PREDICTING AXIALLY AND LATERALLY LOADED PILE BEHAVIOUR USING IN-SITU TESTING METHODS

by

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ABSTRACT

The prediction of axial and lateral pile behaviour is a complex engineering problem. Traditional methods of data collection and subsequent analyses are frequently in error when compared to full-scale load tests. In-situ testing, using advanced electronic tools, provides a means by which representative field data may be obtained. This study investigates the use of such in-situ data in predicting axially loaded pile capacity and laterally loaded pile load-deflection behaviour.

A total of twelve static axial pile capacity methods were evaluated to predict the results obtained from eight full-scale pile load tests on six different piles. These methods, separated into direct and indirect classes, used data obtained from the cone penetration test. Extensive use of commercially available microcomputer software significantly simplified the analyses. In addition, several dynamic pile capacity predictions are presented including results from in-situ dynamic measurements obtained with a pile driving analyzer during pile emplacement. An attempt has been made, with the use of tell-tales, to differentiate the shaft resistance and end-bearing components of the load test results. These results are then compared to the prediction methods investigated.

Two methods of predicting lateral load-deflection behaviour using in-situ data have been investigated. One method uses pressuremeter test data and the other, a new method proposed in this study, uses full-displacement flat plate dilatometer test data. These predictions are compared with full-scale lateral load tests on three piles of differing size.

In both the axial and lateral load cases, the preferred method(s) of analyses are identified. It is shown that excellent agreement can be obtained for predicting measured pile behaviour using several methods. The limitations of this study are noted, and recommendations for further research are proposed.

Advisors:

Dr. Peter K. Robertson

Dr. Richard G. Campanella

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CHAPTER 1

INTRODUCTION

1.1 Outline

In order that a piled foundation may be designed safely and economically, either an accurate prediction of its behaviour under load is made or a full-scale pile load test is performed. Full-scale load tests are very expensive and are therefore often impractical. Predictive methods require an accurate assessment of the soil properties into which the pile is to be placed. In-situ testing methods offer an excellent means by which to accurately obtain these soil properties.

In 1984, the British Columbia Ministry of Transportation and Highways (BCMOTH) performed pile testing, axial and lateral, on a 915 mm diameter pile as part of the design phase for the Alex Fraser Bridge project. The University of British Columbia (UBC) In-Situ Testing Group became involved in the evaluation of the testing data and the subsequent prediction of pile behaviour using in-situ testing methods (Robertson et al., 1985). Due in part to the encouraging results of the UBC predictions, the BCMOTH agreed to support a research program whereby several 324 mm diameter piles would be installed and tested both axially and laterally. This study is the result of that research program.

This thesis is organized in the following manner: Chapter 2 presents an overview of pile design and the role in-situ testing can play in providing more accurate data than most traditional methods. Chapter 3 introduces the research site used for this study. In Chapter 4, a description of the in-situ tests performed and of the data obtained is presented. Details of the installation and load testing of the piles investigated comprises

Chapter 5. Chapter 6 presents predicted versus measured axial pile capacity results using both static and dynamic predictive methods. In Chapter 7, the results of the lateral pile prediction methods investigated are compared to the measured test behaviour. Chapter 8 presents the recommended method(s) of predicting both axial and lateral pile behaviour from in-situ testing data. The thesis closes with a summary, conclusions, and recommendations for areas of further study.

1.2 Thesis Objectives

The major objectives of this study are listed as follows:

a) Perform and interpret several full-scale axial and lateral pile load tests

b) Compare the results of both the axial and lateral pile load tests to the predictions made from in-situ testing data

c) Propose and evaluate a method of determining lateral pile behaviour from flat plate dilatometer data

d) Recommend the preferred methods for predicting axial and lateral pile behaviour using in-situ testing data

CHAPTER 2

PILE DESIGN

The use of piles, dating back to prehistoric lake villages, is man's oldest method of overcoming the difficulties of inadequate earth materials (Poulos and Davis, 1980). Efforts have been reported in literature since the publication of "Piles and Pile Driving" edited by Wellington of the "Engineering News" in 1893. Since this time, pile design has progressed from being purely empirical to having an ever increasing theoretical basis.

Traditionally, pile design has meant predicting the ultimate axial load capacity of the given foundation and to assess whether tolerable settlements will be exceeded. This ultimate load is calculated either by "static" methods, which use empirical and theoretical bearing and shaft capacity formulae; or by "dynamic" methods, which use measured or modelled pile driving data. Pile settlement is generally predicted from empirical correlations (Peck et al., 1974). Extensive experience exists in the area of axial pile design as can readily be deduced by the large number of both technical papers written and analytical methods proposed. In addition to axial loads, however, piles are often required to resist lateral loads . The lateral behaviour of piles has not received nearly as much attention as the axial pile problem although since the mid-1970's this has been changing.

Vesić (1977) summarized the principal situations where piles may be needed (Fig. 2.1). The most common situation requiring a piled foundation is where the upper soil stratum is either too compressible and/or generally too weak to support the desired structure. In addition, piled foundations

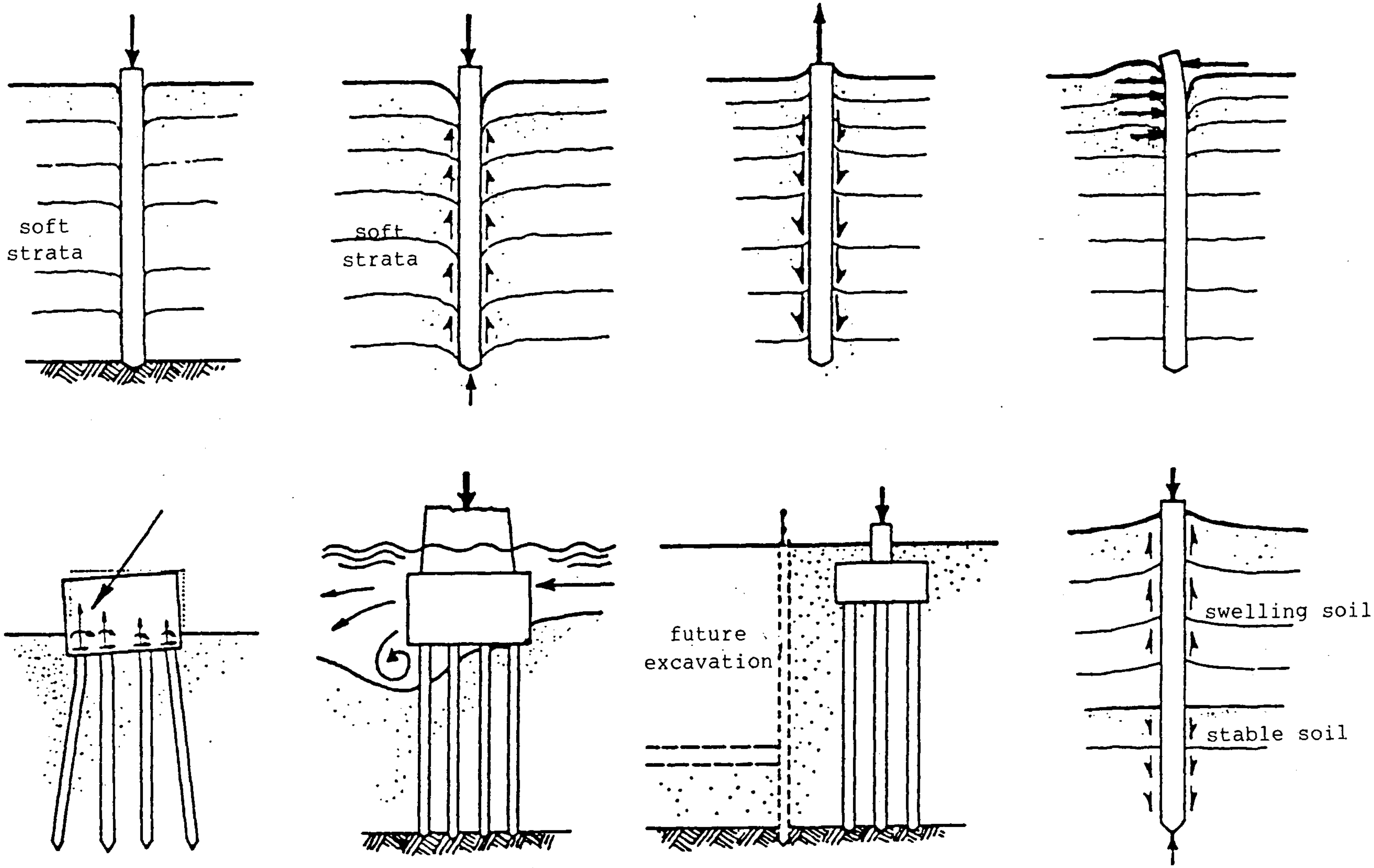


FIG. 2.1. SITUATIONS IN WHICH PILES MAY BE REQUIRED
(Adapted from Vesić, 1977)

are also frequently required because of the relative inability of shallow footings to transmit inclined, horizontal, or uplift forces and overturning moments (Vesić, 1977). Once it has been determined that a piled foundation is required, design of that foundation must reflect the selection of pile type. There are basically three main material pile types used (either separately or together to form composite materials). Table 2.1 lists the principal design advantages and disadvantages of each type. As well as pile type, the emplacement technique used to install the pile must be considered in the design. There are four main methods of pile installation:

i) Driven piles

ii) Bored or cast-in-place piles

iii) Driven and cast-in-place piles

iv) Screw piles.

In Fig. 2.2, an example of each of these methods is presented.

In this chapter, a brief review of methods of designing piles subject to both axial and lateral loads will be presented. For each loading case the general behaviour mechanism developed during the application of load will also be presented. In addition, a brief justification for the use of in-situ testing methods for axial and lateral pile design is included.

2.1 Axially Loaded Piles

2.1.1 Introduction

All piles, due to their own self-weight, impart an axial load on the soil even when isolated from any external forces. There are likely an infinite number of examples where vertical piles could be used to support structural loads. However, in each case, their use is generally for the

PILE TYPE	ADVANTAGES	DISADVANTAGES
Timber	Easy to handle or cut-off. Relatively inexpensive material Readily available (N.A.) Naturally tapered	Decay above water table Limited in size and bearing capacity Prone to damage by hard driving Difficult to extend Noisy to drive
Steel	Easy to handle, cut off, extend Available in any size Can penetrate hard strata Convenient to combine with steel superstructure	Subject to corrosion Flexible H-piles may deviate from axis of driving Relatively expensive Noisy to drive
Concrete: Precast	Durability in almost any environment Convenient to combine with concrete superstructure	Cumbersome to handle and drive Difficult to cut off or extend Noisy to drive
Concrete: Cast-in-place i) casing left in ground	Allows inspection before concreting Easy to cut off or extend	Casing cannot be re-used Thin casing may be damaged by impact or soil pressure
ii) casing withdrawn or no casing	No storage space required Can be finished at any elevation Can be made before excavation Some types allow larger displacements in weaker soils	In soft soils shaft may be damaged by squeezing In case of heavy compaction of concrete, previously completed piles may be damaged If concrete is placed to fast there is danger of creation of a void

TABLE 2.1. PRINCIPAL ADVANTAGES AND DISADVANTAGES OF DIFFERENT PILE TYPES (Adapted from Vesic, 1977)

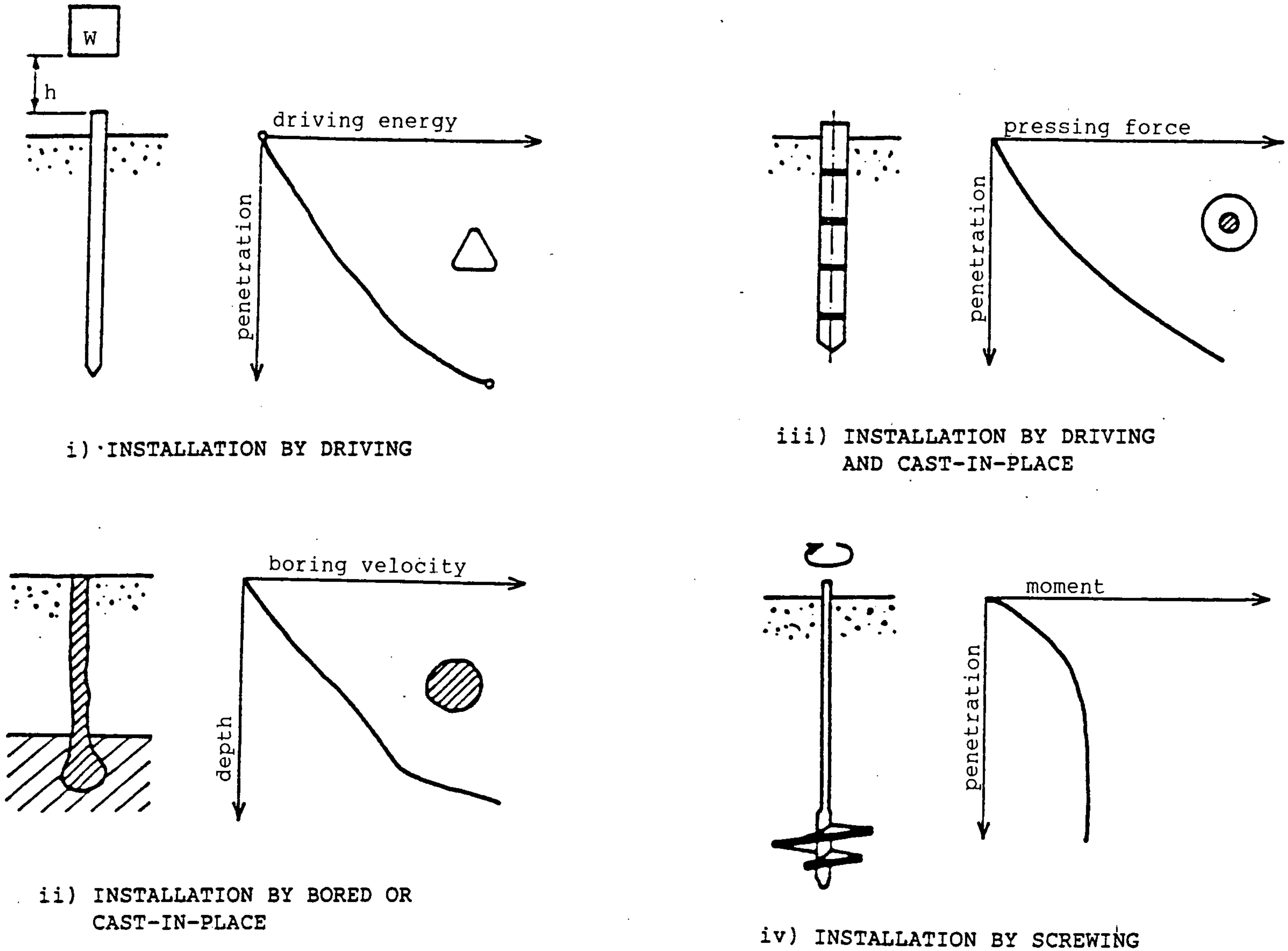


FIG. 2.2. METHODS OF INSTALLING PILES
(Adapted from Kézdi, 1975)

same reason; to transfer the structural loads to more competent and/or less compressible earth material(s).

In designing axially loaded piles the following three criteria must be considered, structural failure of the pile, bearing capacity failure of the soil, settlement of the piled foundation. Excluding buckling and bending due to lateral loads and failure due to excessive energy input during pile driving, structural failure is assumed to occur when the stress in the foundation equals the critical stress for the shaft material (e.g., the yield stress for steel pipe piles). Structural failure is seldom a concern unless very dense soil or rock is encountered. In many cases it is the bearing capacity of the soil or the settlement which determines the maximum foundation load. For predicting axial pile capacity both static and dynamic capacity predictions are available.

2.1.2 Static Capacity Prediction Methods

For this study, only the prediction of axial capacity of driven piles will be addressed. The problem of estimating the settlement of axially loaded piles will not be addressed. Brief descriptions of possible failure mechanisms under axial loading and the prediction of axial capacity are presented in this section.

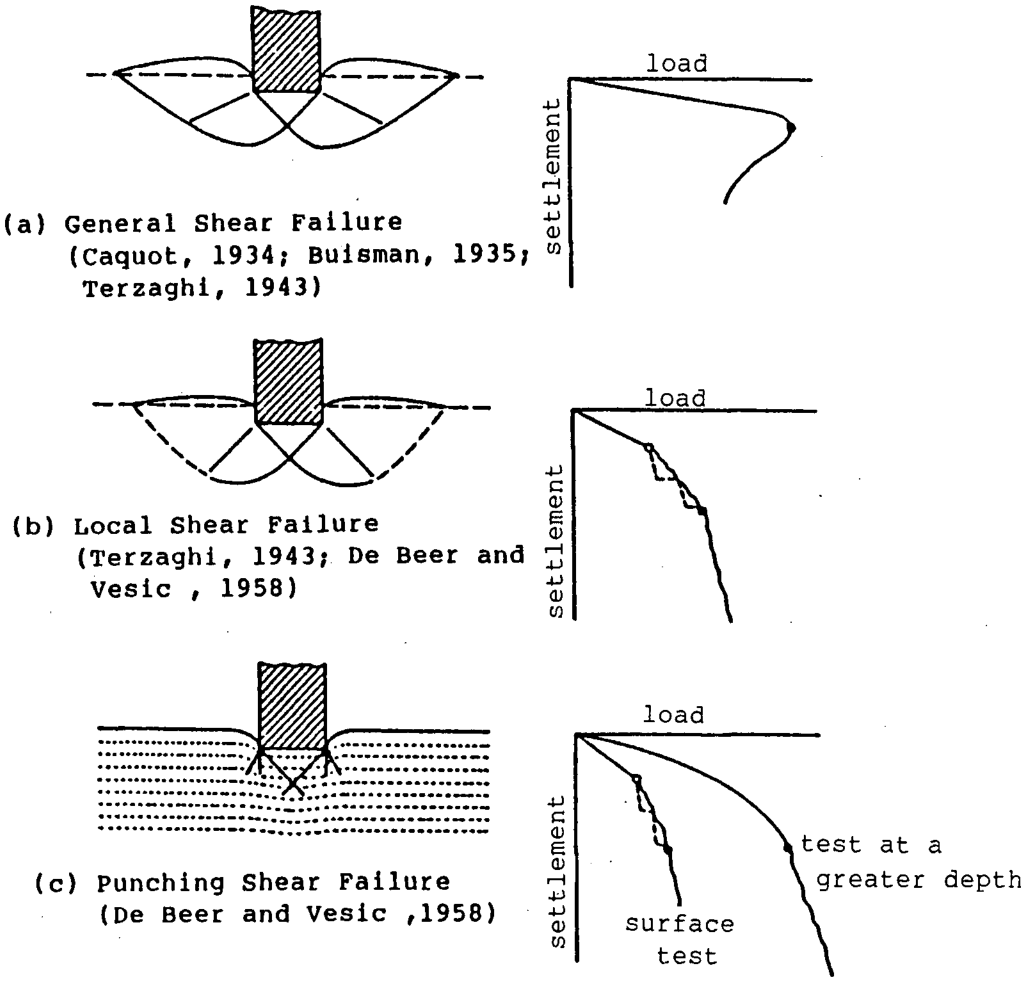
2.1.2.1 Failure Mechanisms

In order to evaluate any bearing capacity prediction method, whether theoretical or empirical, it is often useful to review whether or not the failure mechanism used in its formulation is representative of the in-situ conditions. The mode of failure depends mainly on; the shear strength of the surrounding soil, the length to diameter ratio of the pile and the pile type (Kézdi, 1975).

It is often assumed that bearing capacity failure occurs as a shear failure in the soil supporting the foundation structure. Three principal modes of shear failure were recognized by Vesić (1963). These failure modes are shown in Fig. 2.3. General shear failure (Fig. 2.3a) is characterized by the existence of some well-defined failure pattern consisting of a continuous slip surface from one edge of the foundation to the ground surface. Local shear failure (Fig. 2.3b) is characterized by a failure pattern defined only beneath the foundation level. A punching shear failure (Fig. 2.3c) is less well-defined and is often difficult to observe. Unlike the general and local shear failure modes, the punching shear failure involves practically no movement of the soil toward the free surface. The punching shear failure generally fits the observed soil behaviour around most piles during driving (Vesić, 1977).

Vesić (1963) conducted extensive laboratory studies in granular soils of variable density to define the various failure mechanisms. These mechanisms are also present in cohesive soils, but are more readily observable in cohesionless soils. Vesić's work is summarized graphically in Fig. 2.4. In Fig. 2.4, D = depth of foundation and b = pile width. It is important to note that the limits of failure zone depend upon material compressibility (Vesić, 1963). More compressible materials will tend to have small D/b ratios to generate a punching shear failure.

It is interesting to note from Fig. 2.4 that for circular foundations (i.e. most piles), a punching failure will occur below a relative depth of 4. Fig. 2.5 presents some of the existing proposed failure patterns for pile foundations. It can be seen that most of the proposed failure



(a) General Shear Failure (Caquot, 1934; Buisman, 1935; Terzaghi, 1943)

(b) Local Shear Failure (Terzaghi, 1943; De Beer and Vesic , 1958)

(c) Punching Shear Failure (De Beer and Vesic ,1958)

FIG 2.3. TYPES OF FAILURE MECHANISMS
(Adapted from Vesić , 1963)

I - General Shear Failure

II - Local Shear Failure

III- Punching Shear Failure

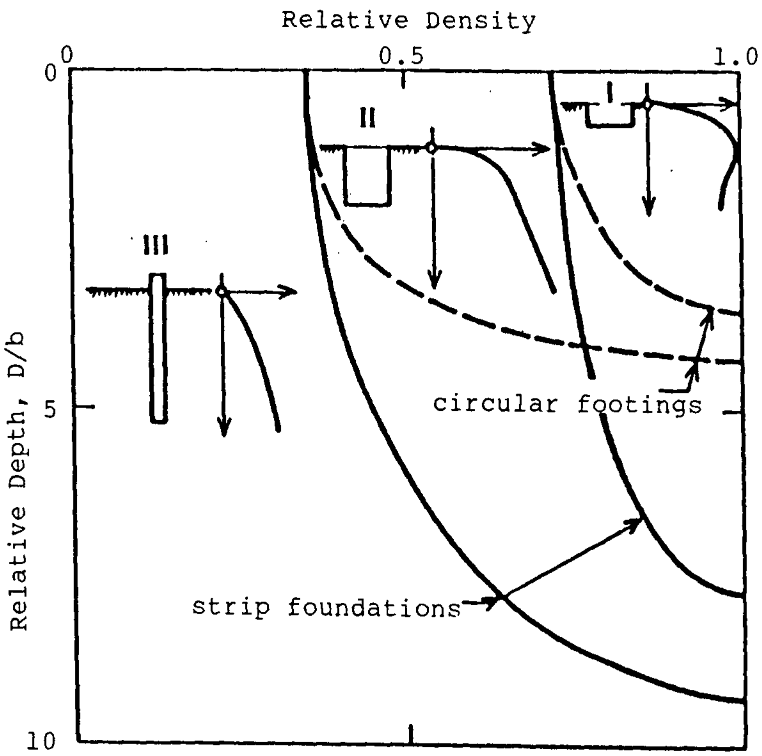
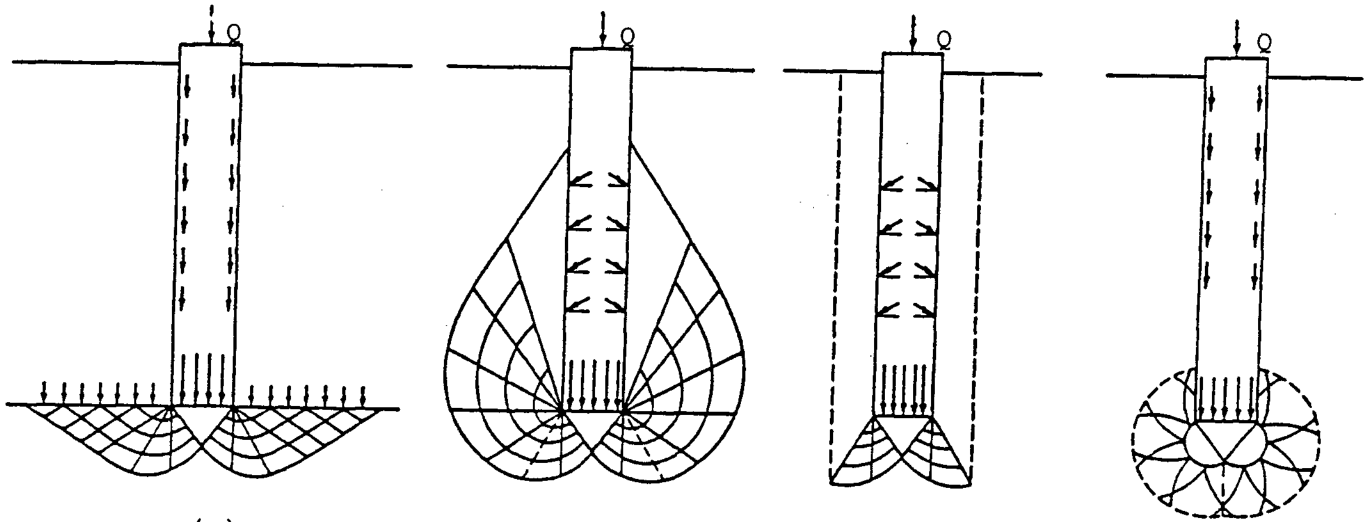


FIG. 2.4. FIELDS FOR DIFFERENT TYPES OF FAILURE FOR SHALLOW AND DEEP FOUNDATIONS (Adapated from Kézdi, 1975)



(a)
Prantl
Reissner
Caquot
Buisman
Terzaghi

(b)
DeBeer
Jaky
Meyerhof

(c)
Bereznantsev
and Yaroshenko
Vesic

(d)
Bishop, Hill, and
Mott
Skempton

FIG. 2.5. **ASSUMED FAILURE MECHANISMS UNDER PILE FOUNDATIONS**
(Adapted from Vesić, 1967)

patterns model either the general shear failure or the local shear failure conditions. Fig. 2.6 shows how much variability results in the derived bearing capacity factor, N_q, due to the use of these different failure mechanisms. For frictional soils the following formula is commonly accepted for the pile point resistance, Q_p:

$$Q_p = A_p \, (\gamma \cdot d \cdot N_q) \tag{2.1}$$

where: A_p = area of pile tip

γ = total unit weight of soil

d = depth of tip embedment

It is therefore distressing that Fig. 2.6 shows a variability in N_q that is in excess of one order of magnitude. Independent studies by Norlund (1963) and Vesić (1967) show that the values of N_q proposed by Berezantsev correlate most closely with measured point resistance at failure. It is worth noting that the assumed failure mechanism proposed by Berezantsev (Fig. 2.5) most closely resembles the description of punching shear failure described earlier.

For cohesive soils, the value of N_q is not important but another bearing capacity factor, N_c, is commonly used to give the following formula for pile point resistance, Q_p:

$$Q_p = A_p \, (S_u \cdot N_c + \gamma \cdot d) \tag{2.2}$$

where: S_u = undrained shear strength.

Although the value of N_c doesn't vary as much as N_q, Ladanyi (1967) shows

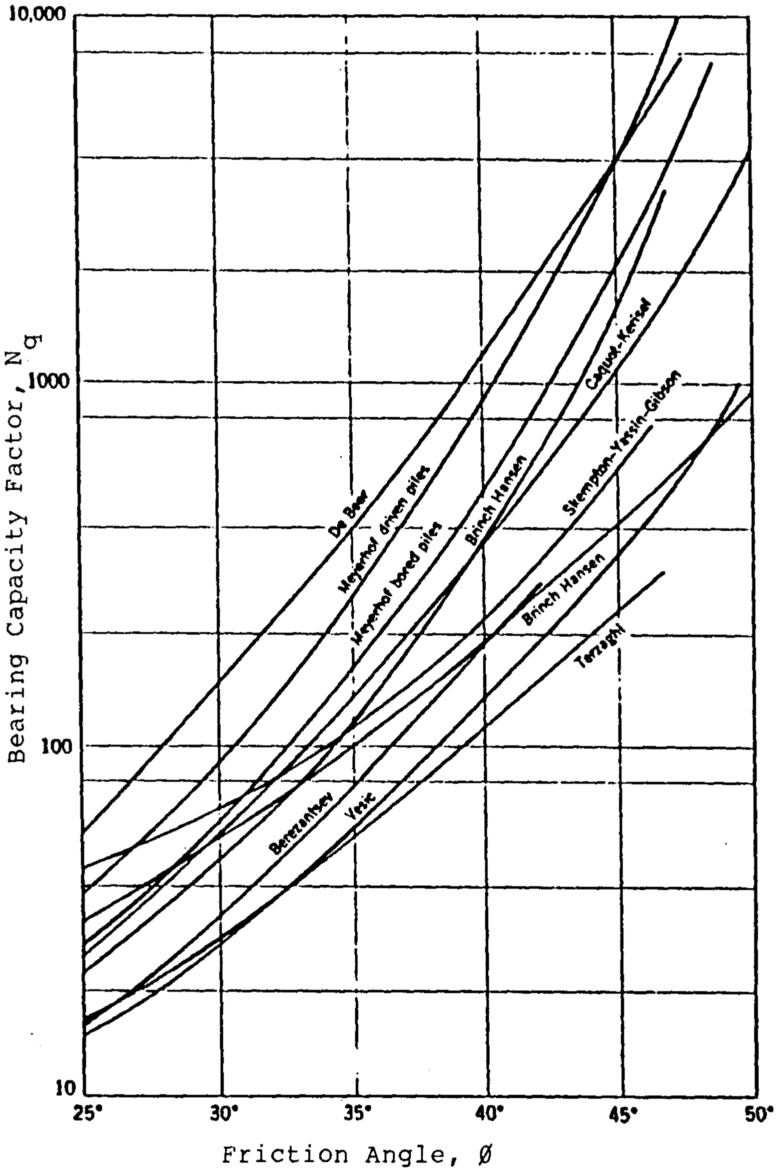


FIG. 2.6. BEARING CAPACITY FACTORS FOR DEEP CIRCULAR FOUNDATIONS (Adapted from Vesić, 1967)

that N_c can vary over a significant range depending on the stress-strain properties of the soil.

2.1.2.2 Prediction Methods

Despite the amount of attention the subject has received, the problem of predicting the axial load carrying capacity of driven piles still challenges engineers.

Static prediction methods are based upon evaluating the properties of the soil into which the pile is to be or has been driven. This is usually done by considering the shaft (or side) resistance and end bearing as independent components of the total pile resistance.

The shaft resistance in cohesive soils is usually estimated using an approach similar to the one proposed by Tomlinson (1957). This method estimates the unit shaft resistance (f_s) as being equal to the undrained shear strength of the soil reduced by a factor dependent on the magnitude of the undrained shear strength in the form:

$$f_s = \alpha \cdot S_u \tag{2.3}$$

where: f_s = unit shaft resistance

S_u = undrained shear strength

α = adhesion coefficient

= func^n (S_u)

The adhesion coefficient, α, is an empirical quantity first proposed by Tomlinson (1957) to correlate the undrained pile cohesion with the undrained shear strength. One problem with the approach in Eq. 2.3 is that the value of undrained strength used will be highly dependent upon the method by which it was obtained. Another problem is that it seems inconsistent to use an undrained strength to predict the drained frictional resistance of a pile. The shaft resistance in cohesionless soils is often estimated using an equation of the following form (Meyerhof, 1976):

$$f_s = K \cdot \sigma'_v \cdot \tan \delta \tag{2.4}$$

where: K = coefficient of lateral earth pressure

σ'_v = average effective vertical stress

δ = friction angle between soil and pile

One problem with this approach is that the value of K is often difficult to select. Investigators have reported values of K ranging from 0.3 to 3.0 (Lambe and Whitman, 1969). Another problem is that Eq. 2.4 suggests that shaft resistance increases linearly with depth. Difficulty also exists in estimating δ.

The end bearing capacity of a driven pile is most commonly predicted using the Buisman-Terzaghi equation which has the form:

$$q_{ult} = c \cdot N_c + \frac{1}{2} B \cdot \gamma \cdot N_\gamma + \gamma \cdot d \cdot N_q \tag{2.5}$$

where: q_{ult} = ultimate unit tip bearing capacity

c = soil cohesion

N_c, N_γ, N_q = bearing capacity factors

γ = unit weight of soil at pile tip

d = depth of pile tip

B = pile width

For cohesionless soils, Eq. 2.5 reduces to:

$$q_{ult} = \gamma \cdot d \cdot N_q \tag{2.6}$$

since c=0 and N_γ is negligible in most cases.

For cohesive soils Eq. 2.5 is usually reduced to:

$$q_{ult} = c \cdot N_c + \gamma \cdot d \cdot N_q \tag{2.7}$$

Note that for cohesive soils N_q=1. The major drawback with using Eq. 2.5, and its reduced forms, is that the Buisman-Terzaghi equation is a general solution for the general shear mode of failure. As was shown in the preceding section, it is the punching shear failure mechanism that appears to govern most pile foundations. As well, the Buisman-Terzaghi equation is not a rigorous solution; it is a superposition of solutions (e.g. Prandtl and Reissner solutions) which leads to an intentionally conservative result. In cohesionless soils another problem that exists is that a value of N_q must be obtained. As was shown in the preceding section, there is a wide variation of opinion concerning the actual form of the ϕ-N_q relationship (ϕ = angle of internal soil friction). As well, an accurate determination of ϕ is often difficult. For cohesive soils the problems are generally less severe, since the value of N_c is known with more confidence than the value of N_q. However, the contribution of end bearing to total resistance in cohesive soils is usually small, especially for long piles, and therefore an accurate prediction of end bearing doesn't improve the accuracy of the total resistance prediction considerably.

Considering the above, it is difficult to understand why these traditional prediction methods are still commonly used. Nottingham (1975) suggests three reasons as to why this is the case:

1. Dynamic prediction methods often do not provide any better results and the predictions are not available until the pile is driven.

2. It is often difficult to justify the cost of a pile load testing program on small projects.

3. Even when pile load testing can be justified, it is desirable to evaluate the probable performance of different pile types, sizes, and lengths during the design stage of a project in order to intelligently plan the field testing program.

In-situ testing, in particular the cone penetration test (CPT), offers an alternative solution to the pile capacity prediction problem. Determination of pile capacity from the CPT was one of the earliest applications of the cone test. The CPT can be thought of as an "in-situ model" of a driven displacement pile. CPT soundings provide a nearly continuous record of cone bearing and sleeve friction data allowing nearly continuous pile resistance profiles to be developed. Laboratory testing and the need for evaluating intermediate values (K, N_q, etc.) are generally eliminated using the CPT "directly" to predict axial pile capacity. The available "direct" methods are empirical and rely upon an accurate assessment of the effects due to the size differential between the cone penetrometer and the pile. The major effects between the CPT and a pile are scale effects, installation effects, and material effects. The study of these effects began with the original work at the Delft Laboratories in Holland by Van

Mierlo and Koppejan (1952). Scaling CPT data to predict pile capacity is now usually done using the method by Begemann (1965) or some variation of his method. An elaboration of scaling CPT data to predict pile capacity is presented in Chapter 6. Other in-situ tests, most notably the pressuremeter (PMT) and the standard penetration test (SPT), -can also be used to predict axial pile capacity. This study, however, only evaluated the use of the cone penetrometer for predicting axial pile capacity.

2.1.3 Dynamic Capacity Prediction Methods

Pile capacity can be determined by dynamic methods using two techniques. The first is a prediction, the second an in-situ test (Rausche et al., 1984).

Prediction methods require that an accurate static soil analysis be performed and that the effects of pile driving on the soil are estimated. Predictions may be done by either dynamic formulae or by the wave equation.

Dynamic formulae have been used for over 100 years by engineers. An astonishing amount of effort and ingenuity had been expended prior to the 1960's in developing pile driving formulas (Smith, 1960). Smith (1960) reports that by 1959 the editors of "Engineering News Record" had on file 450 such formulas. These original formulae all had the same form:

$$W_H \cdot H = Q_{dynamic} \cdot [(\text{Set}) - (\text{Energy Losses})] \qquad (2.8)$$

where: W_H = hammer weight

H = hammer drop height

$Q_{dynamic}$ = dynamic capacity.

These formulae considered the pile as a rigid mass experiencing motion caused by Newtonian impact of a mass. The energy delivered per blow, $W_H \cdot H$, can be equated with the sum of energy spent in displacing the pile over a distance (set) against the soil resistance ($Q_{dynamic}$) and the energy lost in elastic rebound and plastic deformations. These formulae, although widely used, rarely supply consistently accurate results as they fail to model the true nature of dynamic stress impact on hammer-pile impact.

In 1950 E.A.L. Smith proposed a numerical solution which could be used to solve extremely complex pile-driving problems. Smith (1960) carried this another step and applied his numerical solution to wave theory; the initial use of the wave equation in pile design. Today, wave equation analyses can be performed using commercially available programs and entering the appropriate values that represent the soil, hammer system and pile system. Fig. 2.7 shows a schematic representation of the wave equation model. The most common commercially available programs for performing wave equation analysis of piles are either the TTI (Texas Transportation Institute) series or the WEAP (Wave Equation Analysis of Piles) series.

The in-situ dynamic pile tests require measurements of the response of a pile to a hammer blow. The most basic of these measurements is the permanent set (permanent pile penetration for a given hammer strike) or blow count. Interpretation is then made by using either dynamic formulae or a wave equation analysis. In-situ pile tests may also be used in a more sophisticated manner by using the measurements of force and motion of the pile near its top during driving. Calculation of pile capacity from these measurements may be accomplished by a simple formulae (e.g., Case method), or by numerical analysis (e.g., CAPWAP). The Case method is a name that refers to the methods developed at the Case Institute of Technology in the

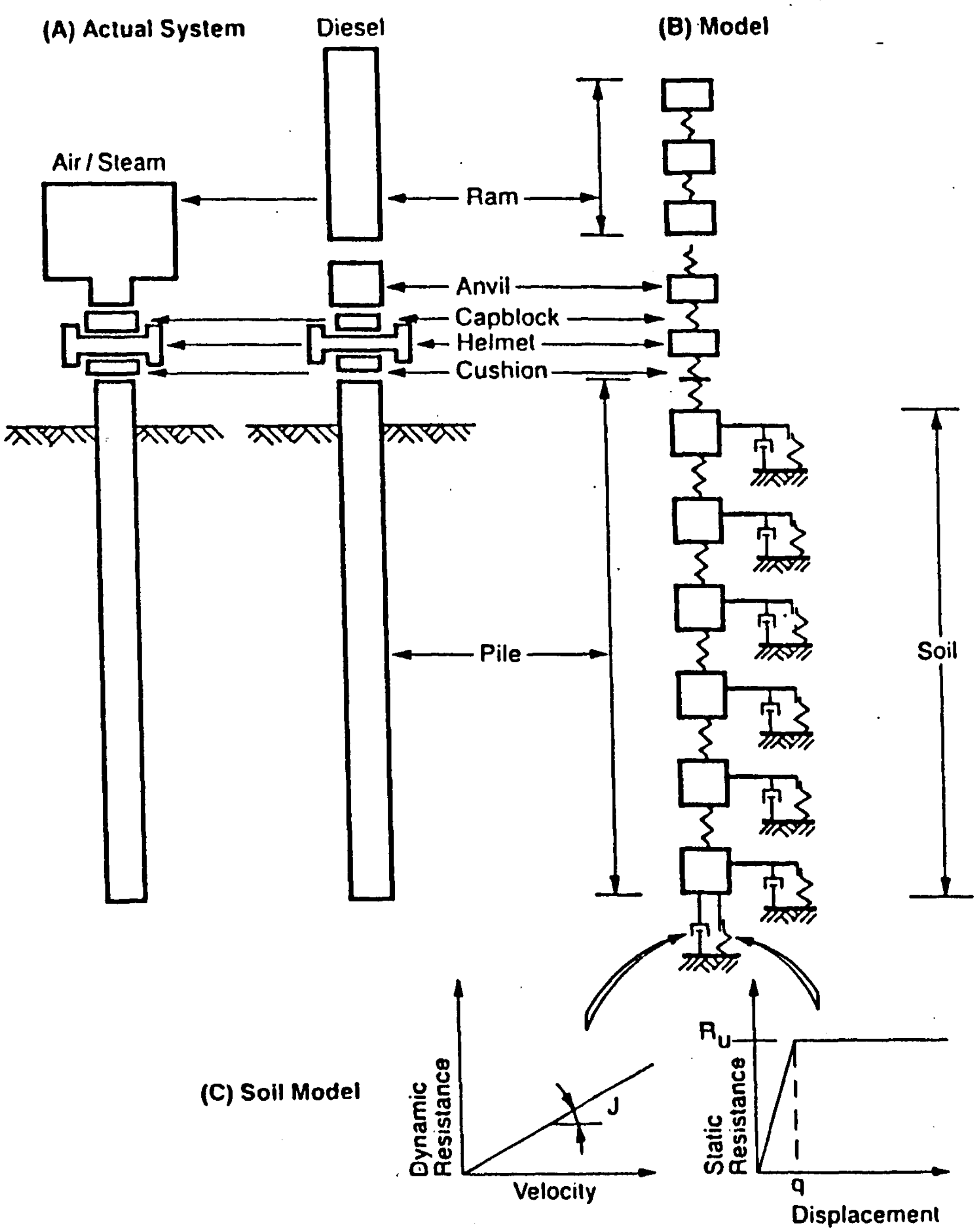


FIG. 2.7. SCHEMATIC REPRESENTATION OF DRIVING SYSTEM FOR WAVE EQUATION MODEL

last 1960's. An excellent summary of the Case Method is given by Gravare et al. (1980). CAPWAP (CAse Pile Wave Analysis Program) was initially developed by Rausche (1970). The CAPWAP analysis uses the same mathematical model of the pile and the soil as is used in the wave equation programs. However, with CAPWAP the model does not include the hammer and driving system, but only that portion of the pile below the measuring gauges. These gauges are used to measure forces and accelerations in the pile (see Fig. 2.8).

Fig. 2.9 presents a summary of the various techniques of predicting pile behaviour using dynamics. Even with the amount of attention pile dynamics has received, however, reliable results are often not realized when comparisons with static load tests are made. This is mainly because the dynamic capacity is seldom equal to the static capacity due to differences in soil strength or resistance. Disregarding this problem a severe limitation of in-situ dynamic methods is that the pile must be driven before a load capacity prediction can be made.

2.2 Laterally Loaded Piles

2.2.1 Introduction

Piles generally tend to be rather slender structural elements, usually vertical or only slightly inclined, and therefore they generally cannot carry high loads which act perpendicularly to their axis. Thus, it is usually not economical to use vertical piles where primarily lateral loads act; batter piles, tiebacks, deadmen or thrust surfaces are preferred. However, piles are primarily used for supporting vertical loads and are therefore placed vertically. This is because, among other reasons, the axial pile capacity decreases markedly due to load inclination (Meyerhof

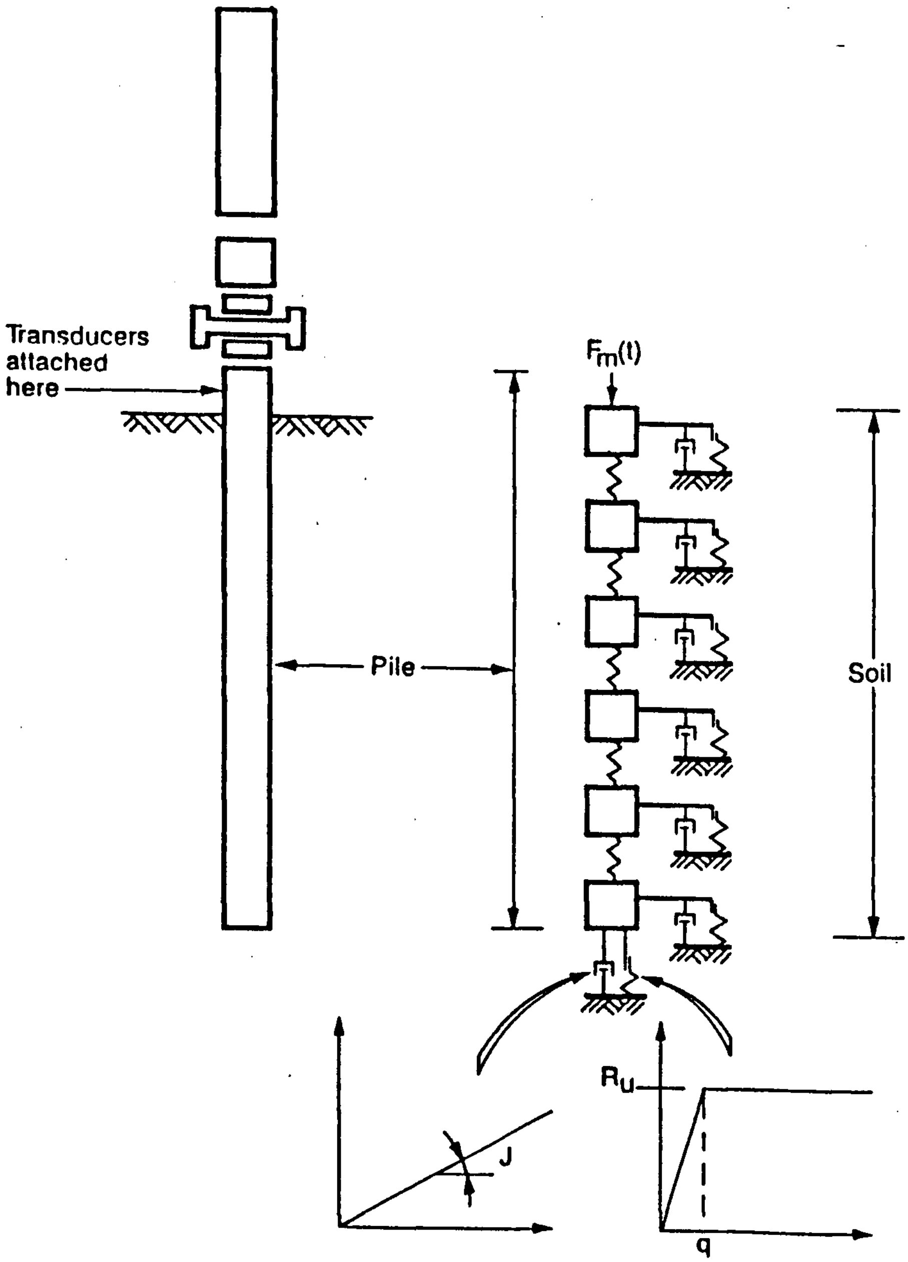


FIG. 2.8. SCHEMATIC REPRESENTATION OF CAPWAP MODEL

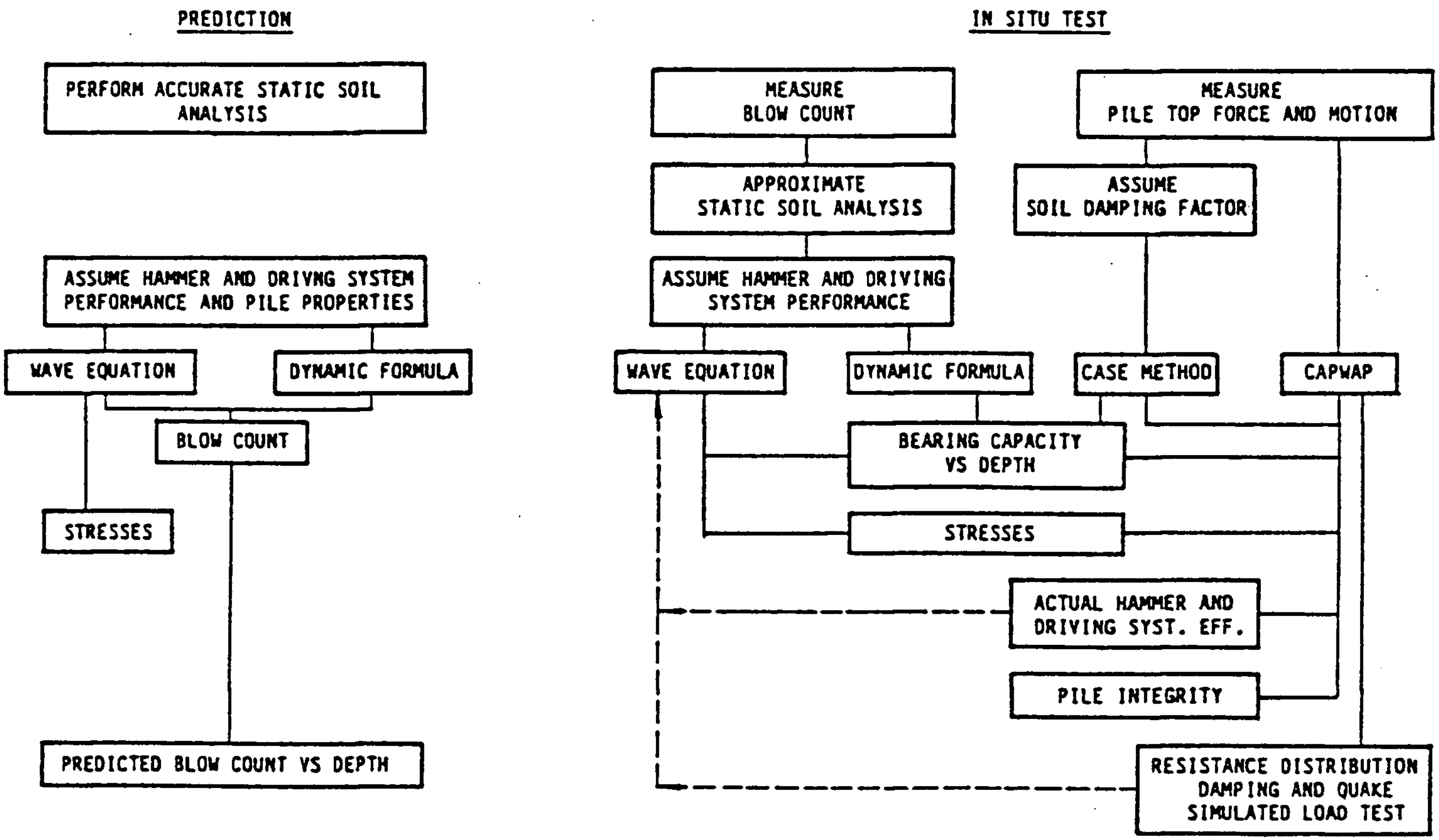


FIG. 2.9. DYNAMIC PILE ANALYSIS: METHODS AND RESULTS

and Sastry, 1985) and the placement of inclined piles is more difficult. Examples of strucures where substantial lateral loads can be induced upon primarily vertical piles include:

i) offshore oil/gas drilling platforms exposed to current, storm, ice and vessel loads

ii) bridge piers/piles exposed to current, ice and vessel loads

iii) electrical transmission towers exposed to wind loading

iv) marine structures such as a dock

v) building foundations subject to wind and earthquake loading.

In designing for lateral loads on piles, the following two criteria must be satisfied, ultimate structural failure of the pile cannot occur; and there must be an acceptable deflection at anticipated working loads. The second criterion is most often used for design as it usually ensures that the first is satisfied.

2.2.2 Mechanism of Behaviour

Horizontal loads on vertical piles are resisted by the mobilization of resistance in the soils confining the pile as the soil deflects.

Based upon field and laboratory observations (Goldsmith, 1979), when a circular pile is loaded the soil moves radially away from the front face and inwards towards the back face (Fig. 2.10). Fig. 2.10 shows that there is little or no slip along the pile sides and hence a very small contribution of side friction to the overall lateral resistance. Smith and Slyh (1986), among others, disagree with this, however, and suggest that a marked amount of slip along the pile sides exists. At depth, below the influence of a free surface, Randolph and Houlsby (1984) offer the concept

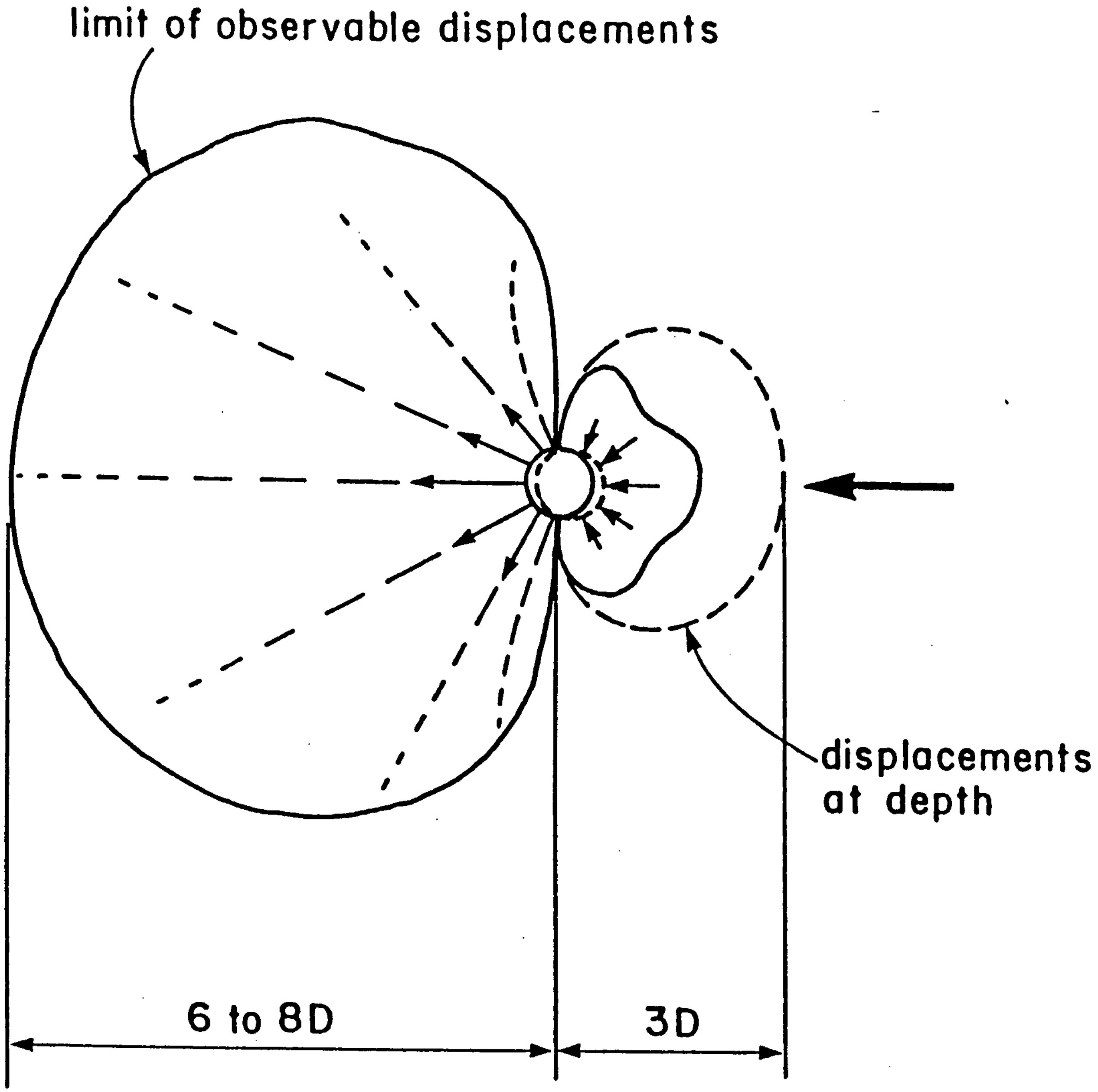


FIG. 2.10. OBSERVED DISPLACEMENTS AROUND LATERALLY LOADED PILE
(After Robertson et al., 1986)

of soil "flowing" around the laterally displaced pile (Fig. 2.11). Near the surface, where confining stresses are low, the soil being stressed by the displacement of the pile moves towards the free surface. This movement of soil at shallow depth is shown in Fig. 2.12. Below some "critical depth" the soil no longer has a vertical component to its movement. This concept of critical depth is also shown schematically in Fig. 2.12. The behaviour mechanisms shown in Figs. 2.10 through 2.12 assume that no torsional component exists in the applied load. Torsional loading, due to eccentricity of the applied load is addressed by Randolph (1981,a), among others, and will not be considered in this study.

2.2.3 Lateral Load Behaviour Prediction Methods

The problem of predicting the behaviour of piles subject to lateral loads is a difficult analytical question. Although not as plentiful as for axially loaded piles, proposed solutions to the lateral pile problem are numerous. The most common of these approaches will be briefly presented in the following section.

The simplest model for the laterally loaded pile problem is that of a vertical elastic beam, loaded transversely and restrained from movement by uniform linear Winkler springs along the beam. The stiffness of these springs is commonly called the subgrade reaction modulus for the soil. Hetenyi (1946) solved closed form solutions for several cases of loading and pile fixity. The model used is as shown in Fig. 2.13. The equation Hetenyi solved was of the form:

$$EI \cdot \frac{d^4y}{dx^2} + P_x \frac{d^2y}{dx^2} + E_s \cdot y = 0 \qquad (2.9)$$

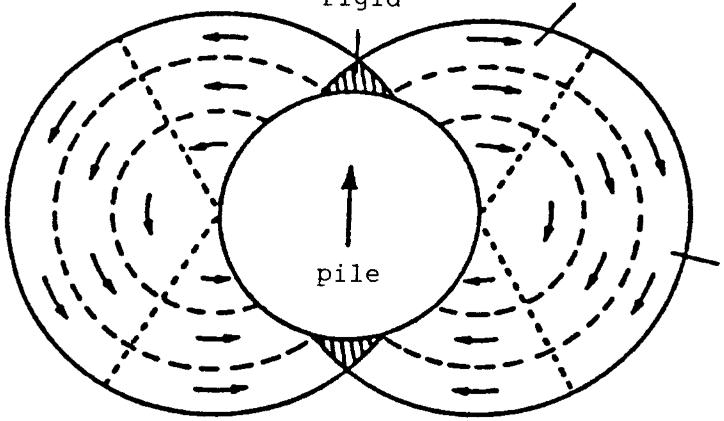


FIG. 2.11. SOIL FLOW AROUND LATERALLY LOADED PILE
AT DEPTH
(Adapted from Randolph and Houlsby, 1984)

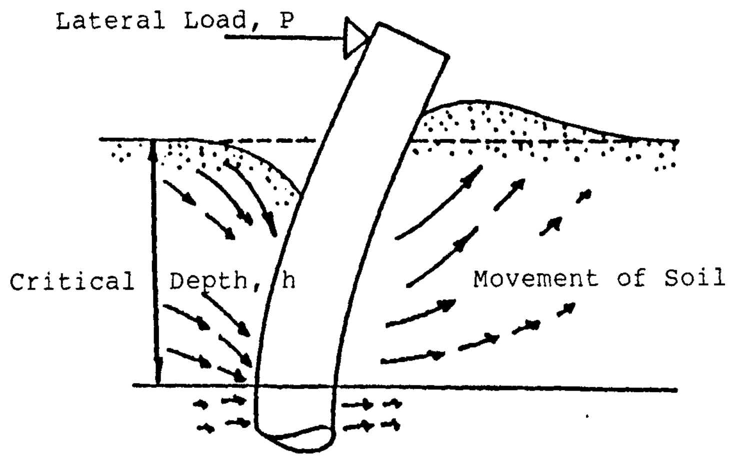


FIG. 2.12. SOIL MOVEMENT AT SHALLOW DEPTH DUE TO LATERAL PILE DISPLACEMENT (Adapted from Broms, 1964)

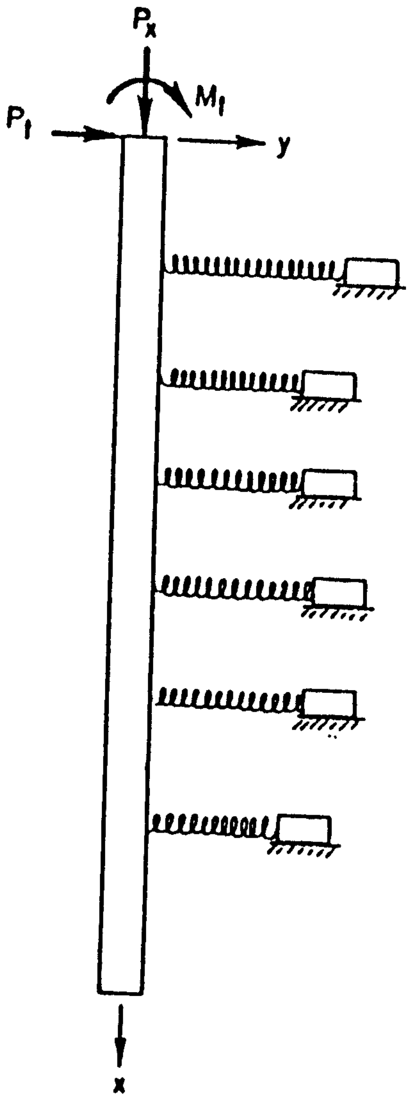


FIG. 2.13. MODEL OF LATERALLY LOADED PILE USING DISCRETE WINKLER SPRINGS

where: EI = flexural stiffness of pile

E_s = subgrade reaction modulus

From this early work, analytical approaches have developed in two separate directions (Randolph, 1981,b).

One development has utilized the integral equation (or boundary integral) method of analysis, modelling the soil as a homogeneous elastic continuum (Poulos, 1971). This method is very computationally intensive and much experience in discretizing boundary elements is necessary for accurate results (Evangelista and Viggiani, 1976). The general use of the integral equation in routine geotechnical practice is seen as still being some time away.

The other development retains the conceptual model of modelling the soil restraint as discrete Winkler springs. Improvements to this model began when spring stiffnesses along the pile were allowed to vary (Reese and Matlock, 1956). The most important improvement came with the introduction of the nonlinear subgrade reaction method proposed by Matlock and Ripperger (1956), among others. The nonlinear subgrade reaction method is now widely used for the design of laterally loaded piles. This method replaces the soil reaction with a series of independent Winkler springs. The nonlinear behaviour of the soil springs is represented by P-y curves which relate soil reaction (P) and pile deflection (y) at points along the pile length. A typical P-y curve is shown in Fig. 2.14.

Most traditional methods of obtaining P-y curves (e.g. Matlock, 1970; API RP2A, 1980) involve using laboratory data from samples that may or may not be representative of the actual in-situ soil conditions around the pile. In-situ testing methods, in particular the pressuremeter, have

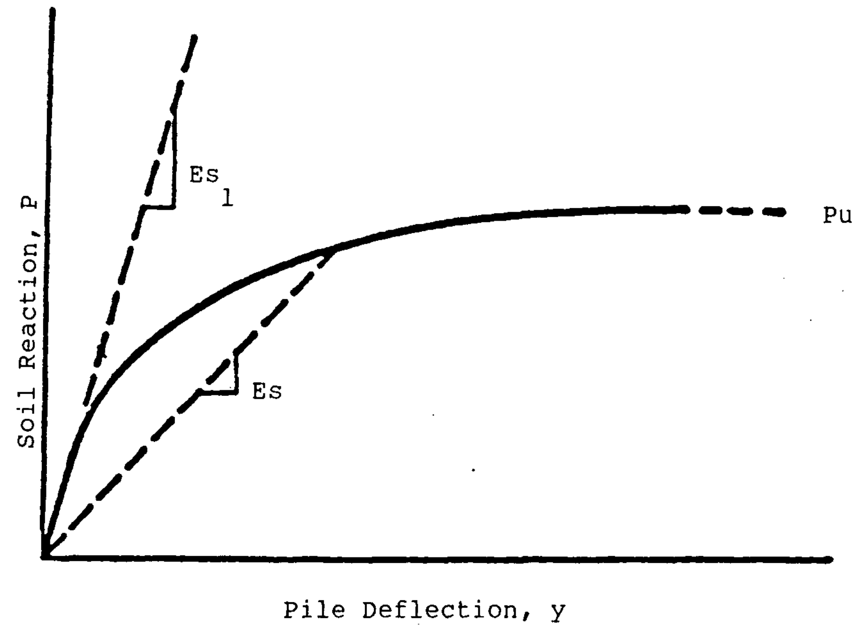


FIG. 2.14. SHAPE OF A TYPICAL P-y CURVE USED FOR NON-LINEAR SUBGRADE REACTION METHOD

allowed the development of semi-empirical methods to obtain P-y curves using data obtained in the field. Several methods have been proposed for the development of P-y curves and subsequent design of laterally loaded piles using pressuremeter data (Briaud et al., 1983; Baguelin et al., 1978; Robertson et al., 1983; Baguelin, 1982). Other in-situ tests, such as the flat plate dilatometer test (using a method developed as a part of this study), can also be used to develop P-y curves.

CHAPTER 3

RESEARCH SITE

In 1984, the British Columbia Ministry of Transportation and Highways (B.C. MOTH) installed a 915 mm diameter steel pipe test pile as part of the design phase for the proposed Alex Fraser Bridge Project. The University of British Columbia (UBC) became involved in the subsequent prediction of the pile's axial and lateral behaviour by the use of in-situ testing methods. Robertson et al. (1985) published these results and demonstrated how accurately the measured load test results could be predicted by the use of in-situ tests. To further study the prediction of pile behaviour using in-situ testing methods, and to provide UBC with a full-scale field teaching site, the B.C. MOTH generously provided six piles for research and teaching on a site directly adjacent to the location of the 1984 load test. The B.C. MOTH provided all piling materials and the labour needed for specially preparing the site and for pile installation. In addition, instruments and personnel were provided for dynamic monitoring during pile installation and for some portions of the load testing program. All data from the 1984 pile load testing was made fully available for inclusion within this study.

Throughout this thesis, the UBC Pile Research Site (UBCPRS) and the MOTH Pile Research Site (MOTHPRS) will mainly be discussed as separate sites. The reason for this is that the pre-planning, pile driving and pile load testing performed at the UBCPRS was done mainly by UBC personnel whereas UBC had little direct involvement with these areas for the MOTHPRS. The two research sites are, however, within 100 m of one another and so in this chapter, especially with respect to the discussion of area geology, the separation will be largely ignored.

3.1 Regional Geology

The research site is located on Lulu Island which is within the post-glacial Fraser River delta (Fig. 3.1). Blunden (1975) correctly identifies the Fraser Delta region sediments as marine deltaic deposits that have been formed upon basal layers that have undergone isostatic rebound for roughly the last 11,000 years at a rate greater than the rate of recent (i.e. post-glacial) marine transgression. The total thickness of the deltaic deposits varies but they are, on average, roughly 200 m thick (Blunden, 1975). The Fraser Delta area now known as Richmond, Delta, and New Westminster has been above mean sea level for approximately 8,000 years when the sea level was about 10 m below present levels.

The surficial geology of the Lulu Island region is typical of a former marine environment no longer dominated by tidal action. There is a prevalent deposit of organic silty clays that has been laid down in a swamp or marsh environment. Below this upper layer, which extends to roughly 15 m depth, a medium dense sand deposit, locally silty, prevails to roughly 25-30 m depth. This deposit is indicative of a very high energy depositional period and most likely represents a former channel bank of the Fraser River. Next, prevailing to roughly 60 m depth, exists a normally consolidated clayey silt containing thin sand layers. These materials were laid down in a much lower energy environment than the sand above. Below this, probably extending for up to 150-200 m depth, is a similar deposit except that the sand layers are much more prevalent and thicker (up to 0.5 m thick). The non-uniformity of the deposits below 30 m indicate a

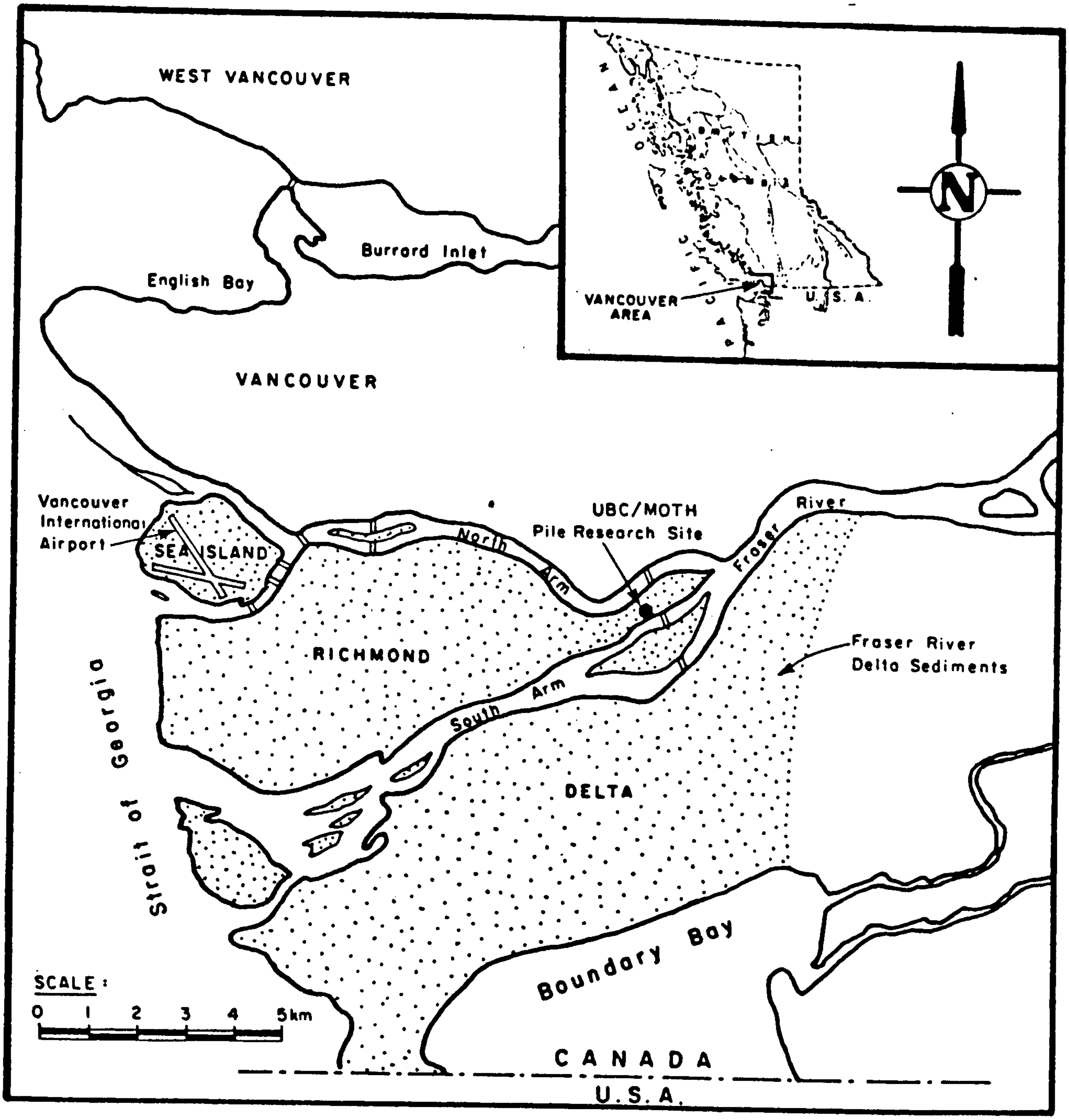


FIG. 3.1. GENERAL LOCATION OF RESEARCH SITE

depositional history most likely consisting of alternating turbulent and quiescent environments associated with either tidal flat facies, marginal bank, or an alluvial floodplain depositional environment. The CPT profiles presented in the following chapter present a clear picture of the stratigraphic detail at the site.

3.2 Site Description

As shown in Fig. 3.1, both the UBCPRS and MOTHPRS are located on the north side of the Annacis Channel within the South Arm of the Fraser River. Fig. 3.2 shows the relative locations of the UBCPRS and the MOTHPRS. Upon the entire site, 2 to 4 m of heterogeneous fill exists at the surface. For the purpose of facilitating in-situ testing, making pile driving possible, and studying lateral pile behaviour, the fill material was removed in the general area of both pile sites. This material was replaced with clean river sand and at the UBCPRS this sand was placed at varying densities (see Chapter 4). The purpose of the different densities for the sand was to allow the behaviour of the piles to be studied under lateral loads with different soil stiffnesses near ground surface. This effect, however, has not been investigated for this study and is left as some of the future suggested research for the site.

The site directly underlies a connector bridge to the new Alex Fraser cable-stayed bridge linking Annacis Island with Surrey and Delta. The piles used for the connector bridge are 1.5 m diameter piles driving to depths in excess of 70 m. The purpose of the MOTHPRS was to assess the capacities of these piles.

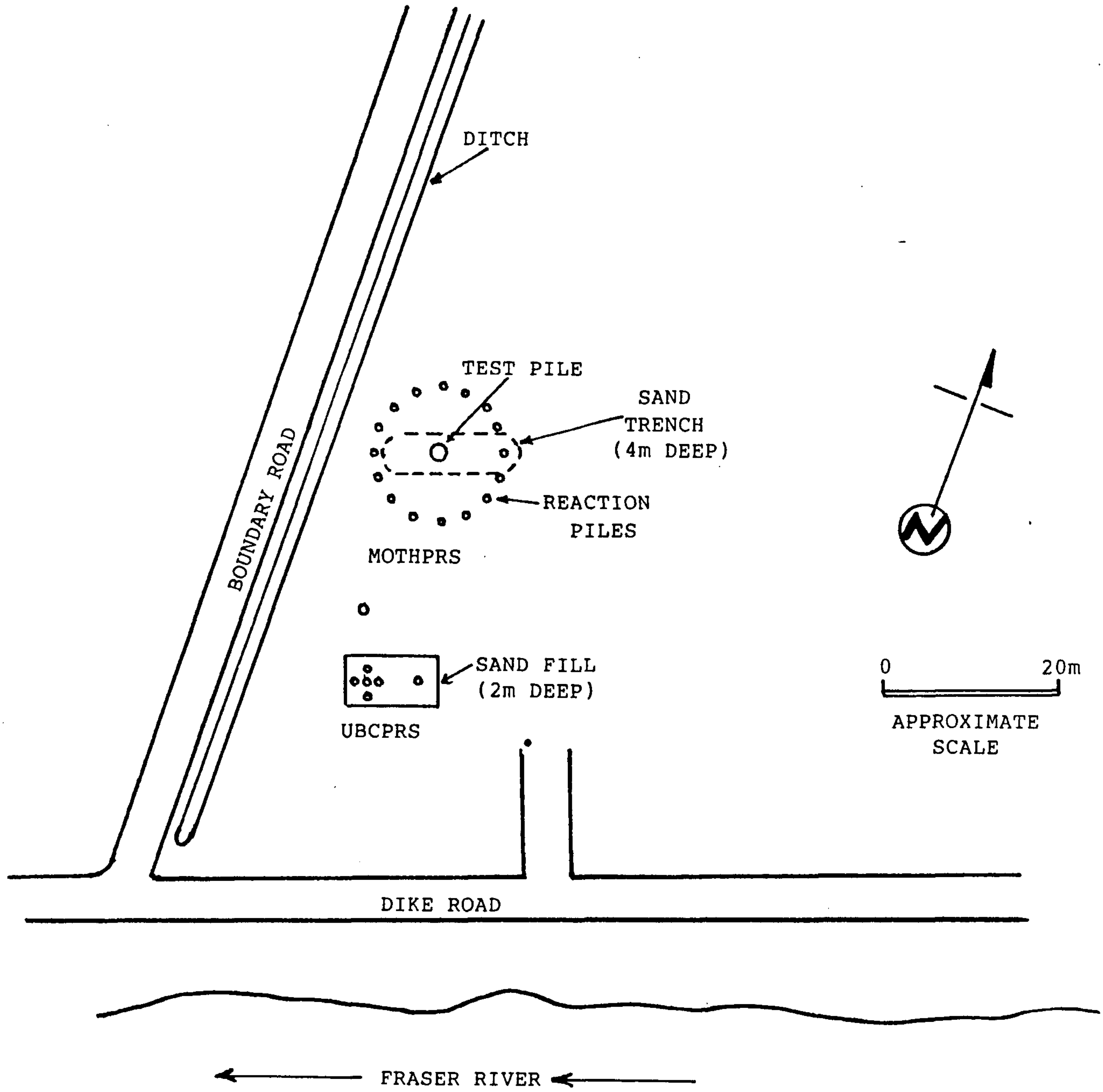


FIG. 3.2. SITE PLAN OF THE UBCPRS
AND THE MOTHPRS

CHAPTER 4

IN-SITU TESTS PERFORMED

4.1 Introduction

In-situ testing, traditionally consisting of geotechnical engineers pushing their heels or a stick into the soil to make qualitative measures, has always played a major role in the art of foundation engineering (Robertson, 1985). Modern in-situ tests that can supply economic and repeatable results are becoming increasingly available to the geotechnical engineer. The four main reasons that these tests are becoming increasingly popular are listed by Mitchell et al. (1978), as follows;

1) The ability to determine properties of soils, such as sands and off-shore deposits, that cannot be easily sampled in the undisturbed state.
2) The ability to avoid some of the difficulties of laboratory testing, such as sample disturbance and the proper simulation of in-situ stresses, temperature, and chemical and biological environments.
3) The ability to test a larger volume of soil than can be conveniently tested in the laboratory.
4) The increased cost effectiveness of an exploration and testing program using in-situ methods.

In addition, a laboratory test must reproduce the in-situ state of stress whereas an in-situ test invariably begins at or close to this state. The fact that an in-situ test must be conducted with reference to the existing in-situ stress state is, however, an important limitation.

In-situ testing somewhat alters the stress field around the device due to the insertion of the device into the ground. However, in contrast to laboratory testing, in-situ testing cannot generally simulate large changes in stress. Robertson (1985) and Wroth (1984) provide excellent discussions of the in-situ testing methods available and the interpretation of these tests for foundation design purposes.

Pile foundations, like any engineered subsurface structure, require an accurate assessment of the properties of the soil from which they are to derive their resistance. In this chapter, several of the most common in-situ testing methods used to design pile foundations are briefly described and the summarized data obtained for this study are presented. Later in this study conclusions will be made regarding the accuracy of the soil properties obtained using these tests. These conclusions will be made by assessing the ability of the data obtained to predict measured pile behaviour using various analytical techniques.

Table 4.1 presents a summary of the in-situ tests performed for this study. The test locations are shown on Fig. 4.1 (full site plan) and Fig. 4.2 (expanded scale for detail of UBCPRS). The numbered locations relate to the numbers listed in Table 4.1. Table 4.1 and Figs. 4.1 and 4.2 should be used as a guide for those wishing to use the research sites in the future.

4.2 In-Situ Testing Methods

In this section only, the three testing procedures used in this study for predicting axial and lateral pile behaviour are described. The summarized results from these tests are also included.

No.	Test	Name	Date Performed
1	Seismic Cone Pressuremeter Test	FDPMT87-1	3 APR 87
2	Self Boring Pressuremeter Test	SBPMT87-3	16 FEB 87
3	Self Boring Pressuremeter Test	SBPMT87-2	12 FEB 87
4	Self Boring Pressuremeter Test	SBPMT87-1	11 FEB 87
5	Seismic Cone Penetration Test	SCPT87-1	7 FEB 87
6	Nilcon Field Vane Test	SPT86-1	31 OCT 86
7	Piezometer Cone Penetration Test	NFVT86-1	31 OCT 86
8	Piezometer Cone Penetration Test	CPT86-2	31 AUG 86
9	Piezometer Cone Penetration Test	CPT86-1	22 AUG 86
10	Piezometer Cone Penetration Test	CPT85-1	13 JUL 85
11	Piezometer Cone Penetration Test	CPT84-1	22 AUG 84
12	Flat Plate Dilatometer Test	DMT85-2	29 AUG 85
13	Flat Plate Dilatometer Test	DMT85-1	22 AUG 85
14	Full Displacement Pressuremeter Test	FDPMT84-1	18 AUG 84
15	Dynamic Cone Penetration Test	DCPT85-1	30 AUG 85
16	Dynamic Cone Penetration Test	DCPT85-2	30 AUG 85
17	Dynamic Cone Penetration Test	DCPT85-3	30 AUG 85
18	Dynamic Cone Penetration Test	DCPT85-4	30 AUG 85
19	Becker Hammer Test	BDT85-2	20 AUG 85
20	Becker Hammer Test	BDT85-1	20 AUG 85

Table 4.1 Pile Research Sites In-Situ Tests Performed

The results from the other tests performed (see Table 4.1) are not included within this study. These results may be found filed at the UBC In-Situ Testing Group Library, Room 1208, in the Civil Engineering Building at U.B.C.

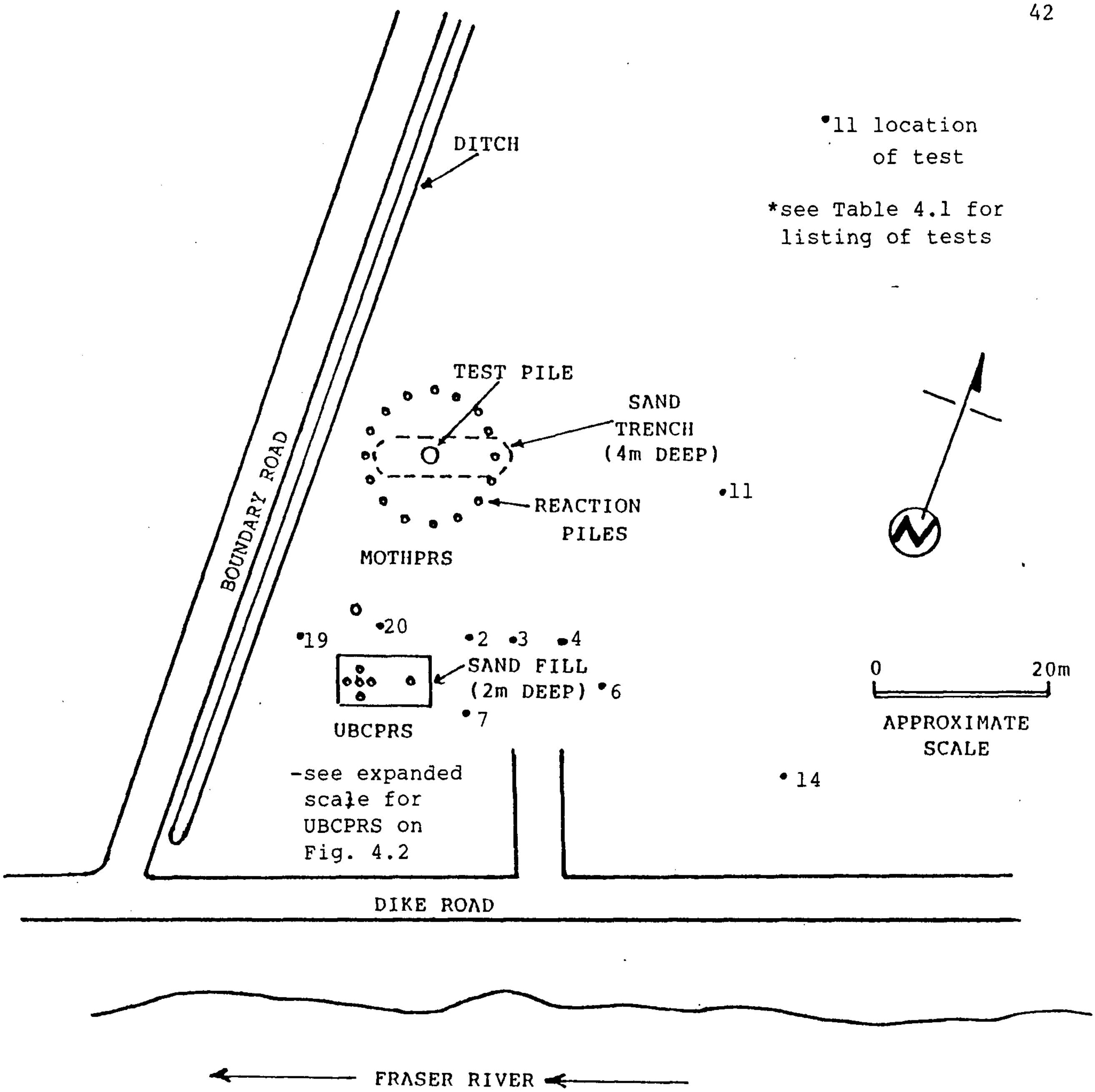
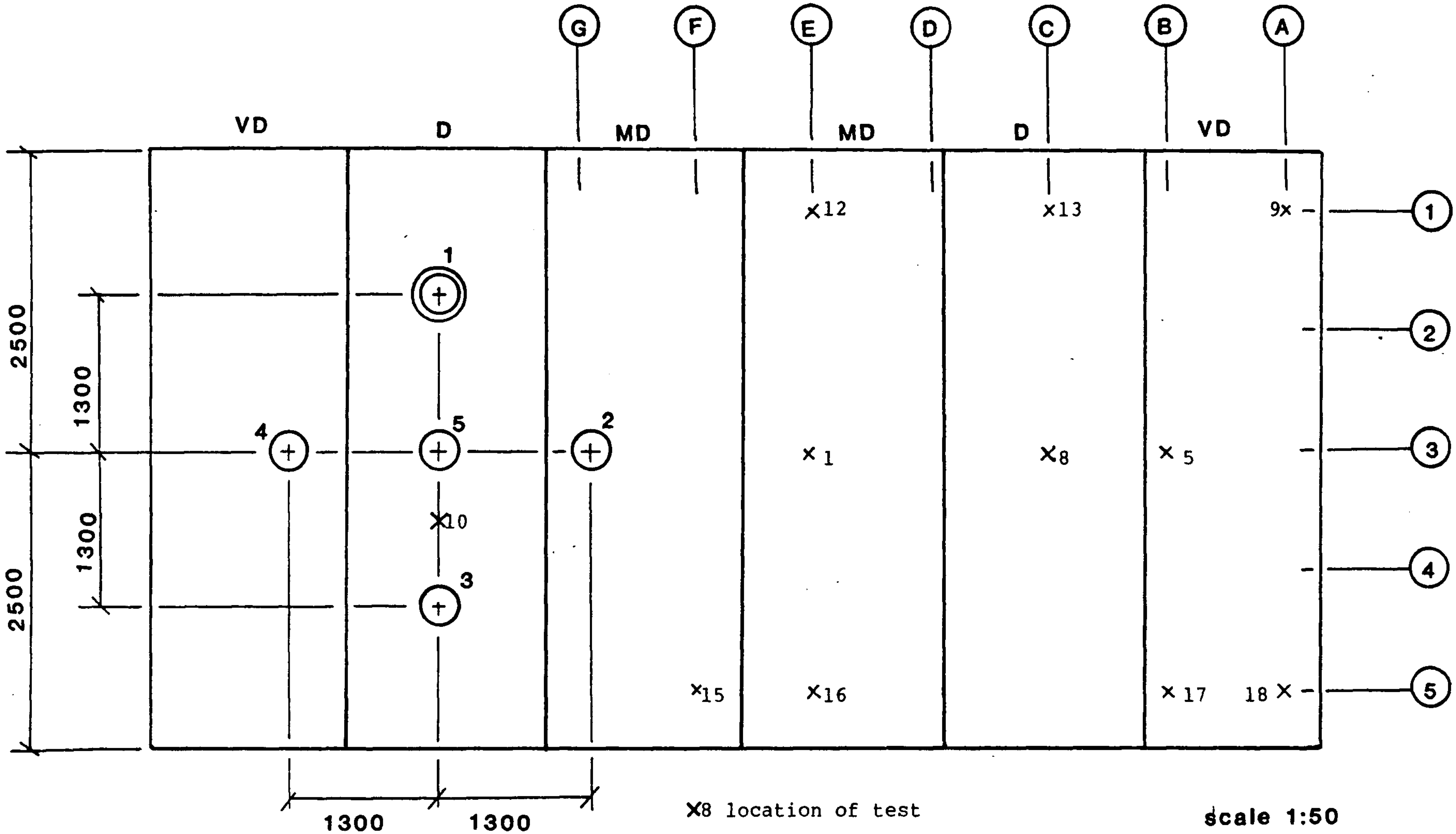


FIG. 4.1. LOCATIONS OF IN-SITU TESTS PERFORMED AT PILE SITES



×8 location of test

*see Table 4.1 for listing of tests

scale 1:50

VD : very dense

D : dense

MD : medium dense

FIG. 4.2. LOCATIONS OF IN-SITU TESTS PERFORMED AT UBCPRS

4.2.1 Piezometer Cone Penetration Testing

4.2.1.1 Test Description

The cone penetration test (CPT) is a quasi-static penetration test. The CPT was originally developed in Europe but is now gaining increasing acceptance in North America and elsewhere.

For this study electric cones with built in load cells that measure the end resistance (q_c) and sleeve friction (f_s) continuously were used. A schematic of UBC6, an electric cone developed at UBC, is shown in Fig. 4.3. It is this cone that was mainly used in this study. This cone, in accordance with ASTM D3441-79, has a 10 cm² cone tip with a 60° conical tip. The friction sleeve has a standard 150 cm² surface area. In addition to the q_c and f_s measurements, many cones (e.g. UBC6) now incorporate a pore pressure transducer. The addition of the pore pressure transducer allows continuous measurement of pore pressures during penetration as well as equilibrium pore pressures obtained from dissipation data.

The advantages of the CPT are: rapid procedure; continuous logging; good repeatability; and easy standardization. Some of its limitations include: inability to penetrate gravel; no sample obtained; high initial cost; and requirement for technical back-up facilities.

As for any electronic instrument, proper calibration and periodic calibration checks are essential to ensure all electric cones are functioning properly.

Robertson and Campanella (1986) provide a comprehensive review of equipment, testing procedures and data interpretation for electric cone testing.

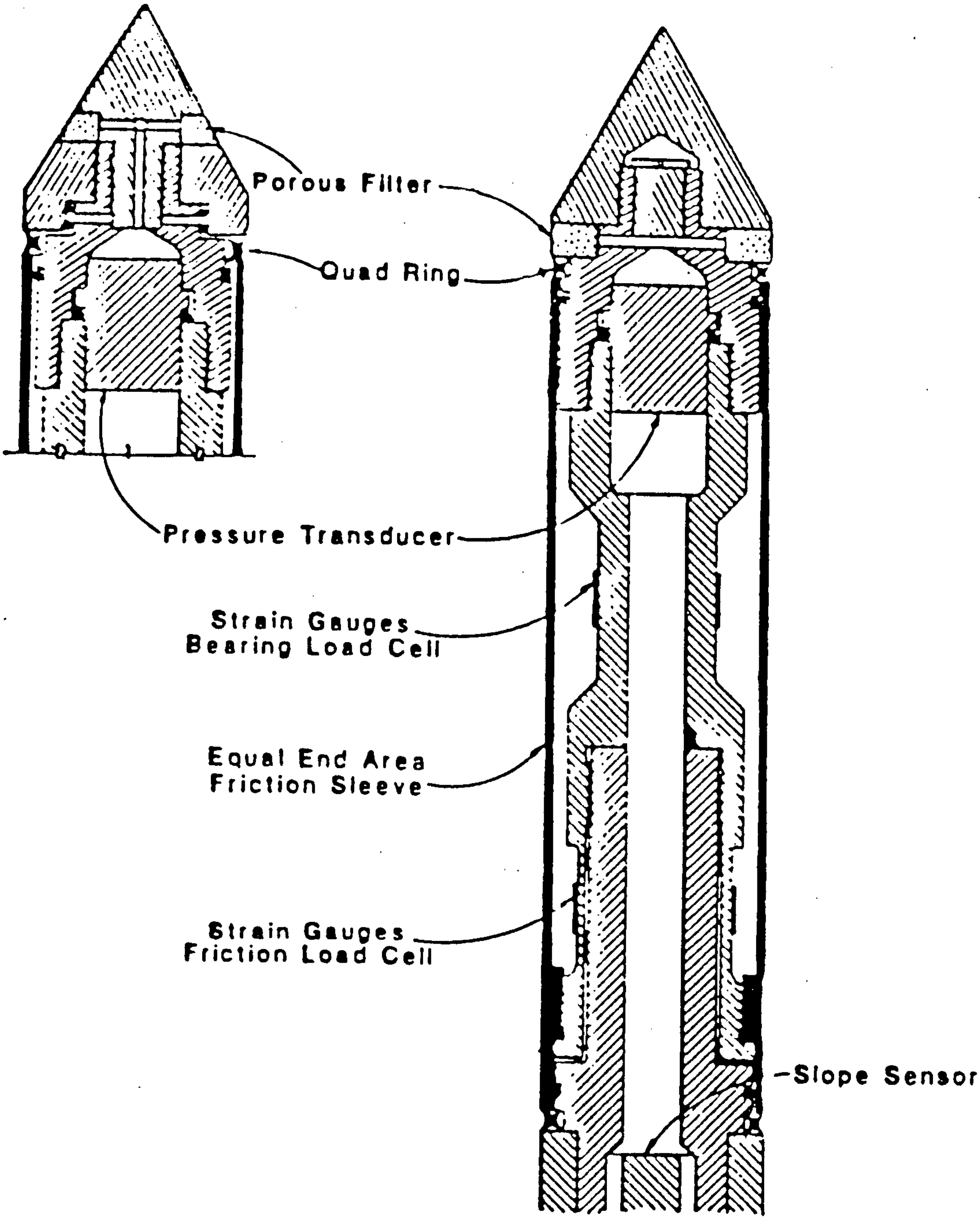


FIG. 4.3. SCHEMATIC OF ELECTRIC CONE DEVELOPED AT UBC

4.2.1.2 Results

Figs. 4.4 and 4.5 show, respectively, interpreted CPT profiles for the UBCPRS and MOTHPRS. It is data from these two CPT profiles that is used in Chapter 6 to predict axial pile capacity.

For the UBCPRS, CPT85-1 (see Table 4.1) is used. As shown in Fig. 4.4, this sounding was carried out to nearly 36 meters in depth. The extremely soft nature of the soft organic silty clay between 2.5 and 14.5 meters is very apparent on Fig. 4.4. See Fig. 4.2 for the location of CPT85-1.

For the MOTHPRS, CPT84-1 (see Table 4.1) is used. This sounding (Fig. 4.5) is as described by Robertson et al. (1985). CPT84-1 is located on Fig. 4.1. Note the differences in scale between Figs. 4.4 and 4.5.

4.2.2 Pressuremeter Testing

4.2.2.1 Test Description

The pressuremeter was initially developed by L. Ménard in 1954 in France as a "specific test" tool to obtain a measure of strength and stiffness of soils and rocks. Ménard-type pressuremeters are generally placed in pre-bored holes and are therefore often difficult to use in cohesionless or swelling soils. Self-boring pressuremeters were then developed in 1972 in an effort to eliminate soil disturbance associated with a pre-bored hole. However, self-boring pressuremeters are usually expensive, require a great deal of technical backup, and are often limited to use in soils where $D_{50} < 5$ mm (where D_{50} is the mean grain size of the material to be tested).

One of the latest developments is a full displacement pressuremeter test (FDPMT). This test does cause soil disturbance due to the full

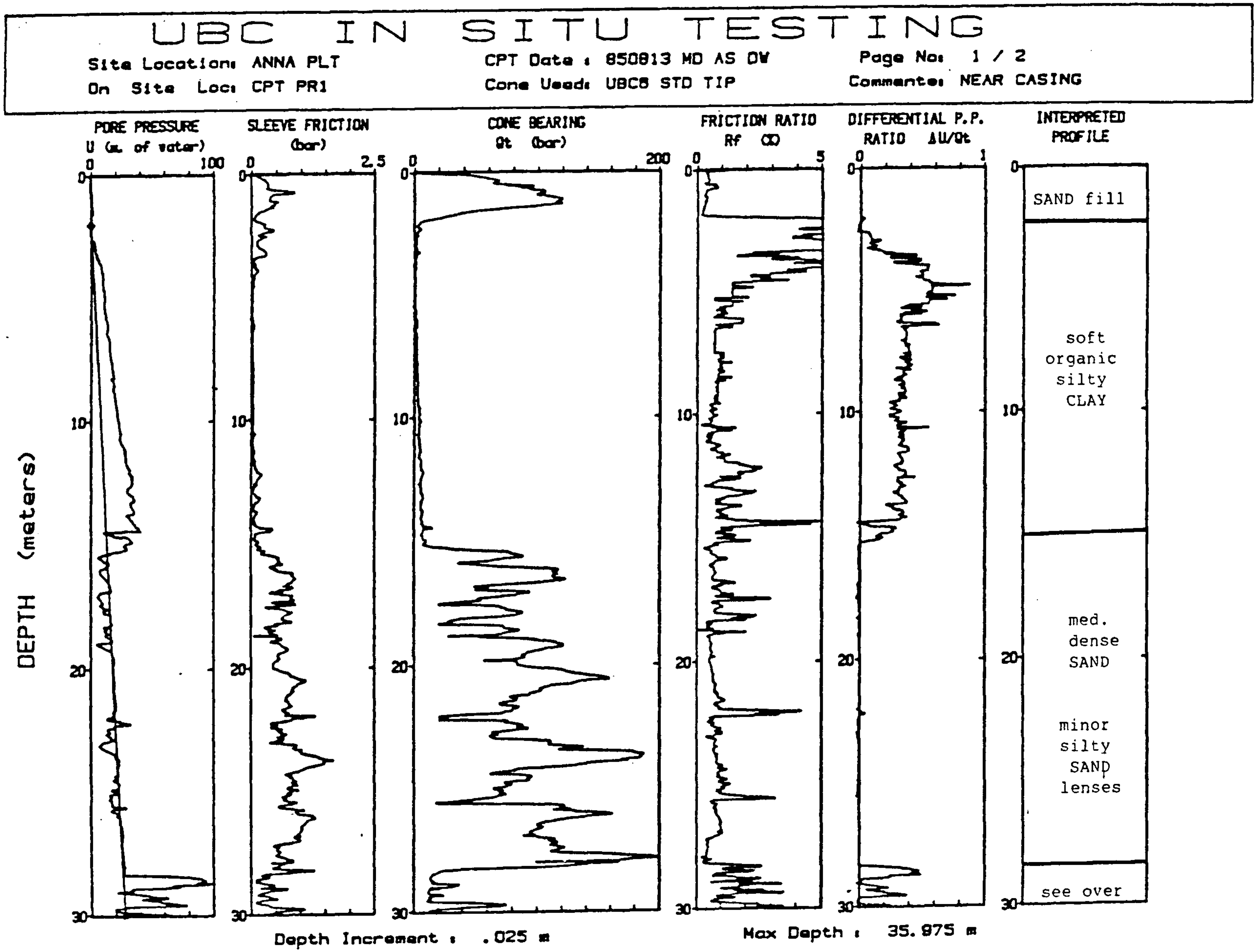


FIG. 4.4. CPT INTERPRETED PROFILE USED FOR UBCPRS

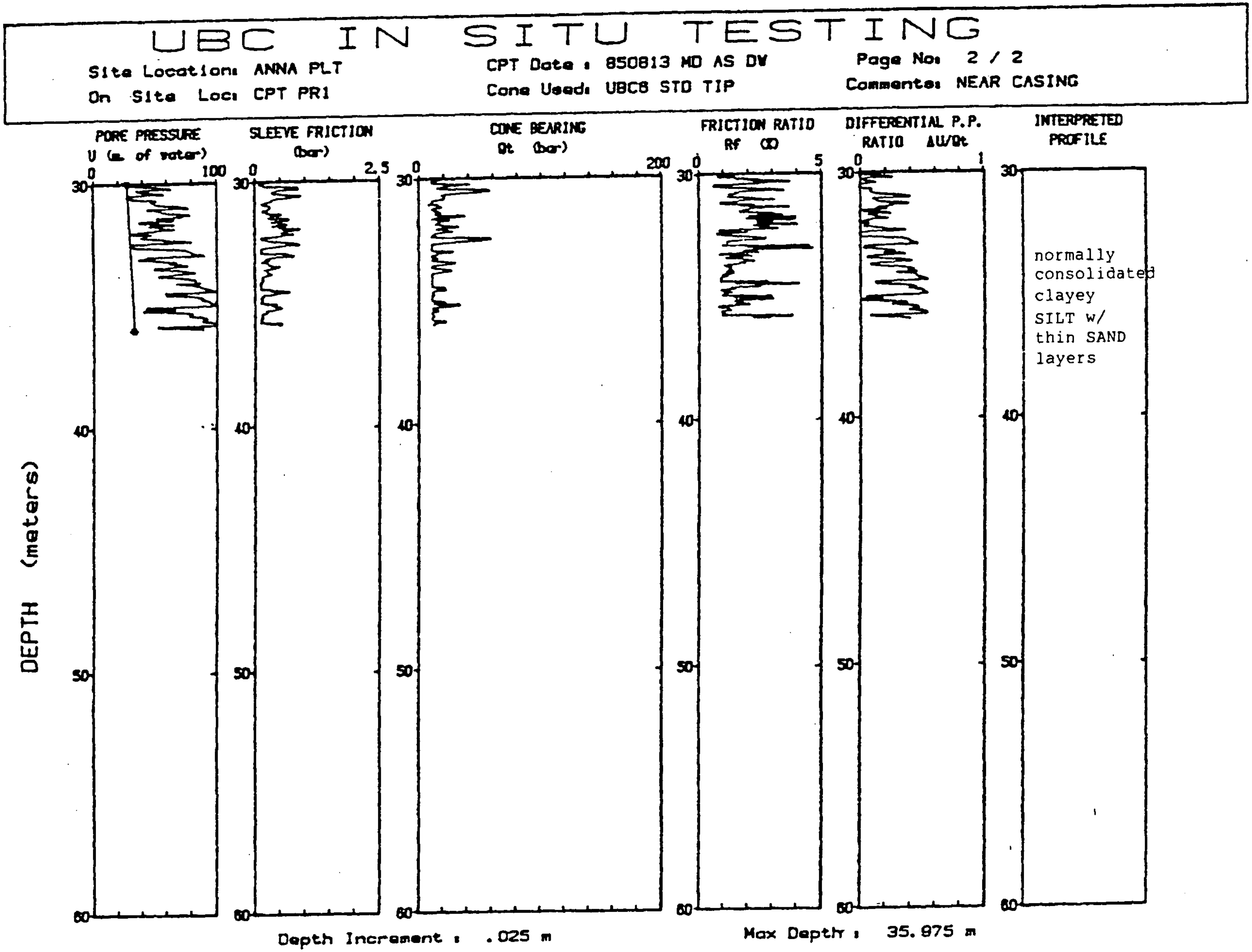

UBC IN SITU TESTING
Site Location: ANNA PLT
On Site Loc: CPT PR1
CPT Date : 850813 MD AS DW
Cone Used: UBC8 STD TIP
Page No: 2 / 2
Comments: NEAR CASING
PORE PRESSURE
U (m. of water)
SLEEVE FRICTION
(bar)
CONE BEARING
Qt (bar)
FRICTION RATIO
Rf (%)
DIFFERENTIAL P.P.
RATIO ΔU/Qt
INTERPRETED
PROFILE
normally
consolidated
clayey
SILT w/
thin SAND
layers
DEPTH (meters)
Depth Increment : .025 m
Max Depth : 35.975 m

FIG. 4.4. CONT.

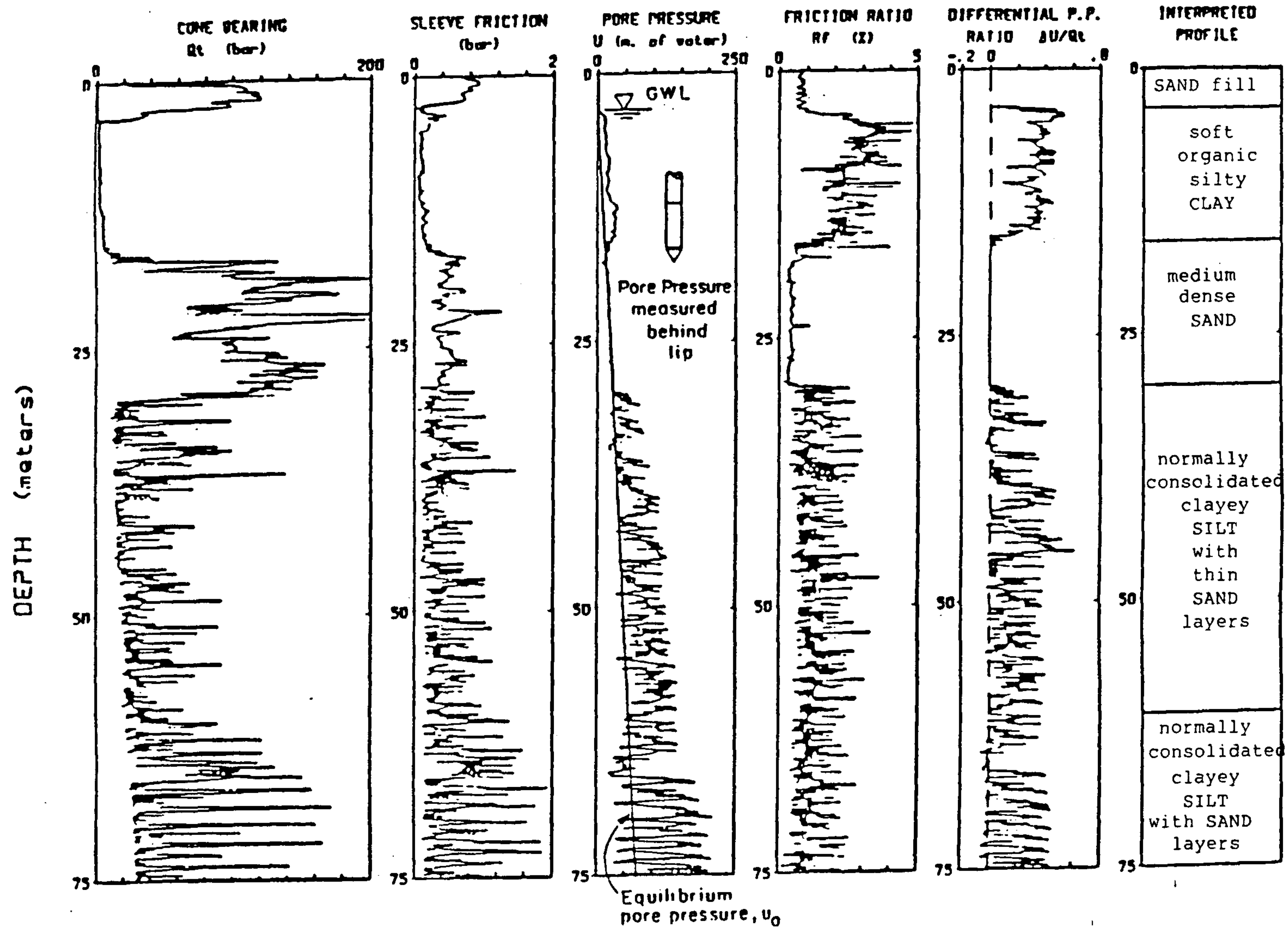


FIG. 4.5. CPT INTERPRETED PROFILE USED FOR MOTHPRS

displacement inflation, but the disturbance is essentially repeatable each time. Hughes and Robertson (1985) suggest that for sands, the stress paths followed by soil elements near the advancing probe are such that before pressuremeter inflation, the radial stress on an element adjacent to the probe has reduced close to the initial in-situ stress state. The pressuremeter test supplies a pressure expansion curve relating applied pressure to cavity strain.

For the UBCPRS, the UBC Cone Pressuremeter (Fig. 4.6) was used. This instrument has a 15 cm^2 cross-sectional area. The cone portion of the probe was not utilized. Campanella and Robertson (1986) briefly summarize the research and development of the UBC Cone Pressuremeter. For the MOTHPRS, a self-boring pressuremeter, pushed in a full-displacement manner, was used. Details of this probe can be found in Hughes and Robertson (1985).

4.2.2.2 Results

The pressuremeter curves used to predict lateral pile behaviour for the UBCPRS piles are from FDPMT87-1 (see Table 4.1 and Fig. 4.2). These pressuremeter curves are included in Appendix I. The depths of the tests in FDPM87-1 were:

i) 0.17 m

ii) 1.0 m

iii) 2.0 m

iv) 3.0 m

v) 4.0 m

vi) 4.8 m

vii) 6.35 m

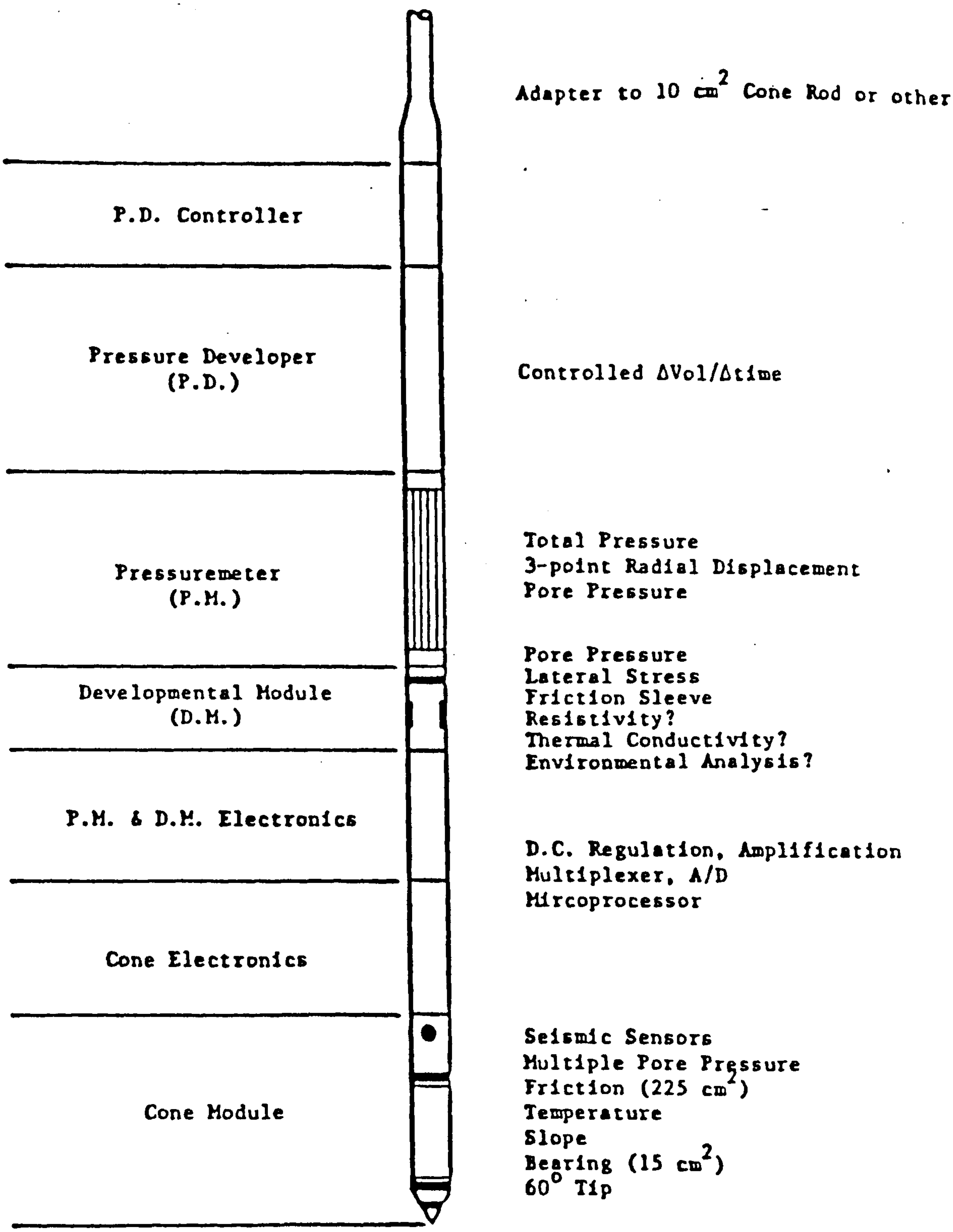


FIG. 4.6. CONCEPTUAL DESIGN : UBC CONE PRESSUREMETER

(After Campanella and Robertson, 1986)

viii) 7.9 m

ix) 9.4 m

x) 10.4 m

xi) 12.4 m

xii) 15.5 m

These test depths can be compared with the stratigraphy for the UBCPRS shown in Fig. 4.4.

The pressuremeter curves used to predict lateral pile behaviour for the MOTHPRS pile are from FDPMT84-1 (see Table 4.1 and Fig. 4.1). Full details of the pressuremeter testing for the MOTHPRS can be found in Brown (1985).

4.2.3 Flat Plate Dilatometer Testing

4.2.3.1 Test Description

The flat plate dilatometer test (DMT) was developed in Italy by S. Marchetti in 1980. The dilatometer is a flat plate 95 mm wide, 14 mm thick and 220 mm in length. A flexible stainless steel membrane 60 mm in diameter is located on one side of the blade. A schematic representation of the dilatometer is shown in Fig. 4.7.

The dilatometer test involves inflating the flexible membrane to achieve a one millimeter deflection. The first reading (A) corresponds to the membrane lift-off pressure and the second reading (B) to the pressure required to cause the one millimetre deflection at the center of the membrane. Readings A and B are corrected for both free-air effects of membrane seating and the effect of membrane curvature. The DMT is performed at 20 cm intervals of depth. This leads to a comprehensive, however discrete, profile.

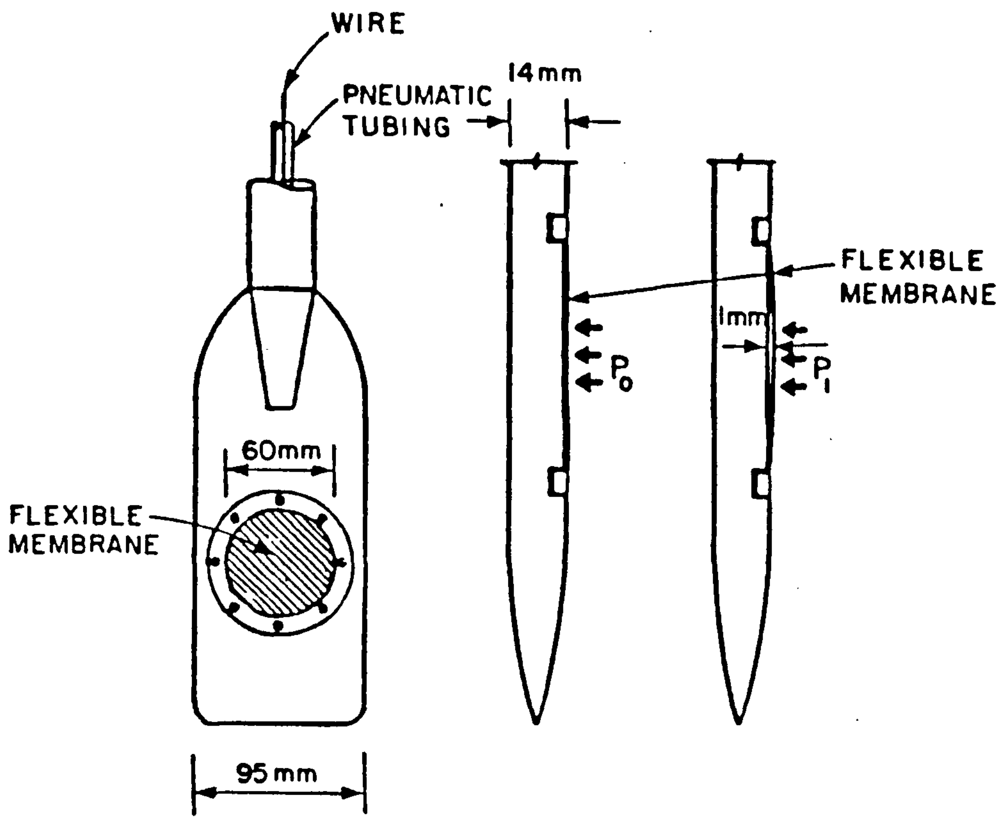


FIG. 4.7. SCHEMATIC REPRESENTATION OF
FLAT PLATE DILATOMETER

Using the corrected dilatometer data of A and B (P_0 and P_1, respectively), Marchetti (1980) developed empirical correlations to find several soil parameters. These correlations are all based upon three index parameters Marchetti gets from P_0 and P_1. These are Material Index, I_d; Horizontal Index, K_d; and Dilatometer Modulus, E_d.

Much more detailed discussions of the DMT and testing procedures are given in Marchetti (1980), Brown (1983), Campanella and Robertson (1983), and in Schmertmann (1986).

4.2.3.2 Results

The DMT results used for both the UBCPRS and the MOTPRS are shown in Figs. 4.8 and 4.9. The "raw" DMT data can be found in Appendix I. Fig. 4.8 shows the intermediate geotechnical parameters obtained from the DMT whereas Fig. 4.9 shows the interpreted geotechnical parameters from the DMT. The DMT test used was DMT85-2 (see Table 4.1 and Fig. 4.2). The intermediate geotechnical parameters and the interpreted geotechnical parameters are obtained by using correlations developed by Marchetti (1980). Details of the computer program used to evaluate these parameters can be found in MacPherson (1984).

4.2.4 Other Methods

As shown in Table 4.1, a number of in-situ tests were performed at the UBCPRS and the MOTHPRS. Due to space restrictions, only the test results used to predict axial and lateral pile behaviour have been included within this dissertation. However, the locations of all tests performed (see Figs. 4.1 and 4.2) are included so that this study can be used as a guide for those wishing to use the research sites in the future.

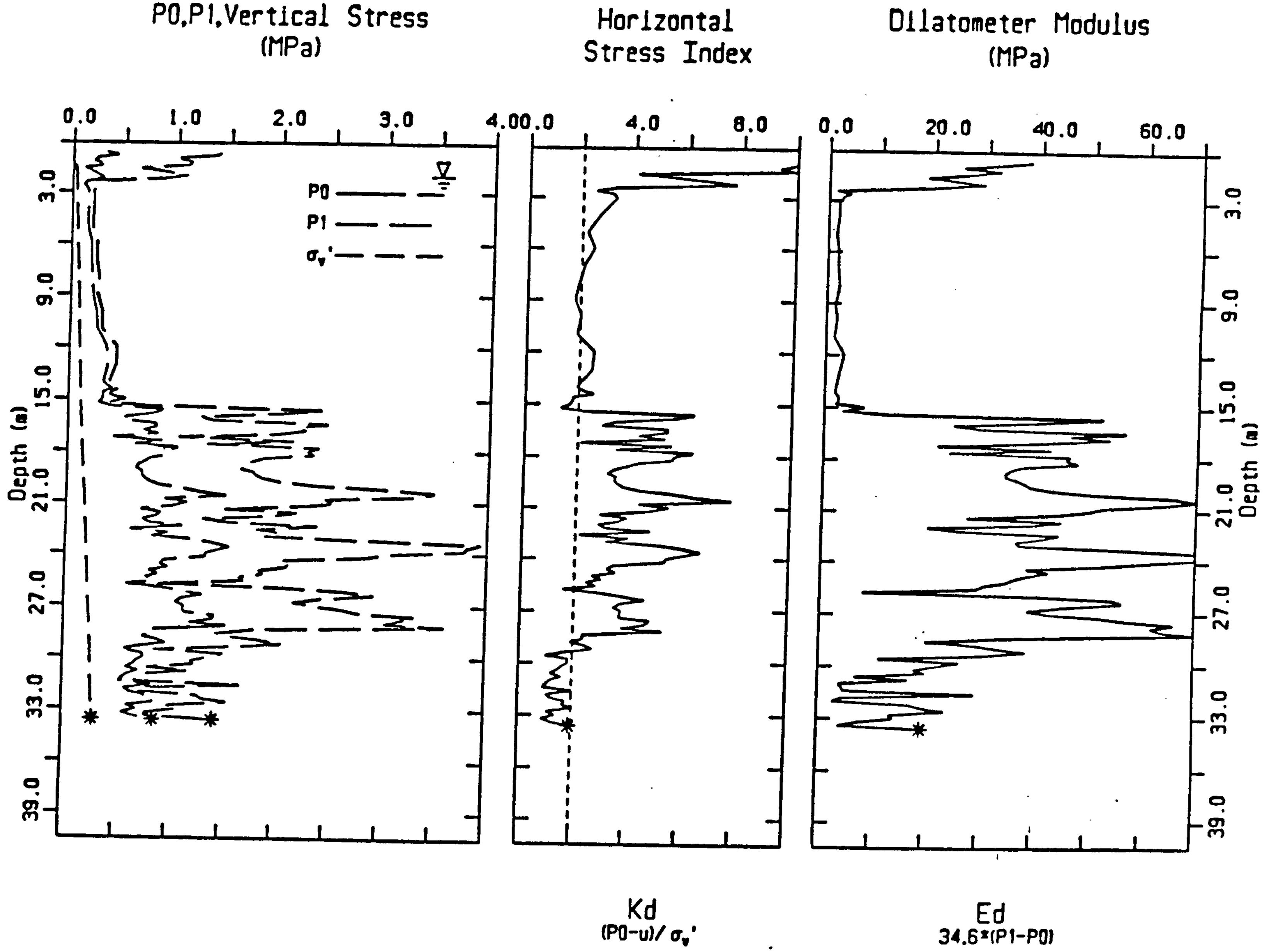


FIG. 4.8. INTERMEDIATE GEOTECHNICAL PARAMETERS FROM DMT
UBCPRS/MOTHPRS

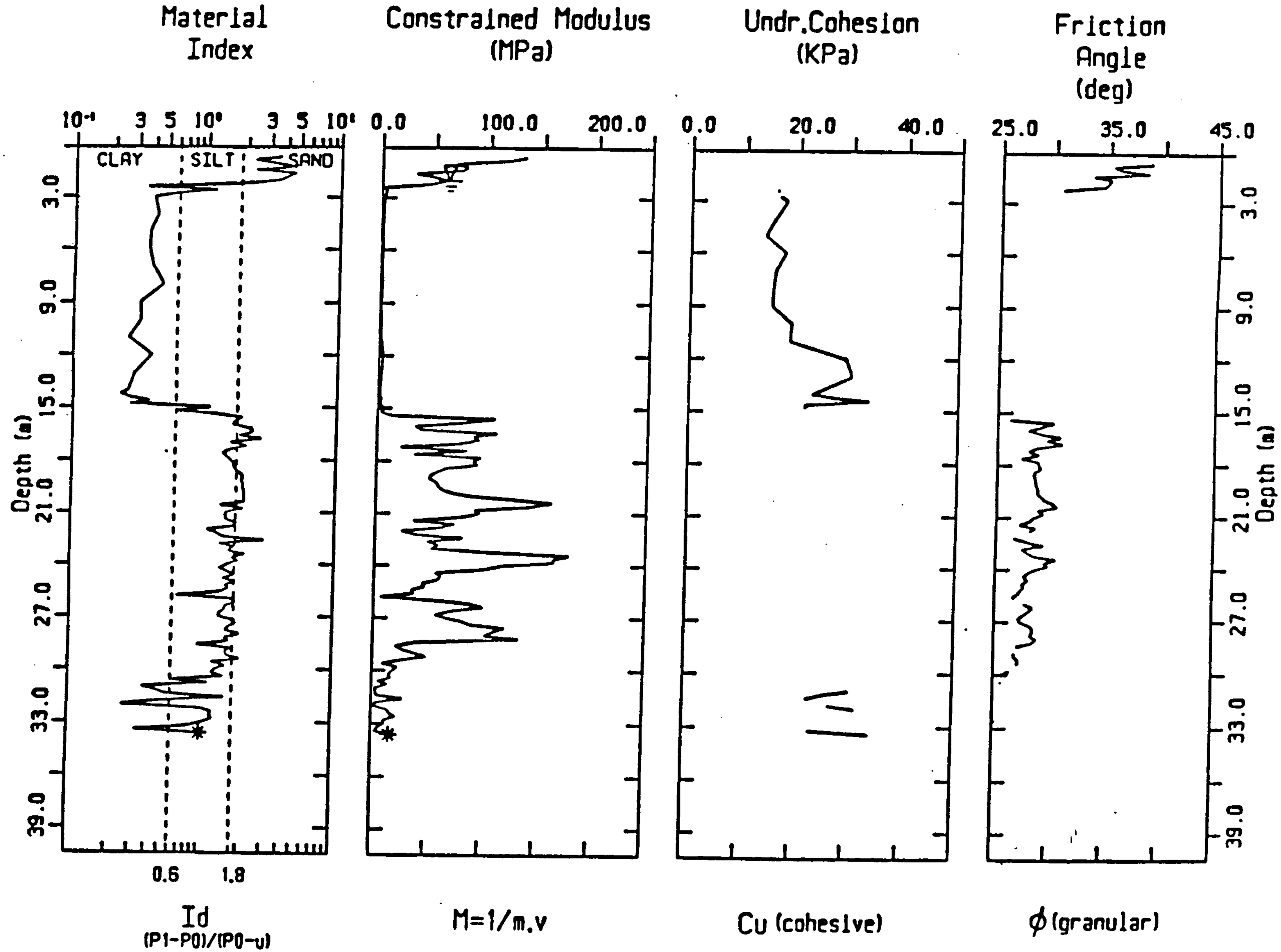


FIG. 4.9. INTERPRETED GEOTECHNICAL PARAMETERS FROM DMT UBCPRS/MOTHPRS

The results for the Nilcon Field Vane Test (NFVT86-1) are presented herein as this test was used indirectly in assessing the capacity of the axially loaded test piles. Fig. 4.10 presents the results of NFVT86-1 along with an estimate of undrained strength from the CPT (CPT85-1) using:

$$S_u = \frac{q_c - \sigma_{vo}}{N_k} \qquad (4.1)$$

where: $N_k = 15$

It is apparent from Fig. 4.10, excepting the material above 5 metres depth, that the undrained strengths estimated from CPT results agree well with measured in-situ NFVT values. The discrepancy in the upper 5 metres is due to the fibrous nature of the organics in this zone. These fibrous organics will have little effect on the CPT values but will cause the NFVT to record excessively high results. Also a spike in the NFVT profile at 13.5 metres is most probably due to a fine sand or silt lense not encountered at the location of CPT85-1. The locations of CPT85-1 and NFVT86-1 are shown in Fig. 4.2.

4.3 Summary

For this study, several in-situ testing methods were performed. In most cases, to determine site homogeneity and ensure instrument repeatability, test repetition has been performed. Only the results used in this study for the prediction of axial and lateral pile behaviour have been included in Appendix I.

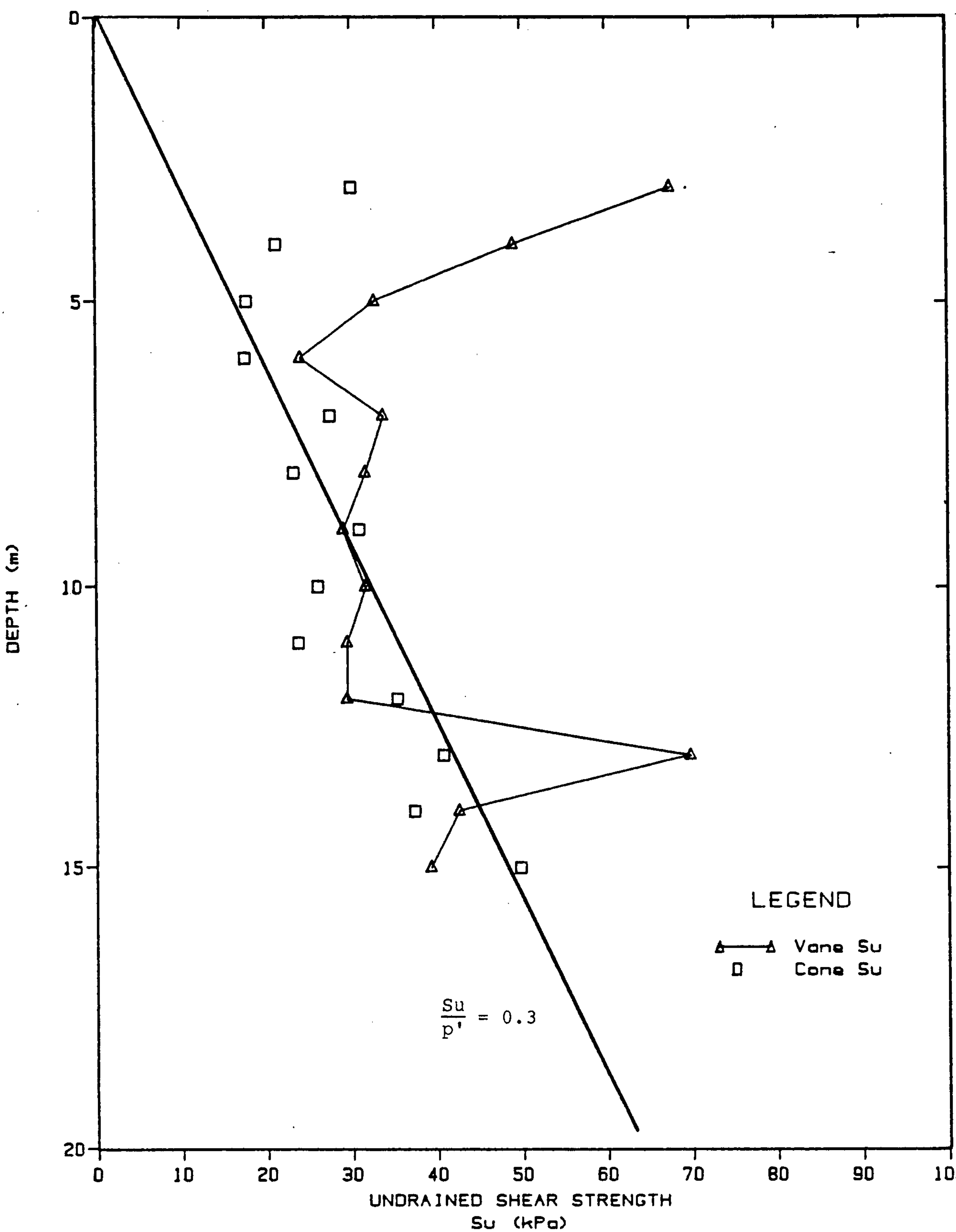


FIG. 4.10. UBC PILE RESEARCH SITE
UNDRAINED STRENGTH PROFILES

The collection of the data was greatly aided by the use of the UBC Geotechnical Research Vehicle (see Campanella and Robertson, 1981, for a detailed description of this vehicle). In all cases possible, ASTM (American Society for Testing and Materials) standard designation testing methods have been used. Where no standard designations were available, testing methods standard to the local geotechnical community were used.

CHAPTER 5

PILE INSTALLATION AND LOAD TESTING

In this chapter, the details of pile installation and the axial and lateral pile testing performed will be presented. More emphasis will be placed on describing the UBCPRS piles although a brief summary of work performed on the MOTHPRS is included.

As mentioned previously, all of the pre-planning, pile driving and pile load testing at the UBCPRS was done mainly by UBC personnel whereas UBC had little direct involvement with these areas for the MOTHPRS.

5.1 Pile Installation

Six piles were driven (four 324 mm dia., 9.5 mm wall thickness; one 324 mm dia., 11.5 mm wall thickness; one 610 mm dia., 11.5 mm wall thickness) at the UCBPRS. The five smaller (324 mm dia.) piles are the focus of this study. The larger (610 mm dia.) pile (pile no. 6) has been left for future instrumentation and testing. In addition, a seventh pile was driven at the UBCPRS to investigate the dynamic pile capacity. This pile will be discussed in Section 5.1.2.

At the MOTHPRS, one pile (915 mm dia., 19 mm wall thickness) was driven. The relative embedments of the five UBCPRS piles and the MOTHPRS pile are shown in Fig. 5.1. Note that pile no. 1 had a larger diameter sleeve for the first 2.5 m to remove any frictional resistance in the upper sand fill.

5.1.1 Driving Records

A summary of the driving records for the UBCPRS piles is shown in Table 5.1. Complete driving records can be found in Appendix II. All

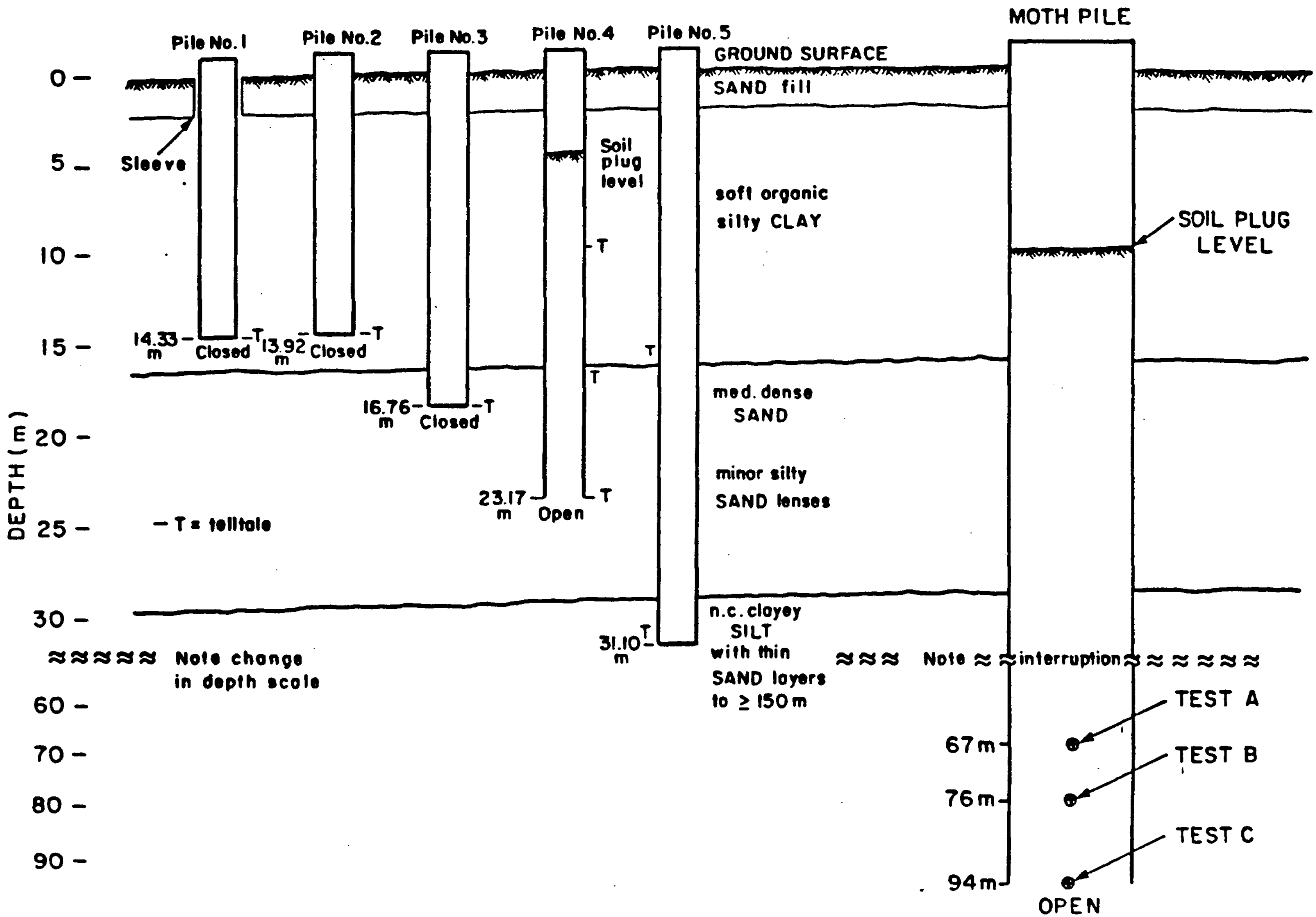


FIG. 5.1. UBC/MOTH TEST PILE EMBEDMENTS

Pile No.	Total Depth Feet (m)	Hammer Weight	Drop Height feet	Total No. of Blows	Driving Date
1	47' (14.33 m)	4,400 lb	∿4'	42	19 AUG 85
2	45' (13.72 m)	6,200 lb	∿3'	69	16 AUG 85
3	55' (16.76 m)	6,200 lb	∿4'	84	16 AUG 85
4	76' (23.17 m)	6,200 lb	∿5'	261	16 AUG 85
5	102' (31.10 m)	6,200 lb	∿6-7'	364	15/16 AUG 85
6	103' (31.39 m)	6,200 lb	∿10' max.	1512	14/15 AUG 85
7	94' (28.65 m)	3,500 lb	∿8'	1457	19 NOV 86

Table 5.1 UBC Pile Research Site Pile Driving Records Summary

piles were driven with a steel drop hammer using a metal helmut and plywood cushion. Piles 1,2,3 and 5 were driven closed-ended with the base-plate flush with the diameter of the piles, pile no. 4 was driven open-ended. Soil plug monitoring on pile no. 4 during driving was performed. After final driving, the top of the soil plug was 8.07 m below ground surface; thus the total length of the soil plug was 15.1 m.

No anomalies such as buckling, splitting or creasing of the piles were encountered during driving. After pile driving, all piles (except no. 4) were inspected for straightness and integrity by lowering a light to the bottom of the pile. In each case the piles were essentially straight and no structural defects were observed.

A summary of the driving records for the MOTHPRS pile is given in Eisbrenner (1985). The pile was driven initially using a 3400 kg drop hammer (average drop height 1.2 m) down to a depth of 19.9 m. Below this depth, the pile was driven using a Delmag D-62-22 single acting diesel hammer. The cap block used was alternating layers of aluminum and canvas reinforced phenolic resin. The pile was driven open-ended and the soil plug monitored. The pile was driven three times; initially to a depth of 67 m and later to 78 m and 94 m after axial load tests to failure had been performed. A more complete account of the MOTHPRS pile installation can be found in Eisbrenner (1985).

5.1.2 Dynamic Measurements

On each of the five UBCPRS piles, pile head acceleration and full-bridge strain gauge information was recorded during driving. This information was recorded using a pile driving analyzer (P.D.A.), Model EBA from Goble, Rausche and Likins (GRL) Associates, supplied by the B.C. M.O.T.H.

Significant difficulties were encountered during the collection of the PDA data. On two of the five piles, the strain gauges and/or the accelerometers became separated from the pile in spite of valiant attempts to protect this instrumentation. A general unfamiliarity with the equipment by the UBC and M.O.T.H. personnel contributed to the rather poor quality data being collected. Studies performed later by Mr. B. Miner (1986) using the data collected indicated a problem with the tape speed and instrument flutter which led to signal distortion. Table 5.2 summarizes the results of a visual review of the data during playback.

Upon further study of the data from pile nos. 2,3, and 4, no meaningful value of ultimate dynamic pile resistance could be calculated. Attempting to remove the undesirable frequencies using a Fast Fourier Transform did not improve the data sufficiently for successful analyses.

As mentioned earlier, an additional pile (pile no. 7) was driven at the UBCPRS. Table 5.1 provides a summary of the driving record. This pile was monitored using a different PDA (Model GC from GRL Associates) than was used for the original five piles. Again the PDA was supplied by the B.C. M.O.T.H. but an engineer from GRL (Mr. B. Miner) was also present. Pile no. 7 (324 mm dia., 11.5 mm wall thickness) was driven to 28.7 m closed

Pile No.	Remarks
1	some consistency in the data, force and velocity measurements not proportional
2	useful data
3	useful data
4	unreliable data
5	unreliable data
6	some consistency but generally unreliable data, force and velocity measurements not proportional

Table 5.2 UBC Pile Research Site PDA Summary

ended and was intended as a model of pile no. 5. Unfortunately, this is not the case because:

i) pile no. 7 was driven nearly 3 m short of the anticipated depth;

ii) the base plate was oversized and not flush with the outside of the pile.

During the time that pile no. 7 was driven, restrike data was also obtained from pile nos. 2,3 and 5. The results and discussion of interpretation of the restrike data on pile no. 5 can be found in Section 6.5.

For the MOTHPRS pile, dynamic monitoring was carried out by Trow Ltd., Whitby, Ontario. In addition to the PDA, CAPWAP analyses were also performed on the MOTH pile. A summary of the monitoring program can be found in B.C. M.O.T.H. report project D470E. A summary of the results of the PDA and CAPWAP analyses are also included within D470E.

5.2 Axial Load Testing

5.2.1 Introduction

For the UBCPRS, the axial pile testing program is summarized in Table 5.3. The driving dates are also included in order to ilustrate the amount of time between driving and pile testing. From the CPT pore pressure dissipation data, the maximum time for 90% of the excess pore pressure to dissipate (t_{90}) was equal to 30 minutes for measurements behind the cone tip. Comparing the 36 mm diameter cone to the 324 mm diameter pile would therefore yield t_{90} values of 2430 minutes (roughly 2 days) using the method outline by Gillespie (1980). Therefore, the CPT pore pressure dissipation data indicate that the time periods between pile driving and pile testing were sufficient to allow all excess pore pressures to dissipate.

Pile No.	Pile Length (m)	Driving Date(s)	Testing Date(s)
1	14.3	19 AUG 85	09 NOV 85
2	13.7	16 AUG 85	01 MAR 85
3	16.8	16 AUG 85	09 NOV 85
4	23.2	16 AUG 85	01 MAR 85
5	31.1	15 AUG 85	22 SEP 85
		16 AUG 85	06 OCT 85
6	31.4	14 AUG 85	NOT YET TESTED
		15 AUG 85	

Table 5.3 UBCPRS Pile Driving and Testing Schedule

The MOTHPRS pile was tested axially to failure when the tip was at depths of 67, 78 and 94 m below the ground surface (see Table 5.4). Calculations by Robertson et al. (1985), based on CPT pore pressure dissipation data, show that t_{90} for the 915 mm pile would be approximately

Test No.	Pile Length (m)	Driving Date(s)	Testing Date
A	67.0	10,11,13,16, 17 APR 84	09 MAY 84
B	78.0	11 MAY 84	01 JUN 84
C	94.0	09 JUN 84	29 JUN 84

Table 5.4 MOTHPRS Pile Driving and Testing Schedule

20 days. This may indicate that the load test values may be slightly affected by transient excess pore pressures as 21 days was taken as the testing interval. The testing sequence was as follows:

i) drive pile to 67 m

ii) wait 21 days

iii) axial load test to failure (Test A)

iv) drive pile to 78 m

v) wait 21 days

vi) axial load test to failure (Test B)

v) drive pile to 94 m

viii) wait 21 days

ix) axial load test to failure (Test C)

5.2.2 Methodology

For the UBCPRS, the "Quick Load Test Method" of axial loading (similar to ASTM D1143-81 Section 5.6) was used with the axial load being applied in roughly 5% increments of the anticipated failure load. The 'Quick Load Test Method' was used to minimize the time-dependent effects in the cohesive soils. The axial load was measured using a 500,000 lb calibrated electronic load cell. The reaction loads on the remaining piles were measured with smaller load cells. Details on the loading system used (e.g.

pump type) and calibration data for the 500,000 lb load cell are given in Appendix III. The deflections were measured by multiple dial gauge installations. A level survey was also conducted but proved to be less sensitive than the dial gauges.

The load test set-ups used for testing the UBCPRS piles are shown in Figs. 5.2 and 5.3. These figures show the set-up used for pile no. 5 and for the four perimeter piles, respectively.

The MOTHPRS pile was also tested using the 'Quick Load Test Method". The load test arrangement is shown schematically in Fig. 5.4. Further details can be found in Robertson et al. (1985) and in Eisbrenner (1985).

5.2.3 Results

Analysis of the results from axially loaded vertical test piles is more complicated than generally realized (Brierley et al., 1978). For a pile (generally assumed to be stronger than the soil), the ultimate failure load is reached when the pile plunges; rapid settlement under sustained or only sligthly increased load. This definition, however, is often inadequate because plunging requires very large displacements and is often less a function of the pile-soil system and more a function of the capacity of the man-pump system (Fellenius, 1980). To be useful, a failure definition should be based on a simple mathematical rule that can generate repeatable results independent of the individual using the method and of the scale relations chosen for plotting the load test data. For example Fig. 5.5 show the results of a hypothetical pile load test plotted to different scales. The hypothetical test pile could be interpreted, based on a visual inspection of results, as a predominately friction or 'floating' pile (upper figure) or a predominately end bearing pile (lower figure). The

SCALE 1:25

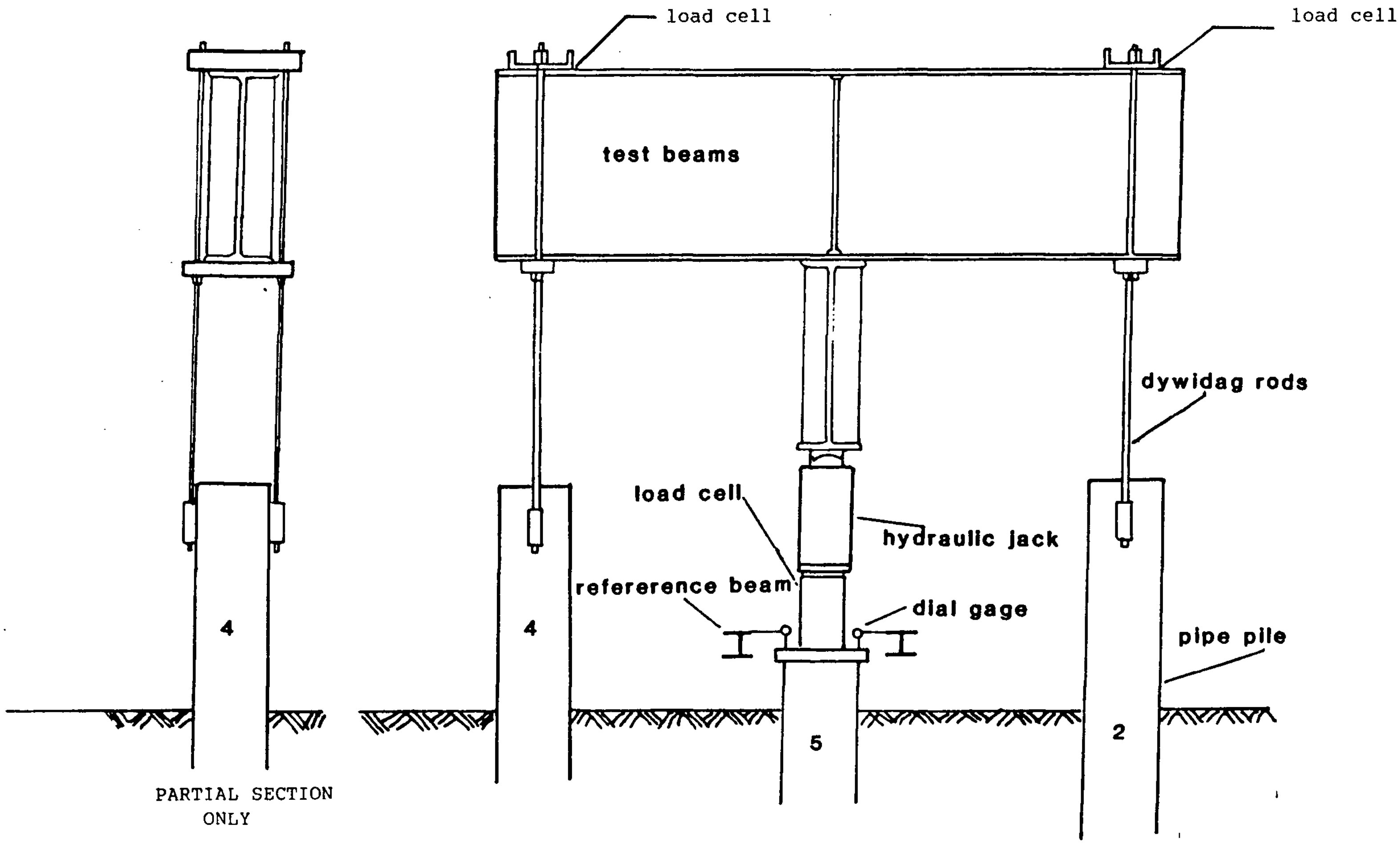


FIG. 5.2. AXIAL PILE LOAD TEST SET UP FOR PILE NO. 5

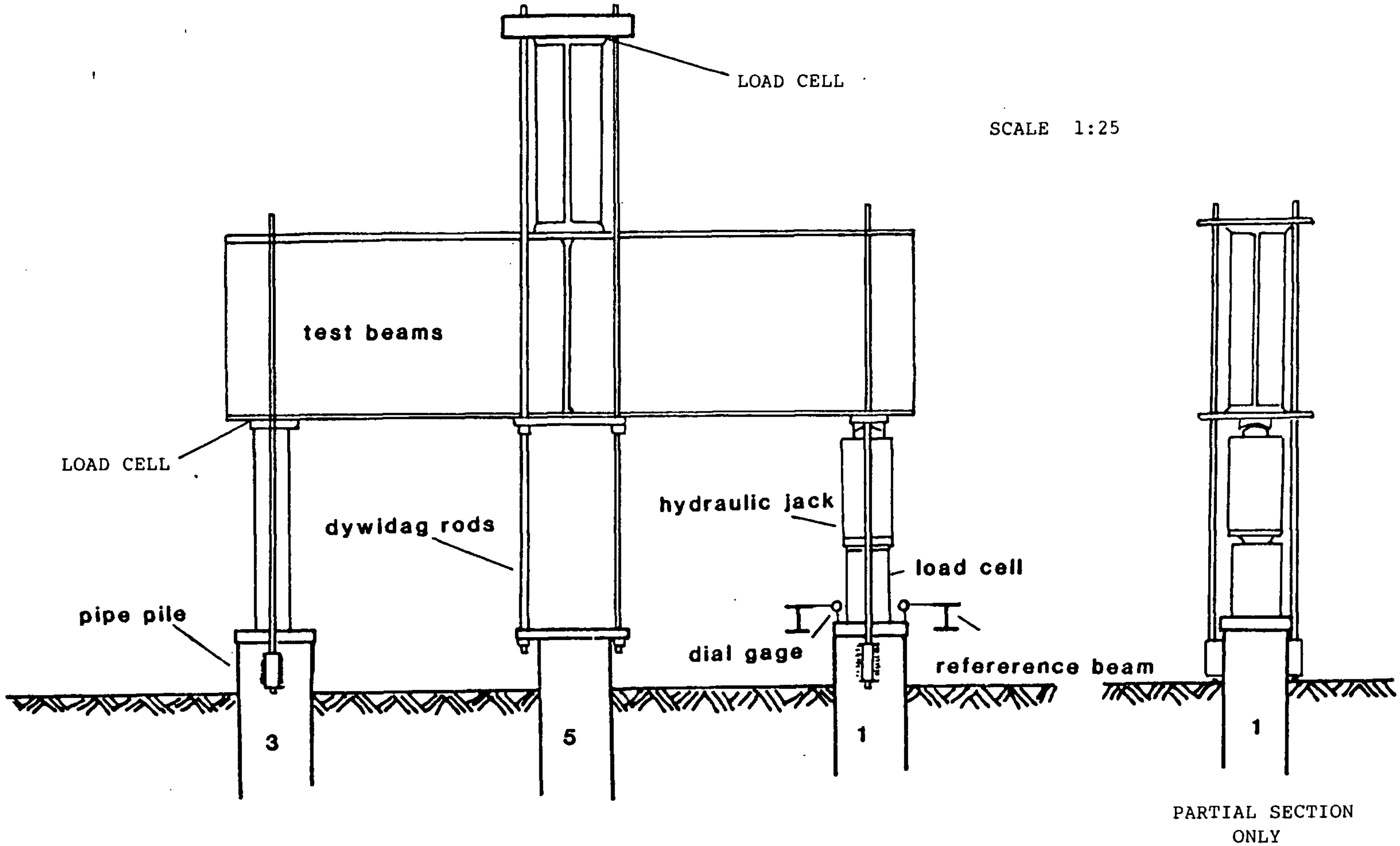


FIG. 5.3. TYPICAL AXIAL PILE LOAD TEST SET UP FOR PILES 1 TO 4 (INCLUSIVE)

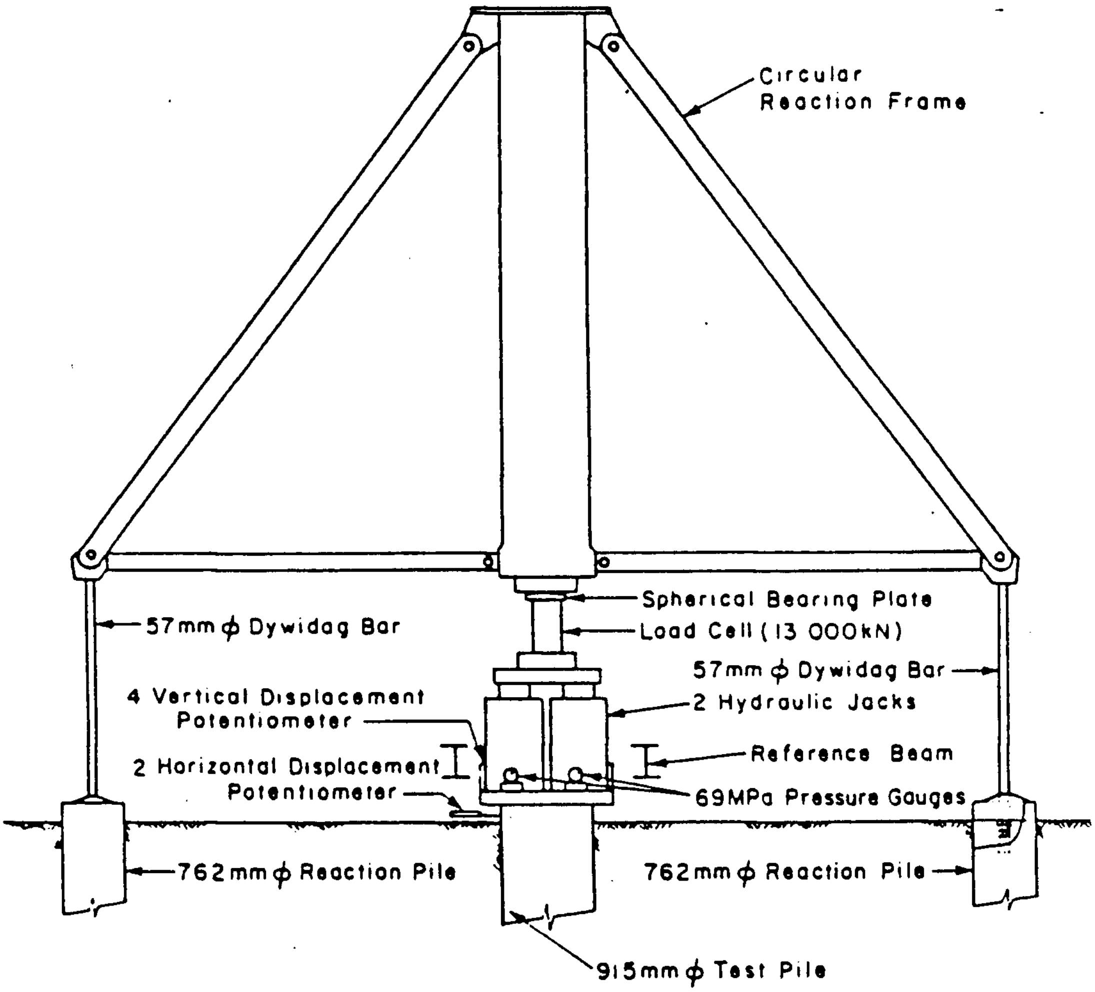


FIG. 5.4. MOTHPRS AXIAL PILE LOAD TEST SET UP
(After Robertson et al., 1985)

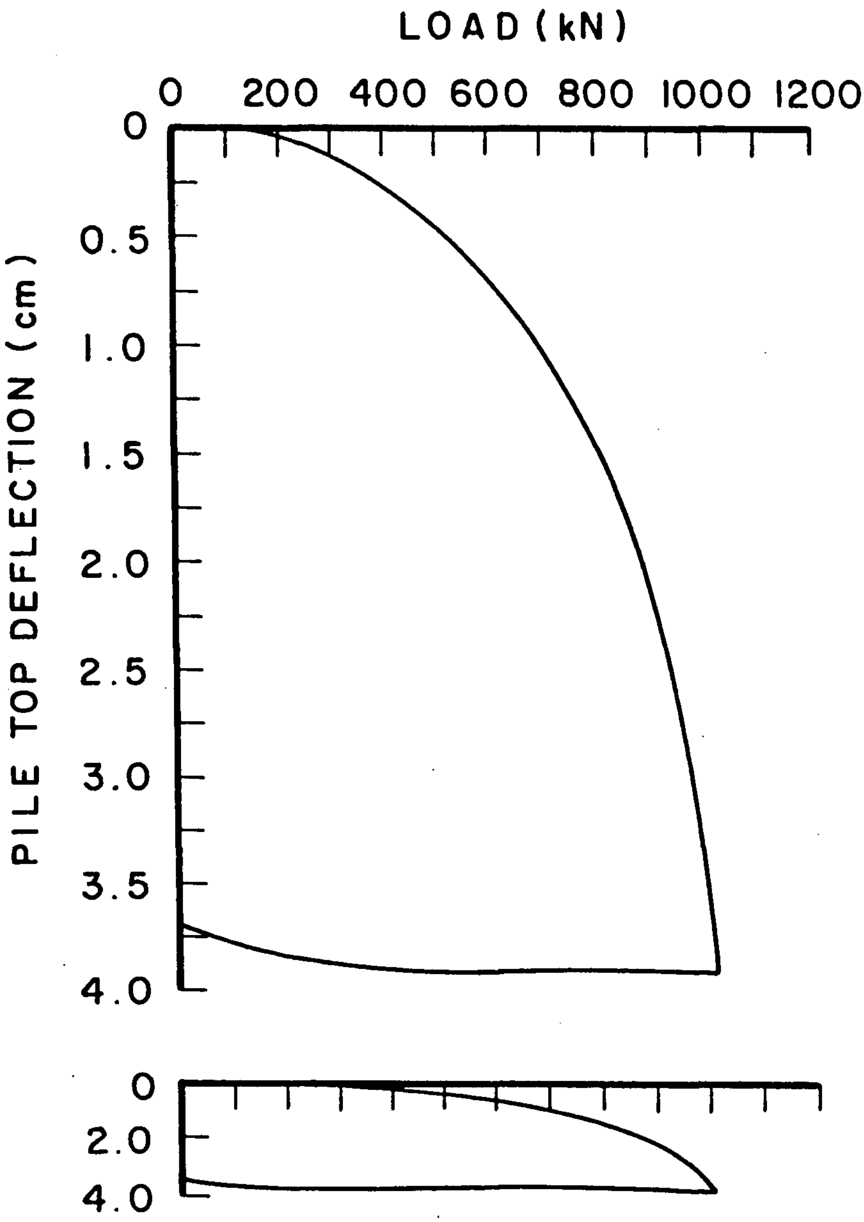


FIG. 5.5. LOAD-DISPLACEMENT DIAGRAM OF A HYPOTHETICAL TEST PILE DRAWN TO TWO DIFFERENT SCALES

method recommended by the Canadian Foundation Engineering Manual (1985, Part 3, Subsection 22.5.1) is that by Davisson (1973) and involves a simple graphical manipulation of the theoretical elastic compression line for the pile in question, (the calculation of the theoretical elastic compression for the UBCPRS piles is included in Appendix III). Davisson's method (1973) has been used in this study to determine failure loads. Fellenius (1980) studied nine commonly used failure criteria and found Davisson's method to be among the most conservative.

Figs. 5.6 through 5.10 present the axial load-displacement test results for the UBCPRS. For each of the five piles complete load-deflection-time records of the testing are shown. The pile top deflection for each pile was taken as an average of two diametrically opposed dial gauges at the pile head. The following are some specific comments about each pile:

i) Pile no. 1 (Fig. 5.6) exhibits unexpected large deflection at low loads such that the theoretical compression line is crossed beyond the first load increment. One possible explanation of this behaviour is that pile No. 1 is cased over the upper 2 m and therefore unrestrained compression can occur near the pile head. But this would not explain large movement at low loads. The "theoretical compression line" is for a pile with no shaft resistance (i.e. a column). Possibly the large movement was related to previous failure in tension, and therefore an unusual load distribution. The overall pile behaviour indicates that it is predominantly a friction pile.

ii) Pile no. 2 (Fig. 5.7) is seen as a predominantly friction pile.

iii) Pile no. 3 (Fig. 5.8) is seen as having both friction and end-bearing components to the total resistance. The slope of the unload-reload

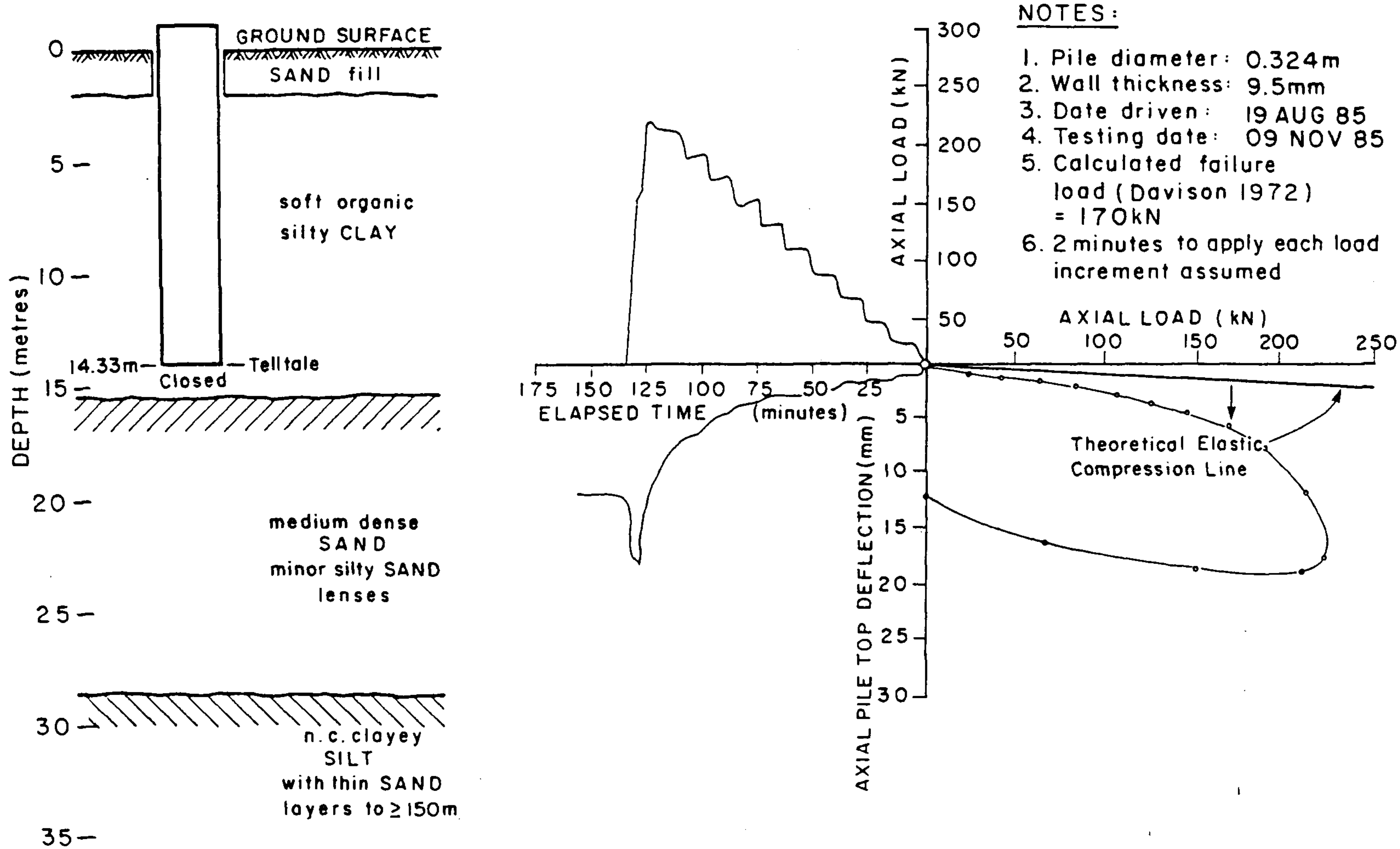


FIG. 5.6. UBC PILE RESEARCH SITE: AXIAL LOAD TEST RESULTS - PILE NO. 1

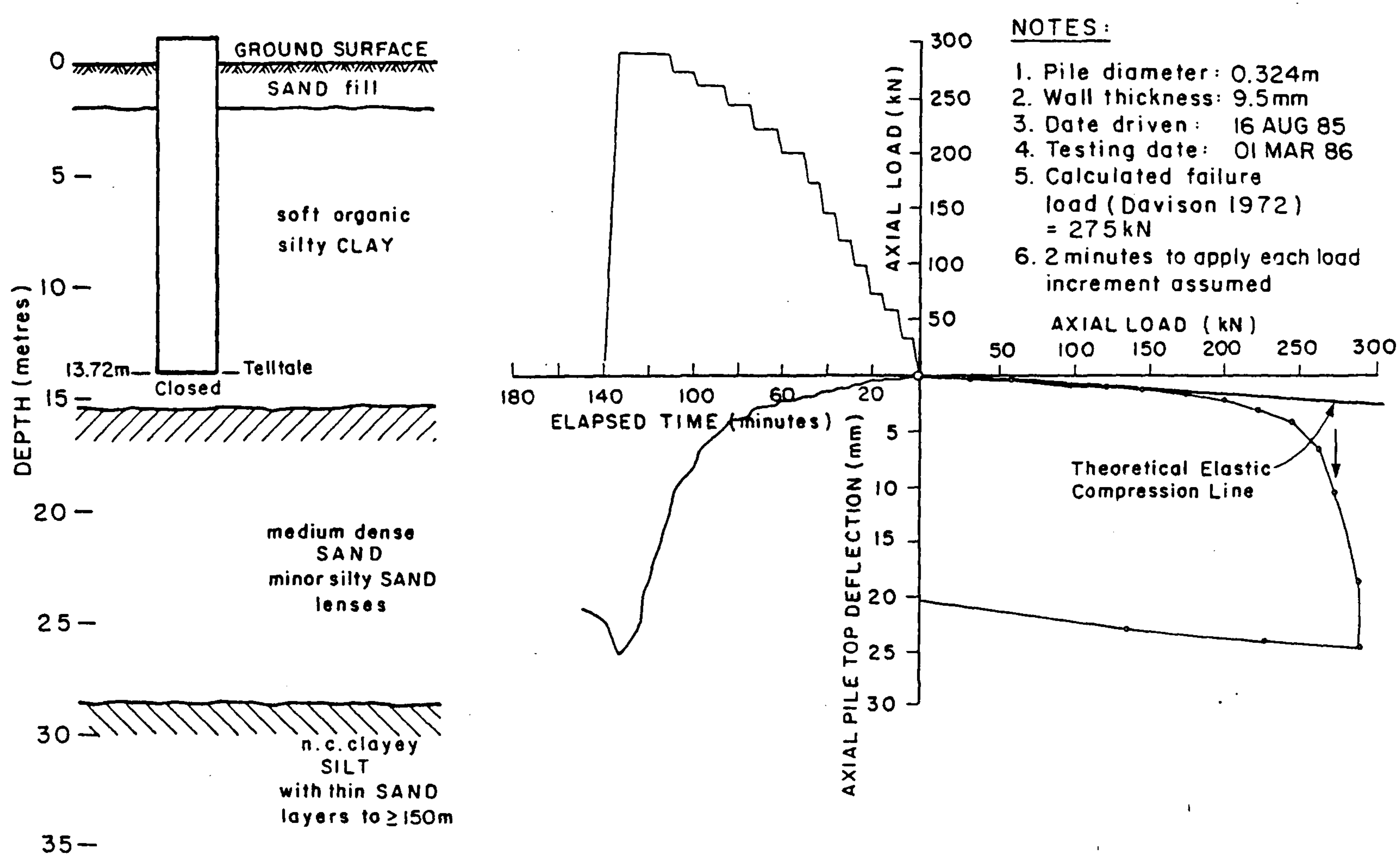


FIG. 5.7. UBC PILE RESEARCH SITE: AXIAL LOAD TEST RESULTS - PILE NO. 2

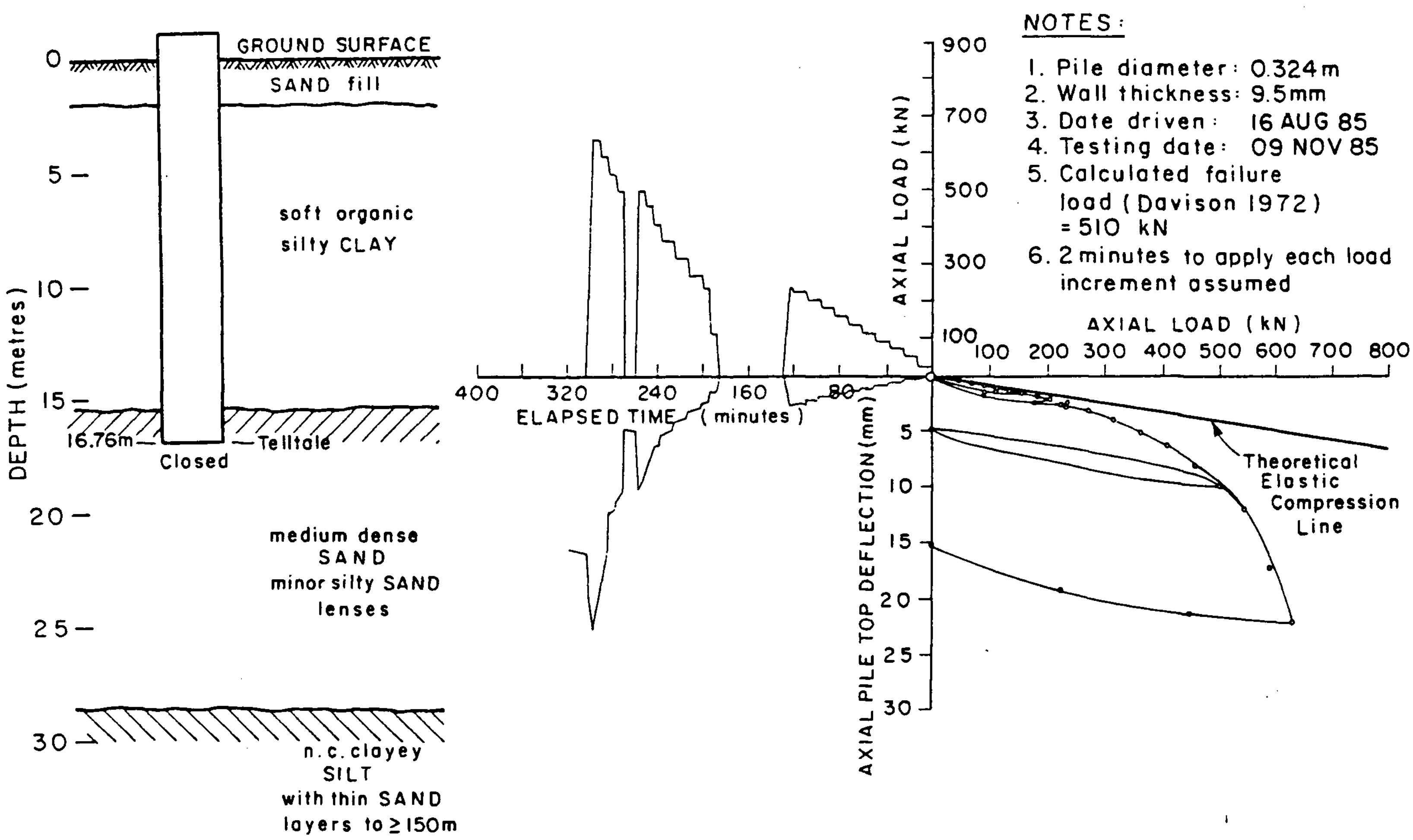


FIG. 5.8. UBC PILE RESEARCH SITE: AXIAL LOAD TEST RESULTS - PILE NO. 3

curves are seen to be approximately parallel the theoretical elastic compression line.

iv) Pile no. 4 (Fig. 5.9) could not be failed in axial compression loading. The reaction frame could not supply the necessary axial force before it began to buckle. The failure load was interpolated as described later in this section.

v) Pile no. 5 (Fig. 5.10) failed predominantly as a friction pile. The plunging nature of the failure is easily observed.

Fig. 5.11(a) presents a summary of the five UBCPRS pile load tests to the same scale. Fig. 5.11(b) presents a summary of the load-displacement results for the three tests on the MOTHPRS pile. The results from the MOTHPRS axial load tests indicate that the pile behaved predominantly as a friction pile. The reduction in measured load observed occurred because, with rapid axial deflections, the hydraulic jacks were unable to sustain the load. Full details of the test program for the 915 mm (MOTHPRS) pile is given by Robertson et al. (1985).

Besides the pile head load-deflection data, extensive tell-tale data was also obtained for the UBCPRS piles. By definition, a tell-tale is a device used to measure the deflection at locations along the pile length other than at the pile head. From tell-tale data it can be possible to estimate the load distribution as well as to infer the load transfer mechanism present, but the interpretation of this data is often difficult because of the complex distribution of residual stresses after driving (Fellenius, 1980). The location of tell-tale placements for the piles is shown in Fig. 5.1. Fig. 5.12 presents a schematic outline of the tell-tale system used for the five piles. Fig. 5.13 presents a schematic concept of

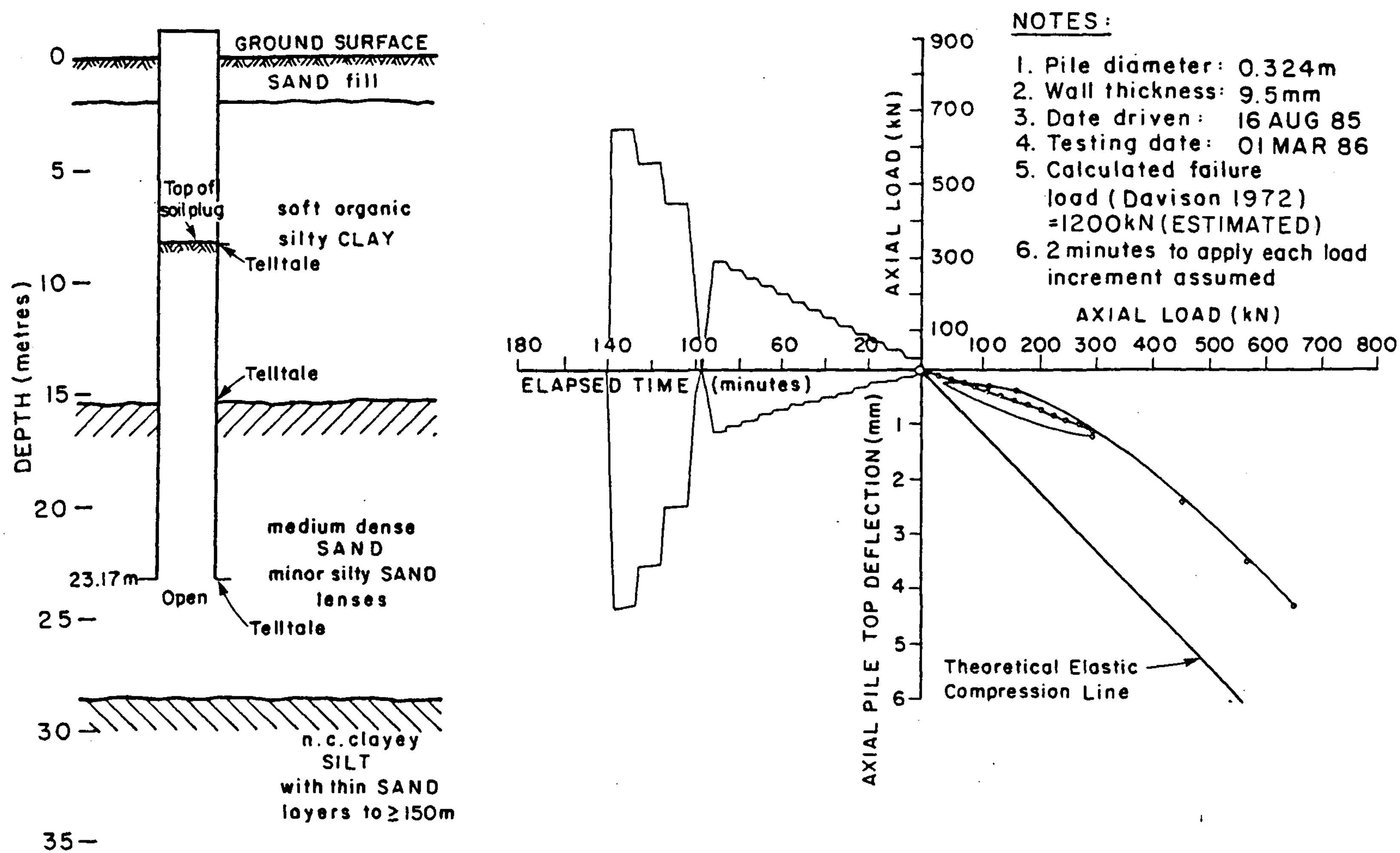


FIG. 5.9. UBC PILE RESEARCH SITE: AXIAL LOAD TEST RESULTS - PILE NO. 4

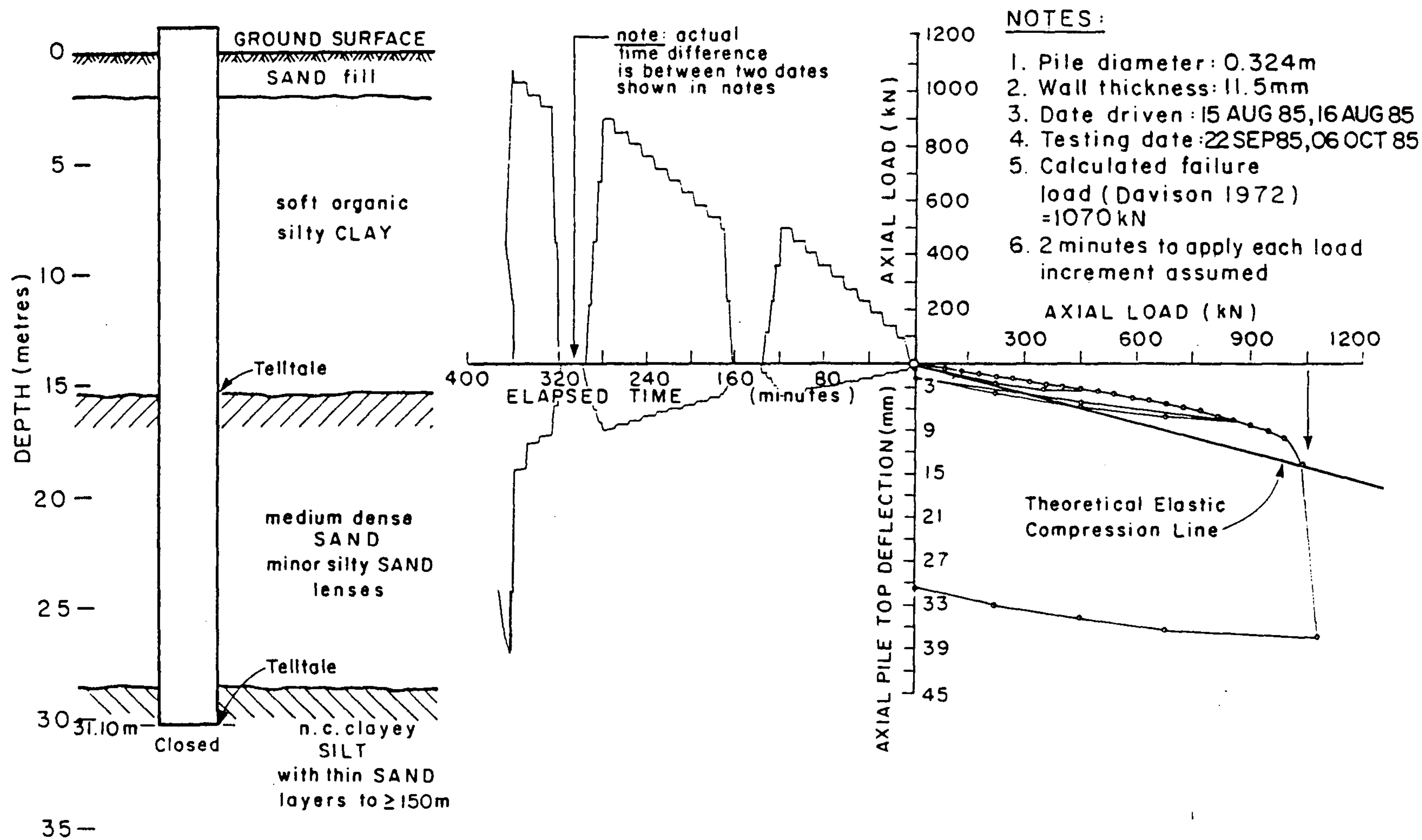
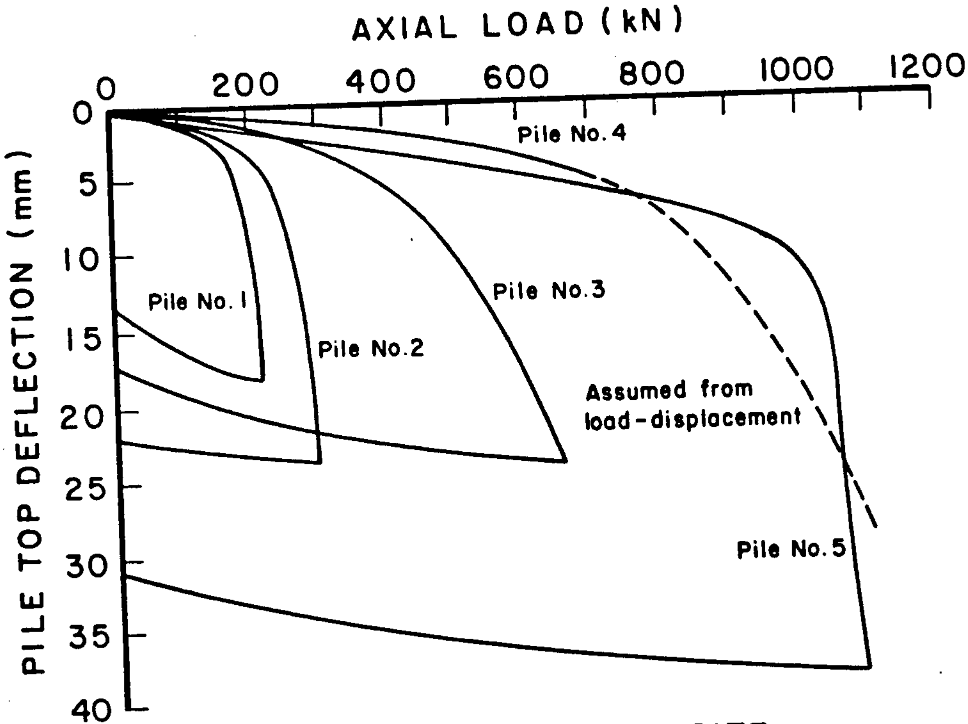
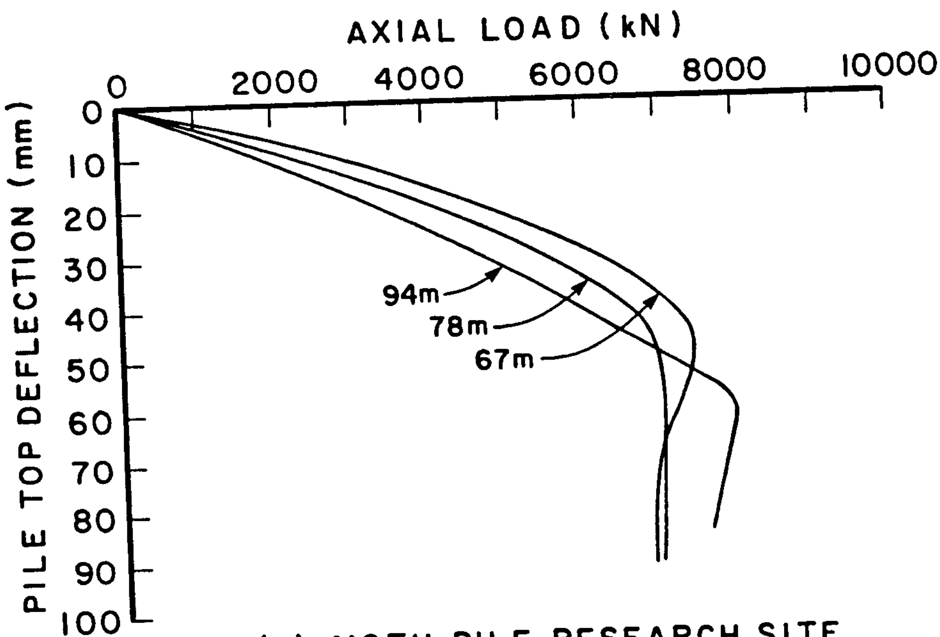


FIG. 5.10. UBC PILE RESEARCH SITE: AXIAL LOAD TEST RESULTS - PILE NO. 5



(a) UBC PILE RESEARCH SITE
AXIAL LOAD TEST RESULTS



(b) MOTH PILE RESEARCH SITE

FIG. 5.11. SUMMARY OF PILE LOAD TEST RESULTS

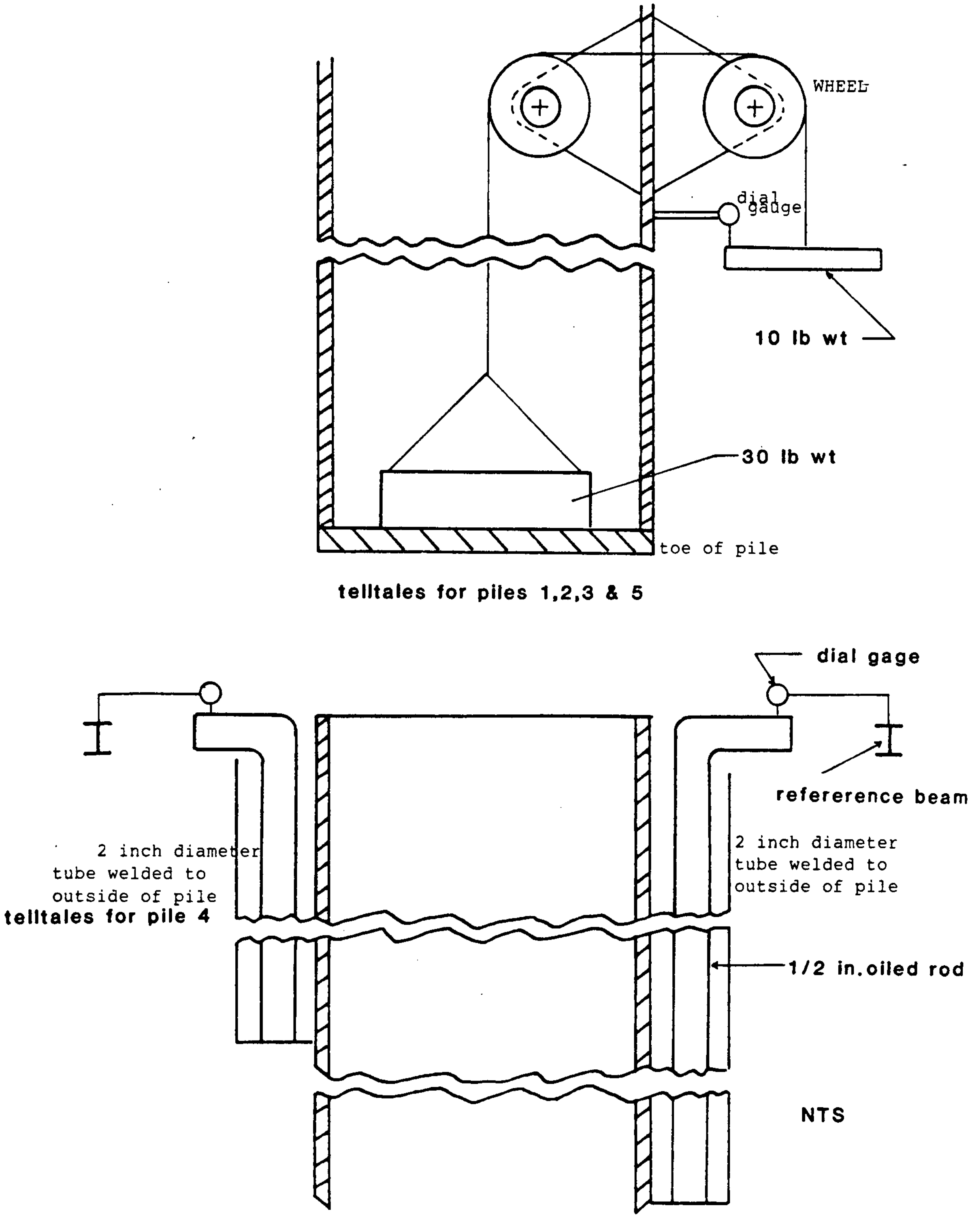


FIG. 5.12. UBCPRS TELLTALE INSTALLATIONS

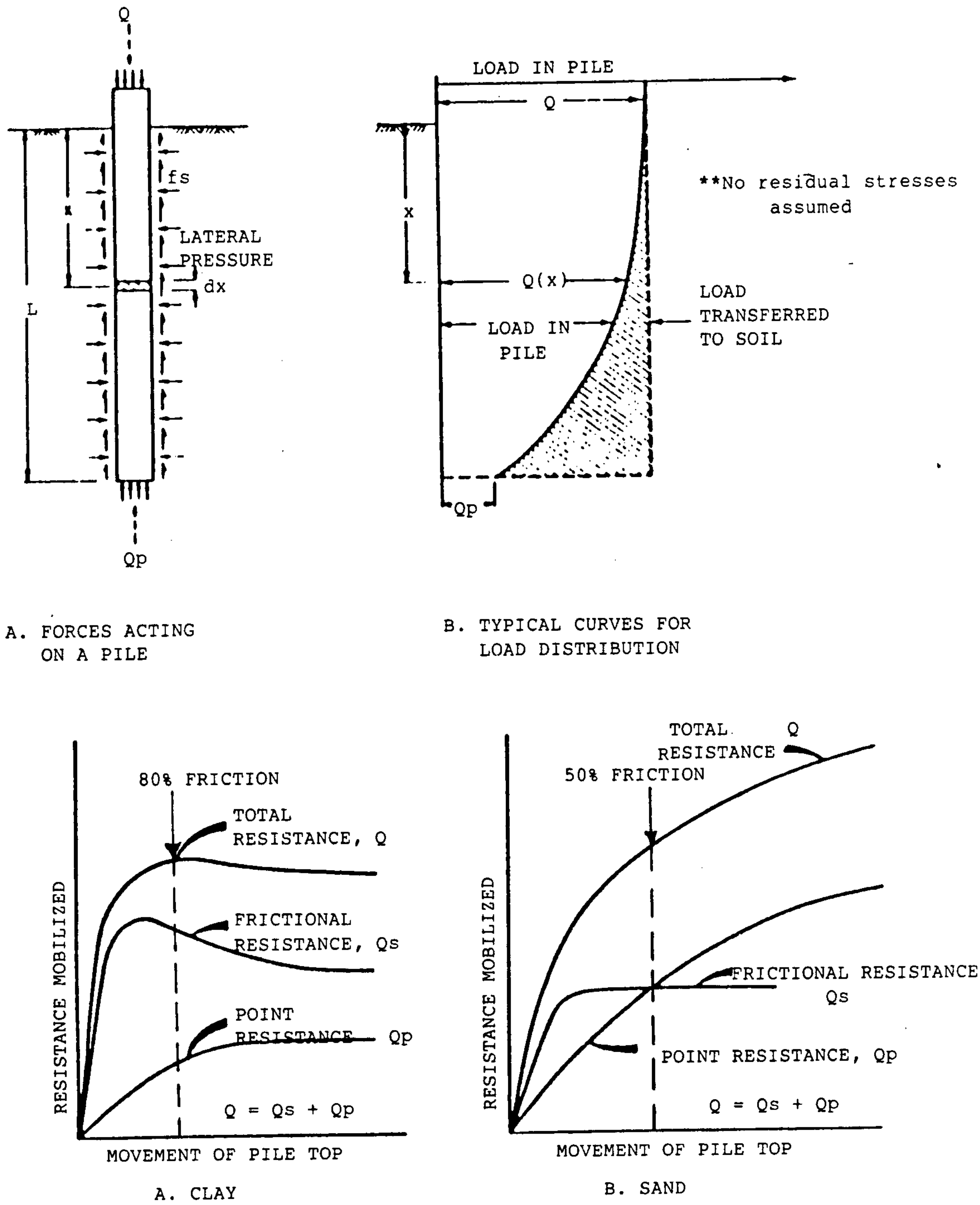


FIG. 5.13. SCHEMATIC CONCEPT OF LOAD-TRANSFER

typical load transfers for axially loaded piles. Note that the peak values of shaft resistance and point resistance do not mobilize simultaneously as many traditional static capacity formulae imply. In other words before the toe of the pile feels the effect of the applied axial load, significant axial deflection must occur at the pile head in most cases. Also, as is seen in Fig. 5.13, the load transfer mobilization relationships depend upon the soil type(s) present. The tell-tale data obtained presented several problems for interpretive purposes, possibly because of the complex loading history for piles 1, 2, 3 and 4. The tell-tale data from pile no. 1 was ultimately regarded as of being little use. Piles 2,3 and 5, however, provided data from which interpretations could be made. Pile 4, because it wasn't failed, provided only data on the soil plug behaviour. The ratio of end bearing to skin friction for piles 2, 3 and 5 piles, as determined from tell-tale data, is shown in Table 5.5. Fig. 5.14 shows a summary plot of the tell-tale data for pile no. 5.

Pile	Ratio of Toe: Shaft Resistance
2	30:70
3	50:50
5	20:80

Table 5.5 Summary of Tell-Tale Data for UBCPRS

As mentioned previously, pile no. 4 was never failed under the application of axial load. This was the only UBCPRS pile to be driven open ended. Two methods have been used to extrapolate the failure load The first method was the method developed by Chin (1970). Chin (1970) proposed a computational method whereby the estimation of the ultimate load of piles

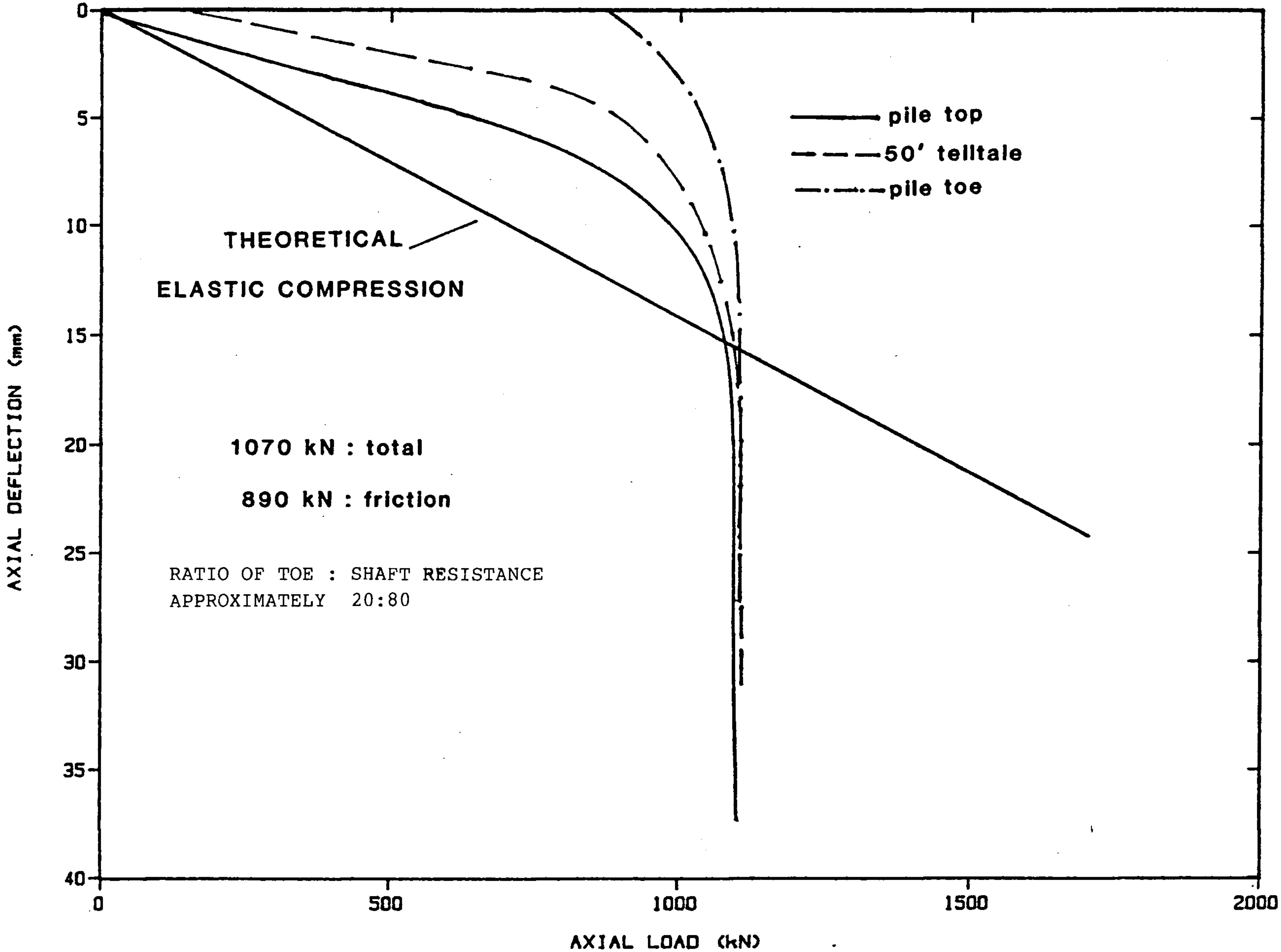


FIG. 5.14. UBC PILE RESEARCH SITE: PILE NO. 5
TELLTALE SUMMARY

not carried to failure can be made by plotting the trend of the normalized load-deflection data. In order to test the method, the test data from pile no. 5 was also analyzed. Fig. 5.15 shows Chin's method plotted for both pile nos. 4 and 5. For pile no. 5 the method estimates 1100 kN, approximately 30 kN larger than the Davisson failure load (1070 kN) obtained from load testing. For pile no. 4 Chin's method predicts 1100 kN.

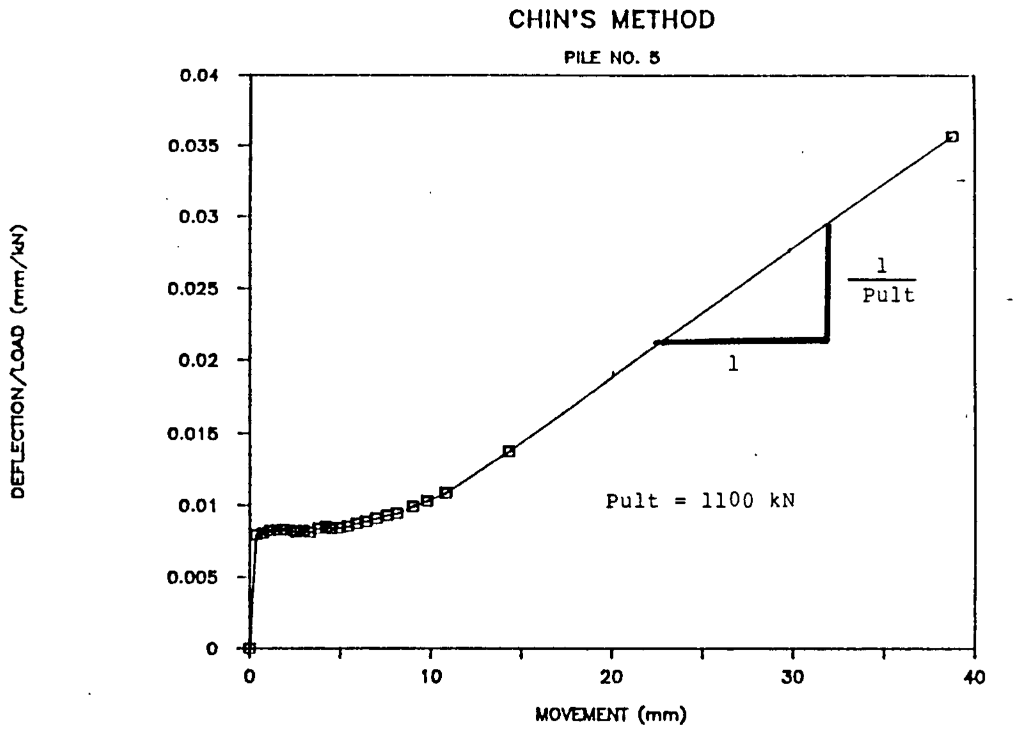
The second method of estimating the failure load of pile no. 4 was by using the shape of the load deflection curves from the other 4 piles. Each pile, being of different lengths, has different components of resistance due to the varying lengths. By assuming that the soil acted the same on all piles at any given depth, the load deflection technique could be applied. One assumption made is that pile no. 4 behaved as a closed-ended pile under static loading. From the tell-tale data taken from the soil plug this assumption appears valid. However, it must be noted that at higher loads the pile may have unplugged. Calculations, however, suggest that this would not be the case. For all calculations carried out for this study, it was assumed that the pile would not unplug at loads up to failure. This second method predicts the failure load of pile no. 4 to be 1250 kN. Therefore, roughly averaging both methods, the failure load of pile no. 4 was assumed to be 1200 kN.

A summary of all calculated capacities from the axial load testing is presented in Table 5.6.

5.3 Lateral Load Testing

5.3.1 Introduction

For the UBCPRS, the lateral pile testing program consisted of load testing one of the 9.5 mm walled pile (pile no. 3) and the larger 11.5 mm



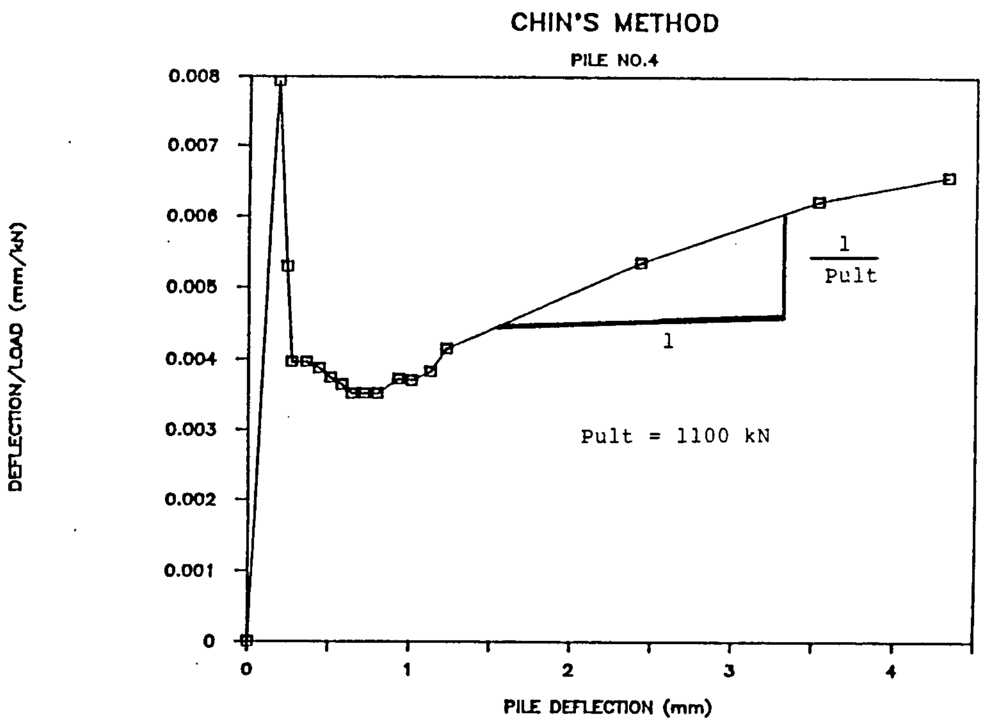


FIG. 5.15. UBCPRS: CHIN'S METHOD TO PREDICT FAILURE LOAD FOR PILE NOS. 4 AND 5

Pile/Test No.	Length (m)	Diameter (m)	Wall Thickness (mm)	L/D	Open/Closed Ended	Capacity (kN)
1	14.3	0.324	9.5	44	C	170
2	13.7	0.324	9.5	42	C	220
3	16.8	0.324	9.5	52	C	610
4	23.2	0.324	9.5	72	O	1200
5	31.1	0.324	11.5	96	C	1070
A	67.0	0.915	19	73	O	7500
B	78.0	0.915	19	85	O	7000
C	94.0	0.915	19	103	O	8000

Table 5.6 Summary of Axial Pile Load Testing at UBCPRS and MOTHPRS

walled pile (pile no. 5). The MOTHPRS pile had also been tested under lateral loading.

5.3.2 Methodology

For the UBCPRS the lateral loading was achieved by jacking between adjacent piles. In this manner two piles were tested at one time. The lateral loads were applied in increments of 20 kN and held for approximately 15 minutes to allow time for readings to be taken. These readings consisted of dial gauge and inclinometer readings. The dial gauge readings were checked by the use of LVDTs (Linear Voltage Displacement Transducer) on the two test piles (pile nos. 3 and 5). A schematic of the load set up is shown in Fig. 5.16. A schematic of the inclinometer casing set-up is shown in Fig. 5.17. The deflection of adjacent piles at ground surface was also measured but, due to measurement resolution difficulties, these values are not considered reliable. The lateral load was measured using a calibrated load cell. Calibration data for the load cell used is given in

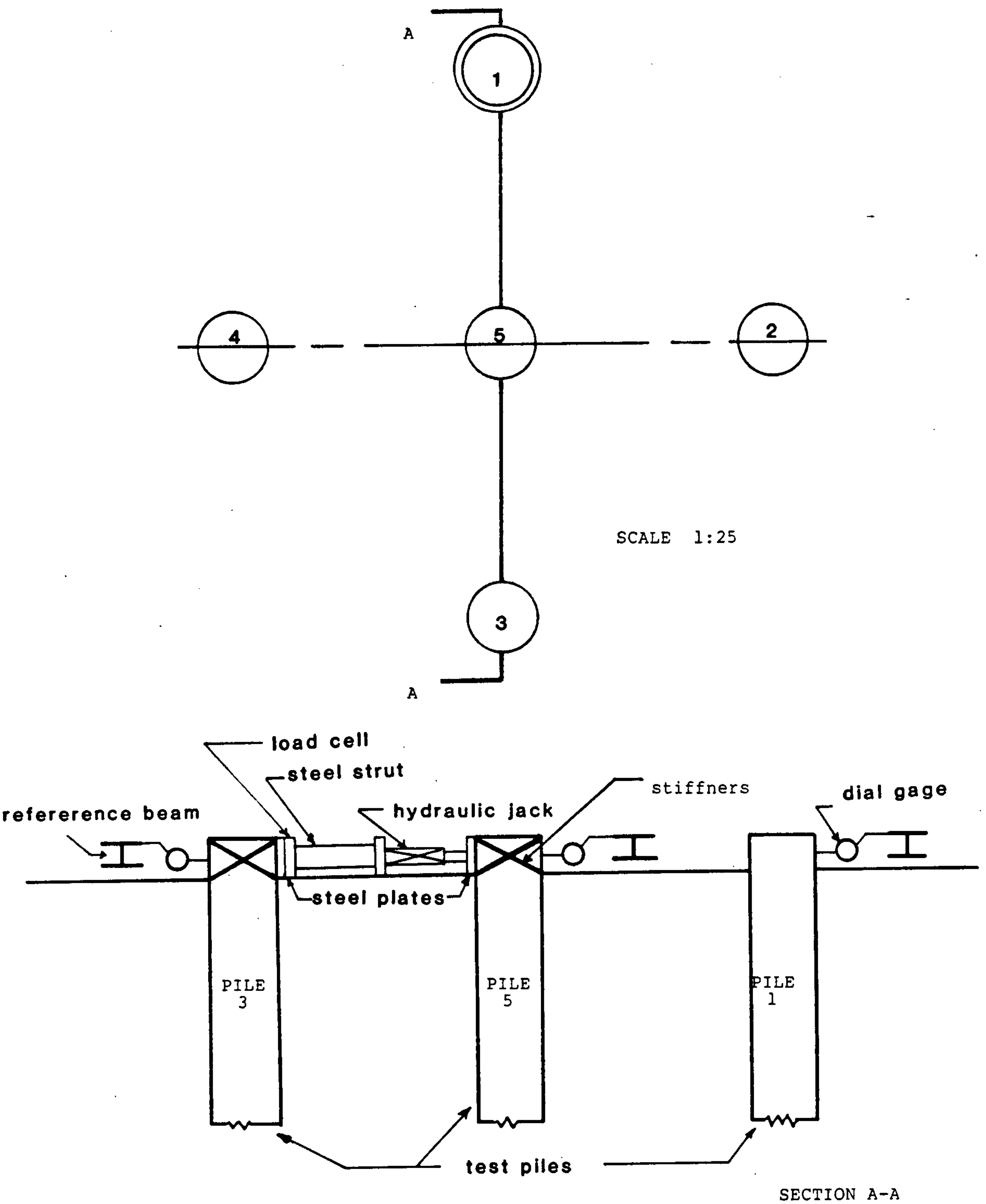


FIG. 5.16. UBC PILE RESEARCH SITE: LATERAL LOAD TEST SET UP

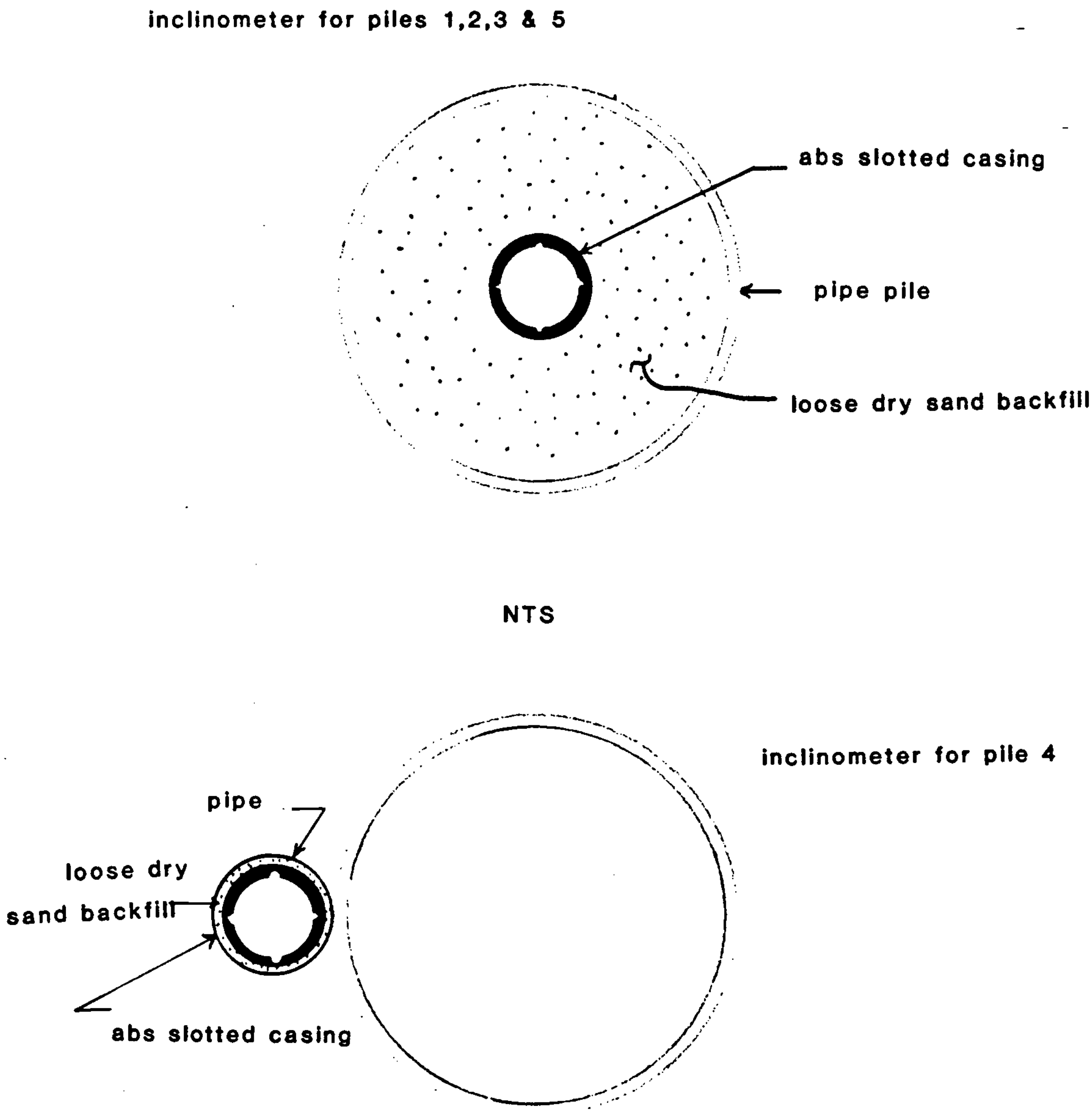


FIG. 5.17. UBC PILE RESEARCH SITE: INCLINOMETER SET UP FOR LATERAL LOAD TESTING

Appendix IV. Stiffners were placed in both piles in order to prevent possible buckling of the piles at the points of load application.

The MOTHPRS pile was loaded as shown in Fig. 5.18. Further details can be found in Robertson et al. (1985) and in Eisbrenner (1985).

5.3.3 Results

Unlike the axial load case, no standard method of interpreting lateral load test results exists. The effects of creep (time effects) can be very pronounced during lateral pile testing. Until standardization of testing is realized, it will remain difficult to compare results between different researchers and hence, difficult to confidently use design methods based on correlations with load test data.

The results of the UBCPRS lateral load tests are shown in Figs. 5.19 and 5.20. In Fig. 5.19 the ground surface deflection and deflected shape versus depth profile is presented for pile no. 3. In each case, any creep present driving any 15 minute load increment has been encorporated in the plots. A maximum deflection at the ground surface of approximately 30 mm is measured under the peak lateral load of 140 kN. Note that 30 mm is nearly 20% of the pile radius and thus would probably be larger than most maximum design deflections. The deflected shape profile for a load of 120 kN indicates that the depth of the first point of contraflexure is at a depth of approximately 3 metres (approximately 9 pile diameters) and that below this point almost no further deflection is evident. For pile no. 5, as shown in Fig. 5.20, the maximum ground surface deflection, under the lateral load of 140 kN, was approximately 22 mm. The deflected shape profile for a load of 120 kN, also shown in Fig. 5.20, indicates that the first point of contraflexure is at a depth of roughly 3.5 metres

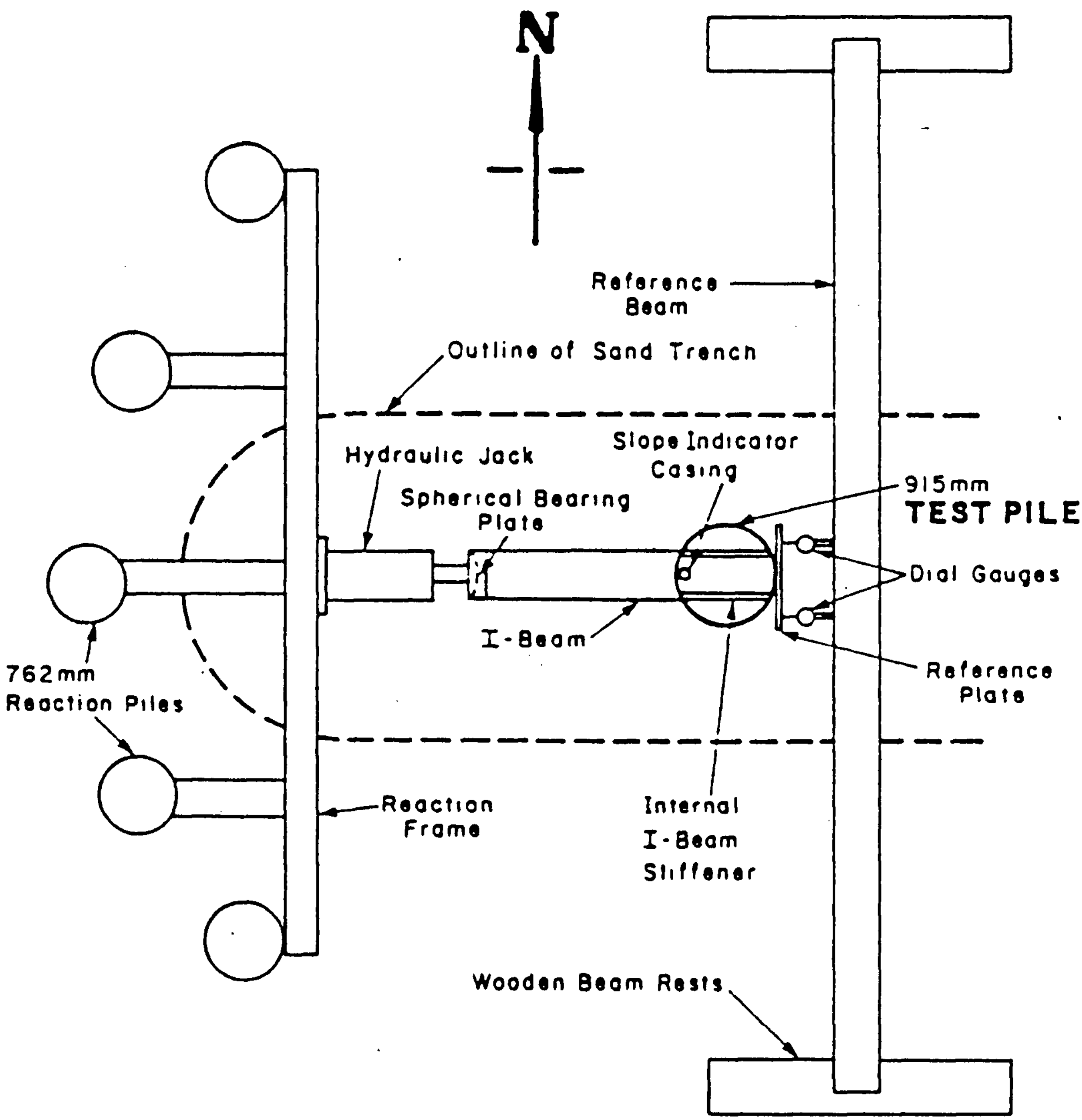
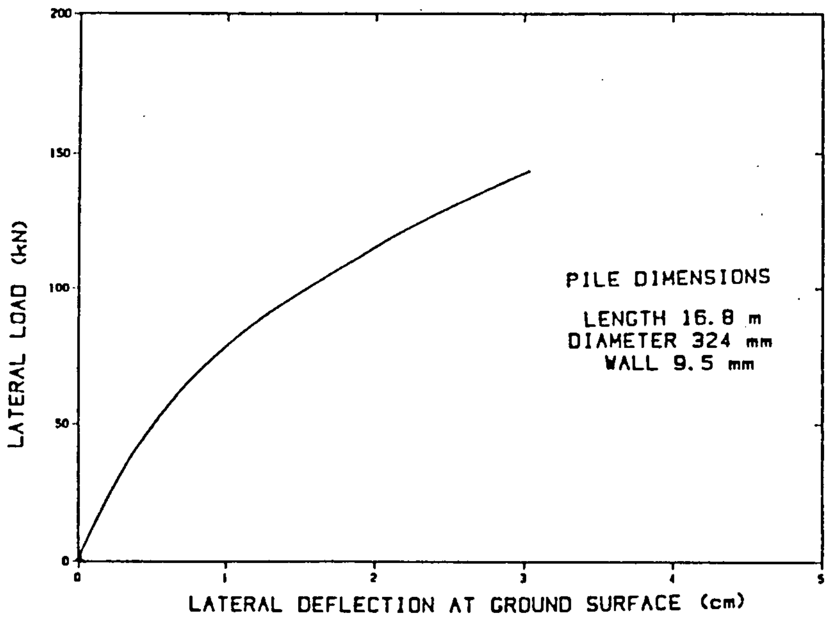


FIG. 5.18. MOTHPRS LATERAL PILE LOAD TEST ARRANGEMENT
(After Robertson et al., 1985)



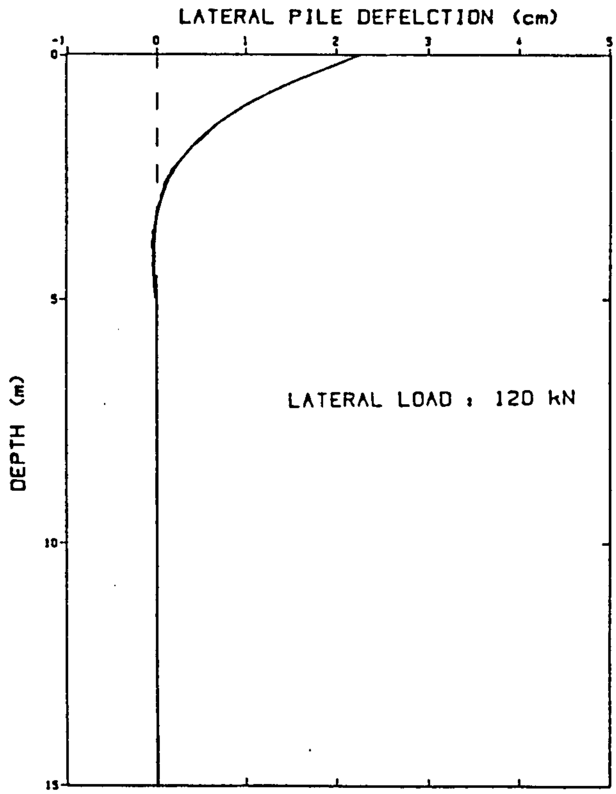
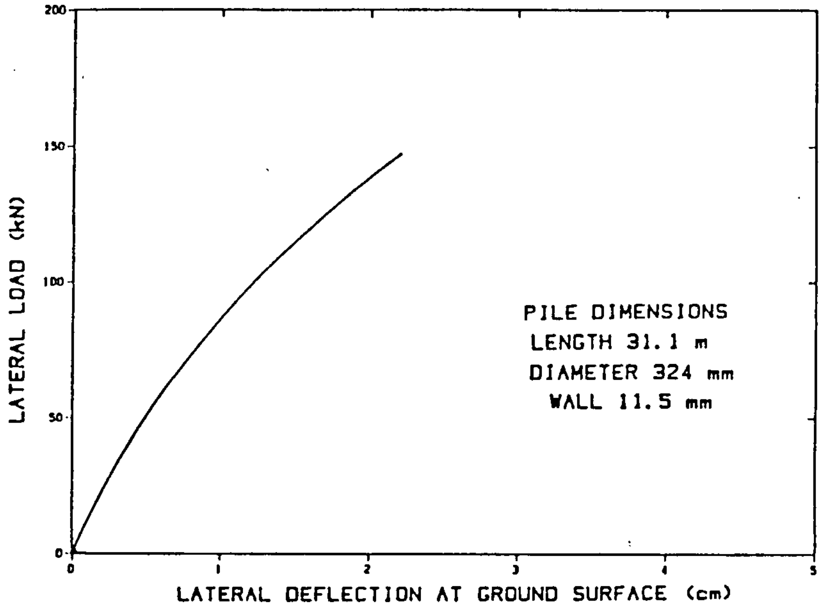


FIG. 5.19. UBCPRS: LATERAL PILE LOAD TEST RESULTS - PILE NO. 3



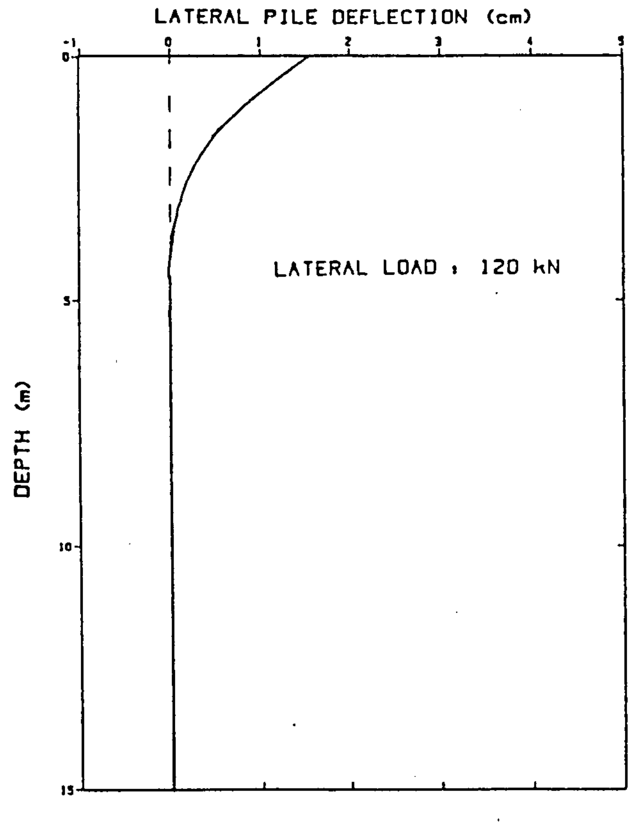
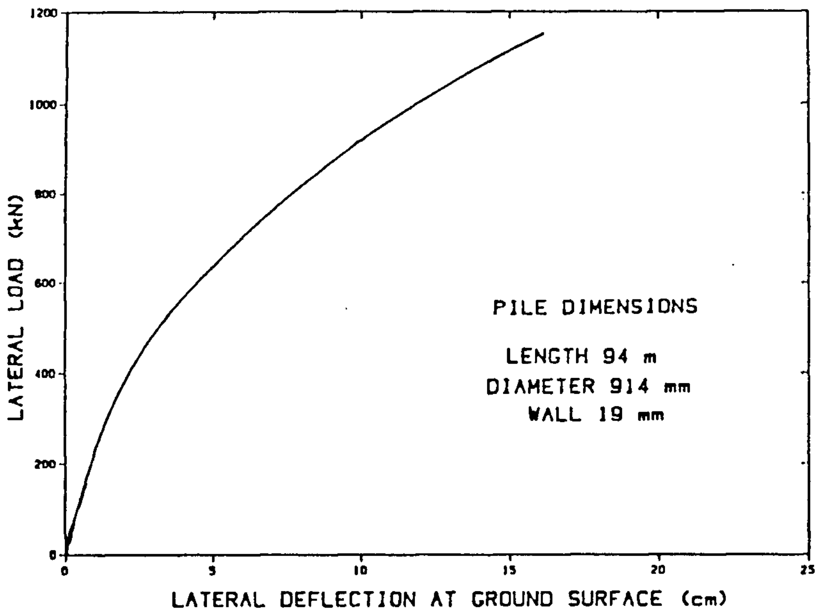


FIG. 5.20. UBCPRS: LATERAL PILE LOAD TEST RESULTS - PILE NO. 5

(approximately 11 pile diameters). Once again, below this point almost no further deflection is apparent.

The ground surface deflection and deflected shape profile for the MOTHPRS pile are both shown in Fig. 5.21. A maximum deflection at the ground surface of approximately 150 mm occurred under an applied load of 1100 kN. The deflected shape profile, at a corresponding 1100 kN load, indicates a first point of contraflexure at a depth of approximately 10 metres. Essentially, no significant deflection is recorded below this depth.



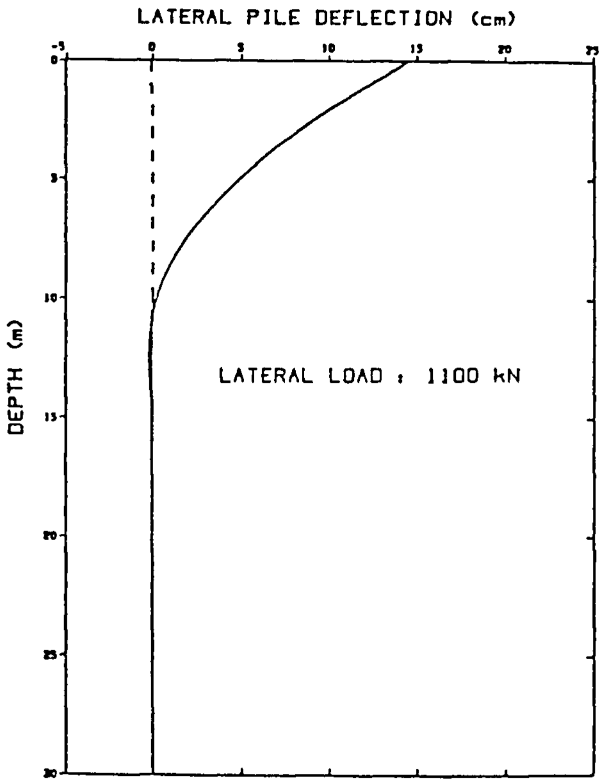


FIG. 5.21. MOTHPRS: LATERAL PILE LOAD TEST RESULTS

CHAPTER 6

PREDICTED VERSUS MEASURED AXIAL CAPACITY

6.1 Introduction

In this chapter, the various methods of predicting axial pile capacity evaluated for this study are compared to the pile load test capacity values obtained and described in Chapter 5. The prediction methods will be separated into groups as follows:

i) Static methods - direct
- indirect

ii) Dynamic methods

Static methods are defined as methods that use static pile capacity formulae to predict capacity. For this study the term "direct method" is applied to any static prediction method that uses CPT data directly without the need to evaluate any intermediate values (coefficients of earth pressure, bearing capacity factors, friction angle, etc.). An "indirect method" is taken to refer to static prediction methods that require intermediate correlations in order to predict pile capacity from CPT data. It must be realized that, unlike the direct methods, most indirect methods were not formulated specifically for use with CPT data. As such, any discrepancies between the predicted and measured pile capacities using the indirect methods may not be due solely to problems inherent to these methods. The correlations between the CPT values and the intermediate parameters may lack sufficient accuracy. This should be kept in mind when comparisons are made between direct and indirect methods.

Dynamic methods are defined as methods that use either predicted or measured pile driving stress wave data to predict pile capacity at the time of driving.

In order to ensure that no bias in imparted to any one method, the same input data set is used in each case. In general this input data set is comprised of two CPT soundings, one for each of the UBCPRS piles (CPTPR85-1) and the MOTHPRS pile (CPTPR84-1). Details of the in-situ testing data used in this chapter is given in Chapter 4. To predict the capacity of the 915 mm diameter (MOTH) pile at depths greater than 75 m the CPT profile was predicted assuming a continued linear increase. Available borehole information supplied by the BCMOTH indicates that a linear increase in parameters is a reasonable assumption.

Details of the dynamic measurements used in the dynamic methods are found in Chapter 5.

For each method, two plots will be presented. One plot will compare the predicted and measured pile capacities for the UBCPRS piles and the other will show the predicted and measured capacities for the MOTHPRS pile. In each case, the components of the predicted shaft resistance and total resistance are presented, the end bearing component being the difference between the two.

Detailed descriptions of each of the twelve prediction methods evaluated, as listed in Table 6.1, will not be presented. However, Table 6.2 summarizes the formulation of the 12 static prediction methods evaluated. Each method will also be briefly outlined in the appropriate section. For a more detailed account of any method evaluated, is consult the complete list of references given in Table 6.1. In addition to the 12

TABLE 6.1. PILE CAPACITY PREDICTION

No.	Method	Reference(s)	Test Data	Type
1	Schmertmann & Nottingham, CPT	Nottingham (1975), Nottingham & Schmertmann (1975), Schmertmann (1978)	CPT	Static-Direct
2	deRuiter & Beringen, CPT	deRuiter & Beringen (1979)	CPT	Static-Direct
3	Zhou et al. (1982), CPT	Zhou et al. (1982)	CPT	Static-Direct
4	Laboratoire Central des Ponts et Chausées (LCPC) CPT	Bustamante & Gianeselli (1982)	CPT	Static-Direct
5	Van Mierlo & Koppejan "Dutch" CPT	Van Mierlo & Koppejan (1952)	CPT	Static-Direct
6	API RP2A	American Petroleum Institute (1980)	CPT	Static-Indirect
7	Dennis & Olson (Modified API)	Dennis & Olson (1983a,b)	CPT	Static-Indirect
8	Vijayvergiya & Focht	Vijayvergiya & Focht (1972)	CPT	Static-Indirect
9	Burland	Burland (1973)	CPT	Static-Indirect
10	Janbu	Janbu (1976)	CPT	Static-Indirect
11	Meyerhof Conventional	Meyerhof (1976)	CPT	Static-Indirect
12	Flaate & Selnes	Flaate & Selnes (1977)	CPT	Static-Indirect
13	Engineering News Record Dynamic Formula	Cummings (1940)	Pile installation blow-counts, hammer size, set	Dynamic-Rigid Pile
14	WEAP86	Goble & Rausche (1986)	Pile installation blow-counts, hammer size	Dynamic-Wave Equation (Prediction)
15	Case Method	Gravare et al. (1980)	Dynamic measurements	Dynamic-Case Method
16	CAPWAP	Rausche (1970)	Dynamic measurements	Dynamic-Wave Equation (In-Situ)

TABLE 6.2. DESIGN METHODS FOR CALCULATING
AXIAL PILE CAPACITY

METHOD*	FORMULATION	
	Shaft Resistance	End Bearing
1. Schmertmann and Nottingham CPT Nottingham (1975), Nottingham and Schmertmann (1975), and Schmertmann (1976)	<u>SAND:</u> f = most appropriate** from f_1, f_2, f_3 $f_1 = K [\sum_{0}^{8D} (\frac{\ell}{8D}) \cdot f_s + \sum_{8D}^{L} f_s]$ K = empirical coefficient = Func^n (L/D, material, shape) f_s = CPT sleeve friction value ℓ = depth to f_s considered D = pile width L = pile length f_2 = 0.12 MPa $f_3 = c \cdot q_c$ c = empirical coefficient = Func^n (pile type) q_c = CPT tip bearing value	<u>SAND:</u> q_p = minimum of q_{p_1} and q_{p_2} q_{p_1} = linear function of q_c above and below pile tip q_c = CPT tip bearing value = 30 MPa maximum cutoff q_{p_2} = 15 MPa clean sands = 10 MPa v. silty sand

* Methods 1 to 5 are "direct" CPT methods.
Methods 6 to 12 are "indirect" CPT methods

** Most appropriate = minimum value where lack of local experience exists.

TABLE 6.2. (cont'd)

METHOD	FORMULATION: Shaft Resistance	FORMULATION: End Bearing
1. (cont'd)	CLAY: f = most appropriate* from f_1, f_2, f_3 $f_1 = \alpha' \; S_u^{**}$ α' = empirical coefficient = Func^n (f_s, material) S_u = undrained shear strength $f_2 = \lambda' \; (\bar{p}' + 2\;\bar{S}_u)$ $\bar{p}'$ = ave. $\bar{\sigma}_{v_o}$ along pile length $\bar{S}_u$ = ave. undrained shear strength along pile length λ' = empirical coefficient = Fun^n (L) $f_3 = \alpha' \; [\sum_{o}^{8D} (\frac{\ell}{8D}) \bullet f_s + \sum_{8D}^{L} f_s]$	CLAY: q_p = minimum of q_{p_1} and g_{p_2} q_{p_1} = linear function of q_c above and below pile tip $q_{p_2} = q_{p_1}$ for NC and slightly OC clays $= \alpha \bullet q_{p_1}$ for highly OC clays α = Woodward's (1961) adhesion ratio ** Schmertmann suggests using $S_u = \frac{q_c - \sigma_{vo}}{N_K}$ where N_K = 10-20, adjust to reflect local experience.
2. de Ruiter and Beringen CPT de Ruiter and Beringen (1979)	SAND: f = most appropriate* from f_1, f_2, f_3, f_4 f_1 = 0.12 MPa (limit value) f_2 = CPT sleeve friction, f_s $f_3 = q_c/300$ (compression) $f_4 = q_c/400$ (tension) CLAY: $f = \alpha \bullet S_u^{***}$ α = 1 for N.C. clay = 0.5 for O.C. clay	SAND: q_p = minimum of q_{p_1} and q_{p_2} q_{p_1} = Func^n (q_c, OCR, D, L) OCR = overconsolidation ratio q_{p_2} = 15 MN/m² CLAY: $q = N_c \bullet S_u^{***}$ N_c = bearing capacity factor for $\emptyset = 0$ = 9

* Most appropriate = minimum value where lack of local experience exists.

***de Ruiter and Beringen suggest S_u from CPT: $S_u = q_c/N_K$ where N_K = 15-20 for North Sea clays. Use appropriate values to reflect local experience.

TABLE 6.2. (cont'd)

METHOD	FORMULATION	
	Shaft Resistance	End Bearing
3. Zhou et al (1982) CPT Zhou, Zie, Zuo, Luo and Tang (1982)	$f = \Sigma(\beta \bullet \bar{f}_s)$ $\bar{f}_s$ = average local CPT friction of the "s" layer β = empirical coefficient = Func^n ($\bar{f}_s$, soil type) soil type I: $\bar{q}_c \geq 2$ MPa $f_s/\bar{q}_c \leq 0.014$ soil type II: other than I $\beta_I = 0.23\ (\bar{f}_s)^{-0.45}$ $\beta_{II} = 0.22\ (\bar{f}_s)^{-0.55}$	$q_p = \alpha \bullet \bar{q}_c$ $\bar{q}_c$ = interpreted cone resistance at toe level (computed over a range of ± 4B about toe α = empirical coefficient = Func^n ($\bar{q}_c$, soil type) B = D = pile width $\alpha_I = 0.71\ (\bar{q}_c)^{-0.25}$ $\alpha_{II} = 1.07\ (\bar{q}_c)^{-0.35}$
4. Van Mierlo and Koppejan "Dutch" CPT Van Mierlo and Koppejan (1952)	$f = \Sigma(0.4\%\ \text{of}\ q_c)$	$q_p = \dfrac{q_b+q_a}{2}$ q_b = average q_c 2xdiameter below pile tip q_a = average q_c 8xdiameter above pile tip

TABLE 6.2 (cont'd)

METHOD	FORMULATION	
	Shaft Resistance	End Bearing
5. Laboratoire Central des Ponts et Chaussées (LCPC) CPT Bustamante and Gianeselli (1982)	$f = \sum_{1}^{i} q_{s_i}$ $^*q_{s_i}$ = the limit skin friction at the level of the layer i $= q_c/\alpha$ q_c = cone resistance corresponding to the given level α = empirical coefficient = Func^n (pile type, soil type, q_c) *limit values exist = Func^n (pile type, soil type, q_c)	$q_p = q_{c_a} \cdot K_c$ q_{c_a} = equivalent cone resistance at the level of the pile point = Func^n (q_c, a) $a = \frac{3}{2}D$ K_c = penetrometer bearing capacity factor = Func^n (pile type, soil type, q_c)

TABLE 6.2. (cont'd)

<table>
<tr><th rowspan="2">METHOD</th><th colspan="2">FORMULATION</th></tr>
<tr><th>Shaft Resistance</th><th>End Bearing</th></tr>
<tr><td>6. API RP2A

American Petrolum Institute (1980)</td>
<td>SAND: $f = K\,\sigma'_{v_o}\tan\delta$
K = coefficient of lateral earth pressure
= 0.5 to 1.0 for compressive axial loading
σ'_{v_o} = effective overburden pressure
δ = angle of soil friction on pile wall

Soil Type | ϕ | δ | N_q
clean sand | 35° | 30° | 40
silty sand | 30° | 25° | 20
sandy silt | 25° | 20° | 12
silt | 20° | 15° | 8

$\emptyset$ = angle of internal friction of soil

CLAY: $f = \alpha \cdot \beta_1\,\sigma'_{v_o}$; $\beta_1 = (S_u/\sigma'_{v_o})$
S_u = undrained shear strength
(i) highly plastic (P.I.>25)
NC: $\alpha = 1$
OC: $\alpha = 1$, but f ≯ the larger of 1 Ksf or $(S_u)_{NC}$
(ii) low to medium plasticity clay

S_u(Ksf) | α
< 0.5 | 1
0.5 - 1.5 | 1 - 0.5 (lin. var^n)
> 1.5 | 0.5</td>
<td>SAND: $q_p = \sigma'_{v_o} \cdot N_q$
N_q = bearing capacity factor
= Func^n ($\emptyset'$) [see skin friction]

CLAY: $q_p = 9 \cdot S_u$

Note: - no defined method to obtain S_u or ϕ</td></tr>
</table>

TABLE 6.2. (cont'd)

<table>
<tr><th rowspan="2">METHOD</th><th colspan="2">FORMULATION</th></tr>
<tr><th>Shaft Resistance</th><th>End Bearing</th></tr>
<tr>
<td rowspan="2">7. Dennis and Olson
(modified API RP2A)

Dennis and Olson (1983 a) and Dennis and Olson (1983 b)</td>
<td><u>SAND:</u> $f = F_{SD} \cdot K \cdot \sigma'_{v_o} \tan \delta$
F_{SD} = empirical coefficient
$= 1/[0.6 \exp (L/60 \cdot D)]$
D = pile diameter
L = embedded length
K = empirical coefficient = 0.8 unless local experience dictates otherwise</td>
<td><u>SAND:</u> $q_p = F_D \sigma'_{v_o} N_q$
F_D = empirical coefficient
$= 1/[0.15 + 0.008 L]$
N_q = bearing capacity factor
= Func^n (Ø')</td>
</tr>
<tr>
<td><u>CLAY:</u> $f = \alpha \cdot \bar{S}_u \cdot F_c \cdot F_L$
F_c = empirical correction for strength
= obtain from local experience, or:
U.C.T. (high quality): $F_c = 1.1$
U.C.T. (driven sampler): $F_c = 1.8$
Field vane: $F_c = 0.7$
U.C.T. = unconfined compression test
α = adhesion factor
= the adhesion factor varies linearly as follows:
<table>
<tr><th>$\bar{S}_u F_c$(psf)</th><th>0</th><th>600</th><th>1200</th><th>5000</th><th>∞</th></tr>
<tr><td>α</td><td>1.0</td><td>1.0</td><td>0.5</td><td>0.3</td><td>0.3</td></tr>
</table>
$\bar{S}_u$ = average undrained shear strength over pile length
F_L = empirical correcton for depth
<table>
<tr><th>L(ft)</th><th>0</th><th>100</th><th>175</th><th>∞</th></tr>
<tr><td>F_L</td><td>1.0</td><td>1.0</td><td>1.8</td><td>1.8</td></tr>
</table>
</td>
<td><u>CLAY:</u> $q_p = 9 \cdot S_u \cdot F_c$
F_c = as per skin friction
S_u = undrained shear strength near the pile tip

Note: - no defined method to obtain S_u or ϕ</td>
</tr>
</table>

TABLE 6.2. (cont'd)

METHOD	FORMULATION	
	Shaft Resistance	End Bearing
8. Vijayvergiya and Focht Vijayvergiya and Focht (1972)	SAND: • recommend use of Dennis and Olson's criteria (6.) CLAY: $f = \lambda\ (\bar{\sigma}_{vm} + 2\ S_{um})$ λ = empirical coefficient = Func^{n} (L) S_{um} = mean undrained shear strength along pile $\bar{\sigma}_{vm}$ = mean σ'_{vo} along pile L = pile penetration	SAND: • recommend use of Dennis and Olson's criteria (6.) CLAY: $q_p = 9 \cdot S_u$ Note: - no defined method to obtain S_u
9. Burland Burland (1973)	SAND: • recommend use of Dennis and Olson's criteria (6.) CLAY: $f = \beta \cdot \sigma'_{vo}$ 1. NC: $\beta = (1-\sin\phi')\tan\phi'$ ϕ' = effective angle of internal friction 2. If ϕ' not known: β = 0.25 - 0.40 (ave = 0.32)	SAND: • recommend use of Dennis and Olson's criteria (6.) CLAY: $q_p = 9 \cdot S_u$ Note: - no define method to obtain ϕ' or S_u

TABLE 6.2. (cont'd)

METHOD	FORMULATION	
	Shaft Resistance	End Bearing
10. Janbu Janbu (1976)	<u>SAND:</u> $f = S_v (\sigma'_{v_o} + a)$ $S_v = \tan \phi' [\sqrt{1 + \mu^2} + \sqrt{1 + r^2}]^{-2}$ $\mu = \tan \phi'/\lvert r \rvert$ r = roughness number = Func^n (L) a = soil attraction = $c \bullet \cot \phi$ c = cohesion <u>CLAY:</u> • as for SAND	<u>SAND:</u> $q_p = (N_q - 1)(\sigma'_{v_o} + a)$ N_q = bearing capacity factor = Func^n (μ, Ψ) Ψ = angle of plastification Note: - no defined method to obtain ϕ' <u>CLAY:</u> • as for SAND
11. Meyerhof Conventional Meyerhof (1976)	<u>SAND:</u> $f = K_s \bullet \sigma'_{v_o} \bullet \tan \delta \leq f_\ell$ K_s = average coefficient of earth pressure on pile shaft = Func^n (ϕ', pile type, installation) δ = angle of skin friction f_ℓ = limiting value of average unit skin friction = from local experience	<u>SAND:</u> $q_p = \sigma'_{v_o} \bullet N_q \leq q_\ell$ N_q = bearing capacity factor = Func^n (ϕ', D, L) q_ℓ = limiting value of unit point resistance = $0.5\ N_q \tan \phi'$ (units = tsf) Note: - no defined method to obtain ϕ' or S_u

TABLE 6.2. (cont'd)

METHOD	FORMULATION	
	Shaft Resistance	End Bearing
11. (cont'd)	CLAY: $f = \beta\ \sigma'_{v_o} \leq S_u$ NC: $\beta = \text{Func}^n(L)$ OC: $\beta = 1.5\ (1-\sin\phi')\ \tan\phi'\ \sqrt{OCR}$ OCR = overconsolidation ratio	CLAY: $q_p = c \cdot N_c + \sigma'_{v_p} \cdot N_q \leq q_m$ N_c = bearing capacity factor $c = \text{Func}^n\ (\phi', D, L)$ c = ave. unit cohesion near pile point = usually taken as $c = S_u$ and $N_q \rightarrow 0$ q_m = limiting value of unit point resistance = empiricial (local experience)
12. Flaate and Selnes Flaate and Selnes (1977)	SAND: • recommend use of Dennis and Olson's criteria (6.) CLAY: $f = \mu_L\ [(0.3 - 0.001\ I_p) \cdot \sqrt{OCR} \cdot \sigma'_{v_o} + 0.008\ I_p \cdot S_u]$ or simplified $f = \mu_L\ [0.3 \text{ to } 0.5] \cdot \sqrt{OCR}\ \sigma'_{v_o}$ μ_L = length function = (L+20)/(2L+20) (L in metres) I_p = plasticity index	SAND: • recommend use of Dennis and Olson's criteria (6.) CLAY: $q_p = 9 \cdot S_u$ Note: - no defined method to obtain ϕ' or S_u

static prediction methods presented four dynamic prediction methods, also listed in Table 6.1, are included.

In total, sixteen methods of predicting axial pile capacity are presented and compared with pile load test data from six different piles at eight different depths.

A discussion of the sensitivity of the prediction methods to the input parameters chosen is also included.

6.2 Use of Spreadsheets

Many pile prediction methods are relatively difficult and time consuming to implement without the aid of a computer. This is especially true when near continuous CPT data is used. For each of the prediction methods used in this study a computer program was written using commercially available spreadsheet software. The spreadsheet is seen as a powerful engineering computational tool that is well suited to geotechnical engineering design. The spreadsheet is particularly well adapted for performing sensitivity analyses and therefore rapid evaluation of input parameters. Perhaps the greatest attraction of using spreadsheets, however, is that the programmer/operator requires little computer programming background. It is doubtful that the number of methods investigated could have been possible within the required time frame without this computational assistance. Davies et al. (1987) provide a more complete discussion on the use of spreadsheets, specifically with CPT input data, for foundation engineering design.

6.3 Direct Methods

In this section, five methods of directly predicting pile capacity using CPT data are presented. As mentioned previously, all of these methods have been formulated specifically for use with CPT data and can therefore be expected to give better results than the indirect methods.

Direct methods apply CPT data directly by the use of theoretical and/or empirical scaling factors without the need to evaluate any intermediate values (coefficients of earth pressure, bearing capacity factors, friction angle, etc.). The scaling factors, in all cases, resemble the original work of de Beer (1963). As shown in Fig. 6.1, if a probe of zero diameter penetrates into a soil layer, the penetration resistance would follow the idealized curve ABCD. This is to say that the device would "feel" the entire effect of the lower soil layer immediately upon penetration. However, if a large diameter pile were pushed into the layer, the point resistance would not equal that of the zero diameter probe until the pile reached a greater depth, at point E. This depth is often termed the critical depth (D_c). De Beer (1963) showed that it is reasonable to assume that the pile resistance curve between points B and E varies linearly; thus, the pile resistance at any intermediate depth could be determined if the idealized penetration resistance curve and D_c were known. Although it is not possible to use a probe of zero diameter, the standard sized electric cones (35.7 mm in diameter) can be assumed to approximate this condition (curve ABC'D), especially for large diameter piles. Meyerhof (1951), de Beer (1963), and others have shown that D_c is a function of foundation size and soil stiffness. Therefore, it is more logical to express critical depth as a ratio $(D/B)_c$ where B is the foundation diameter. This concept is complicated in highly layered materials where layer thicknesses can be less than D_c for the large diameter piles. In these situations the full penetration resistance may be mobilized on the cone but may not be realized for the pile before the influence of another layer is

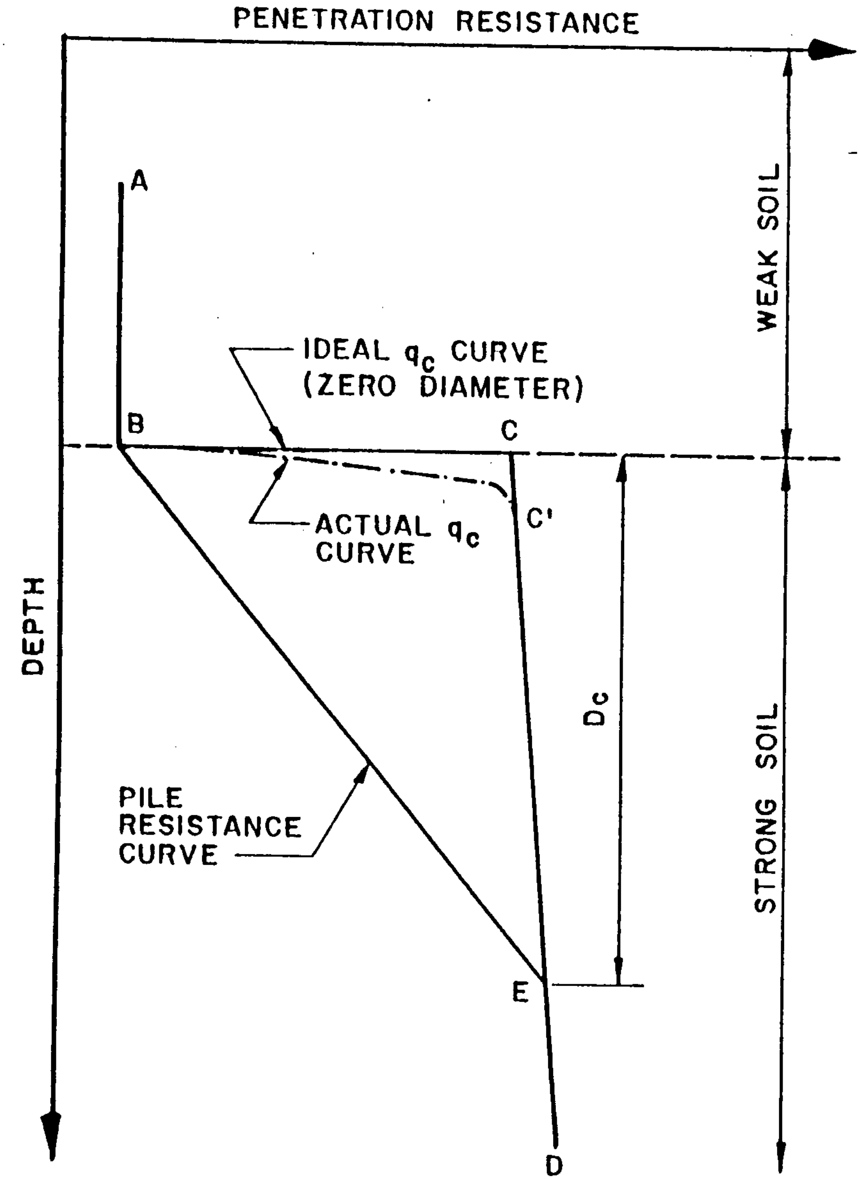


FIG. 6.1. DE BEER SCALE EFFECT DIAGRAM FOR CPT PILE PREDICTIONS
(ADAPTED FROM NOTTINGHAM, 1975)

felt. The way in which the different direct methods define the critical depth and layering effects for both sleeve friction and point resistance is, for the most part, what separates the methods available.

6.3.1 Schmertmann and Nottingham CPT Method

6.3.1.1 Outline

The Schmertmann and Nottingham CPT Method (Schmertmann, 1978) is a summary of the work on both model and full-scale piles presented by L. Nottingham (1975) in his doctoral dissertation at the University of Florida. This method uses both CPT values of cone bearing and sleeve friction.

Although seen as a direct method, an estimate of undrained shear strength, S_u, is required. Schmertmann (1978) suggests that the CPT-S_u relationship used should reflect local experience. Based upon local experience and data obtained for this study (see Chapter 4), the undrained strength was taken to be:

$$S_u = \left(\frac{q_c - \sigma_{vo}}{15}\right) \qquad (6\text{-}1)$$

where: S_u = undrained strength

q_c = cone bearing

σ_{vo} = in-situ vertical total stress

This formulation of undrained shear strength was used in all CPT methods investigated for this study that required an evaluation of the undrained strength profile.

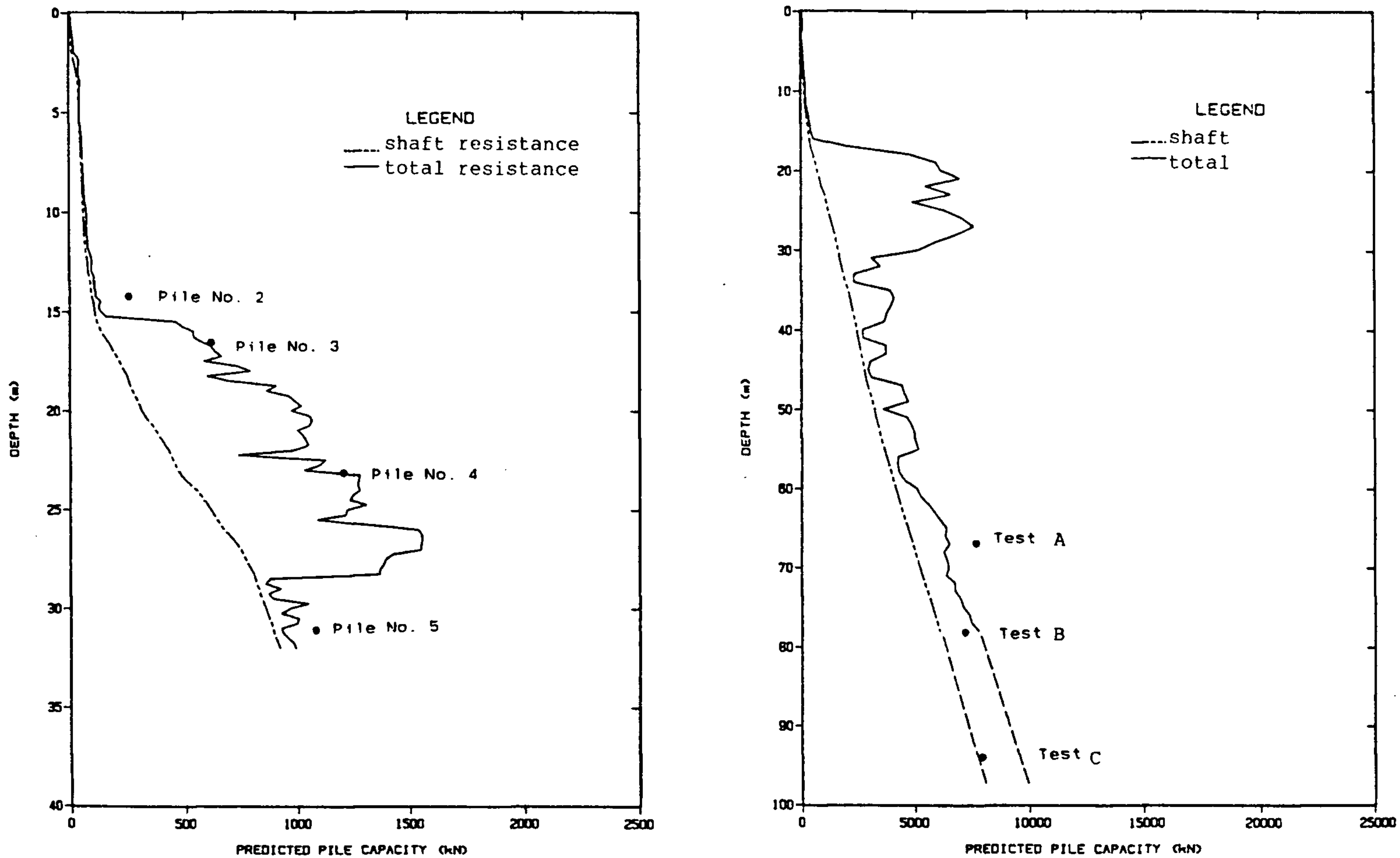
Being a combination of many previous works, precise limitations of the Schmertmann and Nottingham method are difficult to ascertain. Various researchers, Robertson et al. (1985), among others, have reported good correlations with full scale pile load test results.

The Schmertmann and Nottingham method is relatively difficult to implement with some of the procedures being open to interpretation. As can be seen in Table 6.2, it requires a great number of calculations and, therefore, without the aid of a computer, errors are likely.

6.3.1.2 Results

The results of predicted versus measured pile capacity for the UBCPRS and the MOTHPRS pile are shown in Figs. 6.2(a) and 6.2(b) respectively. For this method, and all subsequent methods, only piles 2,3,4 and 5 will be plotted for the UBCPRS since, the predicted capacities include the shaft resistance from the 2 m of sand fill. Pile no. 1 and pile no. 2 behaved essentially the same except that pile no. 1, being cased at the surface, had no contribution to capacity from the upper 2 m of sand fill. Both the skin friction and total resistance profiles are presented for each method. The difference between these two components is the end bearing component of the total resistance. Note that for the MOTHPRS pile below a depth of 78 m the skin friction and total resistance were projected to depth using the trend of the plot above 78 m. This seems justified due to the consistent nature of the deposit as verified by deep drill hole testing carried out at the site by B.C.M.O.T.H.

As shown in Fig. 6.2(a), the predicted capacity agrees very well with the load test results at the UBCPRS. For pile nos. 3 and 4, the predictions are almost identical to the measured capacities. For pile nos. 2 and



UBC PILE RESEARCH SITE
Schmertmann and Nottingham CPT Method
(a)

MOTH PILE LOAD TEST SITE
Schmertmann and Nottingham CPT Method
(b)

FIG. 6.2. SCHMERTMANN AND NOTTINGHAM CPT METHOD

5 there is some discrepancy but the error in prediction is of a conservative nature.

Noting the scale changes to both axes in Fig. 6.2(b), the MOTHPRS also shows good agreement between predicted and measured pile capacity. For test A and B the results are very good with only slight discrepancies. For test C, however, a larger degree of disagreement exists with a non-conservative prediction resulting. Nevertheless, the error is small ($\sim$25%) and, it is suggested, within acceptable limits.

6.3.2 de Ruiter and Beringen CPT Method

6.3.2.1 Outline

The de Ruiter and Beringen (1979) method is based upon experience gained in the North Sea by Fugro Consultants International. The original development of the method can be found in de Ruiter (1971) and de Ruiter (1975). It is also commonly referred to as the "European Method" by North American Engineers.

The de Ruiter and Beringen CPT Method is an empirical method that, as can be seen in Table 6.2, utilizes both CPT cone bearing and sleeve friction. This method, as was the case with Schmertmann and Nottingham's, requires correlating CPT data to undrained shear strength. The inaccuracies introduced by this correlation are discussed in Section 6.6.

de Ruiter and Beringen make no comment as to the method of validation for their method and therefore it is difficult to note specific limitations.

The de Ruiter and Beringen method is relatively simple to implement as it is well explained by the authors. However, the method requires a great number of computations and is therefore best suited for use with the aid of a computer.

6.3.2.2 <u>Results</u>

For the UBCPRS, as shown in Fig. 6.3(a), the measured axial capacity was predicted extremely well by the deRuiter and Beringen method. For pile nos. 2,4 and 5 there is essentially no difference between predicted and measured capacity. For pile no. 3 a slight overprediction exists.

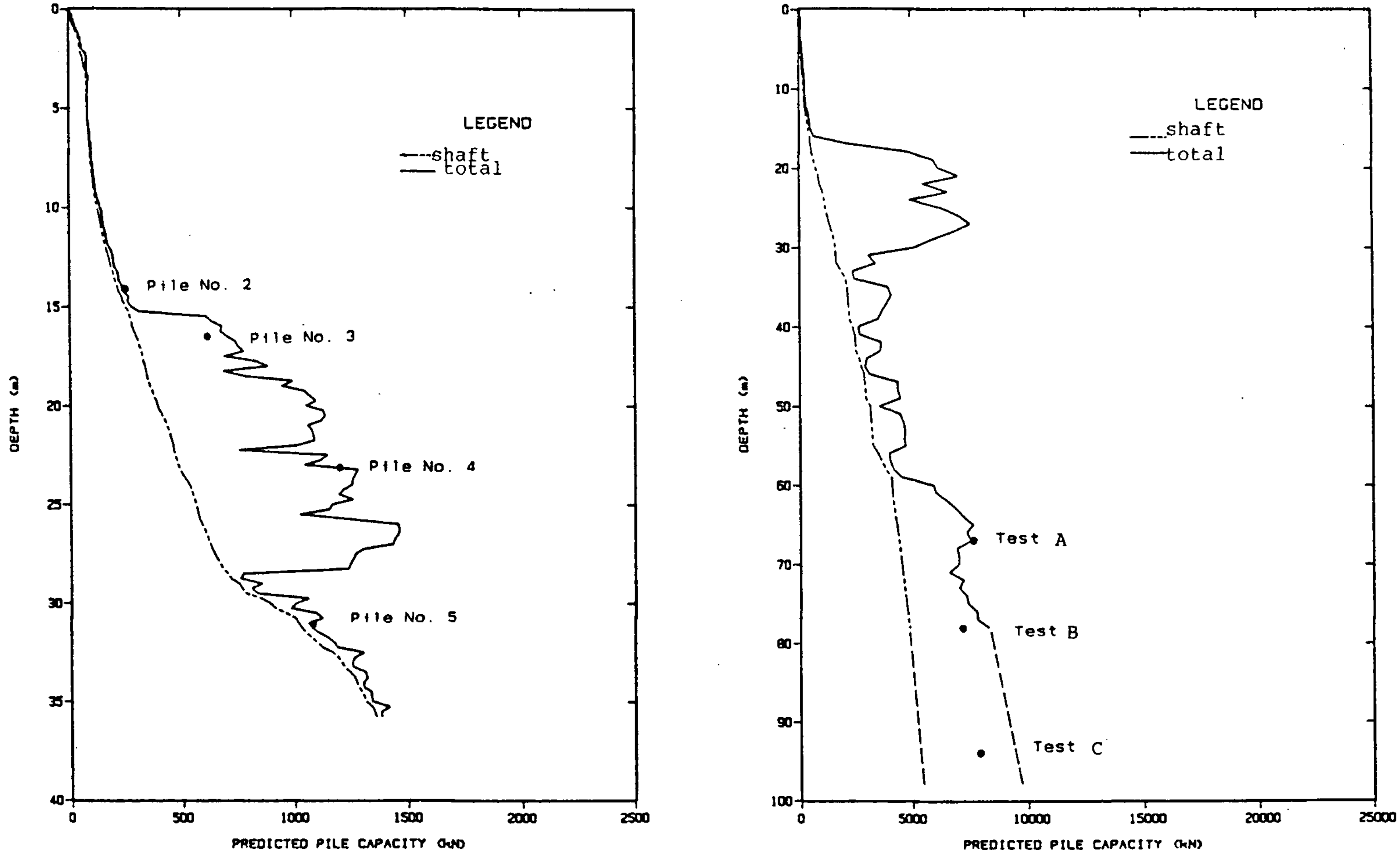
For the MOTHPRS, the predicted versus measured capacities yield good agreement a shown in Fig. 6.3(b). Tests B and C had their capacities slightly overpredictd but by less than 20 percent in each case. For test A the measured capacity was almost identical to the predicted value.

6.3.3 <u>Zhou, Zie, Zuo, Luo and Tang CPT Method</u>

6.3.3.1 <u>Outline</u>

The Zhou et al. (1982) CPT Method is based upon Chinese experience gained using the cone bearing and the sleeve friction from the CPT to predict axial pile capacity. This experience consists of empirically relating the CPT values with 96 full scale pile load tests in various stratigraphic profiles. The majority of this work was performed by the China Academy of Railway Sciences in Beijing.

As can be seen in Table 6.2, the Zhou et al. (1982) method is relatively simple to understand and it is simple to implement. A limitation, noted by Zhou et al. (1982), is that neither debris fill or loess has yet been validated with this method. Another limitation is that the only piles to have been used for validation were driven precast concrete piles. The size of the piles used ranged form 0.25 to 0.55 m in diameter and were from 6.5 to 31.25 m in length.



UBC PILE RESEARCH SITE
deRuiter and Beringen CPT Method
(a)

MOTH PILE LOAD TEST SITE
deRuiter and Beringen CPT Method
(b)

FIG. 6.3. DERUITER AND BERINGEN CPT METHOD

6.3.3.2 Results

As can be seen in Fig. 6.4(a), the predicted pile capacities agreed quite well with the measured capacities for the UBCPRS piles. This is especially true of pile nos. 2,3 and 4. Pile no. 5 had its capacity over-predicted by approximately 30 percent.

The MOTHPRS results, also shown in Fig. 6.4(b), show relatively poor agreement between predicted and measured behaviour. In fact, test C is overestimated by nearly one hundred percent.

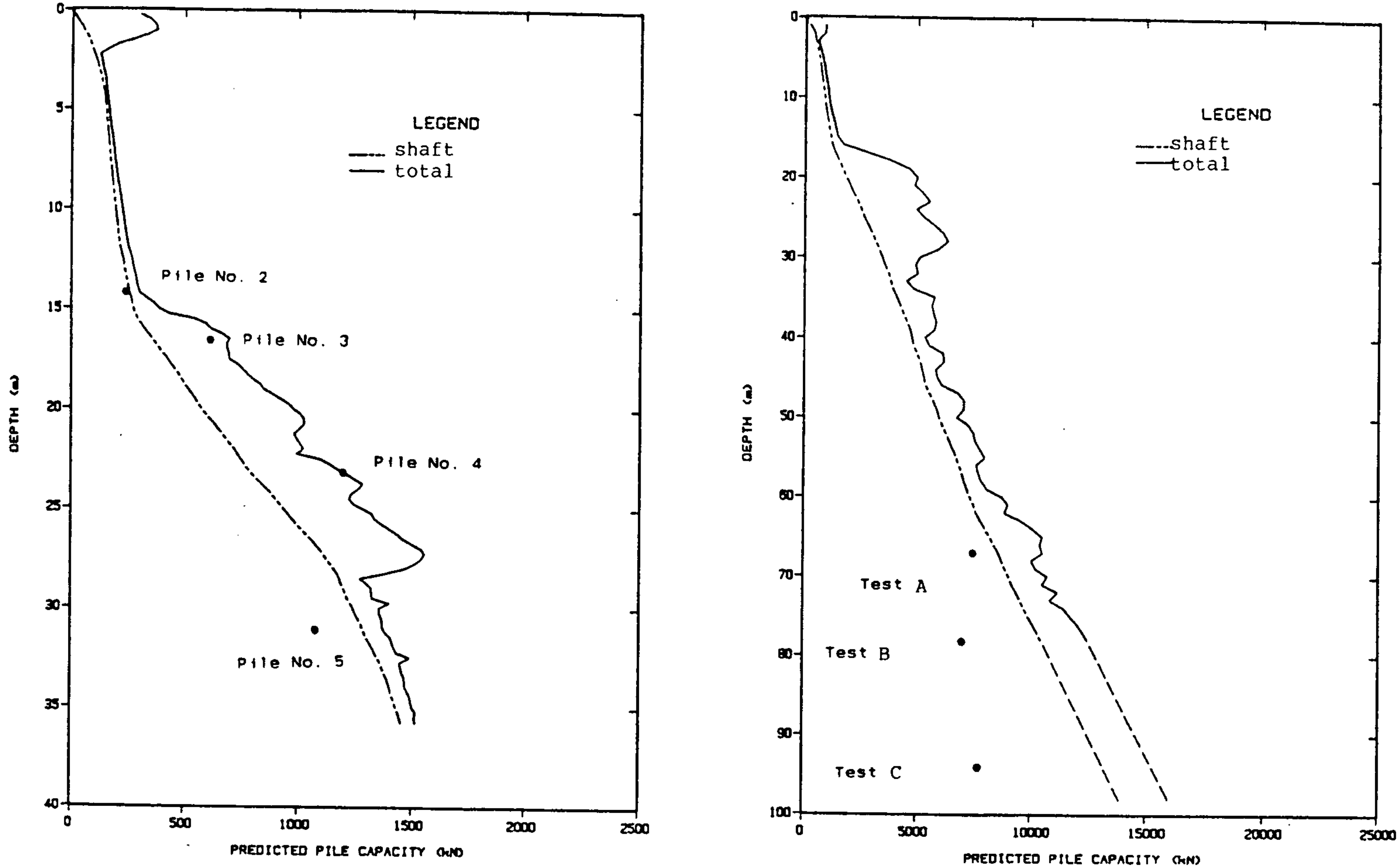
6.3.4 Van Mierlo and Koppejan "Dutch" CPT Method

6.3.4.1 Outline

The Van Mierlo and Koppejan "Dutch" Method represents what was probably the first comprehensive CPT pile capacity method to be formulated in the Netherlands, Van Mierlo and Koppejan (1952) did their studies in conjunction with Delft Laboratories, Holland.

This method is based upon purely empirical observations comparing CPT results with static pile load tests. As can be seen in Table 6.2, this is an extremely simple method to use and has the advantage of only needing CPT bearing values. This advantage is important as obtaining accurate sleeve friction values from CPT data is often an area of concern.

One major limitation of this method is that it was developed solely with mechanical cone data. Using electric cone data, as in this study, is not completely valid but acceptable for comparative purposes. For commercial design using this method it may be advisable to use equivalent mechanical cone values are determined from the electric cone data using the method outlined by Schmertmann (1978).



UBC PILE RESEARCH SITE
Zhou et al CPT Method
(a)

MOTH PILE LOAD TEST SITE
Zhou et al (1982) CPT Method
(b)

FIG. 6.4. ZHOU ET AL. (1982) CPT METHOD

6.3.4.2 Results

From Fig. 6.5(a) it can be seen that the Van Mierlo and Koppejan method predicted the actual capacities of the UBCPRS piles quite well. The capacities were somewhat underpredicted for pile nos. 2 and 5 and overpredicted for pile nos. 3 and 4.

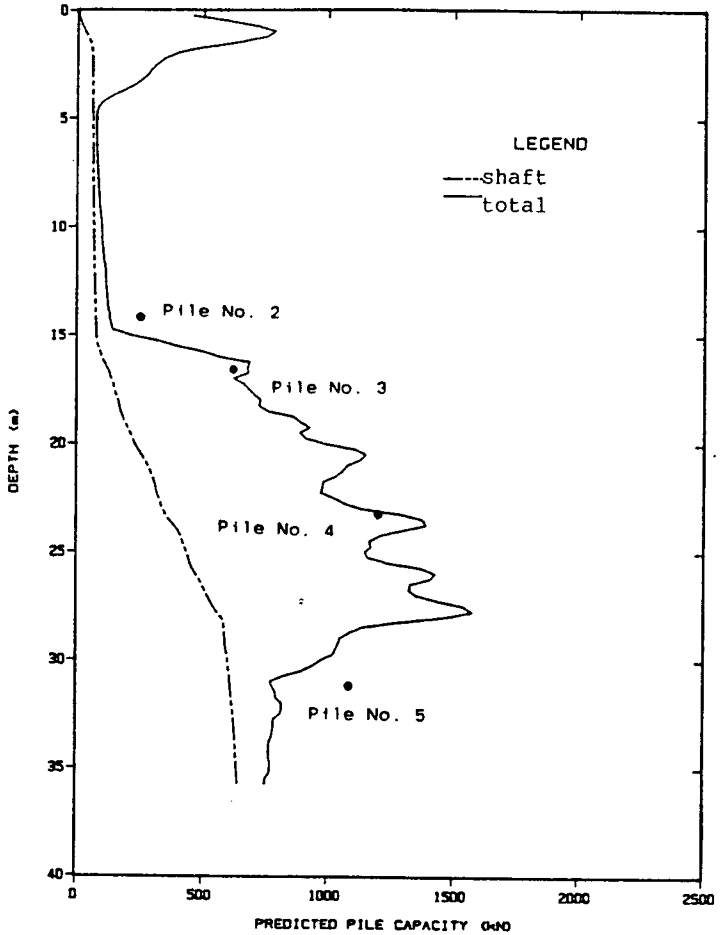
The predicted behaviour for the MOTHPRS, shown in Fig. 6.5(b), is such that all three load test results are underpredicted. Test A was underpredicted by approximately twenty-five percent whereas the measured capacities of tests B and C were within ten percent of the predicted values.

6.3.5 Laboratoire Central des Ponts et Chausées (LCPC) CPT Method

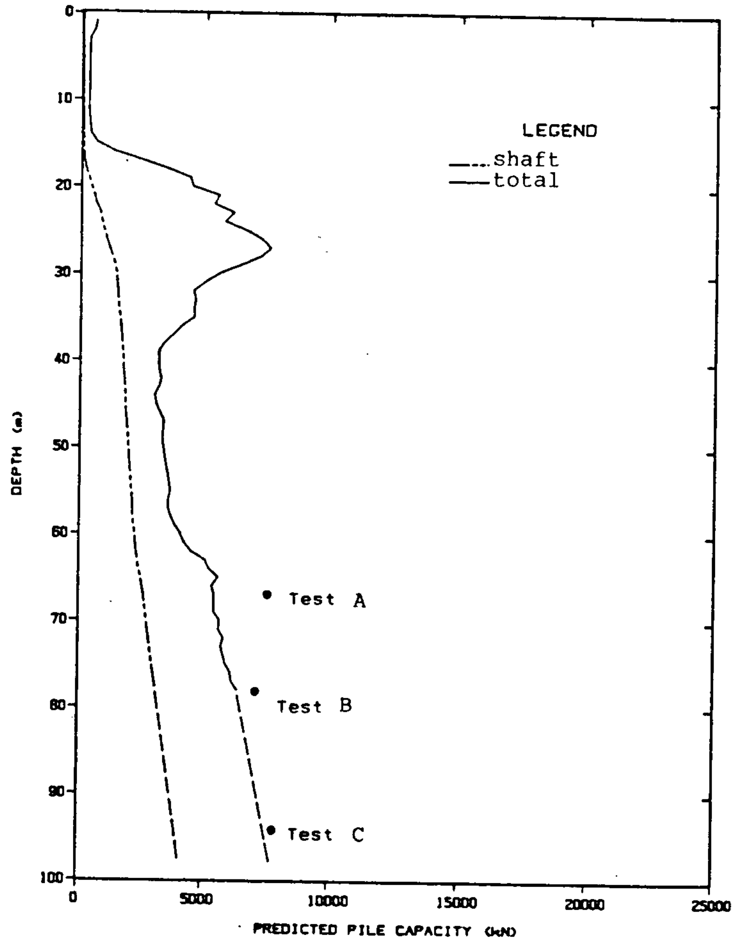
6.3.5.1 Outline

The LCPC CPT Method (Bustamante and Gianeselli, 1982) is a result of experimental work by the French Highway Department to validate the original French CPT pile prediction method (which can be found in the FOND72 (1972) document). The experimental data, consisting of a large number of full-sale loading tests, resulted in the re-adjustment of the original French method and the formation of the LCPC CPT method.

The LCPC CPT method has the same advantage as the original Dutch methods in that only CPT bearing values are needed (except to define soil type). This method is based on a series of 197 full-scale static pile loading (or extraction) tests. The tests involved 96 deep foundations distributed on 48 sites containing materials such as: clay, silt, sand, gravel, weathered rock, mud, peat, weathered chalk, and marl. The types of piles included driven, bored, grouted, barrettes and piers. The sizes used for the driven piles were 300 to 640 mm in diameter and 6 to 45 m in



UBC PILE RESEARCH SITE
Van Mierlo and Koppejan "Dutch" CPT Method
(a)



MOTH PILE LOAD TEST SITE
Van Mierlo and Koppejan "Dutch" CPT Method
(b)

FIG. 6.5. VAN MIERLO AND KOPPEJAN "DUTCH" CPT METHOD

length. However, it is interesting to note that very few of the piles were driven pipe piles.

The LCPC CPT Method is very simple to use and understand and offers no ambiguities. The validation of the method is well documented by Bustamante and Gianeselli (1982).

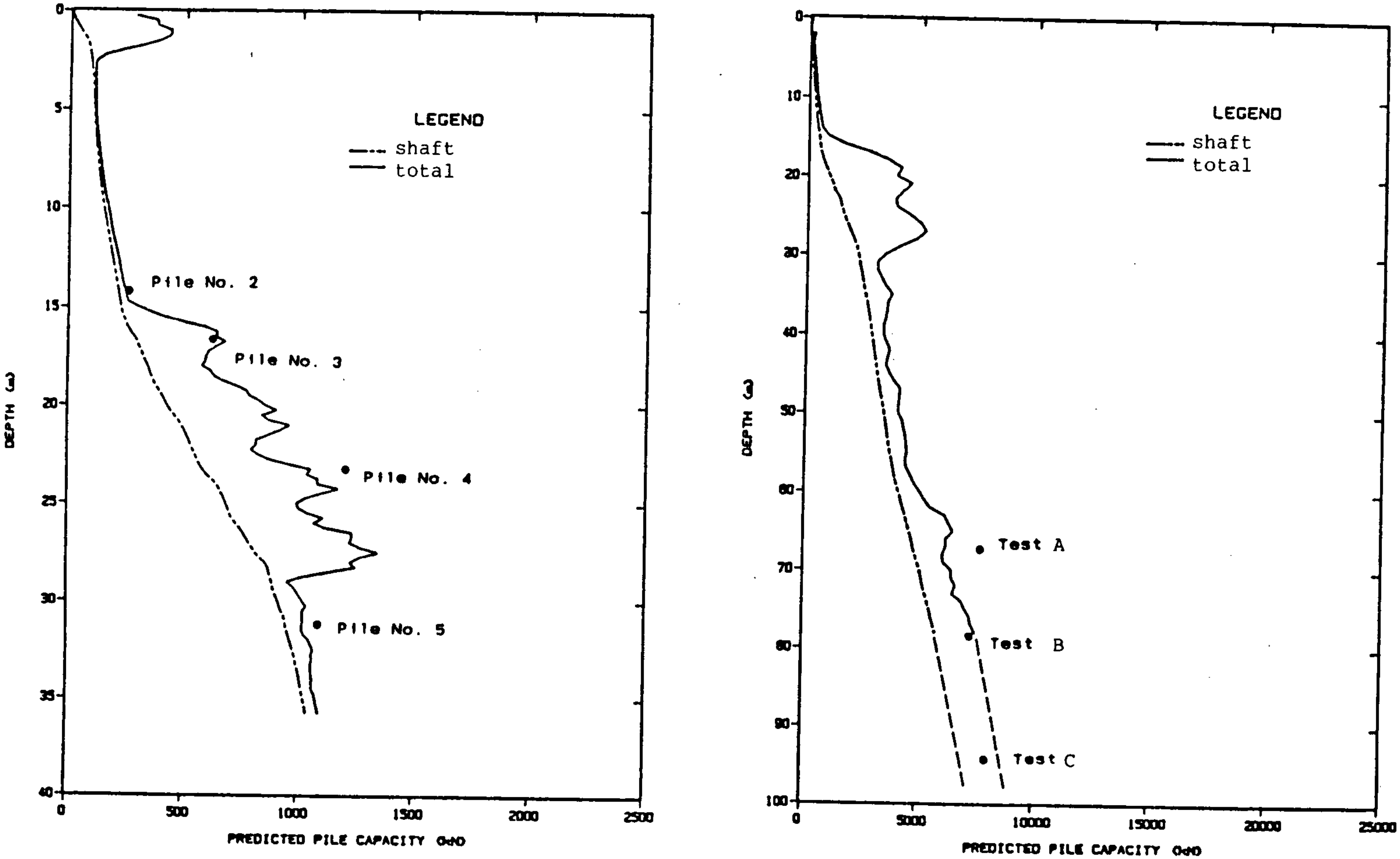
6.3.5.2 Results

The comparison between predicted and measured capacities by the LCPC method for the UBCPRS piles is shown in Fig. 6.6(a). Excellent agreement between predicted and measured pile capacity is evident for all piles. The capacities for pile nos. 2,3 and 5 are all slightly underpredicted whereas pile no. 3 is slightly overpredicted.

Excellent agreement between predicted and measured pile capacity also exists for the MOTHPRS as shown in Fig. 6.6(b). The capacity of test A is slightly underpredicted while the capacities of tests B and C are slightly overpredicted.

6.4 Indirect Static Prediction Methods

In this section twelve methods commonly used by foundation engineers are presented. In each case, all of the input parameters required have been obtained from in-situ testing (usually using the CPT unless otherwise specified) using appropriate correlations. Several of the methods (e.g. Vijayvergiya and Focht) have originally been formulated for use solely in cohesive soils. In these cases, the cohesionless soil contribution to the pile capacity has been obtained using the Dennis and Olson method (Section 6.4.2). The justification of this is that many of these "cohesive soil" methods suggest using the API RP2A (Section 6.4.1) cohesionless soil



UBC PILE RESEARCH SITE
LCPC CPT Method
(a)

MOTH PILE LOAD TEST SITE
LCPC CPT Method
(b)

FIG. 6.6. LCPC CPT METHOD

recommendations. The Dennis and Olson method is a modified API RP2A method and is seen by many as a preferred method to the original API RP2A. As well, in engineering pratice this combination of methods can be used for comparison purposes and to define critical input parameters.

This section wil also briefly examine empirical design-methods for penetration tests not as common as the CPT or SPT (e.g. Becker Hammer test).

6.4.1 American Petroleum Institute (API) RP2A Method

6.4.1.1 Outline

The API RP2A (1980) method was created by the American Petroleum Institute for piled offshore drilling platforms. This method is used extensively for onshore design and is considered by many as the major offshore prediction method.

As can be seen in Table 6.2, this method requires an estimation of the angle of internal friction (ϕ) for cohesionless soils and an estimation of undrained shear strength (S_u) for cohesive soils. The values of ϕ can be obtained from CPT data using the correlation proposed by Robertson and Campanella (1983). This correlation is used throughout this study for indirect CPT methods requiring friction angle values. The undrained strength is determined as described in Section 6.2.1.1.

The API RP2A method has been used for design in many offshore piling projects. The major limitation of this method, and all of the indirect methods used in this study, is that the accuracy of the parameters used in the implementation of the method (e.g. ϕ, S_u) are highly dependent upon the accuracy and reliability of the empirical relationships used to obtain the parameter from the in-situ test data.

The API RP2A method is simple to use and computer assistance is not necessary. However, the method is subject to different levels of interpretation and therefore no unique answer is possible between individual users.

6.4.1.2 Results

As shown in Fig. 6.7(a), the API RP2A method was somewhat successful in predicting the capacity of the UBCPRS piles while pile no. 2 had its measured load slightly underpredicted, the measured capacities for pile nos. 3,4 and 5 were all overpredicted.

For the MOTHPRS (as seen in Fig. 6.7(b)), the predicted pile capacity was considerably overpredicted when compared to the measured test results.

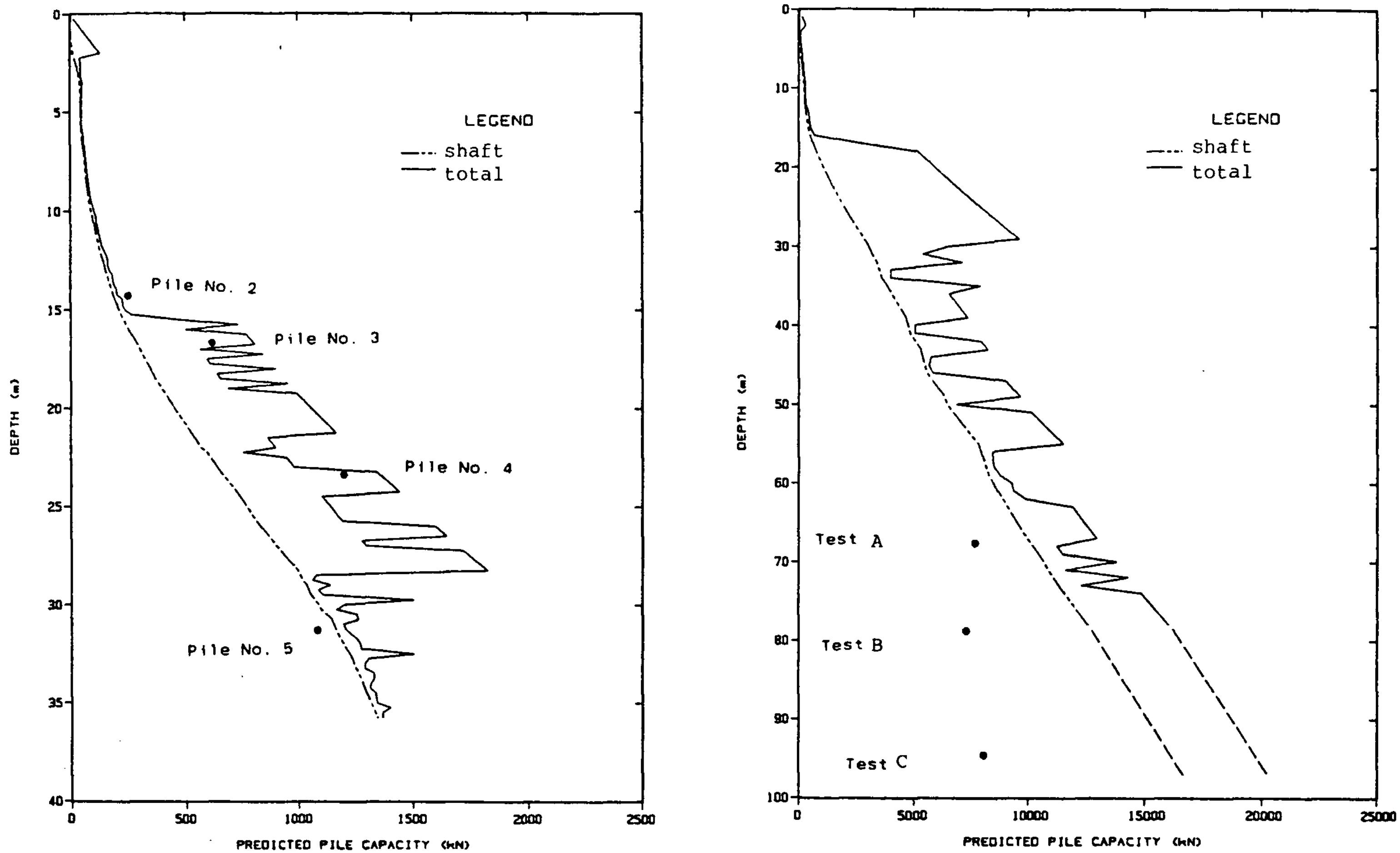
6.4.2 Dennis and Olson Method

6.4.2.1 Outline

The Dennis and Olson Method (Dennis and Olson, 1983a and 1983b) is a modification of the API RP2A method.

From Table 6.2, it is seen that the main difference between the Dennis and Olson and API RP2A method is the use of empirical correction factors by the former. These correction factors are functions of pile embedment for both cohesive and cohesionless soils, and of undrained shear strength for cohesive soils. For cohesionless soils, the value of the angle of soil friction on the pile wall (δ) is obtained as outlined for the API RP2A method.

The validation of this method consisted of comparing the results of 84 full-scale pile load tests in cohesive soils and 66 full-scale pile load tests in cohesionless soils with those predictd by the method. All of the



UBC PILE RESEARCH SITE
American Petroleum Institute RP2A Method
(a)

MOTH PILE LOAD TEST SITE
American Petroleum Institute RP2A Method
(b)

FIG. 6.7. AMERICAN PETROLEUM INSTITUTE RP2A METHOD

piles tested were steel pipe piles with pile diameters ranging from 0.3 m to 1.0 m and test embedments up to 83 m.

The Dennis and Olson method is simple to use and not open to interpretation like the API RP2A method.

6.4.2.2 Results

As can be seen in Fig. 6.8(a), Dennis and Olson's method underpredicted the measured capacity of all the UBCPRS piles except pile no. 3. The capacity of pile no. 3 was slightly overpredicted. Still, all four predictions are quite good.

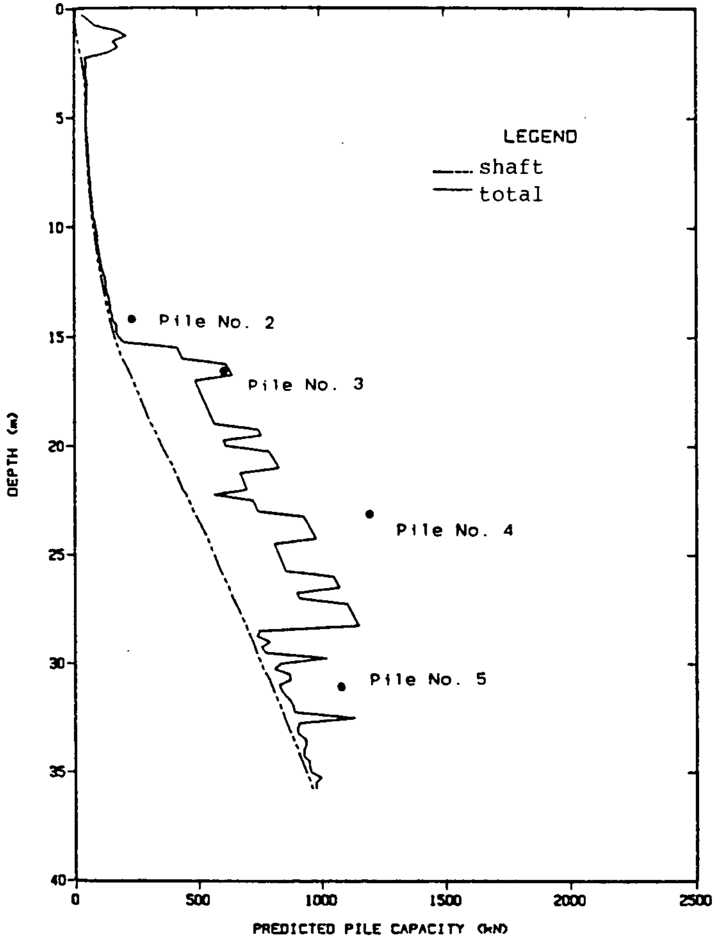
For the MOTHPRS, as shown in Fig. 6.8(b), all three test results are overpredicted by a large amount. In particular, tests B and C are overpredicted by more than 100%. This is somewhat surprising since the method was developed and validated for large diameter, long steel pipe piles.

6.4.3 Vijayvergiya and Focht Method

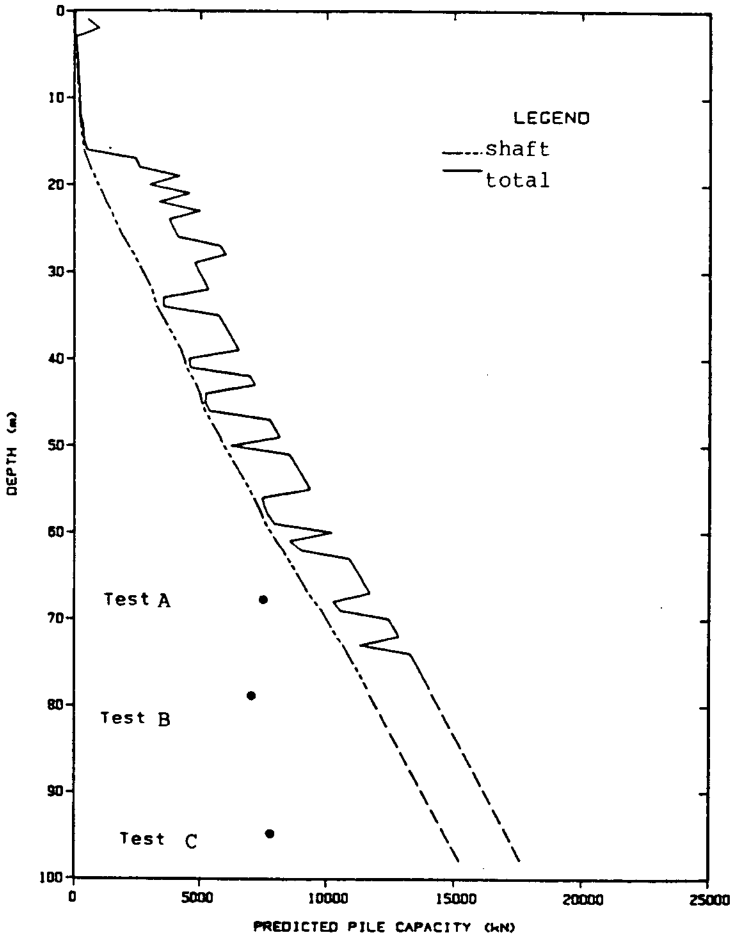
6.4.3.1 Outline

The Vijayvergiya and Focht method (Vijayvergiya and Focht, 1972) was the first widely used method to encorporate two concepts now considered essential to pile design in cohesive materials.

Firstly, the prediction of pile capacity was not solely based upon undrained shear strength but upon effective vertical stress as well. Vijayvergiya and Focht realized that, under static loading, drained friction will govern pile capacity. Secondly, this method encorporates a term (λ) which is a dimensionless coefficient dependent upon pile penetration. In effect, a term to address scale effects is included. Unfortunately, pile diameter was excluded from the original formulation of



UBC PILE RESEARCH SITE
Dennis and Olson Method
(a)



MOTH PILE LOAD TEST SITE
Dennis and Olson Method
(b)

FIG. 6.8. DENNIS AND OLSON METHOD

the method. Schmertmann (1978), among others, suggests that λ be evaluated as a function of pile length and pile diameter.

The Vijayvergiya and Focht method is based upon 47 full-scale pile load tests on piles ranging in length from 2.5 to 100 m in length and in capacity from 27 to 7800 kN. No mention of pile diameter is included. As shown by Table 6.2, this method has been developed for cohesive soils only and therefore Dennis and Olson's method has been used for the cohesionless soils.

The Vijayvergiya and Focht method is both simple to understand and to implement. A large advantage of the method is that it is straightforward and hence different users should obtain approximately the same results.

6.4.3.2 Results

For the UBCPRS, as shown in Fig. 6.9(a), this method did a reasonable job of predicting the measured pile capacities. Pile nos. 2,3 and 5 were all overpredicted but never by much more than 25 percent. Pile no. 4 was slightly underpredicted.

As can be seen in Fig. 6.9(b), the MOTHPRS measured capacities were all greatly overpredicted. Tests B and C were overpredicted by more than 100%. The performance of this method on these tests is very poor, and worse than the method by Dennis and Olson.

6.4.4 Burland Method

6.4.4.1 Outline

The Burland Method (Burland, 1973), like the Vijayvergiya and Focht method, was originally developed only for cohesive soils (see Table 6.2).

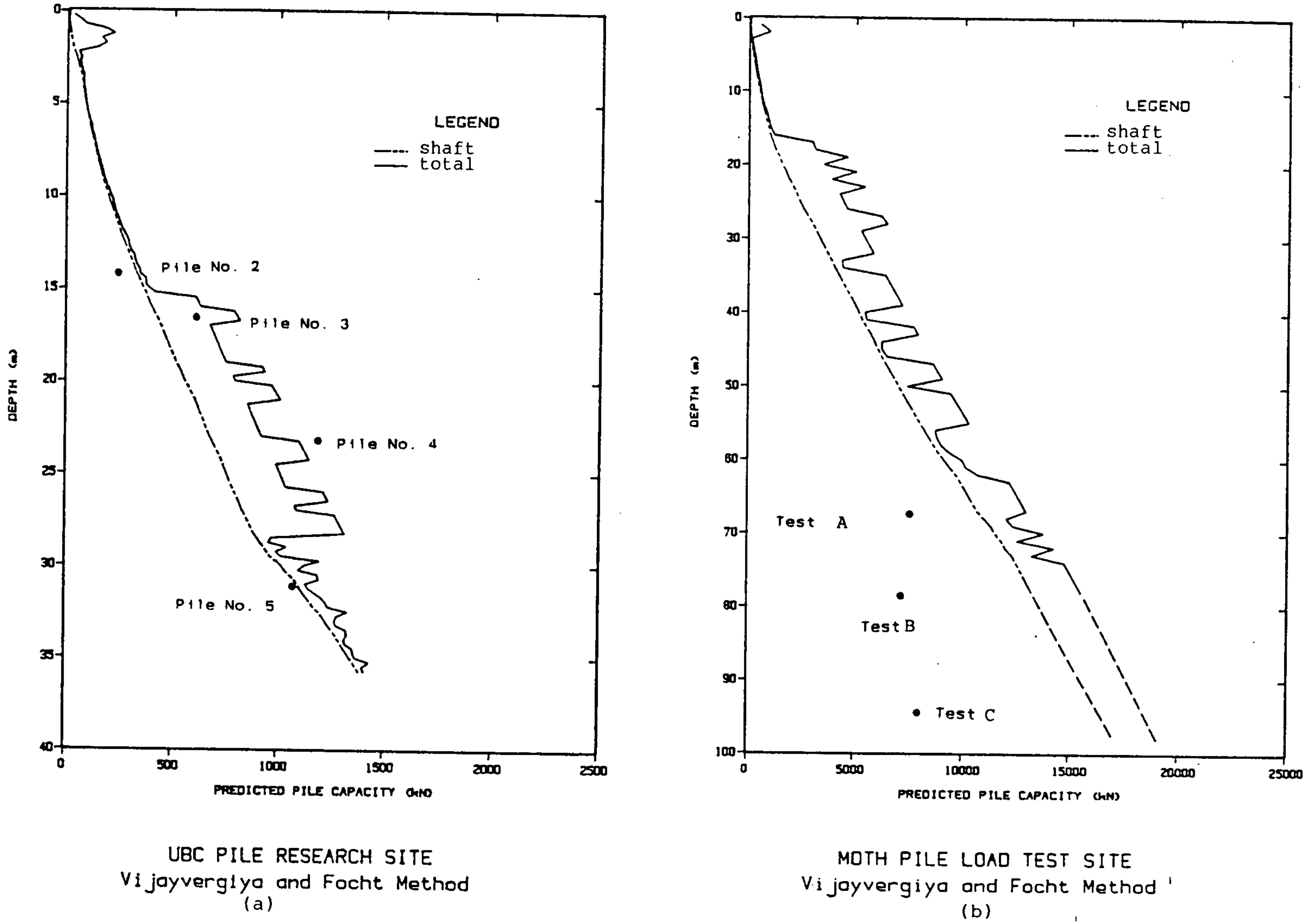


FIG. 6.9. VIJAYVERGIYA AND FOCHT METHOD

This method formulated an expression to determine shaft resistance in terms of effective stress. An empirical factor, β, was defined to be equal to the ratio of the unit skin friction over the effective overburden pressure. Burland found that β ranged from 0.25 to 0.40 (average = 0.32) for driven piles and that it is approximately independent of clay type.

This method was validated using reults from 41 full-scale load tests. The size of the piles used is not reported. Pile types included steel, concrete and timber.

This method is simple to use but the range of β given can cause a variation in results of up to 60%. For this study the recommended value of 0.32 was used.

6.4.4.2 Results

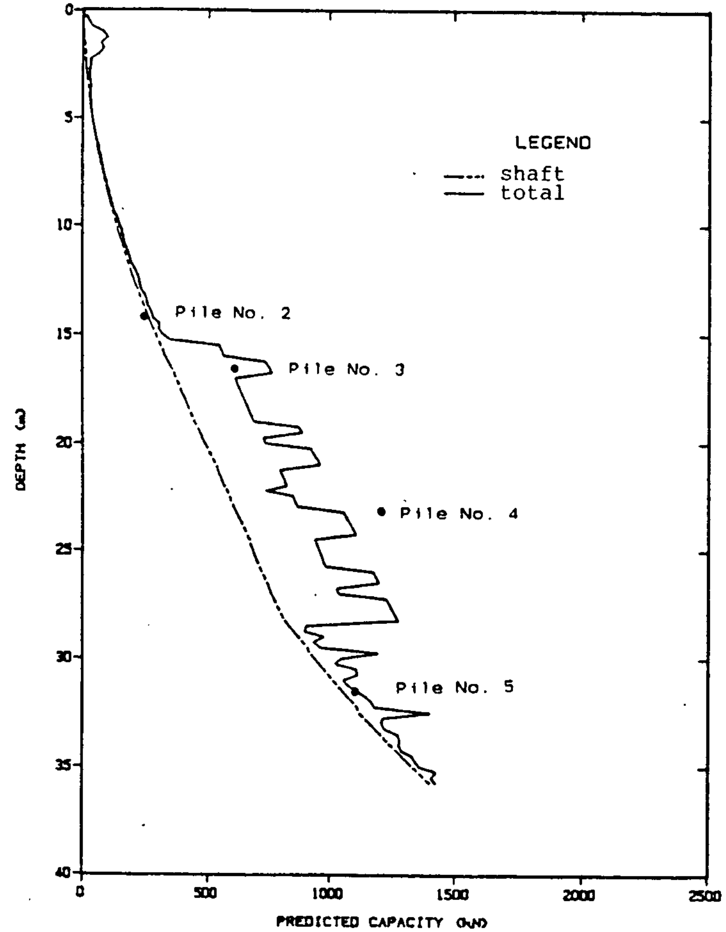
The results for the prediction of pile capacity at the UBCPRS using the Burland method are shown in Fig. 6.10(a). Good agreement between predicted and measured behaviour is seen, especially for pile no. 5.

For the MOTHPRS, shown in Fig. 6.10(b), the predicted results grossly overpredict the measured capacity. For all three tests the predicted capacity is generally at least 100% too large. These results were the poorest obtained for any method applied to the MOTHPRS.

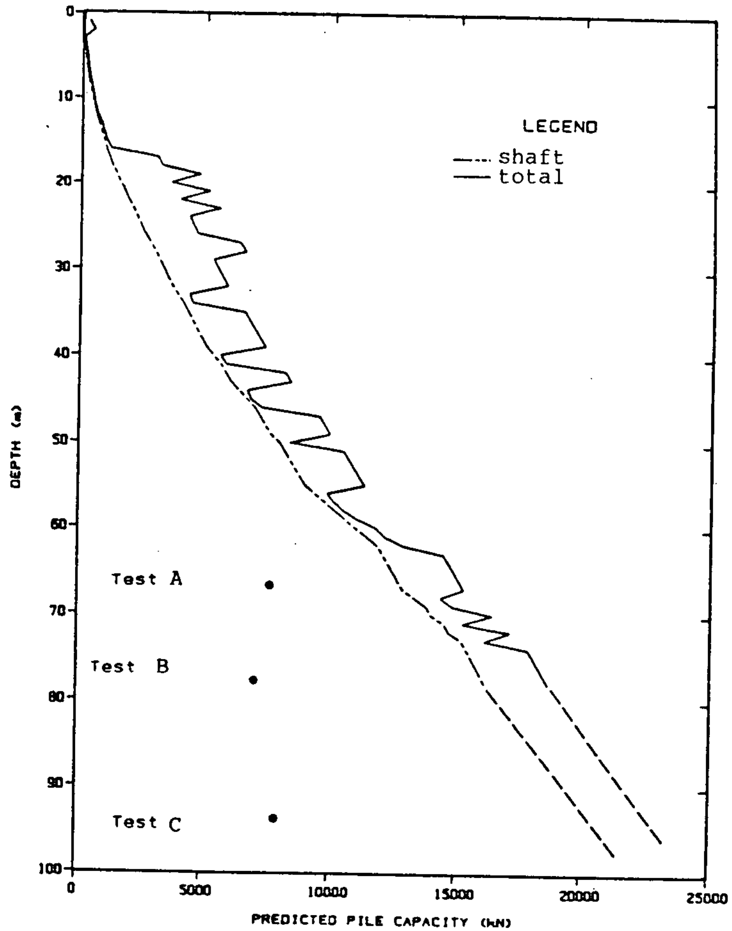
6.4.5 Janbu Method

6.4.5.1 Outline

The Janbu method (Janbu, 1976) uses an effective stress analysis. As seen in Table 6.2, both the skin friction and end bearing formulations are in terms of effective overburden stress level. Janbu (1976) makes no



UBC PILE RESEARCH SITE
Burland Method
(a)



MOTH PILE LOAD TEST SITE
Burland Method
(b)

FIG. 6.10. BURLAND METHOD

reference to any validation of his method. In addition, no specific limitations of the method are noted.

Although computationally straightforward, this method requires the evaluation of several uncommon parameters (e.g. ψ = angle of plastification). Janbu (1976) is somewhat vague about how to obtain these parameters and no direct references are supplied. For this reason unique answers between individual users of this method are unlikely.

6.4.5.2 Results

As shown in Fig. 6.11(a), the Janbu method overpredicted all of the measured capacities for the UBCPRS piles. This overprediction ranged from 15 to nearly 100 percent. The method predicted very large end bearing capacities in the sand (15 m to 30 m).

For the MOTHPRS, larger overpredictions result. As shown in Fig. 6.11(b), the predicted pile capacity is greater than 200% larger the actual measured capacity for test C.

6.4.6 Meyerhof Conventional Method

6.4.6.1 Outline

The Meyerhof conventional method is as presented in 1976 as the eleventh Terzaghi lecture to the American Society of Civil Engineers (Geotechnical Engineering Division). This method, as can be seen in Table 6.2, is similar to that of the American Petroleum Institute (API RP2A). The main difference is that Meyerhof suggests the use of limiting skin friction and end bearing values based upon field observations.

Meyerhof's method is validated by comparing measured field results from many authors. Unfortunately, Meyerhof offers no mention as to the size range of the piles involved in the field load tests.

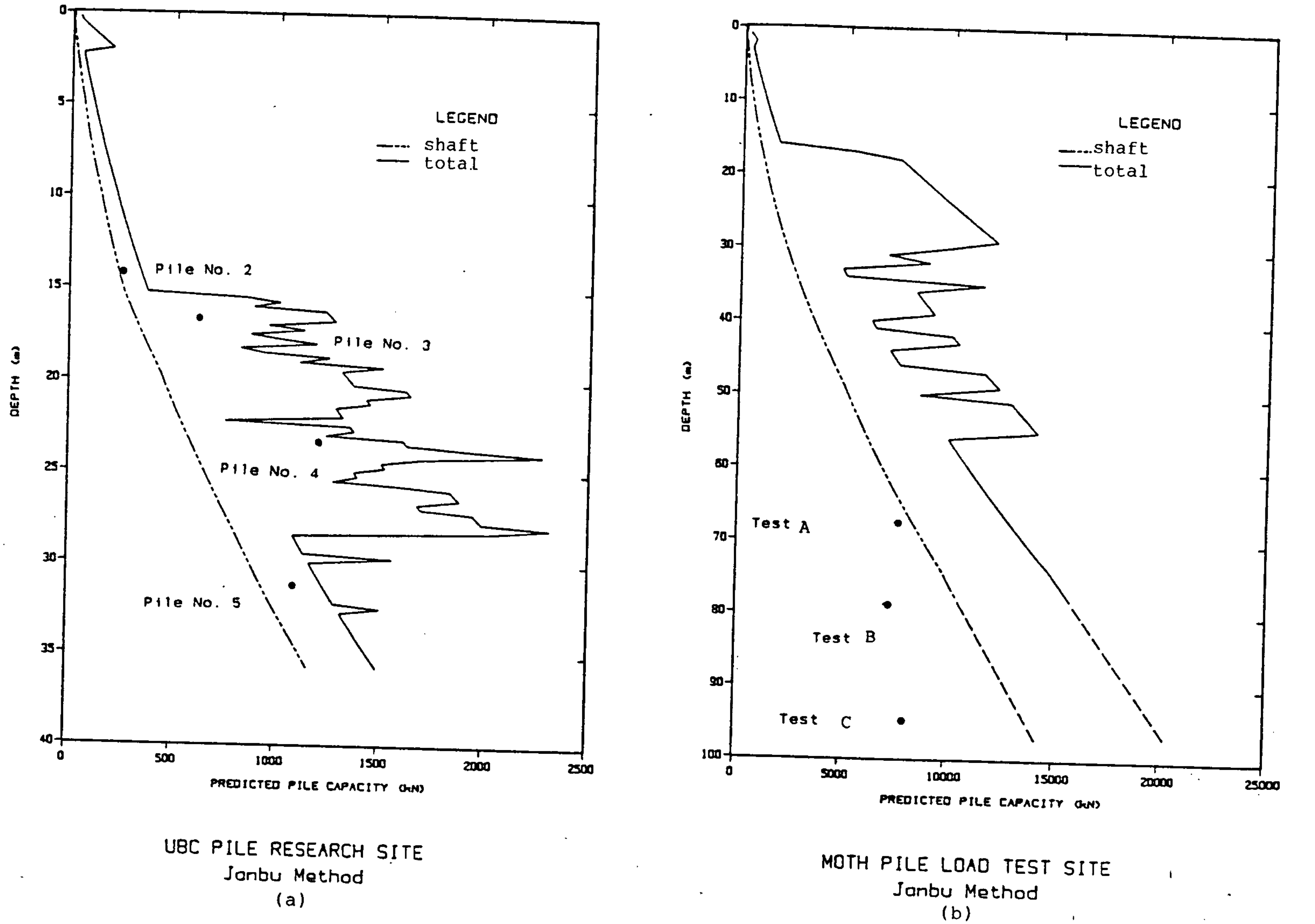


FIG. 6.11. JANBU METHOD

This method is simple to use and, unlike the API RP2A method, recommended values for parameters such as the coefficient of lateral earth pressure are clearly presented.

6.4.6.2 Results

As shown in Fig. 6.12(a), the Meyerhof conventional method predicted the capacities for the UBCPRS piles quite well. The capacity of pile no. 2 was almost precisely predicted whereas the capacities of the other three piles were only slightly overpredicted.

For the MOTHPRS, large overpredictions of measured capacity result. As shown in Fig. 6.12(b), the predicted pile capacity is in the order of 200-300% of the measured capacity for tests A, B and C.

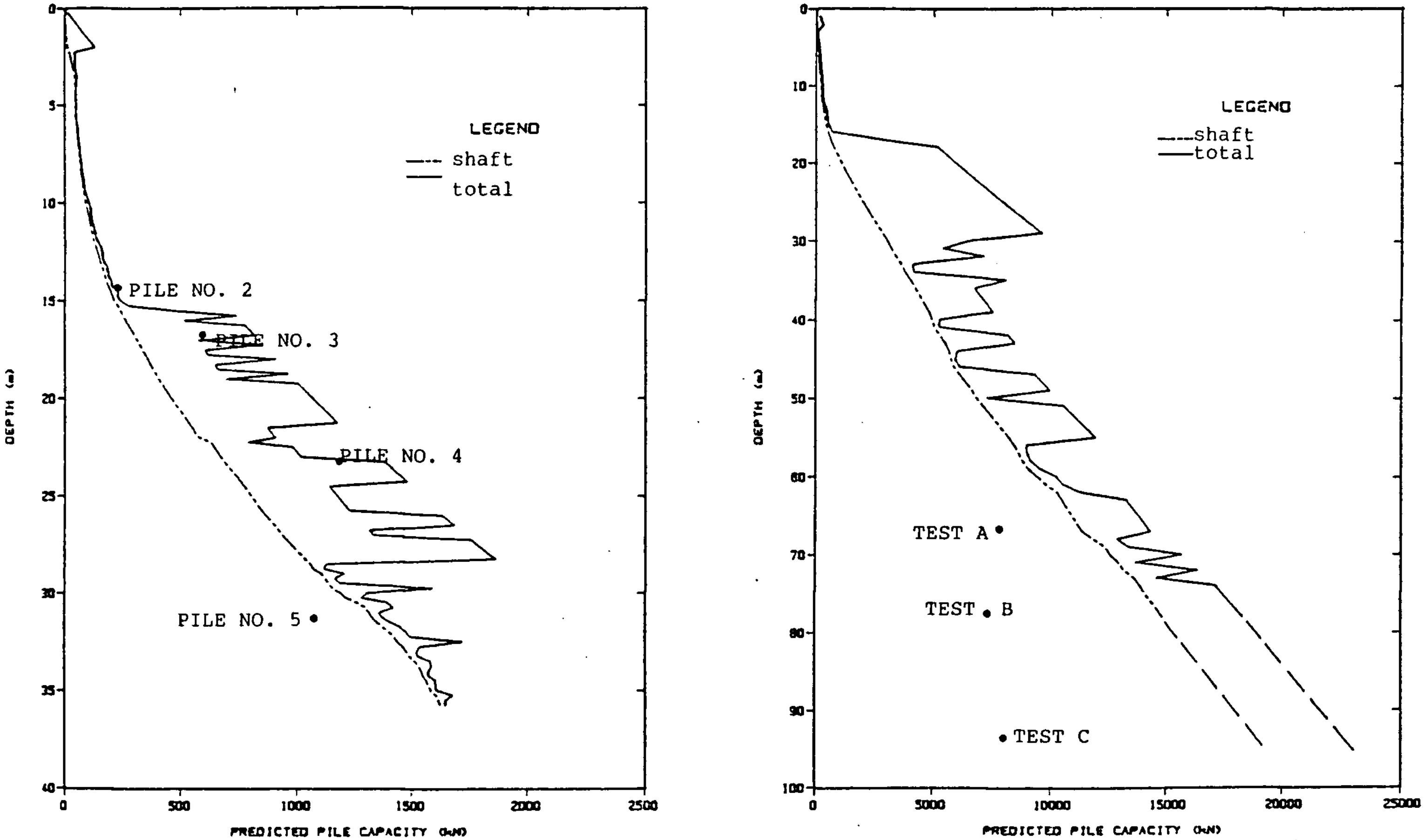
6.4.7 Flaate and Selnes Method

6.4.7.1 Outline

The Flaate and Selnes method (Flaate and Selnes, 1977), like the Vijayvergiya and Focht and Burland methods, was originally developed only for cohesive soils (see Table 6.2).

This method formulated an expression to determine shaft resistance in terms of effective stress, plasticity and overconsolidation ratio. An empirical factor, μ_L, was defined as a factor to relate the above parameters and the pile length. Pile length was included so that the reduction in mobilized side friction with increased pile length could be included.

This method was validated using results from 44 full-scale load tests. The piles were mainly timber pies up to 200 mm in diameter and ranging in length from 7 to 24 metres. In addition several concrete and steel pipe



UBC PILE RESEARCH SITE
Meyerhof Conventional Method
(a)

MOTH PILE LOAD TEST SITE
Meyerhof Conventional Method
(b)

FIG. 6.12. MEYERHOF CONVENTIONAL METHOD

piles, up to 470 mm in diameter and 23 metres in length, were also investigated by the authors.

This method is simple to use but requires obtaining values of plasticity index and overconsolidation ratio; two quantities that cannot yet be determined confidently with CPT.

6.4.7.2 Results

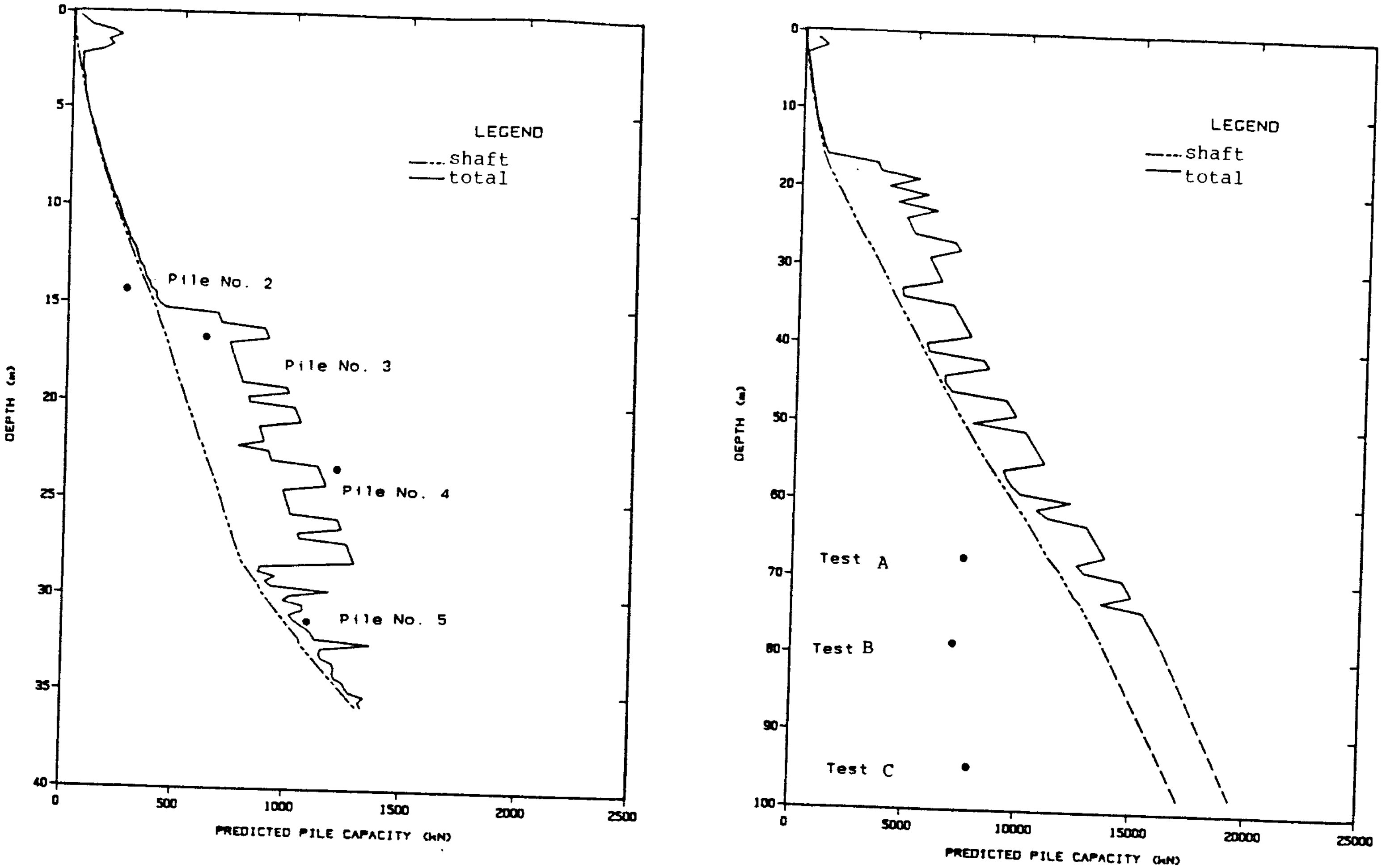
The results for the prediction of pile capacity at the UBCPRS using the Flaate and Selnes method are shown in Fig. 6.13(a). Good agreement between predicted and measured behaviour is seen, especially for pile no. 5.

For the MOTHPRS, shown in Fig. 6.13(b), the predicted results greatly overpredict the measured capacity. For all three tests the predicted capacity is generally at least 100% too large. These results were almost as poor as for the Janbu method, which was considered the worst method evaluated.

6.5 Dynamic Methods

6.5.1 Introduction

As discussed in Chapter 2, dynamic methods can be divided into "prediction" and "in-situ" classes. For this study the prediction methods used were the Engineering News Record (ENR) dynamic formula and the wave equation. WEAP86, is an interactive wave equation program to simulate the soil-pile system. The "in-situ" measurements used for dynamic prediction were the Case Method (using Goble, Raushe and Likens Ltd. PDA) and CAPWAP. In all cases, pile no. 5 from the UBCPRS was used. A summary of the calculations performed for all methods are presented in Appendix VI.



UBC PILE RESEARCH SITE
Floate and Selnes Method
(a)

MOTH PILE LOAD TEST SITE
Floate and Selnes Method
(b)

FIG. 6.13. FLAATE AND SELNES METHOD

6.5.2 Results

The ENR formula has the following form:

$$R = \frac{2 \cdot W_H \cdot H}{S + 0.1} \qquad (6.1)$$

where: R = capacity under working conditions (kips)

W_H = weight of hammer (kips)

H = hammer drop height (feet)

S = set (inches)

In the above equation a factor of safety of 6 is recommended. Therefore to get the predicted ultimate capacity the result obtained must be multiplied by 6. For the initial driving of pile no. 5 the predicted ultimate capacity was 1944 kN. During restrike, when the result should be more indicative of the static capacity, the predicted ultimate capacity was 3114 kN.

The results of the wave equation analysis on pile no. 5, using WEAP86, are presented in Figs. 6.14 and 6.15. In Fig. 6.14 the effect of varying hammer efficiency is illustrated. Depending upon whether a hammer efficiency of 60% or 70% is chosen (typical ranges for drop hammers), a different dynamic capacity will result. The other problem that arises is that a tip resistance to shaft resistance ratio must be chosen. As can be seen in Fig. 6.15, the influence of the value of this ratio chosen has a significant effect on the result. From the static analysis using CPT, an approximate tip resistance to shaft resistance ratio of 20:80 for pile no 5 was determined. Using initial restrike hammer blowcount data it is justifiable to assume that this ratio can be used to calculate dynamic

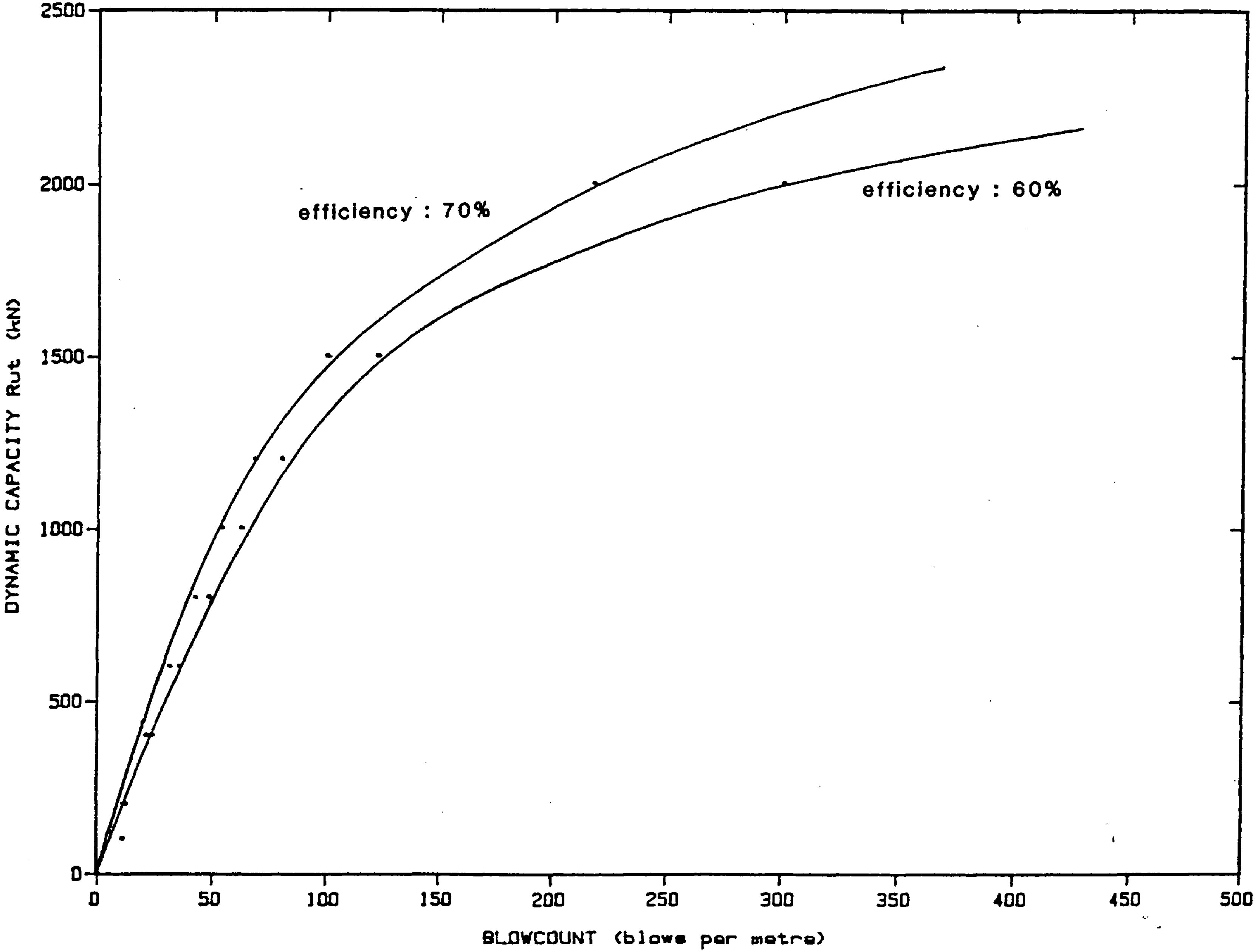


FIG. 6.14. UBC PILE RESEARCH SITE WEAP86: PILE NO. 5
VARYING HAMMER EFFICIENCY

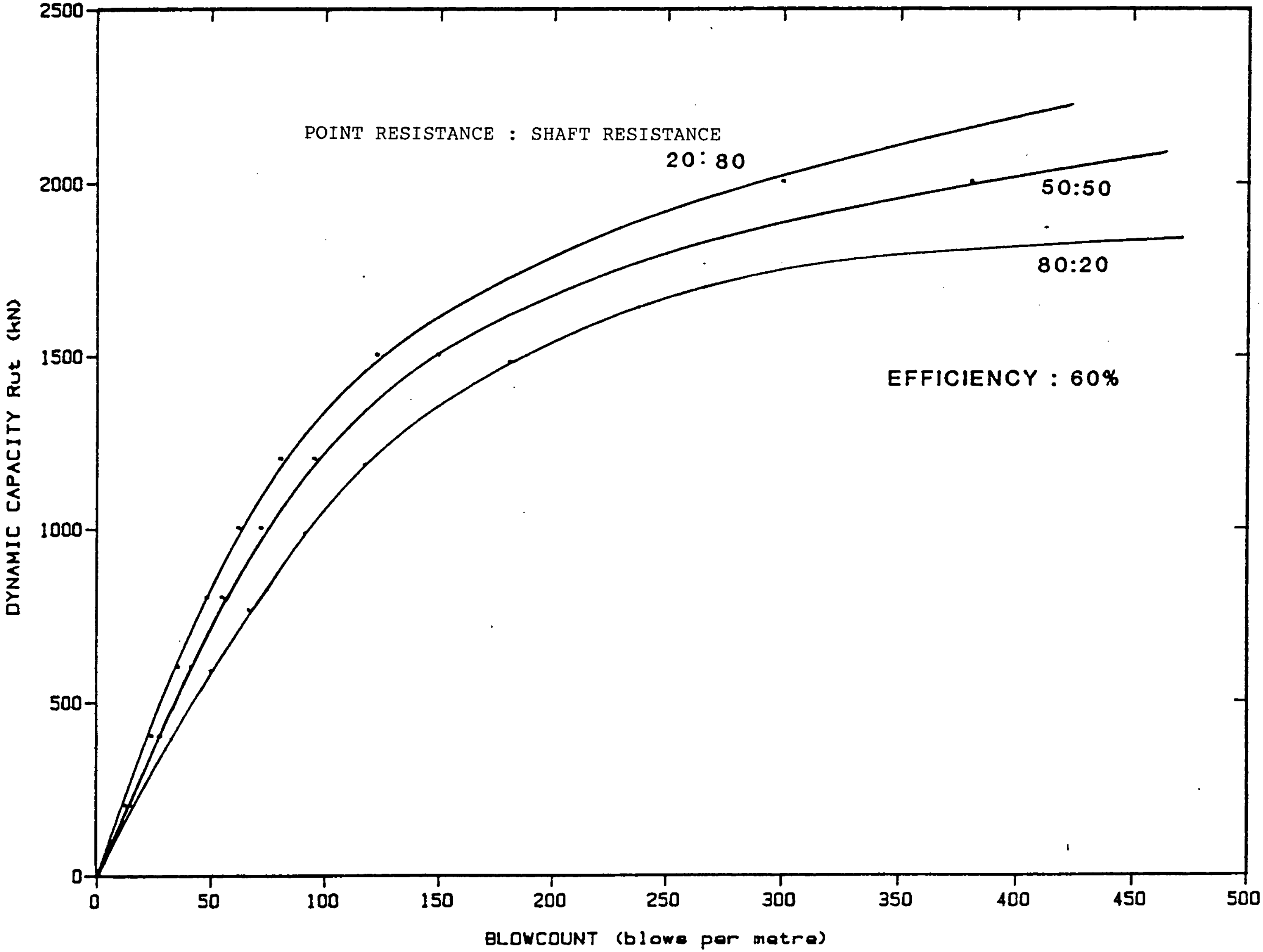


FIG. 6.15. UBC PILE RESEARCH SITE WEAP86 : PILE NO. 5
VARYING SHAFT RESISTANCE TO TIP RESISTANCE RATIO

capacity. Also, during initial restrike the effects of pile set up are reflected in the measured blow count, therefore it appears reasonable that this will approximate a static capacity prediction. Therefore, assuming that the efficiency of the hammer is 60% and that the tip resistance:shaft resistance ratio is 20:80, a capacity can be predicted. From initial restrike data on pile no. 5 the blowcount was 80 blows per metre. This results in a predicted capacity (using Fig. 6.15) of 1230 kN. Note that all other input values (damping, quake, etc.) used were as suggested by the WEAP86 manual.

Using in-situ data, the Case Method and CAPWAP capacities were also obtained for pile no. 5 using a pile analyzer (PDA). The case method provided predicted results of 1903 kN and 1080 kN depending upon the damping value, J_c, used (see Appendix VI) for calculation details). Using a J_c value of 0.70, the 1903 kN (and overpredicted) result is obtained. The value of 0.70 was suggested by a Goble, Rausche and Likens (GRL) representative present during the dynamic measurements upon the restriking of pile no. 5. A value of J_c equal to 1.07, as suggested by static load test results using the GRL PDA manual (see Appendix VI) yields the 1080 kN (and highly accurate) result. As will be discussed further in section 6.6.2, the choice of J_c is the single largest factor affecting accurate predictions of static pile capacity using dynamic methods. A CAPWAP capacity of 1646 kN (50% overprediction) was predicted using J_c = 0.70. Unfortunately, CAPWAP program results using a more appropriate damping value were not available for inclusion within this dissertation.

6.6 Sensitivity to Input Parameters

6.6.1 Static Methods

The accuracy of the results for any prediction method will always depend not only upon the method used but upon the "quality" of the input parameters used in that method. A pile capacity prediction method cannot be expected to perform well (give accurate predictions of measured behaviour) unless input parameters representative of the existing subsurface conditions are used. For the indirect methods, estimates of parameters such as undrained strength, angle of internal friction, and others are required by each method. For the direct methods only the accuracy of the CPT is of concern (except for the de Ruiter and Beringen, and Schmermann and Nottingham methods where both require undrained shear strength estimates).

For this study, only CPT data has been used to estimate (directly or indirectly) input parameters. To ensure the accuracy of the actual CPT results, careful field techniques and properly calibrated equipment is essential. Regardless of the CPT data being assumed accurate (i.e. repeatable and representative), the correlations used to estimate parameters using CPT data must be accurate as well or non-representative results will result. Most of the CPT parameter correlations are empirical and cannot therefore be expected to be universal. Local correlations will almost always be preferred (unless the method used indicates otherwise). As an example, the value of undrained shear strength (S_u) has been calculated using three different CPT correlations. These results have then been used to check the sensitivity of the de Ruiter and Beringen method (assumed to be a direct method as explained in Section 6.3) to the value of S_u.

Figure 6.16 presents the results of the de Ruiter and Beringen method using:

$$S_u = \frac{q_c}{15} \tag{6.2}$$

This is the value (where local correlations are unavailable) proposed by de Ruiter and Beringen (1979) but is based upon North Sea data. As can be seen in Fig. 6.16, non-conservative predictions generally result. However, if a value that is more appropriate for local conditions is used, the result is much better. Figure 6.17 shows this method using:

$$S_u = \frac{q_c - \sigma_{vo}}{15} \tag{6.3}$$

This value was chosen from field vane correlations obtained in similar soils at the UBCPRS (Greig, 1985) and in comparison with vane results at the UBCPRS as described in Chapter 4. With Eqn. 6.3 used as an estimate of undrained shear strength this method predicts the measured pile capacity very well. It was this value of undrained strength, as noted earlier, that was used for this study wherever an undrained strength estimate was required. Finally, Fig. 6.18 presents the de Ruiter and Beringen method using yet another formulation for S_u:

$$S_u = \frac{q_c - \sigma_{vo}}{10} \tag{6.4}$$

Once again, as with Eqn. 6.2, a significant descrepancy between predicted and measured pile capacity is evident (non-conservative predictions).

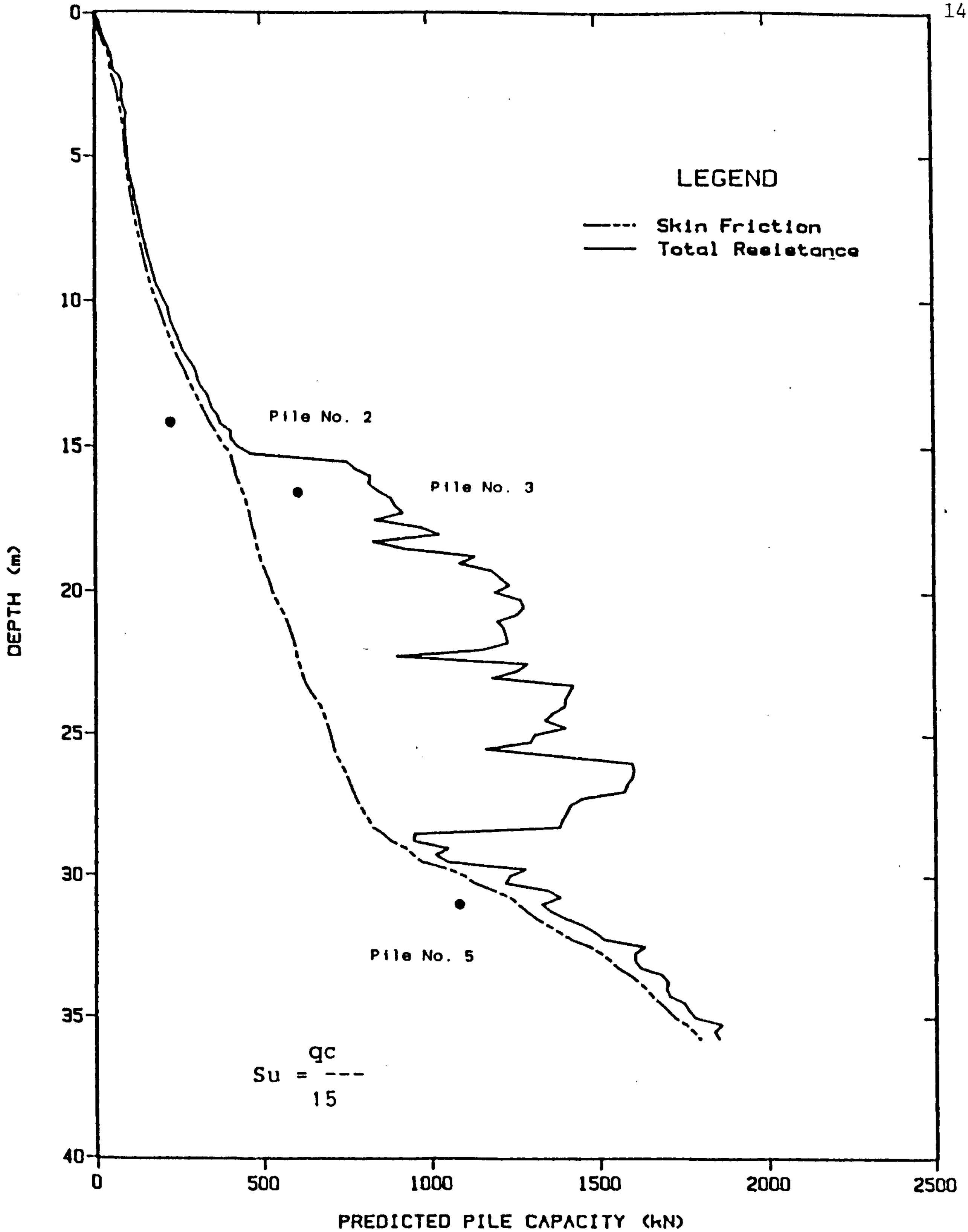


FIG. 6.16. UNDRAINED STRENGTH NO. 1

UBC PILE RESEARCH SITE
deRuiter and Beringen CPT Method

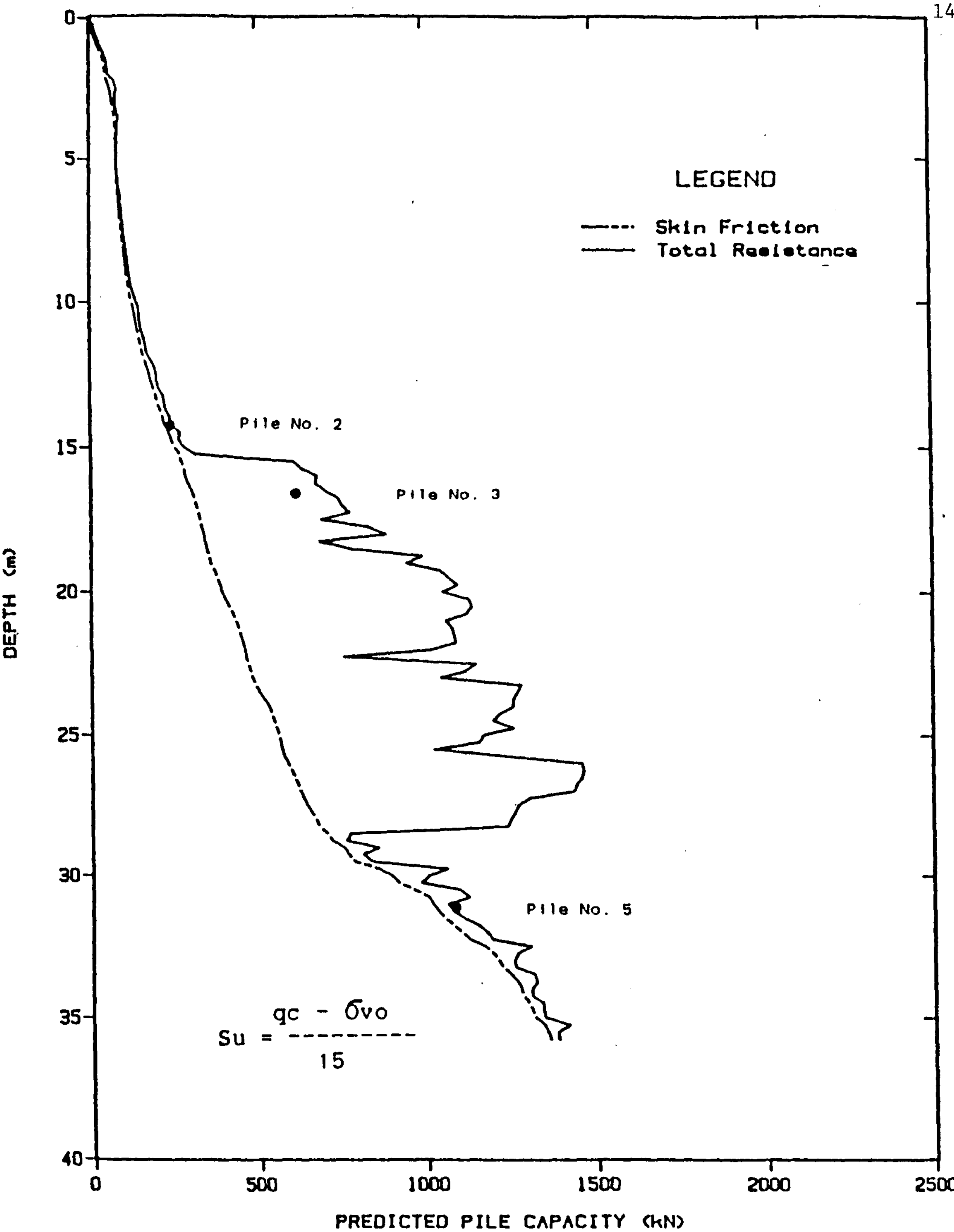


FIG. 6.17. UNDRAINED STRENGTH NO. 2

UBC PILE RESEARCH SITE
deRuiter and Beringen CPT Method

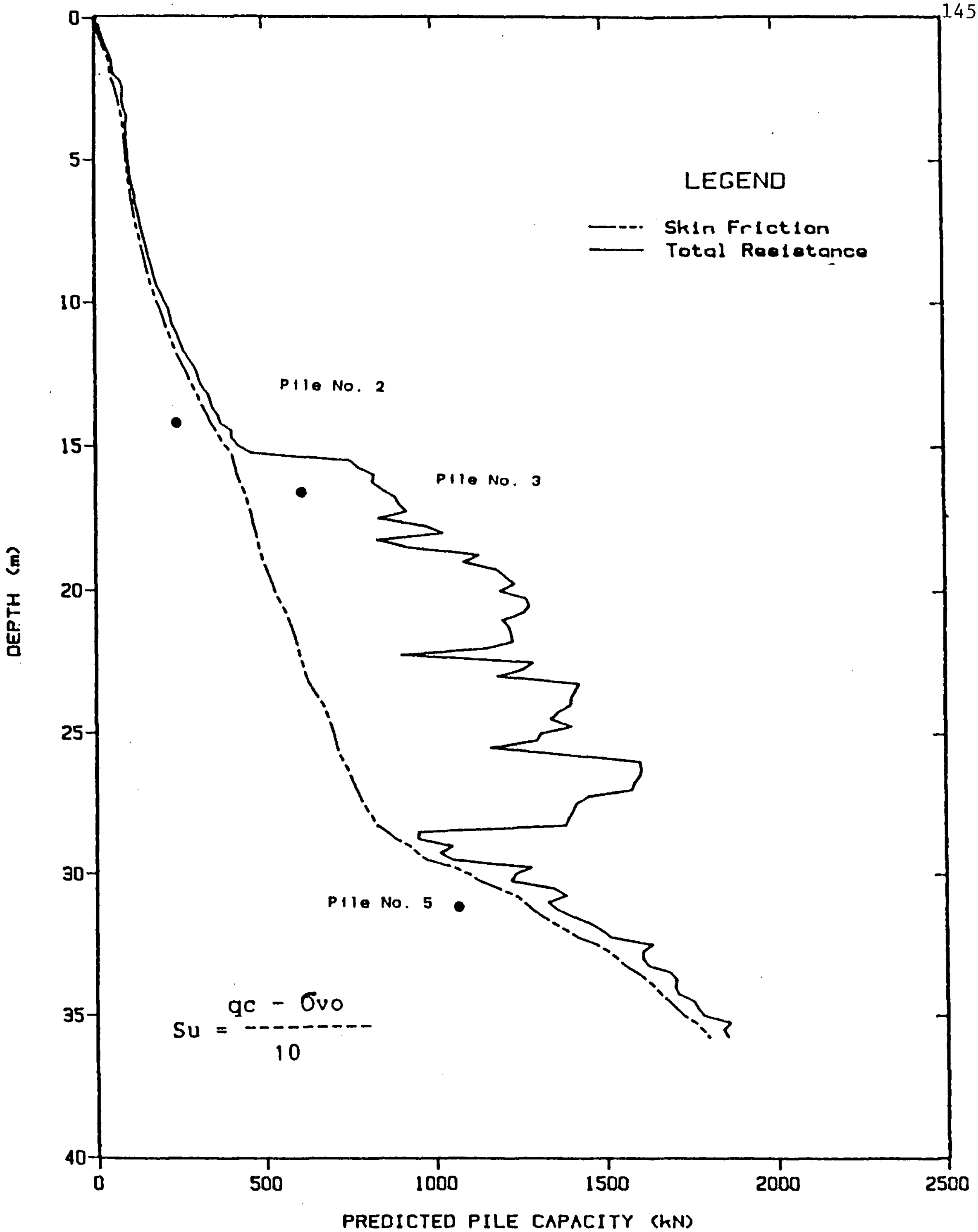


FIG. 6.18. UNDRAINED STRENGTH NO. 3

UBC PILE RESEARCH SITE
deRuiter and Beringen CPT Method

This simple example illustrates the importance of performing sensitivity analyses when performing pile capacity analyses. This is especially true when, as is the case using CPT data, correlations that may not reflect local conditions are to be used.

6.6.2 Dynamic Methods

As was shown in Section 6.5.2, using damping values obtained from correlations with full-scale load test results, the Case Method can provide an excellent prediction of static axial capacity when initial restrike data is analyzed. It was also shown that choosing a value without the advantage of load test results can lead to significant error. For accurate dynamic analyses of piles, the damping characteristics of the soil must be properly evaluated. Unfortunately, little improvement in the manner by which damping values are chosen can be noted in published literature over the past 10-15 years. Damping values are also important input parameters for other wave equation analysis of piles (e.g., WEAP86 or CAPWAP). In addition, wave equation methods require accurate assessments of soil quake and skin friction distrbution profile along the pile length if accurate predictions are to result. Unfortunately, these values are seldom determined on a site specific basis and "recommended" values from operations manuals are usually used. These values are generally quoted in ranges such that over 100% in variation can result from using extreme values. In addition, these recommended values may not reflect at all the actual site charateristics of interest.

In-situ testing methods, particularly the CPT, have the potential to vastly improve the accuracy of input parameters such as soil damping, soil quake and skin friction distribution. For the soil damping, a simple empirical correlation between the case damping constant, J_c, and the ratio between cone resistance (q_c) and friction ratio (FR,%), q_c/FR,%, can be proposed as shown in Fig. 6.19. The data used for J_c is after Rausche et al. (1985). Note that Fig. 6.19 should be adjusted to reflect local experience. It is interesting to note that for UBCPRS pile no. 5, the (q_c/FR,%) ratio near the pile tip ranges from 7 to 11. Thus, using a conservative upper bound trend line, Fig. 6.19 yields a case damping value of approximately 1.0. This is in close agreement with the J_c value computed from static load test results, as was shown in Section 6.5.2.

Soil quake, or the elastic ground compression, is a concept based on a simplistic elasto-plastic soil model proposed by Chellis (1951). The quake, Q, is the displacement at which the soil becomes plastic as shown in Fig. 6.20. Note also in Fig. 6.20 that a determination of ultimate static soil resistance, R_u, is also required. Traditionally a standard quake value of 0.1 inch (2.5 mm) is generally used for all soils, based on the original work of Smith (1960). However, real soil does not behave in such a simplistic manner. Using either CPT data, to develop parameters for a more representative soil model, or modified PMT curves seems a more logical approach of evaluating the stress-strain soil behaviour necessary for wave equation analysis.

Finally, the shaft resistance distribution profile for the pile-soil system must be estimated in order to perform a wave equation analysis. Usually either a constant value with depth or a triangular distribution is chosen with little regard for the prevailing stratigraphy. Using the CPT sleeve friction values, scaled from 0 to 100, provides a profile (with

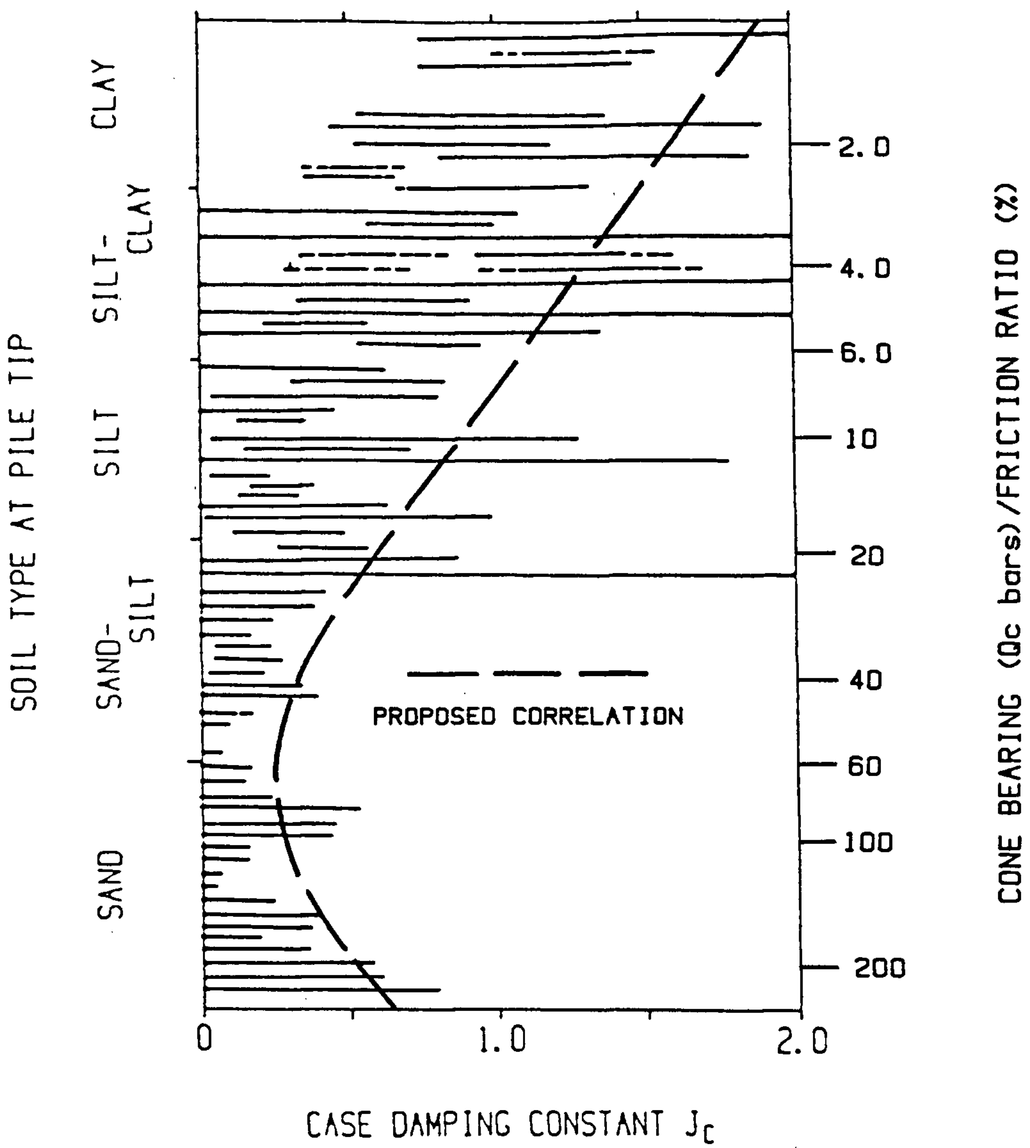


FIG. 6.19. PROPOSED CORRELATION BETWEEN CPT DATA AND CASE DAMPING CONSTANT, Jc

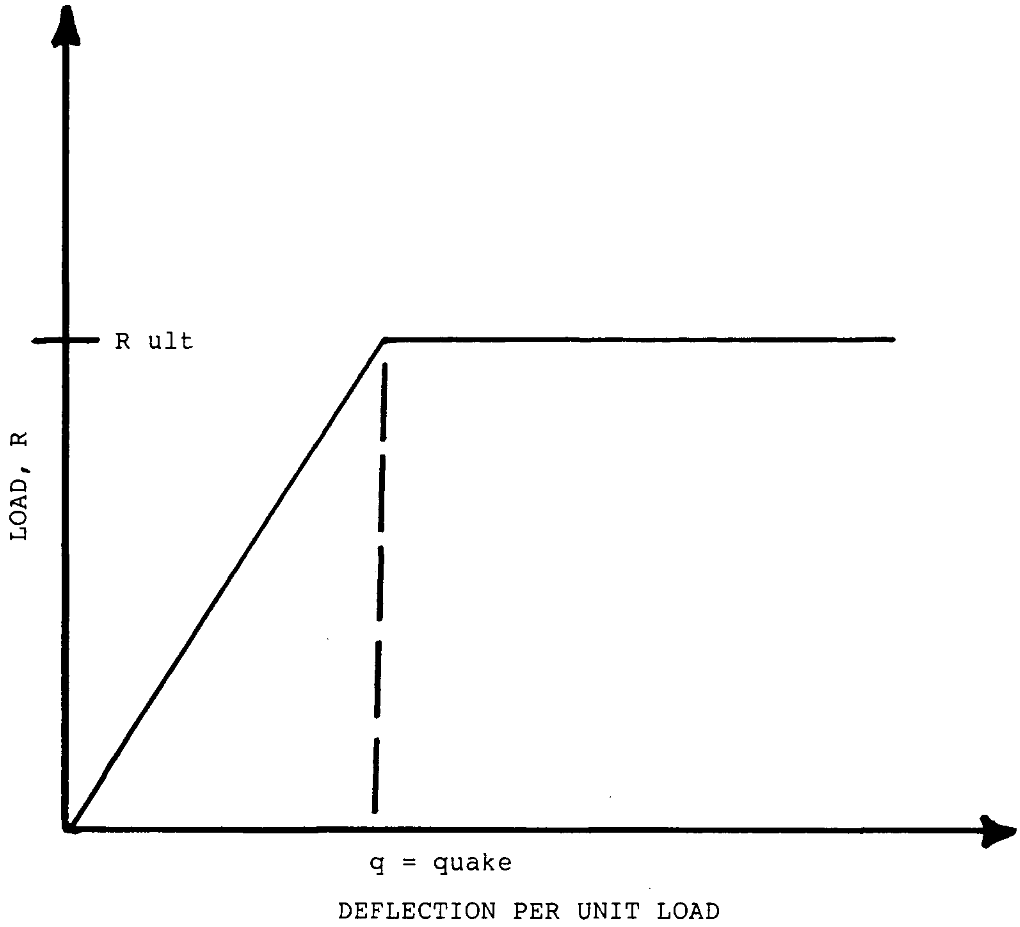


FIG. 6.20. ELASTO-PLASTIC SOIL MODEL
(ADAPTED FROM CHELLIS, 1951)

appropriate scaling) of pile-soil interaction. These values of CPT sleeve friction should only be used, however, for analyzing the start of restrike condition when approximately static resistance is measured. This is because the CPT value will generally not accurately model the dynamic pile-soil condition. However, this is not strictly correct because the CPT is not a truly static penetration test but must be considered a "quasi-static" penetration test. This is especially true in soft clays where the CPT penetration will cause large excess pore pressures to be generated.

All of the above is presented to demonstrate that careful selection of input parameters for dynamic pile analysis is crucial. The use of pile dynamics, particularly in-situ dynamic measuring methods, will increase if the methods can be shown to provide accurate results. At present, this accuracy is inhibited by the poor quality of the soil input parameters. More representative soil parameters and soil models must be adapted.

6.7 Discussion of Axial Pile Capacity Prediction

Figure 6.21 summarizes the results of all the static methods evaluated in the form of bar charts for each method. Note that, with few exceptions, both the direct and the indirect methods provided reasonable predictions of the measured capacities of the smaller piles. The direct methods, the Zhou et al. (1982) method to a lesser extent, also predicted the capacity of the larger pile quite satisfactorily. However, without exception, the indirect methods had predictions that were significantly in error and non-conservative when compared to the measured results for the large pile. Since the indirect methods generally did reasonably well in predicting the capacity of the smaller piles, and since the piles are all in the same deltaic soil deposits, the results suggest that scale effects are extremely

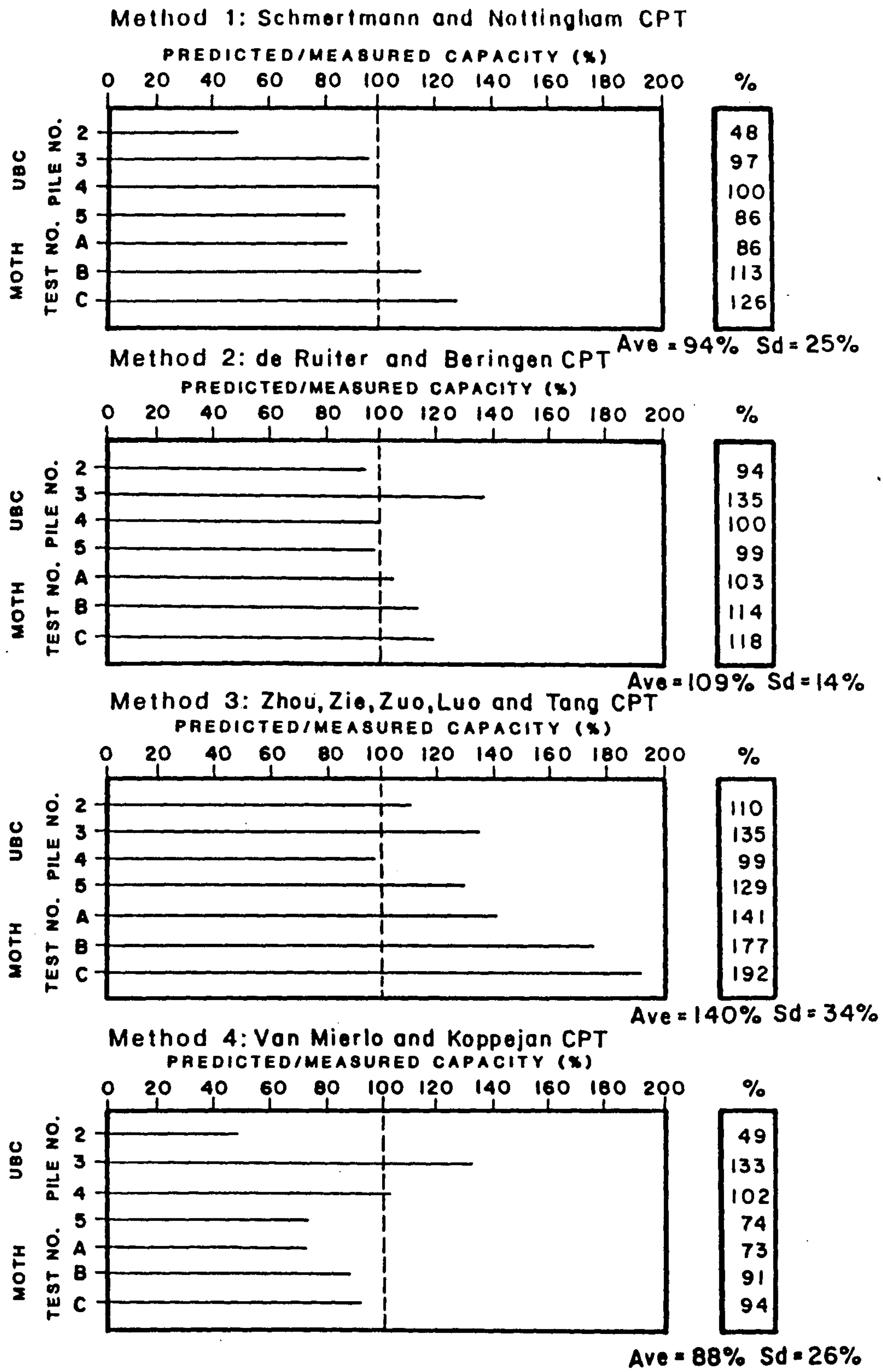
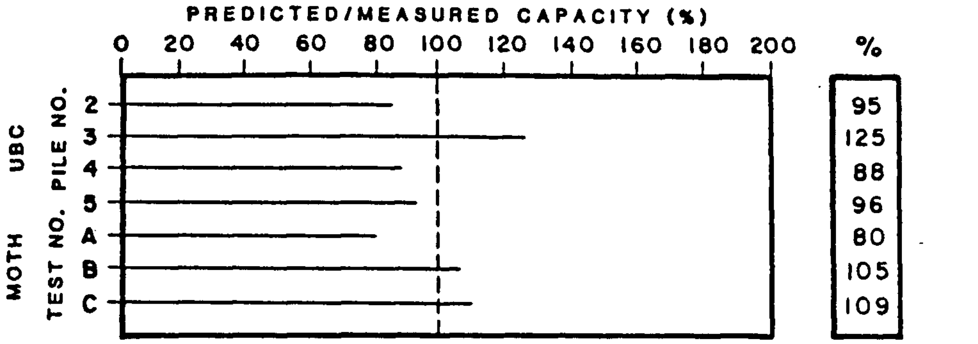
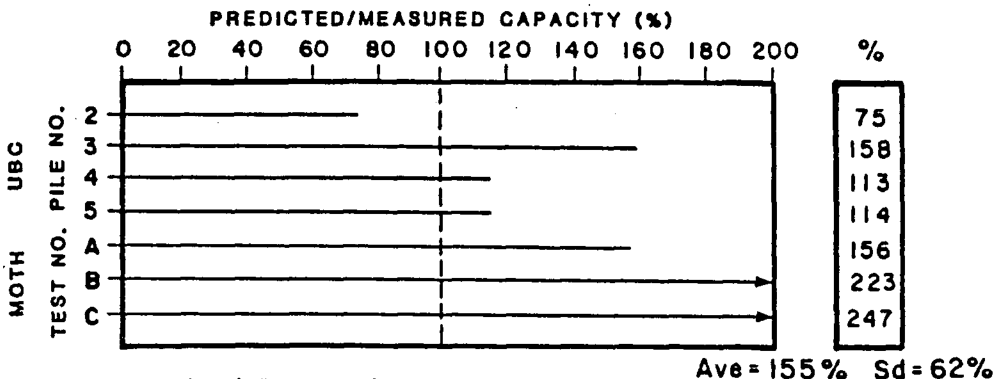


FIG. 6.21. BAR CHARTS OF PREDICTED VERSUS MEASURED PILE CAPACITY FOR STATIC PREDICTION METHODS EVALUATED

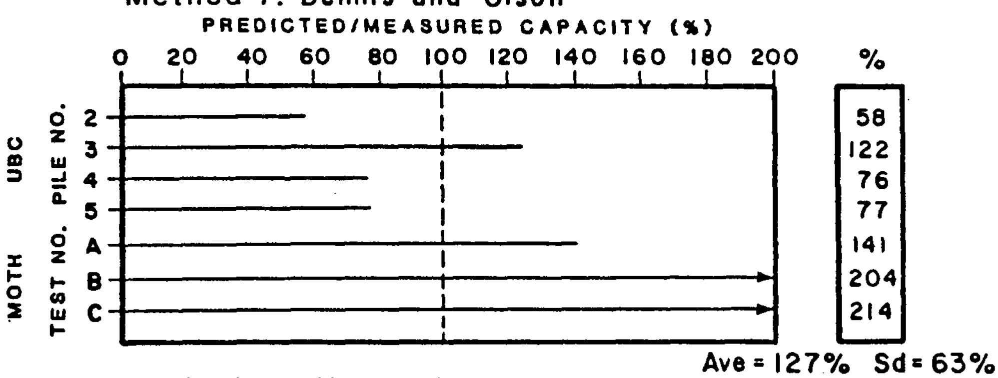
Method 5: LCPC CPT



Method 6: API RP2A



Method 7: Dennis and Olson



Method 8: Vijayvergiya and Focht

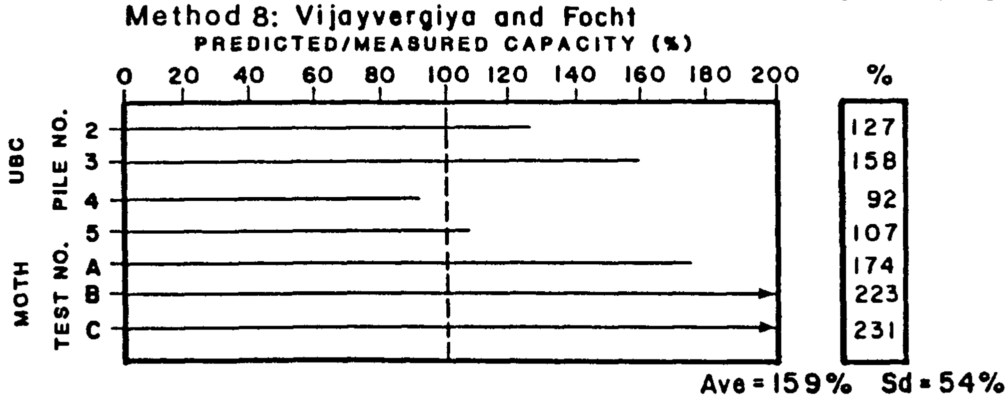
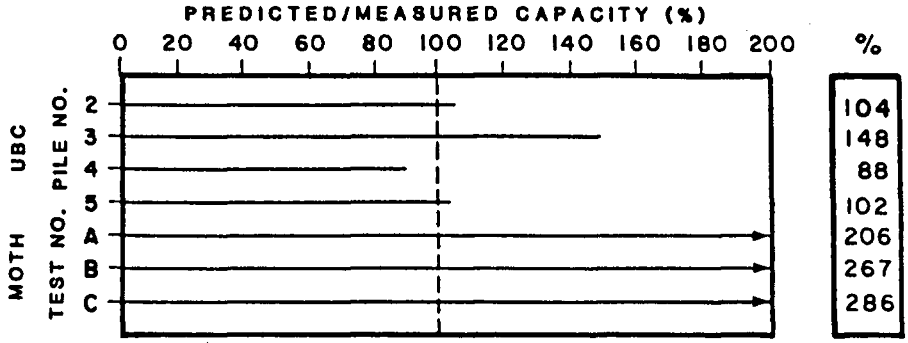


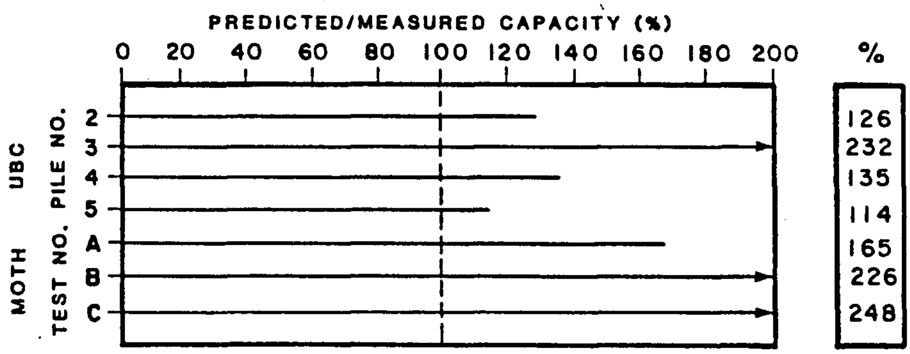
FIG. 6.21. CONT...

Method 9: Burland



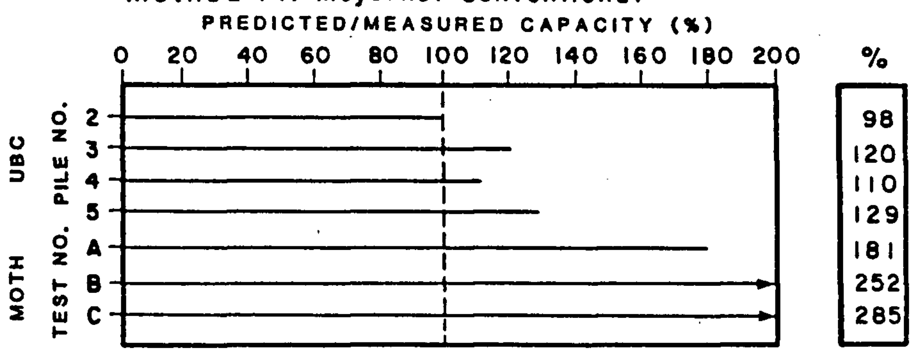
Ave = 172% Sd = 82%

Method 10: Janbu



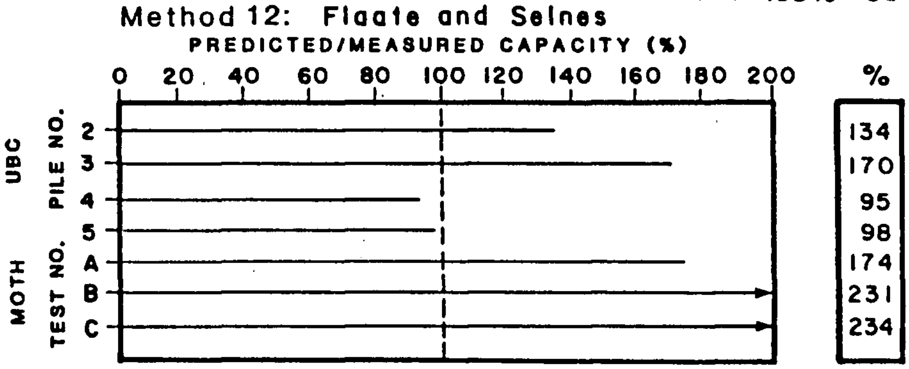
Ave = 178% Sd = 56%

Method 11: Meyerhof Conventional



Ave = 168% Sd = 74%

Method 12: Flaate and Selnes



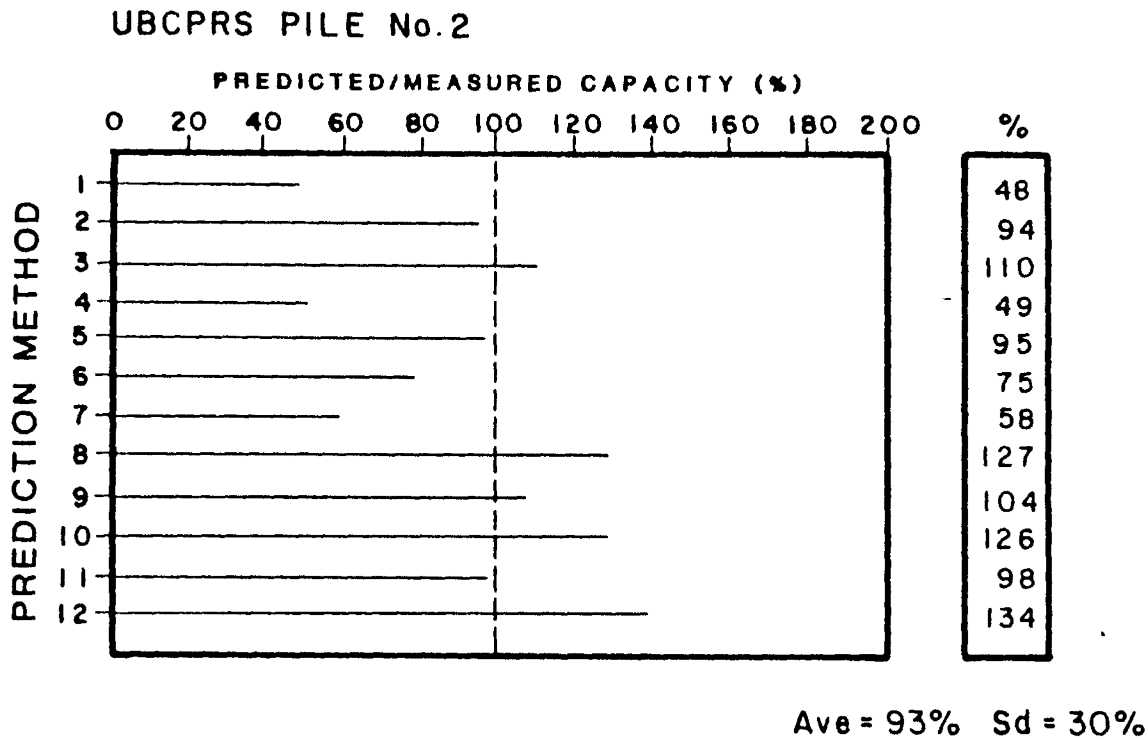
Ave = 162% Sd = 57%

FIG. 6.21. CONT.

important for the large diameter pile. Most of the indirect methods are empirical in nature and based upon observed results from piles considerably smaller than 915 mm in diameter and 100 m in length. The direct methods, on the other hand, while also themselves generally empirical, all have scaling factors in their make-up (as described in Section 6.3) that allow the problem of pile size to be addressed in a consistent fashion.

When the bar charts are drawn for each pile (Fig. 6.22, see Table 6.1 for the prediction method corresponding to each number listed) the effect described above becomes even more apparent.

In Chapter 5, when the tell-tale data was analyzed, the calculated raito of shaft resistance to total resistance was shown to be approximately 80%. To further evaluated the twelve static predictions methods, Tables 6.3 and 6.4 present the predicted shaft resistance ratios versus measured and predicted total resistance respectively. It is interesting to note that, as shown in Table 6.3, the average ratio for all twelve methods is quite close to the calculated value (93% versus 80%). The method that was closest to the actual shaft resistance/total resistance ratio was the Schmertmann and Nottingham CPT method. This method ws also shown earlier to predict very well the capacities of all the piles investigated. Table 6.4 shows that, with only two exceptions, the predicted shaft resistance to predicted shaft resistance to predicted total resistance ratios were all greater than 90%. Tables 6.3 and 6.4 demonstrate that while many methods were shown to predict the total resistance of pile no. 5 quite well, few actually predicted the assumed correct (as calculated by tell-tale data) ratio of resistance between the shaft resistance and end bearing components.



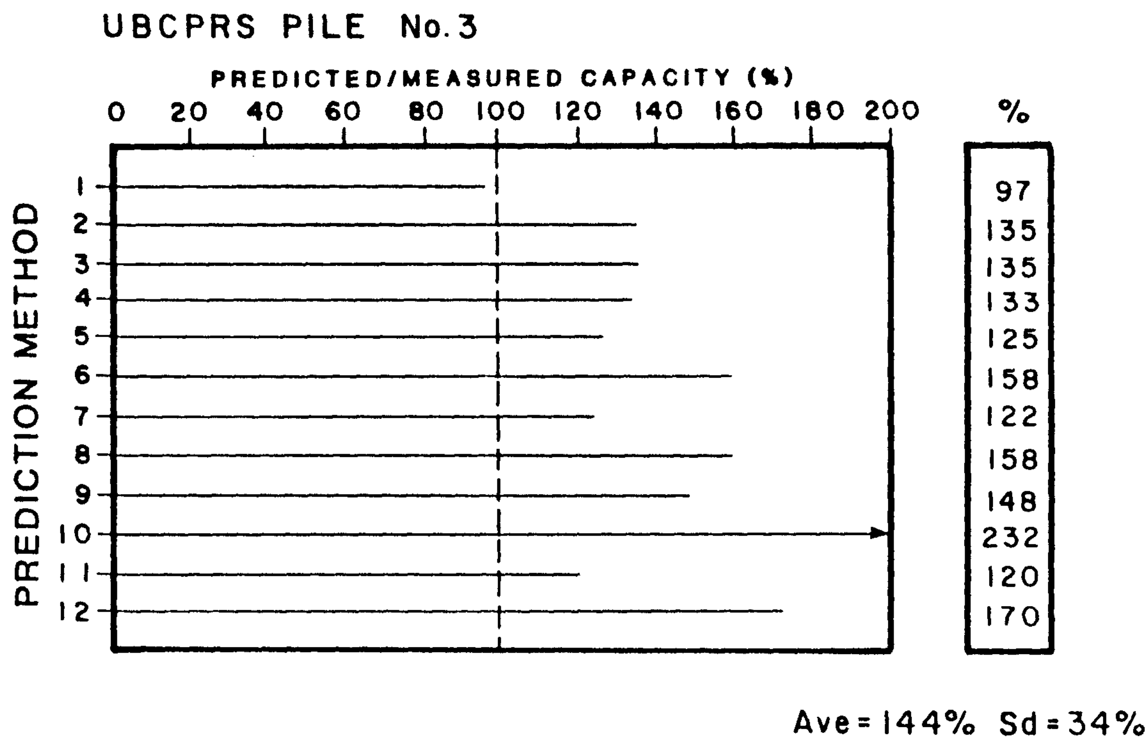
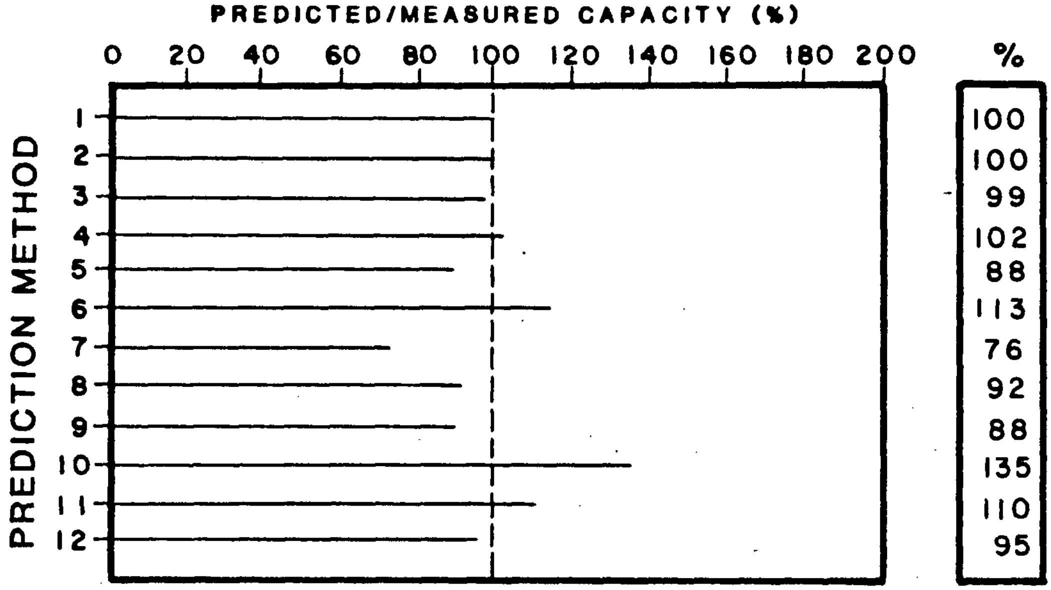


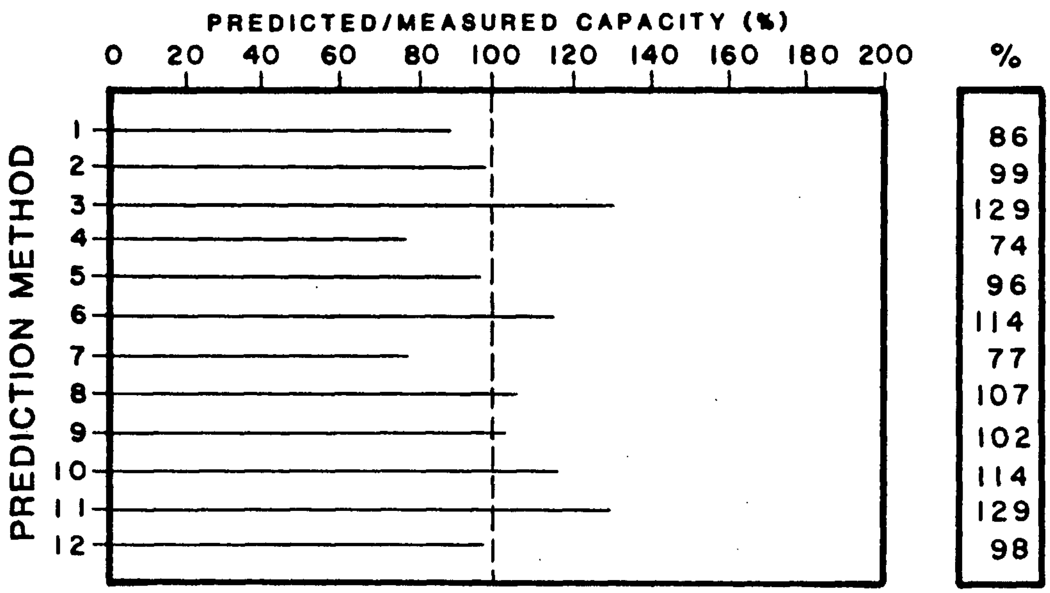
FIG. 6.22. BAR CHARTS OF PREDICTED VERSUS MEASURED PILE CAPACITY FOR PILES ANALYZED

UBCPRS PILE No. 4



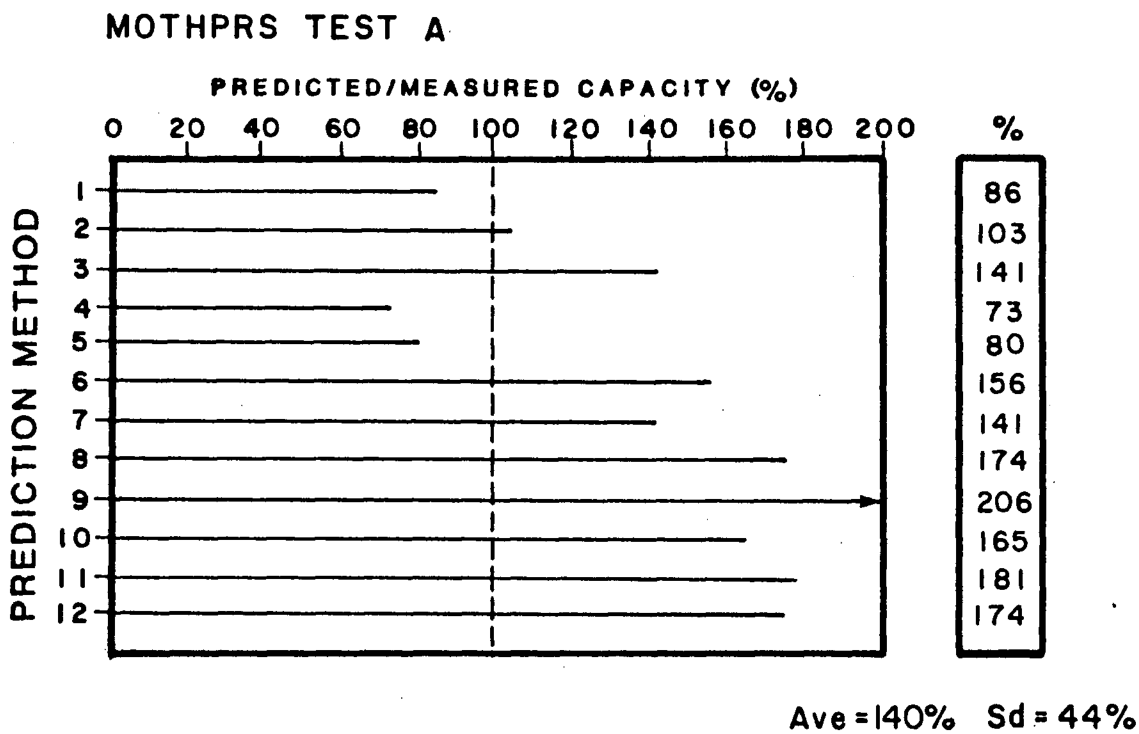
Ave = 100% Sd = 15%

UBCPRS PILE No.5

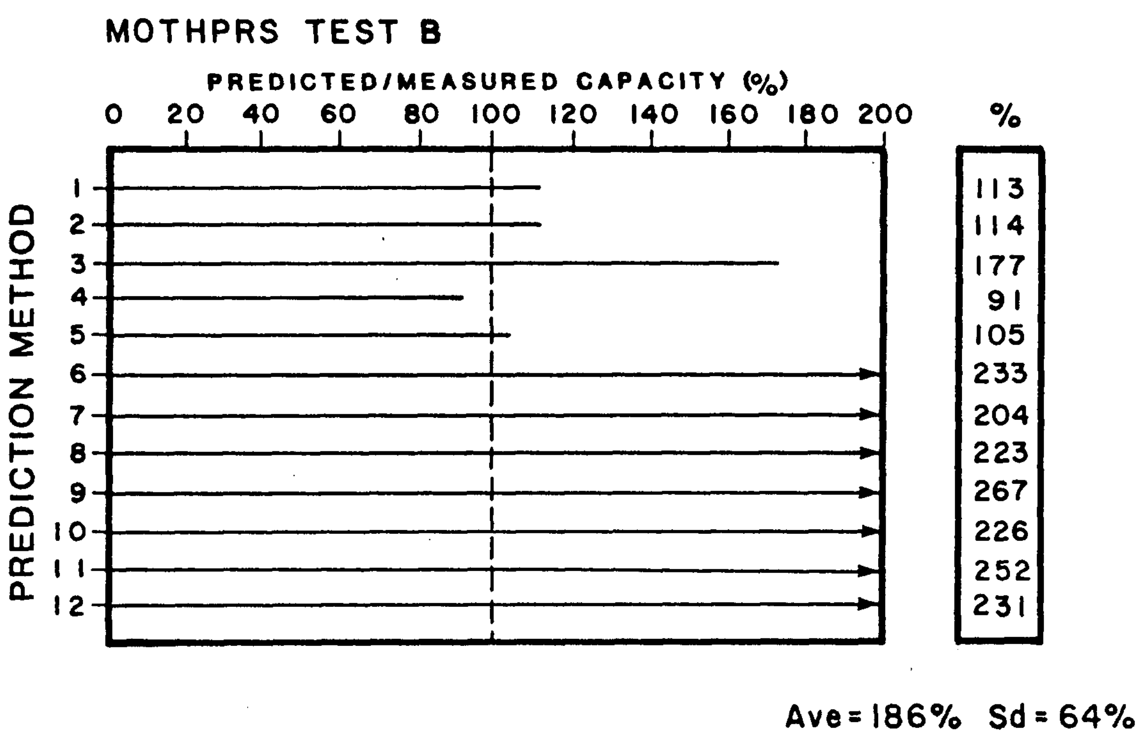


Ave = 102% Sd = 18%

FIG. 6.22. CONT...



MOTHPRS TEST A
PREDICTED/MEASURED CAPACITY (%)
0 20 40 60 80 100 120 140 160 180 200
%
PREDICTION METHOD
1 2 3 4 5 6 7 8 9 10 11 12
86
103
141
73
80
156
141
174
206
165
181
174
Ave = 140% Sd = 44%



MOTHPRS TEST B
PREDICTED/MEASURED CAPACITY (%)
0 20 40 60 80 100 120 140 160 180 200
%
PREDICTION METHOD
1 2 3 4 5 6 7 8 9 10 11 12
113
114
177
91
105
233
204
223
267
226
252
231
Ave = 186% Sd = 64%

FIG. 6.22. CONT...

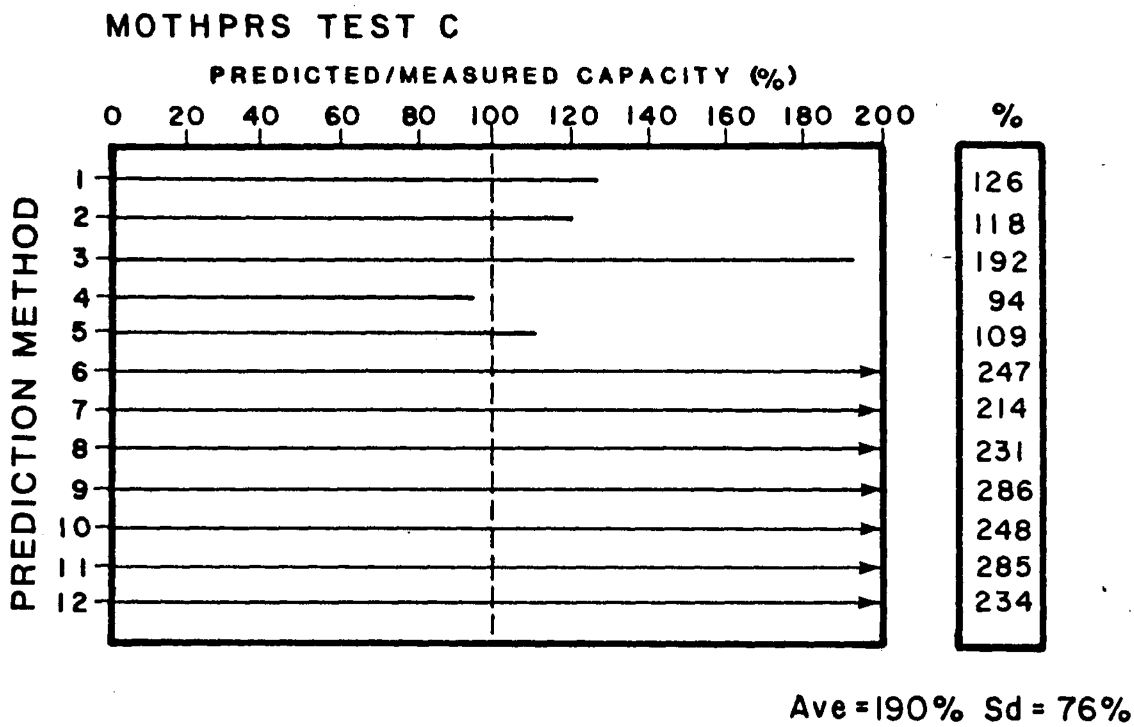

MOTHPRS TEST C
PREDICTED/MEASURED CAPACITY (%)
0 20 40 60 80 100 120 140 160 180 200
%
PREDICTION METHOD
1 2 3 4 5 6 7 8 9 10 11 12
126
118
192
94
109
247
214
231
286
248
285
234
Ave = 190% Sd = 76%

FIG. 6.22. CONT.

TABLE 6.3 PREDICTED SHAFT RESISTANCE AS A PERCENTAGE OF TOTAL MEASURED AXIAL CAPACITY FOR PILE NO. 5

	Method	Predicted Shaft Resistance/ Measured Total Resistance (%)
1.	Schmertmann & Nottingham CPT	82
2.	deRuiter & Beringen CPT	95
3.	Zhou et al. CPT	120
4.	Van Mierlo & Koppejan CPT	49
5.	LCPC CPT	86
6.	API RP2A	108
7.	Dennis & Olson	75
8.	Vijayvergiya & Focht	102
9.	Burland	95
10.	Janbu	86
11.	Meyerhof Conventional	123
12.	Flaate & Selnes	90
	Ave:	92.6
	Sd:	20.1

TABLE 6.4 PREDICTED SHAFT RESISTANCE AS A PERCENTAGE OF TOTAL PREDICTED AXIAL CAPACITY FOR PILE NO. 5

	Method	Predicted Shaft Resistance/ Predicted Total Resistance (%)
1.	Schmertmann & Nottingham CPT	96
2.	deRuiter & Beringen CPT	95
3.	Zhou et al. CPT	93
4.	Van Mierlo & Koppejan CPT	77
5.	LCPC CPT	91
6.	API RP2A	95
7.	Dennis & Olson	94
8.	Vijayvergiya & Focht	96
9.	Burland	95
10.	Janbu	76
11.	Meyerhof Conventional	97
12.	Flaate & Selnes	94
	Ave:	91.6
	Sd:	7.2

The dynamic methods, only considered for UBCPRS Pile no. 5, also showed a considerable scatter of results. With the dynamic methods it was shown that the in-situ measurement methods (such as the Case Method) can only be expected to give results as accurate as the simpler predictive analyses (such as a Wave Equation Analysis) if appropriate values of parameters such as soil damping are used.

CHAPTER 7

PREDICTED VERSUS MEASURED LATERAL BEHAVIOUR

7.1 Introduction

In this chapter, methods of predicting lateral pile behaviour will be compared to pile load test values obtained as described in Chapter 5.

The two in-situ test methods used are the full-displacement pressuremeter test (FDPMT) method and the flat plate dilatometer test (DMT) method. The former method is only briefly described here. Full details are given by Robertson et al. (1986). The DMT methods is a new method proposed in this study. Both of these methods use the nonlinear discrete Winkler spring approach (P-y curves) described in Chapter 2. In each case, the P-y curves obtained were analyzed with the program LATPILE (Reese and Sullivan, 1980). This program is briefly described in this chapter.

The two methods of predicting the lateral behaviour of driven displacement piles are presented and the results obtained compared with pile load test data from 3 different piles (piles 3 and 5, UBCPRS, and the MOTHPRS pile). In each case predicted versus measured results are included for both pile head deflection and deflected shape versus depth profiles. In addition, other available and potential methods of predicting laterally loaded pile behaviour are briefly discussed.

7.2 Program LATPILE

The P-y curves developed as described in the following sections are used as input data for the program LATPILE. Reese (1977) developed COM622 which was the original program. Reese and Sullivan (1980) then created the first version of LATPILE. The version of LATPILE used for this study is a microcomputer version modified at UBC to be used with IBM-PC and compatible microcomputers.

LATPILE is a finite difference program that can handle up to 20 different P-y curves. The program can analyze any one of three boundary conditions at the pile top along with any combination of 1) lateral deflections along the free field, 2) lateral loads along the pile, 3) a lateral load at the pile top, and 4) axial load. Soil response is interpolated between P-y curves. Full details of the system documentation, operating documentation and governing difference equations can be found in Reese (1977).

The use of LATPILE is straightforward and a minimum of input data is required. There are some disadvantages to using this program in lieu of a finite element program. The finite element method can permit realistic three-dimensional effects and computation of stresses and deformations in and around the piles. LATPILE, however, is seen as adequate for this study as only load-deflection behaviour is of interest. Reese and Desai (1979) have shown that no major differences of pile deflection are seen when comparing the finite difference method to the finite element method with comparable input data.

7.3 Lateral Pile Behaviour

As mentioned previously, two methods of predicting lateral pile behaviour are compared with lateral load test results from three different piles. The DMT and the PMT methods are presented. In addition, other possibilities of using in-situ data to predict lateral pile behaviour are briefly discussed.

7.3.1 Full Displacement Pressuremeter Test P-y Curve Method

The method proposed by Robertson et al. (1983) for obtaining P-y curves from FDPMT results is used. This method is briefly outlined and the results presented. Robertson et al. (1986) document four case histories where this method has been shown to provide very good preditions of measured behaviour.

7.3.1.1 Outline

The method by which FDPMT curves are developed into P-y curves is shown in Fig. 7.1. From the original data, σ_r (radial pressure) and $\Delta R/R$ (cavity strain), three steps are necessary in order to obtain a P-y curve:

i) The pressuremeter curve must be corrected for the lift-off pressure. This is done to remove the effects of the in-situ lateral soil pressure present upon the pressuremeter before expansion. This value (lift-off) is subtracted in order that the lateral stresses around the pile, the vector sum of which are zero, can be accurately modelled.

ii) The presuremeter curve must then be converted into the units of a P-y curve. The radial pressure (σ_r) is converted to a lateral load (P) per unit length of pile by multiplying the radial pressure by the pile diameter, D. To convert the cavity strain ($\Delta R/R$) to displacement units (y), the cavity strain is multiplied by the pile radius.

These two steps in themselves create a P-y curve. However, when the resulting curves have been compared with measured pile behaviour, discrepancies have been noted. The main reason for these discrepancies is that it requires a difference force to expand a pressuremeter than it does to deflect a pile laterally. Therefore a third step becomes necessary:

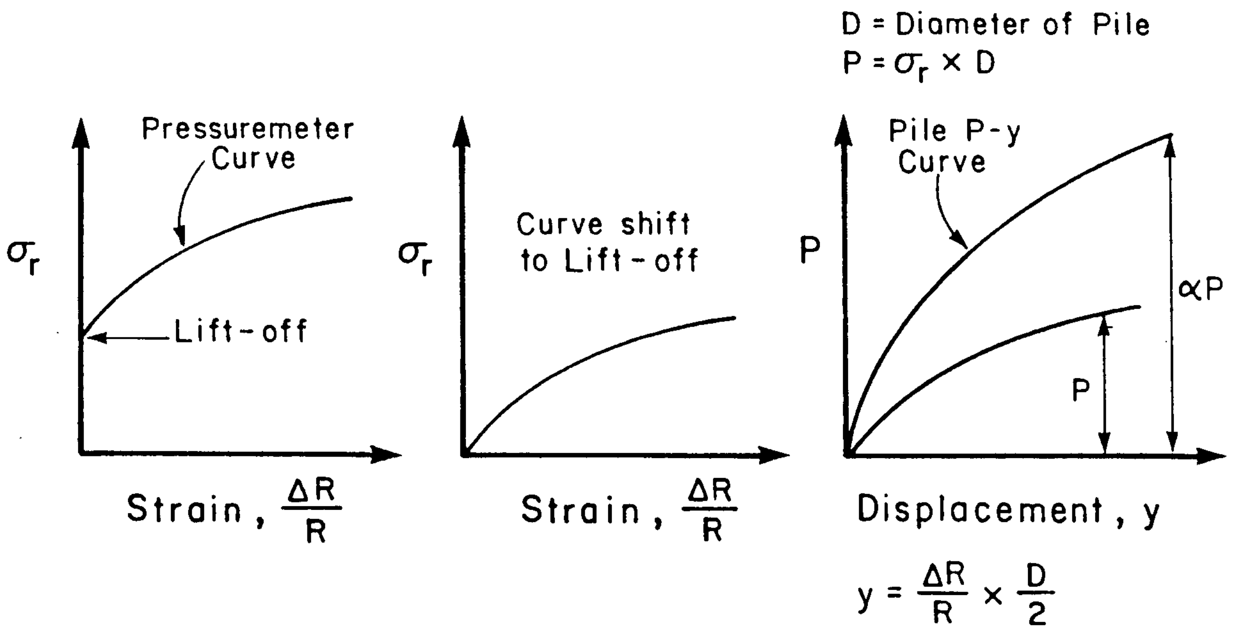


FIG. 7.1. SCHEMATIC REPRESENTATION OF DEVELOPMENT OF PILE P-y CURVES FROM PRESSUREMETER CURVES

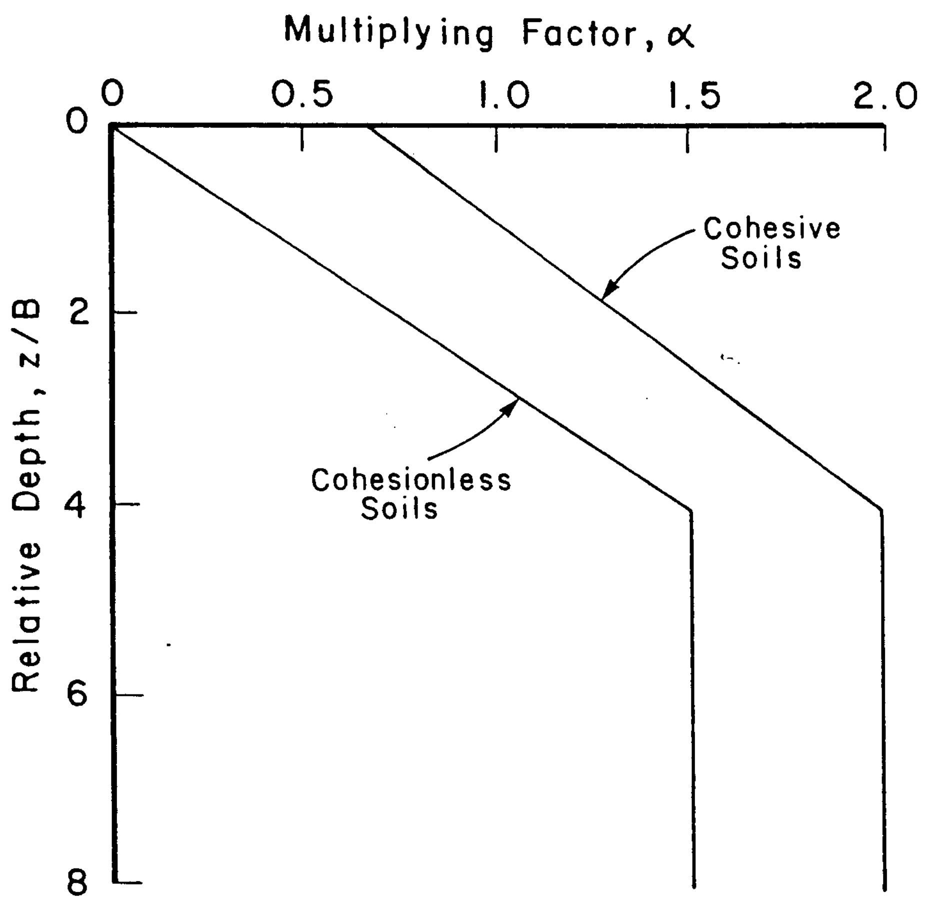


FIG. 7.2. VARIATION OF MULTIPLYING FACTOR WITH RELATIVE DEPTH

iii) Due to the reason noted above, a soil multiplication factor, α, must be applied. Based upon field observations, Robertson et al. (1983) suggest soil multiplication factors of 2.0 for cohesive soils and 1.5 for cohesionless soils (see Figure 7.2). The α factor chosen is then multiplied by the P value obtained in step (ii).

The values of α for cohesive and cohesionless soil suggested above are shown to be appropriate by finite element pressuremeter modelling (Atukorala and Byrne, 1984). This modelling found soil multiplication factors of between 1.9 and 2.6 for cohesive soils and of between 1.4 to 1.7 for cohesionless soils. The range in values is due to changes in the radial strain level assumed for the pressuremeter test.

As was discussed in Chapter 2, an understanding of the concept of a critical depth for lateral pile response is important for a correct prediction of lateral behaviour under loading. Above the critical depth, the free (ground) surface will allow a vertical component of movement to exist in the soil in front of and behind the pile. The influence of the free surface thus reduces the lateral resistance that the soil applies to the pile. Fig. 7.2 shows the variation of the soil multiplication factor, α, with relative depth (depth = z, pile diameter = B) proposed by Robertson et al. (1986). The reduction of lateral resistance is reflected in reductions in α below a relative depth of 4. The reduction values presented in Fig. 7.2 are similar to those proposed by Briaud et al. (1983). Thus when a pressuremeter test is performed within four pile diameters of the ground surface the soil multiplication factor is to be reduced as shown in Fig. 7.2, otherwise no reduction is applied. Note that this method does not consider variations in pile stiffness. However, Briaud et al. (1983) offer a method that encorporates pile stiffness but this was not used for this study.

In addition to correcting the pile P-y curve for a critical depth, the pressuremeter test results themselves must be corrected for surface effects. The critical depths (z_c) for a pressuremeter were proposed by Baguelin et al. (1979), as follows,

$$z_c = 15\ D_{PMT} \text{ for cohesive soils}$$

$$z_c = 30\ D_{MPT} \text{ for cohesionless soils}$$

where: D_{PMT} = diameter of unexpanded pressuremeter.

The pressuremeter curve is then corrected using:

$$P' = \frac{P}{\beta} \qquad (7.1)$$

where: P' = corrected pressure

β = reduction in mobilized pressure at all strains

Fig. 7.3 presents the values of β suggested by Briaud et al. (1983).

7.3.1.2 Results

The results of computed versus measured lateral pile behaviour using the FDPMT method are shown in Figs. 7.4 to 7.6. In Fig. 7.4 the MOTHPRS pile's deflection at ground surface and deflected shape versus depth profile both show very good agreement between mesured and predicted

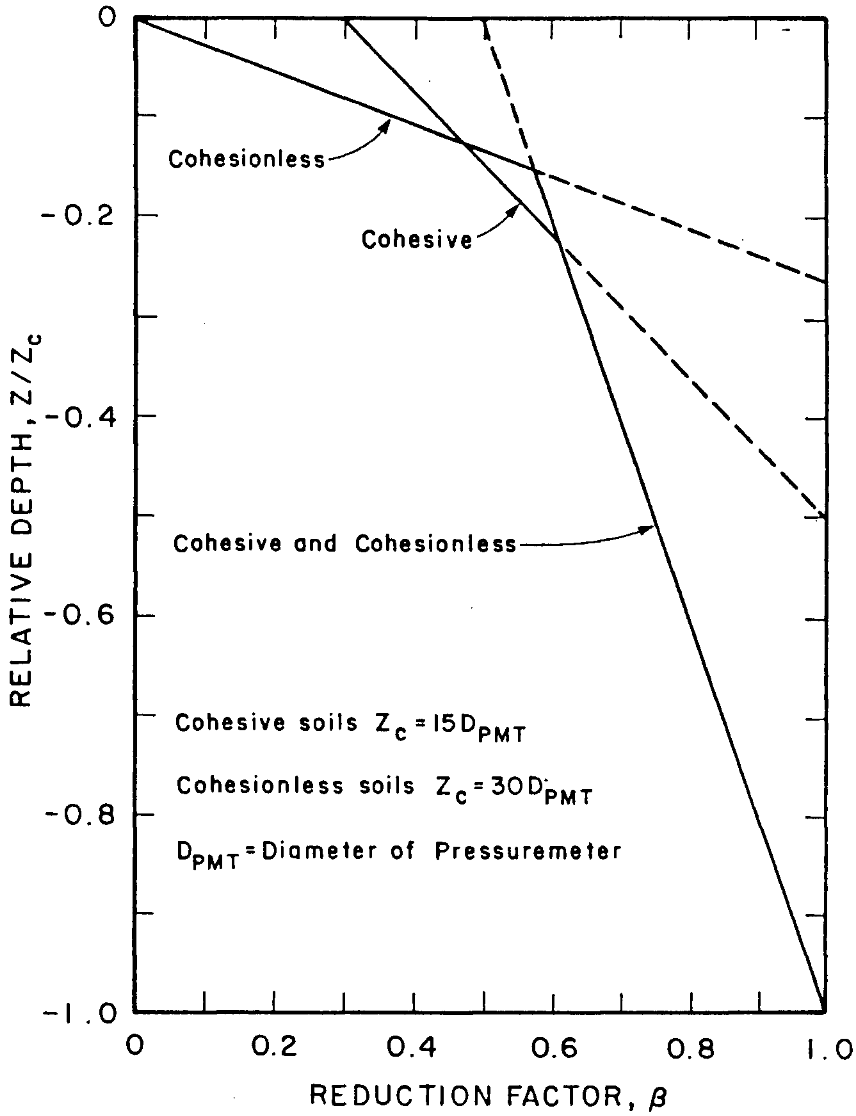


FIG. 7.3. REDUCTION FACTORS FOR PRESSUREMETER TEST RESULTS AT SHALLOW DEPTH
(ADAPTED FROM ROBERTSON ET AL., 1986)

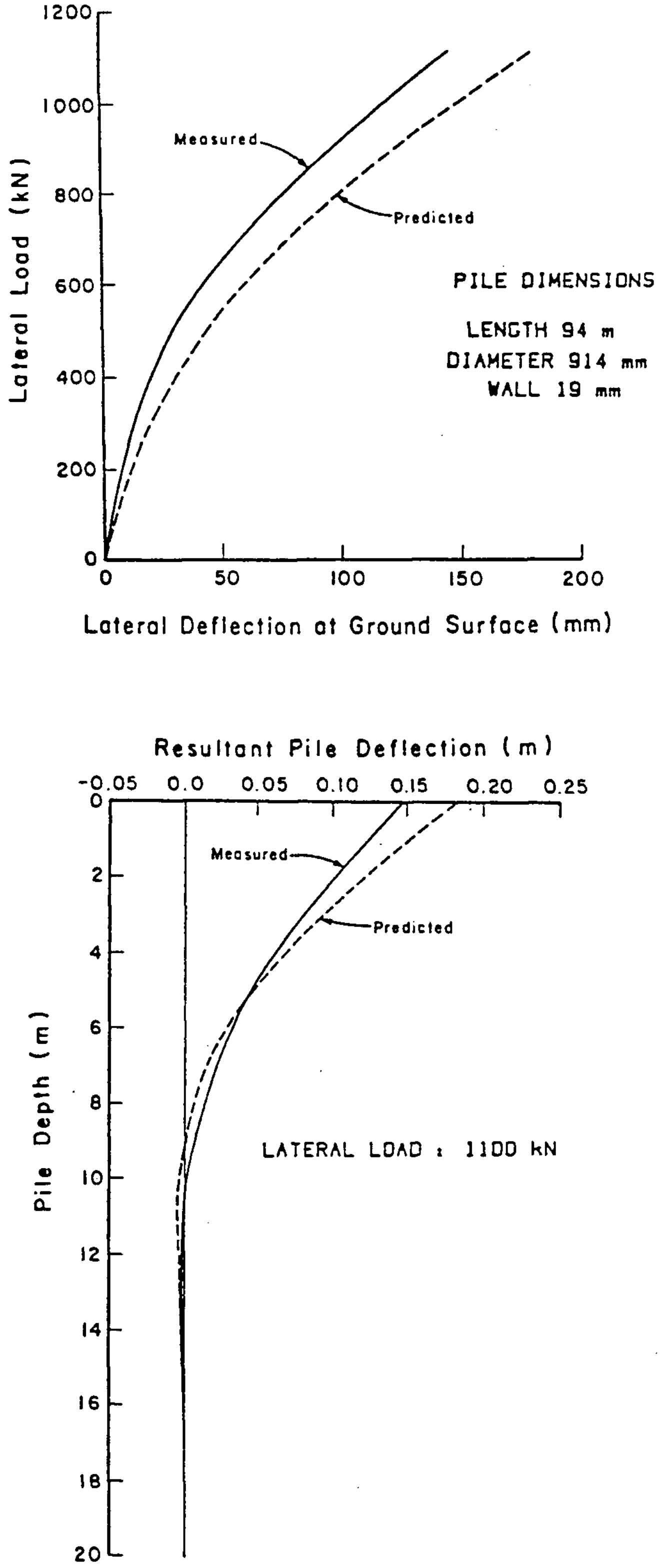
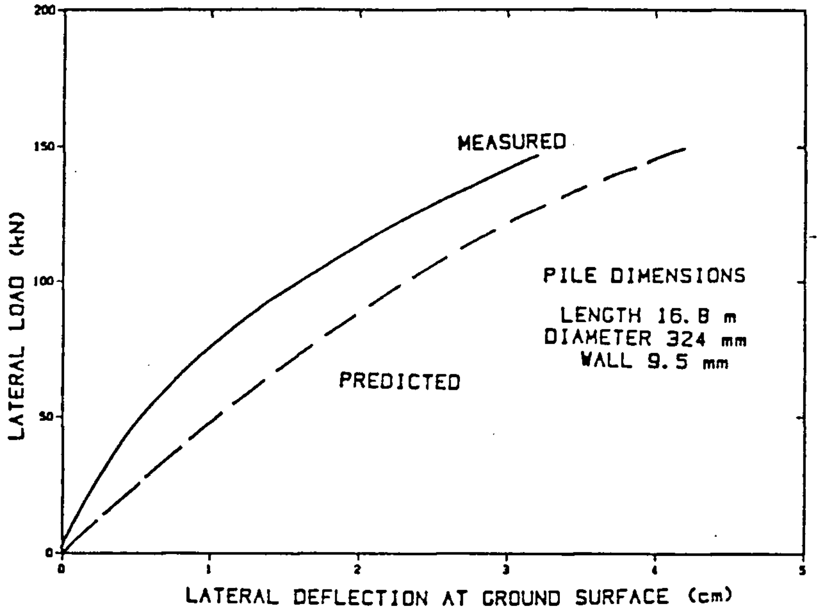


FIG. 7.4. FDPMT METHOD: PREDICTED VERSUS MEASURED LATERAL PILE BEHAVIOUR MOTHPRS PILE



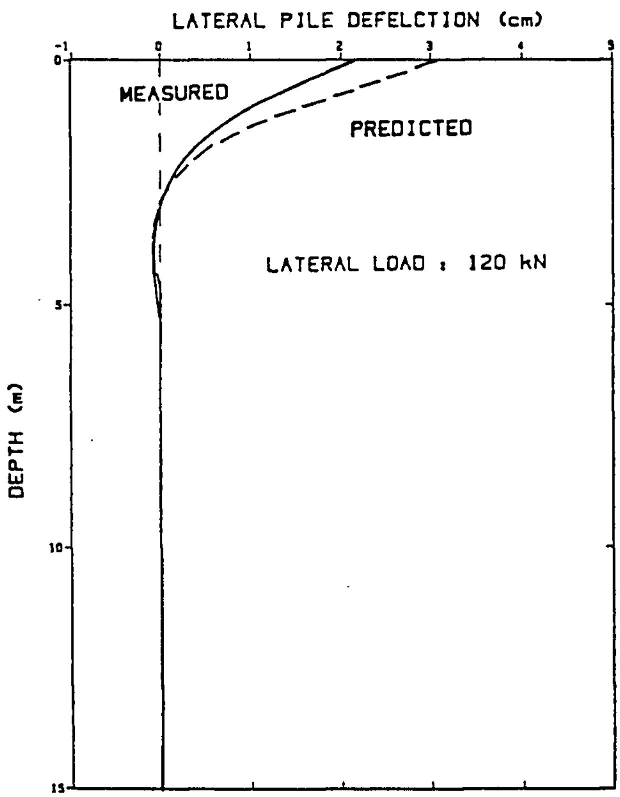
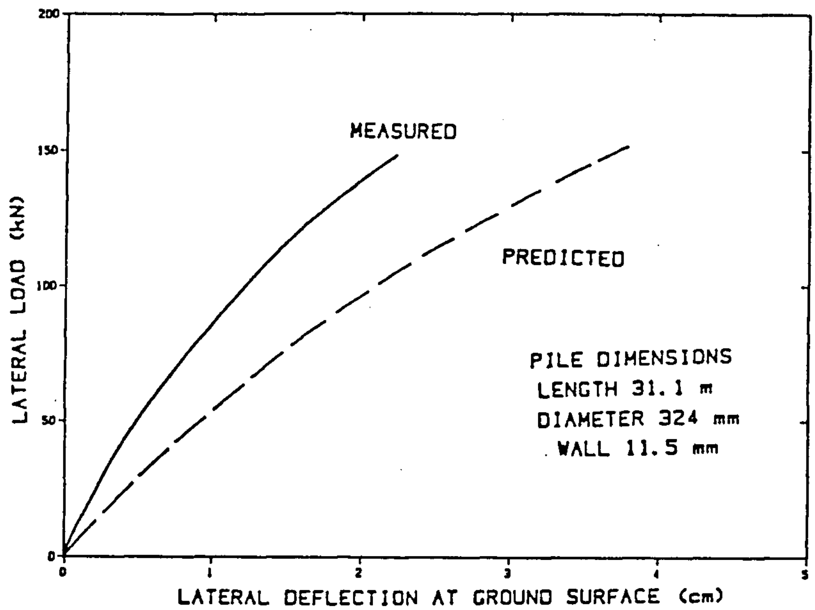


FIG. 7.5. FDPMT METHOD: PREDICTED VERSUS MEASURED LATERAL PILE BEHAVIOUR UBCPRS PILE NO. 3



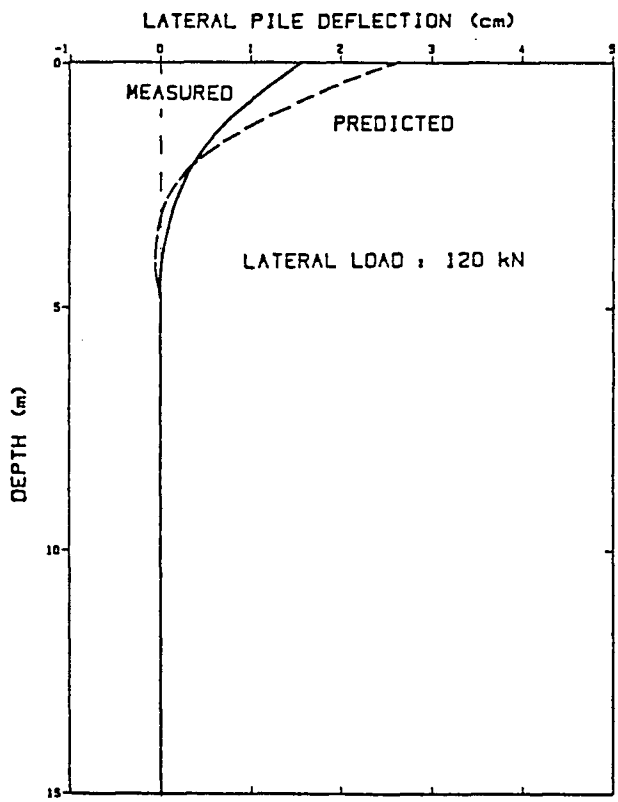


FIG. 7.6. FDPMT METHOD: PREDICTED VERSUS MEASURED LATERAL PILE BEHAVIOUR - UBCPRS PILE NO. 5

behaviour. The predicted deflection at the pile head is within 20% of the measured values. Any discrepancy in prediction is generally shown as being conservative in nature.

For the UBCPRS, Figs. 7.5 and 7.6 show that good agreement between predicted and measured behaviour is again evident with the predictions of pile head deflection generally being within 30 to 50% of the measured values. The predicted values for pile no. 3 (Fig. 7.5) were closer to the measured results than those for pile no. 5 (Fig. 7.6).

For both the UBCPRS and MOTHPRS piles, the predicted versus measured depths of contraflexure agree very well.

7.3.2 <u>Flat Plate Dilatometer P-y Curve Method</u>

Several methods of determining P-y curves from in-situ testing methods exist using the pressuremeter. One approach, using the FDPMT to model driven piles, has been outlined in the previous section and shown to provide good results. However, in general, several problems exist in using the pressuremeter to obtain P-y curves. Some of these difficulties can be stated as follows: the PMT is a difficult and costly test to perform, the pressuremeter has a large installation size and therefore it is difficult to assess the results close to the ground surface (where lateral pile response is most influenced); there are usually only a small number of test results; and there are differences in the soil failure mechanisms during loading between laterally loaded piles and the PMT (symmetric versus non-symmetric).

The flat plate dilatometer test (DMT) is seen as avoiding many of the problems that exist with the PMT. Because of this, the use of DMT data to derive P-y curves is postulated. Being a new method, both the theoretical development and a detailed description of how to implement it are presented.

7.3.2.1 Theoretical Development

Cohesive Soils

Matlock (1970) performed lateral load tests on a steel pipe pile, 324 mm in diameter, using 35 pairs of electric resistance strain gauges installed along the 12.8 metre embedded portion. Using both data from these tests and existing data, Matlock proposed the use of a cubic parabola to predict P-y curves in the form

$$P/P_u = 0.5\ (y/y_c)^{1/3} \tag{7.2}$$

where: P/P_u = ratio of soil resistance

y/y_c = ratio of soil deflection.

This cubic parabola is only valid for short-term, one-way static loading and for soils that behave in a strain hardening manner under this loading. Fig. 7.7, shows the cubic parabolic P-y curve. This curve is in non-dimensional form with P_u to be obtained as described later. The horizontal coordinate is the pile deflection divided by the deflection at a static resistance equal to one-half of the ultimate resistance, P_u. The form of the pre-plastic portion of the static resistance curve, up to point 2 on Fig. 7.7, is based on semilogarithmic plots of the experimental curves which Matlock found to fall roughly along straight lines at slopes yielding the exponent 1/3.

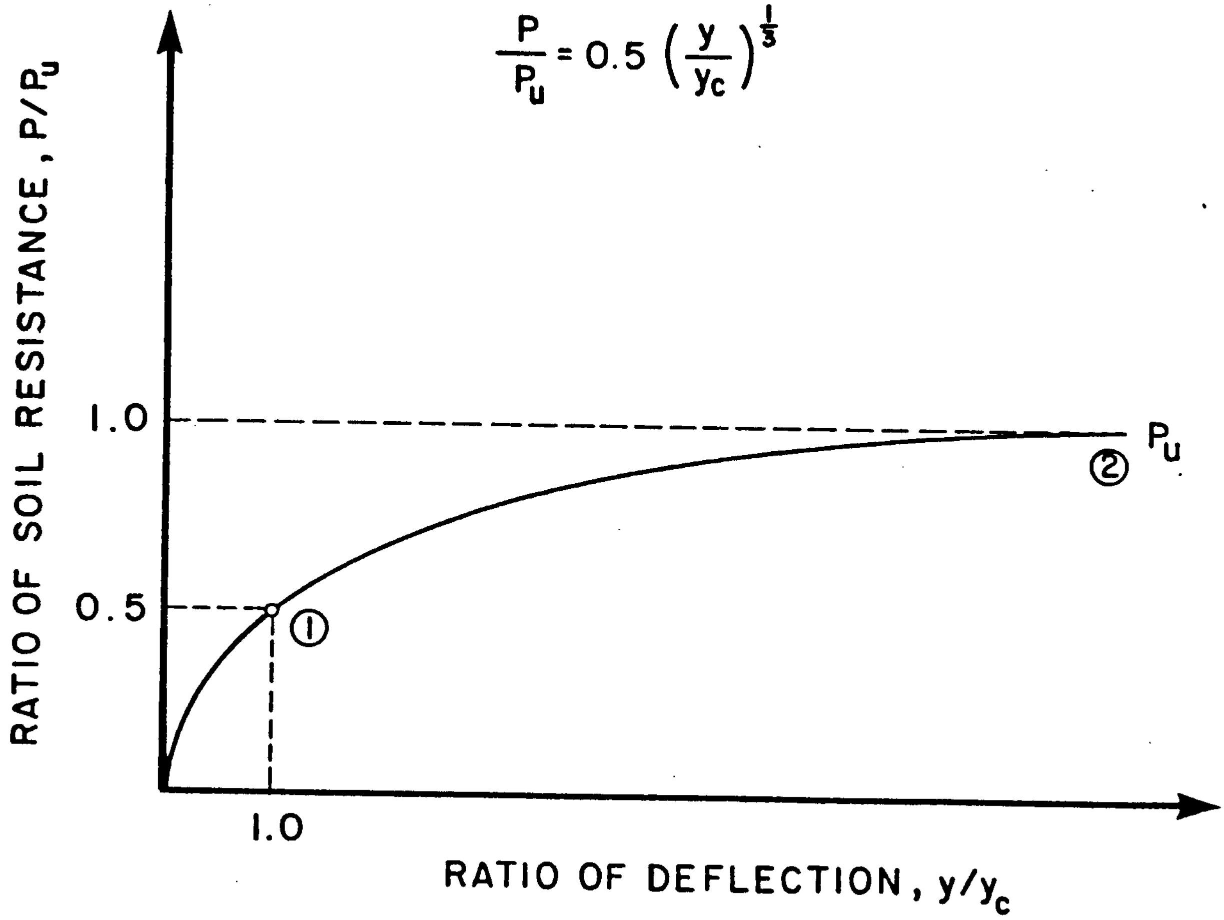


FIG. 7.7. CUBIC PARABOLIC P-y CURVE FOR STRAIN HARDENING SOILS (ADAPTED FROM MATLOCK, 1970)

The value of pile deflection at point 1 in Fig. 7.7 ($y=y_c$) is based upon a concept proposed by Skempton (1951). This concept combined elasticity theory, ultimate strength methods, and laboratory soil properties and showed that the strain ϵ_c, related to y_c, is that which occurs at 50% ultimate stress from the laboratory unconfined compression stress strain curve. From the work of Skempton, Matlock (1970) proposed his "Soft Clay Method" which had the form:

$$y_c = A \cdot \epsilon_c \cdot D \tag{7.3}$$

where: D = pile diameter

A = empirical coefficient

= 6.35 for pile diameter in cm and y_c in cm.

An important consideration when using empirical relationships is the scale effect. Piles commonly in use for supporting offshore structures are up to 15 times larger than those upon which Matlock based his linear "Soft Clay Method", (Stevens and Audibert 1979). It is not reasonable to expect this linear relationship to exist over such a large range of pile dimensions. Studies by Stevens and Audibert (1979) among others, suggest that in cohesive soils the reference deflection, y_c, is not linearly dependent upon pile diameter but is instead approximately defined as:

$$y_c = B \cdot \epsilon_c \cdot D^{0.5} \tag{7.4}$$

where: B = empirical coefficient

= 14.2 for cm

D = pile diameter in cm.

However, Stevens and Audibert (1979) compared Matlock's linear method with their nonlinear approximation on several full-scale lateral load tests with varying pile diameter and showed that their method agreed more closely with observed results (see fig. 7.8). Therefore, Stevens and Audibert's equation has been used for this study to determine y_c for cohesive soils.

The value of ϵ_c (or ϵ_{50}) must be evaluated from a stress-strain curve for the soil in question. Using the hypobolic curve fitting expression proposed by Duncan and Chang (1970), the following relationship can be derived (see Appendix VII):

$$\epsilon_c = \left(\frac{1}{2-R_f}\right) \cdot \frac{\sigma_f}{E_i} \tag{7.5}$$

where: R_f = ratio of deviatoric failure stress over deviatoric ultimate stress (take equal to 0.8)

σ_f = deviatoric failure stress

= $2 \cdot S_u$ for cohesive soil

S_u = undrained shear strength

E_i = initial tangent modulus

which simplifies to:

$$\epsilon_{50} = \frac{1.67 \cdot S_u}{E_i} \tag{7.6}$$

The initial tangent modulus, E_i, can be estimated from the DMT as:

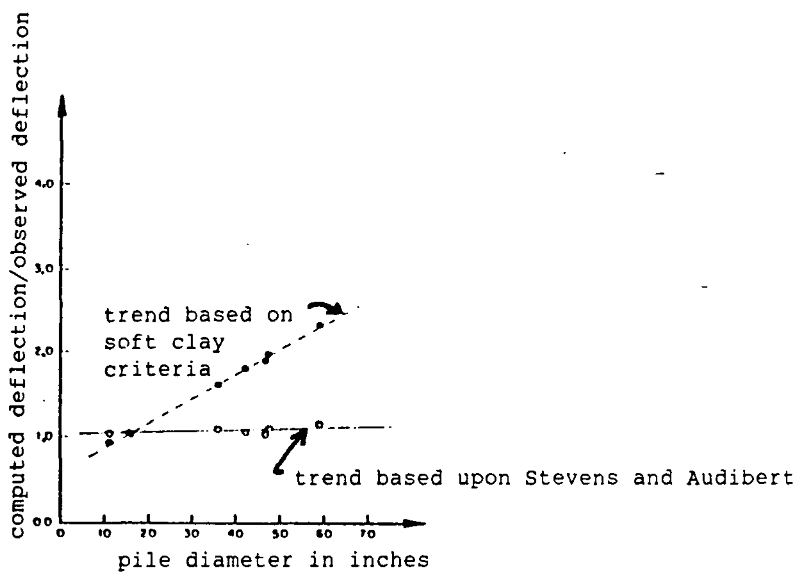


FIG. 7.8. EFFECT OF MAKING REFERENCE DEFLECTION A FUNCTION OF $D^{0.5}$ FOR COHESIVE SOILS
(ADAPTED FROM STEVENS AND AUDIBERT, 1979)

TABLE 7.1. VALUES OF J RECOMMENDED BY MATLOCK (1970)

Value of J	Soil Type	Soil Tested
0.5	Soft clay	Sabine clay
0.25	Stiff clay	Lake Austin clay

$$E_i = F_C \cdot E_D \tag{7.7}$$

where: F_C = empirical stiffness factor

E_D = dilatometer modulus (Marchetti, 1980)

From experience gained within the UBC In-Situ Testing Group (e.g. by McPherson, 1985) a F_C value of approximately 10 is suggested and this value is supported by this study. The undrained strength of the soil, S_u, can be obtained from DMT results using the correlation proposed by Marchetti (1980). Therefore, combining Eqs. 7.4, 7.6 and 7.7 yields:

$$y_c = \frac{23.71 \cdot S_u \cdot D^{0.5}}{F_C \cdot E_D} \tag{7.8}$$

where: y_c = in cm.

D in cm

F_c = 10 (cohesive soils)

The evaluation of the static ultimate resistance, P_u , is based upon plasticity theory. In clay, soil is confined so that plastic flow around a pile (at depth) occurs only in horizontal planes (Matlock, 1970). This may be expressed as follows:

$$P_u = N_p \cdot S_u \cdot D \tag{7.9}$$

where: N_p = non-dimensional ultimate resistance coefficient

S_u = undrained soil strength (from DMT)

D = pile diameter.

At considerable depth it is generally accepted that the coefficient, N_p, should be equal to 9. Near the surface, due to the lower confining stress level, the value of N_p reduces to the range of 2 to 4. Matlock (1970), among others, proposed the following equation to describe this variation:

$$N_p = 3 + \frac{\sigma'_{vo}}{S_u} + J \cdot \frac{x}{D} \leq 9 \qquad (7.10)$$

where: $N_p \leq 9$

x = depth

σ'_{vo} = effective vertical stress level at x

J = empirical coefficient.

Eq. 7.10 closely resembles that presented by Reese (1958). Reese, however, proposed a value of 2.8 for J which does not agree with experimental results. Matlock (1970) proposed values for J as shown in Table 7.1. It is these values that have been used for this study.

Cohesionless Soils

It has been suggested that for cohesionless soils the continuous hyperbolic tangent function is to be used to describe P-y curves (O'Neill and Murchison, 1983). This, however, requires a determination of the modulus of lateral soil reaction, K_r. Preliminary studies into determining K_r from DMT data have been presented (Marchetti, 1980; Motan and Gabr, 1984) but sufficient validation does not exist and therefore, for this study, the simpler cubic parabolic P-y curve (Eq. 7.2) function has been used. This, however, probably isn't fundamentally correct as the use of an ultimate pressure, P_u , in cohesionless soils is not supported by recent research using nonlinear finite element analyses (Yan, 1986). Yan (1986) found that the P-y curves for cohesionless soils closely approximate the bilinear model proposed by Scott (1980); and, in fact, can be represented by a simple power function in the form:

$$\frac{P}{E \bullet D} = a\left(\frac{y}{D}\right)^{b} \tag{7.11}$$

where: E = elastic deformation modulus

a = power function mutliplier ≈ 0.4

b = power function exponent ≈ 0.5

It is suggested that future refinements of this DMT method should attempt to include either the continuous hyperbolic tangent function and/or a form of the above power function so that critical comparisons with the cubic parabolic function can be made.

As for cohesive soils, the values of P_u and y_c must be determined in terms of values obtained from DMT test data. The lateral ultimate soil resistance, P_u, is determined from the lesser value given by the following two equations:

$$P_u = \gamma \bullet x\ [D(K_p - K_a) + x \bullet K_p \bullet \tan\phi \bullet \tan\beta] \tag{7.12}$$

or

$$P_u = \gamma \bullet D \bullet x\ (K_p^3 + 2K_o \bullet K_p^2 \bullet \tan\phi - K_a) \tag{7.13}$$

where: x = depth below the ground surface

γ = unit weight of soil (buoyant or total, as appropriate)

D = pile diameter

ϕ = angle of internal friction

K_a = Rankine active coefficient

$= \frac{1-\sin\phi}{1+\sin\phi}$

K_p = Rankine passive coefficient

$= 1/K_a$

K_o = coefficient of earth pressure at-rest

$\beta = 45° + \phi/2$

Eqs. 7.12 and 7.13 are after Reese et al. (1974) and Murchison and O'Neill (1984). The value ϕ can be estimated by correlation from DMT inflation results (Marchetti, 1980). However, experience gained at UBC (e.g. Robertson, 1982 and McPherson, 1985) suggests increasing the friction angle determined using Marchetti's original correlation from the DMT by some value between 3 and 9 degrees. An increase of 5 degrees was used for this study. It is recognized that the friction angle could also have been determined more accurately using Durgunoglu and Mitchell's bearing capacity theory (Schmertmann, 1982) but the DMT pushing force needed for this method was not recorded. The coefficient of earth pressure at-rest, K_o, was taken to be 0.5. Further refinements of this method could include using the K_o value obtained from DMT results by correlation.

The reference pile deflection, y_c, for cohesionless soils is evaluated from:

$$y_c = 2.5 \cdot \epsilon_{50} \cdot D \qquad (7.14)$$

where: y_c = in cm

D = pile diameter in cm.

The value of ϵ_{50} is evaluated, as for cohesive soils, using Eq. 7.5. The failure deviatoric stress, σ_f, is taken to be (Duncan and Chang, 1970):

$$\sigma_f = \left(\frac{2 \cdot \sin\phi}{1-\sin\phi}\right) \sigma'_v \tag{7.15}$$

The value of ϕ (with the 5° increase) is estimated from the DMT test. As for cohesive soils, R_f is taken to be equal to 0.8. The initial tangent modulus, E_i, can be determined from the DMT as:

$$E_i = FS \cdot E_D \tag{7.16}$$

where: FS = empirical stiffness factor

E_D = dilatometer modulus (Marchetti, 1980)

From experience gained at UBC (e.g. by McPherson, 1985), a FS value of approximately 1 is suggested. However, for the prediction of lateral pile response, the use of an FS value of 2 is supported by this study (Section 7.3.1.3). Therefore, combining Eqs. 7.14 through 7.16 yields:

$$y_c = \frac{4.17 \cdot \sin\phi \cdot \bar{\sigma}_v}{E_D \cdot FS \cdot (1-\sin\phi)} \tag{7.17}$$

where: y_c = in cm.

7.3.2.2 Programs LATDMT.UBC

Programs LATDMT.UBC refers to a series of four FORTRAN programs that are required for the DMT method. These four programs are:

1) DMT.UBC

2) PU-YC.UBC

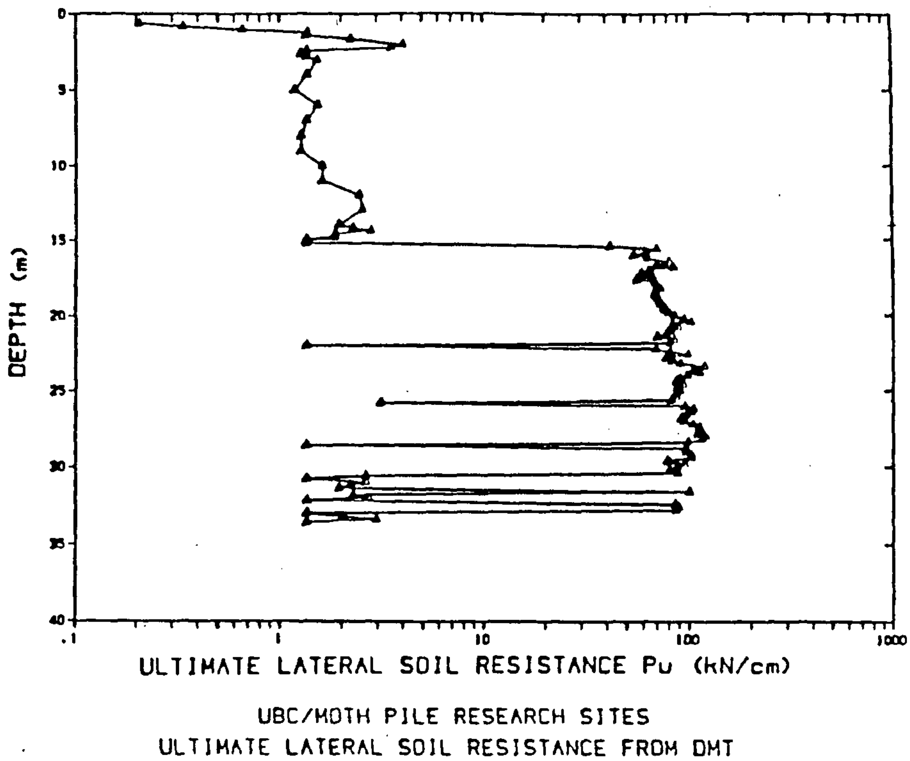
3) PY.UBC

4) LATPILE.UBC

The program DMT.UBC is a program to interpret " dilatometer data based upon the correlations of Marchetti (1980). This program was originally written by John Schmertmann but has been updated at UBC by Ian McPherson. LATPILE.UBC is an available program that has been modified at UBC (see Section 7.2). The other two programs, PU-YC.UBC and PY.UBC, were developed by the writer. PU-YC.UBC takes DMT.UBC output and creates semi-continuous (every 20 cm) profiles of both P_u (ultimate resistance) and y_c (reference deflection) with depth (see Fig. 7.9). From these continuous profiles, average value (trend) lines must be chosen and the profiles discretized as LATPILE can only accept up to 20 P-y curves. Once this discretization is complete, program PY-UBC can be used to generate P-y curves based upon the cubic parabola. Both PU-YC.UBC and PY.UBC listings are appended to this dissertation (Appendix VII). Once the P-y curves have been generated, LATPILE.UBC is then used to generate the predicted pile behaviour. A flowchart describing the steps involved in producing P-y curves using DMT data and then predicting lateral pile behaviour using LATPILE is presented in Fig. 7.10. In Fig. 7.10 it cn be seen that engineering judgement is necessary to discretize the results of PU-YC.UBC into a maximum of 20 layers.

7.3.2.3 <u>Results</u>

As described earlier, the averaged P_u and y_c values must be chosen from those computed for the DMT data (DMT85-2, see Chapter 4). Figure 7.11



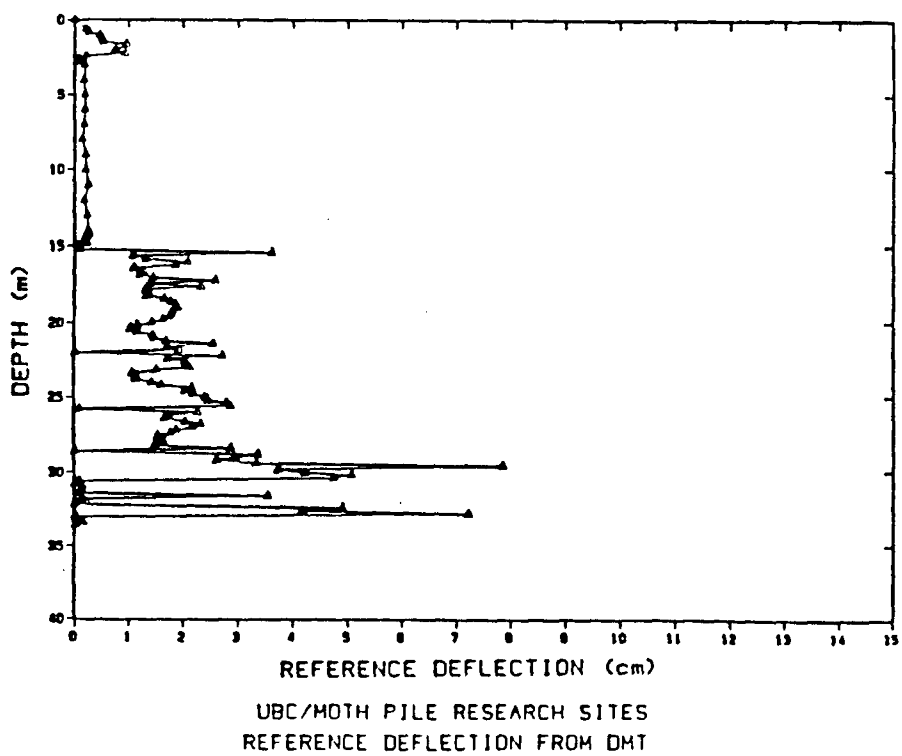


FIG. 7.9. Pu AND Yc CALCULATED OUTPUT FROM DMT

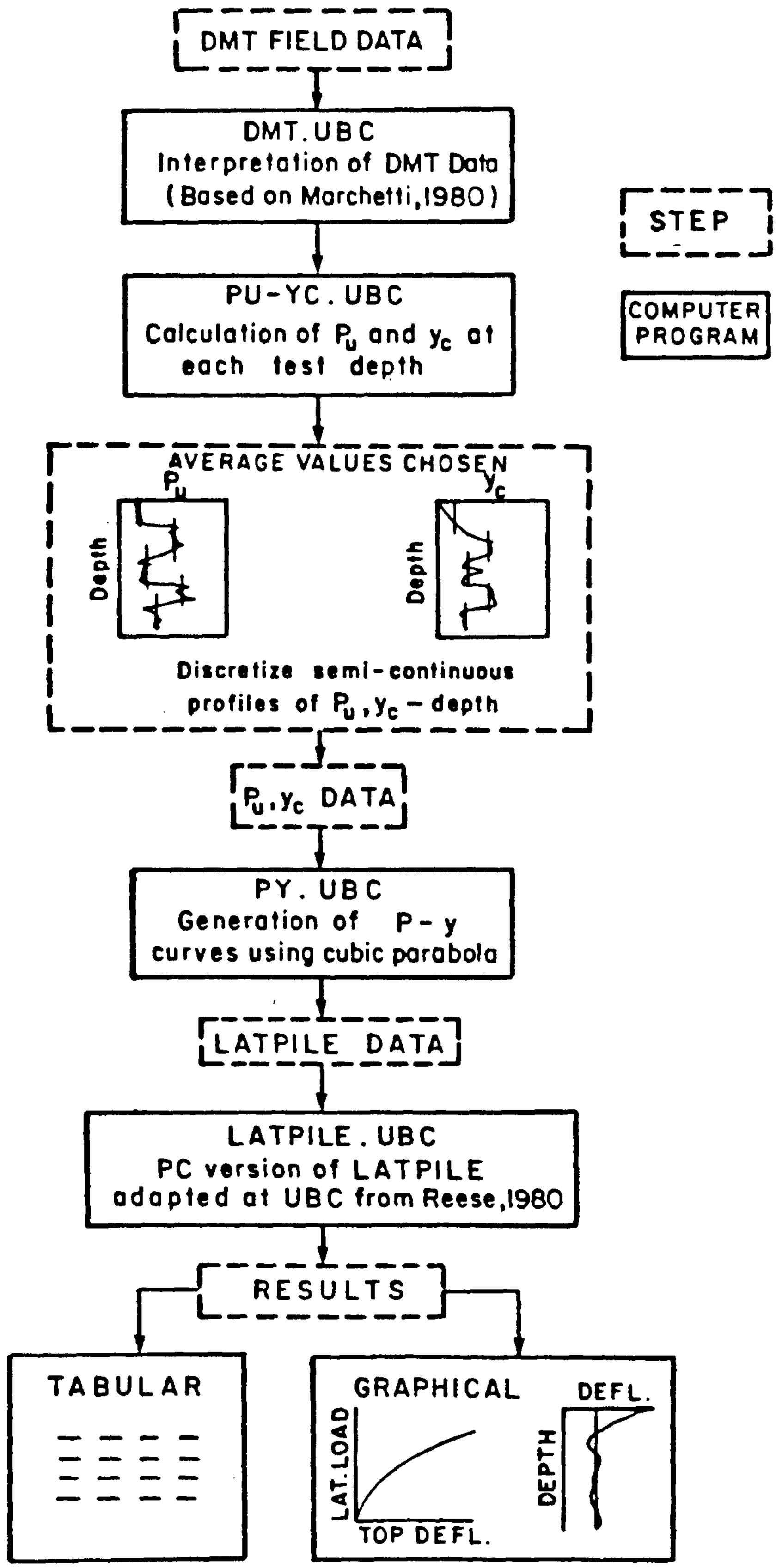
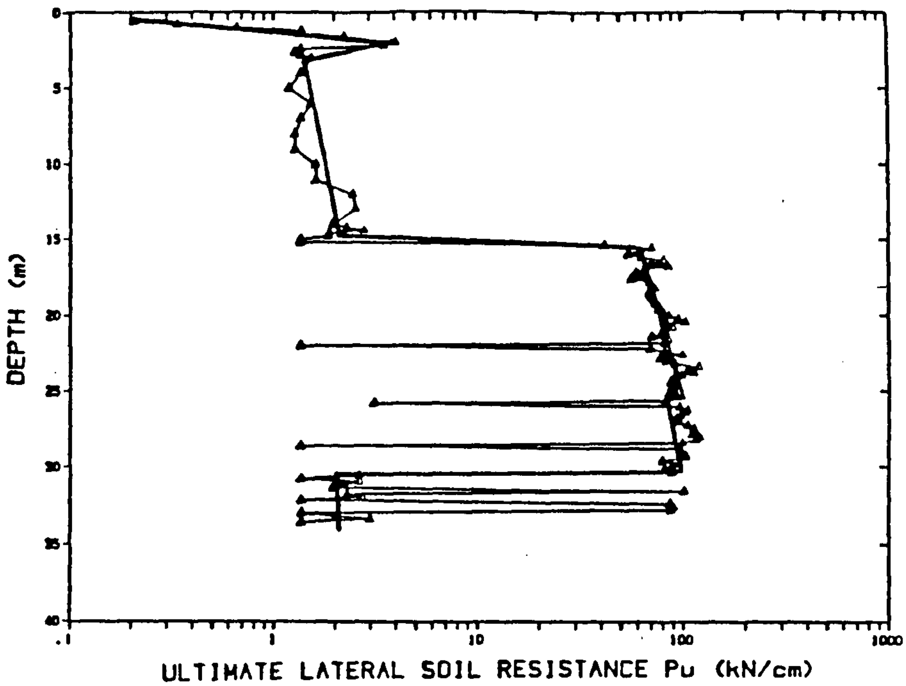
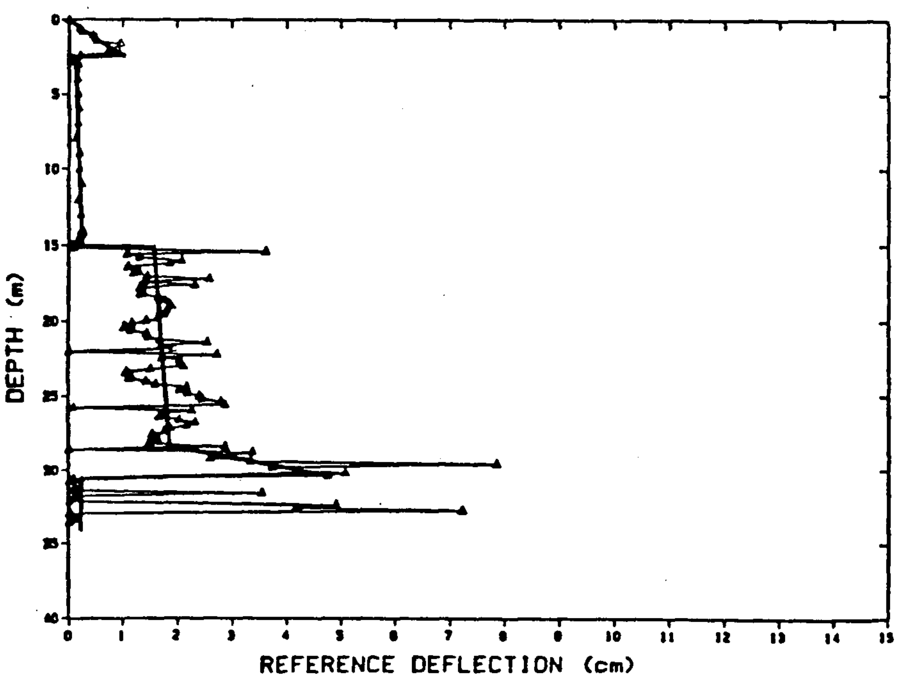


FIG. 7.10. FLOWCHART FOR DETERMINING P-y CURVES FROM DMT DATA



UBC/MOTH PILE RESEARCH SITES
ULTIMATE LATERAL SOIL RESISTANCE FROM DMT



UBC/MOTH PILE RESEARCH SITES
REFERENCE DEFLECTION FROM DMT

FIG. 7.11. AVERAGE VALUES OF Pu and Yc CHOSEN FROM DMT

shows the average values chosen from the P_u, y_c profiles for the data used. These values were used as input P-y curves as calculated according to the equations presented earlier.

A summary of the calculated and measured load deflection curves is shown in Fig. 7.12 and Figs. 7.13 and 7.14 for the MOTHPRS and UBCPRS respectively. The three piles in question are all of differing sizes as noted. In each case two values of FS (1 and 2) are used in the evaluation of both the pile head and deflected shape deflection profiles. This is to show that while previous work with the DMT suggested an FS value of close to 1, the results of this study suggest that a value of 2 may be more appropriate. Studies showed that the value of FC was, as was predicted by previous work, about equal to 10.

The results in Fig. 7.12 for the MOTHPRS pile show that the predicted deflection agrees well with the measured deflection. Not much difference was seen here between FS=1 and FS=2, especially a higher loads. The curve for FS=1, however, resembled the measured load deflection curve shape better than did the curve for FS=2. For both modulus factors (FS=1 and 2), the predicted deflection is approximately 25% larger than the measured deflecton at the pile head under large load (1100 kN0 and agreement is generally closer at lower loads. The deflected shape versus depth profiles at a load of 1100 kN also agree closely with the points of contraflexure both occurring at about a depth of 11 metres.

The results in Fig. 7.13 for the smaller (pile no. 3) of the two UBCPRS piles tested again show excellent agreement between predicted and measured deflection. This is particularly true for the curve corresponding to the modulus factor FS=2. For the FS=2 curve, the difference between the predicted and measured results is generally never more than 25% for the

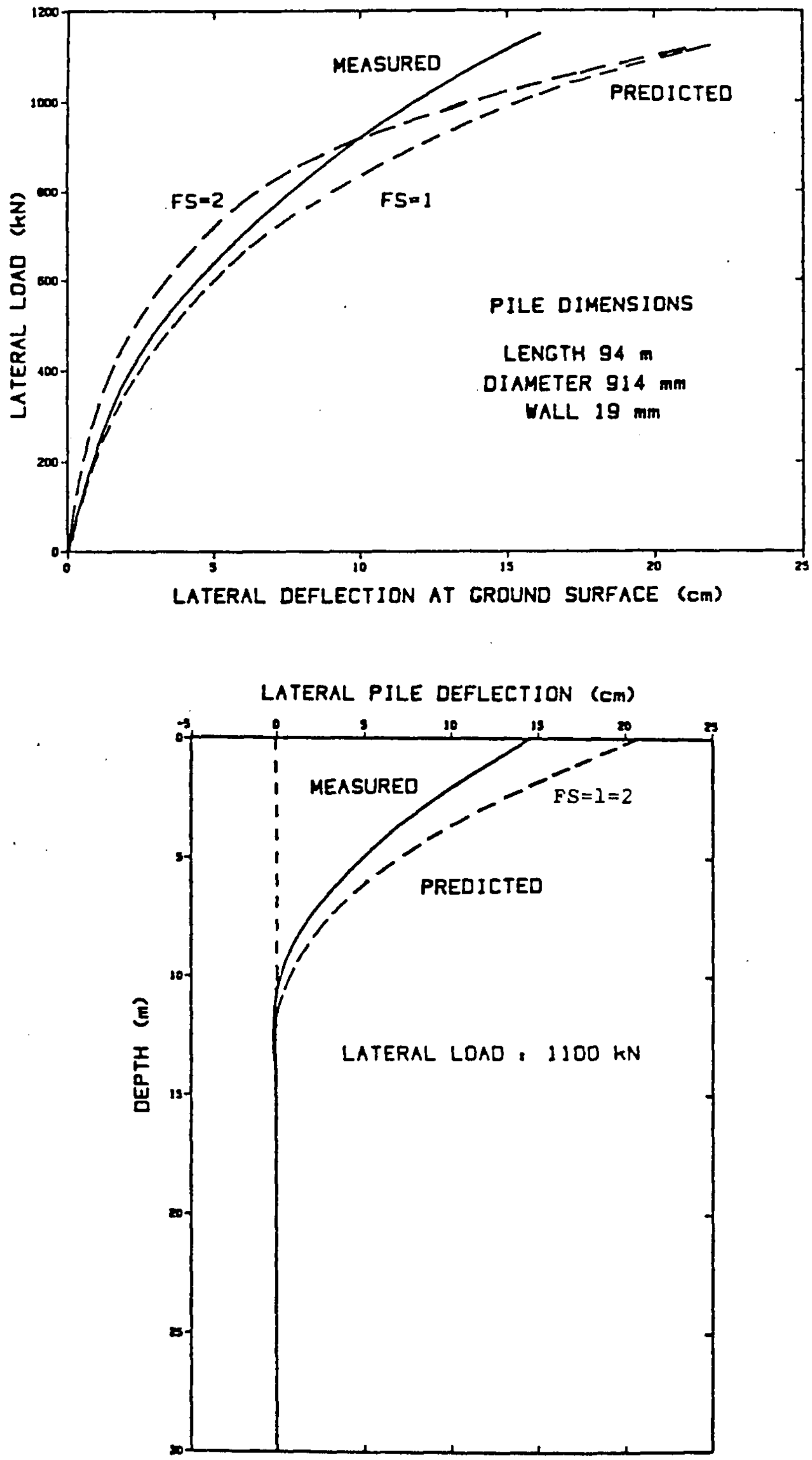


FIG. 7.12. DMT METHOD: PREDICTED VERSUS MEASURED LATERAL PILE BEHAVIOUR - MOTHPRS PILE

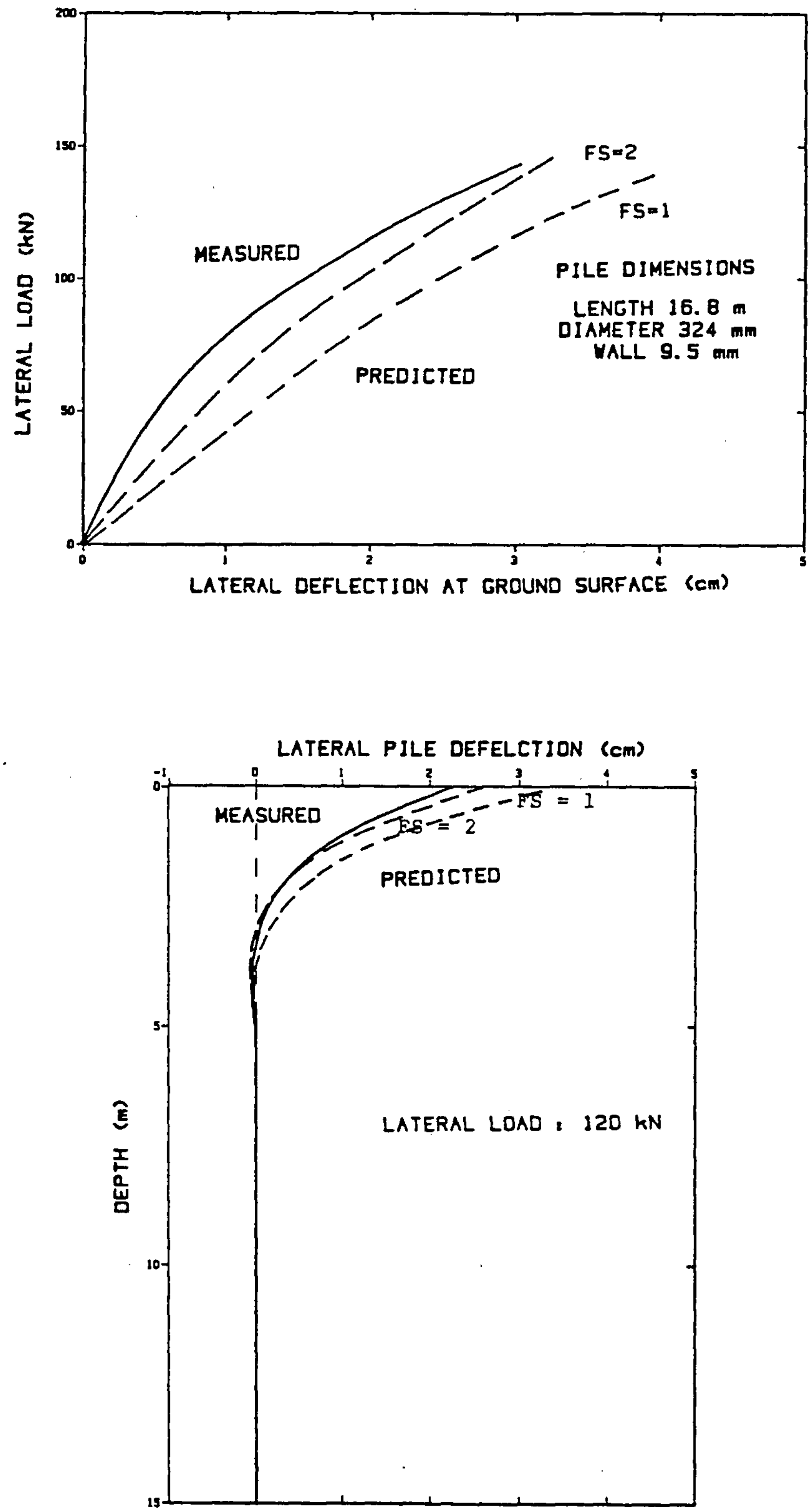


FIG. 7.13. DMT METHOD: PREDICTED VERSUS MEASURED LATERAL PILE BEHAVIOUR - UBCPRS PILE NO. 3

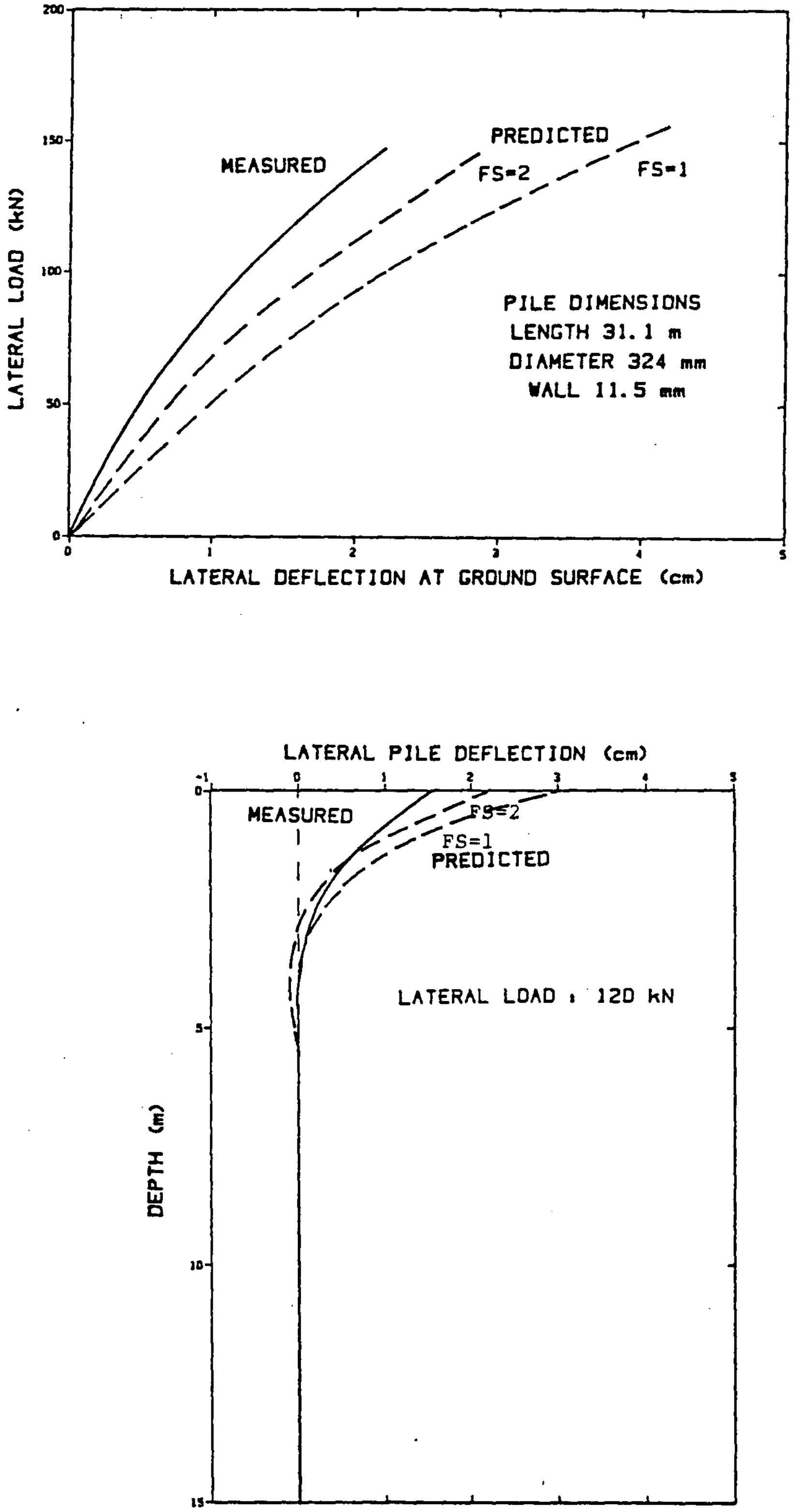


FIG. 7.14. DMT METHOD: PREDICTED VERSUS MEASURED LATERAL PILE BEHAVIOUR - UBCPRS PILE NO. 5

entire range of loads with the predicted values being higher. The deflected shape versus depth profiles for 120 kN load are of similar good agreement with all three curves showing essentially the same depth of contraflexure.

The results in Fig. 7.14 for UBCPRS pile no. 5 showed poorer agreement between predicted and measured deflection. However, with the value of FS=2 (the better prediction) being used, the pile head deflection predictions were generally only 35% larger than the measured results. This must still be regarded as fairly good agreement. The deflected shape versus depth profiles also show similar good agreement between predicted and measured behaviour.

7.3.3 Other Methods

Other in-situ methods are available for predicting laterally loaded pile behaviour. However, these are mainly pressuremeter methods. Besides the FDPMT method present, methods using self-boring pressuremeter test data (e.g. Baguelin, 1982) or pre-bored pressuremeter test data, using a Ménard type pressuremeter, (e.g. Briaud et al., 1983) also exist. Schmertmann (1978) has attempted to correlate CPT data with the Ménard PMT and then use the values obtained for an appropriate PMT design method. Schmertmann's method was briefly examined but meaningful results could not be obtained and therefore none are presented. Schmertmann (1978) readily admits that this method should only be used for the most preliminary of design.

Potential exists for using DMT, PMT and CPT data in new methods for predicting laterally loded pile behaviour. Beyond the traditional two data points obtained with the Marchetti dilatometer, a research DMT that supplies a continuous load-deflection curve is available (Tsang, 1987). From this continuous curve, which resembles a FDPMT curve, a method of constructing a P-y curve is possible. This method would probably not be unlike the current FDPMT method.

Both the PMT and CPT could be used to predict laterally loaded pile behaviour using the method presented for the DMT in the previous section. This method requires estimates of undrained strength, friction angle, and initial tangent Young's modulus. Both the CPT and PMT offer several means by which these parameters can be obtained. The value of performing this exercise with PMT data seems small, however, due to the more direct and proven methods available. On the other hand, this would be of great interest as far as the CPT is concerned. Being the preferred in-situ testing instrument for predicting axial pile capacity, having the capability of also estimating lateral behaviour would mean that a single instrument for pile foundation design would be available. The CPT has shown good ability in estimating drained friction angle and unrained shear strength. However, the accuracy of modulus estimates from CPT data are highly affected by the stress and strain history of the soil (Baldi et al., 1985).

Other methods of predicting lateral pile behaviour from in-situ testing methods, using not only the previously mentioned tests but other in-situ testing methods are possible. As in-situ testing becomes more commonly used in geotechnical practice for foundation design, many of these methods will be realized.

7.4 Discussion of Lateral Pile Behaviour Predictions

Both the FDPMT and the DMT methods performed well in predicting the measured lateral behaviour of the three piles investigated. The FDPMT method, as proposed by Robertson et al. (1983), is a proven method that was further validated by this research. The DMT method, however, is a new method proposed by this study. Further field studies are necessary in order to evaluate the DMT method for other soil profiles and pile types.

Overall, this study has shown that in-situ testing is a reliable method of accurately predicting laterally loaded pile behaviour in the soil types as investigated.

CHAPTER 8

RECOMMENDED CORRELATIONS

8.1 Axial Pile Capacity

As was shown in Chapter 6, due mainly to their ability to deal with the scale differences between piles of differing size, a preference for using the direct static prediction methods is apparent. Based upon the results presented in Chapter 6 the following three direct methods are preferred:

1. LCPC CPT (Bustamante and Gianeselli, 1982)
2. de Ruiter and Beringen CPT (1979)
3. Schmertmann and Nottingham CPT (1978)

For the piles tested, these three methods supplied a maximum error of 52% and an average error of 5% when compared with measured axial pile capacities. The LCPC (French) method is shown to be the best method with a maximum error of 25%, an average error of 0%, and a standard deviation (S_d) of 15%. In addition, the LCPC method does not directly require the CPT sleeve friction value other than to define soil type. This is a desirable feature since the cone bearing is generally obtained with more accuracy and confidence than the sleeve friction.

The results of this study indicate that indirect CPT methods to predict axial pile capacity may significantly overpredict the capacity of large diameter, long piles (L/D > 75) supported in deltaic soils.

No preference was seen between the dynamic methods briefly evaluated however the dynamic formula investigated (Engineering News Record) was shown to easily be the most unreliable.

8.2 Lateral Pile Behaviour

Both the full-displacement pressuremeter and the flat plate dilatometer methods were shown to be very effective in predicting the measured lateral pile behaviour. The dilatometer method, being a new method, needs further validation and hence this method must be used with caution. At this time it is therefore felt that a preference must be shown for using the pressuremeter method.

8.3 Limitations and Precautions

Any emprical prediction method (axial or lateral pile behaviour) can be expected to yield accurate results only if the conditions under which it is applied resemble those in the data bank used to formulate the method. When determining the suitability of any empirical design method, the intended application should be compared with the method's data bank conditions such as:

i) pile installation technique
ii) pile material type
iii) pile shape
iv) pile size (diameter and embedment)
v) soil conditions
vi) special considerations

Designers should use any empirical method with caution.

CHAPTER 9

SUMMARY AND CONCLUSIONS

The major objective of this study was to evaluate methods of predicting axial and lateral pile behaviour as measured from full-scale pile load tests. The following sections present a sumary of the significant findings from this research.

9.1 Pile Installation and Load Testing

The "Quick Load Test Method" of axial loading (similar to ASTM D1143-91 Section 5.6) was used for axial pile load testing. The "Quick Load Test Method" was used to minimize time-dependent effects. This method was found to work well with an average testing time of 4 to 6 hours per pile.

To calculate the axial pile load test failure load, the method by Davisson (1973) was found to be repeatable.

The tell-tale data obtained at the UBCPRS, other than for pile no. 5 (which ws load tested first) presented several problems for interpretive purposes. This is possibly because of the complex loading history for the other piles.

Unlike with the axial load case, no standard method of interpreting lateral load test results exists. The effects of creep (time effects) can be very pronounced during lateral pile testing. Until standardization of testing is realized, it will remain difficult to compare results between researchers.

9.2 Axial Pile Capacity Prediction Methods

This thesis compared twelve static axial pile capacity prediction methods with the results from eight full-cale pile load tests on six different piles. The piles were steel pipe piles driven into deltaic soil deposits. The length to diameter ratios (L/D) for the piles ranged from 40 to 100. The measured axial capacities ranged from 170 kN to 8,000 kN in soils that included organic silt, sand and clay.

CPT data was used for the prediction of pile capacity for the twelve methods evaluated. The direct methods, which incorporate CPT-pile scaling factors, provided the best predictions for the piles and methods evaluated. Based on the results of this research the following three direct methods are preferred:

1. LCPC CPT
2. de Ruiter and Beringen CPT
3. Schmertmann and Nottingham CPT

The results of this research indicate that indirect CPT methods used to predict axial pile capacity may significantly overpredict the capacity of large diameter, long piles (L/D > 75) supported in clayey silt soils.

The main conclusion from the brief evaluation of dynamic prediction methods is that the accuracy of the prediction is extremely dependent on the input parameters chosen. Unfortunately, systematic and reliable methods for choosing these input parameters are not yet available.

9.3 Lateral Pile Behaviour Prediction Methods

Both the full-displacement pressuremeter and the flat plate dilatometer are seen as useful tools for assesing laterally loaded pile behaviour.

The pressuremeter method is an existing method (Robertson et al., 1983) with significant validation. The results of this research are seen as further validation of this method.

Further field studies are necessary in order to evaluate the dilatometer method for other soil profiles and pile types. The proposed method must be used with caution until further validation has taken place. However, due to both the ability of the dilatometer to obtain a near continuous profile of soil response and to its small size, the DMT offers an excellent means of obtaining considerable data even at shallow depths below the ground surface. This is very important for the design of laterally loaded piles since very little deflection occurs below a depth of approximately five pile diameters under typical design loads (Poulos and Davis, 1980).

9.4 Recommendations for Further Research

The areas listed below are some of those which the author believes additional research could improve the ability to make accurate predictions of axially and laterally loaded pile behaviour from in-situ testing data.

i) Development of a standard method of performing lateral pile load tests so that data between researchers can be easily compared.

ii) Further validation of the preferred direct axial pile capacity prediction methods. Local correlations would be especially beneficial.

iii) Development of a systematic and repeatable method of obtaining parameters for pile dynamic analyses from in-situ tests.

iv) Further validation of the proposed DMT method for predicting lateral pile behaviour.

v) Continued development of equipment like UBC's cone pressuremeter from which axial and lateral pile behaviour can be predicted from one test.

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APPENDIX I

REDUCED IN-SITU TEST DATA

```
DMT PR 8
QUEENSBOROUGH, LULU ISLA
23-08-85
 0.08 0.55 0.0  2.00
 0.060
34.600
  0
 0.40 1.40 7.40
 0.60 1.70 4.20
 0.80 1.70 6.00
 1.00 1.60 6.40
 1.20 1.90 8.50
 1.40 1.90 9.00
 1.60 1.80 5.80
 1.80 1.50 5.50
 2.00 1.50 5.90
 2.20 1.00 4.20
 2.40 1.10 2.30
 2.60 0.80 2.00
 2.80 1.35 2.20
 3.00 1.35 2.10
 3.20 1.10 2.10
 3.40 1.35 2.20
 3.60 1.40 2.20
 3.80 1.50 2.40
 4.00 1.50 2.40
 4.20 1.50 2.40
 4.40 1.40 2.20
 4.60 1.55 2.30
 4.80 1.60 2.40
 5.00 1.40 2.30
 5.20 1.50 2.30
 5.40 1.40 2.10
 5.60 1.40 2.20
 5.80 1.50 2.20
 6.00 1.60 2.30
 6.20 1.70 2.40
 6.40 1.60 2.30
 6.60 1.70 2.50
 6.80 1.80 2.55
 7.00 1.75 2.55
 7.20 1.80 2.55
 7.40 1.80 2.60
 7.60 1.80 2.50
 7.80 1.90 2.70
 8.00 1.90 2.70
 8.20 2.05 3.00
 8.40 2.10 2.90
 8.60 2.00 2.80
 8.80 2.10 2.90
 9.00 2.05 2.90
 9.20 2.20 3.10
 9.40 2.20 3.10
 9.60 2.10 3.00
 9.80 2.30 3.20
10.00 2.30 3.10
10.20 2.20 3.00
10.40 2.30 3.10
```

FILE NAME:DMT-PR-85-1
LOCATION:QUEENSBOROUGH, LULU ISLADATE:23-08-85   TEST NUMBER:DMT PR 85-1
INTERMEDIATE DILATOMETER PARAMETERSFROM 0.40M TO13.40M.
NUMBER OF DATA POINTS: 66
ZW= 2.00M. DA= 0.08 DB= 0.55 ZM= 0.0

DEPTH	A	B	PO	P1	ED
0.4	1.4	7.4	1.2	6.8	195.1
0.6	1.7	4.2	1.7	3.6	67.9
0.8	1.7	6.0	1.6	5.4	133.3
1.0	1.6	6.4	1.5	5.8	151.5
1.2	1.9	8.5	1.7	7.9	216.9
1.4	1.9	9.0	1.7	8.4	235.1
1.6	1.8	5.8	1.7	5.3	122.4
1.8	1.5	5.5	1.4	4.9	122.4
2.0	1.5	5.9	1.4	5.3	137.0
2.2	1.0	4.2	1.0	3.6	93.4
2.4	1.1	2.3	1.2	1.8	20.7
2.6	0.8	2.0	0.9	1.4	20.7
2.8	1.4	2.2	1.4	1.6	8.0
3.0	1.4	2.1	1.4	1.6	4.4
3.2	1.1	2.1	1.2	1.6	13.4
3.4	1.4	2.2	1.4	1.6	8.0
3.6	1.4	2.2	1.5	1.6	6.2
3.8	1.5	2.4	1.6	1.8	9.8
4.0	1.5	2.4	1.6	1.8	9.8
4.2	1.5	2.4	1.6	1.8	9.8
4.4	1.4	2.2	1.5	1.6	6.2
4.6	1.6	2.3	1.6	1.8	4.4
4.8	1.6	2.4	1.7	1.8	6.2
5.0	1.4	2.3	1.5	1.8	9.8
5.2	1.5	2.3	1.6	1.8	6.2
5.4	1.4	2.1	1.5	1.6	2.5
5.6	1.4	2.2	1.5	1.6	6.2
5.8	1.5	2.2	1.6	1.6	2.5
6.0	1.6	2.3	1.7	1.8	2.5
6.2	1.7	2.4	1.8	1.8	2.5
6.4	1.6	2.3	1.7	1.8	2.5
6.6	1.7	2.5	1.8	1.9	6.2
6.8	1.8	2.6	1.9	2.0	4.4
7.0	1.8	2.6	1.8	2.0	6.2
7.2	1.8	2.6	1.9	2.0	4.4
7.4	1.8	2.6	1.9	2.1	6.2
7.6	1.8	2.5	1.9	1.9	2.5
7.8	1.9	2.7	2.0	2.1	6.2
8.0	1.9	2.7	2.0	2.1	6.2
8.2	2.1	3.0	2.1	2.4	11.6
8.4	2.1	2.9	2.2	2.3	6.2
8.6	2.0	2.8	2.1	2.3	6.2
8.8	2.1	2.9	2.2	2.3	6.2
9.0	2.1	2.9	2.1	2.3	8.0
9.2	2.2	3.1	2.3	2.6	9.8
9.4	2.2	3.1	2.3	2.6	9.8
9.6	2.1	3.0	2.2	2.4	9.8
9.8	2.3	3.2	2.4	2.6	9.8
10.0	2.3	3.1	2.4	2.6	6.2
10.2	2.2	3.0	2.3	2.4	6.2
10.4	2.3	3.1	2.4	2.6	6.2
10.6	2.3	3.2	2.4	2.6	9.8
10.8	2.4	3.4	2.5	2.8	13.4
11.0	2.5	3.4	2.6	2.9	11.6

U.B.C. INSITU TESTING RESEARCH GROUP.

File Name:DMT-PR-85-1
Location:QUEENSBOROUGH, LULU ISLAND

Record of Dilatometer test No:DMT PR 85-1
Date:23-08-85

Calibration Information:DA= 0.08 Bars DB= 0.55 Bars ZM= 0.0 Bars ZW= 2.00 metres

Gamma=Bulk unit weight
Sv =Effective over.stress
Uo =Pore pressure
Id =Material index
Ed =Dilatometer modulus
Kd =Horizontal stress index

INTERPRETED GEOTECHNICAL PARAMETERS
Ko =Insitu earth press.coeff.
OCR=Overconsolidation Ratio
M =Constrained modulus
Cu =Undrained cohesion(cohesive)
PHI=Friction Angle(cohesionless)

Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)
0.40	1.21	6.85	195.	0.0	4.65	1.80	0.060	20.2	*****	8.69	2.79		40.6	607.	SAND	CEMENTED	0.40
0.60	1.69	3.65	68.	0.0	1.16	1.70	0.094	17.9	30.65	2.88	2.61			207.	SILT	LOW DENSITY	0.60
0.80	1.60	5.45	133.	0.0	2.41	1.80	0.130	12.3	56.00	7.28	2.09		33.6	359.	SILTY SAND	LOW RIGIDITY	0.80
1.00	1.47	5.85	151.	0.0	2.98	1.80	0.166	8.9	30.05	4.99	1.70		34.2	363.	SILTY SAND	LOW RIGIDITY	1.00
1.20	1.68	7.95	217.	0.0	3.73	1.80	0.202	8.3	26.65	5.38	1.64		35.9	508.	SAND	LOW RIGIDITY	1.20
1.40	1.66	8.45	235.	0.0	4.10	1.80	0.238	7.0	18.93	4.51	1.46		36.2	514.	SAND	LOW RIGIDITY	1.40
1.60	1.71	5.25	122.	0.0	2.07	1.80	0.274	6.2	15.40	4.22	1.36		30.5	253.	SILTY SAND	LOW RIGIDITY	1.60
1.80	1.41	4.95	122.	0.0	2.51	1.80	0.310	4.6	8.42	2.61	1.09		30.7	219.	SILTY SAND	LOW RIGIDITY	1.80
2.00	1.39	5.35	137.	0.0	2.84	1.80	0.346	4.0	6.64	2.30	0.99		31.1	233.	SILTY SAND	LOW RIGIDITY	2.00
2.20	0.95	3.65	93.	0.02	2.90	1.70	0.360	2.6	2.86	1.03	0.69		29.8	123.	SILTY SAND	LOOSE	2.20
2.40	1.15	1.75	21.	0.04	0.54	1.60	0.372	3.0	1.87	0.70	0.78	0.14		26.	SILTY CLAY	SOFT	2.40
2.60	0.85	1.45	21.	0.06	0.76	1.60	0.384	2.1	1.05	0.40	0.56	0.09		19.	CLAYEY SILT	COMPRESSIBLE	2.60
2.80	1.42	1.65	8.	0.08	0.17	1.50	0.394	3.4	2.29	0.90	0.87	0.17		11.	MUD		2.80
3.00	1.42	1.55	4.	0.10	0.10	1.50	0.404	3.3	2.16	0.87	0.84	0.16		6.	MUD		3.00
3.20	1.16	1.55	13.	0.12	0.37	1.60	0.416	2.5	1.42	0.59	0.67	0.12		15.	SILTY CLAY	SOFT	3.20
3.40	1.42	1.65	8.	0.14	0.18	1.50	0.426	3.0	1.88	0.80	0.79	0.16		10.	MUD		3.40
3.60	1.47	1.65	6.	0.16	0.14	1.50	0.436	3.0	1.89	0.82	0.79	0.16		8.	MUD		3.60
3.80	1.57	1.85	10.	0.18	0.20	1.50	0.446	3.1	1.99	0.89	0.81	0.17		13.	MUD		3.80
4.00	1.57	1.85	10.	0.20	0.21	1.50	0.456	3.0	1.88	0.86	0.78	0.17		12.	MUD		4.00
4.20	1.57	1.85	10.	0.22	0.21	1.50	0.466	2.9	1.78	0.83	0.76	0.16		12.	MUD		4.20
4.40	1.47	1.65	6.	0.24	0.14	1.50	0.476	2.6	1.49	0.71	0.69	0.14		7.	MUD		4.40
4.60	1.62	1.75	4.	0.26	0.09	1.50	0.486	2.8	1.70	0.82	0.74	0.16		5.	MUD		4.60
4.80	1.67	1.85	6.	0.28	0.13	1.50	0.496	2.8	1.70	0.84	0.74	0.17		7.	MUD		4.80
5.00	1.47	1.75	10.	0.30	0.24	1.50	0.506	2.3	1.25	0.63	0.62	0.13		10.	MUD		5.00
5.20	1.57	1.75	6.	0.32	0.14	1.50	0.516	2.4	1.35	0.70	0.65	0.14		6.	MUD		5.20
5.40	1.48	1.55	3.	0.34	0.06	1.50	0.526	2.2	1.13	0.59	0.59	0.13		2.	MUD		5.40
5.60	1.47	1.65	6.	0.36	0.16	1.50	0.536	2.1	1.06	0.57	0.56	0.12		5.	MUD		5.60
5.80	1.58	1.65	3.	0.38	0.06	1.50	0.546	2.2	1.15	0.63	0.60	0.13		2.	MUD		5.80
6.00	1.68	1.75	3.	0.40	0.06	1.50	0.556	2.3	1.24	0.69	0.62	0.15		3.	MUD		6.00
6.20	1.78	1.85	3.	0.42	0.05	1.50	0.566	2.4	1.33	0.75	0.65	0.16		3.	MUD		6.20
6.40	1.68	1.75	3.	0.44	0.06	1.50	0.576	2.1	1.12	0.64	0.58	0.14		2.	MUD		6.40
6.60	1.77	1.95	6.	0.46	0.14	1.50	0.586	2.2	1.19	0.70	0.61	0.15		6.	MUD		6.60

Z (m) PO (Bar) P1 (Bar) Ed (Bar) Uo (Bar) Id Gamma (t/CM) Sv (Bar) Kd OCR Pc (Bar) KO Cu (Bar) PHI (Deg) M (Bar) Soil Type Description Z (m)

Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)
6.80	1.87	2.00	4.	0.48	0.09	1.50	0.596	2.3	1.28	0.76	0.63	0.16		4.	MUD		6.80
7.00	1.82	2.00	6.	0.50	0.14	1.50	0.606	2.2	1.14	0.69	0.59	0.15		6.	MUD		7.00
7.20	1.87	2.00	4.	0.52	0.09	1.50	0.616	2.2	1.16	0.71	0.60	0.15		4.	MUD		7.20
7.40	1.87	2.05	6.	0.54	0.13	1.50	0.626	2.1	1.10	0.69	0.58	0.15		6.	MUD		7.40
7.60	1.88	1.95	3.	0.56	0.06	1.50	0.636	2.1	1.06	0.67	0.56	0.15		2.	MUD		7.60
7.80	1.97	2.15	6.	0.58	0.13	1.50	0.646	2.2	1.12	0.73	0.59	0.16		6.	MUD		7.80
8.00	1.97	2.15	6.	0.60	0.13	1.50	0.656	2.1	1.07	0.70	0.57	0.15		6.	MUD		8.00
8.20	2.11	2.45	12.	0.62	0.22	1.50	0.666	2.2	1.20	0.80	0.61	0.17		11.	MUD		8.20
8.40	2.17	2.35	6.	0.64	0.12	1.50	0.676	2.3	1.21	0.82	0.61	0.17		6.	MUD		8.40
8.60	2.07	2.25	6.	0.66	0.13	1.50	0.686	2.1	1.05	0.72	0.56	0.16		5.	MUD		8.60
8.80	2.17	2.35	6.	0.68	0.12	1.50	0.696	2.1	1.11	0.78	0.58	0.17		6.	MUD		8.80
9.00	2.12	2.35	8.	0.70	0.16	1.50	0.706	2.0	1.01	0.71	0.55	0.16		7.	MUD		9.00
9.20	2.27	2.55	10.	0.72	0.18	1.50	0.716	2.2	1.13	0.81	0.59	0.17		9.	MUD		9.20
9.40	2.27	2.55	10.	0.74	0.19	1.50	0.726	2.1	1.08	0.78	0.57	0.17		9.	MUD		9.40
9.60	2.17	2.45	10.	0.76	0.20	1.50	0.736	1.9	0.93	0.69	0.52	0.15		8.	MUD		9.60
9.80	2.37	2.65	10.	0.78	0.18	1.50	0.746	2.1	1.10	0.82	0.58	0.18		9.	MUD		9.80
10.00	2.37	2.55	6.	0.80	0.11	1.50	0.756	2.1	1.06	0.80	0.57	0.17		5.	MUD		10.00
10.20	2.27	2.45	6.	0.82	0.12	1.50	0.766	1.9	0.92	0.70	0.52	0.16		5.	MUD		10.20
10.40	2.37	2.55	6.	0.84	0.12	1.50	0.776	2.0	0.98	0.76	0.54	0.17		5.	MUD		10.40
10.60	2.37	2.65	10.	0.86	0.19	1.50	0.786	1.9	0.94	0.74	0.52	0.16		8.	MUD		10.60
10.80	2.46	2.85	13.	0.88	0.25	1.60	0.798	2.0	0.99	0.79	0.54	0.17		11.	CLAY	SOFT	10.80
11.00	2.56	2.90	12.	0.90	0.20	1.50	0.808	2.1	1.05	0.85	0.56	0.18		10.	MUD		11.00
11.20	2.67	2.95	10.	0.92	0.16	1.50	0.818	2.1	1.11	0.91	0.58	0.20		9.	MUD		11.20
11.40	2.77	3.05	10.	0.94	0.16	1.50	0.828	2.2	1.17	0.96	0.60	0.21		9.	MUD		11.40
11.60	2.87	3.15	10.	0.96	0.15	1.50	0.838	2.3	1.22	1.02	0.62	0.22		10.	MUD		11.60
11.80	3.25	3.85	21.	0.98	0.26	1.60	0.850	2.7	1.57	1.34	0.71	0.27		24.	CLAY	SOFT	11.80
12.00	3.70	4.40	24.	1.00	0.26	1.60	0.862	3.1	2.01	1.73	0.81	0.33		32.	CLAY	SOFT	12.00
12.20	4.14	4.95	28.	1.02	0.26	1.70	0.876	3.6	2.46	2.16	0.90	0.40		40.	CLAY	LOW CONSISTENCY	12.20
12.40	4.05	4.65	21.	1.04	0.20	1.60	0.888	3.4	2.28	2.02	0.87	0.38		29.	CLAY	SOFT	12.40
12.60	3.85	4.45	21.	1.06	0.21	1.60	0.900	3.1	1.98	1.78	0.81	0.34		27.	CLAY	SOFT	12.60
12.80	3.25	3.85	21.	1.08	0.28	1.60	0.912	2.4	1.31	1.20	0.64	0.25		21.	CLAY	SOFT	12.80
13.00	3.65	4.35	24.	1.10	0.28	1.60	0.924	2.8	1.65	1.52	0.73	0.30		29.	CLAY	SOFT	13.00
13.20	4.04	4.85	28.	1.12	0.28	1.70	0.938	3.1	2.00	1.87	0.81	0.36		36.	CLAY	LOW CONSISTENCY	13.20
13.40	3.56	4.05	17.	1.14	0.20	1.60	0.950	2.5	1.46	1.38	0.68	0.28		19.	CLAY	SOFT	13.40

Z (m) | PO (Bar) | P1 (Bar) | Ed (Bar) | Uo (Bar) | Id | Gamma (t/CM) | Sv (Bar) | Kd | OCR | Pc (Bar) | KO | Cu (Bar) | PHI (Deg) | M (Bar) | Soil Type | Description | Z (m)

NOTES: 1.For 0.9>Id>1.2 neither Cu nor Phi calculated.
2.1Bar=100KPa
3.# =1mm Deflection not reached.

<u>COMMENTS</u>

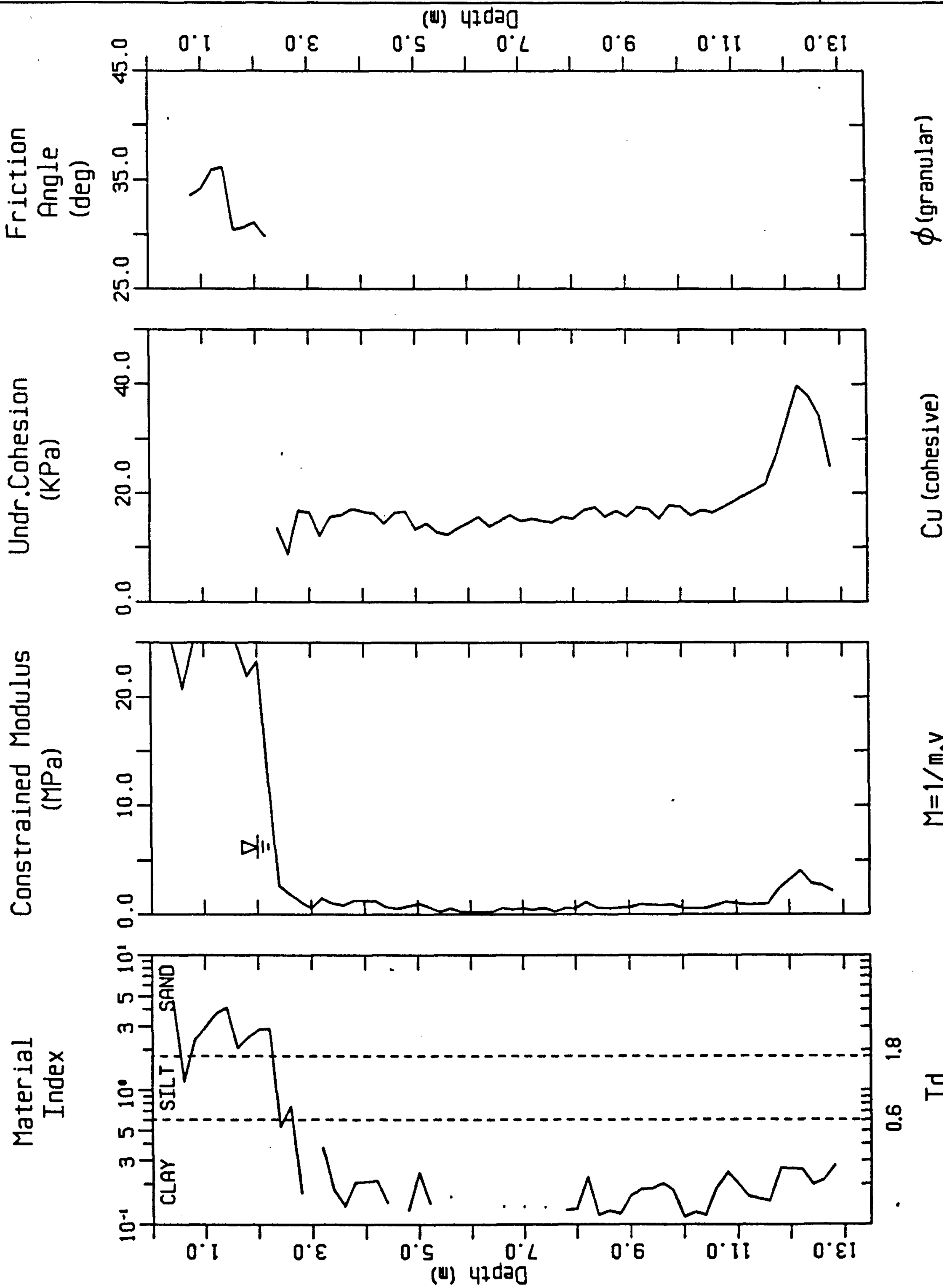
U.B.C. INSITU TESTING.

Location: QUEENSBOROUGH PILE RESEARCH SITE

Test No. DMT 85-1

INTERPRETED GEOTECHNICAL PARAMETERS.

Test Date: 23-08-85



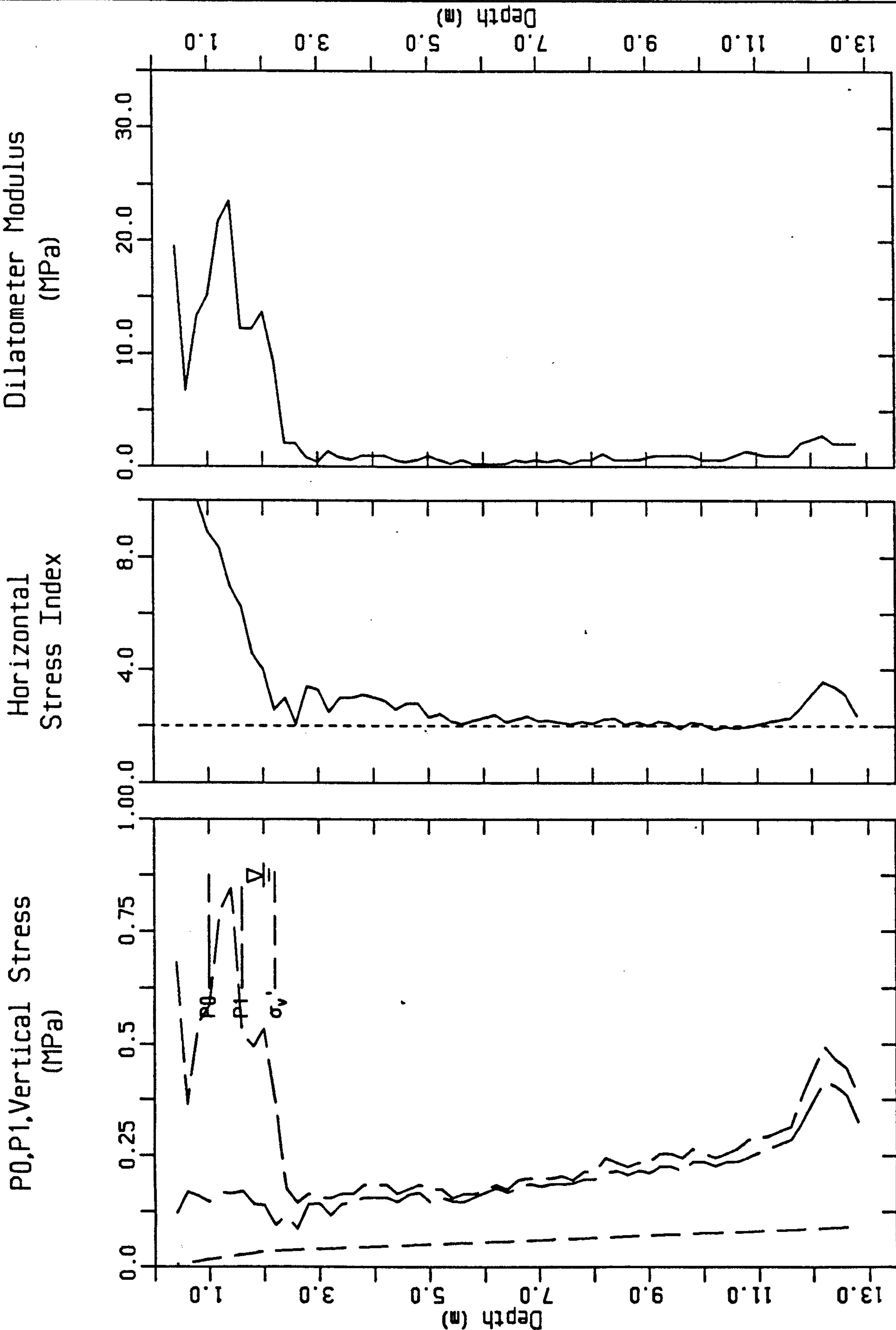
U.B.C. INSITU TESTING.

Location: QUEENSBOROUGH PILE RESEARCH SITE

Test No. DMT 85-1

Test Date: 23-08-85

INTERMEDIATE GEOTECHNICAL PARAMETERS



```
dmt 85-2
queensborough
29-08-85
 0.14 0.13 0.0 2.00
 0.100
34.600
  0
 0.60 3.5014.20
 0.80 4.5013.80
 1.00 2.50 9.70
 1.20 2.3011.40
 1.40 3.6011.20
 1.60 1.30 6.70
 2.00 2.7011.00
 2.20 3.10 8.80
 2.40 1.20 1.90
 2.60 1.00 2.40
 2.80 1.30 2.30
 3.00 1.40 2.20
 4.00 1.40 2.20
 5.00 1.40 2.10
 6.00 1.80 2.60
 7.00 1.80 2.60
 8.00 1.90 2.80
 9.00 2.00 2.70
10.00 2.40 3.20
11.00 2.50 3.20
12.00 3.40 4.60
13.00 3.60 4.60
14.00 3.20 4.00
14.20 3.50 4.30
14.40 4.00 5.00
14.60 3.20 4.20
14.80 3.20 4.00
15.00 3.20 5.50
15.20 2.80 4.00
15.40 4.00 7.60
15.60 9.6024.20
15.80 8.8020.40
16.00 5.8012.80
16.20 5.4013.80
16.40 9.0024.80
16.60 9.0022.00
16.80 7.8022.80
17.00 9.0020.20
17.20 4.6010.80
17.40 9.6021.60
17.60 6.6013.40
17.8011.0024.00
18.0010.2023.00
18.2010.2023.60
18.40 8.4019.00
18.60 7.8017.80
18.80 7.1016.80
19.00 6.9016.60
19.20 7.5017.80
19.40 7.5018.20
19.60 7.8018.80
```

117	31.40	5.40	7.00
118	31.60	8.70	17.00
119	31.80	5.80	7.30
120	32.00	6.20	7.30
121	32.20	8.00	13.00
122	32.40	7.70	13.40
123	32.60	8.90	15.70
124	32.80	6.20	10.20
125	33.00	7.10	11.30
126	33.20	5.70	8.00
127	33.40	6.60	8.00
128	33.60	8.80	14.40

FILE NAME:dmt-pr-85-2
LOCATION:queensborough DATE:29-08-85 TEST NUMBER:dmt 85-2
INTERMEDIATE DILATOMETER PARAMETERSFROM 0.60M TO33.60M.
NUMBER OF DATA POINTS: 121
ZW= 2.00M. DA= 0.14 DB= 0.13 ZM= 0.0

DEPTH	A	B	PO	P1	ED
0.6	3.5	14.2	3.1	14.1	378.9
0.8	4.5	13.8	4.2	13.7	328.1
1.0	2.5	9.7	2.3	9.6	251.8
1.2	2.3	11.4	2.0	11.3	320.8
1.4	3.6	11.2	3.4	11.1	266.3
1.6	1.3	6.7	1.2	6.6	186.4
2.0	2.7	11.0	2.4	10.9	291.7
2.2	3.1	8.8	3.0	8.7	197.3
2.4	1.2	1.9	1.3	1.8	15.6
2.6	1.0	2.4	1.1	2.3	41.1
2.8	1.3	2.3	1.4	2.2	26.5
3.0	1.4	2.2	1.5	2.1	19.3
4.0	1.4	2.2	1.5	2.1	19.3
5.0	1.4	2.1	1.5	2.0	15.6
6.0	1.8	2.6	1.9	2.5	19.3
7.0	1.8	2.6	1.9	2.5	19.3
8.0	1.9	2.8	2.0	2.7	22.9
9.0	2.0	2.7	2.1	2.6	15.6
10.0	2.4	3.2	2.5	3.1	19.3
11.0	2.5	3.2	2.6	3.1	15.6
12.0	3.4	4.6	3.5	4.5	33.8
13.0	3.6	4.6	3.7	4.5	26.5
14.0	3.2	4.0	3.3	3.9	19.3
14.2	3.5	4.3	3.6	4.2	19.3
14.4	4.0	5.0	4.1	4.9	26.5
14.6	3.2	4.2	3.3	4.1	26.5
14.8	3.2	4.0	3.3	3.9	19.3
15.0	3.2	5.5	3.2	5.4	73.8
15.2	2.8	4.0	2.9	3.9	33.8
15.4	4.0	7.6	4.0	7.5	121.0
15.6	9.6	24.2	9.0	24.1	520.6
15.8	8.8	20.4	8.4	20.3	411.6
16.0	5.8	12.8	5.6	12.7	244.5
16.2	5.4	13.8	5.1	13.7	295.4
16.4	9.0	24.8	8.4	24.7	564.2
16.6	9.0	22.0	8.5	21.9	462.5
16.8	7.8	22.8	7.2	22.7	535.1
17.0	9.0	20.2	8.6	20.1	397.1
17.2	4.6	10.8	4.4	10.7	215.4
17.4	9.6	21.6	9.2	21.5	426.2
17.6	6.6	13.4	6.4	13.3	237.2
17.8	11.0	24.0	10.5	23.9	462.5
18.0	10.2	23.0	9.7	22.9	455.2
18.2	10.2	23.6	9.7	23.5	477.0
18.4	8.4	19.0	8.0	18.9	375.3
18.6	7.8	17.8	7.5	17.7	353.5
18.8	7.1	16.8	6.8	16.7	342.6
19.0	6.9	16.6	6.6	16.5	342.6
19.2	7.5	17.8	7.1	17.7	364.4
19.4	7.5	18.2	7.1	18.1	378.9
19.6	7.8	18.8	7.4	18.7	389.8
19.8	8.3	20.4	7.8	20.3	429.8
20.0	9.5	23.8	8.9	23.7	509.7
20.2	11.8	30.2	11.0	30.1	658.7

U.B.C. INSITU TESTING RESEARCH GROUP.

File Name:dmt-pr-85-2
Location:queensborough

Record of Dilatometer test No:dmt 85-2
Date:29-08-85

Calibration Information:DA= 0.14 Bars DB= 0.13 Bars ZM= 0.0 Bars ZW= 2.00 metres

Gamma=Bulk unit weight
Sv =Effective over.stress
Uo =Pore pressure
Id =Material index
Ed =Dilatometer modulus
Kd =Horizontal stress index

INTERPRETED GEOTECHNICAL PARAMETERS
Ko =Insitu earth press.coeff.
OCR=Overconsolidation Ratio
M =Constrained modulus
Cu =Undrained cohesion(cohesive)
PHI=Friction Angle(cohesionless)

Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)
0.60	3.12	14.07	379.	0.0	3.51	1.90	0.100	31.2	*****	33.21	3.56		38.8	1322.	SAND	CEMENTED	0.60
0.80	4.19	13.67	328.	0.0	2.26	1.90	0.138	30.4	*****	43.52	3.51		35.3	1165.	SILTY SAND	CEMENTED	0.80
1.00	2.29	9.57	252.	0.0	3.17	1.90	0.176	13.0	62.72	11.04	2.16		35.9	687.	SILTY SAND	CEMENTED	1.00
1.20	2.00	11.27	321.	0.0	4.64	1.90	0.214	9.3	33.19	7.10	1.76		38.4	783.	SAND	CEMENTED	1.20
1.40	3.37	11.07	266.	0.0	2.28	1.90	0.252	13.4	66.03	16.64	2.20		33.4	739.	SILTY SAND	MEDIUM RIGIDITY	1.40
1.60	1.18	6.57	186.	0.0	4.55	1.80	0.288	4.1	6.92	1.99	1.01		35.0	322.	SAND	LOW RIGIDITY	1.60
2.00	2.44	10.87	292.	0.0	3.46	1.90	0.364	6.7	17.60	6.41	1.42		34.5	628.	SAND	MEDIUM RIGIDITY	2.00
2.20	2.97	8.67	197.	0.02	1.93	1.90	0.382	7.7	21.13	8.07	1.56		30.7	445.	SILTY SAND	MEDIUM RIGIDITY	2.20
2.40	1.32	1.77	16.	0.04	0.35	1.60	0.394	3.2	2.13	0.84	0.84	0.16		21.	SILTY CLAY	SOFT	2.40
2.60	1.08	2.27	41.	0.06	1.16	1.60	0.406	2.5	1.43	0.58	0.68			47.	SILT	COMPRESSIBLE	2.60
2.80	1.40	2.17	27.	0.08	0.58	1.60	0.418	3.2	2.05	0.86	0.82	0.16		35.	SILTY CLAY	SOFT	2.80
3.00	1.51	2.07	19.	0.10	0.39	1.60	0.430	3.3	2.17	0.93	0.85	0.18		26.	SILTY CLAY	SOFT	3.00
4.00	1.51	2.07	19.	0.20	0.42	1.60	0.490	2.7	1.58	0.77	0.71	0.16		22.	SILTY CLAY	SOFT	4.00
5.00	1.52	1.97	16.	0.30	0.37	1.60	0.550	2.2	1.17	0.65	0.60	0.14		15.	SILTY CLAY	SOFT	5.00
6.00	1.91	2.47	19.	0.40	0.37	1.60	0.610	2.5	1.40	0.85	0.67	0.18		21.	SILTY CLAY	SOFT	6.00
7.00	1.91	2.47	19.	0.50	0.39	1.60	0.670	2.1	1.09	0.73	0.57	0.16		17.	SILTY CLAY	SOFT	7.00
8.00	2.01	2.67	23.	0.60	0.47	1.60	0.730	1.9	0.95	0.69	0.53	0.15		19.	SILTY CLAY	SOFT	8.00
9.00	2.12	2.57	16.	0.70	0.32	1.60	0.790	1.8	0.85	0.67	0.49	0.15		13.	CLAY	SOFT	9.00
10.00	2.51	3.07	19.	0.80	0.32	1.60	0.850	2.0	1.01	0.86	0.55	0.19		17.	CLAY	SOFT	10.00
11.00	2.62	3.07	16.	0.90	0.26	1.60	0.910	1.9	0.91	0.83	0.51	0.19		13.	CLAY	SOFT	11.00
12.00	3.49	4.47	34.	1.00	0.39	1.70	0.980	2.5	1.46	1.43	0.68	0.29		37.	SILTY CLAY	LOW CONSISTENCY	12.00
13.00	3.70	4.47	27.	1.10	0.29	1.60	1.040	2.5	1.42	1.48	0.67	0.30		29.	CLAY	SOFT	13.00
14.00	3.31	3.87	19.	1.20	0.26	1.60	1.100	1.9	0.94	1.03	0.52	0.23		16.	CLAY	SOFT	14.00
14.20	3.61	4.17	19.	1.22	0.23	1.60	1.112	2.2	1.12	1.25	0.58	0.27		18.	CLAY	SOFT	14.20
14.40	4.10	4.87	27.	1.24	0.27	1.70	1.126	2.5	1.45	1.64	0.68	0.33		29.	CLAY	LOW CONSISTENCY	14.40
14.60	3.30	4.07	27.	1.26	0.38	1.60	1.138	1.8	0.85	0.96	0.49	0.22		23.	SILTY CLAY	SOFT	14.60
14.80	3.31	3.87	19.	1.28	0.27	1.60	1.150	1.8	0.83	0.95	0.48	0.22		16.	CLAY	SOFT	14.80
15.00	3.24	5.37	74.	1.30	1.10	1.70	1.164	1.7	0.75	0.87	0.45			63.	SILT	LOW DENSITY	15.00
15.20	2.89	3.87	34.	1.32	0.62	1.60	1.176	1.3	0.53	0.63	0.35	0.16		29.	CLAYEY SILT	COMPRESSIBLE	15.20
15.40	3.97	7.47	121.	1.34	1.33	1.70	1.190	2.2	1.28	1.53	0.60		26.1	124.	SANDY SILT	LOW DENSITY	15.40
15.60	9.02	24.07	521.	1.36	1.96	2.00	1.210	6.3	15.11	18.28	1.37		30.2	1079.	SILTY SAND	RIGID	15.60
15.80	8.37	20.27	412.	1.38	1.70	1.95	1.229	5.7	9.06	11.14	1.27		29.1	808.	SANDY SILT	DENSE	15.80

Z (m) PO (Bar) P1 (Bar) Ed (Bar) Uo (Bar) Id Gamma (t/CM) Sv (Bar) Kd OCR Pc (Bar) KO Cu (Bar) PHI (Deg) M (Bar) Soil Type Description Z (m)

Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)
16.00	5.60	12.67	245.	1.40	1.68	1.80	1.245	3.4	3.51	4.37	0.86		27.9	358.	SANDY SILT	MEDIUM DENSITY	16.00
16.20	5.13	13.67	295.	1.42	2.30	1.90	1.263	2.9	3.65	4.61	0.77		29.0	408.	SILTY SAND	MEDIUM RIGIDITY	16.20
16.40	8.36	24.67	564.	1.44	2.36	2.00	1.283	5.4	11.64	14.94	1.23		30.8	1094.	SILTY SAND	RIGID	16.40
16.60	8.50	21.87	462.	1.46	1.90	2.00	1.303	5.4	10.37	13.51	1.23		29.6	889.	SILTY SAND	RIGID	16.60
16.80	7.20	22.67	535.	1.48	2.70	2.00	1.323	4.3	7.63	10.10	1.05		31.0	940.	SILTY SAND	RIGID	16.80
17.00	8.59	20.07	397.	1.50	1.62	1.95	1.342	5.3	7.23	9.71	1.21		28.7	750.	SANDY SILT	DENSE	17.00
17.20	4.44	10.67	215.	1.52	2.13	1.90	1.360	2.1	2.01	2.73	0.58		27.9	232.	SILTY SAND	MEDIUM RIGIDITY	17.20
17.40	9.15	21.47	426.	1.54	1.62	1.95	1.379	5.5	7.80	10.76	1.24		28.8	823.	SANDY SILT	DENSE	17.40
17.60	6.41	13.27	237.	1.56	1.41	1.80	1.395	3.5	2.88	4.02	0.89		27.3	350.	SANDY SILT	MEDIUM DENSITY	17.60
17.80	10.50	23.87	462.	1.58	1.50	1.95	1.414	6.3	8.53	12.07	1.36		28.8	950.	SANDY SILT	DENSE	17.80
18.00	9.71	22.87	455.	1.60	1.62	1.95	1.433	5.7	8.19	11.73	1.27		28.9	890.	SANDY SILT	DENSE	18.00
18.20	9.68	23.47	477.	1.62	1.71	1.95	1.452	5.6	8.76	12.72	1.25		29.1	925.	SANDY SILT	DENSE	18.20
18.40	8.02	18.87	375.	1.64	1.70	1.95	1.471	4.3	5.58	8.21	1.05		28.5	640.	SANDY SILT	DENSE	18.40
18.60	7.45	17.67	353.	1.66	1.76	1.95	1.490	3.9	4.90	7.30	0.96		28.4	567.	SANDY SILT	DENSE	18.60
18.80	6.77	16.67	343.	1.68	1.95	2.00	1.510	3.4	4.50	6.79	0.86		28.5	507.	SILTY SAND	RIGID	18.80
19.00	6.57	16.47	343.	1.70	2.03	2.00	1.530	3.2	4.25	6.50	0.82		28.6	491.	SILTY SAND	RIGID	19.00
19.20	7.14	17.67	364.	1.72	1.94	2.00	1.550	3.5	4.81	7.45	0.89		28.6	552.	SILTY SAND	RIGID	19.20
19.40	7.12	18.07	379.	1.74	2.04	2.00	1.570	3.4	4.89	7.68	0.87		28.8	569.	SILTY SAND	RIGID	19.40
19.60	7.40	18.67	390.	1.76	2.00	2.00	1.590	3.5	5.21	8.29	0.90		28.7	597.	SILTY SAND	RIGID	19.60
19.80	7.85	20.27	430.	1.78	2.05	2.00	1.610	3.8	5.87	9.45	0.94		29.0	684.	SILTY SAND	RIGID	19.80
20.00	8.94	23.67	510.	1.80	2.06	2.00	1.630	4.4	7.81	12.74	1.05		29.4	883.	SILTY SAND	RIGID	20.00
20.20	11.03	30.07	659.	1.82	2.07	2.15	1.653	5.6	12.39	20.48	1.25		30.1	1289.	SILTY SAND	VERY RIGID	20.20
20.40	12.58	34.87	771.	1.84	2.08	2.15	1.676	6.4	16.16	27.09	1.38		30.6	1609.	SILTY SAND	VERY RIGID	20.40
20.60	15.07	33.27	630.	1.86	1.38	2.10	1.698	7.8	10.42	17.69	1.57		28.9	1420.	SANDY SILT	VERY DENSE	20.60
20.80	9.41	24.67	528.	1.88	2.03	2.00	1.718	4.4	7.83	13.46	1.06		29.3	914.	SILTY SAND	RIGID	20.80
21.00	11.45	25.87	499.	1.90	1.51	2.10	1.740	5.5	6.83	11.88	1.24		28.5	958.	SANDY SILT	VERY DENSE	21.00
21.20	10.34	22.87	433.	1.92	1.49	1.95	1.759	4.8	5.29	9.31	1.13		28.1	775.	SANDY SILT	DENSE	21.20
21.40	7.16	15.07	274.	1.94	1.51	1.95	1.778	2.9	2.38	4.22	0.77		27.2	360.	SANDY SILT	DENSE	21.40
21.60	9.02	21.97	448.	1.96	1.83	2.00	1.798	3.9	5.35	9.62	0.97		28.6	725.	SILTY SAND	RIGID	21.60
21.80	8.60	19.87	390.	1.98	1.70	1.95	1.817	3.6	4.10	7.45	0.92		28.1	600.	SANDY SILT	DENSE	21.80
22.00	7.26	13.07	201.	2.00	1.10	1.80	1.833	2.9	1.76	3.22	0.76			254.	SILT	MEDIUM DENSITY	22.00
22.20	7.78	15.27	259.	2.02	1.30	1.95	1.852	3.1	2.17	4.02	0.81		26.7	351.	SANDY SILT	DENSE	22.20
22.40	11.03	23.87	444.	2.04	1.43	1.95	1.871	4.8	5.00	9.35	1.13		27.9	795.	SANDY SILT	DENSE	22.40
22.60	6.17	18.17	415.	2.06	2.92	2.00	1.891	2.2	2.05	3.87	0.59		29.4	484.	SILTY SAND	RIGID	22.60
22.80	9.74	20.27	364.	2.08	1.38	1.95	1.910	4.0	3.51	6.71	0.99		27.4	586.	SANDY SILT	DENSE	22.80
23.00	8.23	18.97	372.	2.10	1.75	1.95	1.929	3.2	3.37	6.50	0.82		27.9	524.	SANDY SILT	DENSE	23.00
23.20	12.09	27.87	546.	2.12	1.58	2.10	1.951	5.1	6.56	12.80	1.18		28.5	1013.	SANDY SILT	VERY DENSE	23.20
23.40	14.03	39.37	877.	2.14	2.13	2.15	1.974	6.0	14.37	28.37	1.32		30.5	1781.	SILTY SAND	VERY RIGID	23.40
23.60	15.47	37.97	779.	2.16	1.69	2.10	1.996	6.7	11.86	23.67	1.42		29.5	1645.	SANDY SILT	VERY DENSE	23.60
23.80	14.42	37.87	811.	2.18	1.92	2.15	2.019	6.1	13.12	26.50	1.33		29.9	1647.	SILTY SAND	VERY RIGID	23.80
24.00	13.30	30.97	611.	2.20	1.59	2.10	2.041	5.4	7.38	15.07	1.23		28.7	1171.	SANDY SILT	VERY DENSE	24.00
24.20	13.31	28.57	528.	2.22	1.38	2.10	2.063	5.4	5.68	11.71	1.22		28.0	1001.	SANDY SILT	VERY DENSE	24.20
24.40	9.41	20.57	386.	2.24	1.56	1.95	2.082	3.4	3.24	6.74	0.88		27.6	569.	SANDY SILT	DENSE	24.40
24.60	9.05	21.37	426.	2.26	1.81	2.00	2.102	3.2	3.67	7.72	0.83		28.1	610.	SILTY SAND	RIGID	24.60
24.80	9.81	20.97	386.	2.28	1.48	1.95	2.121	3.5	3.18	6.74	0.90		27.5	578.	SANDY SILT	DENSE	24.80
25.00	8.04	18.47	361.	2.30	1.82	2.00	2.141	2.7	2.62	5.60	0.71		27.7	453.	SILTY SAND	RIGID	25.00

Z (m) | PO (Bar) | P1 (Bar) | Ed (Bar) | Uo (Bar) | Id | Gamma (t/CM) | Sv (Bar) | Kd | OCR | Pc (Bar) | KO | Cu (Bar) | PHI (Deg) | M (Bar) | Soil Type | Description | Z (m)

Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)
25.20	8.86	18.87	346.	2.32	1.53	1.95	2.160	3.0	2.54	5.48	0.79		27.3	467.	SANDY SILT	DENSE	25.20
25.40	7.72	16.57	306.	2.34	1.65	1.95	2.179	2.5	1.96	4.27	0.66		27.2	356.	SANDY SILT	DENSE	25.40
25.60	8.54	16.97	292.	2.36	1.36	1.95	2.198	2.8	1.95	4.28	0.74		26.7	368.	SANDY SILT	DENSE	25.60
25.80	5.93	8.27	81.	2.38	0.66	1.80	2.214	1.6	0.71	1.57	0.43	0.37		69.	CLAYEY SILT	MEDIUM DENSITY	25.80
26.00	9.09	20.67	401.	2.40	1.73	1.95	2.233	3.0	2.97	6.64	0.78		27.8	542.	SANDY SILT	DENSE	26.00
26.20	11.08	26.97	550.	2.42	1.83	2.00	2.253	3.8	5.15	11.61	0.96		28.5	878.	SILTY SAND	RIGID	26.20
26.40	13.06	29.37	564.	2.44	1.53	2.10	2.275	4.7	5.33	12.14	1.11		28.2	996.	SANDY SILT	VERY DENSE	26.40
26.60	11.11	24.27	455.	2.46	1.52	1.95	2.294	3.8	3.65	8.38	0.94		27.7	710.	SANDY SILT	DENSE	26.60
26.80	10.60	21.87	390.	2.48	1.39	1.95	2.313	3.5	2.86	6.61	0.89		27.2	577.	SANDY SILT	DENSE	26.80
27.00	11.35	23.67	426.	2.50	1.39	1.95	2.332	3.8	3.26	7.60	0.95		27.4	664.	SANDY SILT	DENSE	27.00
27.20	11.33	26.27	517.	2.52	1.70	1.95	2.351	3.7	4.29	10.08	0.94		28.1	809.	SANDY SILT	DENSE	27.20
27.40	11.35	27.87	571.	2.54	1.87	2.15	2.374	3.7	5.03	11.93	0.93		28.6	895.	SILTY SAND	VERY RIGID	27.40
27.60	14.23	33.37	662.	2.56	1.64	2.10	2.396	4.9	6.43	15.40	1.14		28.6	1200.	SANDY SILT	VERY DENSE	27.60
27.80	13.99	31.87	619.	2.58	1.57	2.10	2.418	4.7	5.62	13.59	1.11		28.3	1100.	SANDY SILT	VERY DENSE	27.80
28.00	11.96	30.47	640.	2.60	1.98	2.15	2.441	3.8	5.93	14.47	0.95		28.9	1027.	SILTY SAND	VERY RIGID	28.00
28.20	15.77	36.07	702.	2.62	1.54	2.10	2.463	5.3	6.77	16.67	1.22		28.5	1331.	SANDY SILT	VERY DENSE	28.20
28.40	8.47	18.27	339.	2.64	1.68	1.95	2.482	2.4	1.85	4.59	0.64		27.2	379.	SANDY SILT	DENSE	28.40
28.60	8.66	14.47	201.	2.66	0.97	1.95	2.501	2.4	1.33	3.32	0.65			215.	SILT	DENSE	28.60
28.80	7.54	15.87	288.	2.68	1.71	1.95	2.520	1.9	1.33	3.36	0.53		26.9	269.	SANDY SILT	DENSE	28.80
29.00	8.88	18.47	332.	2.70	1.55	1.95	2.539	2.4	1.78	4.51	0.66		26.9	378.	SANDY SILT	DENSE	29.00
29.20	9.71	20.87	386.	2.72	1.60	1.95	2.558	2.7	2.25	5.76	0.73		27.3	484.	SANDY SILT	DENSE	29.20
29.40	7.12	15.87	303.	2.74	2.00	1.90	2.576	1.7	1.28	3.30	0.46		27.2	257.	SILTY SAND	MEDIUM RIGIDITY	29.40
29.60	5.38	8.67	114.	2.76	1.25	1.70	2.590	1.0	0.35	0.91	0.23		25.0	97.	SANDY SILT	LOW DENSITY	29.60
29.80	7.58	15.17	263.	2.78	1.58	1.80	2.606	1.8	1.12	2.93	0.50		26.5	230.	SANDY SILT	MEDIUM DENSITY	29.80
30.00	7.63	14.07	223.	2.80	1.33	1.80	2.622	1.8	0.96	2.51	0.50		25.7	189.	SANDY SILT	MEDIUM DENSITY	30.00
30.20	6.99	12.17	179.	2.82	1.24	1.80	2.638	1.6	0.71	1.87	0.43		25.0	152.	SANDY SILT	MEDIUM DENSITY	30.20
30.40	6.66	12.47	201.	2.84	1.52	1.80	2.654	1.4	0.71	1.89	0.38		25.7	171.	SANDY SILT	MEDIUM DENSITY	30.40
30.60	6.04	8.07	70.	2.86	0.64	1.70	2.668	1.2	0.45	1.19	0.30	0.31		60.	CLAYEY SILT	LOW DENSITY	30.60
30.80	7.11	11.97	168.	2.88	1.15	1.80	2.684	1.6	0.69	1.85	0.42			143.	SILT	MEDIUM DENSITY	30.80
31.00	6.08	7.27	41.	2.90	0.37	1.70	2.698	1.2	0.44	1.18	0.29	0.31		35.	SILTY CLAY	LOW CONSISTENCY	31.00
31.20	5.68	6.97	45.	2.92	0.47	1.70	2.712	1.0	0.35	0.94	0.23	0.26		38.	SILTY CLAY	LOW CONSISTENCY	31.20
31.40	5.47	6.87	48.	2.94	0.55	1.70	2.726	0.9	0.30	0.82	0.20	0.23		41.	SILTY CLAY	LOW CONSISTENCY	31.40
31.60	8.44	16.87	292.	2.96	1.54	1.95	2.745	2.0	1.25	3.44	0.54		26.5	276.	SANDY SILT	DENSE	31.60
31.80	5.88	7.17	45.	2.98	0.45	1.70	2.759	1.1	0.37	1.01	0.25	0.27		38.	SILTY CLAY	LOW CONSISTENCY	31.80
32.00	6.30	7.17	30.	3.00	0.26	1.70	2.773	1.2	0.44	1.23	0.30	0.32		26.	CLAY	LOW CONSISTENCY	32.00
32.20	7.90	12.87	172.	3.02	1.02	1.80	2.789	1.8	0.81	2.27	0.48			146.	SILT	MEDIUM DENSITY	32.20
32.40	7.57	13.27	197.	3.04	1.26	1.80	2.805	1.6	0.74	2.08	0.44		25.0	168.	SANDY SILT	MEDIUM DENSITY	32.40
32.60	8.71	15.57	237.	3.06	1.21	1.95	2.824	2.0	1.01	2.85	0.55		25.3	217.	SANDY SILT	DENSE	32.60
32.80	6.15	10.07	136.	3.08	1.27	1.80	2.840	1.1	0.40	1.12	0.26		25.0	115.	SANDY SILT	MEDIUM DENSITY	32.80
33.00	7.04	11.17	143.	3.10	1.05	1.80	2.856	1.4	0.56	1.60	0.36			121.	SILT	MEDIUM DENSITY	33.00
33.20	5.74	7.87	74.	3.12	0.81	1.70	2.870	0.9	0.29	0.84	0.19	0.24		63.	SILT	LOW DENSITY	33.20
33.40	6.68	7.87	41.	3.14	0.33	1.70	2.884	1.2	0.47	1.35	0.31	0.35		35.	SILTY CLAY	LOW CONSISTENCY	33.40
33.60	8.67	14.27	194.	3.16	1.02	1.80	2.900	1.9	0.92	2.68	0.52			165.	SILT	MEDIUM DENSITY	33.60
Z (m)	PO (Bar)	P1 (Bar)	Ed (Bar)	Uo (Bar)	Id	Gamma (t/CM)	Sv (Bar)	Kd	OCR	Pc (Bar)	KO	Cu (Bar)	PHI (Deg)	M (Bar)	Soil Type	Description	Z (m)

NOTES: 1.For 0.9>Id>1.2 neither Cu nor Phi calculated.
2.1Bar=100KPa
3.# =1mm Deflection not reached.

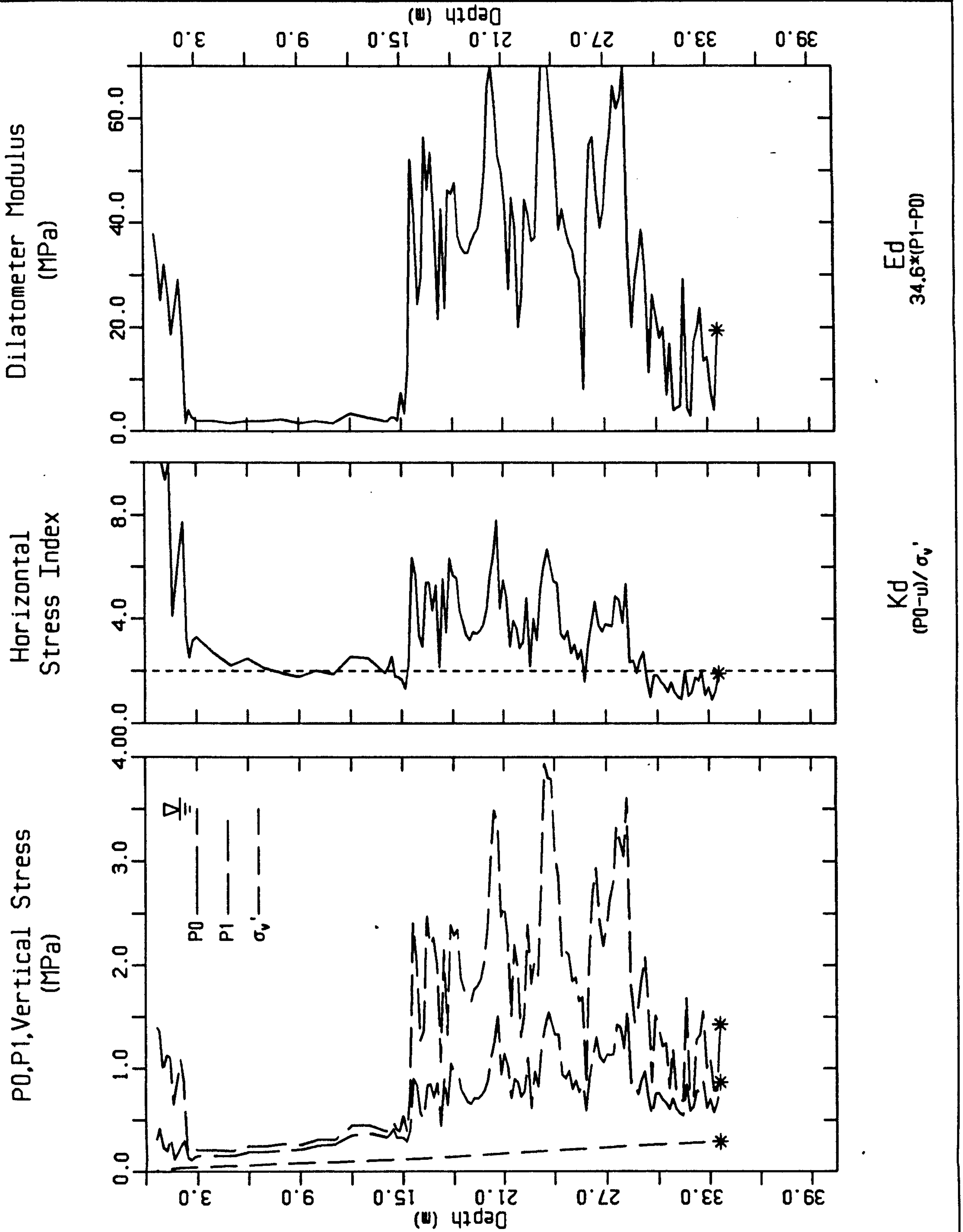
U.B.C. INSITU TESTING.

Location: QUEENSBOROUGH PILE RESEARCH SITE

Test No. DMT 85-2

Test Date: 29-08-85

INTERMEDIATE GEOTECHNICAL PARAMETERS



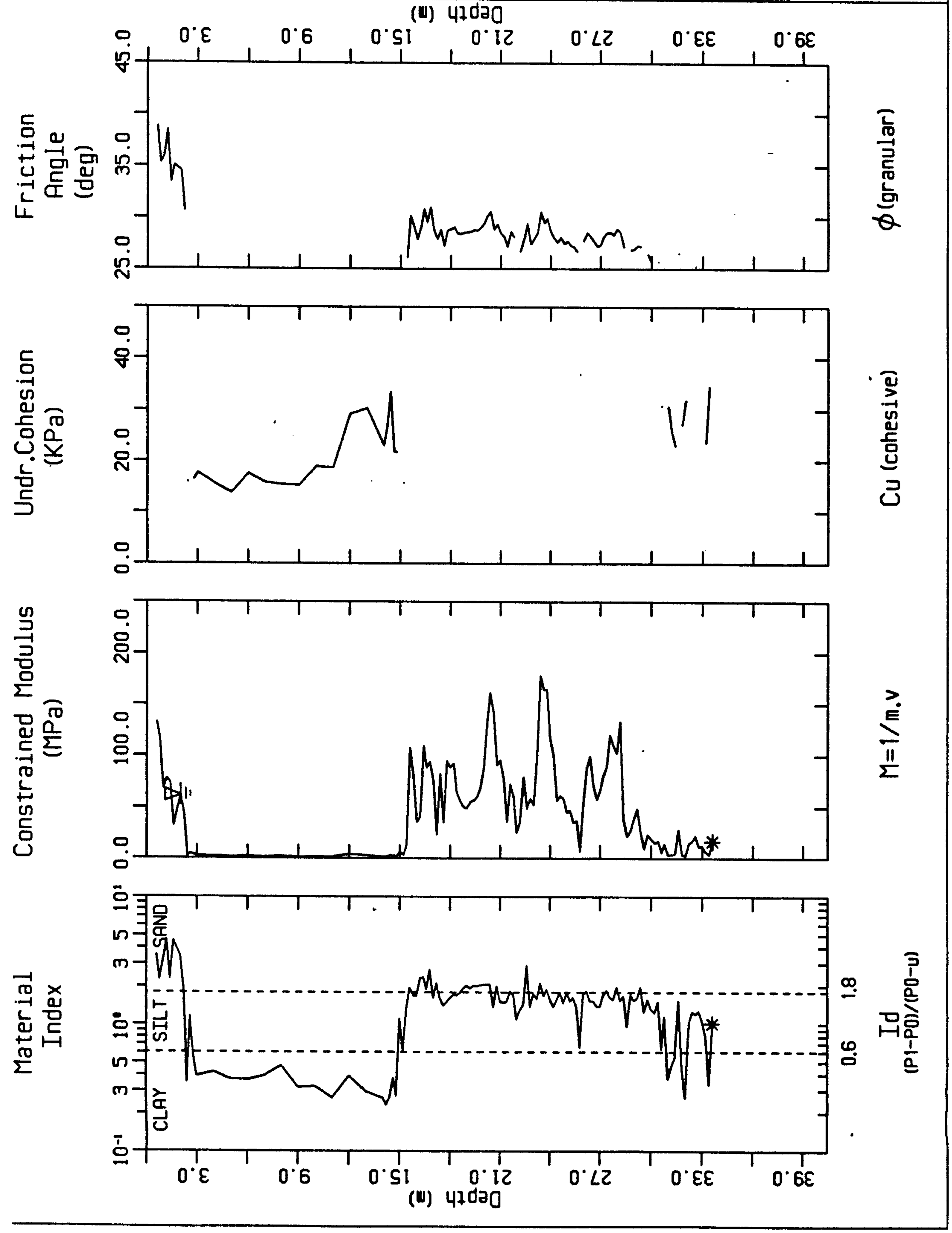
U.B.C. INSITU TESTING.

Location: QUEENSBOROUGH PILE RESEARCH SITE

Test No. DMT 85-2

Test Date: 29-08-85

INTERPRETED GEOTECHNICAL PARAMETERS.



ASDMT

SUPPLEMENTARY DMT DATA

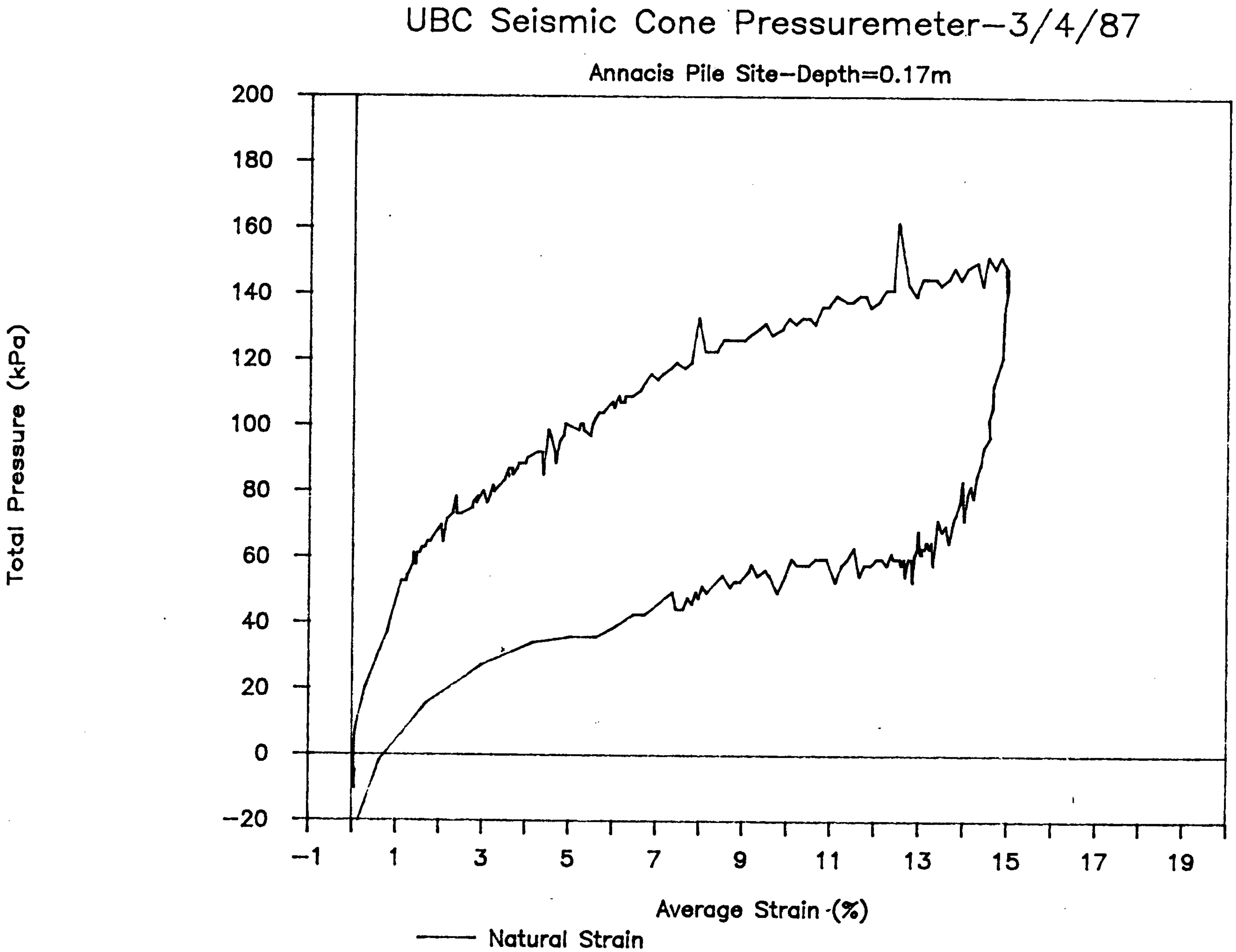
UBC PILE RESEARCH SITE, QUEENSBOROUGH, LULU ISLAND

DMT-PR-85-1 DELA=0.08 DELB=0.55 22Aug85

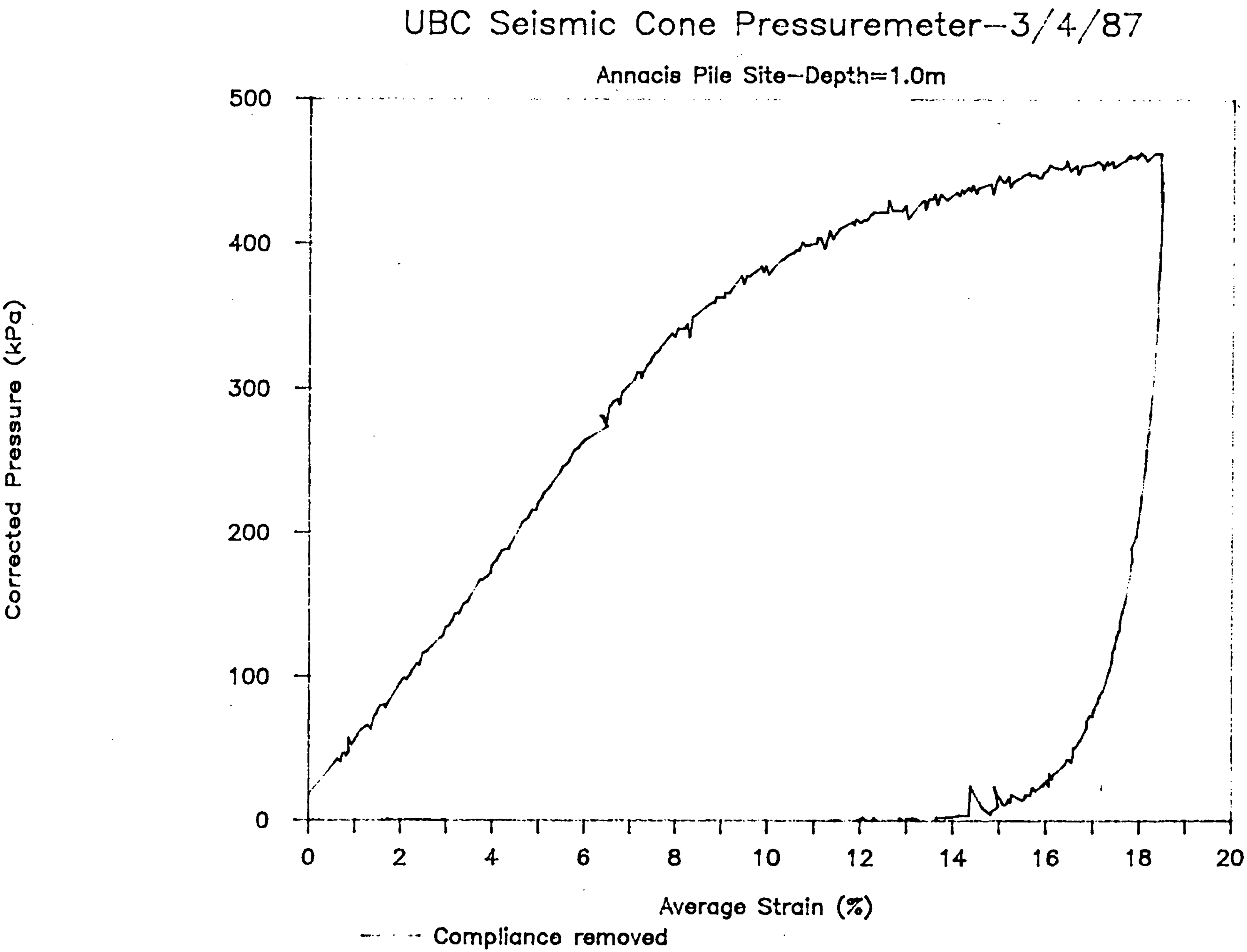
Depth(m)	A	B	C	Time(min)
3.4	1.35			0
	1.1			7
	1.0			17
	1.0	2.0	0.4	36
4.0	1.5			0
	1.2			5
	1.1			10
	1.1			15
	1.05	2.0	0.6	20
5.0	1.4			0
	1.2			5
	1.1			11
	1.1	2.0	0.65	16
6.0	1.6			0
	1.4	2.25	0.8	5
7.0	1.75			0
	1.5	2.4	0.9	5
8.0	1.9			0
	1.7			5
	1.7	2.8	0.8	7
9.0	2.05			0
	1.8	2.75	1.0	5
10.0	2.3			0
	1.75			10
	1.6			20
	1.65	2.8	0.8	31
11.0	-			0
	2.1	3.2	1.4	5
12.0	-			0
	2.75	4.0	1.9	20
13.0	-			0
	3.2	4.6	2.4	5

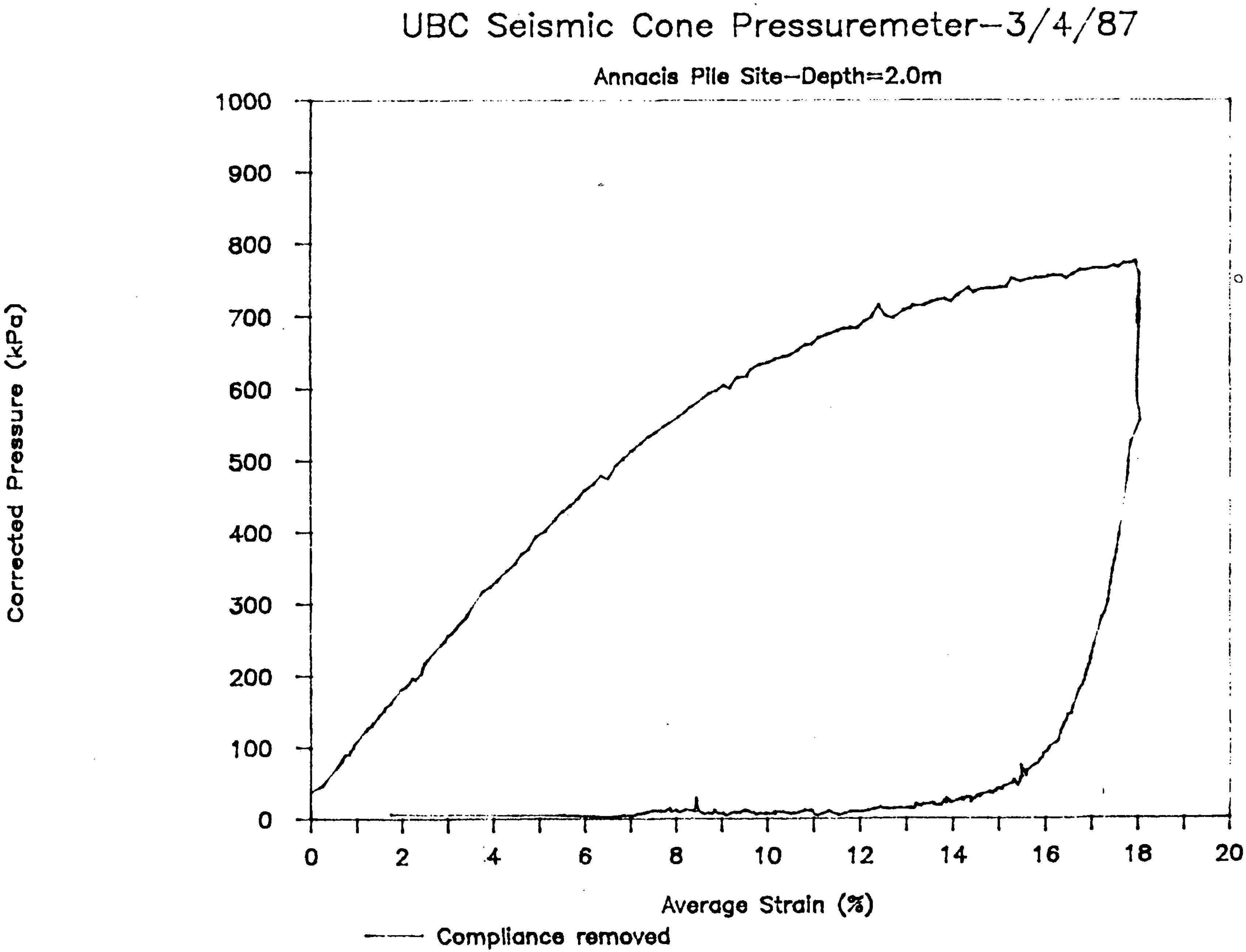
DMT-PR-85-2 DELA=0.14 DELB=0.13 29Aug85

Depth(m)	A	B	C	Time(min)
33.2	5.7			0
	5.0			13
	4.9			24
	4.9			40
	4.8			50
	4.7	7.6	2.5	62

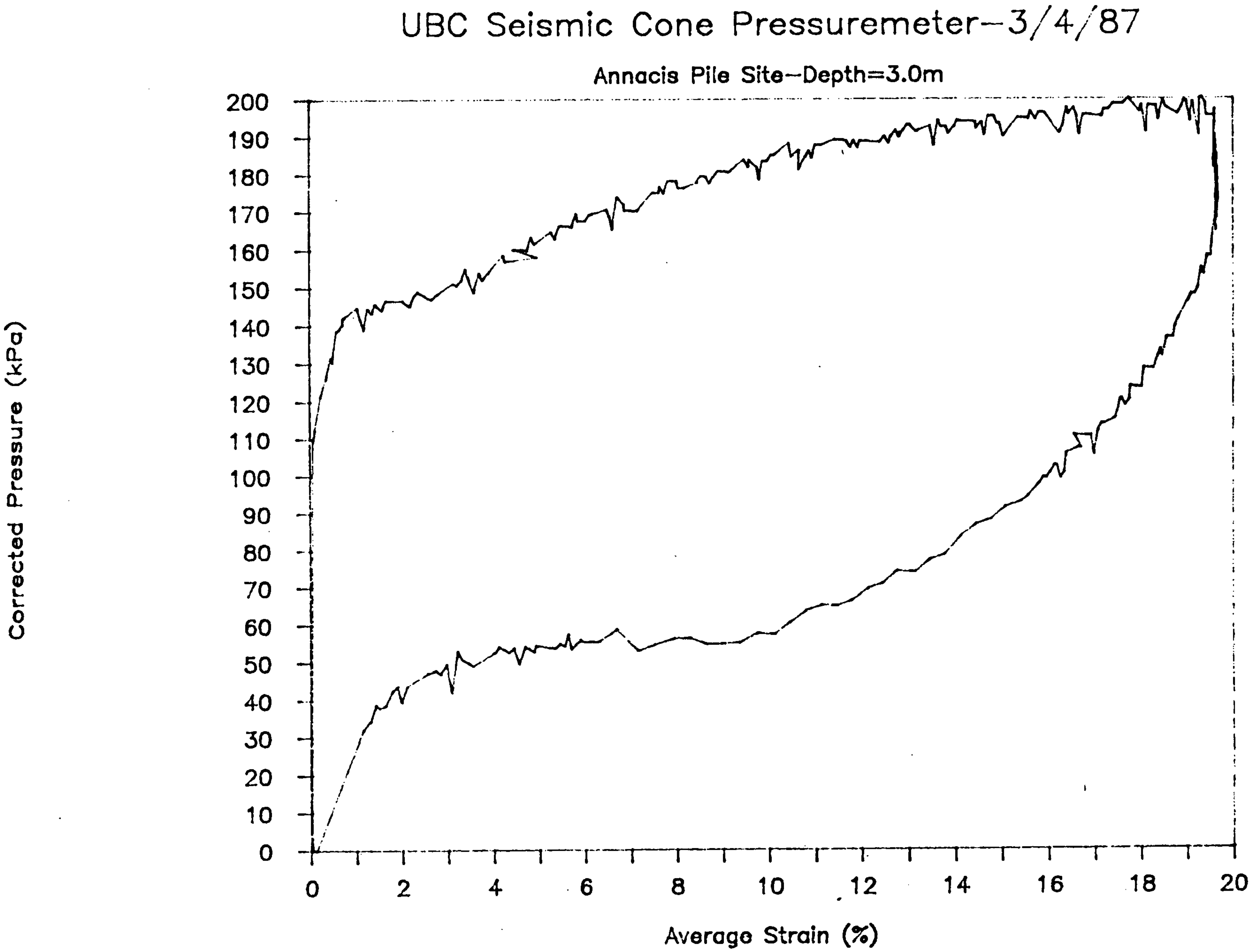


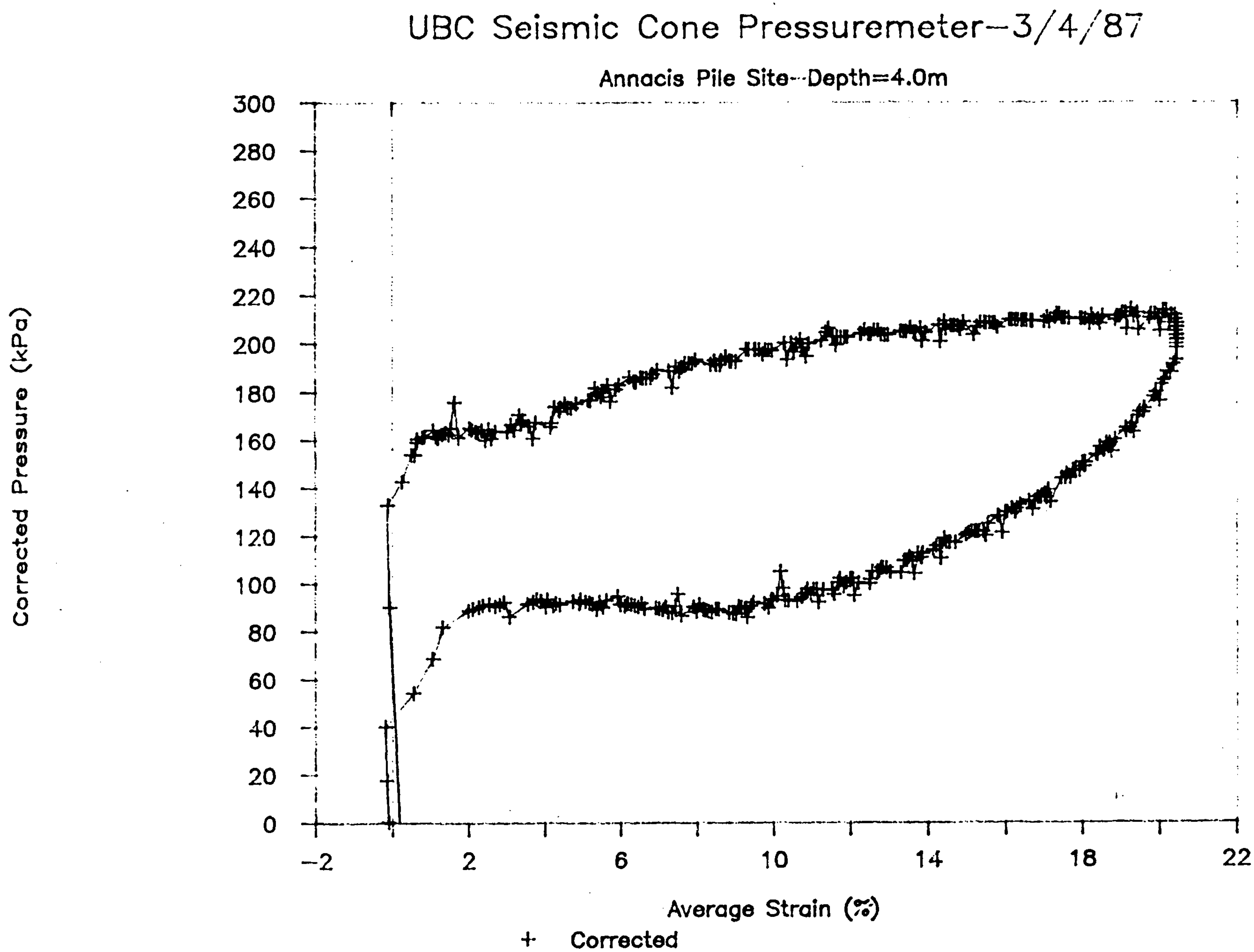
UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=0.17m
Total Pressure (kPa)
200
180
160
140
120
100
80
60
40
20
0
–20
–1
1
3
5
7
9
11
13
15
17
19
Average Strain (%)
Natural Strain


UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=1.0m
Corrected Pressure (kPa)
500
400
300
200
100
0
0
2
4
6
8
10
12
14
16
18
20
Average Strain (%)
Compliance removed

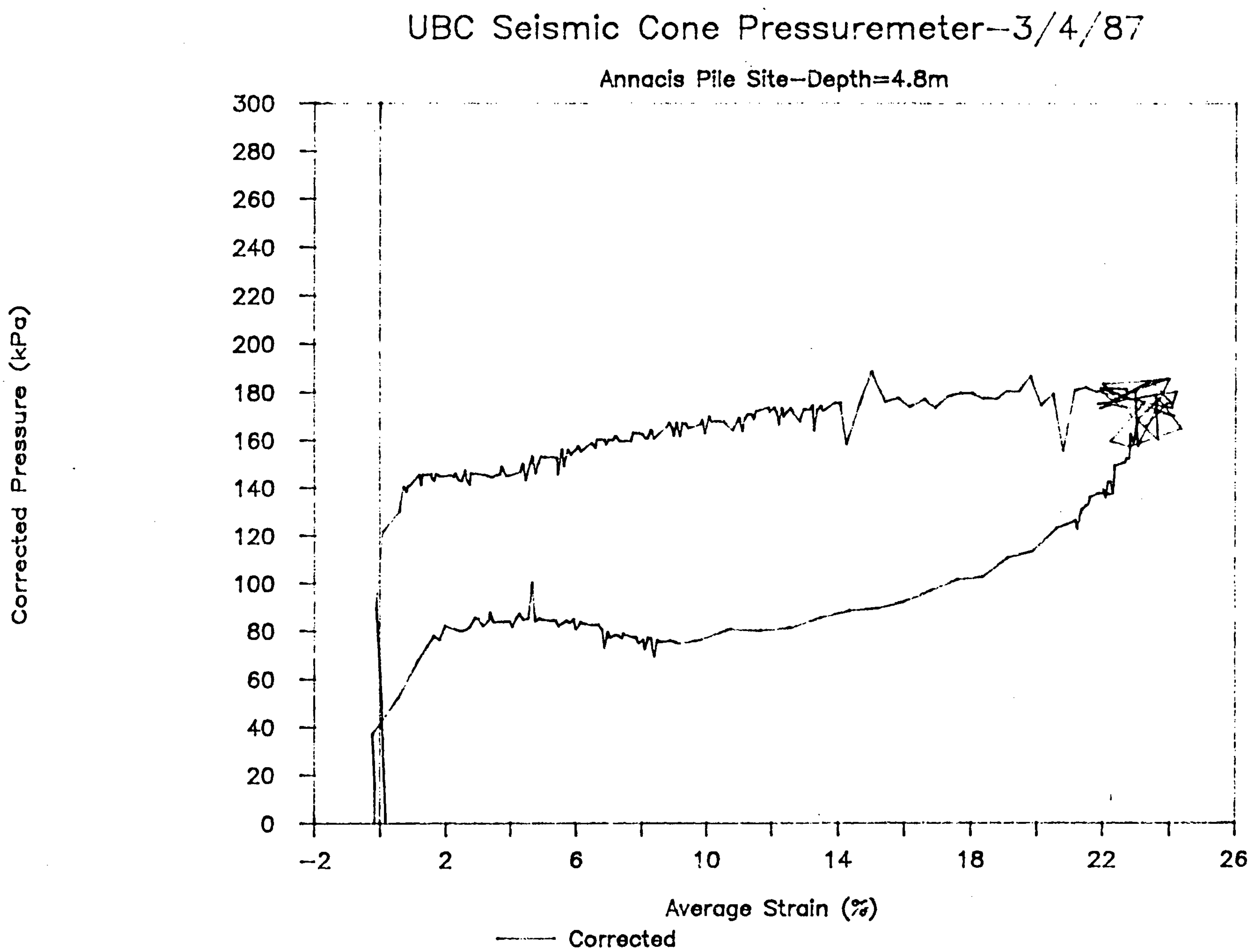


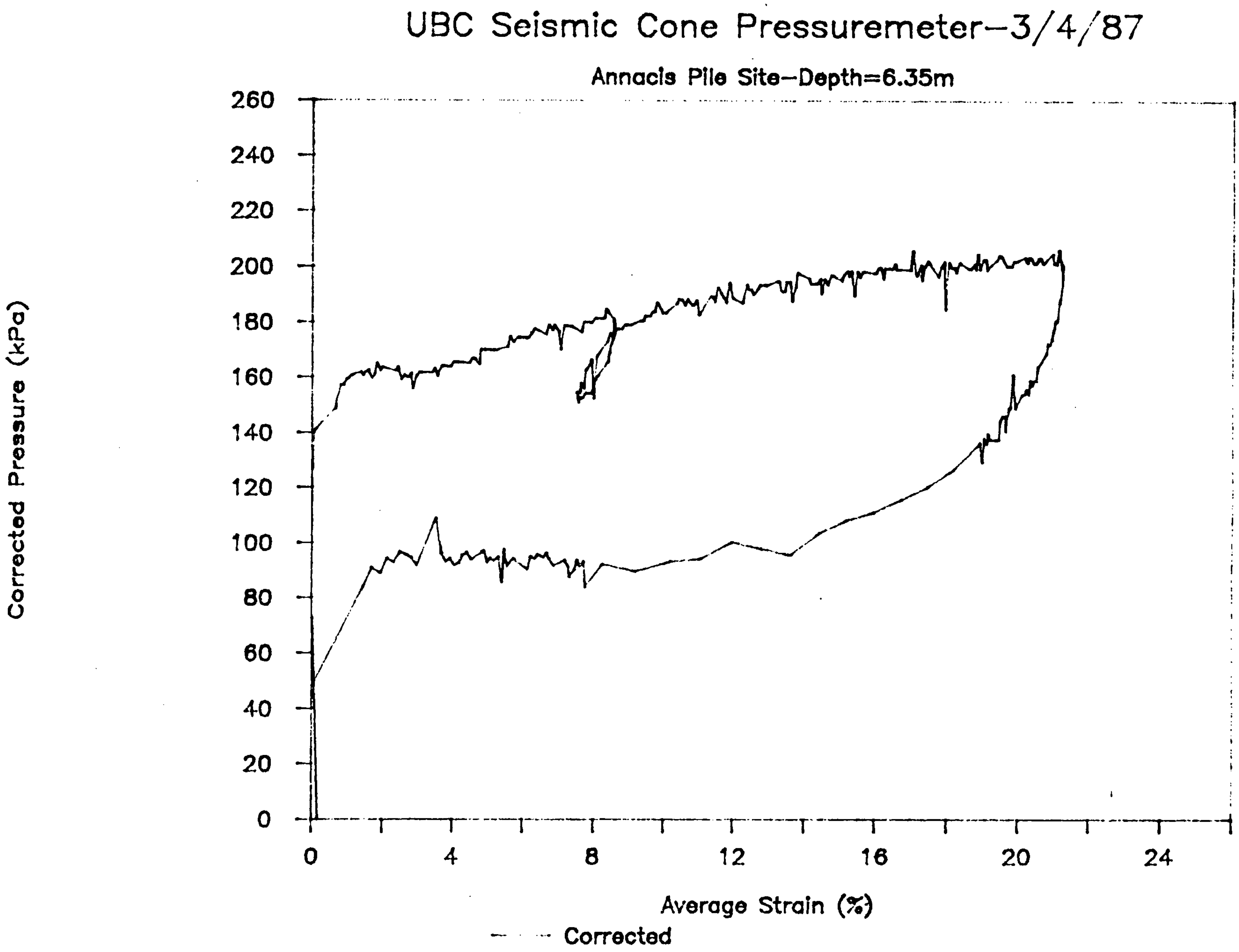
UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=2.0m
Corrected Pressure (kPa)
1000
900
800
700
600
500
400
300
200
100
0
0
2
4
6
8
10
12
14
16
18
20
Average Strain (%)
Compliance removed


UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=3.0m
Corrected Pressure (kPa)
Average Strain (%)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
0
2
4
6
8
10
12
14
16
18
20

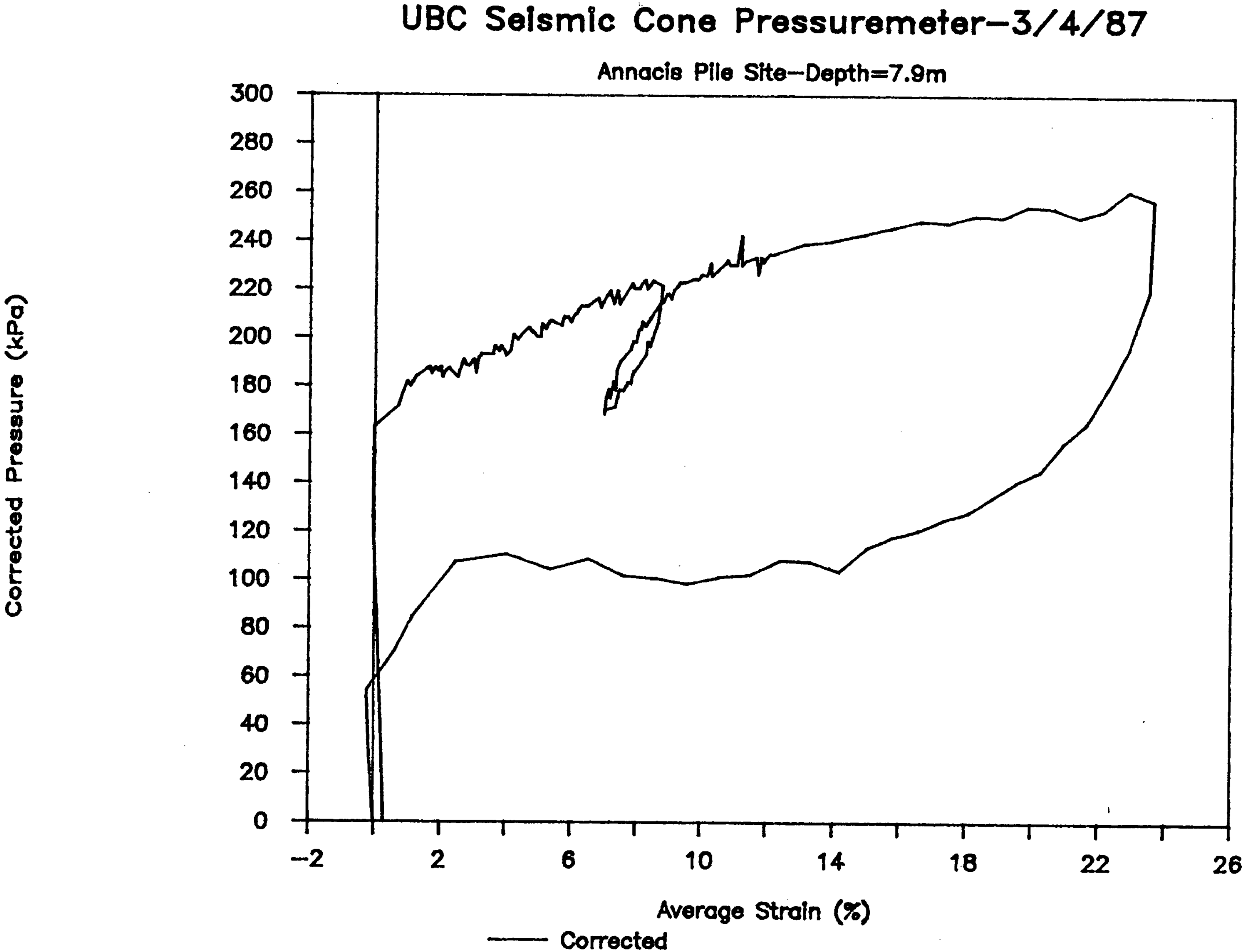


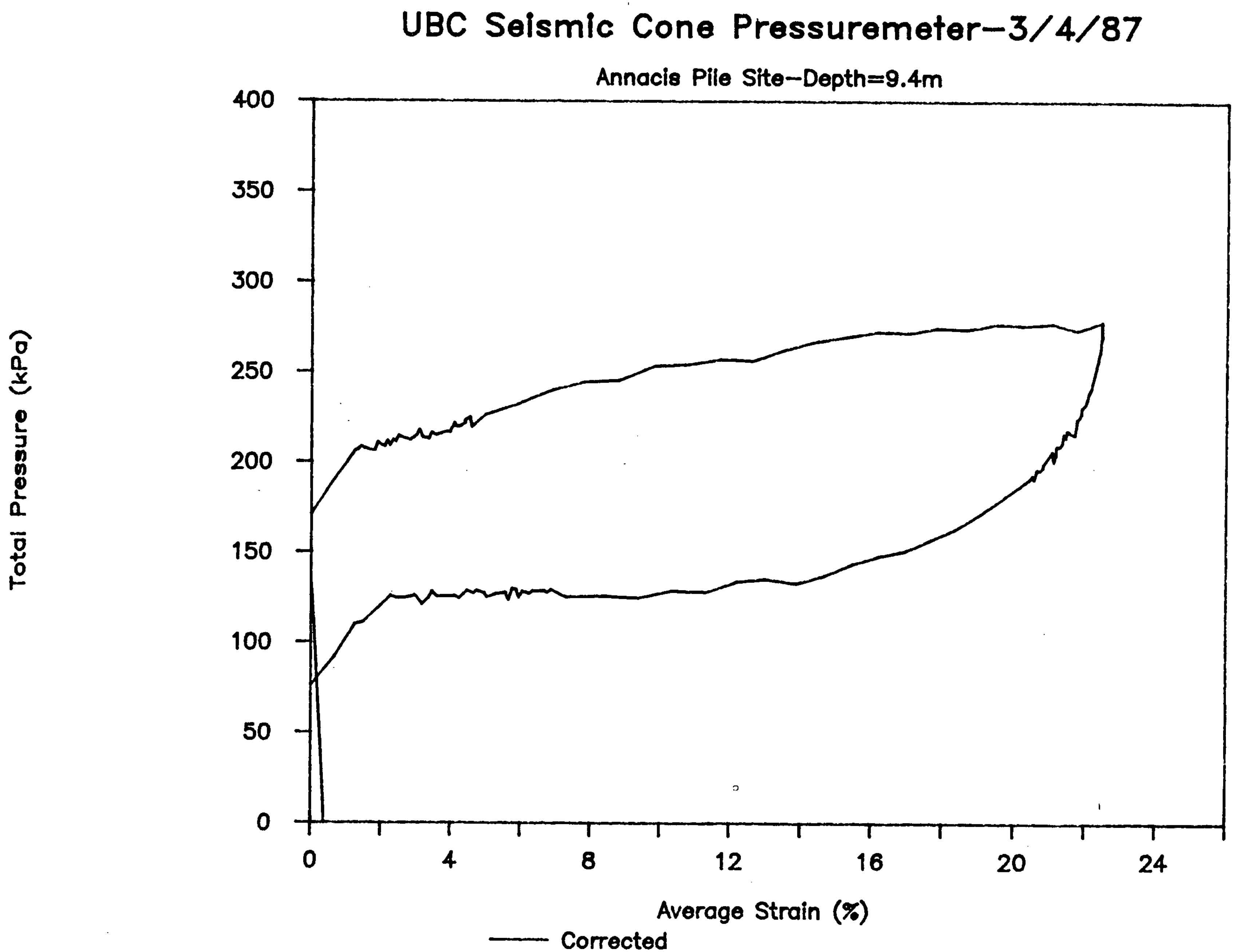
UBC Seismic Cone Pressuremeter—3/4/87
Annacis Pile Site--Depth=4.0m
Corrected Pressure (kPa)
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
−2
2
6
10
14
18
22
Average Strain (%)
+ Corrected

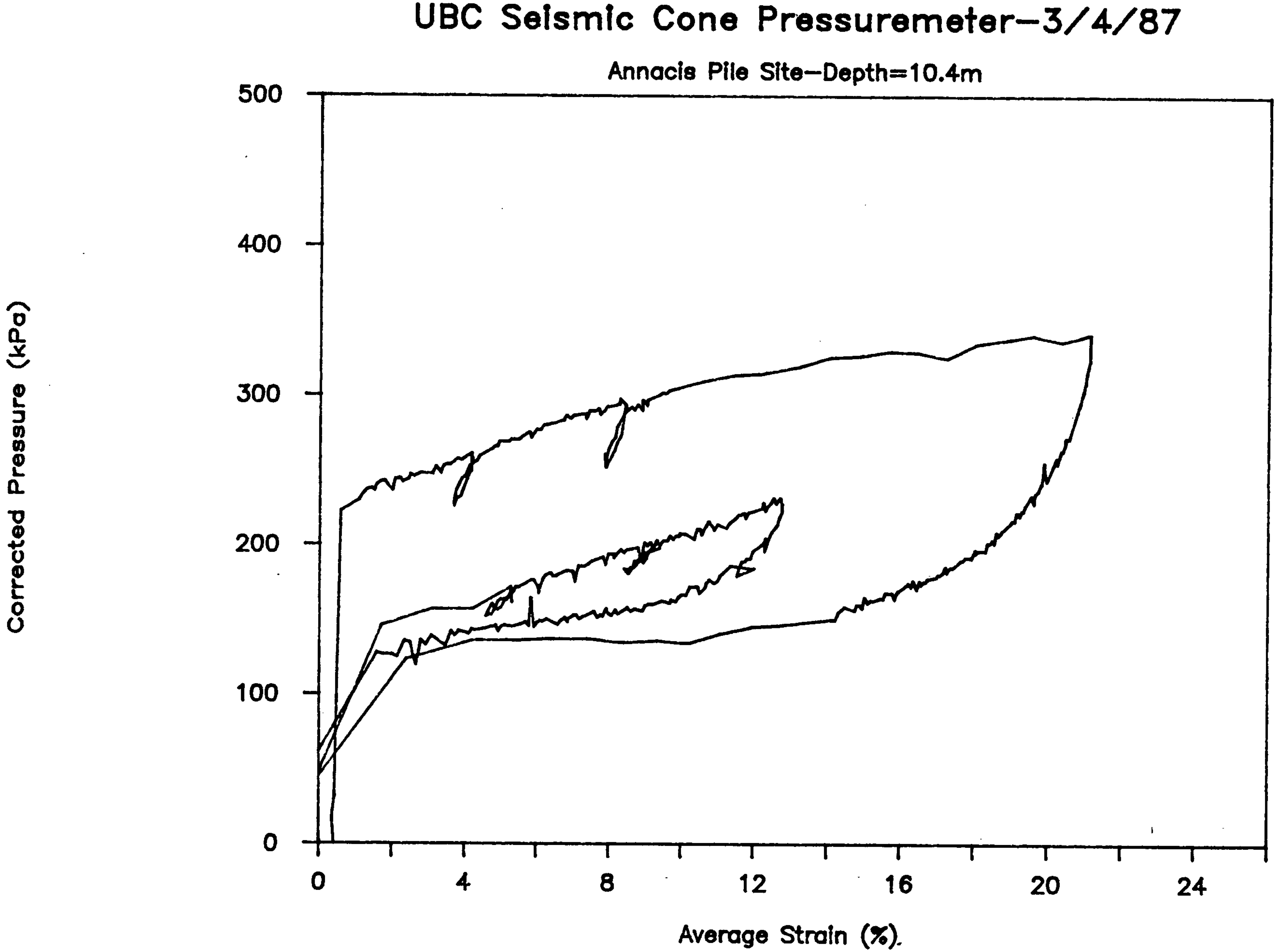

UBC Seismic Cone Pressuremeter—3/4/87
Annacis Pile Site—Depth=4.8m
Corrected Pressure (kPa)
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
−2
2
6
10
14
18
22
26
Average Strain (%)
Corrected



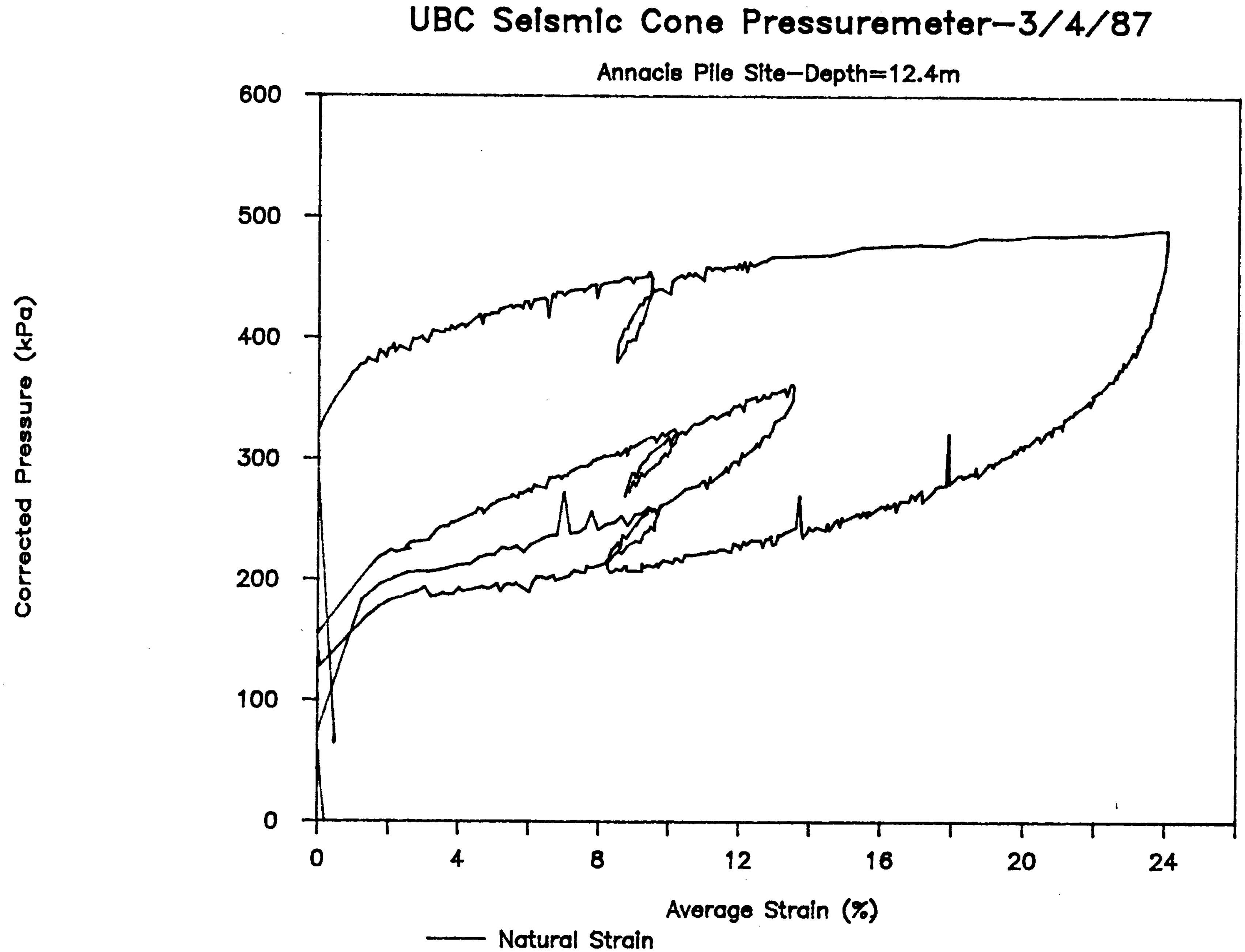
UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=6.35m
Corrected Pressure (kPa)
260
240
220
200
180
160
140
120
100
80
60
40
20
0
0
4
8
12
16
20
24
Average Strain (%)
Corrected

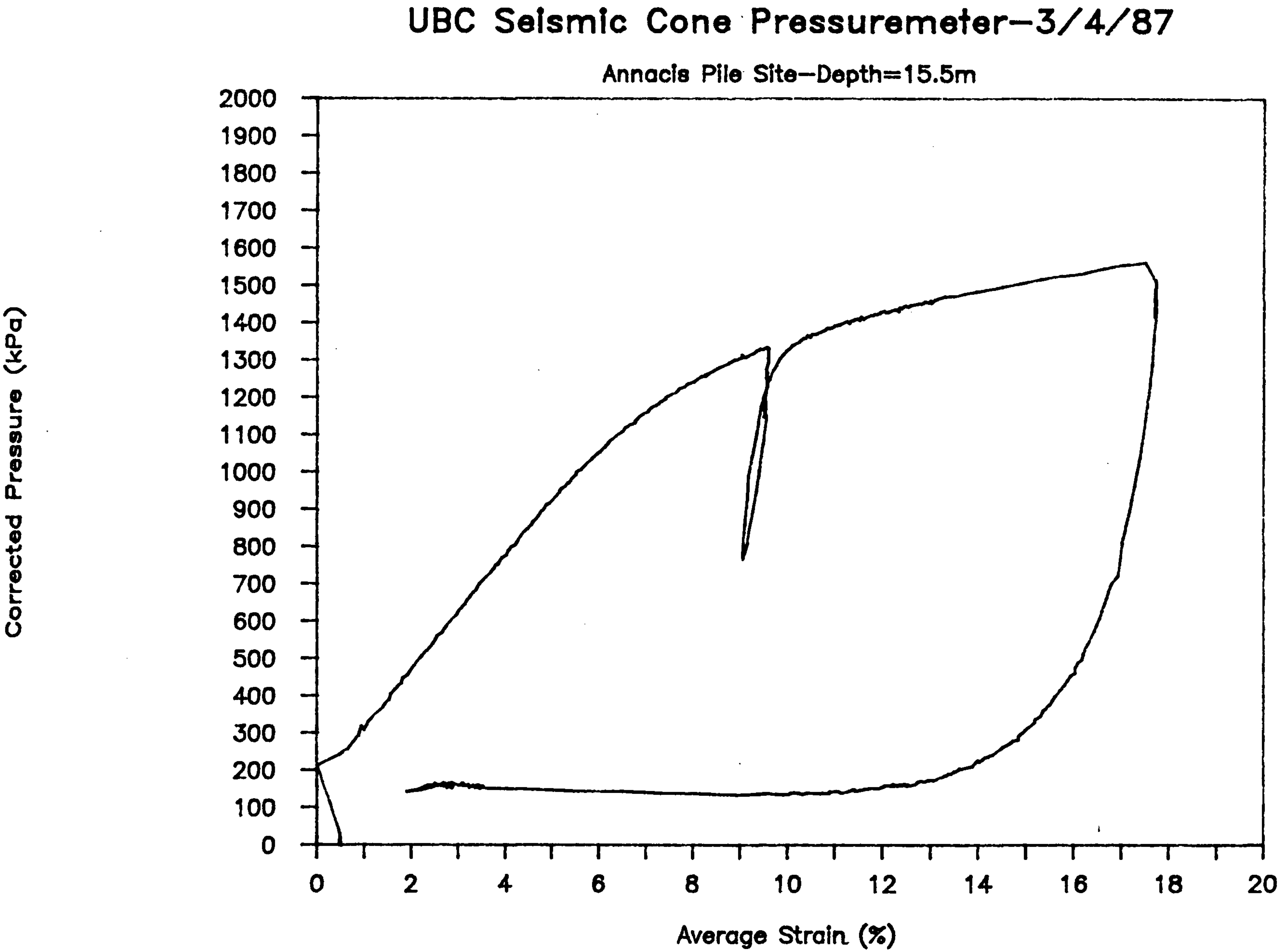

UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=7.9m
Corrected Pressure (kPa)
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
–2
2
6
10
14
18
22
26
Average Strain (%)
Corrected


UBC Seismic Cone Pressuremeter—3/4/87
Annacis Pile Site—Depth=9.4m
Total Pressure (kPa)
400
350
300
250
200
150
100
50
0
0
4
8
12
16
20
24
Average Strain (%)
Corrected



UBC Seismic Cone Pressuremeter—3/4/87
Annacis Pile Site—Depth=10.4m
Corrected Pressure (kPa)
500
400
300
200
100
0
0
4
8
12
16
20
24
Average Strain (%)


UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=12.4m
Corrected Pressure (kPa)
600
500
400
300
200
100
0
0
4
8
12
16
20
24
Average Strain (%)
Natural Strain


UBC Seismic Cone Pressuremeter–3/4/87
Annacis Pile Site–Depth=15.5m
Corrected Pressure (kPa)
2000
1900
1800
1700
1600
1500
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
0
0
2
4
6
8
10
12
14
16
18
20
Average Strain (%)

APPENDIX II

PILE DRIVING RECORDS FOR UBCPRS

PILE PENETRATION DIAGRAM

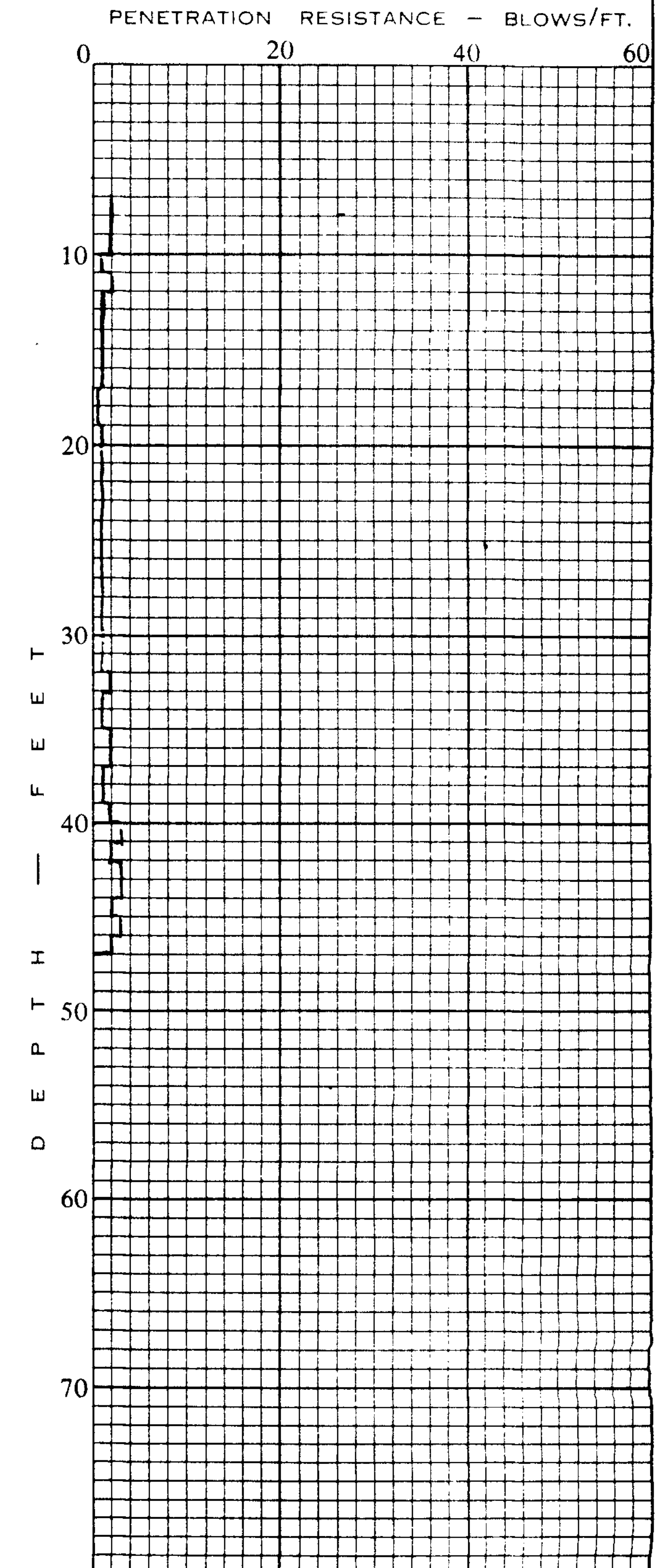
236

DATE 19 AUG 85 TECHNICIAN AS PILE NO. 1

START 7:28 am

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
0-1		21	1	41	3	61	
1-2		22	1	42	2	62	
3		23	1	43	3	63	
4	6' g.l. below	24	1	44	3	64	
5	ref level	25	1	45	2	65	
5- 6		26	1	46	3	66	
7	2/4"	27	1	47	2	67	
8	2	28	1	~~48~~	END OF PILE 1	68	
9	2	29	1	49	8 am	69	
10	2	30	1	50		70	
11	1	31	1	51		71	
12	2	32	1	52		72	
13	1	33	2	53		73	
14	1	34	1	54		74	
15	1	35	1	55		75	
16	1	36	2	56		76	
17	1	37	2	57		77	
18	1	38	1	58		78	
19		39	1	59		79	
20	1	40	2	60		80	

Outer casing went down 1 ft with pile & tapped & pulled back casing. Restart 7:46 am



PILE DATA

ELEVATION GROUND ≈ 1.84 m

TYPE HAMMER DROP

WT. HAMMER 440 lb

HT. DROP 4 ft

TYPE PILE CLOSED ENDED PIPE

DIMENSIONS PILE 12 3/4" OD × 3/8" wall

REMARKS Ref. level ≈ 1'8" above G.L. i.e. Elev. 2.349

THE UNIVERSITY OF BRITISH COLUMBIA

UBC IN-SITU TESTING

JOB No.	TECH. AS
PROJECT	UBC PILE RESEARCH
LOCATION	QUEENSBOROUGH, LULU IS
HOLE No.	PILE 1
DATE	19 AUG 85 PLATE

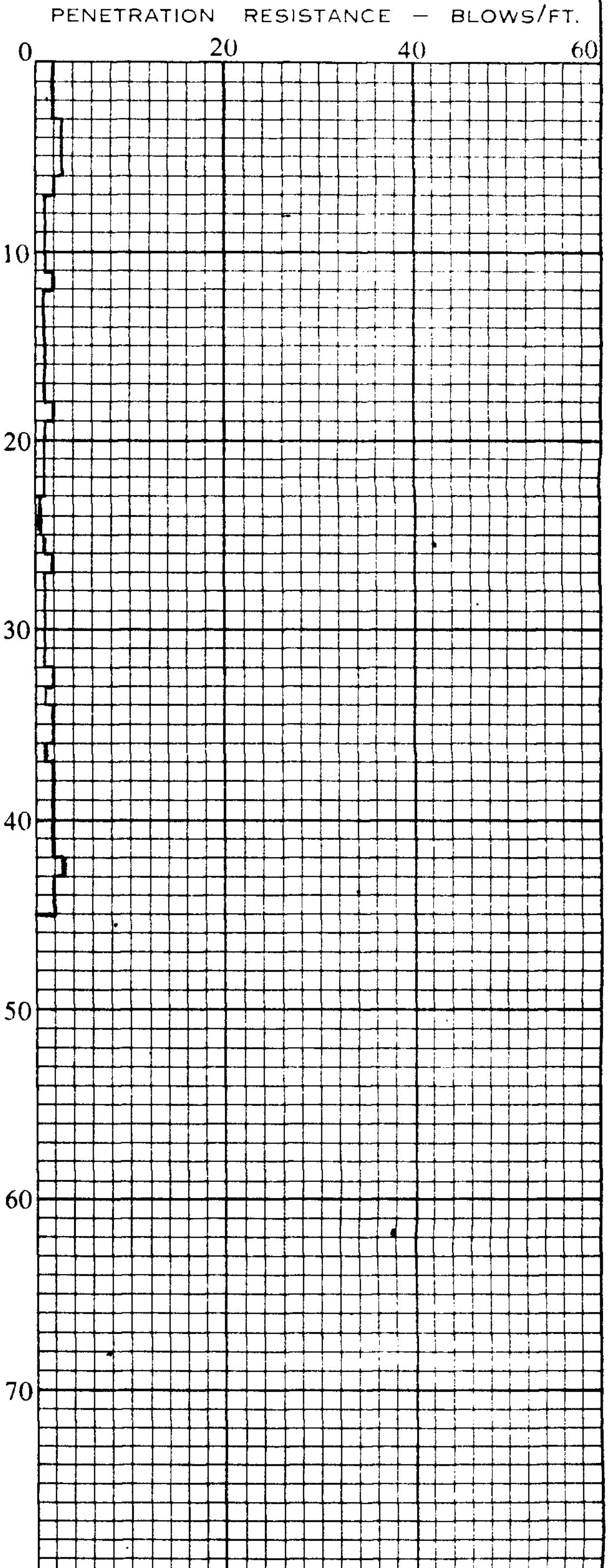
PILE PENETRATION DIAGRAM

237

DATE 16 AUG 85 TECHNICIAN AS PILE NO. 2

START
1.52 pm

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
0-1	2	21	1	41	2	61	
1-2	2	22	1	42	2	62	
3	2	23	1	43	3	63	
4	3	24	[illegible]	44	2	64	
5	3	25	[illegible]	45	2	65	[illegible]
6	3	26	1	~~46~~	END OF PILE	66	67 ... to ...
7	2	27	2	~~47~~	2:00pm 16 Aug	67	
8	1	28	1	~~48~~		68	
9	1	29	1	49		69	
10	1	30	1	50		70	
11	1	31	1	51		71	
12	2	32	1	52		72	
13	1	33	2	53		73	
14	1	34	1	54		74	
15	1	35	2	55		75	
16	2	36	2	56		76	
17	1	37	1	57		77	
18	1	38	2	58		78	
19	2	39	2	59		79	
20	1	40	2	60		80	



PILE DATA

ELEVATION GROUND ≈ 1.83 m

TYPE HAMMER DROP

WT. HAMMER 620 lb

HT. DROP 3

TYPE PILE CLOSE ENDED PIPE PILE

DIMENSIONS PILE 12 3/4" OD × 3/8" wall

REMARKS 3 × 3/4" plywood cushion (old)

THE UNIVERSITY OF BRITISH COLUMBIA

UBC IN-SITU TESTING

JOB No. TECH. AS

PROJECT UBC PILE RESEARCH

LOCATION QUEENSBOROUGH, LULU IS

HOLE No. PILE 2

DATE 16 AUG 85 PLATE

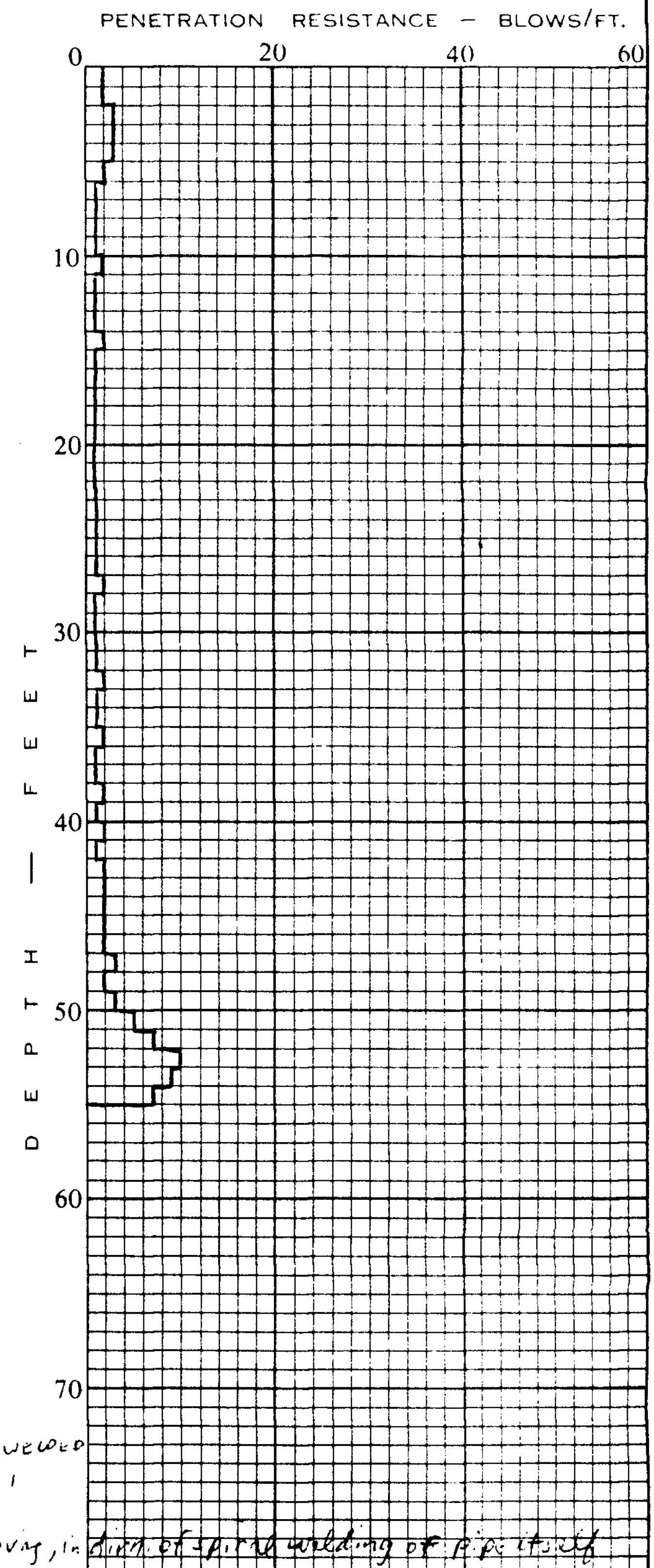
PILE PENETRATION DIAGRAM

238

DATE 16 AUG 85 TECHNICIAN AS PILE NO. 3

START 1.02 pm

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
0-1	2 4' drop	21	1	41	2	61	
1-2	2	22	1	42	1	62	
3	3	23	1	43	2	63	
4	3	24	1	44	2	64	
5	3	25	1	45	2	65	
6	2	26	1	46	2	66	
7	1	27	1	47	2	67	
8	1	28	2	48	3	68	
9	1	29	1	49	2	69	
10	1	30	1	50	3	70	
11	2	31	1	51	5	71	
12	1	32	1	52	7	72	
13	1	33	2	53	10	73	
14	1	34	1	54	9	74	
15	2	35	1	55	7	75	
16	1	36	2	~~56~~	END OF PILE	76	
17	1	37	1	~~57~~	1.14 pm 16 Aug	77	
18	1	38	1	58		78	
19	1	39	2	59		79	
20	1	40	1	60		80	



PILE DATA

ELEVATION GROUND ≃ 1.88 m

TYPE HAMMER DROP HAMMER

WT. HAMMER 620 lb

HT. DROP ~~3~~ 4 ft

TYPE PILE CLOSE ENDED PIPE PILE - spiral welded

DIMENSIONS PILE 12 3/4" OD x 3/8" wall x 60'

REMARKS Pile rotated ≃ 120° clockwise during driving, in dirn of spiral welding of pipe itself

THE UNIVERSITY OF BRITISH COLUMBIA

UBC IN-SITU TESTING

JOB No.	TECH. AS
PROJECT	UBC PILE RESEARCH
LOCATION	QUEENSBOROUGH, LULU IS.
HOLE No.	PILE 3
DATE	16 AUG 85 PLATE

PILE PENETRATION DIAGRAM

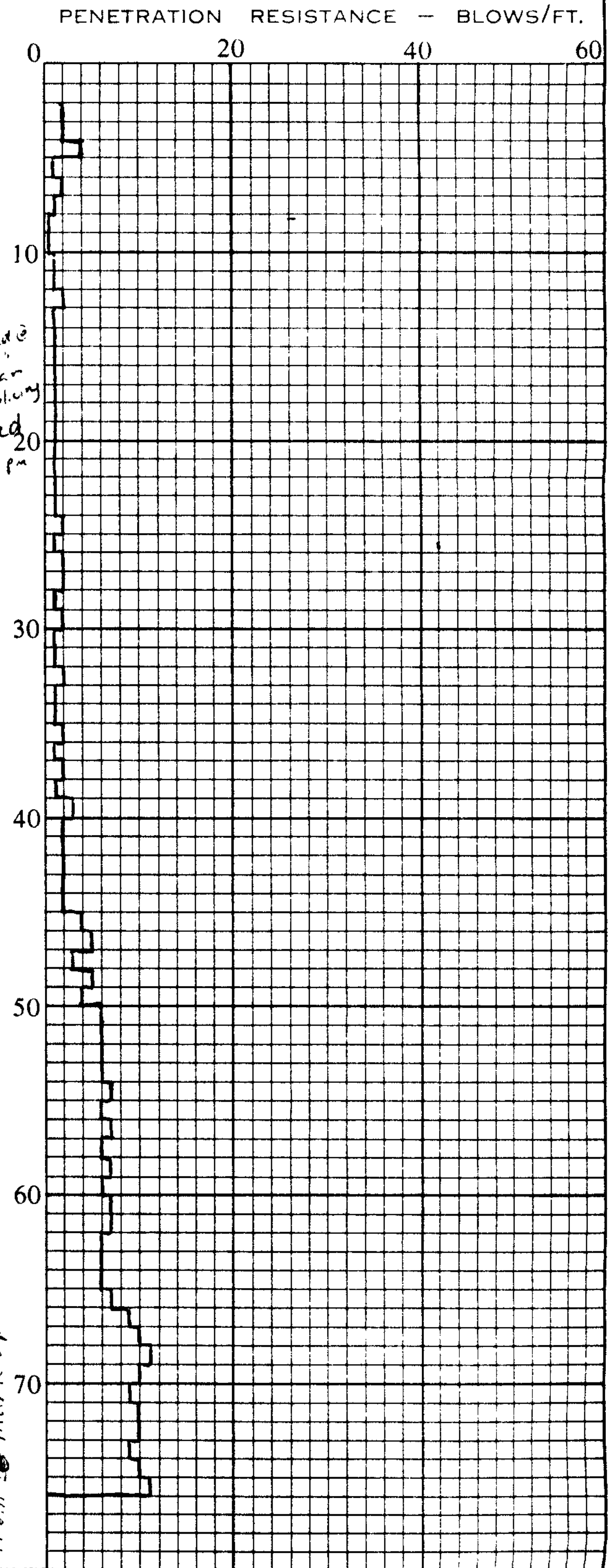
DATE 16 AUG 85 TECHNICIAN AS PILE NO. 4

START 0925

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
0-1	2 2' drop	21	1	41	2	61	7
1-2	3 1 ft	22	1	42	2	62	7
3	2 5'	23	1	43	2	63	6
4	2	24	1	44	2	64	6
5	2/6" 2/6"	25	2	45	2	65	6
6	1	26	1	46	2/8" 2/4"	66	7
7	2	27	2	47	5	67	9
8	1	28	2	48	3 stopped to weld 3/4"Ø pipe	68	10
9	1	29	1	49	5	69	11
10		30	2	50	4	70	10
11	1	31	1	51	6	71	9
12	1	32	1	52	6	72	10
13	2	33	2	53	6	73	10
14	1	34	1	54	6	74	9
15	1	35	1	55	7	75	10
16	1	36	2	56	6	76	11
17	1	37	1	57	7	77	END OF PILE
18	1	38	2	58	6	78	12:26 pm 16 Aug 85
19	1	39	1	59	7	79	
20	1	40	3	60	6	80	

stopped @ 45'8" 9:36 am for splicing
resumed 11:57 pm

1st pile length = 50'2" with 3½"Ø pipe welded from tip up & 3/4"Ø pipe from 20' mark up



PILE DATA

ELEVATION GROUND ≈ 1.79 m

TYPE HAMMER DROP HAMMER

WT. HAMMER 620 lb

HT. DROP 5 ft

TYPE PILE OPEN ENDED PIPE PILE

DIMENSIONS PILE 12 3/4" O.D. × 3/8" wall

REMARKS 2 new × 3/4" plywood + 1 old 3/4" plywood

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UBC IN-SITU TESTING

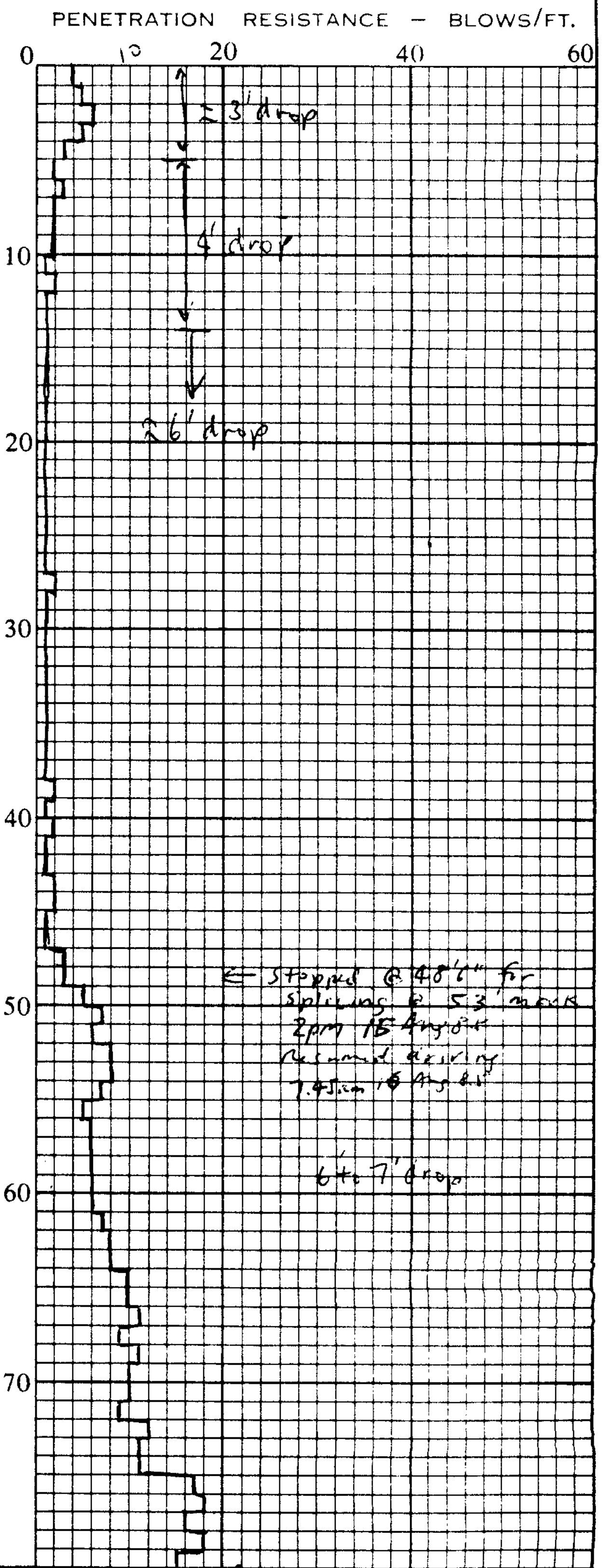
JOB No.	TECH. AS
PROJECT	UBC PILE RESEARCH
LOCATION	QUEENSBOROUGH, LULU IS.
HOLE No.	PILE 4
DATE	16 AUG 85 PLATE

PILE PENETRATION DIAGRAM

DATE 15 AUG 85 TECHNICIAN AS PILE NO. 5 sheet 1

START 1330 15 AUG 85

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
0-1	4 3' drop	21	1	41	2	61	6
1-2	5	22	1	42	1	62	7
3	6 1½' drop	23	1	43	1	63	8
4	5 ×3' drop	24	1	44	2	64	8
5	3 ×3' drop	25	1	45	2	65	10
6	2 4' drop	26	1	46	1	66	10
7	3	27	1	47	1	67	11
8	2	28	2	48	1/6" 2/6" change = 15 mins	68	9
9	2	29	1	15 Aug 49 16 Aug	1/6" 2/6" splicing	69	6/6" 5/6"
10	2	30	1	50	5	70	10
11	1	31	1	51	7	71	10
12	2	32	1	52	6	72	9
13	1	33	1	53	8	73	12
14	2	34	1	54	8	74	11
15	1 6' drop	35	1	55	7	75	11
16	1	36	1	56	5	76	17
17	1	37	1	57	6	77	18
18	1	38	1	58	6	78	16
19	1	39	2	59	6	79	5/3" 13/9"
20	1	40	1	60	6	80	15



PILE DATA

ELEVATION GROUND ≈ 1.84 m

TYPE HAMMER DROP

WT. HAMMER 6200 lb

HT. DROP up to 7 ft

TYPE PILE CLOSED-ENDED PIPE (Seamless)

DIMENSIONS PILE 12 ¾" O.D. × ½" wall

REMARKS Crane operator has tendency to go a bit higher than specified

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UBC IN-SITU TESTING

JOB No. TECH. AJ

PROJECT UBC PILE RESEARCH

LOCATION QUEENSBOROUGH, LULU I.

HOLE No. PILE 5

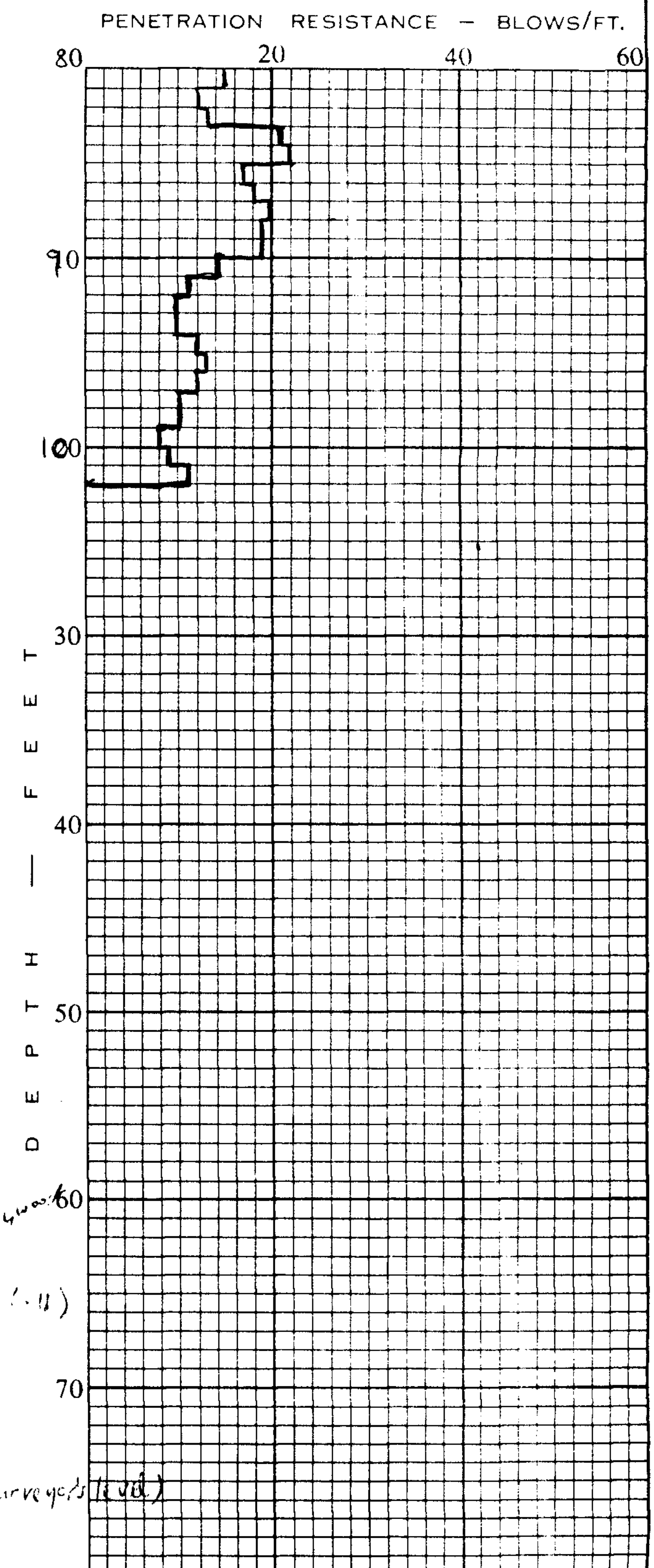
DATE 15-16 AUG 85 PLATE 1 of 2

PILE PENETRATION DIAGRAM

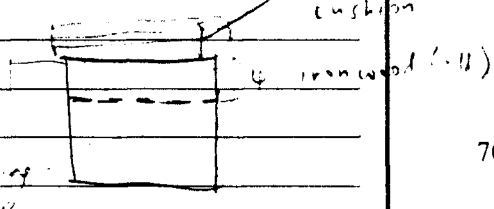
DATE 16 AUG 85 TECHNICIAN AS PILE NO. 5 Sheet 2

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
81	15	101	9	41		61	
82	12	102	11	42		62	
83	13	~~23~~	END OF PILE 5	43		63	
84	21	~~24~~	8:40 am 16 Aug	44		64	
85	22	~~25~~		45		65	
86	17	26		46		66	
87	18	27		47		67	
88	20	28		48		68	
89	19	29		49		69	
90	19	30		50		70	
91	14	31		51		71	
92	11	32		52		72	
93	10	33		53		73	
94	10	34		54		74	
95	12	35		55		75	
96	13	36		56		76	
97	12	37		57		77	
98	10	38		58		78	
99	10	39		59		79	
100	8	40		60		80	

CRANE OPERATOR WAS TOLD TO DROP 6 FT BUT HE HAD TENDENCY TO DROP 1 FT MORE.



PILE DATA



ELEVATION GROUND

TYPE HAMMER

WT. HAMMER

HT. DROP

TYPE PILE

DIMENSIONS PILE TOP OF PILE 5 MONITORED (surveyor's level) DURING DRIVING OF ADJACENT PILES BUT DID NOT MOVE.

REMARKS

THE UNIVERSITY OF BRITISH COLUMBIA

UBC IN-SITU TESTING

JOB No.	TECH. AS
PROJECT	UBC PILE RESEARCH
LOCATION	QUEENSBOROUGH, LULU IS
HOLE No.	PILE 5
DATE 15-16 AUG 85	PLATE 2 of 2

PILE PENETRATION DIAGRAM

DATE 14 AUG 85 TECHNICIAN DAMIKA, TO PILE NO. E Sheet 1

START PILING 11:45 hrs

2 x 3/4" plywood cushion (new) 5 1/4" used at start

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
0-1	5 ft drop	21	2	41	2	61	22
1-2	3	22	2	42	4	62	19
3	3	23 (break 10 mts)	1 for 8" / 1 for 4"	43	4	63	20
4	2	24	5	44	4	64	17
5	4	25	3/4" / 2/8"	45	4	65	15
6	2	26	2	46	4	66	20
7	2	27	3	47	4	67	30
8	2	28	3	48 (wait 10 mts)	3/6" / 2/6"	68	38
9	2	29	2	49	5	69	41
10	2	30	2	50	4	70	36
11	2	31	3	51	7	71	28
12	2	32	2	52	15	72	26
13	~~1~~ 1	33	3	53	15	73	24
14	2	34	2	54 (stopped at 12:35 for splicing)	13/8" / 7/4"	74	21
15	3	35	3	55	24	75	16-8" / 7-4"
16	2	36	3	56	22	76	31
17	2	37	3	57 (BREAK 5 min at 56'8")	20	77	29
18	2	38	4	58	20	78	31
19	1/6" / 1/6"	39	3	59	19	79	31
20	3	40	3	60	21	80	29

5 ft drop; 10 ft drop; 10 ft drop; yielding of pile top observed; 8 ft drop; 6 ft drop

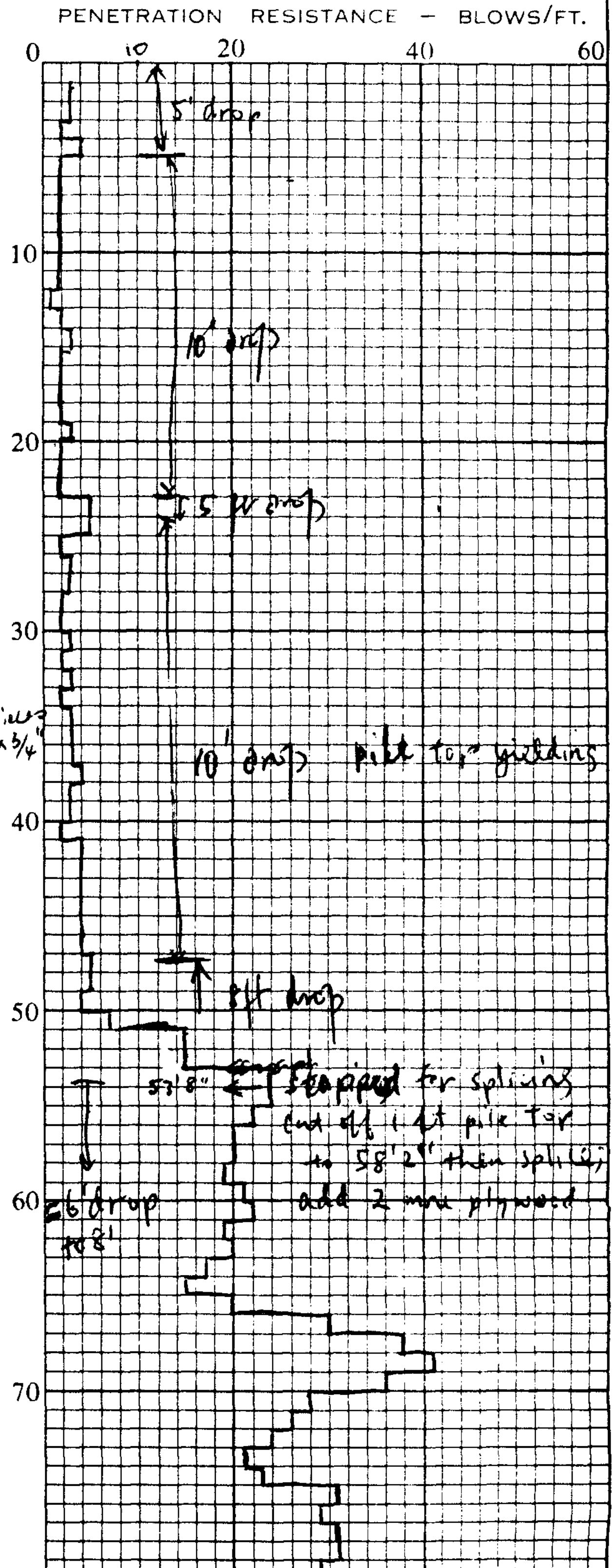
stopped for 10 mts

15 mins break

plywood added

3 pieces 1 x 3/4"

START 9:25 A.M. 08/15/85



PILE DATA

ELEVATION GROUND ≈ 1.83 m

TYPE HAMMER DROP

WT. HAMMER 6200 lb

HT. DROP 10 ft max.

TYPE PILE 24" ϕ x 1/2" wall closed

DIMENSIONS PILE ended pipe pile

REMARKS

THE UNIVERSITY OF BRITISH COLUMBIA

UBC IN-SITU TESTING

JOB No. TECH. DW, TO

PROJECT UBC PILE RESEARCH

LOCATION QUEENSBOROUGH, LULU IS

HOLE No. PILE 6

DATE 14-15 AUG 85 PLATE 1 of 2

PILE PENETRATION DIAGRAM

DATE 15 AUG 85 TECHNICIAN TO PILE NO. 6 sheet 2

DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS	DEPTH	NO. BLOWS
80-81	24	101	23	41		61	
81-82	24	102	25	42		62	
83	23	103	25	43	STOP @ 10 55	63	
84	24	24	END OF PILE 6	44		64	
85	31	25	10.55 am 15 Aug	45		65	
86	—	26		46		66	
87	—	27		47		67	
88	39	28		48		68	
89	43	29		49		69	
90	37	30		50		70	
91	28	31		51		71	
92	27	32		52		72	
93	23	33		53		73	
94	26	34		54		74	
95	26	35		55		75	
96	25	36		56		76	
97	31	37		57		77	
98	31	38		58		78	
99	29	39		59		79	
100	27	40		60		80	

PILE DATA

ELEVATION GROUND

TYPE HAMMER END OF DRIVING

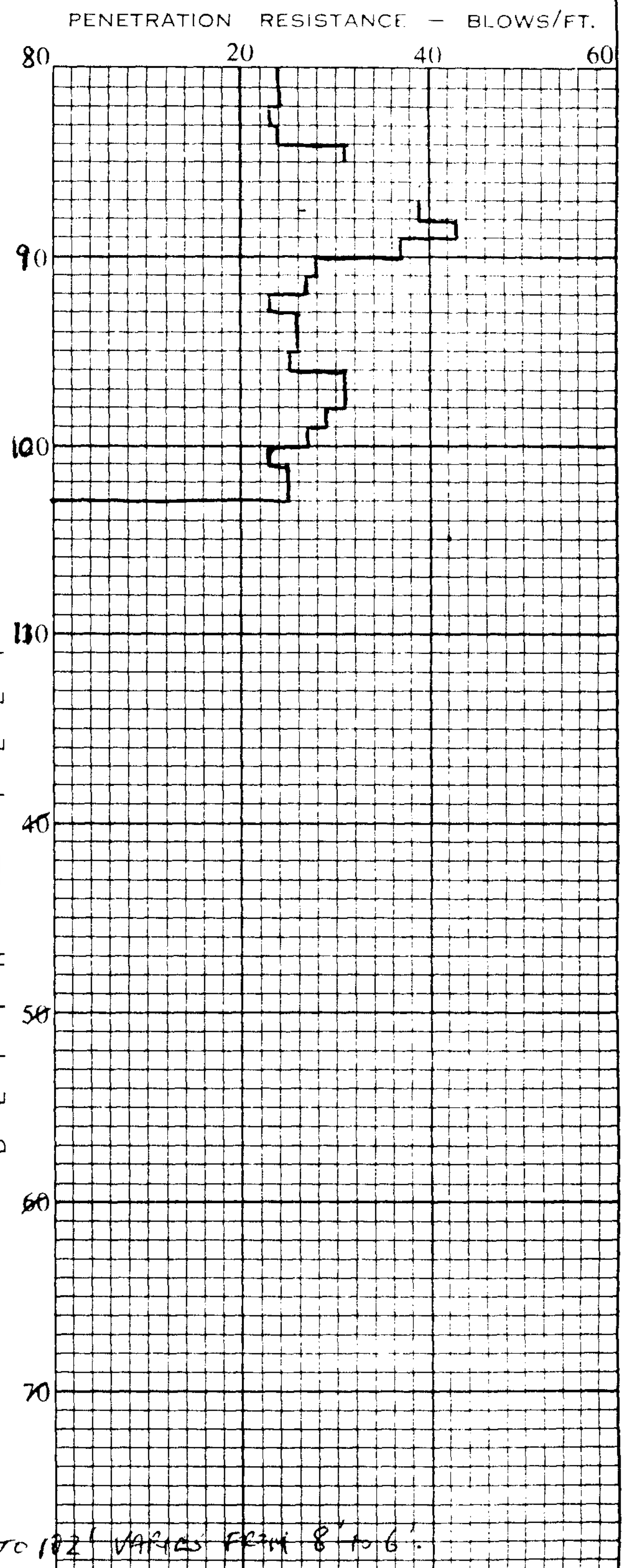
WT. HAMMER @ 103 ft

HT. DROP

TYPE PILE Cut off @ 106' 4"

DIMENSIONS PILE i.e. 1m above G.L

REMARKS HEIGHT OF HAMMER DROP FROM 55.7' TO 102' VARIES FROM 8' to 6'.



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UBC IN-SITU TESTING

JOB No. TECH. DW, TO

PROJECT UBC PILE RESEARCH

LOCATION QUEENSBOROUGH, LULU IS.

HOLE No. PILE 6

DATE 14-15 AUG 85 PLATE 2 of 2

MD [illegible]

SAXIMETER NO. ______ BLOW COUNT/STROKE PILE DRIVING RECORD 244

PILE NO. 7 [illegible] DRIVING ORDER NO. ______ DATE [illegible]

PROJECT MDTH PDA DEMO PROJECT LOCATION [illegible]

PILE TYPE/SIZE [illegible] LENGTH ~100' BATTER 0°

ELEVATION: GROUND ~2m PILE TIP ______ CUTOFF ______

HAMMER TYPE/SIZE [illegible] THROTTLE SETTING ______

CAP/HELMET/CUSHION [illegible]

CONTRACTOR [illegible] FOREMAN GEORGE OBSERVER [illegible]

Depth ft	Blows foot	Stroke	Depth ft	Blows foot	Stroke	Depth ft	Blows foot	Stroke	Depth ft	Blows foot	Stroke
0-1	10	[illegible]	25-26	5		50-51	3 4	~8'	75-76	14	~10'
1-2	7	~8'	26-27	5		51-52	25	~8'	76-77	14	
2-3	7		27-28	5		52-53	25		77-78	12	
3-4	7		28-29	5		53-54	20 15		78-79	15	8
4-5	7		29-30	5		54-55	42	~5'	79-80	13	
5-6	6	~8	30-31	6	~8'	55-56	23 20	~4-5' *3	80-81	12	
6-7	5		31-32	5		56-57	46	~5'	81-82	12	
7-8	3		32-33	6		57-58	40	~4-6'	82-83	[illegible]	
8-9	2		33-34	5		58-59	31	~5-6'	83-84	[illegible]	
9-10	2		34-35	6		59-60	27	~10'	84-85	32	
10-11	2	~9'	35-36	7		60-61	23		85-86	34	*5
11-12	3		36-37	3 1	~16'	61-62	27	~10'	86-87	43	
12-13	2		37-38	4	~16'	62-63	27	~12'	87-88	[illegible]	
13-14	2		38-39	2 4	~16 *2 ~5'	63-64	33	~11-12'	88-89	57	
14-15	3		39-40	7	~8'	64-65	38	~10'	89-90	53	
15-16	3		40-41	7		65-66	17 9 *4	~9-10' ~10'	90-91	50	
16-17	2		41-42	6		66-67	11	~10'	91-92	38	
17-18	4	~9'	42-43	9		67-68	11	~12'	92-93	40	
18-19	3		43-44	7	~9'	68-69	12	~11	93-94	39	
19-20	3		44-45	7		69-70	8	~12'	94-95		
20-21	4		45-46	8		70-71	9		95-96		
21-22	3		46-47	8		71-72	10		96-97		
22-23	2 3	~15 3.5 *1	47-48	7	~8'	72-73	9	~10'	97-98		
23-24	4		48-49	10		73-74	10		98-99		
24-25	5	~9'	49-~~50~~ 49.7	3 5	~4.5'	74-75	13		99-100		
			49.7-50	4	~2'						

REMARKS: *1) [illegible]

*2) [illegible] 6 [illegible]

*3) E LOSS ~ 1.5 [illegible]

*4) [illegible] = 5000 lb

*5) NEW HAMMER = 3500 lb

TIME OF START [illegible] PM STOP ______

DEPTH	MIN.	INTERRUPTION REASON
		[illegible]

APPENDIX III

AXIAL PILE LOAD TESTS FOR UBCPRS

UBCPRS : ELASTIC COMPRESSION CALCULATIONS

PILE NO. 1

O.D = 0.32385 metres   I.D. = 0.3048 metres

Length = 14.326 metres   Elastic Modulus, $E = 2.065 \times 10^{8}$ kPa

$$\text{Delta} = \frac{P\ L}{A\ E}$$

P = applied axial load during load testing
A = cross-sectional area of pile
L = pile length
E = elastic modulus of pile material

$$\text{Delta} = \frac{P\ (14.326 \text{ metres})\ (1000 \text{ millimetres/metres})}{(0.32385^2 - 0.3048^2)\ PI/4\ (2.0565 \times 10^{8} \text{ KN/ metres squared})}$$

Delta (mm.) = P(kN) (7.4063×10^{-3} mm./kN)

similar calculations for piles 2 to 5

PILE NO. 2

Delta (mm.) = P(kN) (7.0910×10^{-3} mm./kN)

PILE NO. 3

Delta (mm.) = P(kN) (8.6667×10^{-3} mm./kN)

PILE NO. 4

Delta (mm.) = P(kN) (1.1976×10^{-2} mm./kN)

PILE NO. 5

Delta (mm.) = P(kN) (1.3862×10^{-2} mm./kN)

UBC PILE RESEARCH PROJECT
HYDRAULIC JACK / LOAD CELL CALIBRATIONS

Calibration Date : 13th September 1985
Calibrated By : Alex Sy
Test Machine : Baldwin 400,000 lb cap.

Hydraulic Jack : 300 ton capacity Rogers Jack
S/N C1300A13; Unit# 8-066
Closed Height 30 ins
Base Diameter 12.125 ins
Weight 590 lbs
(Franki Canada Limited)

Hydraulic Pump : Enerpac Model P462
Unit# 8-035
(Franki Canada Limited)

Pressure Gauge : 10,000 psi capacity Enerpac
Glycerine filled
(Franki Canada Limited)

Load Cell : 500,000 lb cap. BLH Electronics
Type C2P1S; S/N 36881
Diam 10 ins; Height 14 ins
(UBC Structural Engin.)

Strain Readout : Budd Instruments
Datran Digital Strain Indicator
(UBC Structural Engin.)

Pres.Gauge (psi)	Load Cell Budd Rdg	Actual Load (lbs)	Load Cell Budd Rdg	Actual Load (lbs)
RUN 1 Minimal ram extension before testing in Baldwin machine				
0	0	0	2	500
500	60	32,500	69	34,500
1000	134	71,500	144	72,500
1500	212	110,500	220	111,000
2000	293	149,000	299	151,000
2500	370	189,750	378	191,000
3000	443	228,000	445	226,500
3500	522	266,500	528	268,000
4000	599	305,000	608	308,500
4500	680	346,500		
5000	753	384,500		
RUN 2 Ram extended 1" prior to jacking against Baldwin machine				
0	0	0	1	0
500	60	30,500	72	37,500
1000	135	69,250	145	76,000
1500	214	110,500	222	114,500
2000	293	150,000	297	154,750
2500	372	190,250	384	195,000
3000	447	229,000	454	233,500
3500	525	268,500	538	273,000
4000	605	309,000	617	314,500
4500	692	351,250	697	355,000
5000	762	388,000		

Pile #1

DATA SHEET

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TEST PILE #1

LOAD CELL #1	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS Right 1 (mm)	Left 2 (mm)		PILE TIP (mm)	LEVEL READINGS PILE	PILE SETTLE.	REMARKS Ref. [illegible]
0	0	11:30		18.13	6.09		4.19			[illegible]
31 k	= 968 lb			18.00	5.67		4.27	275.2	cm -	78.4 cm 77.8 cm
					AP 2	= 27.1 cm	AP 5 = 80.3 cm	AP 4 = [illegible]	[illegible] cm	TS [illegible] = 28.1 cm
145 k		11.47	0	17.00	4.23		4.34	275.3		
146 k			1	16.99	4.22		4.78			
131 k			2½	16.98	4.21		4.78	275.2		
136 k	(31 [illegible])		5	16.98	4.21		4.75			
	42575 lb		10					275.2		78.4 77.8
276 k	(8625)				AP 2	= 27.1	AP5 = 80.3		AP4 = 25.3	
		12.02	0	15.88	3.31		4.38	275.3		
265 k	(8281)		1	15.85	3.30		4.25			
271 k			2½	15.85	3.29		4.25			
271 k			5	15.84	3.275		4.25			
276 k			10	15.81	3.27		4.25			78.4 77.8
					AP2 =	27.1	AP5 = 80.4		AP4 = 25.2	
426 k		12.15	0	15.05	2.71		4.64			
423	(132151)		1	15.02	3.70		4.64			
434			2½	15.01	3.69		4.67	275.3		
430			5	14.99	3.69		4.67	275.3		
432			10	14.98	3.69		4.90	275.3		78.2 77.8
					AP2 =	27.1	AP5 =	80.4	AP4 = 25.2	
364	(11762)	12.27	0	14.45	2.27		5.15			
376			1	14.41	2.26		5.15	275.3		
[illegible]			2½	14.40	2.25		5.33	275.3		
364			5	14.38	2.24		5.32			
[illegible]			10	14.30	2.21		5.30	275.35		78.[illegible] 77.8
					AP2 =	27.1	AP5 =	80.3	AP4 =	25.[illegible]

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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE #1 | DATE 9 Nov 85 | SHEET 1 of 3

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (mm)	2 (mm)		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
760		12.40	0	13.68	1.68		6.22	275.4		
75[illegible]	23,468	(117 kn)	1	13.64	1.67		6.21			
[illegible]			2½	13.61	1.65		6.20	275.4		
73[illegible]			5	13.60	1.61		6.20			
[illegible]			10	13.58	1.60		6.40			78.2 77.6
				AP2 =	27.0	AP5.	80.2	AP4 =	25.1	T[illegible]
910	[illegible]	12.52	0	13.10	1.14		6.60	275.3		
89[illegible]	[illegible]		1	13.05	1.09		6.60			
[illegible]			2½	13.02	1.07		6.60			
[illegible]			5	13.00	1.04	Add spacer	6.60			
892			10	12.96	1.00 ⇒	9.07	6.92	275.3		78.1 77.6
				AP2 =	27.0	AP5 =	80.4	AP4 =	25.0	
1060	(116 kn)	13.04	0	12.35	8.53		7.02	275.3		
			1	12.23	8.42		7.12			
1048			2½	12.19	8.39		7.12			
			5	12.14	8.34		7.13			
1020			10	12.06	8.28		6.88			78.2 77.6
				AP2 =	26.9	AP5 =	80.3	AP4 =	25.1	
1220		13.16	0	11.35	7.60		6.86	275.5		
1218	(196 kn)		1	11.20	7.45		6.86			
12[illegible]			2½	11.05	7.30		6.85			
[illegible]92			5	10.90	7.15		6.87			
1[illegible]			10	10.65	6.90		6.84			78.[illegible] 77.[illegible]
				AP2 =	26.9 cm	AP5 =	80.3 cm	AP4 =	25.1 cm	

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AXIAL PILE LOAD TEST

TEST PILE #1 | DATE [illegible] | SHEET 2 of 3

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DATA SHEET

TEST PILE #1

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS R 1 mm	L 2 mm		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
1376	2150 psi	13.29	0	9.75	6.00		7.17			
1340			1	9.59	5.88		7.17			
1330			2½	9.48	5.78		7.17			
1330			5	9.25	5.54		7.13	275.7		78.2 77.8
[illegible]42			10	8.88	5.14		7.10			
				AP2 =	26.9	AP3 =	80.3	AP4 =	25.1	
1520		13.41	0	7.61	3.82		6.92	275.85		
14[illegible]			1	7.24	3.57		6.85			
1510	2366 psi		2½	6.82	3.13		6.84			
1514			5	6.09	2.40		6.90			
15[illegible]			10	4.60	0.90		6.90	276.1		78.2 77.7
				AP2 =	26.9	AP3 =	80.3	AP4 =	25.1	
	NO!			3.00 ⇒ 13.00						Add 10mm thick steel
					0.00 ⇒ 10.00					plates to both DG's
1584										COULD NOT JACK
1583	(2480 psi)		0	9.20	5.30		6.59			TO HIGHER LOAD,
										TOO MUCH MOVE-
1510	(2366 psi)		1	8.17	4.30		6.59			MENT ON PILE #1
										ALLOWED JACK
1490			2½	7.48	3.60		6.59			LOAD TO REACH
	unloading									EQUILIBRIUM.
1076			0	8.00	4.12		6.68			
1068			1	8.07	4.15			276.9		
466				9.75	5.80		5.74	276.8		
13	(2.06 pm)		0	14.3	9.25		5.63	275.5		
13	2 30		20m	14.80	9.68					

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AXIAL PILE LOAD TEST

P.R. Pile #1 | DATE [illegible] | SHEET 3 of 3

DATA SHEET

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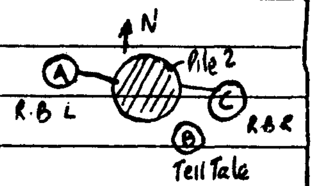
LOAD CELL ON PILE 4 (micro strains)	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS to Tip: Pile Top 1 A (inches)	Tell Tale 2 B (mm)	Pile Top C (mm)	Level Rdg. PILE TOP (2) Level (cm)	LEVEL READINGS: ~~PILE~~ Ref. Beam L	~~PILE SETTLE.~~ Ref Beam R	REMARKS
										Anchor Pile 3 ~~4~~ 5
0		0		1.410	1.09	2.20	42.20	98.5	92.8	94.35 98.5 97.05
10			0	1.400	1.23	2.28				
			1	1.399	1.23	2.28	42.20	98.5	92.7	94.35 98.5 97.10
			5	1.398	1.28	2.30				
20			0	1.394	1.41	2.38				
			.5	1.394	1.41	2.38				
			1	1.394	1.42	2.38				
			5	1.393	1.44	2.39		98.5	92.7	94.4 98.45 97.0
30			0	1.390	1.57	2.48				
			.5	1.388	1.58	2.49	42.30	98.55	92.70	
			5	1.386	1.61	2.51				N.B.
										(A)
40			0	1.380	1.715	2.585				Ref. Beam Left
			.5	1.378	1.75	2.60				has magnetic base
			1	1.377	1.77	2.62	42.2	98.5	92.7	& Strain Gauge on
			5	1.375	1.85	2.72				Pile
										(C)
50			0	1.367	2.06	2.87				
			.5	1.366	2.09	2.88				Magnetic Base on
			1	1.365	2.10	2.88				Pile and Strain
			5	1.363	2.17	2.97				Gauge on Ref Beam
										Right.
60			0	1.355	2.38	3.15				
			.5	1.355	2.40	3.15				
			1	1.350	2.40	3.16				
			5	1.350	2.53	3.27				



1.410 1.354 .056 | 1.410 1.395 .015 | 1.410 1.252 .158

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AXIAL PILE LOAD TEST

TEST PILE #2 | DATE 1 Mar 86 | SHEET 1 of 3

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS inches 1 A	mm 2 B	mm C	Level of PILE TOP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
								Ref. Beam L	Ref. Beam R	Anchor Piles 3 1 5 (may have moved)
70			0	~~1.94~~ 1.34	2.86	3.55			-	
			1	1.34	2.95	3.61	42.35	98.5	92.7	94.5 98.4 96.4
			5	1.334	2.98	3.64				
80			0	1.322	3.24	3.87				
			.5	1.322	3.27	3.88				
			1	1.321	3.28	3.90				
			5	1.319	3.31	3.96				
90			0	1.298	3.89	4.44				
			.5	1.295	3.94	4.48				
			1	1.295	3.95	4.50				
			5	1.289	4.12	4.66	42.50" (13.2)	98.5	92.7	
			10	1.280	4.32	4.86				* New scale
										attached to pile 2.
100			0	1.259	4.88	5.36				42.50 ≡ 30.5
			.5	1.255	4.96	5.43				1" = 50 div. (×4)
			1	1.252	5.04	5.49	(13.2)			
			5	1.240	5.37	5.82	42.65	98.5	92.7	
			10	1.233	5.58	6.05	(1375)			
120			0	1.203	6.35	6.78				
			.5	1.191	6.62	7.05	(13.95)			
			1	1.181	6.81	7.20	(14.0)			
			5	1.153	7.54	7.96	(14.8)			
			10	1.130	8.10	8.50	(14.3)			

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AXIAL PILE LOAD TEST

TEST PILE # 2	DATE 1 Mar 86	SHEET 2 of 3

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	2		Level PILE TOP	LEVEL READINGS PILE Ref. Beam L	PILE SETTLE Ref. Beam R	REMARKS Anchor Piles 3 1 5
				A	B	C				
120			0	1.080	8.53	8.95	(~~15.1~~ 14.45)		-	
			.5	1.066	8.82	9.26				
			1	1.050	9.15	9.54	(~~15.4~~ 14.6)			
			5	1.008	11.10	11.58	(~~15.8~~ 14.9)			
			10	.970	11.65	12.40				
					no travel					
130			0	.870		14.30	(15.1)			
			.5	.815		15.30	(15.8)			
			1	.770		17.40				
			2.5	.710		18.80	(16.4)			
			5	.672		19.72	(16.6)			
			10	.630	★ Rdgs unreliable	(20.43	≡ 18.28) *			* Reset
										★ Readings probably
~~140~~ 130			0	.525		17.90	(17.4)			unreliable due to
										dial gauge arm
	UNLOAD						(17.8)			catching ref. beam
100			0	.417			44.7	~~93~~ 98.45	92.8	94.3 98.25 96.7
50			0	.560			(17.6)			
0		(2.50 pm)	0	.545	(9.56)		(17.2)			
		(3.00 pm)		.556						

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AXIAL PILE LOAD TEST

TEST PILE #2 | DATE 1 Mar 86 | SHEET 3 of 3

DATA SHEET

Pile #3

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UBC LOAD CELL	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS (inches) 1 South	(inches) 2 North		PILE TIP (mm)	LEVEL READINGS * (cm) PILE	PILE SETTLE.	REMARKS
0				1.894	1.624		4.49	~~374.1~~ 374.2		* Level [illegible]
	Ref Beam Left =			~~78.8~~	78.9 cm					[illegible] #3
	Ref Beam Right				80.1 cm					
pc										
10			0	1.891	1.619		5.01	~~374.2~~ 374.2		
			1	1.891	1.617		5.01			
			2 1/2	1.891	1.617		5.01			
			5	1.891	1.617		5.02	374.2		
			10	1.890	1.616		5.02			
	Ref Beam		Left	= 78.9						
			Right	= 80.0						
22			0	1.881	1.604		5.03	374.2		
			1	1.881	1.604		~~5.03~~ 6.79			
			2 1/2	1.881	1.604		6.79			
			5	1.881	1.604		6.80	374.2		
			10	1.880	1.604		6.85			
		Ref Beam	Left =	~~78.8~~ 78.8						
		Ref Beam	Right =	80.1						
32			0	1.873	1.592		~~5.92~~ ~~6.85~~	374.3		
			1	1.874	1.592		5.92			
			2 1/2	1.874	1.592		5.92			
			5	1.874	1.592		6.55	374.3		
			10	1.873	1.591		6.10	374.2		

Ref. Beam Left = 78.8
Right = 80.1

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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

[illegible] | DATE 9/6/85 | SHEET [illegible]

DATA SHEET

TEST PILE # 3 255

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	2		PILE TIP	LEVEL READINGS PILE m	PILE SETTLE.	REMARKS
40			0	1.866	1.582		6.64		-	
			1	1.866	1.582		6.64	374.3		
			2½	1.866	1.582		6.70			
			5	1.866	1.582		6.70			
			10	1.865	1.584		6.70	374.4		
		Ref. Beam	Left	= 78.9						
			Right	= 80.0						
50			0	1.857	1.571		7.39	374.3		
			1	1.857	1.571		7.38			
			2½	1.856	1.571		7.38	374.3		
			5	1.856	1.572		7.38			
			10	1.855	1.570		7.43			
		Ref Beam		Left = 78.8						
				Right = 80.0						
60			0	1.847	1.562		~~7.45~~ 7.70	374.2		
			1	1.847	1.562		7.70			
			2½	1.847	1.562					
			5	1.847	1.562		8.65			
			10	1.847	1.562			374.2		
		Ref Beam		Left	= 78.8					
				Right	= 80.0					

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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

DATE 1 Nov 85 | SHEET 2

DATA SHEET TEST PILE # 3 256

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	2		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
70			0	1.837	1.553		~~374~~ 9.39	374.2	-	
			1	1.837	1.553		9.34			
	500.0		2½	1.836	1.552					
			5	1.836	1.552		9.16			
			10	1.836	1.551		9.28	374.2		
		Ref. Beam Left = 78.8								
		Ref. Beam Right = 80.0								
80			0	1.827	1.542		9.28	374.3		
			1	1.827	1.542					
			2½	1.827	1.540		9.28			
			5	1.825	1.540					
			10	1.824	1.539		9.38			
		Ref. Beam Left = 78.8								
		Right = 80.0								
90			0	1.814	1.529			374.4		
			1	1.814	1.529		9.52			
			2½	1.814	1.529					
			5	1.813	1.528		9.50	374.4		
			10	1.811	1.526		9.57			
		Ref. Beam Left - 78.9								
		Right - 80.0								

DATA SHEET TEST PILE #3

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	EXTENSOMETER READINGS 2		PILE TIP	LEVEL READINGS PILE	LEVEL READINGS PILE SETTLE.	REMARKS
100	700.0		0	1.800	1.516			374.4		
			1	1.800	1.517		9.66			
			2½	1.800	1.515					
			5	1.798	1.513					
			10	1.798	1.512			374.4		
		Ref Beam	Left = 78.8							
				Right = 80.0						
COULD NOT JACK BEYOND THIS LOAD DUE TO TOO MUCH MOVEMENTS ON PILE #1										
108			0	1.790	1.507					PILE #1, WHICH WAS
103			1	1.791	1.508		9.64	374.4		SIMULTANEOUSLY
			2½	1.792	1.510					TESTED, FAILED
										AT THIS LOAD.
78 ~~90~~			0	1.807	1.525		9.25	374.4		∴ UNLOAD ↓
40			0	1.838	1.553		8.92	374.4		
0			0	1.867	1.580		8.54			
		14:05	1	1.870	1.580					
		14.45		1.874	1.583			374.3		
								374.4		
		Ref Beam		Left = 79.0						
				Right = 80.1						

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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

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DATA SHEET

TEST PILE #3 258

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	2		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
0		15:00		1.874	1.583		8.40	374.4	-	RESUME LOAD TEST
										FOR PILE [illegible]
			Anchor Pile		#2	27.2 cm		Level Readings		Opposite end of beam
					#5	80.5 cm				rest on
					#4	25.3 cm				timber cribbing
			Timber Footings			94.9 cm				
50			0	1.849	1.555		8.79	374.4		
			1	1.849	1.555					
			$2\frac{1}{2}$	1.849	1.555					
			5	1.849	1.555		8.83	374.4		
			10							
			Ref. Beam		Left = 79.0			Level Readings		
					Right = 80.1					
			Timber Footings = 95.5							
				Anch Pile #2 = 27.2						
					#5 = 80.4					
					#4 = 25.4					
104	750 psi		0	1.808	1.517		9.30	374.5		
			1	1.808	1.515					
			$2\frac{1}{2}$	1.806	1.514		9.28			
			5	1.804	1.513			374.5		
			Ref. Beam		Left = 79.0			Anchor Pile #2 = 27.2		
					Right = 80.1			Anchor Pile #5 = 80.1		

Timber Footing = 96.2 Anch Pile #4 = 25.3

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Pile #3 footing

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

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DATA SHEET

TEST PILE #3

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	2		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
120			0	1.790	1.498		9.49	374.6		
			1	1.785	1.493					
			$2\frac{1}{2}$	1.782	1.491					
			5	1.781	1.490					
			10	1.779	1.485		9.40	374.6		
				Ref Beam Left = 79.0						
				Right = 80.1						
				Timber Footings = 96.4				Level Readings.		
				Anchor Pile #2 = 27.2						
				#5 = 80.1						
				#4 = 25.3						
140	1000 psi		0	1.755	1.467		9.72	374.7		
			1	1.750	1.461		9			
			3	1.746	1.456		9.75			
			5	1.744	1.455			374.7		
			10	1.739	1.450		9.65			
				Ref Beam Left		=	79.0			
				Ref Beam Right		=	80.1			
				Timber Footing		=	96.7	Level Readings.		
				Anchor Pile	#2	=	27.15			
					#5	=	80.05			
					#4	=	25.3			

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

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DATA SHEET

TEST PILE # 3

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	2		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
160	1100.0		0	1.711	1.424		10.03	374.8	-	
			1	1.709	1.420					
			2 1/2	1.704	1.416					
			5	1.700	1.412			374.83		
			10	1.694	1.405		9.84			
				Ref. Beam		Left =	79.00		Level Reading	
						Right =	80.20			
				Timber Footing		=	96.90			
				Anchor Pile # 2		=	27.1			
						# 5 =	80.00			
						# 4 =	25.2			
180	1250.0		0	1.667	1.379		9.93	375.0		
			1	1.663	1.376					
			2 1/2	1.655	1.368					
			5	1.650	1.362			375.00		
				Ref Beam		Left =	79.1			
						Right =	80.2			
				Timber Footing		=	97.0			
					Anchor Pile	# 2 =	27.1			
						# 5	= 80.0			
						# 4	= 25.3			

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

DATE [illegible] SHEET 7

DATA SHEET

TEST PILE #3

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	EXTENSOMETER READINGS 2		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
220	1400		0	1.612	1.327		10.03	375.1		
			1	1.592	1.307				-	
			2½	1.583	1.302					
			5	1.577	1.290					
			10							
				Ref	Beam	Left	= 79.0			
						Right	= 80.2			
				Timber	Footing		=			
				Anchor	Pile	# 2	= 27.0			
						# 5	= 79.9			
						# 4	= 25.4			
220			0	1.537	1.255		10.29	375.4		
			1	1.518	1.232					
			2½	1.505	1.220					
			5	1.502	1.216					
										← UNLOAD HERE TO
0			0	1.720	1.425					READJUST [illegible]
			2½	1.728	1.428					OF BEING & RELIEVE
			5	1.727	1.428					LOAD ON PILE
			10	1.730	1.430			374.8		DYNAMIC BALL.
220			0	1.481	1.215		10.12	374.0		
			1	1.468	1.200					
			5	1.455	1.188					

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

DATE 1 Nov 85 | SHEET 3 of 4

DATA SHEET

TEST PILE #3

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1	2		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
240			0	1.428	1.163	10.37		375.4		
			2	1.415	1.150				-	
			5	1.405	1.140					
262			0	1.375	1.112			375.7		(374.4)
			$2\frac{1}{2}$	1.358	1.096					THESE [illegible] ARE
			5	1.351	1.091		10.31			TOO [illegible]! [illegible]
			15	1.335	1.075					ANCHOR PLATE ABOVE
										BEAM WAS TIGHT
				Ref	Beam	74.0	Left			AGAINST BEAM
						80.15	Right			"Pulling up on [illegible]
		*								Bootstrap
~~280~~			~~0~~	~~1.315~~	~~1.061~~					*
UNLOAD MOMENTARILY			TO 150	E Ra[illegible]		TO LOOSEN		NUTS ABOVE BEAM		Remedy [illegible]
260			0	1.275	1.002					[illegible] on Pile # 3
			1	1.235	0.968					[illegible] to 150 reload to 260
			$2\frac{1}{2}$	1.212	0.947		11.65			
			5	1.190	.928			376.1		
280	1850		0	1.130	.868					
			1	~~1.12~~ 1.100	.850					
			$2\frac{1}{2}$	1.050	.780			376.6		
			5	1.000	.740					
200			1 min	~~1.330~~ 1.90	.770		11.00			
100				1.122	.846		10.85			
0		17.45		1.282	.989		8.45	375.9		
0		18.00		1.302	1.004					Final [illegible]

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

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DATA SHEET

PILES 1 & 3 TESTING

WEIR JONES' LOAD CELLS

UBC LOAD Cell	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	Channel 1 Load Cell 1	Channel 2 Load Cell 2	Channel 3 Load Cell 3	Channel 4 Load Cell 5	Channel 5 Load Cell 6		REMARKS
				6-89	3-93	1-04	88	5-92	←	BALANCE
				1	2	4	5 west	5 east	←	PILE NO
0				0	0	0	0	0		
				31 x 31.25 (lb/...) = 968.75						
				Loading ... weight of beam						
10 [illegible]		0		145	46	0	0	0		Load
		1		126	46	0	0	0		
		2.5		131	56	0	0	0		
		6		136	62	0	0	0		
		15		139	63	0	0	0		
22 ε		0		276	316	0	0	0		
		1		265	296	0	0	0		
		[illegible]		264	[illegible]	0	0	~~3~~ 0		
		[illegible]		271	390	0	0	0		
		10		276	388	0	0	76		
32 ε		0		476	586	0	0	96		
		1		423	592	0	0	78		
		2½		434	588	0	0	49		
		5		430	576	0	0	100		
		10		432	660	0	0	72		
40 ε		0		564	870	80	0	92		
		1		576	780	78	0	165		
		2½		560	804	82	0	127		
		5		564	764	54	0	120		
		15		574	755	48	0	74		

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

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SHEET 1 of 7

DATA SHEET

TEST PILES 1 & 3

Weir Jones' Load Cells

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	Ch. 1 Pile 1	Ch. 2 Pile 2	Ch. 3 Pile 4	Ch. 4 Pile 5W	Ch. 5 Pile 5E		REMARKS
50με			0	760	906	100	160	153		
			1	751	944	106	149	164		
			2½	744	943	54	124	150		
			5	737	942	66	139	174		
			10	739	926	84	135	160		
60με			0	910	1074	162	178	197		
			1	870	1004	160	182	187		
			2½	886	1014	163	172	178		
			5	890	1090	194	190	190		
			10	872	1020	152	164	156		
70με			0	1060	1218	262	268	270		
			1	1044	1228	254	234	250		
			2½	1048	1180	230	214	224		
			5	1040	1126	232	228	229		
			10	1020	1160	210	214	204		
80με			0	1220	1320	308	306	304		
			1	1218	1336	286	300	276		
			2½	1212	1274	345	306	292		
			5	1192	1300	338	334	336		
			10	1180	1355	356	338	350		
90με			0	1376	1426	408	374	384		
			1	1340	1470	422	394	394		
			2½	1330	1454	399	381	318		
			5	1330	1412	398	395	334		
			10	1342	1424	390	370	366		

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AXIAL PILE LOAD TEST

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DATA SHEET

TEST PILES #1 & 3
Weir Jones' Load Cells

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	Ch 1 Pile 1	Ch 2 Pile 2	Ch Pile 4	Ch Pile 5W	Ch 5 Pile 5E		REMARKS
100 [illegible]			0	1520	1568	540	518	398		Lo[illegible]
			1	1488	1580	530	488	402		
			2½	1510	1590	510	480	426		
			5	1514	1558	548	432	434		
			10	1516	1518	536	440	400		
108 [illegible]			0	1588						max load for
			1	1510						Pile #1
			2½	1490						
			4	1458						
	unload			1076						
				1068						
		unload		466	unloading					
				13						

PILE #1 FAILED.
SUPPORT THAT END OF BEAM ON TIMBER CRIBBING
SO GROUND WILL PROVIDE REACTION FOR
TESTING PILE #3.

DATA SHEET

TEST PILE #3 ONLY
WEIR JONES' LOAD CELL

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	Ch 1 Timber Cribbing	Ch 2 Pile 2	Ch 3 Pile 4	Ch 4 Pile 5 W	Ch 5 Pile 5E		REMARKS
0 ?				-42	-240	52	48	95		[illegible]
51 µε			0	890	-206	0	554	652		[illegible]
			1	890	-214	0	570	640		
			2½	896	-174	0	494	670		
			5	922	-216	0	436	658		
			1½							
104 µε			0	2020	-30	0	870	1248		
			1	1990	0	0	880	1260		
			2½	2020	12	0	896	1300		
			5	2036	37	0	886	1236		
120 µε			0	2412	196	~~180~~ 130	[illegible]	1384		
			1	2436	232	98	923	1300		
			2½	2447	240	84	940	1430		
			5	2447	240	98	950	1396		
			10	2458	280	102	956	1342		
140 µε			0	2882	462	204	1070	1466		
			1	2870	518	180	1060	1506		
			2½	2866	538	192	1020	1524		
			5	2872	552	202	1022	1495		
			10	2849	588	196	1046	1564		
160 µε			0	3166	824	280	1130	1574		
			1	3034	804	296	1122	1536		
			2½	3078	862	300	1104	1620		
			5	3080	878	336	1140	1594		
			10	2932	892	304	1106	1554		

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

DATE [illegible] |

DATA SHEET

TEST PILE #3

Weir Jones' Load Cells

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	Ch.1	Ch.	Ch.	Ch.#	Ch.		REMARKS
[illegible] kips			0	3046	1094	450	[illegible]	1698		
			1	2802	1096	[illegible]	[illegible]	1770		
			2½	2954	1156	462	1234	1686		
			5	2830	1150	480	1262	1708		
			10							
200 kips			0	2966	1286	646	1316	1770		
			1	2982	1410	600	1283	1790		Load Cell
			2½	2950	1390	636	1308	1788		is not [illegible]
			5	2886	1454	642	1300	1820		[illegible]
										loading [illegible]
220 kips			0	3010	1610	784	1400	1896		[illegible]
			1	2934	1586	786	1372	1850		
			2½	2870	1586	810	1400	1876		
			5	2860	1600	812	1382	1866		

Open the valve of pressure [illegible] on jack

[illegible]

Load cell #1 taken off timber cribbing footing

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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

DATE

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DATA SHEET

TEST PILE #3

Weir Jones' Load Cells

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.							REMARKS
[illegible]			0		1320	880	1484	1930	-	-
			1		1378	886	1496	1922		
			2½		1376	914	14[illegible]	1900		
			5		1374	886	1482	1900		
240[illegible]			0		1420	1254	1800	1920		
			1		1420	1384	1752	1936		
			2½		1446	1086	1574	1834		
			5		1422	1216	1592	1786		
26[illegible]			0		1558	1166	1706	2036		
			1		1538	1834	1644	2066		
			2½		1535	1254	1708	1928		
			5		1530	1240	1680	1980		
					1666	1414	1705	1980		[illegible]
										=> [illegible]
										[illegible]
260,4[illegible]			0		1736	1702	1916	2168		
			1		1786	1372	1738	2026		
			2½		1756	1554	1716	2084		
			5		1764	1548	1760	2050		
[illegible]			[illegible]		[illegible]	1784	1970	2156		
			[illegible]		1978	1730	1964	2104		
			[illegible]		1972	1722	1972	2130		
			[illegible]		1984	1750	1950	2160		

DATA SHEET

TEST PILE #3
Weir Jones' Load Cells

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.		Ch2	Ch3	Ch4	Ch5		REMARKS
[illegible]			6		1418	1290	1442	1144	-	[illegible]
100 k			0		698	320	910	922		
0					-250	0	-40	-80		

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AXIAL PILE LOAD TEST

DATA SHEET

1 March 19[illegible]

Pile No 4

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	PILE TOP L (inches)	PILE TOP R (inches)	T/T SOIL PLUG (mm)	T/T 1 (75')	T/T 2 (50')	T/T 3 (26')	REMARKS
0	0	12:43	0	1.785	1.790	0.05	4.08	0.27	1.73	T/T¹ Large Dia.
F=10			1/2	1.781	1.781	0.19	4.08	0.36	1.85	(75' Inclinometer
			1	1.781	1.781					T/T² [illegible]
			5	1.781	1.780	0.20	4.08	0.38	1.87	T/T³ (26')
	5min L. Ref Beam =				97.4					Level on Pile
	R Ref Beam =				98.3				Zero's	337.00 cm = Zero
										Left Ref 97.50
										Right Ref 98.20
F=20		12:51	0	1.795	1.760	0.31	4.04	0.37	1.89	Level on Pile
			1/2	1.795	1.760					
			1	1.795	1.760					338.10
			5	1.805	1.77					
			~~10~~							
		5(min)	L. Ref Beam			97.35				
			R Ref Beam			98.35				
F=30		12:57	0	1.785	1.750	0.41	4.01	0.32	1.89	338.10
			1/2	1.785	1.750					
			1	1.785	1.750					
			5	1.79	1.750	0.42	3.96	0.30	1.89	
			L. Ref Beam			97.35				
			R. Ref Beam			98.20				
F=40		13:04	0	1.765	1.720	0.56	3.96	0.29	1.89	338.10
			1/2	1.765	1.710					
			1	1.765	1.710					[illegible] T/T [illegible]
			5	1.770	1.700	0.57	3.89	0.29	1.96	
		15min	L	Ref Beam		97.30	cm			
			R	Ref Beam		98.20	cm			

L. Ref. ⇒ T/T 1 & L Pile & Soil Plug

R Ref ⇒ T/T 2 & 3 & R Pile

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

2

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	PILE TOP L (inches)	PILE TOP R (inches)	T/T SOIL PLUG (mm)	T/T 1 (mm)	T/T 2 (mm)	T/T 3 (mm)	REMARKS Pile level
ε = 50		13:11	0	1.774	1.7650	0.71	3.89	0.29	1.97	338.1
(110kN)			1/2	1.7745	1.7655				1	
			1	1.7748	1.7652					
			5	1.7755	1.7650	0.71	3.89	0.28	1.	
			L.	Ref Beam						
			R.	Ref Beam						
ε = 60	(600psi)	13:16	0	1.773	1.7620	0.85	3.86	0.25	1.99	338.12
			1/2	1.7735	1.7620					
			1	1.7735	1.7620					
			5	1.772	1.7625	0.89	3.79	0.21	1.99	
			L	Ref Beam						
			R	Ref Beam						
ε = 70	(670psi)	13:25	0	1.7690	1.7590	1.04	3.78	0.23	1.99	338.13
			1/2	1.7685	1.7585					
			1	1.7680	1.7585					
			5	1.7685	1.7610	1.07	3.73	0.16	1.97	
			L	Ref. Beam		97.4				
			R	Ref. Beam		98.25				
ε = 80	(720psi)	13:30	0	1.765	1.757					
			1/2	1.765	1.757					
			1	1.7655	1.757	1.20	3.705	0.11	1.96	
			5	1.766	1.759					
			L	Ref Beam						
			R	—						
ε = 90	(8[illegible]psi)	13:39	0	1.762	1.754	1.37	3.67	0.08	1.93	338.13
			1/2	1.762	1.754					
			1	1.762	1.755					
			5	1.762	1.756	1.37	3.64	0.01	1.92	

10 1.762 1.756

L. Ref Beam : 97.3

R. Ref Beam : 98.2

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE #4 | DATE 1 Mar 86 | SHEET 2 of 5

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	PILE TOP L (Inches)	PILE TOP R (Inches)	T/T SOIL PLUG (mm)	T/T 1 (mm)	T/T 2 (mm)	T/T 3 (mm)	REMARKS Pile end
Σ=100	(870psi)	13:50	0	1.7585	1.7525	1.56	3.59	0.00	1.92	337.16
			1/2	1.7585	1.7525					
			1	1.7585	1.7525					
			5	1.759	1.753					
			10	1.759	1.753	1.59	3.55	-.96	1.92	
			L. Ref Beam		97.3					
			R Ref Beam		98.2					
Σ=110	(940psi)	14:01	0	1.7550	1.749	1.79	3.55	-.99	1.92	
			1/2	1.7545	1.7475					
			1	1.7545	1.7475					338.13
			5	1.7545	1.7475					
			10	1.755	1.7485	1.83	3.50	-.95	1.92	
Σ=120	1010psi	14:12	0	1.7515	1.7440	1.97	3.49	-.95	1.92	338.20
(264kN)			1/2	1.7515	1.7440					
			1	1.7515	1.7440					
			5	1.7515	1.7440					
			10	1.7490	1.7430	2.04	3.40	-0.92	1.92	
Σ=130	1070psi	14:25	0	1.745	1.738	2.23	3.37	-0.90	1.92	
			1/2	1.745	1.738					
			1	1.745	1.737					
			5	1.749	1.737					
			10	1.750	1.737					
Σ=133			0	1.7480	1.7350					
Σ=130				1.7490	1.7370					
Σ=100				1.761	1.7485	1.65	3.36	-0.91	1.89	

~~Σ=130~~

R Ref Beam 98.20
L. Ref Beam 97.35

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE #4 | DATE 1 Mar 86 | SHEET 3 of 5

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	PILE TOP L	PILE TOP R	T/T SOIL PLUG	T/T 1	T/T 2	T/T 3	REMARKS
[illegible]				1.7820	1.7725	0.67	3.34	-0.91	1.53	
[illegible]		2 50		1.8080	1.8035	-0.87	3.34	-0.91	1.05	
		3 00		1.8050	1.800	-0.85	3 34	-0 91	1.03	
				Testing	Pile	No 4 only				
0				1.824	1.828	-0.84	3.06	-0.91	1.03	338.05
				L. Ref Beam		97.4				
				R. Ref Beam		98.2				
70		15:40	0	1.796	1.794	0.43	3.02	-0.90	1.06	338.10
			1	1.797	1.794					
			2	1.797	1.794					
130		15.44	0	1.766	1.759	1.78	3.00	0.00	1.49	338.12
			1	1.766	1.758					
			2	1.766	1.757	1.78	3.02	0.00	1.54	
200		15.52	0	1.715	1.716	3.22	3.35	0.0	2.14	338.17
			1	1.715	1.716					338.[illegible]
			2	1.714	1.7145					
									Pile 5	= 96.25 cm
250	1900 psi	16.03	0	1.676	1.668	4.59	3.61	+0.08	3.02	
			1	1.676	1.668					
			2	1.67[illegible]	1.666					

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	PILE TOP L	PILE TOP R	T/T SOIL PLUG	T/T 1	T/T 2	T/T 3	REMARKS
ε=	290	16.10	0	1.649	1.630					
ε=0		16.15	0	1.808	1.808	0.84	3.63	0.60	1.02	
			1	1.808	1.808					
Restart										
		Note-	Beam	was readjusted,	Reference Beam	could	have	moved		
										Level Readings.
~~17 00~~	0	17.20 hr.		1.827	1.826	(0.71)	3.47	1.67	1.30	338.30
				~~1.822~~	~~1.818~~					
~~100~~	~~100~~									
100	850		0	1.778	1.767	2.48	3.37	2.17	2.04	338.40
200	1500		0	1.716	1.702	4.86	3.48	2.55	2.92	338.50
300	2100		0	1.6535	1.6285	7.14	3.60	3.12	3.92	338.80
			1	1.6503	1.627	7.24	3.63	3.15	4.01	338.85
			5	1.649	1.624	7.28	3.68	3.22	4.20	338.85
340			0	1.610	1.579	8.52	4.09	3.69	4.97	338.87
	Unload									
300				1.627	1.592	8.68	4.12	3.60	4.97	338.90
200				1.685	1.654	(5.92) 5.92	3.98	3.24	4.04	338.70
100				1.746	1.726	3.43	3.94	2.87	2.89	338.60
0			0	1.800	1.795	1.10	3.88	2.24	2.05	338.40
			5	1.809	1.806	0.97	3.84	2.13	1.87	338.40

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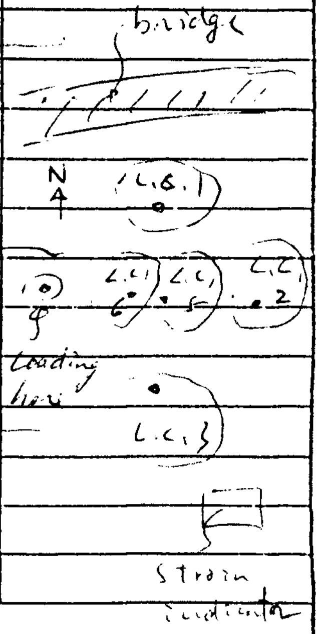
AXIAL PILE LOAD TEST

TEST PILE #1 | DATE [illegible] 81 | SHEET 5 of 5

DATA SHEET

Gag Factor 1.0

LOAD CELL GAUGE 4 ε Rdgs	DIAL PRESSURE P.S.I.	TIME ~~HRS.~~ min	ΔT MINS.	Pile No. 1 Load Cell No. 1	Pile No. 2 Load Cell No. 2	Pile No. 3 Load Cell No. 3		Pile No. 5 EAST ~~WEST~~ Load Cell No. 5	Pile No. 5 WEST ~~EAST~~ Load Cell No. 6	REMARKS
0				0	0	0		0	0	
10		0		0	320	64		88	342	
		1		0	280	40		78	364	
		5		0	240	24		82	306	
20		0		28	374	46		~~374~~ 100	518	
		1		28	442	26		106	546	
		5		26	492	68		266	590	
30		0		0	774	190		294	978	
		1		0	680	160		84	780	
		5		0	556	30		74	898	
40		0		0	940	96		4	1320	
		1		~~40~~ 0	796	54		56	1116	
		5		0	746	0		42	1070	
50		0		0	996	0		84	1292	
		1		0	926	0		76	1292	
		5		0	890	0		102	1396	
60		0		0	1116	40		166	1864	
		1		0	1214	0		154	1582	
		5		0	1108	138		198	1606	
70		0		0	1320	290		234	1930	
		1		0	1368	384		282	1960	
		5		0	1320	320		268	1834	



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AXIAL PILE LOAD TEST

TSC | DATE March 1, 86 | SHEET 1 of 2

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME ~~HRS.~~ (min)	ΔT MINS.	Load Cell No.1	Load Cell No.2	L.C. No.3		L.C. No.5	L.C. No.6	REMARKS
80		0		0	1690	320		350	2000	
		1		0	1620	354		310	2024	
		5		0	1374	296		294	2020	
90		0		56	1560	354		454	2176	
		1		54	1554	336		450	2186	
		5		56	1580	364		420	2200	
		~~10~~		~~58~~	~~1526~~	~~402~~		~~440~~	~~2284~~	
100		0		62	1760	544		614	2440	
		1		60	1764	456		608	2434	
		5		58	1834	550		640	2530	
		10		82	1870	452		594	2352	
110		0		96	2018	520		806	2598	
		1		96	1986	586		832	2600	
		5		80	2006	522		798	2596	
		10		84	2008	520		740	2570	
120		0		136	2120	576		996	2720	
		1		94	2170	596		1010	2860	
		5		94	2120	616		962	2752	
		10		104	2098	592		902	2750	
~~130~~ 130		0		1[illegible]0	2362	604		11[illegible]6	2970	
		1		126	2268	590		1032	2950	
		5		136	2280	616		976	3020	
		10		120	2230	640		924	3016	

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AXIAL PILE LOAD TEST

T...	DATE ... 86	SHEET 2 of 2

(0, 1/2, 1, 2 1/2, 5, 10 min)

DATA SHEET

[illegible] #5 Pile — PKR, [illegible]

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS TOP 1 (LEFT)	TOP 2 (RIGHT)	50' Telltale (A)	PILE TIP (A)	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
[illegible]	0		DIAL	GAUGE						[illegible]
[illegible]			5.045	mm	([illegible])			AP2	[illegible] cm	
			2.01	0 mm	(1.910)			AP[illegible]	[illegible]	
[illegible]			2.160	mm	(2.140)			AP4	[illegible]	10:[illegible]
			5.111 mm		(5.070)			AP3	91.8[illegible] cm	
TP5				1.8391"	1.8798"	1.00 mm	3.00 mm	TP 5	391.55 cm	17°C
BM			* →	(1.8360")	([illegible])	* 10:26			37.30 cm	← Bench mark
Left	[illegible]								97.15 cm	
Rt.	[illegible]								36.10	
20E		10:33		1.8360	1.8540	[illegible]	[illegible]			10:28
[illegible]		1/2		1.8300	1.8540	"	"			
		2		1.8291	1.8538	"	"			
40E		10:36		1.8225	1.8350	"	"			
[illegible]		1/2		1.8225	1.8348	"	"			
		1		1.8224	1.8347					
		2 1/2		1.8215	1.8340				391.50	
		5		1.8205	1.8338					
[illegible]		10:42		1.8128	1.8182	1.00	2.955			
[illegible]		1/2		1.8126	1.8180	"	"			
		1		1.8124	1.8180	"	2.985		391.53	
		2 1/2		1.8120	1.8178	"	"			
		5		1.8107	1.8170	"	"		391.52	
		10		1.8085	1.8152				391.52	
AP1 =	[illegible]	AP2 =	5.135	AP3 = 5.470		[illegible]	2.152			

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE #5 | DATE | SHEET 1 of 3

DATA SHEET

1.8360 1.8355

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (L)	2 (R)	(C)	PILE TIP (A)	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
20E	400psi	10 48		1.7995	1.8032	1.0	2.985	391.59		
		1		1.7988	1.8032	"	2.985			
		2		1.7986	1.8029	"	"	391.59		
	500psi	5		1.7971	1.8018	"	2.985	391.59	0.4mm	
		10		1.7939	1.7998	"	"	391.59		
AP1 = 2.500	AP2 = 5	.140	AP3 =	5.490	AP4 =	?				
100E	(... kN)	11.05	0	1.7845	1.7865	1.010	2.985	391.60		
	660psi		1	1.7845	1.7862	-	-			
			2½	1.7833	1.7850	1.011	2.985			
			5	1.7825	1.7839	1.012	2.982	391.52		
			10	1.7814	1.7819					
AP1 =	2.801	AP2 =	5.120	AP3 =	5.530	AP4 =	2.160	391.59		
	Rt Ref	Beam	= 95.72 ?	Left	Ref	B =	95.185 ?			Ref Beam
										Level ...
120E	(660psi)	11.17	0	1.7715	1.76675	1.022	2.980			
			½	1.7712	1.7675					
	8..psi		1	1.7705	1.7663	1.025	2.980	391.60		
			2½	1.7695	1.7649	1.025	-"-			
			5	1.7715	1.7649	1.028	2.981			
			10	1.7733	1.7659	1.030	2.981			
AP1 =	3.250	AP2 =	4.945	AP3 =	5.600	AP4 =	2.163	391.70		
	Rt Ref	Beam	= 95	.93 (1.7mm) Left	Ref	Beam		96.93	(2.2mm)	
140E	(915kN)	11.30	0	1.7620	1.7512	1.040	2.980			
		½	½	1.7611	1.7501	1.040	2.980	391.67		
			1	1.7610	1.7498	1.040	-"-			
			2½	1.7604	1.7492	1.041	-"-			
	900psi		5	1.7605	1.7482	1.042	2.95	391.69		
			10	1.7621	1.7488	1.045	2.980	391.71		

AP1 = 3.543 AP2 = 4.900 AP3 = 5.35 AP4 = 2.153

Rt Ref Beam = 96.00 Left Ref Beam = 97.00

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE #5 | DATE 22-09-85 | SHEET 2 of 8

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (L)	2 (R)	(B)	PILE TIP (L)	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
160 t		11:41	0	1.7520	1.7344	1.055	2.980			
			½	1.7521	1.7341	-"-	-"- —	391.70		
			1	1.7520	1.7337	-"-	2.980			
			2½	1.7506	1.7315	1.056	2.980	391.71		
	[illegible]		5	1.7491	1.7290	1.060	2.980	391.68		
			10	1.7485	1.7275	1.058	2.980	391.78		
AP1 =	3.259	AP2 =	4.840	AP3 =	5.672	AP4 =	2.151			
	Rt Ref	Beam	=95.9	2 Left	Ref	Beam	96.92			
180 t	([illegible])	11:53	0	1.7400	1.7175	1.063	2.980			
			½	1.7401	1.7175	1.068	"			
			1	1.7405	1.7174	1.068	"	391.85		
			2½	1.7402	1.7162	1.070	2.980			
	1150 psi		5	1.7402	1.7153	1.072	2.965	391.70	1.5mm	
	(4.297)		10	1.7393	1.7111	1.072	2.966	391.70		
AP1 =	4.299	AP2 =	4.669	AP3 =	5.815	AP4 =	2.065			
	Rt Ref	Bea	= 95.	95 Left	Ref	Bea	= 96.89			
200 t	([illegible])	12.04	0	1.7271	1.6921	1.075	2.966	391.70		
			½	1.7270	1.6918	1.075	-"-			
	1250 psi		1	1.7270	1.6918	1.075	-"-			
			2½	1.7259	1.6905	1.079	2.965	391.75		
			5	1.7251	1.6898	1.080	2.965	391.69		
			10	1.7255	1.6869	1.081	2.965	391.69		
AP1 =	4.715	AP2 =	4.575	AP3 =	5.810	AP4 =	2.040			
	Rt	Ref. Beam	= 95.83		Left	Ref Beam	= 96.	88		

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE #5 | DATE 22-05-85 | SHEET 3 of 8

(4)

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (L)	2 (R)	(B)	PILE TIP (A)	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
220E	1400 psi	11·20	0	1·7120	1·6687	1·31	2·970			
			½	–	–	–	–			
	(1400 psi)		1	1·7120	1·6687	1·315	2·975	391·77		
			2½	1·7119	1·6682	1·315	2 979			
			5	1·7108	1·6664	1·315	2·980	391·76		
			10	1·7108	1·6665	1·315	2·980			
AP1 =	5·299	AP2 =	4·440	AP3 =	5·900	AP4 =	2·000	391·73		
		Rt Ref	Beam	= 95·85	Left	Ref.	Beam =	96·87		
										Unload !
160E	(990 psi)	12·31	0	1·7379	1·6927	1·310	2·980	391·65		
157E			2	1·7385	1·6942	1·310	2·980			
AP1 =	5·109	AP2 = 4	·428	AP3 =	5·8..	AP4 =	2·000			
100E	600 psi		0	1·770	1·7271	1·290	2 98	391·69		
			2	1·7715	1·7316	1·291	2·980			
AP1 =	4 725	AP2 =	4 50.	AP3 =	5 780	AP4 =	2·000			
		Left	Ref	Beam	97·02	Rt. Ref	Beam =	96·50		
50E	300 psi		0	1·7990	1·7600	1·281	2·980	391·71		
			2	1·8008	1·7638	1·25	2 980			
0E	0 psi		0	1·8350	1·7960	1·240	2 990	391·64		
			2	1·8390	1·7981	1·24	2·990			
AP1 =	3·532	AP2 =	4·551	AP3 =	5·558	AP4 =	1·965			
	12:45	Left	Ref Beam	=	97·06 Rt	. Ref	Beam =	96·11		
0E	0 psi	~~12:14~~	0	1·8418	1·7975	1·240	2·990	391·42		
		1:09						391·49		

Left Ref Beam = 96·90 Rt Ref Beam = 95·91

Bench Mark = 36·65 (Tunnel Rail)

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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE # 5 | DATE 22-09-85 | SHEET 4 of 8

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (L)	2 (R)		PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
100E	65 psi	13:15	0	1.7915	1.7529	1.250	2.95	391.57		[illegible]
			2	1.7910	1.7530	1.253	2.95	391.57		
220E	145 psi	13:20	0	1.7173	1.6779	1.300	[illegible]			
			½	1.7173	1.6778	1.30	[illegible]			
			1	1.7172	1.6779	—	—			
			2	1.7172	1.6779	1.302	2.98			
240E	158 psi	13:23	0	1.7031	1.6625	1.305	2.98			
			½	1.7015	1.6615	1.310	-"-			
			1	1.7010	1.6604	1.310	-"-	391.80		Level [illegible]
			2½	1.7010	1.6603	1.310	-"-	391.80		[illegible]
			5	1.7005	1.6601	1.310	-"-	391.90		[illegible]
			10	1.6968	1.6573	1.310	-"-	392.00		for [illegible]
AP1 =	6.091	AP2 = 3	.950	AP3 =	6.29	AP4 =	1.728			test
		Left	Ref	Beam	= 97.02	Rt	Ref Beam = 96.05			
260E	16[illegible] psi	13:35	0	1.6838	1.6435	1.390	2.980			
			½	1.6835	1.6436	1.390	2.980	392.02		
			1	1.6835	1.6434	1.390	2.980			
			3	1.6832	1.6434	1.390	-"-			
			5	1.6805	1.6405	1.390	-"-	392.05		
			10	1.6797	[illegible]	1.39[illegible]	-"-			
AP1 =	6.7[illegible]	AP2 =	3.67	AP3 =	6.370	AP4 =	1.622	392.08		
		Left	Ref	Beam =	97.0[illegible]	Rt	Ref Beam = 96.[illegible]			

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PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

TEST PILE # 5 | DATE 22-09-85 | SHEET 5 of 8

(6)

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (L)	2 (R)	(E)	PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
[illegible]		13:47	0	1.6655	1.6261	1.650	2.98		-	
([illegible])			1/2	1.6642	1.6245	1.650	2.98			
			1	1.6641	1.6242	1.650	2.98			
			2 1/2	1.6641	1.6243	1.650	2.98	392.10		
			5	1.6640	1.6243	1.650	2.98	392.10		
			10	1.6620	1.6229	1.650	2.98	392.10		
AP1:	7.315	AP2 =	3.410	AP3 =	6.46	AP4:	1.545			AP1: 7.4[illegible]
	(7.355)	Lef	Ref. Beam =		97.10	Rt Ref	Beam =	96.13		
	(7 min)									
300 E	1960 psi	13.58	0	1.6460	1.60[illegible]	1.650	2.980			AP1:
(1.75 hrs)			1/2	1.6455	1.6056	1.650	2.9[illegible]			7.835
			1	1.6460	1.6056	1.65	2.980	392.12		7.870
			2 1/2	1.6439	1.6039	1.650	2.990			8.050
			5	1.6441	1.6039	1.650	2.992	392.14		8.090
			10	1.6422	1.6021	1.65	2.995			8.250
AP1:	8.250	AP2 =	3.045	AP3 =	6.580	AP4:	1.472	392.15		
		Lef.	Ref	Beam =	97.10	Rt. Ref.	Bea. =	96.11	Bench	mark: 37.30
320 E	2030 psi	14:11	0	1.6280	1.5870					
([illegible])			1/2	1.6270	1.5					
			1	1.6270	1.5860	1.970	2.995			8.730
			2 1/2	1.6270	1.5860	1.960	3.000			8.820
			5	1.6245	1.5823	1.950	3.000	392.19		9.016
			10	1.6245	1.5815	1.9[illegible]	3.0[illegible]	392.20		9.205
AP1:	9.2[illegible]	AP2:	2.61	AP3:	6.70	AP4:	1.280			
		Lef	Ref. Beam:		97.10	Rt	Ref. Beam = 96.	12		

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AXIAL PILE LOAD TEST

TEST PILE #5 | DATE 22-07-85 | SHEET 6 of 8

DATA SHEET

(7)

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (L)	2 (R)	(B)	PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
3406		14:20	0	1.6078	1.5631	1.950	3.000			AP1 = 9.660
(76 kn)	218 psi		1/2	1.6077	1.5632	1.950	3.00	392.23		= 9.750
			1	1.6077	1.5630	1.952	-"-			
			2½	1.6051	1.5609	"	"			= 9.9[illegible]
			5	1.6028	1.5578	"	"	392.26		3 min 10.0[illegible] 0.110
			10	1.6040	1.5578					=10.272 (5 min) =10.3[illegible] =10.470 (10 min)
AP1 =	10.470	AP2 =	2.060	AP3 =	6.84	AP4	= 1.090			
			Left Ref	Beam	= 97.10	Rt. Ref	Beam	= 96.12		
3606	2300 psi	14:35	0	1.5862	1.5410	2.055	3.050			AP1 = 11.100
[illegible]			1/2	1.5860	1.5402	2.060	-"-			= 11.240
			1	1.5860	1.5401	2.058	-"-	392.2[illegible]		11.—
			2½	1.5831	1.5380	2.060	-"-			11.555
			5	1.5810	1.5361	2.060	-"-	392.[illegible]		11.90
			10	1.5790	1.5350	2.055		392.30		12.8[illegible]
AP1 =	12.8[illegible]	AP2 =	1.200	AP3 =	6.940	AP4 =	0.870			
			Left	Ref Beam	= 97.11	Rt.	Ref. Beam	= 96.16		
38[illegible]	[illegible] psi	14:48	0	1.5640	1.5210	2.060	3.00			= 12.950
[illegible]			1/2	1.5621	1.5195	2.062	"	392.37		13.20[illegible]
			1	1.5625	1.5190	2.062	"			13.300
			2½	1.5585	1.5150	2.065	"			13.78[illegible]
			5	1.5570	1.5138	2.065		392.38		14.256
			10	1.5582	1.5150	2.065		392.32		[illegible]
AP1		AP[illegible]	9.55 -0.11	AP3	6.970	AP4	0.650			
			Left	Ref Beam	97.07	Rt	Ref. Beam	96.13		
	AP2 =	9.55	-0.11			B. M	ark =	37.30		
	AP1	[illegible]								

[illegible]

AP1 = 93.91
AP2 = 44.8[illegible]
AP3 = 91.52
AP4 = 42.85

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UBC IN-SITU TESTING

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AXIAL PILE LOAD TEST

TEST PILE #5	DATE 22-09-85	SHEET 7 of 8

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (L)	2 (R)	(C)	PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
[illegible]		15.01	0	1.5365	1.4930	2.165	3.0[illegible]		-	16.[illegible]
[illegible]			1/2	1.5360	1.491			392.40		16.37
			1	1.536	1.4[illegible]	2.17				
			2 1/2	1.531	1.4872	2.17				17.[illegible]
			5	1.529	1.485	2.17	[illegible]	392.40		21.[illegible]
Could not pump fast enough to keep up 400 E load because										Pile [illegible]
anchor pile #1 coming off the ground!										Fail!
300E		15.06	0	1.573	1.520			392.30		
	1900 psi		2	1.5740	1.5290		AP1 =	93.00		
		Lt. Ref. Beam 97.09				Rt. Ref. Beam		96.11		
			3	1.5742	1.529[illegible]		AP2 =	44.01		
							AP3 =	91.68		
							AP4 =	42.65		
200E	1250 p	15.11	0	1.6385	1.5901			392.14		
			2	1.6395	1.5907	2.14	AP1 =	93.10		
150E		15.17	0	1.7100	1.6578	2.100	3.00	392.00		
			2	1.7110	1.6591	2.096	[illegible]			
							AP1 =	93.23		
0E			0	1.7935	1.7380	1.955	3.00	391.80		
							AP1 =	93.48		
		reset loading*					AP1 =	93.47		
							AP[illegible]	44.39		
							AP[illegible]	91.80		
							AP[illegible]	42.75		

* [illegible] readings [illegible] appeared [illegible]

TPS = 391.79

Bench Mark = 37.30

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AXIAL PILE LOAD TEST

TEST PILE #5 | DATE [illegible]-85 | SHEET 8 of 8

6 OCT. 1985
#5

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS (in) 1 LEFT	(in) 2 RIGHT	MID-POINT*	PILE TIP*	LEVEL READINGS (cm) PILE	PILE SETTLE.	REMARKS
0	TP#5			1.2350"	1.8905"	0.380 mm	5.14 mm	382.5 cm	-	
	AP 1			7.459	mm			—		
	[illegible]			7.215	mm			35.4 cm		
	[illegible]			13.66	mm			27.2 cm		[illegible]
	[illegible]			9.442	mm			33.6 cm		[illegible]
	Ref. Beam Left							37.9 cm		[illegible]
	Ref. Beam Right							90.3 cm		[illegible]
[illegible]		11:35	0	1.779	1.8322	0.76	6.38			* [illegible]
[illegible]			$2\frac{1}{2}$	1.779	1.8322	0.76	6.48	382.6		[illegible]
			5	1.7808	1.8379	0.76	6.48	382.6		[illegible]
			10	1.779	1.8370	0.75	6.475	382.6		[illegible]
	AP1 = 6.672		AP2 =	8.205	AP3 =	13.00	AP4 =	9.481		
			Ref Beam Left	= 87.9	Ref. B.R.	= 90.4				
20 TE		11:45	0	1.7151	1.7740	1.38	8.56	382.8		[illegible]
1280 psi			$2\frac{1}{2}$	1.7145	1.7728	1.38	8.61	382.8		
			5	1.714	[illegible]	[illegible]	8.50	382.8		
			10	1.713	1.7703	1.700	8.50	382.8		
	AP1 =	5.757	AP2 =	7.97	AP3 =	12.505	AP4 =	9.338		
			Ref Beam Left	= 87.9			Ref Beam	Rt =	90.4	
30 TE		11:53	0	1.6423	1.7015	2.15	10.10	383.0		
1920 psi			$2\frac{1}{2}$	1.6405	1.6982	2.16	10.10	383.0		
			5	1.639	1.6970	2.16	10.10	383.0		
			10	1.6385	1.6952	2.15	10.0[illegible]	383.[illegible]		
	AP1 =	4.625	AP2 =	7.31	AP3 =	12.075	AP4 =	9.109		
			Ref.	Beam Left	= 87.9		Ref. Beam	Rt =	90.0	

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AXIAL PILE LOAD TEST

TEST PILE #5 | DATE 6-10-85 | SHEET 1 of 6

DATA SHEET

2

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 (LEFT)	2 (RIGHT)	MID-POINT	PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
400E		12:09	0	1.5640	1.6198	3.15	12.00	383.2		
(2500psi)			2½	1.5575	1.6115	3.15	12.[illegible]	383.2		
			5	1.5531	1.6075	3.19	12.36	383.2		
			10	1.5518	1.6053	3.22	12.43	383.2		
	AP1 =	3.600	AP2 =	6.32	AP3 =	11.580	AP4 =	8.732		
			Ref	Beam Left : 87.	8	Ref.	Beam Right	: 90	.2	
420E		12:21	0	1.5372	1.5910	3.40	13.270	383.3		
			2½	1.5331	1.5850	3.356	13.260	383.3		
			5	1.5305	1.5827	3.35	13.26	383.3		[illegible]
			10	1.5275	1.5781	3.40	13.35	383.3		[illegible]
	AP1 =	3.36[illegible]	AP2 =	5.88	AP3 =	11.38	AP4 =	8.62[illegible]		[illegible]
			Rf. Be.	Left : 87	8		Rf. Beam	Right :	90.2	
200E		12.31	0	1.657	1.707	2.155	12.62	383.0		
0E		12.33	0	1.808	1.855	1.15	11.52	382.6		ZERO [illegible]
			5	1.816	1.863	1.07	11.45	382.6		25 min
		12.56	25	1.8215	1.8660	1.09	11.69			[illegible]
	AP1	: 6.950	AP2 =	7.595	AP3 =	12.205	AP4 = 8.8	90		ZERO [illegible]
200E		12.59	0	1.7045	1.7550	2.05	12.36	382.9		
			2½	1.7011	1.7531	2.06	12.34			
420E		13.05	0	1.5385	1.5888	3.21	14.30	383.35		
(2780psi)			2½	1.5325	1.5827	3.21	14.28	383.35		
			5	1.5275	1.5775	3.32	13.8[illegible]			
			10	1.5220	1.5725	3.32	13.88	383.5		
	AP1	= 3.26	AP2 =	5.71	AP3 =	10.850	AP4 =	8.420		
				Rf. Bea	Left : 87.8		Ref. Bea.	Rt. =	90.1	

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AXIAL PILE LOAD TEST

TEST PILE #5 | DATE 6-10-85 | SHEET 2 of 6

DATA SHEET

Bench Mark 6.75

3)

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	EXTENSOMETER READINGS 1 LEFT	2 RIGHT	MID-POINT (RIGHT)	PILE TIP	LEVEL READINGS PILE	PILE SETTLE.	REMARKS
440 E		13:16	0	1.5065	1.5580	3.42	14.23	383.4	-	
(2250 psi)			2½	1.4976	1.5486	3.56	14.30			
			5	1.4901	1.5415	3.56	14.26	383.35		
			10	1.4809	1.5326	3.65	14.17			
	AP1 =	2.999	AP2 =	5.740	AP3 =	10.671	AP4 =	8.349		
			Left Beam =		87.8		Right Beam = 90.2			
460 E		13:28	0	1.4640	1.5160	3.71	14.15	383.5		
(2980 psi)			2½	1.4342	1.4860	3.71	14.40			
			5	1.4100	1.4615	3.70	14.39	383.65		
			10	1.3430	1.3960	3.70	14.20	383.80		
	AP1	= 2.71	AP2 =	5.75	AP3 =	10.40	AP4 = 8.232			
			Left	Beam =	87.9	Rt Beam =		90.2		
480 E		13:38	0	1.2850	1.350	3.90	14.28	384.0		Constant
			2½	1.110	1.155	3.88	14.36	384.3		Pumping
475 E			3	0.935	0.975	3.86	14.35	384.6	(3 min)	
470 E			4	0.750	0.790	3.88	14.38	384.1	(385.5)	
460 E			5	0.53	0.580	3.94	14.96	386.2		STOP
										(FAILED)
400 E			0	0.38	0.445	3.68	14.42	38		
350 E			0	0.436	0.493	3.04	14.05	386.1		
250 E			0	0.505	0.562	2.48	13.64	385.9		[illegible]
										AP1 = -
100 E		13:50	0	0.585	0.641	1.79	12.92	385.7		AP2 = 35.4
0			0	0.686	0.734	1.29	12.88	385.5		AP3 = 89.2

AP1 = 6.640 AP2 = 7.59 AP3 = 11.34 AP4 = 8.470 AP4 = 33.55

0 E 10 min 0.695 0.744 1.17 12.52

Left Ref Co. = 87.9 Rt Ref 90.3

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AXIAL PILE LOAD TEST

TEST PILE #5	DATE 6-10-85	SHEET 3 of 6

ANCHOR PILE LOAD CELLS

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	Load cell #1	#2	#3	#4	#5		REMARKS
				[illegible]	[illegible] 2	[illegible] 5	3	4		
				7.38	[illegible]	[illegible]	[illegible]	[illegible]		
				Channel						
				1	2	3	4	5	6	
1000			0	[illegible]	[illegible]	[illegible]	0	0		
			5	1182	[illegible]	[illegible]	0	0		
			10	1176	[illegible]	[illegible]	0	0		
[illegible]			0	[illegible]	[illegible]	1960	93	[illegible]		
			5	1968	[illegible]	[illegible]	94	[illegible]		
			10	[illegible]	[illegible]	1960	98	17		
[illegible]			0	2570	798	2474	381	0		
			5	[illegible]	[illegible]	[illegible]	388	0		
			10	[illegible]	803	2457	383	0		
[illegible]			0	[illegible]	1272	[illegible]	[illegible]	5		
		2.	55	2972	1287	2815	854	0		
		5	~~10~~	3070	1306	2875	853	0		
			10	3002	[illegible]	2807	805	0		
[illegible]			~~0~~							
			~~5~~							
			~~10~~							

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AXIAL PILE LOAD TEST

PILE # 5 | DATE [illegible] | SHEET 4 of 6

ANCHOR PILE LOAD CELLS

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.	[illegible] 1	[illegible]	[illegible]	[illegible]	[illegible]		REMARKS
[illegible]			0	[illegible]	[illegible]	2872	984	0		
			2.5	3038	1279	2884	[illegible]	0		
			10	[illegible]	1337	2870	[illegible]	0		
7008			0	1932	432	2016	0	-48		[illegible]
08			0	0	0	-64	+24	-48		
	12:58									
08			0	0	0	0	0	0		
2008			0	1516	550	1666	341	0		
			2.5	1516	560	1650	342	0		
4008			0	2804	1472	2660	1238	62		
			2.5	2778	1456	2650	1259	67		
			5	2770	1509	2636	1254	58		
				2780	1472	2664	1266	64		
[illegible]			0	2870	1531	2734	1360	66		
			2.5	2897	1553	2730	1340	62		
			5	2868	1574	2725	1347	57		
			10	2866	1601	2715	1343	[illegible]		

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AXIAL PILE LOAD TEST

PILE #5 | DATE | SHEET 5 of 6

ANCHOR PILE LOAD CELLS

DATA SHEET

LOAD	DIAL PRESSURE P.S.I.	TIME HRS.	ΔT MINS.							REMARKS
6608			0	2897	[illegible]	[illegible]	[illegible]	56		
			2.5	2936	1670	2766	1458	70		
			5	2941	1662	2782	[illegible]	66		
			10	2946	[illegible]	2780	[illegible]	78		
[illegible]			0	3058	[illegible]	2818	[illegible]	78		
			2.5	3060	1774	2852	1533	86		
			~~x~~4	3013	1665	2794	1472	72		
			5	2897	1534	2708	[illegible]	74		
9008			0	2597	1397	2590	1156	64		[illegible]
3008			0	2176	998	[illegible]	694	16		
2008			0	1610	604	1791	216	60		
1008			0	1012	318	1210	~~144~~ 0	0		
0			0	0	0	0	0	0		

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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.

AXIAL PILE LOAD TEST

PILE #5	DATE	SHEET 6 of 6

APPENDIX IV

LATERAL PILE LOAD TESTS

LOAD CELL JACK PRES GAUGE

DATA SHEET

LOAD με	DIAL PRESSURE ~~P.S.I.~~ lbs	TIME ~~HRS.~~ mins	ΔT MINS.	PILE NO. 3 N-S	PILE NO. 3 E-W	PILE NO. 5 N-S	PILE NO. 5 E-W	PILE NO. 4 N-S + E-W	PILE NO. 2 N-S + E-W	REMARKS (PILE NO. 1) (eg. reference beam deflection)
				DIAL GAUGE READINGS (LOADING #5 and #3 N-S)						
1000	5000 = ZERO LOAD			Teclock 2"x.001"	Mitutoyo 10x.01mm	Teclock 2"x.001"	Compac 10mmx.01mm	Teclock 10x.01mm	Teclock 10mm×0.01mm	Pile casing
INITIAL RDG		1.35pm		0.115	4.74	0.219	~~4.90~~	3.43/2.77	4.07/3.05	Teclock 10x.01mm 7.49
		1.55pm					5.17			
		2.41pm		0.115	4.68	0.218	5.17	3.43/2.77	4.10/3.02	Piano Wire 74.3cm
1432	10000	2.43pm	0	0.154	4.69	0.268	5.13			Pile 1 = 7.50 mm
			2m	0.158		0.268		3.42/2.77	4.08/3.01	
			5	0.160	4.74	0.269	5.12	3.41/2.77	4.08/3.01	
			10	0.162	4.74	0.270	5.11			
			18	0.163	4.74	0.271	5.11			
1845	15,000	3.08	0	0.223	4.74	0.328	5.11	3.29/2.77	3.97/3.00	
UNLOAD TO ZERO LOAD										
984		3.11		0.148	4.74	0.242	5.11	3.29/2.77	3.97/3.01	Pile 1 = 7.47 mm
										Piano Wire 74.3cm
	Fill up jack with hydraulic oil									
	DIRECTION OF PILE MOVEMENT WITH DGR									
				↑DGR	↑DGR	↑DGR	↑DGR	↑DGR	↑DGR	Pile 1 ↑DGR = South
				= South	= East	= North	= East	= North	= North	
								↑DGR	↑DGR	
								= West	= East	
	RE ZERO DGR									
		3.21pm		0.144	4.74	0.239	4.98	3.23/3.16	4.00/2.89	Pile 1 = 7.45
				0						
		4.11pm		0.145	4.66	0.241	4.98	~~3.26/3.01~~	4.07/2.95	Pile 1 = 7.43
								3.64/2.00		Piano Wire 74.3cm

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LATERAL PILE LOAD TEST

PILES 1+3 | DATE | SHEET 1 OF 4

DATA SHEET

LOAD $\mu\varepsilon$	DIAL PRESSURE ~~P.S.I.~~ lbs	TIME ~~HRS.~~ mins	ΔT MINS.	DIAL GAUGE READINGS (LOADING #5 and #3 N-S) PILE No. 3 N-S	PILE NO. 3 E-W	PILE NO. 5 N-S	PILE NO. 5 E-W	PILE NO. 4 N-S + E-W	PILE NO. 2 N-S + E-W	REMARKS (PILE NO. 1) (eg. reference beam deflection)
1816	15,000	4.14 pm	0	0.227	4.66	0.326	4.98			-
			2	0.230	4.66	0.327	4.98	3.63/2.00	3.97/2.94	Pile 1 = 7.47
			5	0.231	4.66	0.329	4.98			
			10	0.233	4.66	0.330	4.98	3.63/2.00	3.97/2.94	
			15	0.233		0.331				
2287	20,000	4.30	0	0.308	4.76	0.393	4.98	3.56/2.03		
			2	0.318		0.398			3.70/2.94	Pile 1 = 7.48
			5	0.320	4.81	0.400	5.02	3.55/2.03	3.69/2.94	Piano Wire = 74.3 cm
2888	25,000	4.37	0	0.449	5.12	0.499	5.32	3.38/2.04	3.15/2.94	
			2	0.468	5.33	0.511	5.38			Pile 1 = 7.43
			5	0.472	5.38	0.513	5.39			
			10	0.478	5.38	0.518	5.41	3.33/2.04	3.04/2.96	Pile 1 = 7.41
			15	0.482	5.40	0.520	5.42			
3468	30,000	4.53	0	0.630	5.77	0.620	5.70	3.22/2.04		Pile 1 = 7.18
			2	0.633	5.75	0.669	6.05		2.21/2.99	Pile 1 = 7.10
			5	0.694	6.05	0.659	5.87	3.13/2.04	2.14/3.00	Piano Wire = 74.3 cm
										Pile 1 = 7.09
4219	35,000	5.02	0	0.907	6.86	0.796	6.49			
		5.04	2	0.948	7.07	0.814	6.54	2.99/2.04	1.25/3.02	Pile 1 = 6.67
			7	0.973	7.13	0.825	6.59			
			10	0.991	7.13	0.834	6.60	2.94/2.05	1.13/3.19	Pile 1 = 6.63
			15	1.000	7.13	0.840	6.61			Piano Wire = 74.3 cm

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LATERAL PILE LOAD TEST

PILES 1 + 3 | DATE | SHEET 1 of 4

DATA SHEET

LOAD $\mu\varepsilon$	DIAL PRESSURE ~~P.S.I.~~ lbs	TIME ~~HRS.~~ mins	ΔT MINS.	DIAL GAUGE READINGS (LOADING #5 and #3 N-S) PILE NO. 3 N-S	PILE NO. 3 E-W	PILE NO. 5 N-S	PILE NO. 5 E-W	PILE NO. 4 N-S + E-W	PILE NO. 2 N-S + E-W	REMARKS (PILE NO. 1) (eg. reference beam deflection)
1778	40,000	5.20	0	1.280	7.78	0.949	6.93			-
			2	1.216	8.02	0.961	6.73	2.81/2.04	0.13/3.22	Pile 1 = 6.25
			5	1.276	8.20	0.982	7.00	2.66/2.04	0.0/3.21	Pile 1 = 6.15
									RESET TO	
									7.76/3.41	
	RAN OUT OF JACK TRAVEL									
	PUT IN 2 SCREW JACKS TO TAKE UP LOAD									
	BUT SCREW JACKS POPPED OUT !									
	LOAD DROPPED TO ZERO.									
ZERO	LOAD	5.37	0	0.275	6.40	0.367	4.83	3.34/2.47	0.58/2.91	Pile 1 = 6.21
983									"hang up"	
		5.42	5	0.274	6.40	0.360	4.83	3.34/2.47	NG/2.70	Pile 1 = 6.17

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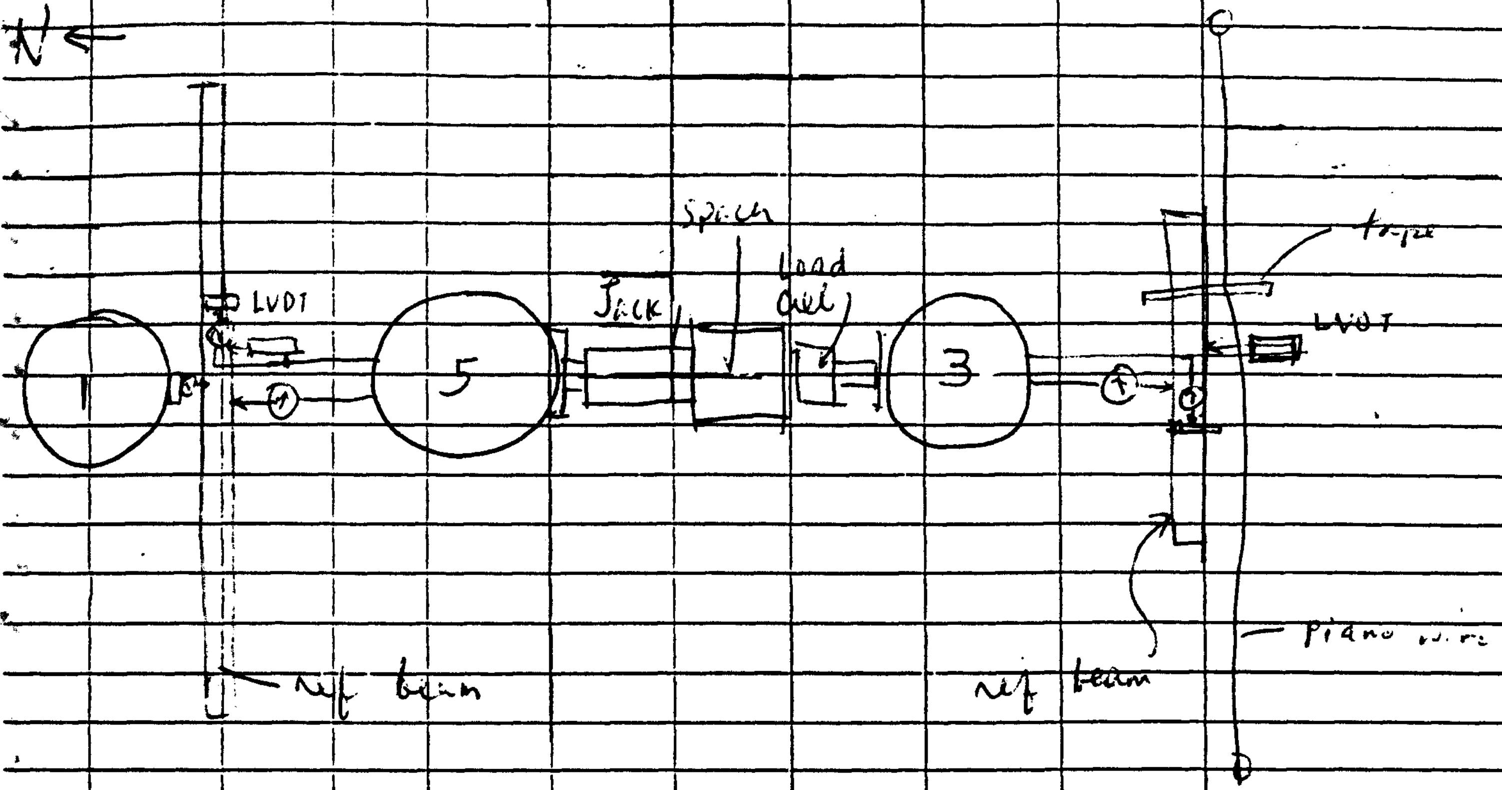
LATERAL PILE LOAD TEST

PILES 1+3 | DATE | SHEET 1 OF 4

DATA SHEET

LOAD $\mu\varepsilon$	DIAL PRESSURE ~~p.s.i.~~ lbs	TIME ~~HRS.~~ mins	ΔT MINS.	DIAL GAUGE READINGS (LOADING #5 and #3 N-S)						REMARKS (PILE NO. 1) (eg. reference beam deflection)
				PILE NO. 3 N-S	PILE NO. 3 E-W	PILE NO. 5 N-S	PILE NO. 5 E-W	PILE NO. 4 N-S + E-W	PILE NO. 2 N-S + E-W	



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UBC IN-SITU TESTING

PILE RESEARCH PROJECT, QUEENSBOROUGH, B.C.		
LATERAL PILE LOAD TEST		
PILES 1+3	DATE	SHEET 1 OF 4

APPENDIX V

DYNAMIC AXIAL CAPACITY PREDICTION METHODS

A. Engineering News Record (ENR) Dynamic Formula

$$R = \frac{2 \bullet W_H \bullet H}{S + 0.1}$$

- see Section 6.5.2 for explanation of symbols.

i) Pile no. 5 original driving

- end of driving: set = 1.09 inches

W_H = 6.2 kips

H = 7 feet

$$R = \frac{(2) \bullet (6.2)(7)}{(1.09 + 0.1)} = 73 \text{ kips}$$

factor of safety = 6

$\therefore$ ultimate capacity = 478 kips

= 1944 kN

ii) Pile no. 5 restrike data

- beginning of restrike: set = 0.5 inches

W_H = 3.5 kips

H = 10 feet

$$R = \frac{2 \bullet (3.5)(10)}{(0.5 + 0.1)} = 117 \text{ kips}$$

factor of safety = 6

$\therefore$ ultimate capacity = 700 kips

= 3114 kN

WEAP86: WAVE EQUATION ANALYSIS OF PILE FOUNDATIONS
1986, VERSION 1.004

HAMMER MODEL OF: UBC   MADE BY: UBC

ELEMENT	WEIGHT (KN)	STIFFNESS (KN/MM)	COEFF. OF RESTITUTION	D-NL. (MM)	CAP DAMPG (KN/M/S)
1	13.790				
2	13.790	99852.3	1.000	3.0480	
CAP/RAM	3.560	483.4	.500	3.0000	33.0

HAMMER OPTIONS:

HAMMER NO.	FUEL SETTG.	STROKE OPT.	HAMMER TYPE	DAMPNG-HAMR
311	1	0	3	2

HAMMER PERFORMANCE DATA

RAM WEIGHT (KN)	RAM LENGTH (MM)	MAX STROKE (M)	STROKE (M)	EFFICIENCY
27.58	1220.00	2.13	2.13	.700

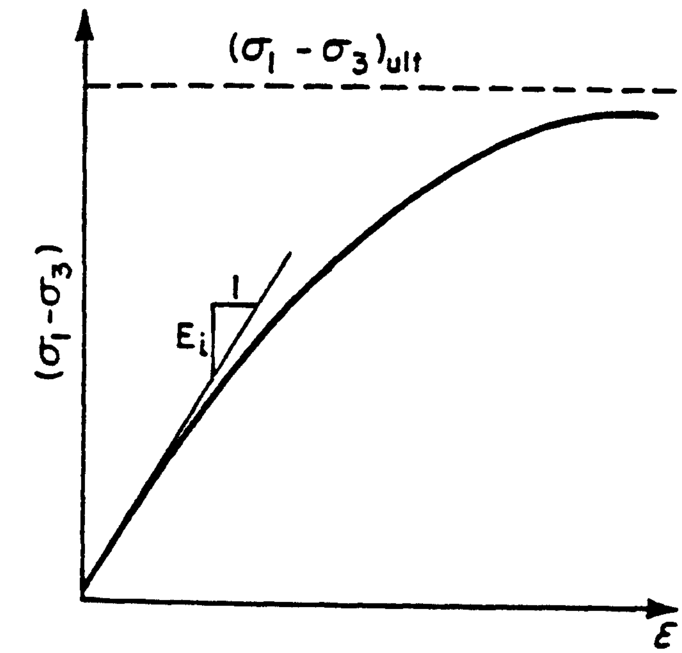
RTD PRESS. (KPA)	ACT PRESS. (KPA)	EFF. AREA (CM2)	IMPACT VEL. (M/S)
.00	.00	.00	5.41

HAMMER CUSHION	AREA (CM2)	E-MODULUS (MPA)	THICKNESS (MM)	STIFFNESS (KN/MM)
	900.00	205.6	38.100	485.8

APPENDIX VI

LATDMT.UBC PROGRAMS LISTING

DERIVATION OF ϵ_{50} RELATIONSHIP FROM HYPERBOLIC STRESS-STRAIN RELATIONSHIP



$$\sigma_d = \sigma_1 - \sigma_3 = \frac{\epsilon}{\frac{1}{E_i} \quad \frac{\epsilon}{(\sigma_f/R_f)}}$$ Duncan and Chang (1970)

$$\frac{\sigma_f}{2} = \frac{\epsilon_{50}}{\frac{1}{E_i} \quad \frac{\epsilon_{50}}{(\sigma_f/R_f)}}$$

$$\frac{\sigma_f}{2} = \frac{\epsilon_{50}}{\frac{1}{E_i} + \frac{R_f \epsilon_{50}}{\sigma_f}}$$

$$\frac{\sigma_f}{2E_i} + \frac{R_f \epsilon_{50}}{2} = \epsilon_{50}$$

$$\frac{\sigma_f}{2E_i} = \epsilon_{50} \left(1 - \frac{R_f}{2}\right)$$

$$\therefore \quad \epsilon_{50} = \frac{\sigma_f}{2E_i} \left(\frac{1}{2 - R_f}\right) \qquad (7.5)$$

```
C*****************************************************************
C         UBC IN-SITU TESTING GROUP                              *
C*****************************************************************
C*****************************************************************
C*** PROGRAM FOR CALCULATON OF PU-YC FROM DILATOMETER RESULTS ****
C*****************************************************************
C                                                                *
C           WRITTEN BY MICHAEL P. DAVIES, OCTOBER, 1986 UPON     *
C           ADAPTATION FROM PROGRAM BY TSO TIEN-HSING            *
C           FORMAL DEVELOPMENT OF ALL EQUATIONS USED CAN BE      *
C           FOUND IN M.A.Sc. THESIS BY PROGRAM AUTHOR            *
C                                                                *
C*****************************************************************
C                                                                *
C           RUNNING INSTRUCTIONS:                                *
C                                                                *
C           RUN *FORTRANVS SCARDS=PU-YC.UBC SPUNCH=-OUT          *
C           RUN -OUT 1=DMT DATA FILE 5=FACTOR FILE 6=-DMT ECHO   *
C           7=(PU,DEPTH) 8=(YC,DEPTH)                            *
C                                                                *
C*****************************************************************
          REAL KO,M,ID,KD,MU
          COMMON /L1/X,D,ESV,PHI,SNPH
          COMMON /L4/PU,KO
          COMMON /L5/CU
          COMMON /L6/ED
          COMMON /L7/EPS50,EIC,FACC
          COMMON /L8/U
          COMMON /L15/PU1,PU2
          COMMON /L22/PO,P1,RK1,RK2,EIS1,EIS2,SF
C         ******************************************************
C         *   FEC, FES=CORRECTION FACTORS TO DILATOMETER MODULII *
C         ******************************************************
          READ(5,*)FEC,FES
          PRINT *,'FEC= ',FEC,'FES= ',FES
C         ******************************************************
C         *  READ IN OUTPUT FROM DMT.UBC (DMT DATA REDUCING      *
C         *  PROGRAM)                                            *
C         ******************************************************
  100     READ(1,*,END=111)X,PO,P1,ED,U,ID,GAMMA,ESV,KD,OCR
     *        ,PC,KO,CU,PHI,M
          WRITE(6,299)X,PO,P1,ED,U,ID,GAMMA,ESV,KD,OCR
     *        ,PC,KO,CU,PHI,M
C         ***********************************************
C         *  D=PILE DIAMETER IN CM                      *
C         *  NOTE: ENSURE THAT THIS IS CHANGED APPROPRIATELY *
C         *  FOR THE PILES BEING USED                   *
C         ***********************************************
          D=91.5
C         *******************************************
C         *****  CHANGE UNIT TO KN-CM, XX=DEPTH,M  ****
C         *****  PU=KPA-M, YC=CM                   ****
C         *******************************************
          FA=0.01035986
          PO=PO*FA
          P1=P1*FA
          ED=ED*FA
          U=U*FA
```

```
            ESV=ESV*FA
            PC=PC*FA
            CU=CU*FA
            M=M*FA
            XX=X
            X=X*100.
            RPH=PHI
C           **********************************
C           *  PHI IS INCREASED BY 5 DEGREES *
C           **********************************
            PHI=PHI+5.
C           ***************************************
C           ***** PU CALCULATION *****************
C           ***************************************
            IF(RPH.EQ.O.)THEN
               CALL PUCLAY
               ELSE
               CALL PUSAND
            END IF
            PPU=PU
C           ****************************************
C           **         YC CALCULATIONS        *****
C           ****************************************
            RF=O.8
            MU=O.4
            RD=O.6
            IF(RPH.EQ.O.)THEN
               SF=2.*CU
               EIC=FEC*38.2*(P1-PO)
               EPS50=(SF/EIC)/(2.-RF)
               YC=14.2*EPS50*(D**O.5)
               ELSE
               SF=(2*SNPH/(1.-SNPH))*ESV
               EIS=FES*38.2*(P1-PO)
               EPS50=(SF/EIS)/(2.-RF)
               YC=2.5*EPS50*D
            END IF
            WRITE(7,99)PPU,XX
            WRITE(8,99)YC,XX
  99        FORMAT(2F15.4)
 199        FORMAT(4F15.4)
 299        FORMAT(7F7.2,F7.3,7F7.2)
            GO TO 100
 111        STOP
            END
C********************************************
            SUBROUTINE PUCLAY
            REAL NP,J
            COMMON /L1/X,D,ESV
            COMMON /L4/PU
            COMMON /L5/CU
            J=O.5
C           ********************************************************
C           * NOTE THAT J SHOULD BE REDUCED TO O.25 FOR STIFF CLAYS *
C           ********************************************************
            NP=(3.+(ESV))/CU+(J*X/D)
            IF(NP.GT.9.)THEN
               NP=9.
```

```
            END IF
            PU=NP*CU*D
            RETURN
            END
C***************************************
           SUBROUTINE PUSAND
           REAL KA,KP
           COMMON /L1/X,D,ESV,PHI,SNPH
           COMMON /L4/PU,RKO
           COMMON /L15/PU1,PU2
           PH=PHI/180.*3.141593
           A=PH/2.
           B=3.141593/4.+A
C          ********************************************
C          *                                          *
C          *  CHECK IF KO FROM DILATOMETER OUTPUT = RKO *
C          *                                          *
C          ********************************************
           RKO=0.5
           TNPH=TAN(PH)
           TNB=TAN(B)
           SNPH=SIN(PH)
           KA=(1-SNPH)/(1+SNPH)
           KP=1./KA
           R=D*(KP-KA)+X*KP*TNPH*TNB
           T=(KP**3.)+2.*RKO*(KP**2.)*TNPH+TNPH-KA
           PU1=ESV*R
           PU2=ESV*D*T
           IF(PU1.GT.PU2)THEN
              PU=PU2
              ELSE
              PU=PU1
           END IF
           DD=4.*D
           IF(X.LE.DD)THEN
              PU=PU/DD*X
           END IF
           RETURN
           END
```

```
C**************************************************************
C***********      UBC IN-SITU TESTING GROUP      **************
C       PROGRAM FOR THE CALCULATION OF P=Y CURVES USING       *
C       A CUBIC PARABOLA FROM PU=YC DATA DERIVED FROM         *
C                   DILATOMETER RESULTS                       *
C**************************************************************
C                                                             *
C       WRITTEN BY MICHAEL P. DAVIES, DECEMBER 1986           *
C       UPON ADAPTATION FROM TSO TIEN-HSING                   *
C                                                             *
C**************************************************************
              DIMENSION Y(50),P(50)
              COMMON/L1/X,PU,YC,K,RK
              COMMON/L2/Y,P,D
C             ***********************************************
C             * LATPILE INPUT DATA. CAN CHANGE `PRINT' VALUES    *
C             * HERE OR LATER FROM P-Y.UBC OUTPUT BEFORE RUNNING *
C             * LATPILE                                          *
C             ***********************************************
              PRINT *,'LATERAL LOAD PILE(DATA-NEW)'
              PRINT *,'5,3'
              PRINT *,'1100 ,0, 91.5,100,0.05,30,1,1,0,0,0,1'
C             **************************
C             *   N=NO. OF P-Y CURVES   *
C             **************************
              N=10
              PRINT *,'9,12'
C             **********************
C             *                    *
C             *    RK=0. FOR CLAYS  *
C             **********************
C             **************************************
C             *     X=0(USING RK=0,PU=0.001,YC=0.01) *
C             *     X(CM), PU(KN/SQ.CM-CM), YC(CM)   *
C             *     RK(KN/CU.CM), RK=0(CLAY)         *
C             *                                      *
C             **************************************
222           READ(5,*,END=88)X,PU,YC
              X=X*100.
              D=91.5
              CALL PARAB
              WRITE(6,99)X
              WRITE(6,99)((Y(I),P(I)),I=1,K)
99            FORMAT(2F20.4)
              GO TO 222
C             ***************************************
              *  MORE LATPILE DATA TO CHANGE HERE OR  *
              *  FROM P-Y.UBC OUTPUT                  *
              ***************************************
88            PRINT *,'1'
              PRINT *,'11134180000.,30'
              STOP
              END
C*********************************************************
              SUBROUTINE PARAB
              DIMENSION Y(50),P(50)
              COMMON/L1/X,PU,YC,K
              COMMON/L2/Y,P,D
```

```
      WY=8.*YC
      YY=0.
      DO 100 I=1,100
         K=I
         A=(YY/YC)**(1./3.)
         PP=0.5*PU*A
         Y(K)=YY
         P(K)=PP
         IF(YY.GT.WY)THEN
            GO TO 5
            ELSE
            YY=YY+WY/10.
         END IF
100   CONTINUE
5     YY=YY+D
      Y(K)=YY
      P(K)=PU
      RETURN
      END
```