests on single piles. Other factors which must be considered in determining pile capacity are negative skin friction in compressible soils, possible stress relaxation in sands, and long-term settlement. Despite these inadequacies, pile load tests provide invaluable information and can result in significant economies in foundation design. The procedure used in performing a pile load test should be the one which gives reliable results with the greatest economy and convenience.

Based upon the pile load tests performed as a part of this project, the following conclusions and recommendations are made.

1. There is no significant difference in failure load produced by the maintained load test, the Texas quick test, and the constant rate of penetration test.
2. The load-settlement relationship is essentially the same up to about 60% of the failure load for all three test procedures used. This covers the normal working load range.
3. The load transfer behavior is essentially the same up to about 60% of the failure load for all three test procedures used.
4. Since there is no significant difference in the observed failure load between the three test procedures, and since the

load-settlement and load transfer behavior are essentially the same in the working load range, it is recommended that the Arkansas Highway Department adopt a rapid load test procedure. The Texas quick test is recommended because less expensive equipment is required (the same equipment used for the ML test may be used) and the test procedure is slightly easier to perform than the constant rate of penetration test. Either the TQ or CRP test would yield satisfactory results, however.

5. The Engineering News formula currently used in the Standard Specifications did not accurately predict the capacity of the test piles on this project. It is recommended that a comprehensive formula such as the Hiley formula be adopted and that the wave equation analysis be implemented also.

REFERENCES

Arkansas State Highway Commission (1972), Standard Specifications for Highway Construction, pp. 388-390.

Coyle, H.M. and L.C. Reese (1966), "Load Transfer for Axially Loaded Piles in Clay", Journal, Soil Mechanics and Foundations Division, ASCE, Volume 92, SM2, pp. 1-26.

Coyle, H.M. and Sulaiman (1967), "Skin Friction for Steel Piles in Sand", Journal, Soil Mechanics and Foundations Division, ASCE, Volume 93, SM2, pp. 261-278.

Federal Highway Administration (1976), Pile Driving Analysis - Wave Equation - TTI Program, Implementation Package 76-13.

Federal Highway Administration (1976), Wave Equation Analysis of Pile Driving, WEAP Program, Implementation Package 76-14.

Federal Highway Administration (1977), The Texas Quick-Load Method for Foundation Load Testing, Implementation Package 77-8.

Fuller, F.M. and H.E. Hoy (1970), "Pile Load Tests Including Quick-Load Test Methods and Interpretations", Highway Research Record No. 333, pp. 74-86.

Goble, G.G. and R. Rausche (1970), "Pile Load Test by Impact Driving", Highway Research Record No. 333, pp. 123-29.

Reese, Lymon C. "Design and Construction of Drilled Shafts", Journal, Geotechnical Engineering Division, ASCE, Volume 104, GT1, pp. 91-116.

Potyondi, J.G.(1961), "Skin Friction Between Various Soils and Construction Materials, Geotechnique, Volume 11, No. 4, pp. 331-353.

Skempton, A.W.(1951), "The Bearing Capacity of Clays", Proceedings, Building Research Congress, Institute of Civil Engineers, London, England, Volume 1, pp. 182-88.

Smith, E.A.L.(1960), "Pile Driving Analysis by the Wave Equation", Journal, Soil Mechanics and Foundation Division, ASCE, Volume 86, SM4.

Sowers, G.B. and G.F. Sowers (1970), Introductory Soil Mechanics and Foundations, Third Edition, Macmillan Publishing Co., Inc., New York, pp. 445-498.

Tomlinson, M.J. (1969), Foundation Design and Construction, Second Edition, Wiley-Interscience, New York, New York, pp. 387-391.

Vesic, A.S. (1967), "Ultimate Loads and Settlements of Deep Foundations in Sand", Bearing Capacity and Settlements of Foundations, Duke University, pp. 53-68.

Whitaker, T. and W. Cooke (1961), "A New Approach to Pile Testing", Proceedings, Fifth International Conference on Soil Mechanics and Foundation Engineering, Volume 2, pp. 171-76.

APPENDIX

SUGGESTED SPECIFICATION FOR QUICK LOAD TESTS

Loading tests to determine the size, length, and number of piles shall be made when called for in the plans or special provisions. Also, when the required bearing resistance, as computed by the specified pile formula, cannot be attained at or near the depth of penetration indicated on the plans, the Engineer may require load tests on one or more piles as necessary to establish the actual bearing capacities of the piles and to develop a modified bearing resistance formula.

Where loading tests are called for in plans or specifications, no piling other than test piling shall be cast or driven until the loading tests governing the structure or portion of structure in question are completed to the satisfaction of the Engineer.

Loading tests shall consist of the application of a test load placed upon the pile with suitable apparatus for accurately measuring the test load and the settlement of the pile under each increment of load. The test load shall be applied by a hydraulic jack acting between the test pile and the reaction. The reaction may consist of a weighted box or platform resting or cribbing and loaded to a total weight greater than the anticipated maximum test load, or a beam attached to anchor piles located as far from the test pile as possible, or other reaction approved by the Engineer.

The applied load shall be measured by a pressure gage connected to the hydraulic jack or by a load cell approved by the Engineer.

The pressure gage and the jack shall have been recently calibrated and certified accurate to within five percent.

Pile settlement shall be measured primarily by two dial gages furnished by the Contractor, capable of being read to an accuracy of 0.001 inch. The gages shall be attached to a fixed beam supported by stakes soundly driven at least 8 feet on either side of the test pile. The gages shall be mounted on opposite sides of the test pile with the stems parallel to the direction of load application. The stems of the gages shall rest on top of the pile or on lugs welded or clamped to the pile.

A check settlement observation shall be made before and after the loading test and at intervals during the test by one of the following methods:

(1) Use a surveyor's level and target rod reading to 0.001 foot with the rod resting on top of a bolt or rod set in the pile head and extending up through the reaction.

(2) Use a wire and scale with the wire stretched between two stakes driven 8 feet on each side of the test pile. The wire shall pass across the face of the scale attached to the test pile. Some suitable device shall be used to maintain constant tension in the wire throughout the test.

The secondary checks shall be completely independent of the dial gage set up.

The amount of time to elapse between driving and test loading shall be established by the Engineer. (A minimum elapsed time of 72 hours will usually be required for piles driven in clay, but piles embedded totally in sand may usually be tested immediately.) The procedure for load testing piles shall be as follows:

The head of the pile shall be cut off level and a plate placed on top of the pile. The load shall be applied in increments of 25 percent of the design load or as directed by the Engineer. Gross settlement readings, loads and other data shall be recorded immediately before and after the application of each load increment. Each load increment shall be held for an interval of 2-1/2 minutes. Each succeeding increment shall be applied immediately after the 2-1/2 minute interval readings have been made. When the load-settlement curve obtained from the test data shows that the pile has failed; i.e., the load can be held only by constant pumping and the pile is being driven into the ground, pumping shall cease. Gross settlement reading, loads and other data shall be recorded immediately after pumping has ceased and again at intervals of 2-1/2 minutes for a total period of 5 minutes. All load shall then be removed and the member allowed to recover. Gross settlement readings shall be made immediately after all loads have been removed and at intervals of 2-1/2 minutes for a total period of 5 minutes.

All test loads shall be carried to failure or to the capacity of the equipment, unless otherwise noted on the plans.

The ultimate pile capacity or failure load shall be taken as the maximum load that can be applied without producing a gross pile head movement in excess of the calculated elastic compression of the pile at that load plus 1/50 of the pile diameter.

In the event test loading does not show a satisfactory bearing value the test pile shall be driven further and again test loaded as directed.