LOAD DISTRIBUTION IN TEST LOADED PRESTRESSED CONCRETE PILES

by

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SUMMARY

In the spring of 1977, a full scale testing programme was conducted by the United States Federal Highway Administration (FHWA) to investigate the behaviour of seven octagonal 60 metre (200 ft) long prestressed concrete piles, driven in very soft, compressible clay layer undergoing settlement. Three of the piles were subjected to a short term static test loading and four were monitored over a period of one year while influenced by downdrag load caused by a 3.7 metre (12 ft) embankment surcharge.

The static test loading was conducted three to four months after the piles were driven. The downdrag measurements were first recorded one month after the piles were driven and continued for 189 days.

The data from measurements of telltale determined deformation in the sleeved portion of the piles were used to formulate the variation of the deformation modulus with stress and with strain. For each of the test loaded piles, the tangent modulus was first plotted versus stress level and versus strain level. Linear regression analysis was used to establish the expression for pile modulus versus stress and strain.

The measured compression was converted to corresponding strain at approximate failure load. The load distribution in the piles was obtained using the corresponding equation of the deformation modulus variation with strain level.

For the test loaded piles, the mobilized shaft unit resistance was computed by differentiating the regression polynomial describing the best curve fit of the distribution of load in the pile and dividing the results by the unit stiffness of the piles. The mobilized shaft resistance was found to increase linearly with depth. A small toe resistance was observed in the three piles.

The shaft resistance mobilized by Piles 1, 2, and 4 was compared to the overburden effective stress to obtain the Bjerrum-Burland beta coefficient and to the undrained shear strength to obtain the alpha factor. The calculated values of the beta coefficient ranged between 0.27 to 0.43 in the clay layer, and was 0.38 in the silty clay layer. The alpha factor was 0.75 in the soft clay layer and 0.71 in the stiff silty clay layer. The obtained results were compared to values presented by Burland (1973) and by Meyerhof (1976).

The beta values were found to be overestimated for the test loaded piles when 100 and 200 ton of residual loads assumed to be present in the piles at the start of the testing.

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CHAPTER ONE

INTRODUCTION

1.1 General

The use of piles is one of man's oldest method of overcoming difficulties of founding concentrated loads on soft soils. Piles are used to either transmit the load to the soil or to reduce settlement in soils where large vertical movement is expected. An ever-increasing use of deep foundation to support loads have increased the demands for developing a good understanding of the behaviour of pile foundation under applied loads. Static and dynamic tests on piles have been considered expensive but used over the years by geotechnical engineers in order to prove a design or to gain information on the interaction between the foundation and the supporting soil.

Static testing of piles is usually conducted by loading the pile or pile group using hydraulic jacks acting against a anchored reaction frame in accordance with the ASTM D1143 designation. The compression of the tested pile is measured by means of rod extensometers (telltales) and strain gauges placed in the pile.

In the spring of 1977, a full scale pile testing programme was initiated by the United States Federal Highway Administration (FHWA) at the site of Keehi Interchange, Honolulu, Hawaii, to investigate the behaviour of a 60 metre (200 ft) long prestressed concrete piles in very soft, compressible clay soil undergoing settlement. The programme consisted of test driving and test loading several prestressed concrete piles. Three of the piles were subjected to a short term static loading test and four were monitored over a period of one year while influenced by downdrag load caused by a 3.7 metre (12 ft) embankment surcharge. This report presents a study on the distribution of axial load in the tested piles.

1.2 Objectives

The objectives of this study are as follows:

(1) to verify the evaluation of the deformation modulus of the Keehi prestressed concrete piles carried out by Yu (1984).

(2) to determine the axial load distribution in the statically tested piles.

(3) to evaluate the Bjerrum-Burland beta-coefficient (ratio of shaft resistance over effective overburden pressure) acting along the shaft of the piles.

1.3 Scope

The scope of this study consists of an evaluation of the measured strain in the three push-tested piles and in the four piles subjected to downdrag load. The analysis proceeded as follows:

(1) verification of the validity of the deformation modulus formulation of the tested prestressed concrete piles, Yu (1984).

(2) preliminary analysis of the accuarcy of telltale data using recomendations by Yu (1984).

(3) determination of the load and shaft resistance distribution in the piles without consideration of residual stresses.

(4) evaluation of the Bjerrum-Burland beta-coefficient.

1.4 Outline of the Report

Chapter 2 presents a literature review and theoretical background of the subject matter.

Chapter 3 presents a description of the site, the site soil profile, the test piles, and the testing procedure.

Chapter 4 presents the field data and the data compilation system.

Chapter 5 contains an analysis of the test loading results. It covers the formulation of the deformation modulus, the axial load distribution along the piles for three levels of applied load, and presents an estimation of residual strain and a computation of the beta-coefficient.

Conclusions of this study are presented in Chapter 6.

Tables and figures are appended at the end of the report.

CHAPTER TWO

LITERATURE REVIEW AND THEORETICAL BACKGROUND

2.1 General

A pile is a foundation unit that provides support for a structure by transmitting the loads through compressible or weak soil to more competent soil. It is used where footings cannot transmit inclined, horizontal, or uplift force-loads to the underlying soil, and where collapsible layers of soil extend to a considerable depth. Pile foundations are classified by material type or by the method of placement. Piles can be pre-manufactured or cast in place. They can be driven, jacked, or jetted. They can be made of wood, concrete, or steel.

2.2 Pile Capacity

The static capacity (R) of a single pile is considered as the sum of two components, the shaft resistance (R_s) and the toe resistance (R_t), as follows:

$$R = R_s + R_t \tag{2.1}$$

2.2.1 Positive shaft resistance

The theoretical approach for evaluating the unit shaft resistance developed along a driven pile, r_s, is similar to that used to analyse the resistance to sliding of a rigid body. Burland (1973) proposed an equation presented in Eq. 2.2, relating the unit shaft resistance r_s to the lateral effective stress acting on the pile. This equation is based on the assumptions that the excess pore pressure developed during pile installation is completely dissipated and the soil arround the pile has non effective cohesion due the remoulding of soil during installation.

$$r_s = \sigma_h' \tan\phi' \tag{2.2}$$

where σ_h' = horizontal effective horizontal stress at depth z.
ϕ' = effective angle of friction between soil and pile shaft.

Employing K_s as the ratio between the horizontal effective stress to the vertical effective stress, and M as the ratio between the tangent of the soil angle of friction to the tangent of the angle of friction between soil and pile shaft, Eq. 2.2 becomes:

$$r_s = \sigma_v' M K_s \tan\phi' \tag{2.3}$$

where M = $\tan\phi' / \tan\delta'$
$\tan\phi'$ = soil friction
$\tan\delta'$ = pile-soil friction

The quantity $M K_s \tan\phi'$ is called the Bjerrum-Burland β factor and is equal to the ratio of unit shaft resistance to the effective overburden stress at any depth along the pile.

The value of M ranges from 0.7 to 1.0 depending on the pile material (steel, concrete, wood) and method of installation (Bozozuk et al., 1978). The values of K_s depend on soil type, the stress history, and the method of pile installation. Eqs. 2.4 and 2.5 are equations reported by Massarsch (1979) relating the earth pressure coefficient at rest K_o to the effective angle of internal friction (ϕ') and to the plasticity index (I_p) of normally consolidated clay.

$$K_o = 1 - \sin\phi' \tag{2.4}$$
$$K_o = 0.44 + 0.42 (I_p / 100) \tag{2.5}$$

Burland (1973) reported that, for a pile driven in soft normally consolidated clay, the value of K_s is higher than K_o and the value obtained from the simple effective stress theory presented in Eq. 2.3 is considered as a lower limit. Fig. 2.1 shows a plot of average shaft resistance versus effective overburden pressure compiled from a large number of pile test carried out on a wide variety of clays. The values of beta obtained range between 0.25 and 0.40. For soft normally consolidated clays with angle of internal friction ranging between 20 and 30 degrees, the beta factor varied only between 0.25 and 0.30. The 0.25 value is considered as the lower limit obtained by Eq. 2.3 in which it is assumed that the value of K_s is equal to the earth pressure coefficient at rest (K_o).

For piles driven in stiff clays, the validity in using Eq. 2.3 to evaluate the shaft resistance along the pile shaft is questionable due to the complex stress conditions during pile driving. The central problem is in estimating the value of K_s which for heavily overconsolidated clay can varies with depth and can have values as high as 3 near the surface to less than 1 at higher depth. The value of beta reported can be as high as 0.8 for stiff clay and for coarse and dense soils.

Meyerhof (1975) reported that the average ultimate unit shaft resistance, r_s, in homogeneous saturated clay soils can be related to the undrained shear strength τ_u as follows:

$$r_s = \alpha \ \tau_u \tag{2.6}$$

where α is an empirical coefficient that depends on the nature and strength of the clay, method of pile installation, and pile diameter. For driven piles, the α-coefficient can vary from unity for soft clay to one half or less in stiff clay.

Vesic (1977) reported that a pile displacement of 10 mm (0.4 inch) is sufficient to mobilize the shaft resistance along a driven pile, regardless of soil and pile type and dimensions. However, Fellenius (1980) reported as a results of field investigations that even smaller movement of 2 to 4 mm (0.1 to 0.2 inch) was enough to generate shear resistance along a driven pile.

2.2.2 Negative skin friction

Negative skin friction develops when piles are driven through soft soils into stiff underlying strata or when a stress increase usually in the form of fill or groundwater lowering has occurred. Bjerrum et al. (1969) reported that the negative skin friction resulting from the consolidation of a clay layer can be related to the effective overburden pressure as presented in Eq. 2.3.

Bozozuk (1981) found that a reversal of shear forces down to a depth of 20 m occurred when loading a pile and generating a relative movement of 4 mm. Fellenius (1984) reported that a 35 mm settlement of the ground surface due to a 3 metre surcharge placed around single piles was sufficient to develop negative skin friction down to depth of 20 m.

A neutral plane shown in Fig. 2.2 as defined by Fellenius (1984) and also given in the CFEM (1985) is the plane where the shaft shear changes from negative skin friction to positive shaft resistance. It is determined by the requirement that the sum of the applied load and the drag load is in equilibrium with the sum of the positive shaft and toe resistances of the pile.

Fellenius (1972), presented measurements on the build-up of negative skin friction on two instrumented precast piles driven through 40 m of soft clay into a firm underlying layer. The results showed a steady increase of negative skin friction with time. The shaft friction was not fully developed at the end of the test. A beta-factor of 0.1 was obtained after a 480-day observation period.

2.2.3 Toe resistance

The toe resistance of a single pile is difficult to determine and, usually, it can only be estimated by an approximative formula. The toe resistance of a single pile R_t can be related to the effective overburden pressure as :

$$R_t = A_t\, r_t = A_t\, \sigma_v'\, N_t \tag{2.7}$$

where A_t = the cross sectional area of the pile at the toe
r_t = the unit toe resistance
N_t = the bearing capacity coefficient.

Meyerhof (1975) reported empirical values relating the static cone pressure, q_c, to the unit toe resistance.

For piles in homogeneous sand the relationship was presented as a function of the angle of internal friction as presented in Eq. 2.8.

$$r_t = q_c = 0.5\, N_q \tan\phi' \tag{2.8}$$

Meyerhof (1975) reported also that the unit toe resistance cannot be estimated by Eq. 2.7 for piles driven into homogeneous soil through compressible material into a thick bearing stratum . The value of r_t becomes independent of the overburden pressure at the pile toe and should be determined by Eq 2.8.

Esrig and Kirby (1979), reported a formula that relates the unit toe resistance of piles driven in clay to the undrained shear strength τ_u as presented in Eq. 2.9:

$$r_t = \tau_u\, N_c \tag{2.9}$$

in which τ_u is the shear strength of the soil at the pile toe and N_c is the bearing capacity factor for cohesion. Esrig and Kirby (1979) reported that the value of N_c can be related to the clay plasticity index and ranged between 10.3 to 11.9 for clay with plasticity index smaller than 30, and between 9.4 to 11 for clay with plasticity index higher than 30.

2.3 Analysis of Data from Telltale Instrumented Piles

Anchors or telltales have been used in the instrumentation of statically tested piles. Telltales are placed in the pile at certain depths for registring axial deformation of the pile under a load applied at the pile head. Methods of analysis of telltale data have been reported by Leonards and Lowell (1979), Reese (1979), Fellenius (1980), and others.

Fig. 2.3 outlines the approach by Reese (1979) for the analysis of telltale instrumented piles. Figs. 2.3A and 2.3B show pile head movement and load in the pile at various levels of head load. The average load Q_z at any depth in the pile is calculated using Eq. 2.10.

$$Q_z = E_z A_{c,z} \frac{du_z}{dz} \tag{2.10}$$

where E_z = the deformation modulus of the pile at depth z.
$A_{c,z}$ = the cross sectional area of the pile at depth z.
du_z/dz = pile axial strain at depth z.

By selecting a particular depth z and a load level, the slope of the load distribution curve dQ/dz divided by the circumference of the pile at that point yields the unit load transfer, or unit shaft resistance, r_s, as follows:

$$r_s = \frac{1}{A_{s,z}} \frac{dQ}{dz} \tag{2.11}$$

Integration of the area (1) under the load distribution curve divided by the appropriate values of axial stiffness (AE) gives the compression of the pile over the depth z selected, which, when subtracted from the pile head movement for the load selected, yields the vertical movement (u) of the pile. Repeating this computation for each load level, the unit load transfer versus movement curves at different telltale levels can be obtained. A plot of the variation of the unit shaft resistance with depth can be obtained as shown in Fig. 2.3d.

Fellenius (1980) presented a method in evaluating the load distribution in telltale instrumented, statically tested piles as shown in Fig. 2.4, where a pile is installed into a two-layer soil. The diagram shows one telltale placed at the pile toe and another placed some distance above. The measured axial deformations of the pile give three values of average load in the pile as marked in the graph. The straight lines, Lines 1 and 2, from the pile head through the two upper average loads are mathematically true considering one telltale at a time and assuming a unit constant shaft resistance distribution. Fellenius (1980) reported that, the true load distribution (Line 3) must be fitted in a way that the areas A' and A" and B' and B" between the true line, Line 3, and the mathematical lines, Lines 1 and 2, are equal. For linearly increasing unit shaft resistance, the same procedure can be applied to find a curved load distribution line as shown by Line 4 of Fig. 2.4.

2.4 Residual Strain

Residual or locked-in strain is the strain induced in the pile as the result of either or both the pile installation and that induced from the consolidation of the surrounding soil after installation of the pile.

Driving a pile into clayey soil causes large displacement that generates excess water pressure in the vicinity of the piles. The dissipation of this excess pressure with time affects the soil strength and stress-strain relationships. These changes in soil stresses during and after installation of the pile are accompanied by the development of forces causing residual strain in the pile.

Cooke (1979) stated that, residual strain arise in the pile because of differences in the rates of mobilization of resistance, on the shaft and at the toe as a pile is displaced, either during installation or under compressive loading during reconsolidation of the soil. Fig. 2.5a shows the soil displacement during the application of the driving forces where the shaft and the toe resistances of the pile are acting upward to resist the downward movement of the pile. As the driving force falls to zero and the pile rebounds, the soil under the pile toe pushes the pile back up while the pile lengthens elastically. These two components of the rebound create enough upward movement to reverse the direction of the pile-soil creating a negative shaft resistance in equilibrium with the pile toe force, as shown in Fig. 2.5b.

This definition leads to the interesting observation that the neutral plane as defined in Section 2.2.2 is located at the toe of Cooke's pile. This is explained by the fact that Cooke's model is based on high pile rebound caused by large toe resistance.

Fig. 2.6 presents results reported by Cooke (1979) on the distribution of residual load in 5 metre long steel piles driven in London clay. Results were obtained by measuring the pile rebound at intervals of 0.1 metre pile penetration. A rebound ranged between 2 to 4 mm was obtained at penetration smaller than 1 m because the mobilized shaft resistance was not sufficient to oppose the rebound force at the pile toe, whereas at larger penetration, the rebound stabilized at 2 mm where sufficient shaft resistance was developed to prevent the pile rebound. It was noticed also that, at large penetrations, the residual load at the toe was approximately 75 percent of the maximum toe load.

Another study on the residual load distribution in steel piles embdded in sands was reported by Hunter and Davisson (1969). A residual load of 25 to 48 tons was obtained for conventionally driven piles, whereas negligible residual load was found for piles driven by a vibratory hammer. This was credited to that vibratory hammers are extremely effective in minimizing shaft resistance during driving.

2.5 Limit Loads

Pile movement and behaviour have been used in empirical procedure to determine pile limit load. A load causing a pile head movement of 10 % pile diameter was considered as failure load. However this definition was considered not fully correct by a number of researchers because it ignores the elastic deformations of the pile which can be significant for long piles. Brinch Hansen (1963), Chin (1971), Davisson (1972), and Butler and Hoy (1977) were among others that proposed numerical methods to evaluate the limit load from a static loading test.

Davissondefined his limit load as the load that corresponds to a movement exceeding the pile elastic compression by a value of 4 mm (0.15 in) plus a value equal to the pile diameter divided by 120. Chin, assumed that the load movement curve is of hyperbolic shape when the load approaches the failure load. By the Chin method, each load is divided with it's corresponding movement value and the resulting value is plotted against the movement. The slope of the linear part of this curve correspond to the Chin limit load. The limit load proposed by Butler and Hoy (1970) was defined as the load at the intersection of the tangent sloping 14 mm/KN (0.05 in/ton), and the tangent to the initial straight portion of the load - deformation curve. Fellenius (1980) suggested that the intersection should be to the elastic line instead. Brinch Hansen defined his limit load - the 80 % criterion - as the load that gives four times the movement of the pile head obtained for 80% of that load. The four mentioned methods along with five others are summarised and discussed by Fellenius (1980).

2.6 Deformation Modulus

In contrast to what is usually assumed, the deformation of piles does not vary linearly with the load level. Fig. 2.7, shows a typical stress-strain relationship for a concrete pile obtained from a static loading test. The deformation modulus of the pile can be determined as the slope of the tangent to the deformation curve at any point defining the tangent modulus, or as the secant modulus defined as the slope of the chord joining any point on the curve to the origin as defined in Eqs. 2.12 and 2.13, respectively.

$$E_s = \frac{\sigma_i - \sigma_o}{\epsilon_i - \epsilon_o} \tag{2.12}$$

$$E_t = \frac{\sigma_i - \sigma_{i-1}}{\epsilon_i - \epsilon_{i-1}} \tag{2.13}$$

where E_s = secant modulus

σ_o = initial stress

σ_i = stress at the considered load level

ϵ_o = initial strain

ϵ_i = strain at the considered load level

E_t = tangent modulus; increment of stress over increment of strain

$\sigma_i - \sigma_{i-1}$ = increment in stress from load level i-1 to load level i

$\epsilon_i - \epsilon_{i-1}$ = increment of strain from load level i-1 to load level i

The variation of the deformation modulus can be presented with respect to either the stress level or to the strain level. To choose one or another depends on various factors such as the goodness of fit of the line or the curve used to model this variation.

A first approach, presented by Yu (1984) consists of the deformation modulus as a function of stress level. The tangent modulus E_t is assumed to vary linearly with stress level as shown in Fig. 2.8 and presented in Eq. 2.14.

$$E_t = A\sigma + B \tag{2.14}$$

Where A is the slope of the line and B is the y intercept.

Using Hookes law, the tangent modulus variation with strain can be obtained as follows:

$$E_t = B / (1 + A\epsilon) \tag{2.15}$$

The stress-strain equation can therefore be obtained by integrating Eq. 2.15 with respect to strain. This was based on the definition thatthe tangent modulus E_t is the derivative of the stress-strain curve at any stress or strain level. Applying this concept, the stress-strain equation is:

$$\sigma = (B/A) \ln (1 + A\epsilon) \tag{2.16}$$

Having the equation of the stress-strain curve and using the definition that the secant modulus is the ratio of the difference in deformation between any load level and the initial conditions, which in most cases is the origin, the secant modulus variation with strain or with stress level can be obtained as.

$$E_s = \frac{\sigma(i) - \sigma(0)}{\epsilon(i) - \epsilon(0)} = \frac{B\ (\ln\ (1 + A\ \epsilon(i)) - \ln\ (\ 1 + A\ \epsilon(0)))}{A\ (\ \epsilon(i)\ -\ \epsilon(0))}$$

Letting $\epsilon(0)$ be equal to zero at the origin of the curve and E_o be the modulus value at zero strain, the above equation can be presented as:

$$E_s = (E_o/A\epsilon\ )\ \ln\ (1 - A\epsilon\ ) \qquad (2.17)$$

An alternative approach can be developed by assuming that the tangent modulus is linear with strain instead of with stress. The same procedure applied above is used and the modulus variation with strain, the stress-strain curve and the secant modulus variation with strain can be obtained as presented in Eqs. 2.18 through 2.22.

$$E_t = A\,\epsilon + E_o \qquad (2.18)$$

$$= (A/2)\,\epsilon^2 + E_o\,\epsilon + C \qquad (2.19)$$

$$E_s = \frac{\sigma(i) - \sigma(0)}{\epsilon(i) - \epsilon(0)} = \frac{(A/2)\,\epsilon^2(i) + E_o\epsilon(i) + C - ((A/2)\,\epsilon^2(0) + E_o\ \epsilon(0) + C)}{\epsilon(i)\ -\ \epsilon(0)} \qquad (2.20)$$

$$E_s = (A/2)\,\epsilon + E_o \qquad (2.21)$$

And as a function of stress, the equation becomes.

$$E_s = (E_o/2) - \sqrt{(1/2)\ (E_o^2 - 2A\sigma\ )} \qquad (2.22)$$

CHAPTER THREE

KEEHI FIELD TESTING PROGRAMME

3.1 Introduction

The Keehi field testing programme (KFTP) is a project-oriented research programme, which was undertaken to confirm or to modify preliminary foundation design assumptions and to establish the final design for the Keehi Interchange project.

3.2 Site Description

3.2.1 Site location

The Keehi Interchange is located at the Keehi Lagoon basin in Honolulu, Hawaii, toward Pearl harbor and covers an approximatly 300 metre (1000 ft) by 1000 metre (3000 ft) area.

The field testing programme consisted of seven instrumented precast prestressed concrete piles. The test area was located about 600 metre (2000 ft) from the junction of Middle Street and Dillingham Boulevard.

3.2.2 Soil conditions

The information on the soil condition at the Keehi site has been obtained from a report prepared by Parsons Brinckerhoff-Hirota Associates (1979) on the Keehi Field testing program.

Geotechnical investigations at the Keehi site revealed four main soil strata. The ground surface was horizontal and at elevation of +1.5 metre (5 ft). The groundwater table was encountered in the fill at the ground surface. A profile of the excess hydrostatic head measured by piezometers at different locations and elevations is shown in Fig. 3.1.

Table 3.1 shows the soil profile as obtained from boreholes drilled at the Keehi site. The soil profile consists of 2 to 5 metre (8 to 15 ft) of medium to compact coral sand fill underlained by a very soft clay layer to a depth of 33 metre (110 ft). The fill was placed in the 1930's and 1940's to reclaim the area for the construction of Kamehameha Highway and Nimitz Highway. Fig. 3.2 shows the undrained shear strength envelope of the soft clay layer obtained from field vane shear tests with Bjerrum's correction and from evaluation of the measurement of a Dutch cone Penetrometer with friction jacket.

At a depth of 41 metre (134 ft), the soft silty clay changes to a firm silty clay. The silty clay generally extends to elevation of 44 metre (144 ft), of which the lower 5 to 8 metre (15 to 25 ft) are firmer than the clay above it. The degree of consolidation and compressibility of this layer has been difficult to assess. It varies from slightly underconsolidated to overconsolidated.

The soft to firm clay is followed by a mixed stratum of coral sand and/or firm to stiff silty clay. The mixed stratum extends to a depth of 58 metre (190 ft). The thickness of the coral sand layer varies from 0 to 11 metre (0 to 35 ft) and the density from very compact to loose. This region is not well defined and often found mixed with the firm clay from above and/or stiff clay from below.

At an depth of 57 metre (187 ft), the mixed stratum is underlain by a stiff to very stiff silty clay. The deepest borings penetrated to depth of 80 metre (260 ft) without encountering competent rock.

The geotechnical proporties of each soil layer (density, water content, angle of internal friction and the plasticity index) as obtained from laboratory testing are presented in Table 3.1. A profile of the effective overburden pressure around pile 1, 2, and 4, calculated from the soil data and from the excess pore pressure profile shown in Fig. 3.1 is presented in Fig. 3.3.

3.3 Test Piles

3.3.1 General

Two pile testing programmes were carried out on seven prestressed precast concrete piles. The first programme consisted of three piles statically tested under compression load, and the second test involved a long term monitoring of the behaviour of four piles under downdrag conditions.

3.3.2 Pile description

The Keehi test piles consisted of 420 mm (16.5 inch) octagonal precast prestressed concrete piles. The piles were equipped with a 125 mm (5 inch) PVC pipe cast in the centre of the pile for inspection and instrumentation purposes. All piles had a net cross section area of 0.13 m^2 (200 in^2), a circumference of 1.4 m (4.6 ft), and were composed of three segments which were spliced during driving with either Herkules splices or tube splices. A standard Herkules rock shoe connected to the pile was provided for the Herkules spliced piles, while the tube spliced piles were equipped with a loose fitting tube-shoe held in place with wooden wedges. All piles were installed through steel sleeves ranging from 5 to 27 metre (16 to 27 ft) to eliminate the effect of shaft friction of the upper soil layer.

The statically tested piles; Piles 1 through 4, were driven between June 7 and June 16 and tested between September 15 and October 4. The downdrag piles ; Piles 6, 7, 8, and 9 were driven between June 6 and June 12. Two of the downdrag piles, Piles 7 and 9, were coated with bitumen in the soft clay layer to reduce the negative skin friction. Table 3.2 presents a full description of all piles used in this testing program. The locations of these piles are shown in Fig 3.4. The pile driving hammer consisted of a Delmag D30 single acting diesel hammer.

3.4 Pile Instrumentation

The information on the pile instrumentation were obtained from Soil and Rock Instrumentation Report (1978). The instrumentation locations in the seven piles are presented in Table 3.3. The pile instrumentation included an inclinometer casing, strain gauges, multi-rod extensometers, multi-wire extensometers, total earth-pressure cells, and pneumatic piezometers. The soil instrumentation consisted of ground settlement platforms, ground control stakes, deep and multi deep settlement points gauges, pneumatic piezometers, and groundwater observation wells.

Piles 1 and 2 were instrumented with multi-rod extensometers and strain gauges. Pile 4 was instrumented with rod extensometers only. The downdrag piles, Piles 6 to 9, were equipped with rod and wire extensometers, strain gauges, total earth pressure cells, and piezometers. Locations of the instrumentation in the seven piles are presented in Table. 3.3.

3.4.1 Rod extensometer

The rod extensometers (telltales) used in the instrumentation of the Keehi testing piles consisted of anchors grouted in the pile centre tube at selected locations. For details, see Yu (1984). Rising to the pile head from these anchors were oil-filled polyethylene tubes into which steel rods were installed to mate with the anchors. A reference plate was mounted on the pile head allowing passage of all extensometer rods within the pile creating stick-up of each rod above the reference plate. Any pair of anchors may be used to define a "gauge length". The change in rod stick-up length is equal to the pile compression over the considered gauge length.

3.4.2 Earth pressure cells

The earth pressure cell consists of two rectangular stainless steel plates welded together around their rims with athin oil-filled space between. For details, see Yu (1984). The cell is cast into the pile such that one face is flush with the pile surface. A pressure acting on the face is transmitted through the oil and acts on the sensor. The sensor operating system consists essantially of balancing total earth pressure on the soil side (pile) of the diaphgram with a measured gas pressure at the opposite side. Gas pressure is applied through an 3 mm (1/8 inch) nylon input tube, with flow returning through a similar return tube. A gas supply tank, flow regulator, flow indicator, and gas pressure gauge were mounted in a portable readout unit.

3.4.3 Pile piezometers

The piezometers cast into the pile were of the same type as the earth pressure cells. For details, see Yu (1984). The outer stainless steel plate is perforated with a series of 2 micron stainless steel filters which allow passage of water into the space between the plates and along the connecting tube to the sensor. A second tube allowed saturation with water prior to pile driving by means of a 3/16 inch nylon tube.

3.5 Soil Instrumentation

The soil instumentation included soil piezometers, multi deep settlement points, benchmarks, and soil inclinometers. The location of this instrumentation is shown in Fig. 3.4.

3.6 Static Axial Compression Loading Test

3.6.1 Test Procedure

The arrangement for the axial compression test followed the ASTM D1143 designation. The quick maintained load method of testing was applied using a small constant increment of load applied every 15 minutes. A 30 ton load increment was used for Piles 2 and 4 and a 25 ton increment was used for Pile 1.

3.6.2 Loading system and test arrangement

The static loading on the Keehi piles was conducted by jacking against a dead weight reaction system supported on wood mats as illustrated schematically in Fig. 3.5. Total available reaction load was approximately 650 tons, composed primarily of large concrete beams. A load transfer assembly allowing access to telltales at pile head was placed on top of the tested piles.

To sleeve off the influence of the fill layer and to provide a length of pile which was unaffected by shaft resistance, Piles 1, 2, and 4 were sleeved off over a length of 5 metre (16 ft), 5 metre (16 ft), and 27 metre (90 ft), respectively. See also Table 3.2.

Vertical pile head movements were measured with two dial gauges mounted on opposite side of the pile. Two reference beams were used, one on each side of the of pile. The stability of the beams was monitored with surveying equipment to check for any possible settlement. Direct measurement of pile head movement was made using 125 mm (5 inch) range dial gauges. To provide remote readout capability, a direct current displacement tranducer (DCDT) was attached in parallel with each dial gauge and connected to an automatic data acquisition system (ADAS). The movement of the coral crust near the corner of the cribbing was also monitored with a dial gauge attached to the reference beam and bearing on a steel rod driven into the crust. The lateral movement of the pile head was monitored with two dial gauges of 125 mm range. For details, see Yu (1984).

3.7 Downdrag Monitoring and Settlement

Four piles designated as Piles 6 through 9 were driven at the same site as the statically tested piles as shown in Fig. 3.4, and monitored for long term downdrag load. Two of these, Piles 7 and 9 were coated with bitumen to check the effectiveness of the coating in reducing the downdrag load. The bitumen coated length ranged between 26 to 31 metre (85 to 100 ft). All piles were instrumented with rod and wire extensometers. Compressions were measured by telltales instrumented in the piles.

To increase the rate of the soil settlement around the downdrag piles, and starting on July 28, 1977, 51 to 59 days after the driving of the first and last of the downdrag test piles, and finishing on August 12, 1977, a 3.7 meter (12 ft) embankment surcharge was built around the piles with a base of 29 metre (95 ft) in length by 18 metre (59 ft) in width and with sides sloping at 2(H):1(V). Settlement of the pile group was monitored by means of five settlement platforms installed at different loaction around the embankment surcharge.

To sleeve off the influence of the fill layer, Piles 6, 7, 8, and 9 were sleeved off over a length of 8.4 metre (27.5 ft).

As shown in Fig. 3.6, a ground surface settlement of 127 to 152 mm (5 to 6 inches) was recorded at the end of the 15 days of embankment construction. By December 5, 1977, the ground surface settlement had increased to 406 mm (16 inches). The multi-deep settlement points at that time indicated less than 25 mm (1 inch) settlement at a depth of 34 m (110 ft) below the soft clay layer. At the closing of the monitoring programme on February 6, 1977, the ground surface had setlled 460 mm (18 inches). Table 3.4 shows the dates of the driving, the testing, and the monitoring of the piles.

CHAPTER FOUR

FIELD DATA

4.1 Loading Test Data

4.1.1 General

The static loading test of the Keehi testing programme consisted of testing three piles, Piles 1, 2 and 4, under compressive loads. The data consisted of pile head movement and pile compression at each telltale level under the incremental head load as presented in Tables 4.1 through 4.3. The tabulated data were plotted as load versus deformation as shown in Figs. 4.1 to 4.3. The pile compression was measured as deformation to the 1/10,000 inch, and the applied head load was measured in tons.

4.1.2 Faulty telltales

In Pile 1, the insertion of the extensometer rods in the polyethylene tube was very difficult beyound a depth of 21 metre (70 ft). The tube was believed to have collapsed or buckled which increased the friction between the rod and the tube. Therefore, the deflections measured by anchors 9 and 10 are believed to be smaller than the real values and they have been disregarded from the analysis. In Pile 2, an electrical malfunction occurred in the DCDT circuits for anchors numbers 10 and 11, therefore, these two anchors were excluded from in the analysis of Pile 2.

4.1.2 Reliability of telltale data

Preliminary analysis on the reliability of telltale data in analysing load distribution in test loaded piles was carried out by Yu (1984) who found that telltale measurements are influenced by gauge length, type of splices, residual load, and shaft resistance distribution along the pile and concluded the following:

i) Measurements from telltales with either short gauge lengths -shorter than 12 meter (40 ft)- or very long gauge lengths -longer than 30 meter (100 ft)- yield highly contradictory modulus values and are unrepresentative. Results from telltale measurements with gauges lengths between 12 and 30 meter are more consistent and the analysis should be restricted to these.

ii) Telltale measurements across the tube splices are influenced by the splices, but the mechanical Herkules splice did not affect the measured deformations.

4.1.4 Applied load

Pile 1 was loaded in 25-ton increments. The pile failed structurally under an applied load of 550 tons. The break was located 16 m (52 ft) below the ground surface in the uppermost tube splice.

Pile 2 was loaded in 30-ton increments. The pile failed structurally under an applied load of 540 tons. The break was located 22 m (74 ft) below ground surface approximately 6 m (20 ft) from the lower end of the second segment.

Pile 4 was loaded in 30-ton increments. The pile failed structurally when the head load was being increased from 510 to 540 ton. The break was located near the pile head at a depth of 2.3 metre (7 ft).

4.1.5 Pile modulus

According to Parsons and al., 1979, the modulus of the concrete was determined in laboratory testing to be 24 GPA (3.5 million psi). Variation of the pile modulus with stress level during static testing was studied by Yu (1984). Four different approaches, namely, linear approach, non-linear approach, Janbu's tangent modulus approach, and hyperbolic approach, were used to model mathematically the tangent modulus variation with stress level. Only measurements and results of telltales with gauge length between 12 and 30 metre and anchored within the sleeved portion were considered in the formulation.

For strain values smaller than 0.08 %, Yu (1984) found little variation between the axial load in the pile determined using modulus from the four approaches. However, for larger strains, the highest axial load values resulted always from the tangent modulus approach, and the lowest values were obtained from hyperbolic approach. The linear approach gave intermediate axial load values. Yu (1984) also found that the non-linear approach has the highest correlation coefficient and the smallest standard deviation. These observations are presented graphically in Figs. 4.4 and 4.5.

Eq. 4.1 is the formula obtained by Yu (1984) which relates the tangent modulus, E_t to the applied axial load Q for Pile 4.

$$E_t = 34.21*10^9 - 473\ (Q/A_c) \tag{4.1}$$

where E_t and Q/A_c are in Pascal. In imperial units, the equation becomes:

$$E_t = 4.97*10^6 - 473\ (Q/A_c) \tag{4.2}$$

where E_t and Q/A_c are in psi.

Expressing Eqs. 4.1 and 4.2 as a function of strain instead of stress, the equations become:

$$E_t = (34.21*10^9) / (1+473\epsilon) \tag{4.3}$$
$$E_t = (4.97*10^6) / (1+473\epsilon) \tag{4.4}$$

for SI and Imperial units respectively.

4.2 Downdrag Monitoring Data

4.2.1 General

As part of the Keehi testing programme, four telltale-instrumented piles were influenced by downdrag load caused by an embankment surcharge while monitored from July 27, 1977 to Feb 2, 1978.

The observed telltale deflections are presented in Appendix A as Tables A4.1 through A4.4 and plotted versus time in Appendix B as Figs. B4.1 to B4.4.

The total horizontal stresses and the pore pressures were measured by earth pressure cells and by piezometers installed in the piles at various locations. The data are presented in Tables A4.5 through A4.8. The horizontal effective stress presented in the tables was calculated by subtracting the measured total horizontal stress from the measured pore pressure and adding 0.63 psi (44 KPa). The added value corrects for an elevation difference of 17.5 inches (0.2 m) between the two pressure sensors. The change of the pore pressure and of the horizontal effective stress in the four piles with time are plotted in Figs. B4.5 through B4.12.

Figs. B4.13 and B4.14 show a comparison between the calculated values of pore pressure and total overburden pressure using soil data, to the values measured by the piezometers and the earth pressure cells installed in the piles. It is clearly observed from the figures that the pore pressure and the horizontal stress recorded from Piles 7, 8, and 9 are not reliable since the data when plotted fit below the hydrostatic line, and below the calculated total overburden pressure.

4.2.2 Reliability of data

The collected data from the telltales measurement were considered reliable since no serious problems were reported during the installation or the monitoring. The data obtained from the earth pressure cells and from the piezometers readings on Piles 7, 8, and 9 were considered unreliable for the reasons stated in Section 4.2.1. The strain gauges in all the piles were considered unreliable and were disregarded from the analysis.

CHAPTER FIVE

ANALYSIS OF RESULTS

5.1 Loading Test Piles

5.1.1 Introduction

Data obtained from test loading piles are mainly used for the analysis of load distribution in the piles. The parameters used in the analysis should be selected in a way to converge with the condition of the testing procedure and with the pile material. For concrete piles, the deformation modulus is considered an important parameter due to its dependency on the applied load, and therefore should be determined using the methods presented in Chapters 2 and 4.

5.1.2 Analysis of Modulus

Applying and repeating the analysis by Yu (1984), the tangent modulus variation with stress level of the three piles was obtained and plotted in Figs. 5.1 through 5.3 for Piles 1, 2, and 4. Linear regression was used to model the modulus variation with stress.

Eqs. 5.1, 5.2, and 5.3 are mathematical relations obtained by linearly regressing the variation of the tangent modulus with stress level of the three piles. The data were taken from telltales located in the sleeved portions of the piles. It is clearly observed from the figures and from the regression coefficient (r) that the data from Pile 4 resulted in the best regression fit.

<u>Pile 1:</u> $E_t = 3.96*10^6 - 500*(Q/A_c)$ $\quad r = 0.9146 \quad$ (5.1)

<u>Pile 2:</u> $E_t = 4.60*10^6 - 500*(Q/A_c)$ $\quad r = 0.8901 \quad$ (5.2)

<u>Pile 4:</u> $E_t = 4.97*10^6 - 473*(Q/A_c)$ $\quad r = 0.9511 \quad$ (5.3)

where E_t and Q/A_c are in units of psi.

Using Eq. 2.15 to express the moduli as a function of strain instead of stress, non linear relations were obtained given by Eqs. 5.4, 5.5, and 5.6.

<u>Pile 1:</u> $E_t = 3.96*10^6/(1+500*\epsilon)$ (5.4)
<u>Pile 2:</u> $E_t = 4.60*10^6/(1+500*\epsilon)$ (5.5)
<u>Pile 4:</u> $E_t = 4.97*10^6/(1+473*\epsilon)$ (4.4) and (5.6)

where E_t is in psi and ϵ in 10^6 inch/inch.

Using Eq. 2.17 in combination with Eqs. 5.1, 5.2 and 5.3, the secant modulus variation with strain level in each of the three piles can be derived according to the information presented in Chapter 2. The resulting equations are as follows:

<u>Pile 1:</u> $E_s = (7.92*10^3/\epsilon)\ \ln(1 + 500*\epsilon)$ (5.7)
<u>Pile 2:</u> $E_s = (9.20*10^3/\epsilon)\ \ln(1 + 500*\epsilon)$ (5.8)
<u>Pile 4:</u> $E_s = (10.5*10^3/\epsilon)\ \ln(1 + 473*\epsilon)$ (5.9)

The tangent modulus of the three piles can also be plotted versus strain instead of stress, as shown in Figs. 5.4 through 5.6, which gives Eqs. 5.10, 5.11 and 5.12, mathematical relations obtained by linearly regressing the variation of the three piles tangent moduli with strain level.

<u>Pile 1:</u> $E_t = 3.51*10^6 - 935*(\epsilon)$ $r = 0.9204$ (5.10)
<u>Pile 2:</u> $E_t = 4.30*10^6 - 1310*(\epsilon)$ $r = 0.9171$ (5.11)
<u>Pile 4:</u> $E_t = 4.78*10^6 - 1650*(\epsilon)$ $r = 0.9650$ (5.12)

where E_t is in psi and ϵ is in 10^6 inch/inch.

Having the above relations, the secant modulus variation with strain level can be derived using the information presented in Chapter 2. The secant modulus varies vary linearly with strain with a slope equal to one half of the slope of the tangent modulus variation with strain. Applying this concept and Eq. 2.12, the followings relations are obtained.

Pile 1: $E_s = 3.51*10^6 - 468 *(\epsilon)$ (5.13)
Pile 2: $E_s = 4.30*10^6 - 655 *(\epsilon)$ (5.14)
Pile 4: $E_s = 4.78*10^6 - 825 *(\epsilon)$ (5.15)

where E_s is in psi and ϵ in 10^6 inch/inch. In Si units, Eqs. 5.13, 5.14, and 5.15 become:

Pile 1: $E_s = 24.16*10^9 - 468 *(\epsilon)$ (5.16)
Pile 2: $E_s = 29.58*10^9 - 655 *(\epsilon)$ (5.17)
Pile 4: $E_s = 32.90*10^9 - 825 *(\epsilon)$ (5.18)

Fig. 5.7 shows a comparison of the six expressions of E_s, Eqs. 5.7 through 5.9 and Eqs. 5.13 through 5.15.

Comparing the two approaches presented above, it was observed that the results obtained from the second approach are more adequate, since the regression coefficient is higher for the three piles and the derived secant moduli vary linearly with strain level as shown in Fig. 5.7.

The secant modulus relations with strain presented by Eqs. 5.13, 5.14, and 5.15 are used in the analysis of the load distributions, since they were derived from the tangent modulus obtained from telltale readings located in the sleeved portions of the piles where no shaft resistance existed. The relations are considered applicable for the rest of the piles (non sleeve portions) and were used in the load distributions analysis.

5.1.3 Limit Load

The limit loads for the three test loaded piles; Piles 1, 2 and 4, were determined using methods proposed by Brinch Hansen (1963), Davisson (1972), Chin (1971), and by Butler and Hoy (1977) as shown in Figs. 5.8 through 5.13.

The Davisson's offset value (x) as defined by a pile head movement of 4 mm

(0.15 in) plus a toe movement equal to the pile diameter divided by 120 was calculated using Eq. 5.19.

$$x = 4.0 + (D/120) \quad \text{(x and D in mm)} \tag{5.19}$$
$$x = 0.15 + (D/120) \quad \text{(x and D in inch)}$$

For the Keehi piles D = 16.5 inch (420 mm).
Therefore x = 0.15 + 16.5/120 = 0.28 inch or 7.3 mm.

The results of the limit load calculations are presented in Table 5.1.

It is clearly observed from Figs. 5.8 through 5.13 that the plastic state on which Davisson based his criterion was not reached yet, and the offset elastic line falls beyond the range of the load movement curve making the determination of the limit load impossible. The elastic line for each pile was calculated by multiplying, the modulus obtained from the relations given in Section 5.1.2, by the correspondent value of A/L.

The maximum head and toe movements obtained for Piles 1, 2, and 4; were 85 mm (3.3 inch) and 10 mm (0.38) for pile 1, 17 mm (2.8 inch) and 4 mm (0.16 inch) for pile 2, and 99 mm (3.9 in) and 9 mm (0.38 inch) for pile 4, respectively. Obviously, the soil resistance in the lower portions of the piles located in the silty clay layer, was not yet fully mobilized. This observation agreed with the reported values by Fellenius (1984), Vesic (1977) as stated in Chapter 2.

A comparison of the limits loads as obtained from the static test loading and from the three considered methods is shown in Table 5.1. A difference ranging between 43 and 55 % was observed between Chin limit load and the static testing failure load.

5.1.4 Load distribution analysis

Tables 5.2, 5.3, and 5.4, present the calculated average loads in piles 1, 2 and 4. Only applied loads near the upper failure load were considered in the analysis. For Piles 1, 2, and 4, loads level of 550 and 500, 540 and

510, 510 and 450 tons were considered, respectively.

For each pile, only telltale combinations with gauge length between 12 and 30 m were considered. Tables 5.2, 5.3, and 5.4 show the average strain at the middle of the considered telltale, the secant modulus as a function of strain calculated by means of the corresponding mathematical relation developed in Section 5.1.3 and the average load at the mid-point of each telltale using Eq. 2.10.

To obtain the load distributions in the piles, the average load at each telltale level was plotted versus the elevation of the telltale mid-point as shown in Figs. 5.14 through 5.18. Approximative load distributions curves were obtained by first joining the points corresponding to the load values at various elevations by a straight line, which assumes that the unit shaft resistance is constant between telltales. The procedure was then repeated assuming a variable unit soil resistance with depth. The shape of the load distribution will be non linear with a curvature distributions of the assumed unit resistance. The true, or the most reasonable load distribution, is obtained when the areas and on both sides of the straight line and the chosen curve are approximately equal.

To find the load distribution curve, a curvilinear regression was applied to the data trying second and third degree curves. The best fit was compared to the straight-line distribution to arrive at the load distributions given in Figs. 5.14 through 5.18.

The load distributions in the three piles show that the soil resistance mobilized in the upper clay layer ranged between 1.2 and 1.6 MN (140 and 190 ton) at maximum applied loads. It was also observed that in the stiff clay, the mobilized resistance was small since the piles failed structurally before fully mobilizing the shaft resistance. A small toe resistance was also observed in the three piles.

5.1.5 Shaft resistance analysis

The unit shaft resistance distributions in the piles were obtained by differentiating the regression polynomials for the load distribution curve and dividing the result by the unit shaft circumferential area of the piles. The calculation of the mobilized unit shaft resistance in Piles 1, 2, and 4 are presented in Tables 5.5 through 5.9.

The calculated unit shaft resistance mobilized by the piles in the upper soft clay layer, from a depth of 3 metre (10 ft) to a depth of 28 metrer (92 ft), was between 33.5 to 43 KPa (0.35 to 0.45 tsf). In the same layer, the in-situ undrained shear strength increases from 12 KPa (0.1 tsf) at 4.5 metre (15 ft) depth, to 57 KPa (0.6 tsf) at 34 metre (110 ft) depth. The ratio between the mobilized shaft resistance and the undrained shear strength, alpha , was 0.75.

In the lower part of the clay layer at a depth of 45 metre (150 ft), the shaft resistance mobilized by Pile 4 was between 38, and 56 KPa (0.4 and 0.6 tsf). The alpha ratio in this layer was also found to be 0.75.

Below a depth of 53 m (153 ft), in the layer of mixed coral sand and of stiff silty clay, an alpha value of 0.71 was obtained. A maximum unit shaft resistance of 58 to 87 KPa (0.6 to 0.9 tsf) was observed.

Fig. 5.19 shows the shaft resistance mobilized around Piles 1, 2, and 4. For Pile 2, no results were obtained beyond a depth of 36 metre (130 ft) due to the failure of the two lowest telltales. It is observed from Fig. 5.19 that the shaft resistance is approximately constant in the upper soft clay layer for Piles 1, and 2 and linearly increasing for Pile 4. It was also observed that a small resistance was mobilized in the lower portions of the piles.

5.1.6 Calculation of the beta coefficient

The Bjerrum-Burland beta coefficient is the ratio of the unit shaft resistance mobilized at a certain depth to corresponding value of the effective overburden pressure as shown in Table 5.10. The effective overburden pressure was calculated using the soil data and the piezometers readings installed in the soil around Piles 1, 2, and 4 (see Fig. 3.1).The locations of the piezometers are shown in Fig. 3.4. The effective overburden pressure was calculated as the difference between the total stress (soil density times soil depth) and the pore pressure (excess pore pressure measured by piezometers plus hydrostatic pressure).

The calculated beta values presented in Table 5.10, ranged between 0.27 to 0.43 in the soft clay layer, and was 0.38 in the stiff silty clay layer. In the dense sand layer, however, the beta values was between 0.21 to 0.28. Beta-values between 0.22 and 0.28 envelope all the calculated values as shown in Fig 5.19.

The calculated beta values are considered lower bound values since the soil resistance around the piles was not fully mobilized. The results agree with reported by Burland (1973).

5.2 Downdrag Piles

5.2.1 General

The downdrag piles were driven in the same area as the test loaded piles (Fig. 3.4). The piles were driven between June 6 and June 20 of 1977 and monitored for a period of almost one year. An embankment was built around the piles as explained in Chapter 4 to accelerate the rate of consolidation of the soft clay layer. Piles 7 and 9 were coated with bitumen to reduce the effect of the settling soil. Pile 6 met a refusal at a depth of 45 meter (150 ft) in the soft clay layer and Piles 7, 8, and 9 were driven down to the stiff clay layer.

5.2.2 Load distribution analysis

Tables A5.1 through A5.4 present the calculated axial loads in the piles as a function of time. Several data sets obtained at different days after driving were selected and used in the analysis. Single and multiple telltales were used. The tables show the difference in deformation reading induced at each telltale level, the calculated corresponding strain, secant modulus, and the resulted axial load at different locations in the piles. The variation of the secant modulus with strain level was calculated using the formula developed for Pile 4 (Eq. 5.15).

Since the first monitored date for all the piles was July 27, 38 to 60 days after the piles were driven, nothing is known about the deformation developed in the piles between the time of driving and the first monitored date. For analysis, it was assumed that a zero deformation is present in the piles at the first day of downdrag monitoring.

The calculated values of axial loads in the piles was plotted as load versus depth for several dates after driving as shown in Fig. B5.1 through B5.4.

By August 12, 1977, at the end of the 15 day construction period of the fill embankment, the settlement platforms near the piles indicated a ground settlement of 177 mm (7 inches). At this stage, downdrag loads of 0.8 and 0.9 MN (85 and 98 tons) occurred in Piles 6 and 8, respectively. The neutral planes for the two piles were located at depths of 34 and 48 metre (112 and 156 ft), respectively, i.e., in the soft clay layer. By October 6, 1977, the location of the neutral planes in Piles 6, 7, 8, and 9 were well defined where a 0.3 metre (12 inches) ground settlement was observed. By February 6, 1978, the settlement platform indicated a ground surface settlement of 0.43 metre (17 inches). The magnitude of the maximum drag load in Piles 6 and 8 was at this stage 1.8 and 2.3 MN (207 and 254 tons) located at an elevation of 35 and 42 metre (117 and 156 ft), respectively. At this date, a maximum downdrag load of 0.6 and 0.5 MN (68 and 56 tons) was obtained in Piles 7 and 9, respectively.

No furthur analysis of the downdrag data will be made as such analysis lies outside the scope of this report.

5.3 Residual Strain Analysis

5.3.1 General

Residual strain as presented in Chpater 2 is strain induced in the pile as the result of either the installation of the pile or induced from the consolidation of the surrounding soil after installation. Available methods of residual strain analysis are either based on theoretical approach such as numerical methods, or based on empirical methods that consist on recording the pile deformation from the initial time of driving to the time of testing, or on carrying out some special tests on the field such as pull tests and loading and unloading tests. Residual strain when obtained should be added to the measured deformation by the instrument assuming zero deformation at the time of monitoring. Residual strain can be either negative or positive (tension or compression).

5.3.2 Analysis

The Keehi results were analysed assuming a zero locked-in deformation in the piles at the time of initial monitoring. It is considered a point of interest to see how the beta-values change when imposing values of residual strain on the analysis.

For this purpose, maximum residual loads of 100 and 200 tons were assumed to be acting in the piles at the time of testing increasing linearly with depth. The assumed residual loads were transferred to deformation that was added to the deformation values otained from the previous analysis (using telltale data). The resulted load distributions were used to calculate the new shaft resistance in the piles. The beta values were therefore calculated for each value of the assumed residual deformation as shown in Table 5.11, where it is observed that a 100-ton residual load which corresponds to 20 % of the maximum load can cause a 30 % decrease in the beta value around Piles 1, 2, and 4.

Therefore, it was concluded that the beta values obtained by neglecting the residual load in the piles, are overestimated for the test loaded piles.

CHAPTER SIX

CONCLUSIONS

This report presents data from a full scale field testing programme conducted on seven 60 metre (200 Feet) long prestressed concrete piles investigated for construction of Keehi-Interchange at Honolulu, Hawaii, in 1977.

The objectives of this study were, to verify the evaluation of the deformation modulus of the Keehi prestressed concrete piles carried out by Yu (1984), to determine the axial load distribution in the test loaded piles, and to evaluate the Bjerrum-Burland Beta coefficient.

The results of the analysis suggest that for the prestressed concrete piles, the deformation modulus should be taken to vary linearly with strain level and not with stress level as carried out by Yu (1984), also the tangent modulus variation with strain level should be used to derive the secant modulus variation with strain level. The derived equations should be used in the analysis of the load distributions in the piles. The following equations resulted for Piles 1, 2, and 4.

	Imperial units	SI units
Pile 1:	$E_s = 3.51*10^6 - 468*(\epsilon)$	$E_s = 24.16*10^9 - 468*(\epsilon)$
Pile 2:	$E_s = 4.30*10^6 - 655*(\epsilon)$	$E_s = 29.56*10^9 - 655*(\epsilon)$
Pile 4:	$E_s = 4.78*10^6 - 825*(\epsilon)$	$E_s = 32.90*10^9 - 825*(\epsilon)$

where E_s is the secant modulus in psi and KPa, and ϵ is the strain in 10^6 inch /inch and mm/mm.

The shaft resistance obtained from the load distribution analysis in the test loaded piles, was compared to the effective ovberburden pressure and to the undrained shear strength of the soil. Beta-values between 0.22 and 0.28 envelope all the calculated value. The alpha-value was almost constant at 0.73 in the soft clay layer and in the stiff clay layer.

The analysis suggested also that when the residual load are not accounted for in the analysis, the calculated beta-values are overestimated in the loading test piles.

More study should be carried out on the downdrag piles to calculate the beta factor around these piles. Furthur study is also required to determine the amount of residual stress the piles.

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TABLES

TABLE 3.1 GENERALISED SOIL PROFILE (AFTER PARSONS, 1979)

Elev m (Ft)	Soil Description	Density kg/m^3 (Pcf)	In-Situ Vane Shear Strength KPa (Psf)	PI	Friction Angle (deg)	K_o
+1.5 (+5.0)	Medium to Compact coral sand fill	1762 (110)				
-2.4 (-8.0)	Very Soft/muck Silty clay	1560 (97.4)	12 (250)			
-34 (-110)	Soft to muck Silty clay	1560 (97.4)	57 (1200)	45 - 47	12 - 15	0.690
-41 (-134)	Firm silty clay	1640 (102.4)	83 (1730)	51	8	0.760
-44 (-144)	Medium to dense coral sand	1800 (112.4)	83 (1730)			
-53 (-175)	Firm to stiff Silty clay	1800 (112.4)		38	11 - 36	0.60
-57 (-187)	Stiff to very stiff silty clay	1800 (112.4)	144 (3000)			

TABLE 3.2: DESCRIPTIONS OF THE KEEHI INTERCHANGE TEST PILES

PILE No	EMBEDDED DEPTH m (Ft)	PILE STICKUP m (Ft)	SLEEVE x LENGTH m (Ft)	SPLICE TYPE	INSTRUMENTATION
1	59 (195)	0.7 (2.2)	5 (16)	tube spliced	Telltales and strain gauges
2	51 (167)	0.6 (2.1)	5 (16)	Herkules spliced	Telltales
4	62 (204)	0.7 (2.7)	27 (90)	Herkules spliced	Telltales
6	50 (164)	0.6 (1.9)	8.4 (27.5)	Herkules spliced	Telltales, strain gauges, pressure cells, piezometers
7*	66 (216)	0.85 (2.8)	8.4 (27.5)	Herkules spliced	Telltales, strain gauges, pressure cells, piezometers
8	63 (206)	0.7 (2.3)	8.4 (27.5)	Herkules spliced	Telltales, strain gauges, pressure cells, piezometers
9*	66 (216)	0.8 (2.6)	8.4 (27.5)	Herkules spliced	Telltales, strain gauges, pressure cells, piezometers

All piles are octagonal prestressed precast concrete piles with a side to side diameter of 420 mm (16.5 inch), and a net area of 0.13 m^2 (200 in^2).

* Piles 7 and 9 were coated with 1/16 and 3/16 inch of bitumen, respectively. The bitumen coated length was between 26 to 31 m (85 to 100 ft), respectively.

x The sleeve length is below the ground surface.

TABLE 3.3: INSTRUMENTATIONS LOCATIONS IN PILE 1 TO PILE 9.

ALL VALUES ARE IN FEET.

INSTRUMENTS DESCRIPTION	PILE 1	PILE 2	PILE 4	PILE 6	PILE 7	PILE 8	PILE 9
Head Elev	+7.15	+7.12	+7.70	+18.86	+19.76	+19.31	+19.62
Toe Elev	-187.85	-159.99	-196.30	-145.14	-196.24	-186.69	-196.38
Bottom of sleeve	-11.00	-11.00	-85.00	-22.50	-22.50	-22.50	-22.50
TELLTALES (FROM THE HEAD OF THE PILES)							
T1	19.31	9.61	17.65	12.95	30.56	28.49	31.00
T2	34.24	19.77	31.63	29.87	40.56	43.95	44.04
T3	44.24	39.82	46.66	39.95	50.52	54.48	59.55
T4	67.90	54.48	56.96	59.86	60.52	68.49	68.52
T5	92.33	69.71	80.55	75.99	83.52	91.98	90.98
T6	114.34	89.63	103.61	84.48	107.46	116.98	115.98
T7	124.35	99.59	126.64	94.5	130.5	138.49	139.48
T8	147.42	124.61	136.64	117.63	140.53	148.49	148.51
T9	167.55	144.61	164.62	139.85	162.48	171.98	171.00
T10	181.77	157.66	194.64	162.43	185.54	193.99	192.09
T11	—	166.69	205.96	—	199.13	—	—
EARTH PRESSURE CELLS AND PORE PRESSURE PIEZOMETERS							
LEVEL 1	-	-	-	20.8	146	154	154
LEVEL 2	-	-	-	50.9	176.2	184.2	184.2
LEVEL 3	-	-	-	80.9	206.1	214.1	214
SPLICES							
SPLICE 1	52	13.9	52	10.9	64	64	64
SPLICE 2	132	93.9	132	90.9	144	144	144

TABLE 3.4 SCHEDULE OF THE KEEHI PILES

PILE	DATE DRIVEN	DATE TESTED	MONITORING PERIOD
1	June 7, 1977	Sept 15, 1977	-
2	June 8, 1977	Sept 27, 1977	-
4	June 16, 1977	Oct 4, 1977	-
6	June 6, 1977	-	July 27, 1977 to Feb 6, 1978
7	June 14, 1977	-	July 27, 1977 to Feb 6, 1978
8	June 13, 1977	-	July 27, 1977 to Feb 6, 1978
9	June 20, 1977	-	July 27, 1977 to Jan 18, 1978

TABLE 4.1A: STATIC LOADING TEST RESULTS, PILE 1, TELLTALES 1 - 5.

HEAD LOAD (Tons)	HEAD MOVEMENT *10000 (Inches)	TELLTALE DEFLECTION *10000 (Inches)					
		TELLTALE DEPTH (Feet)	T1 19.31	T2 34.24	T3 44.24	T4 67.9	T5 92.33
25	672		171	239	235	256	233
50	1465		345	522	569	552	686
75	2452		537	822	961	886	1385
100	3747		788	1237	1487	1322	2314
125	4805		981	1563	1914	1670	3132
150	5379		1079	1724	2128	1840	3512
175	6631		1312	2122	2632	2254	4427
200	7719		1501	2441	3048	2591	5173
225	8771		1688	2754	3448	2981	5868
250	9888		1898	3114	3903	3534	6693
275	10988		2111	3476	4368	4189	7513
300	12272		2332	3845	4835	4873	8341
325	13521		2553	4216	5309	5553	9182
350	15390		2864	4737	5979	6525	10414
375	17361		3181	5268	6664	7493	11647
400	18999		3462	5738	7282	8360	12738
425	20811		3746	6214	7912	9225	13836
450	23841		4117	6831	8701	10299	15222
475	25894		4441	7377	9410	11250	16479
500	28439		4851	8051	10248	12418	18006
525	30997		5285	8761	11132	13665	19671
550	33187		5651	9348	11865	14718	21098

TABLE 4.1B: STATIC LOADING TEST RESULTS, PILE 1, TELLTALES 6 - 10.

HEAD LOAD (Tons)	HEAD MOVEMENT *10000 (Inches)	TELLTALE DEFLECTION *10000 (Inches)				
	TELLTALE DEPTH (Feet)	T6 19.31	T7 124.3	T8 147.4	T9 167.6	T10 181.7
25	672	222	378	269	238	229
50	1465	806	1056	907	556	534
75	2452	1623	1908	1788	933	987
100	3747	2730	3085	3010	1461	2026
125	4805	3740	4148	4160	1887	3171
150	5379	4206	4632	4689	2098	3736
175	6631	5359	5866	6025	2593	5052
200	7719	6303	6848	7997	3003	6163
225	8771	7189	7767	8087	3416	7188
250	9888	8242	8860	9274	3917	8404
275	10988	9276	9939	10437	4441	9623
300	12272	10321	11022	11587	5130	10828
325	13521	11374	12101	12736	6172	12041
350	15390	12890	13646	14436	7878	13816
375	17361	14392	15218	16174	9668	15654
400	18999	15753	16632	17726	11359	17329
425	29811	17127	18033	19310	13062	19091
450	23841	18807	19793	21277	15120	21104
475	25894	20320	21351	23011	16991	22929
500	28439	22141	23206	25107	19242	25118
525	30997	24042	25170	27281	21643	27384
550	33187	25662	26843	29205	23639	29399

TABLE 4.2A: STATIC LOADING TEST RESULTS, PILE 2, TELLTALES 1-6.

HEAD LOAD (Tons)	HEAD MOVEMENT *10000 (Inches)	TELLTALE DEPTH (Feet)	T1 9.6	T2 19.8	T3 39.8	T4 54.5	T5 69.7	T6 89.6
			TELLTALE DEFLECTION *10000 (Inches)					
30	765		8	113	316	422	473	558
60	1638		172	373	696	955	1086	1243
90	2626		291	597	1073	1490	1704	1970
120	3596		387	789	1411	1985	2310	2675
150	4739		486	986	1792	2514	2941	3435
180	5859		576	1175	2158	3029	3560	4191
210	7227		673	1385	2560	3607	4236	5061
240	8549		762	1578	2930	4149	4900	5901
270	10011		853	1788	3334	4713	5594	6809
300	10319		946	1996	3713	5275	6231	7726
330	12904		1035	2216	4133	5869	6981	8702
360	14618		1135	2451	4597	6532	7816	9782
390	16324		1241	2662	5016	7157	8581	10820
420	18269		1378	2925	5554	7914	9492	12041
450	20172		1518	3164	6070	8640	10376	13229
480	22295		1678	3438	6639	9437	11351	14536
510	25471		1843	3742	7270	10380	12467	16066
540	28013		2018	4047	7886	11154	13507	16541
---	32728		1008	1856	3359	3991	7397	7909

TABLE 4.2B: STATIC LOADING TEST RESULTS, PILE 2, TELLTALES 7 - 11.

HEAD LOAD (Tons)	HEAD MOVEMENT *10000 (Inches)	TELLTALE DEPTH (Feet)	T7 99.6	T8 124.6	T9 144.6	T10 157.7	T11 166.7
			TELLTALE DEFLECTION *10000 (Inches)				
30	765		586	608	318	572	240
60	1638		1330	1411	1031	1109	907
90	2626		2131	2127	1628	1972	1916
120	3596		2921	3095	2566	2976	2716
150	4739		3783	4040	3742	4034	3848
180	5859		4645	5089	4788	5108	4929
210	7227		5639	6278	6010	6455	6266
240	8549		6604	7416	7092	7756	7604
270	10011		7645	8668	8384	8725	8972
300	10319		8691	9922	9673	10689	10554
330	12904		9800	11270	11122	12271	12005
360	14618		11026	12776	12753	13975	13832
390	16324		12191	14156	14212	15530	15476
420	18269		13537	15754	15919	17357	17254
450	20172		14841	17337	17551	18555	18962
480	22295		16276	19058	19334	20913	20881
510	25471		17935	21013	21368	22988	23046
540	28013		19497	22864	23342	24950	25590
---	32728		25797	25226	23777	24809	26375

TABLE 4.3A: STATIC LOADING TEST RESULTS, PILE 4, TELLTALES 1 - 6.

HEAD LOAD (Tons)	HEAD MOVEMENT *10000 (Inches)	TELLTALE DEPTH (Feet)	TELLTALE DEFLECTION *10000 (Inches) T1 17.6	T2 31.6	T3 46.7	T4 56.6	T5 80.6	T6 103.6
30	571		72	190	321	350	442	467
60	1546		231	441	667	781	1053	1146
90	2559		387	693	1021	1227	1711	1895
120	3809		620	978	1427	1718	2448	2726
150	5141		775	1247	1807	2182	3152	3531
180	6460		916	1510	2184	2639	3862	4341
210	8005		1075	1792	2586	3123	4631	5211
240	9378		1215	2065	2986	3596	5396	6091
270	11105		1382	2367	3418	4199	6212	7032
300	12772		1544	2678	3872	4621	7052	8024
330	14698		1713	2993	4332	5148	7876	9018
360	16604		1924	3347	4841	5727	8762	10120
390	18627		2141	3722	5385	6343	9684	11307
420	20974		2380	4130	5977	7007	10677	12596
450	23642		2655	4571	6608	7709	11725	13954
480	26659		2988	5068	7314	8483	12855	15439
510	29722		3371	5615	8105	9345	14072	17053
---	39276		18818	18249	19518	19412	22078	26402

TABLE 4.3B: STATIC LOADING TEST RESULTS, PILE 4, TELLTALES 7 - 11.

HEAD LOAD (Tons)	HEAD MOVEMENT *10000 (Inches)		TELLTALE DEFLECTION *10000 (Inches)				
		TELLTALE	T7	T8	T9	T10	T11
		DEPTH (Feet)	126.6	136.6	164.6	194.6	206
30	571		565	524	649	592	914
60	1546		1330	1322	1534	1447	1742
90	2559		2187	2220	2545	2427	2696
120	3809		3160	3230	3657	3523	3610
150	5141		4098	4218	4778	4640	4757
180	6460		5053	5236	5960	5825	5985
210	8005		6109	6355	7280	7132	7366
240	9378		7169	7468	8638	8471	8778
270	11105		8272	8631	10090	9934	10305
300	12772		9412	9866	11640	11514	11894
330	14698		10567	11117	13186	13124	13474
360	16604		11841	12475	14857	14927	15234
390	18627		13187	13943	16636	16892	17164
420	20974		14672	15561	18538	19085	19337
450	23642		16271	17297	20622	21460	21718
480	26659		17996	19208	22863	24104	24302
510	29722		19903	21273	25283	26999	27096
---	38276		26371	28917	32213	32740	35637

TABLE 5.1: LIMIT LOAD

	PILE 1	PILE 2	PILE 4
LIMIT LOAD IN KN (TONS).			
MAXIMUM LOAD	4900 (550)	4800 (540)	4500 (510)
DAVISSON	-	-	-
CHIN	9800 (1100)	9800 (1100)	8000 (900)
BUTLER & HOY	N/A	N/A	N/A
MAXIMUM MOVEMENT IN MM (INCHES).			
HEAD	85 (3.33)	71 (2.80)	99 (3.90)
TOE	10 (0.38)	4 (0.16)	9 (0.36)
SLOPE OF ELASTIC LINE IN KN/M (TONS/INCH).*			
AE / L	42 (121)	59 (171)	54 (156)

* The calculations of EA / L for Piles 1, 2, and 4 are shown below.

<u>Pile 1:</u> Maximum strain $= 1348 \cdot 10^{-6}$ Inch/Inch.
$E_s = 2.87 \cdot 10^{6}$ psi. (Eq. 5.13)
$L = 197$ Feet.
$A = 200\ In^2$

---> $E\ A/L = 121$ Tons/Inch.

<u>Pile 2:</u> Maximum strain $= 1279 \cdot 10^{-6}$ Inch/Inch.
$E_s = 3.46 \cdot 10^{6}$ psi. (Eq. 5.14)
$L = 169$ Feet.
$A = 200\ In^2$

---> $E\ A/L = 171$ Tons/Inch.

<u>Pile 4:</u> Maximum strain $= 1096 \cdot 10^{-6}$ Inch/Inch.
$E_s = 3.88 \cdot 10^{6}$ psi. (Eq. 5.15)
$L = 207$ Feet.
$A = 200\ In^2$

---> $E\ A/L = 156$ Tons/Inch.

Table 5.2A: AXIAL LOAD COMPUTATION IN PILE 1, HEAD LOAD OF 550 TONS.

Pile Length = 197 Feet.
Secant Modulus Es = 3.51 - 0.000468*strain.
Derived from
Tangent Modulus Et = 3.51 - 0.000935*strain.

TELLTALES NO#		READING DIFFERENCE E-4(In)	GAUGE LENGTH (Feet)	STRAIN *E-6 (In/In)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	9348.00	34.24	2275.12	2.45	556.32	17.00
0	3	11865.00	44.24	2234.97	2.46	550.70	22.00
0	4	14718.00	67.90	1806.33	2.66	481.32	34.00
0	5	21098.00	92.33	1904.22	2.62	498.68	46.00
1	5	15447.00	73.02	1762.87	2.68	473.33	56.00
2	5	11750.00	58.09	1685.60	2.72	458.68	63.00
3	5	9233.00	47.99	1603.29	2.76	442.45	68.00
2	6	16341.00	80.10	1700.06	2.71	461.46	74.00
3	6	13797.00	70.00	1642.50	2.74	450.26	79.00
3	7	14978.00	80.01	1560.01	2.78	433.67	84.00
4	8	14487.00	79.52	1518.17	2.80	425.01	108.00
5	8	8107.00	55.09	1226.33	2.94	360.06	120.00
6	8	3543.00	33.08	892.53	3.09	276.00	131.00
7	8	2362.00	23.08	852.83	3.11	265.30	136.00

TABLE 5.2B: AXIAL LOAD COMPUTATION IN PILE 1, HEAD LOAD OF 500 TONS.

Pile Length = 197 Feet.
Secant Modulus Es = 3.51 - 0.000468*strain.
Derived from
Tangent Modulus Et = 3.51 - 0.000935*strain.

TELLTALES NO		READING DIFFERENCE E-4(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	8051.00	34.24	1959.45	2.59	508.08	17.00
0	3	10248.00	44.24	1930.38	2.61	503.17	22.00
0	4	12418.00	67.90	1524.05	2.80	426.24	34.00
0	5	18006.00	92.33	1625.15	2.75	446.82	46.00
1	5	13155.00	73.02	1501.30	2.81	421.47	56.00
2	5	9955.00	58.09	1428.10	2.84	405.82	63.00
3	5	7758.00	47.99	1347.16	2.88	387.92	68.00
2	6	14090.00	80.10	1465.88	2.82	413.96	74.00
3	6	11893.00	70.00	1415.83	2.85	403.14	79.00
3	7	12958.00	80.01	1349.62	2.88	388.47	84.00
4	8	12689.00	79.52	1329.75	2.89	383.99	108.00
5	8	7101.00	55.09	1074.15	3.01	323.03	120.00
6	8	2966.00	33.08	747.18	3.16	236.13	131.00
7	8	1901.00	23.08	686.38	3.19	218.87	136.00

TABLE 5.3A: AXIAL LOAD COMPUTATION IN PILE 2, HEAD LOAD OF 540 TONS.

Pile Length = 169 Feet.
Secant Modulus Es = 4.3 - 0.000655*strain.
Derived from
Tangent Modulus Et = 4.3 - 0.00131*strain.

TELLTALES NC		READING DIFFERENCE E-4(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	4047.00	19.80	1703.28	3.18	542.38	9.90
0	3	7886.00	39.80	1651.17	3.22	531.43	19.90
0	4	11154.00	54.50	1705.50	3.18	542.84	27.25
0	5	13507.00	69.70	1614.90	3.24	523.59	34.85
1	5	11489.00	60.10	1593.04	3.26	518.78	39.65
0	6	17541.00	89.60	1631.42	3.23	527.18	44.80
1	6	15523.00	80.00	1616.98	3.24	524.04	49.60
1	7	17479.00	90.00	1618.43	3.24	524.36	54.60
2	7	15450.00	79.80	1613.41	3.24	523.26	59.70
3	7	11611.00	59.80	1618.03	3.24	524.27	69.70
4	7	8343.00	45.10	1541.57	3.29	507.22	77.05
3	8	14978.00	84.80	1471.89	3.34	491.01	82.20
5	8	9357.00	54.90	1420.31	3.37	478.60	97.15
6	9	5801.00	55.00	878.94	3.72	327.34	117.10
7	9	3845.00	45.00	712.04	3.83	272.97	122.10

TABLE 5.3B: AXIAL LOAD COMPUTATION IN PILE 2, HEAD LOAD OF 510 TONS.

Pile Length = 169 Feet.
Secant Modulus Es = 4.3 - 0.000655*strain.
Derived From
Tangent Modulus Et = 4.3 - 0.00131*strain.

TELLTALES NO		READING DIFFERENCE E-4(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	3742.00	19.80	1574.92	3.27	514.75	9.90
0	3	7270.00	39.80	1522.19	3.30	502.78	19.90
0	4	10380.00	54.50	1587.16	3.26	517.48	27.25
0	5	12467.00	69.70	1490.55	3.32	495.41	34.85
1	5	10624.00	60.10	1473.10	3.34	491.30	39.65
0	6	16066.00	89.60	1494.23	3.32	496.28	44.80
1	6	14223.00	80.00	1481.56	3.33	493.30	49.60
1	7	16092.00	90.00	1490.00	3.32	495.28	54.60
2	7	14193.00	79.80	1482.14	3.33	493.43	59.70
3	7	10665.00	59.80	1486.20	3.33	494.39	69.70
4	7	7555.00	45.10	1395.97	3.39	472.63	77.05
3	8	13743.00	84.80	1350.53	3.42	461.26	82.20
5	8	8546.00	54.90	1297.21	3.45	447.58	97.15
6	9	5302.00	55.00	803.33	3.77	303.16	117.10
7	9	3433.00	45.00	635.74	3.88	246.90	122.10

TABLE 5.4A: AXIAL LOAD COMPUTATION IN PILE 4, HEAD LOAD OF 510 TONS.

Pile Length = 207 Feet.
Secant Modulus Es = 4.78 - 0.000825*strain.
Derived from
Tangent Modulus Et = 4.78 - 0.001650*strain.

TELLTALES NO		READING DIFFERENCE E-4(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD. (Feet)
0	3	8105.00	46.70	1446.00	3.59	518.69	23.35
0	4	9345.00	56.60	1376.00	3.64	501.52	28.30
0	5	14072.00	80.60	1455.00	3.58	520.84	40.30
1	5	10701.00	63.00	1415.00	3.61	511.19	49.10
2	5	8457.00	49.00	1438.00	3.59	516.77	56.10
1	6	13862.00	86.00	1326.00	3.69	488.77	60.60
2	6	11438.00	72.00	1324.00	3.69	488.25	67.60
3	6	8949.00	56.90	1310.00	3.70	484.60	75.15
2	7	14288.00	95.00	1253.00	3.75	469.41	79.10
4	6	7708.00	47.00	1367.00	3.65	499.26	80.10
3	7	11798.00	79.90	1230.00	3.77	463.13	86.65
			End of The Sleeved Portion.				89.00
4	7	10558.00	70.00	1257.00	3.74	470.49	91.65
4	8	11928.00	80.00	1242.00	3.76	466.41	96.65
5	7	5831.00	46.00	1056.00	3.91	412.77	103.60
5	8	7201.00	56.00	1072.00	3.90	417.61	108.60
5	9	11211.00	84.00	1112.00	3.86	429.52	123.00
6	9	8230.00	61.00	1124.00	3.85	433.04	134.10
6	10	9946.00	91.00	911.00	4.03	366.99	149.10
7	10	7096.00	68.00	870.00	4.06	353.42	160.60
8	10	5726.00	58.00	823.00	4.10	337.51	165.60
8	11	5823.00	69.40	699.00	4.20	293.81	171.30
9	11	1813.00	41.40	365.00	4.48	163.48	185.40

TABLE 5.4B: AXIAL LOAD COMPUTATION IN PILE 4, HEAD LOAD OF 450 TONS.

Pile Length = 207 Feet.
Secant Modulus Es = 4.78 - 0.000825*strain.
Derived from
Tangent Modulus Et = 4.78 - 0.001650*strain.

TELLTALES NO		READING DIFFERENCE E-4(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD. (Feet)
0	3	6608.00	46.70	1179.16	3.81	448.93	23.35
0	4	7709.00	56.60	1135.01	3.84	436.25	28.30
0	5	11725.00	80.60	1212.26	3.78	458.22	40.30
1	5	9070.00	63.00	1199.74	3.79	454.73	49.10
2	5	7154.00	49.00	1216.67	3.78	459.44	56.10
1	6	11299.00	86.00	1094.86	3.88	424.45	60.60
2	6	9383.00	72.00	1086.00	3.88	421.81	67.60
3	6	7246.00	56.90	1061.22	3.90	414.35	75.15
2	7	11700.00	95.00	1026.32	3.93	403.68	79.10
4	6	6245.00	47.00	1107.27	3.87	428.13	80.10
3	7	9663.00	79.90	1007.82	3.95	397.94	86.65
			End of The Sleeved Portion.				89.00
4	7	8526.00	70.00	1015.00	3.94	400.18	91.65
4	8	9588.00	80.00	998.75	3.96	395.11	96.65
5	7	4546.00	46.00	823.55	4.10	337.70	103.60
5	8	5572.00	56.00	829.17	4.10	339.62	108.60
5	9	8897.00	84.00	882.64	4.05	357.63	123.00
6	9	6668.00	61.00	910.93	4.03	366.97	134.10
6	10	7506.00	91.00	687.36	4.21	289.58	149.10
7	10	5189.00	68.00	635.91	4.26	270.60	160.60
8	10	4163.00	58.00	598.13	4.29	256.39	165.60
8	11	4421.00	69.40	530.86	4.34	230.50	171.30
9	11	1096.00	41.40	220.61	4.60	101.44	185.40

Table 5.5: CALCULATION OF SHAFT RESISTANCE, PILE 1, Q = 550 TONS

Equation presenting the load distribution with depth:

$$y = -207.7 + 0.052\ Q + 0.000559\ Q^2$$

$$\frac{dy}{dQ} = 0.052 + 0.0011\ Q$$

$$r_s = \frac{1}{A_s} \frac{dQ}{dy}$$

LOAD (Q) (Tons)	DEPTH (y) (Feet)	dy/dQ (Feet/Tons)	A_s (Ft^2/Ft)	r_s (tsf)
550	10	.657	4.55	0.334
548	11	.655	4.55	0.335
542	15	.648	4.55	0.339
498	43	.599	4.55	0.366
473	58	.572	4.55	0.384
458	66	.556	4.55	0.395
442	75	.538	4.55	0.408
433	81	.528	4.55	0.416
425	84	.519	4.55	0.423
360	116	.448	4.55	0.490
320	134	.404	4.55	0.544
275	151	.355	4.55	0.619

Table 5.6 : CALCULATION OF SHAFT RESISTANCE, PILE 1, Q = 500 TONS

Equation presenting the load distribution with depth:

$$y = -225 + 0.231\ Q + 0.000369\ Q^2$$

$$\frac{dy}{dQ} = 0.231 + 0.000793\ Q$$

$$r_s = \frac{1}{A_s}\ \frac{dQ}{dy}$$

LOAD (Q) (Tons)	DEPTH (y) (Feet)	dy/dQ (Feet/Tons)	A_s (Ft^2/Ft)	r_s (tsf)
490	23	.198	4.55	0.354
446	48	.585	4.55	0.376
422	62	.565	4.55	0.388
405	71	.552	4.55	0.398
387	80	.538	4.55	0.408
383	82	.535	4.55	0.411
323	92	.487	4.55	0.451
250	144	.429	4.55	0.512
236	150	.418	4.55	0.525
219	157	.405	4.55	0.543

Table 5.7: CALCULATION OF SHAFT RESISTANCE, PILE 2, Q = 500 TONS

Equation presenting the load distribution with depth:

$$y = -250 + 2.26\ Q - 0.00327\ Q^2$$

$$\frac{dy}{dQ} = 2.26 - 0.00654\ Q$$

$$r_s = \frac{1}{A_s}\ \frac{dQ}{dy}$$

LOAD (Q) (Tons)	DEPTH (y) (Feet)	dy/dQ (Feet/Tons)	A_s (Ft^2/Ft)	r_s (Tsf)
540	17	-	4.55	-
542	24	1.28	4.55	0.171
531	28	1.21	4.55	0.181
523	38	1.16	4.55	0.189
518	43	1.12	4.55	0.195
507	55	1.06	4.55	0.208
491	71	0.95	4.55	0.231
450	104	0.68	4.55	0.321
400	131	0.35	4.55	0.628

Table 5.8: CALCULATION OF SHAFT RESISTANCE, PILE 4, Q = 510 TONS

Equation presenting the load distribution with depth:

$$y = -198 - 0.0955\ Q + 0.000646\ Q^2$$

$$\frac{dy}{dQ} = -0.0955 + 0.00129\ Q$$

$$r_s = \frac{1}{A_s}\ \frac{dQ}{dy}$$

LOAD (Q) (Tons)	DEPTH (y) (Feet)	dy/dQ (Feet/Tons)	A_s (Ft^2/Ft)	r_s (tsf)
470	102	.504	4.55	0.430
466	103	.505	4.55	0.434
412	129	.432	4.55	0.509
417	127	.442	4.55	0.496
429	121	.457	4.55	0.479
366	147	.376	4.55	0.583
353	152	.359	4.55	0.611
337	157	.338	4.55	0.650
293	171	.282	4.55	0.778

Table 5.9: CALCULATION OF SHAFT RESISTANCE, PILE 4, Q = 450 TONS

Equation presenting the load distribution with depth:

$$y = -205 + 0.0426\ Q + 0.00053\ Q^2$$

$$\frac{dy}{dQ} = 0.0426 + 0.00107\ Q$$

$$r_s = \frac{1}{A_s}\ \frac{dQ}{dy}$$

LOAD (Q) (Tons)	DEPTH (y) (Feet)	dy/dQ (Feet/Tons)	A_s (Ft^2/Ft)	r_s (tsf)
450	89			
400	103	.471	4.55	0.467
395	105	.465	4.55	0.472
390	107	.459	4.55	0.478
370	116	.437	4.55	0.502
366	117	.433	4.55	0.507
357	122	.424	4.55	0.581
289	148	.352	4.55	0.625
270	155	.332	4.55	0.662
256	159	.316	4.55	0.694
230	167	.288	4.55	0.761
171	182	.255	4.55	0.975
101	194	.150	4.55	1.460

TABLE 5.10: CALCULATION OF THE BETA-COEFFICIENT IN PILES, 1, 2, AND 4.

Depth (Feet)	Soil Type	Soil Density (pcf)	Total Stress (tsf)	Pore[1] Pressure (tsf)	Effective Stress (tsf)	Shaft[2] Resistance (tsf)	Beta
13	Fill	110.0	0.72				
15	Soft Silty Clay	97.4	0.82	0.31	0.51	0.35	0.74-
43			2.17	1.33	0.84	0.36	0.43
66			3.29	2.26	1.03	0.39	0.38
81			4.03	2.72	1.31	0.42	0.32
116			5.73	3.90	1.83	0.49	0.27
134	Firm Silty Clay	102.4	6.61	4.07	2.54	0.54	0.21
152	Medium to Dense Sand	112.4	7.53	4.58	2.93	0.61	0.21
157			7.81	4.74	3.07	0.65	0.21
182	Firm to Stiff Silty Clay	112.4	9.20	5.52	3.67	0.98	0.28
194			9.88	5.89	3.99	1.46	0.37

(1) Obtained from piezometers data installed around Piles 1, 2, and 4.
(2) Average from Piles 1, 2, and 4.

TABLE 5.11 : VARIATION OF THE BETA-COEFFICIENT WITH RESIDUAL LOADS

Depth (Feet)	r_s (tsf)			σ_v' (tsf)	Beta Coefficient		
Residual Load (Tons)	0	100	200		0	100	200
43	0.36	0.35	0.32	0.84	0.43	0.42	0.38
66	0.39	0.37	0.35	1.03	0.38	0.36	0.34
81	0.42	0.38	0.35	1.31	0.32	0.29	0.27
116	0.49	0.43	0.38	1.83	0.27	0.23	0.21
134	0.54	0.46	0.39	2.54	0.21	0.18	0.15
152	0.61	0.48	0.32	2.93	0.21	0.16	0.11
182	0.98	0.68	0.52	3.67	0.27	0.18	0.14
194	1.46	0.85	0.61	3.99	0.37	0.21	0.15

F I G U R E S

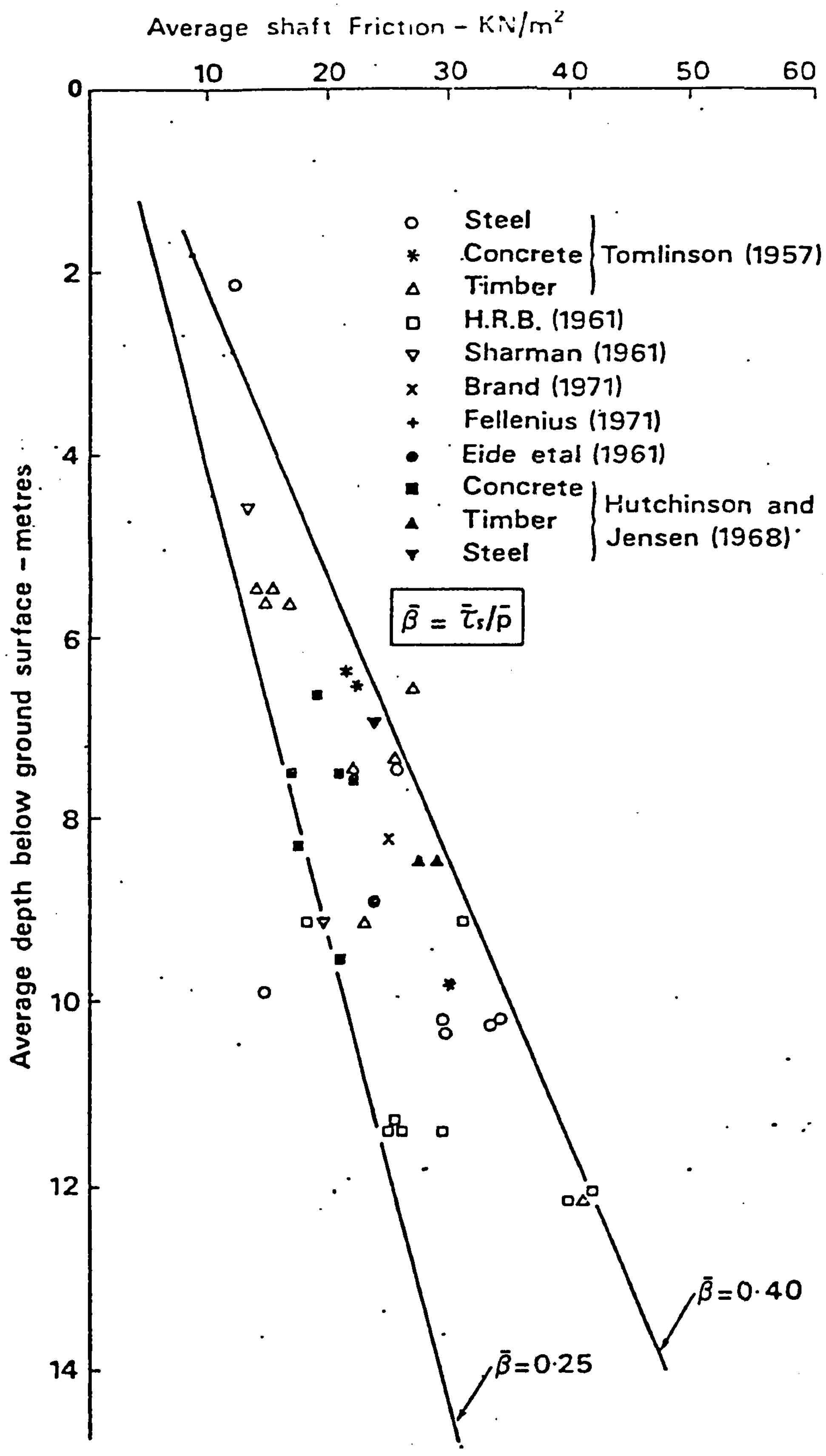


Fig.2.1: Relationship between the average shaft resistance and average depth for piles driven in soft clay (after Burland, 1973).

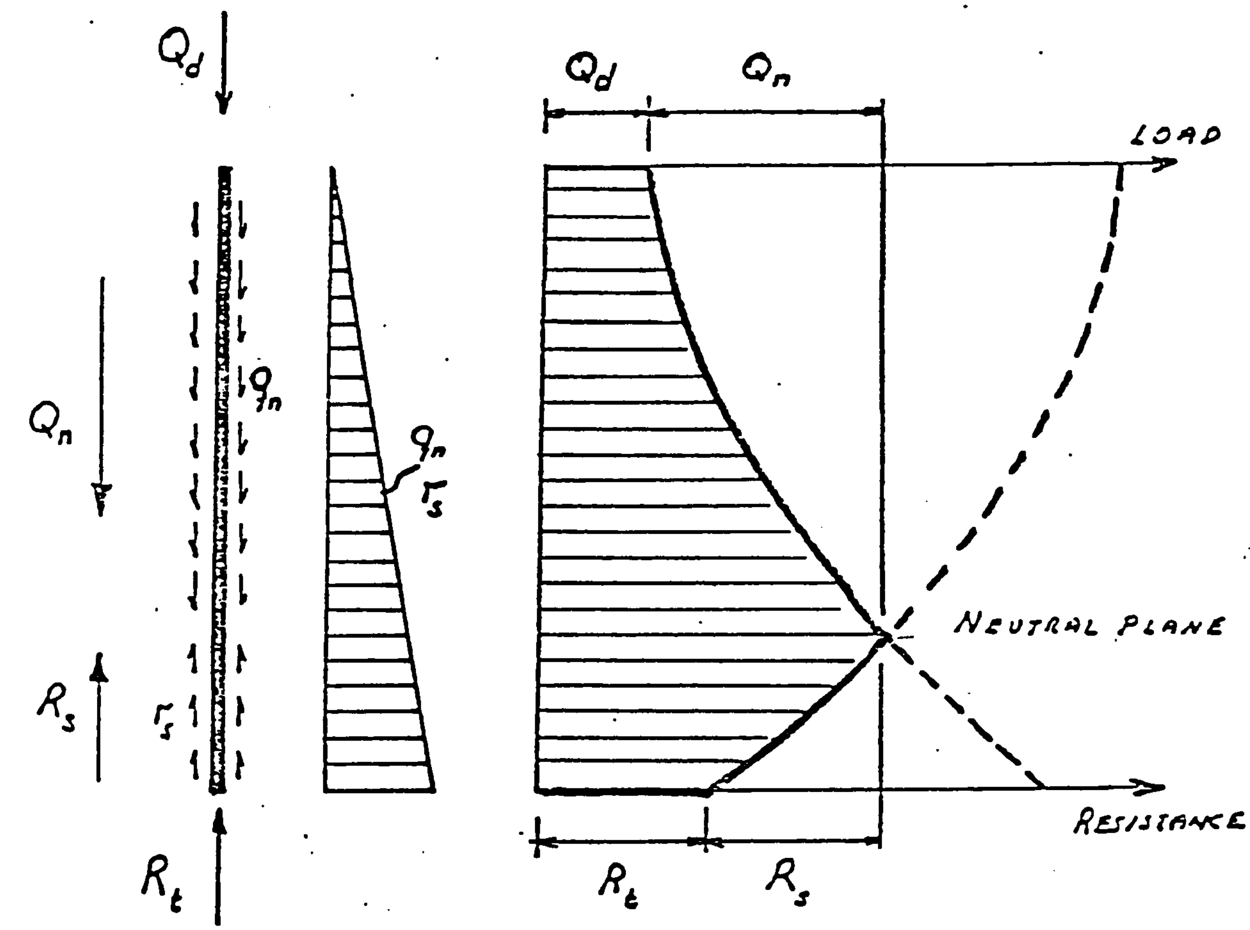


Fig. 2.2: Definition and construction of the neutral plane (after Fellenius, 1984).

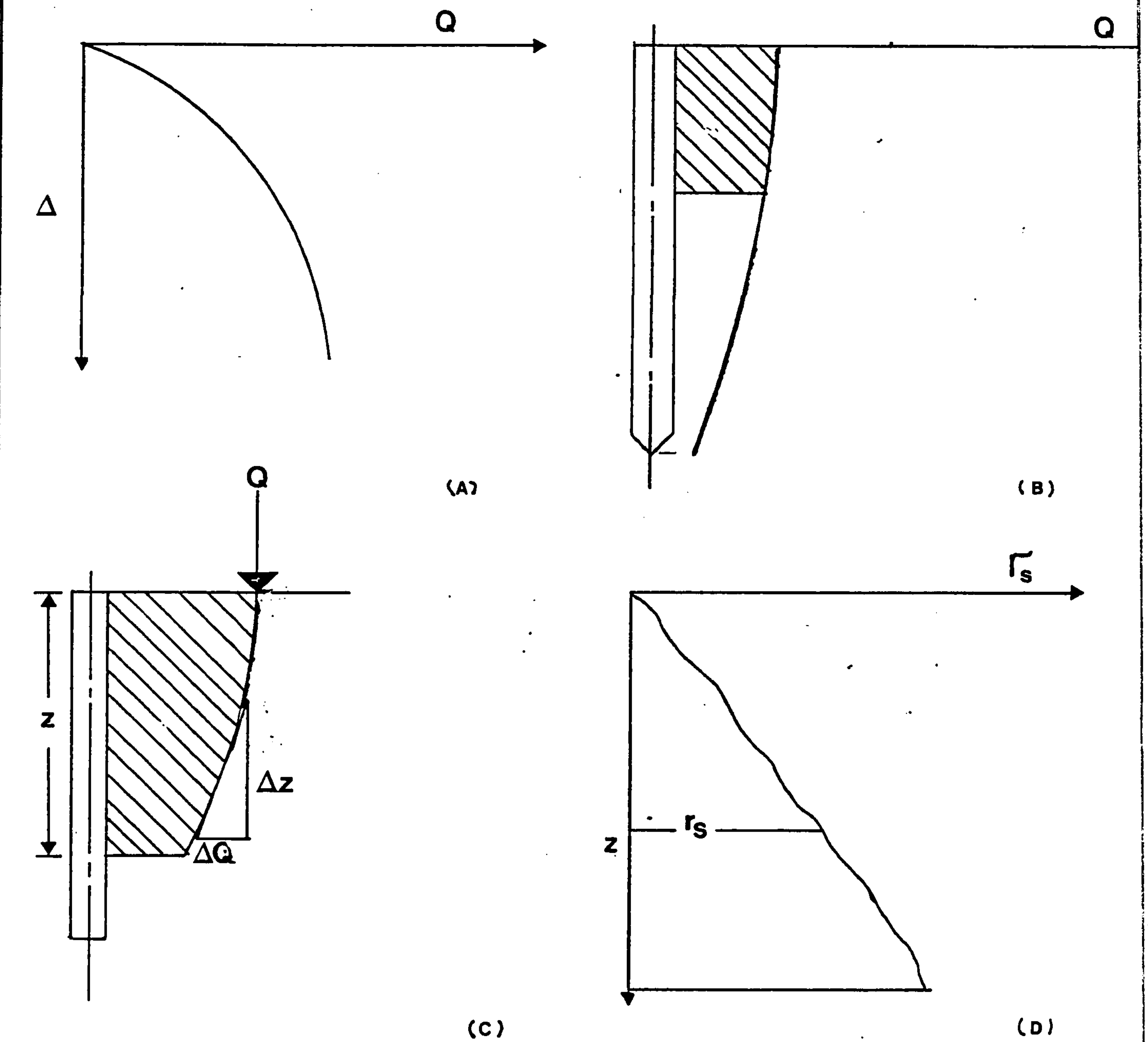


Fig. 2.3: Determination of load transfer versus depth for telltale instrumented piles (after Reese, 1979).

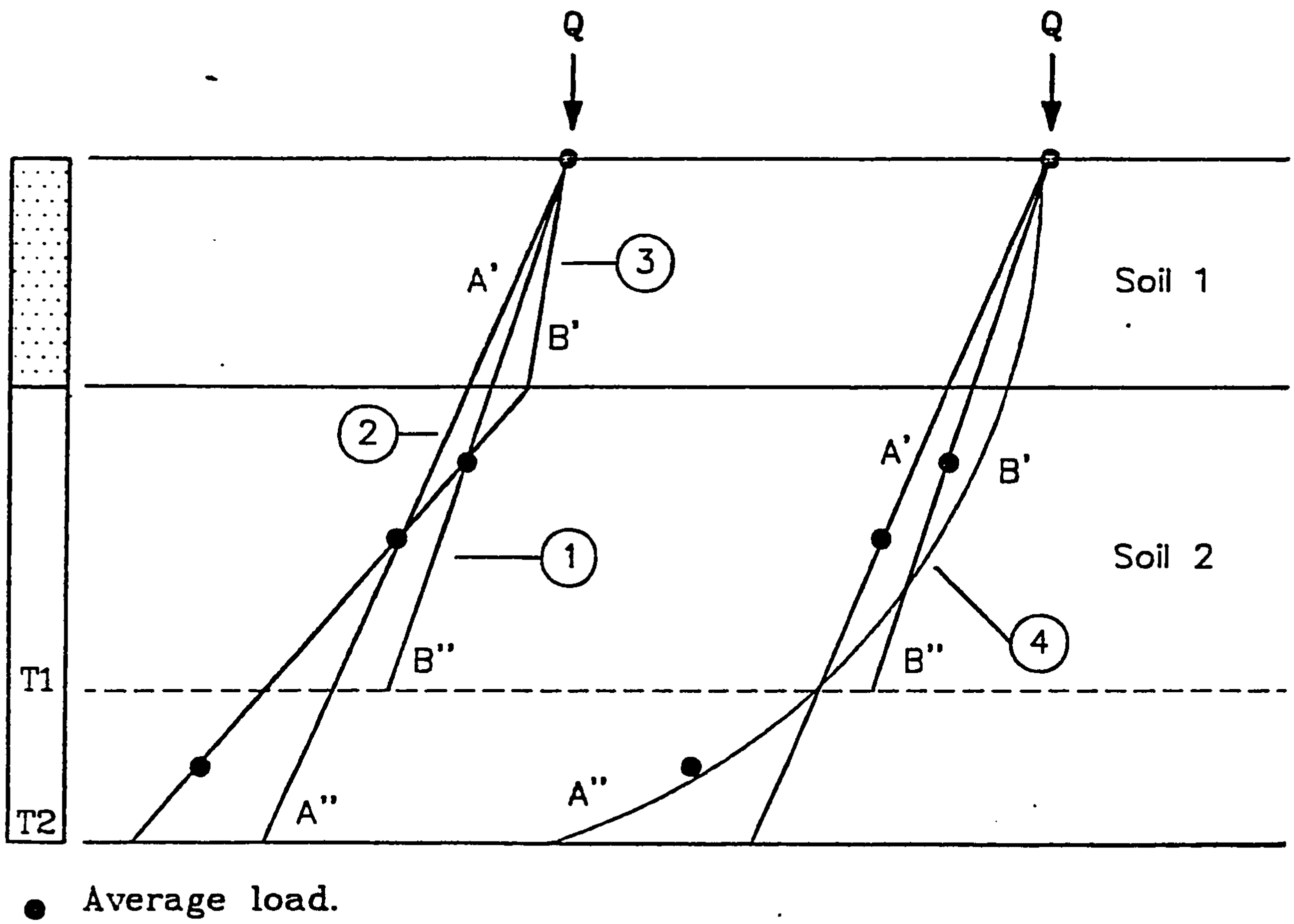


Fig. 2.4: Determination of load distribution from telltale instrumented piles (after Fellenius, 1980).

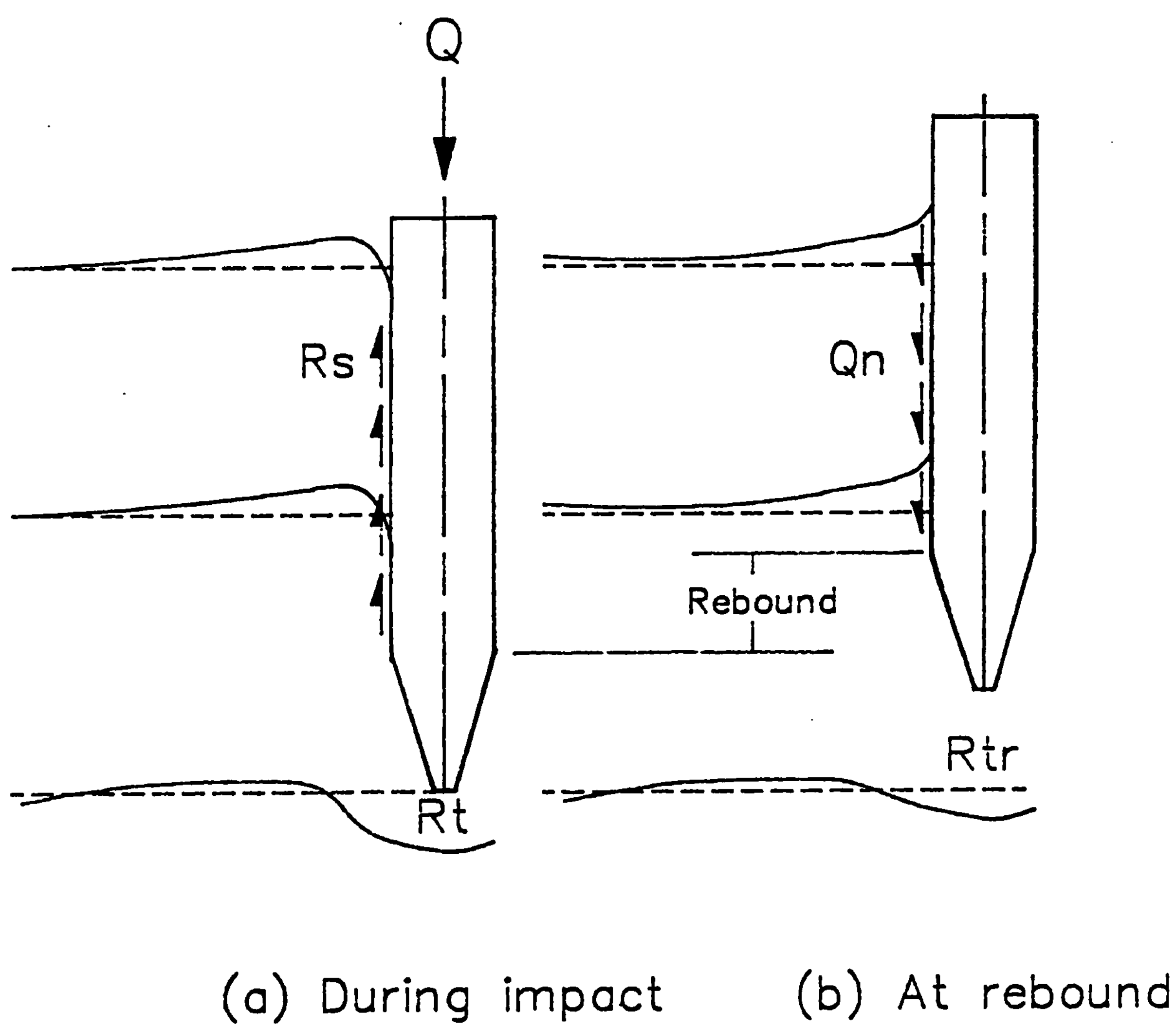


Fig. 2.5: Typical soil displacement near the toe of a jacked or bored pile (after Cooke, 1979).

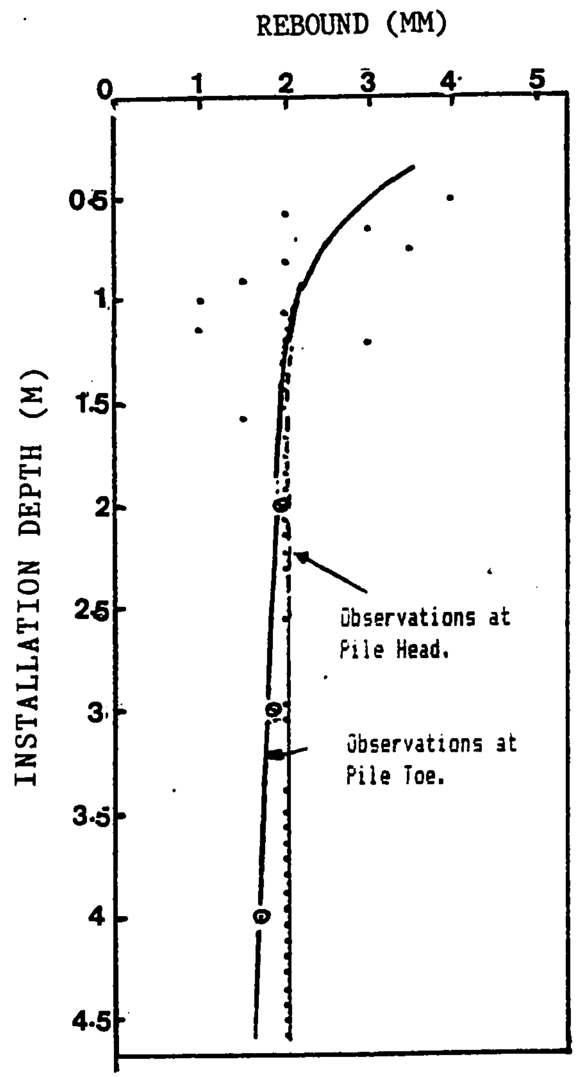


Fig. 2.6: Reported measurement of pile rebound (after Cooke, 1979).

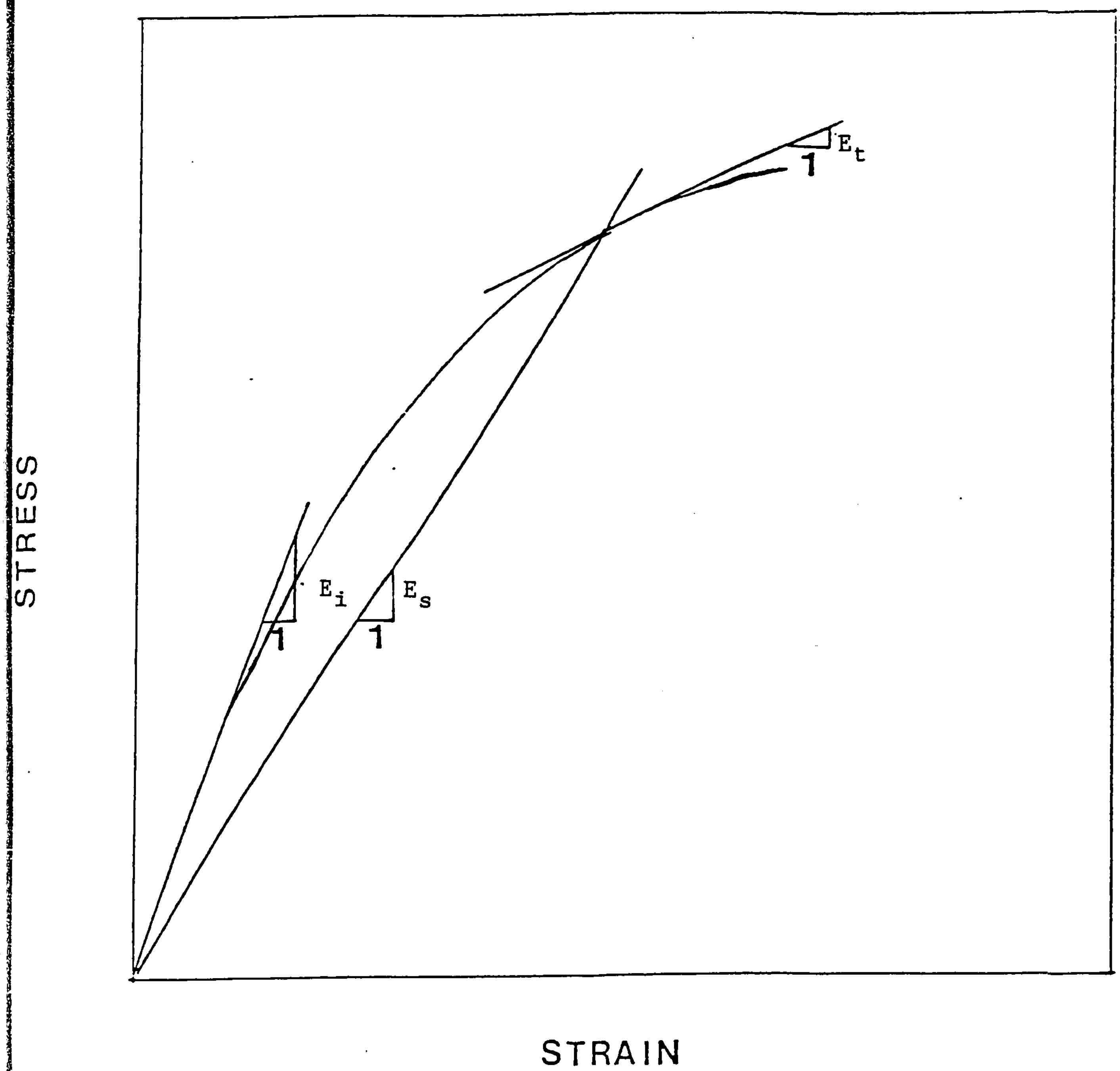


Fig. 2.7: Idealized stress-strain curve of concrete piles in axial compression.

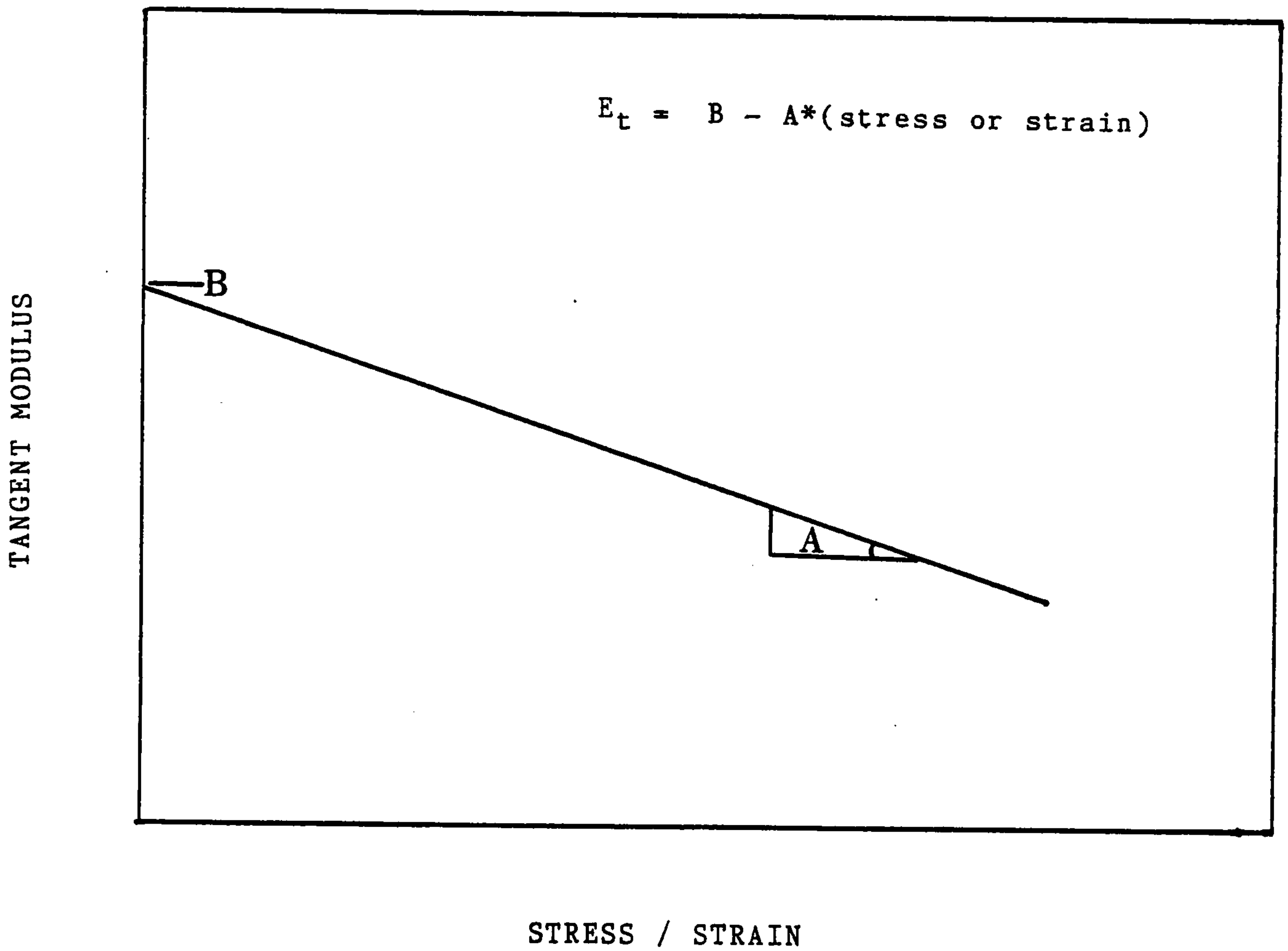


Fig. 2.8: Modulus variation with stress/ strain linear approach.

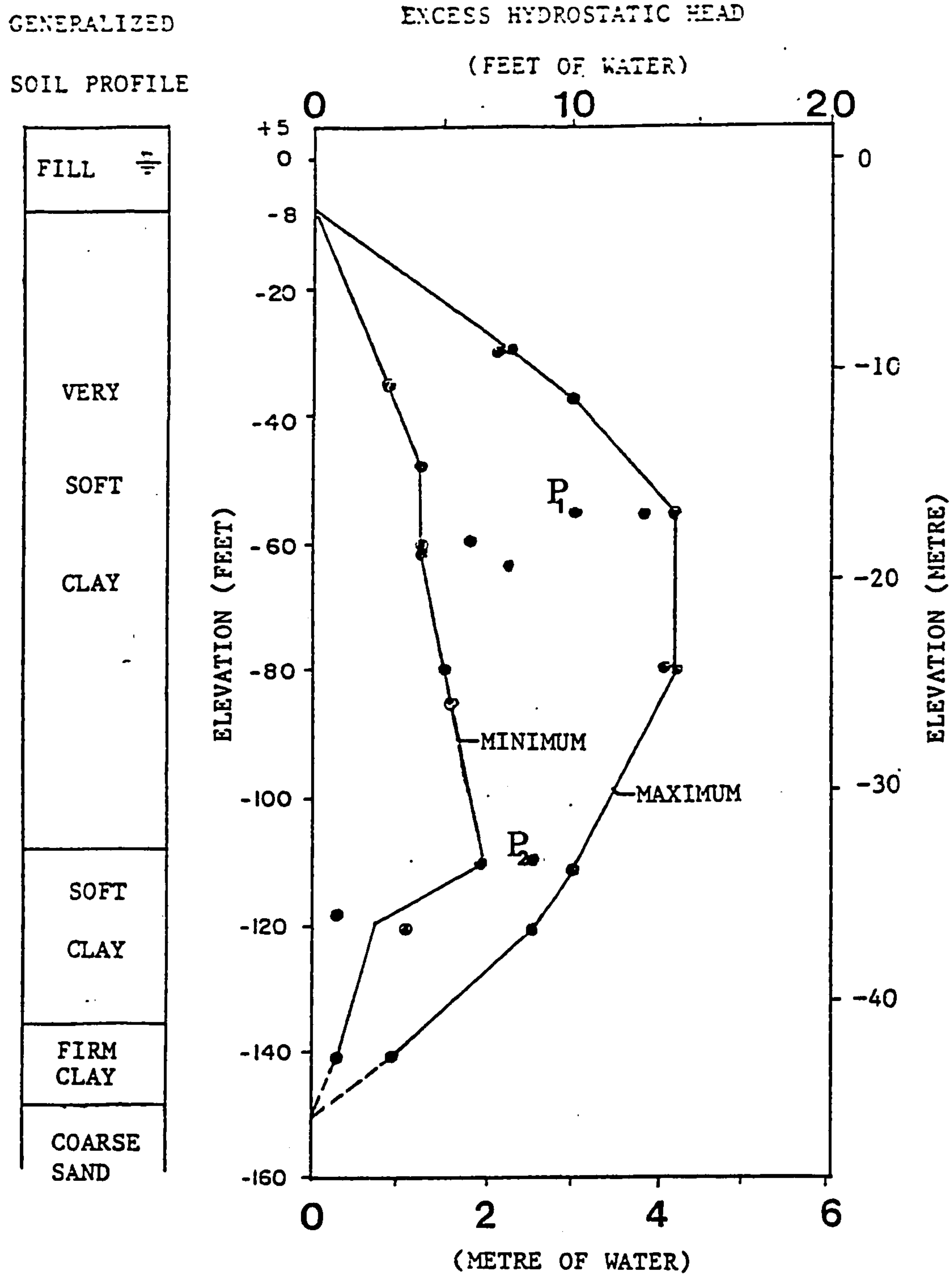


Fig. 3.1: Generalized excess hydrostatic head profile.
(after Parsons, 1979).
P_1 and P_2 were installed near Piles 1, 2, and 4.

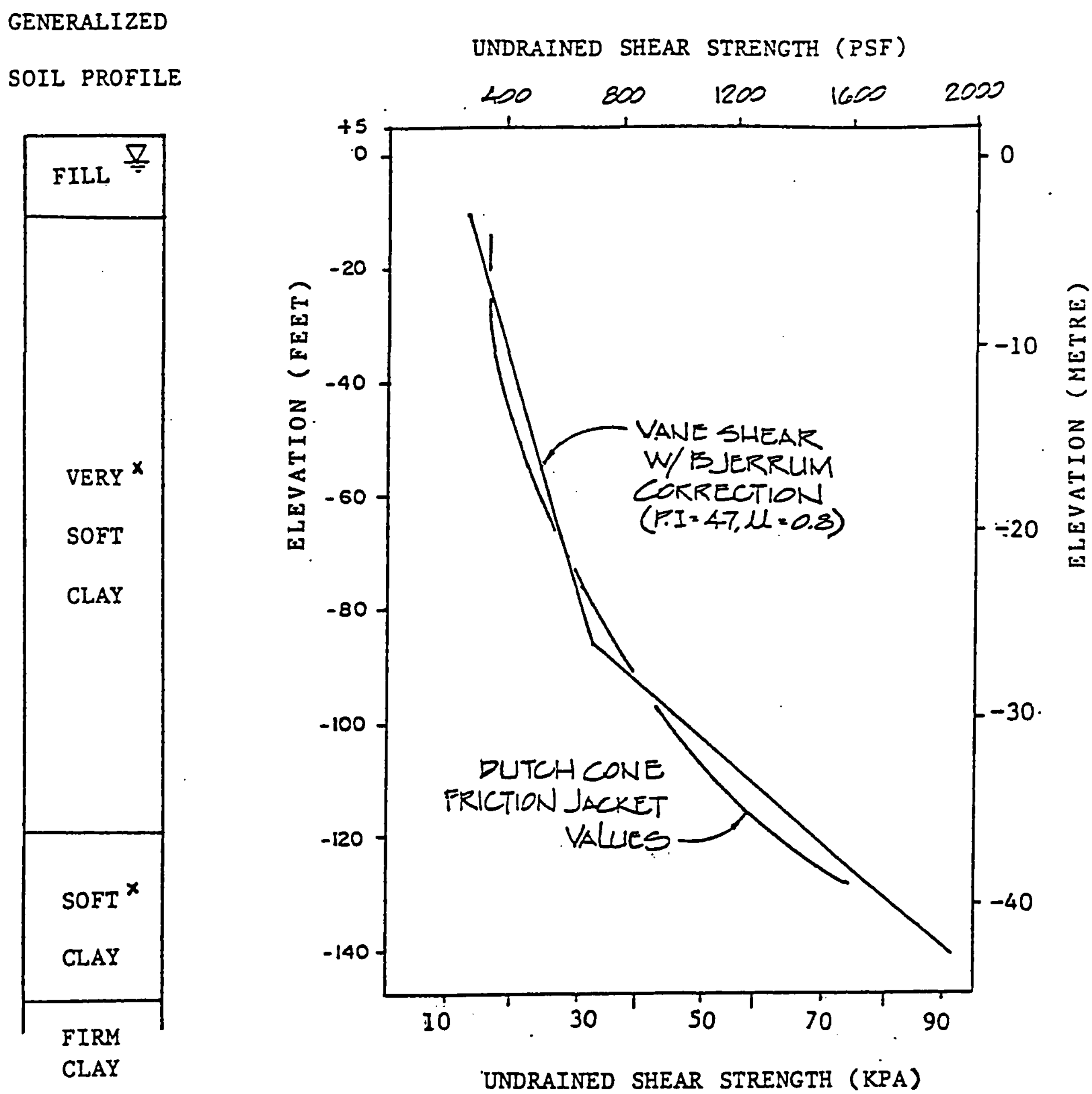


Fig. 3.2: Undrained shear strength envelope (after Parsons et al., 1979).

x) Consistency classification by Parsons, Brinckerhoff, Hirota Associates, 1979.

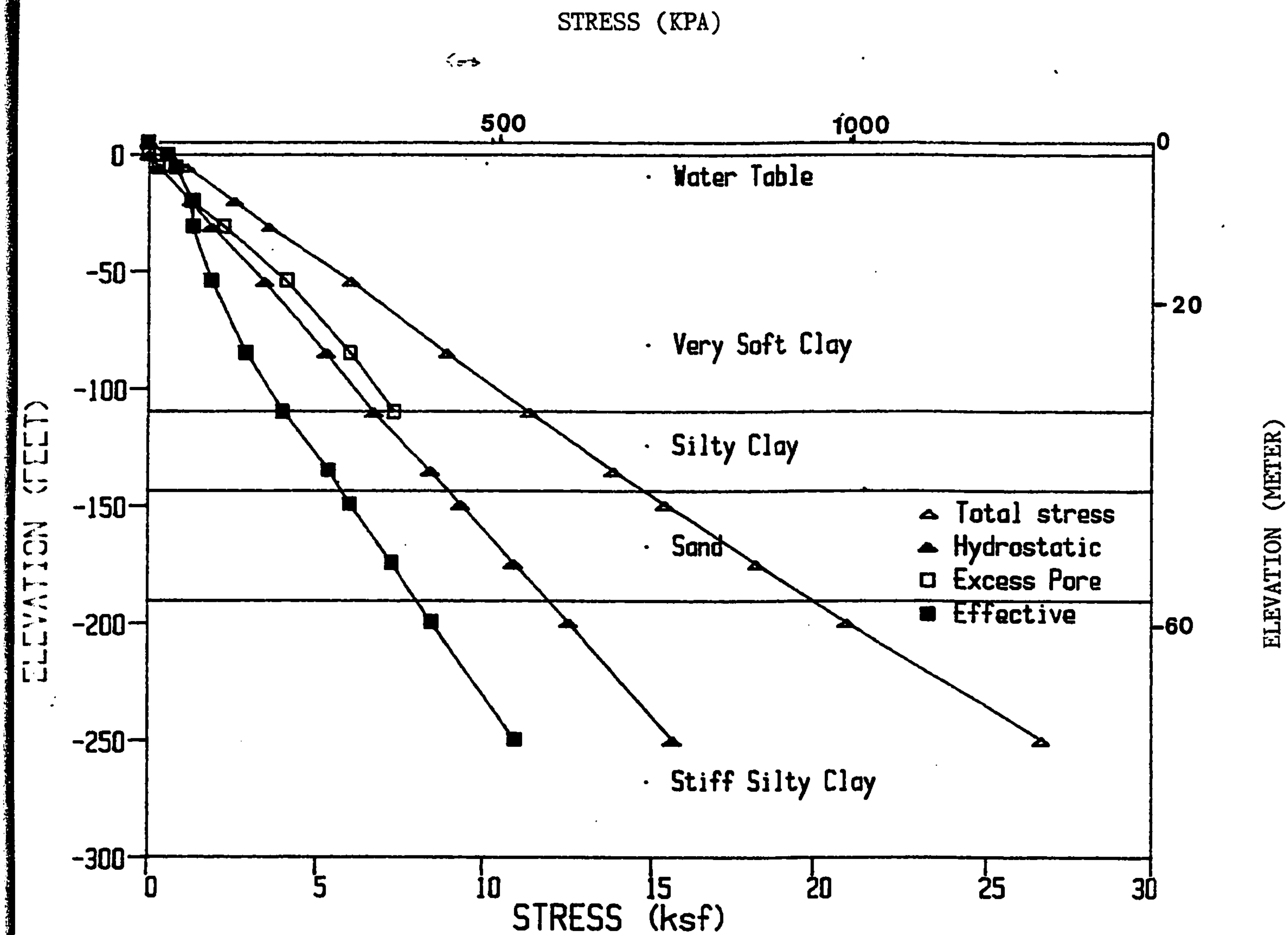


Fig. 3.3: Pressure versus depth around Piles 1, 2, and 4
after Parsons et al., 1979).

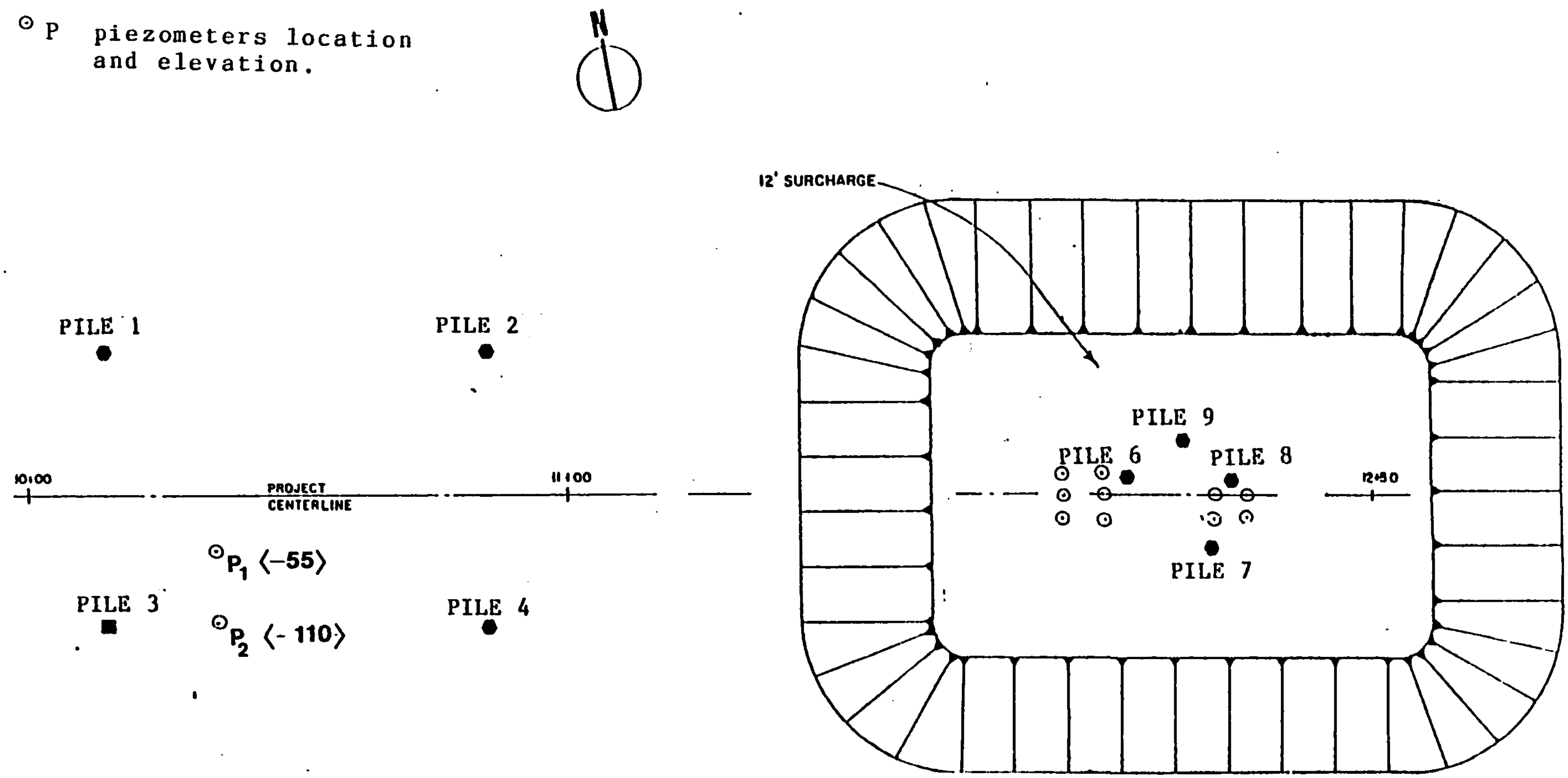


Fig. 3.4: Loaction of the test Piles (after Parsons et al., 1979).

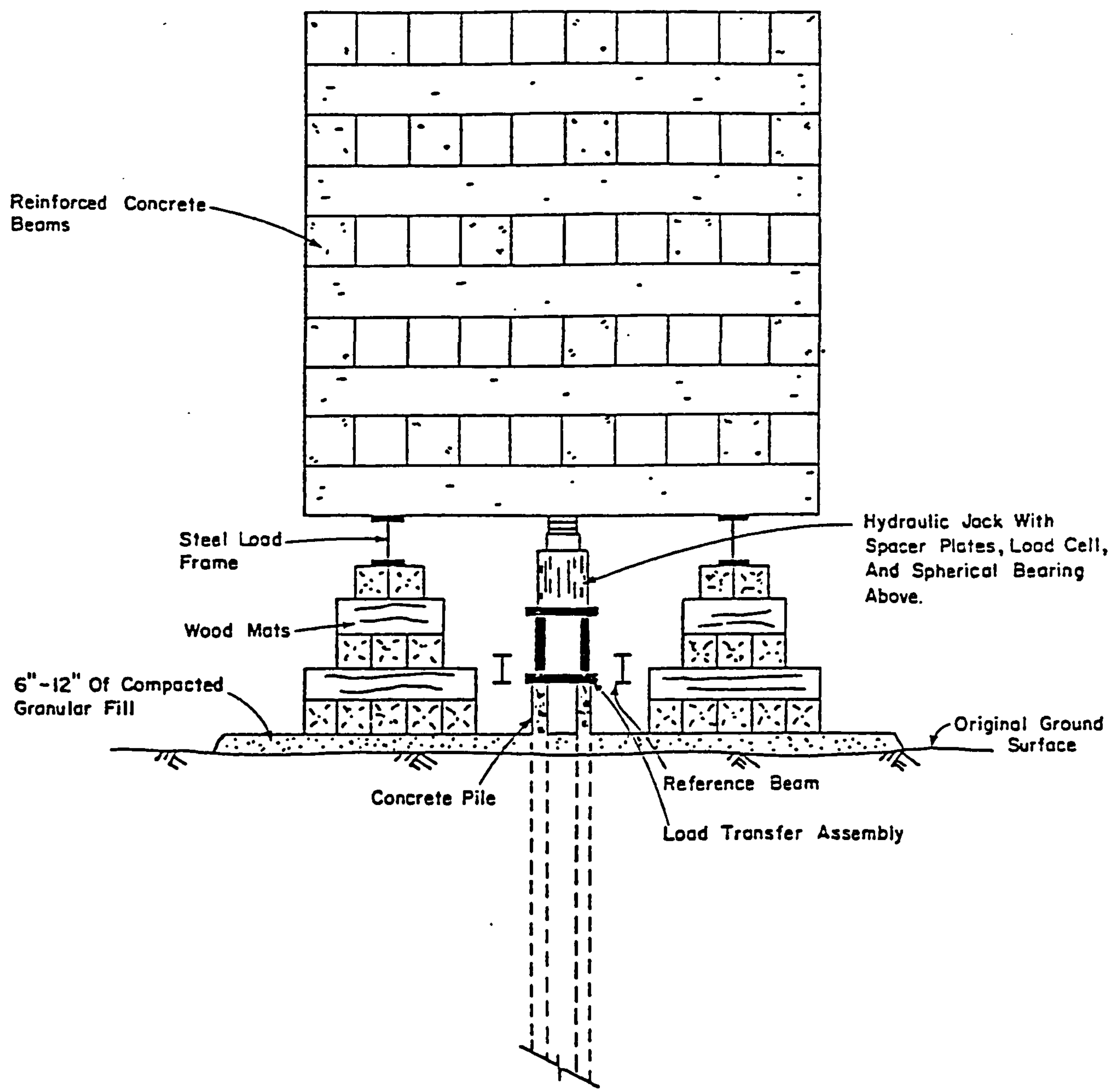


Fig. 3.5: Shematic view of test loading arrangement (after Soil and Rock Instrumentation, 1978).

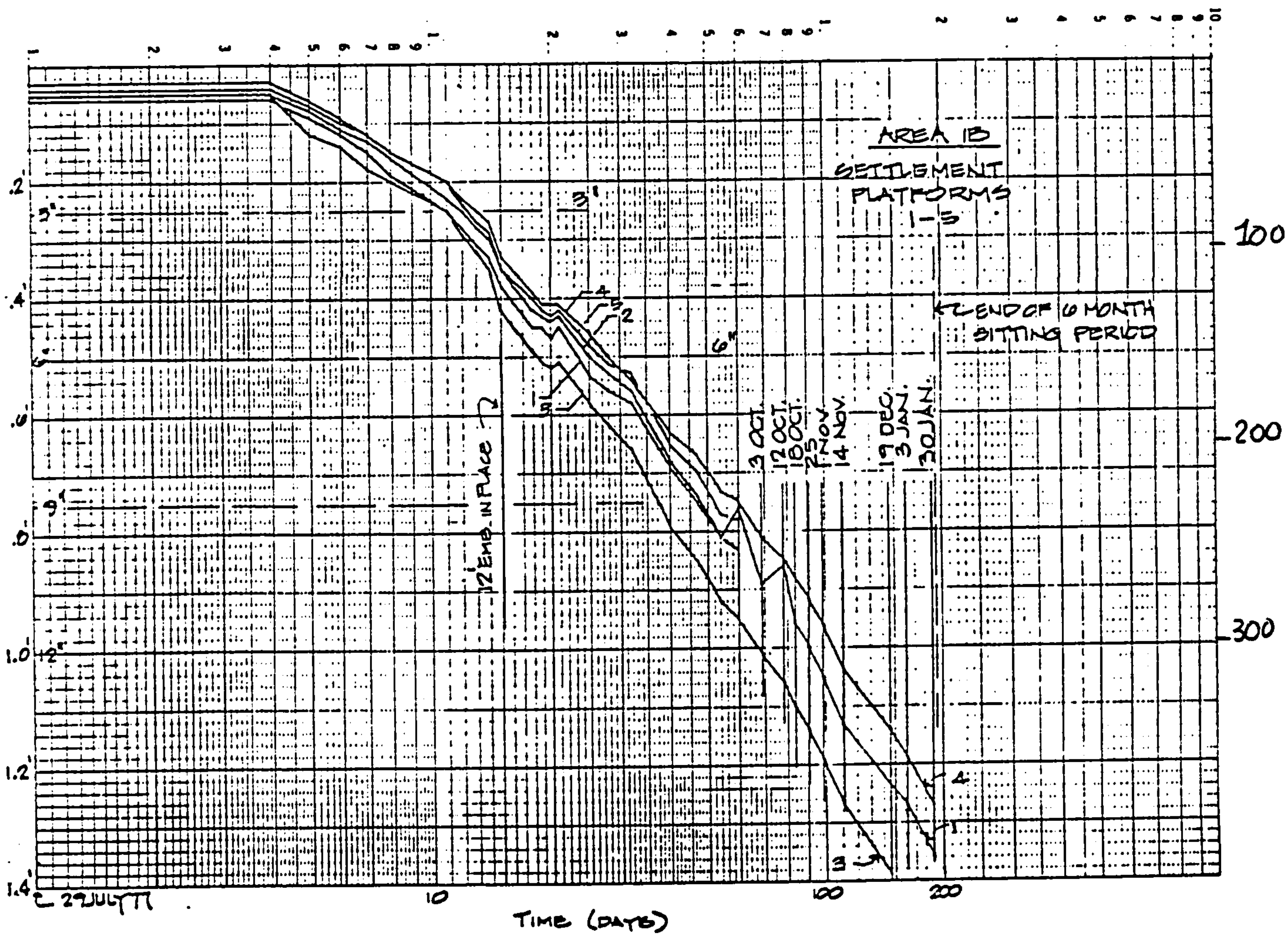


Fig. 3.6: Settlement platforms versus time around Piles 6 to 9
(after Parsons et al., 1979).

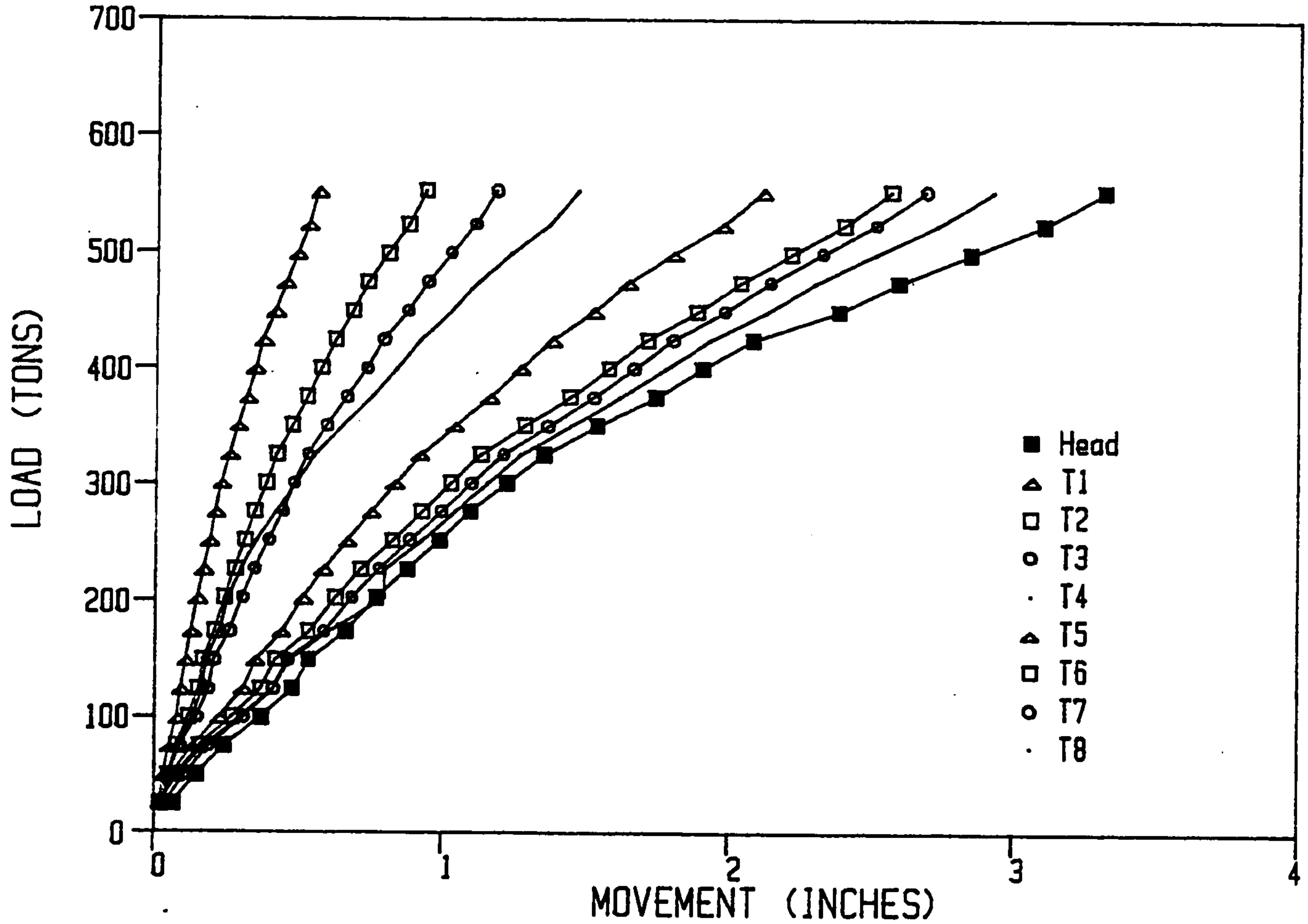


Fig. 4.1: Load movement diagram for Pile 1.

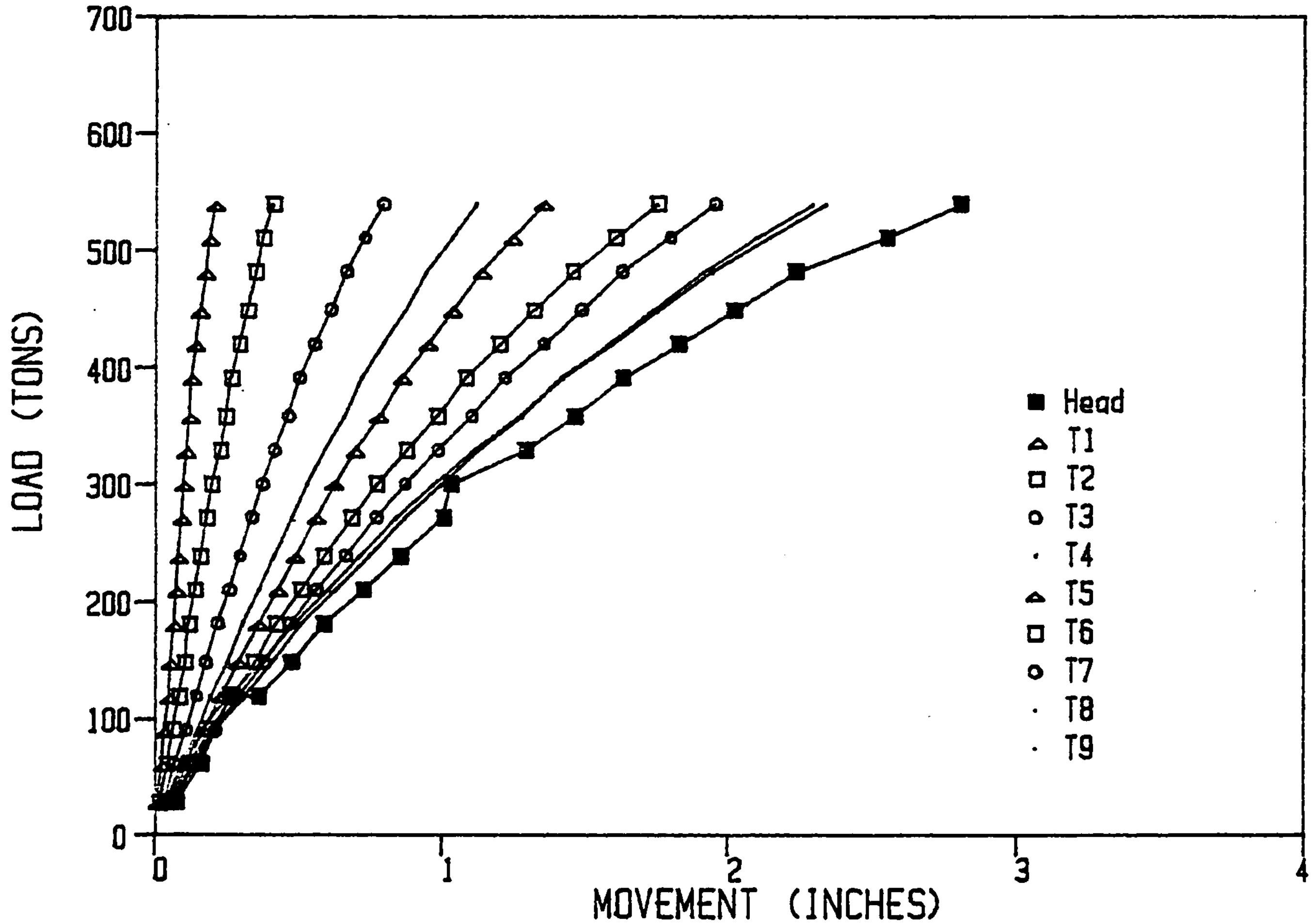


Fig. 4.2: Load movement diagram for Pile 2.

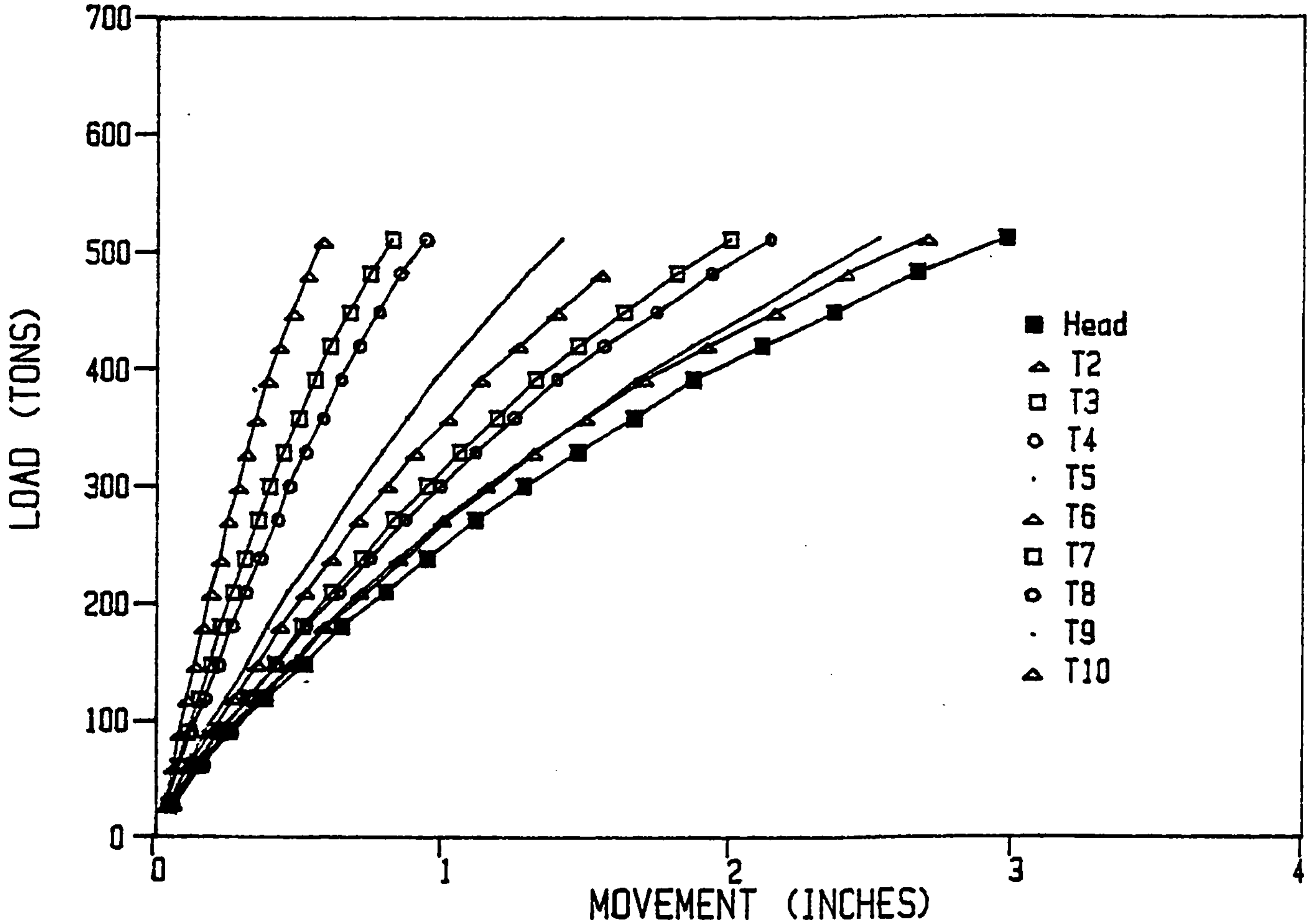
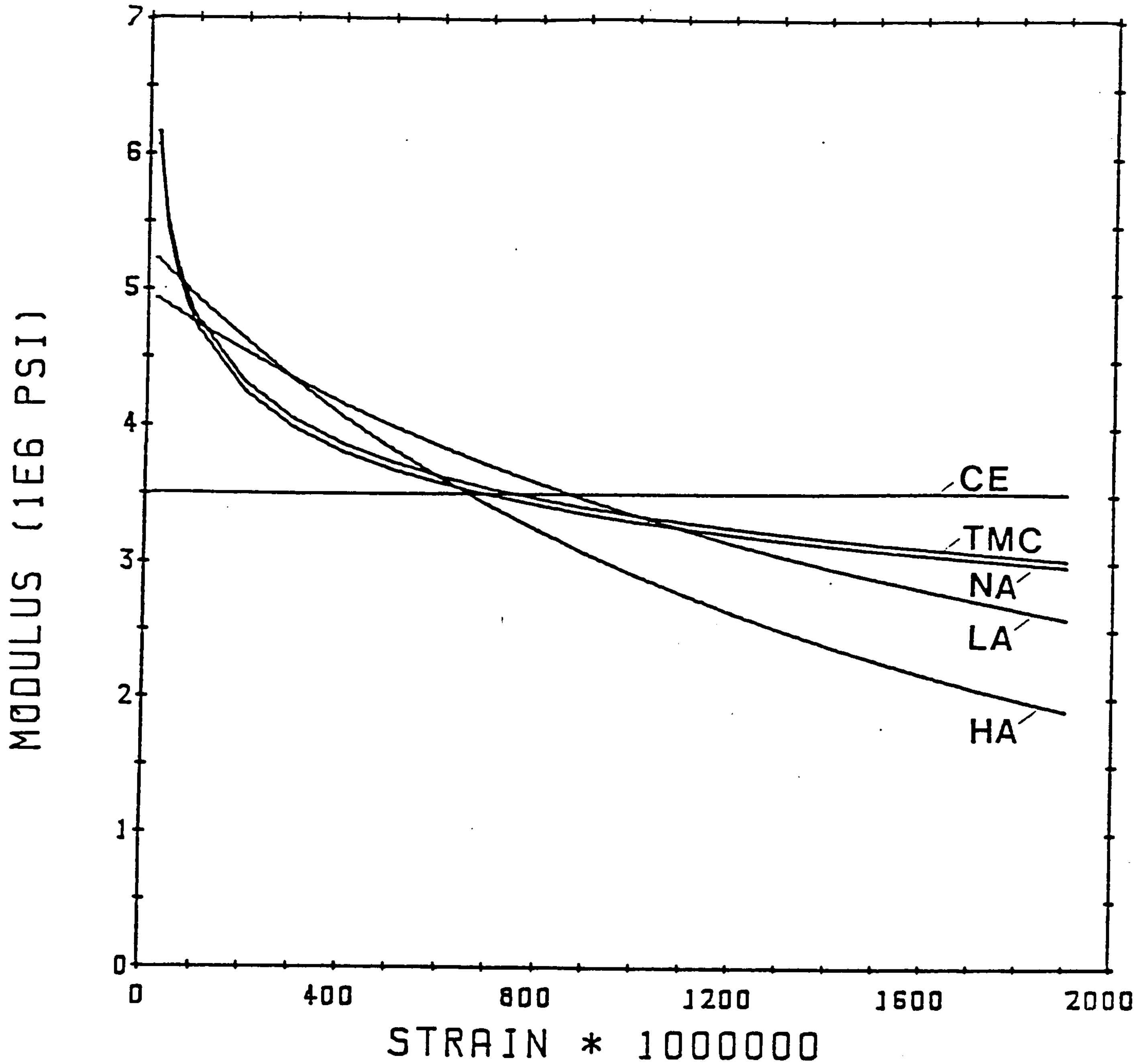


Fig. 4.3: Load movement diagram for Pile 4.



NOTE: CE = CONSTANT MODULUS
LA = LINEAR APPROACH
NA = NON-LINEAR APPROACH
TMC = TANGENT MODULUS CONCEPT
HA = HYPERBOLIC APPROACH

Fig. 4.4: Comparison of modulus versus strain for different approaches (after Yu, 1984)

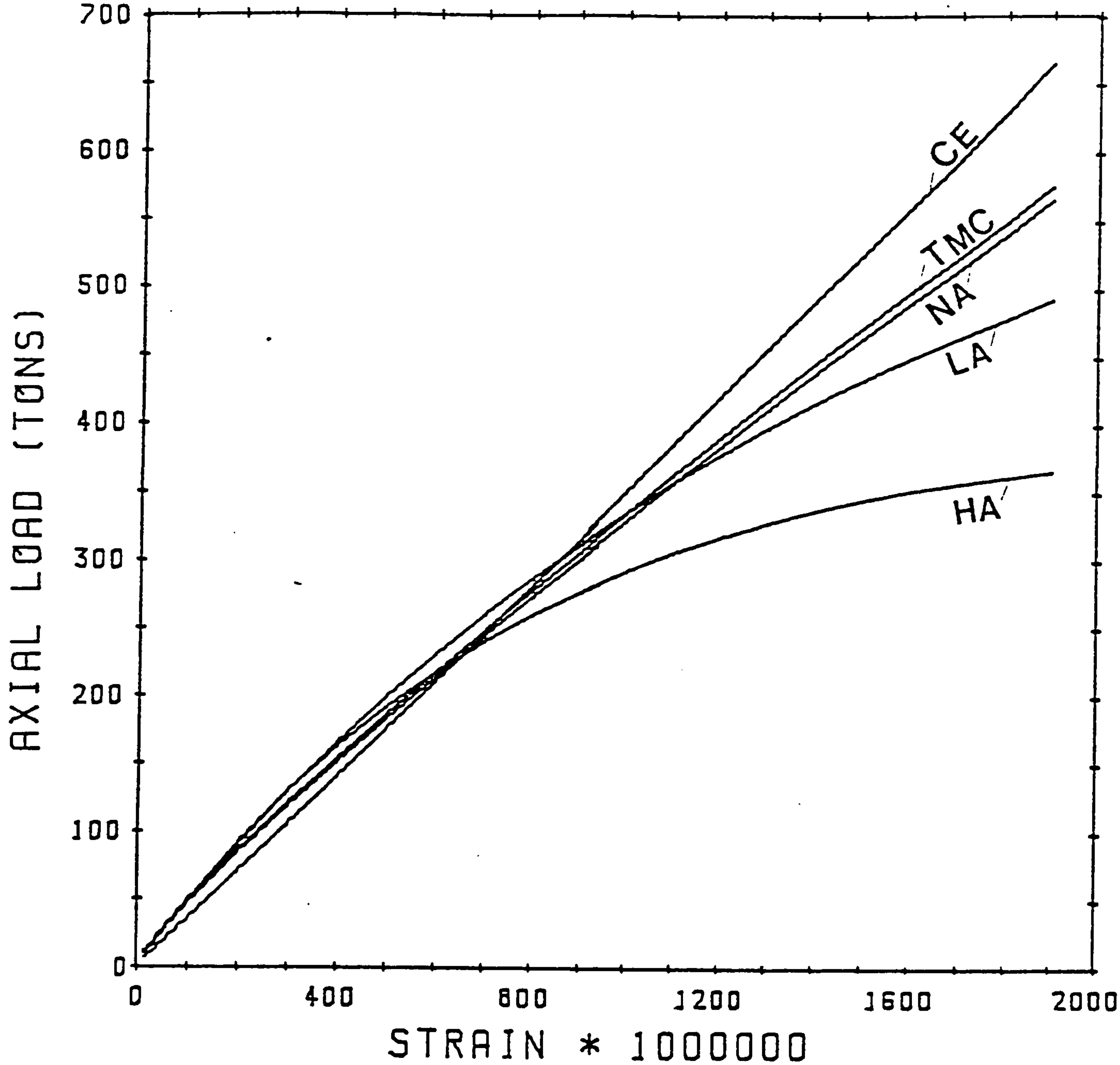


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Fig. 4.5: Comparison of axial load versus strain for different approaches (after Yu, 1984).

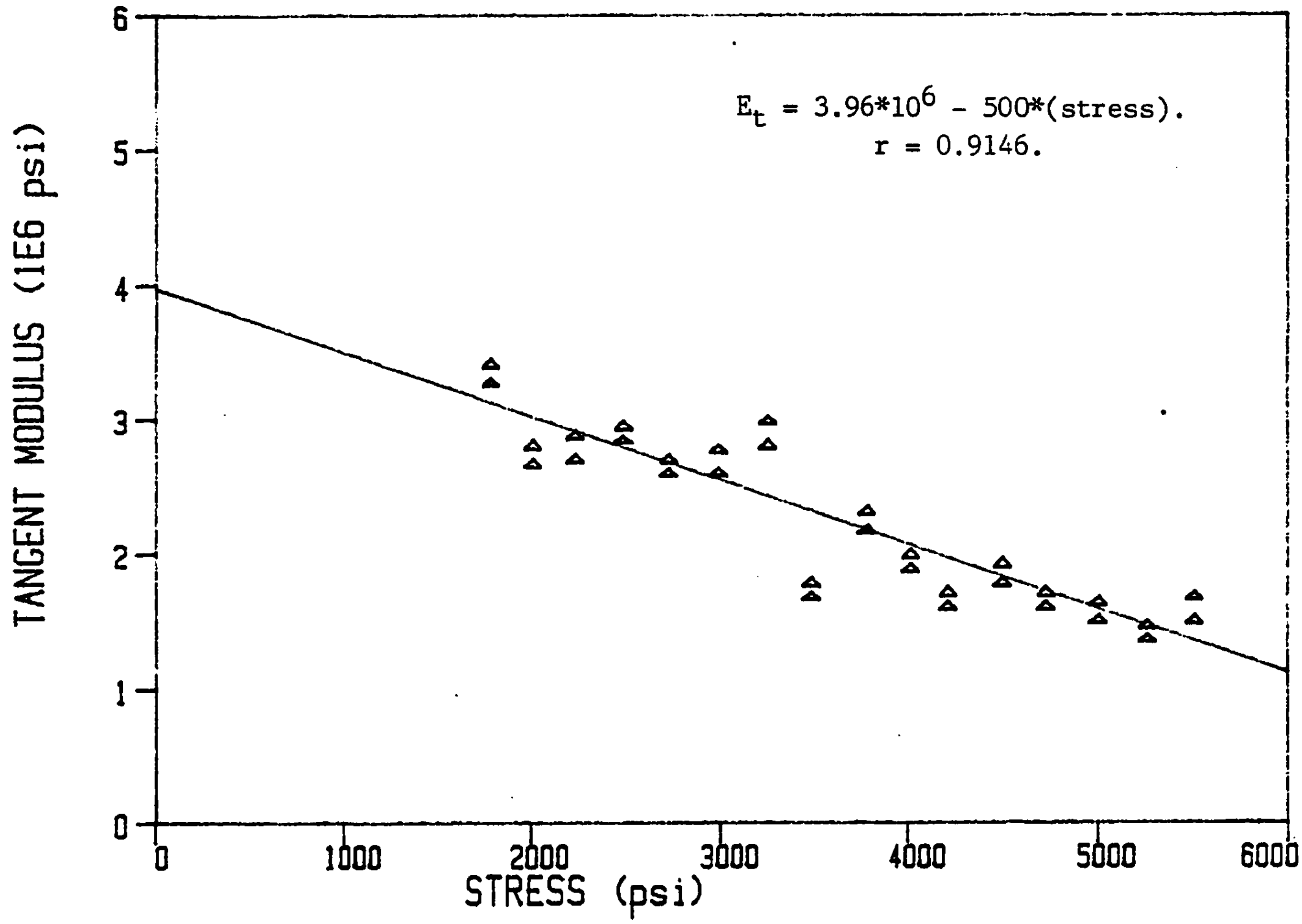

Fig. 5.1: Tangent modulus variation with stress, Pile 1.

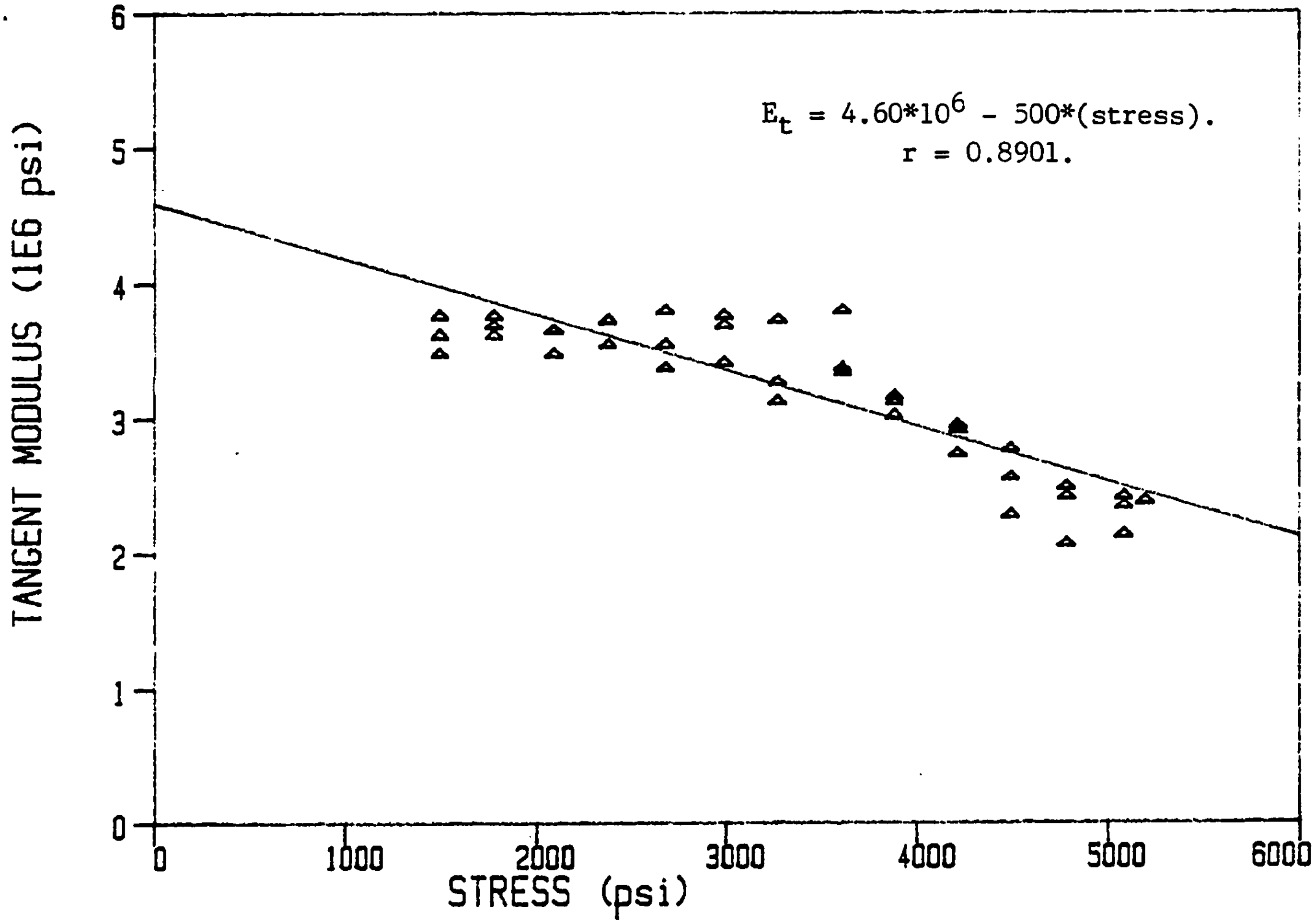

Fig. 5.2: Tangent modulus variation with stress, Pile 2.

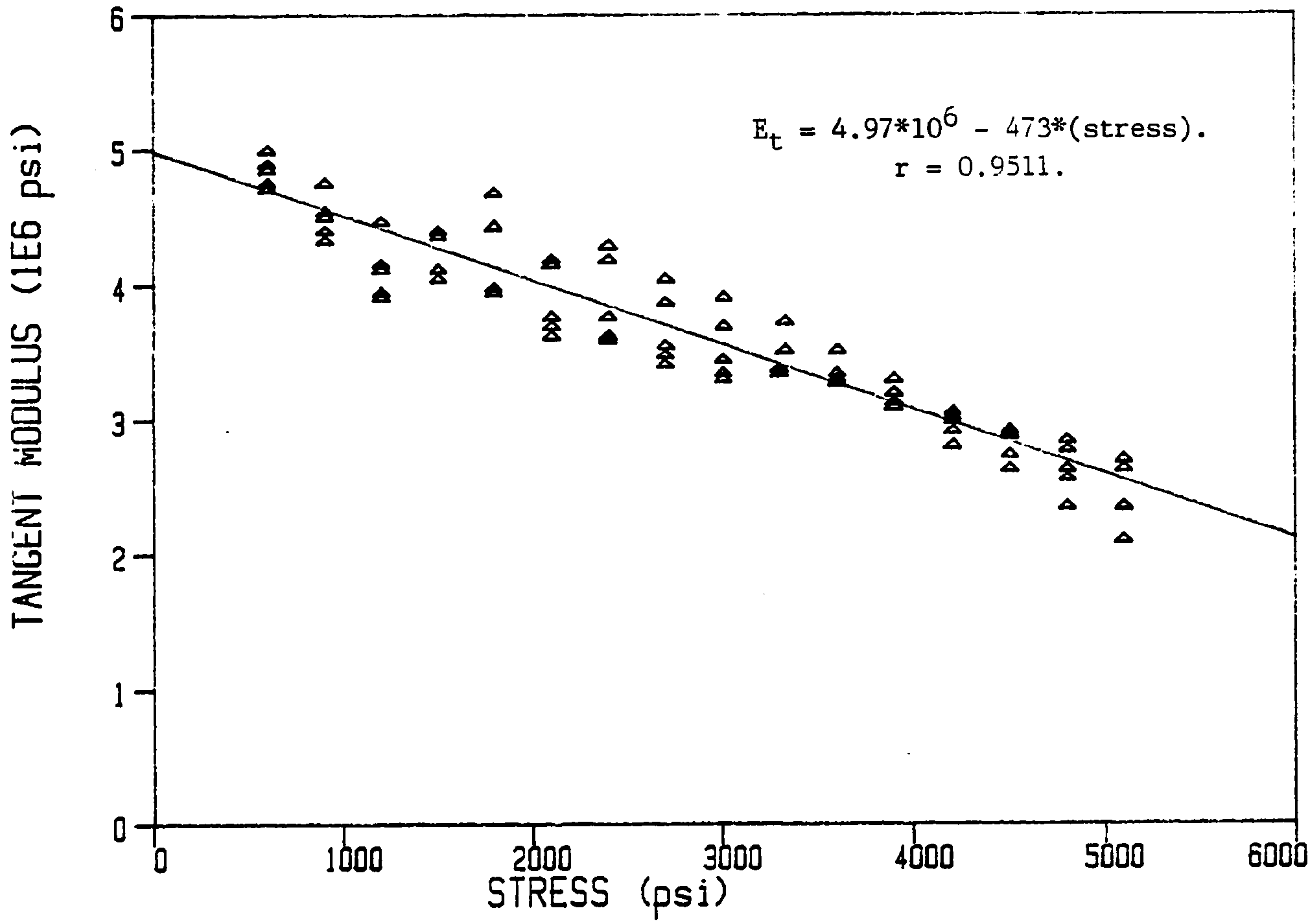

Fig. 5.3: Tangent modulus variation with stress, Pile 4.

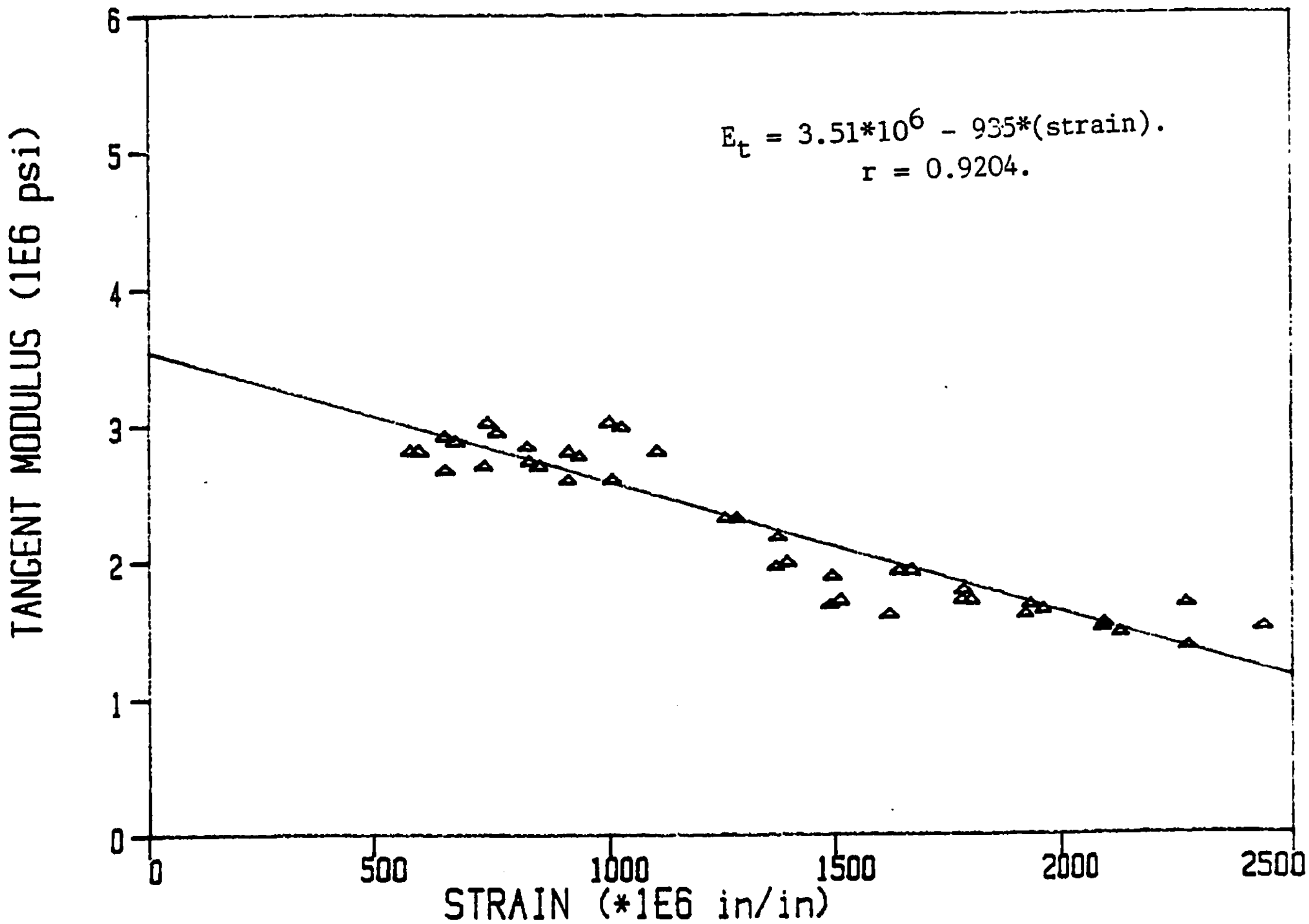

Fig. 5.4: Tangent modulus variation with strain, Pile 1.

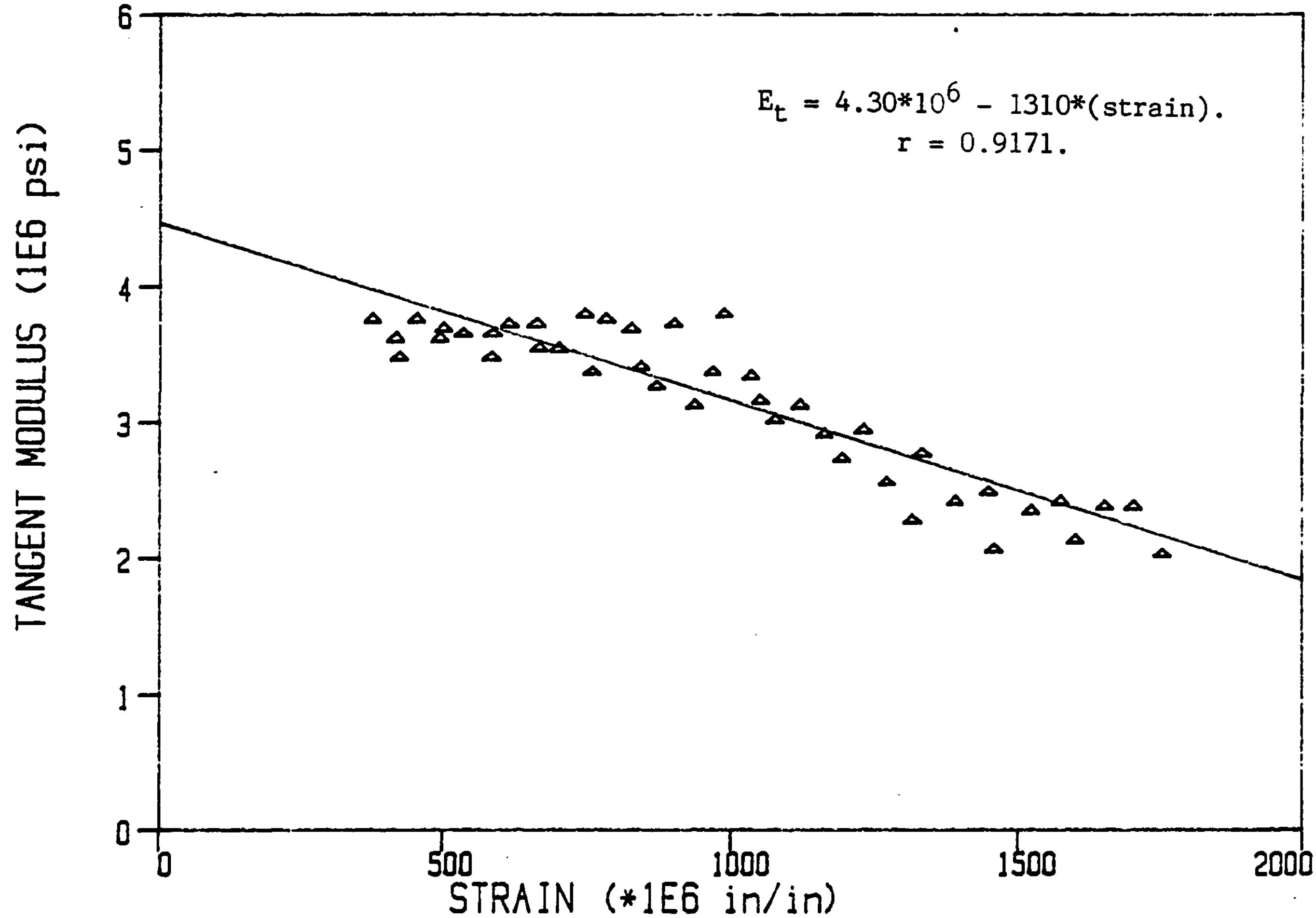

Fig. 5.5: Tangent modulus variation with strain, Pile 2.

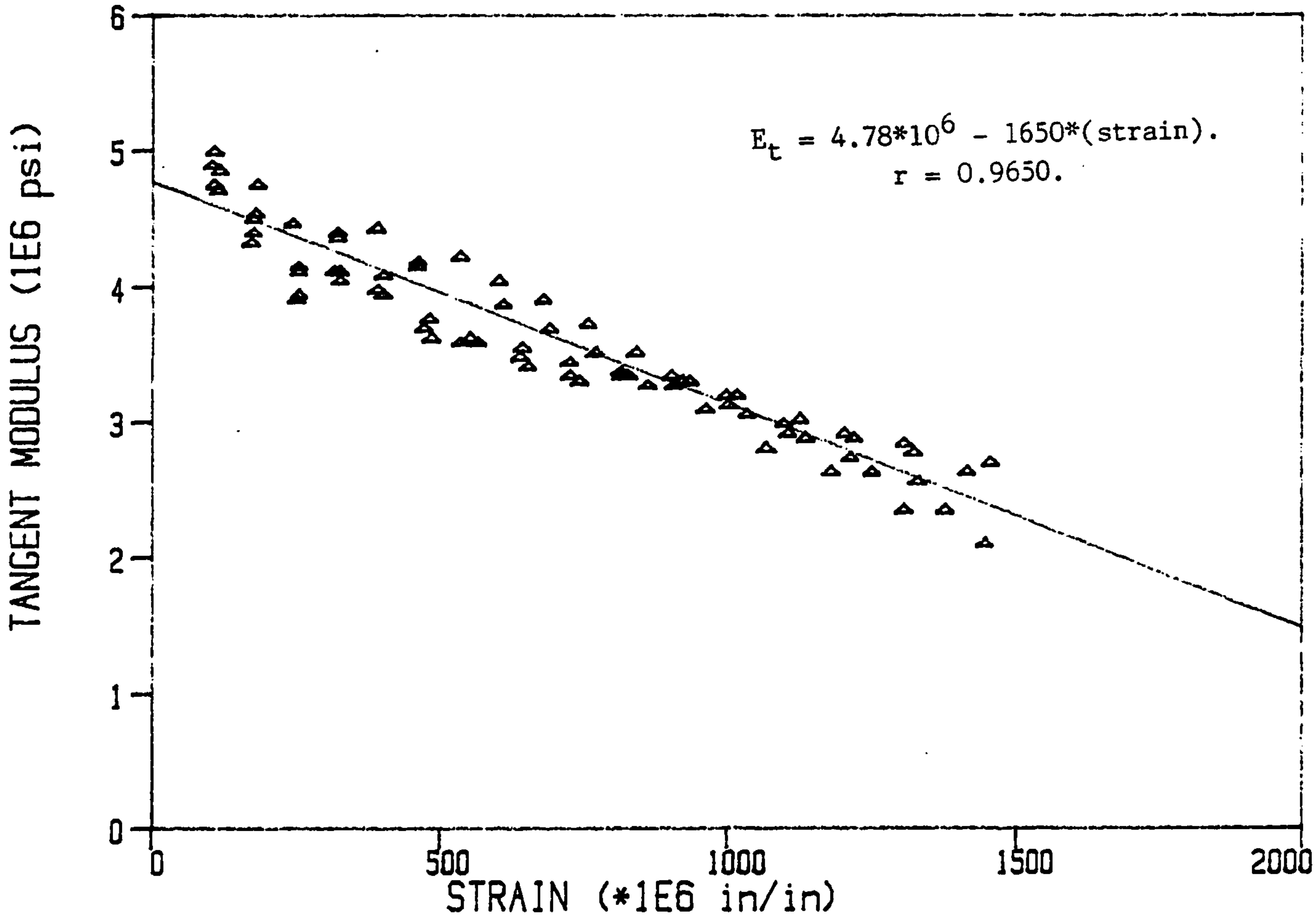

Fig. 5.6: Tangent modulus variation with strain, Pile 4.

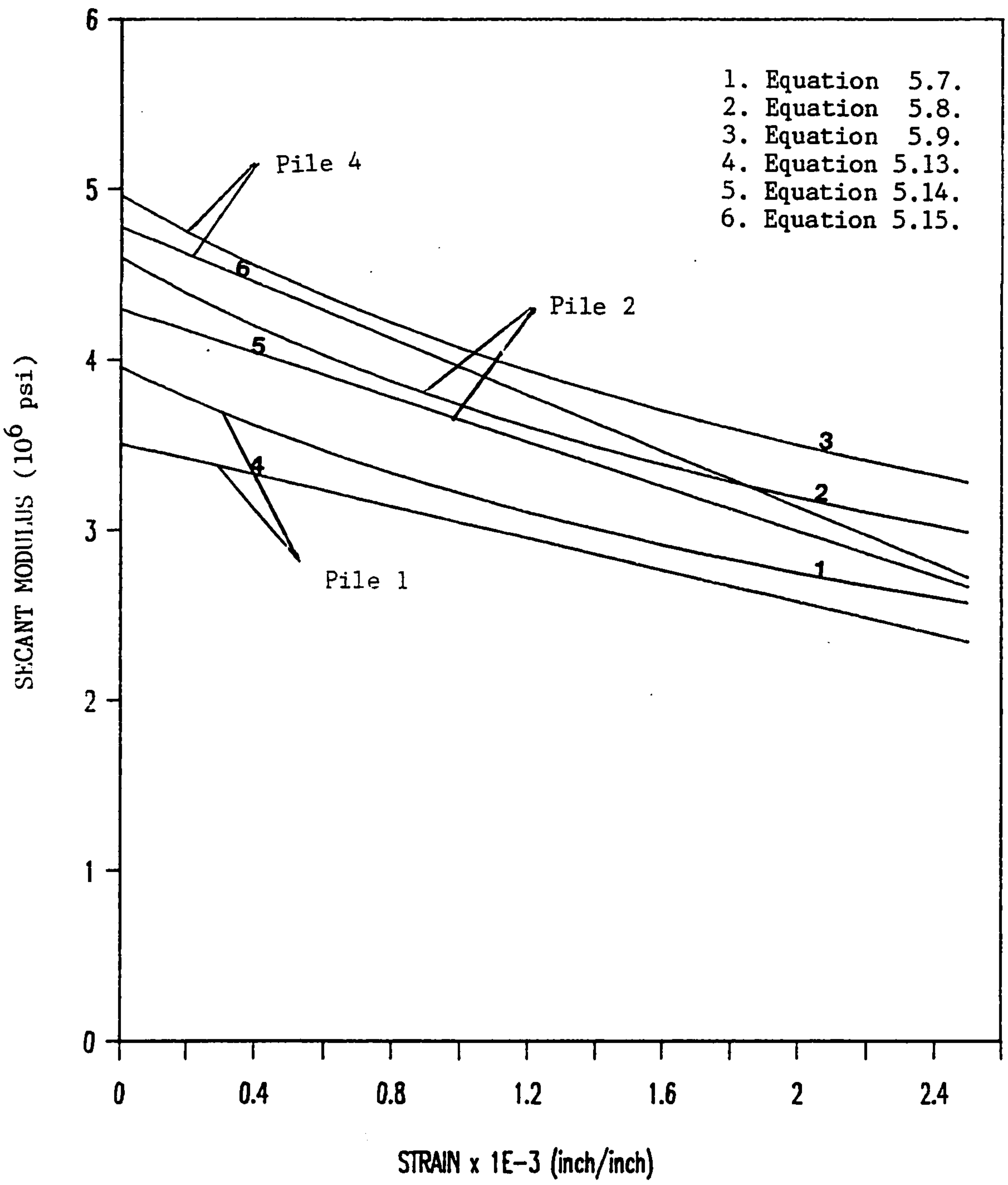

Fig. 5.7: Comparison between the derived secant modulus from stress and the derived secant modulus from strain, for Piles 1, 2, and 4.

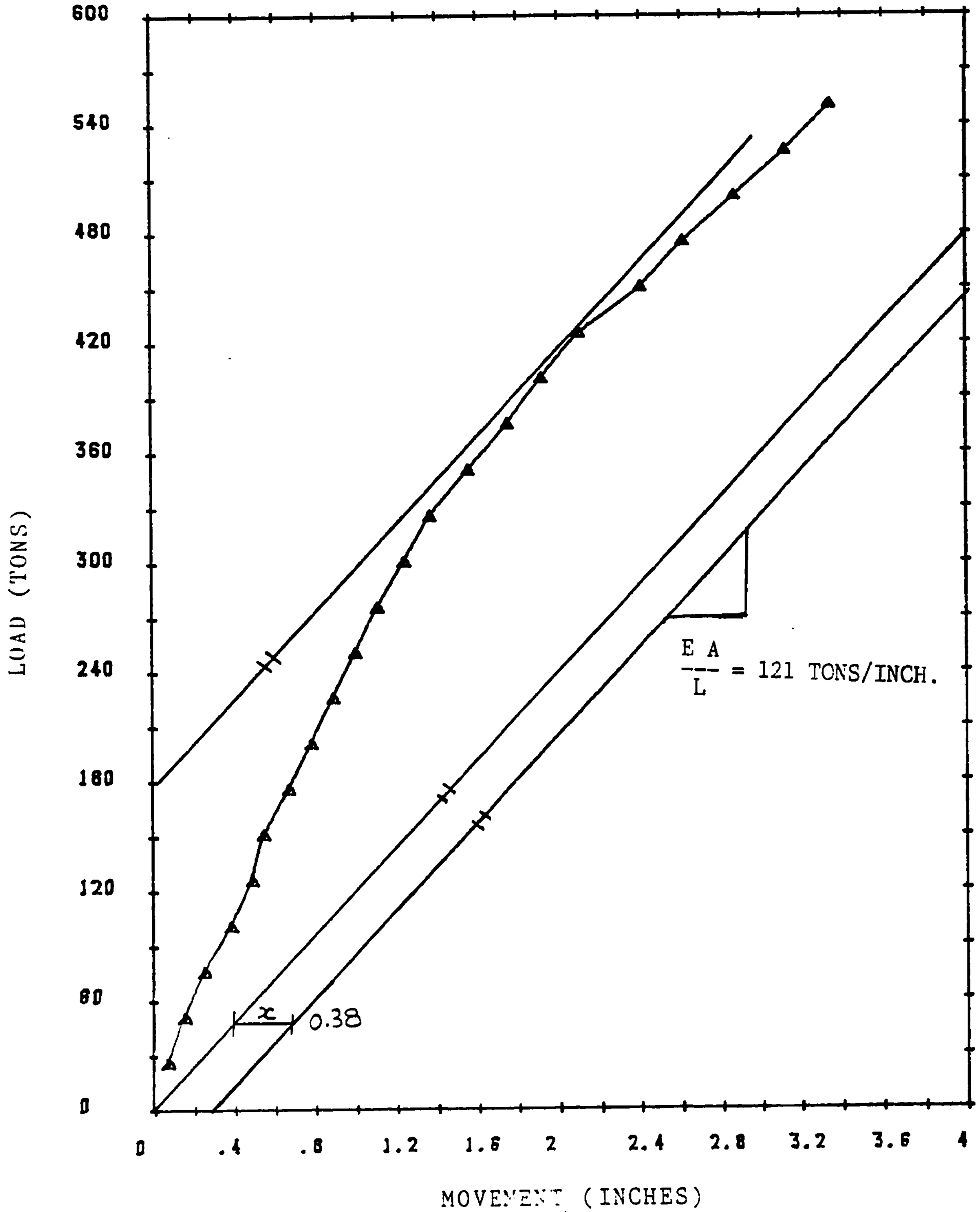

Fig. 5.8: Construction of Davisson limit load, Pile 1.

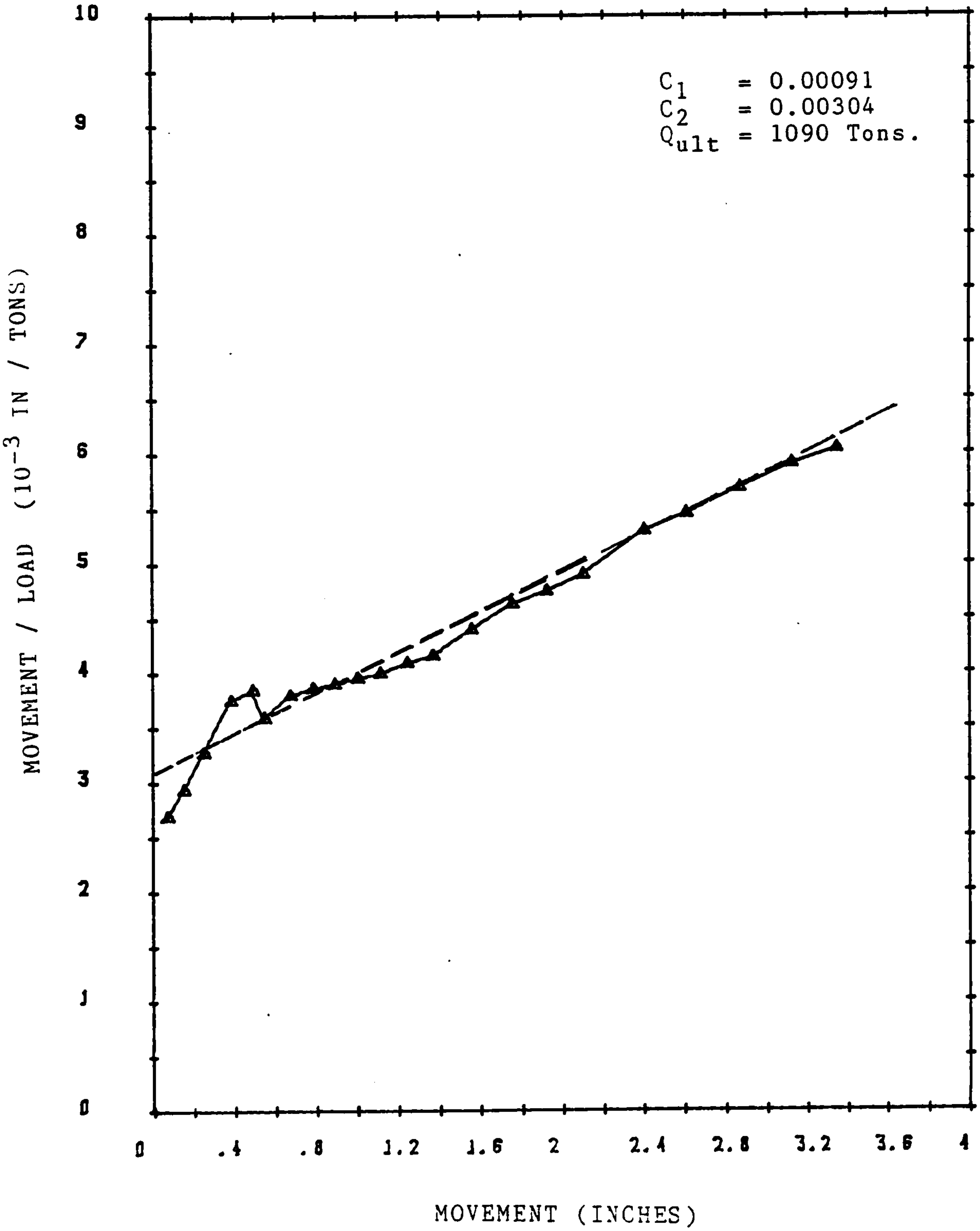

Fig. 5.9: Construction of Chin limit load, Pile 1.

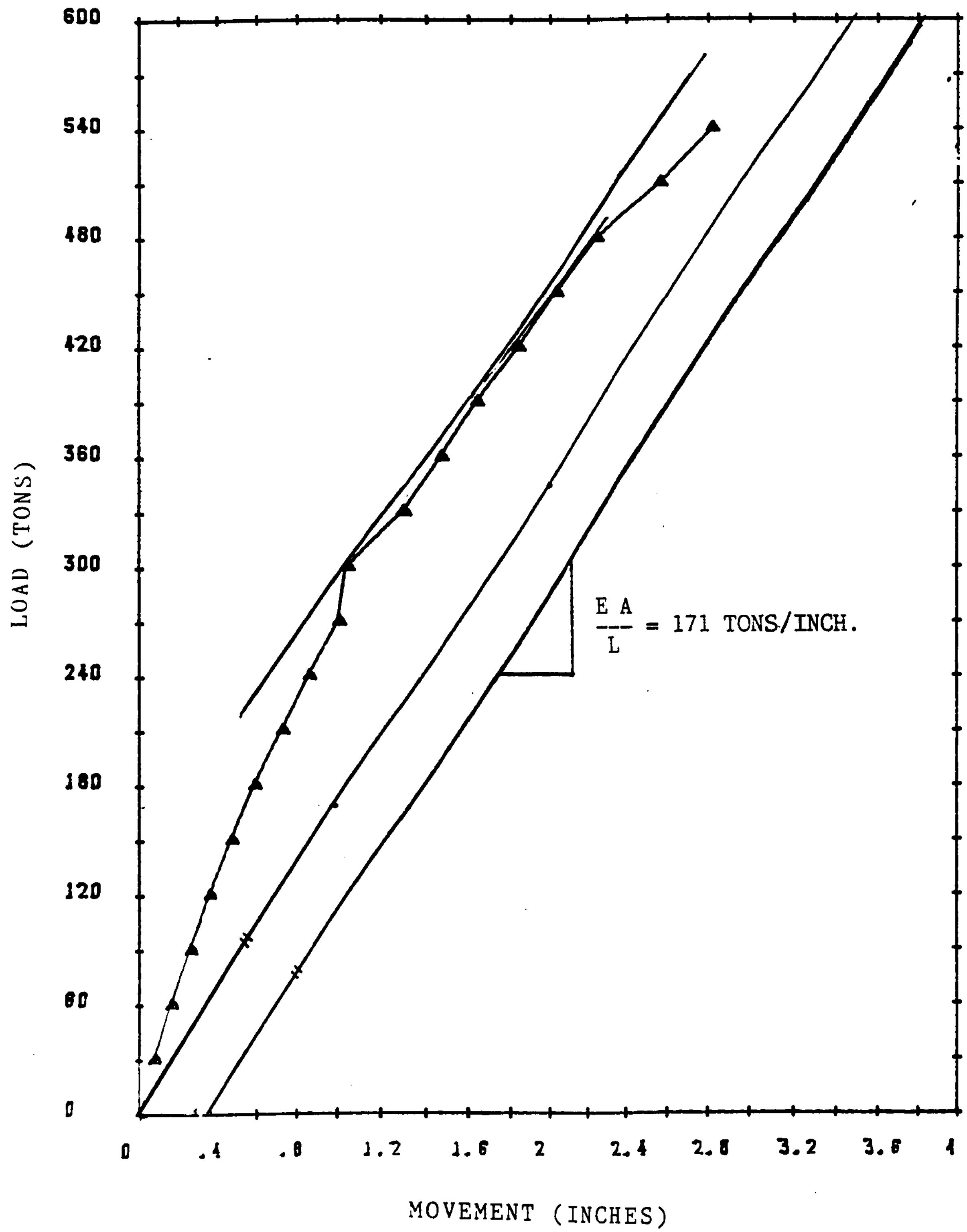

Fig. 5.10: Construction of Davisson limit load, Pile 2.

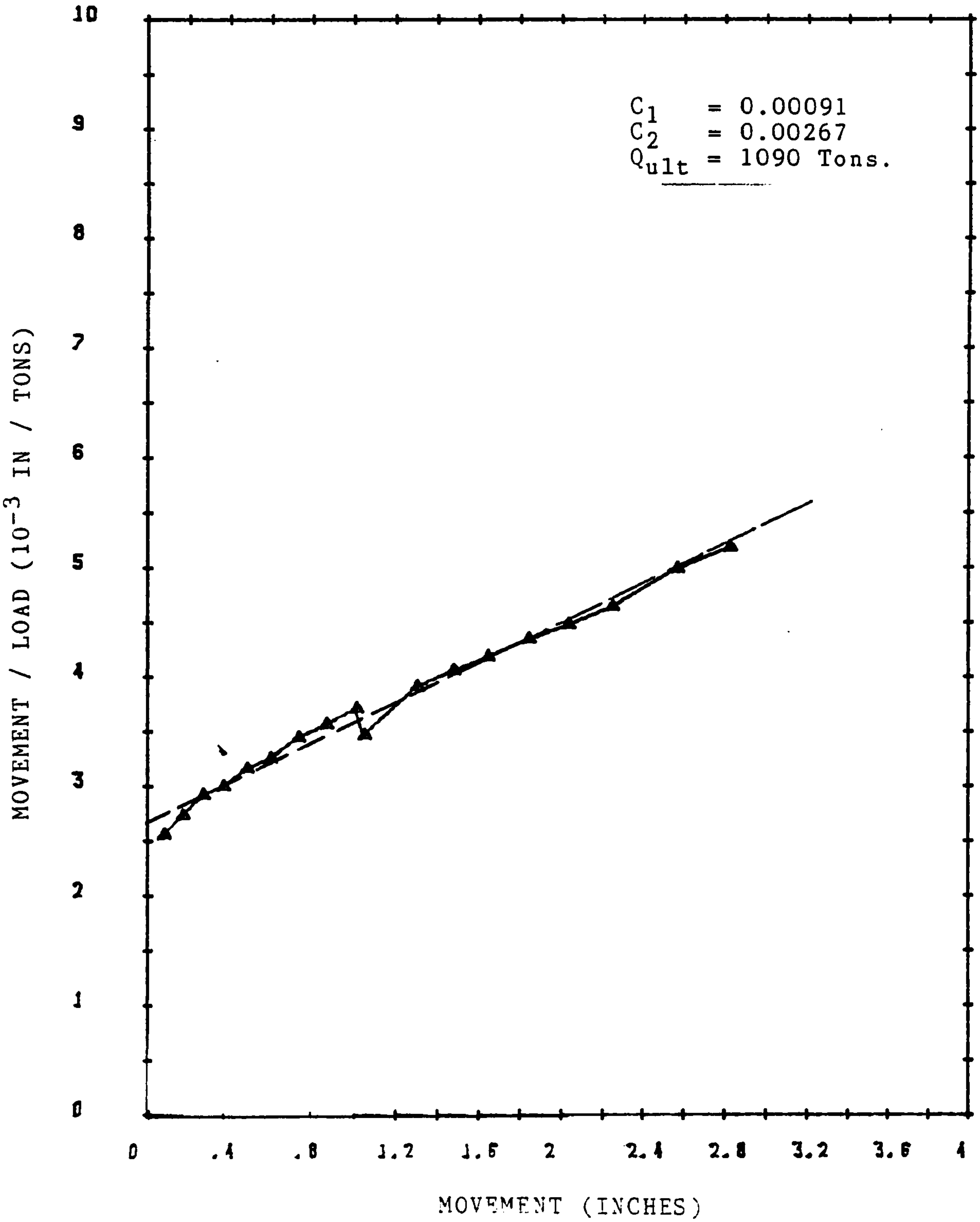

Fig. 5.11: Construction of Chin limit load, Pile 2.

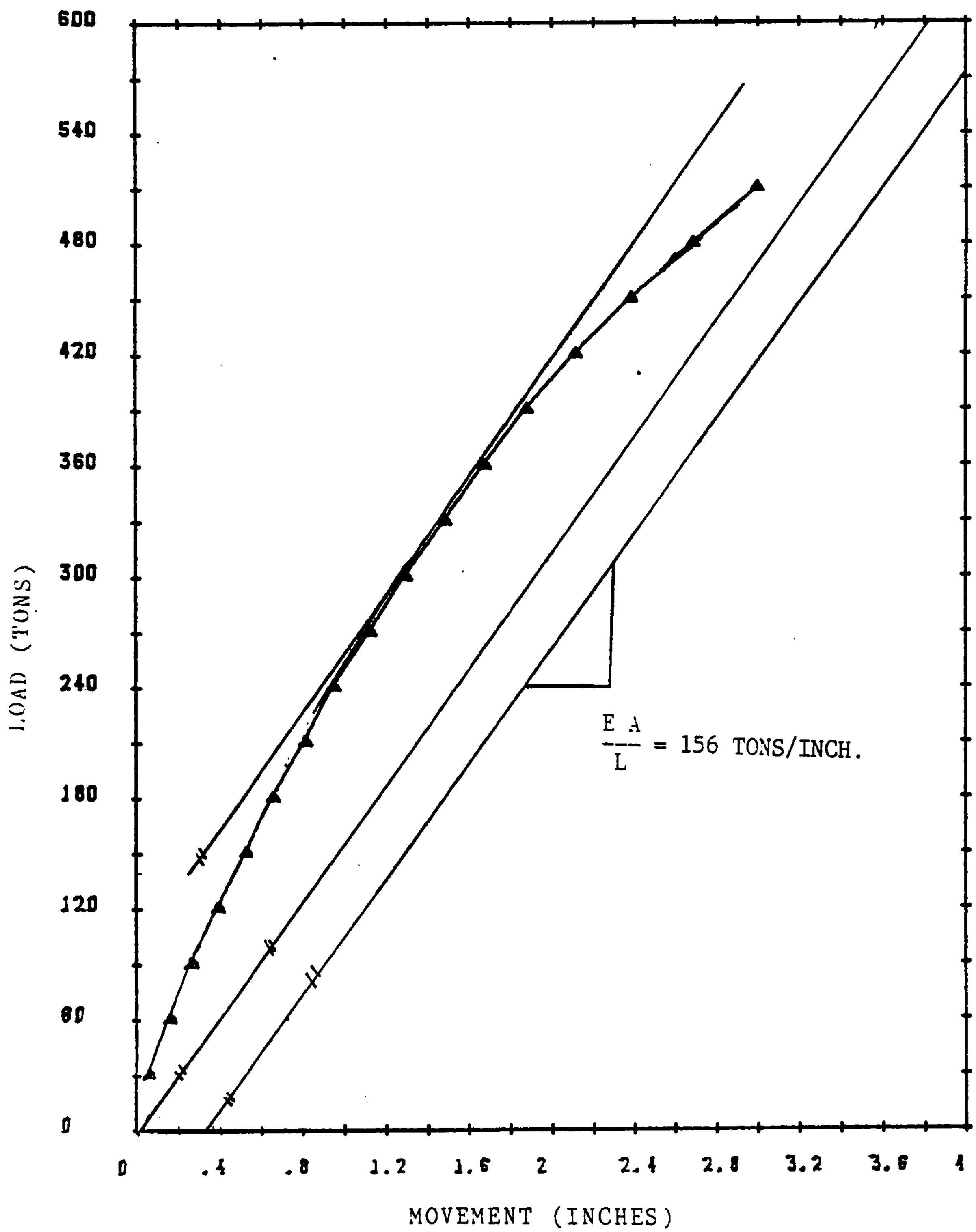

Fig. 5.12: Construction of Davisson limit load, Pile 4.

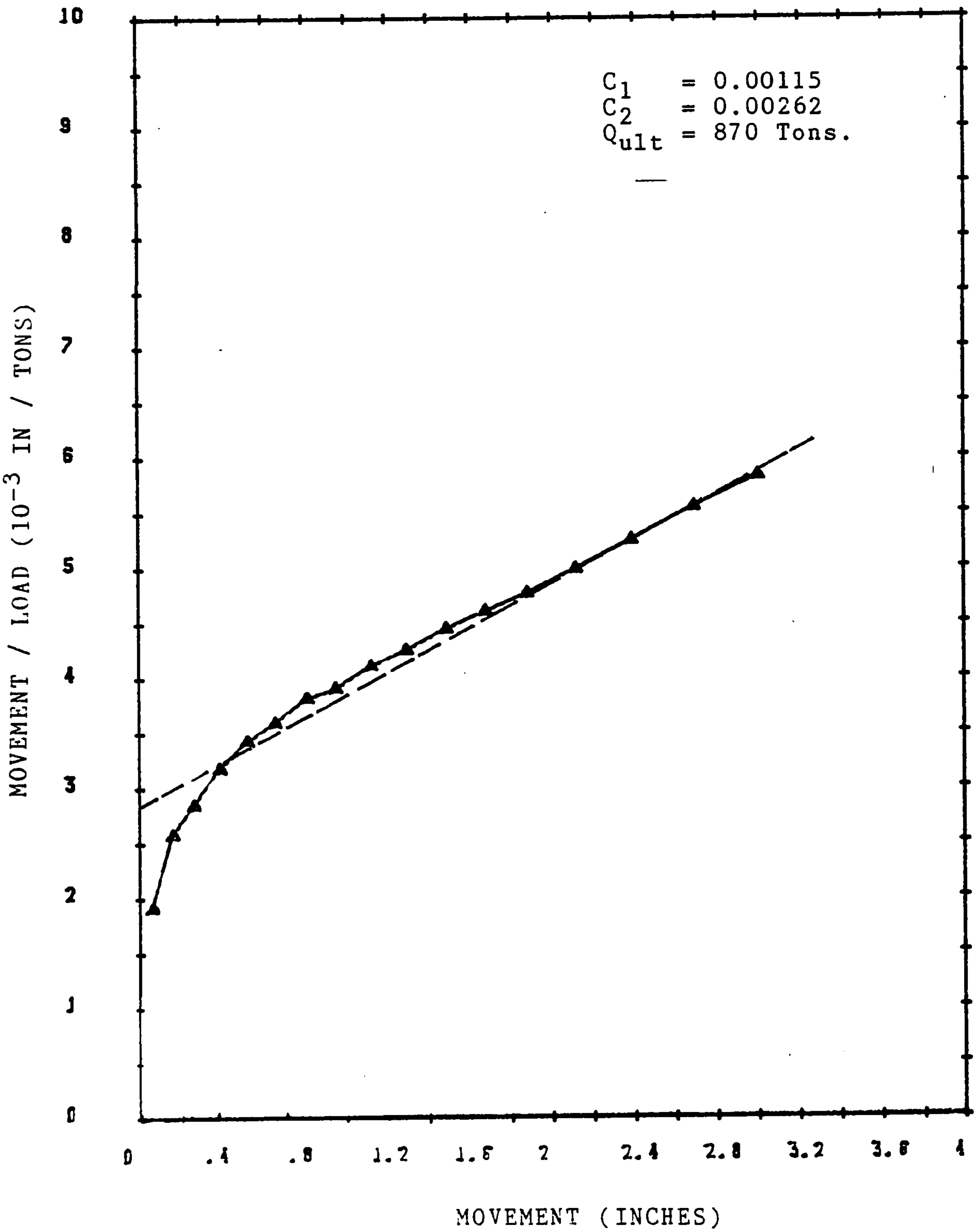

Fig. 5.13: Construction of Chin limit load, Pile 4.

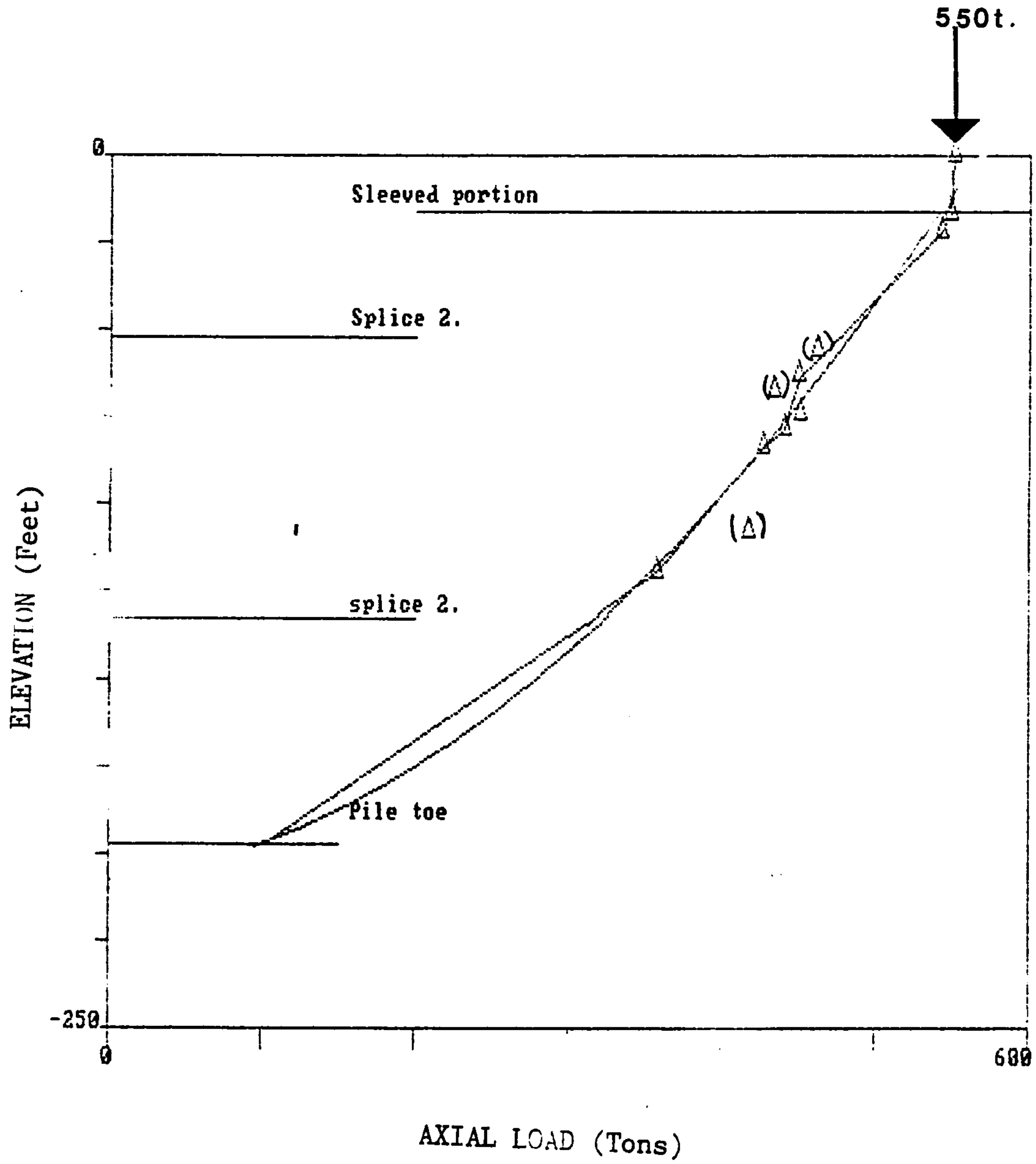

Fig. 5.14: Load distribution versus depth, Pile 1.
Head load, Q = 550 tons.

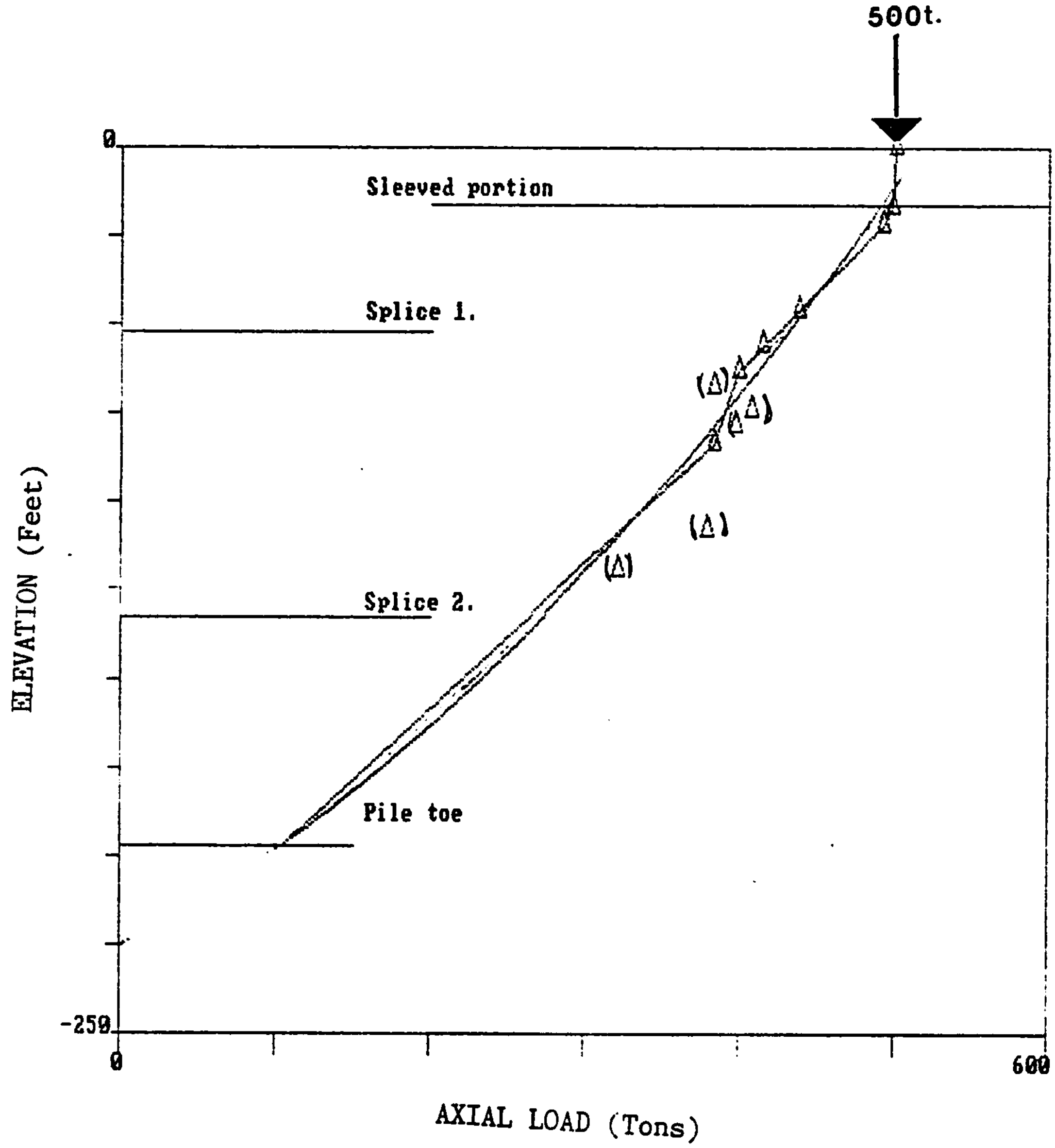

Fig. 5.15: Load distribution versus depth, Pile 1.
Head load, Q = 500 tons.

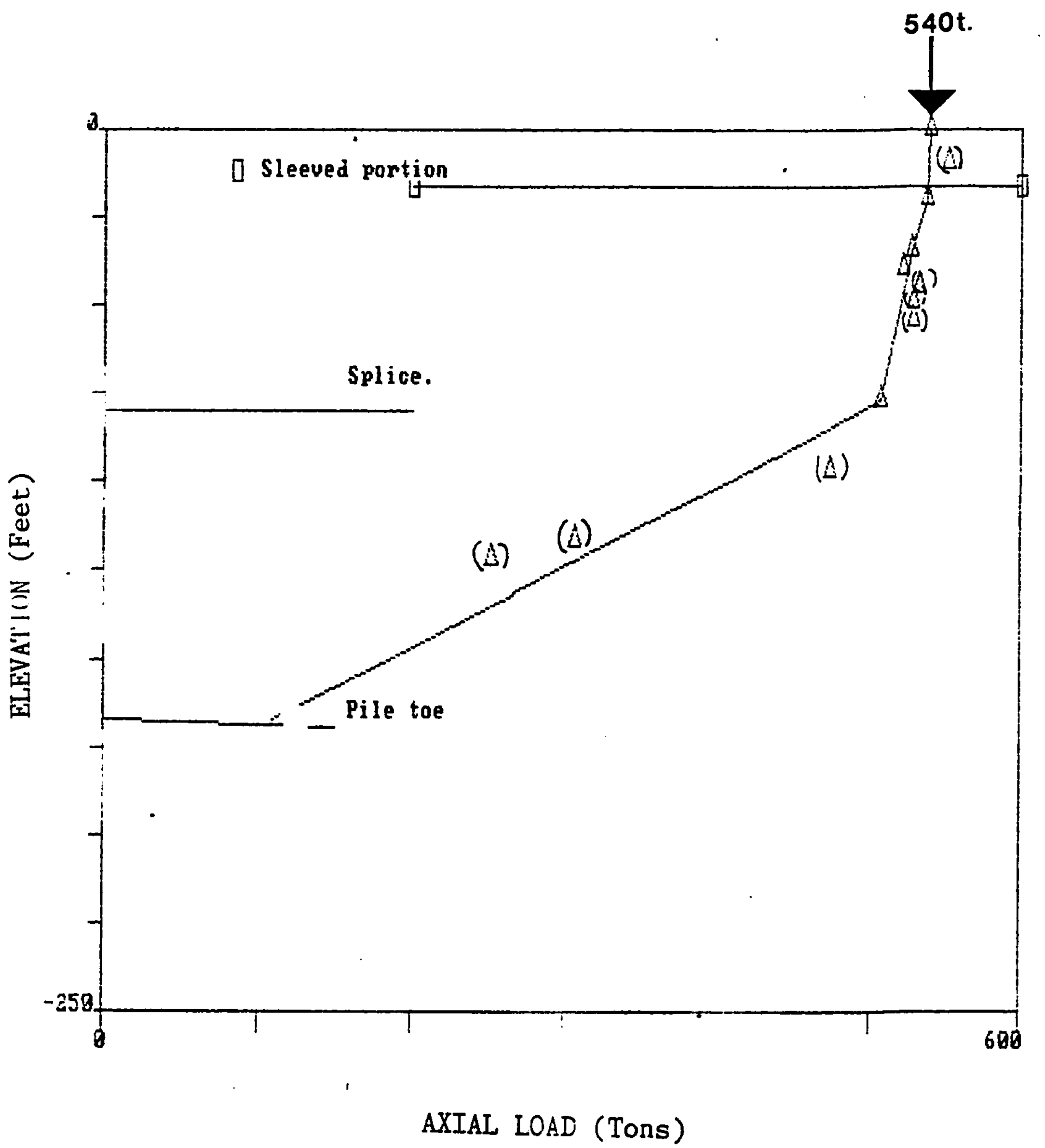

Fig. 5.16: Load distribution versus depth, Pile 2.
Head load, Q = 540 tons.

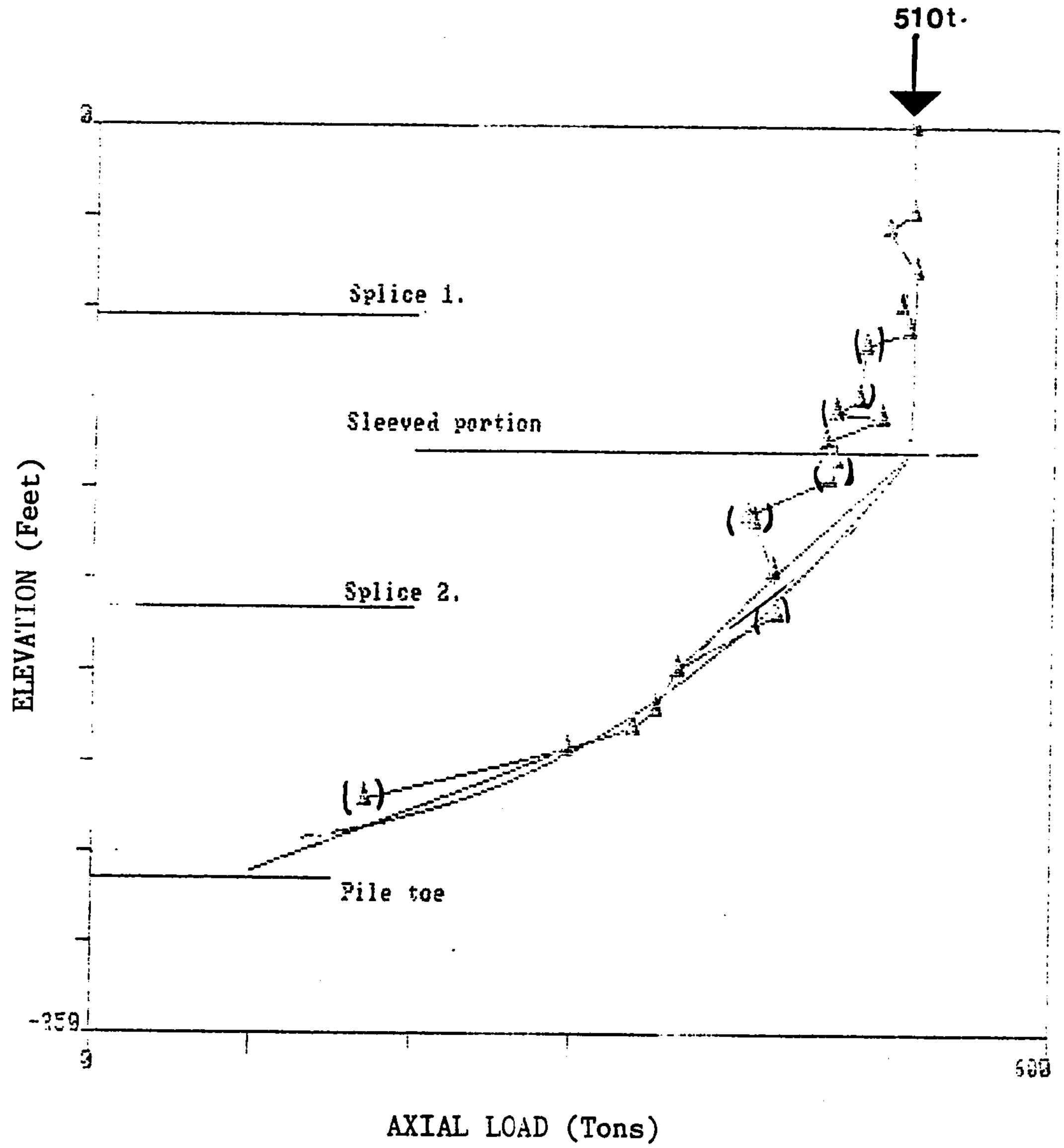

Fig. 5.17: Load distribution versus depth, Pile 4.
Head load, Q = 510 tons.

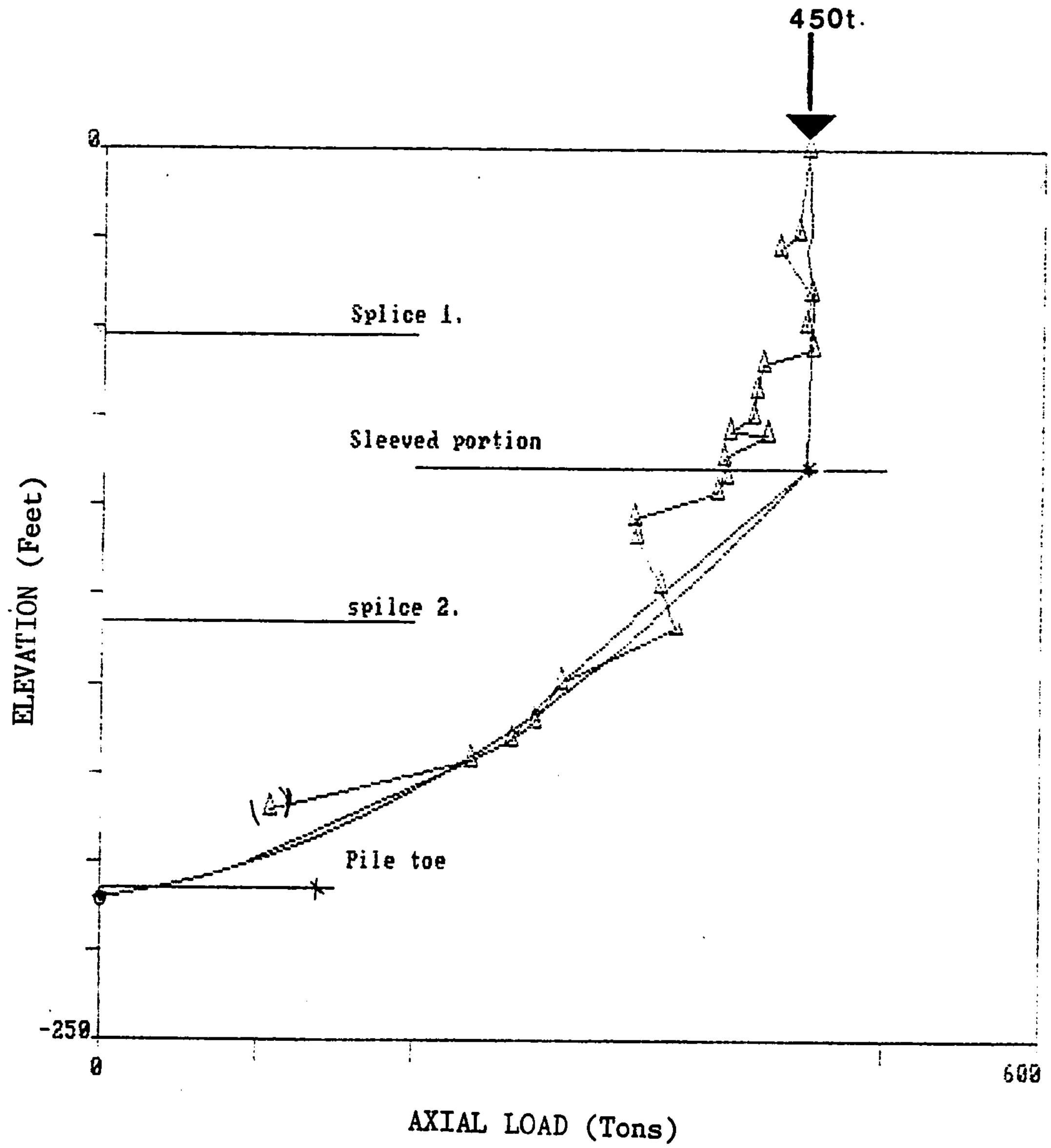

Fig. 5.18: Load distribution versus depth, Pile 4.
Head load, Q = 450 tons.

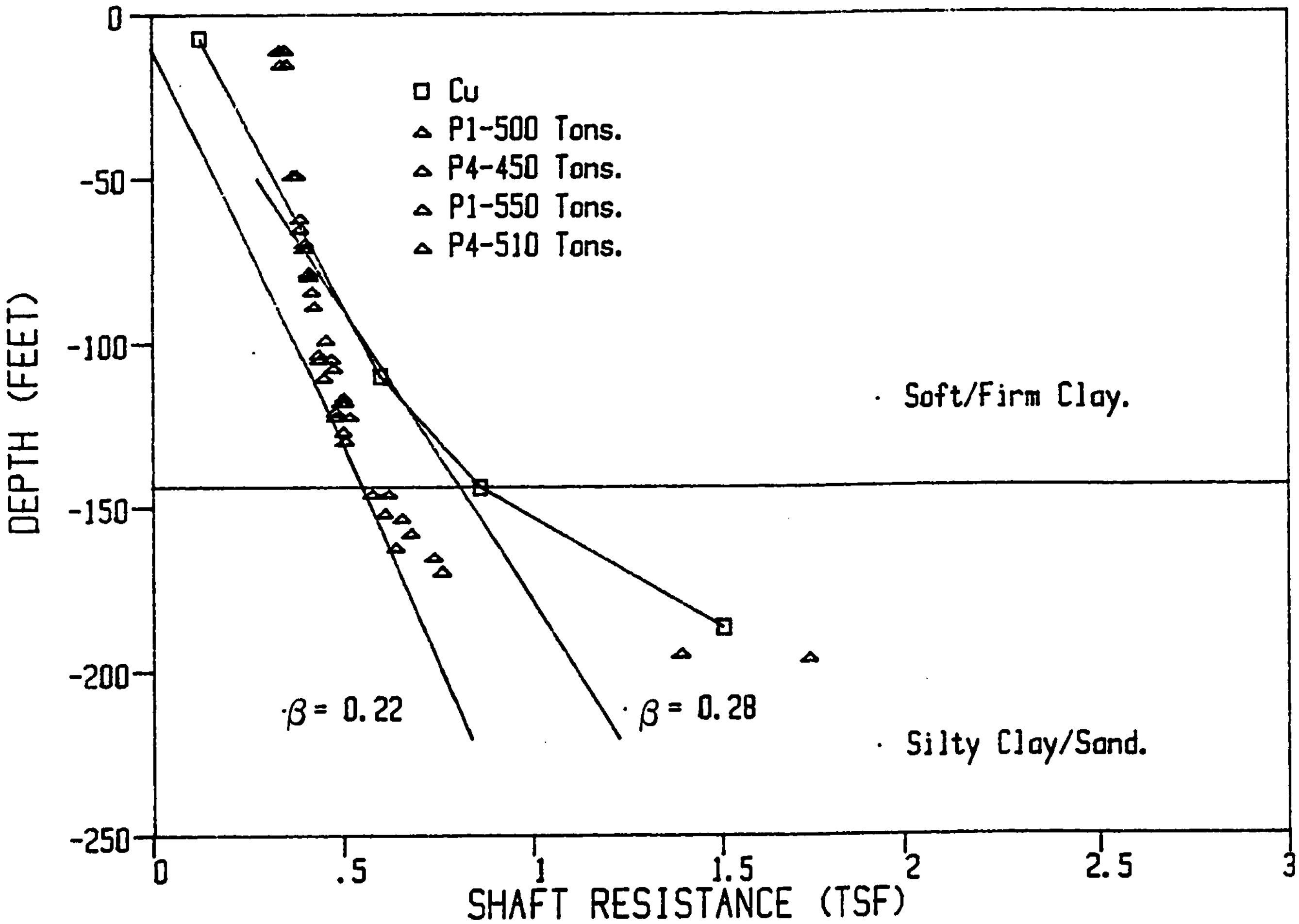

Fig. 5.19: Average shaft resistance versus depth around Piles 1, 2, and 4.

APPENDIX A

DATA AND ANALYSIS TABLES
FOR
PILES 6, 7, 8, AND 9.

TABLE A4.1A: DOWNDRAG MONITORING OF PILE 6, TELLTALES 1 - 5.

MONITORING DATE			TELLTALE DEFLECTION (Inches)				
	DAYS	TELLTALE	T1	T2	T3	T4	T5
		DEPTH (Feet)	12.95	29.87	39.95	59.86	75.99
77/06/06	0	DAY	0	0	0	0	0
77/07/27	51	1	.0038	.0028	.0066	.0064	.003
77/07/28	52	2	.0016	.0018	.0049	.0057	.0017
77/07/29	53	3	.0023	.0022	.0054	.0068	.0031
77/08/01	54	4	.0013	.0008	.0035	.0055	.0019
77/08/02	55	5	.0016	.0012	.0054	.0097	.0053
77/08/03	56	6	.0022	.0019	.0068	.0119	.0118
77/08/04	57	7	.0018	.0015	.0081	.0158	.0164
77/08/05	58	8	.0018	.0018	.0088	.0171	.0201
77/08/08	61	11	.0004	.0008	.0087	.0195	.0339
77/08/09	62	12	.0014	.0019	.011	.0215	.0271
77/08/10	63	13	.0015	.0019	.011	.0229	.029
77/08/11	64	14	.0016	.0023	.0114	.0239	.0306
77/08/12	65	15	.0012	.002	.0113	.0245	.0425
77/08/15	68	18	.0011	.0014	.0108	.0238	.0438
77/08/17	70	20	.0009	.0023	.0117	.0263	.0457
77/08/22	75	25	.0017	.0029	.0131	.0293	.05
77/08/25	78	28	.0015	.0023	.0132	.0303	.0518
77/08/29	82	32	.0015	.0025	.0124	.0288	.0527
77/09/01	85	35	.0023	.0028	.0127	.0287	.0529
77/09/07	91	41	.0018	.0058	.0146	.0304	.0532
77/09/15	99	49	-.001	.0042	.0127	.0308	.056
77/09/20	104	54	-.0003	.0046	.0137	.0329	.0329
77/09/27	111	61	.0009	.006	.0158	.036	.0638
77/10/06	120	70	-.0003	.0052	.0154	.0376	.0677
77/10/11	125	75	.0008	.0066	.0169	.0395	.0707
77/10/18	132	82	.001	.0073	.0177	.0411	.074
77/10/25	139	89	.0016	.0078	.0192	.0438	.0769
77/11/01	145	95	.0017	.0086	.0201	.0456	.0801
77/11/07	151	101	.0014	.0089	.208	.0472	.0827
77/11/14	158	108	.0026	.0097	.0225	.0462	.0871
77/11/21	165	115	.0016	.0096	.0221	.0508	.089
77/11/28	172	122	.0007	.009	.022	.0517	.0913
77/12/05	179	129	.0029	.0115	.0246	.0553	.0963
77/12/19	193	143	.0049	.0144	.0144	.0282	.0614
78/01/03	207	157	.0063	.0162	.031	.0658	.111
78/01/18	222	172	.0058	.0164	.0319	.0685	.1157
78/02/06	240	190	.0052	.0169	.0326	.0721	.1218

TABLE A4.1B: DOWNDRAG MONITORING OF PILE 6, TELLTALES 6 - 10.

MONITORING DATE			TELLTALE DEFLECTION (Inches)				
	DAYS	TELLTALE	T6	T7	T8	T9	T10
		DEPTH (Feet)	84.48	94.5	117.6	139.9	162.4
77/06/06	0	DAY	0	0	0	0	0
77/07/27	51	1	.0071	.0093	.0146	.0179	.0181
77/07/28	52	2	.0102	.0153	.0263	.0331	.0362
77/07/29	53	3	.0143	.021	.0368	.0481	.0545
77/08/01	54	4	.0158	.023	.0433	.0554	.0643
77/08/02	55	5	.0226	.0331	.058	.0768	.0884
77/08/03	56	6	.0284	.0408	.0717	.0935	.118
77/08/04	57	7	.0364	.0516	.0866	.1142	.1323
77/08/05	58	8	.0417	.0585	.0941	.1314	.1515
77/08/08	61	11	.0478	.0672	.1123	.1494	.1736
77/08/09	62	12	.0507	.0706	.1183	.1587	.1844
77/08/10	63	13	.0533	.0746	.1243	.1664	.1926
77/08/11	64	14	.0559	.0776	.129	.1727	.1993
77/08/12	65	15	.058	.0804	.1343	.1803	.2085
77/08/15	68	18	.0589	.0811	.1375	.1863	.2177
77/08/17	70	20	.0632	.0871	.1447	.1962	.2283
77/08/22	75	25	.0693	.0941	.1565	.213	.2485
77/08/25	78	28	.0723	.0988	.1627	.2219	.2594
77/08/29	82	32	.0713	.0974	.1633	.2234	.2624
77/09/01	85	35	.0705	.0965	.1634	.2248	.2656
77/09/07	91	41	.0737	.1006	.1696	.2248	.2815
77/09/15	99	49	.0791	.1078	.1837	.2562	.304
77/09/20	104	54	.0832	.112	.1904	.2665	.3166
77/09/27	111	61	.0886	.1189	.2003	.2798	.333
77/10/06	120	70	.0939	.1285	.2127	.2947	.3523
77/10/11	125	75	.0972	.1292	.2177	.3033	.3623
77/10/18	132	82	.1009	.1335	.2252	.3133	.3744
77/10/25	139	89	.1058	.1389	.2353	.3246	.3878
77/11/01	145	95	.1093	.1435	.2407	.3352	.4003
77/11/07	151	101	.1129	.147	.2468	.3439	.4101
77/11/14	158	108	.1179	.1532	.2564	.3557	.4248
77/11/21	165	115	.1202	.1567	.2611	.3629	.4333
77/11/28	172	122	.1236	.1599	.2681	.3726	.4445
77/12/05	179	129	.1292	.167	.2773	.3842	.4575
77/12/19	193	143	.1386	.1774	.293	.4048	.4814
78/01/03	207	157	.1463	.1862	.3067	.4232	.503
78/01/18	222	172	.1519	.1965	.3182	.4391	.5221
78/02/06	240	190	.1605	.204	.3341	.4603	.5471

TABLE A4.2A: DOWNDRAG MONITORING OF PILE 7, TELLTALES 1 - 5.

MONITORING DATE			TELLTALES DEFLECTION (Inches)				
	DAYS	TELLTALE	T1	T2	T3	T4	T5
		DEPTH (Feet)	30.56	40.56	50.52	60.52	83.52
77/06/14	0	DAY	0	0	0	0	0
77/07/27	43	1	.0007	.0013	.0026	.0021	.0032
77/07/28	44	2	.0006	.0014	.0016	.002	.0025
77/07/29	45	3	.0009	.0016	.0021	.0023	.0028
77/08/01	47	5	.0017	.0024	.0007	.0032	.0035
77/08/02	48	6	.0022	.0029	.0038	.0043	.0046
77/08/03	49	7	.0021	.0031	.0041	.0044	.007
77/08/04	50	8	.0027	.0041	.005	.0055	.008
77/08/05	51	9	.0031	.0044	.0051	.0056	.0078
77/08/08	54	12	.0032	.0047	.0037	.0044	.0084
77/08/09	55	13	.0039	.0054	.0052	.0061	.0081
77/08/10	56	14	.0042	.0054	.0058	.0066	.0085
77/08/11	57	15	.0046	.0059	.0065	.0072	.0092
77/08/12	58	16	.005	.0064	.0062	.007	.0088
77/08/15	61	19	.005	.0062	.0054	.0066	.0084
77/08/17	63	21	.0045	.0064	.0057	.0068	.0085
77/08/22	68	26	.0058	.0073	.0069	.008	.0094
77/08/25	71	29	.0063	.0079	.0073	.0082	.0094
77/08/29	75	33	.0077	.0091	.0099	.0107	.0116
77/09/01	77	35	.0085	.0099	.0117	.0122	.0134
77/09/07	84	42	.0098	.0111	.0134	.0145	.0156
77/09/15	92	50	.0101	.0113	.0314	.012	.0132
77/09/20	97	55	.01	.011	.0117	.0132	.0155
77/09/27	104	62	.0114	.013	.0164	0	.0208
77/10/06	113	71	.0113	.0135	.0154	.0174	.0216
77/10/11	118	76	.012	.0138	.0171	.0195	.0247
77/10/18	125	83	.0129	.0153	.0189	.0212	.0276
77/10/25	132	90	.0146	.0173	.021	.0237	.0311
77/11/01	138	96	.0151	.0178	.0233	.0248	.0336
77/11/07	144	102	.0153	.0184	.0234	.0259	.0358
77/11/14	151	109	.0168	.0198	.0248	.0275	.0382
77/11/21	158	116	.0166	.0197	.024	.0264	.0381
77/11/28	165	123	.0179	.0207	.025	.0271	.0398
77/12/05	172	130	.0189	.022	.0284	.0307	.0446
77/12/19	186	144	.0211	.025	.0335	.0355	.0513
78/01/03	200	158	.0225	.0269	.0363	.0384	.056
78/01/18	215	173	.0231	.0283	.0374	.0402	.0595
78/02/06	235	193	.0248	.0299	.0382	.0414	.0626

TABLE A4.2B: DOWNDRAG MONITORING OF PILE 7, TELLTALES 6 - 11.

MONITORING DATE			TELLTALE DEFLECTION (Inches)				
	DAYS	TELLTALE	T6	T7	T8	T9	T11
		DEPTH (Feet)	107.46	130.5	140.53	162.48	199.13
77/06/14	0	DAY	0	0	0	0	0
77/07/27	43	1	.0041	.0052	.0047	.006	.0034
77/07/28	44	2	.0048	.005	.0049	.0057	.004
77/07/29	45	3	.006	.0061	.0064	.006	.0058
77/08/01	47	5	.0073	.0071	.0077	.0073	.0078
77/08/02	48	6	.0099	.0102	.0105	.016	.0108
77/08/03	49	7	.0101	.0113	.0116	.0123	.0154
77/08/04	50	8	.0117	.0128	.0135	.0146	.021
77/08/05	51	9	.0121	.0136	.0145	.0167	.0237
77/08/08	54	12	.0141	.0132	.0143	.0194	.0267
77/08/09	55	13	.0136	.0159	.0168	.0213	.0341
77/08/10	56	14	.0147	.0169	.0181	.0241	.0365
77/08/11	57	15	.0151	.0173	.0186	.0262	.0377
77/08/12	58	16	.016	.0196	.0207	.0279	.0426
77/08/15	61	19	.0164	.0199	.0211	.0301	.0446
77/08/17	63	21	.0164	.0209	.0223	.0319	.0446
77/08/22	68	26	.0189	.0242	.0261	.0368	.0517
77/08/25	71	29	.0214	.0266	.0292	.0402	.0577
77/08/29	75	33	.0249	.0315	.0338	.0459	.0643
77/09/01	77	35	.0289	.036	.0385	.0511	.0719
77/09/07	84	42	.0323	.0411	.0441	.0601	.084
77/09/15	92	50	.0341	.0436	.0467	.0677	.0912
77/09/20	97	55	.0365	.0487	.0526	.0736	.0969
77/09/27	104	62	.0446	.0577	.0619	.0842	.1096
77/10/06	113	71	.0461	.0611	.066	.092	.1212
77/10/11	118	76	.0498	.0655	.0701	.0965	.1267
77/10/18	125	83	.0543	.0715	.0766	.1041	.1347
77/10/25	132	90	.0586	.0772	.0824	.1125	.1445
77/11/01	138	96	.0625	.0817	.0878	.1193	.1522
77/11/07	144	102	.0657	.0863	.0923	.1251	.159
77/11/14	151	109	.0693	.0905	.0967	.132	.1683
77/11/21	158	116	.0698	.0922	.0922	.1363	.1749
77/11/28	165	123	.073	.0964	.1029	.1435	.1833
77/12/05	172	130	.079	.1025	.1092	.1485	.1888
77/12/19	186	144	.0878	.1132	.1211	.1615	.2024
78/01/03	200	158	.0945	.1212	.13	.1741	.2167
78/01/18	215	173	.0996	.1274	.1368	.1837	.2311
78/02/06	235	193	.1045	.1351	.1464	.1964	.2492

TABLE A4.3A: DOWNDRAG MONITORING OF PILE 8, TELLTALES 1 - 5.

MONITORING DATE			TELLTALE DEFLECTION (Inches)				
	DAYS	TELLTALE	T1	T2	T3	T4	T5
		DEPTH (Feet)	28.49	43.95	58.48	68.49	91.98
77/06/13	0	DAY	0	0	0	0	0
77/07/27	44	1	.0032	.005	.0059	.0085	.0159
77/07/28	45	2	.0035	.0053	.0076	.0121	.0203
77/07/29	46	3	.0034	.0046	.0079	.0128	.0239
77/08/01	48	5	.0031	.0044	.0085	.0138	.0285
77/08/02	49	6	.0012	.0041	.009	.017	.0347
77/08/03	50	7	.005	.0096	.0158	.0253	.0465
77/08/04	51	8	.0049	.0098	.0159	.026	.0491
77/08/05	52	9	.0052	.012	.0192	.0301	.0556
77/08/08	55	12	.0044	.0141	.0238	.0381	.0688
77/08/09	56	13	.0065	.0171	.027	.0422	.0937
77/08/10	57	14	.0073	.0187	.029	.0441	.076
77/08/11	58	15	.008	.0195	.03	.045	.076
77/08/12	59	16	.0076	.0165	.0255	.0379	.68
77/08/15	62	19	.0079	.0186	.0284	.0424	.0747
77/08/17	64	21	.0083	.0194	.0298	.0443	.078
77/08/22	69	26	.0102	.0229	.0348	.0503	.0807
77/08/25	72	29	.01	.233	.0304	.0526	.091
77/08/29	76	33	.0114	.026	.0393	.0556	.0963
77/09/01	78	35	.0123	.0268	.0393	.0553	.0953
77/09/07	84	41	.0132	.0268	.0378	.0514	.0884
77/09/15	92	49	.0114	.0259	.0372	.0518	.0912
77/09/20	97	54	.0121	.0278	.0381	.0532	.0941
77/09/27	104	61	.0142	.0296	.0409	.0566	.0985
77/10/06	113	70	.0141	.0371	.044	.0613	.1071
77/10/11	118	75	.0156	.032	.0448	.0628	.1084
77/10/18	125	82	.0166	.0346	.047	.0652	.1135
77/10/25	132	89	.0178	.0379	.0507	.0702	.121
77/11/01	139	96	.0188	.0389	.0532	.073	.1252
77/11/07	145	102	.0188	.0404	.0548	.0749	.1287
77/11/14	152	109	.02	.0422	.0577	.0788	.1348
77/11/21	159	116	.0184	.0414	.0576	.0792	.1358
77/11/28	166	123	.0197	.0432	.0599	.0816	.1399
77/12/05	173	130	.0233	.0467	.0639	.0861	.1459
77/12/19	187	144	.0256	.0512	.0697	.0924	.1555
78/01/03	201	158	.027	.0544	.0741	.09798	.1637
78/01/18	216	173	.0286	.0571	.0785	.1025	.1712
78/02/06	234	191	.0297	.06	.083	.1087	.1804

TABLE A4.3B: DOWNDRAG MONITORING OF PILE 8, TELLTALES 6 - 10.

MONITORING DATE			TELLTALE DEFLECTION (Inches)				
	DAYS	TELLTALE	T6	T7	T8	T9	T10
		DEPTH (Feet)	116.98	138.49	148.49	171.98	193.99
77/06/13	0	DAY	0	0	0	0	0
77/07/27	44	1	.021	.0461	.0283	.0354	.0341
77/07/28	45	2	.0306	.0372	.0418	.05	.0486
77/07/29	46	3	.0375	.05	.0582	.0695	.07
77/08/01	48	5	.0472	.0632	.0744	.0884	.0925
77/08/02	49	6	.0583	.0776	.091	.1099	.1143
77/08/03	50	7	.0757	.1007	.1169	.1411	.149
77/08/04	51	8	.0826	.1146	.1311	.1627	.1745
77/08/05	52	9	.0915	.1286	.1473	.1915	.1956
77/08/08	55	12	.1123	.1554	.176	.216	.2338
77/08/09	56	13	.1193	.1644	.1855	.23	.2498
77/08/10	57	14	.1239	.1703	.1921	.2388	.2592
77/08/11	58	15	.1253	.1744	.1973	.2458	.2677
77/08/12	59	16	.1117	.1644	.1868	.2362	.2539
77/08/15	62	19	.1231	.1755	.1999	.2536	.2784
77/08/17	64	21	.1285	.1829	.2083	.2643	.29
77/08/22	69	26	.1433	.2025	.23	.2962	.3214
77/08/25	72	29	.1501	.2122	.241	.307	.3371
77/08/29	76	33	.1562	.2214	.2517	.319	.3514
77/09/01	78	35	.1554	.2214	.2542	.3241	.3576
77/09/07	84	41	.1482	.2152	.2491	.3224	.3584
77/09/15	92	49	.1565	.2285	.2643	.344	.3836
77/09/20	97	54	.1604	.2345	.2726	.3551	.3968
77/09/27	104	61	.1702	.2471	.2837	.3773	.4171
77/10/06	113	70	.1818	.265	.3038	.3993	.4449
77/10/11	118	75	.1845	.2699	.3092	.4059	.4547
77/10/18	125	82	.1924	.2803	.3191	.4201	.4705
77/10/25	132	89	.2044	.295	.3343	.4417	.494
77/11/01	139	96	.2114	.3047	.3452	.4552	.5082
77/11/07	145	102	.2168	.3127	.3547	.468	.5227
77/11/14	152	109	.2261	.3255	.3673	.4841	.5405
77/11/21	159	116	.2293	.3315	.3741	.4937	.5551
77/11/28	166	123	.2357	.3405	.384	.5066	.5652
77/12/05	173	130	.244	.3503	.3951	.5199	.5804
77/12/19	187	144	.2579	.3687	.4149	.5447	.609
78/01/03	201	158	.273	.3862	.434	.5692	.6368
78/01/18	216	173	.2845	.4018	.4522	.592	.6616
78/02/06	234	191	.2988	.4425	.4742	.6198	.6931

TABLE A4.4A: DOWNDRAG MONITORING OF PILE 9, TELLTALES 1 - 5.

MONITORING DATE			TELLTALES DEFLECTION (Inches)				
	DAYS	TELLTALE	T1	T2	T3	T4	T5
		DEPTH (Feet)	31	44.04	59.55	68.52	90.98
77/06/20	0	DAY	0	0	0	0	0
77/07/27	37	1	.0014	.0015	.0022	.0025	.0039
77/07/28	38	2	.0022	.0029	.0029	.0038	.005
77/07/29	39	3	.0009	.0009	.0019	.0028	.004
77/08/01	40	4	.001	.0005	.0019	.0031	.005
77/08/02	41	5	.0017	.0005	.0027	.0042	.0063
77/08/03	42	6	.0021	.0005	.0039	.0047	.0075
77/08/04	43	7	.0025	.0023	.0048	.0065	.0091
77/08/05	44	8	.0032	.0034	.006	.0078	.0102
77/08/08	47	11	.0013	.0039	.0075	.0085	.0098
77/08/09	48	12	.0028	.002	.0064	.0089	.0123
77/08/10	49	13	.0033	.0037	.0066	.0092	.0118
77/08/11	50	14	.0047	.0041	.0078	.0101	.0132
77/08/12	51	15	.0051	.0054	.0089	.0111	.0143
77/08/15	54	18	.0041	.0062	.0079	.0105	.0135
77/08/17	56	20	.0037	.0054	.0068	.0108	.0143
77/08/22	61	25	.0066	.005	.0109	.0159	.0194
77/08/25	64	28	.0065	.0086	.0128	.0167	.0201
77/08/29	68	32	.0077	.0087	.0139	.0188	.023
77/09/01	71	35	.0097	.0102	.0161	.022	.0267
77/09/07	78	42	.0085	.0125	.0159	.0236	.0288
77/09/15	86	50	.0073	.0112	.0148	.0247	.0308
77/09/20	91	55	.0087	.0139	.0168	.0274	.0338
77/09/27	98	62	.0106	.0134	.0188	.0308	.0383
77/10/06	107	71	.0109	.0134	.0201	.0332	.0415
77/10/11	112	76	.0129	.0141	.0229	.0361	.0454
77/10/18	119	83	.0142	.0235	.0254	.0392	.0491
77/10/25	126	90	.0166	.0181	.0291	.0433	.0537
77/11/01	132	96	.0169	.021	.03	.0451	.0555
77/11/07	139	103	.017	.0215	.0305	.0458	.0571
77/11/14	146	110	.019	.0218	.034	.05	.0616
77/11/21	153	117	.017	.024	.0326	.0478	.0608
77/11/28	160	124	.017	.0221	.0335	.0496	.0632
77/12/05	167	131	.0209	.0225	.0383	.0544	.0687
77/12/19	181	145	.0255	.0265	.0456	.0625	.0777
78/01/03	195	159	.0277	.0319	.0496	.0672	.0841
78/01/18	210	174	.0266	.0346	.0498	.068	.0865

TABLE A4.4B: DOWNDRAG MONITORING OF PILE 9, TELLTALES 6 - 10.

MONITORING DATE			TELLTALES DEFLECTION (Inches)				
	DAYS	TELLTALE	T6	T7	T8	T9	T10
		DEPTH (Feet)	116	139.5	148.5	171	192.1
77/06/20	0	DAY	0	0	0	0	0
77/07/27	37	1	.0048	.005	.0048	.0081	.0054
77/07/28	38	2	.0058	.0065	.0067	.0096	.0078
77/07/29	39	3	.0053	.0063	.0083	.0124	.012
77/08/01	40	4	.0073	.0091	.0099	.0158	.0163
77/08/02	41	5	.0091	.0115	.0129	.0188	.0203
77/08/03	42	6	.0106	.0131	.0153	.0224	.0257
77/08/04	43	7	.0123	.0146	.0167	.0248	.0296
77/08/05	44	8	.0133	.0156	.0183	.0263	.0333
77/08/08	47	11	.0137	.0166	.0166	.0267	.0339
77/08/09	48	12	.0162	.0187	.0204	.0302	.0338
77/08/10	49	13	.0159	.0186	.0209	.0308	.0402
77/08/11	50	14	.0172	.0201	.0221	.0328	.042
77/08/12	51	15	.0185	.022	.0248	.036	.0469
77/08/15	54	18	.0181	.0215	.0248	.0355	.0468
77/08/17	56	20	.0195	.0219	.0248	.0356	.0466
77/08/22	61	25	.0252	.0279	.029	.0419	.0553
77/08/25	64	28	.027	.0301	.0316	.0454	.0596
77/08/29	68	32	.0309	.0346	.0362	.0501	.0651
77/09/01	71	35	.0357	.0404	.0326	.0565	.0721
77/09/07	78	42	.0397	.0452	.0464	.0616	.0783
77/09/15	86	50	.0435	.0499	.0517	.0682	.0862
77/09/20	91	55	.0474	.0549	.0568	.0735	.093
77/09/27	98	62	.0538	.0625	.0648	.0822	.103
77/10/06	107	71	.0584	.0689	.0737	.091	.1154
77/10/11	112	76	.0636	.0745	.0768	.0968	.1191
77/10/18	119	83	.0677	.0793	.0825	.1038	.126
77/10/25	126	90	.0733	.0861	.0879	.1102	.1345
77/11/01	132	96	.0768	.0903	.0929	.1151	.1403
77/11/07	139	103	.0803	.0939	.0971	.1202	.1459
77/11/14	146	110	.0849	.1001	.1029	.1269	.154
77/11/21	153	117	.0852	.1014	.1047	.1295	.1573
77/11/28	160	124	.888	.1053	.1085	.1339	.1624
77/12/05	167	131	.0952	.1128	.1166	.1422	.1711
77/12/19	181	145	.107	.1262	.1296	.1561	.1862
78/01/03	195	159	.1156	.1363	.1349	.1673	.1996
78/01/18	210	174	.1201	.1426	.147	.1758	.2094

TABLE A4.5: EFFECTIVE HORIZONTAL EARTH PRESSURE DATA IN PILE 6.

Effective Pressure = Total Pressure - Pore Pressure + 0.63 psi.
All values are in psi.

MONITORING DATE	LEVEL 1			LEVEL 2			LEVEL 3		
	σ_{th}	u	σ'_h	σ_{th}	u	σ'_h	σ_{th}	u	σ'_h
Depth (Feet)	20.8			50.9			80.9		
77/7/27	7.59	8.02	.2	18.26	16.49	2.4	40.91	39.49	2.05
/7/28	7.4	7.89	.14	18.63	16.53	2.73	40.91	39.58	1.96
/7/29	7.09	7.44	.28	18.37	16.04	2.96	40.82	39.06	2.39
/8/01	6.78	7.19	.22	18.49	15.97	3.15	40.7	38.89	2.44
/8/02	6.97	7.41	.19	19.09	16.07	3.65	40.81	38.71	2.73
/8/03	7.04	7.57	.1	19.71	16.08	4.26	41.09	38.81	2.91
/8/04	6.87	7.39	.11	20.65	16.18	5.1	41.28	38.69	3.22
/8/05	6.14	7.39	-.62	21.23	16.05	5.81	41.41	38.51	3.53
/8/08	6.71	7.29	.05	21.16	16.61	5.18	41.26	38.08	3.81
/8/10	6.44	7.14	-.07	21.16	15.72	6.07	41.29	37.64	4.28
/8/11	6.69	7.19	.13	21.41	15.77	6.27	41.34	37.69	4.28
/8/12	7.04	7.74	-.07	22.14	15.98	6.79	41.56	37.69	4.5
/8/15	7.04	7.74	-.07	22.06	15.98	6.71	41.46	37.25	4.84
/8/16	6.94	7.56	.01	21.73	15.72	6.64	41.41	36.91	5.13
/8/17	6.79	7.49	-.07	21.58	15.53	6.68	41.36	36.82	5.17
/8/18	6.74	7.39	-.02	21.66	15.48	6.81	41.41	36.72	5.32
/8/22	6.99	7.68	-.06	21.72	15.8	6.55	41.41	36.29	5.75
/8/25	6.99	7.84	-.22	21.96	16.21	6.38	41.95	36.28	6.3
/8/29	6.94	7.74	-.17	21.76	16.01	6.38	41.36	35.75	6.24
/9/06	7.29	8.09	-.17	22.57	16.25	6.95	41.76	35.22	7.17
/9/13	7.19	8.09	-.27	22.13	16.46	6.3	41.6	34.78	7.45
/9/20	6.89	7.59	-.07	21.46	15.77	6.32	41.41	34.15	7.89
/9/27	6.99	7.79	-.17	21.41	16.01	6.03	41.21	33.86	7.98
/10/04	7.09	7.71	.01	21.53	16.4	5.76	41.11	34.3	7.44
/10/11	6.99	7.84	-.22	21.26	15.87	6.02	40.96	34.39	7.2
/10/19	7.24	8.14	-.27	21.66	16.01	6.28	41.7	36.96	5.37
/10/25	6.99	7.87	-.25	21.4	15.58	6.0	40.99	39.68	1.94
/11/01	6.97	7.8	-.2	21.22	15.46	6.39	40.89	40.34	1.18
/11/07	6.74	7.49	-.12	20.85	14.85	6.63	40.7	40.17	1.16
/11/14	6.97	7.19	.41	20.42	14.39	6.66	40.55	39.42	1.76
/11/22	6.5	7.27	-.14	20.6	14.42	6.81	40.49	39.81	1.31
/11/28	6.33	6.91	.05	20.25	14.32	6.56	39.44	38.84	1.23
/12/05	6.26	7.24	-.35	20.28	14.14	6.77	40.15	39.39	1.39
/12/19	6.4	6.94	.09	20.19	14.52	6.3	40.11	39.63	1.11
78/1/23	5.29	6.52	-.6	18.81	12.97	6.47	39.39	37.97	2.05

σ_{th} = Total Horizontal Earth Pressure.
u = Pore Pressure.
σ'_h = Effective Horizontal Pressure.

TABLE A4.6: EFFECTIVE HORIZONTAL EARTH PRESSURE DATA IN PILE 7.

Effective Pressure = Total Pressure - Pore Pressure + 0.63 psi.
All values are in psi.

MONITORING DATE	LEVEL 1			LEVEL 2			LEVEL 3		
	σ_{th}	u	σ'_h	σ_{th}	u	σ_t	σ_{th}	u	σ'_h
Depth (Feet)		146			176.2			206.1	
77/7/27	36.47	29.72	7.38	52.90	44.50	9.03	39.42	52.55	-12.50
/7/28	37.29	29.89	8.03	52.80	44.26	9.17		52.45	
/7/29	37.18	29.58	8.23	52.52	43.11	10.04	37.66	52.17	-13.88
/8/01	36.38	29.89	7.12	52.52	43.31	9.84	38.30	52.06	-13.13
/8/02	36.89	30.52	7.00	52.78	43.40	10.01	38.09	52.07	-13.35
/8/03	38.12	30.91	7.84	52.88	43.80	9.71	38.51	52.18	-13.04
/8/04	37.83	31.59	6.87	53.15	43.86	9.92	39.32	52.36	-12.41
/8/05	37.91	32.00	6.54	53.22	43.77	10.08	39.57	52.38	-12.18
/8/08	38.01	31.96	6.68	53.00	43.22	10.41	38.05	52.07	-13.39
/8/10	39.24	32.39	7.48	53.15	43.20	10.58	39.13	52.16	-12.40
/8/11	37.32	32.29	5.66	53.98	43.73	10.88	37.53	51.89	-13.73
/8/12	39.29	32.80	7.12	53.18	42.83	10.98	38.96	52.08	-12.49
/8/15	37.55	32.73	5.45	53.00	42.37	11.26	37.70	51.87	-13.54
/8/16	39.32	33.27	6.68	52.95	42.17	11.41	37.30	51.82	-13.89
/8/17	39.13	32.58	7.18	53.00	42.04	11.59	37.24	51.77	-13.90
/8/18	39.19	32.63	7.19	52.90	41.88	11.65	36.39	51.67	-14.65
/8/22	37.50	32.56	5.57	52.81	42.03	11.41	37.59	51.49	-13.27
/8/25	39.39	32.63	7.39	52.80	42.24	11.19	39.16	51.53	-11.74
/8/25	39.08	32.39	7.32	52.55	42.14	11.04	38.46	51.28	-12.19
/9/01	39.33	32.58	7.38	52.61	43.27	9.97	38.86	51.23	-11.74
/9/06	39.49	33.02	7.10	52.75	42.17	11.21	39.06	51.28	-11.59
/9/13	37.92	32.78	5.77	52.42	42.32	10.73	38.71	51.10	-11.76
/9/20	38.47	32.58	6.52	52.40	41.73	11.30	37.55	50.89	-12.71
/9/27	38.62	32.34	6.91	52.10	40.73	12.00	37.30	50.62	-12.69
/10/04	38.93	32.48	7.08	52.17	41.85	10.95	38.36	50.77	-11.78
/10/11	38.54	32.27	6.90	51.95	41.68	10.90	38.66	50.65	-11.36
/10/14	38.83	32.44	7.02	52.05	41.34	11.34	38.76	50.60	-11.21
/10/25	38.54	32.29	6.88	51.80	41.34	11.09	37.90	50.50	-11.97
/11/01	38.60	32.34	6.89	51.90	40.84	11.69	38.71	50.48	-11.14
/11/07	38.90	33.85	5.68	52.65	42.15	11.13	39.20	51.95	-12.12
/11/14	38.22	32.09	6.76	51.65	40.77	11.51	38.56	50.35	-11.16
/11/22	37.99	31.85	6.77	51.55	40.51	11.67	37.90	50.25	-11.72
/11/28	37.96	31.90	6.69	51.60	40.85	11.38	37.85	50.35	-11.87
/12/05	37.86	31.90	6.59	51.55	40.80	11.38	38.00	50.25	-11.62
/12/19	37.60	31.71	6.52	51.41	41.56	10.48	38.71	50.20	-10.86
78/1/23	36.58	30.36	6.85	51.10	40.57	11.16	34.06	49.82	-15.13

σ_{th} = Total Horizontal Pressure.
u = Pore Pressure.
σ'_h = Horizontal Effective.

TABLE A4.7: EFFECTIVE HORIZONTAL EARTH PRESSURE DATA IN PILE 8.

Effective Pressure = Total Pressure - Pore Pressure + 0.63 psi.
All values are in psi.

MONITORING DATE	LEVEL 1 σ_{th}	LEVEL 1 u	LEVEL 1 σ_h'	LEVEL 2 σ_{th}	LEVEL 2 u	LEVEL 2 σ_h'	LEVEL 3 σ_{th}	LEVEL 3 u	LEVEL 3 σ_h'
Depth (Feet)		154			184.2			214.1	
770727	41.12	32.79	8.96	53.23	50.24	3.62	73.01	52.79	20.85
770728	41.25	32.90	8.98	53.40	50.22	3.81	72.93	52.79	20.77
770729	41.29	33.11	8.81	53.04	50.09	3.58	72.24	52.36	20.51
770801	41.17	32.99	8.81	52.87	49.86	3.64	72.73	52.58	20.78
770802	41.65	33.56	8.72	53.08	50.12	3.59	72.71	52.74	20.60
770803	41.86	33.89	8.60	53.10	50.19	3.54	72.93	52.68	20.88
770804	42.20	34.30	8.53	53.16	50.44	3.35	72.84	52.90	20.57
770805	42.49	34.60	8.52	53.51	50.36	3.78	72.62	52.76	20.49
770808	42.43	34.47	8.59	52.66	50.06	3.23	72.56	52.53	20.66
770810	42.94	34.95	8.62	52.56	49.99	3.20	72.50	52.48	20.65
770811	42.94	34.82	8.75	52.46	49.94	3.15	72.45	52.48	20.60
770812	43.29	35.33	8.59	53.46	50.09	4.00	72.52	52.55	20.60
770815	43.34	35.14	8.83	52.46	49.89	3.20	72.43	52.43	20.63
770816	43.19	35.04	8.78	52.26	49.70	3.19	72.32	52.33	20.62
770817	43.19	35.04	8.78	52.31	49.70	3.24	72.37	52.33	20.67
770818	43.24	35.04	8.83	52.21	49.65	3.19	72.32	52.28	20.67
770822	43.20	34.90	8.93	52.37	49.37	3.63	72.32	52.28	20.67
770825	43.24	34.85	9.02	52.31	48.51	4.43	72.44	52.43	20.64
770829	43.17	34.71	9.09	51.97	49.00	3.60	72.22	52.03	20.82
770901	43.27	35.14	8.76	52.11	49.15	3.59	72.32	51.84	21.11
770906	43.44	35.09	8.98	51.92	49.20	3.35	72.37	52.23	20.77
770913	43.31	34.88	9.06	51.82	48.87	3.58	72.25	52.01	20.87
770920	43.19	34.71	9.11	51.52	48.51	3.64	72.12	51.19	21.56
770927	43.14	34.59	9.18	51.37	48.26	3.74	72.10	50.70	22.03
771004	43.17	34.41	9.39	51.45	48.19	3.89	72.12	51.94	20.81
771011	43.01	36.05	7.59	51.22	47.97	3.88	71.93	51.64	20.92
771019	43.04	82.23	-38.56	51.32	48.01	3.94	72.07	51.94	20.76
771025	43.04	34.06	9.61	51.13	47.86	3.90	71.81	51.72	20.72
771101	41.09	34.08	7.64	51.37	47.95	4.05	72.02	51.97	20.68
771107	42.84	33.89	9.58	50.93	47.65	3.91	71.76	51.59	20.80
771114	42.69	33.74	9.58	50.76	47.38	4.01	71.61	51.39	20.85
771122	42.69	33.57	9.75	50.83	47.37	4.09	71.66	52.50	19.79
771128	42.63	33.60	9.66	50.73	47.29	4.07	71.66	51.44	20.85
771205	42.53	33.46	9.70	51.03	47.22	4.44	71.66	51.44	20.85
771219	42.49	33.27	9.85	50.89	46.98	4.54	71.21	51.38	20.46
780123	41.90	33.82	8.71	50.36	46.45	4.54	71.21	52.63	19.21

σ_{th} = Total Horizontal Pressure.
u = Pore Pressure.
σ_h' = Effective Horizontal Pressure.

TABLE A4.8: EFFECTIVE HORIZONTAL EARTH PRESSURE DATA IN PILE 9.

Effective Pressure = Total Pressure - Pore Pressure + 0.63 psi.
All values are in psi.

MONITORING DATE	LEVEL 1			LEVEL 2			LEVEL 3		
	σ_{th}	u	σ'_h	σ_{th}	u	σ'_h	σ_{th}	u	σ'_h
ELEV (Feet)		154			181.4			214	
77/7/27	95.96	36.22	60.37	59.63	49.45	10.81	79.17	50.68	29.12
/7/28	97.35	36.41	61.57	59.78	49.52	10.89	79.2	50.53	29.3
/7/29	96.09	36.83	59.89	59.59	49.35	10.87	79.15	50.21	29.57
/8/01	95.99	36.52	60.1	59.26	48.88	11.01	78.89	51.98	2.44
/8/02	95.9	36.84	59.69	59.46	49.01	11.08	78.82	50.01	29.44
/8/03	95.59	37.11	59.11	59.47	49.08	11.02	78.88	49.59	29.92
/8/04	97.53	37.7	60.46	59.75	49.18	11.2	78.8	49.58	29.85
/8/05	96.94	38.2	59.37	59.78	49.18	11.23	78.8	49.07	30.36
/8/08	98.24	37.67	61.2	59.61	48.78	11.46	78.7	48.76	30.57
/8/10	96.39	37.7	59.32	59.37	48.64	11.36	78.5	48.25	30.88
/8/11	96.49	37.6	59.52	59.32	48.54	11.41	78.45	48.33	30.75
/8/12	96.89	38.39	59.13	59.57	48.69	11.51	78.45	48.57	30.51
/8/15	41.79	37.55	4.87	59.37	48.41	11.59	78.3	46.98	31.95
/8/16	95.14	38.8	56.97	59.07	48.24	11.46	78.15	46.69	32.09
/8/17	95.64	38.25	58.02	59.12	48.17	11.58	78.15	56.58	22.2
/8/18	96.14	37.65	59.12	59.19	48.1	11.72	78.18	46.45	32.36
/8/22	96.85	38.21	59.27	59.26	47.81	12.08	78.09	45.93	32.79
/8/25	97.64	37.31	60.96	59.07	47.77	11.93	78	45.97	32.66
/8/29	97.14	37.75	60.02	58.77	47.29	12.11	77.75	45.97	32.41
/9/06	47.49	37.45	10.67	58.77	47.31	12.09	77.6	45.45	32.78
/9/13	42.56	38.09	5.1	58.58	45.43	13.78	77.5	45.83	7.45
/9/20	96.54	36.81	60.36	58.26	46.53	12.36	77.25	44.53	33.35
/9/27	96.24	37.41	59.46	58.06	46.54	12.15	77.08	44.63	33.08
/10/04	97.89	36.61	61.91	58.04	46.58	12.09	77.05	44.23	7.44
/10/11	97.99	37.36	61.26	57.71	45.94	12.4	76.8	43.77	33.66
/10/19	43.59	36.81	7.41	57.86	46.19	12.3	76.75	42.23	5.37
/10/25	98.82	37.21	62.24	57.67	46.24	12.06	76.68	41.04	36.27
/11/01	98.94	36.91	62.66	57.36	45.76	12.23	76.61	40.93	36.31
/11/07	97.74	36.32	62.05	57.3	45.4	12.53	76.4	39.69	1.16
/11/14	41.42	34.34	7.71	57.15	45.35	12.43	76.3	38.65	38.28
/11/22	43.94	36.21	8.36	57.14	46.04	11.73	76.2	37.96	38.87
/11/28	98.34	36.02	62.95	57.05	46.19	11.49	76.15	37.62	39.16
/12/05	43.24	35.82	8.05	57.05	46.38	11.3	76.05	36.76	39.92
/12/19	98.93	35.23	64.33	57.3	47.07	10.86	75.85	36.32	40.16
78/01/2				55.14	48.95	6.82	75.4	30.51	45.52

σ_{th} = Total Horizontal Pressure.
u = Pore Pressure.
σ'_h = Effective Horizontal Pressure.

TABLE A5.1A: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 6.

Pile length = 166 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 07 - 27.
Number of Days = 51 Days.
Day 1

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	Secant MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	3	3800	39.95	7.93	4.77	3.78	20.00
0	4	6400	59.86	8.91	4.77	4.25	30.00
1	4	2600	46.91	4.62	4.78	2.21	36.00
0	5	3000	75.99	3.29	4.78	1.57	38.00
0	6	7100	84.48	7.00	4.77	3.34	42.00
1	5	-800	63.04	-1.06	4.78	-0.51	44.00
0	7	9300	94.50	8.20	4.77	3.91	47.00
1	6	3300	71.53	3.84	4.78	1.84	49.00
2	5	2000	46.12	3.61	4.78	1.73	53.00
1	7	5500	81.55	5.62	4.78	2.68	54.00
2	6	3300	54.61	5.04	4.78	2.40	57.00
2	7	6500	64.63	8.38	4.77	4.00	62.00
3	7	2700	54.55	4.12	4.78	1.97	67.00
2	8	11800	87.76	11.20	4.77	5.35	74.00
4	7	2900	34.64	6.98	4.77	3.33	77.00
3	8	8000	77.68	8.58	4.77	4.10	79.00
4	8	8200	57.77	11.83	4.77	5.64	89.00
3	9	11300	99.90	9.43	4.77	4.50	90.00
5	8	11600	41.64	23.21	4.76	11.05	97.00
4	9	11500	79.99	11.98	4.77	5.71	100.00
5	9	14900	63.86	19.44	4.76	9.26	108.00
6	9	10800	55.37	16.25	4.77	7.75	112.00
7	9	8600	45.35	15.80	4.77	7.53	117.00
6	10	11000	86.44	10.60	4.77	5.06	123.00
7	10	8800	67.93	10.80	4.77	5.15	128.00
8	10	3500	44.80	6.51	4.77	3.11	140.00

TABLE A5.18 :AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 6.

Pile length = 166 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 08 - 12.
Number of Days = 65 Days.
Day 15

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	Secant MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	3	11300	39.95	23.57	4.76	11.22	20.00
0	4	24500	59.86	34.11	4.75	16.21	30.00
1	4	23300	46.91	41.39	4.75	19.64	36.00
0	5	42500	75.99	46.61	4.74	22.10	38.00
0	6	58000	84.48	57.21	4.73	27.08	42.00
1	5	41300	63.04	54.59	4.73	25.85	44.00
0	7	80400	94.50	70.90	4.72	33.48	47.00
1	6	56800	71.53	66.17	4.73	31.27	49.00
2	5	40500	46.12	73.18	4.72	34.54	53.00
1	7	79200	81.55	80.93	4.71	38.15	54.00
2	6	56000	54.61	85.45	4.71	40.24	57.00
2	7	75600	64.63	97.48	4.70	45.81	62.00
3	7	69100	54.55	105.56	4.69	49.54	67.00
2	8	132300	87.76	125.63	4.68	58.75	74.00
4	7	55900	34.64	134.48	4.67	62.79	77.00
3	8	123000	77.68	131.95	4.67	61.64	79.00
4	8	109800	57.77	158.39	4.65	73.64	89.00
3	9	169000	99.90	140.97	4.66	65.75	90.00
5	8	91800	41.64	183.72	4.63	85.03	97.00
4	9	155800	79.99	162.31	4.65	75.41	100.00
5	9	137800	63.86	179.82	4.63	83.29	108.00
6	9	122300	55.37	184.06	4.63	85.19	112.00
7	9	99900	45.35	183.57	4.63	84.97	117.00
6	10	150500	86.44	145.09	4.66	67.62	123.00
7	10	128100	67.93	157.15	4.65	73.08	128.00
8	10	74200	44.80	138.02	4.67	64.40	140.00

TABLE A5.1C: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 6.

Pile length = 166 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 09 - 07.
Number of Days = 91 Days.
Day 41

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	Secant MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	3	14600	39.95	30.45	4.75	14.48	20.00
0	4	30400	59.86	42.32	4.75	20.08	30.00
1	4	28600	46.91	50.81	4.74	24.07	36.00
0	5	53200	75.99	58.34	4.73	27.61	38.00
0	6	73700	84.48	72.70	4.72	34.31	42.00
1	5	51400	63.04	67.95	4.72	32.10	44.00
0	7	100600	94.50	88.71	4.71	41.76	47.00
1	6	71900	71.53	83.76	4.71	39.46	49.00
2	5	47400	46.12	85.65	4.71	40.33	53.00
1	7	98800	81.55	100.96	4.70	47.42	54.00
2	6	67900	54.61	103.61	4.69	48.64	57.00
2	7	94800	64.63	122.23	4.68	57.20	62.00
3	7	86000	54.55	131.38	4.67	61.37	67.00
2	8	163800	87.76	155.54	4.65	72.35	74.00
4	7	70200	34.64	168.88	4.64	78.37	77.00
3	8	155000	77.68	166.28	4.64	77.20	79.00
4	8	139200	57.77	200.80	4.61	92.65	89.00
3	9	210300	99.90	175.43	4.64	81.31	90.00
5	8	116400	41.64	232.95	4.59	106.87	97.00
4	9	194400	79.99	202.53	4.61	93.42	100.00
5	9	171600	63.86	223.93	4.60	102.90	108.00
6	9	151100	55.37	227.41	4.59	104.44	112.00
7	9	124200	45.35	228.22	4.59	104.79	117.00
6	10	207800	86.44	200.33	4.61	92.45	123.00
7	10	180900	67.93	221.92	4.60	102.01	128.00
8	10	111900	44.80	208.15	4.61	95.92	140.00

TABLE A5.1n: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 6.

Pile length = 166 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 10 - 06.
Number of Days = 120 Days.
Day 70

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	Secant MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	3	15400	39.95	32.12	4.75	15.27	20.00
0	4	37600	59.86	52.34	4.74	24.79	30.00
1	4	40600	46.91	72.12	4.72	34.05	36.00
0	5	67700	75.99	74.24	4.72	35.03	38.00
0	6	93900	84.48	92.63	4.70	43.57	42.00
0	7	128500	94.50	113.32	4.69	53.11	47.00
1	6	93900	71.53	109.39	4.69	51.30	49.00
2	5	62500	46.12	112.93	4.69	52.93	53.00
1	7	131500	81.55	134.38	4.67	62.74	54.00
2	6	88700	54.61	135.35	4.67	63.19	57.00
2	7	123300	64.63	158.98	4.65	73.91	62.00
3	7	113100	54.55	172.78	4.64	80.12	67.00
2	8	207500	87.76	197.03	4.62	90.98	74.00
4	7	90900	34.64	218.68	4.60	100.58	77.00
3	8	197300	77.68	211.66	4.61	97.48	79.00
4	8	175100	57.77	252.58	4.57	115.47	89.00
3	9	279300	99.90	232.98	4.59	106.89	90.00
5	8	145000	41.64	290.19	4.54	131.76	97.00
4	9	257100	79.99	267.85	4.56	122.11	100.00
5	9	227000	63.86	296.22	4.54	134.35	108.00
6	9	200800	55.37	302.21	4.53	136.92	112.00
7	9	166200	45.35	305.40	4.53	138.29	117.00
6	10	258400	86.44	249.11	4.57	113.96	123.00
7	10	223800	67.93	274.55	4.55	125.02	128.00
8	10	139600	44.80	259.67	4.57	118.56	140.00

TABLE A5.1E: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 6.

Pile length = 166 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 12 - 05.
Number of Days = 179 Days.
Day 129

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	Secant MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	3	24600	39.95	51.31	4.74	24.31	20.00
0	4	55300	59.86	76.99	4.72	36.31	30.00
1	4	52400	46.91	93.09	4.70	43.78	36.00
0	5	96300	75.99	105.61	4.69	49.56	38.00
0	6	129200	84.48	127.45	4.67	59.58	42.00
1	5	93400	63.04	123.47	4.68	57.76	44.00
0	7	167000	94.50	147.27	4.66	68.60	47.00
1	6	126300	71.53	147.14	4.66	68.55	49.00
2	5	84800	46.12	153.22	4.65	71.30	53.00
1	7	164100	81.55	167.69	4.64	77.84	54.00
2	6	117700	54.61	179.61	4.63	83.19	57.00
2	7	155500	64.63	200.50	4.61	92.52	62.00
3	7	142400	54.55	217.54	4.60	100.08	67.00
2	8	265800	87.76	252.39	4.57	115.39	74.00
4	7	111700	34.64	268.72	4.56	122.49	77.00
3	8	252700	77.68	271.09	4.56	123.52	79.00
4	8	222000	57.77	320.24	4.52	144.61	89.00
3	9	359600	99.90	299.97	4.53	135.96	90.00
5	8	181000	41.64	362.23	4.48	162.32	97.00
4	9	328900	79.99	342.65	4.50	154.10	100.00
5	9	287900	63.86	375.69	4.47	167.94	108.00
6	9	255000	55.37	383.78	4.46	171.30	112.00
7	9	217200	45.35	399.12	4.45	177.64	117.00
6	10	328300	86.44	316.50	4.52	143.02	123.00
7	10	290500	67.93	356.37	4.49	159.87	128.00
8	10	180200	44.80	335.19	4.50	150.95	140.00

TABLE A5.1F: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 6.

Pile length = 166 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1978 - 02 - 06.
Number of Days = 240 Days.
Day 190

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	Secant MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	3	32600	39.95	68.00	4.72	32.12	20.00
0	4	72100	59.86	100.37	4.70	47.15	30.00
1	4	66900	46.91	118.84	4.68	55.64	36.00
0	5	121800	75.99	133.57	4.67	62.37	38.00
0	6	160500	84.48	158.32	4.65	73.61	42.00
1	5	116600	63.04	154.13	4.65	71.72	44.00
0	7	204000	94.50	179.89	4.63	83.32	47.00
1	6	155300	71.53	180.93	4.63	83.78	49.00
2	5	104900	46.12	189.54	4.62	87.64	53.00
1	7	198800	81.55	203.15	4.61	93.70	54.00
2	6	143600	54.61	219.13	4.60	100.78	57.00
2	7	187100	64.63	241.25	4.58	110.51	62.00
3	7	171400	54.55	261.84	4.56	119.50	67.00
2	8	317200	87.76	301.20	4.53	136.49	74.00
4	7	131900	34.64	317.31	4.52	143.37	77.00
3	8	301500	77.68	323.44	4.51	145.97	79.00
4	8	262000	57.77	377.94	4.47	168.87	89.00
3	9	427700	99.90	356.77	4.49	160.04	90.00
5	8	212300	41.64	424.87	4.43	188.20	97.00
4	9	388200	79.99	404.43	4.45	179.82	100.00
5	9	338500	63.86	441.72	4.42	195.05	108.00
6	9	299800	55.37	451.21	4.41	198.88	112.00
7	9	256300	45.35	470.97	4.39	206.82	117.00
6	10	386600	86.44	372.71	4.47	166.69	123.00
7	10	343100	67.93	420.90	4.43	186.57	128.00
8	10	207600	44.80	386.16	4.46	172.28	140.00

TABLE A5.2A: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 7.

Pile length = 216 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 07 - 27.
Number of Days = 43 Days.
Day 1

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	1300	40.56	2.67	4.78	1.28	20.00
0	3	2600	50.52	4.29	4.78	2.05	25.00
0	4	2100	60.52	2.89	4.78	1.38	30.00
0	5	3200	83.52	3.19	4.78	1.53	42.00
1	5	2500	52.96	3.93	4.78	1.88	57.00
2	5	900	42.96	1.75	4.78	0.83	62.00
1	6	3400	76.90	3.68	4.78	1.76	69.00
2	6	2800	66.90	3.49	4.78	1.67	74.00
3	6	1500	56.94	2.20	4.78	1.05	97.00
1	7	4500	99.94	3.75	4.78	1.79	81.00
2	7	3900	89.94	3.61	4.78	1.73	86.00
2	8	3400	99.97	2.83	4.78	1.35	91.00
3	8	2100	90.01	1.94	4.78	0.93	96.00
4	8	2600	80.01	2.71	4.78	1.29	101.00
5	7	2000	46.98	3.55	4.78	1.69	107.00
5	8	1500	57.01	2.19	4.78	1.05	112.00
5	9	2800	78.96	2.96	4.78	1.41	123.00
6	9	1900	55.02	2.88	4.78	1.37	135.00
6	11	-700	91.67	-0.64	4.78	-0.30	153.00
7	11	1800	68.63	2.19	4.78	1.04	165.00
8	11	1300	58.60	1.85	4.78	0.88	170.00

TABLE A5.2B: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 7.

Pile length = 216 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 08 - 12.
Number of Days = 58 Days.
Day 16

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	6400	40.56	13.15	4.77	6.27	20.00
0	3	6200	50.52	10.23	4.77	4.88	25.00
0	4	7000	60.52	9.64	4.77	4.60	30.00
0	5	8800	83.52	8.78	4.77	4.19	42.00
1	5	3800	52.96	5.98	4.78	2.86	57.00
2	5	2400	42.96	4.66	4.78	2.22	62.00
1	6	11000	76.90	11.92	4.77	5.69	69.00
2	6	9600	66.90	11.96	4.77	5.70	74.00
3	6	9800	56.94	14.34	4.77	6.84	97.00
1	7	14600	99.94	12.17	4.77	5.81	81.00
2	7	13200	89.94	12.23	4.77	5.83	86.00
2	8	14300	99.97	11.92	4.77	5.69	91.00
3	8	14500	90.01	13.42	4.77	6.40	96.00
4	8	13700	80.01	14.27	4.77	6.80	101.00
5	7	10800	46.98	19.16	4.76	9.13	107.00
5	8	11900	57.01	17.39	4.77	8.29	112.00
5	9	19100	78.96	20.16	4.76	9.60	123.00
6	9	11900	55.02	18.02	4.77	8.59	135.00
6	11	26600	91.67	24.18	4.76	11.51	153.00
7	11	23000	68.63	27.93	4.76	13.29	165.00
8	11	21900	58.60	31.14	4.75	14.81	170.00

TABLE A5.1C : AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 7.

Pile length = 216 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 09 - 07.
Number of Days = 84 Days.
Day 42

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	11100	40.56	22.81	4.76	10.86	20.00
0	3	13400	50.52	22.10	4.76	10.53	25.00
0	4	14500	60.52	19.97	4.76	9.51	30.00
0	5	15600	83.52	15.57	4.77	7.42	42.00
1	5	5800	52.96	9.13	4.77	4.36	57.00
2	5	4500	42.96	8.73	4.77	4.17	62.00
1	6	22500	76.90	24.38	4.76	11.61	69.00
2	6	21200	66.90	26.41	4.76	12.57	74.00
3	6	18900	56.94	27.66	4.76	13.16	97.00
1	7	31300	99.94	26.10	4.76	12.42	81.00
2	7	30000	89.94	27.80	4.76	13.22	86.00
2	8	33000	99.97	27.51	4.76	13.09	91.00
3	8	30700	90.01	28.42	4.76	13.52	96.00
4	8	29600	80.01	30.83	4.75	14.66	101.00
5	7	25500	46.98	45.23	4.74	21.45	107.00
5	8	28500	57.01	41.66	4.75	19.77	112.00
5	9	44500	78.96	46.96	4.74	22.27	123.00
6	9	27800	55.02	42.11	4.75	19.98	135.00
6	11	51700	91.67	47.00	4.74	22.28	153.00
7	11	42900	68.63	52.09	4.74	24.68	165.00
8	11	39900	58.60	56.74	4.73	26.86	170.00

TABLE A6.2D: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 7.

Pile length = 216 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 10 - 06.
Number of Days = 113 Days.
Day 71

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	13500	40.56	27.74	4.76	13.19	20.00
0	3	15400	50.52	25.40	4.76	12.09	25.00
0	4	17400	60.52	23.96	4.76	11.41	30.00
0	5	21600	83.52	21.55	4.76	10.26	42.00
1	5	10300	52.96	16.21	4.77	7.73	57.00
2	5	8100	42.96	15.71	4.77	7.49	62.00
1	6	34800	76.90	37.71	4.75	17.91	69.00
2	6	32600	66.90	40.61	4.75	19.27	74.00
3	6	30700	56.94	44.93	4.74	21.31	97.00
1	7	49800	99.94	41.52	4.75	19.71	81.00
2	7	47600	89.94	44.10	4.74	20.92	86.00
2	8	52500	99.97	43.76	4.74	20.76	91.00
3	8	50600	90.01	46.85	4.74	22.21	96.00
4	8	48600	80.01	50.62	4.74	23.98	101.00
5	7	39500	46.98	70.07	4.72	33.09	107.00
5	8	44400	57.01	64.90	4.73	30.68	112.00
5	9	70400	78.96	74.30	4.72	35.06	123.00
6	9	45900	55.02	69.52	4.72	32.83	135.00
6	11	75100	91.67	68.27	4.72	32.25	153.00
7	11	60100	68.63	72.98	4.72	34.44	165.00
8	11	55200	58.60	78.50	4.72	37.01	170.00

TABLE A6.2 E: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 7.

Pile length = 216 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 12 - 05.
Number of Days = 172 Days.
Day 130

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	22000	40.56	45.20	4.74	21.44	20.00
0	3	28400	50.52	46.85	4.74	22.21	25.00
0	4	30700	60.52	42.27	4.75	20.06	30.00
0	5	44600	83.52	44.50	4.74	21.11	42.00
1	5	25700	52.96	40.44	4.75	19.20	57.00
2	5	22600	42.96	43.84	4.74	20.80	62.00
1	6	60100	76.90	65.13	4.73	30.78	69.00
2	6	57000	66.90	71.00	4.72	33.52	74.00
3	6	50600	56.94	74.05	4.72	34.95	97.00
1	7	83600	99.94	69.71	4.72	32.92	81.00
2	7	80500	89.94	74.59	4.72	35.19	86.00
2	8	87200	99.97	72.69	4.72	34.31	91.00
3	8	80800	90.01	74.81	4.72	35.30	96.00
4	8	78500	80.01	81.76	4.71	38.53	101.00
5	7	57900	46.98	102.70	4.70	48.22	107.00
5	8	64600	57.01	94.43	4.70	44.40	112.00
5	9	103900	78.96	109.65	4.69	51.42	123.00
6	9	69500	55.02	105.26	4.69	49.40	135.00
6	11	109800	91.67	99.81	4.70	46.89	153.00
7	11	86300	68.63	104.79	4.69	49.18	165.00
8	11	79600	58.60	113.20	4.69	53.05	170.00

TABLE A5-2F: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 7.

Pile length = 216 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1978 - 02 - 06.
Number of Days = 235 Days.
Day 193

TELLTALES NO		READING DIFFERENCE *E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	29900	40.56	61.43	4.73	29.05	20.00
0	3	38200	50.52	63.01	4.73	29.79	25.00
0	4	41400	60.52	57.01	4.73	26.98	30.00
0	5	62600	83.52	62.46	4.73	29.53	42.00
1	5	37800	52.96	59.48	4.73	28.14	57.00
2	5	32700	42.96	63.43	4.73	29.99	62.00
1	6	79700	76.90	86.37	4.71	40.67	69.00
2	6	74600	66.90	92.92	4.70	43.71	74.00
3	6	66300	56.94	97.03	4.70	45.60	97.00
1	7	110300	99.94	91.97	4.70	43.26	81.00
2	7	105200	89.94	97.47	4.70	45.81	86.00
2	8	116500	99.97	97.11	4.70	45.64	91.00
3	8	108200	90.01	100.17	4.70	47.06	96.00
4	8	105000	80.01	109.36	4.69	51.29	101.00
5	7	72500	46.98	128.60	4.67	60.11	107.00
5	8	83800	57.01	122.49	4.68	57.31	112.00
5	9	133800	78.96	141.21	4.66	65.85	123.00
6	9	91900	55.02	139.19	4.67	64.94	135.00
6	11	144700	91.67	131.54	4.67	61.45	153.00
7	11	114100	68.63	138.54	4.67	64.64	165.00
8	11	102800	58.60	146.19	4.66	68.12	170.00

TABLE A5.3A: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 8.

Pile length = 208 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 07 - 27.
Number of Days = 44 Days.
Day 1

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6 Psi	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	5000	43.95	9.48	4.77	4.52	22.00
0	3	5900	58.48	8.41	4.77	4.01	29.00
0	4	8500	68.49	10.34	4.77	4.93	34.00
0	5	15900	91.98	14.41	4.77	6.87	46.00
1	4	5300	40.00	11.04	4.77	5.27	48.00
1	5	12700	63.49	16.67	4.77	7.94	60.00
2	5	10900	48.03	18.91	4.76	9.01	68.00
1	6	17800	89.49	16.58	4.77	7.90	73.00
2	6	17100	74.03	19.25	4.76	9.17	81.00
3	6	16200	59.50	22.69	4.76	10.80	88.00
4	6	13600	49.49	22.90	4.76	10.90	93.00
3	7	40200	80.01	41.87	4.75	19.87	98.00
3	8	22400	90.01	20.74	4.76	9.88	103.00
4	8	19800	80.00	20.63	4.76	9.82	108.00
5	8	12400	56.51	18.29	4.76	8.71	120.00
5	9	19500	54.00	30.09	4.76	14.31	132.00
6	9	13300	76.01	14.58	4.77	6.95	145.00
6	10	13100	55.50	19.67	4.76	9.37	156.00
7	10	12000	45.50	21.98	4.76	10.47	166.00
8	10	5800	44.80	10.79	4.77	5.15	171.00

TABLE A5.3B: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 8.

Pile length = 208 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 08 - 12.
Number of Days = 59 Days.
Day 16

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6 Psi	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	16500	43.95	31.29	4.75	14.87	22.00
0	3	25500	58.48	36.34	4.75	17.26	29.00
0	4	37900	68.49	46.11	4.74	21.87	34.00
0	5	68000	91.98	61.61	4.73	29.14	46.00
1	4	30300	40.00	63.13	4.73	29.85	48.00
1	5	60400	63.49	79.28	4.71	37.38	60.00
2	5	51500	48.03	89.35	4.71	42.05	68.00
1	6	104100	89.49	96.94	4.70	45.56	73.00
2	6	95200	74.03	107.16	4.69	50.28	81.00
3	6	86200	59.50	120.73	4.68	56.51	88.00
4	6	73800	49.49	124.27	4.68	58.13	93.00
3	7	138900	80.01	144.67	4.66	67.43	98.00
3	8	161300	90.01	149.34	4.66	69.54	103.00
4	8	148900	80.00	155.10	4.65	72.16	108.00
5	8	118800	56.51	175.19	4.64	81.21	120.00
5	9	168200	54.00	259.57	4.57	118.51	132.00
6	9	124500	76.01	136.50	4.67	63.71	145.00
6	10	142200	55.50	213.51	4.60	98.30	156.00
7	10	89500	45.50	163.92	4.64	76.14	166.00
8	10	67100	44.80	124.81	4.68	58.38	171.00

TABLE A5.3C: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 8.

Pile length = 208 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 09 - 07.
Number of Days = 84 Days.
Day 41

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6 Psi	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	26800	43.95	50.82	4.74	24.08	22.00
0	3	37800	58.48	53.86	4.74	25.51	29.00
0	4	51400	68.49	62.54	4.73	29.57	34.00
0	5	88400	91.98	80.09	4.71	37.75	46.00
1	4	38200	40.00	79.58	4.71	37.52	48.00
1	5	75200	63.49	98.70	4.70	46.38	60.00
2	5	61600	48.03	106.88	4.69	50.15	68.00
1	6	135000	89.49	125.71	4.68	58.79	73.00
2	6	121400	74.03	136.66	4.67	63.78	81.00
3	6	110400	59.50	154.62	4.65	71.94	88.00
4	6	96800	49.49	163.00	4.65	75.72	93.00
3	7	177400	80.01	184.77	4.63	85.50	98.00
3	8	211300	90.01	195.63	4.62	90.35	103.00
4	8	197700	80.00	205.94	4.61	94.94	108.00
5	8	160700	56.51	236.98	4.58	108.64	120.00
5	9	234000	54.00	361.11	4.48	161.85	132.00
6	9	174200	76.01	190.98	4.62	88.28	145.00
6	10	210200	55.50	315.62	4.52	142.65	156.00
7	10	143200	45.50	262.27	4.56	119.69	166.00
8	10	109300	44.80	203.31	4.61	93.77	171.00

TABLE A6.3 D: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 8.

Pile length = 208 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 10 - 06.
Number of Days = 113 Days.
Day 70

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6 Psi	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	37100	43.95	70.35	4.72	33.22	22.00
0	3	44000	58.48	62.70	4.73	29.65	29.00
0	4	61300	68.49	74.59	4.72	35.19	34.00
0	5	107100	91.98	97.03	4.70	45.60	46.00
1	4	47200	40.00	98.33	4.70	46.21	48.00
1	5	93000	63.49	122.07	4.68	57.12	60.00
2	5	70000	48.03	121.45	4.68	56.84	68.00
1	6	167700	89.49	156.16	4.65	72.63	73.00
2	6	144700	74.03	162.88	4.65	75.67	81.00
3	6	137800	59.50	193.00	4.62	89.18	88.00
4	6	120500	49.49	202.90	4.61	93.59	93.00
3	7	221000	80.01	230.18	4.59	105.65	98.00
3	8	259800	90.01	240.53	4.58	110.20	103.00
4	8	242500	80.00	252.60	4.57	115.48	108.00
5	8	196700	56.51	290.07	4.54	131.71	120.00
5	9	292200	54.00	450.93	4.41	198.77	132.00
6	9	217500	76.01	238.46	4.58	109.29	145.00
6	10	263100	55.50	395.05	4.45	175.96	156.00
7	10	117900	45.50	215.93	4.60	99.37	166.00
8	10	141100	44.80	262.46	4.56	119.77	171.00

TABLE A53E: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 8.

Pile length = 208 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 12 - 05.
Number of Days = 173 Days.
Day 130

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6 Psi	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	46700	43.95	88.55	4.71	41.68	22.00
0	3	63900	58.48	91.06	4.70	42.84	29.00
0	4	86100	68.49	104.76	4.69	49.17	34.00
0	5	145900	91.98	132.18	4.67	61.74	46.00
1	4	62800	40.00	130.83	4.67	61.13	48.00
1	5	122600	63.49	160.92	4.65	74.78	60.00
2	5	99200	48.03	172.11	4.64	79.83	68.00
1	6	220700	89.49	205.52	4.61	94.75	73.00
2	6	197300	74.03	222.09	4.60	102.09	81.00
3	6	180100	59.50	252.24	4.57	115.32	88.00
4	6	157900	49.49	265.88	4.56	121.26	93.00
3	7	286400	80.01	298.30	4.53	135.24	98.00
3	8	331200	90.01	306.63	4.53	138.81	103.00
4	8	309000	80.00	321.88	4.51	145.31	108.00
5	8	249200	56.51	367.49	4.48	164.52	120.00
5	9	374000	54.00	577.16	4.30	248.40	132.00
6	9	265900	76.01	291.52	4.54	132.33	145.00
6	10	336400	55.50	505.11	4.36	220.39	156.00
7	10	230100	45.50	421.43	4.43	186.79	166.00
8	10	185300	44.80	344.68	4.50	154.96	171.00

TABLE A5.3F: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 8.

Pile length = 208 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1978 - 02 - 06.
Number of Days = 234 Days.
Day 191

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6 Psi	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	60000	43.95	113.77	4.69	53.31	22.00
0	3	83000	58.48	118.27	4.68	55.38	29.00
0	4	108700	68.49	132.26	4.67	61.78	34.00
0	5	180400	91.98	163.44	4.65	75.92	46.00
1	4	79000	40.00	164.58	4.64	76.44	48.00
1	5	150700	63.49	197.80	4.62	91.32	60.00
2	5	120400	48.03	208.90	4.61	96.25	68.00
1	6	269100	89.49	250.59	4.57	114.60	73.00
2	6	238000	74.03	267.91	4.56	122.14	81.00
3	6	215800	59.50	302.24	4.53	136.93	88.00
4	6	190100	49.49	320.10	4.52	144.55	93.00
3	7	339500	80.01	353.60	4.49	158.71	98.00
3	8	391200	90.01	362.18	4.48	162.30	103.00
4	8	365500	80.00	380.73	4.47	170.03	108.00
5	8	293800	56.51	433.26	4.42	191.61	120.00
5	9	439400	54.00	678.09	4.22	286.19	132.00
6	9	321000	76.01	351.93	4.49	158.00	145.00
6	10	394300	55.50	592.04	4.29	254.08	156.00
7	10	270600	45.50	495.60	4.37	216.63	166.00
8	10	218900	44.80	407.18	4.44	180.95	171.00

TABLE A6.4A: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 9.

Pile length = 219 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 07 - 27.
Number of Days = 38 Days.
Day 1

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	1500	44.02	2.84	4.78	1.36	22.00
0	3	2200	59.55	3.08	4.78	1.47	30.00
0	4	2500	68.52	3.04	4.78	1.45	34.00
0	5	3900	90.98	3.57	4.78	1.71	45.00
1	4	1100	37.52	2.44	4.78	1.17	50.00
1	5	2500	59.98	3.47	4.78	1.66	61.00
2	5	2400	46.94	4.26	4.78	2.04	68.00
1	6	3400	85.00	3.33	4.78	1.59	74.00
2	6	3300	71.96	3.82	4.78	1.83	80.00
3	6	2600	56.45	3.84	4.78	1.83	88.00
2	7	3500	95.44	3.06	4.78	1.46	92.00
3	7	2800	79.93	2.92	4.78	1.39	100.00
3	8	2600	88.96	2.44	4.78	1.16	104.00
4	8	2300	79.99	2.40	4.78	1.14	109.00
5	7	1100	48.50	1.89	4.78	0.90	115.00
5	8	900	57.53	1.30	4.78	0.62	120.00
5	9	4200	80.02	4.37	4.78	2.09	131.00
6	9	3300	55.00	5.00	4.78	2.39	144.00
6	10	600	76.09	0.66	4.78	0.31	154.00
7	10	400	52.61	0.63	4.78	0.30	166.00
8	10	600	43.58	1.15	4.78	0.55	170.00

TABLE A5.48: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 9.

Pile length = 219 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 08 - 12.
Number of Days = 51 Days.
Day 15

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	5400	44.02	10.22	4.77	4.88	22.00
0	3	8900	59.55	12.45	4.77	5.94	30.00
0	4	11100	68.52	13.50	4.77	6.44	34.00
0	5	14300	90.98	13.10	4.77	6.25	45.00
1	4	6000	37.52	13.33	4.77	6.36	50.00
1	5	9200	59.98	12.78	4.77	6.10	61.00
2	5	8900	46.94	15.80	4.77	7.53	68.00
1	6	13400	85.00	13.14	4.77	6.27	74.00
2	6	13100	71.96	15.17	4.77	7.23	80.00
3	6	9600	56.45	14.17	4.77	6.76	88.00
2	7	16600	95.44	14.49	4.77	6.91	92.00
3	7	13100	79.93	13.66	4.77	6.51	100.00
3	8	15900	88.96	14.89	4.77	7.10	104.00
4	8	13700	79.99	14.27	4.77	6.81	109.00
5	7	7700	48.50	13.23	4.77	6.31	115.00
5	8	10500	57.53	15.21	4.77	7.25	120.00
5	9	21700	80.02	22.60	4.76	10.76	131.00
6	9	17500	55.00	26.52	4.76	12.62	144.00
6	10	28400	76.09	31.10	4.75	14.79	154.00
7	10	24900	52.61	39.44	4.75	18.72	166.00
8	10	22100	43.58	42.26	4.75	20.05	170.00

TABLE A5.4c: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 9.

Pile length = 219 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 09 - 07.
Number of Days = 78 Days.
Day 42

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	12500	44.02	23.66	4.76	11.26	22.00
0	3	15900	59.55	22.25	4.76	10.59	30.00
0	4	23600	68.52	28.70	4.76	13.65	34.00
0	5	28800	90.98	26.38	4.76	12.55	45.00
1	4	15100	37.52	33.54	4.75	15.94	50.00
1	5	20300	59.98	28.20	4.76	13.42	61.00
2	5	16300	46.94	28.94	4.76	13.76	68.00
1	6	31200	85.00	30.59	4.75	14.54	74.00
2	6	27200	71.96	31.50	4.75	14.97	80.00
3	6	23800	56.45	35.13	4.75	16.69	88.00
2	7	32700	95.44	28.55	4.76	13.58	92.00
3	7	29300	79.93	30.55	4.75	14.52	100.00
3	8	30500	88.96	28.57	4.76	13.59	104.00
4	8	22800	79.99	23.75	4.76	11.31	109.00
5	7	16400	48.50	28.18	4.76	13.40	115.00
5	8	17600	57.53	25.49	4.76	12.13	120.00
5	9	32800	80.02	34.16	4.75	16.23	131.00
6	9	21900	55.00	33.18	4.75	15.77	144.00
6	10	38600	76.09	42.27	4.75	20.06	154.00
7	10	33100	52.61	52.43	4.74	24.83	166.00
8	10	31900	43.58	61.00	4.73	28.85	170.00

TABLE A540: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 9.

Pile length = 219 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 10 - 06.
Number of Days = 107 Days.
Day 71

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	13400	44.02	25.37	4.76	12.07	22.00
0	3	20100	59.55	28.13	4.76	13.38	30.00
0	4	33200	68.52	40.38	4.75	19.17	34.00
0	5	41500	90.98	38.01	4.75	18.05	45.00
1	4	22300	37.52	49.53	4.74	23.47	50.00
1	5	30600	59.98	42.51	4.74	20.17	61.00
2	5	28100	46.94	49.89	4.74	23.64	68.00
1	6	47500	85.00	46.57	4.74	22.08	74.00
2	6	45000	71.96	52.11	4.74	24.69	80.00
3	6	38300	56.45	56.54	4.73	26.76	88.00
2	7	55500	95.44	48.46	4.74	22.97	92.00
3	7	48800	79.93	50.88	4.74	24.11	100.00
3	8	53600	88.96	50.21	4.74	23.79	104.00
4	8	40500	79.99	42.19	4.75	20.02	109.00
5	7	27400	48.50	47.08	4.74	22.32	115.00
5	8	32200	57.53	46.64	4.74	22.12	120.00
5	9	49500	80.02	51.55	4.74	24.42	131.00
6	9	32600	55.00	49.39	4.74	23.41	144.00
6	10	57000	76.09	62.43	4.73	29.52	154.00
7	10	46500	52.61	73.66	4.72	34.76	166.00
8	10	41700	43.58	79.74	4.71	37.59	170.00

TABLE A5.4E: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 9.

Pile length = 219 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1977 - 12 - 05.
Number of Days = 167 Days.
Day 131

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	22500	44.02	42.59	4.74	20.21	22.00
0	3	38300	59.55	53.60	4.74	25.38	30.00
0	4	54400	68.52	66.16	4.73	31.26	34.00
0	5	68700	90.98	62.93	4.73	29.75	45.00
1	4	33500	37.52	74.40	4.72	35.11	50.00
1	5	47800	59.98	66.41	4.73	31.38	61.00
2	5	46200	46.94	82.02	4.71	38.65	68.00
1	6	74300	85.00	72.84	4.72	34.38	74.00
2	6	72700	71.96	84.19	4.71	39.66	80.00
3	6	56900	56.45	84.00	4.71	39.57	88.00
2	7	90300	95.44	78.85	4.71	37.18	92.00
3	7	74500	79.93	77.67	4.72	36.63	100.00
3	8	78300	88.96	73.35	4.72	34.62	104.00
4	8	62200	79.99	64.80	4.73	30.63	109.00
5	7	44100	48.50	75.77	4.72	35.75	115.00
5	8	47900	57.53	69.38	4.72	32.77	120.00
5	9	73500	80.02	76.54	4.72	36.10	131.00
6	9	47000	55.00	71.21	4.72	33.62	144.00
6	10	75900	76.09	83.13	4.71	39.16	154.00
7	10	58300	52.61	92.35	4.70	43.44	166.00
8	10	54500	43.58	104.21	4.69	48.92	170.00

TABLE A5.4F: AXIAL LOAD COMPUTATION FROM TELLTALE READINGS IN PILE 9.

Pile length = 219 Feet.
Secant Modulus E_s = 4.78 - 0.000825*strain.
Monitoring date = 1978 - 01 - 18.
Number of Days = 210 Days.
Day 174

TELLTALES NO		READING DIFFERENCE E-6(in)	GAUGE LENGTH (Feet)	STRAIN *E-6 (in/in)	SECANT MODULUS *E6(Psi)	AXIAL LOAD (Tons)	DEPTH FROM HEAD (Feet)
0	2	34600	44.02	65.50	4.73	30.96	22.00
0	3	49800	59.55	69.69	4.72	32.91	30.00
0	4	68000	68.52	82.70	4.71	38.97	34.00
0	5	86500	90.98	79.23	4.71	37.35	45.00
1	4	41400	37.52	91.95	4.70	43.26	50.00
1	5	59900	59.98	83.22	4.71	39.21	61.00
2	5	51900	46.94	92.14	4.70	43.34	68.00
1	6	93500	85.00	91.67	4.70	43.12	74.00
2	6	85500	71.96	99.01	4.70	46.52	80.00
3	6	70300	56.45	103.78	4.69	48.72	88.00
2	7	108000	95.44	94.30	4.70	44.34	92.00
3	7	92800	79.93	96.75	4.70	45.47	100.00
3	8	97200	88.96	91.05	4.70	42.84	104.00
4	8	79000	79.99	82.30	4.71	38.78	109.00
5	7	56100	48.50	96.39	4.70	45.31	115.00
5	8	60500	57.53	87.64	4.71	41.26	120.00
5	9	89300	80.02	93.00	4.70	43.74	131.00
6	9	55700	55.00	84.39	4.71	39.75	144.00
6	10	89300	76.09	97.80	4.70	45.96	154.00
7	10	66800	52.61	105.81	4.69	49.65	166.00
8	10	62400	43.58	119.32	4.68	55.86	170.00

APPENDIX B

FIGURES FOR
PILES 6, 7, 8, AND 9.

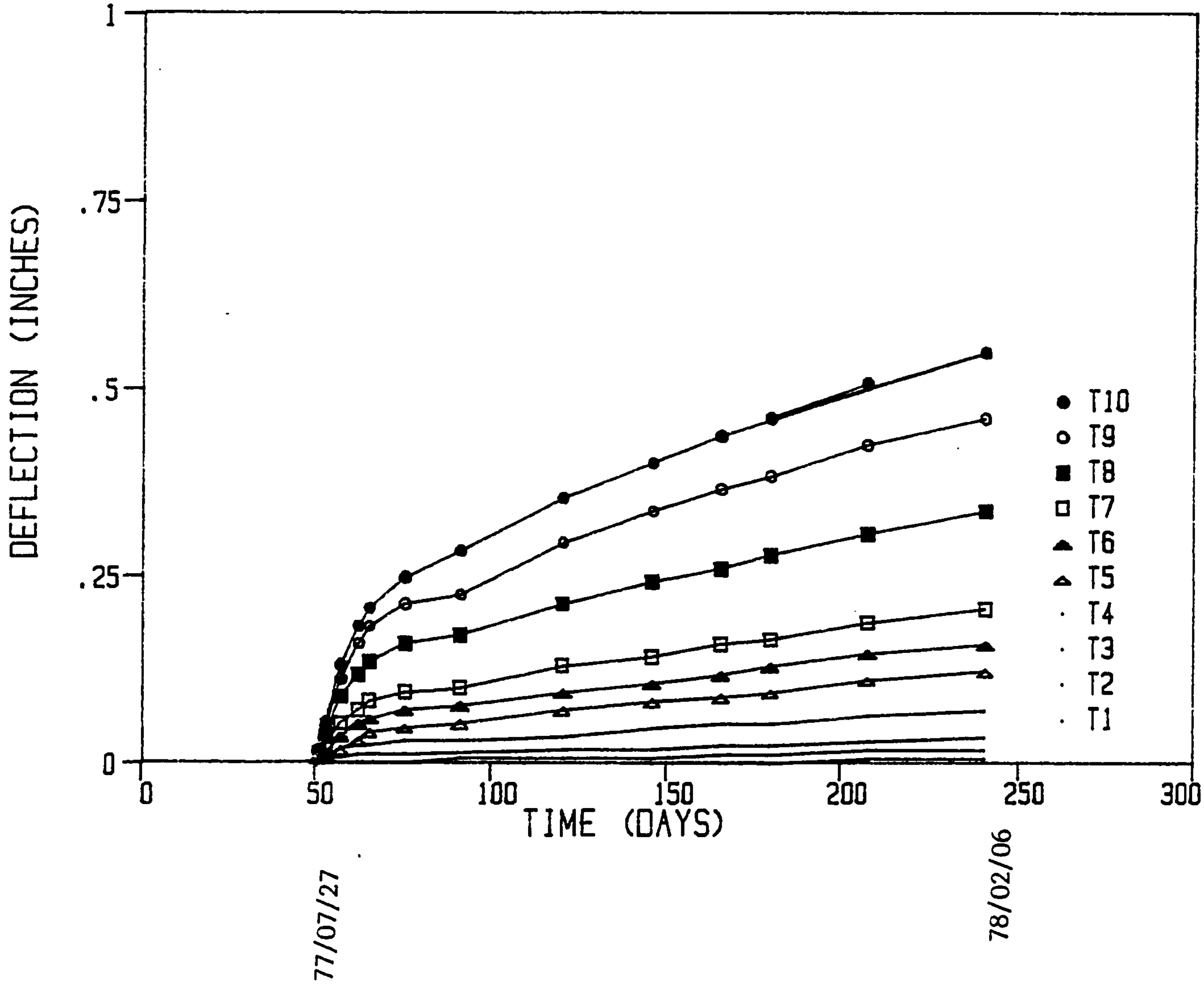

Fig. B4.1: Telltale deflection versus time, Pile 6.

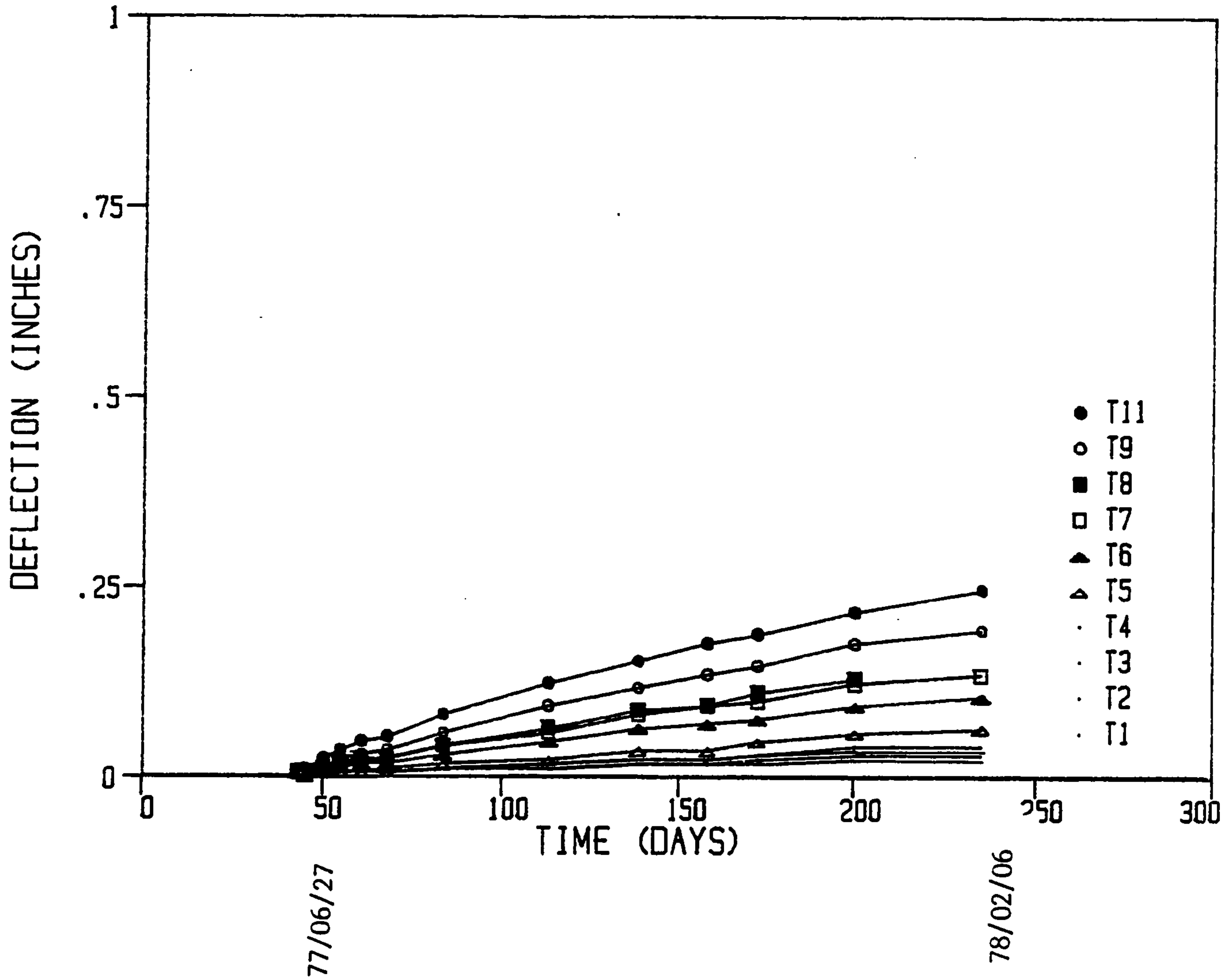

Fig. B4.2: Telltale deflection versus time, Pile 7.

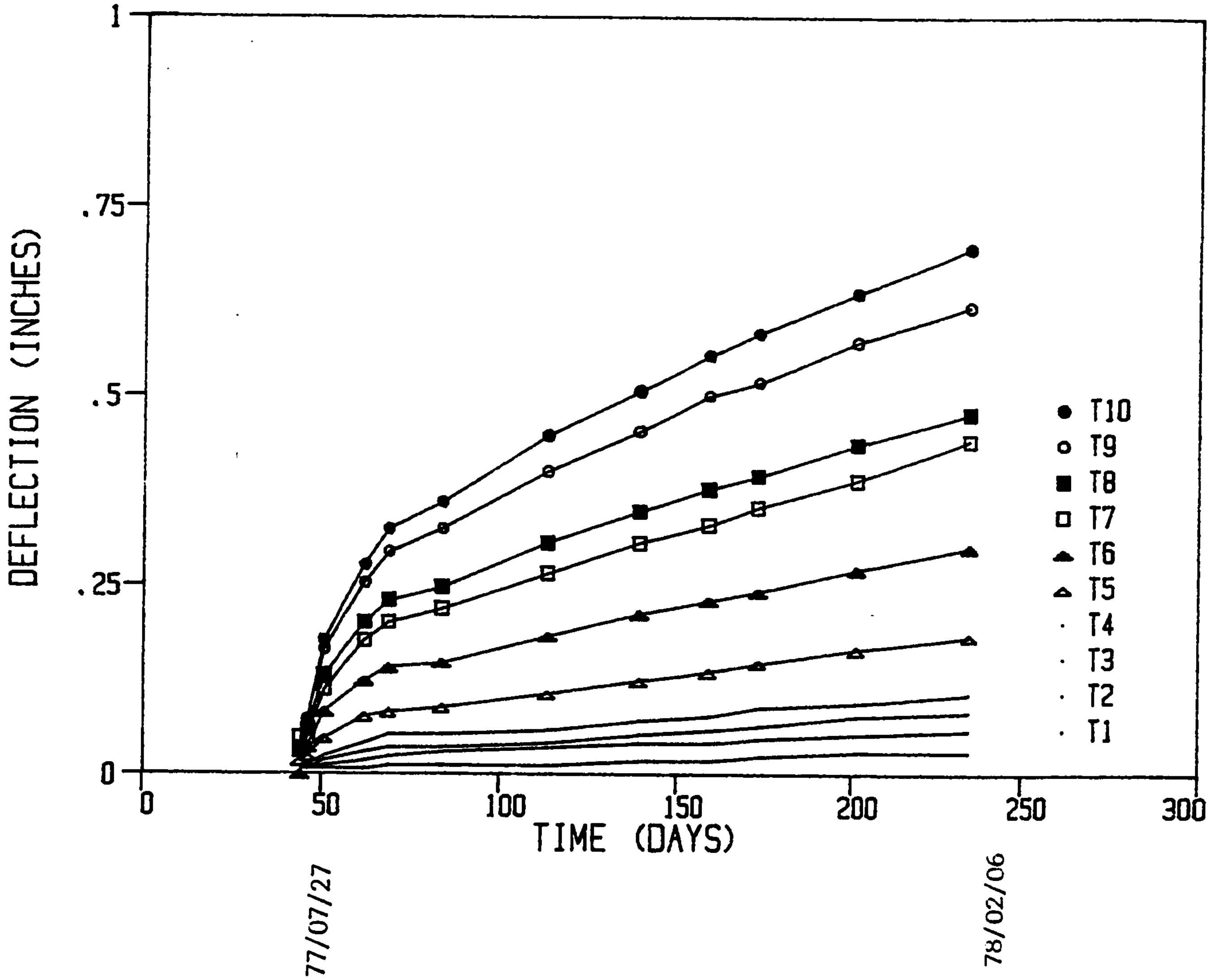

Fig. B4.3: Telltale deflection versus time, Pile 8.

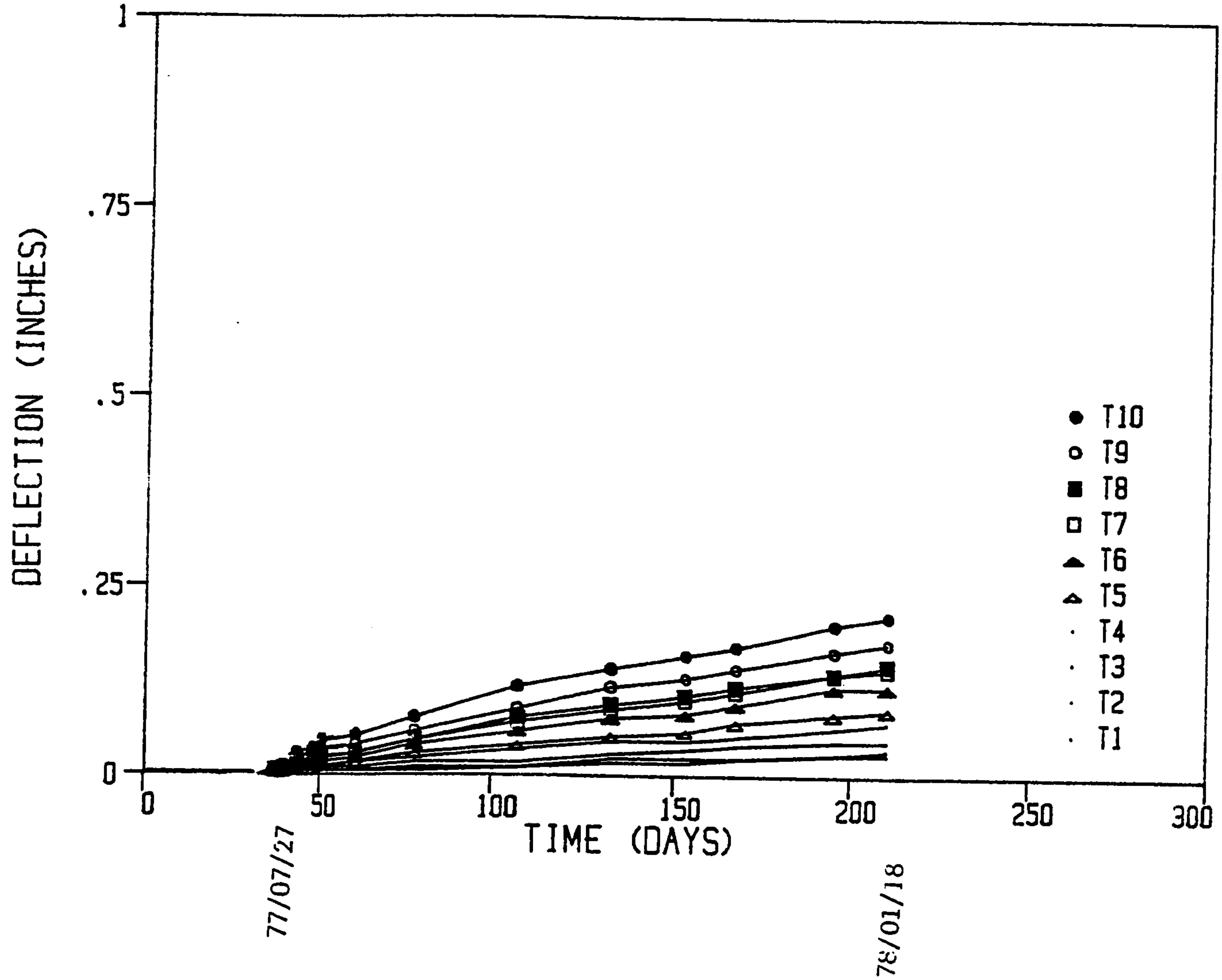

Fig. B4.4: Telltale deflection versus time, Pile 9.

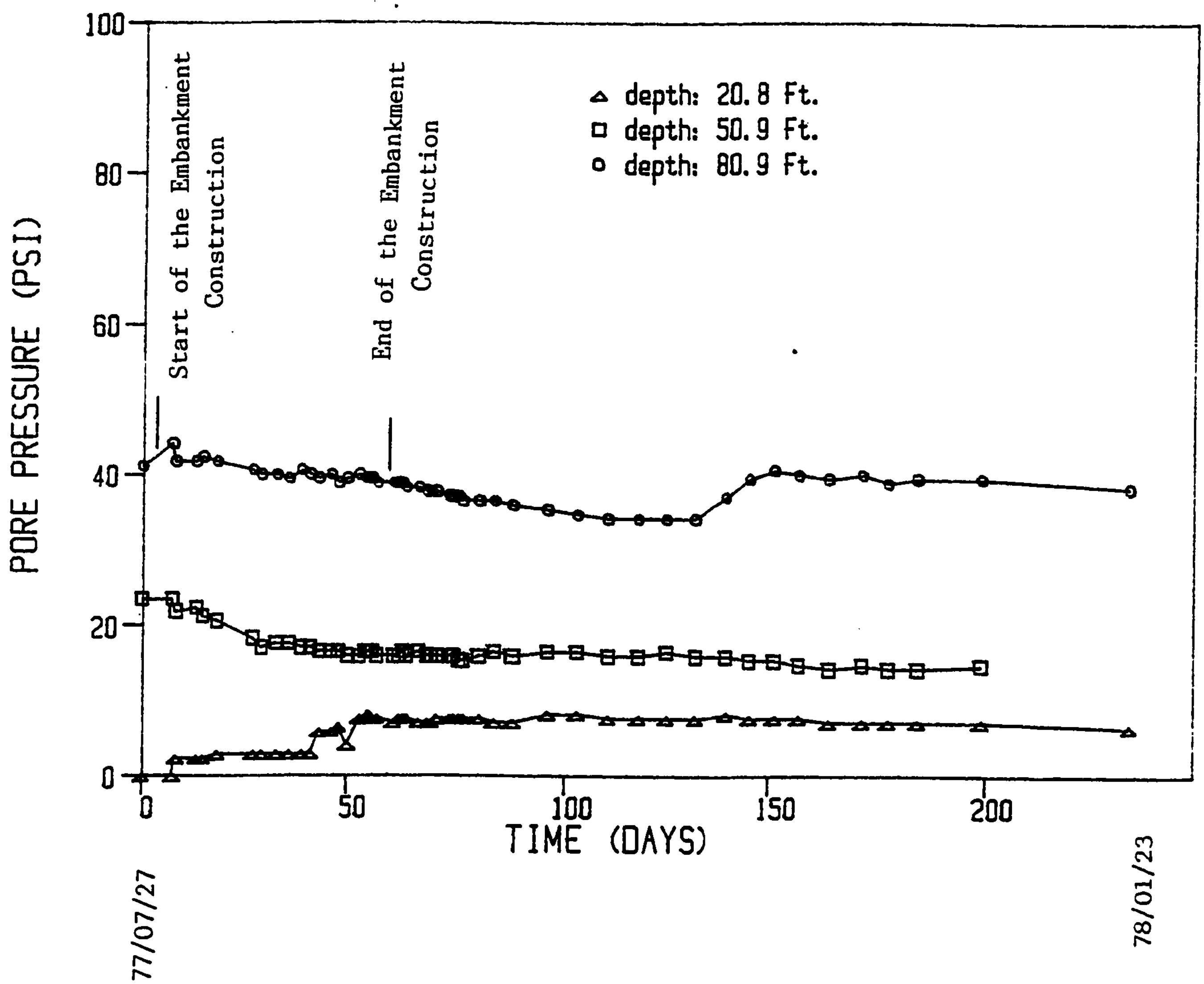

Fig. B4.5: Pore pressure versus time, Pile 6.

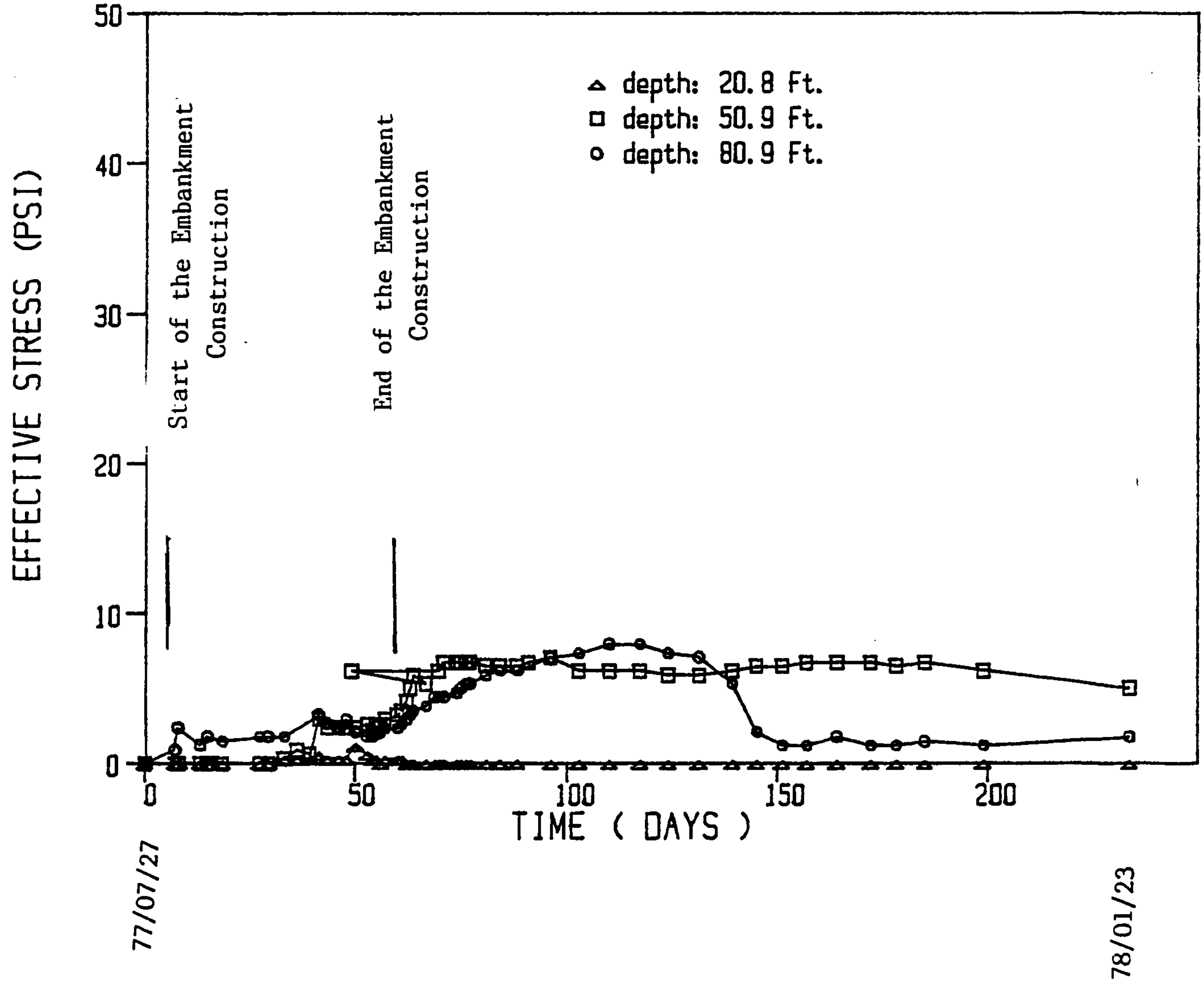

Fig.B4.6: Horizontal effective stress versus time, Pile 6.

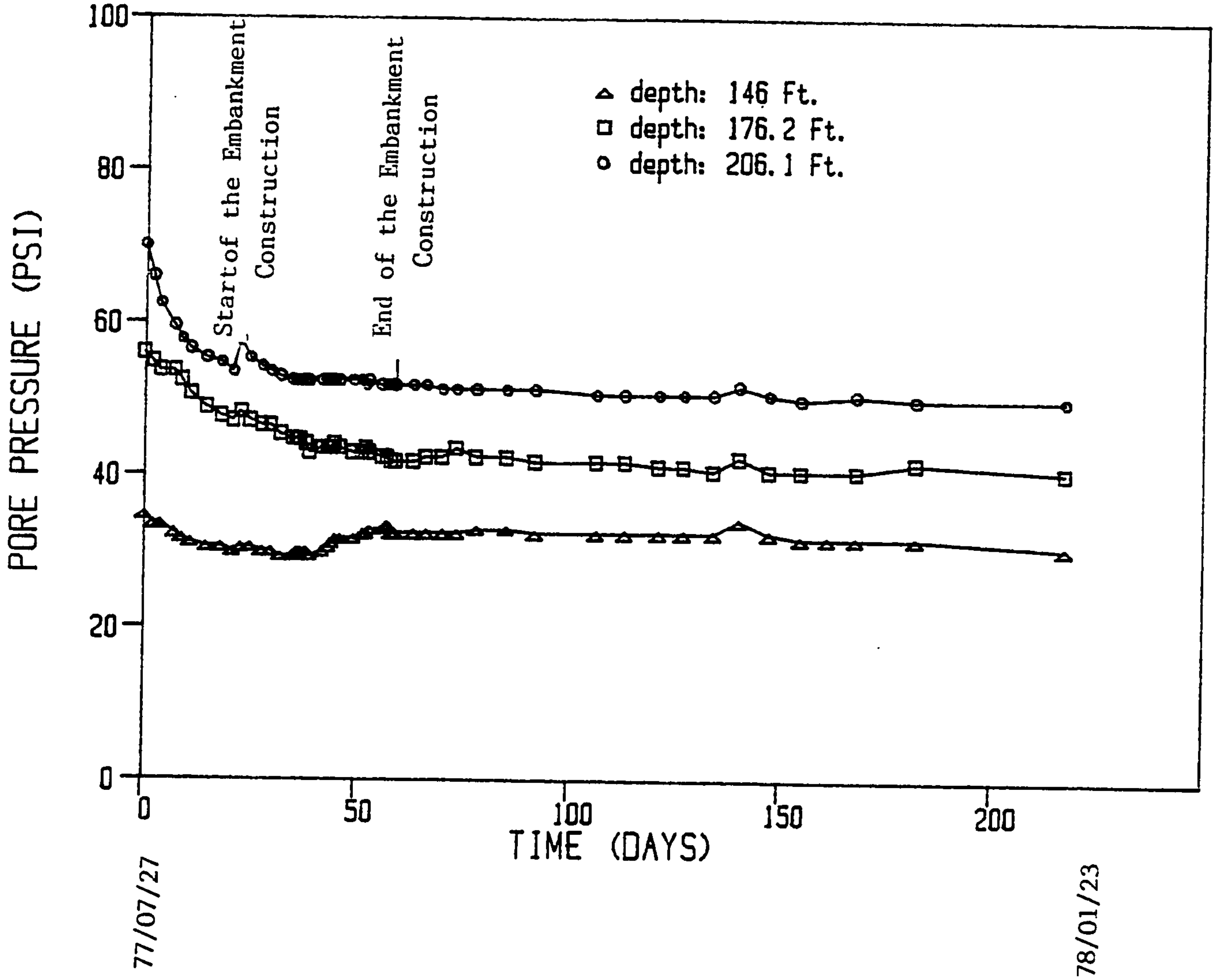

Fig. B4.7: Pore pressure versus time, Pile 7.

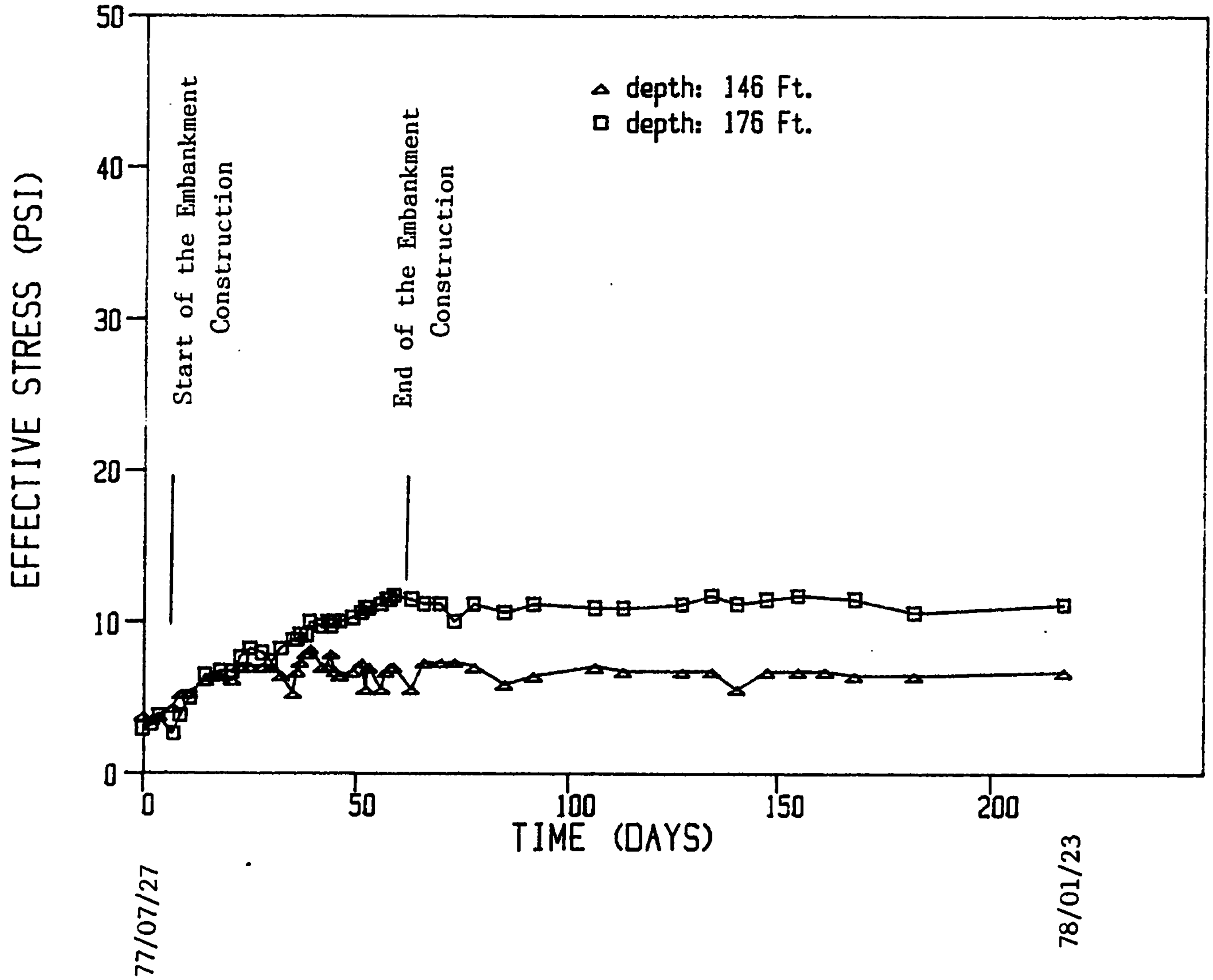

Fig. B4.8: Horizontal effective stress versus time, Pile 7.

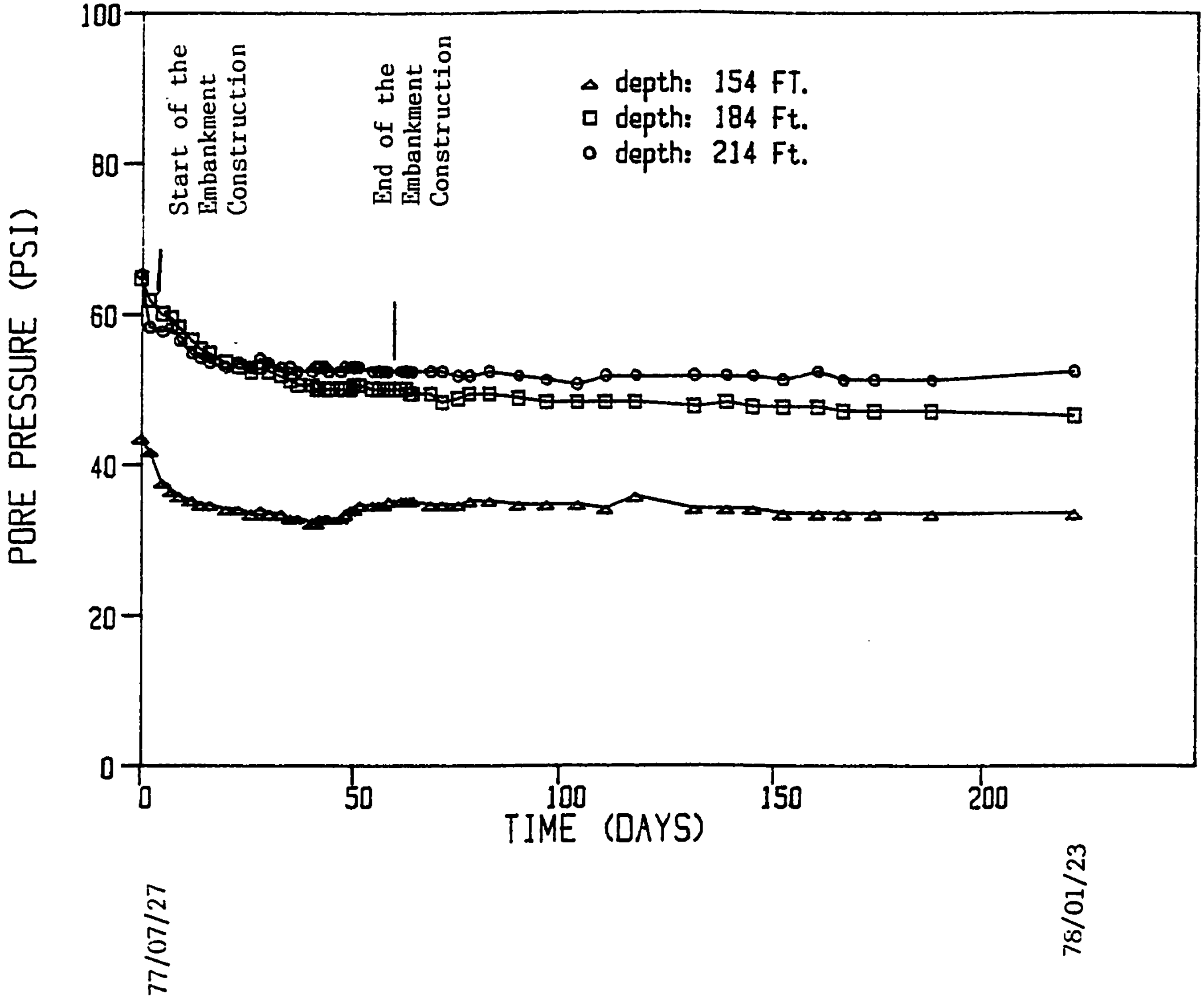

Fig. B4.9: Pore pressure versus time, Pile 8.

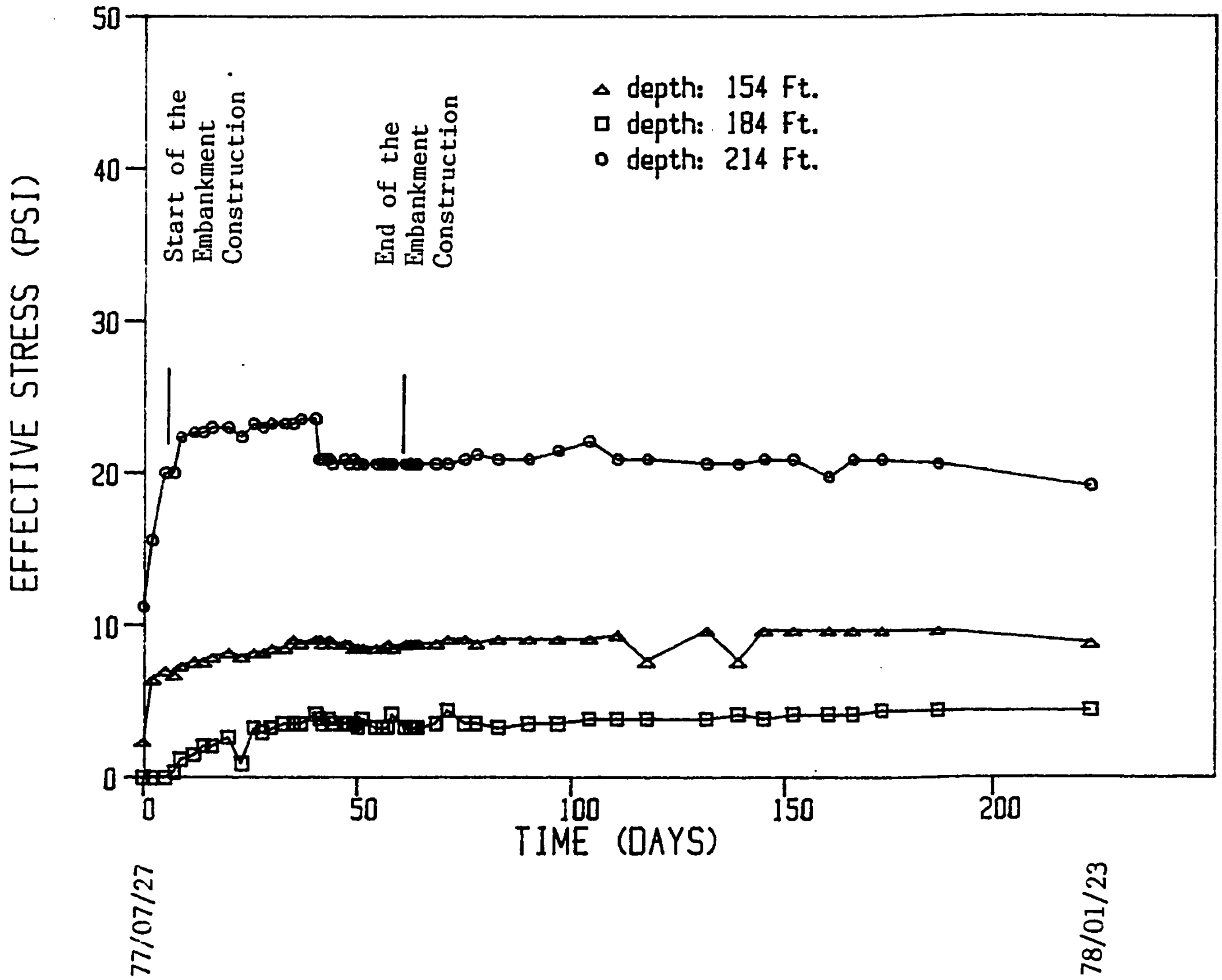

Fig. B4.10: Horizontal effective stress versus time, Pile 8.

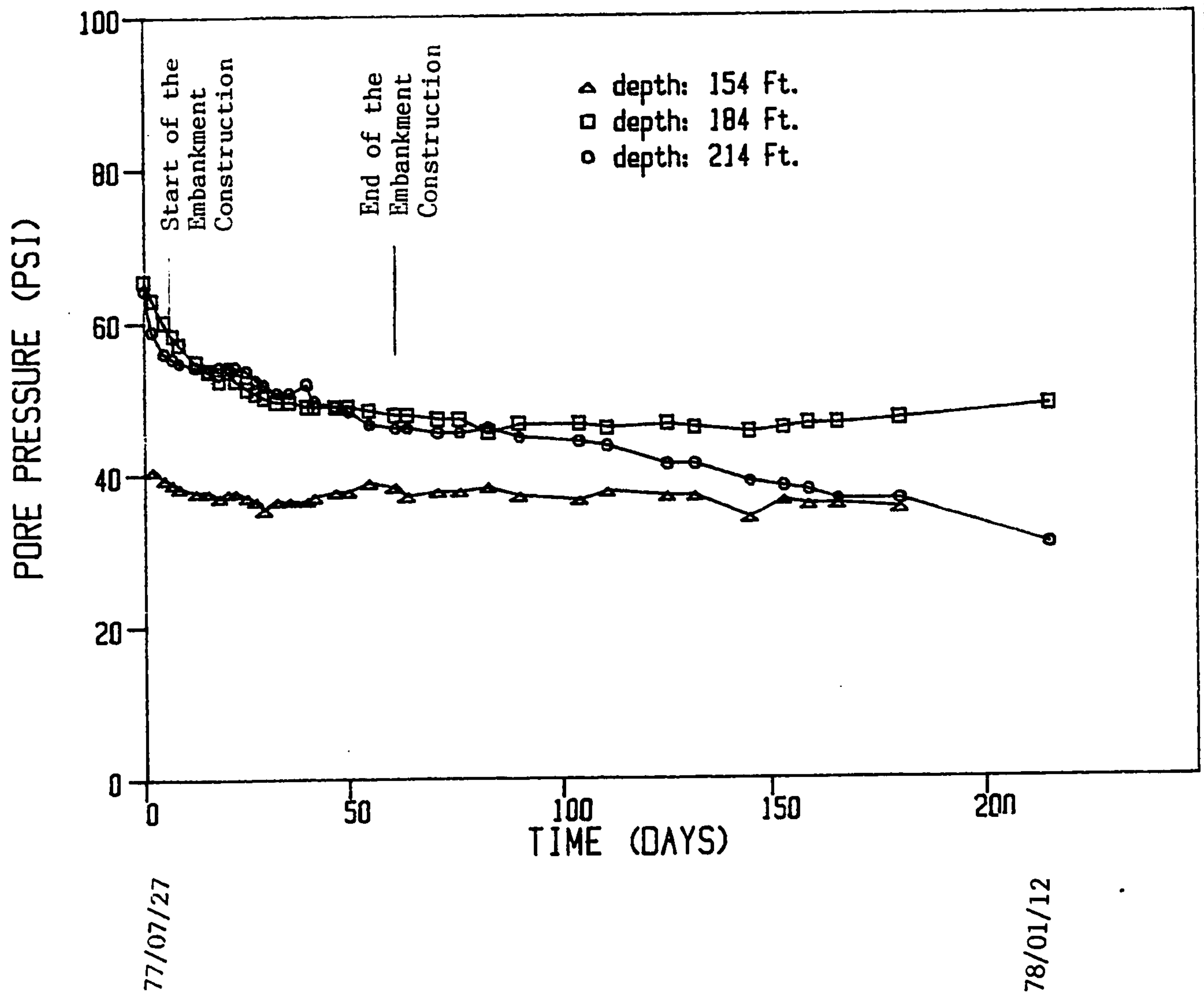

Fig. B4.11: Pore pressure versus time, Pile 9.

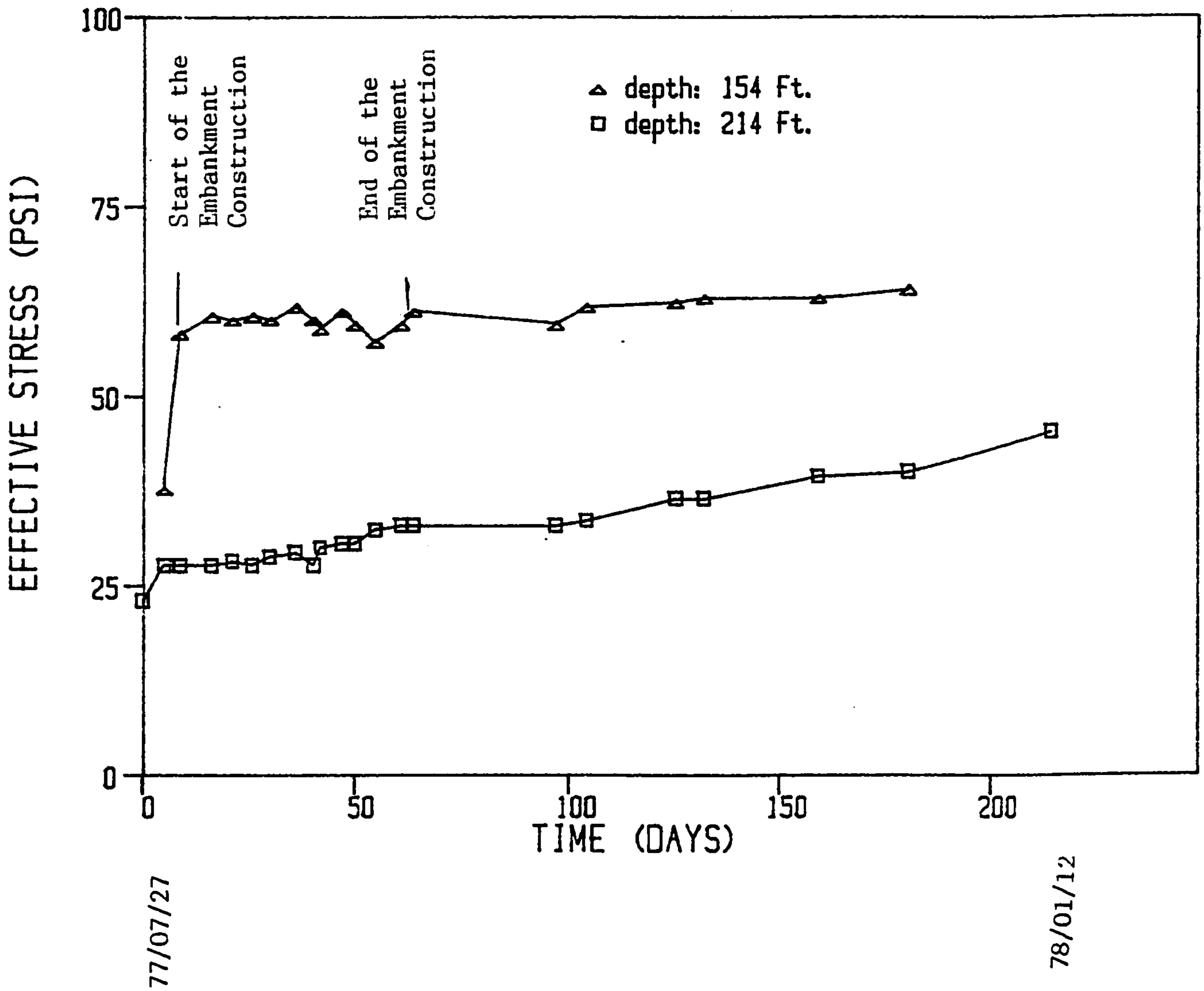

Fig. B4.12: Horizontal effective stress versus time, Pile 9.

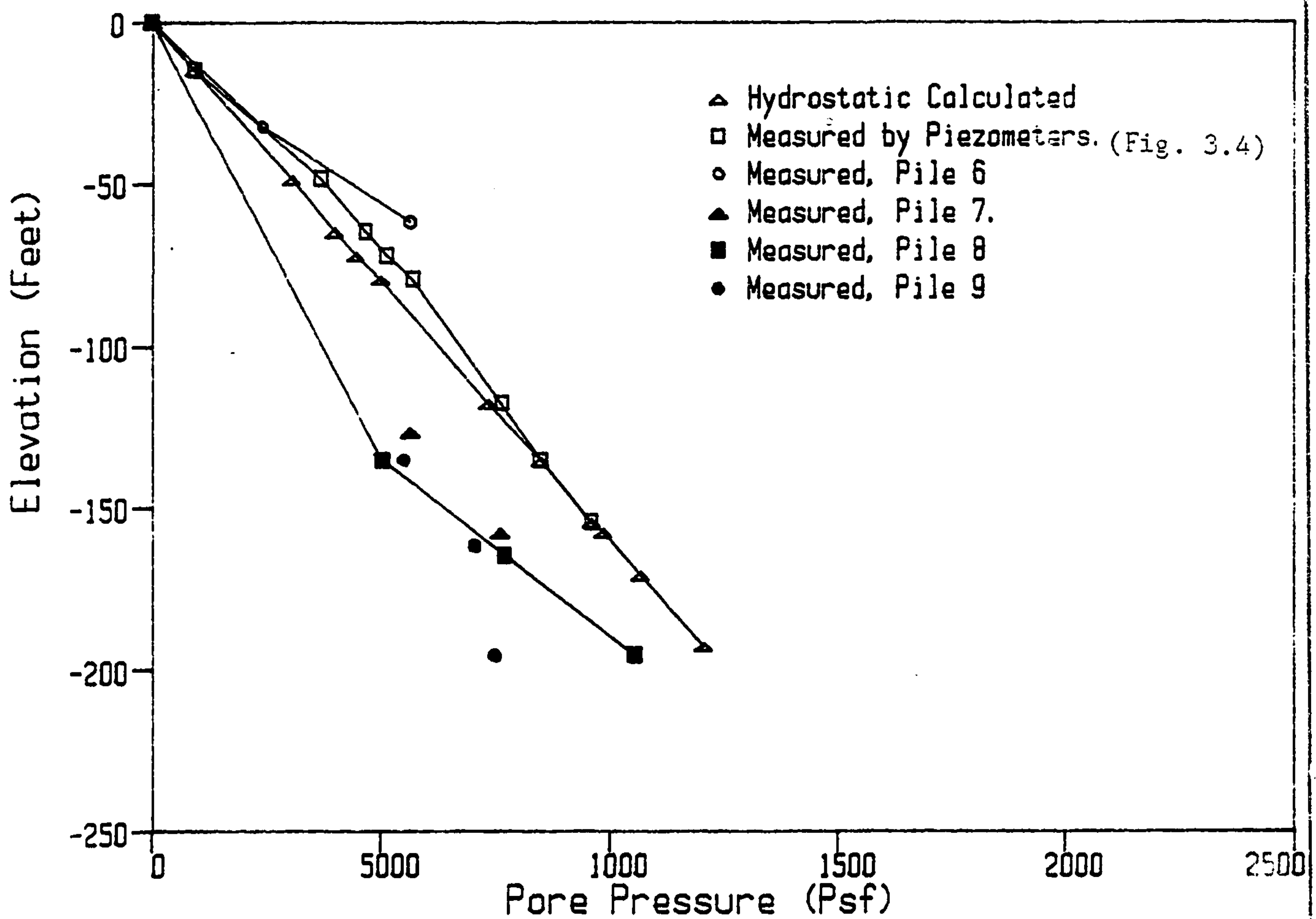

Fig. B4.13: Comparison between the measured and the calculated pore pressure in Piles 6, 7, 8, and 9.

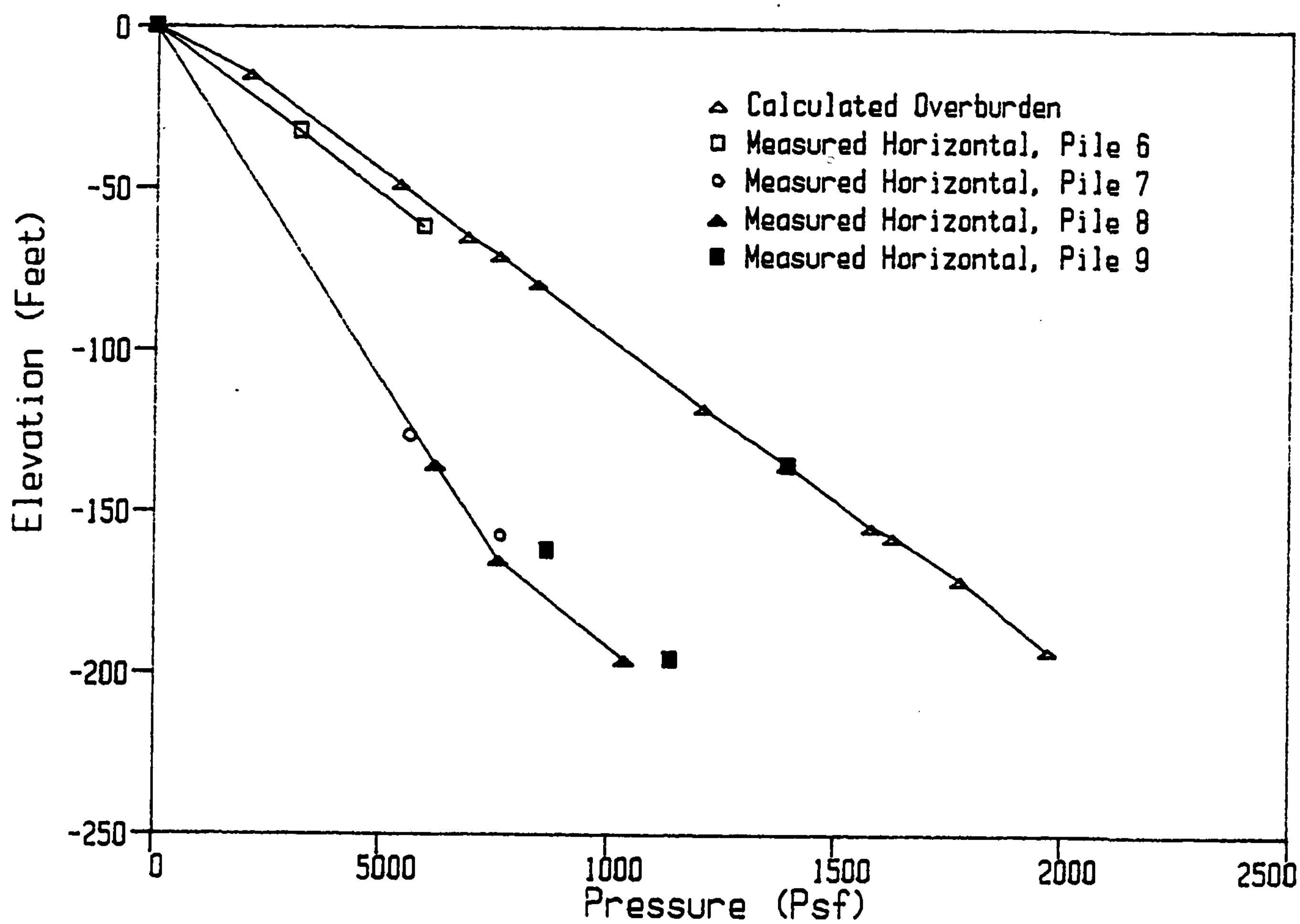

Fig. B4.14: Comparison between the measured total horizontal stress and the calculated total overburden stress in Piles 6, 7, 8, and 9.

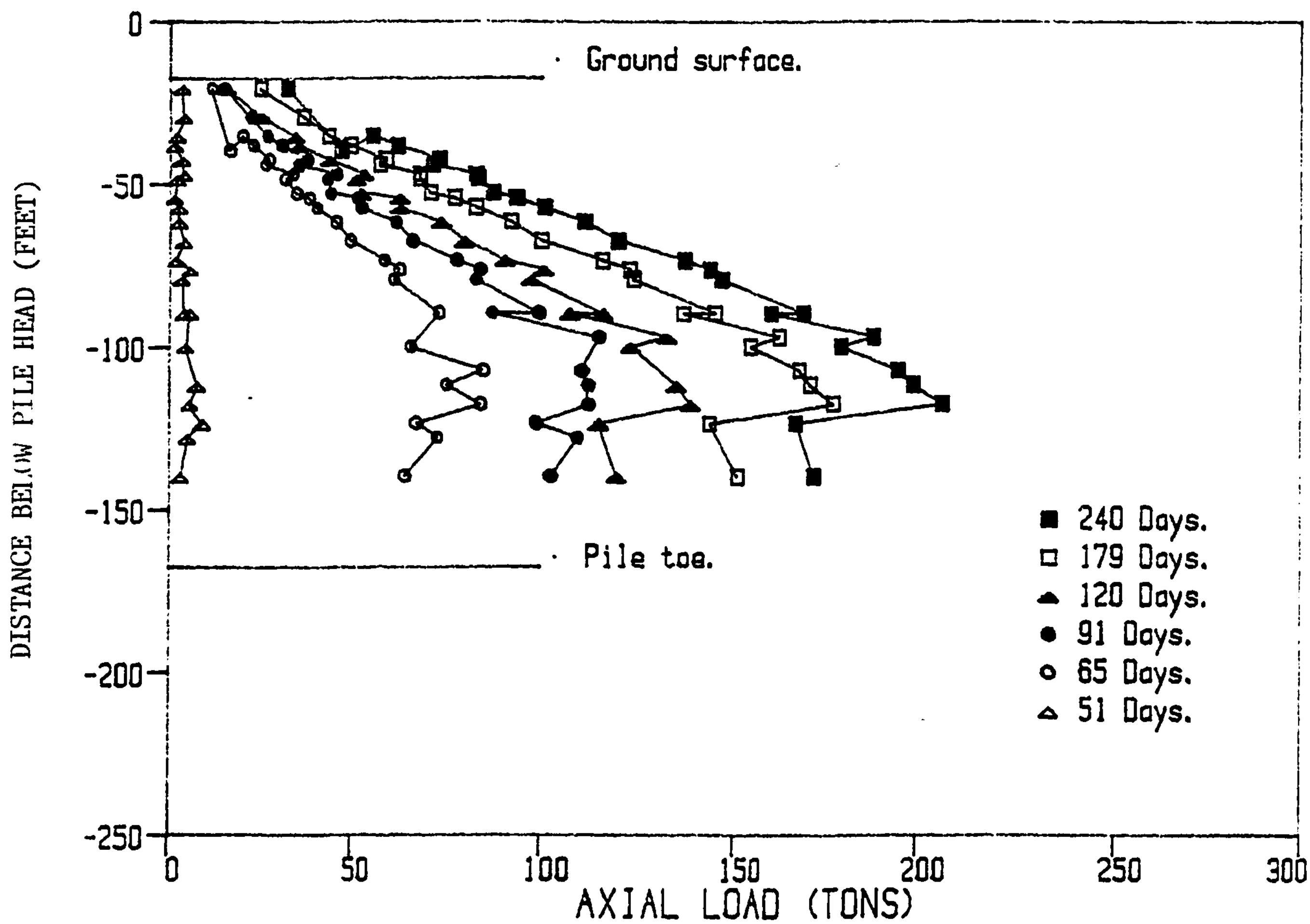

Fig. B5.1: Axial load distribution versus depth, Pile 6.

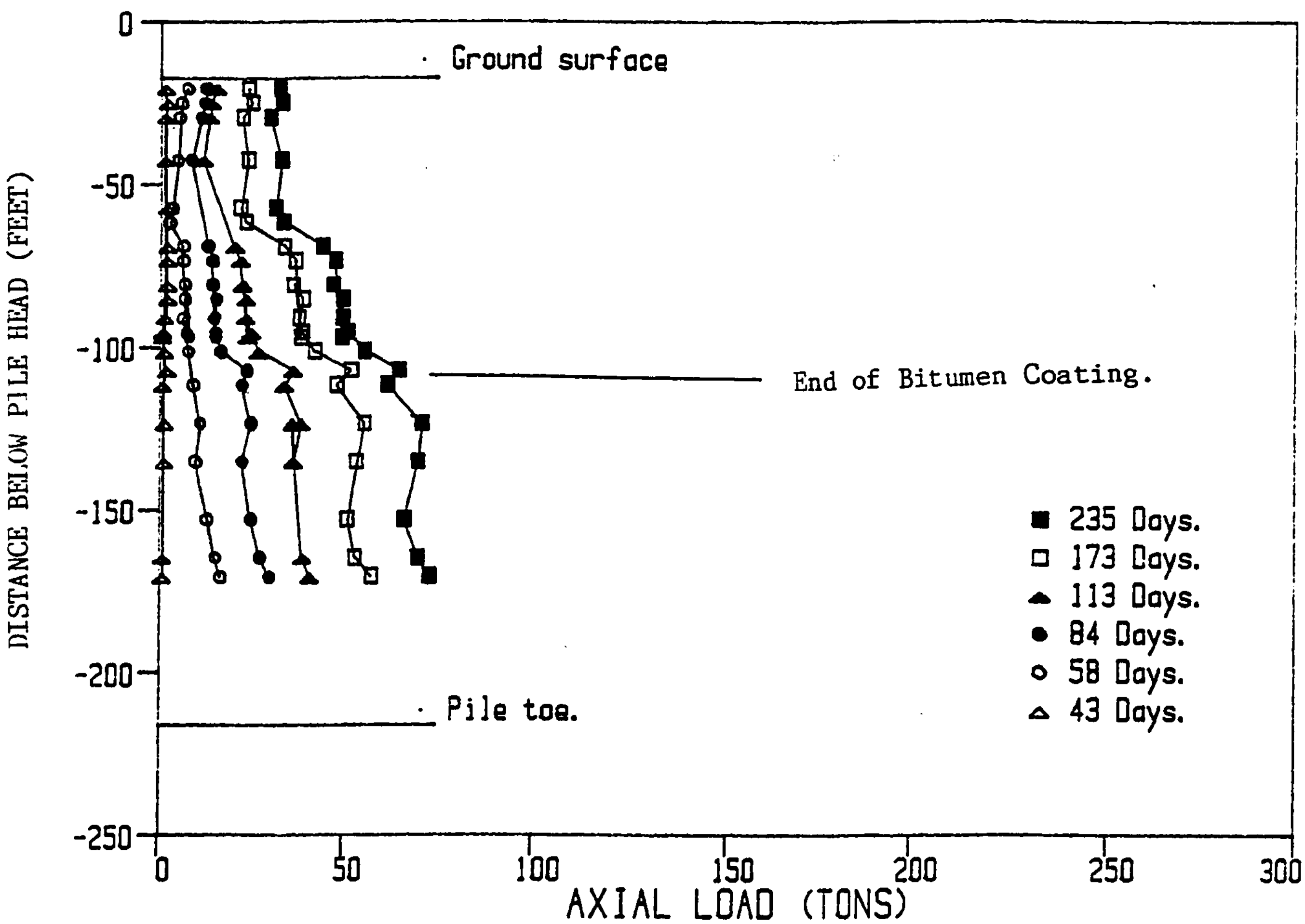

Fig. B5.2: Axial load distribution versus depth, Pile 7.

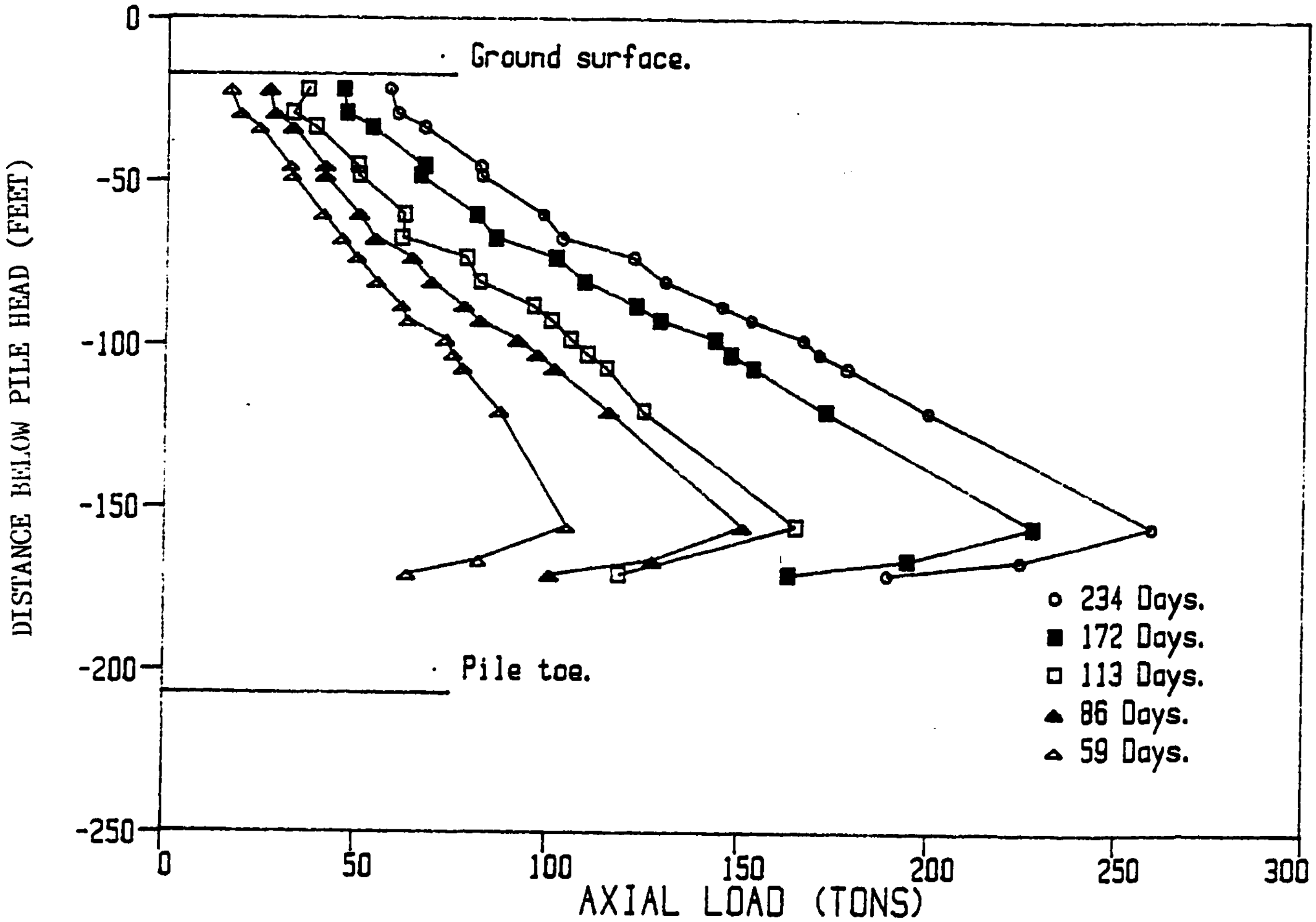

Fig. B5.3: Axial load distribution versus depth, Pile 8.

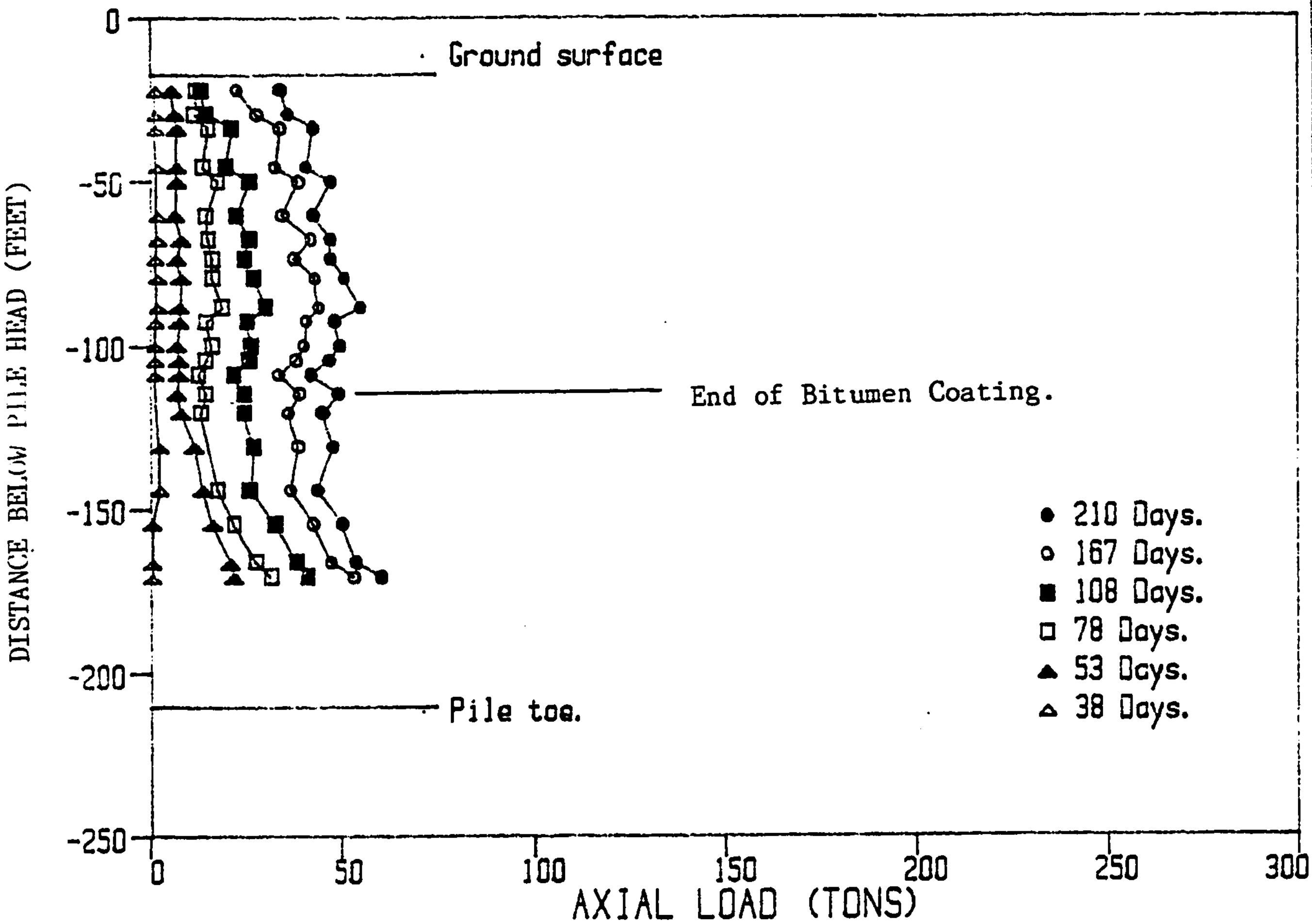

Fig. B5.4: Axial load distribution versus depth, Pile 9.