INSTALLATION OF BORED PILES IN STIFF CLAYS : AN EXPERIMENTAL STUDY OF LOCAL CHANGES IN SOIL CONDITIONS

by

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SUMMARY

This thesis attempts to identify some of the fundamental mechanisms involved in the modification of soil properties and conditions due to pile installation. Pile behaviour is dependent on soil conditions during pile loading, and at that stage the soil properties existing prior to pile installation have experienced some important changes.

Pile design methods, pile installation and the effects of pile installation are critically reviewed with a summary of all factors controlling the behaviour of bored piles.

Different experimental techniques are used to study local changes in soil conditions.

Both laboratory and field studies of moisture content and undrained shear strength variation due to the contact of soil with fresh concrete are presented. A fall cone test was used to study the variation in strength.

A method involving the use of a radioactive labelling procedure to study the migration of water from fresh concrete to the surrounding soil is proposed, with a detailed description of the testing procedures. Results obtained when using such a technique are presented for specimens of London Clay in contact with a concrete mix containing labelled water.

The pore water pressure regime in the fresh concrete is studied and some new facts concerning the boundary conditions at the interface between the fresh concrete and the surrounding soil emerge, leading to the study of the variation in pore water pressure at different distances inside clay specimens in contact with fresh concrete.

A field experiment, involving the design of instrumentation to study the horizontal stress regime variation during and after the installation of a model bored pile, is presented.

Suggestions for further research are made, including laboratory model studies, with the presentation of the proposed apparatus.

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To my wife Neila

To my sons Leandro and Marcio

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1.1. - INTRODUCTION

1.1.1. - RESEARCH OBJECTIVE

The technical and economical advantages that can be obtained from the use of large bored piles have resulted in their widespread adoption.

The allowable load is chosen basically from considerations of the ultimate load and settlement under working load.

The available methods for the prediction of pile bearing capacity are based on pile load test results, relating the pile performance with some soil properties existing prior to pile installation.

The information about soil conditions is usually obtained in a site investigation programme. As a result a certain "design strength profile" is established. With the obtained data, a simple model is adopted, using the evaluated "design strength profile" and correction factors derived from case records of pile load test results. Sometimes a direct correlation between "in situ" tests and unit strengths is used.

The first problem that arises from the present approach to solve the problem of prediction of pile capacity is that of a realistic evaluation of existing soil properties. The proposed methods and coefficients are strongly related to the methods of obtaining the soil properties (and the "design strength profile" adopted), the type of pile testing programme and the definition of the ultimate or limit load in the pile load test.

Pile behaviour is dependent on soil conditions during pile loading, and at that stage the soil properties existing prior to pile installation have experienced some important changes, not just due to pile installation (the most influential single factor) but due to pile loading itself. The effects of pile installation on the soil properties and stress conditions is a subject of major relevance for the understanding of pile behaviour.

This thesis attempts to identify some of the fundamental mechanisms involved in the modification of soil properties and conditions due to pile installation.

1.1.2. - THESIS ORGANIZATION

The thesis is presented in eight chapters and three appendices.

The first chapter presents a brief review of pile design methods and pile installation procedures, followed by a comprehensive "state-of-the art" presentation of the effects of pile installation on the surrounding soil.

Chapters two to eight cover the experimental programme undertaken to study local changes in soil conditions.

In Chapter 2 the laboratory studies of moisture content and undrained shear strength variation due to the contact of clay specimens with fresh concrete are presented. To study the variation in strength a fall cone test was used.

Results of similar studies performed on test specimens obtained from a site in London, 12 m depth, adjacent to bored piles, 4 months after pile installation are presented in Chapter 3.

A method involving the use of a radioactive labelling procedure to study the migration of water from fresh concrete to the surrounding soil is proposed in Chapter 4. A detailed description of the testing procedures, together with results is presented.

In Chapter 5 the pore water pressure regime in the fresh concrete is studied and some new facts concerning the boundary conditions at the interface between the fresh concrete and the surrounding soil emerge, leading to the study of the variation in pore water pressure at different distances inside clay specimens in contact with fresh concrete (Chapter 6).

A field experiment, involving the design of instrumentation to study the horizontal stress regime variation during the installation of a bored pile is presented in Chapter 7. The New Pressure Cell design, calibration, installation and results, with the associated ground instrumentation are presented.

At the end of each chapter the appropriate conclusions and recommendations are drawn.

Chapter 8 presents the main conclusions of the whole research and general recommendations for future work are included.

Appendix 1 presents a review on the existing knowledge on concrete formwork pressure.

Among the suggestions for further research, an experimental approach using a special triaxial to test model piles cast in beds of soil under any stress conditions was included. The equipment was designed and built, the photographs are presented in Appendix 2.

Appendix 3 includes the papers produced during the period of the present research.

1.2. - PILE DESIGN

There are many forms and conditions of instability that can affect a deep foundation and no set of simple calculation procedures can be expected to cover all the variety of problems that can occur. The basic design considerations can be presented as:

(i) - the foundation must possess sufficient safety against overall failure;

(ii) - the foundation should not undergo excessive displacement under working loads.

The nominal design load (D_L) should not exceed a specified fraction of the ultimate load (Q_o), so that, at any time:

$$D_L = Q_o \,/\, F_o$$

where:

F_o = overall safety factor

At the same time, the displacement of the foundation under working load should not exceed specified limits set by usage requirements and structural tolerances of the supported structure (for details see Burland, Broms and De Mello, 1977).

The ultimate load (Q_o) is the load that can cause either the structural failure of the foundation itself or the bearing capacity failure of the soil. In large bored piles the ultimate load is usually determined from considerations of bearing capacity failure of the soil. It is important to point out that a large number of problems with bored piles have been associated with defects of the pile itself (Baker and Khan, 1971, Thorburn and Thorburn, 1977, L.C.P.C., 1978, Reese, 1978). Due to the usual high load carried and the problems involved in testing a significant number of piles on a site (using load tests), there is an increasing interest and development of integrity testing of bored piles (Preiss, 1971, Weltman, 1974, Preiss et al, 1978, Vold and Hope, 1978, Heorne, Stokoe and Reese, 1981).

Basic definitions:

a) Design Load: according to Weltman (1980), "the Design Load is that which may be applied safely to an isolated pile unaffected by any considerations of downdrag. It is thus the ultimate bearing capacity divided by a factor of safety", (i.e., is related just to the pile, without considering the structure).

$$\text{Design Load } (D_L) = Q_o \,/\, F_o$$

b) Applied Load: the Applied Load is the theoretical calculated load to be imposed on a pile by the works which it is required to support.

$$\text{Applied Load } (A_L) = Q_{al} \text{ (Allowable Load)}$$

c) Allowable Load: the Allowable Load (Q_{al}) is the load which may be safely applied to a pile after reducing its Design Load to take into account such considerations as downdrag, pile spacing, overall bearing capacity of the ground below the pile toe and allowable settlement (Weltman, 1980).

$$\text{Allowable Load } (Q_{al}) = \text{Design Load x Reduction Factor (RF)}$$

From the basic equation $Q_o = Q_s + Q_b$ Skempton, 1966, suggested as a first approximation, for large bored piles in clay, considering the mobilization aspects, Figure 1.1b (Q_s= shaft; Q_b= base):

for base diameter $B \leq 2$ m

$$Q_{al} = Q_o / 2 \quad \text{(no base enlargement)}$$

$$Q_{al} = Q_o / 2.5 \quad \text{(with base enlargement)}$$

or

$$Q_{al} = (Q_s / 1.5) + (Q_b / 3)$$

For B greater than 2 m he suggested that the working load (allowable load) should be evaluated from settlement calculation.

Different proposals have been suggested for factors of safety.

From:

$$Q_o = Q_s + Q_b$$

$$\left.\begin{array}{l} Q_{al} = Q_o / F_o \quad \text{or} \\ Q_{al} = Q_s / F_s + Q_b / F_b \end{array}\right\} \quad \text{(the least value)}$$

a) F_o (overall ultimate)

2 to 2.5 (London Clay)*

3 (certain codes of practice) (NB 51 - ABNT)

b) F_b (base)

3 to 3.5 (London Clay)*

c) F_s (shaft)

1 to 1.5 (London Clay)*

The choice of F_o is a function of the uncertainties usually involved in a foundation problem, but F_b and F_s also reflect the mobilization aspects. The use of both conditions imposes an overall factor of safety and an additional assurance that the factor of safety in end bearing is not exceeded when the shaft friction is fully mobilized (when F_s is equal to one).

* Observation: references on London Clay by Whitaker and Cooke, 1966; Burland, Butler and Dunican, 1966a; 1966b; District Surveyors Association, 1973 and Burland and Cooke, 1974.

1.2.1. - COMPUTATION OF THE ULTIMATE LOAD

The basic problem of computation of the ultimate load for a deep foundation can be formulated as follows: a cylindrical shaft of diameter B is placed to depth L inside a soil mass. A static, vertical, compressive, central load Q is applied at the top and increased until a shear failure in the soil is produced. The problem is to determine the ultimate load that this foundation can support (Q_o). (fig. 1.1 A - pg 68)

The first problem that arises is that even if the properties of the soil are known before the pile is installed, the bearing soil above and below the foundation base is always disturbed.

For design purposes the ultimate load is conventionally separated into two components, the shaft or skin load (Q_s) and the base load (Q_b), which are superimposed as follows:

$$Q_o = Q_b + Q_s = q_b A_b + f_s A_s \qquad \text{Eq 1.1}$$

where:

A_b = bearing area of the foundation base

A_s = bearing area of the shaft

q_b = unit base resistance

f_s = unit shaft resistance

The unit resistances depend on a number of factors related to the soil conditions, installation and geometry of the pile and loading conditions.

The bearing areas of the foundation base (A_b) and shaft (A_s) are, by definition, the exposed areas of the pile body in contact with soil. In the case of large bored piles with enlarged bases, the side friction of the base is always disregarded in computing skin friction.

There are many practical ways of solving the problem of the determination of the unit resistances or the overall pile resistance. A detailed survey can be found in Milititsky, 1980.

1.2.2. - PILE RESISTANCE

1.2.2.1. - GENERAL METHODS

In this section the methods using basic soil properties to evaluate the pile resistance will be briefly presented and commented on. For the computation of skin resistance two basic approaches have been proposed : the total and effective stress methods . The base resistance is usually evaluated using total stress.

The major problem of any method using soil properties is the evaluation of these properties. All methods are based on pile load tests performed in certain areas with some knowledge of the soil conditions, after which adjustments or reduction factors are proposed. The classic English school (Skempton, 1959; Whitaker and Cooke, 1966; Skempton, 1966; Burland, 1973; Burland and Cooke, 1974) proposes values to "calibrate" the soil strength, due to a considerable knowledge of London Clay properties and the effects of sample size and method of testing on the strength values. The American school (Reese and Hudson, 1968; O'Neill and Reese, 1970; 1972; Reese and O'Neill, 1971; Engeling and Reese, 1974) until 1976 adopted a different approach: sometimes a crude correlation was used to evaluate the soil strength and no attempt was made to correct that strength, but the installation aspect was used to justify adjusting the soil strength to the results obtained from pile load tests. More recently (Wright and Reese, 1977; 1979) they have adopted the limit state design method.

TOTAL STRESS ANALYSIS - BASE RESISTANCE (Q_b)

There are a number of theoretical solutions for the problem of the ultimate bearing capacity of deep foundations. The available solutions can be classified into two main groups :

a) - solutions using only strength parameters (conventional or classic plasticity solutions), and

b) - solutions which use both strength and deformation parameters (cavity expansion solutions).

All the solutions can be presented in the form (total stress analysis, "$\phi_u = 0$") :

$$Q_b = q_b A_b \qquad \text{Eq 1.2}$$

$$q_b = s_c d_c N_c S_u + \sigma_o \qquad \text{Eq 1.3}$$

or

$$q_b = N_c^C S_u + \sigma_o \qquad \text{Eq 1.4}$$

where:

q_b = unit base resistance

s_c = shape factor with respect to cohesion

d_c = depth factor with respect to cohesion

N_c = reference bearing capacity factor

S_u = undrained shear strength

A_b = bearing area of the foundation base

σ_o = total overburden stress at foundation level

ϕ = angle of internal friction of the soil

N_c^c = comprehensive bearing capacity factor

Usually the contribution of the total overburden stress at the base level σ_o is assumed to be balanced by the stresses generated by the pile weight.

a) Classical plasticity solutions

Figure 1.2 presents the solutions by classical plasticity, grouped according to the failure pattern which they assume. In the first group, which considers that the soil above the foundation level would act as a surcharge only, there is the solution by Reissner, 1924. According to the approach suggested, a value of $N_c = 6.2$ for circular foundations and a σo stress equal to the vertical total stress must be used. The second group consider failure lines reverting to the shaft, mobilizing the soil strength above the foundation level. Using this pattern, Meyerhof, 1951, obtained for circular areas in clays, in the case of no interface (soil versus foundation) resistance $N_c^c = 9.34$ and in the case of an interface resistance (C_a) equal to the soil strength ($C_a = S_u$) $N_c^c = 9.74$.

Skempton, 1951, reviewed the theoretical solutions available at the time (including the "cavity expansion" proposed by Gibson, 1950) and comparing it with field evidence concluded that the chart of Figure 1.3 could be used in the design of foundations in clay.

b)"Cavity expansion" solutions

These solutions consider that failure under a pile end is analogous to the expansion of a spherical or cylindrical cavity. Bishop et al, 1945, obtained for the expansion of a spherical cavity in connection with their studies of metal a solution for the limiting pressure at which continuous expansion of a spherical cavity occurs in a uniform isotropic material. The expression for N_c obtained was :

$$N_c = 4/3 \ (\log_e G/S_u + 1 \ ) \qquad \text{Eq 1.5}$$

where :

G = shear modulus of the material

Gibson, 1950, modified this solution, in order better to represent conditions under the base of a deep foundation. The following expression was proposed :

$$q_b = (\ 4/3 \ (\ \log_e G/S_u + 1 \ ) + 1 \ ) \ S_u + \sigma_{v,o} \qquad \text{Eq 1.6}$$

where:

$$N_c = 4/3 \ (\ \log_e G/S_u + 1) + 1 \qquad \text{Eq 1.7}$$

Vesic, 1975, using a different pattern, proposed the expression:

$$N_c = 4/3 \ (\ \log_e Ir + 1 \ ) + (\ \frac{2 + \pi}{2} \ ) \qquad \text{Eq 1.8}$$

where:

Ir = "rigidity" index

$$= \frac{Eu}{2 \ (\ 1 + \nu u \ ) S_u} = \frac{G}{S_u} \qquad \text{Eq 1.9}$$

Eu = Young's modulus (undrained)

νu = Poisson ratio

and also suggested that the characteristic overburden stress should be taken as the octahedral normal stress at the pile level :

$$\sigma_o = \sigma_{oct,o} = \frac{1 + 2 \ Ko}{3} \ \sigma_{v,o} \qquad \text{Eq 1.10}$$

as a way of including the effect of the initial state of stress on bearing capacity (this suggestion was made based on experimental evidence with model piles in sand, by Al Awkati, 1975).

Table 1.1 presents the bearing capacity factors obtained by the various expressions of the cavity expansion approach.

Values of G/S_u back calculated from measurements of structures founded on normally consolidated and lightly overconsolidated clays with a plasticity index less then 30 are reported by D'Appolonia et al, 1971, to be about 400. The values were found to be lower (about 200) for clays with a plasticity index greater than 30 and substantially lower (about 40) for a single case history involving a slightly organic plastic clay. Laboratory data were presented that suggest that G/S_u decreases with increasing overconsolidation ratio (OCR). The value of G/S_u at an OCR of 8 was found to be about one third the value of G/S_u for normally consolidated specimens.

One must realize that the application of the analysis for expansion of a cavity in an ideal elastic-plastic medium to the end bearing capacity of piles is just an approximation to the real problem, and certainly estimates of G/S_u from field measurements of initial settlements under surface loadings should be applied with caution to the cavity expansion problem. Nevertheless, according to Esrig and Kirby, 1979, these analyses provide the following insights to the range in values for N_c :

a) for a given soil, N_c for overconsolidated clays is expected to be lower than N_c for normally consolidated clays;

b) for a given OCR, N_c for a clay of low plasticity is expected to be higher than N_c for a clay of high plasticity.

There has been a great deal of argument on the adequacy of the available bearing capacity solutions on the basis of whether they succeed or fail to predict the failure of actual foundations. Lambe, 1965; De Mello, 1969 and Wright and Reese, 1979, have compiled data from the literature and load tests which suggest that N_c can vary within extremely wide limits.

For any practical purpose, the value of $N_c = 9$ for deep foundations is widely used. The major source of discrepancy is probably the assessment of the proper value of undrained shear strength, which is very sensitive to factors such as test conditions, soil brittleness, sample disturbance, sample size and others.

TOTAL STRESS ANALYSIS - SHAFT RESISTANCE (Q_s)

The evaluation of the shaft (or skin) resistance in a large bored pile is generally similar to that used to analyse the resistance to sliding of a rigid body in contact with soil :

$$Q_s = A_s f_s \qquad \text{Eq 1.11}$$

where:

A_s = bearing area of the pile shaft

f_s = unit shaft resistance

Using total stress analysis, in a saturated clay the unit shaft resistance is expressed by the following relationship :

$$f_s = \alpha S_u \qquad \text{Eq 1.12}$$

where:

α = non dimensional adhesion factor, the shear strength reduction factor

Table 1.2 presents a comprehensive review of adhesion factors available in the literature, as a results of comparison between results of pile load tests and undrained shear strength average values. It is important to point out that there are various approaches in the

I_r	$N_c = \log_e I_r + 1$ (Gibson & Anderson 1961)	$N_c = \frac{4}{3}(\log_e I_r + 1)$ (Bishop et al, 1945)	$N_c = \frac{4}{3}(\log_e I_r + 1)$ (Gibson, 1950)	$N_c = \frac{4}{3}(\log_e I_r + 1) + \frac{2+\pi}{2}$ (Vesic, 1975)
10	3.30	4.40	5.40	7.00
20	4.00	5.30	6.30	7.90
40	4.70	6.30	7.30	8.80
60	5.10	6.80	7.80	9.40
80	5.40	7.20	8.20	9.80
100	5.60	7.50	8.50	10.00
200	6.30	8.40	9.40	11.00
300	6.70	8.90	9.90	11.50
400	7.00	9.30	10.30	11.90
500	7.20	9.60	10.60	12.20
800	7.70	9.90	10.90	12.80

Table 1.1 - N_c values for $\phi = 0$ condition, as produced by different expressions of the 'cavity expansion approach'.

evaluation of S_u, effective bearing areas of pile shafts, installation techniques and pile load test programmes in that survey.

Skempton, 1959, after assessing the results of 34 pile loading tests from 10 sites around London (Figure 1.4), recommended that the value of alpha would be 0.3 to 0.6 in this material (London Clay) for bored piles, depending on the installation aspects. He also concluded that a good working value of alpha was 0.45 (with a limit of 2000 lb/sq ft (96 kN/m^2) for f_s). Skempton's recommendation is widely used all over the world.

Looking carefully at this assessment of the alpha value one must realize that none of the piles was instrumented, and in some cases he had to estimate the ultimate load. The shear strengths used were from tests on 1 ½ inch (38 mm) diameter specimens. Knowing the ultimate load he calculated the expected end bearing, thus obtaining the shaft load. He could then calculate the average adhesion and obtain an average alpha. There is also an additional factor: the installation techniques at the time were different from those presently used, and the biggest load was 650 tonnes. Only 3 piles reached bearing values greater than 300 tonnes (Figure 1.4b).

EFFECTIVE STRESS ANALYSIS

The effective stress approach, assuming no excess pore water pressure, was first used for the analysis of the long term shaft friction (Eide et al, 1961) or negative skin friction of piles in clays (Zeevaert, 1959; Johannessen and Bjerrum, 1965). Various forms of the effective stress analysis are given by Chandler, 1966; 1968; Hanna, 1971; Clark and Meyerhof, 1972; 1973; Bjerrum, 1973; Burland, 1973; Meyerhof, 1976; Janbu, 1976 ; Parry and Swain , 1976 ; 1977a and 1977b ;

REFERENCE	TYPE OF CLAY AND LOCATION	MEAN SHEAR STRENGTH (kN/m^2)	TEST	CONSTRUCTION METHOD	ALPHA FACTOR		REMARKS
					RANGE	MEAN	
Harris, 1951	Beaumont Clay, Texas	110	Triaxial	Auger with casing		0.6	
Meyerhof and Murdock, 1953	London Clay	165	Triaxial and unconf	Dry (W/C=0.2) (W/C=0.4)	0.4-0.6 0.2-0.4		
Golder and Leonard, 1954	London Clay	125	Triaxial and plate	-	0.5-0.6*		*By Skempton, 1959
Du Bose, 1955, 1956	Layered sandy clay, Texas	55	Unconfined	Dry		1.0	
Skempton, 1959	London Clay	-	Various*	Various (Dry)	0.3-0.6	0.45	*Mainly triaxial
Mohan and Jain, 1961	Expansive Clay, India	150	Unconfined	Hand auger, dry		0.3	
Woodward, Lundgreen and Boitano, 1961	Layered silty and sandy clay, California	110	Unconfined	Bucket with casing		0.5	
Mohan and Chandra, 1961	Expansive Clay, India	80-170			0.4-0.6	0.48	
Frishman and Fleming, 1962	London Clay	150		Power auger	0.2-0.35		
Fleming and Salter, 1962	London Clay	160		Power auger		0.6*	*Assumed
Turner, 1962	Silty clay, Texas	40-55			0.48-1.2	0.75	
Deb and Chandra, 1964	Expansive Clay, India	110-220				0.5	
Patterson and Urie, 1964	Keilor Clay, Australia Kerang Clay, Australia	110-160 125-190			0.2-0.36 0.28-0.46	0.3 0.4	
Whitaker and Cooke, 1966	London Clay	130	Triaxial	Caldwel		0.44	
Burland, Butler and Dunican, 1966	London Clay	150	Triaxial	Power auger	0.4-0.45		
Taylor, 1966	Glacial till, Grimsby, UK	140		Dry	0.4-0.6		

TABLE 1.2. - REVIEW OF THE REFERENCES ON ADHESION FACTORS OF BORED PILES IN CLAY AVAILABLE IN THE LITERATURE

REFERENCE	TYPE OF CLAY AND LOCATION	MEAN SHEAR STRENGTH (kN/m^2)	TEST	CONSTRUCTION METHOD	ALPHA FACTOR		REMARKS
					RANGE	MEAN	
Matick and Kozicki, 1967	Glacial Till, Nova Scotia	110-270	Triaxial and unconf.	Dry		0.64*	*Failure not reached
Komornik and Wiseman, 1967	Sandy fat clay, Israel	160	Vane	Benoto		0.17*	*Failure not reached
Reese and Hudson, 1968	Stiff clay, Austin, Texas	260	Unconfined	Dry (mechanical aug)		0.55	
U.S. Army Engineers, 1968	Expansive clay and Shale	110-300	Triaxial	Mech.auger,dry	0.2-0.6* 0.3-0.5^{+}		*Clay $^{+}$Shale
Bhanot, 1968	Silty clay, Edmont, Canada Glacial till, Edmont, Can	110 330	Triaxial Triaxial	Mech.auger,dry Mech.auger,dry		0.43 0.65	
Watt,Kurfurst and Zernan,1969	Clay, Canada Silty clay, Canada	100 65	Shear box and torvane	Mech.auger		0.3 1.0	
Sowa, 1970	Stiff clay, Ontario	110		Percussion,casing		0.31	
Adams and Radhakrishna, 1970	Stiff clay, Ontario	95-280	Triaxial	Dry		0.4	
O'Neill and Reese, 1970	Houston, Texas	110	Triaxial	Dry and slurry	0.24-0.56		
Barker and Reese, 1970	Houston, Texas	110	Triaxial	Slurry		0.6	
Touma and Reese, 1972	Houston, Texas	100	Triaxial	Dry and Slurry	0.3-0.9		
Holtz and Baker, 1972	Chicago Silty Clay	110-300	Unconfined	Dry		0.75	
Wooley and Reese, 1974	Houston, Texas	120	Triaxial	Slurry		0.18	
Engeling and Reese, 1974	Texas and Puerto Rico			Dry with casing and slurry	0.59-0.7		
Zolkov, 1975	Israel	200-350	Vane		0.3-0.5		
Jelinek,Koreck and Stocker, 1977	Munich Clay, Germany	200-260	Various	Dry	0.27-0.76		

TABLE 1.2 - Continuation

REFERENCE	TYPE OF CLAY AND LOCATION	MEAN SHEAR STRENGTH (kN/m^2)	TEST	CONSTRUCTION METHOD	ALPHA FACTOR		REMARKS
					RANGE	MEAN	
Tomlinson, 1977	Lias Clay, Lutterworth,UK	200			0.1-0.3*		*Very sensitive to water
	Glacial Till,Glasgow, UK	160				0.85	
	Lemore, USA	96			0.49-0.52		
Fearenside and Cooke, 1978	London Clay	140	Triaxial	Bentonite	0.26-0.46		
				Dry	0.23-0.43		
Weltman and Healy, 1978	Glacial Till, Hull, UK	110		Dry		0.61	
	Glacial Till, Hartlepool	130		Dry	0.46-0.65		
Ismael and Klyn, 1978	Silty Clay, Ontario	60-120	Triaxial	Dry		0.64	
Hodgson, 1979	London Clay	150	Triax.and*	Dry		0.7	*Plate load
Lopes, 1979	London Clay	170-220	Triaxial	Various	0.3-0.52	0.38	
Ottaviani and Marchetti, 1979	Sandy silty clay, Rome	80-180	Triax.and*	Slurry	0.55-0.87	0.69	*Vane
Pearce and Brassow, 1980	Beaumont Clay, Texas	190	Triaxial	Slurry		0.6	
Poulos, 1980	Stiff clay, Australia	160-260	Dutch cone	Injected		0.2	
Promboni and Brenner, 1981	Bangkok Clay	20-60	Dutch cone	Benoto		0.8	
		170-200	Dutch cone	Benoto		0.5	
O'Riordan, 1982	Woolwich and Reading beds, London	200-280	Triaxial(?) SPT	Dry	0.36-0.4		

TABLE 1.2 - Continuation

Vesic, 1977; Flaate and Selnes, 1977 and others. In this method the skin friction is related to the coefficient of friction between the pile and the soil and the normal effective stress.

Assuming drained conditions, the following equations have been used:

$$f_s = S' + \sigma'_r \tan \delta' \qquad \text{Eq 1.13}$$

or

$$f_s = S' + \sigma'_h \tan \delta' \qquad \text{Eq 1.14}$$

where:

f_s = effective unit shaft resistance

S' = unit adhesion between the soil and the pile, which is independent of the normal stress

σ'_r = effective radial stress

σ'_h = effective horizontal stress

δ' = effective angle of friction between the pile and the soil

If the interface strength is assumed equal to the soil strength (e.g. the failure is not pile to soil, but soil to soil):

$$f_s = c' + \sigma'_h \tan \phi' \qquad \text{Eq 1.15}$$

or

$$f_s = c' + K\,\sigma'_v \tan \phi' \qquad \text{Eq 1.16}$$

if $c'=0$

$$f_s = K\,\sigma'_v \tan \phi' \qquad \text{Eq 1.17}$$

where:

c' = effective cohesion intercept

ϕ' = effective angle of friction of the surrounding soil

K = coefficient of earth pressure on pile shaft (σ'_h / σ'_v)

Chandler, 1966, proposed the use of effective atress analysis in the interpretation of short-term skin friction measured in pile tests for bored piles in London Clay, reported by Whitaker and Cooke, 1966. Assuming drained conditions, Chandler used the average of K'_o obtained by Skempton, 1961 and Bishop, Webb and Lewin, 1965 and remoulded soil strength parameters.

Burland, Butler and Dunican, 1966b, based on the analysis of pile load tests reported by Skempton, 1959; Whitaker and Cooke, 1966 and Burland, Butler and Dunican, 1966a, argued that the assumption of full re-establishment of the K'_o condition gave an "upper bound" to the measured resistance and that a "lower bound" could be defined using a $K' = 1.25$.

Chandler, 1968, extended the effective stress analysis to piles in normally consolidated clays. Introducing Jaky's, 1944, simplified expression for assessing K'_o, the author proposed the expression:

$$f_s = c' + (1 - \sin \phi')\, \sigma'_v \tan \phi' \qquad \text{Eq 1.18}$$

or, as $c' = 0$ in these soils :

$$f_s = (1 - \sin \phi')\, \sigma'_v \tan \phi' \qquad \text{Eq 1.19}$$

Burland, 1973, compiled considerable data supporting the effective stress analysis and presented a method of calculating shaft friction from effective stress analysis on the shaft, making the assumptions:

(i) before loading, the excess pore water pressures set up during installation are completely dissipated,

(ii) because the zone of major distortion around the shaft is relatively thin, loading takes place under drained conditions during a pile load test (this is the more controversial assumption),

(iii) as a result of remoulding during installation the soil has no effective cohesion.

Hence the shaft friction f_s at any point is given by:

$$f_s = \sigma'_h \tan \delta' \qquad \text{Eq 1.20}$$

(iv) the further simplifying assumption is made that σ'_h is proportional to the vertical effective overburden pressure σ'_v, so:

$$\sigma'_h = K \sigma'_v \qquad \text{Eq 1.21}$$

Thus, from Eqs 1.20 and 1.21:

$$f_s = K \sigma'_v \tan \delta' \qquad \text{Eq 1.22}$$

If $K \tan \delta'$ is denoted by β , then :

$$\beta = f_s / \sigma'_v = K \tan \delta' \qquad \text{Eq 1.23}$$

β is thus similar to the empirical alpha factor, but is related to the fundamental effective stress parameters K and δ'. Average values of β can be obtained empirically from pile load tests, provided sufficient time has elapsed after installation and the tests are carried out sufficiently slowly. It is also possible to make estimates of K and δ' and thus obtain β . In the case of piles in soft clays it is assumed that failure takes place in the narrow zone of remoulded soil close to the pile surface, so that $\delta' = \phi'_d$, where ϕ'_d is the remoulded drained angle of friction of the soil.

For piles in stiff clay, the major problem of the effective stress method is the estimation or determination of the value of K . For a heavily overconsolidated clay such as London Clay, K can vary significantly from the surface to depth (Figure 1.5 and also Chapter 7,Figure 7.1). Burland took the mean curve of K_o versus depth for London Clay from the results of Skempton, 1961 and Bishop, Webb and

Lewin, 1965, as had Chandler, 1966, before him.

Thus the total shaft resistance is given by:

$$Q_s = \pi \; d \sum_{o}^{L} \sigma'_v \; K_o \; \tan \; \delta' \; \Delta L \qquad \text{Eq 1.24}$$

and the mean shaft friction is:

$$f_s = \frac{Q_s}{\pi d \; L} = \frac{1}{L} \sum_{o}^{L} \; \sigma'_v \; K_o \; \tan \; \delta' \; \Delta L \qquad \text{Eq 1.25}$$

Figure 1. 6 identifies the variables used by Burland.

Lines indicating Eq 1.25 and $\beta = 0.8$ are shown in Figure 1.6 using $\delta' = \phi'$, Burland estimated the effect of 10 m of overburden above the clay, and that effect is also plotted on the same figure.

For bored piles Burland recommended that β = 0.8 can be used as a conservative preliminary design value for London Clay, because it is doubtful if the initial at-rest horizontal stresses can be fully re established at the pile surface (thus it would be expected that the curve represented by Eq 1.25 would form an upper limit for values of f_s).

In 3 publications Parry and Swain, 1976; 1977a; 1977b, presented a comprehensive discussion of failure conditions of a soil element adjacent to the pile shaft, suggesting new expressions for the skin friction and a link between the total and effective stress approaches. They assumed that the vertical and horizontal effective stresses σ'_v and σ'_h may vary during the loading, and that δ' may be a function of the stress ratio K (σ'_h / σ'_v). They showed that the first failure of an element of soil close to the pile would, in general, occur on planes inclined to the pile axis. The maximum angle of friction that could be mobilized parallel to the pile axis was shown to be sensitive to the inclination of the failure planes, and this inclination was shown to be dependent on the assumption made about the variation of the stress ratio during loading. It was concluded that if K is greater than unity, δ' must be less than ϕ', and they showed that if K approached the limiting passive state δ' tended to zero.

Hui, 1977 ; 1979 , in two technical notes presented some discussion related to the failure conditions near the pile shaft.

Meyerhof, 1976, suggested that the following equation can be used to estimate the shaft bearing capacity of bored piles in soft clays (normally consolidated clays):

$$f_s = \beta \sigma'_v < S_u \qquad \text{Eq 1.26}$$

where:

$$\beta = K_o \tan \phi'$$

$$K_o = 1 - \sin \phi'$$

and for stiff overconsolidated clays, he suggests for the assessment of K_o, if not known, the empirical equation:

$$K_o = (1 - \sin \phi') \sqrt{OCR} \qquad \text{Eq 1.27}$$

then

$$f_s = (1 - \sin \phi') \sqrt{OCR} \tan \phi' \sigma'_v \qquad \text{Eq 1.28}$$

When the over-consolidation ratio is not known he suggests the use of :

$$f_s = S_u \tan \phi' \qquad \text{Eq 1.29}$$

as a good approximation.

The design of bored piles in overconsolidated clays using effective stress was also advocated by Searle, 1980, and further analysis of the published case histories was presented.

In the recent years, a large number of references referring to the use of effective stress analysis and critical state soil mechanics for driven piles can be found in the literature (Esrig, Kirby and Bea, 1977; Vijayvergiya, 1977; Esrig and Kirby, 1977; Kirby and Wroth, 1977; Flaate and Selnes, 1977; Parry, 1978; Kirby and Esrig, 1979; Esrig and Kirby, 1979; Randolph and Wroth, 1981).

The effects of pile installation are a dominant feature in the modern approach and as differences between driven and bored piles are substantial, the presentation of such references is not relevant to the present work.

1.2.2.2. - PILE RESISTANCE FROM "IN SITU" SOIL TESTING

The determination of the base and shaft resistance of piles using the methods described under the title GENERAL METHODS requires detailed knowledge of strength and general properties of soil strata involved in transmitting pile loads. Sometimes to take the necessary number of samples from the soil mass and perform the appropriate laboratory tests to determine soil characteristics needed for design is not possible or justified, due to cost problems, available time, lack of laboratory facilities and non uniform soil situations. Under these circumstances it is necessary to estimate the unit resistances directly from "in situ" soil testing, such as the Standard Penetration Test (SPT), static cone and pressuremeter.

The correlations obtained by comparing the results of these small "in situ" tests with the large bored pile behaviour must be "calibrated" for each soil condition, taking into account the difference between the values measured in the tests and the large scale strengths relevant to the pile behaviour.

Sometimes the "in situ" tests are used to obtain soil strength and those values are introduced in the classical methods. One problem that must be treated carefully is the fact that usually the bearing capacity formulae have been developed using the experience obtained from pile load tests performed on soils with the properties measured using a certain technique; when using a correlation this must be taken into account, e.g. the correlation must be related to the technique, if the same coefficients are to be used (the correlation must be "calibrated" according the testing technique used to establish the design method).

In some special cases, like uncommon soil conditions, special installation techniques, high load levels or special requirements for the foundation's behaviour, the data for the pile design are obtained from plate bearing tests in boreholes and pile load tests.

The aspects of the use of "in situ" testing for design of large bored piles were reviewed (Milititsky, 1980) with comments about relevance and shortcomings.

1.3 - BORED PILE INSTALLATION

The construction of a bored pile basically consists of drilling a hole in the ground to a predetermined depth and then concreting the hole, with or without reinforcing steel. Shaft construction may proceed by a number of different methods (Woodward, Gardner and Greer, 1972; Weltman and Little, 1977; Reese and Wright, 1977; LCPC, 1978; Reese, 1978). The simplest case occurs in stiff clays in holes to limited depth, where the excavation remains stable and permits free-fall concrete placement. More difficult situations arise whenever casing or bentonite slurry must be used to prevent caving or water flow into the excavation.

Under the broad description of bored piles is included the wide range from the small diameter percussion bored pile to diaphragm wall elements. A detailed description of installation procedures of all piles is beyond the scope of this presentation. Only the basic aspects of bored pile installation will be presented. As a simplification for ease of presentation, the bored pile installation has been divided in 3 stages: (i) boring process, (ii) support used, and (iii)concreting.

1.3.1 - BORING PROCESS

Basically a distinction can be made on the basis of the method used to excavate, either by percussive or rotary means (considering just the mechanical methods).

PERCUSSION

The pile is excavated using percussive means: a hole is formed by repeatedly dropping a tool into the ground and bringing the spoil to the surface.

In the construction of small diameter percussion bored piles it is usual to employ light tripod rigs. The piles are formed using cable operated tools designed for use with the tripod rig. In clay soils a "clay cutter" consisting of a cylinder with a cutting edge at its base is used. The borehole is advanced by a constant repetition of dropping the tool into the base of the bore and emptying the contents alongside the pile excavation. This technique is still used due to the versatility of the equipment and low cost involved.

Systems used for the construction of large diameter percussion bored piles are variable depending on the specialist contractor. Generally skid, crane or tripod mounted rigs are used to excavate the spoil from inside a heavy casing. A hammer grab (cable operated clamshell) may be used to advance the borehole. Insertion of casing may involve oscillating the casing to facilitate penetration and extraction.

ROTARY

In the rotary bored piles, boring is carried out by a rotary rig. A short flight auger or drilling bucket is used. The pile bore is made by lowering the rotating boring tool into the ground. When enough soil moves into the flights, the tool is withdrawn, the spoil removed and the tool is reinserted into the hole for another pass.

Another method of excavation of small diameter piles is the use of continuous flight augers.

Rotary drilling is the most used method all over the world in modern piling.

HAND EXCAVATION

In some places of the world the labour intense method of hand excavation is still used (Far East and Latin America).

UNDER-REAMING

In certain circumstances piles are constructed with under-reams (or with bells). The under-reams or enlarged bases are cut by mechanical means using different under-ream tools. Sometimes hand excavation is used to form the enlargement.

1.3.2. - SUPPORT

In some types of ground or layer sequence, as the hole is advanced, the walls cave in and create a larger hole than intended. Sometimes the soil caves in and fills up the bottom as fast as it can be removed. To excavate to the desired depth under these circumstances, some form of support must be used to prevent collapse of the hole.

For easy of presentation, the support methods can be presented in the following categories:

- NO SUPPORT: the simplest situation. The soil is strong and stable enough in order to permit the excavation to remain stable without any form of support.

- CASING: in some instances, casing may have to be advanced with the hole to keep it open during excavation (many problems have been associated with the use of casing, and withdrawal during concreting the pile, O'Neill and Reese, 1970).

- SLURRY (Bentonite suspension): when large bored piles are constructed through unstable ground to reach soil layers at great depth, a bentonite suspension may be used to stabilise the borehole. When the hole instability is due to seepage forces the stabilization effect can be achieved by any fluid filling the hole (some contractors use just natural ground water mixed with cuttings). A good practice is to use a controlled mix of bentonite. The thixotropic properties of a bentonite suspension, combined with its ability to form an impermeable filter cake on permeable borehole walls under a positive hydrostatic head, render it suitable to stabilise boreholes in granular soils and to retain in suspension fine particles which would otherwise settle in the base of the pile hole.

- CONTINUOUS FLIGHT AUGER: a hollow continuous flight auger is used in the boring operation and mantained during concreting as a support technique.

- COMBINED METHOD: in some circumstances both casing and slurry may be used as complementary technique to stabilize the hole.

1.3.3. - CONCRETING

The technique to be used to fill the drilled hole with concrete is dependent on the support used. Concreting can be accomplished in the following ways: free falling, under slurry and grout-intruded piles.

FREE FALL - When no support is used or a casing is necessary, filling the drilled hole with concrete is fairly straight forward. Usually high slump mixes are used to avoid problems of integrity.

SLURRY - Concreting under slurry requires that concrete must be placed from below and flow upward, displacing the fluid. Usually a tremie is used to place the concrete in the bottom of the hole. The concrete discharge is always kept well below the interface between concrete and slurry to prevent mixing. Before concreting checks must be made to assess the quality of the slurry. Where necessary contaminated bentonite is replaced by a fresh suspension before concreting. The displaced bentonite is usually returned to a desanding plant prior to its re-use.

GROUT-INTRUDED - in this type of pile the borehole is formed by means of a hollow- stem continuous flight auger. The auger remains in the hole until concreting commences, and therefore no lining tubes are required. The special mix is usually pumped under pressure through the hollow body of the auger.

TREMIE - under difficult circumstances , such as presence of water, the concreting can be achieved successfully by the use of a tremie.

1.4. - EFFECTS OF PILE INSTALLATION

1.4.1. - INTRODUCTION

When a pile is installed in the ground, there are a number of conditions existing prior to the installation that will change, such as: state of stress, soil structure, and moisture content. Although the construction procedures for bored piles are usually considered less influential on the surrounding soil than the driven piles (see Figure 1.7 from Vesic, 1977) they certainly affect pile performance. The process of load transfer from the pile to the soil must be dependent on the final soil conditions, after installation. According to some references, there is an increasing interest in the installation effects on pile performance (see Chadeisson, 1961; Endo, 1977; Zliwinski et al, 1979; Bustamante et al, 1979; Curtis,1980).

There has been a large amount of research done on the topic "effects of driven pile installation", covering both experimental techniques (for example: Cummings, Kerkoff and Peck, 1950; Bjerrum and Johannessen, 1960; Lo and Stermac, 1965; Orrge and Broms, 1967; Airhart, 1967; Airhart et al, 1969; Adams and Hanna, 1971; Hagerty and Peck, 1971; Flaate, 1972; Clark and Meyerhof, 1972; 1973; Hagerty et al, 1972; Cooke and Price, 1973; Massarch, 1976; Bozozuk et al, 1978;Blanchet et al, 1980; Cooke et al, 1980; Steenfelt, Randolph and Wroth, 1981; Roy et al, 1981) and analytical procedures (for example : Soderberg, 1962; Nishida, 1964; Desai, 1978; Randolph, Carter and Wroth, 1979; Randolph and Carter, 1979; Wroth, Carter and Randolph, 1979; Randolph and Wroth, 1979; Carter, Randolph and Wroth, 1980; Leifer, Kirby and Esrig, 1980) but just a few publications are concerned with the effects of installation of bored piles on the properties of the clay-pile system.

For ease of presentation the topic will be covered under the following titles : SHAFT EXCAVATION, FRESH CONCRETE AND CLAY SYSTEM and RE-ESTABLISHMENT OF HORIZONTAL STRESSES.

1.4.2. - SHAFT EXCAVATION

The process of boring a vertical hole to form a cast-in-situ bored pile affects the properties of the clay around the pile in a manner difficult to quantify. During shaft excavation the stresses near the hole are reduced by an amount depending upon the wall support, if any, that is provided. No matter the technique used, some amount of stress relief will occur. During the boring of the shaft hole, a relatively narrow zone of soil surrounding the pile must undergo some remoulding due to soil removal, horizontal radial total stresses at the surface of the pile fall to zero, pore water pressures change , and water migrates from the soil mass towards less stressed zones around the borehole so that swelling and softening start to occur. In overconsolidated fissured clays, the fissures may open. A similar phenomenon occurs at the base of the shaft, where the vertical total stresses are reduced to zero.

The magnitude of the swelling, softening and eventual soil movements is influenced by a number of factors, such as: dimensions of the excavation, ground water regime, soil properties (coefficient of earth pressure at rest (Ko), unit weight, pore pressure parameters, OCR, permeability, fabric and macro-structure), lenght of time the hole remains open before concrete is poured and support used (if any). The influence of three factors - soil properties, ground water regime , and time are closely interrelated. If an instantaneous excavation were made, the soil would strain in an undrained condition. If an excavation were made at an infinitely slow rate or were kept open without concreting or support for a long period, the soil would strain in a fully drained condition. Actual drilling occurs over a finite period and the soil conditions are thus partially drained. Typical excavations in clays, as a practical matter, are close to undrained, except near the clay boundaries.

Comparing the effects of using different support systems: the use of casing or drilling mud does not completely restore the initial stress conditions and some amount of stress relief occurs. When a casing is used, it is typical to excavate a shaft somewhat greater than the outside diameter of the casing to facilitate sinking of the casing (American practice, Woodward, Gardner and Greer, 1972) or advance the excavation in front of the casing (general practice). The resulting stress relief is probably smaller than the case of no support, but it is very likely that the extension of the remoulded zone is greater and there is little lateral displacement allowed until the concrete is poured. Under some conditions, the ground may slowly squeeze in around the casing and make extraction difficult. Lukas and Baker, 1978, observed slow squeezing when the ratio of overburden stress, γZ divided by the undrained shear strength, S_u, exceeded 6:

$$\frac{\gamma Z}{S_u} \geq 6 \qquad \text{Eq 1.30}$$

when the values of this ratio exceeded 8 or 9, rapid inward movement or instability of the hole was observed. In the case of using a bentonite slurry, the lateral pressure of the mud is usually less than the lateral pressure required to maintain zero lateral displacement. Accordingly, inward movement occur resulting in some lateral stress relief before complete installation of the pile.

As pile practice is variable and soil parameters assume a wide range in values in nature, a wide range in the results of installation related changes can be expected.

To illustrate the effect of using different techniques on the final pile diameter, Table 1.4 presents the American experience on the typical ratio of installed to designed diameter of bored piles when different techniques are used, for various soil conditions (after Stewart and Kulhawy, 1981).

SOIL	CONSTRUCTION METHOD	TYPICAL RATIO OF INSTALLED TO DESIGNED DIAMETER *
Uniform soft clays	Casing	1.10
Uniform soft clays	Open hole	1.00 to 1.05
Uniform stiff clays	Casing	1.10
Uniform stiff clays	Open hole	1.00 to 1.05
Fine cohesionless	Slurry	1.10 to 1.15
Fine cohesionless	Casing vibrated ahead of excavation	1.00
Stiff soil with cobbles	Open hole	1.10 to 1.15
Loess	Open hole	1.00 to 1.05
Cavernous limestone	Slurry or open hole	Overpour** maybe 100 %
Residual soils	Open hole	Overpour** as much as 50 %

* Most significant factors are groundwater stratification and lenses, construction methods, equipment and soil conditions.

** Overpour = volume of concrete placed in excess of as-designed quantities given as percentage of designed quantity.

TABLE 1.4 - American experience of average bored pile diameter as installed in various soil conditions (After Stewart and Kulhawy, 1981)

1.4.2.1. - STRESS RELIEF

The stress relief caused by a vertical hole in the ground has been treated in the past (Westergaard, 1940; Terzaghi, 1943) using approximate elastic solutions to solve the problem. No general procedure exists, to the author's knowledge, to handle the problem simulating the different excavation procedures or representing a realistic stress-strain behaviour of soil.

A three-dimensional photo-elastic study was made by Galle and Wilhoit, 1961, to determine the stress state around the wall and bottom of a wellbore due to the fluid pressure within the wellbore and unequal geostatic stresses. The results were presented in the form of contour curves for each stress component. A systematic method was given for calculation of these stress components for any combination of fluid pressure within the hole and system of principal geostatic stresses. References were given from previous analytical work. The obtained experimental results were compared to the analytical prediction with reasonable agreement.

Reese and Hudson, 1968b, were unsuccessful in a field attempt to measure the stresses acting upon the shaft of a bored pile. Problems of installation of the total pressure cells inside the pile shaft proved to be insoluble.

For a normally consolidated soil, the stress and strain and stress path for soil elements near an excavation was presented by Lambe, 1970, in a "State-of-the-Art Review" for the problem of braced excavations.

A pattern of stress distribution around the hole is shown in

Figure 1.8, suggested by Touma and Reese, 1972. It is readily apparent that, for the case shown ($K_o < 1$), σ_r is the minor principal stress.

Lopes, 1979, suggested the pattern presented in Figure 1.9 as a possible variation of the stress regime during pile installation.

The only measurements available in the literature are those by Yong, 1979 (also referred to by Anderson and Yong, 1980). Figures 1.10a and 1.10b shows the results obtained using miniature (38 mm diameter, 100 mm long) instrumented model pile, bored in normally and overconsolidated beds of kaolin and filled with micro-concrete (0.6 of water: 2.4 of aggregate:1 of cement). The total stress was measured at the pile-soil interface using a purpose built miniature pressure cell (strain-gauge-diaphragm type). The pore pressure at the interface was measured using an external pressure transducer connected to an interface probe (porous stone inside a brass casing) by nylon tubes. As the interface probe was built in the pile, the effects of drilling had to be assumed. A detailed examination of the results obtained and assumed conditions can raise some questions about the adequacy of techniques used to measure pore pressure and total stresses on the pile-soil interface, the scale of the experiment, the material used to simulate the concrete and even the assumed lines of pore pressure variation. The merit of the work however, is the fact that an attempt was made to study key factors affecting pile behaviour.

1.4.2.2. - REMOULDING

During the installation procedures of a bored pile the soil is disturbed by the boring tool over a zone adjacent to the sides of the excavated hole. When a casing is used additional disturbance can be expected. The thickness of the affected disturbed zone and the degree of disturbance to the soil structure is dependent on the type of equipment being used and probably on other details of installation.

To the author's knowledge there is no field study of the topic in the literature.

The only reference found refers to the effect of cleaning the base of the hole to perform plate load tests at depth, by Marsland, 1971, on the deformation moduli of London Clay (Figure 1.11). In these tests it was evident the influence of the disturbance of the clay below the test level caused by the drilling operations. The load settlement curves for tests made on a surface which had been machine finished and from which only loose clay had been removed, are much flatter than those where extra hand digging had been carried out. Much closer agreement was found between the slope of the initial and reloading curves for the tests where extra clay was removed by hand digging.

1.4.2.3. - DRILLING MUD

As already discussed, drilling mud affects the extent of changes in soil stresses during a boring operation, and thus, affects changes in moisture content in the shaft wall. The mud probably interferes both in the water absorption and ion - exchange phenomenon (setting concrete and clay) and in addition it can influence ultimate pile capacity by increasing or decreasing the available skin resistance, as a result of mudcake formation on the walls of the borehole.

The use of a drilling mud involves other aspects that must be considered: the different tool used can change the final size and surface conditions of the pile, making it difficult to compare the behaviour of piles made using different techniques.

The effect of bentonite on the skin friction of bored piles is a matter of discussion and different opinions can be seen in the literature.

Bentonite suspensions are extensively used to support the soil in diaphragm wall construction and a similar technique may be applied to bored pile construction to stabilize the pile wall, both by prevention of groundwater inflow and by limiting the collapse of unstable soils in the sidewall.

In permeable soil, bentonite forms a 'filter cake' layer (Fleming and Sliwinski, 1977) at the interface with the soil which may not be fully removed by the action of the fresh concrete being placed by tremie. The possible presence of this layer of low strength bentonite raises doubts about the friction values that apply for a foundation constructed using such a method. There is no established knowledge but the sparse evidence suggests a reduction in the friction value for granular soils of 10 to 30 per cent (Fleming and Sliwinski,1977). Some tests in model piles in sand have shown a larger reduction.

In soils of low permeability, such as clay, there is no evidence of the formation of a significant thickness of bentonite at the soil surface (Burland, 1963; Fearenside and Cooke, 1978). In boreholes for piles in saturated clays, only a very thin coating of bentonite cake may be expected to form, and this is confirmed in practice by observations on excavated piles and diaphragm walls. At times a filter cake of a few millimeters thickness has been found (Fleming and Sliwinski, 1977) and its formation is not completely explained. According to Nash and

Jones, 1963, experience with walls constructed at London using the I.C.O.S method showed that for 6 per cent bentonite suspensions in contact with London Clay, a mud cake is formed, at the rate of ¼ in/day (6 mm/day). There are explanations relating the formation of such a layer to large delays in concreting, flocculation and electrical phenomena (Veder, 1961; Nash and Jones, 1963). Reese and O'Neill , 1971, reported a case of reduction in friction probably due to a thick layer of bentonite left at the soil interface because a temporary steel casing was also used (this is not a very common practice). The opinion that skin friction in clays is virtually unaffected by bentonite is supported by a large number of results of pile tests (Chadeisson, 1961; Burland, 1963; Komornik and Wiseman, 1967; Farmer et al, 1970; O'Neill and Reese, 1972; Reese et al, 1973; Corbet et al, 1974; Kiemberger, 1974; Slivinski and Fleming, 1974; Everett and Mc Millan, 1975; Fleming and Sliwinski, 1977; Fearenside and Cooke, 1978; Rosenberg and Journeaux, 1978).

The series of tests on instrumented piles constructed using different techniques, by Fearenside and Cooke, 1978, carried out specially to get a better understanding of the bentonite influence on the behaviour of bored piles in clay concluded that there is no evident difference, due to the use of bentonite.

Regarding the stabilizing action of bentonite suspension, field observations and laboratory experiments have been presented in many publications: Veder, 1953; Nash and Jones, 1963; Veder, 1963; Schneebeli, 1964; Morgenstern and Amir-Tahmasseb, 1965; Elton, 1968; Mesri and Olson, 1970; Muller-Kirchnbauer, 1972; Meyerhof, 1972; Hude, 1972; Puller, 1974; Nash, 1974; Nash, 1978; Xantakos, 1979.

From a practical viewpoint, control of both the quality and level of bentonite slurry is necessary to ensure stability (Littlejohn et al, 1971; Nash, 1974; Hanna, 1978).

The stabilization effect of the bentonite can be simply explained as follows: the suspension acts on the side of the wall exerting hydrostatic pressure (corresponding to the depth of the suspension at this point); the suspension acts through the bentonite cake which forms a diaphragm (to assure stability of a borehole, the hydrostatic pressure of the bentonite suspension must be greater than the pressure of the soil less the arching effect). In addition to the hydrostatic pressure (the main stabilizing factor), there are some secondary factors cited by reserchers:

- the increased resistance to shearing of the zone penetrated by bentonite (in granular soils);
- the passive resistance of the bentonite slurry;
- electro - osmotic forces.

The suspension must form a seal on the surface with which it comes into contact, in order to exert a stabilizing pressure: in this way hydrostatic pressure can be exerted on the walls. In soils of low permeability such as clays, where filtration is very limited (or does not occur), hydrostatic pressure may be exerted directly on the walls of the borehole.

The displacement of bentonite suspension from the face of the borehole by the fluid concrete consists of replacement of one fluid by a heavier one. The properties of fresh (fluid) concrete and of bentonite suspension are similar in type but different in magnitude. A comparison of the usual ranges of the yield stress and plastic viscosity is given on the table below (from Fleming and Sliwinski, 1977).

Material	Yield stress (N/m^2)	Plastic viscosity (cP)	Density (Mg/m^3)
Concrete	250/1000	-	2.2/2.5
Bentonite	0/25	10/50	1.02/1.3

Table 1.3 - Comparison of properties of fluid concrete and bentonite, typical maximum and minimum values encountered in practice, from Fleming and Sliwinski, 1977.

The yield stress of a bentonite suspension is much lower than that of concrete, and as a result the concrete will be efficient in scouring slurry from vertical surfaces since the concrete itself will not be sheared until only very thin mud layers on the borehole surface irregularities remain (Hutchinson cited by Fleming and Sliwinski, 1977).

Some measurements of shear strength of the cake (Veder, 1963; Mesri and Olson, 1970) suggest that the order of shearing resistance of the cake formed on a borehole wall is greater than the shear strength of fresh concrete, and one can expect an inefficient scouring action from the concrete in removing such a layer. Observation of the excavated faces of diaphragm walls in pervious soils confirms the fact that part of the bentonite cake remains.

1.4.2.4. - GROUND MOVEMENT

The only reference found in the literature concerning ground movements associated specifically with bored pile installation was by Lukas and Baker, 1978, referring to cases in Chicago, USA. It is important to note that the typical soil profile in all reported cases presents a layer approximately 10 m thick of soft to medium silty clay overlaying the very stiff clay and bedrock where usually the bored pile stops. Another important aspect of the paper is the installation procedure used: an oversized shaft was drilled (usually 6 inches bigger in diameter than the temporary casing) as a standard technique, using a bentonite slurry, up to the top of the stiff layer. Figure 1.12a shows the foundation settlement of an adjacent structure in site 3, due to the excavation of piles of different diameter. Figure 1.12b shows the ground movement monitored at another site.

As there is a growing use of slurry wall panels as foundation elements ("pieux barrettes" in French literature) and there are some records of the ground movements related to diaphragm wall construction, the most relevant findings will be briefly presented.

A field test carried out by Dibiagio and Myrvoll, 1972, in soft clays in Oslo showed that the lowering of the density of the slurry and its final replacement by water increased the surface settlement of adjacent ground.

Cunningham and Fernandez, 1972, found no significant ground movement due to the installation of two diaphragm walls in interbedded sands and clays in Chicago.

A field test carried out to monitor ground movement caused by a bentonite - supported excavation in London Clay was reported by Farmer and Attewell, 1973. During and after the excavation and concreting of a deep trench (0.8 m wide, 6.1 m long, and 15 m deep) in three panels of approximately 2 m long each, the horizontal movements were measured with inclinometers and vertical movements in one of the boreholes were measured with Building Research Establishment magnetic ring settlement gauges (Figure 1.13a). According to the authors significant deformation of the instrumented ground did not occur until excavation of the centre panel had started. Immediately after excavation, a maximum deformation of 15 mm at borehole 1 was observed at a depth of 5 m. Minor changes occurred up to 7 days, when the excavation was concreted. Maximum deformations of 6 and 2.6 mm were observed at a depth of 6 m in borehole 2 and 3 respectively. No significant horizontal deformation was observed at borehole 4. The vertical settlement was measured at the surface, and at 3 m increments of depth in borehole 1. Maximum vertical settlements of 6 mm occurred at a depth of 7.7 m. Deformation vectors for borehole 1 are illustrated in Figure 1.13b. Apparently the deformations were affected by the excavation guide wall used (reducing the horizontal movement at ground level).

During excavation for a 17 m deep wall through glacial till in Montreal (Canada), Rosenberg et al, 1977, recorded a maximum lateral movement of 1.3 mm at 7.6 m depth.

According to Lambe et al, 1972, a 24 m deep diaphragm wall excavation in the highly overconsolidated Boston Blue Clay showed a maximum inward movement of 19 mm, about half was recovered during concreting. The net movement measured during installation contributed about one third of the total lateral movement after excavation.

Observations on diaphragm wall excavations in London Clay suggest that significant ground movements occur during excavation of the slurry trench (St John, 1975).

Observations of the ground movements associated with the construction of an underground car park in London (Burland et al, 1977) showed that the vertical and horizontal ground movements outside the excavation, due to the installation of the diaphragm walls and piling, amounted to approximately fifty per cent of the total movements recorded on completion of the main excavation to 18.5 m. The area affected by settlements had a length of approximately 30 m and horizontal movements towards excavation were monitored up to 40 m away from the wall. At a distance of 10 m from the south wall a settlement of 5 mm was recorded at the time of completion of the diaphragm wall, which had increased to 10 mm just prior to commencement of the main excavation, four months later. The corresponding maximum horizontal displacement was about 6 mm.

1.4.3. - FRESH CONCRETE AND CLAY SYSTEM

During a typical bored pile installation, following shaft excavation the borehole is filled with a high slump concrete mix. Different effects can result . For ease of presentation these can be grouped as being related to the state of stresses and to the effects of water migration. In reality all effects are related.

When a bored hole is filled with fresh concrete there is an increase in both total radial stresses and pore water pressure. During the setting time of the concrete there is a change both in volume and in the state of stresses. Usually, for the mixes employed in bored piles, the available water in the concrete mix is more than required for cement hydration and may serve as a moisture source for materials having a tendency to swell. To study the interaction between the concrete and the surrounding soil it is fundamental to identify and understand the properties and changes of the concrete mix from the fresh state to the hardened one. A survey of the concrete technology literature shows that the properties of concrete at an early age are not covered by the extensive volume of publications usually devoted to fresh or hardened concrete (Bergstrom and Byfors, 1980). The first attempt to organize the available knowledge was made by the RILEM in April 1982 ("Properties of Concrete at Early Ages", Paris).

1.4.3.1. - DELAY IN CONCRETING

To the author's knowledge there has been no successful field test performed to assess the effect of delay in concreting on the performance of bored piles. It is well known in the piling practice that delays are bad practice and must be avoided at any cost if a good performance is to be expected.

The effect of the delay in concreting can be assessed from the response of the soil in some "in situ" testing. In addition to the loading level and pile geometry, the elastic modulus E (or the shear modulus G) of the supporting soil has a fundamental effect on the settlement of a pile under working conditions. According to Marsland, 1971, the ratio E/S_u (where S_u is the undrained shear strength) both determined from plate loading tests carried out in the bottom of shafts in London clay is dependent on the time taken to set up the tests. For short times, E/S_u values of about 500 were reported, whereas for longer than 8 hours E/S_u dropped to between 100 and 200. The representative in situ value of E/S_u for London clay is probably at least 500 and lower values indicate boring disturbance, inadequate cleaning of the base, and swelling.

A laboratory study of the effects of delay between boring and concreting was performed by Yong, 1979. Using model piles (38mm ∅) the following load reductions were found: for 12 hours a reduction of about 10 %, for 24 hours delay a reduction of about 20 % in shaft bearing capacity, when comparing with the results obtained with no delay.

1.4.3.2. - STATE OF STRESSES

The pattern of stress development throughout the concrete and the adjacent soil mass is a very complex problem. Proposed approaches to evaluate the concrete stress distribution at this stage have been developed for the design of concrete formwork. The normal design methods assume that the concrete will reach a maximum pressure. The pressure envelope follows the fluid pressure to the depth at which this maximum pressure occurs and then remains constant. The maximum design pressure is obtained from design charts (American Concrete Institute, 1958; Civil Engineering Research Association, 1965; CIRIA, 1969) or equations and it is dependent on the rate of placing, height of lift, section being filled, concrete density, temperature and consistency. These approaches were developed for placement of concrete in formwork; there are major differences such as placement rate, consistency and height of lift from one situation to the other. The applicability of the proposed approaches is, at the least, questionable. The mechanisms involved in the development and variation of formwork pressure do not appear clearly identified as yet (Harrison, 1979).

Amongst others, the following variables have been found to affect the lateral pressure of fresh concrete (ACI - Pressures on Formwork, 1958 , CERA - Research Report, 1965):

- Rate of placement: this factor has a primary effect on lateral pressures regardless of the over-all height of the lift and pressure is directly proportional to rate of placement;

- Consistency of the concrete: (an approximate idea of the consistency of the concrete can be obtained from the slump test). The consistency can be expressed as a function of cohesion and internal friction and varies with time as set occurs (the main difference between the concrete used for structures and for large bored piles is that for the second purpose it is common practice to use plasticizers to obtain an almost fluid material);

- Depth of placement: all the research done on the subject is referred to the usual height of one storey (18 ft or 5.40 m), and a major difference arises because piles with depths of 25 m or more are usual;

- Stiffening of the concrete: stiffening of the concrete may be defined as the progressive increase in the strength of the concrete. This is partly due to chemical changes in the cement matrix but it is also dependent upon the degree of mechanical interlocking between aggregate particles. As stiffening develops, the concrete becomes capable of supporting additional surcharge without increase in lateral pressure; this phenomenon might therefore, be related to the rate of increase of shear strength of the fresh concrete. That part of the stiffening process which is due to the chemical reaction of cement depends upon time, temperature and the type of the cement itself. That part of the stiffening process which depends upon the physical interlock beween particles is a function of total stress, workability and vibration. In a large bored pile, the time consumed in concreting all the pile can be significant, and the stiffening of the concrete must be taken into account;

- Arching effects: in all the measurements available, the vertical load at any horizontal section is reduced by frictional forces developed between the concrete and the formwork faces. The arching effect is dependent on the dimensions of the section, the distribution of lateral pressure over formwork and the variation of the coefficient of friction between the concrete and the formwork faces. In a bored pile the face of the pile will be dependent on the installation procedures and soil conditions (and it will normally be much rougher than a timber shutter).

- Other factors: impact (the effect of impact of discharge is to increase the pressure above that normally due to the static surcharge), weight of concrete, maximum aggregate size, temperature of concrete mix, ambient temperature, smoothness and permeability of container, cross section of container, placing procedures, and type of cement.

A full review of the literature on the studies of formwork pressure is outside the scope of this study, but the basic aspects (referring to the mechanisms and proposed approaches to predict the pressure)of the most relevant papers will be presented in Appendix 1.

DATA FROM LITERATURE

The available information in the literature is scarce, since the effects of installation was not a major question in the research programmes performed up to recently.

Field measurements reported by DiBiagio and Roti, 1972, for a trench 18 m deep show that upon completion of the pour the pressure exerted by the fresh concrete was the hydrostatic pressure of liquid concrete only in the upper portion of the panel and specifically for a depth of 5 m (Figure 1.14). When the column of overlying concrete exceeded 5 m, the lateral pressure on the face gradually became less than the total overburden pressure. At a depth of 10 m or more the pressure was between 0.6 and 0.8 times the overburden pressure.

In a paper describing the behaviour of a trench wall, Uriel and Otero, 1977, present the results of two instrumented panels (0.80 m thick, 3.40 m long, 34 m depth). The observed total pressures on one panel are presented in Figure 1.14. On the same figure are presented the theoretical total pressures, corresponding to the case in which the panel is completely full of bentonite (γ_b = 12 kN/m^3), with fresh concrete (γ_c = 23 kN/m^3).

According to the results, for practical purposes all measured values are included between the pressure of the slurry and the hydrostatic pressure of concrete. During the phase with only bentonite in the trench, measured pressures were very similar to the theoretical ones taking γ_b = 12 kN/m^3, except in cells situated at 9 and 12 m depth, where they were slightly higher. When the concrete was poured, pressures increased almost to the line for γ_c = 23 kN/m^3, up to a depth of 12 m. As the depth increased thereafter a difference appeared. Pressures measured at greater depths corresponded to a maximum fresh concrete head that oscillated between 13 and 15 m (almost 20 times the minimum thickness of the panel). The speculation is that a silo effect takes place inside trenches as the concrete level is raised or the effect of concrete set-up with time shows its effect. In the next stage (24 hours after concreting) pressures decreased and stabilized, at values of the same order of magnitude as those of the bentonite phase. According to the authors, there was no major difference between the two panels that were tested.

Data concerning the concrete pressure in a large bored pile obtained from field measurement are presented by Cooke, 1979. Reexamination of the test data obtained by Whitaker and Cooke, 1966, on a very large investigation of the behaviour of bored piles at Wembley, revealed that the vertical forces indicated by the load cells at the pile bases immediately after the concrete was poured were between one - third and one - half of the weight of wet concrete. In all but two of the 13 piles tested, these forces decreased during the period prior to the start of each incremental loading test. According to Cooke, this was attributed to increased arching between the concrete and the clay around the shaft, which could have been initiated by shrinkage of the concrete away from the clay beneath the base.

Recent contacts with the author (Cooke, 1980) did not succeed in obtaining the values of the recorded pressures, because the original data have been lost.

Touma and Reese, 1972, presented a speculative discussion about the possibility of the effect of shear stress development along the shaft wall during placement. It seems, however, that due to the fluid nature of the fresh concrete, any development of shear stresses should be small owing to the small resistance to shear offered by a high slump concrete mix (Powers, 1968).

In a paper on piles in sand, when trying to establish the horizontal stresses due to the fresh concrete, O'Neill and Reese, 1978, using the American Concrete Institution approach, proposed what they called a "critical relative depth" $(Z/d)c$. The concrete pressure was assumed to vary linearly from zero to the maximum at this point and remain constant thereafter. According to the authors this approach could take into account the diameter of the pile, as the factor was evaluated from Equation 1.31:

$$\left\{\frac{Z}{d}\right\}_c = \frac{3000}{\gamma_c' \, d} \qquad \text{Eq 1.31}$$

where:

Z = critical depth (ft)

d = diameter of the pile (ft)

3000 = limit value of horizontal pressure (Psf)

γ_c' = buoyant unit weight of concrete (Pcf)

Since the diameter is present on both sides of the equation it is difficult to understand the effect of the diameter in the variation of the critical depth (Z critical will always equal 23 ft).

Field measurements reported by Reynaud and Riviere, 1981, describing the behaviour of a trench wall (each panel with the following dimensions: 1.20 m thick, 7.20 m long, 30 m depth) in Le Havre, France, are presented in Figure 1.14. Using Glötzl total stress cells it was found that the pressure of the bentonite used was measured with accuracy. When the concrete was poured, it was found that the maximum pressure occurred when the fresh concrete was at 8, 11.5 and 13m above the cells located respectively at 16, 22.1 and

28.1 m depth and then stabilized. The maximum recorded pressures were lower than the hydrostatic calculated with $\gamma_c = 23.1$ kN/m^3 (measured by the authors).

1.4.3.3 - WATER MIGRATION

The available water in the concrete mix is usually more than required for cement hydration and may serve as a moisture source for materials having a tendency to swell. According to Nash and Jones, 1963, differences between the ions and the ion concentration in the soil water compared with the water in the concrete will result in a tendency of water movement due to a difference in chemical potential.

A literature survey of results of measured changes in soil adjacent to the shaft of bored piles and diaphragm walls are presented in Table 1.5.

It is important to note that only part of the observed changes in field experiments are due to the migration of water from concrete. The following factors also play some part: stress release due to drilling, fluid used for borehole stabilization and free water flowing out of the soil mass towards the pile through cracks, fissures or permeable seams during boring operations and up to the time of stabilization of stresses around the pile.

Considering that the survey covered all published data the volume of available information is scarce. The range of initial moisture content was from 19 to 32 and just three types of soils have been referred to: London Clay, Beaumont Clay from Texas, USA and Black Cotton Clay from India. The affected zone varies from a minimum of 25 to a maximum of 100 mm, with an average range from 50 to

70 mm. The maximum recorded increases in moisture content were in the range of 2 to 10 %, most commonly between 2 and 5 %. In some cases the maximum increase did not occur at the interface pile and soil. The variation obtained in moisture content near the interface was in the range 0.5 to 8 %. Figure 1.15 shows the area covered by the changes in a graph of moisture content increase versus distance from contact between pile and soil, using the available data from the literature.

A review of the existing laboratory studies of water migration was made and from the existing references (Dubose, 1956; Taylor, 1966; Chuang and Reese, 1969; O'Neill and Reese, 1972; Rice, 1975 and Yong, 1979) the relevant findings are present as follows. Results of any laboratory simulation of the effect of pile installation must be treated with caution. Apart from the scale effects, there are many installation related effects that are difficult or even impossible to simulate in laboratory conditions, such as: natural structure of the soil, remoulding due to boring, effect of bentonite, relief in stress, pressure acting on the fluid concrete due to self weight, heat generated by hydration of a large mass of concrete.

Results obtained by Chuang and Reese, 1969, are presented in Figures 1.16 to 1.18. The technique used was to pour a cement and sand mortar into a tube containing a sample of compacted soil (one series was performed using natural soil). After sealing the tube, a triaxial cell was used to house the experiment and to apply a convenient confining pressure (usually 10 psi). The effects on moisture migration of factors such as water cement ratio, initial moisture content of the soil, type of cement and time were studied. The main conclusions were: the initial moisture content affects moisture

Reference	Location	Soil type	Method of construction	Concrete	Original moisture content (%)	Affected zone (mm)	Maximum increase in moisture content (%)	Increase at interface (%)
Meyerhof and Murdock, 1953	London	London Clay	Dry (piles)	W/C=0.4	28 to 31	30 to 60	2 to 6	1 to 6
Rodin and Tomlinson, 1953	London	London Clay	Dry (piles)		27 to 31		3.5	3.5
Mohan and Chandra, 1961	India	Black Cotton Clay	Dry (piles)		19 to 20	50 to 75	2 to 3	2 to 3
Burland, 1963	London	London Clay	"Diaphragm" dry		27	50 to 75	4	4
			bentonite		27	30 to 50	3	3
O'Neill and Reese, 1972	Texas	Beaumont Clay	Dry (piles)		20		8	8
Chandler, 1977	London	London Clay			32	40	10	4
Fearenside and Cooke, 1978	London	London Clay	Dry (piles) 7	* 300kg ce/m^3 slump 125mm	22	70	3	3
			8		22	50	2	0.5
			10	400kg ce/m^3 slump 160mm	29	50	5	4
					23	50	2	2
			11	400kg ce/m^3 slump 175mm	27	70	9	9
					21	50	4	1
			Bentonite 13	400kg ce/m^3 slump 175mm	26	100	2	1
					22	25	4	4

* ce/m^3 = cement per cubic meter

TABLE 1.5 - Literature survey of results of measured changes in moisture content in soils adjacent to piles and diaphragm walls

increase (the higher initial moisture content, the lower is the increase); water/cement ratio is a key factor in the process (available free water); the initial moisture content affects the length of moisture migration, the values varying from 1 to $2\frac{1}{2}$ inches (25 to 68 mm) for compacted and natural samples. It was suggested that the use of cements with higher rates of hydration will reduce the moisture migration, if all other variables are kept constant.

A comprehensive laboratory experimental research on model bored piles (25, 38 and 50 mm diameter, 100 mm long) installed (using micro concrete) in beds of anisotropically consolidated kaolin (internal cell dimensions of 200 mm diameter, 400 mm long) was performed by Yong, 1979.

Figure 1.19 shows the results obtained in the study of the influence of initial moisture content migration in the soil surrounding the pile, at mid-height of pile for a mix of 0.6 of water; 2.4 of aggregate; 1 of cement in weight. The region affected by moisture migration is confined to 60 mm from the pile-soil interface. The higher the initial moisture content, the lower is the average increase in moisture content.

The influence of the water/cement ratio on moisture migration is presented in Figure 1.20. As expected, the higher the water/cement ratio, the higher increase in moisture content (certainly due to the greater amount of free water available).

The variation of moisture content at different positions from pile-soil interface at different ages is presented in Figure 1.21. Changes in moisture content can be observed up to 60 mm from the pile-soil interface. After 60 days the measured values of moisture content are slightly below the original value. It is questionable if the experiment simulated a realistic model for a long term experiment, specially due to scale effects.

There are suggestions that the water migrating from the fresh concrete towards the surrounding soil carries cement particles (in highly permeable soil) or cement components dissolved or in solution (Chuang and Reese, 1969) but there is no experimental evidence for the case of clay soils surrounding bored piles of such phenomenon. It is very unlikely that the water migrating from the fresh concrete is clean or does not carry any cement components, the main difficulty in studying the possible cement contamination in the water is the similar chemical components existing in the clay and in the cement.

1.4.3.4 - CONCRETE CURING EFFECTS

The curing or stiffening of the concrete is accomplished by the process of chemical hydration of the cement. When water is added to anhydrous cement a complex series of reactions is started which continues for many years. A presentation of the various theories and possible chemical reactions taking place is out of the scope of this research. A brief reference to the possible effects will be made in this section.

A typical example of the possible effect of concrete set on the measured lateral pressures is shown in Figure 1.14 (Uriel and Otero, 1977) by the difference in values between the curves labelled "after concreting" and "24 hours after placement".

A second effect of the hydration process is the development of thermal strains caused by heat resulting from the chemical reaction of cement and water. According to Touma and Reese, 1972, due to this process longitudinal strains along the shaft may reach values of 100 microstrain, although values as high as 200 may be possible. These strains are typically contractive and depend on the subsurface soil and groundwater conditions, consistency of the concrete and the ambient soil temperature conditions. The result of field measurements by the authors showed that the maximum thermal strain is located near the midheight of the shaft and decreases to zero at both the limits of the shaft (top and bottom). Due to these strains, shear stresses are developed along the soil and shaft interface, caused by the relative movements that occurs between the soil and the strained concrete.

Figure 1.22, from Touma and Reese, 1972, shows an example of these shear stresses for the compressive thermal strains.

At present, there have been no detailed studies made to evaluate the effects of the thermal strains or of the heat generated in the conditions or behaviour of bored piles.

1.4.4 - RE-ESTABLISHMENT OF HORIZONTAL STRESSES

After installation of the pile, the stresses will gradually change and the yield value will depend on the degree of softening that takes place in the clay around the shaft prior and during concreting. Even with perfect conditions, it seems doubtful if the initial at rest horizontal stresses can ever be fully re-established at the pile shaft face (Burland, 1973). The re-establishment of the lateral stresses in the neighbourhood of the pile shaft will reconsolidate the softened zone around the pile. There are no field measurements of the long term stresses (Reese and Hudson, 1968b, failed to measure horizontal stresses during and after pile installation due to instrumentation problems). The strength of the soil in this zone reduces as a result of the initial increase in moisture content, remoulding and swelling and then increases again as reconsolidation progresses (considering only the stresses).

Re-analysis of some of the results presented by Whitaker and Cooke, 1966, Taylor, 1966, and Combarieu, 1975, by the author (Clayton and Milititsky, 1983) has demonstrated that the shaft friction component can increase over a period of many months (Figure 1.23), presumably as a result of changes occurring on the period, including the increase of horizontal effective stresses.

This has serious implications for back-analysis, on which virtually all design methods are based. Load tests are ofter carried out shortly after the piles are formed, when strengths can be still changing.

1.5. - COMMENTS AND CONCLUSIONS

In this chapter pile design methods, pile installation and the effects of pile installation on the surrounding soil were presented. Emphasis was given to the "State-of-the-Art" review on the effects of pile installation, mainly on experimental or field work referring to the topic.

For ease of presentation, the effects of pile installation were classified into three groups : SHAFT EXCAVATION, FRESH CONCRETE AND CLAY SYSTEM, and RE-ESTABLISHMENT OF HORIZONTAL STRESSES.

Effects of shaft excavation have been treated under the titles Stress Relief, Remoulding, Drilling Mud and Ground Movement. Different references were found in the literature concerning the stress regime changes due to excavation, but no real solution exists that covers the problem of pile installation considering both total and effective stress changes, and no measurements exist of local stress changes during pile installation. Remoulding is a factor referred to by many authors but no field measurement or research programme deals with the topic extensively. The use of drilling mud is widely covered by the literature and it appears that the effects of it's use is assumed to be known not to affect pile behaviour. Ground movements reported by different researchers when diaphragm walls were built show that significant deformations occur during excavation and concreting. The information concerning bored piles is practically non existent, particularly if the volume of piling work is considered.

The study of the interaction between fresh concrete and clay is one of major importance in understanding the mechanisms of change in soil conditions during pile installation. Delay in concreting is known from pile practice to affect pile behaviour but no comprehensive work has been done to quantify the effect of delay. State of stresses is a very complex problem. When treating the problem the practice of using empirical approaches is followed by many geotechnical researchers and recent improvements and mechanisms involved in pressure variation are not used. For this reason a separate review was made and is presented in Appendix 1. Water migration is a subject scarcely covered by the references. Both in field determinations near foundations and laboratory simulations it's existence is widely recognized, but the effects of such a phenomenon and the mechanisms responsible for the development of moisture content variation are not clearly understood. There is some speculation about the possible stabilising effect very close to the concrete/soil interface due to cement particles, or cement components in solution, or dissolved in the water migrating from the concrete to the soil, but no experimental evidence exists. The effects of time dependent changes in the properties of the fresh concrete is a topic which rarely appears in the literature and is the subject merely of speculation.

Finally, the possibility of partial re-establishment of horizontal stresses was discussed. The author re-analized some of the sparse data on the topic and demonstrated that, for the cases studied, the shaft friction component can substantially increase over a period of many months.

Pile behaviour depends on the state of stresses acting on the surrounding soil during loading, thus Table 1.6 summarizes all factors controlling the behaviour of a bored pile under vertical compressive load.

The development of effective stress approaches to compute the shaft friction of bored piles can provide an improved understanding of factors that affect shaft friction and should lead in the future to improved techniques, but at present the knowledge of installation effects on soil properties and conditions is far from satisfactory and insufficiently understood to allow them to be simulated by theoretical models or numerical techniques.

In future theoretical models could provide a useful insight into the factors that affect pile capacity and could be useful in practice to provide a framework for continued development. Routine pile design will probably continue to be made based on past experience, using empirical factors and the very simplified total stress approach.

1. GROUND CONDITIONS
   - 1.1 - Soil profile
   - 1.2 - Soil properties (properties of the different layers)
   - 1.3 - State of initial stresses
   - 1.4 - Ground water conditions

2. PILE CONDITIONS
   - 2.1 - Installation
     - 2.1.1 - Drilling process
       - 2.1.1.1 - Manual
       - 2.1.1.2 - Mechanical : Percussion
Rotary drilling
(Type of tool)
     - 2.1.2 - Support used
       - 2.1.2.1 - Unsupported excavation (Dry method)
       - 2.1.2.2 - Casing
       - 2.1.2.3 - Mud
       - 2.1.2.4 - Continuous flight auger
     - 2.1.3 - Base cleaning
       - 2.1.3.1 - Manual
       - 2.1.3.2 - Mechanical
     - 2.1.4 - Excavation time
     - 2.1.5 - Concreting
       - 2.1.5.1 - Delay
       - 2.1.5.2 - Mix composition
       - 2.1.5.3 - Mix properties
       - 2.1.5.4 - Additives
       - 2.1.5.5 - Temperature
       - 2.1.5.6 - Integrity
   - 2.2 - Geometry
     - 2.2.1 - Length
     - 2.2.2 - Cross section
     - 2.2.3 - Base condition
       - 2.2.3.1 - Enlarged
       - 2.2.3.2 - Not enlarged

3. LOADING CONDITIONS
   - 3.1 - Pile age
   - 3.2 - Pile load test
     - 3.2.1 - Type of loading programme
       - 3.2.1.1 - Slow maintained load
       - 3.2.1.2 - Constant rate of penetration
       - 3.2.1.3 - Cyclic
       - 3.2.1.4 - Others
   - 3.3 - Under a real structure
     - 3.3.1 - Build up of the permanent load
     - 3.3.2 - Type of live load

TABLE 1.6 - Factors controlling the behaviour of a single bored pile under vertical compressive load

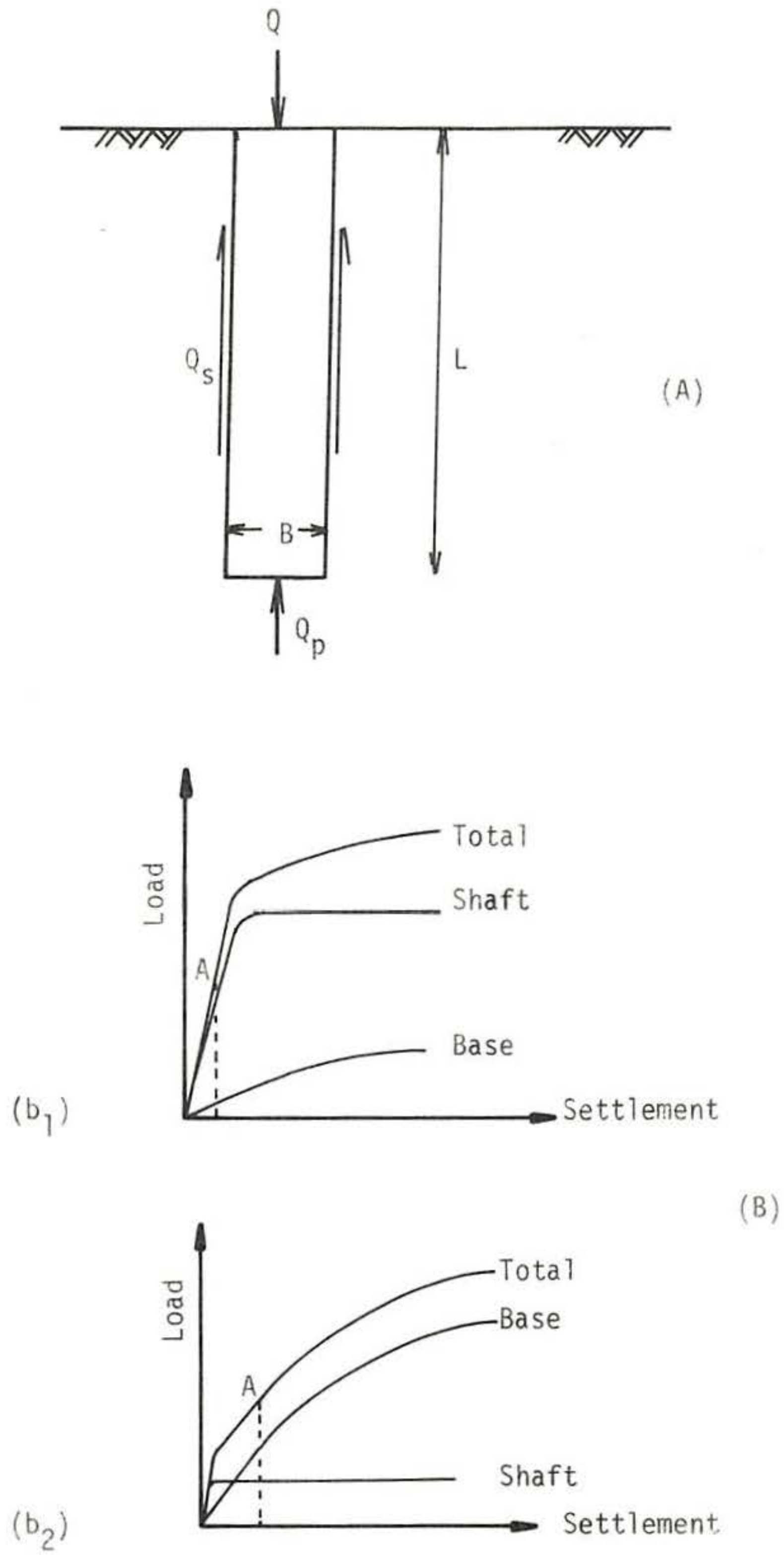


FIGURE 1.1 - LARGE BORED PILES IN CLAYS : (A) COMPUTATION OF THE ULTIMATE LOAD. (B) LOAD-SETTLEMENT DIAGRAMS (b_1) PILE WITH NO BASE ENLARGEMENT, (b_2) PILE WITH ENLARGED BASE. POINTS A INDICATE THE WORKING LOAD AT A FACTOR OF SAFETY OF 2

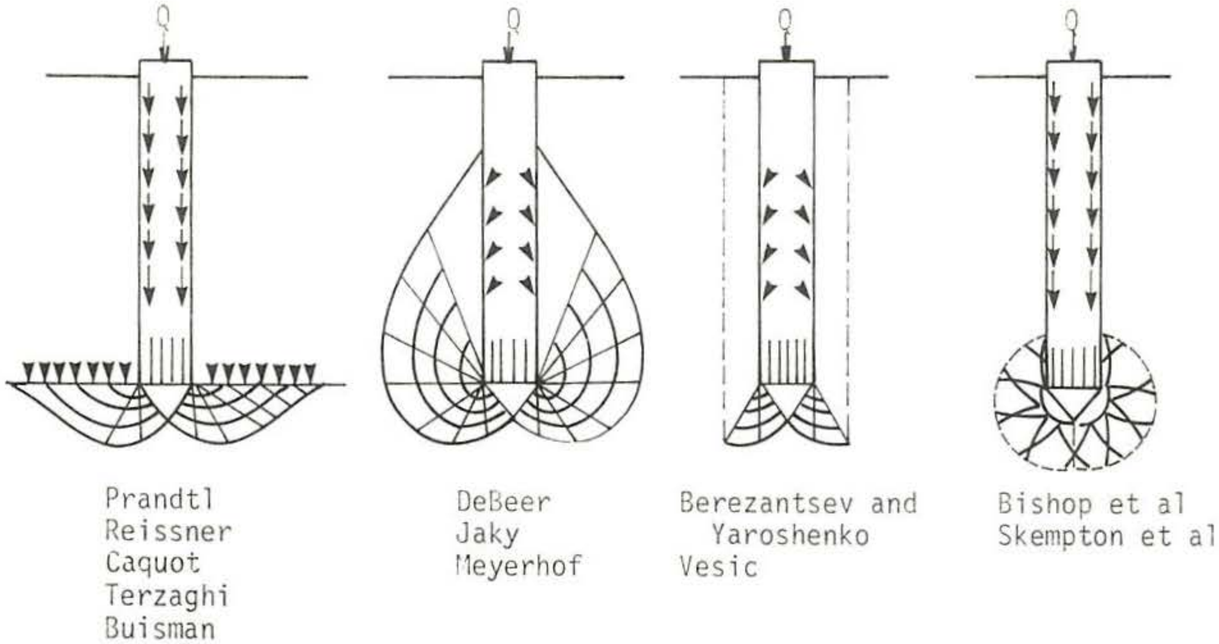


FIGURE 1.2 - FAILURE PATTERNS ASSUMED IN THE DIFFERENT BEARING CAPACITY SOLUTIONS, AFTER VESIC, 1975

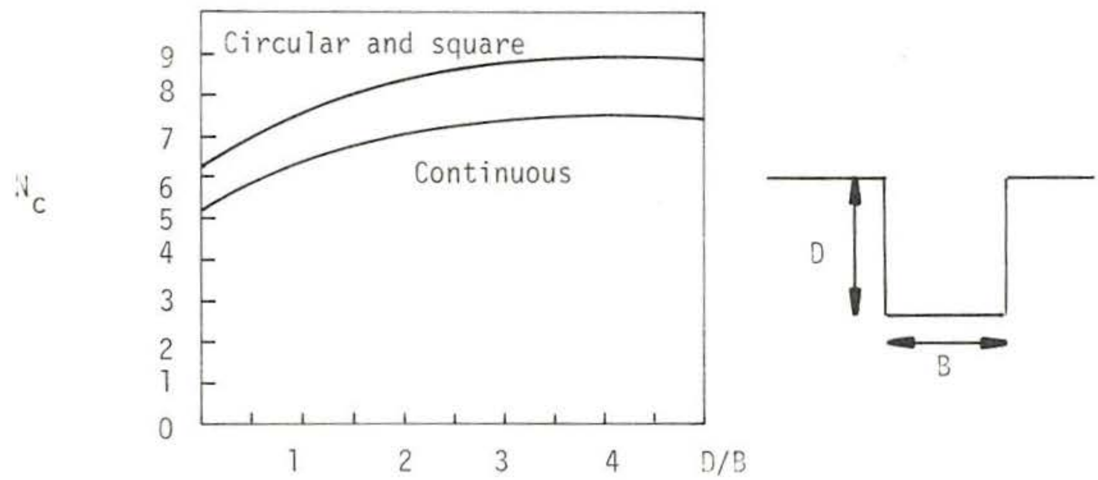


FIGURE 1.3 - BEARING CAPACITY FACTOR FOR FOUNDATIONS IN CLAY, SKEMPTON,1951

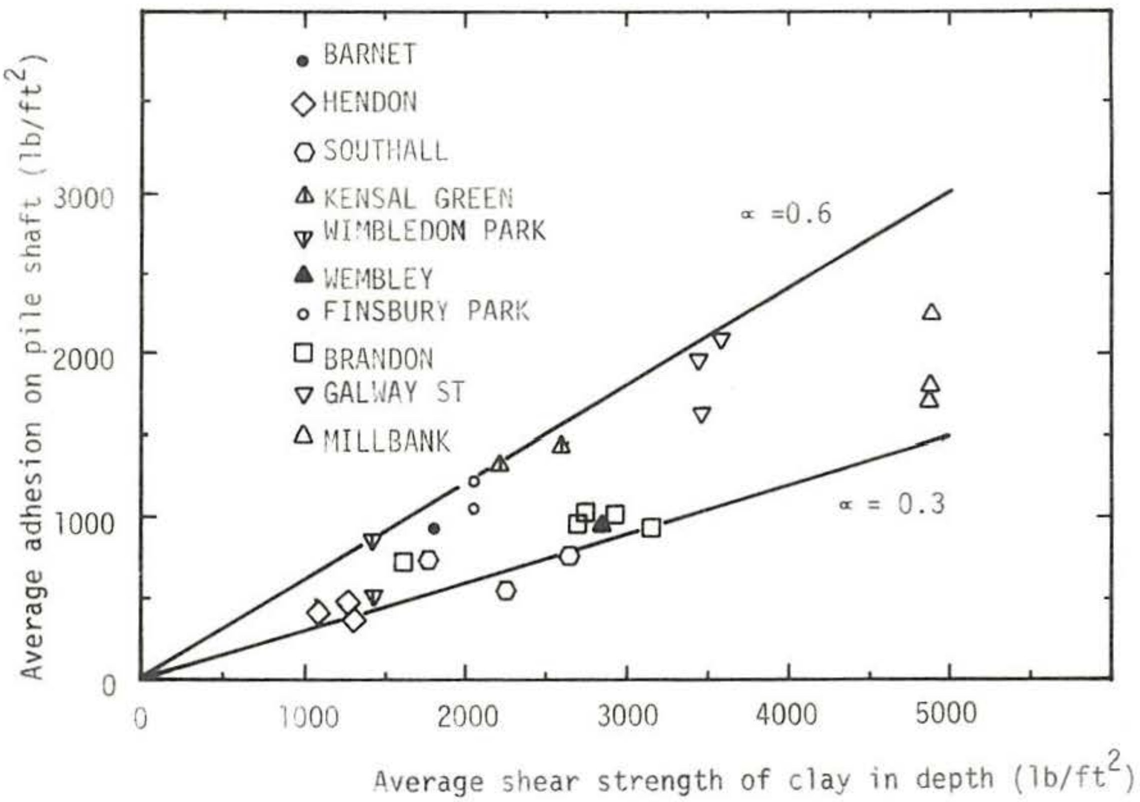


FIGURE 1.4a - COMPARISON MADE BY SKEMPTON, 1959, TO DERIVE THE ALPHA FACTOR METHOD

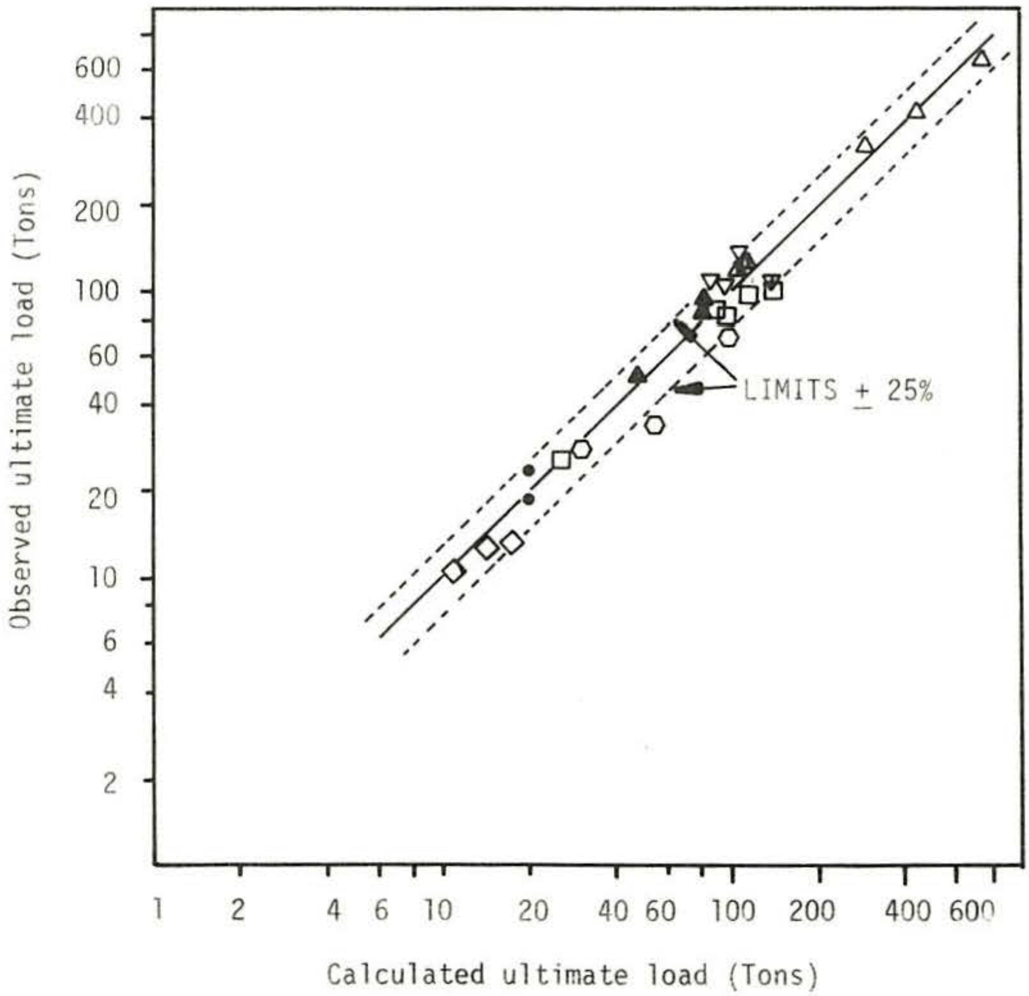


FIGURE 1.4b - OBSERVED VERSUS CALCULATED ULTIMATE LOADS (SKEMPTON, 1959)

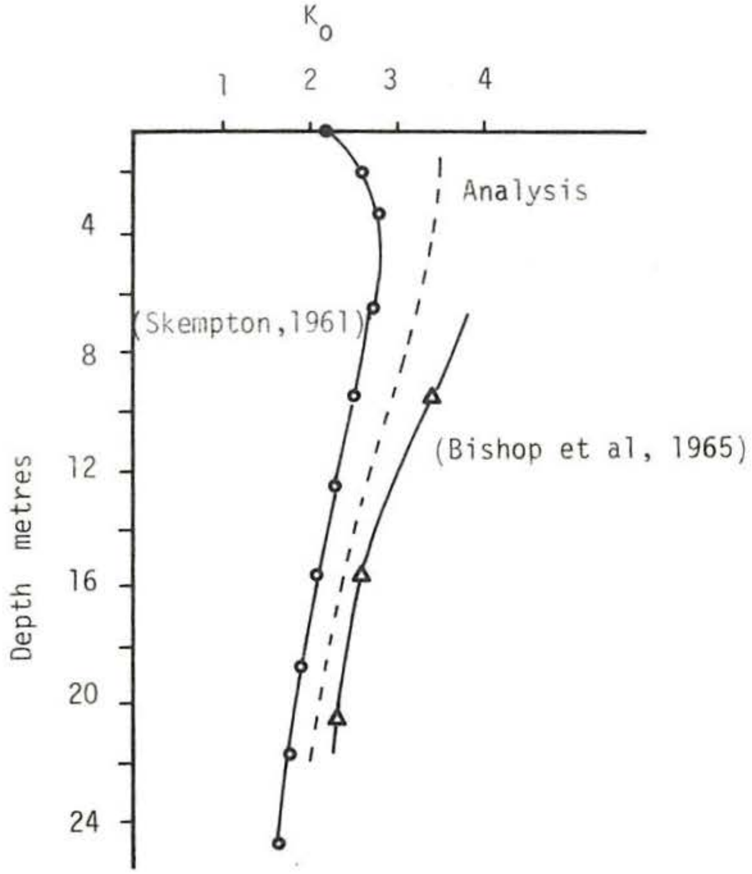
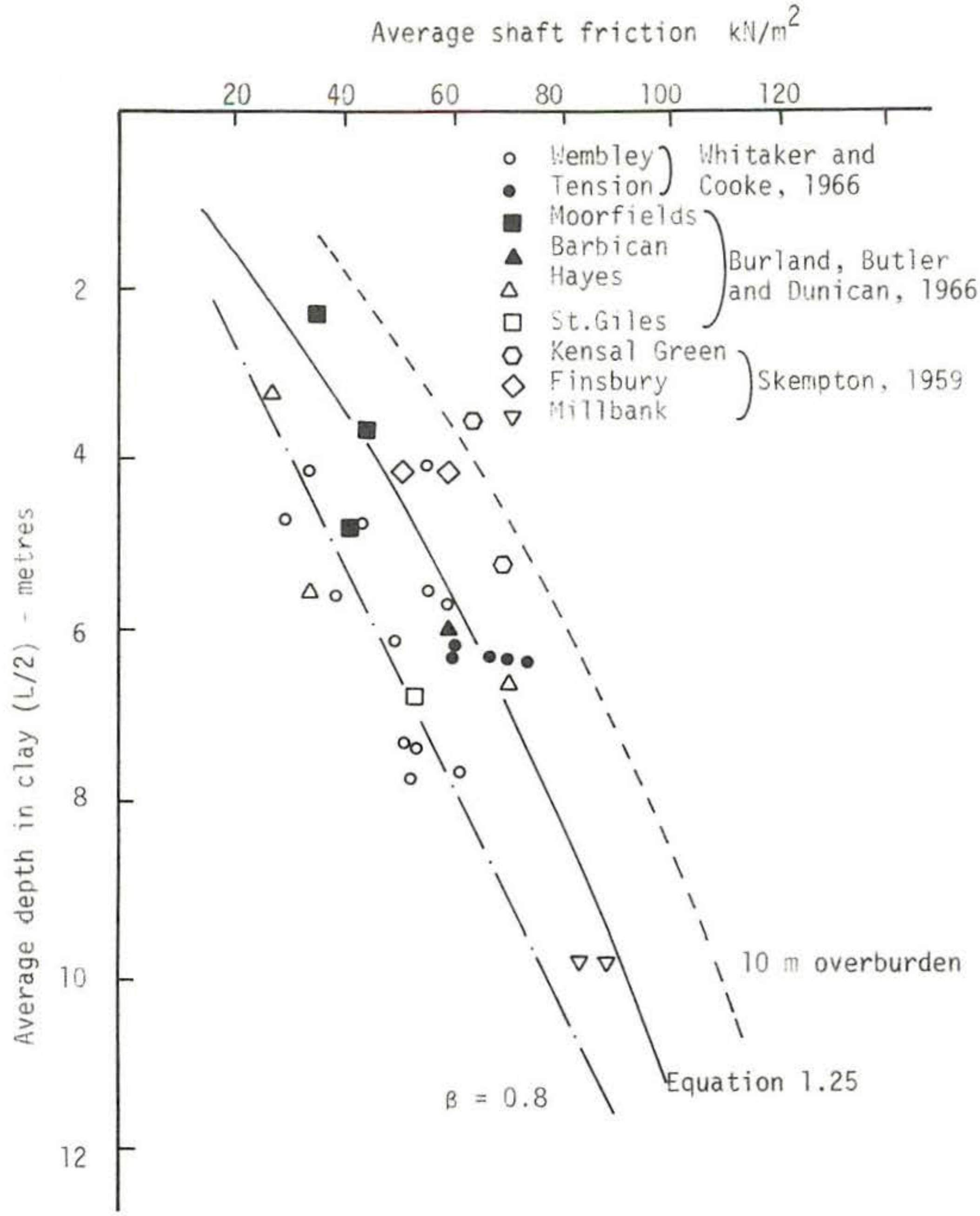


FIGURE 1.5 - VALUES OF K_o VERSUS DEPTH FOR LONDON CLAY



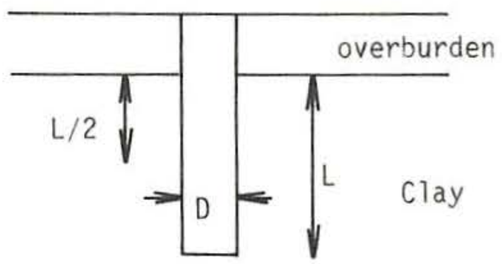


FIGURE 1.6 - OBSERVED VALUES OF AVERAGE SHAFT FRICTION FOR BORED PILES IN LONDON CLAY, BURLAND, 1973

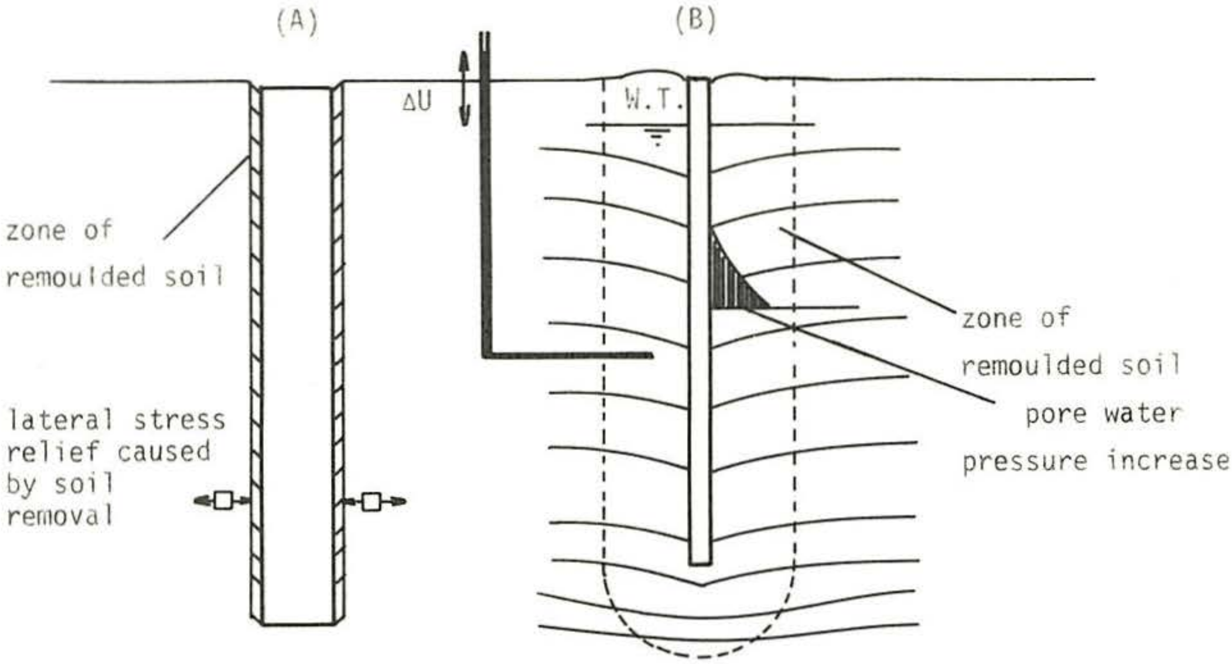
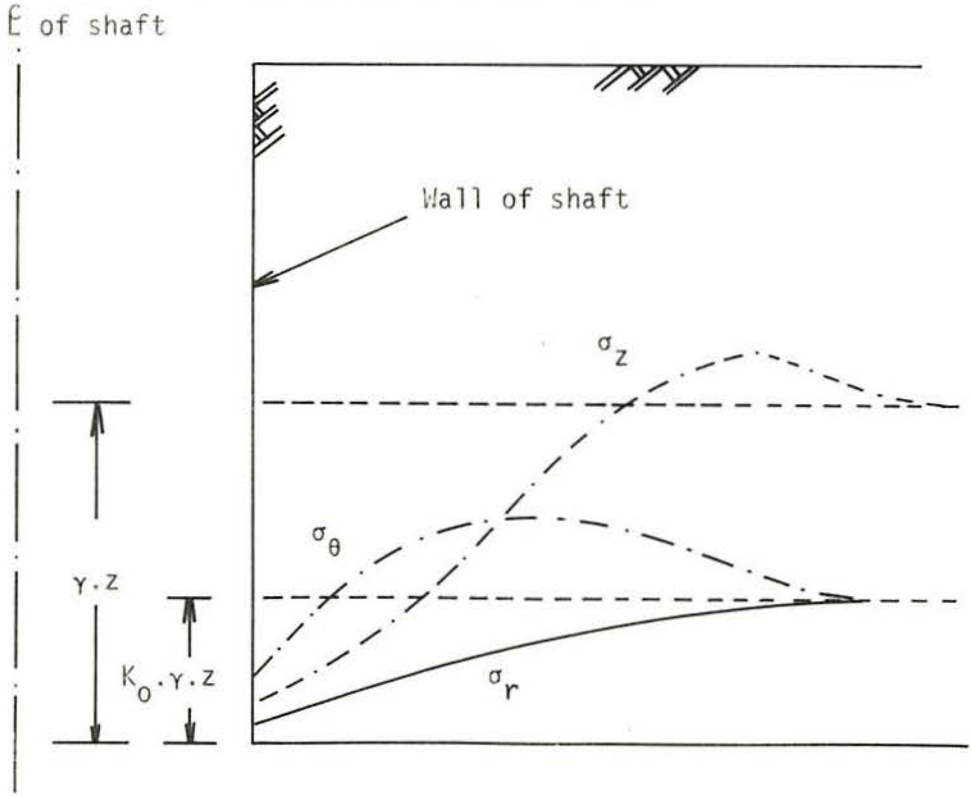
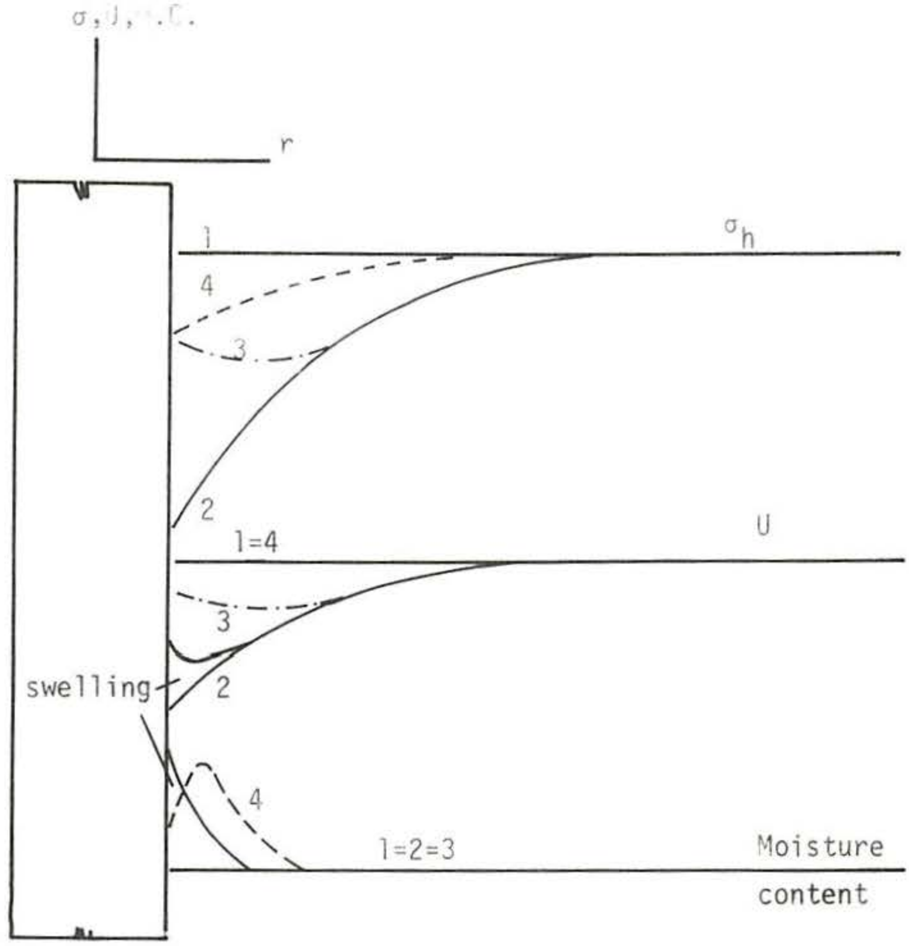


FIGURE 1.7 - EFFECTS OF PILE INSTALLATION, (A) BORED PILE, (B) DRIVEN PILE (AFTER VESIC, 1977)

FIGURE 1.8 - TYPICAL STRESS PATTERN AROUND A SHAFT SUPPORTED BY SLURRY (SUGGESTED BY TOUMA AND REESE, 1972)





LEGEND

1 - ORIGINAL

2 - AFTER BORING

3 - IMMEDIATELY AFTER CASTING

4 - LONG TERM

FIGURE 1.9 - IDEALIZED DIAGRAMS SHOWING STRESS, PORE-PRESSURE AND MOISTURE CONTENT CHANGES IN THE VICINITY OF A BORED PILE DUE TO INSTALLATION (LOPES, 1979)

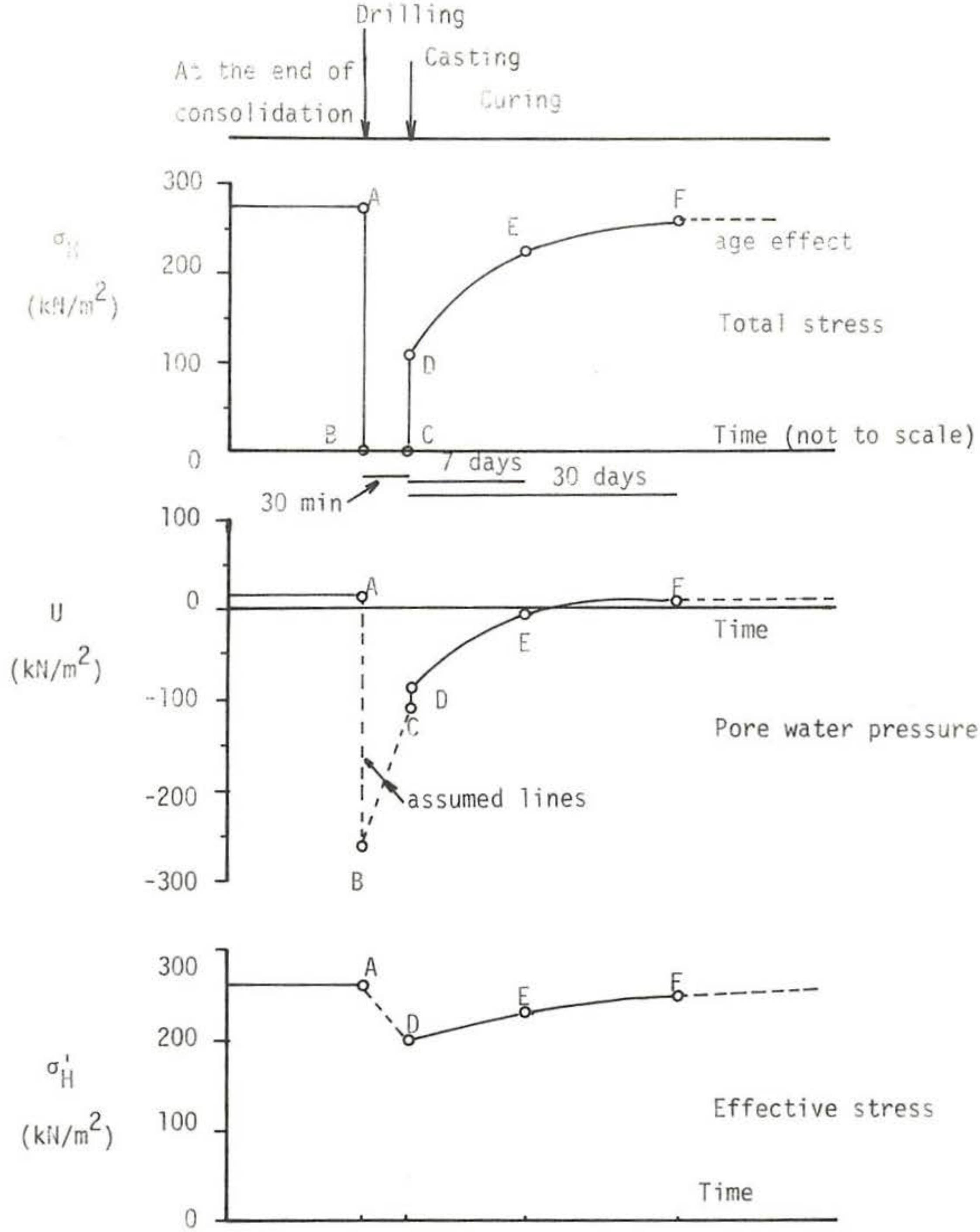


FIGURE 1.10a - VARIATION OF TOTAL HORIZONTAL STRESS, PORE WATER PRESSURE AND EFFECTIVE HORIZONTAL STRESS WITH TIME, FOR NORMALLY CONSOLIDATED KAOLIN, $p_o = 420$ kN/m^2, OWING TO THE INSTALLATION OF A MINIATURE BORED PILE, AFTER YONG, 1979

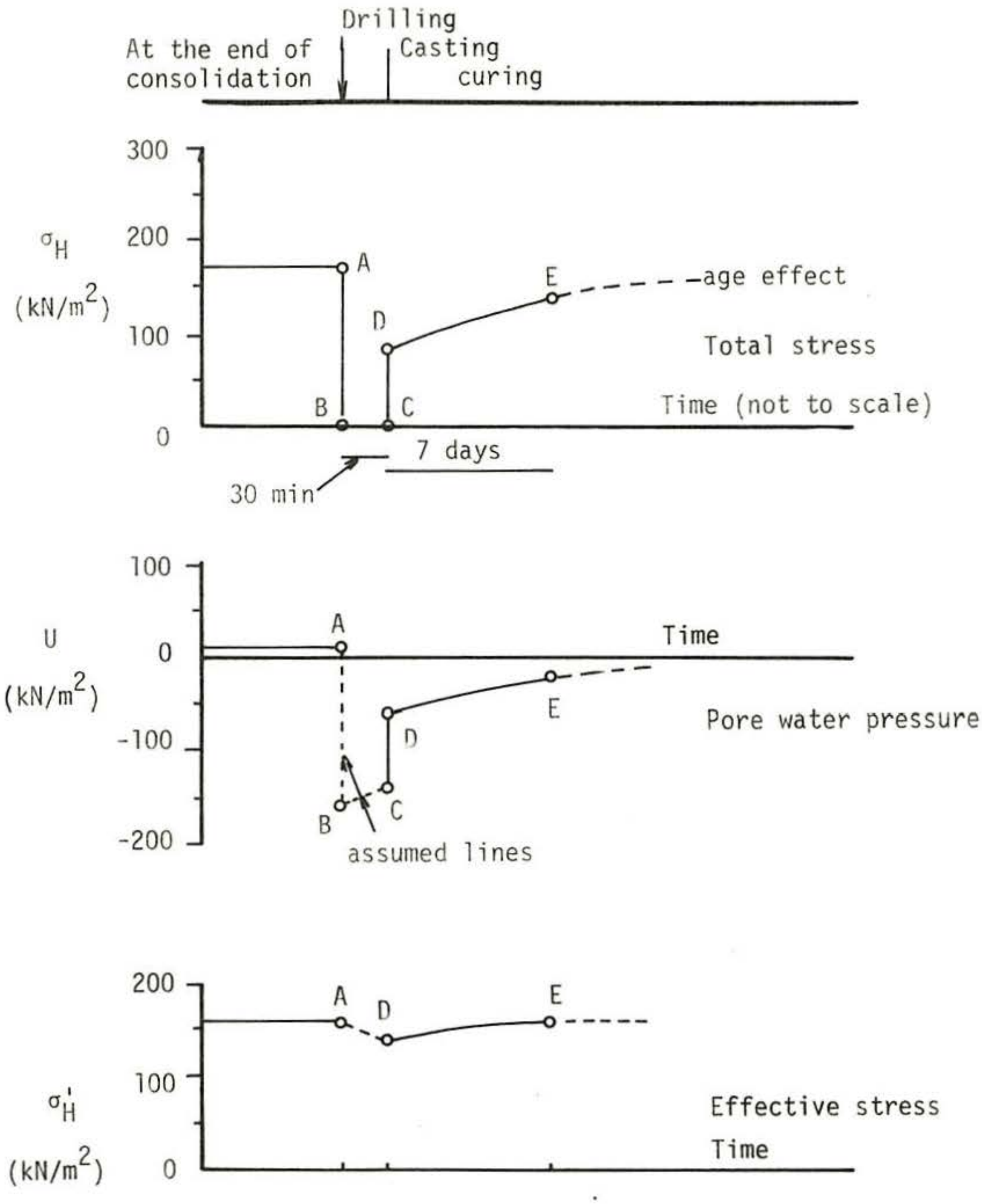


FIGURE 1.10b - VARIATION OF TOTAL HORIZONTAL STRESS, PORE WATER PRESSURE AND EFFECTIVE HORIZONTAL STRESS WITH TIME, FOR OVERCONSOLIDATED KAOLIN, OCR=4, AFTER YONG, 1979

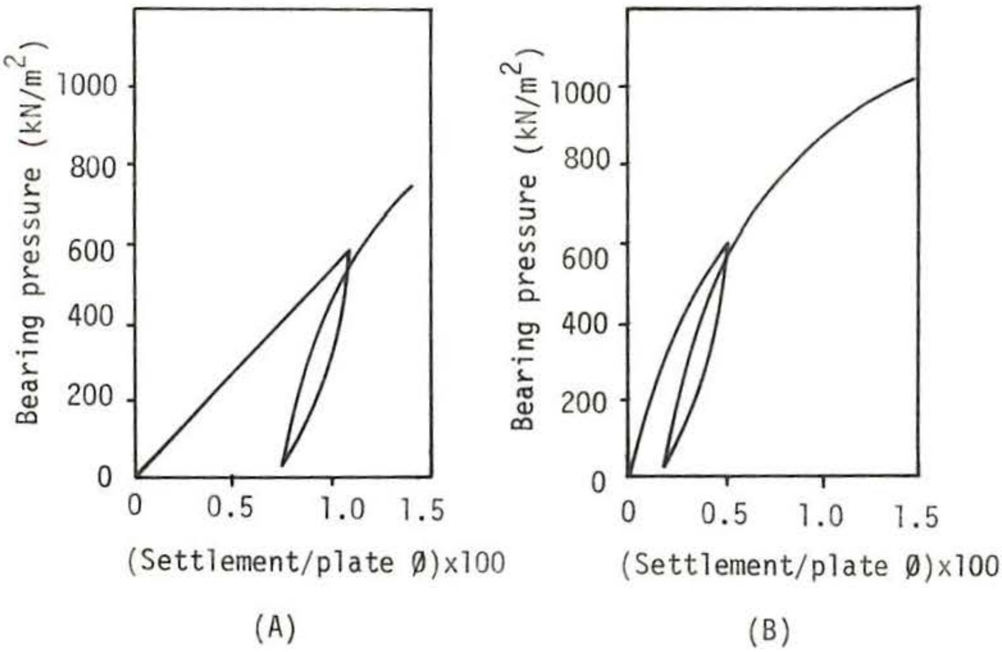


FIGURE 1.11 – TYPICAL LOAD SETTLEMENT CURVES OBTAINED FROM 865 mm DIAMETER PLATE TESTS IN LONDON CLAY SHOWING THE EFFECTS OF THE METHOD OF PREPARATION: (A) MACHINE FINISHED ; (B) HAND FINISHED (EXTRA 50-70 mm OF SOIL REMOVED BY HAND DIGGING (MARSLAND, 1971)

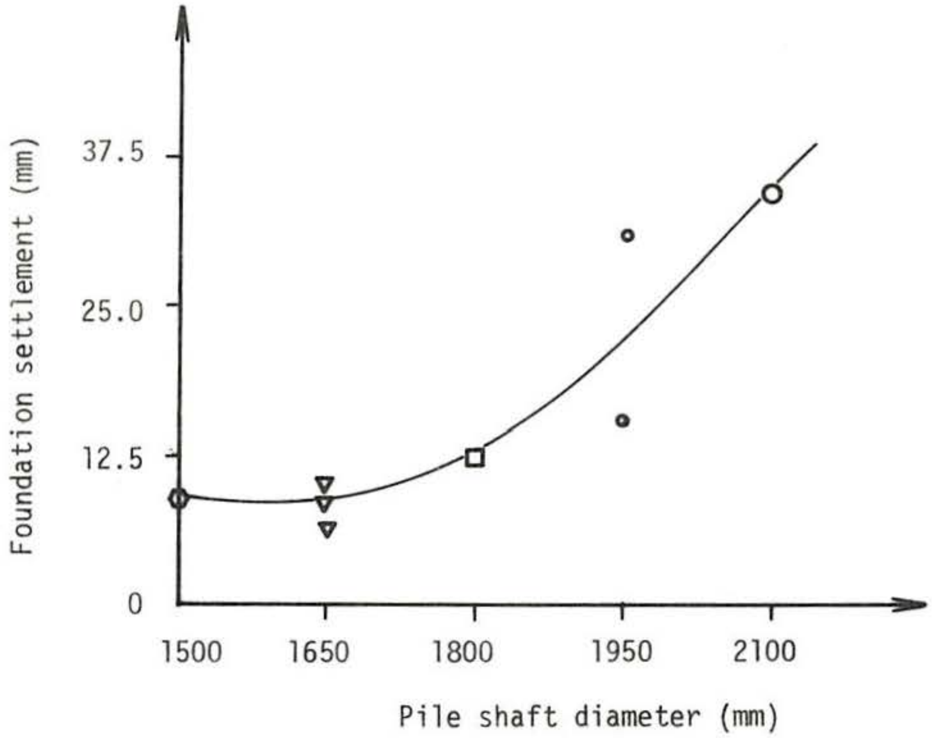


FIGURE 1.12a - FOUNDATION SETTLEMENT VERSUS SHAFT DIAMETER (EDGE OF PILE SHAFTS LOCATED AT A DISTANCE OF 1.20 TO 2.10 m FROM EDGE OF FOUNDATION), AFTER LUKAS AND BAKER, 1978

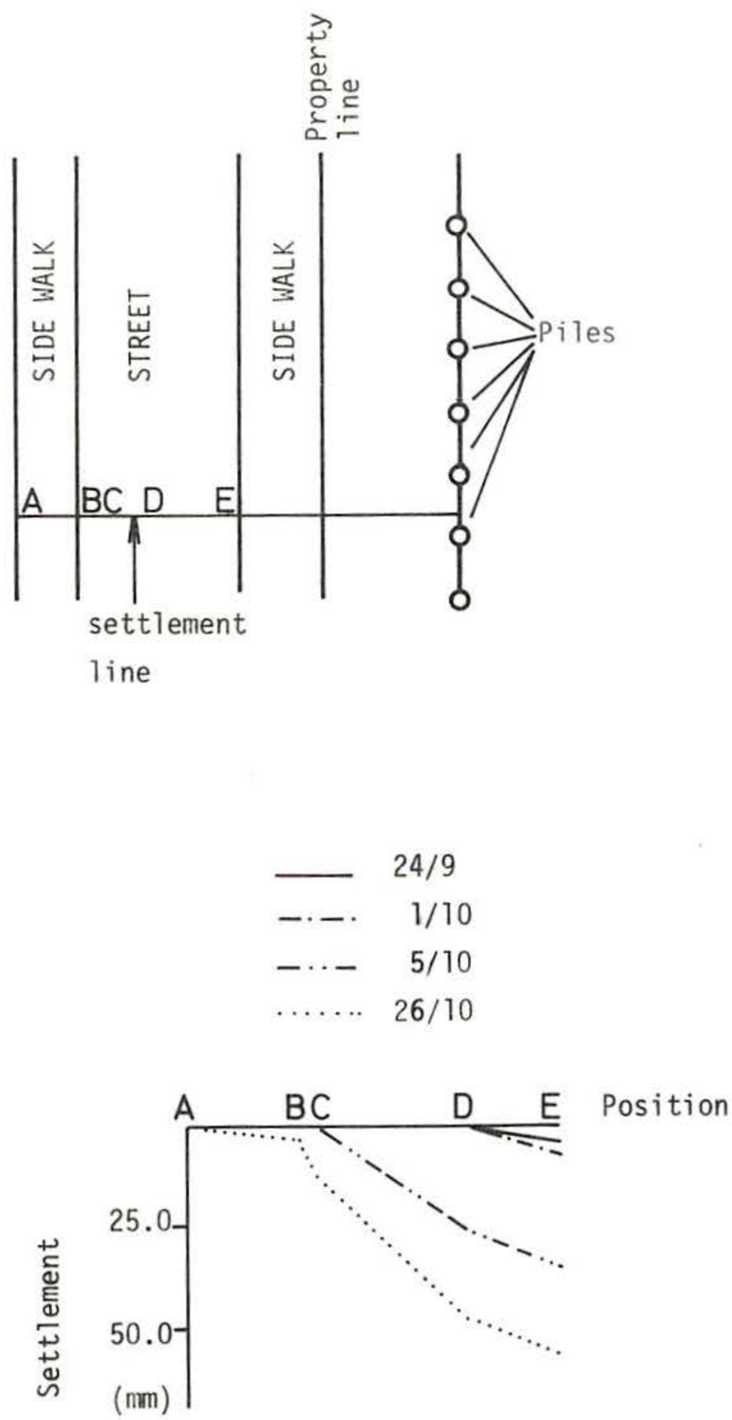


FIGURE 1.12b - STREET SETTLEMENT PROFILE DUE TO BORED PILE INSTALLATION (SITE 3), AFTER LUKAS AND BAKER, 1978

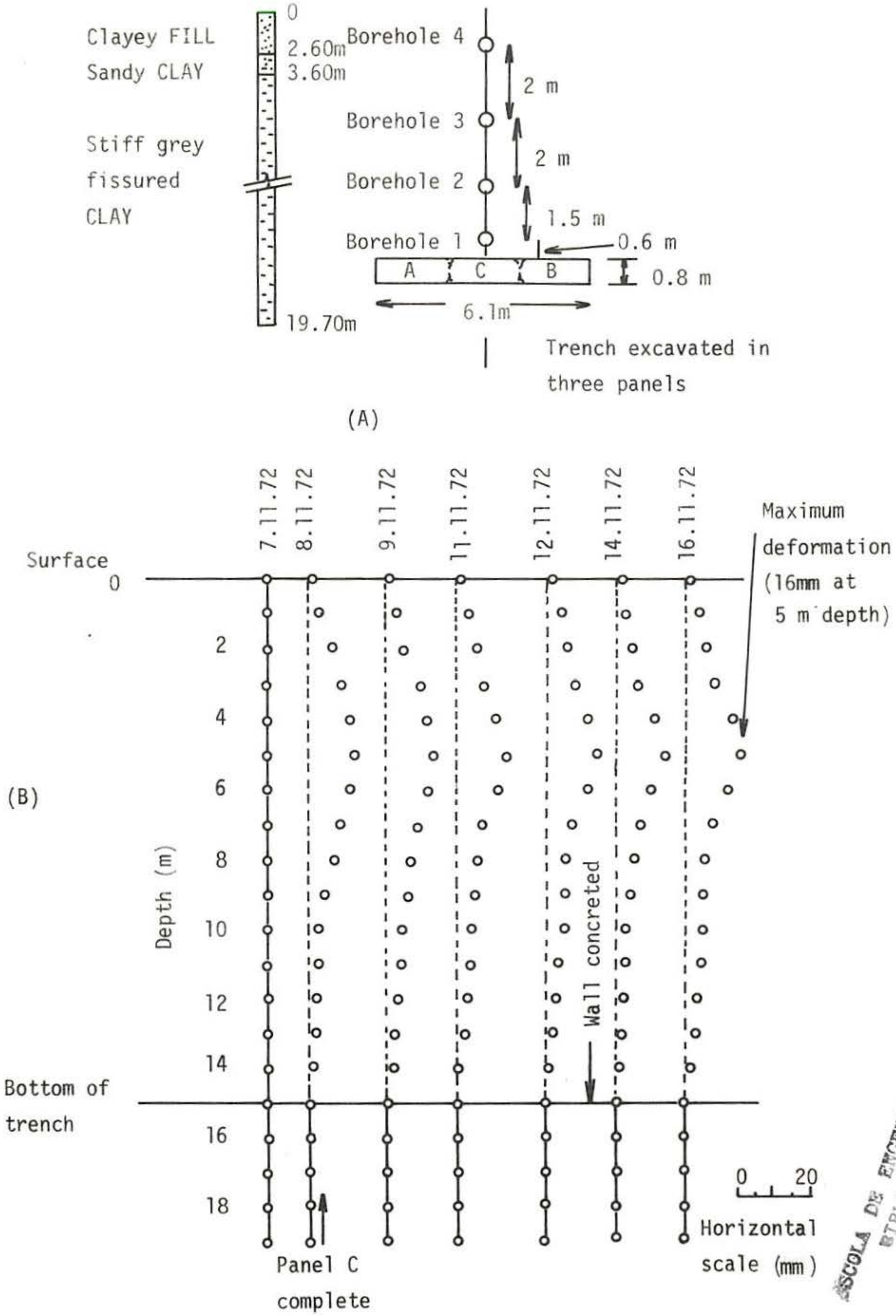


FIGURE 1.13 - FIELD TEST CARRIED OUT TO MONITOR GROUND MOVEMENT CAUSED BY A BENTONITE SUPPORTED EXCAVATION IN LONDON CLAY, BY FARMER AND ATTEWELL, 1973.(A)Experimental layout, (B)Horizontal deformation at borehole 1.

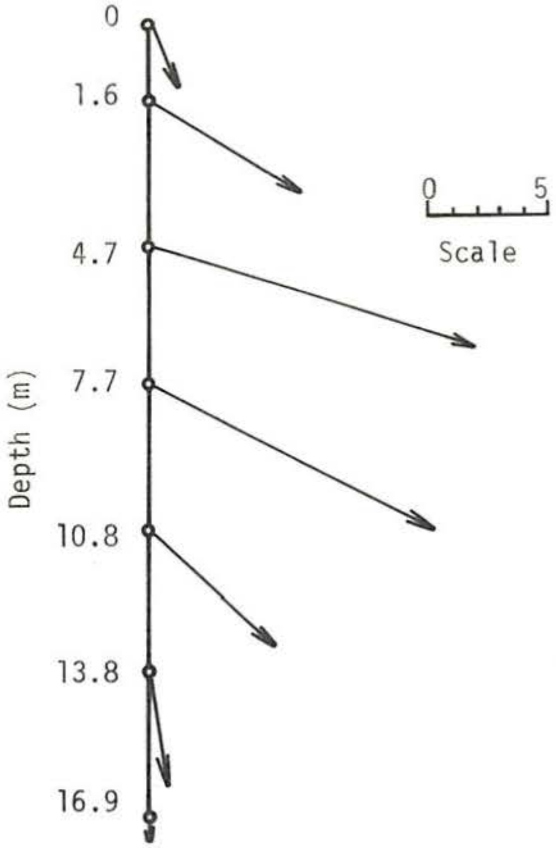


FIGURE 1.13 - CONT. (C) MAXIMUM DEFORMATION VECTORS-BOREHOLE 1

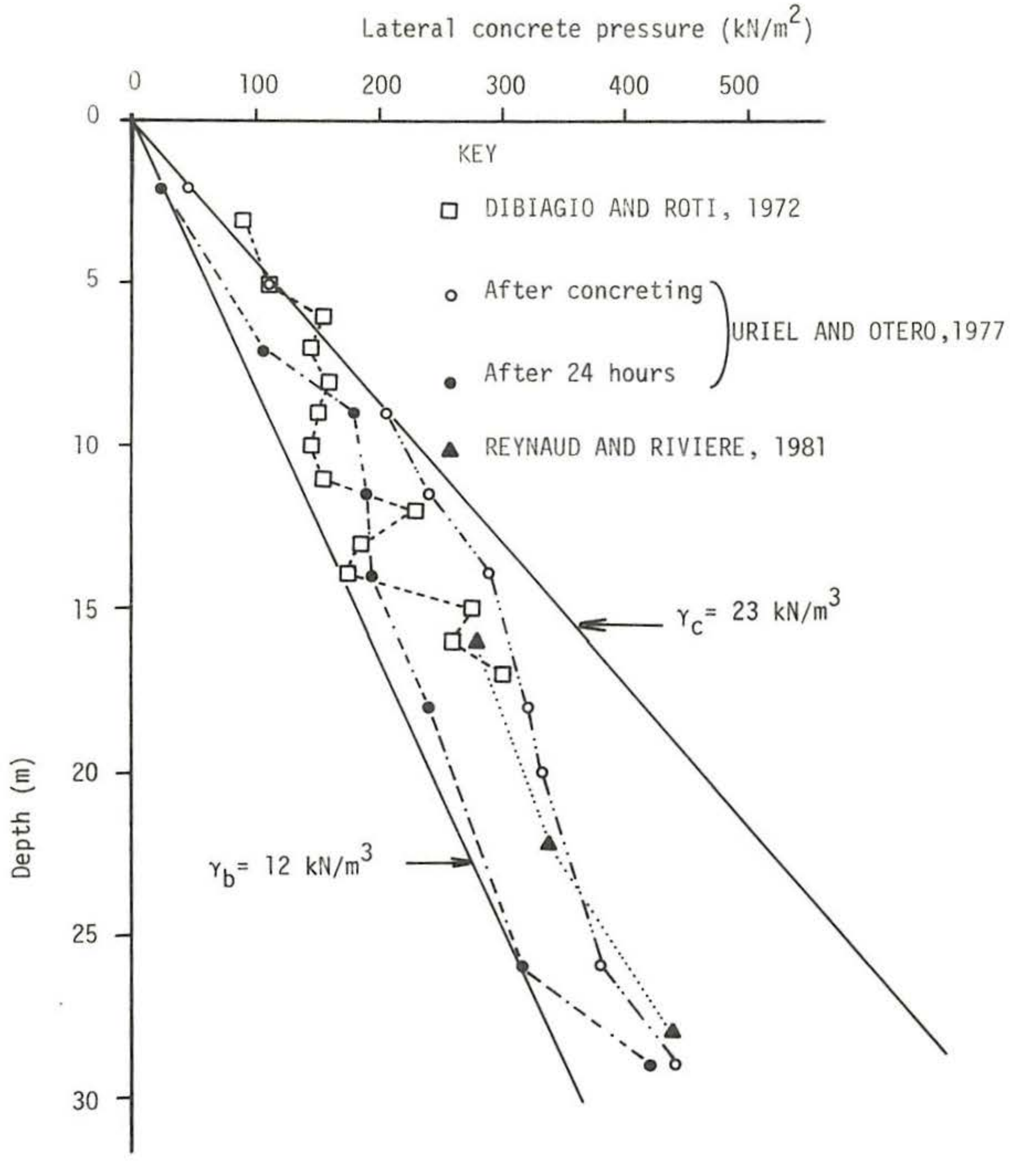


FIGURE 1.14 - AVAILABLE DATA OF TOTAL STRESSES BETWEEN FRESH CONCRETE AND SOIL, AFTER DIBIAGIO AND ROTI, 1972, URIEL AND OTERO, 1977, AND REYNAUD AND RIVIERE, 1981

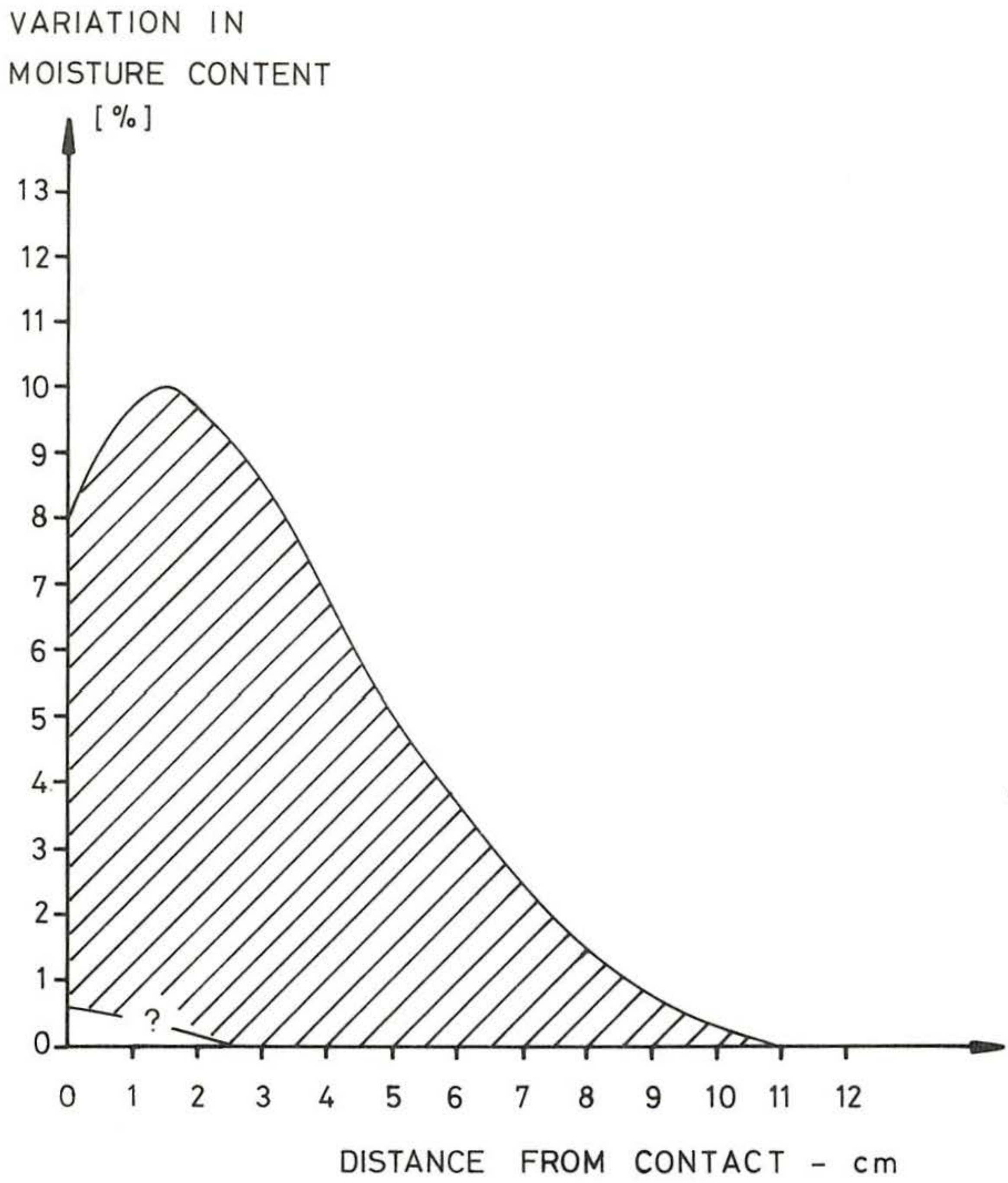


FIGURE 1.15 - AREA COVERED BY ALL AVAILABLE DATA IN THE LITERATURE RELATED TO FIELD MEASUREMENTS OF MOISTURE CONTENT VARIATION NEAR BORED FOUNDATIONS

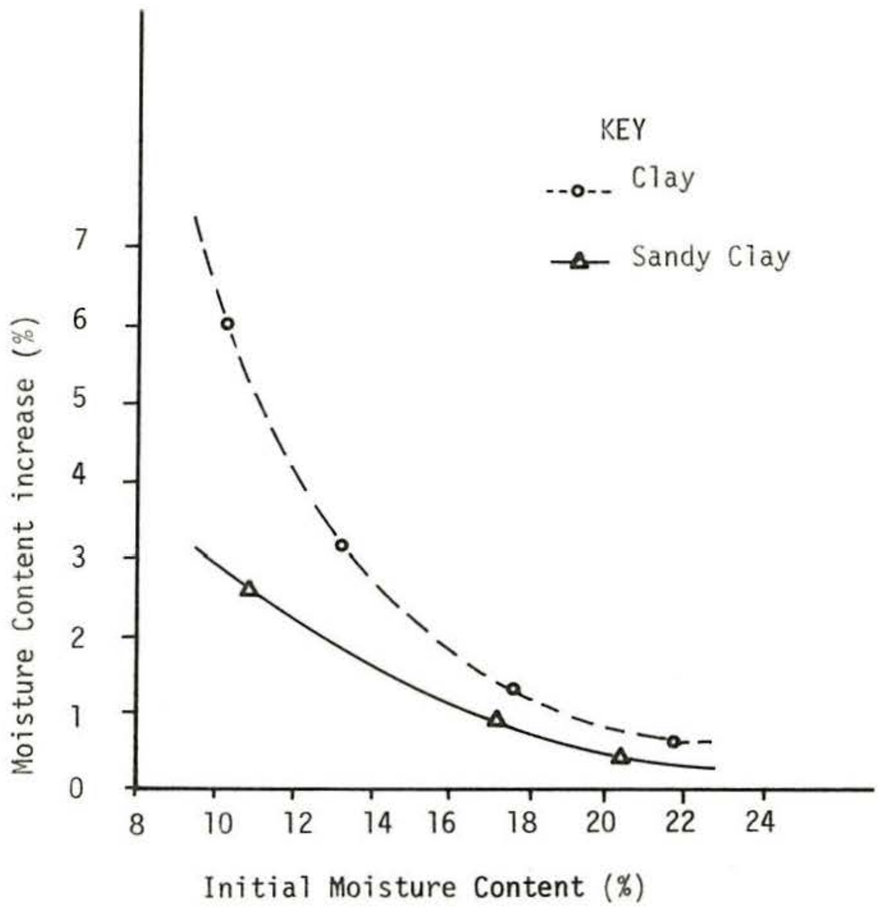
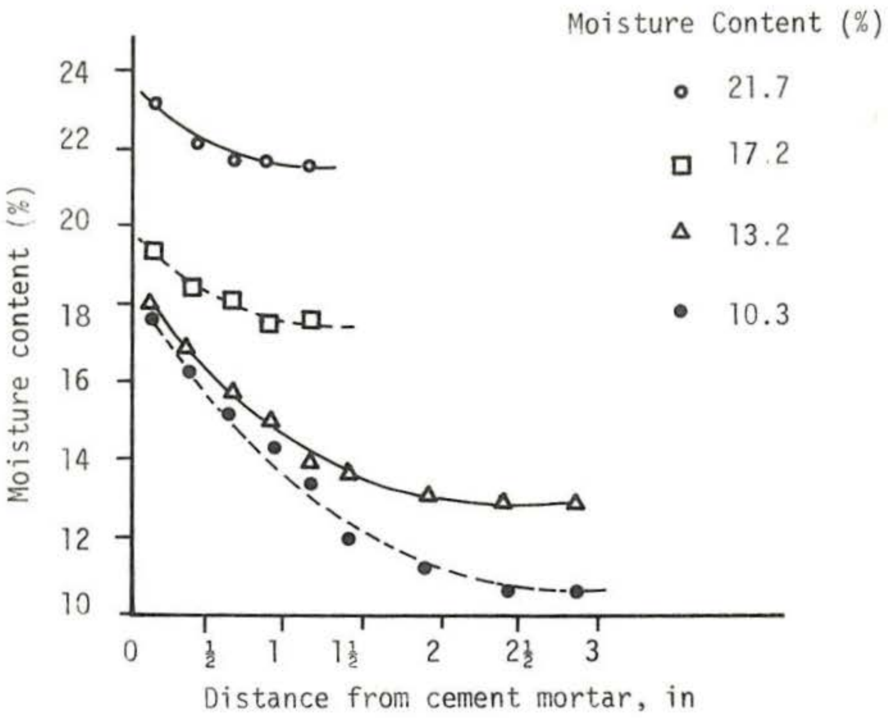
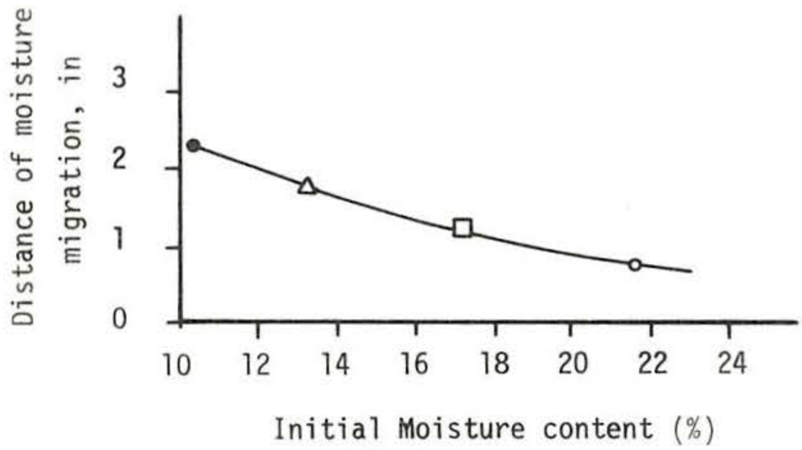


FIGURE 1.16 - RELATION BETWEEN INITIAL MOISTURE CONTENT AND THE AVERAGE MOISTURE CONTENT INCREASE IN THE INCH OF SOIL NEAREST THE INTERFACE BETWEEN CEMENT MORTAR AND SOIL DUE TO MOISTURE MIGRATION (CHUANG AND REESE, 1969)



(A)



(B)

FIGURE 1.17 - (A) DISTRIBUTION OF MOISTURE CONTENT IN CLAY AT DIFFERENT INITIAL MOISTURE CONTENTS,(B) RELATION BETWEEN DISTANCE OF MOISTURE MIGRATION AND INITIAL MOISTURE CONTENT (CHUANG AND REESE, 1969)

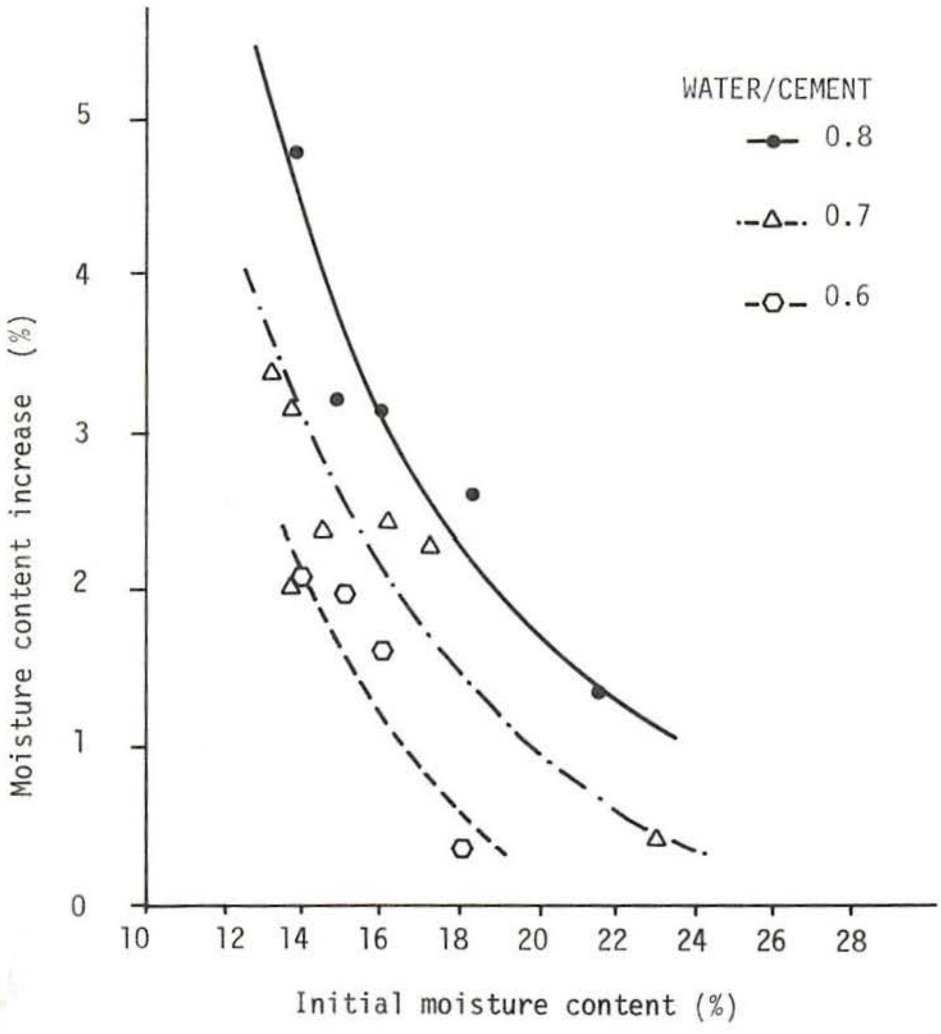
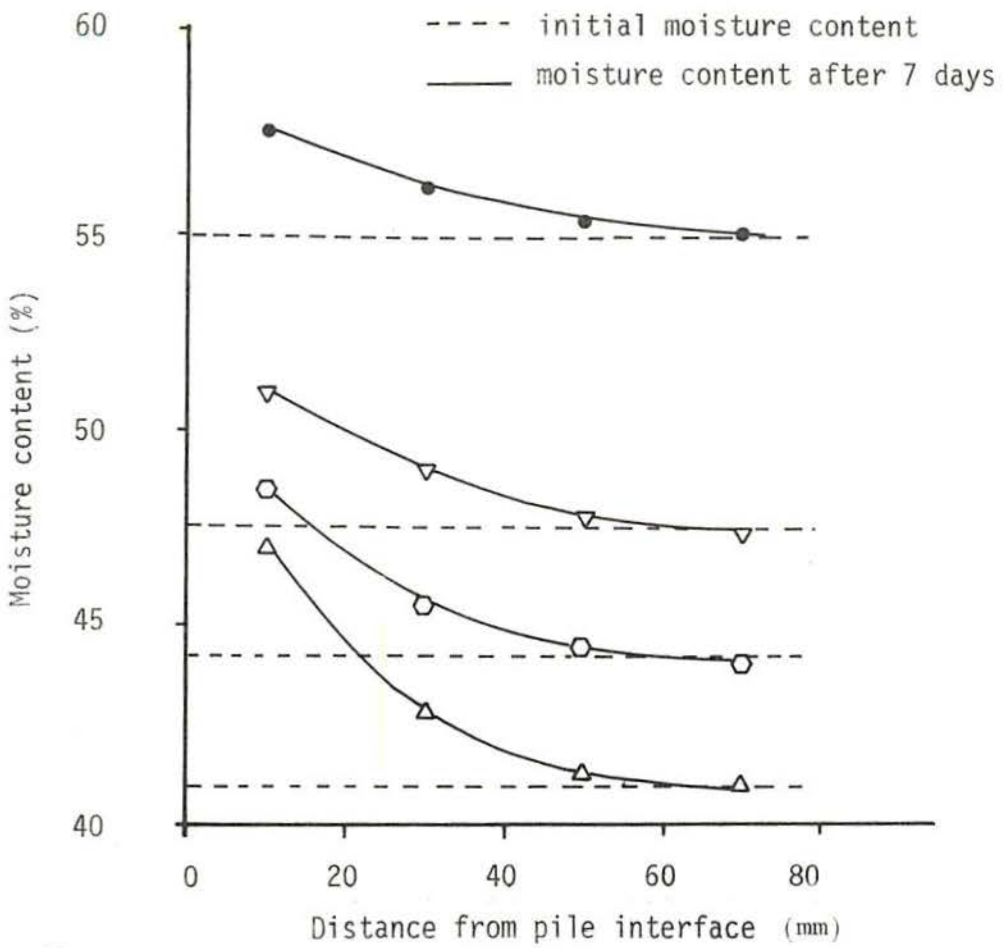


FIGURE 1.18 - RELATIONSHIP BETWEEN INITIAL MOISTURE CONTENT AND MOISTURE CONTENT INCREASE MIGRATED FROM FRESH MORTAR TO UNDISTURBED SOIL (MONTOPOLIS CLAY), AFTER CHUANG AND REESE, 1969



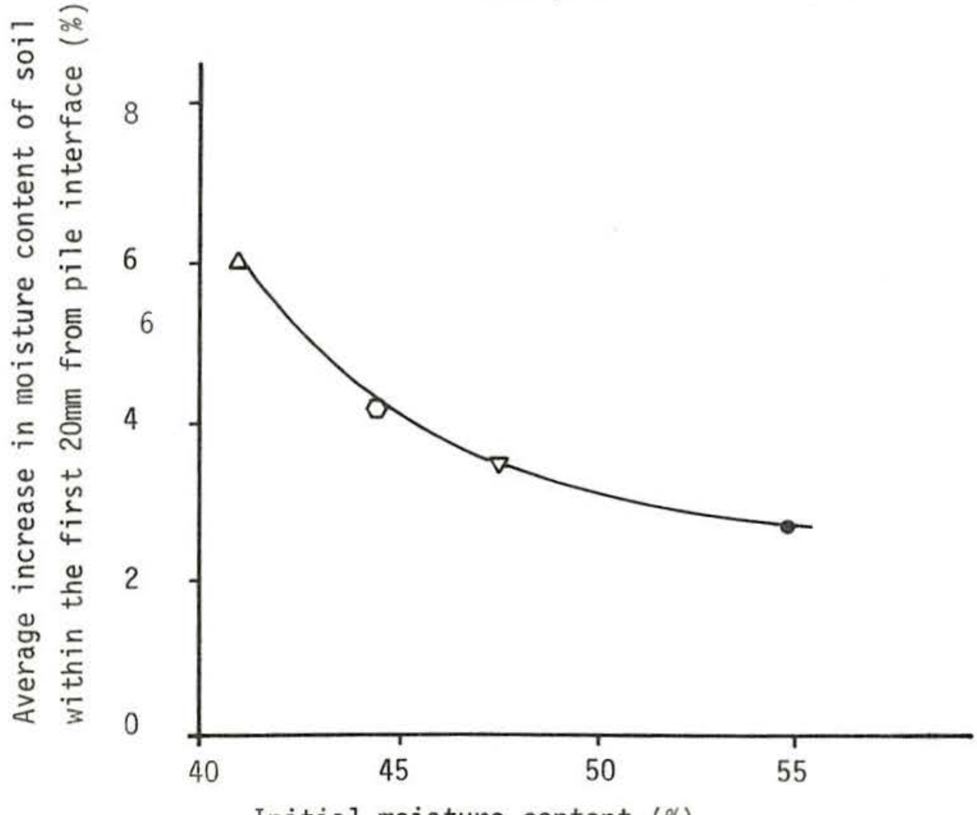


FIGURE 1.19 - INFLUENCE OF INITIAL MOISTURE CONTENT ON MOISTURE MIGRATION IN SOIL SURROUNDING THE PILE, FOR A WATER CEMENT RATIO OF 0.6, AT MID-HEIGHT OF A 38 mm DIAMETER PILE (YONG,1979)

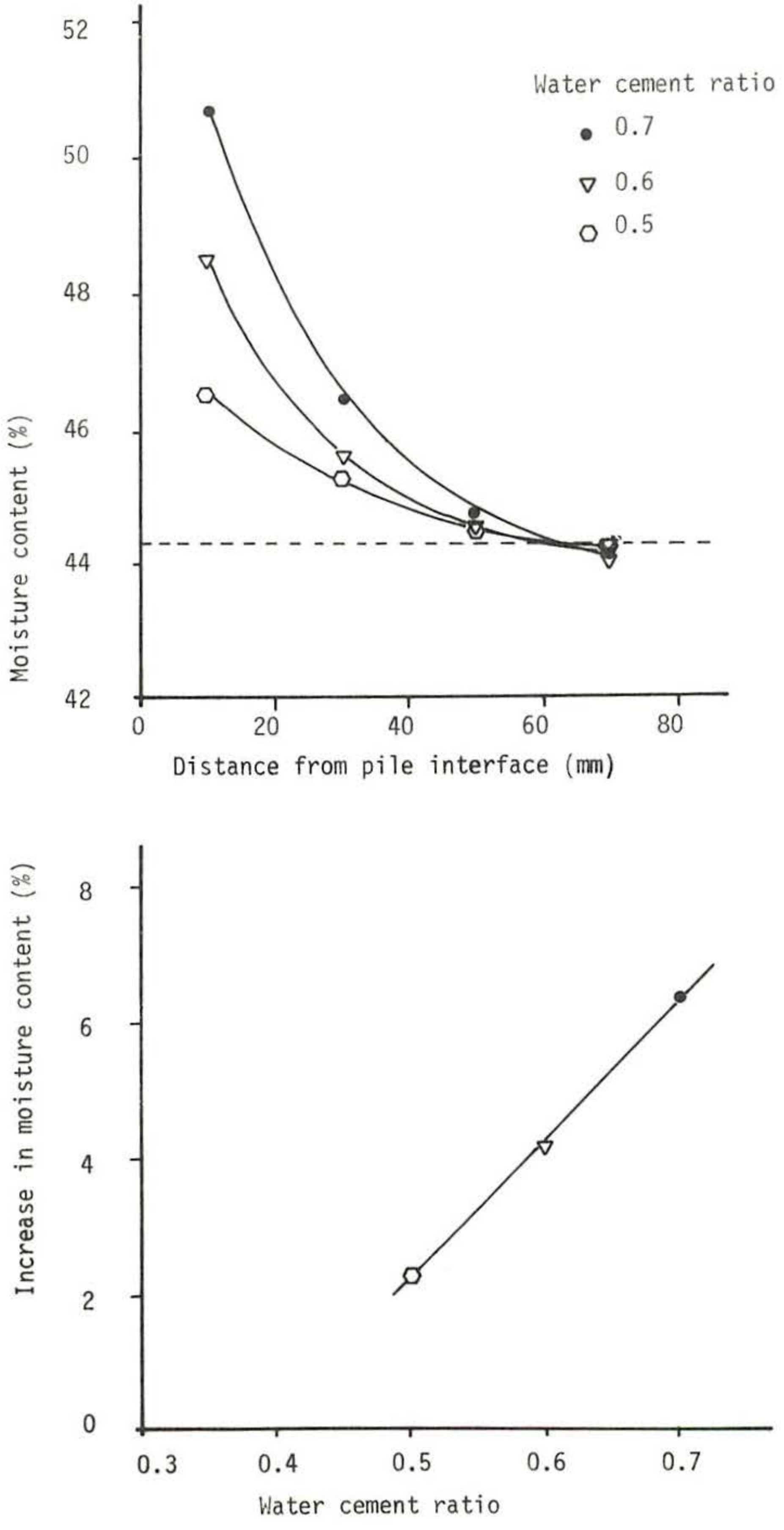


FIGURE 1.20 - INFLUENCE OF WATER-CEMENT RATIO ON MOISTURE MIGRATION IN SOIL (KAOLIN) SURROUNDING THE PILE, AT MID-HEIGHT OF A 38 mm DIAMETER PILE (YONG, 1979)

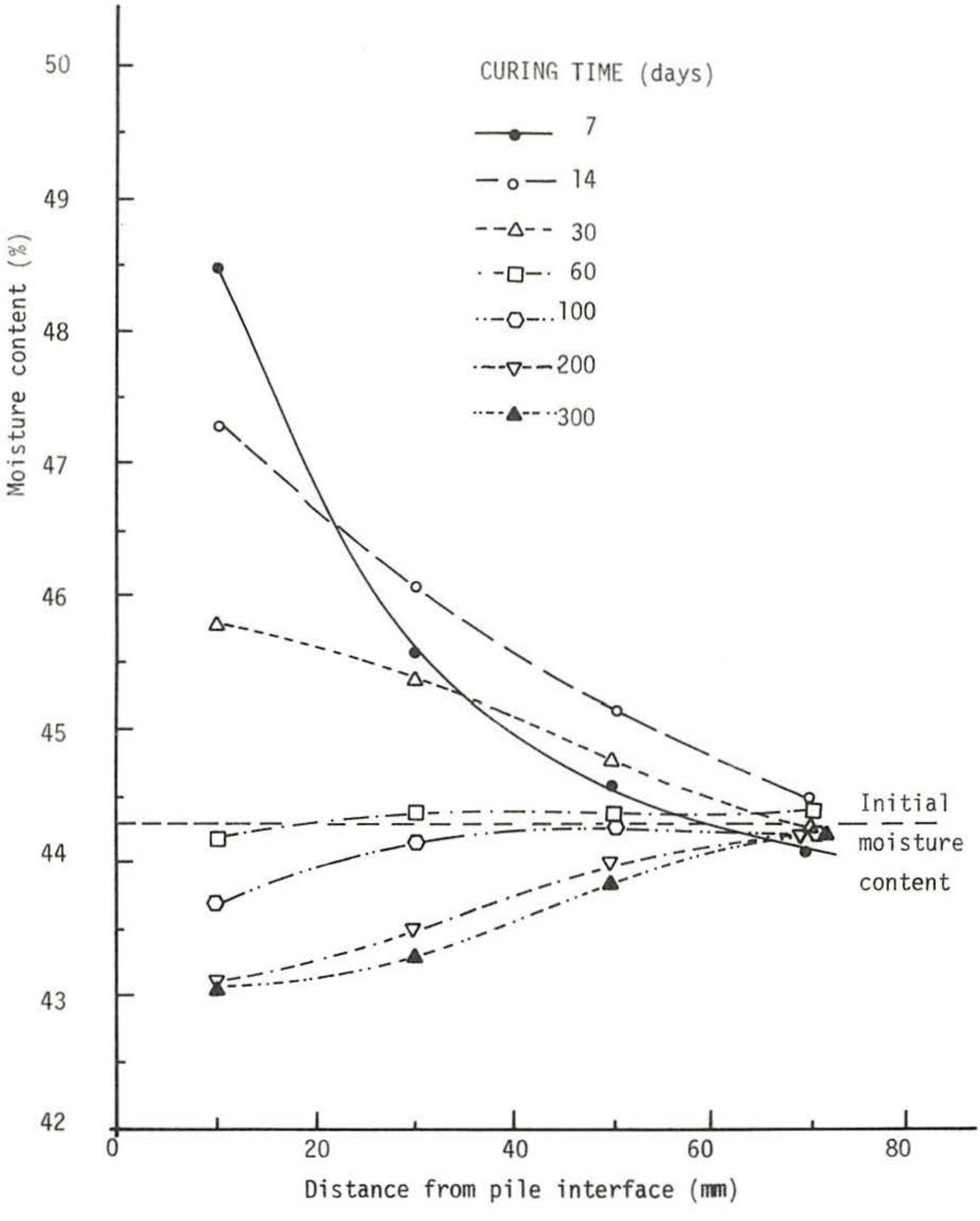


FIGURE 1.21 - VARIATION OF MOISTURE CONTENT WITH CURING TIME (WATER CEMENT RATIO OF 0.6, PILE DIAMETER OF 38 mm), AFTER YONG, 1979

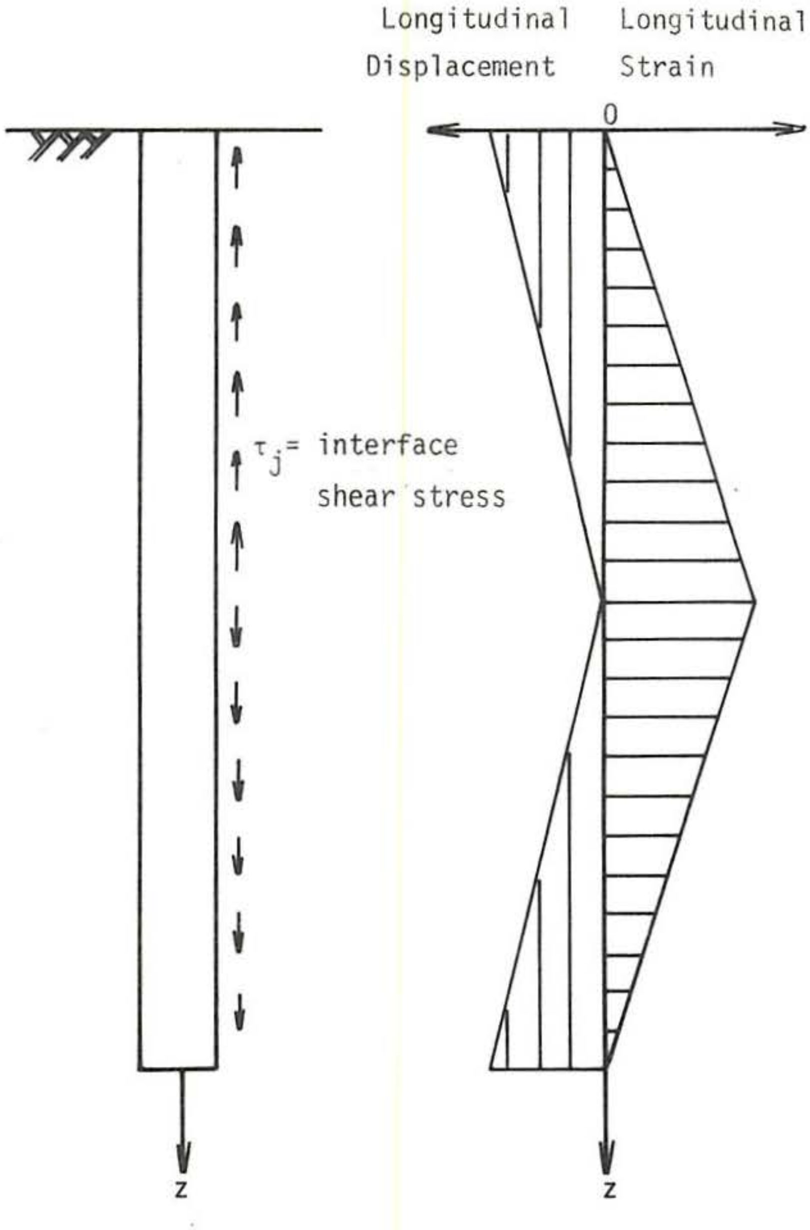


FIGURE 1.22 - TYPICAL SHEAR STRESS AND DISPLACEMENT PATTERN CAUSED BY COMPRESSIVE THERMAL STRAIN DURING CONCRETE CURING (AFTER TOUMA AND REESE, 1972)

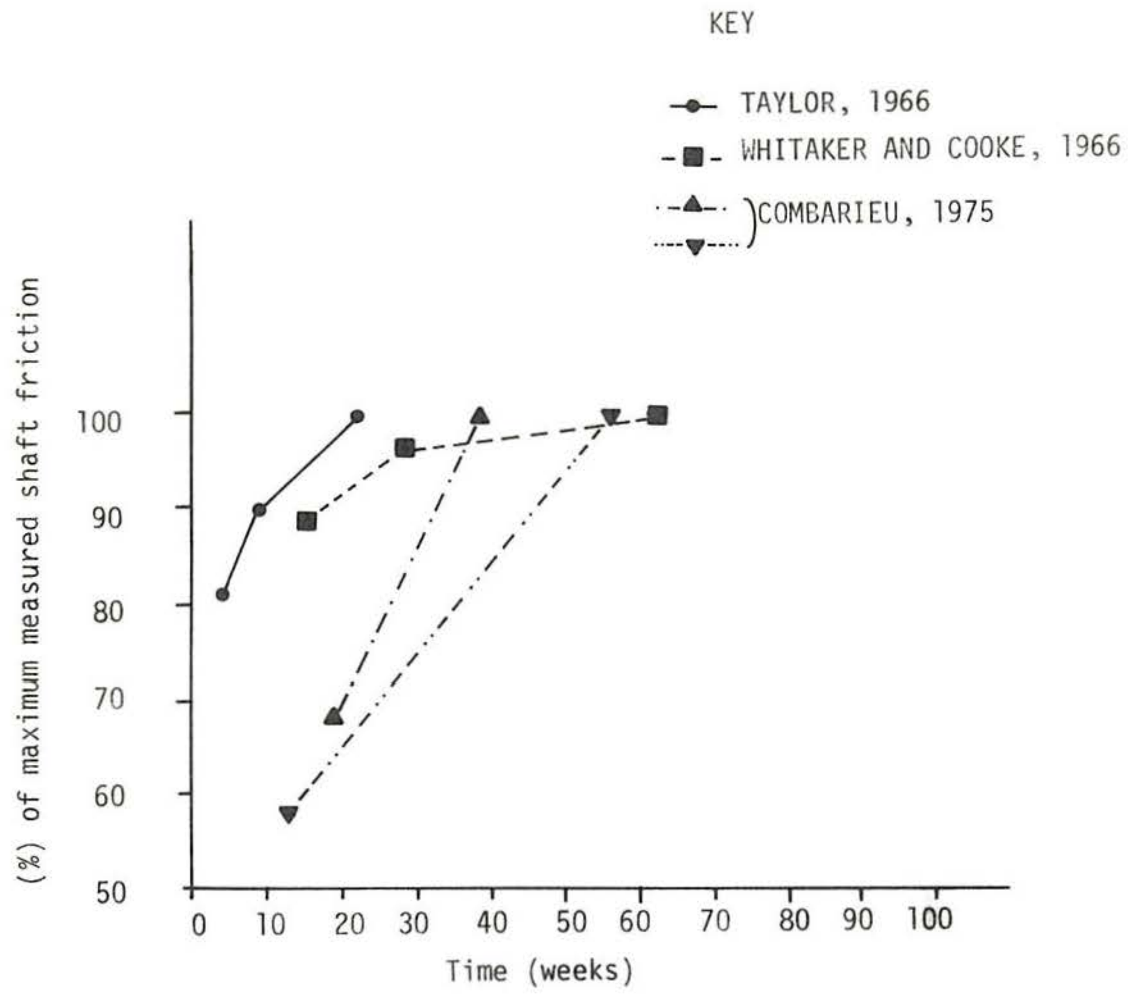


FIGURE 1.23 - EVOLUTION OF SHAFT FRICTION ON BORED CAST IN SITU PILES WITH TIME

CHAPTER 2 - LABORATORY STUDIES OF MOISTURE CONTENT AND UNDRAINED SHEAR STRENGTH VARIATIONS DUE TO THE CONTACT OF CLAY SPECIMENS WITH FRESH CONCRETE.

CHAPTER 2 - LABORATORY STUDIES OF MOISTURE CONTENT AND UNDRAINED SHEAR STRENGTH VARIATIONS DUE TO THE CONTACT OF CLAY SPECIMENS WITH FRESH CONCRETE.

2.1. - INTRODUCTION

To start the study of local changes in soil conditions due to the installation of bored piles in clays, it was decided to simulate some of the conditions arising from pile construction using laboratory experiments.

Investigations of moisture content and undrained shear strength variations in clays due to the presence of fresh concrete are reported in this chapter.

The idea of simulating complex events in simple laboratory experiments to study specific aspects is not new. Referring to the specific topic under consideration the following references were found: Taylor, 1966; Chuang and Reese, 1969; O'Neill and Reese, 1970 and Yong, 1979.

In order to study the variation in strength, the cone test device (BS 1377-Test 2 a), but with a different weight, was used. An "Index strength" was proposed to compare changes in undrained shear strength at different distances from the contact between concrete and soil. The major advantage of such an approach is the possibility of performing a number of tests in one specimen due to the small area affected by each test.

Both natural and compacted soils in contact with fresh concrete were tested. The effects of age of testing and water cement ratio were investigated.

In this chapter the technique proposed the definition and justification of use of the "Index strength", tests performed, results, comments and recommendations will be presented.

Table 2.1 summarizes the test programme performed.

Series	Material used	Specimen used diameter (mm)	length (mm)	water/cement ratio	Time of testing (days)	Remarks
A	Undisturbed Boulder Clay	100	140	0.8	7	Effect of fresh concrete on moisture content and cone penetration
B	Compacted Boulder Clay	100	80	0.8	7	Effect of fresh concrete on moisture content and cone penetration
C	Undisturbed London Clay	100	80	0.8	7	Effect of fresh concrete on moisture content and cone penetration
D	Undisturbed London Clay	100	100	-	no contact	Natural variation of properties
E	Undisturbed and	100	80	-	0	Natural variation of properties
	Compacted London Clay			0.8 0.8	1 6	Effect of time on test results, natural versus compacted specimen
F	Undisturbed London Clay	100	80	0.6	0 and 7	Effect of fresh concrete on moisture content and cone penetration
G	Undisturbed London Clay	38	80	no concrete 0.5 0.8	0 7	Natural variation of properties Effect of water/cement ratio on test results

Table 2.1 - Summary of the test programme performed.

2.2. - CONE PENETRATION AS A MEASUREMENT OF "INDEX STRENGTH" - I_s

2.2.1. - INTRODUCTION

Shear strength of soil can be measured directly or indirectly with many different instruments. The results, however, depend on the actual conditions imposed by the type of test. In the case of fissured materials, like the London Clay, the so called "undrained shear strength" is not just test dependent but, even for the same test (undrained triaxial compression, for example) it depends on the size of the specimens being tested (Ward, Marsland and Samuels, 1965; Bishop, 1966; Simons, 1967; Bishop and Little, 1967; Agarwal, 1968).

When a decision was made to study the effect of the fresh concrete on the strength of London Clay the next step was to decide on the test to be used to measure strength. The necessity of studying possible variations of strength in the same sample at different positions and of using the specimen for testing at different ages required the choice of a testing technique involving a small amount of material.

As the main concern of the research was to identify mechanisms and phenomena, instead of providing numerical values of properties, an index test was chosen. The Cone Penetrometer or Fall Cone test used for many years as a quick measurement of strength (Hansbo, 1957, Kezdi, 1980) was chosen.

The cone penetrometer test was developed by the Geotechnical Commission of the Swedish State Railways between 1914 and 1922. There are a number of references in the literature covering its use as a way of measuring shear strength (Skempton and Bishop, 1950; Caldenius and Lundstrom, 1956; Hansbo, 1957; Youssef et al, 1965; Wroth and Wood,1978;

Kezdi, 1980) and liquid limit (Terzaghi, 1927; Sowers et al, 1959; Scherrer, 1961; Sherwood and Ryley, 1968; 1970; Skopek and Ter-Stepanian, 1975; Littleton and Farmillos, 1977; Wood and Wroth, 1978; Clayton and Jukes, 1978; Nagarev et al, 1981 and Wood, 1982).

2.2.2. - THE TEST

Basically the test is carried out as follows: a metal "standard" cone is placed vertically with its apex just in contact with the top surface of the clay sample to be tested (Figure 2.1b). The cone is then dropped freely into the clay and the depth of penetration measured.

The cone used in the present research was defined in BS 1377:1975 Test 2 (a). It has an apex angle of 30^{o} and a weight of 80 grammes.

The "standard" cone proved to be too light for use in the material being tested. Hansbo, 1957, suggested the use of a 400 grammes cone for stiff clays. It was decided in the present work to use a weight of 650 grammes in order to avoid the possible effect of disturbance (referred to by Hansbo, 1957) when preparing the sample surface for testing.

2.2.3. - SHEAR STRENGTH DETERMINATION USING THE CONE

The relationship derived by Hansbo, 1957, with detailed considerations of the mechanics of the cone penetration and supported by experimental evidence was :

$$\frac{S_u\ P^2}{W} = \text{constant} \qquad \text{Equation 2.1}$$

Where:

S_u = undrained shear strength

P = cone penetration

W = weight of the cone

Equation 2.1, according to Wroth and Wood, 1977, is also obtainable by dimensional analysis (neglecting rate effects).

Hansbo stated that the constant depends mainly on the cone angle but is also influenced by the rate of shear and by the sensitivity of the clay. Comparisons between cone results and other tests (Figure 2.1c, after Hansbo, 1957) can provide values of the constant in Equation 2.1, allowing the determination of the undrained shear strength in a simple and quick way.

The London Clay used in this research was subjected to heavy compaction (BS - 1377 : 1975, Test 13) at different moisture contents. Three specimens (38 mm diameter, 76.2 mm high) were extruded from each mould and tested both using the cone and the standard procedure for undrained triaxial test (rate of deformation of 1.52 mm/min). The results obtained are presented in Table 2.1a and Figure 2.1d.

The scatter of the results obtained is comparable with that measured by Hansbo, with a clearly recognizable trend. The number of test results and range of undrained shear strength values is not enough to define a relationship between cone penetration and undrained shear strength determined with the triaxial. For the results obtained, upper and lower limits are suggested in Figure 2.1d.

Due to the special nature of the material used in the present research (stiff fissured clay), it was decided to adopt the result of the penetration as a measurement of a comparative strength, i.e. an index value.

Sample	D			M			S		
Specimen	D1	D2	D3	M1	M2	M3	S1	S2	S3
Bulk density (Mg/m^3)	1.90	1.90	1.90	1.87	1.89	1.86	1.85	1.86	1.88
Dry density (Mg/m^3)	1.41	1.43	1.43	1.39	1.42	1.39	1.35	1.36	1.38
Moisture content (%)	34.3	32.8	33.0	34.7	33.1	33.9	36.7	36.5	36.0
Undrained triaxial test									
Confining pressure (kN/m^2)	100	200	400	100	200	400	100	200	400
Undrained shear strength (S_u) (kN/m^2)	56	68	67	50	62	62	46	50	55
Cone test									
Cone penetration, average (mm)	7.0	6.3	6.0	7.2	6.8	6.4	9.0	7.3	8.4
Number of tests	9	9	9	6	9	9	6	6	6

TABLE 2.1a – Comparison of test results for specimens of compacted London Clay essayed for cone penetration and undrained triaxial tests

2.2.4. - THE "INDEX STRENGTH" (I_s)

From equation 2.1, expressing the undrained shear strength as a function of the other variables :

$$S_u = \frac{W \cdot K}{P^2} \qquad \text{Equation 2.2}$$

where:

K = constant for a given soil

When performing a series of tests on a sample the weight of the cone and K does not change. A more general constant can be considered:

$$S_u = \frac{\overline{K}}{P^2} \qquad \text{Equation 2.3}$$

where:

$\overline{K}$ = constant for a given soil and a given cone weight

As the interest in the present research is to make comparisons of strength, it was decided to establish an "Index strength" , I_s, as a way of measuring the effect of the fresh concrete in the strength of the material (soil).

The proposed index can be defined as follows:

$$I_s = \frac{1000}{P^2} \qquad \text{Equation 2.4}$$

where:

I_s = "Index strength"

P = cone penetration (mm)

1000 = reference constant

Cone penetration (mm)	I_s
1	1000
2	250
3	111
4	63
5	40
6	28
7	20
8	16
9	12
10	10

TABLE 2.1b - Proposed "Index strength" I_s versus cone penetration (mm)

2.3. - TESTING PROGRAMME

2.3.1. - OBJECTIVES OF THE STUDY

The main objective of the programme was to establish a procedure capable of producing information about the effects of the contact of fresh concrete on moisture content and shear strength of adjacent clay specimens. The control of stress conditions and the possibility of testing the same specimen at different positions and different age was a priority in the establishment of the procedure.

The available studies were made using direct shear boxes (Chuang and Reese, 1969; O'Neill and Reese, 1970) or laboratory vane (Yong, 1979) to measure the shear strength at different distances from the contact between soil and concrete.

As the test conditions in the shear box are difficult to control, stresses and displacements are non-homogeneous and, according to Morgenstern and Tchalenko, 1967, the major discontinuities which appear are Riedel shears and these structures are oblique to the horizontal. Kinematic restraint is an important feature of the test. Probably there is no way of comparing values obtained at the contact between concrete and soil and close to it with those obtained far from that region using direct shear box tests. Apart from those problems each sample could be tested just once; for any comparison different specimens should be tested.

The major limitations of the work done with the laboratory vane are the area affected by each tests (the tests being made at 20, 40, 60 and 80 mm from the contact of soil and concrete), it being impossible to test the soil at a closer distance from the interface, and the materials used to simulate the pile-soil interaction (kaolin and micro-concrete).

Considering all these aspects it was concluded that a new evaluation of the effect of fresh concrete in clay specimens was needed.

2.3.2. - TESTING PROCEDURES

Two types of procedure were used in the programme: the first was to test the clay specimens on the surface (when the same specimen was tested at different ages) and no moisture content was determined. The second was to test the sample for strength and moisture determination. In the second case the sample was sliced, tested with the cone on the new surface created by the cut and the entire section oven dried for moisture content determination.

The test setup is shown in Figure 2.2. A standard triaxial cell was used to mantain a cylindrical specimen under an isotropic state of stresses.

Sample formation was obtained by compaction or extrusion of natural samples from U 100 samplers.

When compacted samples were used, usually a reference specimen was made to study the variation of initial moisture content. In the case of extruded samples the moisture content of both ends was measured.

The first step after sample formation was dependent on the purpose of the test. Some tests were made to study the natural variability of moisture and "Index strength". In these cases the specimens were tested immediately after the specimen was obtained. When the effect of contact with fresh concrete was the feature under question, the specimen was placed inside a standard triaxial cell and reconsolidated up to the desired pressure (usually during one week). After reconsolidation the confining pressure was released (with drainage closed), the cell opened and a cylindrical mould was placed on the top of the sample. The mould was filled with a cement-sand mortar (1 of cement and 3 of sand), at a specific water-cement ratio.

The top cap was replaced, after careful cleaning of the membrane, and the confining pressure re-established. During that phase of the experiment the drainage was closed. At the desired moment, the confining stresses were relieved, the cell was opened, the membrane was unfolded to expose the side of the sample to testing with the cone. The sample was replaced in the same conditions and the confining pressure was re-established.

For the determination of moisture content (when that was required), the sample was split and cone testing was performed inside the sample before sampling for moisture evaluation (all cross section used).

At each distance from the contact between the mortar and clay, three penetration tests were made, averaged, and presented as a single value in the results.

Tables 2.1c and 2.1d presents a general description of the soil strata for both materials used for testing (London Clay and Boulder Clay).

DEPTH	SOIL DESCRIPTION
0.00	Made ground
1.50	Stiff brown fissured silty CLAY with some grey staining in the fissures, occasional lenses of brown sand in the upper levels and selenite crystals. (LONDON CLAY)
7.00	Stiff to very stiff dark grey fissured silty CLAY (LONDON CLAY)
20.00	Bottom of borehole

Table 2.1c - Soil profile identified from the site investigation report, Wimbledon, UK

DEPTH	SOIL DESCRIPTION
0.00	Topsoil
0.35	Made ground
1.20	Soft to firm red-brown silty sandy CLAY with partings of sand
4.00	Stiff red-brown sandy CLAY (BOULDER CLAY)
12.50	Bottom of borehole

Table 2.1d - Soil profile identified from the site investigation report, Derby, UK

2.4. - RESULTS

In this section the description of all tests performed and results obtained will be presented. Some aspects of the cone penetration test discovered during the tests were:

- the presence of a fissure strongly affects the penetration measured, reducing the strength;

- results are affected by selenite crystals, common in London Clay, which increase the measured strength;

- precise location of the cone's point on the surface to be tested is an essential part of the test (otherwise misleading results can be obtained);

- tests must be performed as quickly as possible to minimize the possibility of variation in moisture content due to evaporation.

2.4.1. - CONE PENETRATION AND MOISTURE CONTENT VARIATION IN SAMPLES WITHOUT CONTACT WITH FRESH CONCRETE.

To study the natural variation in moisture content and cone penetration, tests D,E and G1 were performed.

Figure 2.3 shows the cone penetration, moisture content and "Index strength" for two samples of "undisturbed" London Clay (tests D1 and D2), sampled from 6.50 to 7.00 m depth. The samples are described as stiff fissured brown silty clay. Table 2.2 presents the results obtained. In sample D1 the penetration test was performed in two orthogonal directions as shown in Figure 2.3a. The average "Index strength" obtained in sample D1 was 74, the maximum difference from average was 9 and 7% respectively in the two directions tested. The test performed in sample D2 (very fissured) showed a bigger scatter, the average "Index strength" obtained being equal to 7.6 and the maximum difference from the average being 21%.

The results of the tests performed in sample E ("undisturbed" London Clay, firm brown silty clay) are shown in Table 2.3 and Figure 2.4.

Table 2.4 and Figure 2.5 present the results obtained with sample G (undisturbed London Clay without contact with fresh concrete).

As a conclusion of the study of the variation of "Index strength" obtained testing samples without contact with fresh concrete it can be said that a scatter in values is observed and the maximum difference from the average varies from sample to sample, the minimum value obtained was 7%, the maximum was 21%. The moisture content variation observed was randomly distributed.

2.4.2. - EFFECT OF FRESH CONCRETE ON MOISTURE CONTENT DISTRIBUTION AND "INDEX STRENGTH".

Three experiments were made to study the effect of the fresh concrete in contact with soil. The description of the tests and results obtained will be presented with comments.

TEST A -

A sample of Boulder Clay (brown sandy silty clay) recovered from 4.50 m depth using a U 100 sampler was tested. The sample preparation and the testing procedures followed the description of section 2.3.2 - "Testing Procedures".

The identification of the sample, moisture content measured before the test and test conditions are presented in Figure 2.6 and Table 2.5, with the results obtained 7 days after the contact was made.

From the results obtained it can be clearly recognized that there

Distance from contact (mm)	Test D 1					Test D 2	
	Penetration(mm)		I_{s1}	I_{s2}	Moisture content (%)	Penetration (mm)	I_s
	P 1	P 2					
10	3.6	3.7	77	73	29.7	3.8	69
20	3.6	3.8	77	69	30.9	3.6	77
30	3.8	3.7	69	73	31.2	3.3	92
40	3.9	3.6	66	77	31.4	3.6	77
50	3.5	3.6	81	77	29.3		
60		3.7		73	30.2	3.8	69
70	3.7	3.6	73	77	30.3		
80	3.6	3.7	77	73	30.1	3.6	69
Average			74	74	30.4		76
Max difference from average			9	7			21
Range			66/81	69/77	29.3/ 31.4		69/ 92

Table 2.2 - Natural variation of moisture content, cone penetration and "Index strength" (I_s) in undisturbed London Clay, test D 1 and D 2.

Distance from contact (mm)	Penetration (mm)	"Index strength" I_s	Moisture content (%)
10	4.1	59	31.1
20	3.8	69	30.2
30	3.6	77	30.1
40	3.9	66	30.7
50	3.7	73	30.4
Average	3.8	68	30.5
Maximum difference from average (%)		16	
Range	3.6/4.1	59/77	30.1/31.1

Table 2.3 - Natural variation of moisture content, cone penetration and "Index strength" (I_s) in undisturbed London Clay, test E.

Distance from contact (mm)	Penetration (mm)	"Index strength" I_s	Moisture content (%)
5	4.5	49	28.8
10	4.8	43	29.8
20	4.8	43	28.9
30	4.7	45	28.6
40	5.1	38	27.6
50	4.9	42	28.2
Average	4.8	43	28.6
Maximum difference from average (%)		14	
Range	4.5/5.1	38/49	27.6/29.8

Table 2.4 - Natural variation of moisture content, cone penetration and "Index strength" (I_s) in undisturbed London Clay, test G.

was a change in moisture content and in "Index strength". The ratio of minimum "Index strength" to maximum was found to be 0.6 and 0.7 for the different testing positions. The increase in moisture measured was of the order of 1.5%. The reduction in the "Index strength" was observed up to 6 cm away from the contact between the clay and fresh mortar.

TEST B -

The material used in test A was compacted and the same overall conditions applied to the test specimen (Compaction used: BS 1377: 1975, Test 13).

Figure 2.7 presents the results and the testing procedures. The analysis of the results is made in Table 2.6, with the identification of the characteristics of the testing program.

The analysis of the results shows that the increase in moisture is identified up to 3 cm from the contact, and 1.5% was the variation. The shape of the "Index strength" curve is similar to the one obtained in sample A and even the two levels (maximum and minimum) are comparable. The observed ratio between minimum and maximum I_s was 0.74. The reduced values of I_s could be found closer than 3 cm.

Comparing the affected area it can be concluded that the compacted sample was less susceptible to the change. A direct comparison cannot be made because in the compacted sample there is always the possibility of pre-existence of systematic variations of strength due to the layers used in the compactions.

TEST C -

A sample of London Clay (stiff brown fissured silty clay), sampled at 6.00 m depth with a U 100 sampler was tested under the conditions shown in Table 2.7. The results obtained and test conditions are shown in Table 2.7 and Figure 2.8. The reduction in the "Index strength" was found to affect up to 4 cm from the contact, the ratio $I_s min/I_s max$ being equal to 0.74. Table 2.8 summarizes the results on the tested samples (A, B and C).

From the results obtained it can be concluded that the cone penetration test is able to measure changes in strength over a small area in one specimen.

The purpose of the testing programme was to study the possible effects of the contact between fresh concrete and clay. The results showed that there is a clear change in strength and an increase in moisture content.

The length of the region affected by the reduction in the measured "Index strength" varied from 3 to 7 cm. The observed ratios between minimum and maximum "Index strength" were in the range 0.6 to 0.74.

The sand and cement mortar used in the tests is not a good simulation of the usual material employed in real piles. In order to get a fluid mix it was necessary to add water up to a water-cement ratio of 0.8, in a paste of 1 of cement and 3 of sand (resulting in a mix where the water represented 16.7% by weight of the total material).

A variation between 1.5 and 2.4% was found when comparing the moisture content existing prior to contact and after 7 days of testing, the trend being clearly recognizeable.

2.4.3. - EFFECT OF TIME ON TEST RESULTS

In order to study the variation of "Index strength" with time series E and F were performed.

Distance from contact (mm)	"Index strength"		Moisture content (%)	
	I_{s1}	I_{s2}	before test	7 days
0			30.3	
10	39	36		31.6
15		39		
20	39	36		31.8
30	44	39		30.8
40	51	39		30.9
45		44		
50	56	48		31.1
60	51	56		31.2
70	51	44		31.1
80	56	60		29.6
90		60		30.2
100	60			30.6
110				
120				
130				29.4
Base			29.7	
$I_s$max*	56	60		
$I_s$min*	39	36		
Ratio min/max	0.7	0.6		

Table 2.5 - Variation of moisture content, "Index strength" and ratios of $I_s$min/$I_s$max in undisturbed Boulder Clay (W/C = 0.8, 7 days, confining pressure of 100 kN/m^2), test A.

Distance from contact (mm)	"Index strength" I_s	Moisture content (%)
5	42	
10	40	34.1
15	40	
20	42	33.3
30	52	32.4
40	56	32.5
50	56	32.2
60	52	32.6
70	54	32.5
80	49	32.4
I_s max*	54	
I_s min*	40	
Ratio min/max	0.74	
*From Figure 2.6		

Table 2.6 - Variation of moisture content, "Index strength" (I_s) and ratio of $I_s$min/$I_s$max in compacted Boulder Clay (W/C = 0.8, 7 days, confining pressure of 100 kN/m^2), test B.

Distance from contact (mm)	"Index strength" I_s	Moisture content (%)
10	57	32.3
20	57	32.4
30	62	31.7
40	66	30.4
50	77	29.5
60	77	30.0
70		30.0
I_s max	77	
I_s min	57	
Ratio min/max	0.74	

Table 2.7 - Variation of moisture content, "Index strength" (I_s) and ratio of I_s min/I_s max in undisturbed London Clay (W/C=0.8, 7 days, confining pressure of 100 kN/m^2), test C.

	Test A		B	C
Initial moisture content (%)	1	2		
	30.2	30.0	32.5	30.00
Max. increase (%)	1.5	1.5	1.5	2.40
Affected length (mm)	50 to 70	50 to 70	20 to 30	30 to 40
I_s max	56	60	54	77
I_s min	39	36	40	57
Ratio $\frac{I_s \text{ min}}{I_s \text{ max}}$	0.7	0.6	0.74	0.74
Affected length (mm)	60 to 70	60 to 70	20 to 30	40

Table 2.8 - Summary of the results of tests A, B and C.

Specimen of 100 mm diameter

A) Sand and cement mortar (1:3 in weight)

Water/cement ratio : 0.6

each 10 mm of mortar :

Volume = 78.54 cm^3

Weight = 180.00 gramme

Free water*= 10 gramme

B) Soil

Volume of soil per 10 mm = 78.54 cm^3

Weight of dry soil (for a density of 1.45Mg/m^3) = 114 gramme

C) Water necessary for moisture content increase (1 %) per 10 mm of soil = 1.14 gramme

* Free water : considering a water/cement ratio of 0.38 the amount necessary to hydrate the cement, then: 0.60–0.38=0.22 or 6.11 % of free water (in weight of the mortar)

TABLE 2.8a - Free water available in the sand and cement mortar (1:3) and possible change in moisture content in the soil specimen

TEST E : Two samples were compacted (BS 1377, Test 13), using London Clay. The first sample was used as a reference for the moisture content and the second one was tested with the cone at different ages: the first time immediately after compaction, without contact with the mortar, and one and 6 days after contact with the mortar and under 100 kN/m^2 confining pressure.

The results obtained are presented in Table 2.9, Figures 2.9 2.9a and b and 2.10.

From Figure 2.9a it can be seen that the original values of the "Index strength" of the sample were scattered, the average in the area covered from 0 to 2 cm from the contact with the mortar being equal to 57 and from 2 to 5 cm equal to 51.

The results obtained one day after contact showed a more clear trend. A general reduction in strength can be identified, up to 4 to 5 cm from the contact. The lower values were found at 2 to 3 cm from contact.

At 6 days after the contact the values of "Index strength" were lower than the measured for the same distance at one day. The length affected was 5 cm.

A comparison of moisture content values obtained from the reference sample and the tested one is presented in Figure 2.9b, with the values obtained for the same material when it was obtained from the U 100 sampler. The maximum increase in moisture content was of the order of 3% after 6 days.

When the ratios of average "Index strength" were determined two regions were chosen : from zero to 2 cm away from the contact and from 2 to 5 cm from the contact.

Distance from contact (mm)	"Index strength" Initial	1 day	6 days	Moisture content (%) Initial*	6 days
0	57	49		29.6	33.0
10	66	43	34	28.5	32.3
20	49	32	23	29.6	32.4
30	57	32	25	30.1	32.4
40	47	40	25	30.3	31.8
50		47	36	30.3	
Average	55.2	42.2	28.6	29.73	32.38
Average up to 20 mm	57.3	41.3	28.5	29.23	32.57
Average from 20 to 50 mm	51.0	37.7	27.2	30.07	32.20
Ratio of I_s	1	0.76	0.52		
Ratio up to 20 mm	1	0.72	0.51		
Ratio from 20 to 50 mm	1	0.73	0.53		
*Control sample					

Table 2.9 - Effect of time on test results, test E, compacted London Clay, confining pressure of 100 kN/m^2, W/C = 0.8.

One day after contact the ratios of "Index strength" were 0.72 and 0.73 for the two regions. Six days measurements presented lower values, 0.51 and 0.53 respectively (Figure 2.10).

TEST F : One sample of undisturbed London Clay (stiff fissured brown silty clay) , obtained with an U 100 sampler (6 m depth) was tested with the cone at different ages. The first tests were performed immediately after extrusion of the sample, without contact with mortar. The second tests were performed 7 days after the test specimen was in contact with the cement sand mortar, under 100 kN/m2 confining pressure in the triaxial cell. The results obtained are presented in Table 2.10 and Figure 2.11.

The original "Index strength" was higher near the position of contact with the mortar than 40 to 50 mm away (52 to 45). The values obtained 7 days after contact shown a general reduction in strength, the lowest value being found at 30 to 40 mm from contact. The ratios of "Index strength" obtained when comparing the values at 7 and zero days were 0.57.

From the moisture content results obtained at 7 days it appears that the affected area extends up to 60 mm from the contact.

From the results obtained on the series E and F it can be concluded that :

- the cone penetration test provided consistent results of the changes in strength , due to the use of the same test specimen for comparison at different ages;

- the variation in strength continues after the first day, when the mortar sets ;

- in both compacted and undisturbed test specimens

Distance from contact (mm)	I_s 0 day	I_s 7 days	Moisture content (%) 7 days
0	52		30.3
10	49	30	30.2
20	49	27	30.2
30	47	23	29.9
40	45	24	30.1
50	45	32	29.3
60			28.6
Average	47.8	27.2	29.8
Average up to 20 mm	50	28.5	30.2
Average from 30 to 50 mm	45.6	26.3	29.8
Ratio of average 7/0 days		0.57	
Ratio up to 20 mm		0.57	
Ratio from 20 to 50 mm		0.58	

Table 2.10 - Effect of time on test results, test F, undisturbed London Clay, 6.00 m depth confining pressure of 100 kN/m^2, W/C = 0.8.

a major decrease in strength was observed, up to 50 mm from the contact with the mortar;

- the calculated ratios of "Index strength" for the compacted sample were respectively 0.76 and 0.52 for one and six days (when compared with the original values);

- the ratio of "Index strength" obtained at seven days/ the original value was 0.57 for the undisturbed London Clay.

2.4.4. - EFFECT OF WATER/CEMENT RATIO ON TEST RESULTS

The effects of using different water/cement (W/C) ratios were tested in series G.

Two samples were compacted (BS 1377, Test 13) using London Clay. One of the samples (G2) was exposed to a mortar with a water/cement ratio of 0.5, after being tested for "Index strength" measurement, and retested at 7 days. The other sample was used to study the effect of 0.8 water/cement ratio mortar on the same material, at 7 days.

Table 2.8 shows the values obtained for the different positions tested, the average values and the calculated ratios for different sections of the specimens. The tests were performed in accordance to Section 2.3.2 - Testing procedures.

A comparison between the "Index strength" values obtained in sample G2 and plotted in Figure 2.12 as initial value and after contact with mortar with W/C = 0.5 shows that there is apparently an improvement in the strength of the sample. A possible explanation for the results obtained is that, for the mix used, a water cement ratio of 0.5 is not enough to allow the hydration of the cement and to fill the voids in the mortar; the mortar avidity for water creates a suction and reduces the moisture content of the sample. Figure 2.13 gives an idea of the possibility of such a phenomenon, the moisture content variation on sample G2 shows a trend where the moisture content near the contact is lower than at 4 or 5 cm away. Another possible explanation for that set of results is that the mortar created is a non saturated media, with the confining pressure acting on the clay sample, the water migrates from the higher stressed area inside the clay to the permeable material. Inspection of the sample after the test showed a very porous material. The observed increases in "Index strength" were 12 and 18% respectively for the areas between zero and 2 and 2 to 5 cm away from contact.

The results obtained with sample G3 are shown in Figures 2.13 and 2.12 and present the expected trends : decrease in "Index strength" near the contact, increase in moisture content in that area, affected zone covering the first 4 cm of the sample. The ratio of "Index strength" for the first 2 cm was 0.71 and 0.85 for the area from 2 to 5 cm, when comparing with the original values.

As a conclusion of the effect of water/cement ratio, the following aspects appear significant:

- as expected, the water/cement ratio has a strong effect on the changes of both moisture content and "Index strength". As the water/cement ratio increases the amount of free water increase ;

- the results obtained with the sample tested with the mortar using a water/cement ratio of 0.5 must be regarded just as an exercise in research. The test bears no relation to real piling techniques. In real piles a rich mix is always used and porous concrete, such as the one obtained in the test, is avoided .

Distance from contact (mm)		G 2	G 2 W/C = 0.5		G 3 W/C = 0.8	
		I_s initial	I_s	moisture content (%) (7 days)	I_s	moisture content (%) (7 days)
5		49				
10		43	49	28.4	34	30.7
20		43	52	28.9	30	31.1
30		45	52	28.7	34	29.9
40		38	52	29.4	36	29.6
50		42	43	29.6	42	28.7
Average		43.3	49.6	29.00	35.2	30.00
Average up to 20 mm		45.0	50.5	28.65	32.0	30.90
Aver. from 20 to 50 mm		42.0	49.8	29.20	35.5	29.83
Ratio	I_s/I_s initial		1.15		0.81	
	up to 20 mm		1.12		0.71	
	from 20 to 50 mm		1.18		0.85	

Table 2.11 - Effect of different water cement ratios on test results, test G, compacted London Clay.

Distance from contact (mm)	Natural specimen		Compacted specimen	
	"Index strength" I_s	moisture content %	"Index strength" I_s	moisture content %
10	59	31.1	57	28.5
20	69	30.1	66	29.6
30	77	30.1	49	30.1
40	66	30.7	57	30.3
50	73	30.4	47	30.3
Average	68	30.48	55	29.76
Maximum difference from average (%)	16		20	
Range	59/77	30.1/31.1	47/66	28.5/30.3

Table 2.12 - Natural soil versus compacted specimen, test E, London Clay.

2.4.5. - NATURAL SOIL VERSUS COMPACTED SPECIMEN

In series E a comparison between the natural structure and the compacted one can be studied. The London Clay tested as a natural material (undisturbed specimen) was compacted and tested in order to study the effect of compaction and age.

Figure 2.14 shows the moisture content variation in the undisturbed and compacted specimens. Figure 2.15 presents the "Index strength" for both specimens without contact with fresh concrete.

Considering the moisture content distribution in both samples, the scatter in the undisturbed specimen (30.1 to 31.1 %) is lower than in the compacted specimen (28.5 to 30.3 %).

A comparison of the "Index strength" values shows that the average value for the undisturbed specimen is greater than for the compacted one (68 and 55 respectively). Both samples presented a wide scatter in the results (16 and 20 % as maximum differences from the average values for the undisturbed and compacted specimens). The observed ranges in "Index strength" were 59 to 77 for the undisturbed specimen and 47 to 66 for the compacted one.

Figure 2.16 shows the moisture content variation on the undisturbed and compacted specimens without contact with the fresh mortar (like Figure 2.10) and the effect of such contact in the moisture distribution at 6 days for the compacted specimen.

2.5. - CONCLUSIONS (from the experimental results)

NATURAL VARIATION IN MOISTURE CONTENT AND CONE PENETRATION

From the tests performed in specimens without contact with fresh concrete a scatter in values of "Index strength" was observed. The maximum difference from the average varies from specimen to specimen, the minimum value obtained was 7 %, the maximum was 21 %. Moisture content variation observed was randomly distributed.

EFFECT OF FRESH CONCRETE ON MOISTURE CONTENT DISTRIBUTION AND "INDEX STRENGTH"

The results obtained showed that there is a clear change in strength and increase in moisture content. The length of the region affected by the reduction in the measured "Index strength" varied from 30 to 70 mm (3 to 7 cm). The observed ratios between minimum and maximum "Index strength" were in the range 0.60 to 0.74. A variation between 1.5 and 2.5 % was found when comparing the moisture content existing prior to contact and after 7 days of testing, the trend being clearly recognizeable.

EFFECT OF TIME ON TEST RESULTS

The cone penetration test provided consistent results of the changes in strength, due to the use of the same test specimen for comparison at different ages. The variation in strength continues after the first day, when the mortar sets. In both compacted and undisturbed test specimens a major decrease in strength was observed, up to 50 mm (5 cm) from the contact with the mortar.

EFFECT OF WATER/CEMENT RATIO ON TEST RESULTS

As expected, the water/cement ratio has a strong effect on the changes of both moisture content and "Index strength". As the water/ cement ratio increases the amount of free water increase resulting in a larger change in moisture content on the adjacent soil .

NATURAL SOIL VERSUS COMPACTED SPECIMENS

The observed scatter in the moisture content distribution in the undisturbed specimen was lower than in the compacted specimen. Both samples presented a wide scatter in the results of cone penetration tests, resulting in a scatter in "Index strength" values. A comparison of "Index strength" values between both samples is difficult due to the different initial moisture content affecting the comparison.

CONE PENETRATION TEST AS A MEASUREMENT OF SOIL STRENGTH

The cone penetration test was used successfuly for the study of changes in strength. Important aspects of the test discovered during the programme were: (i) the presence of a fissure strongly affects the penetration measured, reducing the strength, (ii) results are affected by selenite crystals, common in London Clay, which increase the measured strength, (iii) precise location of the cone's point on the surface to be tested is an essential part of the test, otherwise misleading results can be obtained, and (iv) tests must be performed as quickly as possible to minimize the possibility of variation in moisture content due to evaporation.

2.6 - COMMENTS AND RECOMMENDATIONS

In this chapter the preliminary study of moisture content and undrained shear strength variations in clay samples due to the contact with fresh cement and sand mortar has been presented.

The moisture content determination was performed using the current practice in Soil Mechanics laboratory work (oven drying samples). As the variations that occur are small the accuracy obtained is not good enough. A new technique is needed to study the changes, especially because the determination of the initial moisture is always a problem.

The use of the cone penetration and the "Index strength" for the study of changes in strength proved to be a successful approach. The possibility of testing the same specimen at different positions and retesting at different ages is a major advantage of the procedure. The cone penetration test is a quick, simple and reliable way of measuring changes in strength or comparing strengths in different positions inside the same specimen or in different specimens. Its use should be adopted as a strength test. With experience, the comparison of results of such tests can provide values for the constants for each material and the test can be used as a very simple index test.

In situations when a completely new material will be found surrounding bored piles, the technique proposed in this chapter can be used as an indication of the sensitivity of the soil to the fresh concrete to be used (prior to the necessary pile load test programme).

Any future laboratory research concerning the effects of fresh concrete on soil properties and conditions should use natural soils (undisturbed specimens) and full concrete mixes. Apart from the fact that different mixes can be studied, the numerical values obtained with a more realistic model can be used as a basis for a better understanding of the various factors affecting pile behaviour. Ideally the laboratory research should be coupled with field work. Samples taken from the vicinity of real piles should be tested and the results compared, and furthermore, pile load test results could be related to the results of these simple experiments, in order to compare real behaviour with the results obtained using this technique.

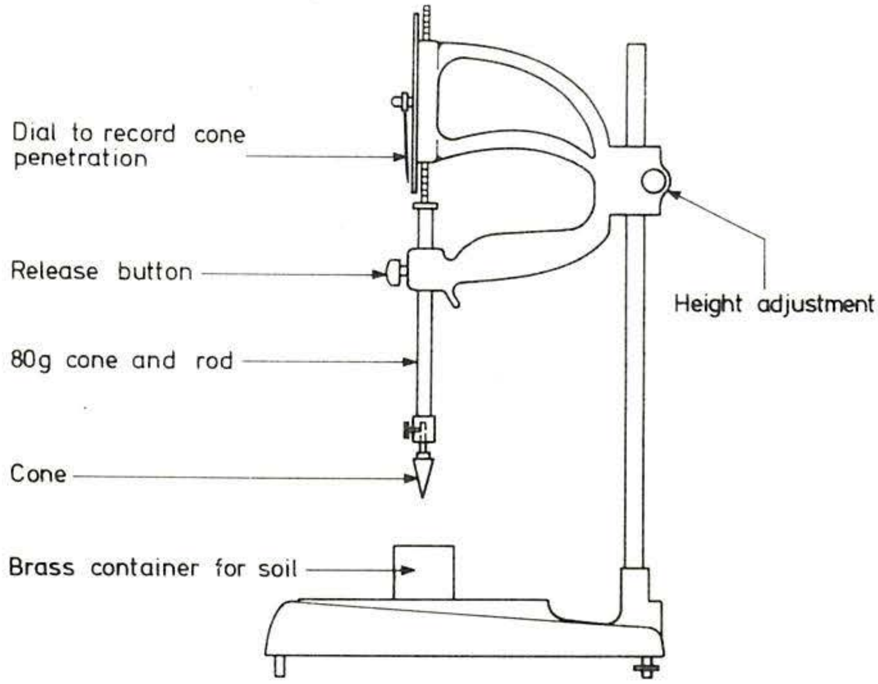


FIGURE 2.1a - 80 g FALL CONE APPARATUS

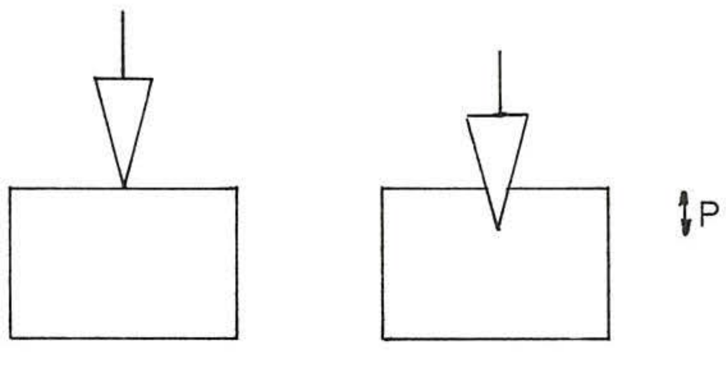


FIGURE 2.1b - CONE PENETRATION TEST

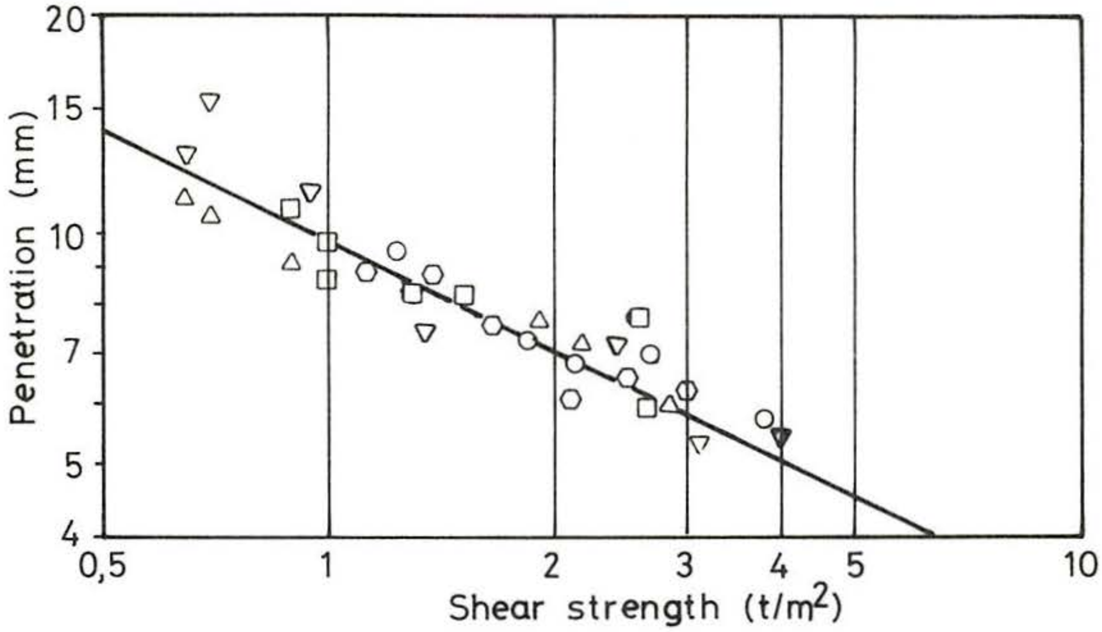


FIGURE 2.1c - RELATIONSHIP BETWEEN CONE PENETRATION (100 gramme, 30^{o}) AND UNDRAINED SHEAR STRENGTH OBTAINED BY THE FIELD VANE TEST OF VARIOUS CLAYS (HANSBO, 1957)

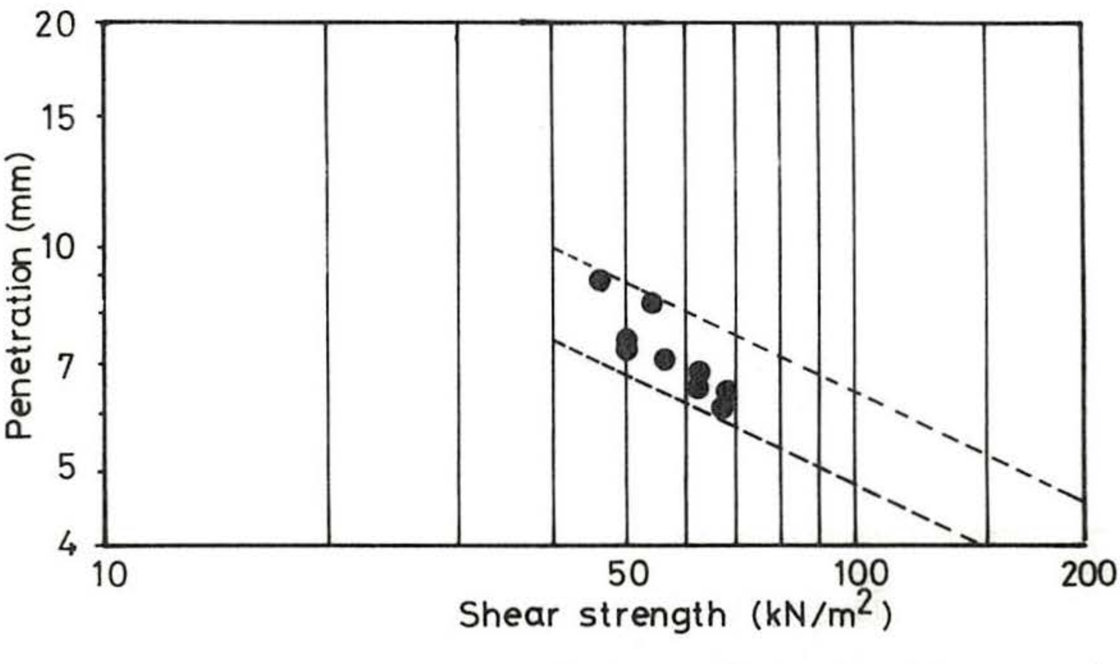


FIGURE 2.1d - RELATIONSHIP BETWEEN CONE PENETRATION (650 gramme, 30^{o}) AND UNDRAINED SHEAR STRENGTH OBTAINED BY THE TRIAXIAL TEST OF SPECIMENS OF COMPACTED LONDON CLAY

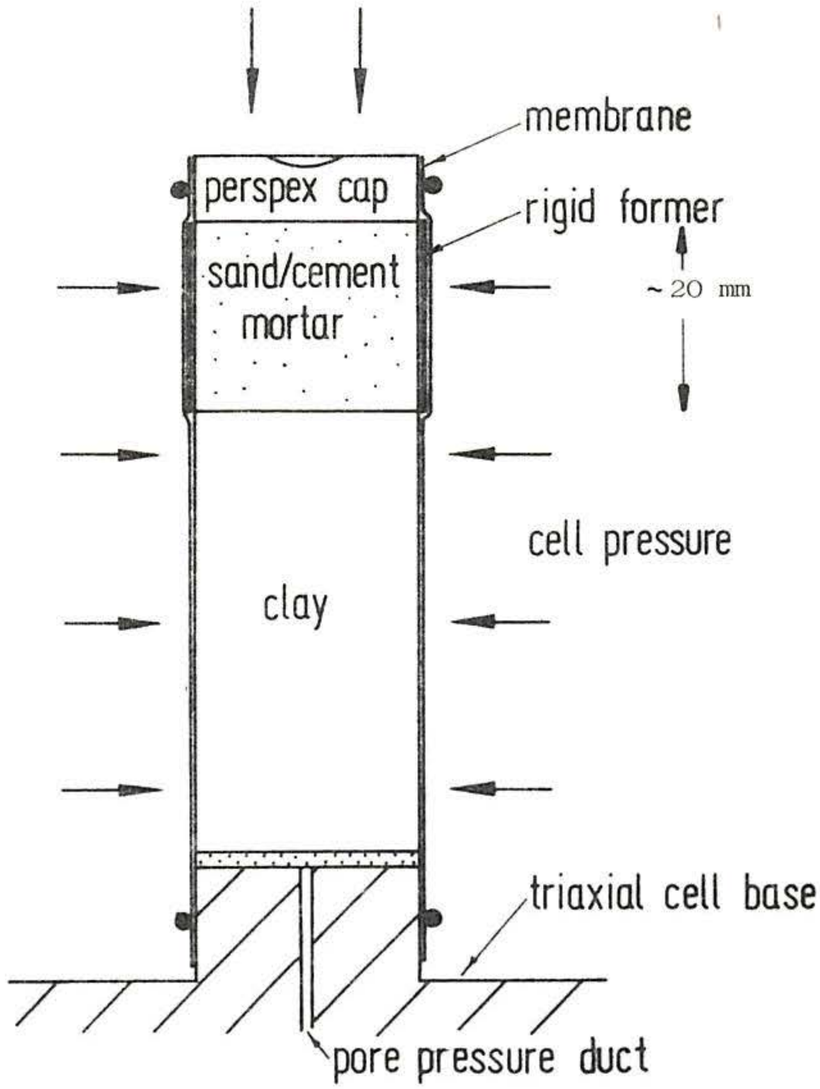


FIGURE 2.2 - TEST ARRANGEMENT USED FOR OBTAINING THE CLAY AND CONCRETE INTERFACE EFFECT

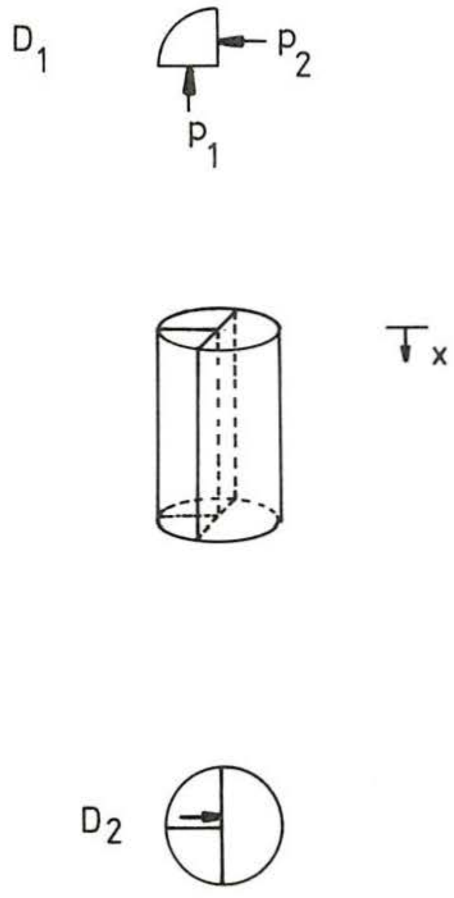


FIGURE 2.3a - TESTING PROCEDURE USED IN SPECIMENS D_1 AND D_2

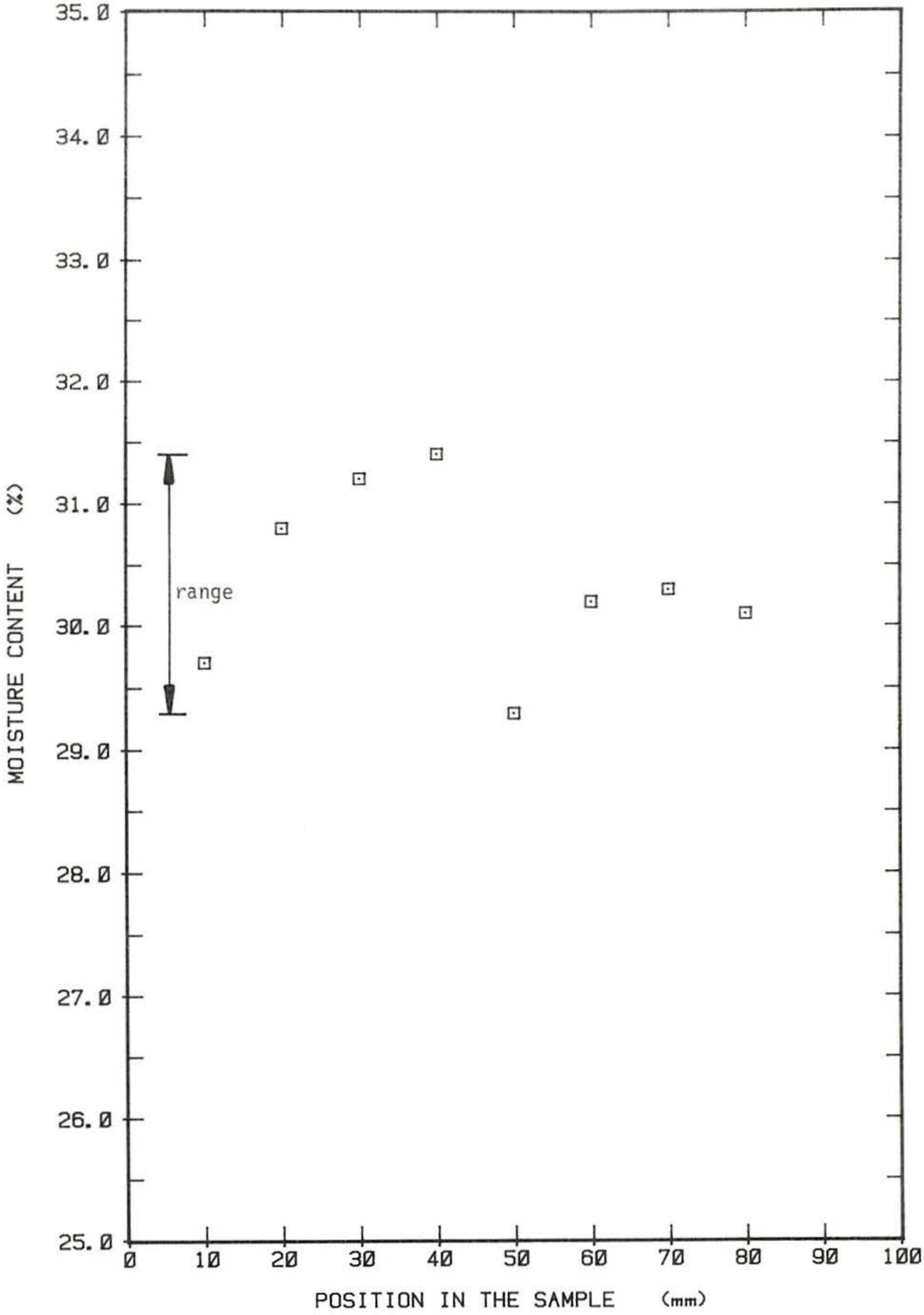


FIGURE 2.3b - THE NATURAL VARIABILITY OF MOISTURE CONTENT IN A SPECIMEN OF UNDISTURBED LONDON CLAY, TEST D_1

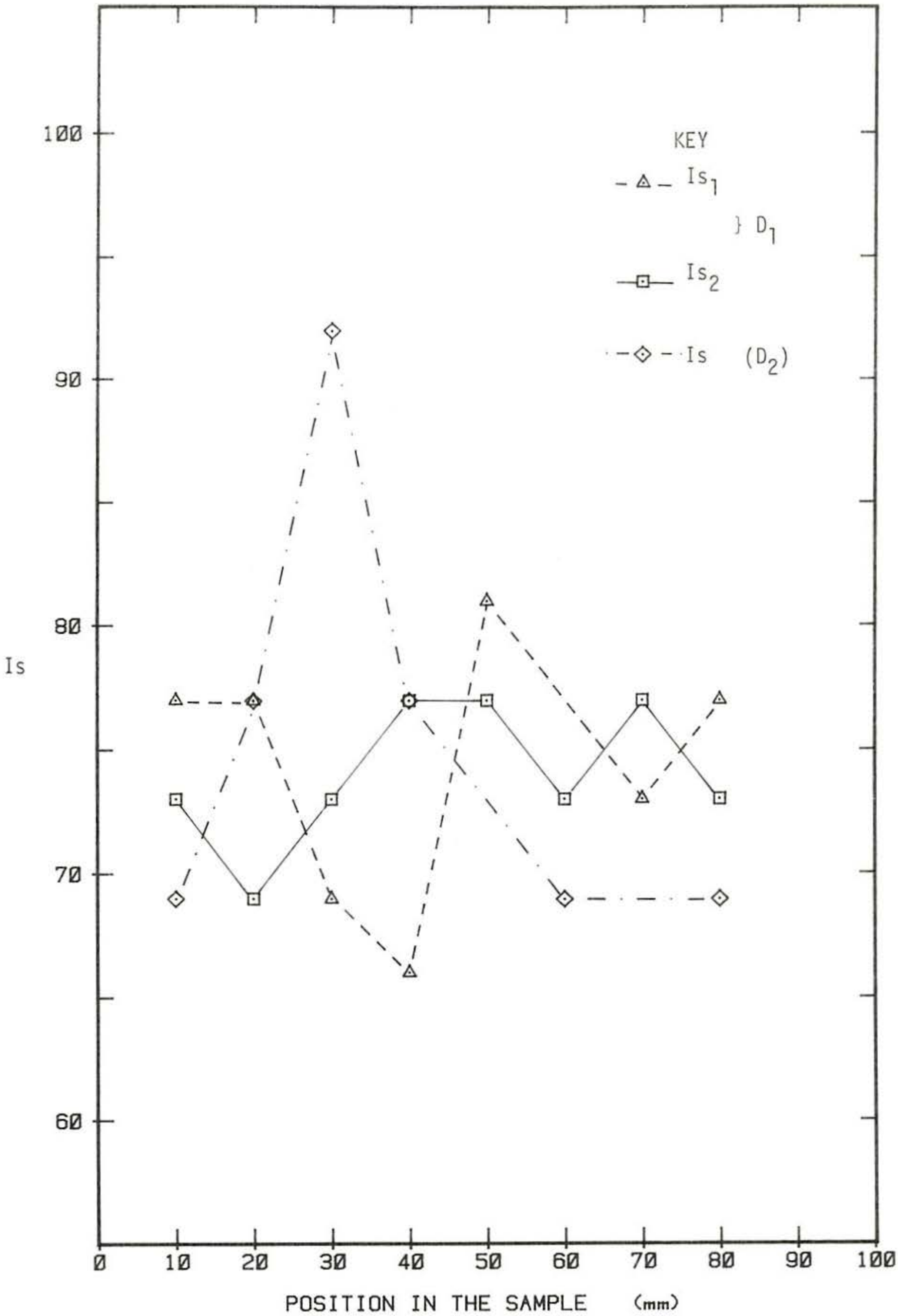


FIGURE 2.3c - THE NATURAL VARIABILITY OF "INDEX STRENGTH" IN SPECIMEN OF UNDISTURBED LONDON CLAY, TESTS D_1 AND D_2

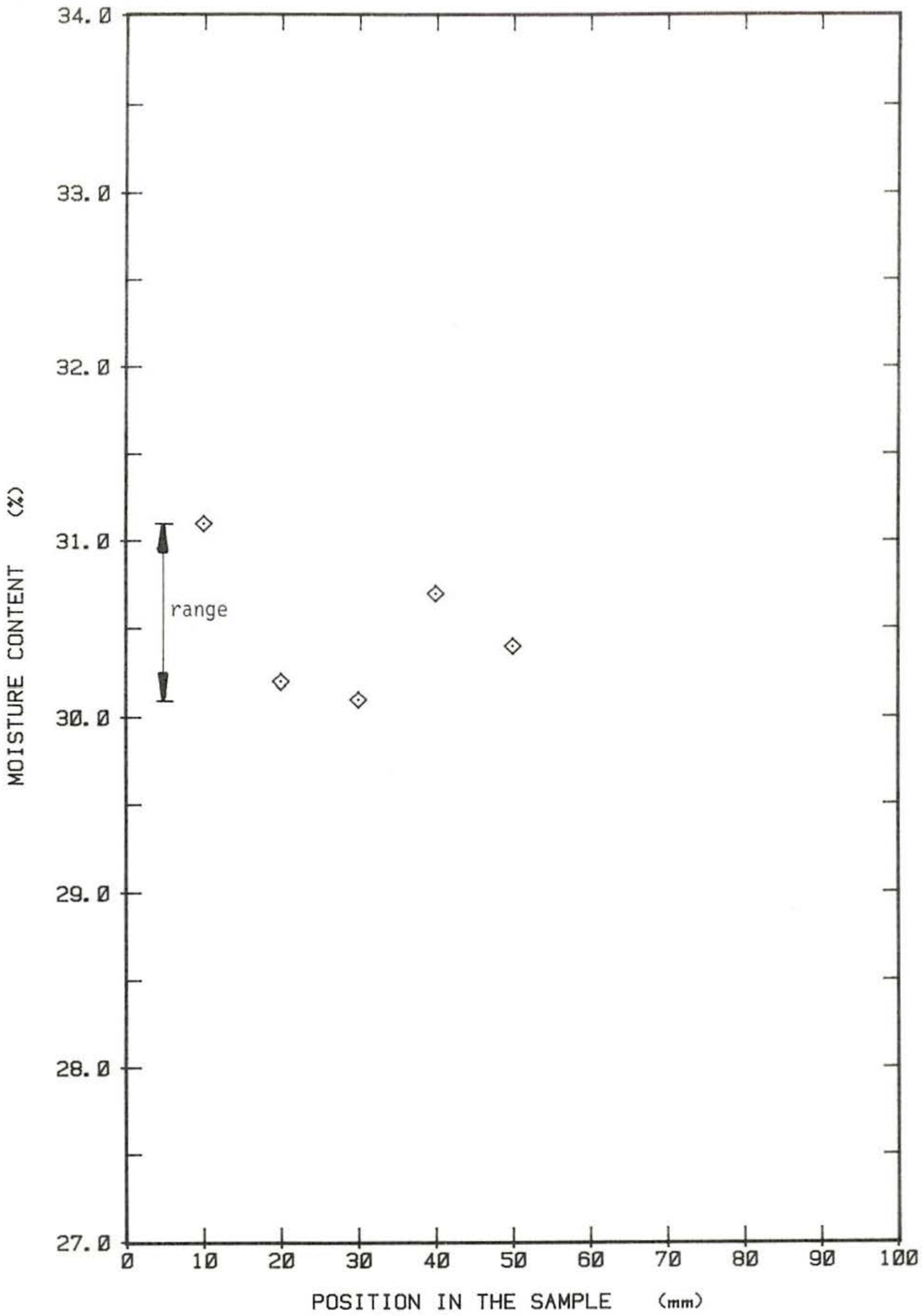


FIGURE 2.4a - THE NATURAL VARIABILITY OF MOISTURE CONTENT IN A SPECIMEN OF UNDISTURBED LONDON CLAY , TEST E

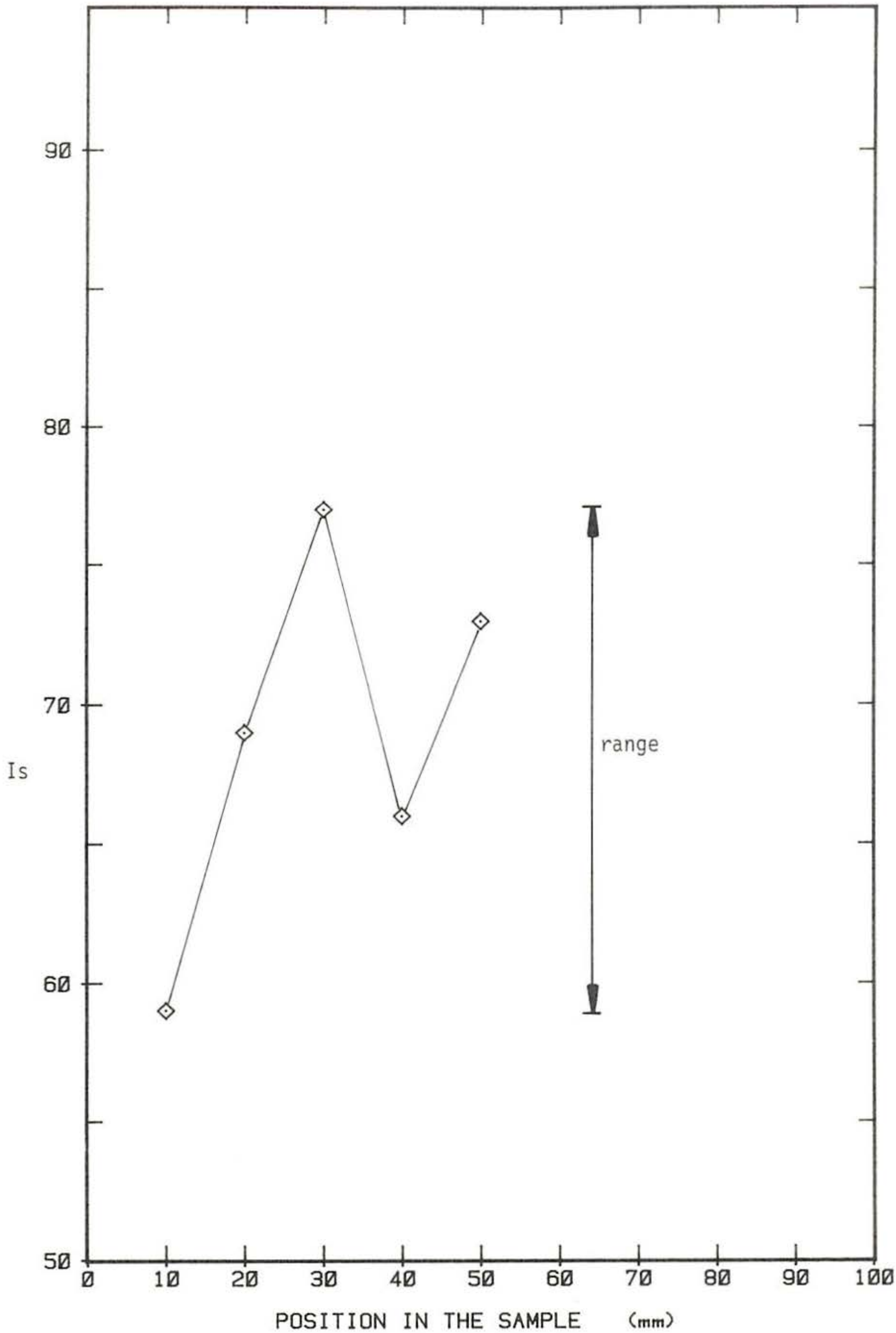


FIGURE 2.4b - THE NATURAL VARIABILITY OF "INDEX STRENGTH" IN A SPECIMEN OF UNDISTURBED LONDON CLAY, TEST E

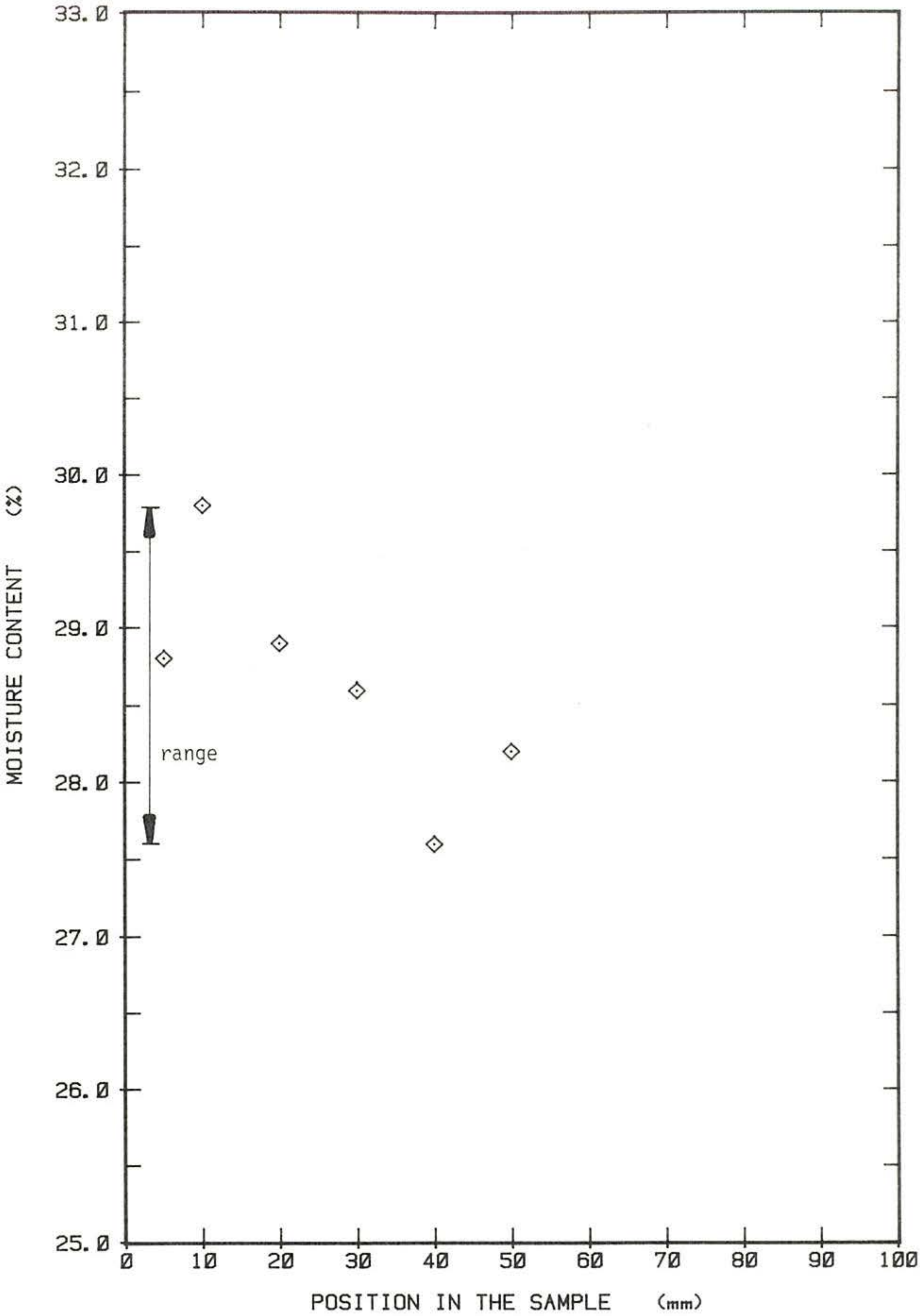


FIGURE 2.5a - THE NATURAL VARIABILITY OF MOISTURE CONTENT IN A SPECIMEN OF UNDISTURBED LONDON CLAY, TEST G

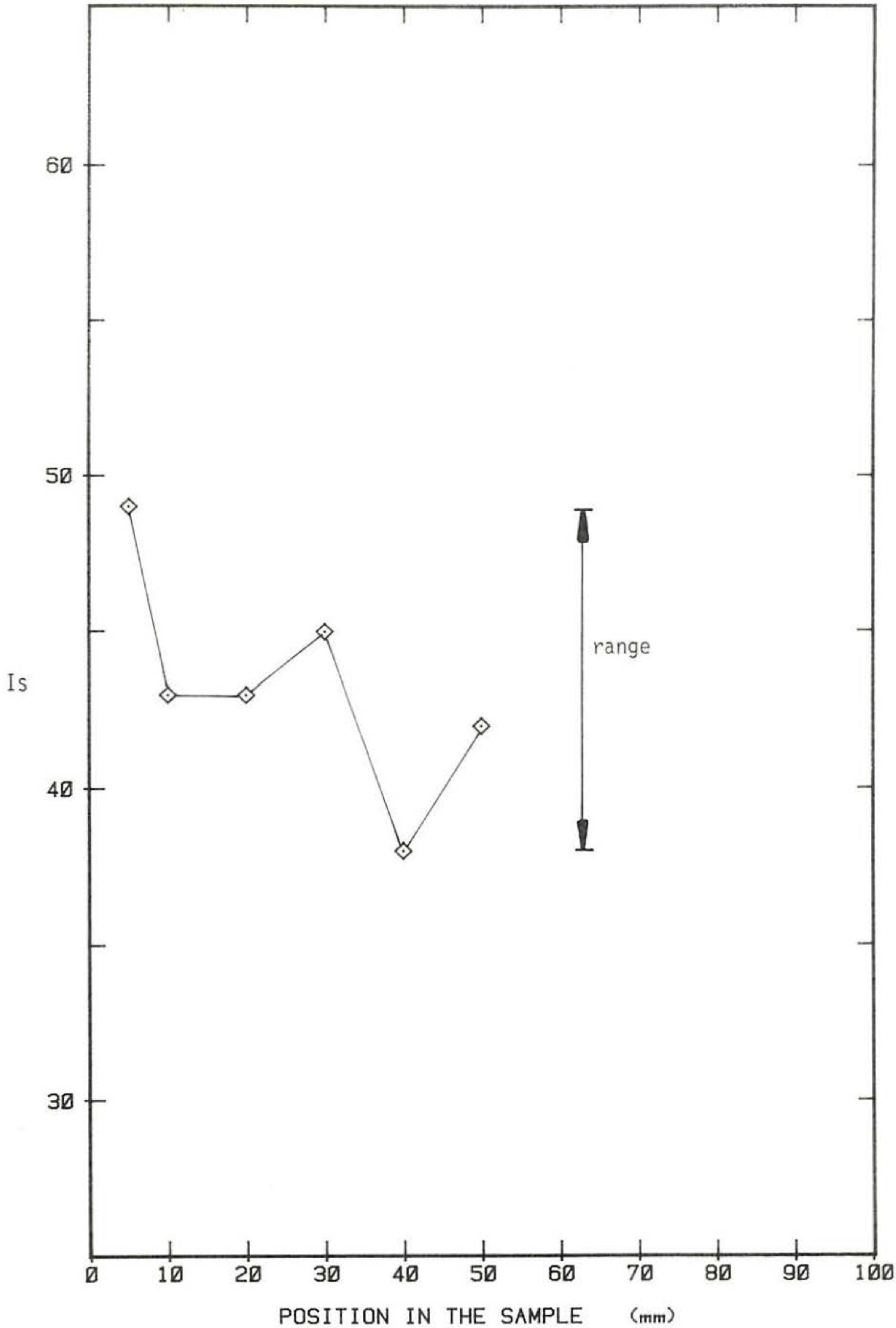


FIGURE 2.5b - THE NATURAL VARIABILITY OF "INDEX STRENGTH" IN A SPECIMEN OF UNDISTURBED LONDON CLAY, TEST G

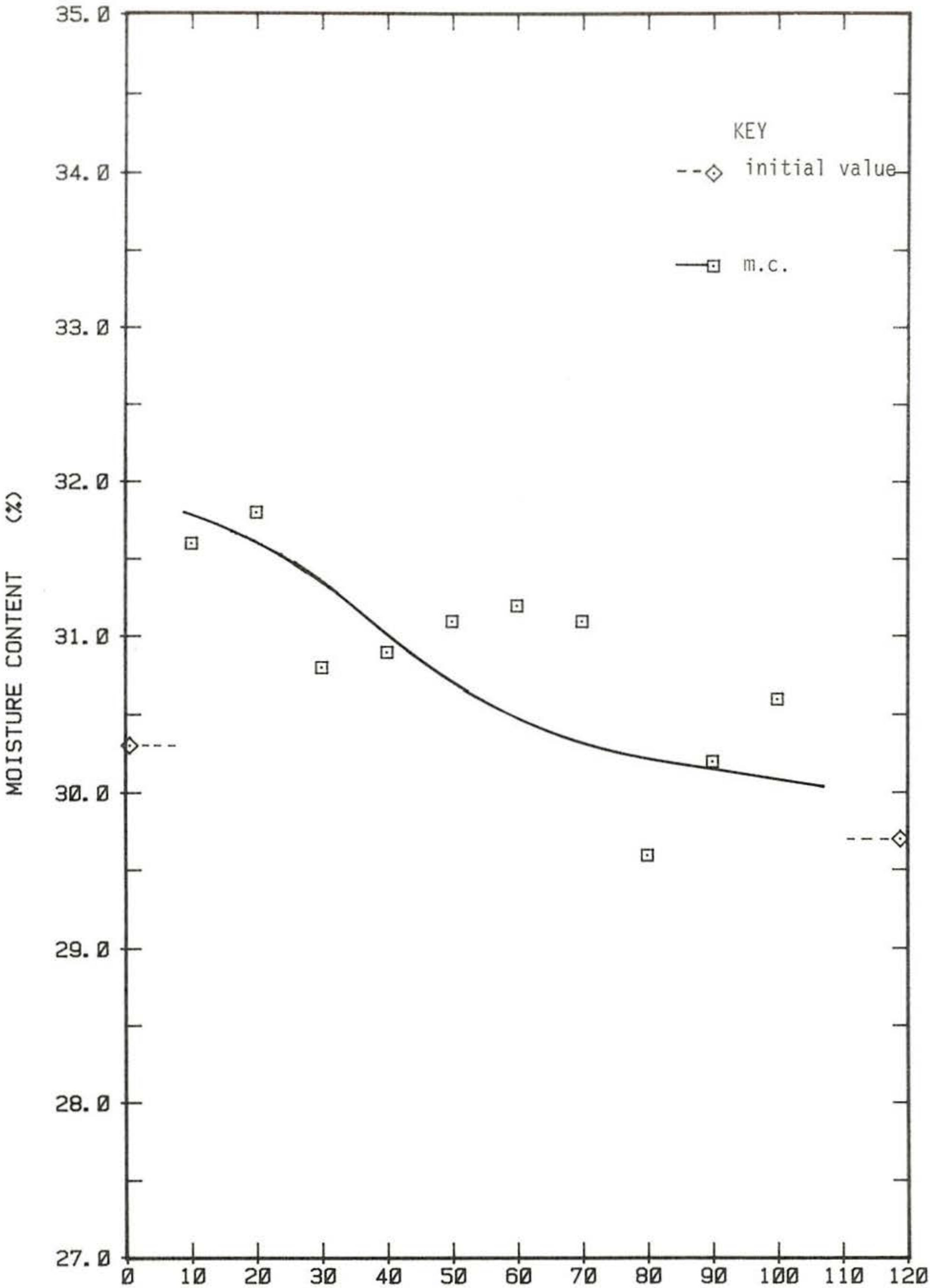


FIGURE 2.6a - EFFECT OF FRESH CONCRETE ON MOISTURE CONTENT DISTRIBUTION IN A SPECIMEN OF UNDISTURBED BOULDER CLAY, TEST A

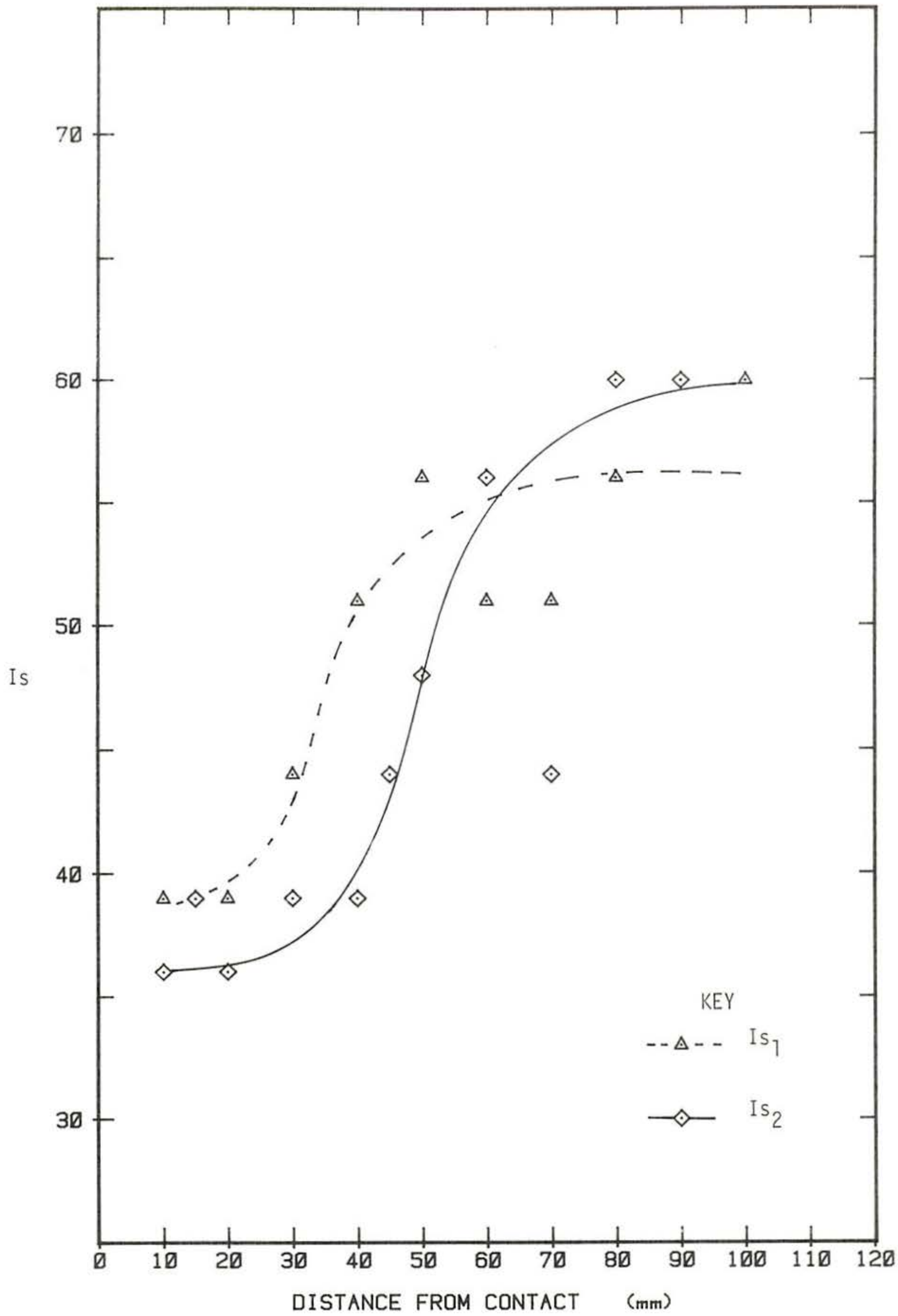


FIGURE 2.6b - EFFECT OF FRESH CONCRETE ON "INDEX STRENGTH" DISTRIBUTION IN A SPECIMEN OF UNDISTURBED BOULDER CLAY, TEST A

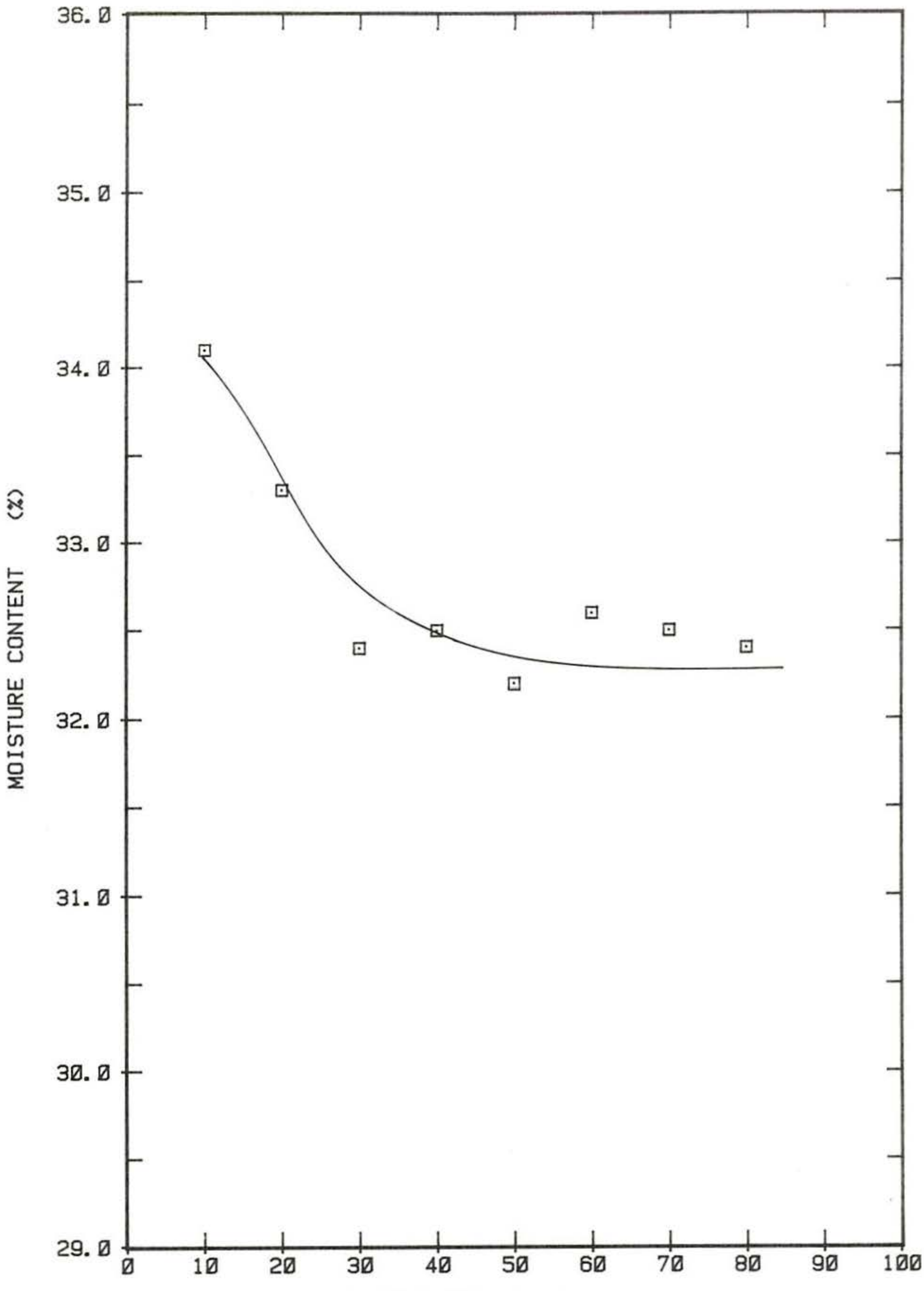


FIGURE 2.7a - EFFECT OF FRESH CONCRETE ON MOISTURE CONTENT DISTRIBUTION IN A SAMPLE OF COMPACTED BOULDER CLAY, TEST B

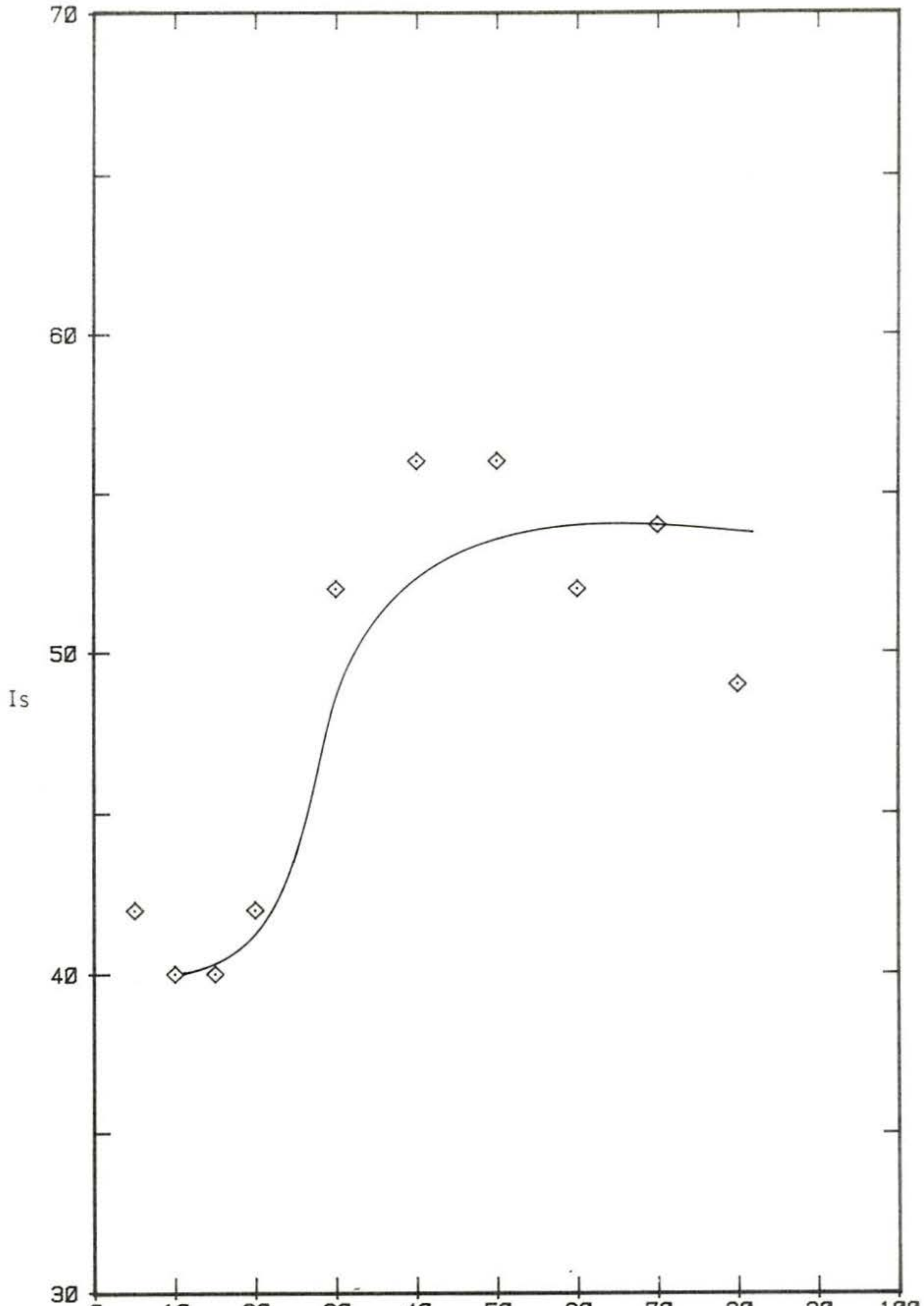


FIGURE 2.7b - EFFECT OF FRESH CONCRETE ON "INDEX STRENGTH" DISTRIBUTION IN A SAMPLE OF COMPACTED BOULDER CLAY, TEST B

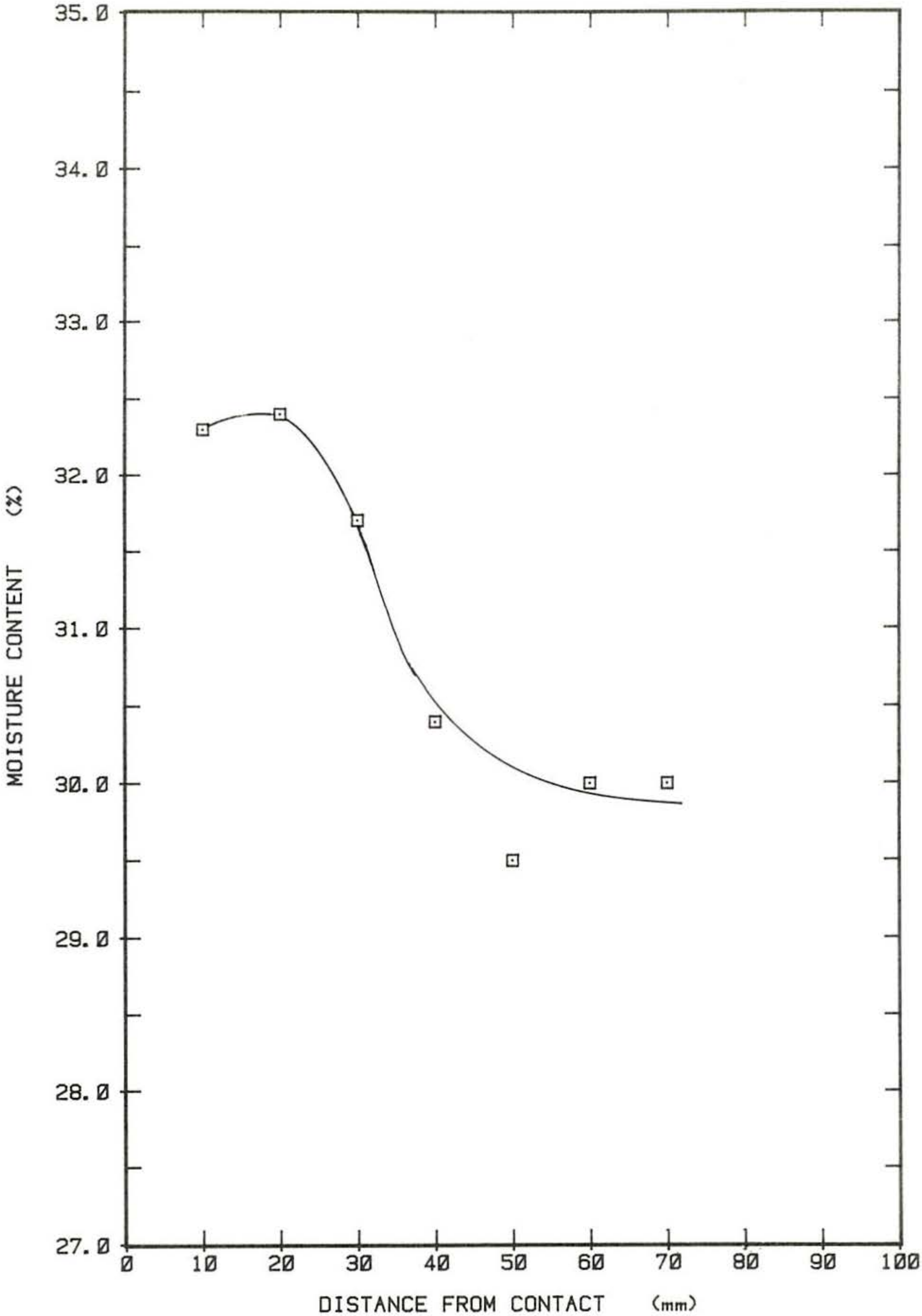


FIGURE 2.8a - EFFECT OF FRESH CONCRETE ON MOISTURE CONTENT DISTRIBUTION IN A SPECIMEN OF UNDISTURBED LONDON CLAY, TEST C

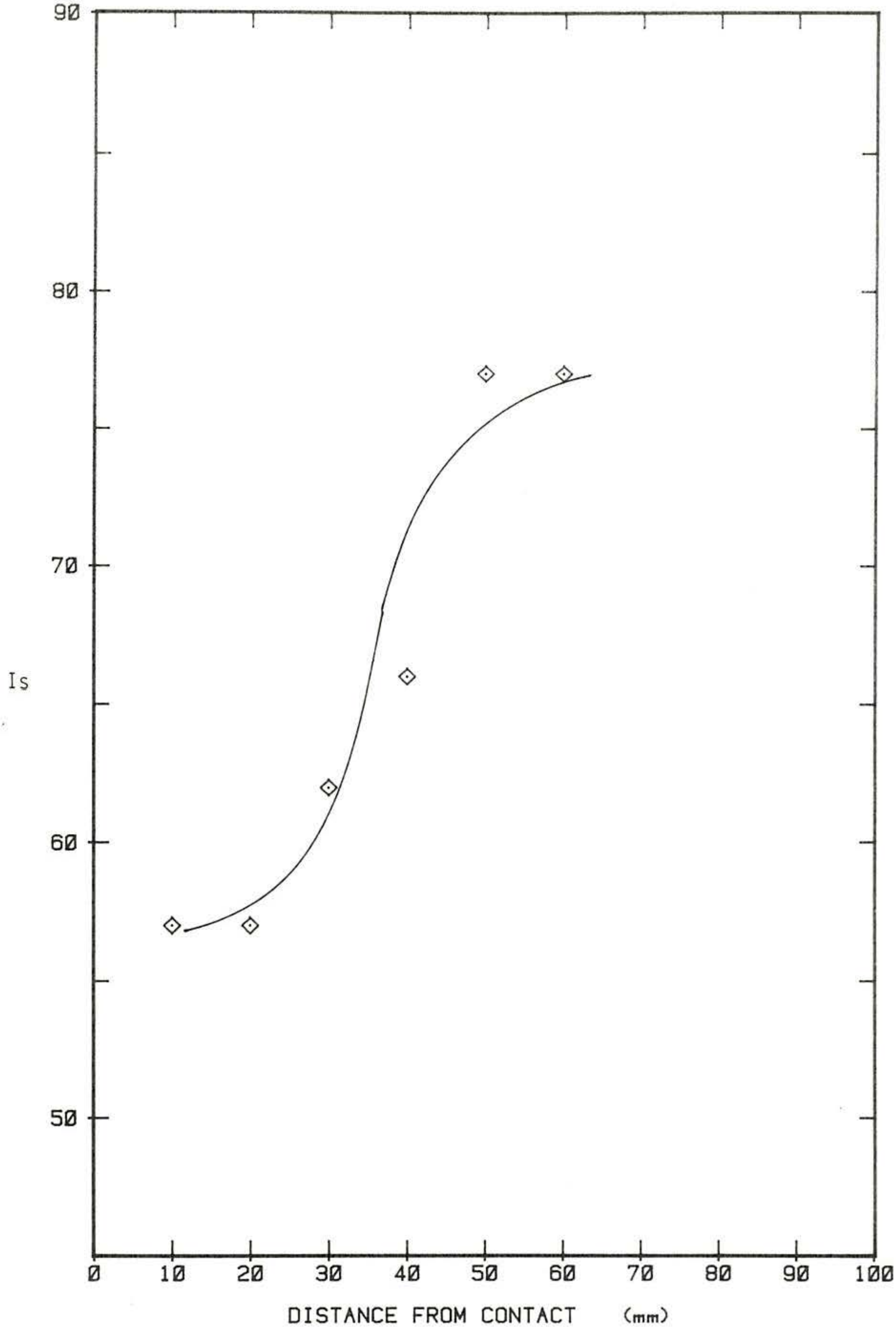


FIGURE 2.8b - EFFECT OF FRESH CONCRETE ON "INDEX STRENGTH" DISTRIBUTION IN A SPECIMEN OF UNDISTURBED LONDON CLAY, TEST C

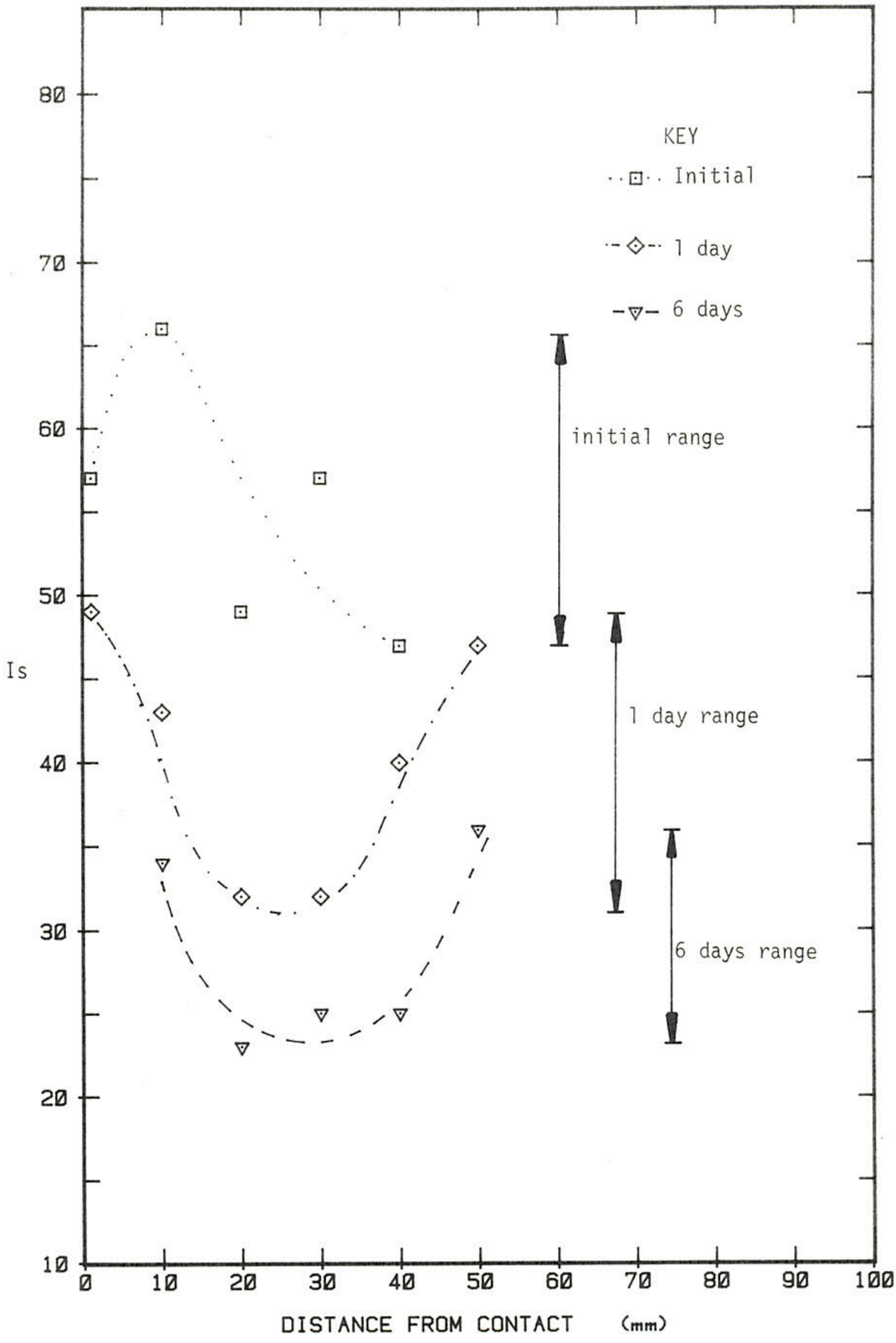


FIGURE 2.9a - EFFECT OF TIME ON TEST RESULTS, COMPACTED LONDON CLAY

SERIES E - VARIATION ON "INDEX STRENGTH"

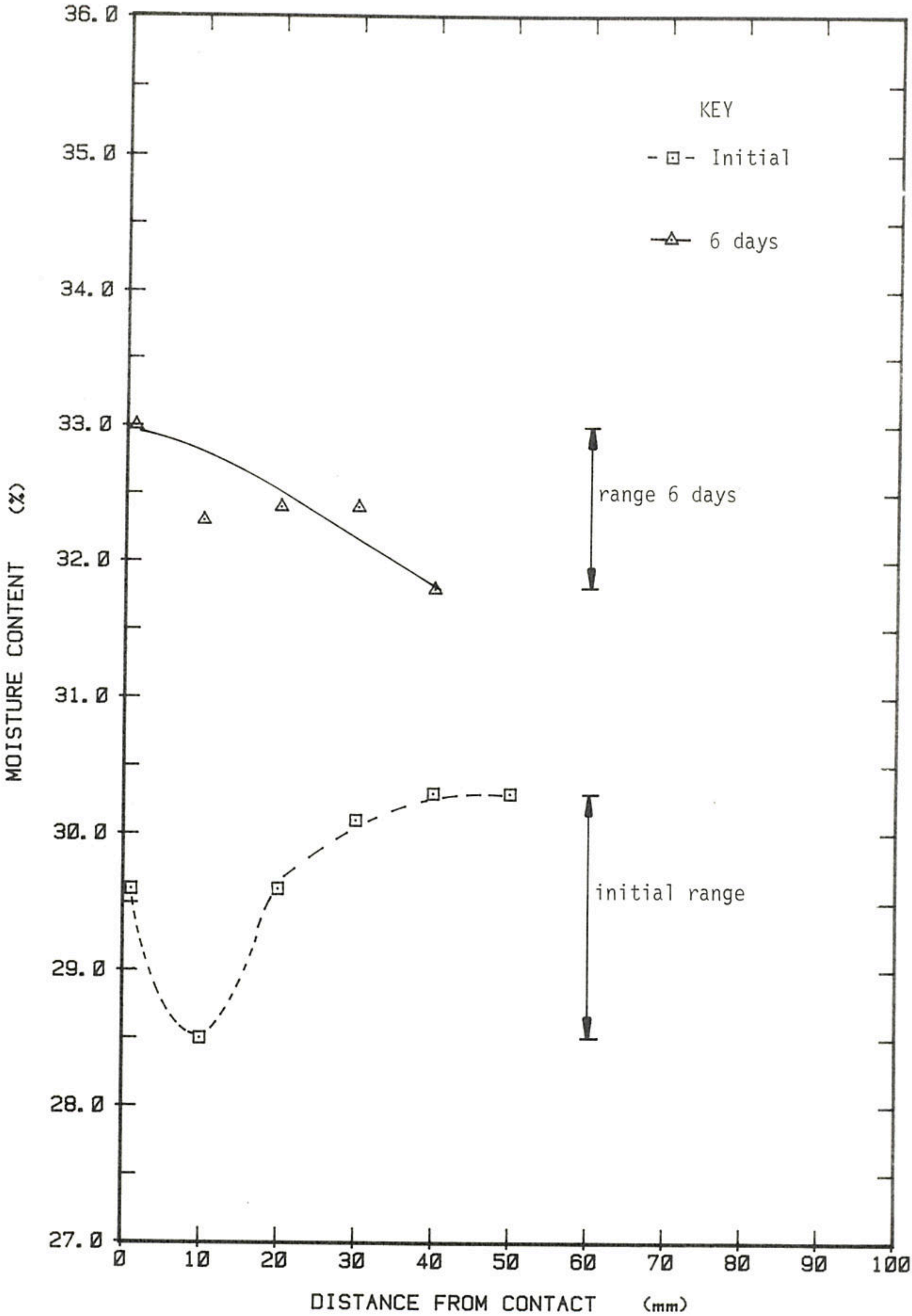


FIGURE 2.9b - EFFECT OF TIME ON TEST RESULTS, COMPACTED LONDON CLAY

SERIES E - VARIATION OF MOISTURE CONTENT

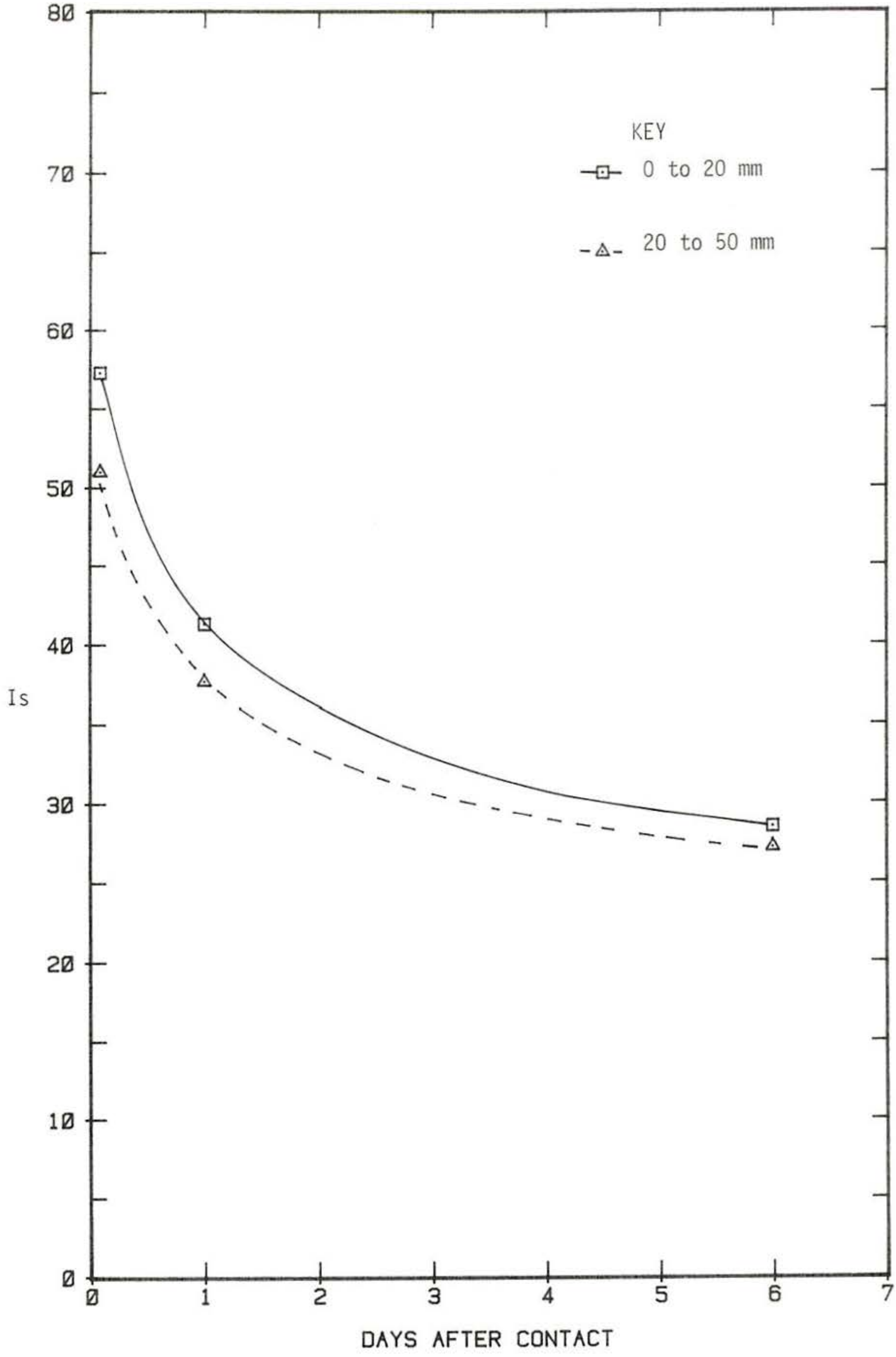


FIGURE 2.10a - VARIATION OF "INDEX STRENGTH" WITH TIME, SERIES E

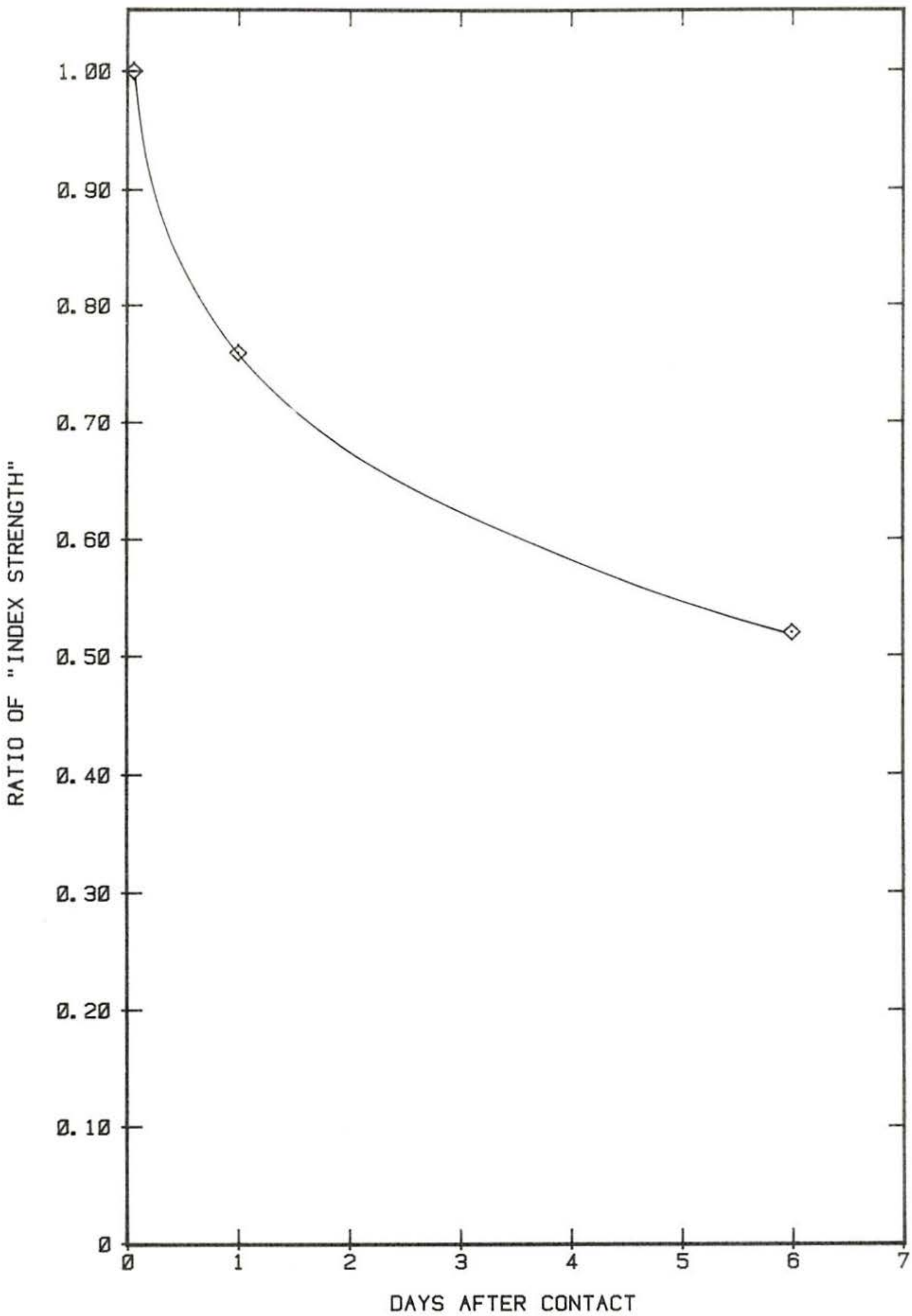


FIGURE 2.10b - RATIOS OF "INDEX STRENGTH" FOR SERIES E , AVERAGE FROM 0 TO 50 mm

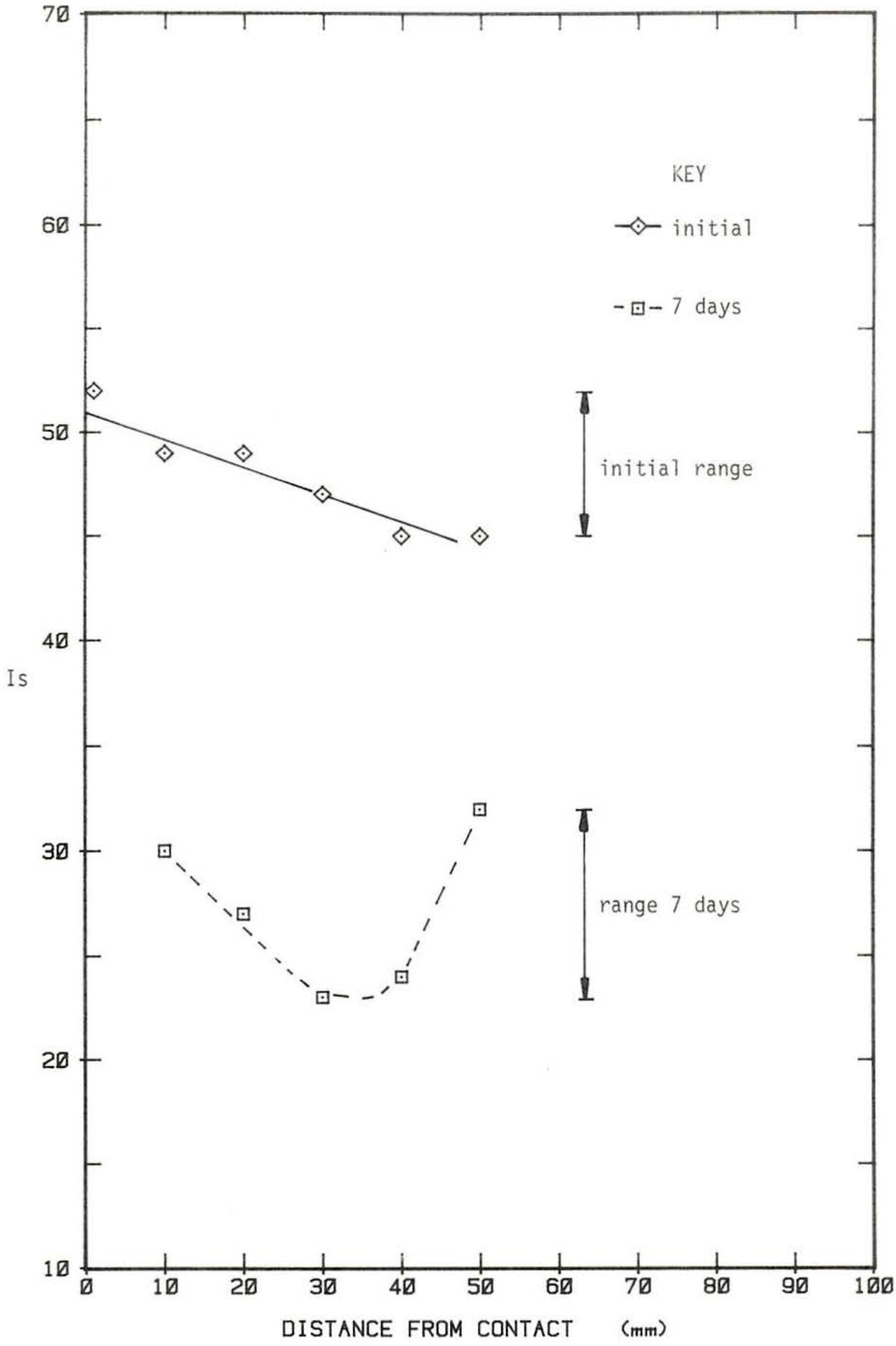


FIGURE 2.11a - EFFECT OF TIME ON TEST RESULTS, UNDISTURBED LONDON CLAY

SERIES F - VARIATION OF "INDEX STRENGTH"

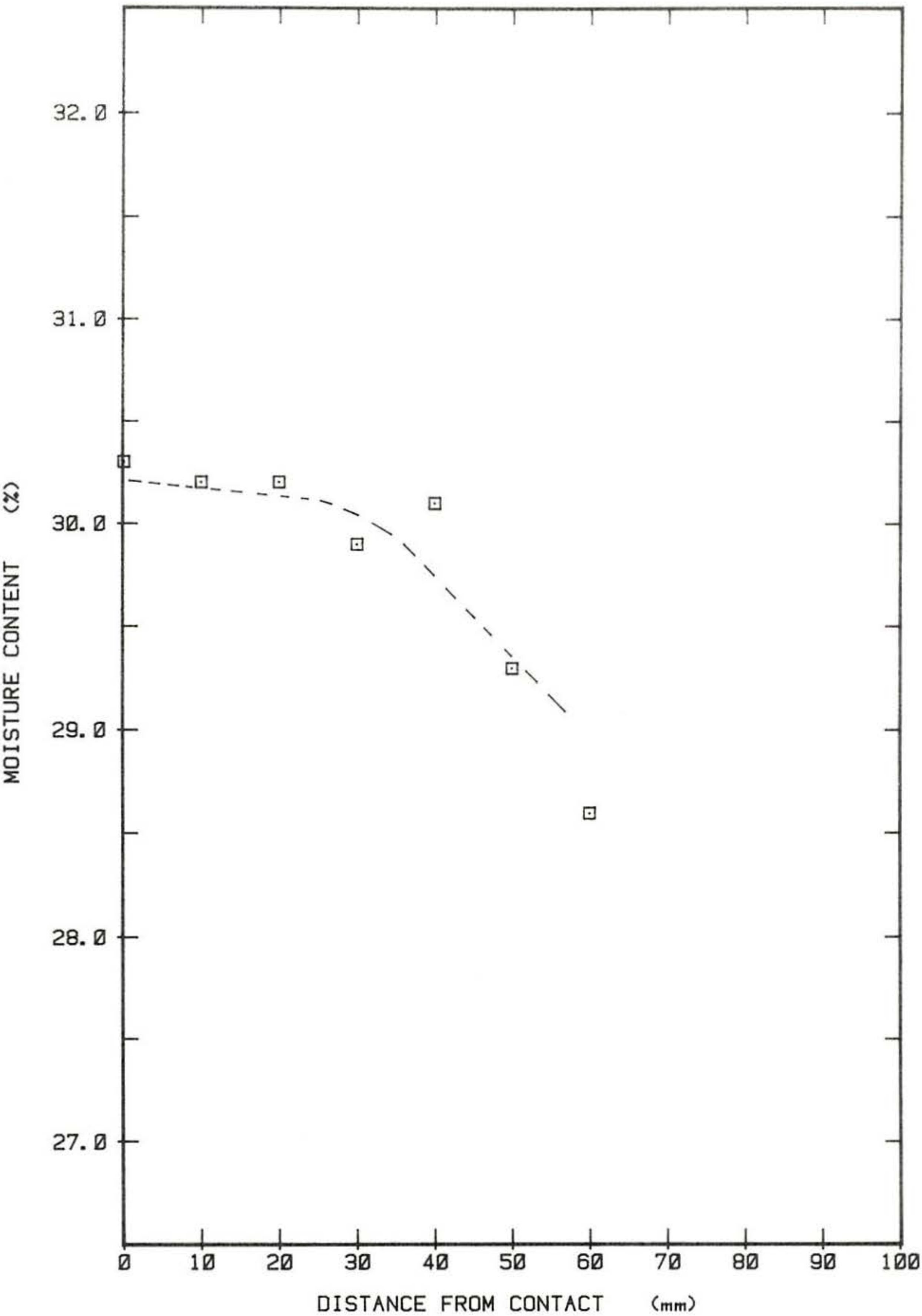


FIGURE 2.11b - MOISTURE CONTENT DISTRIBUTION AT 7 DAYS, SERIES F

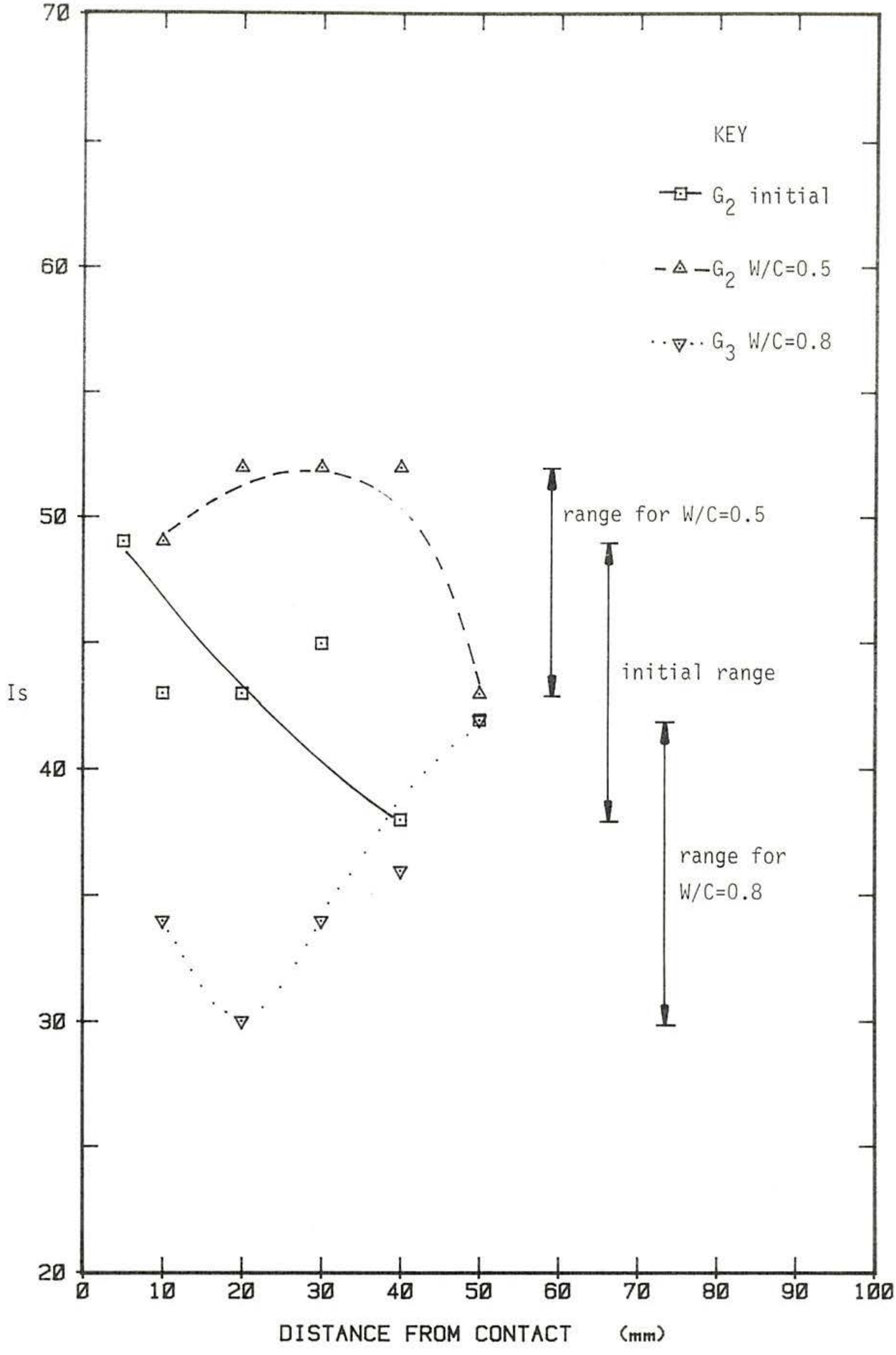


FIGURE 2.12 - EFFECT OF WATER CEMENT RATIO ON "INDEX STRENGTH", COMPACTED LONDON CLAY, SERIES G

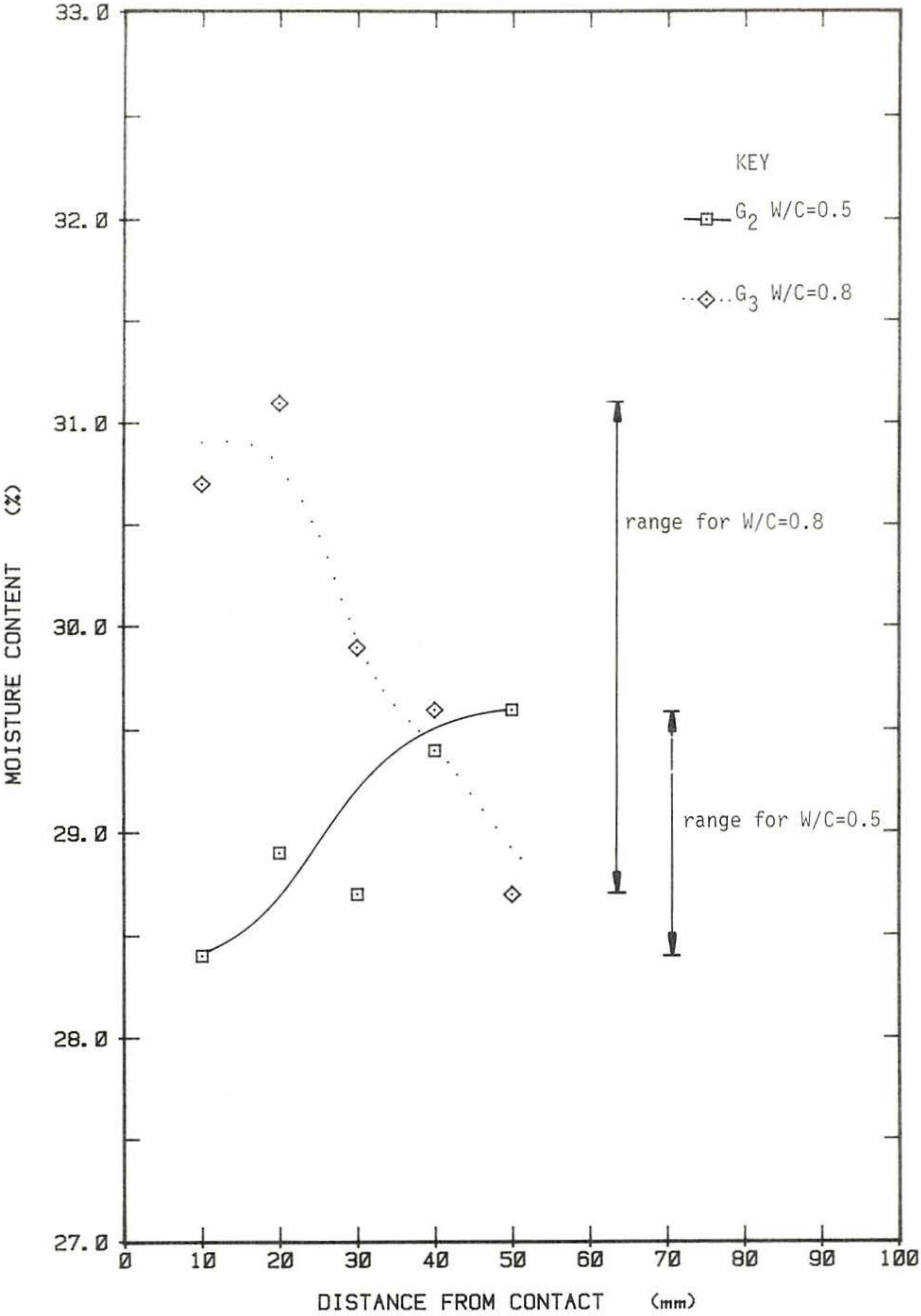


FIGURE 2.13 - EFFECT OF WATER CEMENT RATIO ON MOISTURE CONTENT, COMPACTED LONDON CLAY, SERIES G

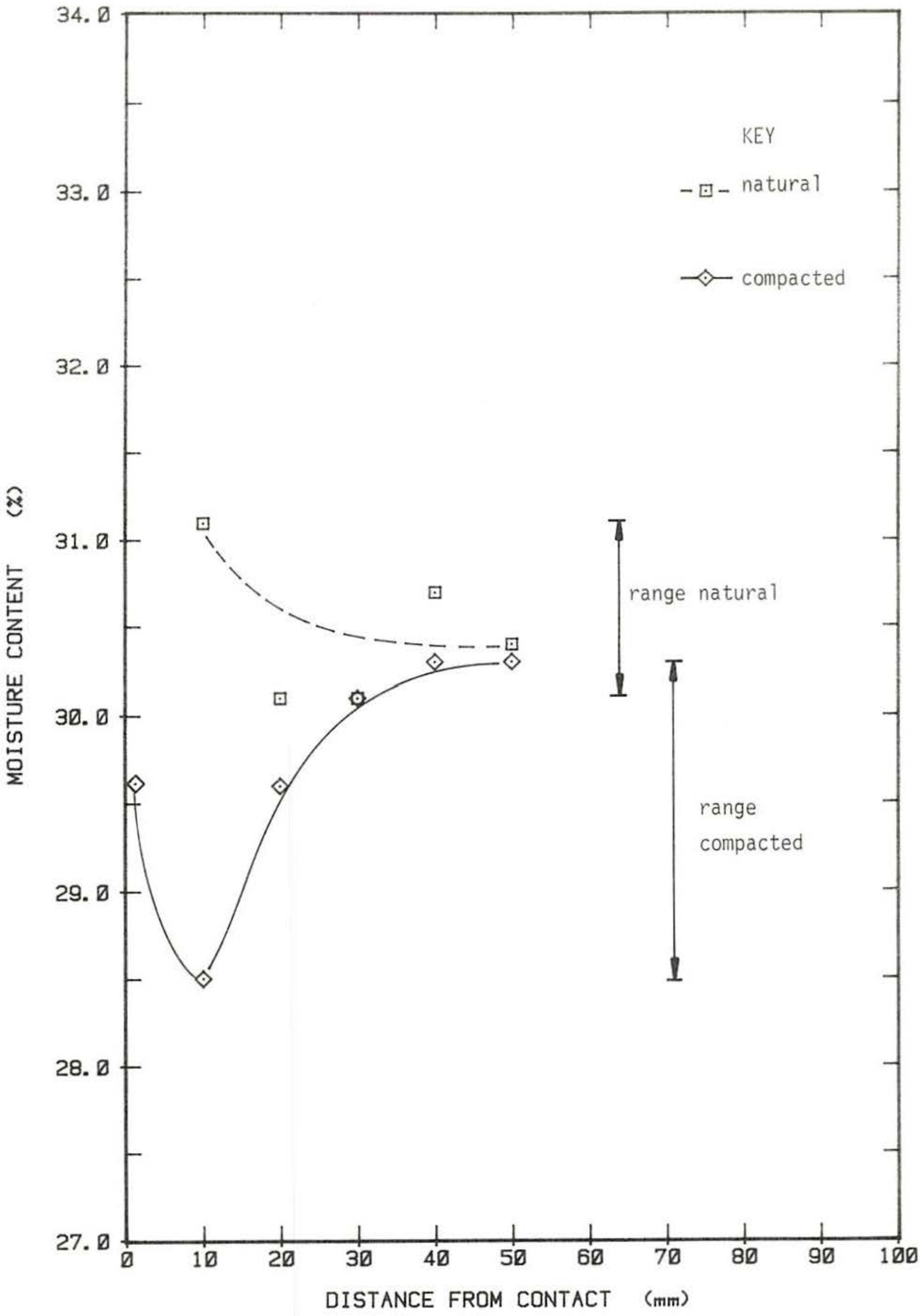


FIGURE 2.14 - MOISTURE CONTENT VARIATION IN UNDISTURBED (NATURAL) AND COMPACTED SPECIMENS OF LONDON CLAY, SERIES E

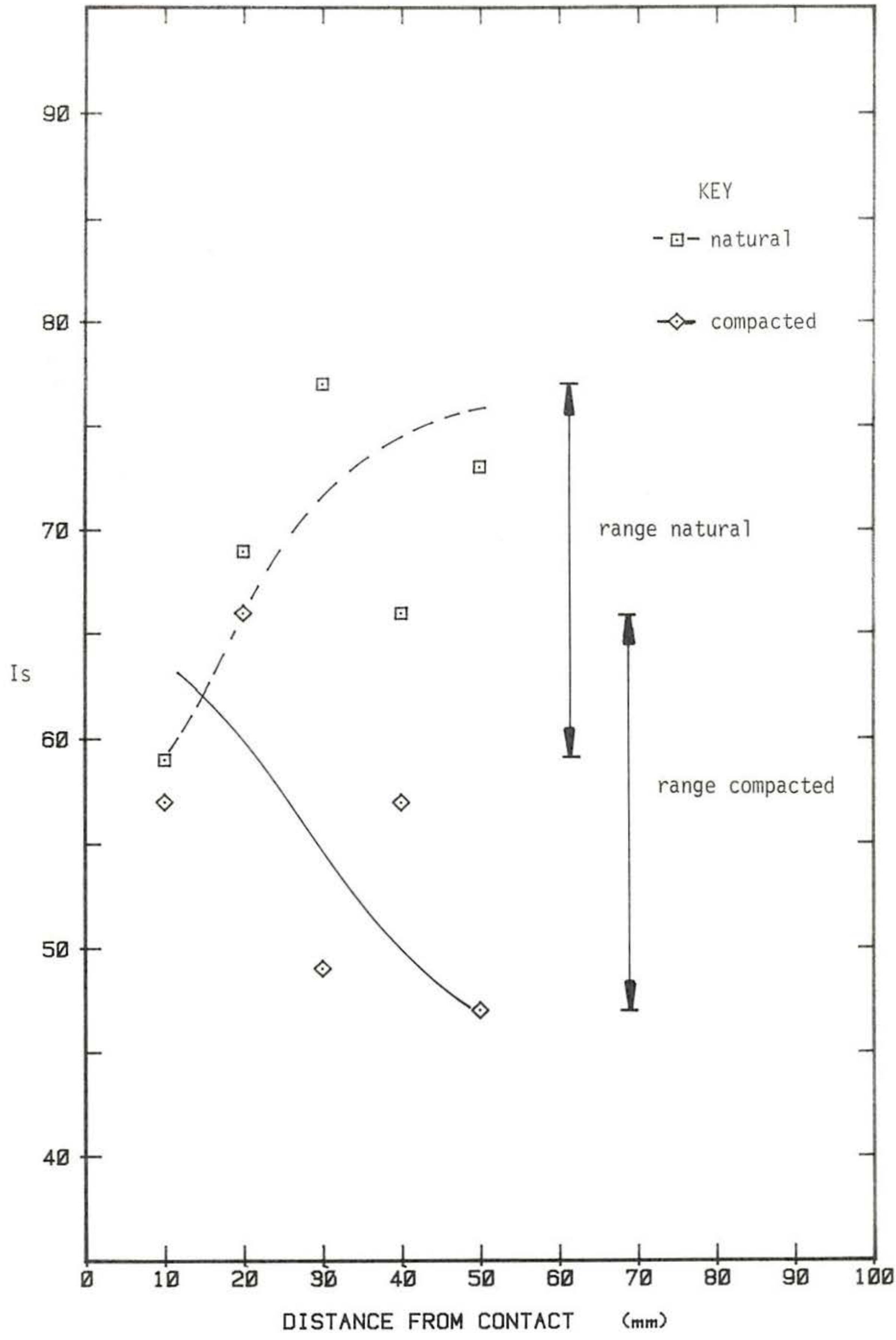


FIGURE 2.15 - "INDEX STRENGTH" VARIATION IN UNDISTURBED (NATURAL) AND COMPACTED SPECIMENS OF LONDON CLAY, SERIES E

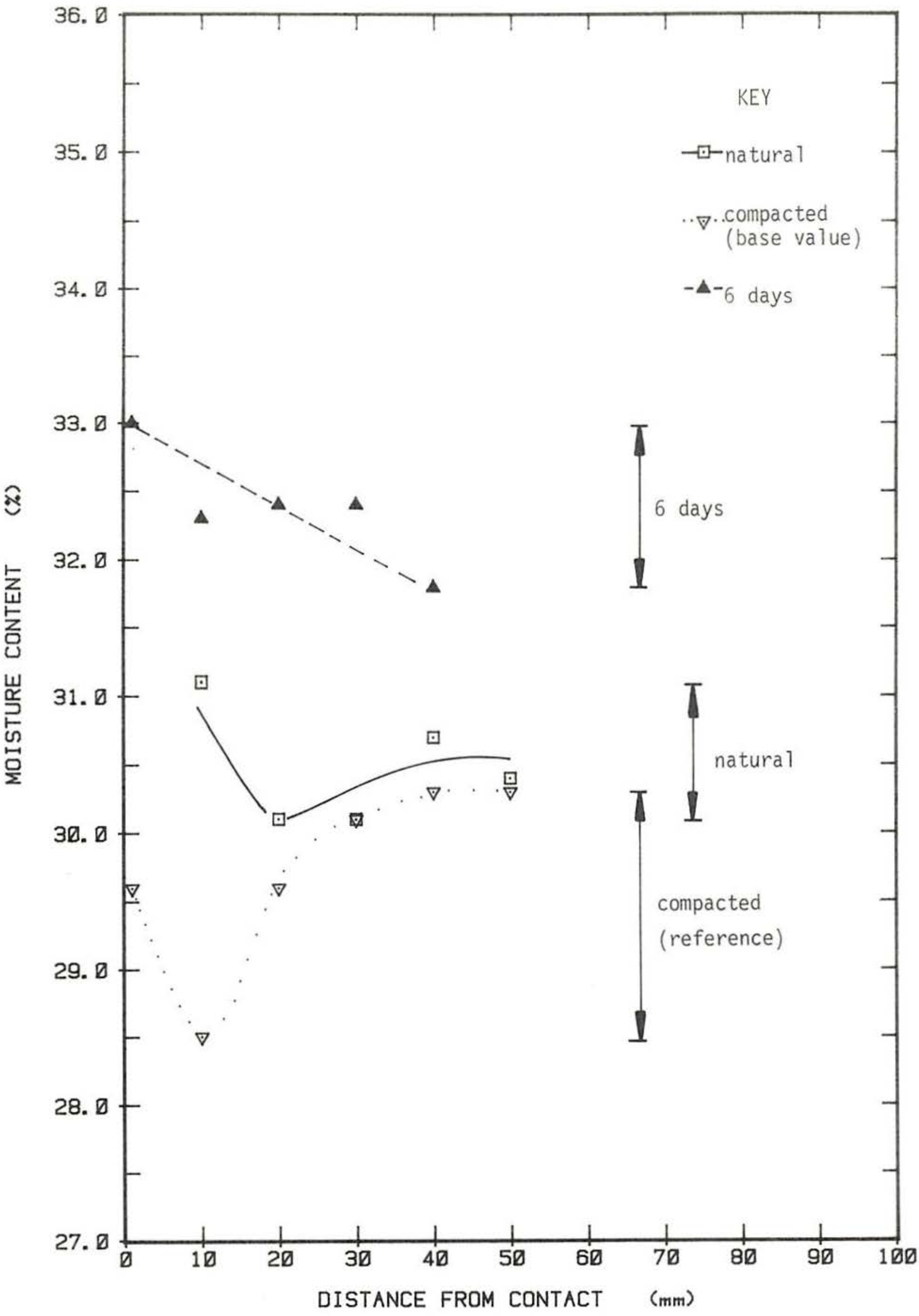


FIGURE 2.16 - MOISTURE CONTENT VARIATION, SPECIMENS OF LONDON CLAY, a)UNDISTURBED,b)COMPACTED,c)COMPACTED IN CONTACT WITH FRESH CONCRETE

CHAPTER 3 - MOISTURE CONTENT AND CONE RESISTANCE CHANGES NEAR BORED PILES

CHAPTER 3 - MOISTURE CONTENT AND CONE RESISTANCE CHANGES NEAR BORED PILES

3.1. - INTRODUCTION

In order to study the local changes in soil conditions due to the installation of bored piles, the ideal situation is to compare laboratory results obtained under controlled conditions with field measurements. It is unlikely that simple experiments performed in the laboratory will simulate the complex series of events taking place during the installation of a real bored pile. The effects of remoulding caused by the boring tool, relief in stress, effect of the use of casing, use of drilling mud, pouring the fresh concrete and setting of the concrete in natural soils can be assessed only in field experiments. A major problem that must be overcome is to find a suitable place, with favourable conditions for soil sampling adjacent to the piles. Routine piling does not present the possibility of access to the soil affected.

A very convenient situation is presented when a contiguous bored pile wall is built. The excavation that follows pile installation allows easy access to the surrounding soil at depth. By the kind suggestion of Dr. W.G.K. Fleming of Cementation (the piling contractor for the site), attention was drawn to a site in London.

The construction of a 10 story block (Peninsular House) with a three story basement on a site bounded by Monument Street, Lower Thames Street and Pudding Lane, Billingsgate, London EC3 gave the opportunity of access to soil layers enabling measurement of the effects of pile installation in real conditions.

The writer gratefully acknowledges the assistance of Pell, Frishmann and Partners, Consulting Engineers who gave permission to go in site and provided local support.

Work was undertaken to obtain samples from London Clay, 12 m below ground level. Samples were taken close to the piles within the site and from the surrounding wall. The main difficulty was timing site visits to fit in with the excavation work. Both block samples and 1" samplers were used. The description of the site, project, location of testing and test results will be presented in this chapter, with the analysis and conclusions.

Cone penetration tests were performed in accordance to the procedures described in Chapter 2, using a 650 grammes, 30^{o} cone.

All block samples were tested for cone penetration in site. The specimens obtained with the one inch sampler were extruded and tested in the Soil Mechanics Laboratory, at the University of Surrey.

3.2. - THE SITE

3.2.1. - LOCATION

Figure 3.1. shows the plan of the site, bounded by Monument Street, Lower Thames Street and Pudding Lane, Billingsgate, London EC3.

3.2.2. - SITE INVESTIGATION

The site investigation was a routine contract, carried out by Terresearch Limited, Foundation Engineers & Contractors (Report No 5.24/738). It consisted of the drilling of five boreholes at the site, during February and March, 1974. The boreholes were 200 mm and 150 mm nominal diameter and were drilled with light percussion equipment. Borehole 2 was drilled to a depth of 15.5 m whilst the remaining boreholes were 30 m deep. The positions of the boreholes are shown in Figure 3.1.

Samples of the clay layer were taken using standard 102 mm diameter open drive sampling tubes. The results of the laboratory testing programme (unconsolidated undrained triaxial compression tests) are shown in Figure 3.2.

3.2.3. - GROUND CONDITIONS

The following is an extract from the site investigation report:

"The Geological Survey of England and Wales, London Sheet No N $\underline{\overline{V}}$ S.E. (scale 6 inch to 1 mile) shows Higher Flood Plain Gravel over London Clay in the area of the site. A borehole for water drilled near King William Street about 100 yards to the North West of the site, proved the London Clay to extend to a depth below surface of about 44 m.

The borehole investigation confirmed this general sequence. Boreholes 1 and 2 on the site of the former Billingsgate Buildings proved made ground comprising concrete over clay with gravel and shell fragments to a depth of between 1.25 and 1.90 m. On the western part of the site Boreholes 3 and 4 were drilled through a basement void and concrete floor, beneath which bricks, clay and timbers were proved to a maximum depth of 7.20 m. In Borehole 5 the made ground comprised concrete over clay, sand and timber to a depth of 4.85 m.

Beneath the made ground the boreholes penetrated medium dense fine to coarse gravel which was 5.35 m thick in Borehole 1 but only 1.75 to 2.00 m thick in Boreholes 4 and 5 and was absent in Boreholes 2 and 3.

All five boreholes entered stiff to very stiff London Clay and were terminated in this stratum at depths of between 15.50 and 30.00 m (-12.00 to -26.65 m O.D). The clay was generally slightly silty, highly fissured and below a level of about - 15 m O.D. frequent partings of fine grey sand typical of the lower parts of the London Clay stratum were observed in most of the boreholes. The clay was for the most part dark grey in colour but was weathered to brown over the upper part to a maximum of 5.80 m in Borehole 3.

Groundwater was struck at a depth of 6.80 m below surface in Borehole 5 but no strikes were observed at shallow depth in the remaining boreholes due to the addition of water to assist drilling through the made ground and gravel.

Additional strikes were noted at depths of between 23.0 and 25.0 m in the fissured London Clay. The standing water level rose significantly in all boreholes when left overnight on completion of drilling, in Borehole 1 for example, the level rose from a depth of 30.0 m to within 1.50 m of the surface. Final standing water levels measured after completion of drilling ranged from 1.50 to 5.70 m depth (1.85 to 2.30 m O.D.)."

Figure 3.2 presents a general description of the strata, according to the site investigation report.

3.3. - THE PROJECT

A general section through the project is presented in Figure 3. 3. The building was designed to house an investment company, part of the basement being used as a bullion storage.

The construction sequence was as follows:

a) - installation of the piles and retaining wall from the initial ground level, the piles being concreted up to the pile cap levels and backfilled up to ground level to avoid stability problems;

b) - general excavation of the ground, with a temporary heavy steel bracing structure supporting the wall;

c) - excavation for each pile cap individually ;concreting the pile cap. After concreting all pile caps, the basement slab was concreted;

d) - followed by concreting of columns and slabs with the subsequent withdrawal of the internal bracing.

Table 3.1 presents details of the concrete used in the foundations. The technique applied for the installation of the wall and piles was the use of casing in the granular top layer and a bentonite slurry for the rest of the piles.

Figure 3.4 presents the position of the testing area, with the identification of the piles in the wall and the pile cap used for the research work.

3.4. - TEST RESULTS

The results obtained will be presented in Figures 3.5 to 3.10 with the identification of the samples being tested. The sampling was performed immediately after excavation for the pile cap. Apart from the samples tested, two block samples were lost due to the presence of fissures.

PILE 35

Figure 3.5 shows the location of the tests made near the pile 35. One inch samples were taken from the bottom of the excavation for the pile cap (15/9/1980) in order to study the variation of moisture content and cone penetration at different points at the same depth. Of the six samples taken, only four were subsequently used; the natural fissures of the soil proved to be a major problem in obtaining samples in a condition suitable for testing with the cone. For the penetration tests the samples were extruded and, at each position on the sample, 3 penetration tests were performed and the average used as a single result. The moisture content and cone penetration results obtained are presented in Table 3.3 and Figures 3.6 a and 3.6 b.

Considering the moisture content results, it appears that the variation in natural moisture is considerable (Figure 3.6 a), for some samples up to 2% of moisture content. When the averages for each sample were plotted (Figure 3.6 b) no significant trend could be identified. The results of cone penetration, plotted on the same figure, showed a clearer trend, being higher near the pile soil contact and lower far from that region. Figure 3.6c shows the average "Index strength" variation near the pile, showing a clear trend.

PILE 36

Figures 3.7 a and b shows the position of the block sample, testing procedures and results obtained near the pile 36 (15/9/1980). A well defined trend is observed in both moisture content changes and cone penetration (carried out on site).

PILE 37

The results obtained when testing the material near pile 37 (15/9/1980) are plotted in Figure 3.8 b. The measured moisture content values showed a small variation (less than 1%) and the penetration values were scattered.

PILE 26 (wall)

The testing procedures used in pile 26 (in the wall) when a one inch sampler was used to obtain a representative specimen of the soil adjacent to the wall (10/9/1980) is presented in Figure 3.9 a. In Figure 3.9 b the results obtained are presented. A well defined trend in both moisture content variation and cone penetration exists.

PILE 28 (wall)

The layout and testing procedures near pile 28 (in the wall) is presented in Figure 3.10 a. A block sample was taken (10/9/1980) and cone testing was carried out on site. The results obtained are presented in Figure 3.10 b. The shapes of the moisture content curve and the cone penetration curve are similar. An increase in moisture in the area close to the concrete (5 cm) with an equivalent increase in cone penetration (decrease in shear strength) in the same region was observed.

A summary of the measured results is presented in Table 3.2. A general description of the results obtained for all piles is as follows:

- after four months from pile installation, the moisture content measured near the piles in 3 out of 5 cases showed a clear trend, with higher values in the zone up to 5 cm thick, adjacent to the piles;

- in all cases there was a variation (from 0.9 to 4.4% moisture content) in moisture content, when comparison is made with values obtained close and far from pile - soil contact;

- the highest moisture content measured was not always in the closest position of contact, being sometimes up to 2 cm away from that point;

- in all cases there was a clear variation in cone penetration, when a comparison of values obtained close and far from the pile - soil contact was made;

- the position of the highest cone penetration values recorded (i.e. the position of lowest shear strength) in each test did not occur close to the pile, being in some cases up to 3 cm away from the contact;

- there is no coincidence between the positions of highest moisture content and lowest shear strength (highest cone penetration) measured;

- the shear strength values close to the interface between pile and soil , in most cases, is higher than might be expected from its moisture content. It seems that the contamination in the water leaving the concrete may penetrate and strengthen soil close by. This process has been suggested by Chuang and Reese, 1969, when analysing the results of shear box tests.

Part of the foundation	Class of concrete (N/mm^2)	Additives	Water/cement ratio	Installation
Piles	30	Plastic.*	0.5	
No 35				7/5/1980
No 36				9/5/1980
No 37				5/1980
Wall	35	Plastic.	0.5	
No 26				5/1980
No 28				5/1980

* Flocrete N , 200ml/50 kg cement

TABLE 3.1 . - Identification of materials used in the foundation's work.

3.5 - ANALYSIS

In order to study the changes in shear strength observed using the cone, the values of the "Index strength" (I_s), presented in Chapter 2, item 2.2, were calculated and presented in Figures 3.7c, 3.8c, 3.9c and 3.10 c.

The symbols (I_s)max and (I_s)min refer to maximum and minimum values of the obtained "Index strength". The value of the penetration obtained when testing nearest to the clay/concrete contact was identified as Pc, and the corresponding I_s as (I_s)c.

The values of maximum and minimum moisture contents for each sample were plotted versus minimum and maximum I_s in Figure 3.11 for all tests. The general trend in "Index strength" variation showed a defined shape, the increase in moisture resulting in a decrease in strength. There is a significant scatter of results, typical of natural soils.

It is important to note at this point that the "Index strength" being measured by the cone is related to the intact shear strength of the material tested. The changes measured are due to installation procedures, the increase in moisture content being a result of a number of factors already discussed in Chapter 1.

To investigate a possible relationship, the changes in moisture content and the ratio of "Index strength" R observed in the tests performed were plotted in Figure 3.12.

$$R = (I_s)\text{min}/(I_s)\text{max}$$

The values of R are in the range 0.36 to 0.74. As the number of experimental points is small, only a trend can be suggested (specially if one considers that for zero change in moisture content R must be 1). The increase in moisture content results in a reduction of R.

Values of Rc were calculated and plotted in Figure 3.13 versus moisture content change for each pile.

$$Rc = (I_s)c/(I_s)max$$

Rc values were in the range 0.62 to 0.88. Test results with the pile 35 were discarded as the Pc value was not obtained in the testing programme. The same kind of trend observed in R variation was obtained, with consistently higher values of Rc.

With the objective of studying the possible effect of the initial strength on the strength reduction, values of R and Rc versus $I_s$max were plotted in Figures 3.14 and 3.15 respectively. No trend could be identified.

A comparison of the obtained R and Rc values for the piles on the wall and on the foundation is made in Table 3.4. The number of tests is not big enough for any conclusion but it appears that the reductions near the foundation were lower than those for the wall. There are various possible reasons for the bigger reduction in the wall, such as: variability in soil sensitivity, group effect on the wall, minor differences in installation conditions at that specific point. In the case of ideal identical conditions, the fact that the piles in the wall were concreted up to the original ground level, raising the pore water pressure at the test level, and the foundations were simply backfilled with soil with no big rise in pore water pressure, is a possible mechanism responsible for the difference.

Test Identification	1	2	3	4	5
Sampling Technique	1"sampler	Block sample	1"sampler	1"sampler	Block sample
Pile	35	36	37	26(wall)	28(wall)
Lowest Moisture Content (%)	26.3(av)	26.8	28.2	26.9	25.3
Highest Moisture Content (%)	27.7(av)	29.8	29.1	31.3	28.1
Variation in Moisture (%)	1.4(av)	3.0	0.9	4.4	2.8
Affected zone (mm)	(?)	60	40	60	70
Lowest Penetration P_{min} (mm)	3.01(av)	3.0	2.5	3.3	2.4
Highest Penetration P_{max} (mm)	3.95(av)	3.5	2.9	5.5	3.4
$\triangle P$ (mm)	0.94	0.5	0.4	2.2	1.0
Pc (mm)	-	3.2	2.8	4.2	3.1
I_s max	110	111	160	92	145
I_s min	64	81	119	33	86
I_s c	-	98	127	57	104
$R = I_s \min/I_s \max$	0.58	0.73	0.74	0.36	0.59
$Rc = I_s c/I_s \max$	-	0.88	0.79	0.62	0.72

TABLE 3.2 - Summary of the measured results, calculated "Index strength" I_s, and Ratios of "Index strength" R and R c .

Pile	R	Rc
Wall		
26	0.36	0.62
28	0.59	0.72
average	0.48	0.67
Foundation		
35	0.58	-
36	0.73	0.88
37	0.74	0.79
average	0.68	0.83

TABLE 3.3 - Comparison of R and Rc values obtained for the piles from the wall and foundations

3.6 - CONCLUSIONS AND RECOMMENDATIONS

Four months after pile installation a variation in moisture content (from 0.9 % to 4.4 % moisture content), when comparing values obtained close and far from pile-soil contact, was found. Three out of five cases showed a clear trend, with higher values in the zone up to 5 cm thick, adjacent to the piles. The highest moisture content measured was not always in the closest position of contact, being sometimes up to 2 cm away from that point. A possible explanation for that reduction is the existence of suction forces in the concrete, generating a further water flow towards the pile.

In all cases there was a clear trend in "Index strength" (I_s), when a comparison of values obtained at different distances of the pile-soil contact was made. The position of the lowest "Index strength" recorded in each test did not occur close to the pile, being in some cases up to 3 cm away from the contact. There was no coincidence in positions of highest moisture content and lowest "Index strength". It seems that the cement components in solution in the water leaving the concrete may penetrate and strengthen soil close by.

The calculated ratios of minimum and maximum "Index strength" (R) are in the range 0.36 to 0.74. When comparing the "Index strengths" obtained in the closest position to the contact between pile and soil with maximum values, the calculated ratios (Rc) were in the range 0.58 to 0.88. It appears from the experimental results that a trend can be suggested, when plotting the variation in moisture content and R for each test, the higher the increase in moisture, the lower measured R.

No effect was found of a possible influence of the initial (maximum) "Index strength" on R or Rc for the tests performed.

When comparing the ratios of "Index strength", R and Rc, in piles from both the wall and from the foundations, it was clear that the reductions in the wall were consistently higher. A number of causes could be responsible for the different behaviour, such as: variability in soil sensitivity, group effect in the wall, minor difference in installation condition at that specific point. The fact that the piles in the wall were concreted up to the original ground level, generating a big rise in the pore pressure at 12 m depth and the foundations were simply backfilled with soil, with no rise in pore pressure is a possible explanation for the difference.

Considering that the "Index strength" for a fissured clay is a measure of the "intact undrained shear strength" and the failure surface on the soil when the pile-soil system fails is most unlikely to coincide with existing fissures, it can be speculated that the cone measurements of variation in strength can provide useful information in the study of the skin friction of bored piles in fissured clays.

The systematic use of cone penetration tests as a technique for studying changes in shear strength in piling practice may lead to the identification of important installation related chenges and provide evidence for the adoption of better techniques and practice.

In an instrumented pile load test where samples could be taken at different positions along the shaft of the pile during installation, a comprehensive research programme could be suggested. It should include the study in the laboratory of "Index strength" changes coupled with results of cone penetration tests performed in specimens (obtained after pile installation) from site and the pile load test, in order to provide a possible "method of estimation" of shaft bearing capacity, based on results from cone penetration tests. Another point that needs investigation is the location of the failure surface related to the strength variation near the pile shaft wall.

Scale: approx 1:500

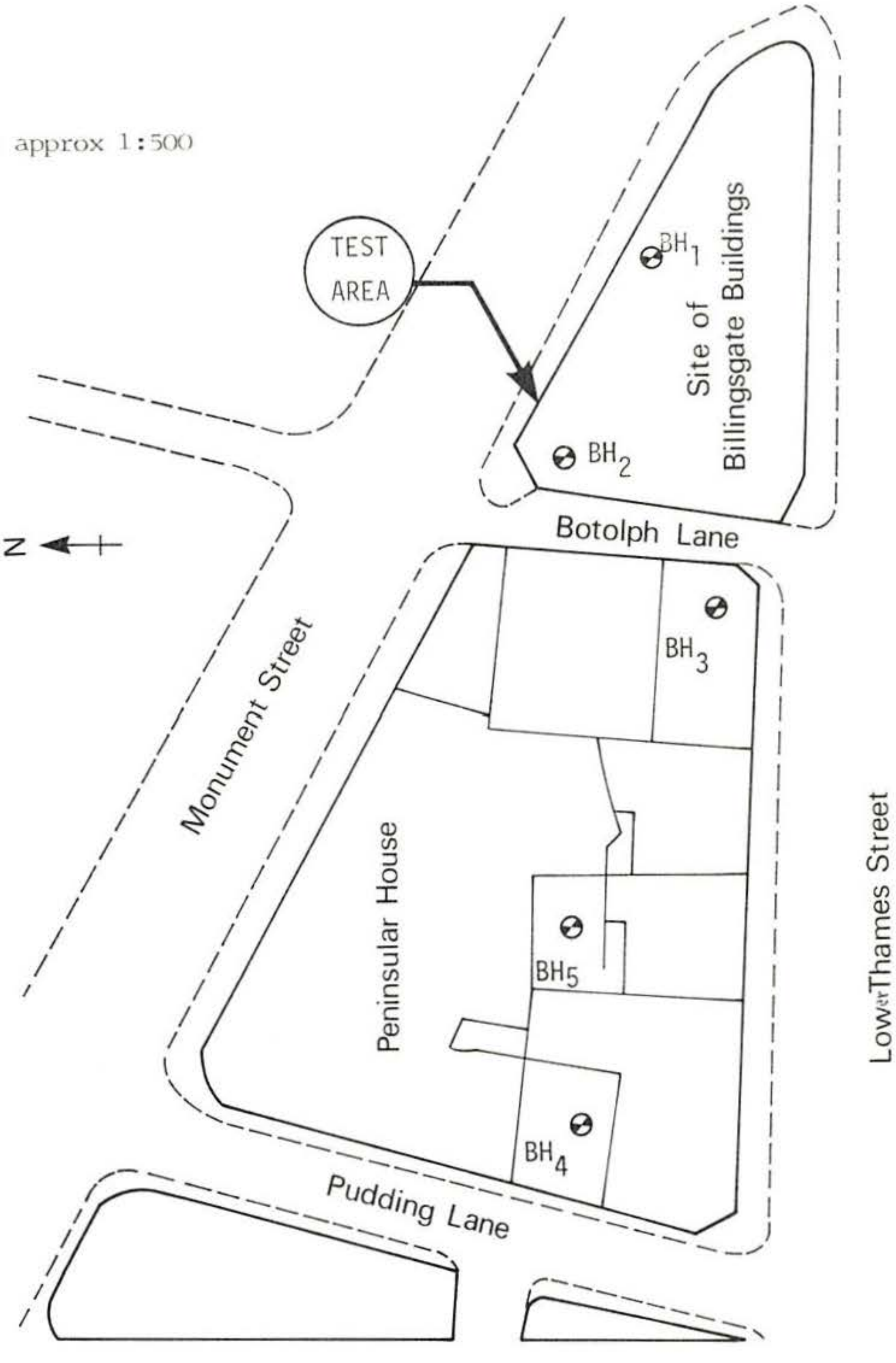


FIGURE 3.1 - PLAN OF THE SITE

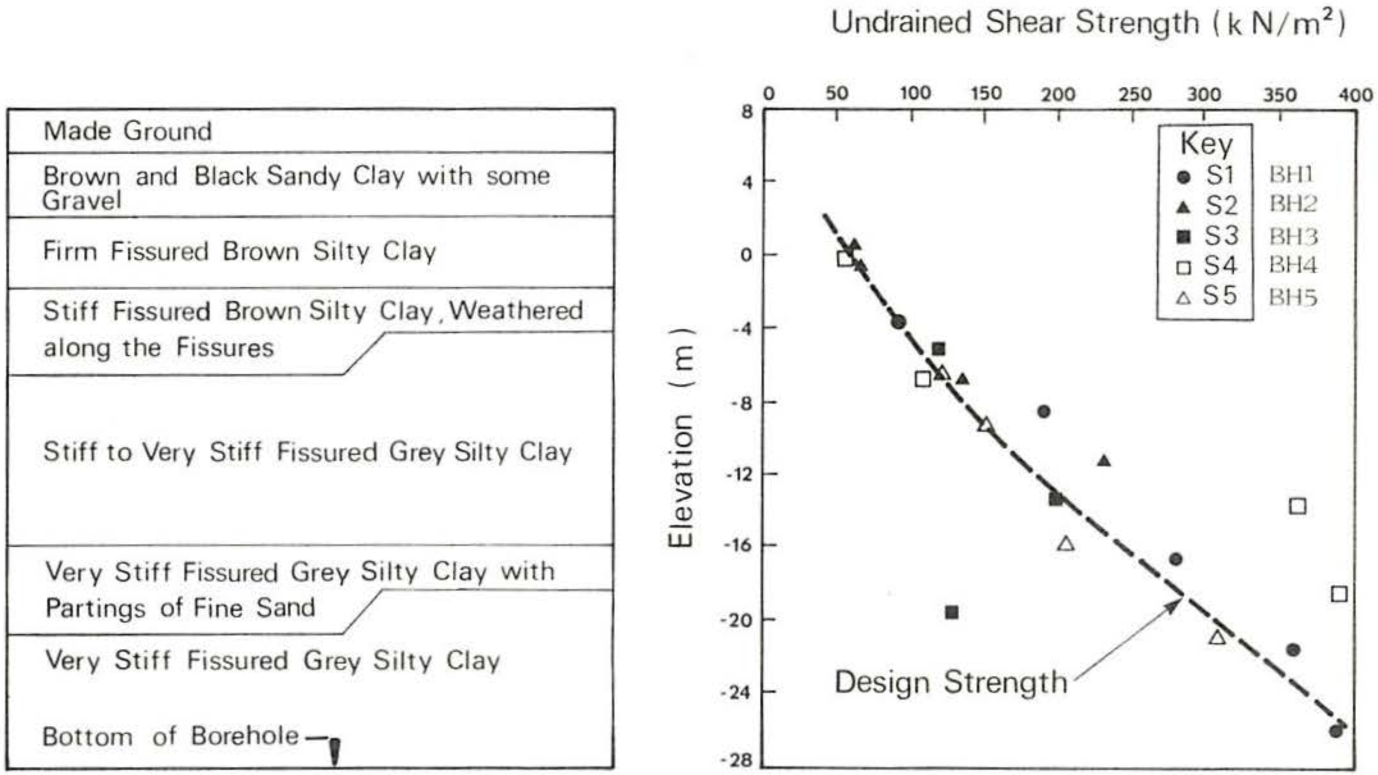


FIGURE 3.2 - GENERAL DESCRIPTION OF THE STRATA AND RESULTS OF THE LABORATORY TESTING PROGRAMME (UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS), from the Site Investigation Report

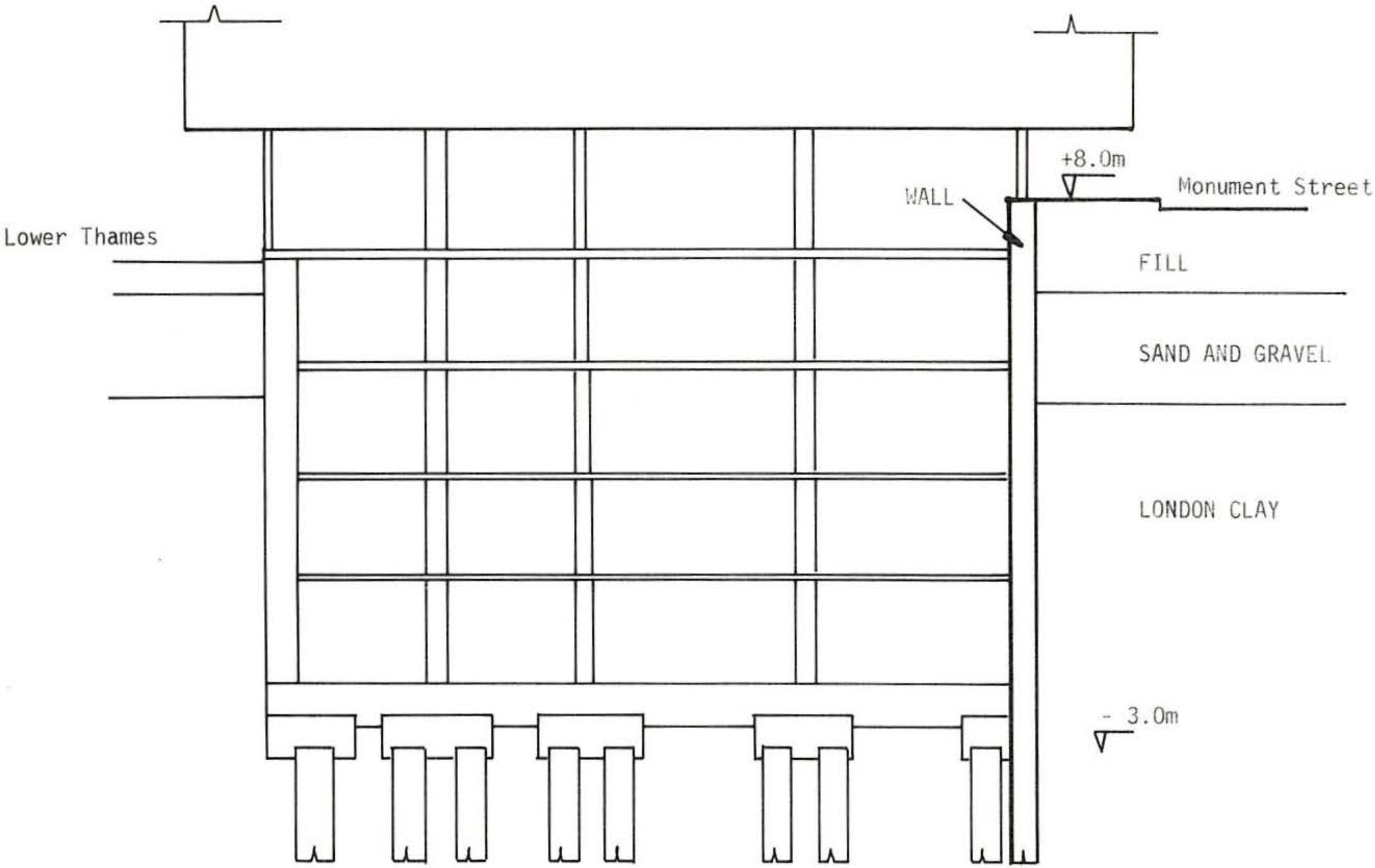


FIGURE 3.3 - GENERAL SECTION THROUGH THE PROJECT

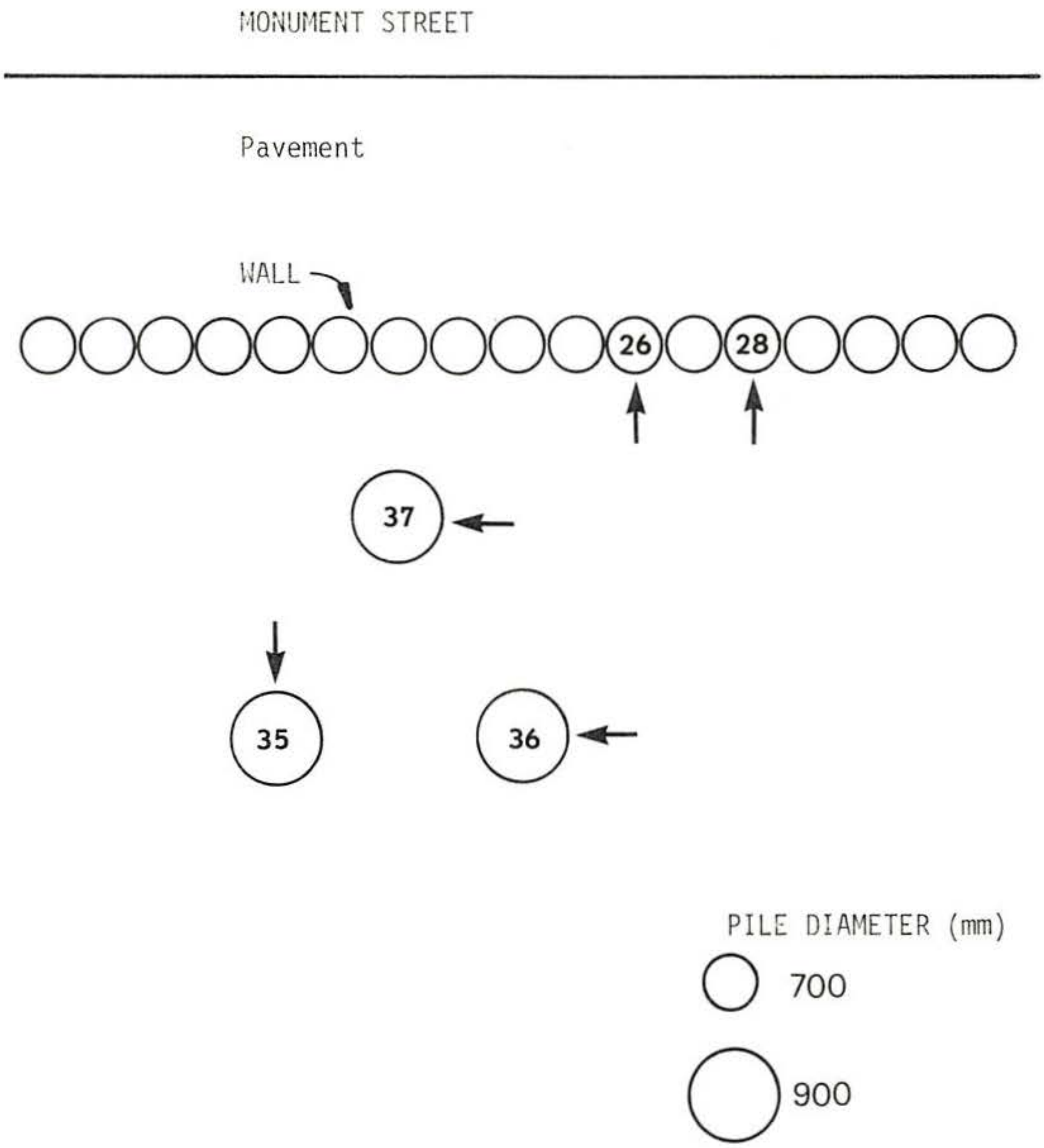


FIGURE 3.4 - POSITION OF THE TESTING AREA (see also FIGURE 3.1)

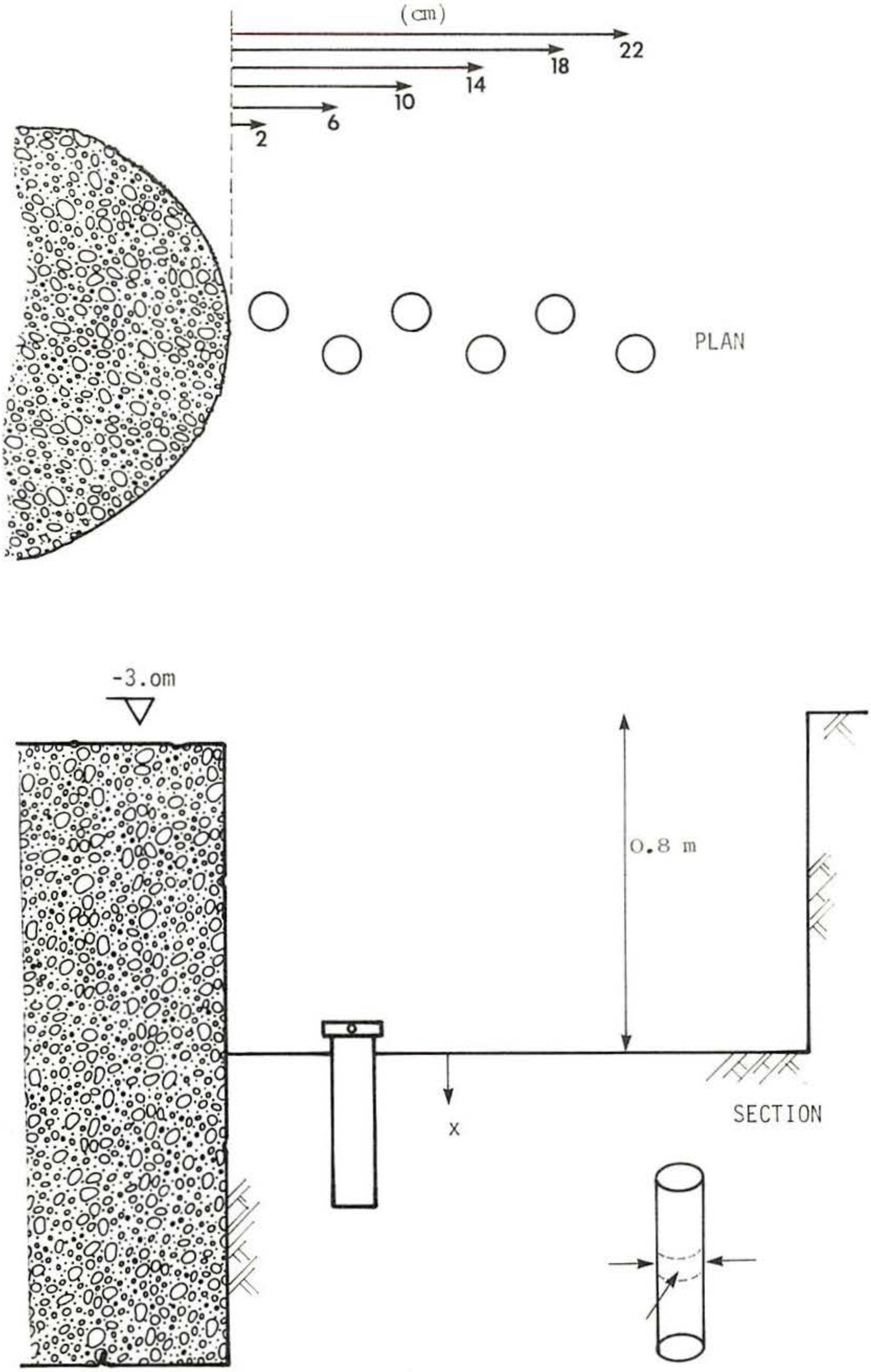


FIGURE 3.5 - LAYOUT OF THE TESTS PERFORMED NEAR PILE 35

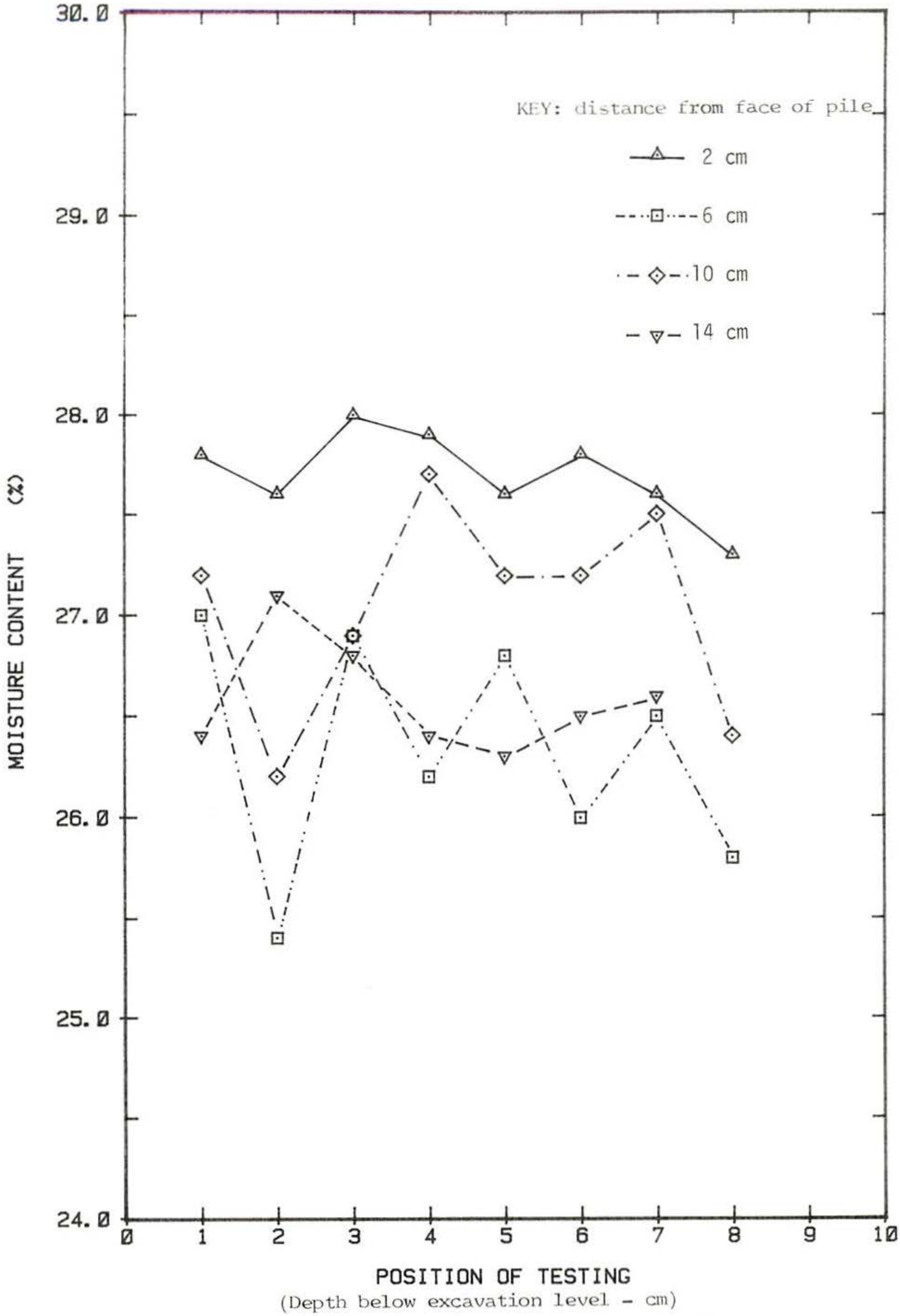


FIGURE 3.6a - MOISTURE CONTENT VARIATION NEAR PILE 35

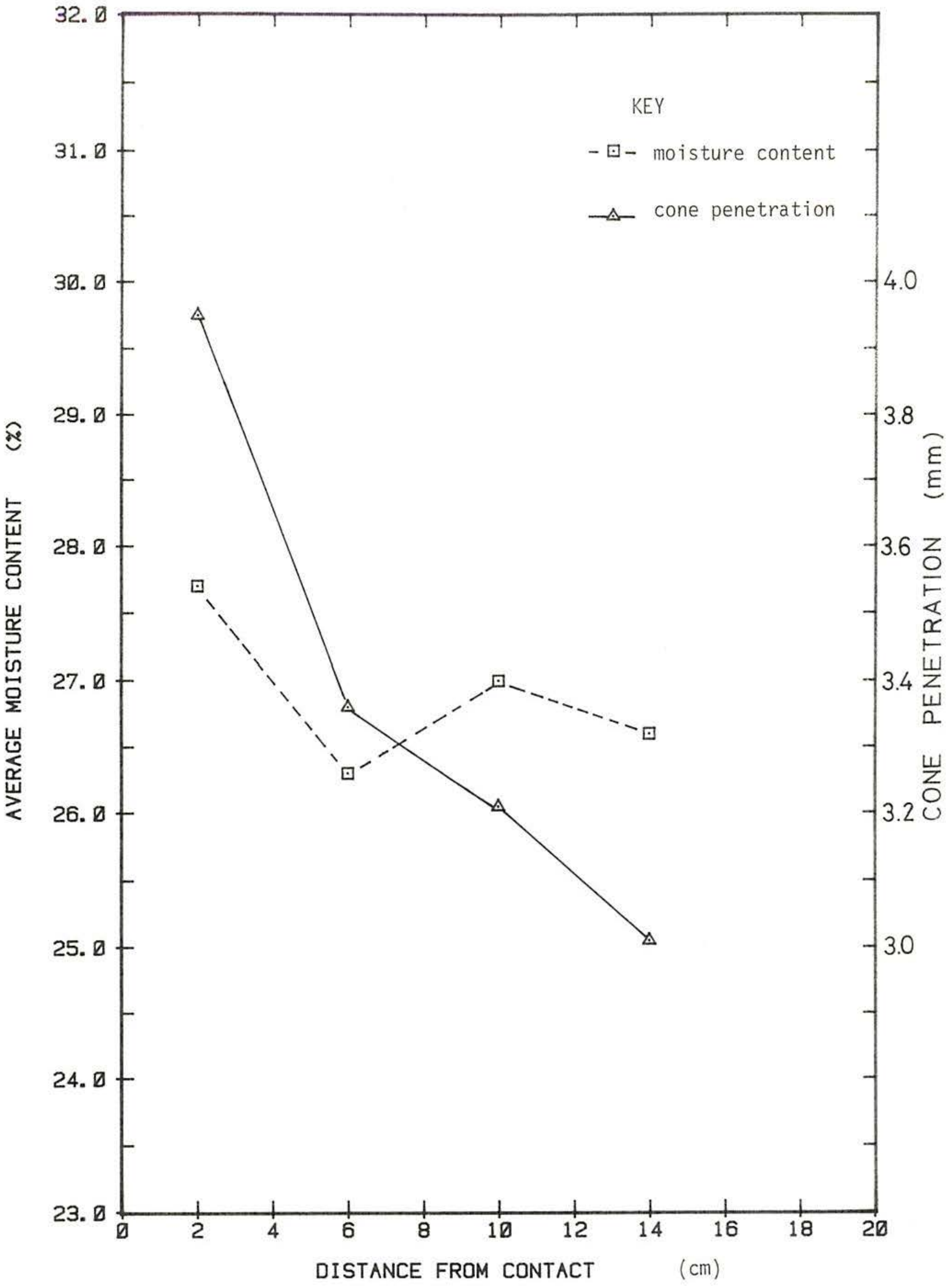


FIGURE 3.6b - AVERAGE RESULTS OF MOISTURE CONTENT AND CONE PENETRATION NEAR PILE 35

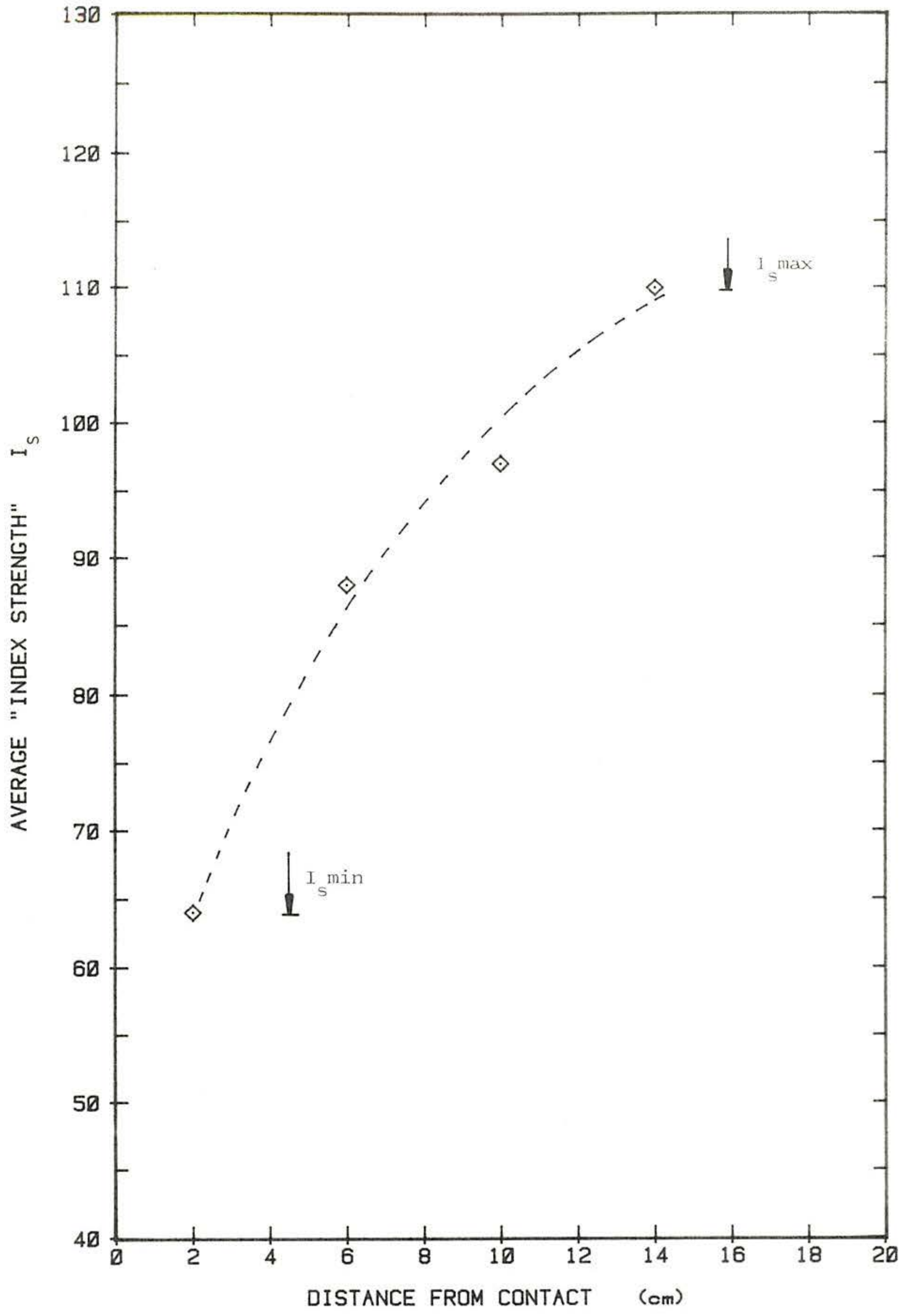


FIGURE 3.6c - AVERAGE "INDEX STRENGTH" VARIATION NEAR PILE 35

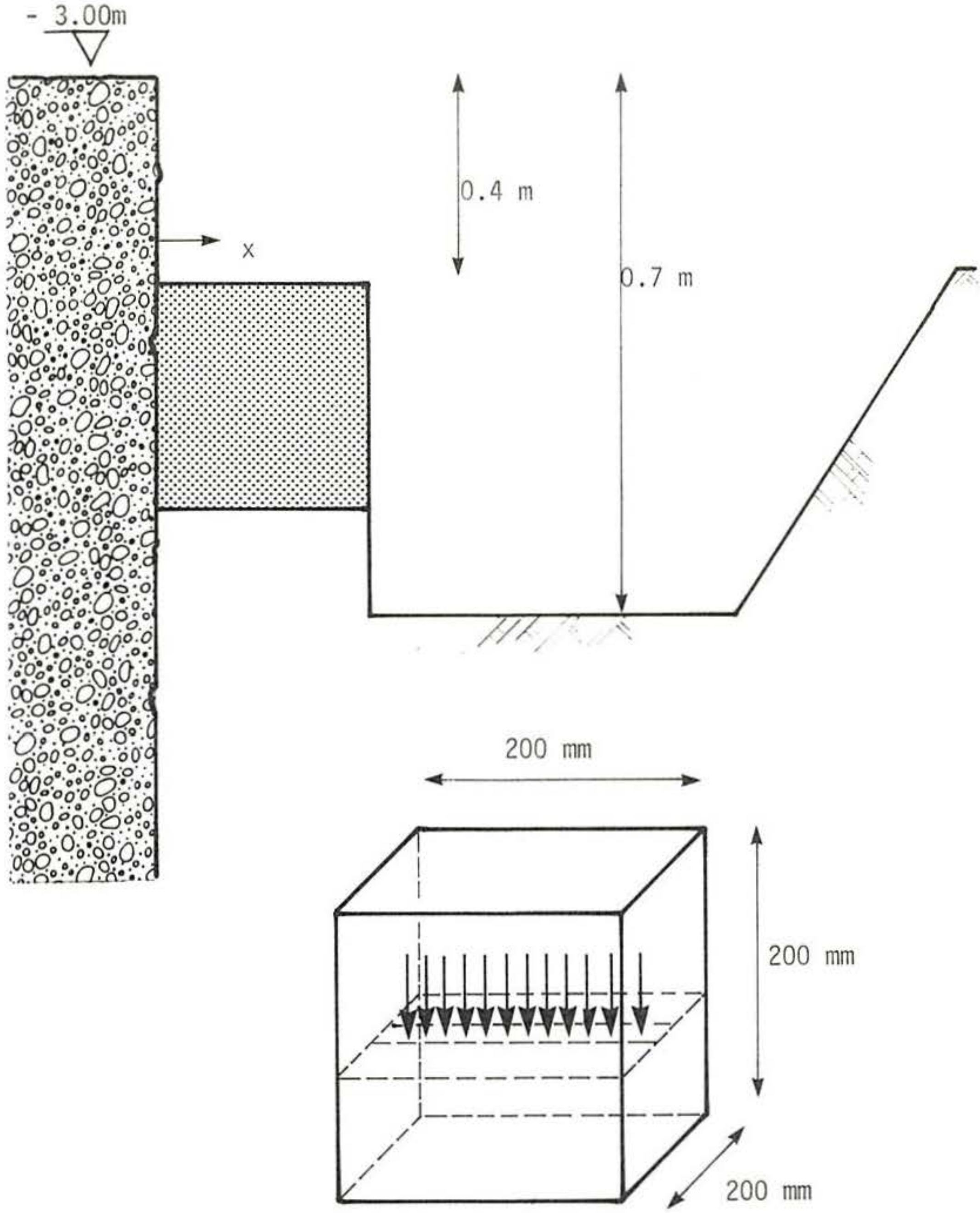


FIGURE 3. 7a - LAYOUT OF THE BLOCK SAMPLE POSITION AND TESTING PROCEDURES NEAR PILE 36

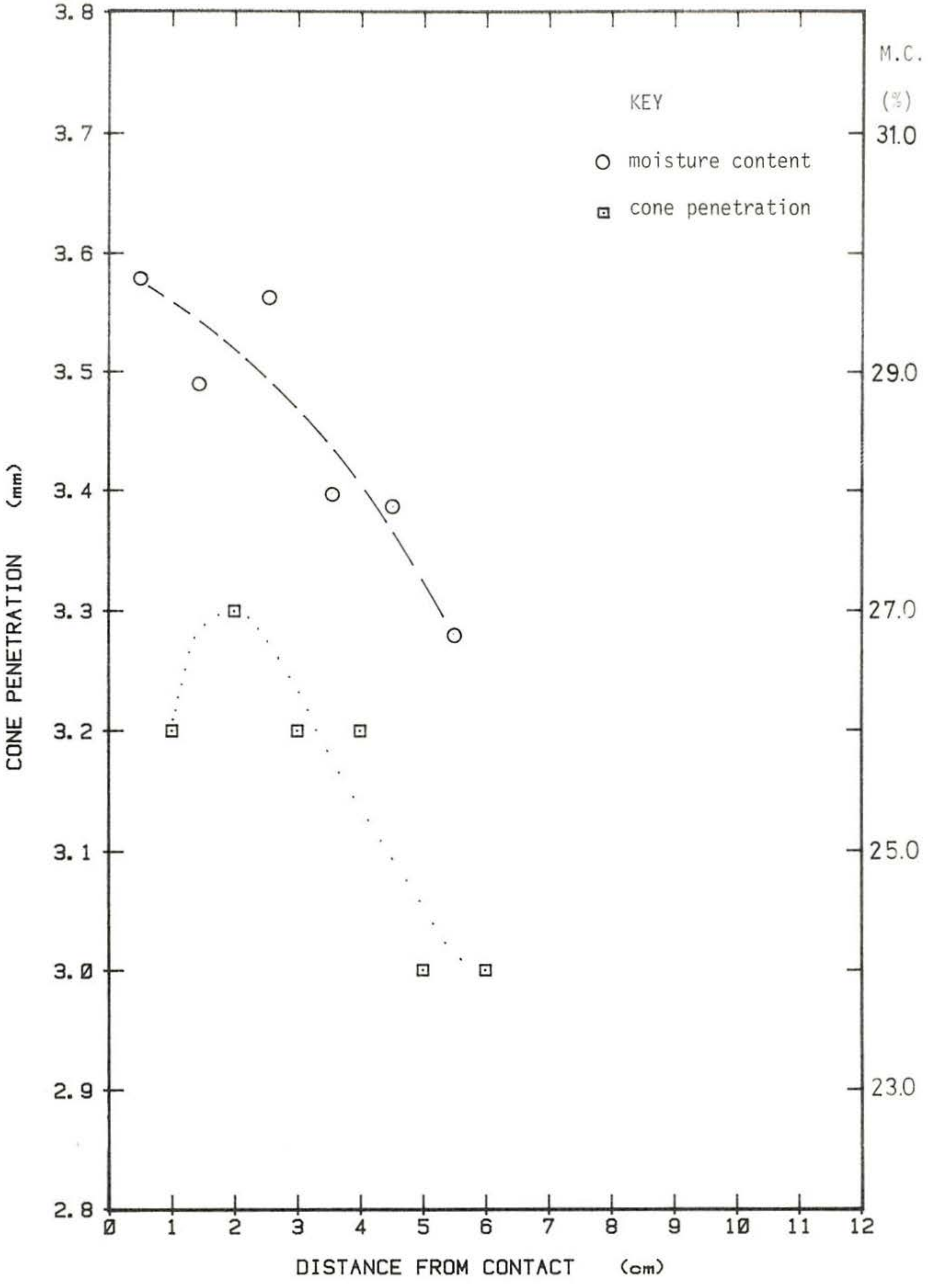


FIGURE 3.7b - MOISTURE CONTENT AND CONE PENETRATION RESULTS NEAR PILE 36

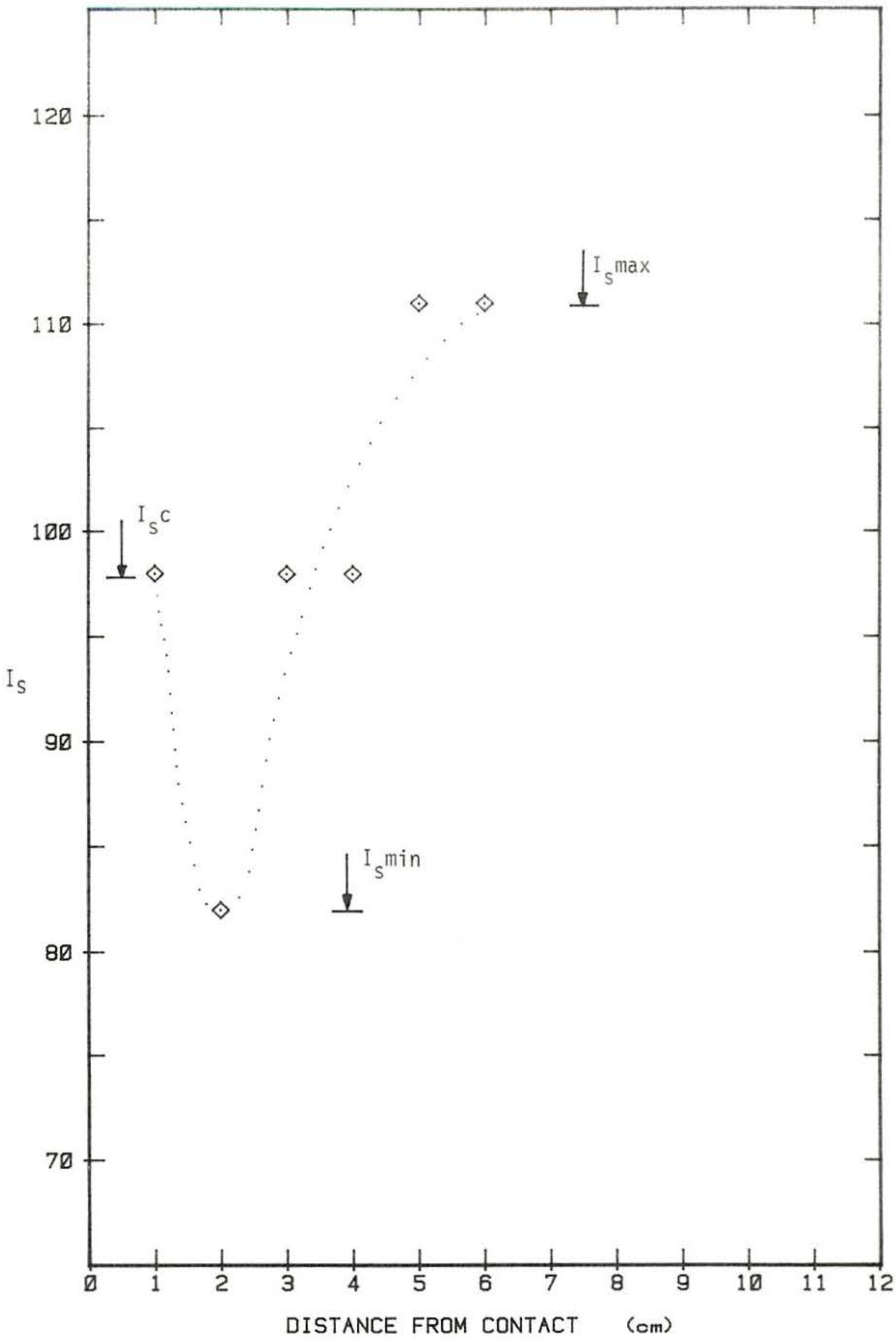


FIGURE 3.7c - "INDEX STRENGTH" VARIATION NEAR PILE 36

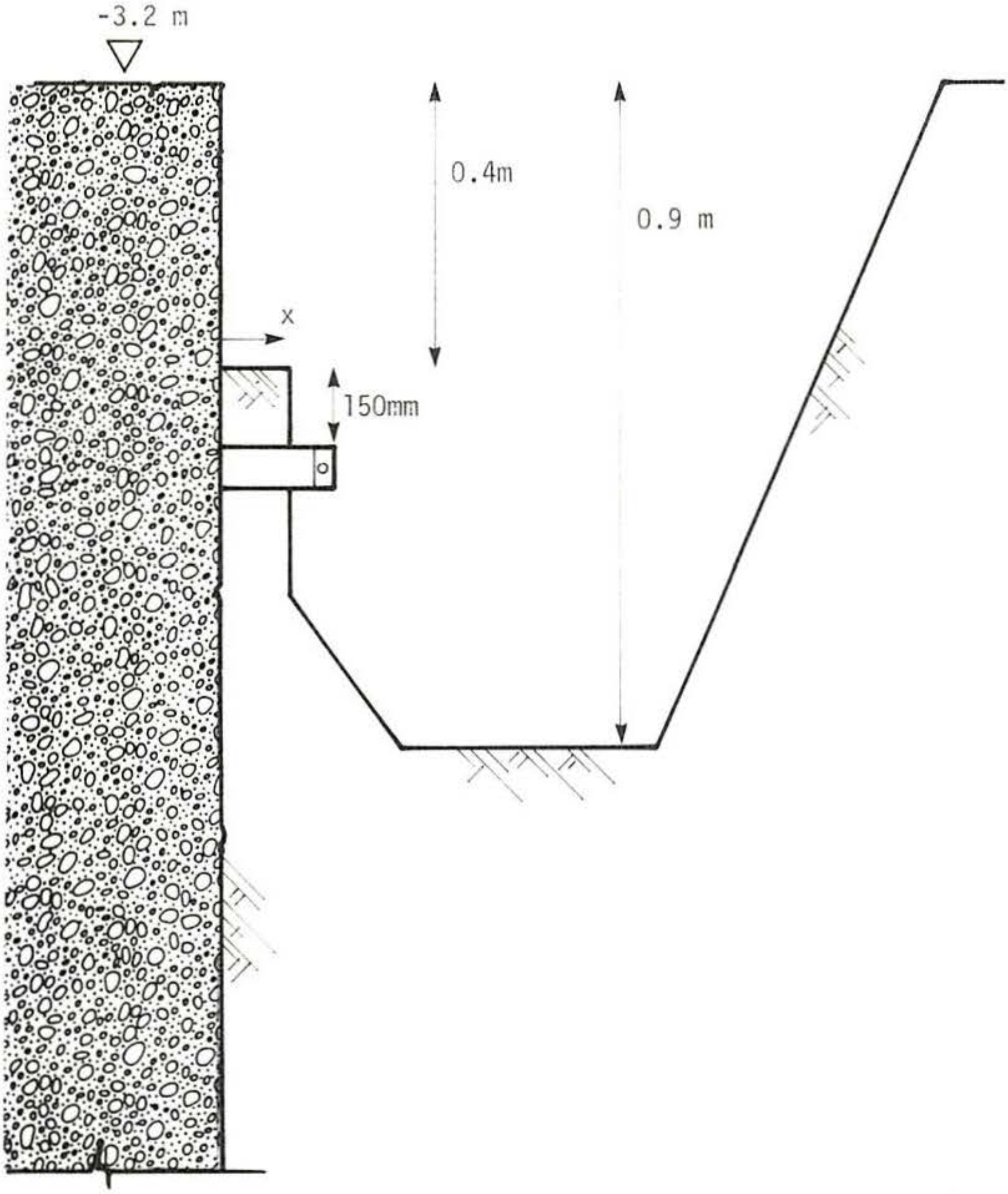


FIGURE 3.8a - LAYOUT OF SAMPLING NEAR PILE 37

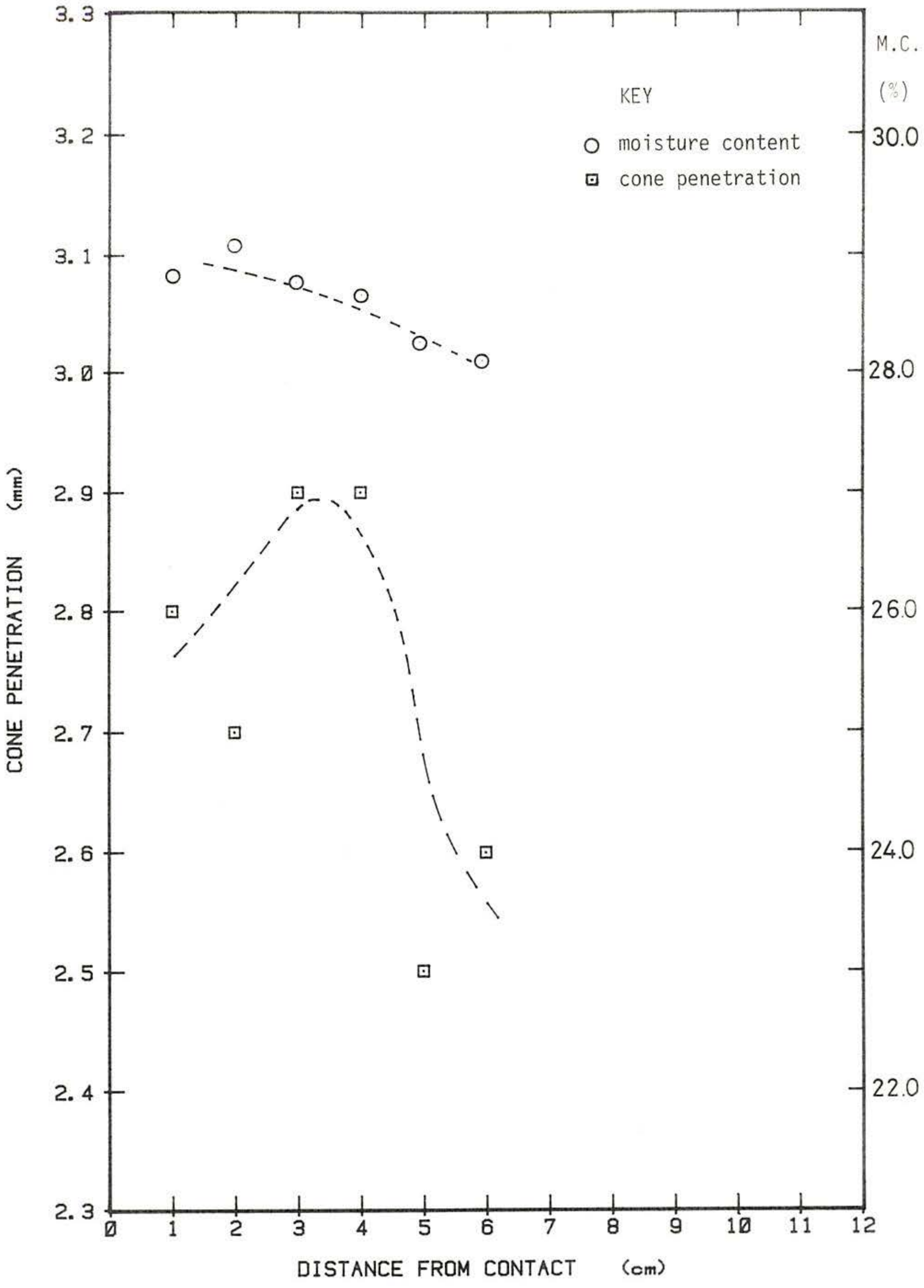


FIGURE 3.8b - MOISTURE CONTENT AND CONE PENETRATION RESULTS NEAR PILE 37

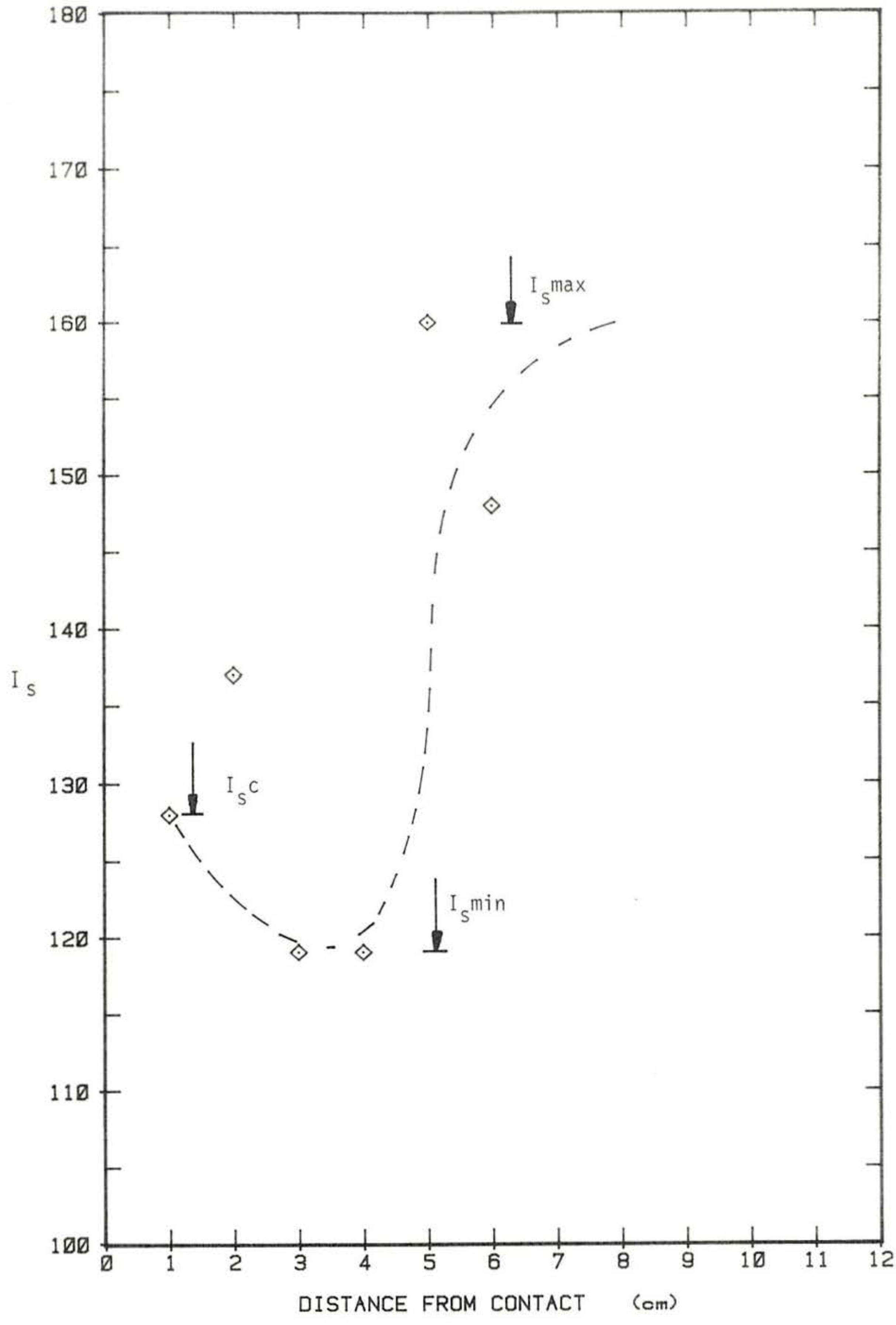


FIGURE 3.8c - "INDEX STRENGTH" VARIATION NEAR PILE 37

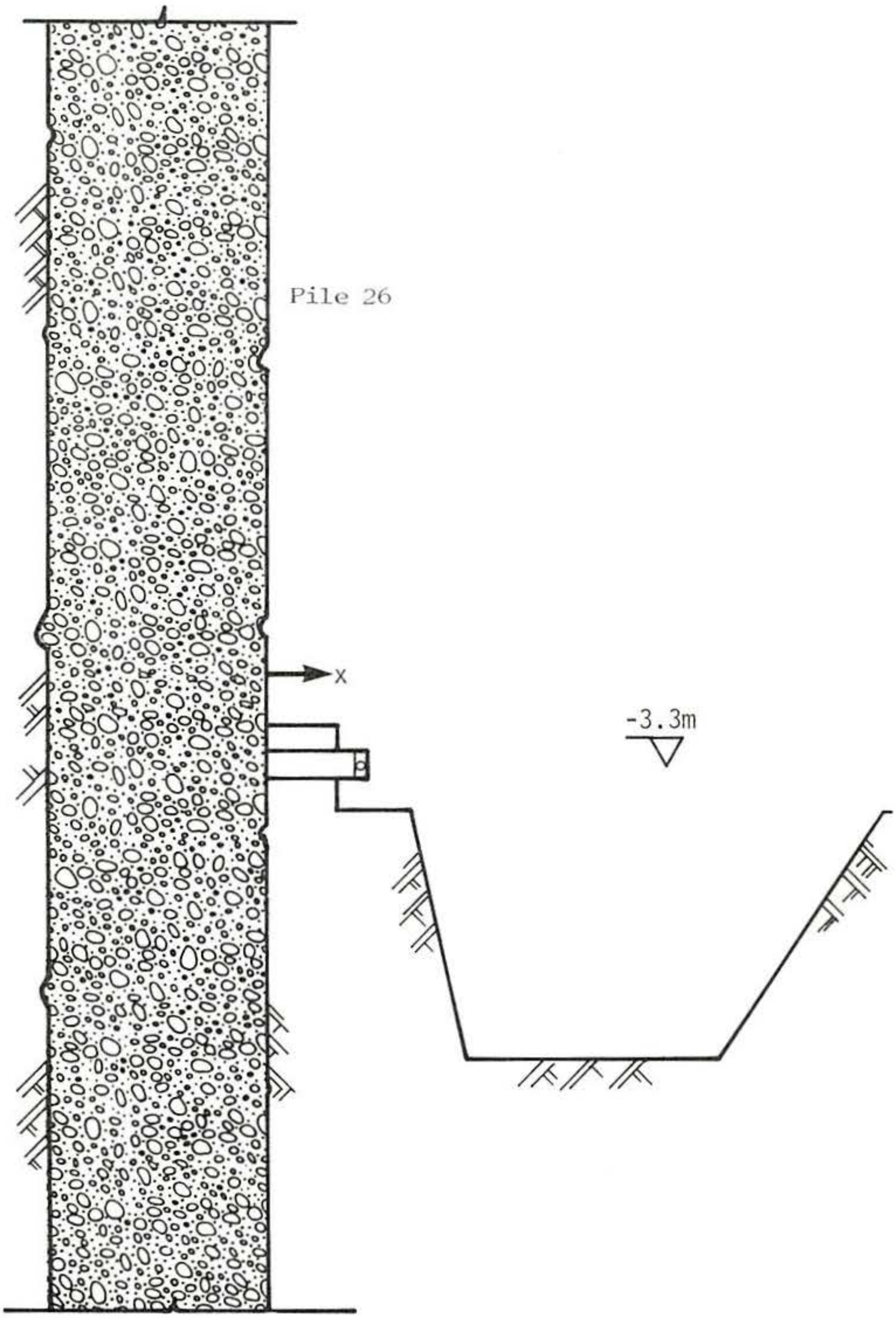


FIGURE 3.9a - LAYOUT OF SAMPLING NEAR PILE 26 (WALL)

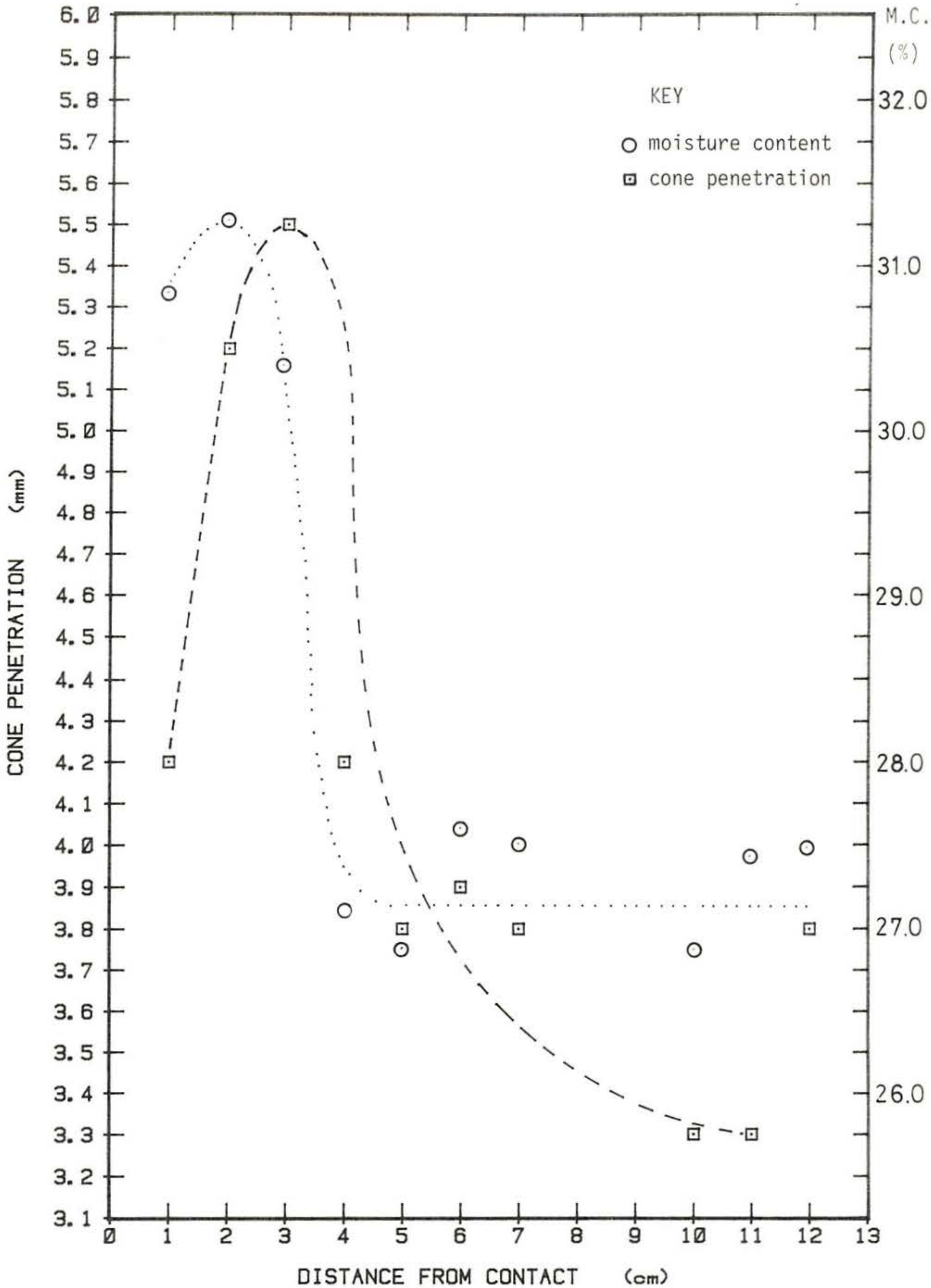


FIGURE 3.9b - MOISTURE CONTENT AND CONE PENETRATION RESULTS NEAR PILE 26 (WALL)

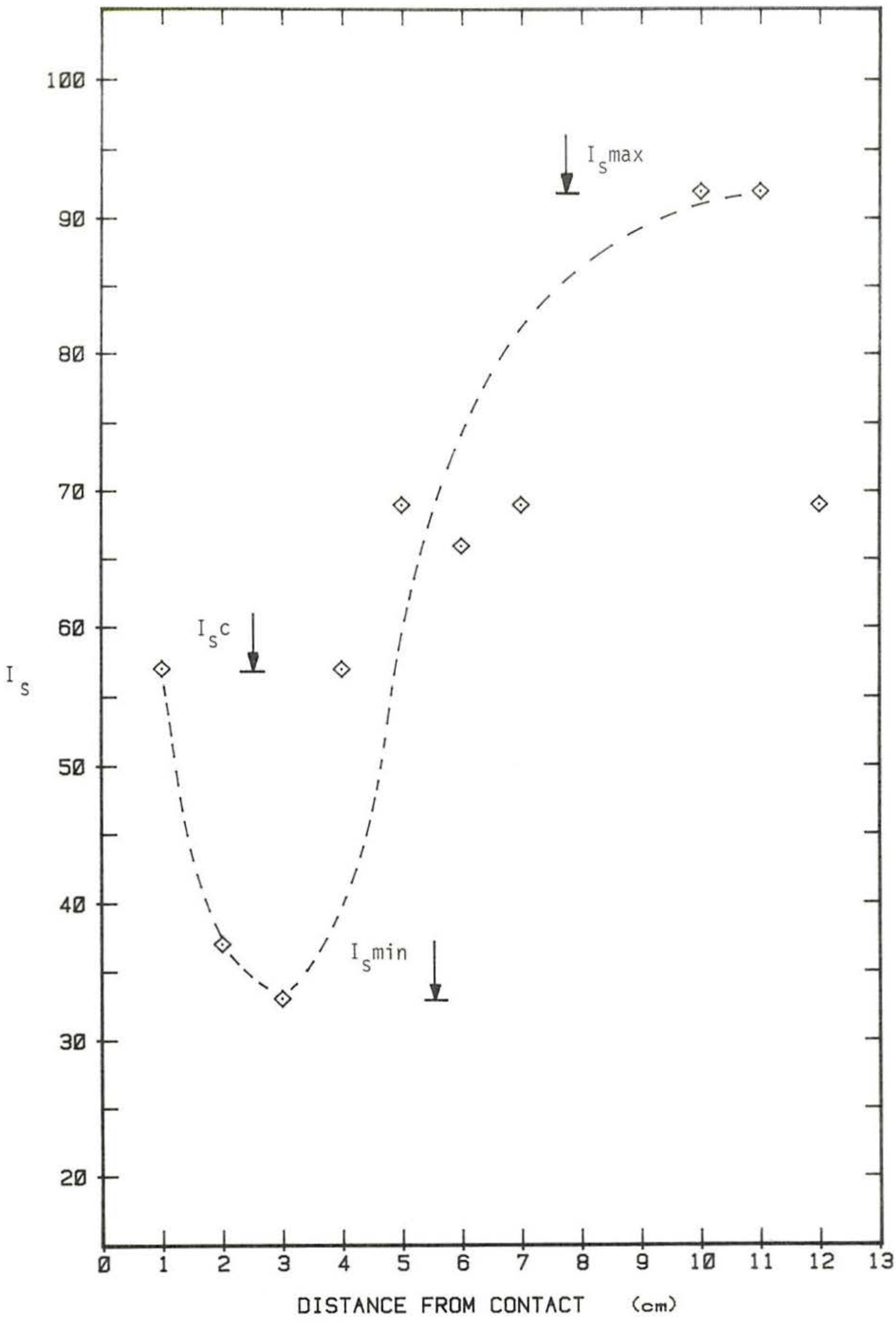


FIGURE 3.9c - "INDEX STRENGTH" VARIATION NEAR PILE 26

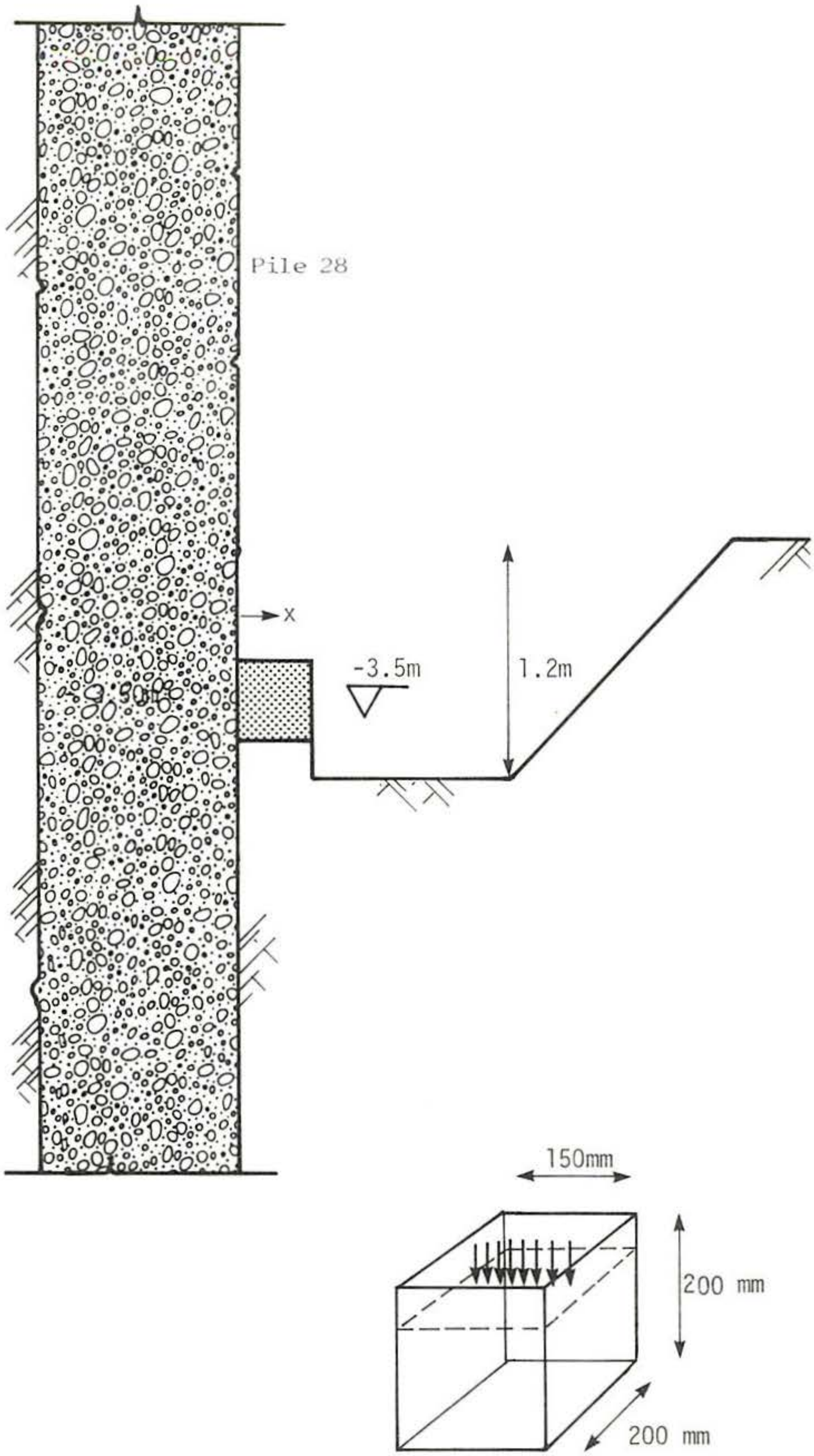


FIGURE 3.10a - POSITION OF THE BLOCK SAMPLE AND TESTING NEAR PILE 28 (WALL)

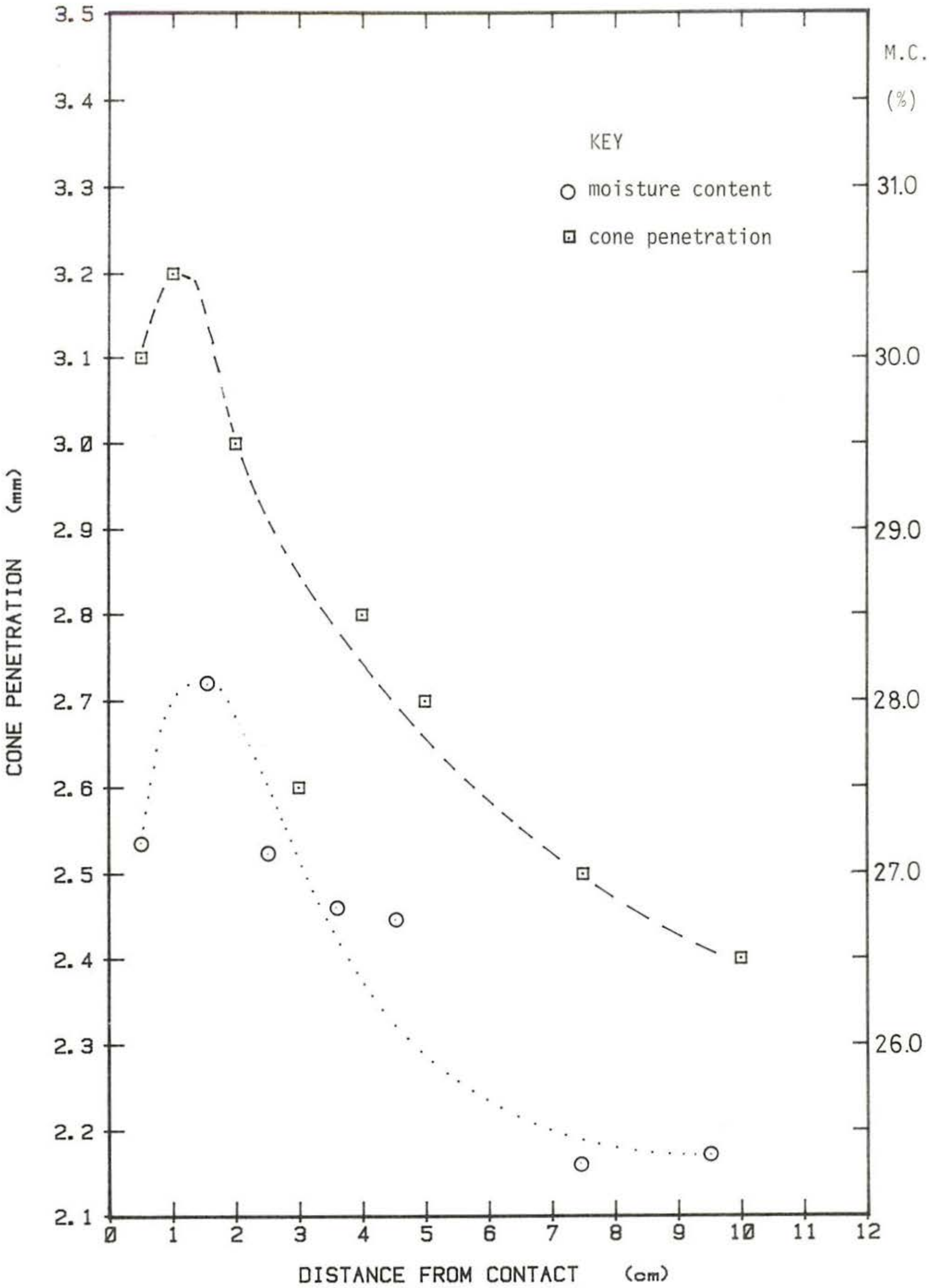


FIGURE 3.10b - MOISTURE CONTENT AND CONE PENETRATION RESULTS NEAR PILE 28 (WALL)

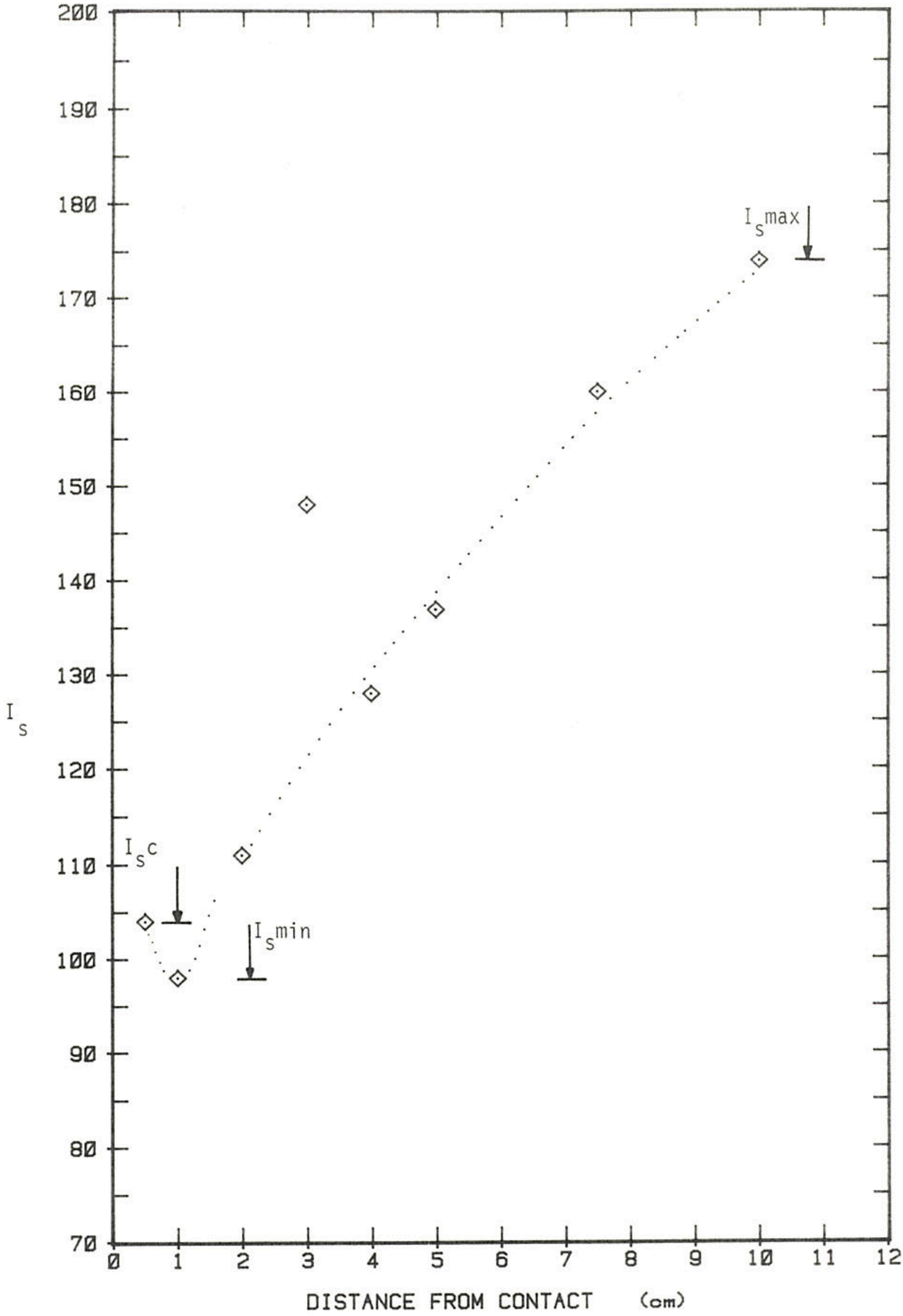


FIGURE 3.10c - "INDEX STRENGTH" VARIATION NEAR PILE 28 (WALL)

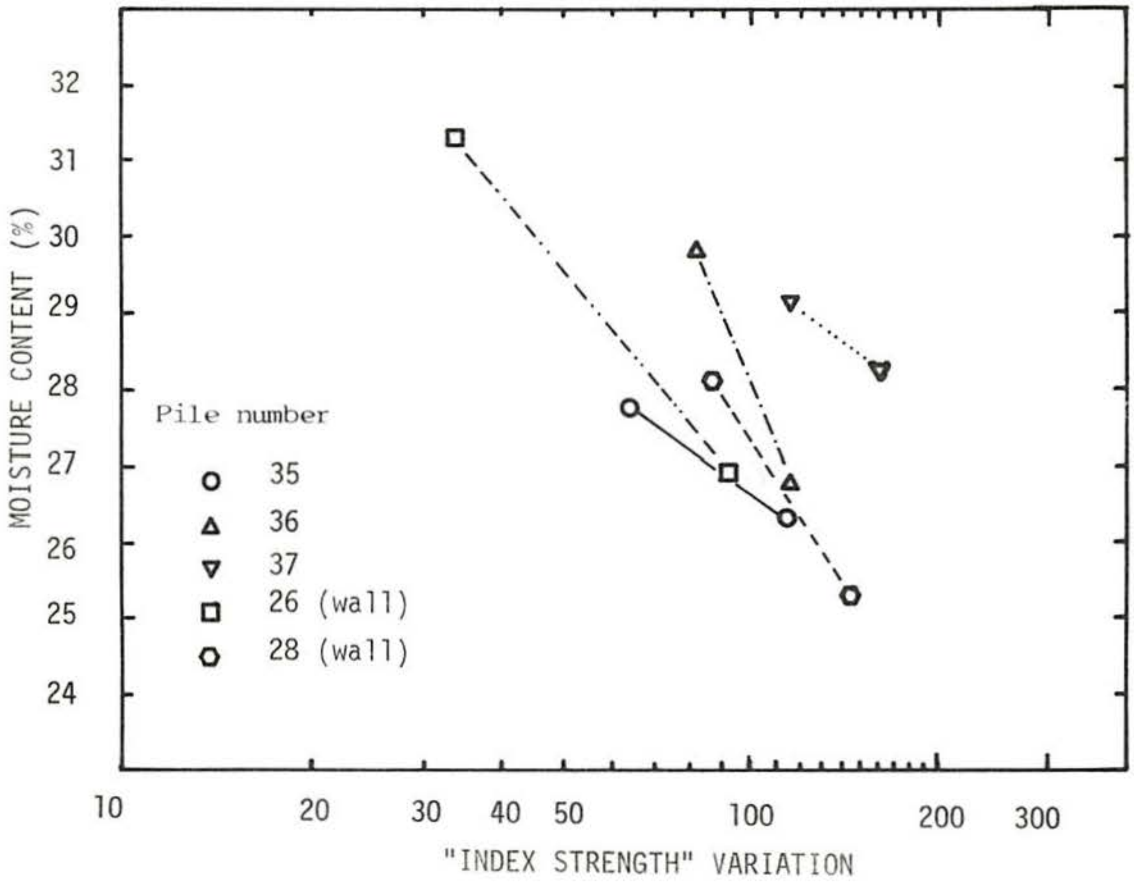


FIGURE 3.11 - SUMMARY OF "INDEX STRENGTHS" AND MOISTURE CONTENTS OBTAINED IN ALL TESTS

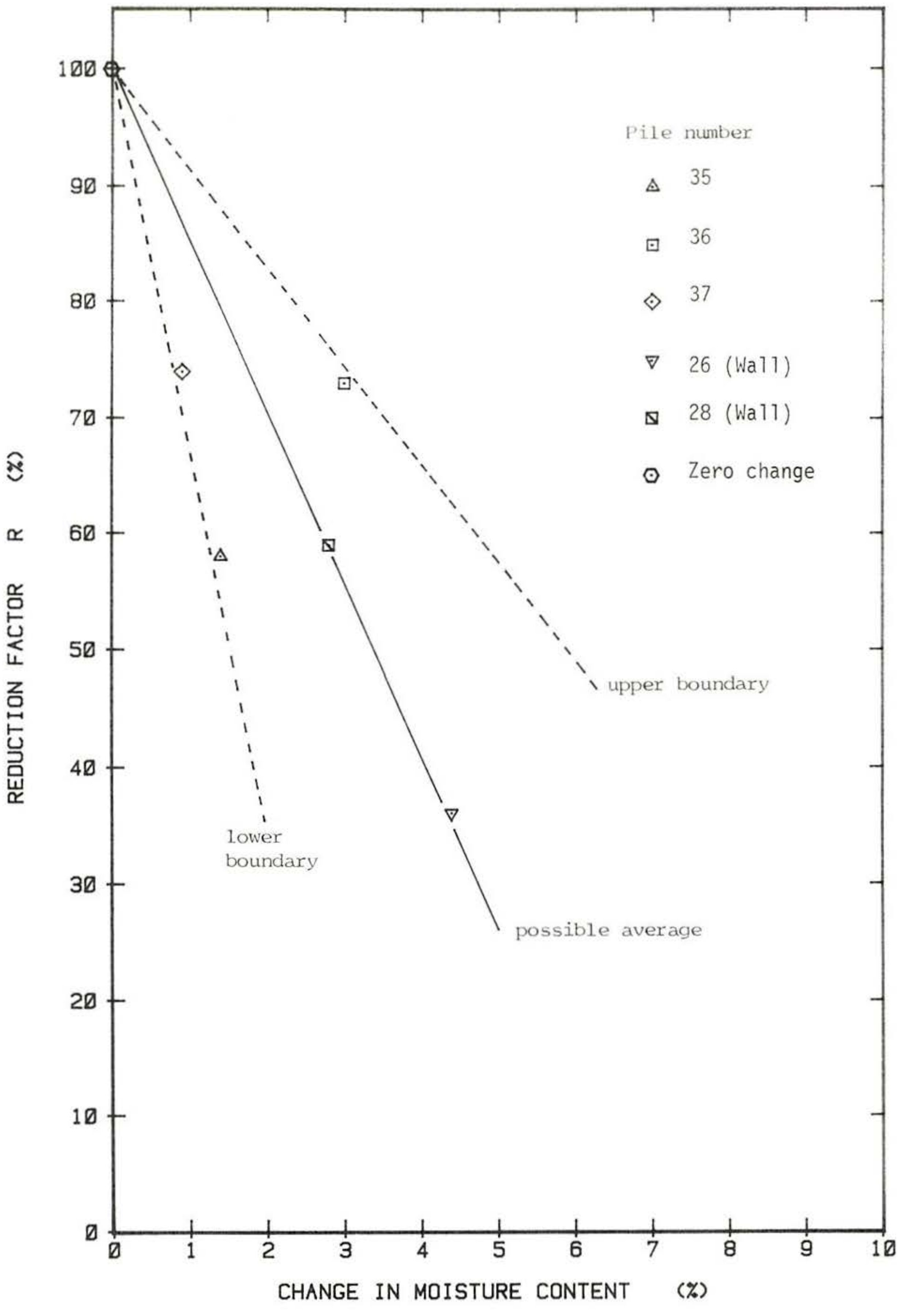


FIGURE 3.12 - COMPARISON OF REDUCTION FACTORS (R) AND CHANGE IN MOISTURE CONTENT

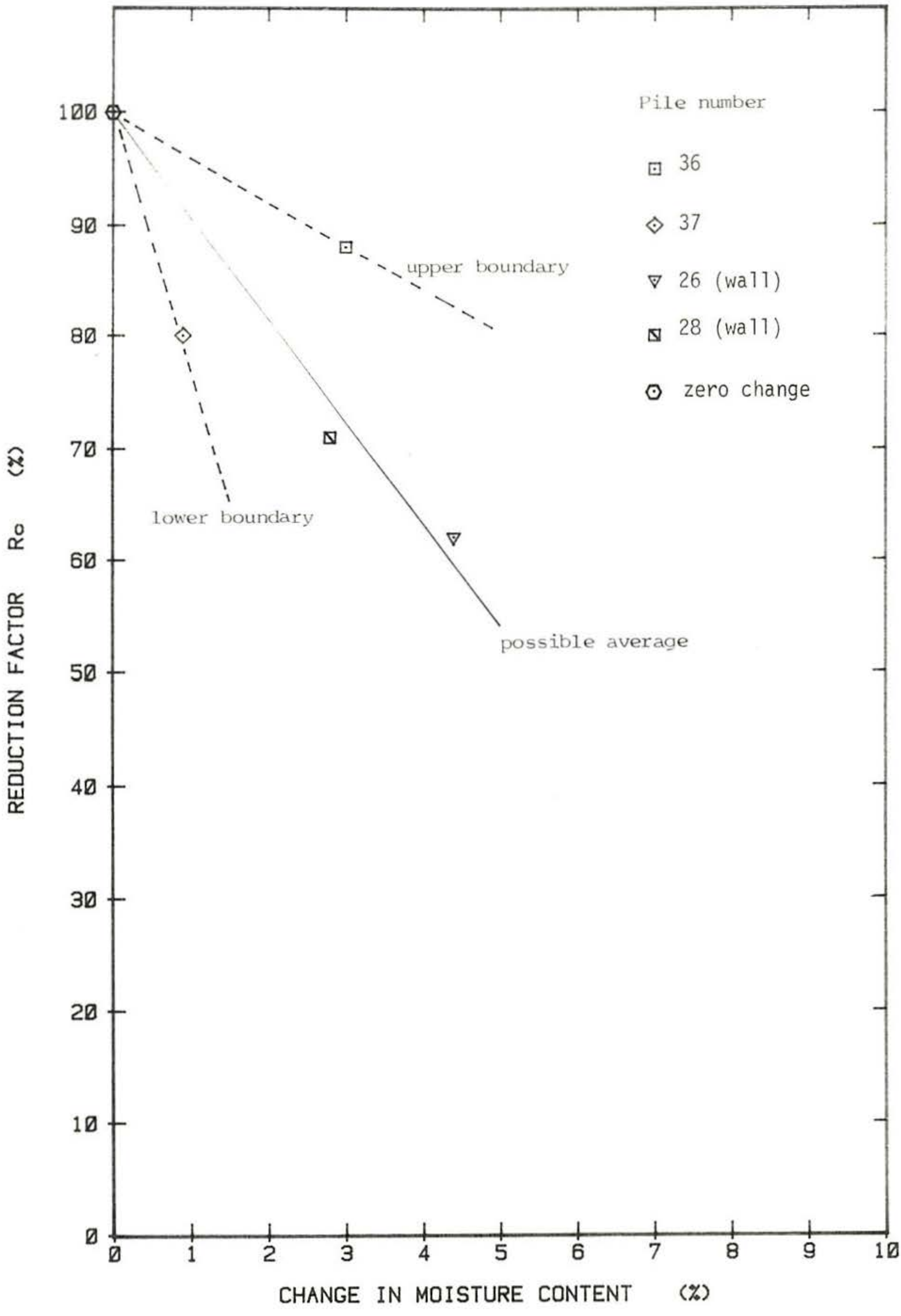


FIGURE 3.13 - COMPARISON OF REDUCTION FACTORS (R_c) AND CHANGE IN MOISTURE CONTENT

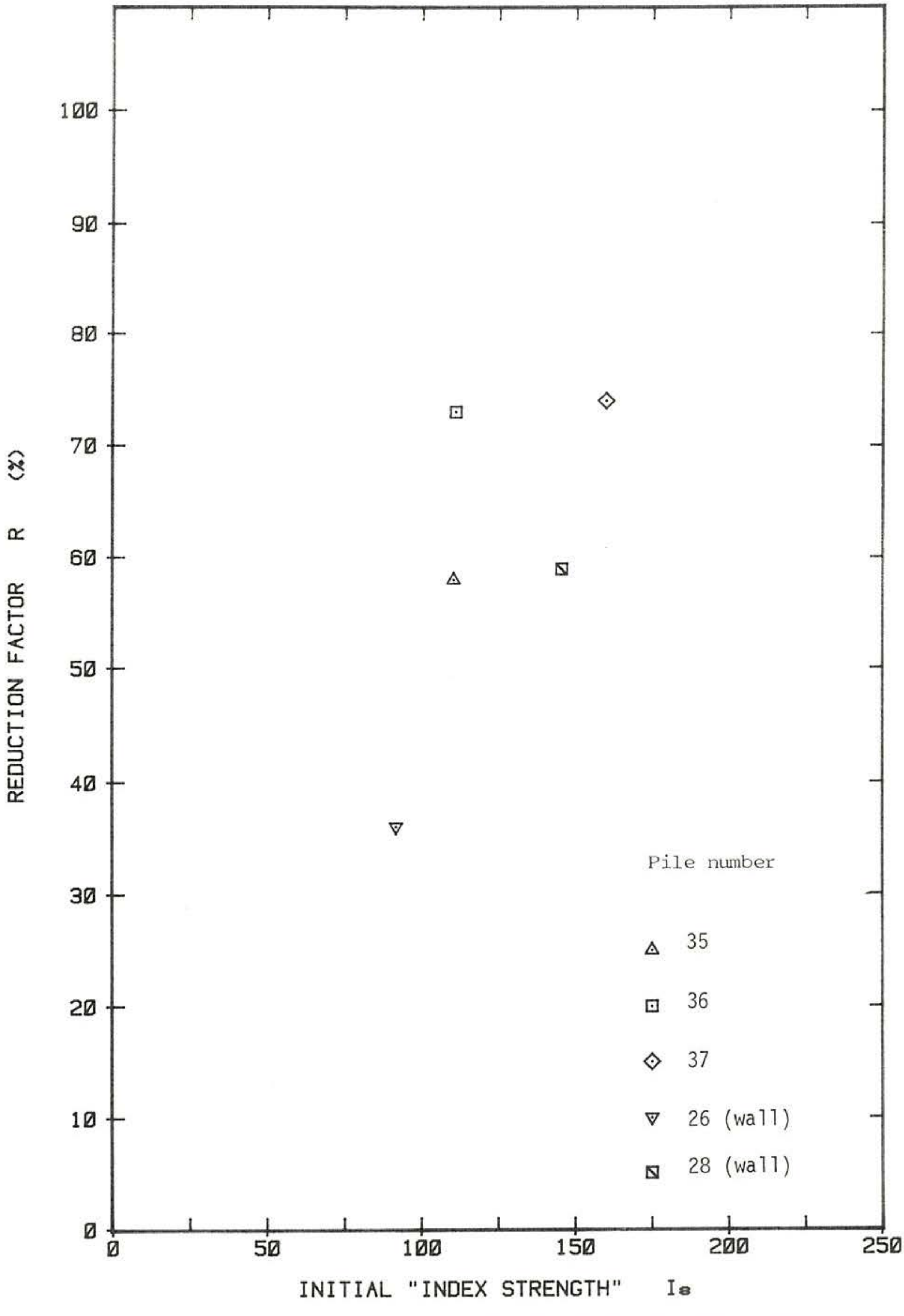


FIGURE 3.14 - COMPARISON OF INITIAL "INDEX STRENGTHS" AND REDUCTION FACTORS (R)

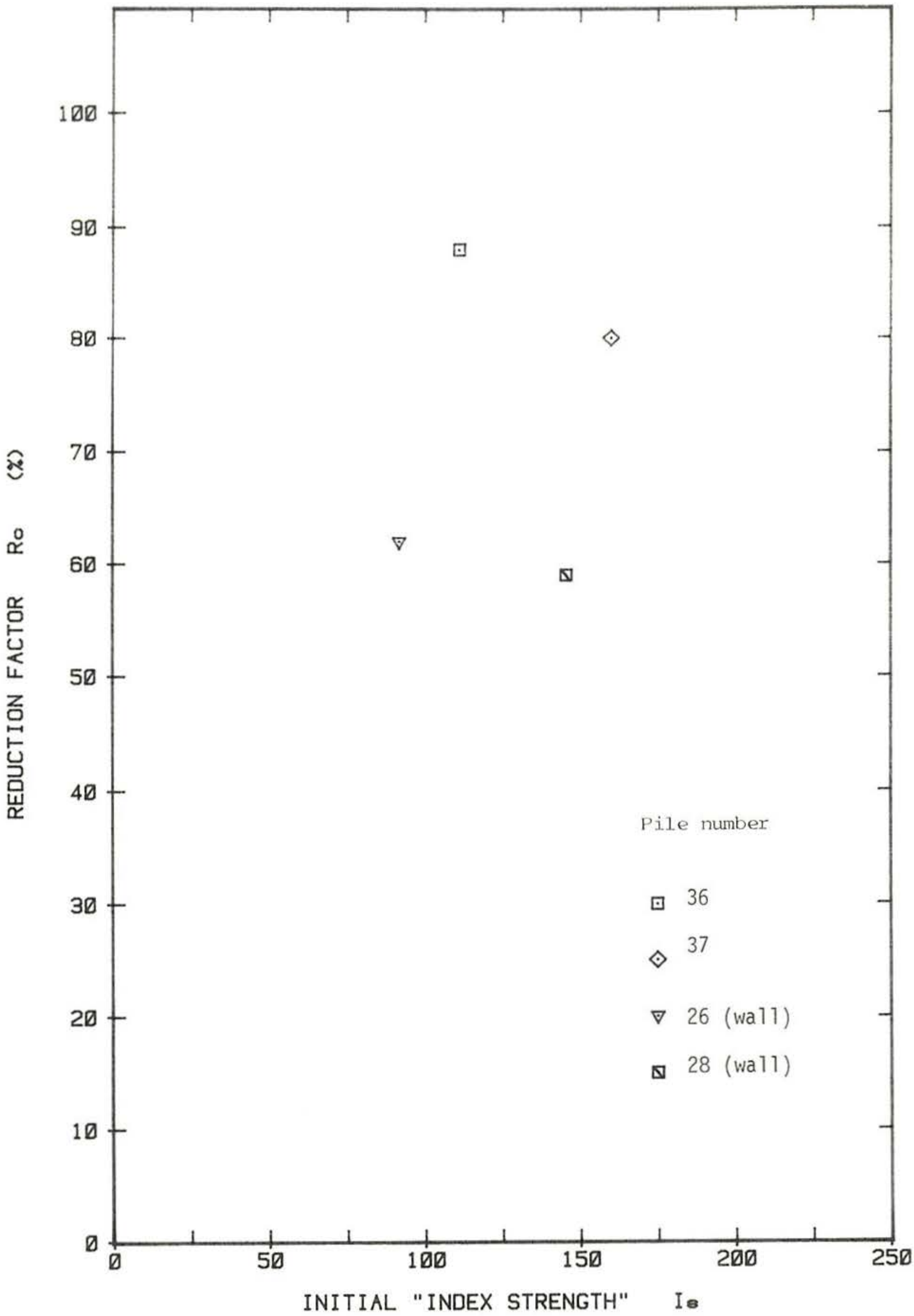


FIGURE 3.15 - COMPARISON OF INITIAL "INDEX STRENGTHS" AND REDUCTION FACTORS (R_c)

CHAPTER 4 - A RADIOCHEMICAL METHOD OF STUDYING MOISTURE MIGRATION

CHAPTER 4 - A RADIOCHEMICAL METHOD OF STUDYING MOISTURE MIGRATION

4.1. - INTRODUCTION

The major problem in attempting to assess the magnitude of local moisture content changes adjacent to piles is the variability of the natural moisture content (see, for example, Fearenside and Cooke, 1978). Although the variability can be reduced when using remoulded soils in the laboratory , other problems remain. Thus, the moisture content test most commonly used (BS 1377 : 1975 Test 1(a)) has very limited accuracy compared with the moisture content increase to be observed. In addition a relatively large specimen needs to be removed and subsequently oven dried. Consequently tests may be affected by the sampling procedure. Furthermore the same soil specimen can not be sampled at different times during the course of an experiment. It is necessary to devise a new procedure that will allow many samples to be taken in a simple reproducible manner without affecting the nearby soil. In addition the analytical method should be sensitive and capable of handling many samples in a short time interval. The present Chapter is concerned with the development of such a procedure.

Isotopes are widely used not only in chemistry but also in the life sciences. The possibility exists of following the transfer of water from one environment to another by means of a labelling procedure, using either a stable (i.e. non-radioactive) or radioactive isotope . Labelled water can be obtained in at least four forms :

a) heavy water or deuterium oxide, D_2O;

b) tritiated water, HTO;

c) $H_2{}^{17}O$;

d) $H_2{}^{18}O$.

Of these isotopes ($D(^2H)$, $T(^3H)$, ^{17}O, ^{18}O) only tritium is radioactive, being a very weak β-emitter (Emax = 18kev) with a half-life of 12.3 years. It is also of low toxicity and can be supplied at very high specific activity (curies per millimole). Furthermore the preferred method of detection, liquid scintillation counting, is extremely sensitive. Even in experiments where the concentration of tritium decreases by a factor of 10^{10} there is no difficulty in measuring its concentration. Consequently it seems that the use of this radioactive isotope of hydrogen as a tracer in studies of concrete and soil interaction is attractive. There are examples of the use of radioactive tracers in civil engineering e.g. in hydrology and in studies of the flow motion of tremied concrete in diaphragm wall construction (Ykuta et al, 1971) but there are no previous examples of the use of radioactive water in studies of water migration from fresh concrete to the surrounding soil.

4.2. - LIQUID SCINTILLATION COUNTING

The detection of radioactivity is based on one of the following methods:

a) the radiation can interact with matter, usually a gas, to give rise to ionisation and the resulting current can then be amplified to give a signal. This is the basis of the ionisation, proportional and Geiger-Muller counters;

b) the second method is based on the interaction of radiation with materials called scintillators. This results in the emission of a light pulse and with the aid of suitable electronic circuitry this again can be converted to an electronic signal. This method can be used for both α, β and γ radiations, a different scintillator being used in each case. Usually the radioactive source is placed adjacent to the solid scintillator but because of the low range associated with weak β-emitters this is not possible. This difficulty is overcome by dissolving the radioactive material directly in the scintillator. The scintillator consists of a solvent such as toluene or xylene, a primary scintillator such as 2.5 - diphenyloxazole, and if need be a blending agent such as ethanol or dioxan, so that the mixture is completely homogeneous. With such a system efficiencies as high as 90 % for ^{14}C- and 55 % for ^{3}H can be achieved. Virtually any type of ^{14}C- or ^{3}H- labelled compound can be detected using this technique. Difficulties arise when the material cannot be dissolved, in which case a gel is prepared, and when the material is highly coloured. If one has a series of samples to count then provided the composition of each vial is the same the detection efficiency will be the same. In those cases in which it is necessary to know the absolute value of the radioactivity (disintegrations per minute) as distinct from some relative value (counts per minute) one employs an internal standard. This has previously been calibrated so that the radioactivity per unit volume (or mass) is known in absolute terms e.g. microcurie per 0.1 ml.

The advantages of the liquid scintillation method of counting are many. Thus sample preparation is easy, the sensitivity is good, and many samples can be counted in a short time interval e.g. as many as 200 per hour.

Some of the available basic books on the subject are : Birks , 1964 ; Horrocks, 1974 ; Neame and Homewood, 1974; Dyer, 1974 and 1980 and Peng 1977.

4.3. - FREEZE-DRYING

The freeze-drying technique originated because of a need to preserve biological specimens in the laboratory. Today the laboratory applications of the technique are used by a large spectrum of research, such as microbiology, virology, histology and chemistry. There are many industrial applications of the technique, foodstuffs being the most important.

The principle of the method is simple and can be illustrated by the phase diagram for water (Figure 4.1). Water can exist in three different forms ; solid, liquid and vapour and the pressure-temperature plot is known as a phase diagram. AB denotes the situation in which vapour and liquid water are in equilibrium, AC that in which ice and liquid are in equilibrium and AD that in which vapour and ice are in equilibrium. A is the water triple point which occurs at Tc =0.0098 C and Pc = 4.58 mm Hg.

When ice at a pressure P_2(which is less than Pc) is heated from a temperature T_2(less than Tc) to T_1 it is converted directly into vapour. In practice the vapour can be removed by condensing it as ice on a cold surface. The apparatus that is used to achieve this change of state is illustrated in Figure 4.2 . Because the amount of water associated with a given mass of soil (in the actual experiments) is small the best procedure is in the first case to add a known amount e.g. one ml of ordinary water to a known weight of soil in the round-bottom flask A. By first of all shaking the flask so that the water becomes fully equilibrated and then rotating in a Dewar flask containing liquid nitrogen a thin film of ice is formed over a large area. At this stage the apparatus was assembled and connected to a rotary vacuum pump and then evacuated with flask A immersed in liquid nitrogen. When the apparatus was fully evacuated the stopcock B was closed and the condenser C immersed in liquid nitrogen, the flask being suspended in air at room temperature. When the drying was completed the vacuum was released cautiously and the melted ice collected in C assayed for tritium.

4.4. - EXPERIMENTAL PROCEDURE

The testing procedure that was used to carry out some of the preliminary experiments and to assess the feasibility of the approach is illustrated in Figure 4.3 . Once satisfactory results had been achieved a more refined laboratory test arrangement was constructed.

4.4.1. - TEST PREPARATION

Figure 4.4 shows the laboratory test arrangement used to allow clay to swell in contact with fresh concrete. A cylindrical specimen of "undisturbed" London Clay (height 200 mm) obtained with a 100 mm thick walled opendrive sampler was placed in the pedestal of a triaxial cell, sealed with a double membrane and consolidated under a desired effective pressure. When the specimen was fully consolidated, the drainage was closed, cell pressure released, the top cap removed and concrete poured into a former on top of the soil specimen.

The concrete in these experiments had a water cement ratio of 0.6 and the mix was 1:2.40:1.90 by weight. Tritiated water (10 μl, of 5 Curie per ml specific activity) was added to 0.5 ml of water and this was equilibrated with the 30 ml of water used in the preparation of the concrete.

The assembly was again sealed in the double membrane and subjected to the previously applied cell pressure.

4.4.2. - SAMPLING FOR RADIOACTIVITY

In order to take a large number of very small samples from the same test specimen, a miniature sampler was designed and constructed (Figure 4.5). The sampler was attached to the end of a plastic syringe for ease of handling, and in an attempt to produce a vacuum behind the sample during withdrawal of the sampler. Initial experiments were made with a sampler with an inside diameter of about 1.5 mm and no inside clearance. As it proved difficult to obtain more than a few millimetres of the very stiff London Clay with this sampler, modifications were required. Inside clearance was provided by rolling the cutting edge inwards to give the sampler the following characteristics :

Area ratio	248 %
Inside clearance	39 %
Inside diameter	1.5 mm
Length of drive	24 mm

There was no difficulty in obtaining samples of the clay with the modified sampler. Samples typically had a volume of about 30 mm^3, and were taken from the specimen by rolling back the outer membrane and driving the sampler through the inner membrane for a distance of about 20 mm, in the required positions (Figure 4.6a). After sampling, the hole in the inner membrane was repaired by applying a small patch of latex rubber to the membrane with adhesive (Figure 4.6b). The second membrane was then replaced in its original position and the cell pressure re-established until a further batch of samples was required. Samples were taken at varying distances from the soil and concrete interface, at different positions around the perimeter of the specimen (Table 4.1).

4.4.3. - RADIOACTIVITY DETERMINATION

The radioactivity of each sample was determined in the following manner. First of all the weight of the sample was determined to the nearest 0.0002 g. The sample weight was usually in the range 0.06 to 0.1 g. Secondly one ml of water was added to each sample, so as to

Distance from concrete (mm)	Days after preparation 1	3	7
10	B	C	A
20	C	D	B
30	D		C
40		A	
50			D
60	A	B	
70			
80	B	C	A
90			
100	C	D	
110			
120			B
130			
140	B	A	
150			
160			
170		B	C
180			
190	D		

TABLE 4.1. - Position of sampling

induce the following isotope exchange reaction :

$$(\text{clay})_{H_2O^*} + H_2O \rightleftharpoons (\text{clay})_{H_2O} + H_2O^*$$

Next the labelled water was removed from the clay solution by freeze-drying using the apparatus and technique previously described. Finally, 2 μl of the radioactive (tritiated) water obtained from condenser C was added to 5 ml of a commercially available liquid scintillator (NE 250) and the radioactivity determined using a Beckman LS 100 liquid scintillation counter. Enough counts were obtained to give an accuracy better than $\pm$ 0.1 % . In order to convert the counts per minute to disintegrations per minute (i.e. absolute radioactivity units) the efficiency of counting was determined by adding a known amount of an internal standard (^{3}H-hexadecane) to a mixture of 2 μl of water and 5 ml of NE 250 liquid scintillator.

4.4.4. - CALCULATIONS

Because the process involves controlled labelling, sampling and testing, it is relatively simple to convert the disintegrations per minute (d.p.m.) obtained from the scintillation counter to an increase in tritiated water content.

Initially, when tritiated water is added to the concrete mix, three methods present themselves as possibilities for determining the disintegrations per unit time per unit volume of this water. These are :

(a) the use of the specific activity of the tritiated water and the various dilution factors;

(b) sampling of the completed mix , and

(c) sampling of the diluted but labelled water, before addition to the cement and aggregate.

Method (a) is only possible if the radioactivity of the tritiated water is accurately known. As supplied, normally, the radioactivity of the tritiated water will only be correct to ± 10 to 20 %. Method (b) requires a large enough sample to be taken to be representative of all the particle sizes present in the concrete. The labelled water is removed by the addition of unlabelled water, as before. Because its radioactivity is higher than can be handled conveniently, it must be diluted. As soon as the labelled water is added to the concrete, it is possible that reactions involving the labelled water would commence. Thus the equilibration process might not remove all the tritium from the concrete sample and a low radioactivity would result, giving an overestimate of labelled water content in the clay samples. Thus the best method of determining the radioactivity of the labelled water would appear to be method (c). After sampling and scintillation counting, the radioactivity of the tritiated water can be expressed in disintegrations per minute per gramme of water. A figure of about 10^{10} to 10^{11} d.p.m./g H_2O^* would be expected in the present case.

In order to obtain the labelled water content of the clay it is necessary to have an approximate knowledge of its overall moisture content before the test. Consider a typical set of data for a sample of clay extracted and tested as described above :

Radioactivity of the tritiated water added to concrete	0.1×10^{12} d.p.m./g H_2O^*
Weight of clay sample	0.05 g
Radioactivity of extracted water	0.1×10^{6} d.p.m.
Moisture content (before test)	30 %

During the preparation of the clay sample to obtain material for scintillation counting the water was removed from the clay by dilution and freeze-drying. The volume of labelled water present in the clay had previously been equilibrated by the addition of one ml of unlabelled water. After sublimation a 2 μl sample was withdrawn. Assuming the labelled water content to be small, this represents a dilution factor of 0.5×10^3. Thus the radioactivity of the full sample would be 50×10^6 d.p.m., and the weight of labelled water transferred from the concrete to the sample would be 0.50 mg. Since a 0.05 g sample of clay with a moisture content of 30 % will have a dry weight of 38.46 mg, this represents an increase in moisture content of 1.30 %. An error of 5 % in the overall initial water content would lead to an error of about 0.05 % in this calculated moisture content increase.

4.5. - SENSITIVITY OF THE TECHNIQUE TO DIFFERENT PROCEDURES

In order to obtain a better understanding of the effects of different procedures on the obtained results, a mix of sand and cement was tested. The mortar used in these experiments was made with 150 g of fine sand, 50 g of ordinary Portland cement, 30 g of water contaminated with 10 μl of tritium. Whenever possible, two samples were taken at each stage of testing to check the repeatability of results.

Tabe 4.2 summarizes the results obtained.

4.5.1. - RADIOACTIVITY OF THE LABELLED WATER

To assess the repeatability of the count rate of the labelled water, series W was tested. The specimens were taken from the water before mixing with the cement. Each specimen was obtained by sampling one ml of the water, one ml of ordinary water was added, and after freeze-drying a specimen of 0.1 ml was withdrawn and added to 5 ml of liquid scintillator (NE 250). The radioactivity was then determined. The results obtained are plotted in Figure 4.7 , presenting a variation of 142 to 153 x 10^3 c.p.m. .

4.5.2. - EFFECT OF INITIAL SAMPLE SIZE ON THE COUNTS

In order to study the effect of the size of the initial sample withdrawn from the labelled water, the series $W_{\underline{i}C}$ was tested. The amount of labelled water taken from the main sample is identified in ml by the digit in the $\underline{i}$ position. The results obtained are presented in Figure 4.7., with the series W for comparison.

4.5.3. - AGEING OF THE MORTAR SAMPLE

A standard procedure was used to study the effect of ageing of the mortar mixture in the results obtained.

A variable amount of material was withdrawn from the mortar at different ages (referred to in Table 4.2 as weight of original sample), and one ml of water was added to each sample. After a period of 5 minutes of mixing, the labelled water was removed by freeze-drying. From that sample of labelled water, 0.1 ml was withdrawn and added to 5 ml of liquid scintillator (NE 250) and the radioactivity determined.

The series $CS_{0/\underline{i}}$ was obtained from the fresh mix, the zero referring to zero hours after mixing the water with the cement. Sample $CS_{0/1}$ was taken immediately after mixing, $CS_{0/2}$ and $CS_{0/3}$ at 20 and 30 minutes, respectively.

To study the possible effect of cement hydration on the count rate, the mix was sampled at 6, 24, 48 hours, 5, 7 and 14 days. At 48 hours 3 samples were taken, mixed with one ml of water but freeze-drying was delayed for 3 and 5 days ($CS_{48/D3}$ and $CS_{48/D5}$), resulting in 5 and 7 days after mixing for the count determination.

4.5.4. - RESULTS AND DISCUSSION

The results obtained are presented in Table 4.2 and in Figures 4.7 to 4.10.

The effect of initial sample size on the counts presented in Figure 4.7 shows that there is a lower efficiency for samples greater than 0.1 ml of water. No difference was found from the results obtained in series W and $W_{\underline{i}C}$ when the same amount of material was tested.

The effect of ageing of the mortar sample in the results is presented in a graphical form in Figure 4.8. As the control of sample size is difficult and the efficiency varies with the amount of water, a basis of comparison must be the weight of the original sample.

Considering the original proportions of the mix (150 g of sand + 50 g of cement + 30 ml of water), the original amount of water in each sample was calculated and is plotted in Figure 4.9. The values obtained in the W series is also presented for comparison.

Figure 4.10 shows the variation obtained in counts/weight of sample with time for the various essayed specimens up to 14 days.

From the values plotted in Figures 4.8 to 4.10 it can be concluded that no major difference in counts is measured when a comparison based in the original labelled water content is made (due to ageing) up to 30 minutes after mixing. The variation in counts obtained up to 24 hours may possibly be due to non homogeneity of water distribution in the mortar. A small reduction was observed up to 48 hours. After 48 hours there is a big reduction in counts/weight of the specimen with time. The reasons for the reduction are probably the hydration of the cement, creating a more stable structure (strong links with the water molecules) and the possibility of bad sample mixing for older samples, due to the larger groups of particles connected together.

Sample	Time after mixing	Approx.weight of original sample (g)	Labelled water (ml)	Counts per minute/10^3	Counts/ weight (/g)
W_A	0	0.1		142	
W_B	0	0.1		153	
$W_{0.5C}$	0	0.05		66	
W_{1C}	0	0.1		151	
W_{2C}	0	0.2		185	
W_{3C}	0	0.3		243	
W_{4C}	0	0.4		275	
$CS_{0/1}$	0	2.8511	0.3583	288	101
$CS_{0/2}$	20 min	3.9577	0.4974	383	97
$CS_{0/3}$	30 min	2.9377	0.3692	293	100
$CS_{6/1}$	6 h	1.8519	0.2327	190	103
$CS_{6/2}$	6 h	2.6113	0.3282	233	89
$CS_{24/1}$	24 h	1.1392	0.1432	130	114
$CS_{24/2}$	24 h	2.2620	0.2843	209	92
$CS_{48/1}$	48 h	0.6837	0.0859	65	95
$CS_{48/D3}$	5 days	0.4792	0.0602	36	75
$CS_{48/D5}$	7 days	0.7898	0.0992	53	67
$CS_{120/1}$	5 days	0.7207	0.0906	56	78
$CS_{120/2}$	5 days	0.8362	0.1051	50	60
$CS_{168/1}$	7 days	0.7171	0.0901	48	67
$CS_{168/2}$	7 days	0.4159	0.0523	29	70
$CS_{336/1}$	14 days	0.8274	0.1040	51	62
$CS_{336/2}$	14 days	0.9172	0.1153	55	60

TABLE 4.2. - Results of the cement x sand testing programme

4.6. - RESULTS

Using the technique described in Section 4.2. two samples of undisturbed London Clay, with the characteristics referred to in Tables 4.2 and 4.3, were tested.

Figure 4.11 to 4.14 presents the results obtained. The increase of moisture content of sample RC_1 after one day of contact is presented in Figure 4.11. The pattern of the increase in moisture content is similar to that obtained using conventional techniques (Chuang and Reese, 1969; O'Neill and Reese, 1970; Yong, 1979). As expected, the moisture content increase falls off rapidly as the sampling point is moved away from the contact between soil and concrete. At a distance of 50 mm the measured radioactivity is only 15 % of its value at a distance of 10 mm.

Results obtained at different ages for sample RC_1 are presented in Figure 4.12. The evolution of moisture content at some points close to the contact is shown in Figure 4.13.

Figure 4.14 shows the results obtained from the sample RC_2. A different trend can be clearly identified from that of the previous sample. The unexpected results canbe explained by the presence of a strong pattern of fissures, probably interfering with the water migration.

The Internal Standard Method was used to check the efficiency of counting, and it was found to be 32 %(see Dyer, 1974, for details of the Internal Standard Method).

From Figure 4.13, obtained from the average lines of Figure 4.12, it is clear the pattern of moisture migration near the contact in the first week. Points close to the contact continue to increase the moisture content after the first day, therefore the measured increase of moisture at 6 cm is very small after the first day.

LOCATION: Wimbledon

Depth (m)	Soil description	Undrained shear strength Depth of sample m	Undrained shear strength kN/m^2	Bulk density kg/m^3
0.00	Made ground			
1.50	Stiff brown fissured silty CLAY with some grey staining in the fissures, occasional lenses of brown sand in the upper levels and selenite crystals			
7.00	Stiff to very stiff dark grey fissured silty CLAY			
		13.50 - 14.00	220	2035
		16.50 - 17.00	310	2050
		19.50 - 20.00	360	2025
20.00	Bottom of borehole			

Table 4.3. - Soil profile and undrained shear strength from the site investigation report.

	RC1	RC2
Sample discription	Very stiff dark grey fissured silty clay	Very stiff dark grey very fissured silty clay
Place of origin	Wimbledon	Wimbledon
Depth	15 m	18 m
Initial moisture content: top	25 %	23 %
bottom	25 %	21 %
Sample diameter	100 mm	100 mm
Sample height	190 mm	180 mm
Remarks		Very fissured (difficult to handle)
Confining pressure	250 kN/m^2	300 kN/m^2

Table 4.4 - Characteristics of samples RC1 and RC2

4.7. - DISCUSSION

The method described involves the use of a radioactive labelling procedure to study the migration of water from fresh concrete to the surrounding soil. The major advantages of the method are:

(A) Very small samples can be used for the measurements with the possibility of close spacing on sampling, great possible frequency and reduced experimental disruption;

(B) Due to the sensitivity of the analytical method used to measure the isotope concentration, in carefully conducted experiments the accuracy of the technique is at least one order of magnitude better than can be obtained by conventional determination of moisture content by oven drying , and

(C) It overcomes the problems existing in the usual approach to studying moisture migration , e.g. variability of moisture content in natural soils and lack of accuracy of the method used to evaluate moisture content compared with the expected variation in moisture.

The experimental technique used in this method is relatively simple to perform, requiring only the usual counting equipment normally found in a radiochemistry laboratory.

Although this method entails the use of radioactive material, there is little or no hazard as the level of radioactivity is low (micro-curies) and the average energy of the tritium β radiation is very weak; consequently the scale of the experiment can easily be increased. Because sampling is carried out remote from the radiochemistry laboratory and the transporting of samples for counting poses no problem, there is the possibility of using the method in full scale experiments in the field (the best way to get quantitative information of the real interaction between bored piles and surrounding soil).

In order to compare the obtained results it is absolutely essential that a standard procedure is followed during all tests; minor changes in the technique can affect the results obtained. The amount of material used and the dilutions must be kept constant. To avoid the possibility of cross-contamination it is important to clean any equipment that is used during sample handling properly, such as pipettes, burettes, and freeze-drying equipment.

Considering the results of the tests made on cement-mortar samples it can be concluded that : (i) a delay in the determination of the radioactivity of the mortar up to 6 hours does not affect the results, (ii) count efficiency is dependent on sample size, and (iii) the ageing of the mortar sample affects the count rate.

The observed increase in moisture content in "undisturbed" London Clay when subjected to contamination under laboratory controlled conditions shows a well defined trend in moisture migration. As expected, the moisture content variation falls off rapidly as the sampling point is moved away from the tritiated concrete.

In very fissured samples the smooth trend is not observed, possibly due to non uniform conditions on the sample, with the fissure pattern providing preferential flow channels.

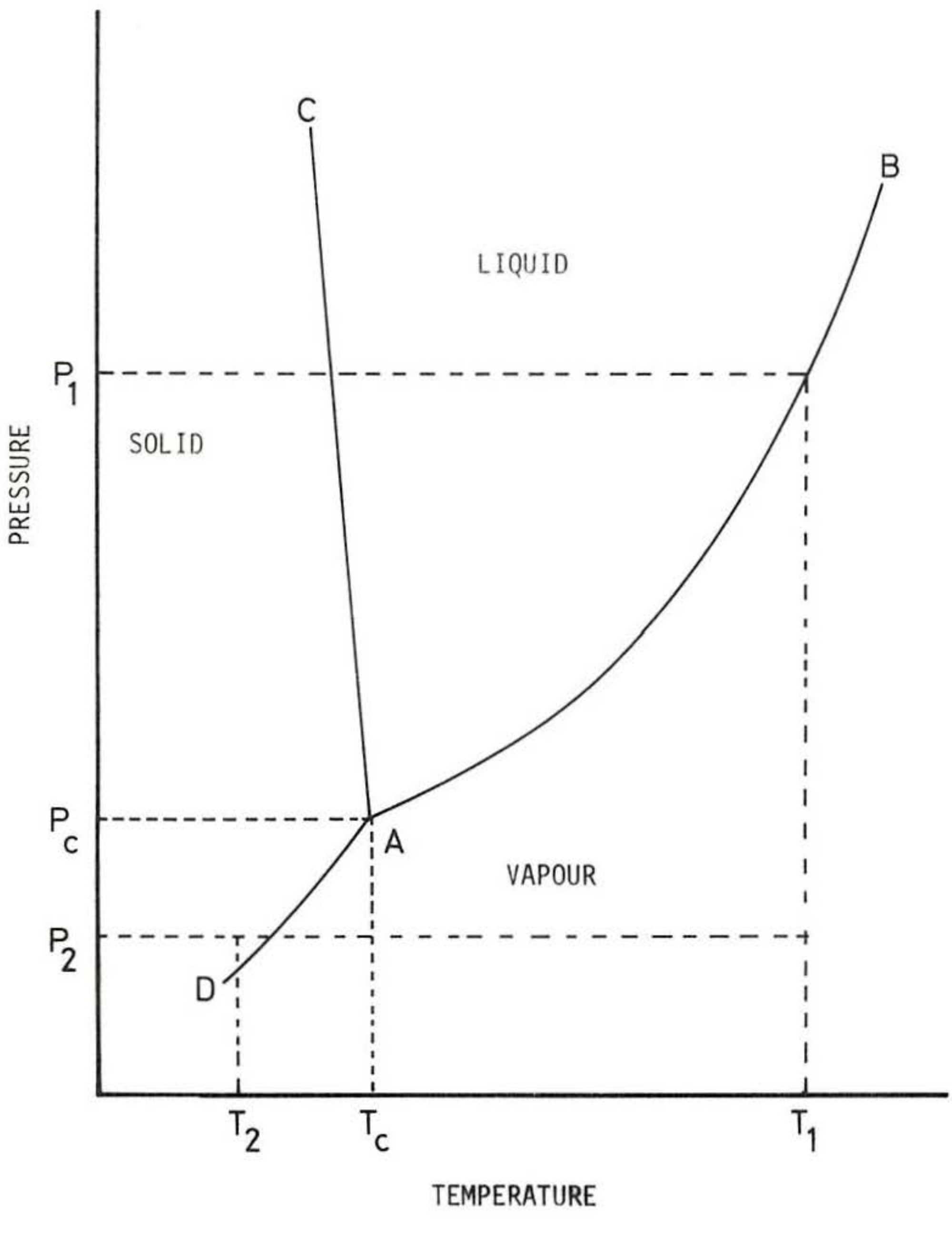


FIGURE 4.1 - PHASE DIAGRAM FOR THE WATER SYSTEM

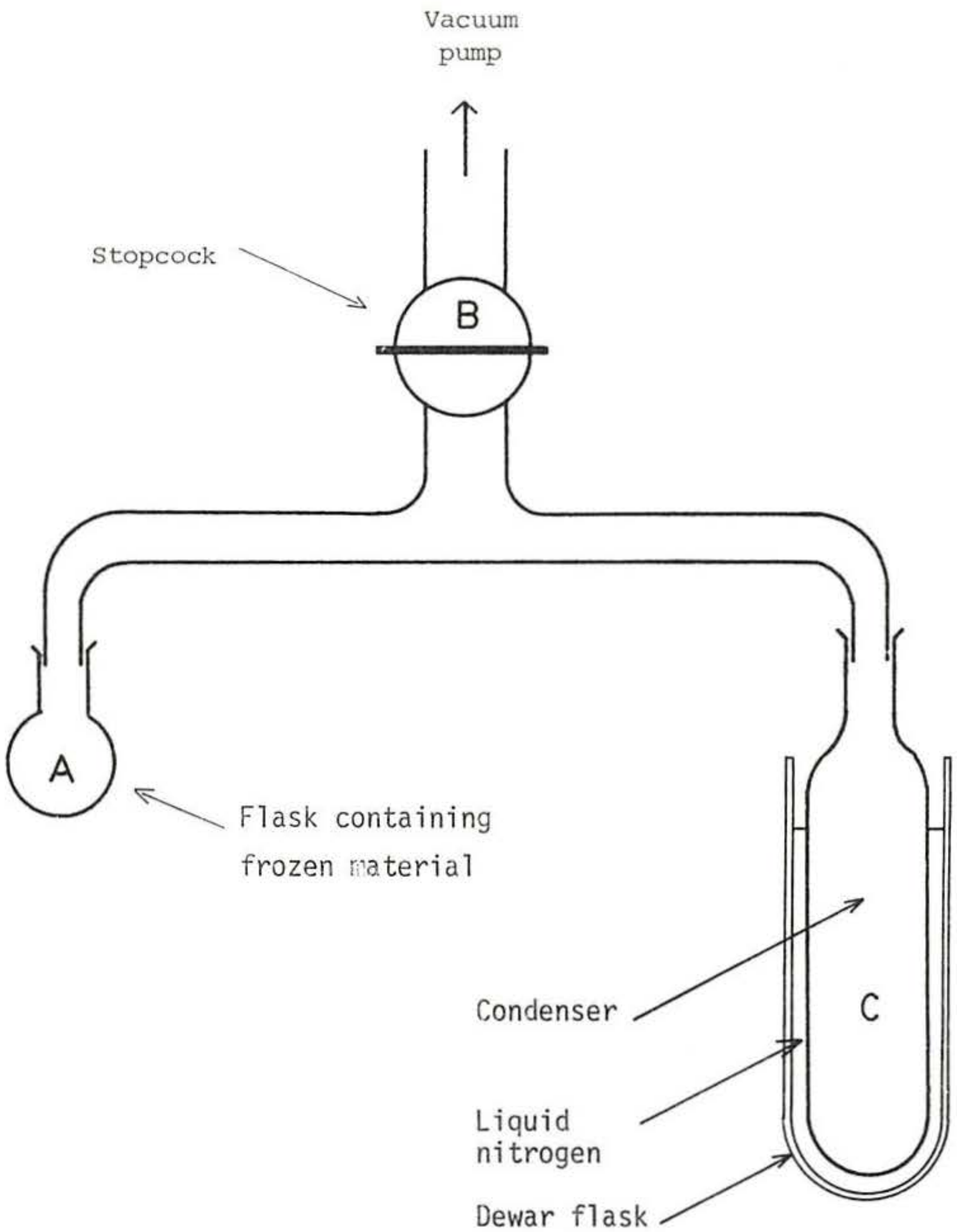


FIGURE 4.2 - FREEZE-DRYING APPARATUS

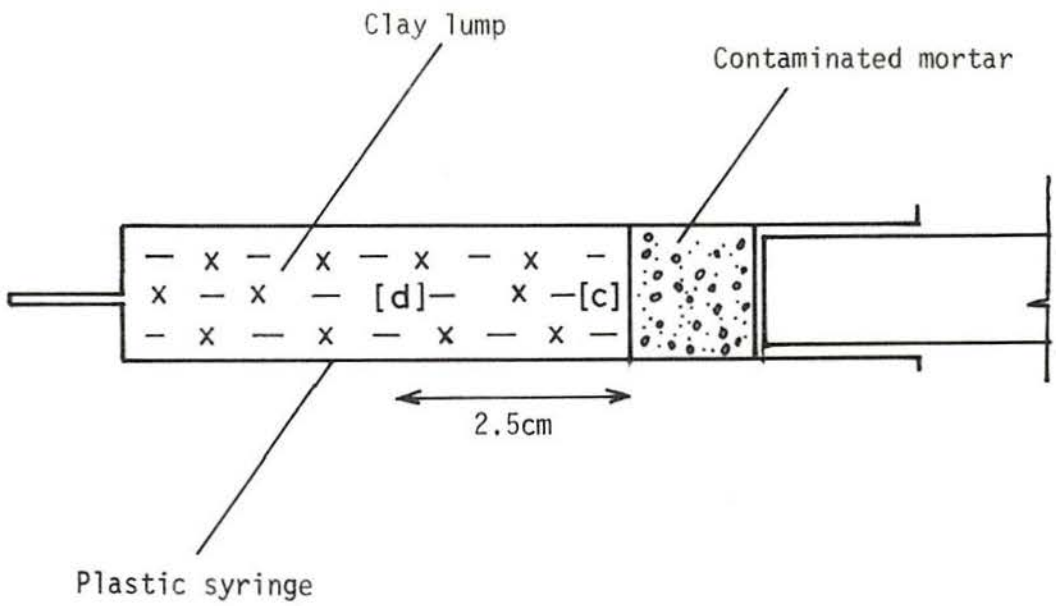


FIGURE 4.3 - PRELIMINARY TEST ARRANGEMENT USED TO CHECK THE PROPOSED TECHNIQUE (c and d are the positions of sampling for radioactivity counting)

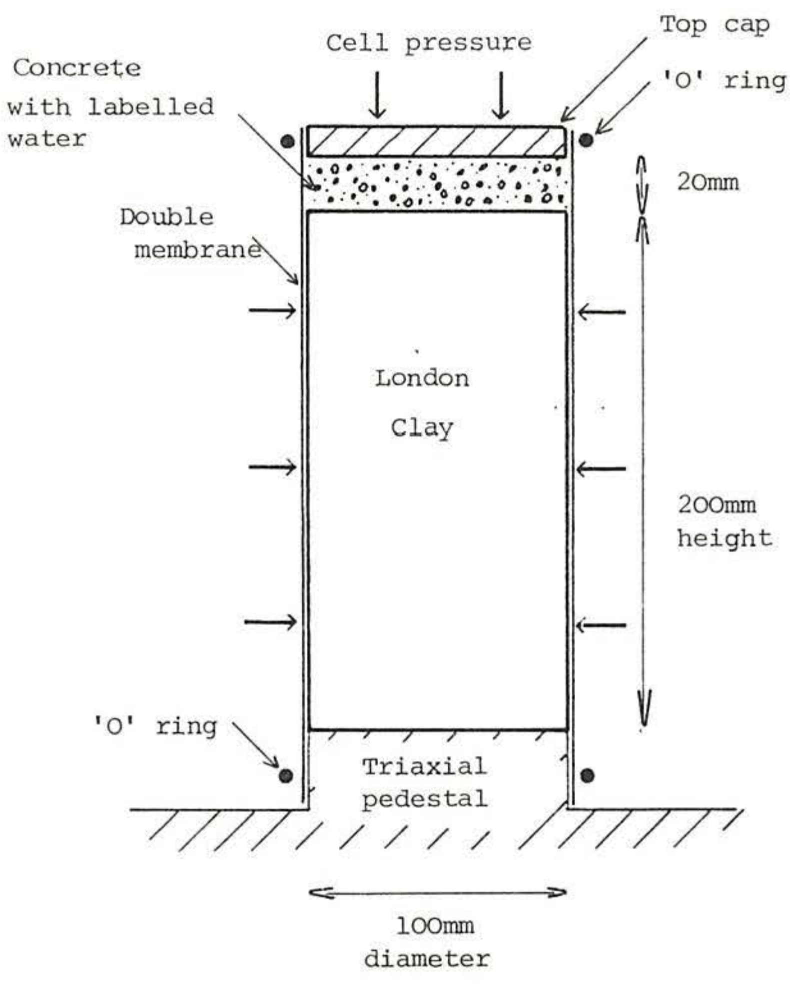


FIGURE 4.4 - LABORATORY TEST ARRANGEMENT

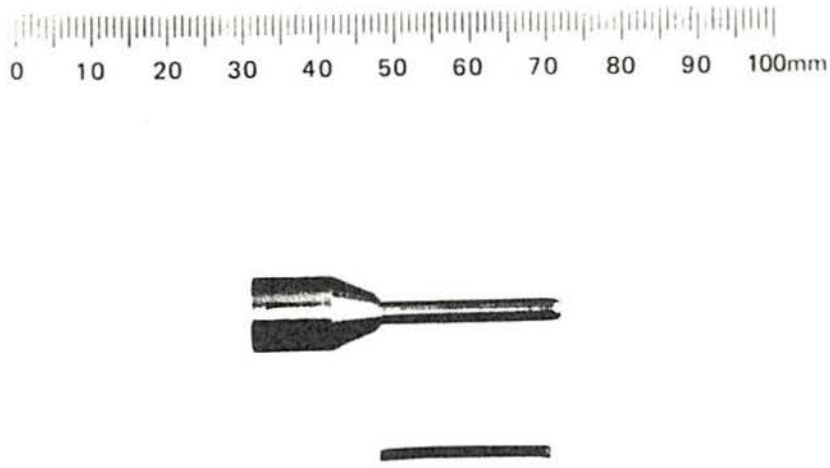


FIGURE 4.5 - MINIATURE SAMPLER WITH A SAMPLE OF CLAY

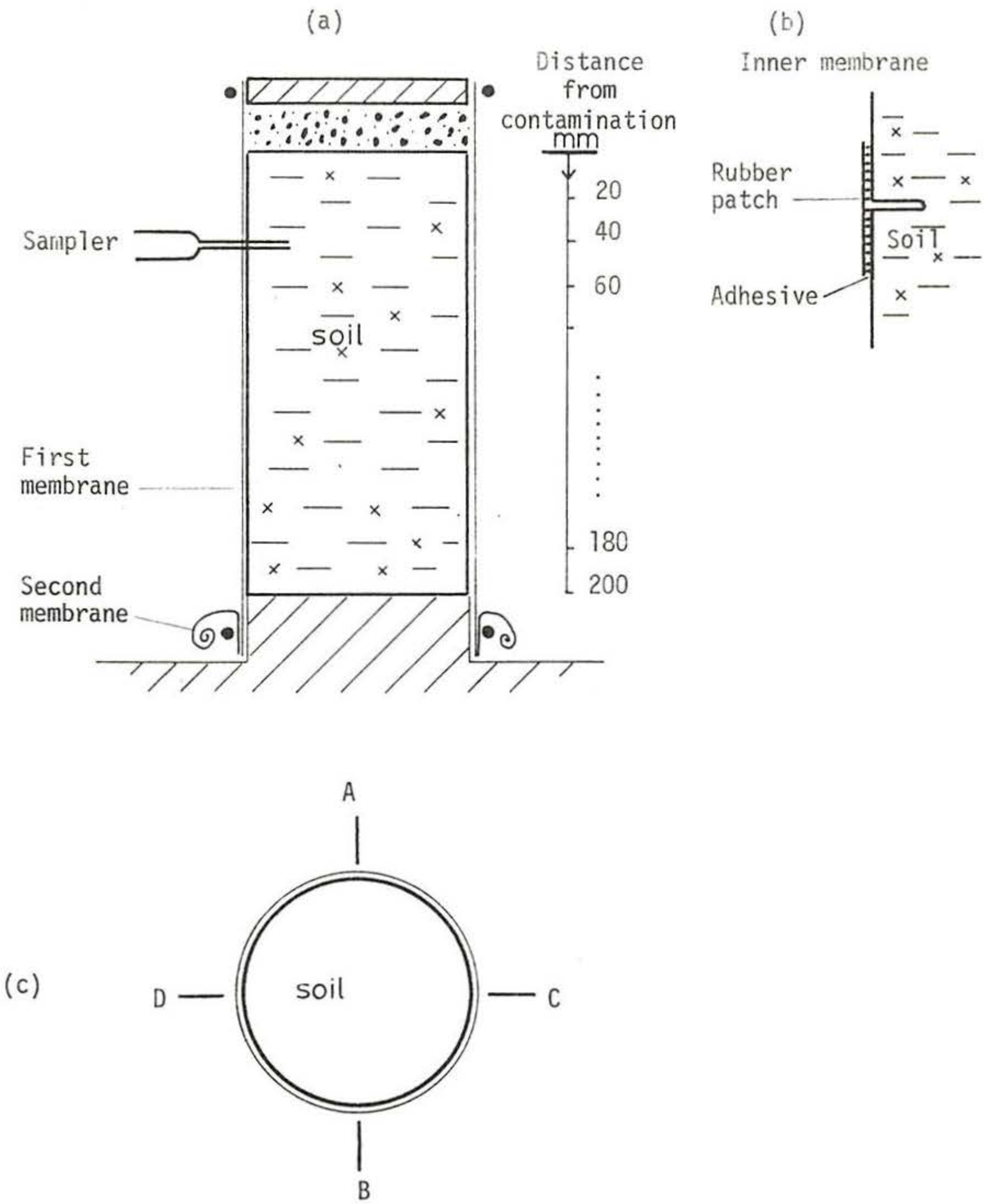


FIGURE 4.6 - DETAILS OF (a) TEST LAYOUT
(b) MEMBRANE REPAIR
(c) SAMPLING LOCATIONS

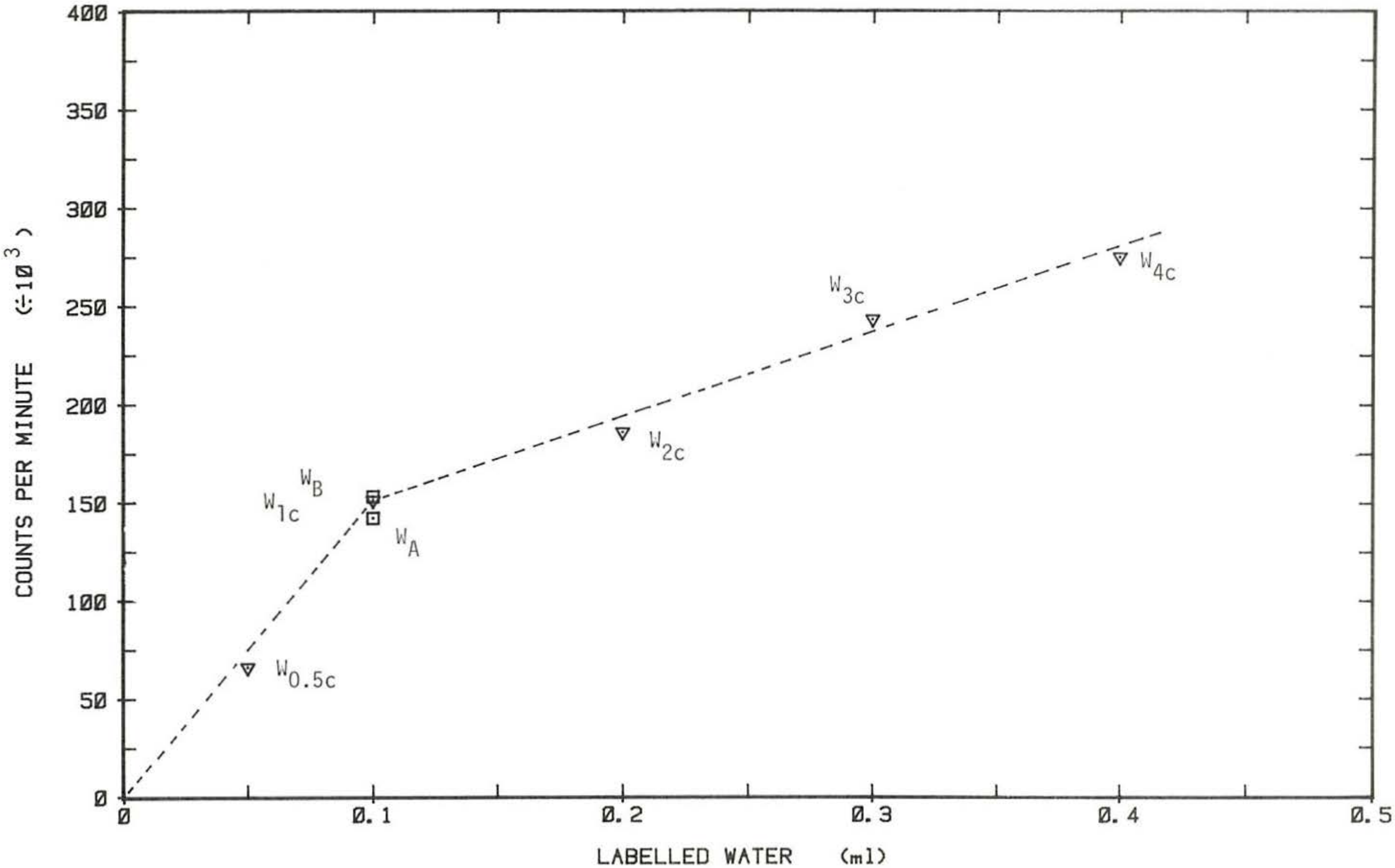


FIGURE 4.7 - EFFECT OF INITIAL SAMPLE SIZE ON THE COUNTS

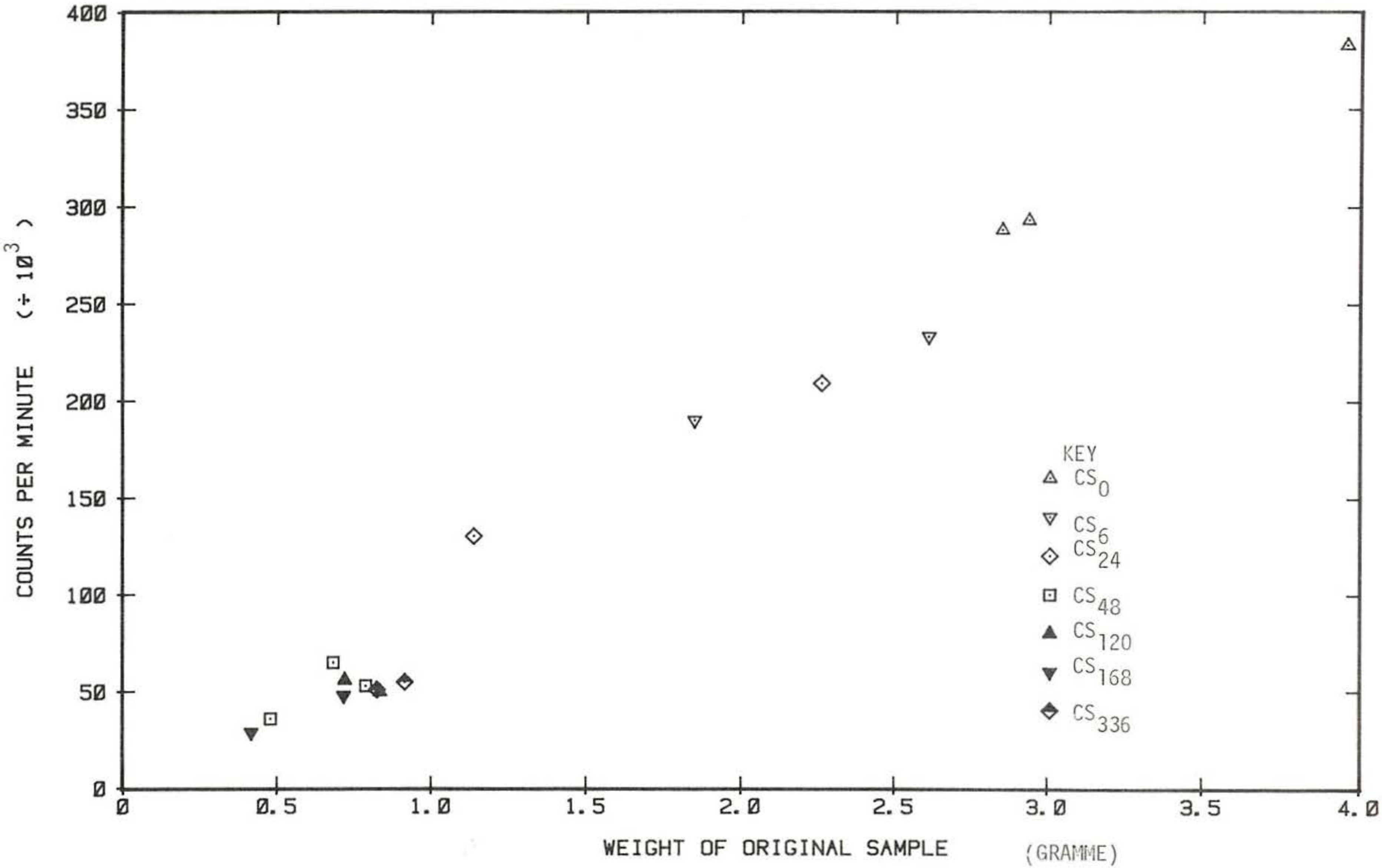


FIGURE 4.8 - EFFECT OF AGEING OF THE MORTAR SPECIMEN

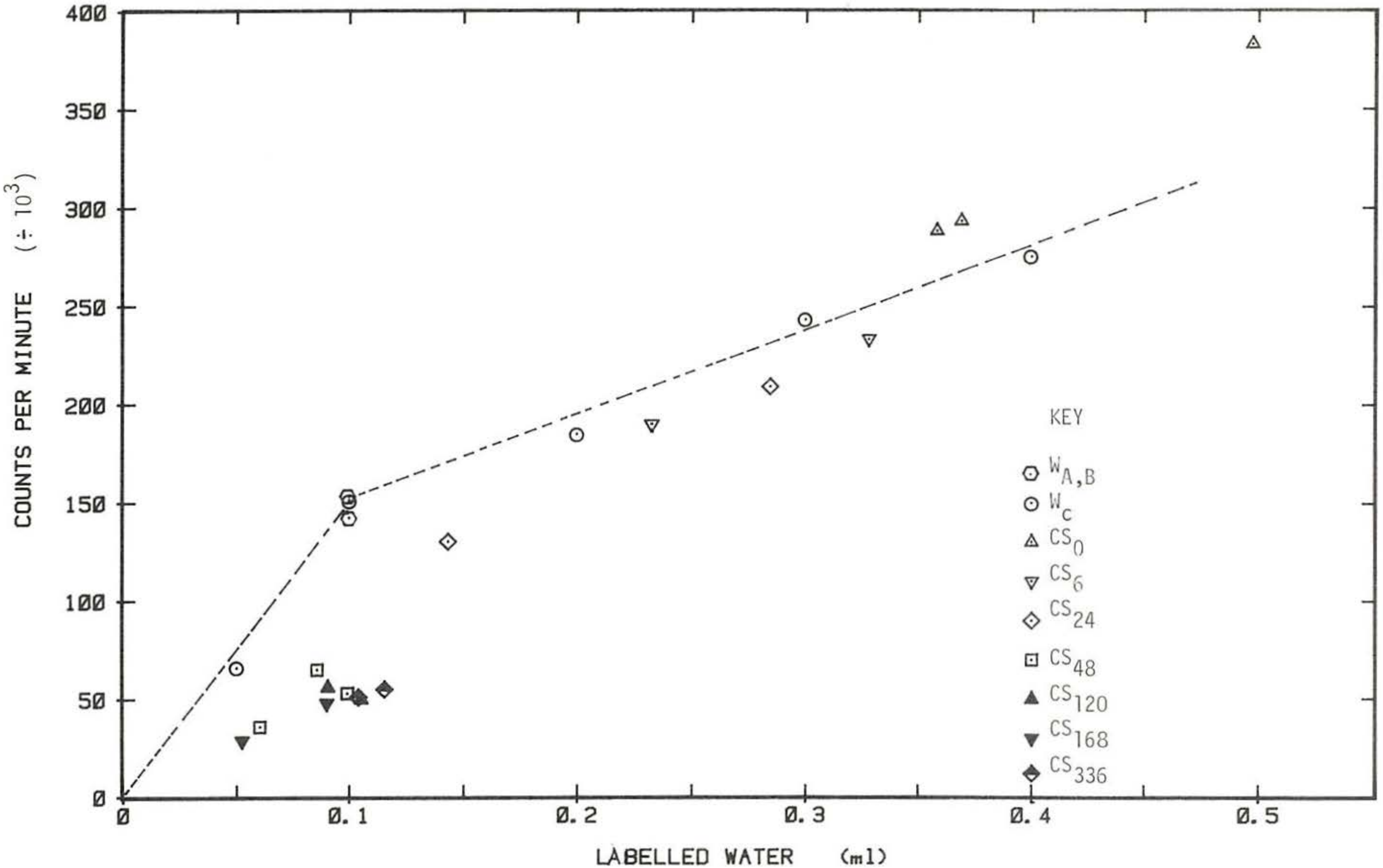


FIGURE 4.9 - EFFECT OF AGEING OF THE MORTAR SPECIMEN, REFERRED TO THE ORIGINAL AMOUNT OF WATER

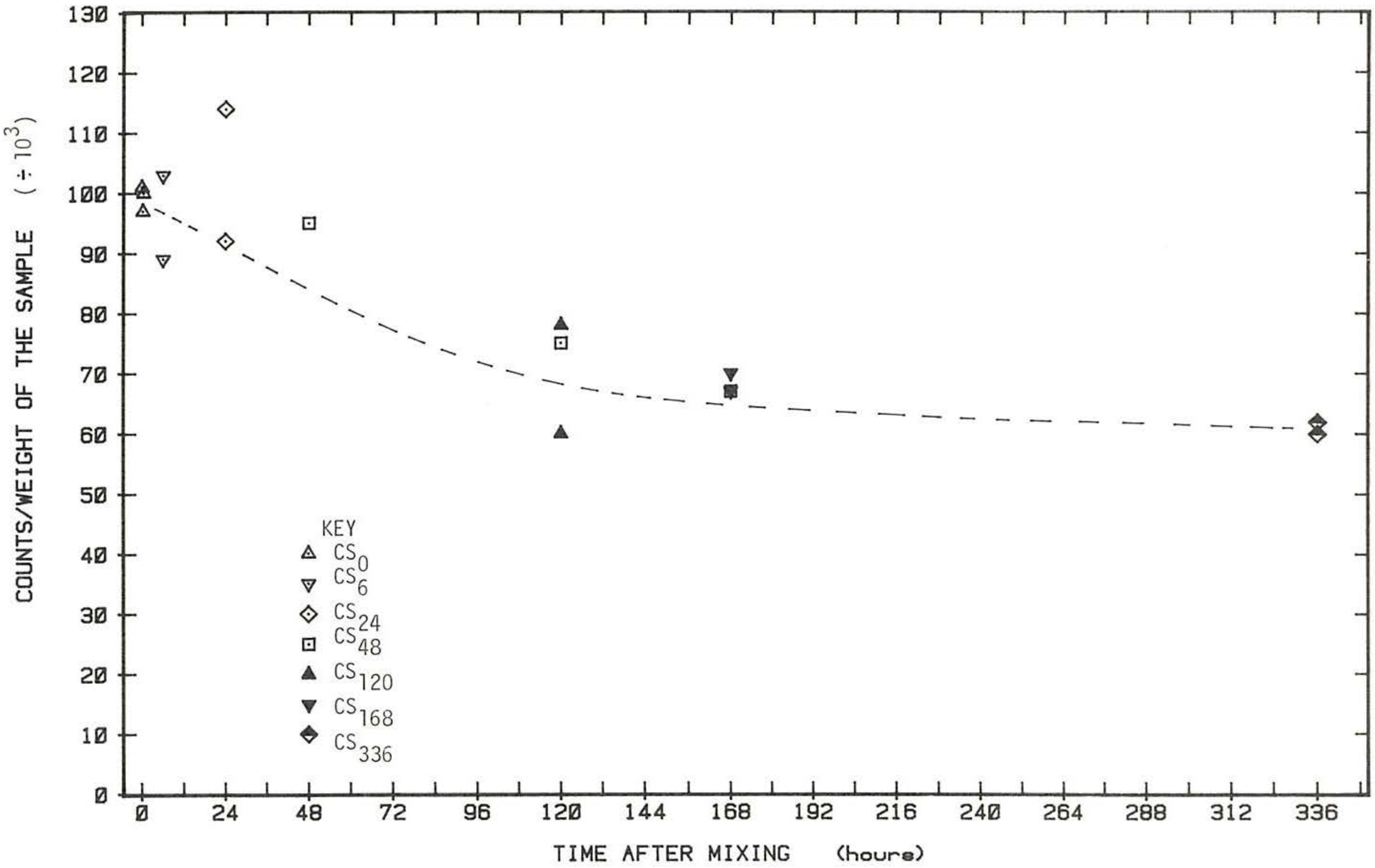


FIGURE 4.10 - VARIATION OF COUNTS PER WEIGHT OF TEST SPECIMEN WITH TIME

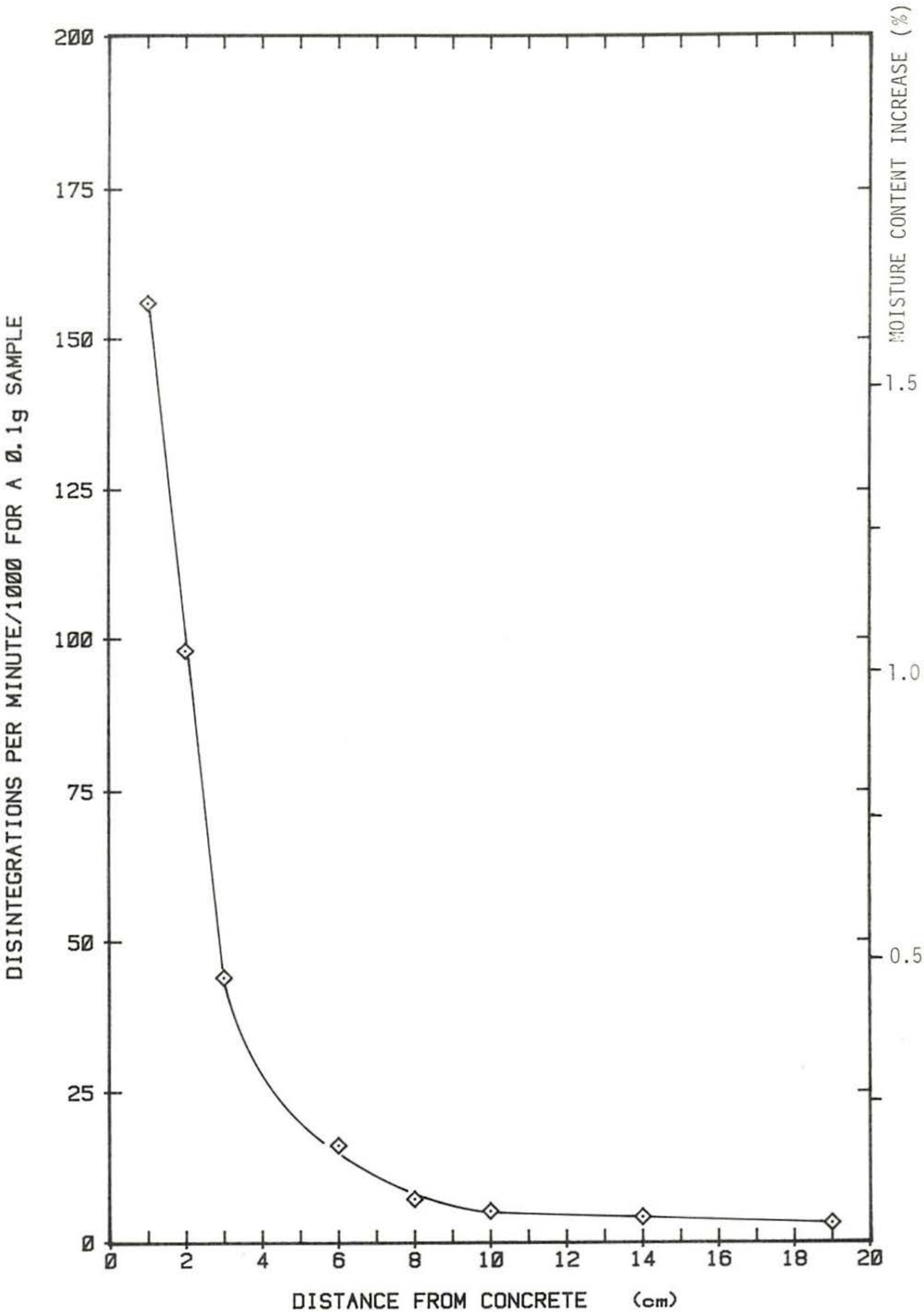


FIGURE 4.11 - CLAY/CONCRETE CONTAMINATION, SPECIMEN RC_1, ONE DAY

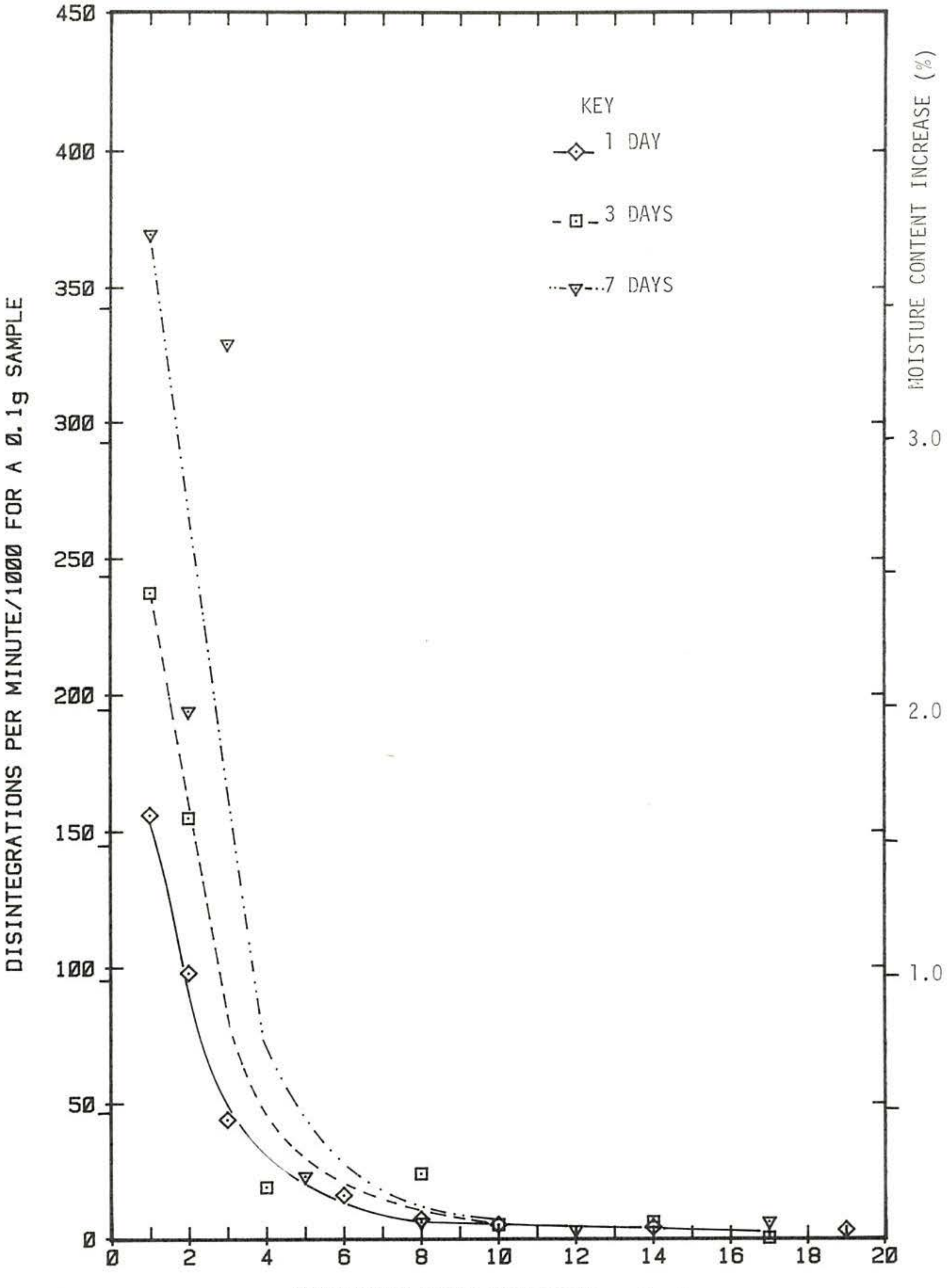


FIGURE 4.12 - CLAY/CONCRETE CONTAMINATION, SPECIMEN RC_1, AT ONE, 3 AND 7 DAYS

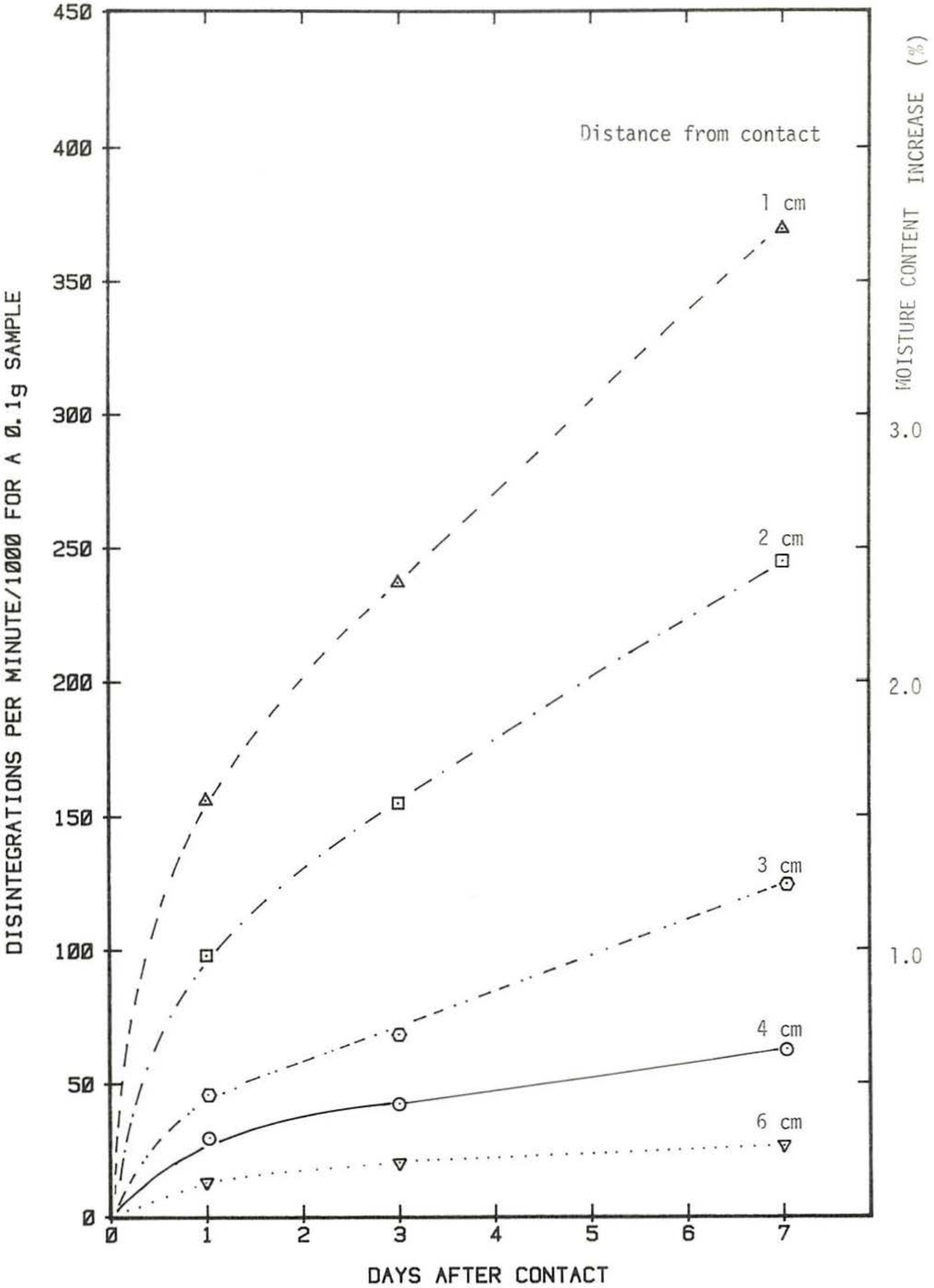


FIGURE 4.13 - EVOLUTION OF MOISTURE CONTENT WITH TIME FOR SPECIMEN RC_1 (TAKEN FROM AVERAGE LINES IN FIGURE 4.12)

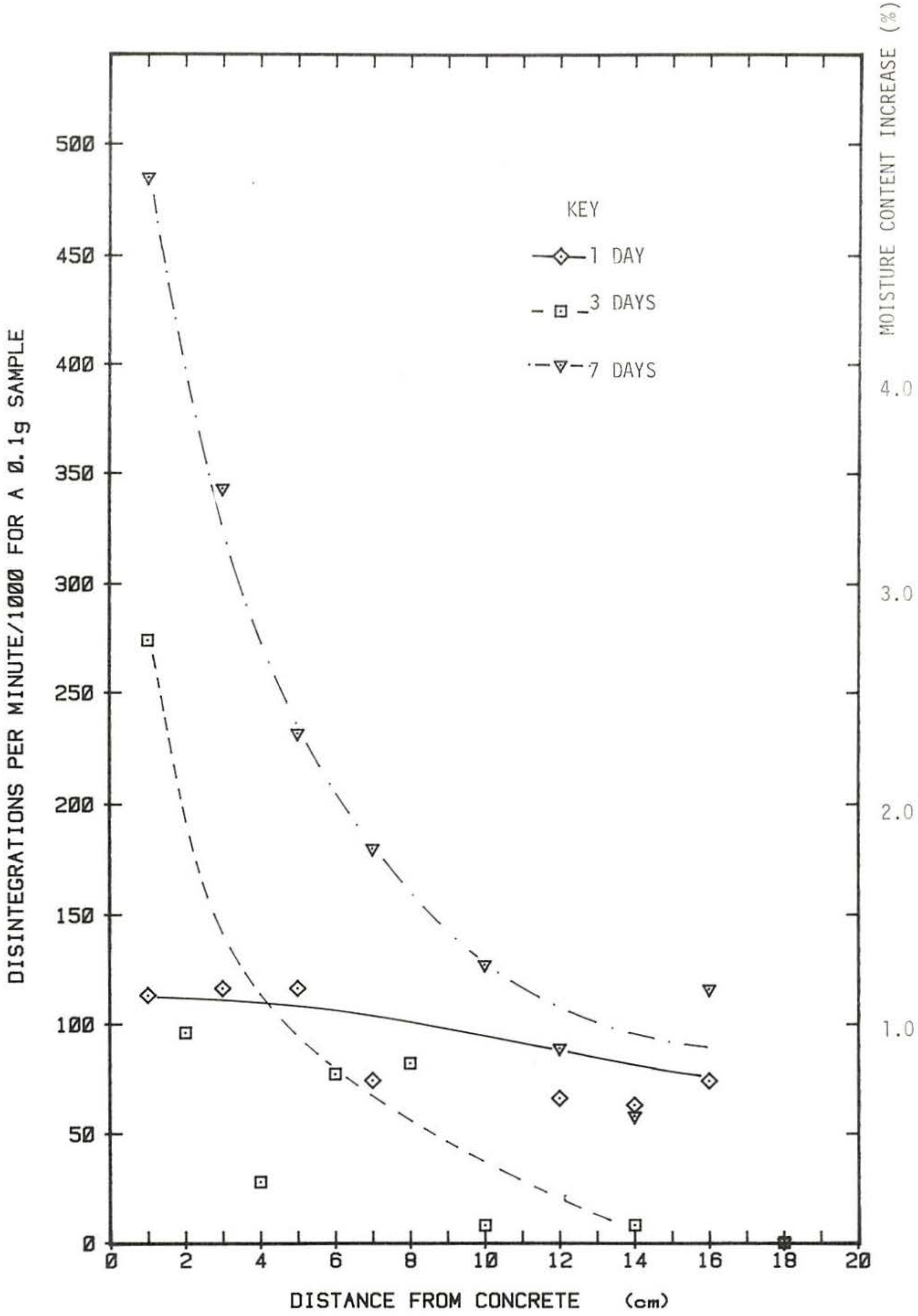


FIGURE 4.14 - CLAY/CONCRETE CONTAMINATION, SPECIMEN RC_2, AT ONE, 3 AND 7 DAYS

C H A P T E R 5 - P O R E P R E S S U R E I N F R E S H C O N C R E T E

CHAPTER 5 - PORE PRESSURE IN FRESH CONCRETE

5.1. - INTRODUCTION

From the identification of all the mechanisms involved during the installation of a bored pile emerged the necessity of studying the pore pressure changes in the fresh concrete.

There is a number of references on attempts to measure the change in shear strength of fresh concrete with time using vane tests (Ritchie, 1962) or triaxial tests (Witte, 1961, Mc Dowell and Ritchie, 1969), in order to provide data for the mathematical theories on pressure in formwork. In 1969 Uzomaka studied the behaviour of fresh concrete using oedometer tests and the triaxial apparatus. In all these studies, total stresses were used and no measurement of pore pressure is mentioned.

Reviewing the literature concerning concrete technology, the topic pore pressure is found related to internal uplift of gravity dams (Moffat, 1970). Some experimental research is concerned with the effect of pore pressure on the strength of concrete (Mc Henry, 1948, Davis, 1950, Leliavsky, 1958, Serafim, 1964, Campus et al, 1969, Cullington et al, 1971, Butler, 1981). The material that was tested in this line of research was not fresh or plastic concrete. Mature specimens were used.

In order to investigate the variation of pore pressure with time during concrete set an experimental programme was undertaken, and the standard procedures used in soil mechanics laboratories were tried. Samples of fluid concrete were tested under a triaxial isotropic state of stresses. The technique used, problems and results will be presented in this chapter. Finally a discussion about the relevance of the subject, factors possibly affecting the results, conclusions and recommendations about future work is included.

5.2. - TECHNIQUE

5.2.1. - ESTABLISHMENT OF THE PROCEDURE

The method proposed to measure the variation of pore pressure with time in a sample of fresh concrete was the utilization of the standard triaxial test equipment with a pressure transducer connected to the base of the cell. Figure 5.1 shows the test arrangement. The cell used to house 100 mm samples was preferred, in order to permit the testing of a complete mix of concrete, instead of a cement / sand mortar.

The first difficulty that arose was the sample preparation. Handling a fluid mix of concrete in order to obtain a cylindrical sample is quite difficult. The first trials were made using two standard 100 mm membranes as a mould without success (Figure 5.2.). A plastic container was introduced (as a permanent mould) in order to form the sample. The base of the pedestal was protected with silicone fluid (DC 200/350 CS) and covered with two saturated filter papers discs. Two membranes were used. The top disc was placed and two "O" rings used as a seal. The ducts in the cell base were previously deaired and saturated. The cell was filled with water and the confining pressure applied. The pressure transducer readings were taken. The observed results showed no clear or defined trend in the first tests, it being impossible to obtain reproducible results when using two identical samples under the same overall conditions.

At first, sample preparation and saturation of the ducts were under close scrutiny but no reasonable results were obtained after improvements. Finally it was discovered that the base of the pedestal, made of aluminium, was suffering chemical attack from the cement present in the fluid, and there was gas generation.

The final test arrangement was established. A perspex disc protection, 5 mm thick was made. Before each test all the ducts were flushed with silicone fluid to avoid any possible contact between contaminated water and the metallic parts. The perspex disc was protected with two filter paper discs, the top of the sample was covered by a filter paper disc and sealed with two "O" rings acting on a perspex cap. Cells with brass pedestal were used.

5.2.2. - SAMPLE PREPARATION

The following steps describe the complete procedure of sample preparation:

1 - Silicone fluid was flushed through the ducts of the pedestal, pressure transducer block and perspex protection disc;

2 - The base of the cell, including all elements such as perspex disc, two filter paper discs, pressure transducer block, etc, were deaired and satured;

3 - The plastic mould was positioned inside two 100 mm standard triaxial membranes fixed to the pedestal with two "O" rings. Usually the plastic mould was fixed to the pedestal with a tape, with some longitudinal cuts on the ends to allow the connection (Figure 5.4);

4 - The solid components of the concrete were mixed, to the point of achieving a homogeneous colour (using a Liner Cumflow mixer);

5 - The necessary amount of water was added to the mix;

6 - From the obtained homogeneous mix, a slump test was performed;

7 - Immediately after the mix was obtained, the material was poured into the mould , with continuous tapping of the concrete with a metallic 10 mm diameter bar to obtain a sample without trapped

air or voids. When a sample of approximately 200 mm high was formed, the top had to be properly prepared for the cap. A trowel was used to obtain a flat surface. During sample preparation it is convenient to protect the cell base with paper in order to avoid any contact of the mix;

8 - A fully saturated filter paper disc was used to protect the top cap;

9 - After getting the top cap in position, with proper consideration to the problems of voids between the sample and the cap and careful cleaning of the sides of the cap, the two membranes were unfolded;

10 - The specimen was sealed using two "O" rings on the top;

11 - The cell was filled with water.

COMMENTS: The successful manipulation of the fluid concrete needs some experience. Sometimes minor details such as lack of cleaning the sides of the top cap and membrane can result in a sample lost due to leaking past the "O" ring. All tools should be cleaned immediately after use.

5.2.3. - TESTING

After the procedures described under the title "Sample preparation" the testing stage was started. The cell pressure was raised in stages up to the desired level. Readings of the pore pressure change were compared with the change in confining pressure, in order to check the value of the pore pressure parameter $B = \frac{\Delta U}{\Delta \sigma}$. The confining pressure was maintained and the pore pressure readings were taken at intervals of time of 1, 2, 4, 8, 15 minutes, ½, 1, 2, 4, 8, 12, 24 hours, 2, 3, 4 days. Usually a datalogger was used and after the first hour automatic data acquisition was provided.

5.3 - RESULTS AND DISCUSSION

The results obtained are presented in Tables 5.1 and 5.2 and Figures 5.5 to 5.7.

The tested mix was composed of :

Cement	1	4 kg
Water	0.59	2.585 kg
Aggregate 10-20mm	1.60	6.400 kg
5-10mm	0.78	3.120 kg
Sand	1.93	7.720 kg

Samples A-1, E-1 and E-2 were tested under the same overall conditions to check the repeatability of the results. Considering the nature of the material being tested, the results show a good level of repeatability.

The tests on Samples F-1 and F-2 were made to study the influence of the confining pressure on the pore water pressure recorded.

In order to compare the results obtained the pore pressures were normalized against confining pressure and plotted for the tests performed in Figure 5.7.

The results of all tests showed that at the begining of the test, when the confining pressure is raised, the concrete mix used behaved as a saturated granular soil, resulting in a B value equal or very close to one. As the concrete set , small changes in the recorded pore pressures were observed, up to 8 hours. Major changes are observed in the first day, the results apparently being dependent on the confining pressure. For the specimens under confining pressures equal or greater to 200 kN/m^2 pore pressures dropped approximately to atmospheric pressure after 48 hours. The specimen subjected to a confining pressure of 100 kN/m^2 had a different behaviour. The reduction of the recorded pore pressures took 4 days.

After the pressure fell to zero further changes were minor. In sample under 300 kN/m^2 a suction was recorded.

Considering these results in the light of the theories of concrete pressure on formwork that considered the pore water pressure (Schjødt, 1955), effective stresses (Harrison, 1979), the mechanisms proposed by the different authors (see Appendix 1) and the field values of measurements of concrete pressure presented in Chapter 1, Section 1.4.3, the following idealization can be proposed as the most likely boundary condition at the interface between the fresh concrete and surrounding soil in a bored pile, due exclusively to the presence of the fresh concrete:

-when a high slump fresh concrete is poured into the borehole it acts as a fluid of density equal to that of the concrete.

-if a lower slump concrete mix is used, a continuous intergranular structure involving the solid constituents may be formed. In such a case the initial horizontal concrete pressure will be the sum of the horizontal component of the pressure of the granular structure and the pore water pressure. As in this case the fluid will have a density between that of the water and the fluid concrete, the resulting pore water pressure will be lower than the case where no initial intergranular structure exists.

-depending on the pile dimensions and concreting procedures, in long and large cross-section piles when concreting can take a long time and concrete set starts before full concreting is achieved, the initial pressure distribution will be different from hydrostatic.

-as the concrete sets, cement hydration will affect the pressure distribution. The pore water pressure will reduce due to the change in the amount of free water available. The pressure due to the granular components of the concrete will change, as cement hydration modifies the mechanical properties of the material. There is a change both in compressibility, and permeability (Figure 5.8), affecting the interaction between soil and concrete.

-once a particle structure is formed, the demand for water can create suction.

Total lateral pressure will consequently change with time in a very complex way.

Sample	Slump (mm)	Test Start	Duration of test (days)	Confining Pressure (kN/m^2)
A-1	150	1/6/81	5	200
E-1	160	25/6/81	5	200
E-2	160	25/6/81	5	200
F-1	170	2/7/81	11	100
F-2	170	2/7/81	11	300

Table 5.1. - Scope of the experiment

SAMPLE	A-1	E-1	E-2	F-1	F-2
CONFINING PRESSURE (kN/m^2)	200	200	200	100	300
Minutes					
0	196	200	191	99	290
5	198	201	195	100	295
10	198	203	197	100	294
30	199	203	197	101	293
Hours					
1	200	203	197	102	292
2	203	203	196	103	290
3	204		195	103	290
4	203		195	101	288
8		179	189	97	267
18	70	108	118		
21				74	115
Days					
1	55	76	89	72	98
2	10			45	22
3	0	-2	6	21	2
4	-6	-2	4	4	-6
5	4	-2	2	-9	-4
6				-9	-15
7				-4	-24
8				-5	-28
9				-4	-24
10				-4	-24
11				-4	-22

Table 5.2 - Summary of the results obtained (Pore water pressure in kN/m^2)

5.4. - CONCLUSIONS AND RECOMMENDATIONS

A procedure using a triaxial cell was established that allowed the study of the variation of pore pressure on fresh concrete with time, giving reproducible results. Specimens of 100 mm diameter were used in order to test a complete mix. When testing fresh concrete specimens it is necessary to use a chemically inert test apparatus in order to obtain meaningful measurements of pore pressure.

The results obtained showed that at the begining of the test the concrete mix used behaved as a saturated granular soil. As the concrete sets, changes in the recorded pore pressures were observed, the values apparently being dependent on the confining pressures.

The present initial study of the variation of the pore pressure regime with time in the concrete provided evidence to support the following idealization of the boundary condition at the interface between the fresh concrete and surrounding soil in a bored pile (due exclusively to the presence of the fresh concrete) :

- when the borehole is filled with concrete, the state of total stresses acting on the pile wall depends on the mix used. When a high slump fluid concrete is used, it acts as a fluid of density equal to that of the concrete. If a lower slump concrete mix is used, a continuous structure involving the solid constituents may be formed. In such a case, the horizontal pressure has two components: the first is due to the horizontal component of the granular structure, the second is the pore pressure (being the density of the fluid in between those of the water and the fluid concrete).

- total lateral pressure will change with time in a complex way as concrete sets, as cement hydration affects the mechanical properties of the material and also the amount of free water. Depending on the overall conditions a suction force can be present at a certain stage (a likely explanation for the apparent reduction in moisture content sometimes observed very close to the interface between concrete and soil).

The study of the development of pore pressure regime and the possibility of the identification of the most relevant factors affecting the pore pressure is of importance for the understanding of the interaction between bored piles and soil.

Factors such as the mix composition and proportions, water/cement ratio, confining pressure, presence of plasticisers and type of cement are likely to affect the pore pressure regime and should be investigated in future work.

The identification of a realistic pore water pressure regime in the fresh concrete and its effects in the surrounding soil is a key factor for any future theoretical study of the interaction between bored piles and soil. If an effective stress approach to pile design is to be used, the changes occurring during and after pile installation should be taken into account.

With the identification of a possible key factor affecting the interaction between pile and soil, a new testing procedure was proposed to investigate not just the boundary conditions but the pore water pressure variation in the adjacent soil (CHAPTER 6 - Pore pressure changes in the vicinity of the interface between soil and concrete).

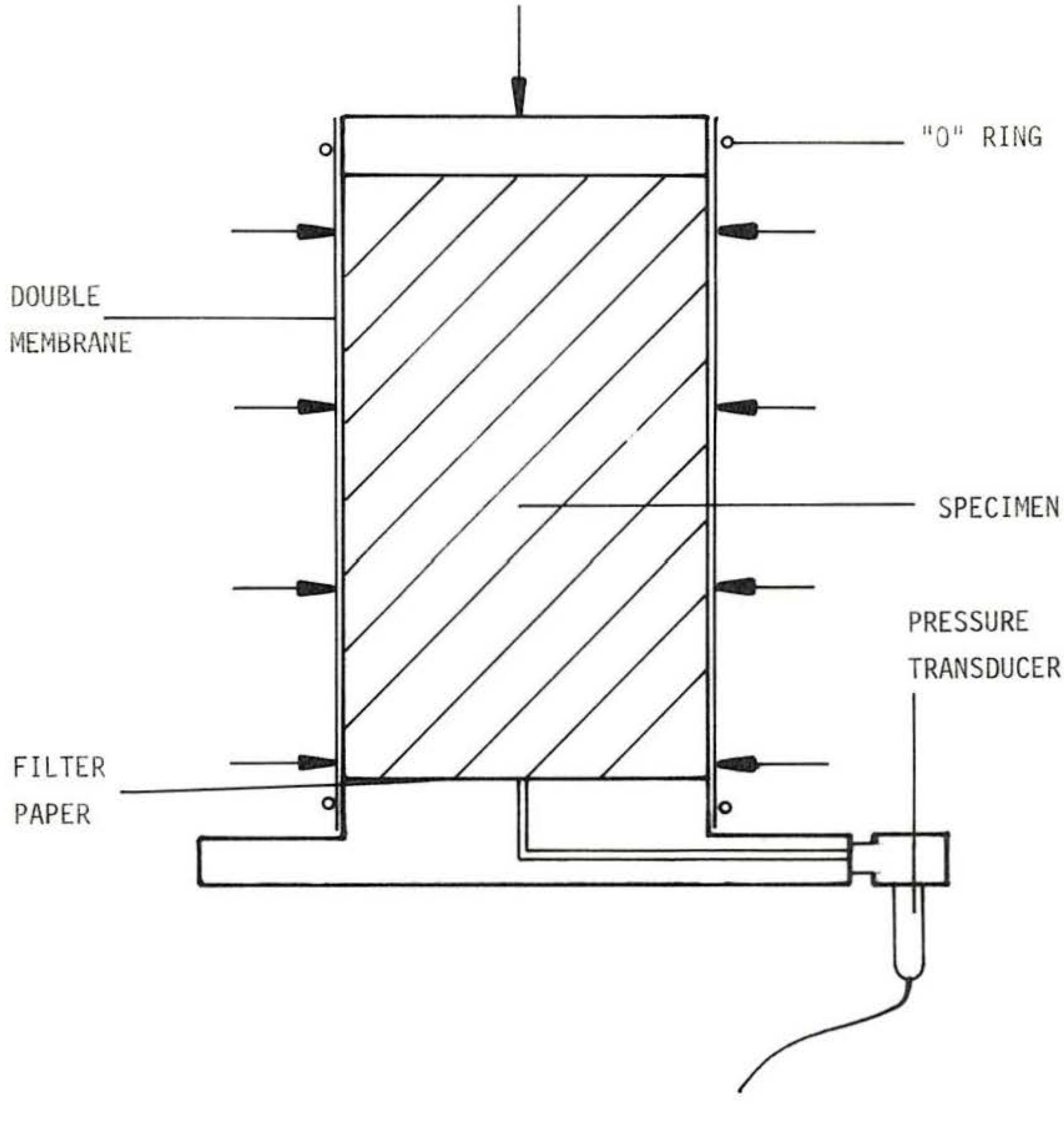


FIGURE 5.1 - PROPOSED TEST ARRANGEMENT

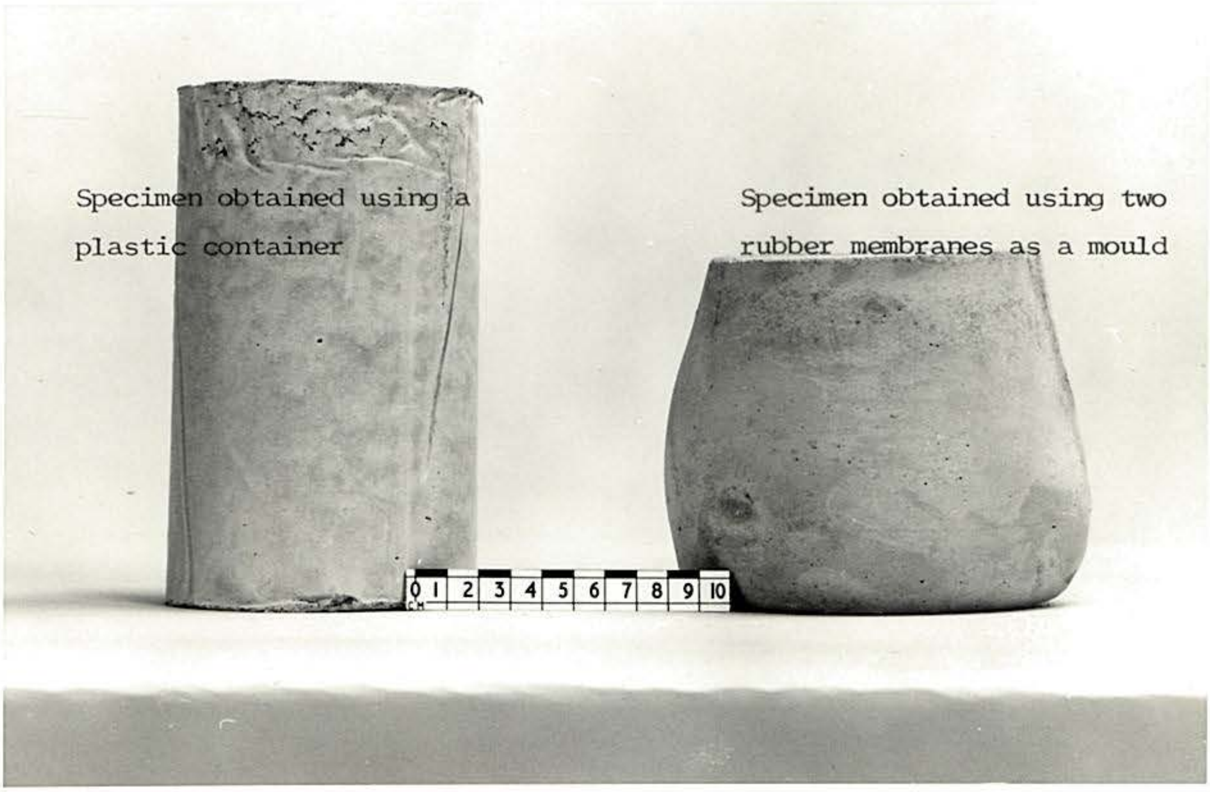


FIGURE 5.2 - COMPARISON OF TEST SPECIMENS OBTAINED USING DIFFERENT MOULDS

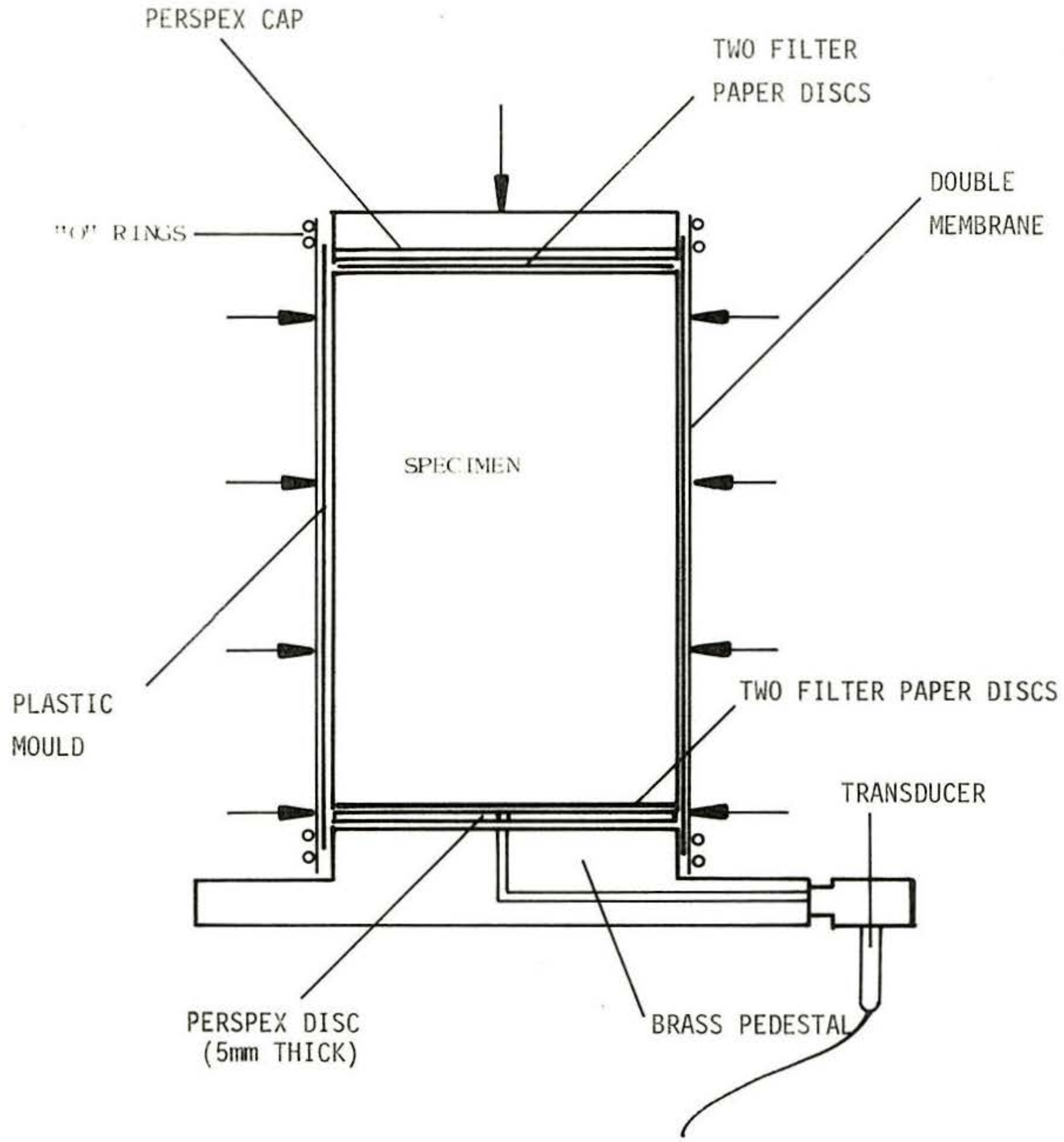


FIGURE 5.3 - DETAILED FINAL TEST ARRANGEMENT USED

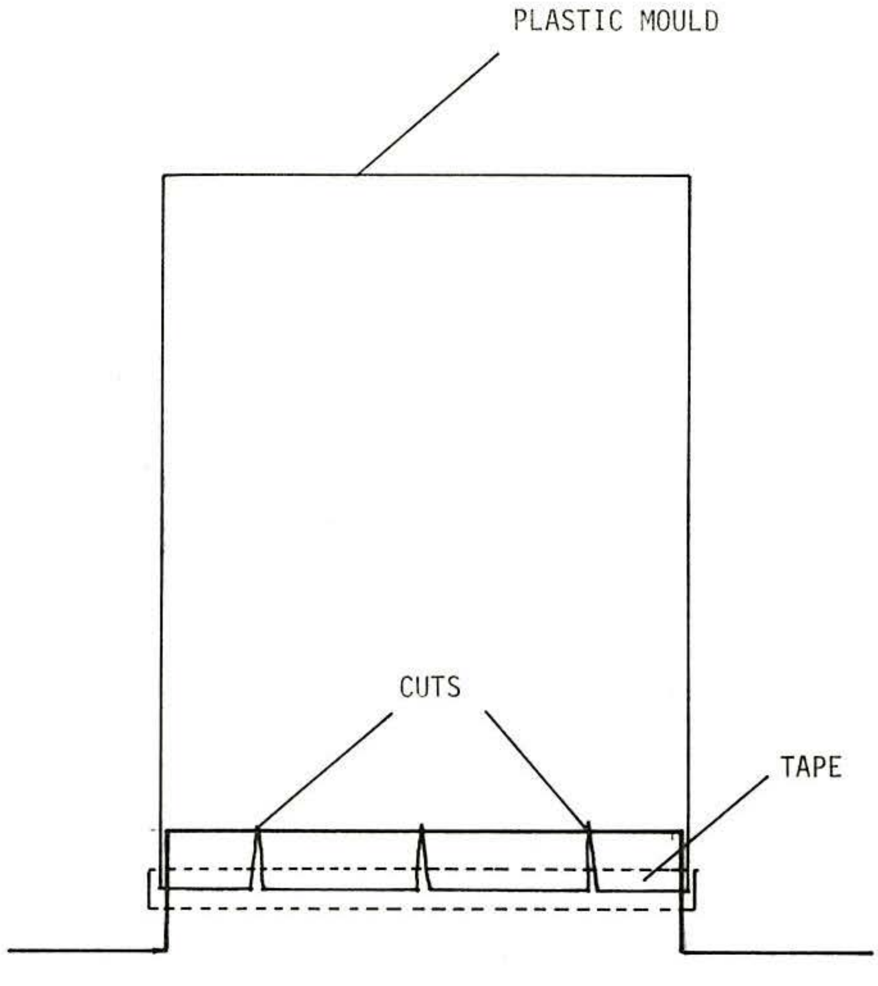


FIGURE 5.4 - DETAIL OF THE FIXING OF THE PLASTIC MOULD

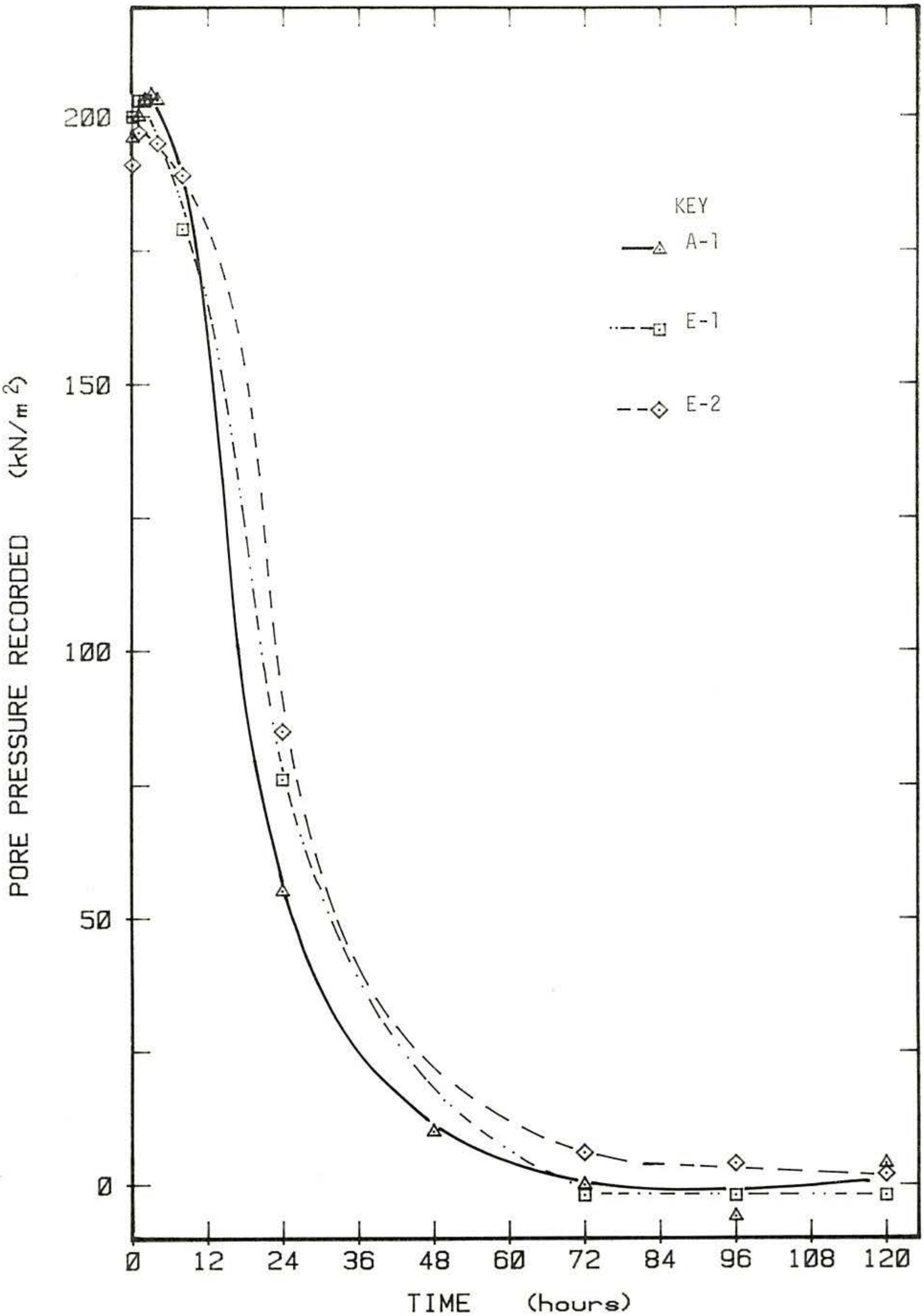


FIGURE 5.5 - RESULTS OBTAINED WITH TEST SPECIMENS UNDER CONFINING PRESSURE OF 200 kN/m^2 (A-1, E-1 and E-2)

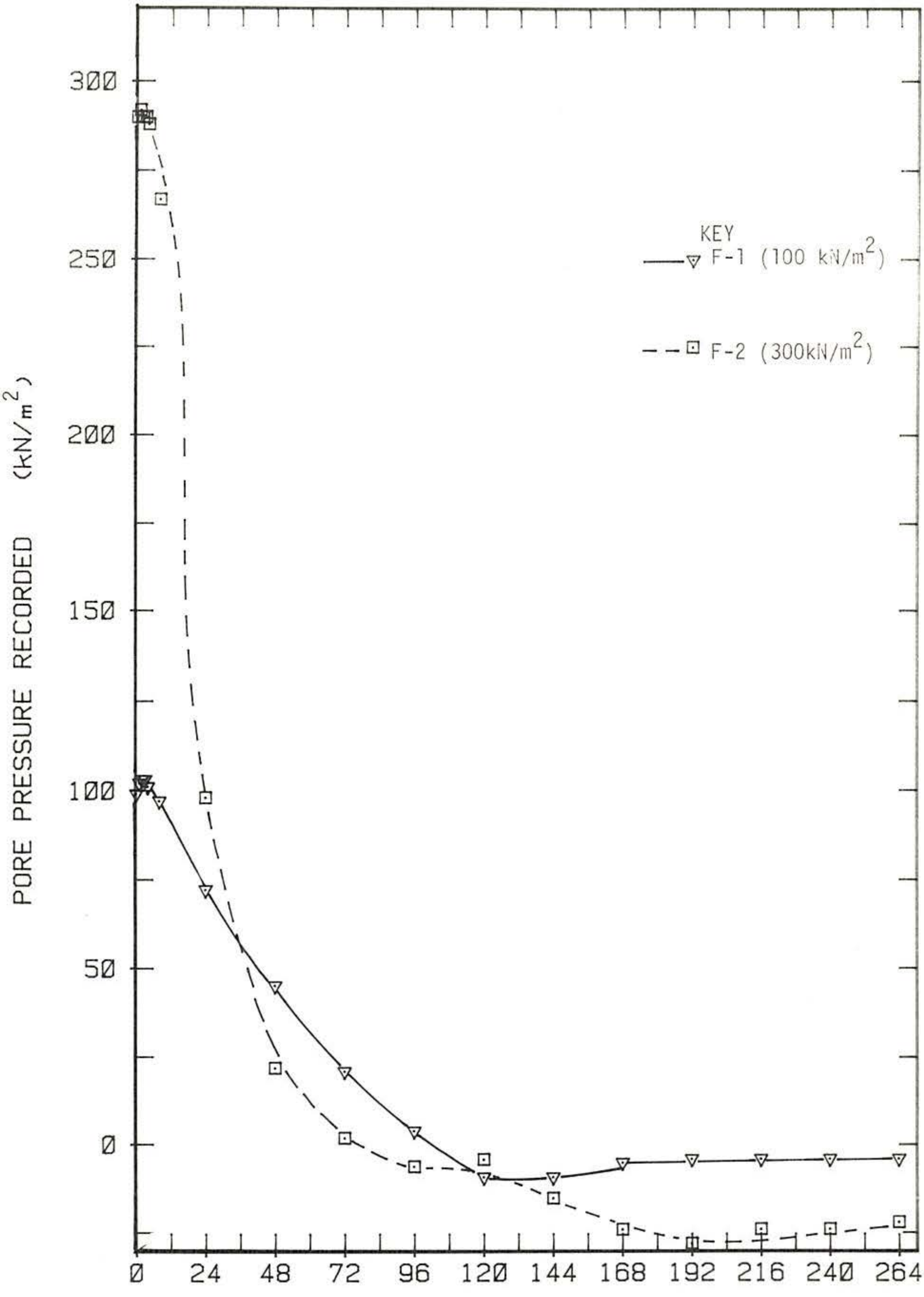


FIGURE 5.6 - RESULTS OBTAINED FOR DIFFERENT CONFINING PRESSURES

(F-1 = 100 kN/m^2, F-2 = 300 kN/m^2)

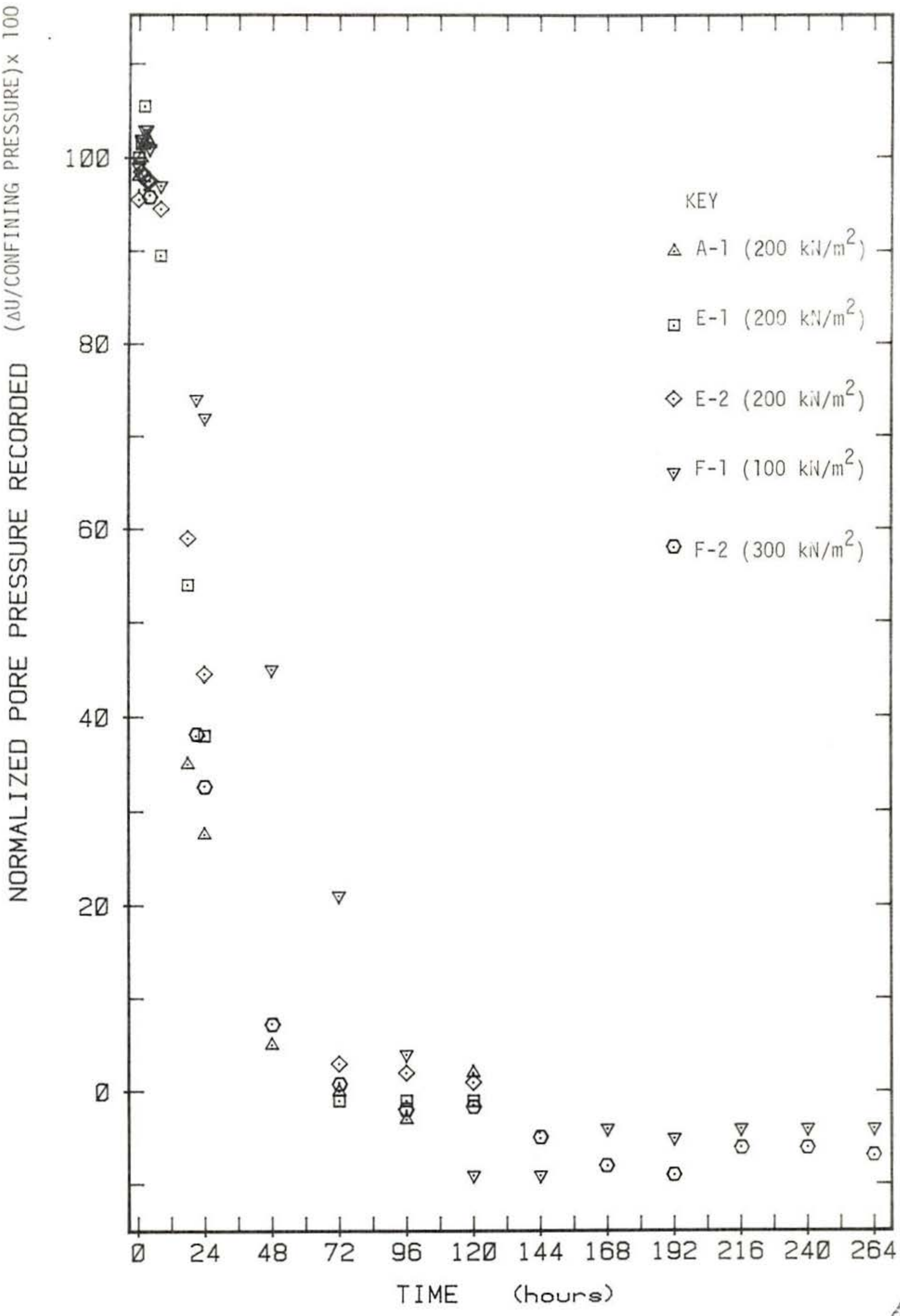


FIGURE 5.7 - COMPARISON OF THE NORMALIZED (ΔU/Confining pressure)

RESULTS

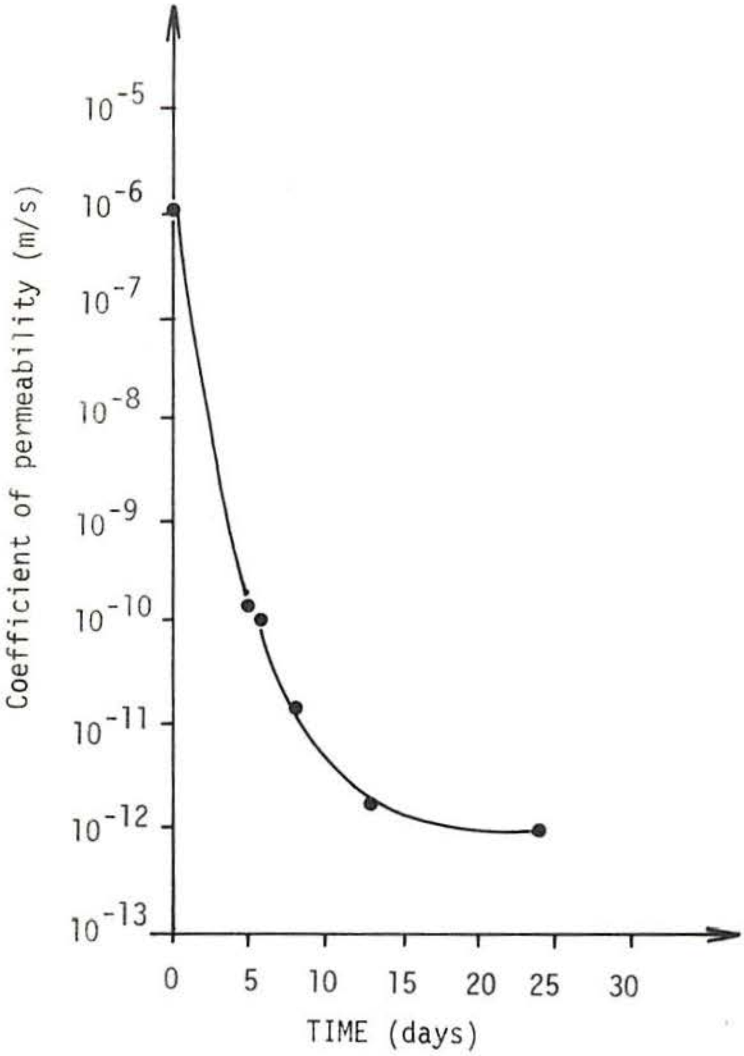


FIGURE 5.8 - VARIATION OF THE COEFFICIENT OF PERMEABILITY OF CEMENT PASTE (0.7 WATER/CEMENT RATIO) WITH TIME (AFTER POWERS, 1954, see POWERS, 1968)

CHAPTER 6 - PORE PRESSURE CHANGES IN THE VICINITY OF THE INTERFACE BETWEEN SOIL AND CONCRETE

CHAPTER 6 - PORE PRESSURE CHANGES IN THE VICINITY OF THE INTERFACE BETWEEN SOIL AND CONCRETE

6.1. - INTRODUCTION

A review of the references concerning the effects of bored pile installation on the properties and conditions of the soil surrounding the shaft of the pile brings to light some suggestions about the stress regime that follows pile concreting (Lopes, 1979, see Figure 1.9, Chapter 1). The only measurement of pore pressure found in the literature was by Yong, 1979, also referred to by Anderson and Yong, 1980, Figures 1.10a and b.

To investigate the variation of the pore water pressure in the surrounding soil due to the presence of fresh concrete it is necessary to simulate the field conditions in the laboratory as closely as possible. It was decided to use specimens of "undisturbed" London Clay and a complete concrete mix.

The standard triaxial cell with minor adaptation was used to house the clay specimen. The experimental programme proposed consisted of the installation of miniature pressure transducers inside a 100 mm diameter clay specimen, at different distances from the top of the specimen , which will be in contact with fresh concrete, reconsolidation of the sample to give an internal equalization of pore pressure, contact of the top of the specimen with fresh concrete, reapplication of the same confining pressure and the recording of the possible variation in pore water pressure with time at the different locations inside the soil specimen.

In this chapter the technique proposed, materials used and results obtained will be presented. A discussion of the significance of such studies and results, conclusions and recommendations about future work is included.

6.2. - TECHNIQUE

6.2.1. - TEST ARRANGEMENT

The proposed approach to study the pore pressure variation at different points inside a clay sample due to the presence of fresh concrete was the use of standard triaxial cells with some adaptation and the insertion of miniature pressure transducers(PDCR - 81/Druck) inside a cylindrical clay sample, 100 mm diameter, 200 mm high, at different distances from the contact with the fresh concrete. The pressure on the base of the sample was measured by an external pressure transducer. All transducers were connected to a datalogger for automatic reading and storage of results. The general test arrangement is shown in Figure 6.1.

6.2.2. - SAMPLE PREPARATION

Sample preparation was achieved by performing the following stages:

1 - The pore pressure transducers previously deaired using a vacuum pump were placed through the special connection designed in such a way that the wires could cross the membrane without leakage, Figure 6. 3;

2 - The wires from the transducers were positioned through the cell base, using the connections(Figure 6.3.) designed for closing the holes in the base of the cell, and connected to the datalogger;

3 - A specimen of clay was placed on the pedestal usually used for performing standard triaxial tests in 100 mm diameter samples. (after deairing and saturation of the ducts of the pedestal, porous stone disc, filter paper and pressure transducer block);

4 - The usual method of fixing the membrane, top cap (without drainage) and "O" rings was used to prepare the sample for the next stage. Care must be taken with the length of the membrane in order to allow the fresh concrete housing on the top of the sample;

5 - Circular holes were made in the membrane (5 mm diameter) in the desired positions, in order to permit the housing of the miniature pore pressure transducers inside the clay sample;

6 - Using miniature samplers (Figure 6.2) cylindrical samples (5 mm in diameter, 12 mm long) were taken from the clay sample, through the holes in the membrane;

7 - In each hole a miniature transducer was introduced, the device designed to permit the wires to cross the membrane was tightened. As an extra protection a layer of Silicone Rubber Compound was applied covering a large area including the hole (Figures 6.2 to 6.5).

8 - After positioning all miniature transducers, the triaxial cell was closed, filled with water and the desired confining pressure applied;

9 - After the first readings were obtained, a decision about the drainage/pore pressure had to be made: the readings obtained were different for the various positions. The first choice was to maintain the drainage closed and wait for pore pressure equalization. The second was to apply a convenient back pressure and again wait for equalization. In the present research the second choice was made.

10 - When the pore pressure readings were stable and uniform (after equilibrium under the applied state of stresses) sample preparation was complete. The entire process typically took 15 days.

6.2.3. - TESTING

The testing procedure used can be described in the following steps (after sample preparation):

1 - The drainage duct was closed. The cell pressure was lowered, the water was taken from the cell and the cell was opened;

2 - The "O" rings on the top cap were taken from position, the top cap was removed;

3 - The designed concrete mix (see Table 6.1) was poured into the top of the clay sample, the final top surface was prepared with care, a filter paper disc was positioned on the top of the fresh concrete. After getting the perspex top cap in position, taking care to avoid the formation of voids between the sample and the cap and careful cleaning of the membrane and top cap to prevent leakage, the membrane was unfolded;

4 - The complete specimen was sealed by using two "O" rings on the top cap;

5 - The cell was refilled with water and the desired confining pressure was applied;

6 - At this stage the drainage duct was mantained closed;

7 - The pore pressure readings were taken for the desired time. A datalogger was used to provide automatic data acquisition (Schlumberger-Solartron 3430 compact Logger).

8 - At the end of the testing period the sample was opened and the moisture content and cone penetration values were determined at different distances from the interface between the concrete and soil.

6.3. - RESULTS

The testing apparatus is presented in Figure 6.1 and the various equipments and details in Figures 6.2 to 6.5.

The tested materials will be identified in table 6.1. All transducers used were calibrated using a Budenberg Dead Weight Tester. Both soil specimens came from the profile described in Table 1.4 (Chapter 4), from 13.5 m depth. After extrusion from U 100 samplers the moisture content of the samples were determined, then the procedure explained under "6.2.2.- Sample preparation" was followed.

Table 6.2 presents the pore water pressure readings at the different stages during sample preparation. It can be seen that the initial readings presented different values from point to point. Considering (i) that the samples used were not recently obtained from the site investigation programme, (ii) the process of inserting the transducers may have changed local conditions and (iii) the usual lack of homogeneity of natural samples (with the fissures playing an important role), this finding was not unexpected.

During the stage of relief of confining pressure and pouring the concrete on the top of the samples, negative pore water pressures were measured (Table 6.2).

The results obtained when the confining pressure was reestablished are presented in Figures 6.7a to 6.8c. Figures 6.7a and 6.8a presents the recorded pore water pressure in samples B-3 and C-2 in the first 4 and 6 hours, respectively. The first day's results are presented in Figures 6.7 and 6.8b. The recorded results in the first week are presented in Figures 6.7c and 6.8c.

In order to study the moisture changes and shear strength variation at the end of the testing period (150 days) the test specimens were opened (Figures 6.5 and 6.6). The moisture content and "Index strength" determined at different distances from the contact between clay and concrete are presented in Figures 6.9 and 6.10 for the test specimen B-3. No tests were performed in test specimen C-2 due to dificulties in handling the material.

The results of pore pressure variation obtained from both samples shows that the re-establishment of the confining pressure initiated an increase in pore pressure (in the clay sample) up to a level higher than the one previously recorded (during the consolidation and equalization period). According to the results obtained in Chapter 5, when the confining pressure is raised, the pore pressure in the concrete is raised too, and is equal to the confining pressure. The difference in pore pressure between the two material causes a migration of water from the fresh concrete towards the clay sample. The maximum recorded values of pressure are variable according to the position inside the sample. The equalization of the pore pressure inside the clay sample was obtained in about one day for specimen B-3 and approximately six days for specimen C-2, after contact with fresh concrete and re-establishment of the confining pressure.

Examination of the results obtained with sample B-3 showed that: - the transducer located at 2 cm from the interface between fresh concrete and soil registered a maximum pressure of 484 kN/m^2 a few minutes after the re-establishment of the confining pressure (500 kN/m^2) and started decaying from that value with time (original back pressure of 325 kN/m^2); - the increase in pore pressure at the other points (5, 10 cm and in the base of the sample) took 7 to 10 hours, with the maximum recorded pressures of 375, 355 and 339 kN/m^2, repectively;

- in the first week the pressure recorded in the base stayed stable, the remaining transducers recorded a slow reduction in the pressures;
- the recorded pressured continued to reduce slowly up to 3 weeks and than stabilizes at 310 kN/m^2.

The behaviour of sample C-2 can be described as follows :
- maximum recorded pressure was at the transducer installed 3 cm from the contact between the fresh concrete and soil and was 315 kN/m^2, four hours after the re-establishment of the confining pressures;
- the increase in pore pressure continued up to 8 hours at the transducer located at 6 cm from the contact and up to 24 hours at the base (slowly after 6 hours), the maximum recorded pressures being 293 and 275 kN/m^2 respectively (original back pressure of 250 kN/m^2);
- one week after the re-establishment of the confining pressures the pore pressures equalized at 255 kN/m^2 at the three recording points.

From Figure 6.9 it can be seen that the variation in moisture content near the contact in the specimen B-3 was up to 3.5 % higher than the moisture content obtained at 7 cm and 2 % from the moisture content measured at 15 cm from the contact. The "Index strength" (I_s) variation is shown in Figure 6.10. The calculated "Index strength" are $I_s c = 73$, $I_s min = 62$, $I_s max = 92$ and the ratios of "Index strength", as defined in Chapter 3 are :

$$R = I_s min \; / \; I_s max = 0.79$$

$$R_c = I_s c \; / \; I_s max = 0.67$$

Figure 6.6 shows the clay/concrete interface after 5 months. The concrete was strongly connected to the clay, being necessary to use a trowel to separate the two materials. The fissured structure of the test specimen used can be identified in Figure 6.6. No apparent difference in soil color, structure or composition could be recognized when comparing the soil specimen at different distances from the interface with the concrete.

Sample identification	B-3	C-2
Sample description		
diameter (mm)	100	100
height (mm)	200	190
Moisture content before testing (%)		
Top	28	27
Bottom	27	26
Thickness of concrete layer on the top (mm)	23	20
Concrete mix used		
cement	1.00	1.00
aggregate 10-20 mm	1.60	1.60
aggregate 5-10 mm	0.78	0.78
sand	1.92	1.92
water	0.60	0.60
Confining pressure (kN/m^2)	500	500
Pore pressure before adding fresh concrete (kN/m^2)	325	250
Number of pressure transducer s inside the sample	3	2
Total number of pressure transducers	4	3
Begining of test	4/11/81	4/11/81
Duration of test	5 months	5 months

Table 6.1. - Identification of test conditions and materials used.

Specimen	B-3				C-2		
Confining pressure (kN/m^2)	500				500		
Distance from concrete/ clay interface (cm)	2	5	10	20*	3	6	19*
Initial pore pressure reading (kN/m^2)	103	74	226	325	176	191	250
Applied back pressure (kN/m^2)	323	325	324	325	250	247	255
Negative pore pressures recorded during relief of confining pressure (kN/m^2)	-40	-76	-84	-3	-25	-17	-18
Readings after 5 months (kN/m^2)	309	311	312	307	255	254	259
Time to reach new equilibrium	3 weeks				1 week		

* Transducer located outside the triaxial cell

TABLE 6.2 - Pore water pressure variation inside the test specimens B-3 and C-2 during the reconsolidation stage, relief of confining pressure and after 5 months

6.4 - DISCUSSION

Considering the results obtained in the present Chapter coupled with the ones of Chapter 5, when the pore water pressure recorded initially in the fluid concrete mix was equal to the confining pressure and the subsequent changes, the following general picture of the pore water pressure variation in the clay specimen emerges:

- when the confining pressure is reduced to zero in the triaxial cell, the pore pressures inside the sample fall to negative values.

- the presence of fresh concrete and re-establishment of confining pressure give rise to a major change in conditions for the clay specimen: the boundary in contact with the fresh concrete presents a pore water pressure level equal to the confining pressure, as the pore pressure inside the clay sample is lower, there is a migration of water from the concrete towards the clay, with a reduction in the pore pressure in the concrete and an increase of pore pressure in the clay sample.

- the water migrates inside the clay sample to equalize the overall pore pressure, the high values on the boundary start to reduce and, at a certain stage (theoretically infinite), a "uniform value" is achieved.

- points very close to the interface (up to 60 mm in the actual results) at first present an increase of pore pressure to a level higher than the original equilibrium and than a reduction takes place towards a uniform value in the sample.

- the rest of the sample presented a trend of increasing pore pressure from an initially lower value towards the original level.

- the most affected area of the test specimen is the one very close to the interface with the concrete, which experiences a higher pore water pressure level than the original equilibrium values, before the concrete contact.

- the variation of the pore water pressure in the clay sample is not just due to the dissipation of the initial higher value in the boundary; as the concrete sets, the amount of free water reduces and the hydration of the cement affects the interaction in a complex and still not completely understood manner (the possibility of the recorded suction forces interfering with the process, the variation of the concrete permeability, stiffness and others should affect the interaction).

Five months after the contact with the fresh concrete, the observed trend in moisture content and shear strength measured with a cone were the same as obtained in samples recovered from the field, near bored piles (Chapter 3), and in the laboratory programme presented in Chapter 2. An increase in moisture content near the contact and a reduction in the undrained shear strength measured with the cone were recorded. The higher moisture content close to the contact was not followed by an equivalent reduction in shear strength, suggesting that water flowing from the fresh concrete might have some components of the cement in suspension that acts as a stabiliser for the soil near the contact. That point is a controversial one, as the initial strength in not known. As it is very difficult to identify cement components in the water, further research is needed to clarify this aspect of the interaction concrete and soil.

The aim of the testing procedure was to simulate, as closely as possible, the general changes in the stress field that a specimen of soil, adjacent to a pile, experiences during pile installation. Certain conditions, such as the axisymmetry, possible arching, the presence of the casing, effects of the drilling tool, use of drilling fluid, effective stress changes and regime, stress levels and heating generated by the large mass of concrete hydrating are not reproducible in such simple experiments and may also affect the process. The results of the performed tests must be viewed critically : there are factors related to the testing procedures that are bound to affect the numerical values obtained, such as :

- Concrete mix : the variation of the pore pressure with time in the concrete is an important boundary condition. In reality this is probably the controlling mechanism of the process. All factors affecting the pressure variation in the concrete (material dependent) should affect the results on such tests (see Chapter 5).

- Scale effect : the amount of fresh concrete related to the volume of soil being tested probably affects the distribution of the pore pressure inside the soil specimen, and will most certainly affect the final equilibrium water content and pore pressure in a laboratory test. It seems that the scale effect has a limited influence : if a small layer (e.g. 4 mm) of concrete is used, certainly the amount of free water will not be enough to create a big change in moisture content and pore water pressure inside the clay specimen and the zone of influence will be restricted. When a certain thickness of concrete is achieved, more concrete will not radically affect the change. The similarity of results between laboratory and field experiments confirms this point of view.

- Stress level : the confining pressure and original pore pressure inside the soil specimens are factors that impose the initial boundary conditions. The new level of confining pressure is another aspect that must be taken into account (due to the effect on the initial pore pressure in the concrete).

- Geometry of the experiment : each experimental geometry will impose a different degree of freedom to the soil specimen (related to volumetric changes) and probably the stress path followed will be affected.

Despite a uniform final pore water pressure and confining pressure, the moisture content distribution inside the test specimen is not uniform. There are several possible explanations for this behaviour :

- Swelling of the soil specimen can occur adjacent to the concrete while it is wet and pore pressures are rising, but once set takes place, shear stresses applied at the concrete/clay interface will reduce reconsolidation in that area. As a result the final state of stress near the contact will be different from that away from contact and the confining pressure will be less than the cell pressure close to the soil/concrete boundary. This mechanism is most unlikely in the field, where the same final difference in moisture content is observed (see Chapter 3).

- The water flowing from the fresh concrete is contaminated with cement components and affects the soil properties near the contact, making the soil structure more stable and preventing reconsolidation.

- The pore water pressure changes are different at the various distances during the process but after re-establishment of final equilibrium (in the laboratory experiment) the various points are under the same effective stress. It is difficult to imagine how these stress path differences might result in a large moisture content change.

- During and after concrete set, suction is developed at the clay/concrete interface, interfering with the general stress equilibrium.

It is most unlikely that a single mechanism is the complete answer to the problem, the possibility is that a combination of factors causes the apparent anomaly.

When a comparison was made of the available studies of soil changes, both in laboratory and in field experiments (with different geometries and installation procedures), Table 1.5 and Section 1.4.3.4., it was noted that the length of soil in which the moisture content increases is always of the order of 50 mm. In the present experiments, that is approximately the length of the sample where there was a large increase in the recorded pore pressure in the hours following concreting, and then a slow reduction towards equalization with the rest of the sample took place. It seems that the zone affected by the rise of pore pressure is the zone were the major changes occurred. If a study of factors possibly affecting the moisture content distribution both in laboratory and field experiments is made, if we eliminate those factors which are not similar we should be left with the factors that control the process.

(a) Similar	(b) Dissimilar
Material	Geometry
Concrete p.w.p behaviour	Effective stress changes and regime
(?)Contamination	Stress levels
of soil with	Remoulding at boundary
cement components	

The pore pressure regime must be the key to the similarity between laboratory and field experiments and it can be suggested that the concrete plays a very important role in the installation related changes in the surrounding soil. Contamination of soil with cement components is a topic that needs further research.

Taking into account all these observations, it is possible to recognize that the general trend observed in the pore water pressure variation in the tests performed represents the mechanistic behaviour of the complex situation near bored piles during and after pile installation.

Considering the results obtained, a completely new image of the pore water pressure regime for the soil surrounding a bored pile emerges. In opposition to the present belief that the pore pressure regime varies from the original level existing before pile installation to a reduced value due to the excavation (negative when no support is used) and then, due to the presence of fresh concrete, starts to increase up to the original level, according to the results obtained in the experimental programme presented in Chapters 5 and 6, Figure 6.11 represents the most likely variation of pore water pressure regime during a bored pile installation.

Immediately after concrete pouring, the pore pressure in the boundary between the pile and soil is equal to the total stress due to the self weight of the fluid concrete, giving a substantial increase in pore water pressure in a narrow zone of the surrounding soil. Due to water migration and hydration of cement the pore pressure reduces in that area and at a certain stage finally equalizes, after some period of time.

The implications of the present findings for future research is that both experimental and theoretical approaches should be used to investigate the effects of the pore water pressure variation on the interaction between pile and soil. Further experimental work is necessary to study the role of the different factors affecting the pore water pressure regime in the fresh concrete and in the surrounding soil.

For the piling practice, concrete mix composition must be considered an important topic not just to avoid integrity problems (Thorburn and Thorburn, 1977, L.C.P.C., 1978, Reese, 1978), but as a possible significant factor affecting the pile behaviour.

6.3. - CONCLUSIONS AND RECOMMENDATIONS

- A technique was established that allowed the study of the pore water pressure regime at different points inside a 100 mm diameter cylindrical clay specimen, due to the contact of fresh concrete on the top of the specimen. A standard triaxial cell, with simple modifications to the base and miniature pressure transducers housed at different points inside the clay specimen, was used.

- After the re-establishment of the confining pressure the measured pore water pressures vary according to their distance from the interface between the soil and concrete. In the proximity of the interface between the soil and concrete the pore pressures are strongly affected by the presence of the fresh concrete in the first hours.

- The pore pressures measured at points close to the contact between the concrete and soil (up to 6 cm) showed that the pore pressures experienced in that area are higher than the rest of the specimen (in the first hours after the contact of the soil with fresh concrete).

- The aim of the testing programme was to simulate the changes in stresses that a specimen of soil, adjacent to a pile, experiences during pile installation. All field conditions are impossible to reproduce in laboratory tests and the numerical results of the tests performed are bound to be influenced by some of the conditions imposed and must be viewed critically. Even so , the general trend in the pore water pressure variation measured represent the mechanistic behaviour of the complex events taking place near real piles during and after installation.

- In opposition to the present belief that the pore water pressure varies from the original level before pile installation to a reduced level due to the excavation and then, due to the presence of the fresh concrete, starts to increase up to the original level, according to the results obtained in Chapters 5 and 6, Figure 6.11 represents the most likely variation of pore water pressure regime during the installation of a bored pile.

Immediately after concrete pouring, the pore pressure at the boundary between the pile and the soil is equal or close to the total stress due to the self weight of the fluid concrete, giving rise to a substantial increase in pore water pressure in a narrow zone of the surrounding soil. Due to the water migration and hydration of cement, the pore pressure reduces in that area and, after some period of time, finally equalizes with the zone less affected.

- It can be speculated that the concrete plays a very important role in installation related changes in the surrounding soil. Apparently the zone where the major variations in pore pressure occur is the same as that of significant moisture content changes.

- An increase in moisture content near the contact and a reduction in the undrained shear strength measured with the cone were recorded 5 months after the contact between fresh concrete and soil. The observed trend in moisture content and shear strength were the same as obtained in samples recovered from the field, near bored piles (Chapter 3), and in the laboratory programme presented in Chapter 2.

- The higher moisture content measured close to the contact was not followed by an equivalent reduction in shear strength, suggesting that water flowing from the fresh concrete might have some components of the cement in suspension that acts as a stabiliser for the soil near the contact. That point is a controversial one and further research is needed to clarify this aspect of the interaction concrete and soil.

- Experimental and theoretical research should be carried out to investigate the effects of the pore pressure variation on the interaction between pile and soil.

- For the piling practice, concrete mix composition must be considered an important topic, not just to avoid integrity problems but as a possible significant factor affecting the pile behaviour. Further experimental work is necessary to study the role of the different factors affecting the pore water pressure regime in the fresh concrete and in the surrounding soil.

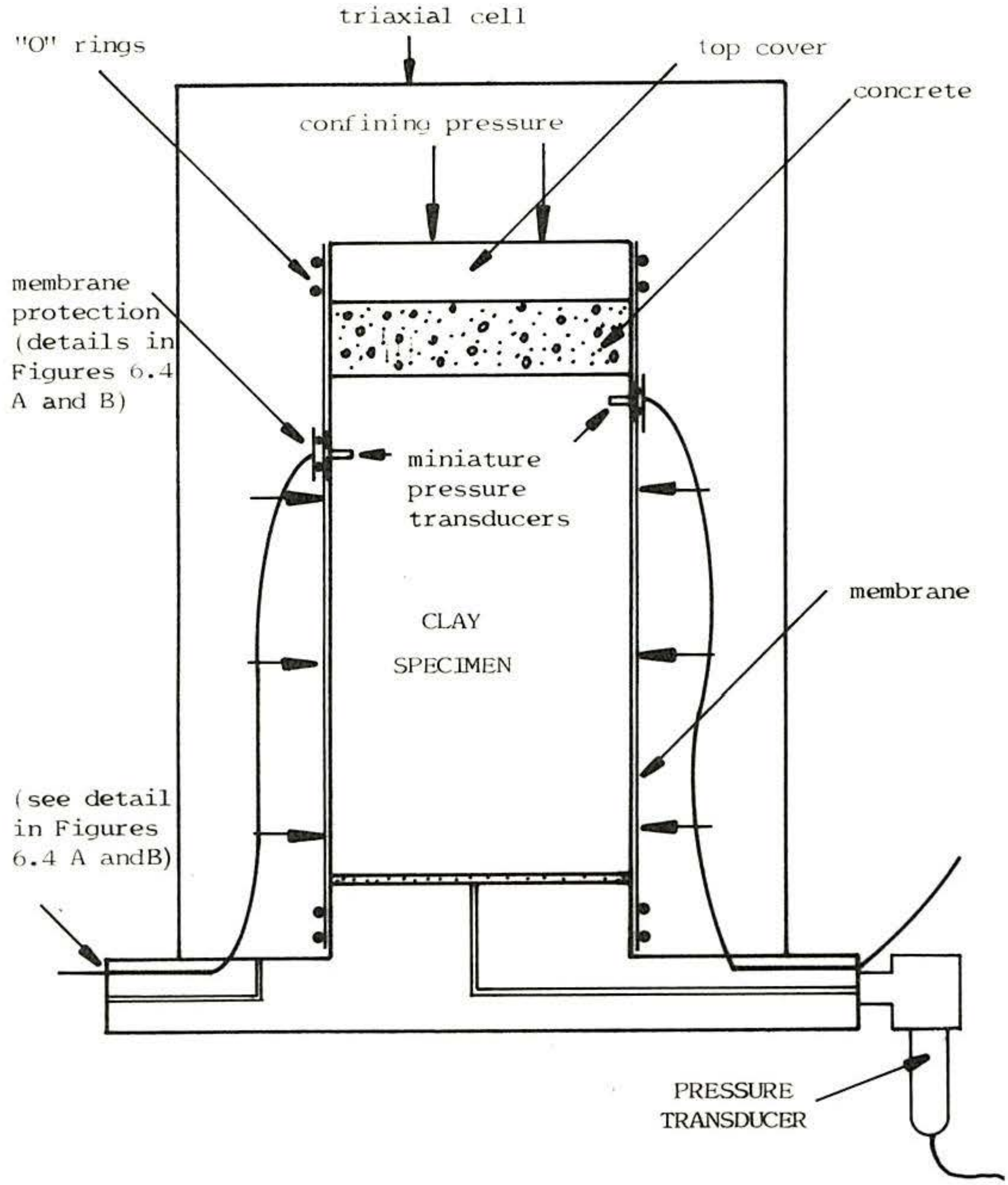
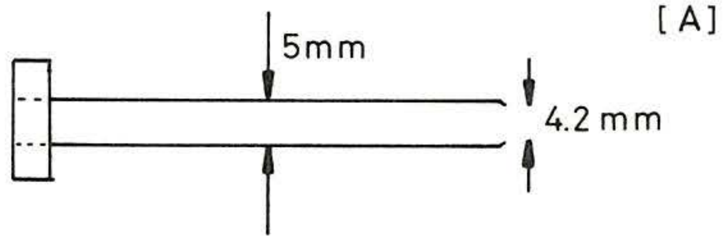


FIGURE 6.1 - GENERAL TEST ARRANGEMENT USED TO MEASURE PORE WATER PRESSURE VARIATION WITH TIME AT DIFFERENT POINTS INSIDE A 100 mm DIAMETER, 200 mm HIGH CLAY SPECIMEN



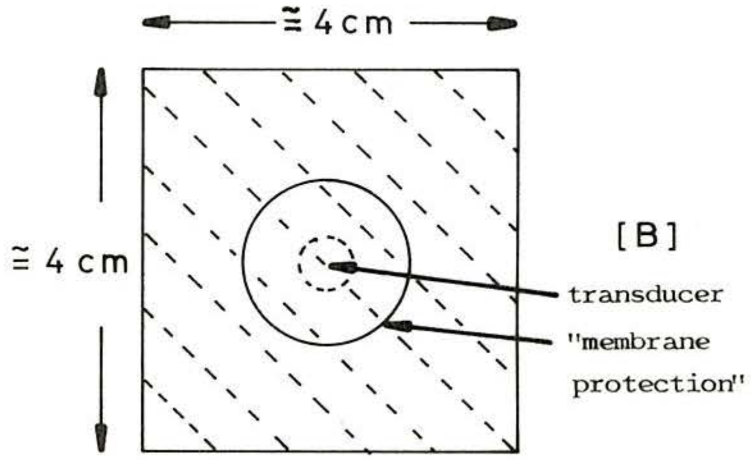
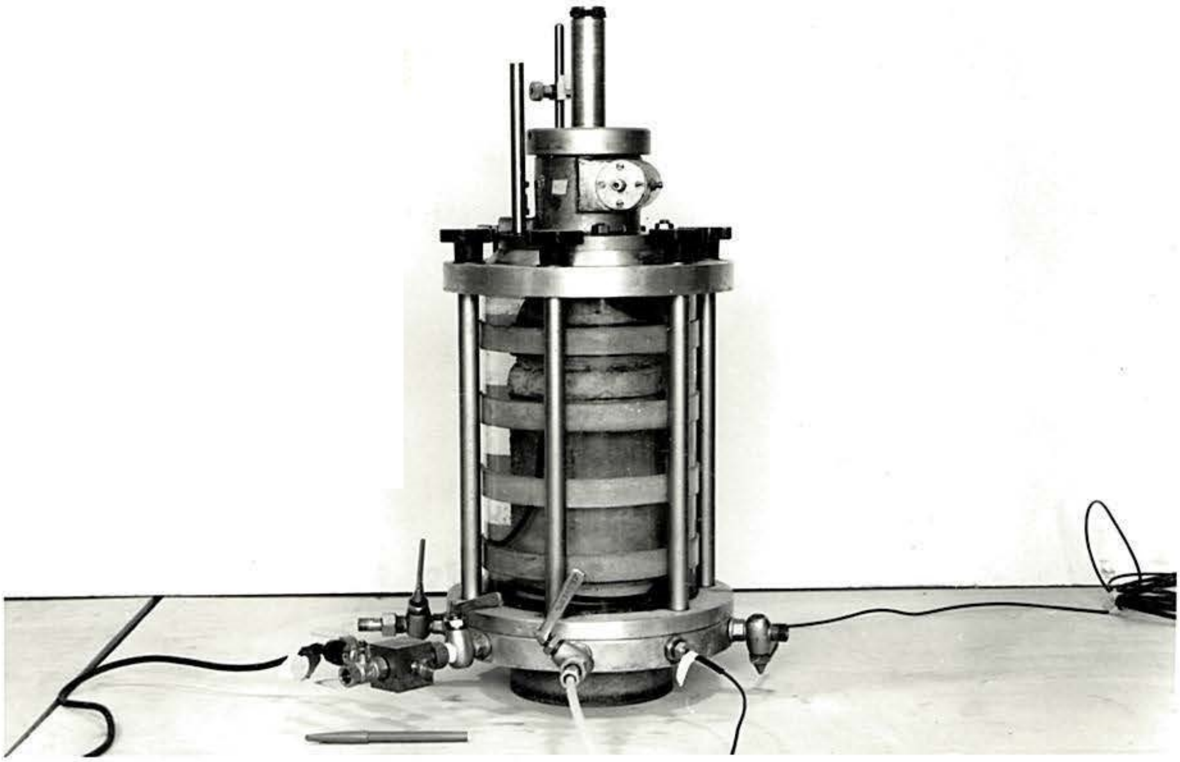


FIGURE 6.2 - DETAILS OF: (A) SAMPLER OF CLAY

(B) AREA COVERED BY SILICONE RUBBER COMPOUND AROUND THE MEMBRANE PROTECTION



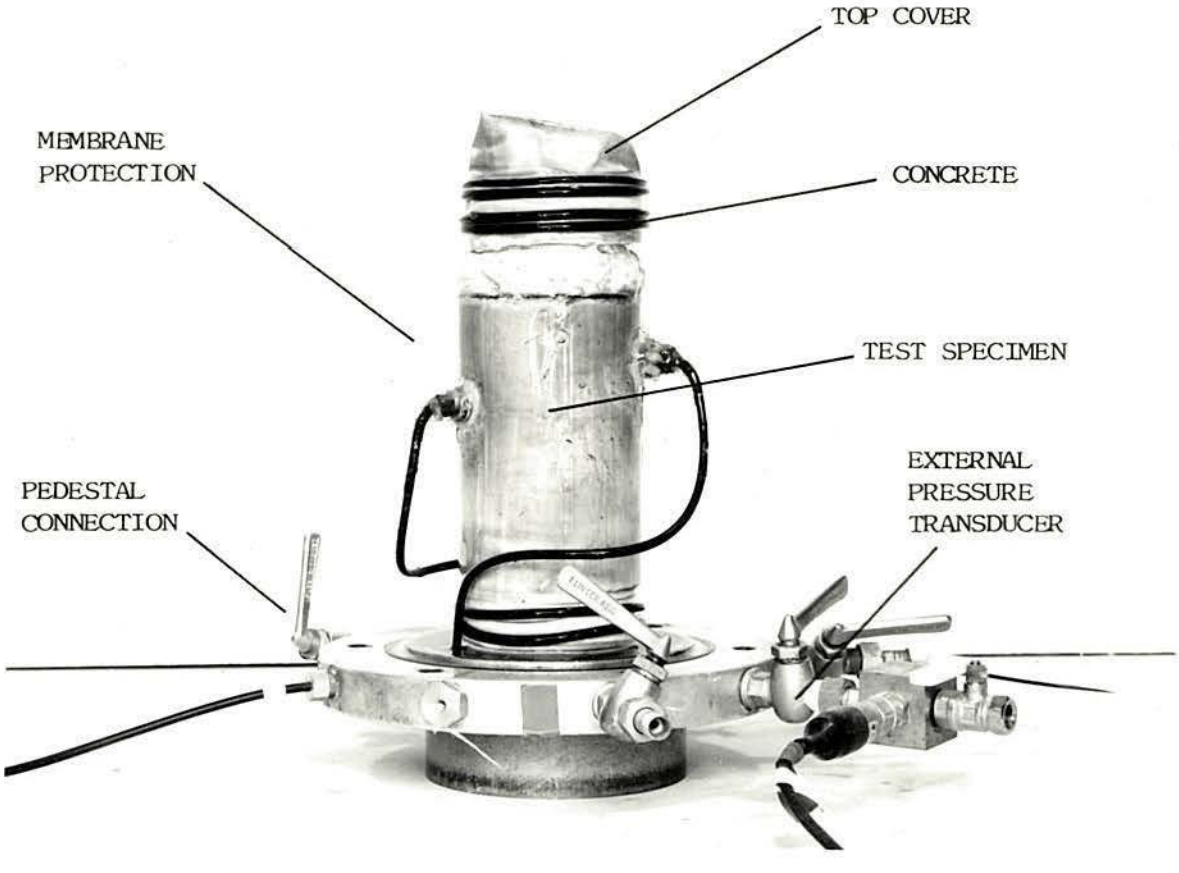
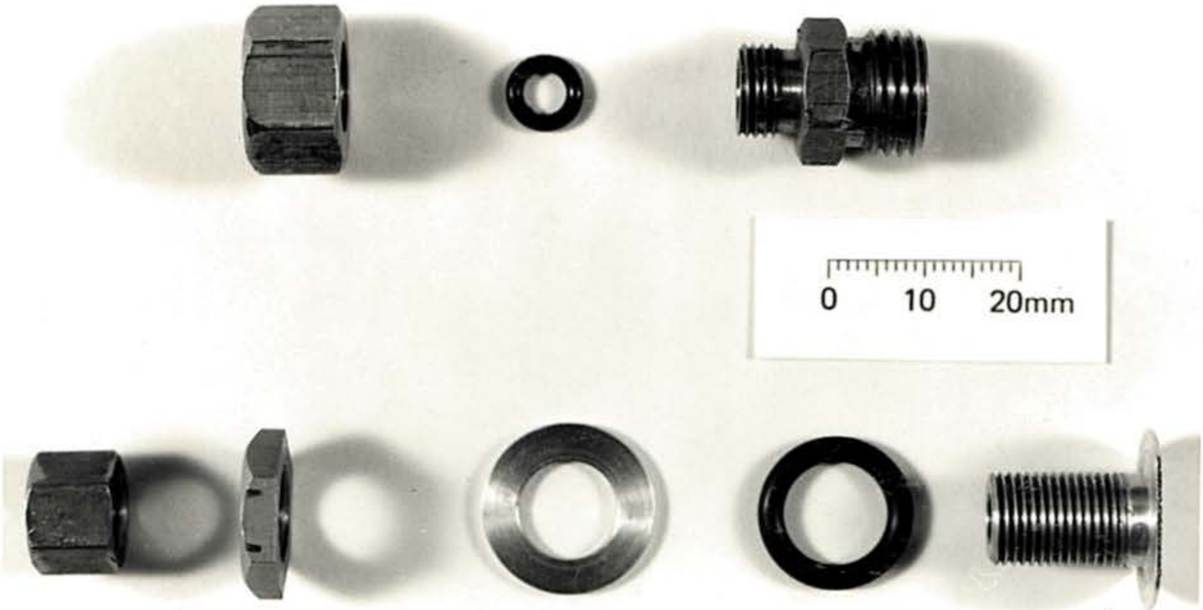


FIGURE 6.3 - PICTURE OF THE GENERAL TEST ARRANGEMENT AND TEST SPECIMEN AFTER 5 MONTHS

FIGURE 6.4 A - PICTURE OF THE MEMBRANE PROTECTION AND SILICONE RUBBER COMPOUND USED AS AN EXTRA PROTECTION

Pedestal connection



Membrane protection

Assembled pedestal connection

Assembled membrane protection

FIGURE 6.4 - DETAILS OF THE MEMBRANE PROTECTION AND PEDESTAL CONNECTION

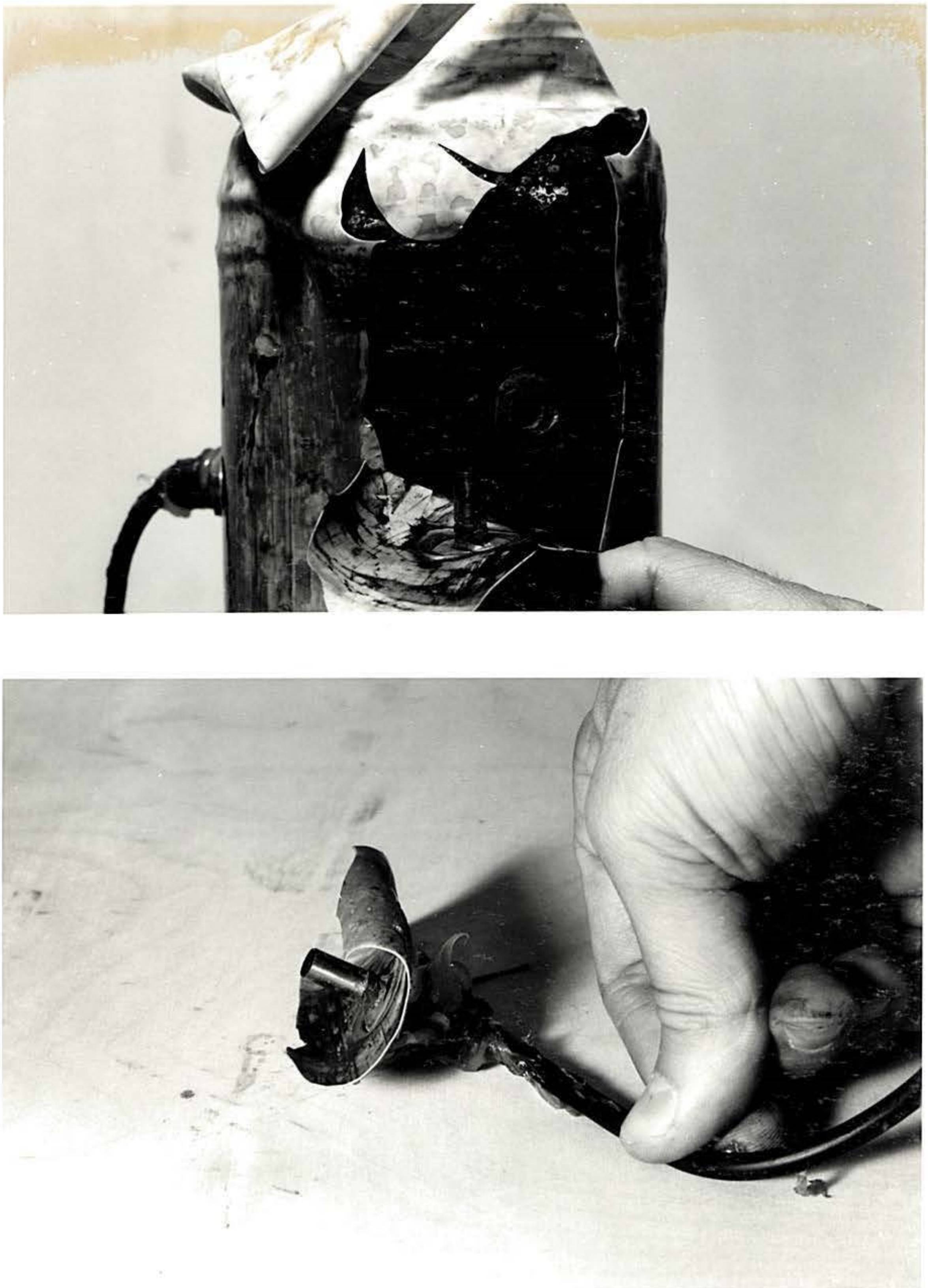


FIGURE 6.5 - CLOSE-UP OF THE MINIATURE TRANSDUCER WITH THE MEMBRANE PROTECTION AND TEST SPECIMEN AFTER 5 MONTHS OF TESTING

FIGURE 6.6 - CLAY / CONCRETE INTERFACE AFTER 5 MONTHS

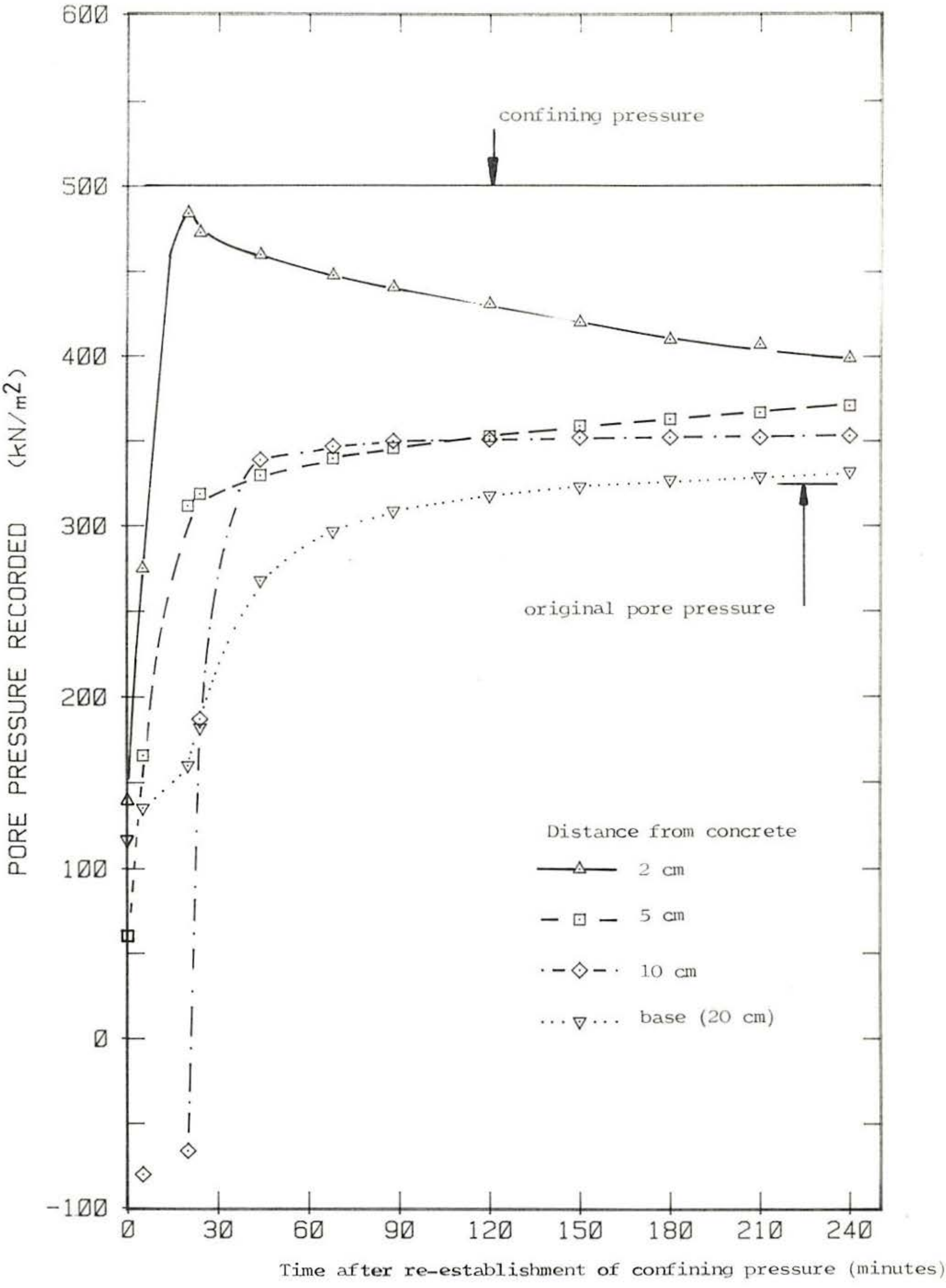


FIGURE 6.7 – TEST SPECIMEN B-3. PORE PRESSURE CHANGES DUE TO CONTACT WITH FRESH CONCRETE. (A) FIRST 240 MINUTES

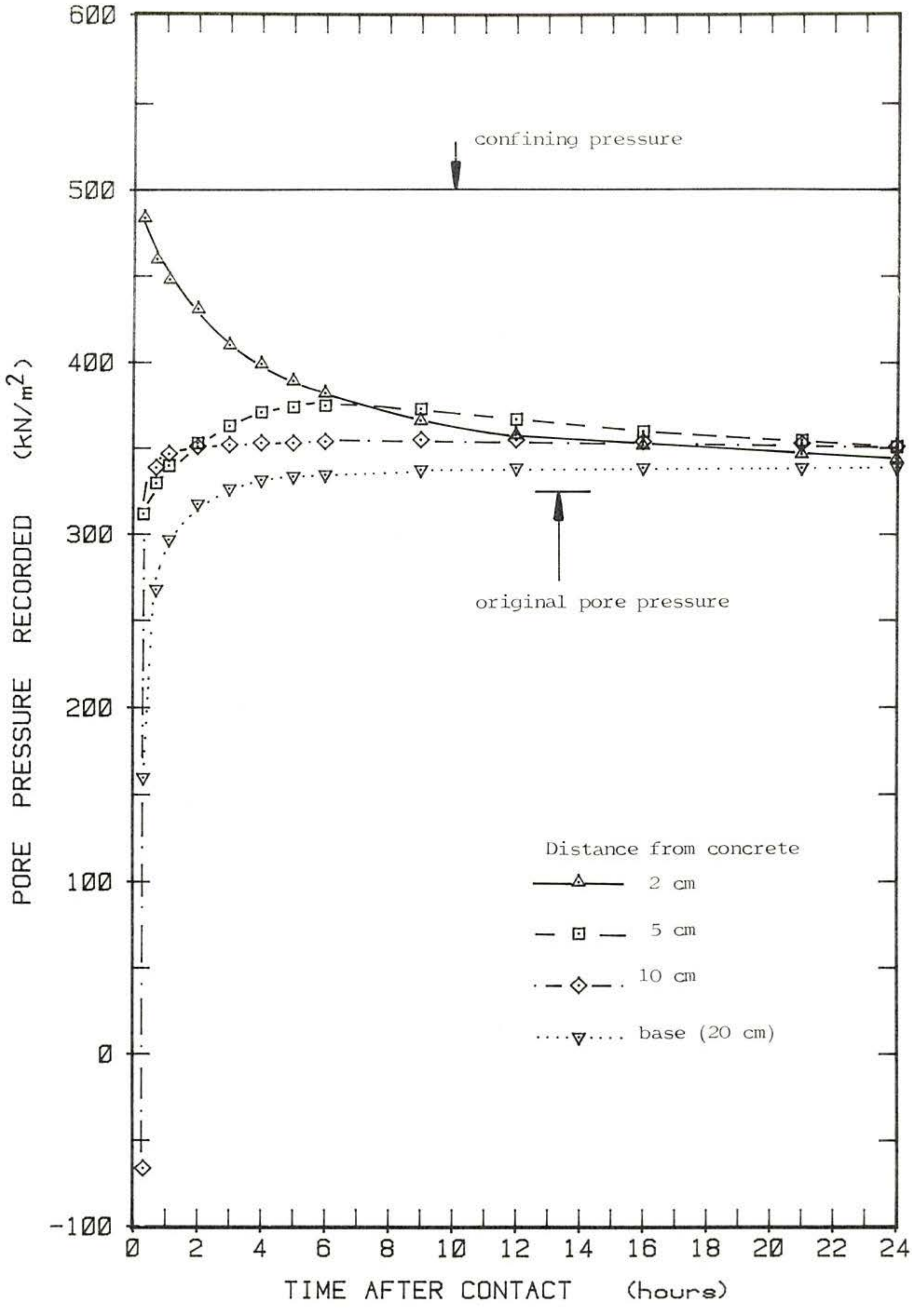


FIGURE 6.7 - (B) FIRST 24 HOURS

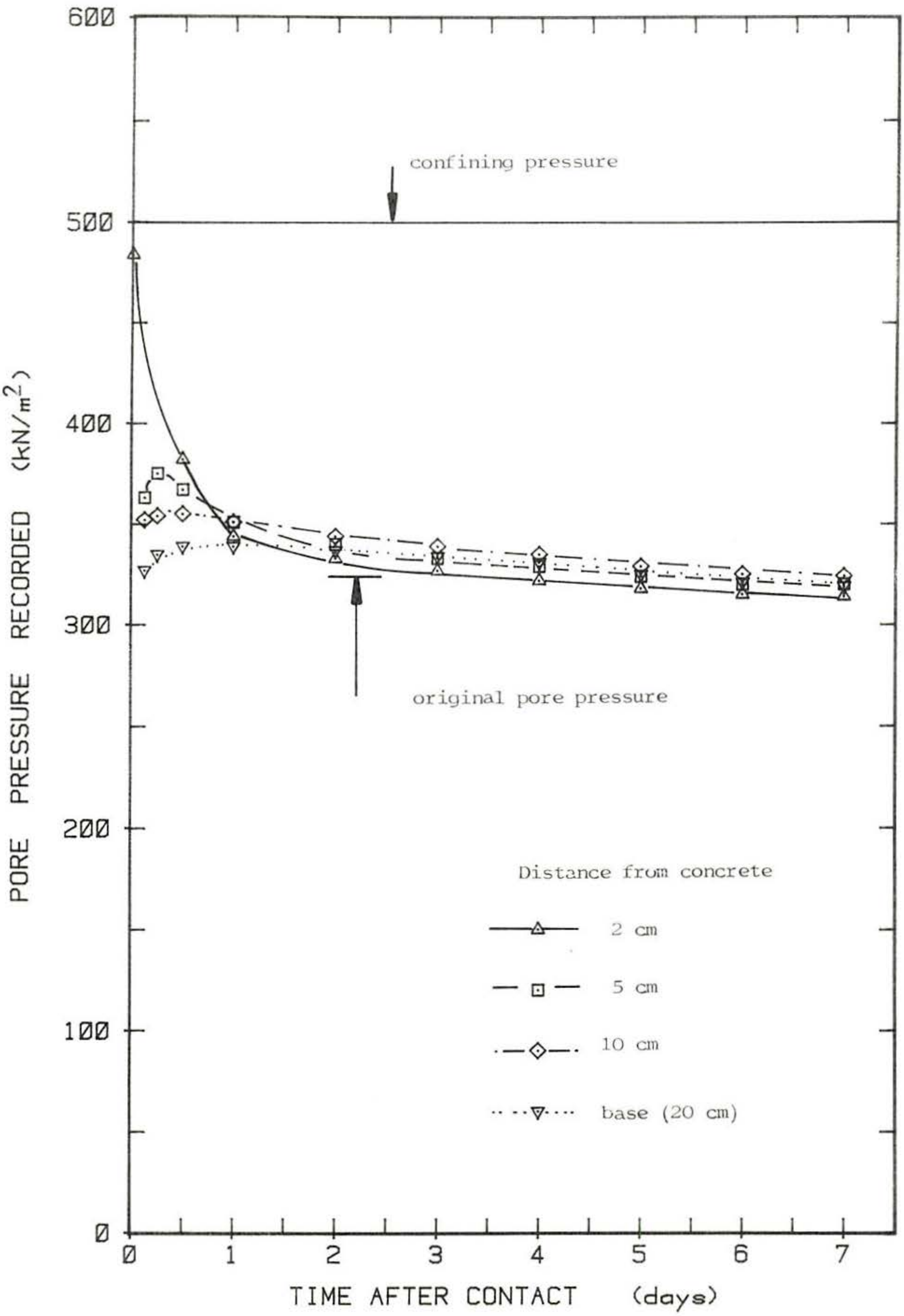


FIGURE 6.7 - (C) FIRST WEEK

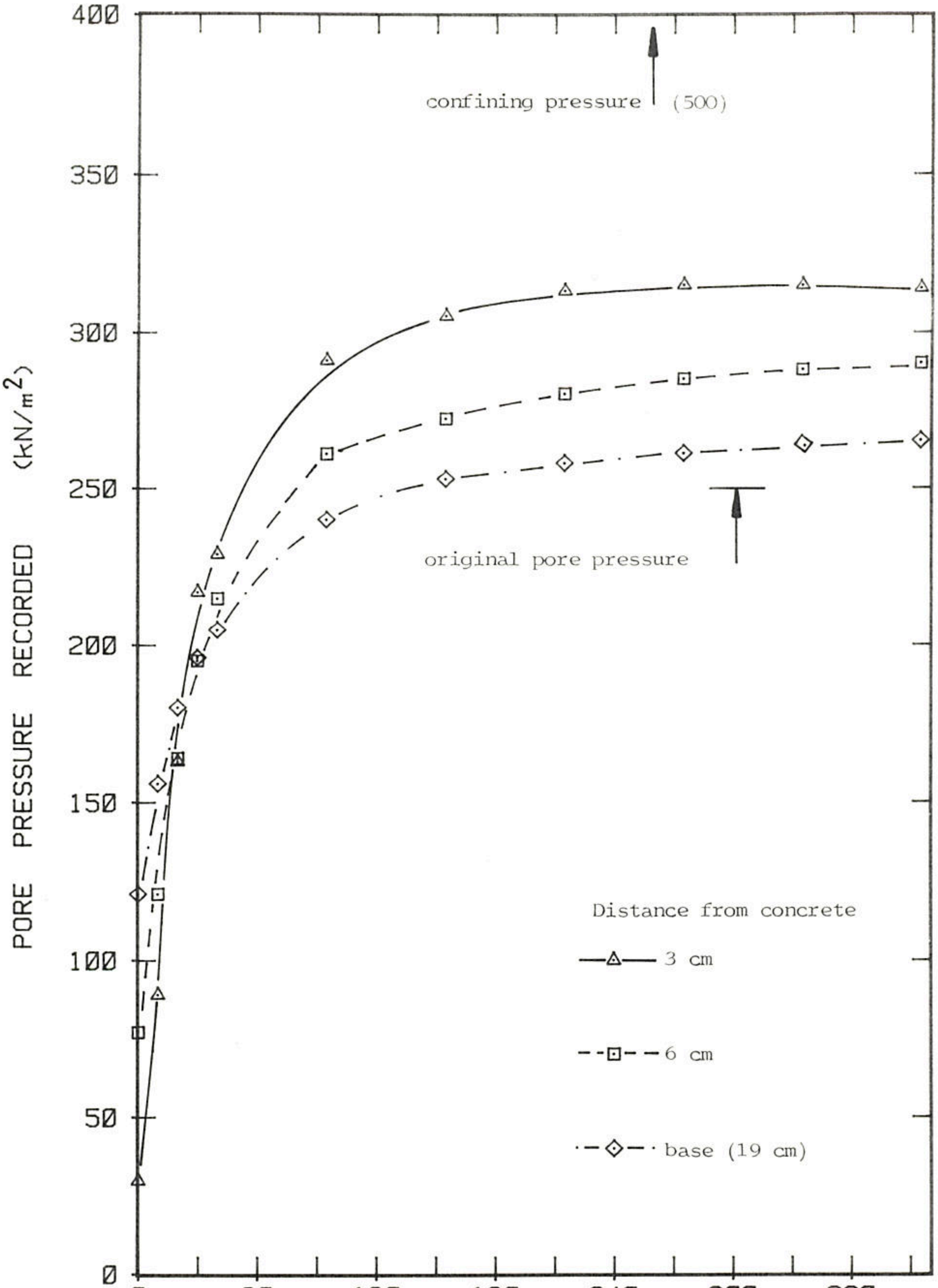


FIGURE 6.8 - TEST SPECIMEN C-2. PORE PRESSURE CHANGES DUE TO CONTACT WITH FRESH CONCRETE. (A) FIRST 400 minutes

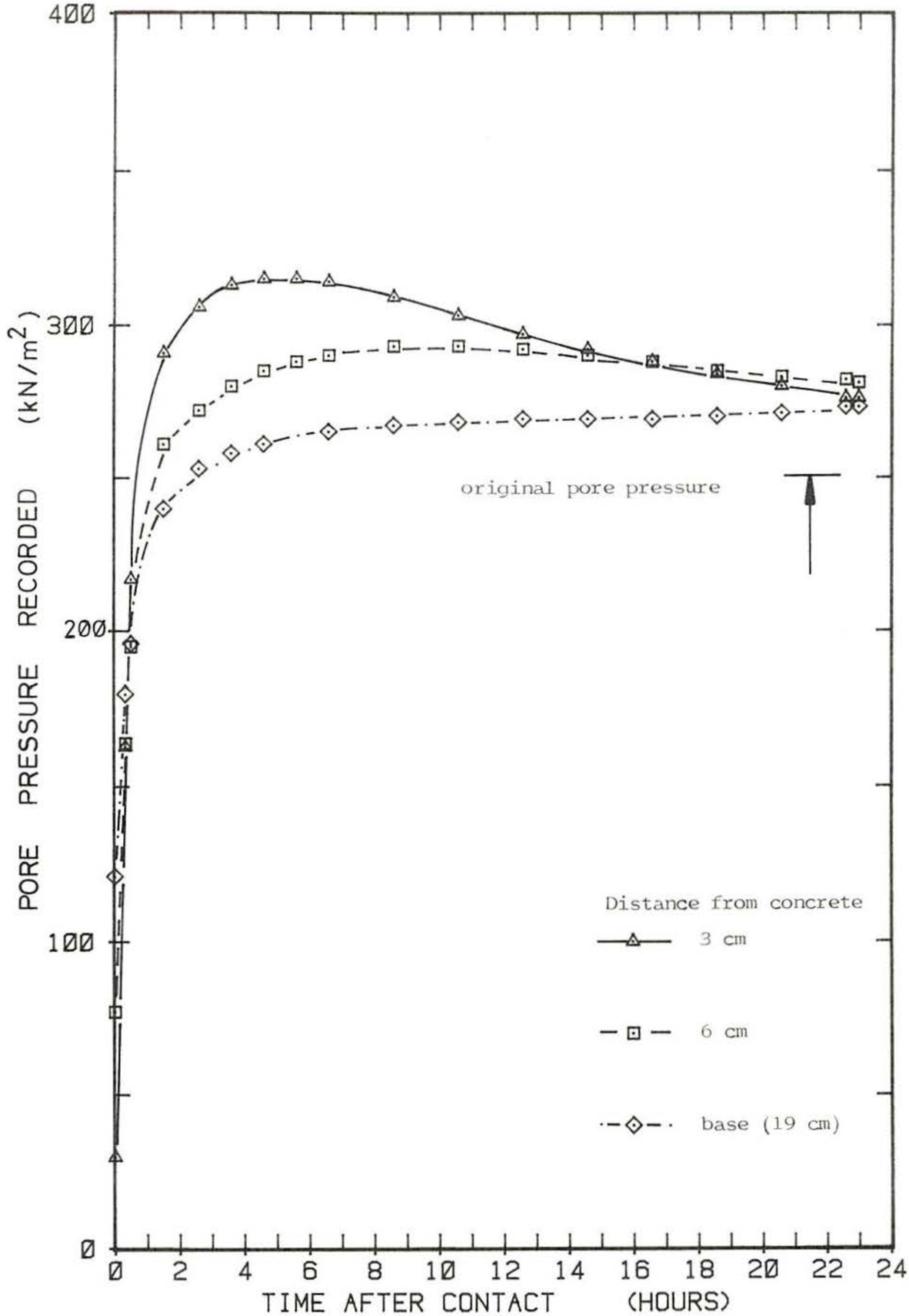


FIGURE 6.8 - (B) FIRST 24 HOURS

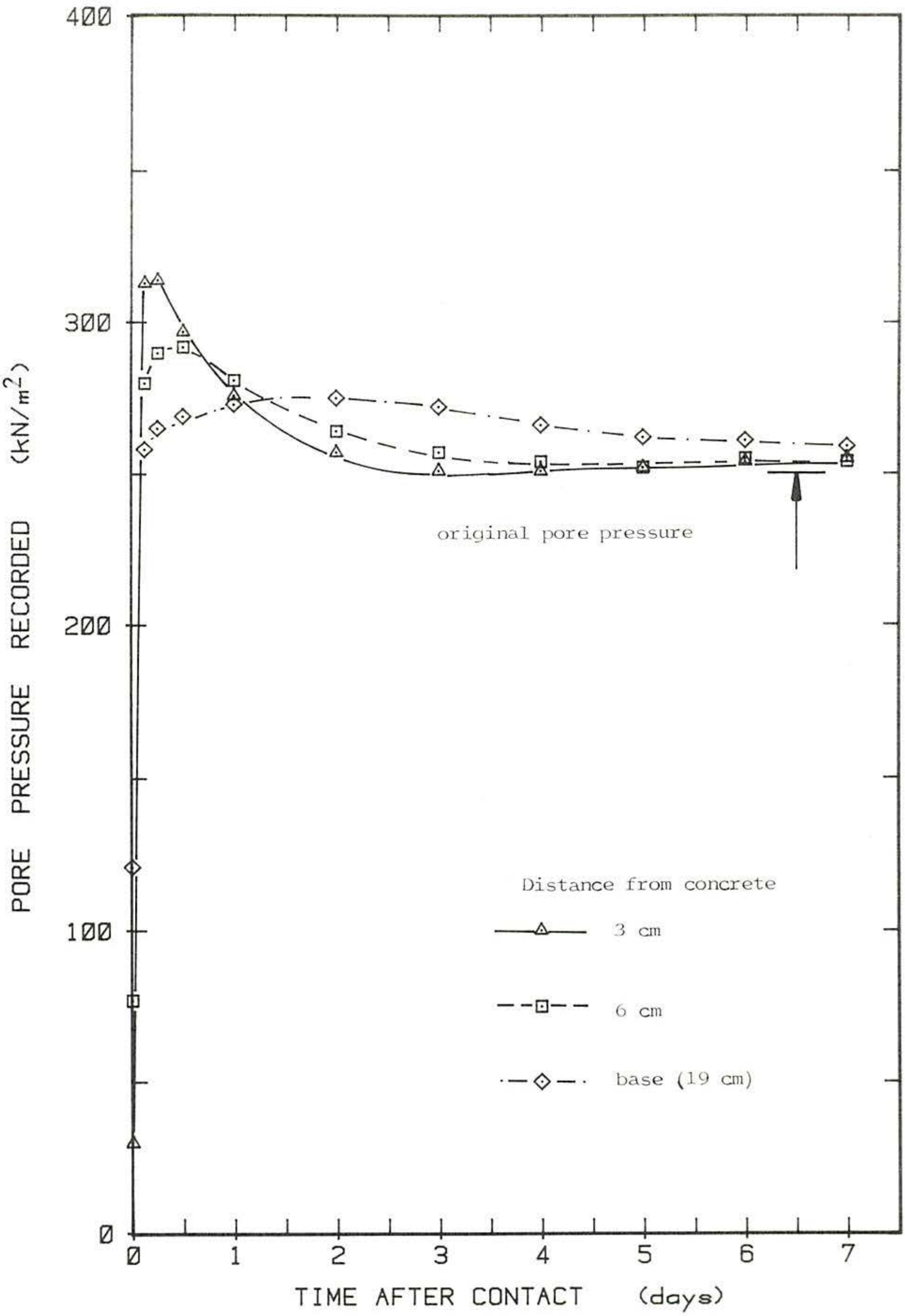


FIGURE 6.8 - (C) FIRST WEEK

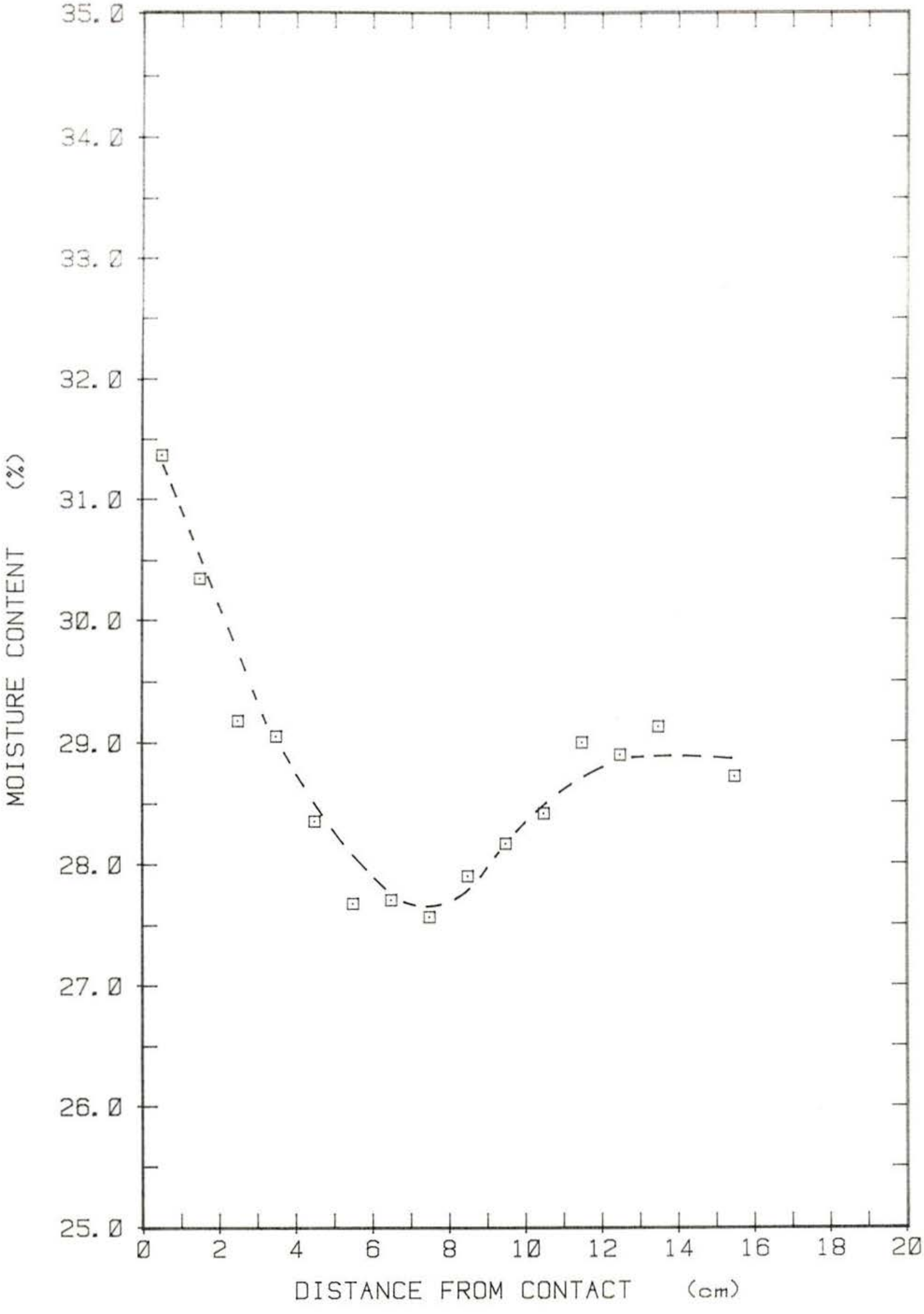


FIGURE 6.9 - MOISTURE CONTENT DISTRIBUTION AFTER 5 MONTHS, TEST SPECIMEN B-3

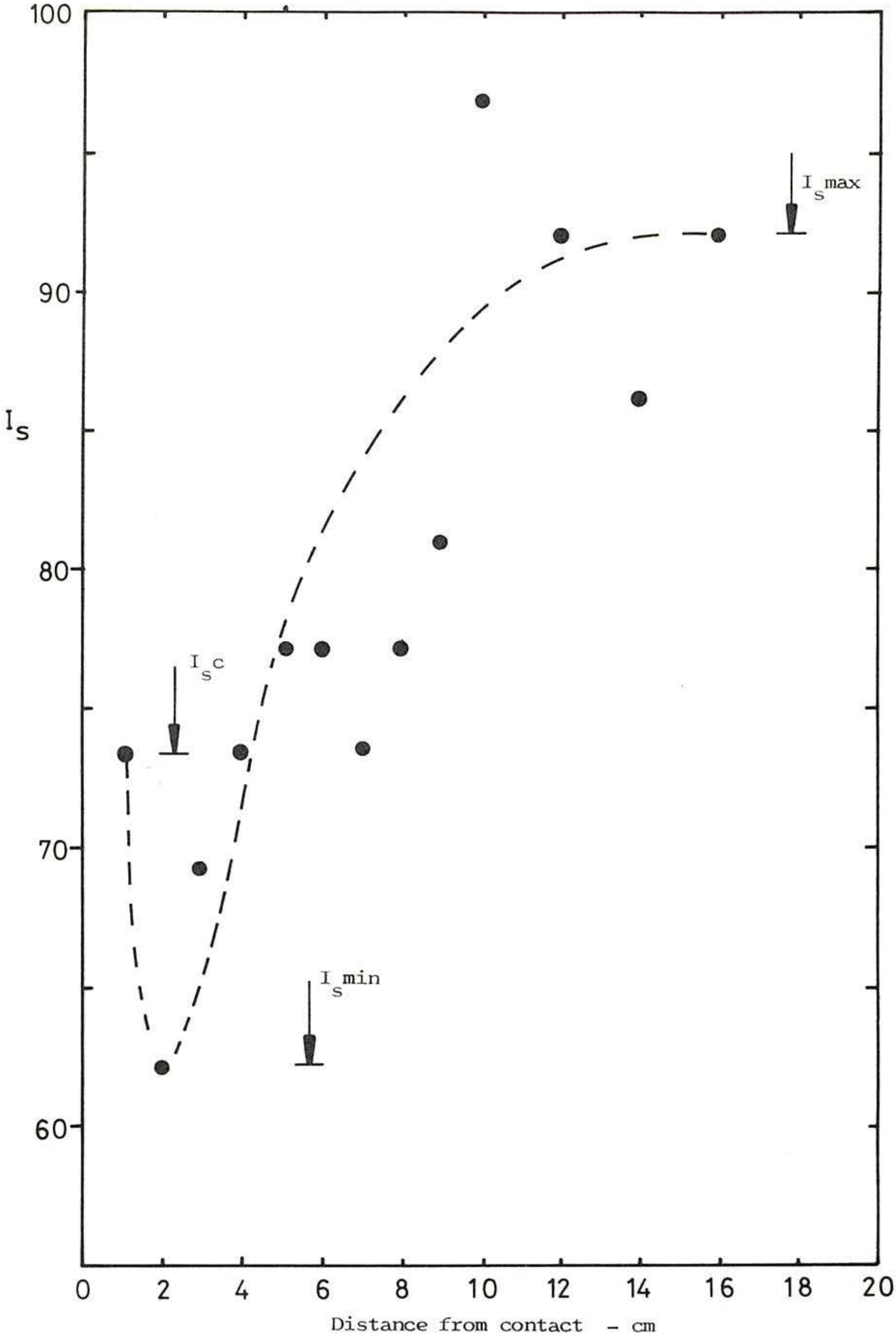


FIGURE 6.10 - "INDEX STRENGTH" DISTRIBUTION AFTER 5 MONTHS IN TEST SPECIMEN B-3

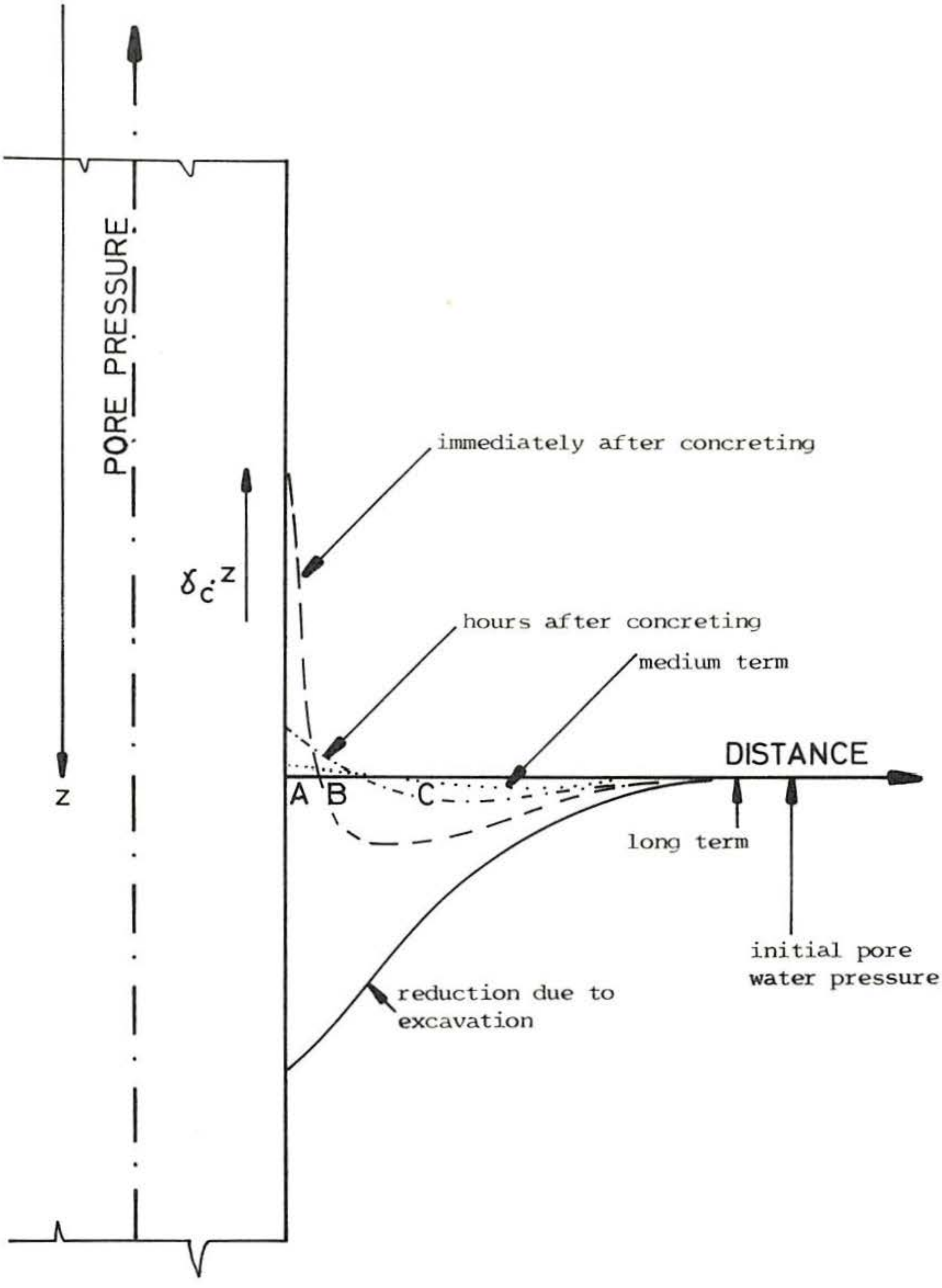


FIGURE 6.11 - SUGGESTED VARIATION OF THE PORE WATER PRESSURE REGIME DURING AND AFTER A BORED PILE INSTALLATION IN THE SURROUNDING SOIL

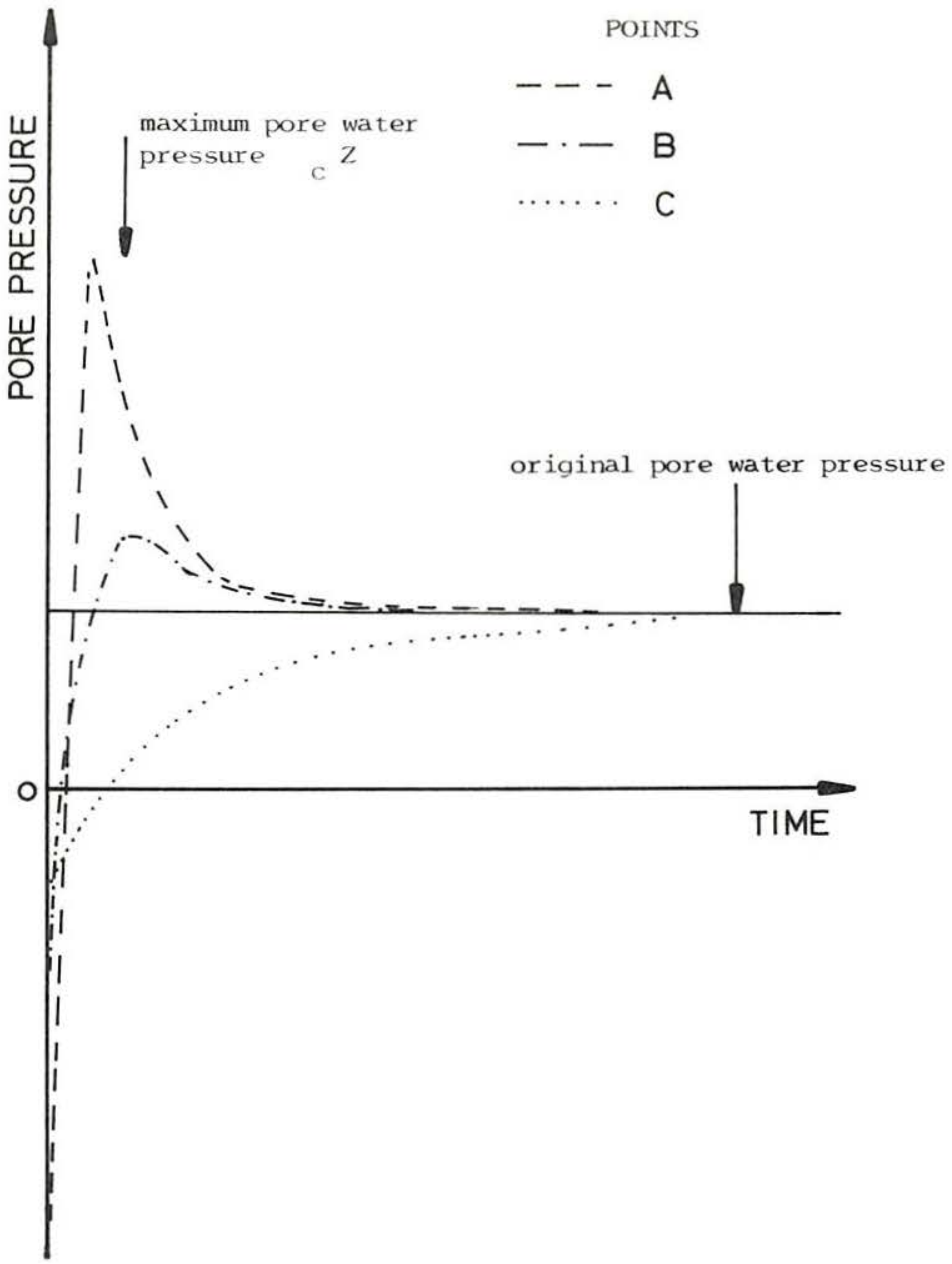


FIGURE 6.11 (cont) - VARIATION WITH TIME OF THE PORE WATER PRESSURE AT DIFFERENT DISTANCES FROM PILE AND SOIL CONTACT

CHAPTER 7 - FIELD EXPERIMENT - MODEL PILE

CHAPTER 7 - FIELD EXPERIMENT - MODEL PILE

7.1. - INTRODUCTION

One of the problems still open to question on the subject of the effects of pile installation on adjacent ground is the stress regime variation. A literature review on the topic was made and presented in Chapter 1. To the author's knowledge there is no reference to measurements of "in situ" horizontal stresses at a site were a pile was built nor any subsequent measurement of some modification in the stresses.

The first problem that arises is the assessment or measurement of the initial horizontal stress . Both laboratory (triaxial test (Bishop and Henkel,1957, Poulos and Davis,1972), capillary pressure suction (Skempton, 1961, Bishop et al, 1965, Som, 1970)) and field tests (self boring pressuremeters (Baguelin et al, 1972, Wroth and Hughes, 1973) flat dilatometers (Marchetti, 1975, 1979), "drive in" total stress cells (Massarch, 1974,1975) and hydraulic fracturing (Bjerrum and Anderson,1972, Penman, 1972, Vaughan, 1972)) can be performed. For practical applications even empirical relationships (Jaky, 1948, Alpan, 1967, Schmidt,1967, Wroth, 1975, Schmertmann, 1975,and Meyerhof, 1976,among others) are widely used. A detailed presentation of all aspects of the determination of "in situ" stresses was made by Wroth (1975) and by Ladd et al (1977).

To give a realistic idea of the difficulty in the estimation of the horizontal "in situ" stresses in stiff clays, Figure 7.1 (from Simpson et al, 1981) is presented. It is clear from the reported values that there is a high dependency on the method of determination. As there is no way of obtaining the "true" value of the horizontal stresses, the assessment of accuracy of the different procedures is very difficult, if not impossible.

The second problem is the reliable measurement of the changes due to pile installation in the vicinity of contact between soil and pile. For this stage the only possibility is the use of continuous field measurements at different positions around the pile.

There is a wide range of approaches to study the problem. Ideally, a comprehensive site investigation should be made, using both laboratory and field tests to study the soil properties and conditions in a restricted area. Following that stage, installation of field equipment of different types to measure the horizontal stresses and piezometric regime should be done at different distances and depths near the position of a real large diameter bored pile to be tested. After the stabilization of the readings, a fully instrumented pile should be built, in order to get the load distribution along the shaft, the load sharing between side and base and pressure in the contact between soil and pile. All instruments used should be monitored during and after pile installation. After stabilization of the readings and allowing time to study subsequent changes, the pile should be loaded up to failure, maintaining all instruments active. After some weeks, the pile should be unloaded and reloaded to study possible changes in behaviour, repeating the procedure in a regular schedule up to the condition of no change in behaviour. An alternative procedure could study the long term behaviour of a bored pile under load.

In order to reconcile the needs of research with available resources, a different line of work was undertaken: a simple experiment was designed.

A new earth pressure cell was designed and build in order to instrument a model pile (15 cm nominal diameter and 7 m long). Data on the "In situ" stresses were obtained using two piezometers and two "drive in" total stress cells. In this Chapter the description of the instrumentation used, together with the New Pressure Cell design, calibration, installation, results obtained and comments about the programme, is presented.

7.2. - EXPERIMENTAL PROGRAMME

To tackle the problem of measurement of the variation of the horizontal stresses due to pile installation it was necessary to design equipment able to simulate pile installation , simple enough to be built by the Civil Engineering Department workshop, strong enough to survive installation, accurate enough to register the possible changes and stable enough to maintain the possibility of reading up to 6 months after installation. It was decided to design a cell with the same diameter as the model pile shaft, flexible to allow for the irregularities of the walls of the excavations, with the possibility of imposing a certain initial pressure (to simulate the action of the fresh concrete). Two cells were designed (one pneumatic, the other using an internal pressure transducer) but just one (with the transducer) was built and used.

It was decided to make the experiments on the University Campus, near the building that houses the Civil Engineering Department laboratories (BB) for easy use of the available facilities.

The experimental program was designed as follows:

- installation of two piezometers (one Casagrande and one Pneumatic) for the measurement of the pore pressure regime. The use of different equipment was decided in order to make possible a comparison of behaviour (to check the reliability of the results) and to have a more flexible system of measurement (adding the advantages of two methods);

- installation of two "drive in" total stress cells. After considering the possibility of using a Camkometer (Cambridge Self Boring Pressuremeter from Cambridge In Situ) it was decided that a better choice was the use of equipment that could be left in the ground to study the long term behaviour. Another important consideration was the high price of the use of the Camkometer, when compared with the cost of buying two "drive in" cells. Two cells were used to measure the horizontal total stresses in orthogonal directions in the ground, at the same depth;

- a deep sounding (Dutch cone) test was performed by Fugro;
- U 100 samplers were taking, using a light percussion drilling rig.

Figures 7.2. and 7.3 shows the location of the site, the distribution of the experiments in plan, the results of the site investigation programme (1965), identification of sub-soil layers and Cone test results, from Fugro (July, 1982).

7.3. - THE PIEZOMETERS

7.3.1. - TYPE

Two piezometers were used to monitor the pore pressure regime in the area of the field test. One was a standard Casagrande, Standpipe piezometer, the other was a standard pneumatic piezometer, both supplied by Geotechnical Instruments Ltd. The reason for the choice of different systems was that results could be checked, rapid changes could be measured, and the advantages of both systems added.

7.3.2. - READOUT

The readings on the Standpipe piezometer were taken using a Water Level Meter (Dipmeter) commercially available from Soil Instruments Ltd.

Figure 7.4 shows the diagram of the pneumatic piezometer measuring system used. A nitrogen air free bottle was used as a pressure supply. The original system was built with a mercury manometer but, in order to use the same system to read the "drive in" cell, a Bourdon Gauge was also included.

7.3.3. - CALIBRATION

The pneumatic piezometer was calibrated using a shaft of water 3.5 m deep in the Hydraulics Laboratory. The piezometer was lowered to different depths below water level and measurements were taken at intervals. It was decided, for simplicity, to use the reading of the pressure obtained when the diaphragm closes (after being open by a pressure greater than the water pressure). This procedure avoided the need of using a flow-meter (the alternative procedure). The results of the calibration are presented in graphical form in Figure 7.5.

7.3.4. - INSTALLATION

The installation procedure used for both piezometers was the same and can be described as follows:

A Portable Minuteman(manufactured by Mobile Drilling Co, USA) was used to drill a 3" (75 mm) diameter, 6.0 m depth hole. Due to the nature of the soil no problem of stability of the hole was observed during installation. PVC Standpipes were used to check the length of the hole. A bed of uniform medium sand was formed by pouring sand and water through the PVC Standpipe to establish a base for the piezometer tip. The sand was compacted with another PVC Standpipe with the end closed. The piezometer tip, saturated in water, was coupled to the Standpipe (in the case of the pneumatic piezometer, special installing rods were used at this stage), and lowered down the hole until the tip reached the sand filter. Further sand was poured through a PVC Standpipe and tamped, until enough material was used to cover the tip by at least 300 mm. (At this stage the installing rods were withdrawn in the case of the pneumatic piezometer). A plug of bentonite made by using stiff bentonite pellets, was formed (approximately 300 mm). The remainder of the hole was backfilled with cement-bentonite grout. In both piezometers a protective standard cover was concreted at the top of the borehole. The plastic tubing of the pneumatic piezometer was connected to Legris fittings in order to make the connections between the readout and the piezometer easy. Figure 7.6 shows the diagram of the piezometer in the ground.

On the day after the installation of the Casagrande Piezometer the PVC Standpipe was filled with water up to ground level.

7.4. - THE "DRIVE IN" TOTAL STRESS CELL

7.4.1. - THE CELL

The "drive in" total stress cell (also called "push-in" cell) was introduced by Massarch (1974, 1975) who developed a spade-like cell using the well-known Glotzl measuring system. The thin cell is pushed down into the ground. When used in soft material the cell is protected by a steel casing. In stiff soils a hole must be made up to 30 cm above the intended measuring level and the cell is pushed from that position.

The thin steel cell, which is filled with oil, is connected to an air operated valve and a pump (Figure 7.7). By slowly increasing the air pressure, the sensitive membrane in the valve will open (at a pressure which slightly exceeds the oil pressure in the cell). According to Massarch (1975) the maximum deflection of the membrane is about 5 mm, corresponding to about 1/30 000 of the diameter of the cell. The volume change of the cell itself is considered to be negligible and does not affect the readings.

The cells used in the program were bought from Soil Instruments Limited (Figure 7.11) and were made according to instructions supplied by the Building Research Establishment.

Values obtained with similar "drive in" cells in London Clay are presented in Figure 7.8b (from Tedd and Charles, 1981) compared with the results of the Camkometer (self boring load cell and pressuremeter) and laboratory study of capillary pressure(Burland and Maswoswe,1982). The dissipation of the pore pressure generated by pushing the cell is shown in Figure 7.8a.

The main problem with the use of these cells is the possible effect of the displacement of soil during installation. The problem of increase of pore pressure due to installation can be overcome by leaving the cell until stabilization of readings is achieved.

7.3.2. - THE READ-OUT

The same read-out employed in the reading of the pneumatic piezometer was used for the "drive in" cells (Figure 7.4) The only difference was the use of a Bourdon gauge (0 to 12 bars) as a pressure measuring device.

7.4.3. - CALIBRATION

The calibration of the "drive in" cells proved to be, with the installation, a major problem to be solved in the present research. When the cells came from the manufacturers (Soil Instruments Ltd) no reference to the calibration was made. A contact was made and it was learned that no calibration was made on such equipment for lack of facilities. As the equipment was built in accordance with Building Research Establishment design , a contact made (Penman, 1982) brought to light that at that institution the calibration was made using a large triaxial cell full of water. When a container filled with clay was used the results obtained were inconclusive, the readings always being higher than the applied confining pressure.

The largest triaxial cell available at the University of Surrey laboratory is for a 15 cm diameter sample, not big enough to house the complete "drive in" cell. The problem was solved by using a large chamber recently built in the laboratory (Clayton and Dikran, 1982, Dikran, in preparation).

A cross section of the chamber is shown in Figure 7.9a. Vertical stresses can be applied to the specimen (435 mm diameter, 650 mm high) by using an air-water pressure system acting on the diaphragm located in the base of the chamber. Lateral stresses are applied by changing the pressure of the water around the specimen, which is enclosed in a 1.2 mm thick latex rubber membrane. Both vertical and horizontal stresses are measured using electrical pressure transducers with digital readout units.

The soil used for the calibration was subangular uniform fine quartz sand (Leighton Buzzard, 100-200 mesh), Figure 7.9b.

Specimen preparation was achieved by hand placing the sand inside a former, with the rubber membrane pulled up inside it and overlapped around the top of the former with two jubilee clips acting on the bottom. After a trial when a compacted specimen proved to give noticeably unreliable results (probably due to the arching of the sand around the cell to be calibrated) it was decided to build a loose sample. During the sample preparation the cell was positioned at mid- height (Figure 7.9a.) of the sand specimen and the tubes taken through the top platen and top cover. When the desired height of the specimen was achieved the rubber membrane was fixed to the top platen by means of two jubilee clips. Before removing the former, a suction of about 20 kN/m^2 was applied to the bottom of the specimen using a vacuum pump. The space around the specimen was filled with water to the top of the specimen. The top cover was placed in position and bolted both to the outer cylinder and to the top cover (the top cover was sealed against the top platen of the specimen).

The chamber used for calibration allows any independent variation of vertical and horizontal stresses. It was decided to use an isotropic state of stresses as a standard calibration procedure. After finishing the calibration, some measurements were made to study the effect of vertical stress changes. It was found that, provided the horizontal stress does not change, there is no effect of the vertical stresses on the readings.

The results obtained are presented in Figures 7.10a and b.

It is evident from the results obtained that the "drive in" cells over read the confining pressures.

7.4.4. - INSTALLATION

The installation of the "drive in" cell is an easy task when testing soft materials. The cell can be pushed down into the ground from the surface; during penetration the cell is usually protected by a steel casing that can be withdrawn while the cell is pushed down the last 300 mm to reach the intended measuring level. When the soils to be tested are stiff or compact, a borehole is necessary. From the bottom of the borehole the cell must be pushed to the desired level (usually 300/400 mm). In a material like London Clay a reaction of about 1.2 tonne was reported necessary (Tedd and Charles, 1981).

A Portable Minutman was used to drill a 4"(100mm) diameter 5.5 m depth hole. An adaptor was designed and built in order to connect the cells to 32 mm square SPT rods (Figure 7.12).

When an hydraulic system (like the one used for the deep sounding) is available,the cell can be pushed easily from the bottom of the borehole to the testing level. In our work it was necessary to use kentledge and an hydraulic jack in order to achieve this objective. The system used is shown in Figure 7.12. It proved to be difficult to build and make it work properly due to problems of levelling, getting the necessary reaction and many others. The first cell (Number 500)was driven using just water as reaction (filling three 250 litre tanks plus approximately 250 kg of beams). The second cell (501) showed a different behaviour during installation. It was necessary to fill two tanks with sand and water, and an additional 500 kg of concrete blocks and 300 kg of steel weights were necessary (after an unsuccessful trial using the same amount of material as the first experiment). During this part of the installation it was suspected that a claystone layer was responsible for the high strength, but later results showed no significant difference in behaviour between the cells. The total load used to drive the two cells was therefore 1000 kg and 2000 kg approximately.

The SPT rods were left in the hole to allow future recovery of the cells. The holes were filled with a bentonite slurry (Fulbent) and protected at ground level by a U 100 sampler (Figure 7.13). The ends of the plastic tubes were fitted with Legris pipe couplings, in order to ease the task of connecting the readout system.

The installation of the first cell took one week, the second 10 days, considering only the time spent on the field work.

7.5. - THE NEW PRESSURE CELL

7.5.1. - CONCEPT AND DESIGN

The basic concept used to develop the design of the "New Pressure Cell" was a field simulation of a bored pile installation. It was decided that the characteristics of the measuring device should include: - possibility of imposing initial stress (to simulate the fresh concrete on the walls of the borehole); - flexibility to allow for the possible irregularities on the walls; - simplicity in order to be economic and possible to build in the Department workshop;-strength and durability to resist the installation and contact with concrete and soil; - accuracy to register even small changes in pressure and stability to allow long term readings.

After a review of the available earth pressure cells principles and equipment,it was decided that the cell should have the same size as the model pile for simplicity and to overcome the basic problem of all devices (change of the state of stresses being measured by imposing a different condition). Basically the same principle was used for the design of two cells. Both cells had the size of the projected model pile, both could be inflated and for manufacturing reasons had the same shape and basic components (internal cylinder and flexible membrane). The main difference was the measuring system.

The cell refered to as the "New Pressure Cell - GP1" (Grout Pneumatic) model was, basically, a rigid cylinder made in duraluminium (100 mmϕ,300 mm long) with two external membranes with pneumatic type of sensors located between the two membranes (Figure 7.14). The idea of proper installation was to lower the cell inside a hole (150 mm in diameter), with the bottom already full of concrete (1m) to simulate a pile construction, apply grout (cement-bentonite mix) through a flexible tube into the space between the rigid cylinder and the inner membrane, up to a desired level (to simulate the pressure of the fresh concrete). The installation of the rest of the pile should continue up to the completion of the pile. Through the tubing coming to the surface the horizontal pressure acting on the device could be measured at any time. The advantages of such cell are that the stiffness is of the same order of magnitude of a pile (and can be changed by altering the cement-bentonite grout), the materials to be used are easy and cheaply found, operating techniques are simple, the device is robust. The major disadvantages are the necessity to master rubber handling techniques in order to produce a reliable device and the problems of grout mix design and installation.

The "New Pressure Cell" ST1 (Solenoid Transducer) model,(Figures 7.15a to 7.17),is a rigid cylinder made in duraluminium (100 mm in diameter, 300 mm long, 10 mm thickness) with top and bottom (10 mm thick plates) with an external rubber membrane. The membrane was specially made by Denber Trading Co, using the specification of Building Research Establishment: pure rubber latex of low protein content, and containing no coagulant (Ca Cl O_3), double leached. The membrane was made by dipping a mould in a tank containing the rubber solution. The mould was cured in the curing solution. After initial problems of irregularities on the surface (causing problems of leakage) and delays in production, a satisfactory product was achieved and delivered (ϕ=140mm,h=300mm).The membrane can be inflated by pumping deaired water in the space between the cylinder and the membrane, it being possible to open and close the inlet valve at the cell using a general purpose solenoid valve Type 200 (Burkert Contromatic

Ltd, Stroud, UK). The pressure can be measured using a transducer with built in amplifier, type PDCR 10/A/S, supplied by Druck Limited, Leicester, UK, giving a 10 Volts output under 300 kN/m^2 pressure. For protection during installation, rubber disks 5 mm thick were used on both ends of the cell and the complete apparatus was covered by two 6 inches standard triaxial rubber membranes (Figures 7.16. and 7.17). After assembly the cell was connected to a 1 inch gas barrel, 6 m long, for installation (Figure 7.24).

During the design of the cell, the use of existing components was a priority. The duraluminium cylinder, the solenoid valve, and the pressure transducer are items commercially available. The rubber membrane was specially produced in order to solve the problem of water permeability on the long period of test, necessary strength and proper size. Figure 7.15b shows the basic components of the cell.

During the assembly of the components it was learned that leakage was the big problem to be faced , apart from deairing. "O" rings and special glues (rubber-metal) were used. The problem of deairing the system was solved by mounting the cell with deaired water being flushed through the solenoid valve and overflowing by the top of the cell.

As the "New Pressure Cell - GP 1" (Grout Pneumatic) was not built, the "New Pressure Cell" - ST 1 (Solenoid Transducer) will be referred to in the remainder of the text as the "New Pressure Cell".

7.5.2. - CALIBRATION

The calibration of the New Pressure Cell involved the following steps:

- calibration of the pressure transducer.
- study of the stability of the transducer in long term measurement.
- calibration of the cell assembly against water, and "creep" test.
- study of the cell behaviour when tested in a calibration chamber with soil.

Each step will be described with the results obtained.

Calibration and stability of the pressure transducer

The pressure transducer was calibrated using a Budenberg Dead-weight pressure gauge tester, with an overall accuracy of the tester better than $\pm$ 0.05 % of the pressure being measured (according to the supplier of the equipment). The results obtained are presented in Figure 7.18b.

To check the stability of the transducer, after the calibration, a pressure of 250 kN/m^2 was applied and maintained for 14 days. The results obtained are presented in Table 7.1. The variation of the recorded pressure was less than 0.5 kN/m^2.

The performance of the pressure transducer during the "warming up" period is presented in Figure 7.18c. No significant change was observed in the readings.

From the results obtained it can be concluded that the performance of the pressure transducer (linearity, stability under constant load and "warming up" effect) is remarkable.

Calibration of the cell assembly against water and "creep" test

To check the ability of the designed New Pressure Cell to respond to applied external pressures and for leakeages, the cell was tested inside a standard triaxial cell , used for 6 inches specimens (Figure 7.18a).

The results obtained on the calibration of the New Pressure Cell are presented in Figure 7.18b. Comparing the results obtained with the transducer calibration it is clear that, after solving the problems of deairing and leakages (it was necessary to take special precautions to overcome these problems, during the mounting of the cell), the New cell is able to measure pressures and pressure variation with the same accuracy as the pressure transducer used.

The volume change characteristic of the cell, when under a constant confining pressure, was tested in an 11 day experiment (Figure 7.19). It was decided to measure the change in volume of the water used to apply the confining pressure inside the triaxial cell. The results obtained are presented in Figure 7.19b. A pressure of 250 kN/m^2 was applied and maintained, being the volume change measured using the system described in Figure 7.19a (paraffin volume change gauge).

To study the "creep" of the triaxial cell used as a container, the test was repeated without the New cell inside the triaxial. The results obtained for the same applied pressure are presented in Figure 7.19b.

The computed volume change under a constant pressure was measured after an "equalization time" of 60 minutes, from the moment that the confining pressure was raised from 200 to 250 kN/m^2, in both experiments.

It is clear from the results obtained that the "creep" measured in the first experiment was largely, if not entirely, originated by the volume change of the triaxial cell and not due to significant volume change of the New cell. The volume change results, when the triaxial cell was tested without the New cell inside, was greater than the original one. The measured difference of about 4 cm^3 is approximately constant during the test.

As it is very difficult to establish the initial volume variation precisely (when the confining pressure is raised from a lower to a higher value) it is doubthful if the difference measured has any special meaning, appart from indicating that the overall volume change is not significantly affected by the presence of the New cell. A more conclusive result can be achieved using a rigid container (instead of a triaxial cell) and a more accurate volume change apparatus (the paraffin gauge used is sensitive to minor temperature changes and the overall capacity is not enough to measure the volume change occurring during the raising of the pressure).

Study of the New Pressure Cell behaviour, when tested in a calibration chamber

In order to gain a better understanding of the sensitivity and general behaviour of the New Pressure Cell, when surrounded by soil, it was decided to use the calibration chamber described in Item 7.4.3. (Figure 7.9) to test the cell. The same soil (fine sand) was used to build the specimen.

After some problems of making the soil bed too compact and then getting arching around the New Pressure Cell, it was achieved a reasonable testing condition. The results obtained are presented in Figure 7.20., with the calibration of the cell plotted in the same Figure, for comparison.

During the testing procedures it was observed that even a minor change in the confining pressure (e.g. 5 kN/m^2) was monitored by the cell. When the vertical stresses were changed, but the horizontal were kept at the same original level, no variation was observed in the readings.

From the results obtained it can be seen that the difference between the applied confining pressure on the sand specimen and the measured pressure with the New Pressure Cell was less than 5 %.

Day	Readings (Volts)
1	8.825
2	8.817
3	8.815
4	8.824
5	8.826
6	8.823
7	8.822
8	8.819
9	8.810
12	8.810
14	8.812

Maximum difference : 0.016 Volts

(0.035 Volts = 1 kN/m^2)

TABLE 7.1. - Stability of the pressure transducer, when under 250 kN/m^2 during 14 days - Readings in Volts.

7.6.- THE SITE

The site used for the experiments was the University Campus. A place near Building BB that houses the Civil Engineering Department Laboratories was chosen in order to allow the use of all necessary facilities (Figure 7.2a). As continuous reading was necessary a power supply near the site was essential.

The records of the Geological Survey indicate the sub-soil conditions to consist of London Clay, Woolwich and Reading beds and Chalk.

From the Report on Site Investigation made by Foundation Engineering Ltd, August, 1965, for the construction of the University Campus, the borehole 105 was the nearest to the site. The description of the layers and the Undrained Triaxial Compression tests on 38 mm test specimens taken from U100 samplers are presented in Figure 7.3a.

From Skempton and Petley, 1967, (study on the slope stability for the building of the Campus) the following values of strengths were taken:

Peak strength of the intact clay

$$c' = 1.6 \ t/m^2 \qquad \phi' = 20^o$$

Residual strength of the slip-surface specimens (approximately equal to those measured in reversal shear box tests on the "intact clay")

$$c' = 0.3 \ t/m^2 \qquad \phi' = 12^o$$

Figure 7.3b presents a description of the subsoil as identified during the installation of the model pile, with the results of the unconsolidated undrained triaxial compression tests on 38 mm samples taken from U 100 samplers obtained some 13 m away from the model pile, using a light percussion rig.

Figure 7.3c presents the results obtained by Fugro, when performing a cone test (June, 1982). According to Fugro's experience, the undrained shear strength of London Clay can be obtained by dividing the cone resistance by 17.

7.7. - RESULTS OBTAINED

In this section all the results obtained from field instrumentation will be presented, with a description of the installation procedures of the "New Pressure Cell".

7.7.1. - PIEZOMETERS

The installation of both piezometers used was described in item 7.3.4. In this section just the results obtained will be referred to.

The Casagrande piezometer was installed on 8/2/82 and the Pneumatic piezometer on 10/2/82. The results obtained since installation are presented in Figure 7.21. Differences in results are significant, if the different methods of obtaining the data are considered. Small seasonal changes can be observed on the figure 7.21.

7.7.2. - "DRIVE IN" TOTAL STRESS CELLS

Installation of cell 500 started in 27/2/82 with the excavation of the hole to 5.30 m. The cell was successfully pushed into the reading position in 5/3/82. Cell 501 was ready for measurements in 12/3/82, after 10 days of installation procedures. The results obtained are presented in Figure 7.22.

Due to the difficulties in the installation procedures the first readings were made days after the installation. No major change was observed in the readings(rise and dissipation of pore pressures) and both cells presented approximately the same outputs (recorded horizontal total stresses).

One of the objectives of the experiment was to study the possible existence of different horizontal stresses acting in different directions (as recorded in other places in London Clay , or even diametrically opposed load cells of the Camkometer, see Simpson et al, 1981, page 115) due to special geological features, but the results obtained showed no significant difference. Figure 7.22 a shows the results obtained with the two cells. Figure 7.22 b presents the estimated vertical stresses for the site, with the values of piezometric level recorded (Standpipe) and horizontal pressures obtained by the "drive in" total pressure cells.

7.7.3. - "NEW PRESSURE CELL" - INSTRUMENTED MODEL PILE

The installation of the "New Pressure Cell" with the model pile started on 21/4/82, with the excavation of the hole. The Portable Minuteman used for drilling the 75 mm and 100 mm diameter holes for the piezometers and "drive in" cells was used. As the required diameter for the model pile was 150 mm and the drilling equipment was not capable of drilling 7 m using a 150 mm continuous auger, pre-drilling was necessary. A 6.5 m depth hole was made using a 100 mm diameter continuous auger. After cleaning the hole, a single auger, 900 mm long and 150 mm diameter, was used to enlarge the overall length and a final level of -7.0 m for the base of the model pile was reached. Drilling the model pile hole took 6 hours and was completed by 15.30 hours on 21/4/82.

The original plan was to concrete the model instrumented pile one day after excavation (22/4), but the water inside the hole rose up to -3.5 m. Cleaning the hole proved to be a problem. After some trials a 100 mm diameter bucket was used, but a decision was made to postpone the pile installation to the next day, due to the unexpected dalay.

Full cleaning of the hole proved impossible, using the available facilities. It was decided to concrete a larger length at the base of the model pile to ensure continuity and a clean bed for the cell.

Concreting of the model pile was initiated at 9.30 h on 23/4/82. The concrete mix used, after some trials in the Material's laboratory to obtain an almost fluid mix, with a slump greater than 150 mm and good workability and a maximum aggregate size compatible with the experiment, was the following (by weight):

Cement:	1
Water:	0.53
Aggregate 10 mm:	1.36
5 mm:	2.04
Melament:	0.0249

The "New Pressure Cell" was carefully introduced into the hole to make contact with the fresh concrete at its base. The system described in Figure 7.23 was used to impose an initial pressure of 80 kN/m^2 on the cell at 11.15 h (simulating 4 m of head of fresh concrete and allowing for a possible small increase due to the real concreting that followed: 4 m x 23 kN/m^3 = 92 kN/m^2).

The solenoid valve was closed and after 15 minutes (when constant readings were achieved) the remaining length of the pile was concreted.

A datalogger was used during the first week for continuous monitoring.

On the first night (23/4/82) after the installation, the equipment was vandalized. After the incident extra precautions were taken to protect the wires, connections and equipment, and successful readings were achieved continuously.

During the model pile installation disturbed samples were taken and tested in the laboratory. The sub-soil profile encountered with identification of the soil type, natural moisture content and Atterberg limits obtained at the respective depths is shown in Table7.2 . Figure 7.26 shows the final configuration of the model pile with the "New Pressure Cell".

Figure 7.27a to d presents the results obtained using the "New Pressure Cell" during the model pile installation and the long term readings. In Figure 7.27a the results obtained during the pile installation, up to 3 hours after the initial pressure was established in the cell are presented. From the imposed initial value of 80 kN/m^2 at 11.15 h, when concreting with the fluid mix started, the pressure reading on the cell reached the peak of 105 kN/m^2 at 11.30 h.

At the end of concreting, at 11.45 h, the pressure reading was 95 kN/m^2. The recorded pressure continued to drop slowly to 87 kN/m^2 three hours after installation started.

Figure 7.27b shows the pressure readings in the 12 hours following installation. It can be seen that no change was observed from 3 to 12 hours.

In Figures 7.27c and d the measured values for the first week and 150 days are reported. In the first week the range of the observed values was, after the first peak, from 87 to 91 kN/m^2. Up to 150 days after installation no significant change was observed.

The rising of the recorded pressures due to installation can be explained by the configuration of the reading device. The "New Pressure Cell", when expanded, has part of the flexible membrane exposed on the top (not covered by the rigid protection discs). From the imposed pressure acting internally the action of the external vertical pressure of the fluid concrete caused an increase in the pressure reading. Under the new state of stresses, the equilibrium between the cell and the surrounding soil was reached after some volume change, giving rise to a change in the recorded pressure. The reading obtained after 2 hours was 88 kN/m^2 and up to 150 days the variation observed was less than 5%.

Due to the events during the installation, the model pile simulated a construction of an unorthodox pile. The delay of 2 days from the excavation to the concreting allowed the soil to reach a new equilibrium under the acting stress regime.

The result obtained, with no major change of pressure following initial equilibrium reached 2 hours after concreting, shows that in the total stress regime no variation occurred in the time that followed the installation.

In Figure 7.28 the "summing up" of field experimentation results and estimated vertical pressure are presented for comparison. The values obtained with the "Drive in" total stress cells when compared with the estimated vertical pressure resulted in the following ratios:

$$\frac{\sigma_h \text{ measured}}{\sigma_v \text{ estimated}} = 2.1$$

$$\frac{\sigma'_h \text{ measured}}{\sigma'_v \text{ estimated}} = 2.5 \qquad (?=K_o)$$

The readings on the "New Pressure Cell" stabilized at a level higher than the estimated vertical total pressure, but probably well below the original "in situ" horizontal total stresses.

Figure 7.27 e presents a typical set of results obtained 100 days after installation of the model pile. It can be observed a small variation in the readings, due probably to the "warming up" effect (transducer) and to the fact that the datalogger was unprotected on the open, suffering the effects of the weather.

7.8. - DISCUSSION

A simple field experiment was performed in order to study the possible effects of pile installation on the existing stresses in the ground. Two piezometers, two "Drive in" total stress cells and a "New Pressure Cell" acting as a model pile instrumentation were installed and presented consistent results during more than 100 days;

Problems during the installation of the model pile resulted in a change of geometry and delayed the concreting. As a result the experiment simulated the installation of a defective (poor workmanship) pile;

At the level of the "Drive in" total stress cells (-5.70 m) the ratios between horizontal stresses measured and vertical stresses estimated resulted in the following values :

$$\frac{\sigma h \text{ measured}}{\sigma_v \text{ estimated}} = 2.1$$

$$\frac{\sigma' h \text{ measured}}{\sigma'_v \text{ estimated}} = 2.5$$

The measured values of total stress from the "New Pressure Cell" (4.00 m depth) reached a maximum of 105 kN/m^2 during installation, when the fresh fluid concrete first acted upon the cell. After 30 minutes the recorded pressure reached a level of 95 kN/m^2 and 2 hours after the installation a final equilibrated stress level of 88 kN/m^2 was recorded, with no significant changes up to 150 days of readings;

The recorded pressures acting on the "New Pressure Cell" were slightly higher than the total vertical pressure estimated for that level. As a result, the ratio of measured horizontal stresses and estimated vertical stresses are lower than the typical range of "in situ" stresses;

The principle, design, sensitivity, strength, accuracy and reliability of the "New Pressure Cell" proved to be suitable for the study of the state of stresses acting in a bored pile. The cell is able to measure the installation pressures and possible variations during long term experiments. As the design is based on a simple configuration and all materials can be easily obtained, the experiment can be repeated on a different scale;

In principle, future field work concerning large bored piles should concentrate on the effects of pile installation on the properties of the surrounding soil. The use of the proposed system, in conjunction with other field equipment can prove to be a valuable tool.

The conclusions of the present findings will be presented in Chapter 8 - General conclusions and recommendations.

DEPTH (m)	SOIL DESCRIPTION	Moisture content (%)	Liquid limit (%)	Plastic limit (%)
0.20	Topsoil			
0.90	Soft brown sandy silty CLAY with some gravel and occasional brick fragments (made ground)			
4.00	Soft to firm fissured brown sandy silty CLAY	35	70	32
6.50	Stiff very fissured brown silty CLAY with some selenite crystals	33	75	35
		30	79	33
		31	67	32
7.00	Very stiff brown silty CLAY possible claystone layer at 6.80 m			
	END OF HOLE			

TABLE 7.2 - Sub-soil profile encountered during the installation of the model pile, with values of natural moisture content and Atterberg limits obtained at the respective depths.

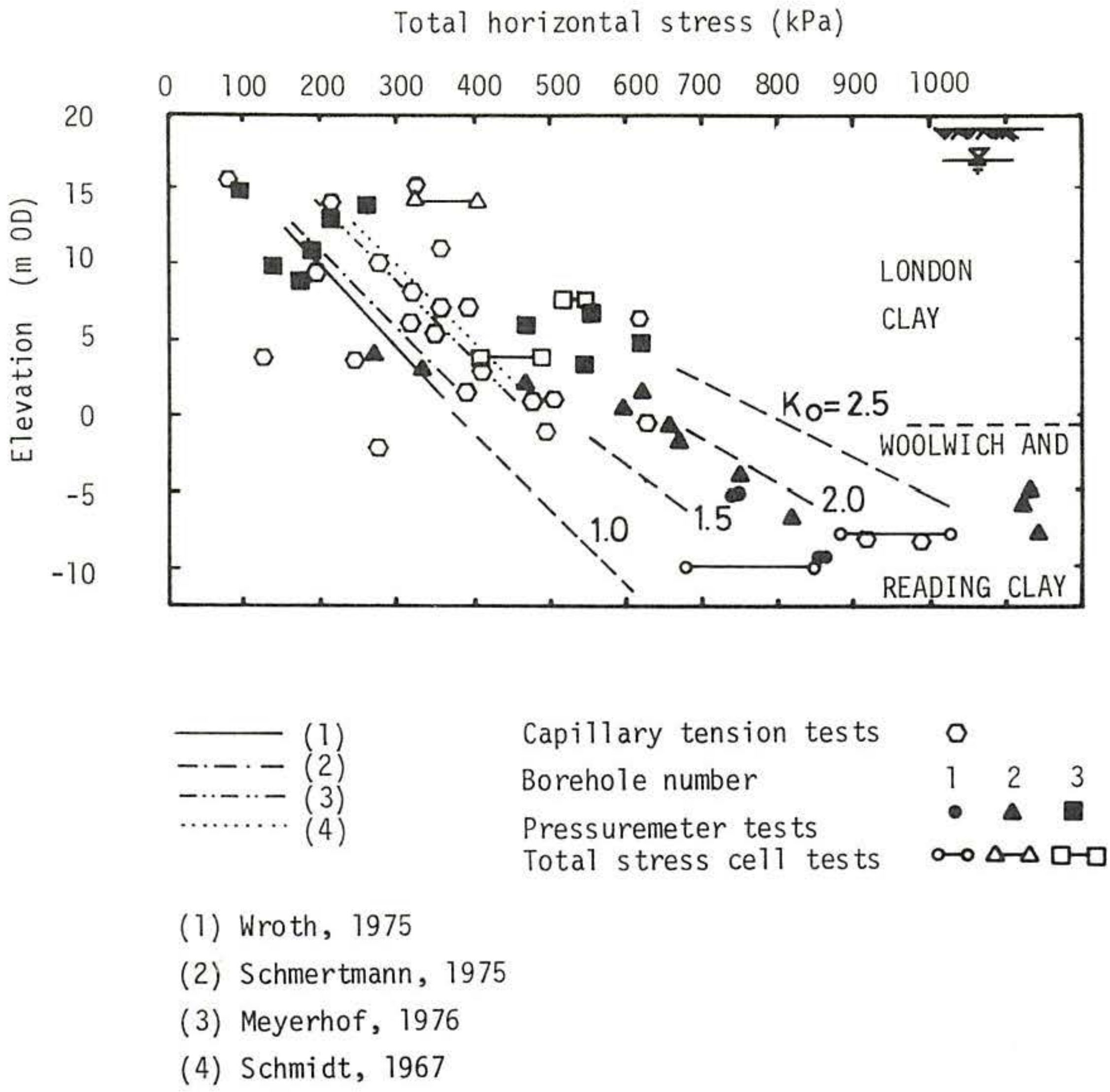
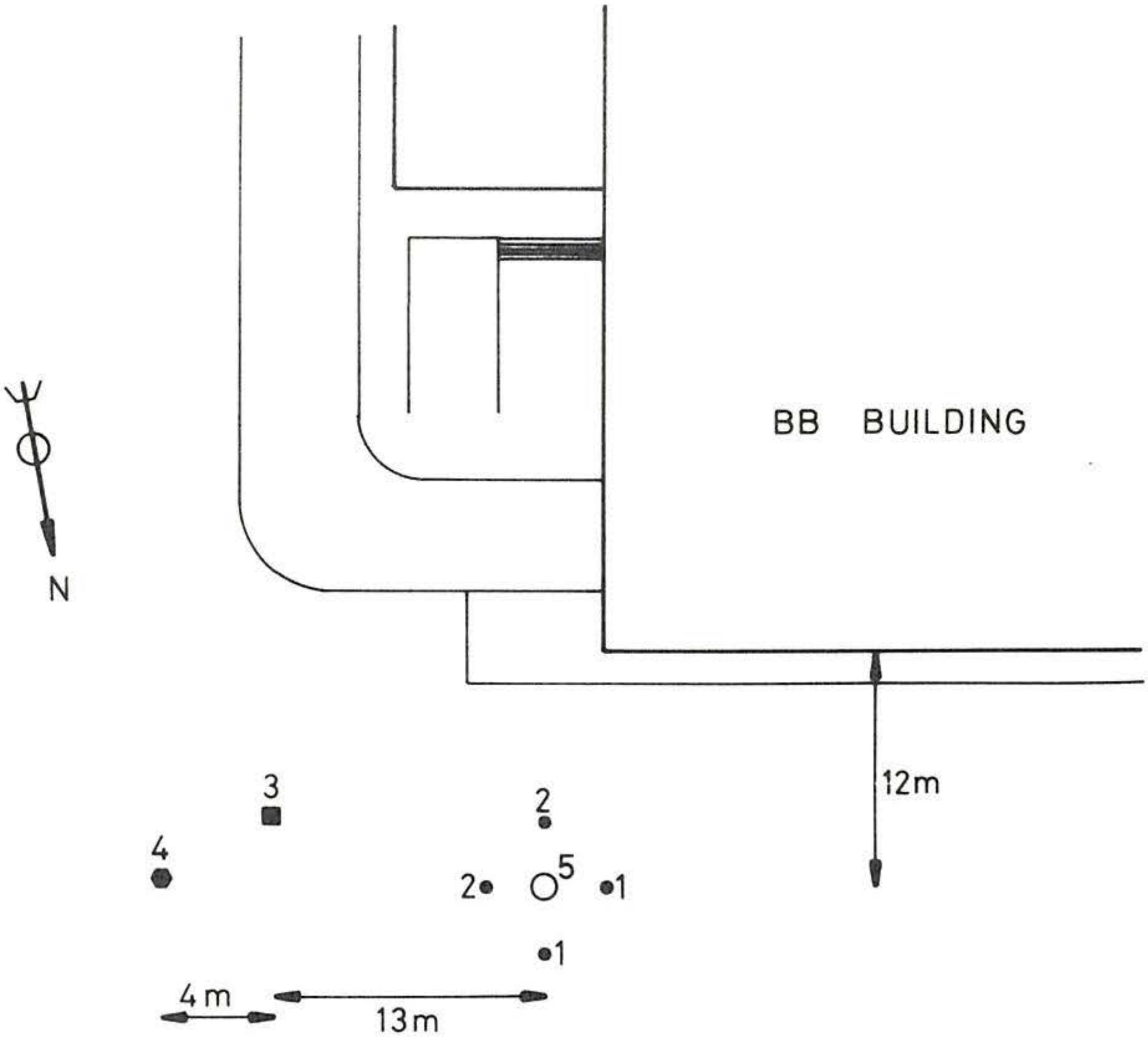


FIGURE 7.1 - COMPARISON BETWEEN CALCULATED AND RESULTS OF LABORATORY AND "IN SITU" MEASUREMENTS OF HORIZONTAL STRESSES (SIMPSON ET AL, 1981)



1 - "Drive in" total stress cells

2 - Piezometers

3 - U 100 samplers recovered with a light percussion rig

4 - Cone test

5 - Model instrumented pile with the "New pressure cell"

FIGURE 7.2a - LOCATION OF THE SITE AND EXPERIMENTAL PROGRAMME

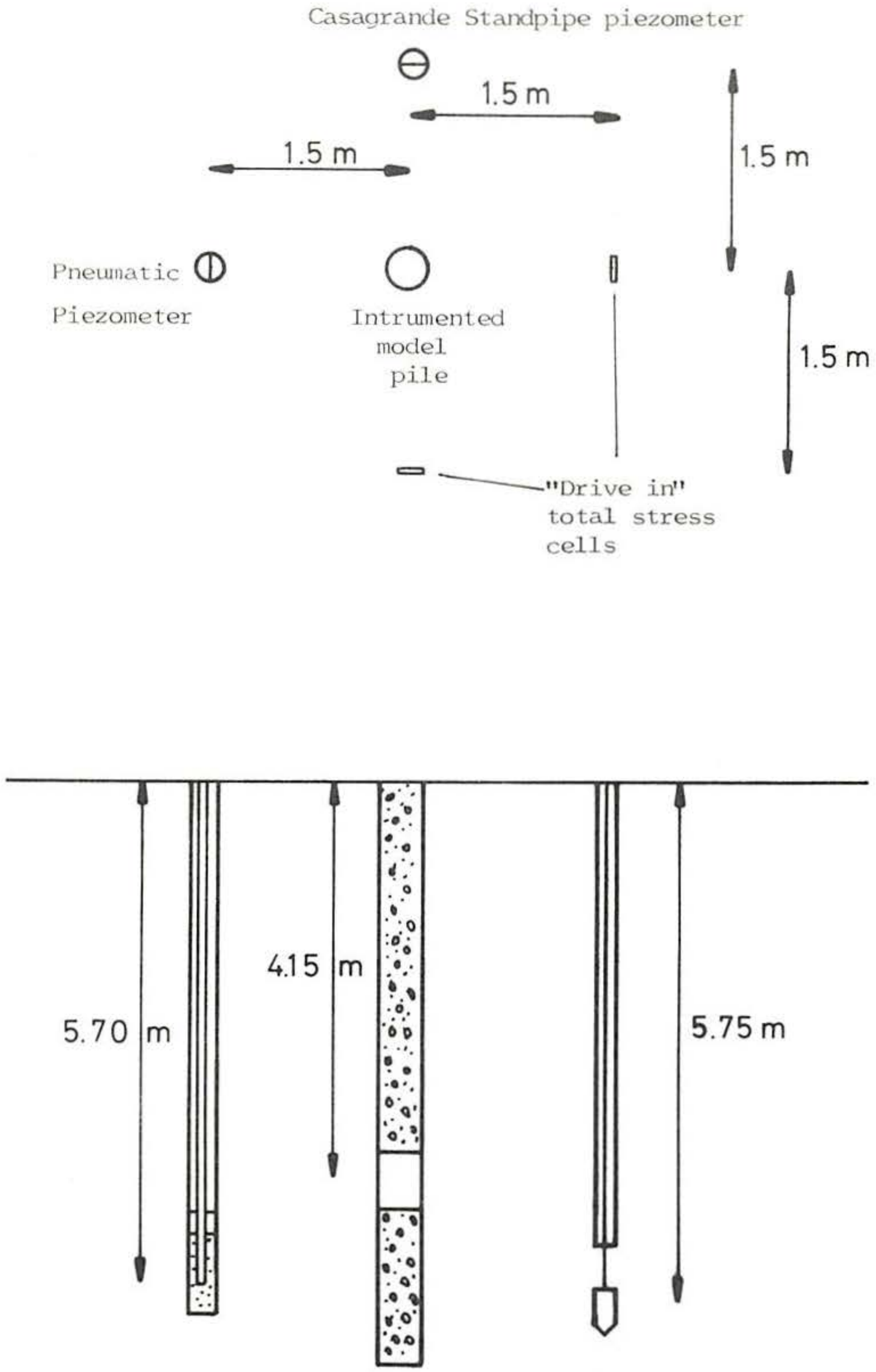


FIGURE 7.2b - DETAILS OF THE LOCATION OF THE INSTRUMENTATION

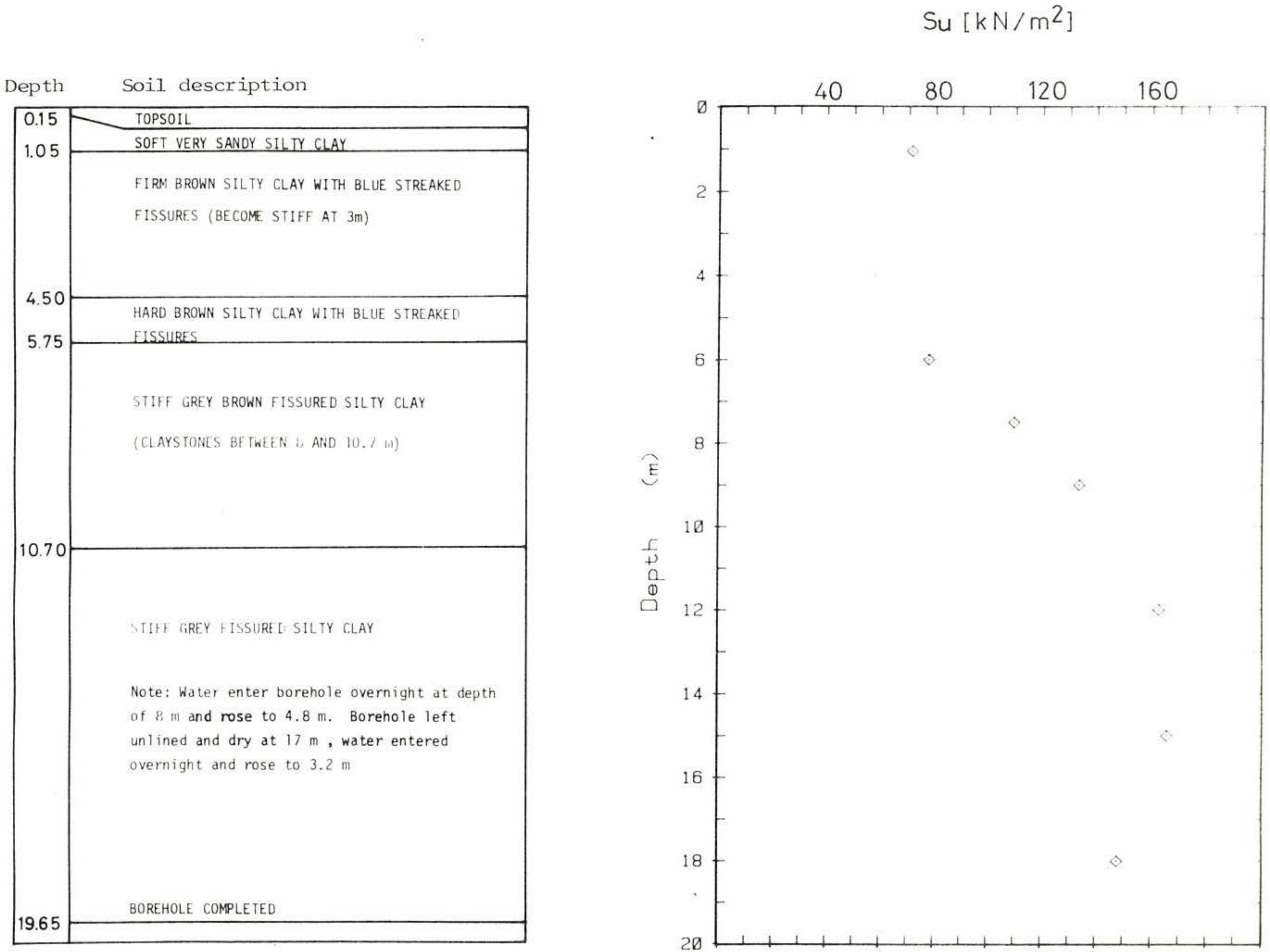
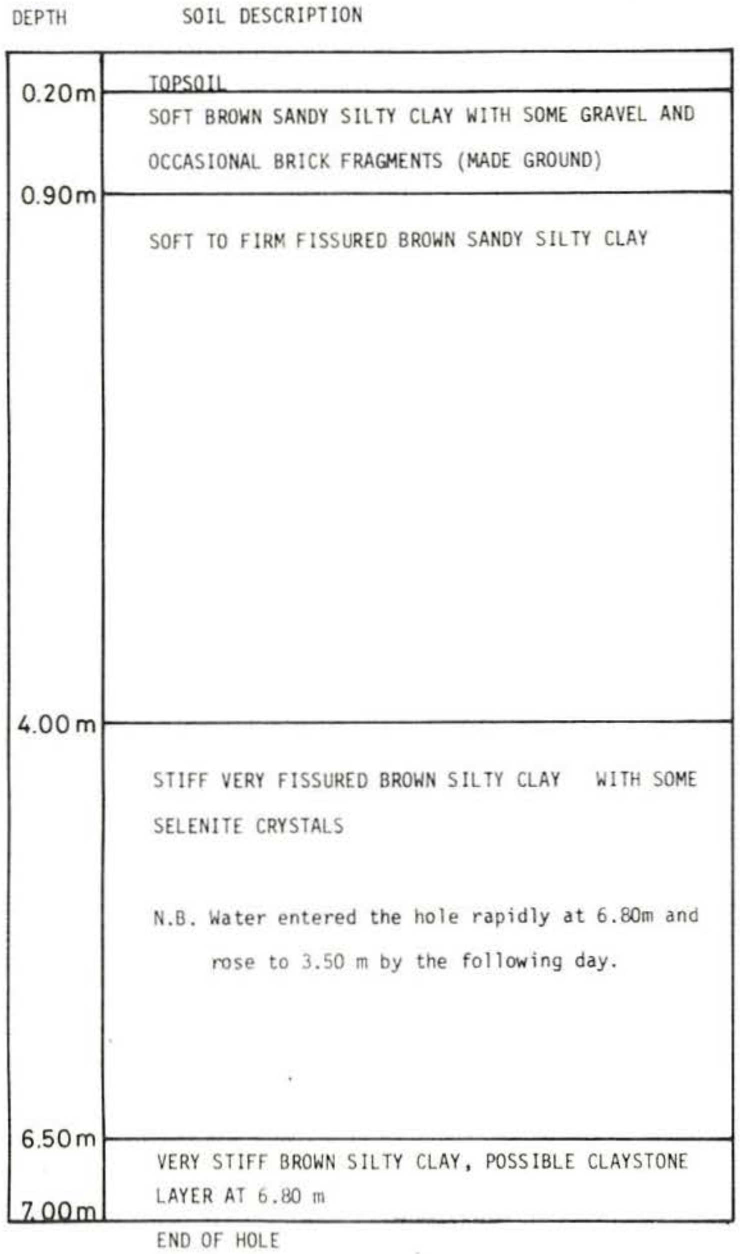


FIGURE 7.3a - SITE INVESTIGATION REPORT BY FOUNDATION ENGINEERING LTD, 1965, SHOWING SOIL DESCRIPTION AND RESULTS OF UNDRAINED TRIAXIAL COMPRESSION TESTS ON 38mm SPECIMENS TAKEN FROM U 100 SAMPLERS (BOREHOLE 105)



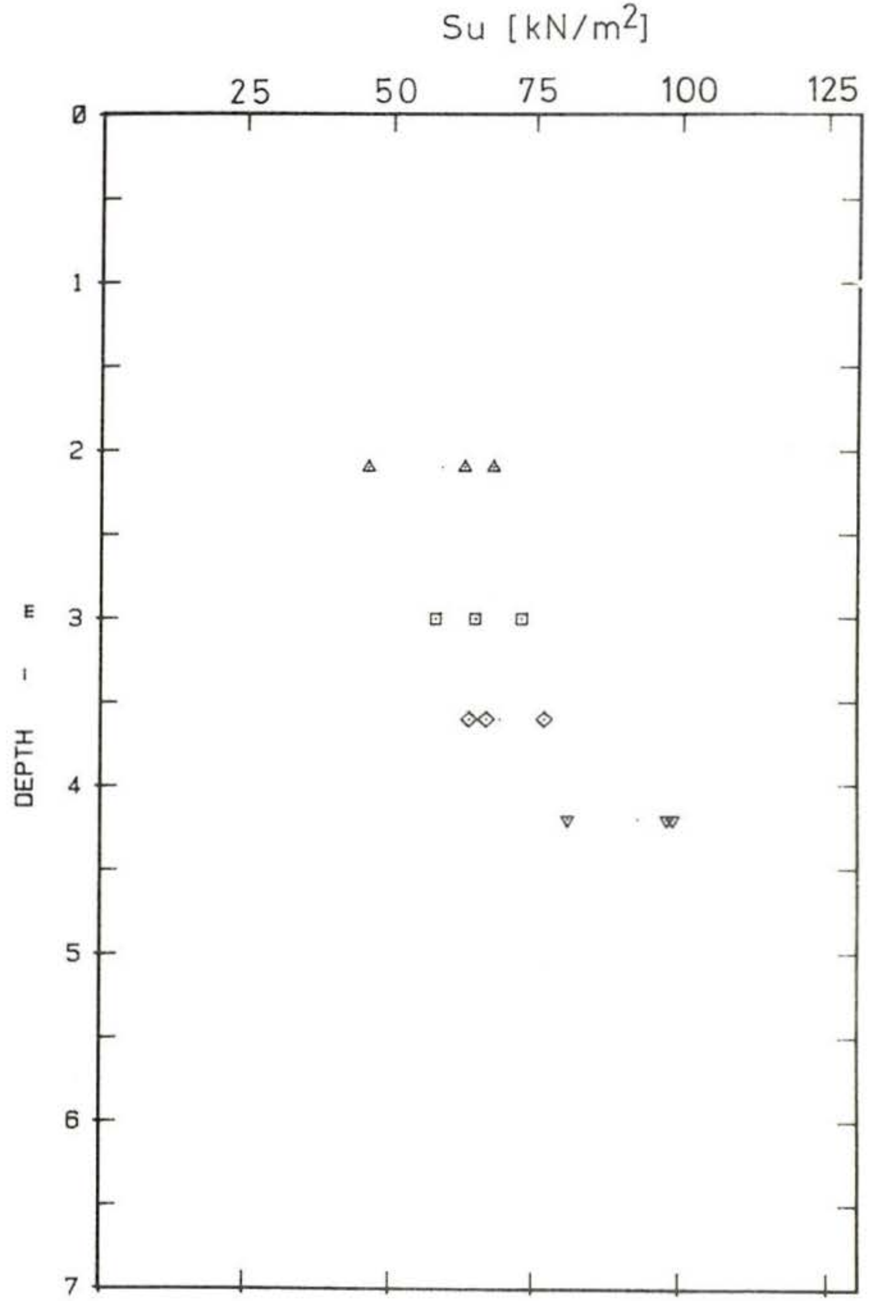


FIGURE 7.3b - SUB-SOIL DESCRIPTION AS FOUND IN THE BOREHOLE MADE FOR THE MODEL PILE AND RESULTS OF UNCONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS MADE IN 38mm SPECIMENS OBTAINED FROM U 100 SAMPLERS TAKEN 13m AWAY FROM THE MODEL PILE

CONE : rate of penetration : approx. 2 cm/sec

cross sectional area: 10 cm

apex : 60

FRICTION SLEEVE :

outside diameter : 3.56 cm

area : 150 cm^2

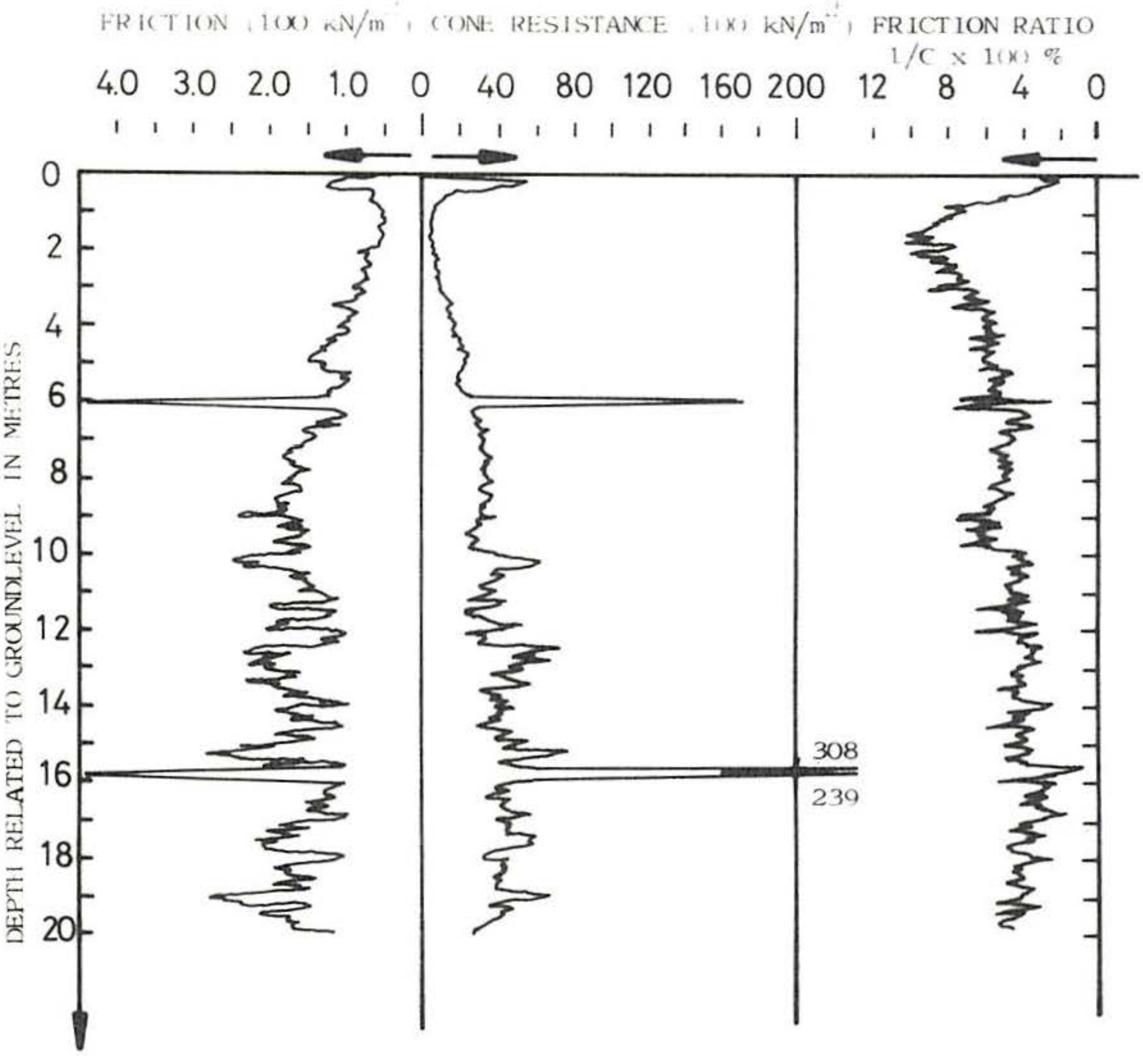


FIGURE 7.3c - CONE TEST RESULTS, BY FUGRO, JULY, 1982

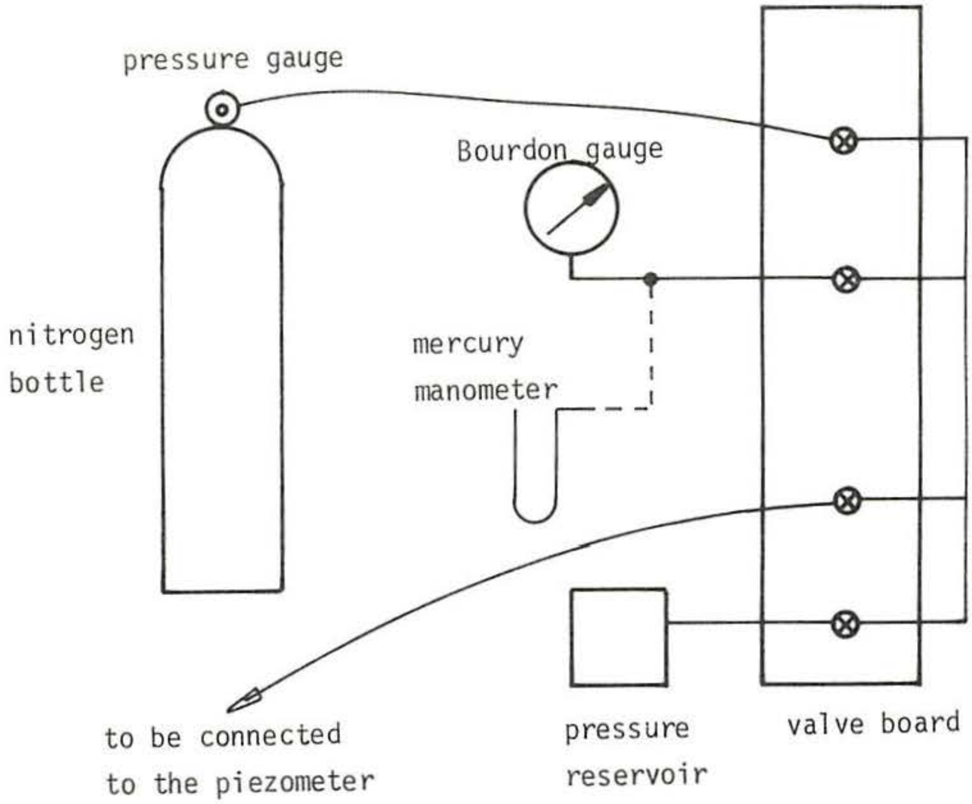


FIGURE 7.4 - DIAGRAMATIC REPRESENTATION OF THE PNEUMATIC MEASURING SYSTEM

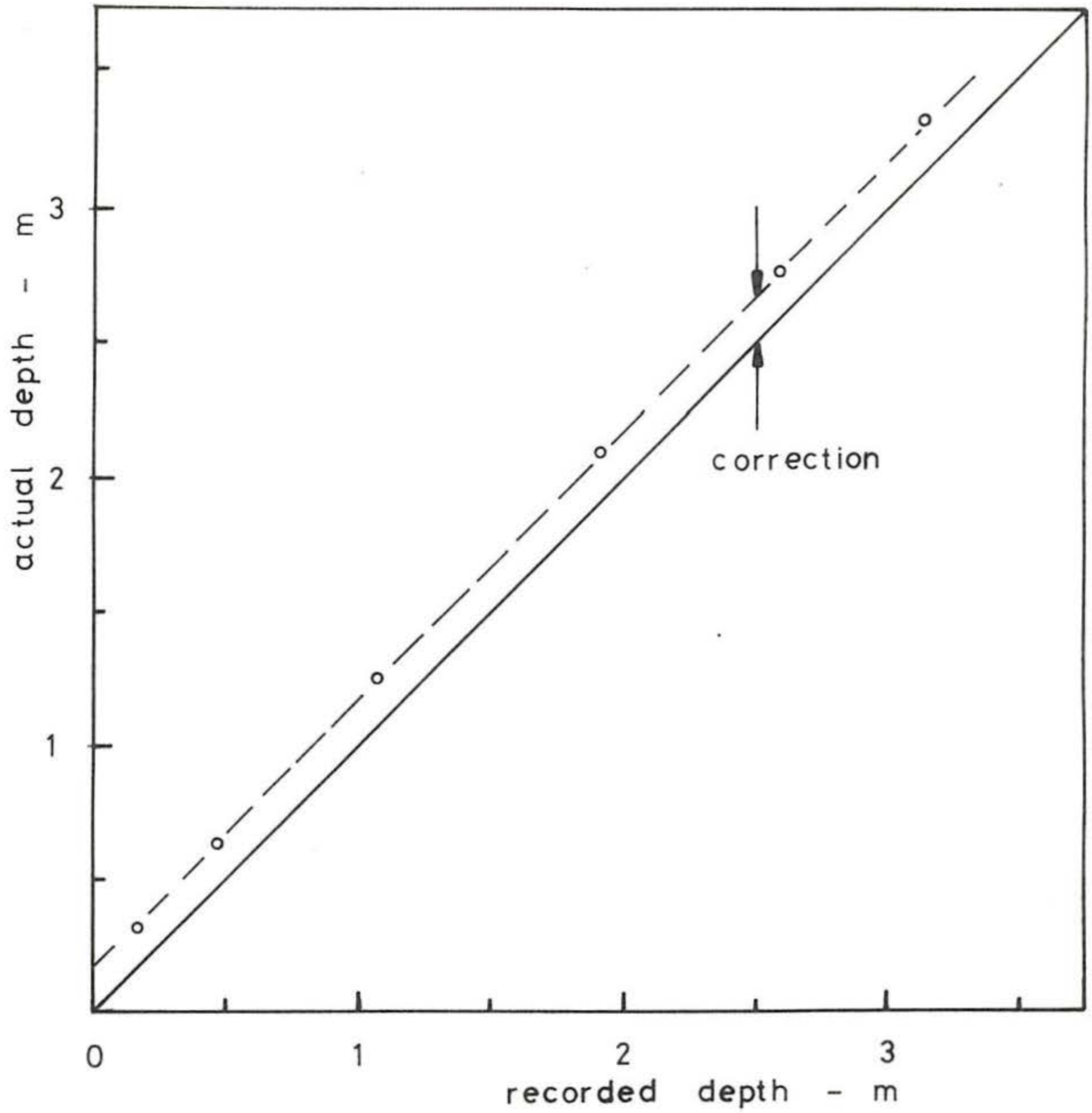
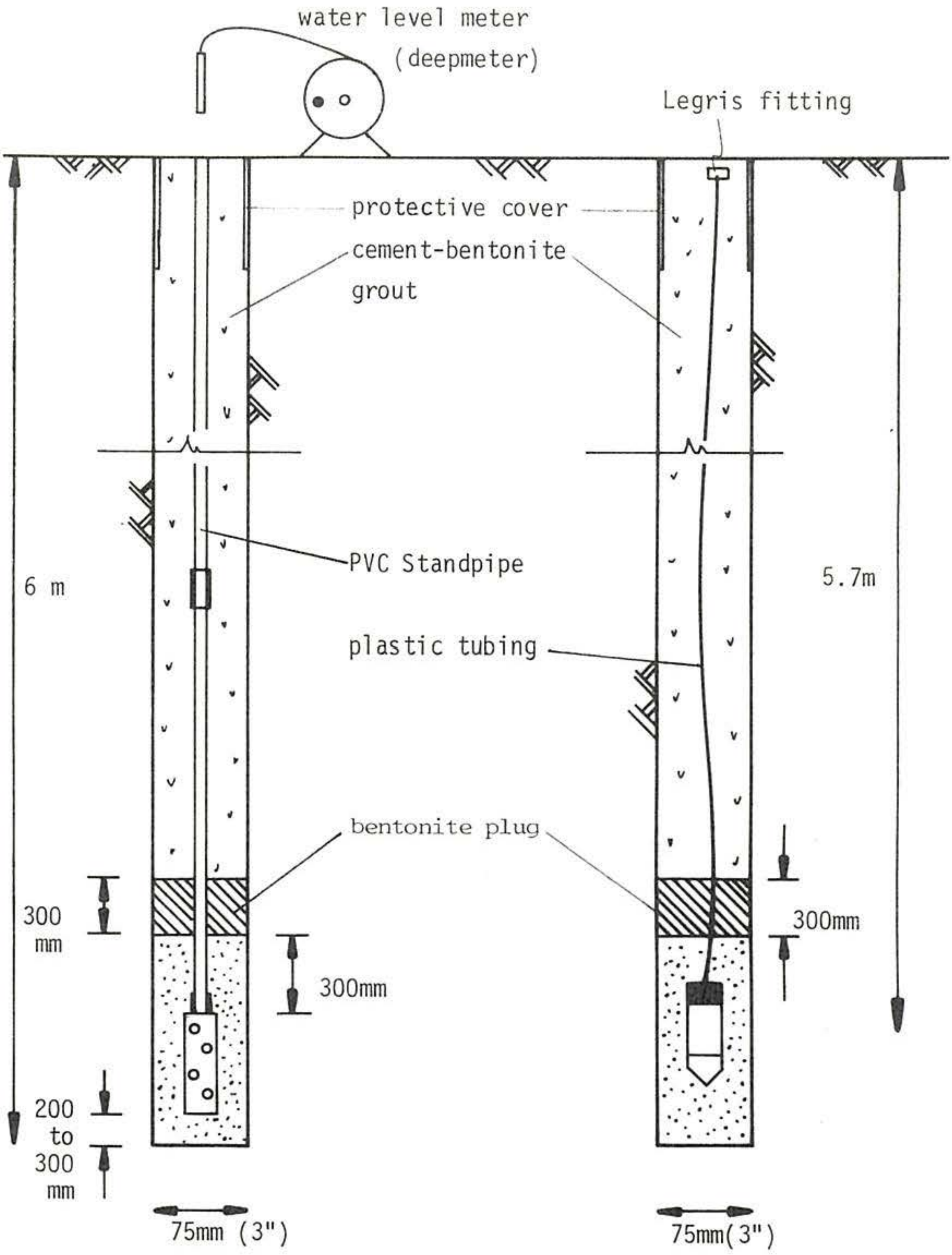


FIGURE 7.5 - CALIBRATION OF THE PNEUMATIC PIEZOMETER



CASAGRANDE STANDPIPE PIEZOMETER PNEUMATIC PIEZOMETER

FIGURE 7.6 - INSTALLATION OF THE PIEZOMETERS

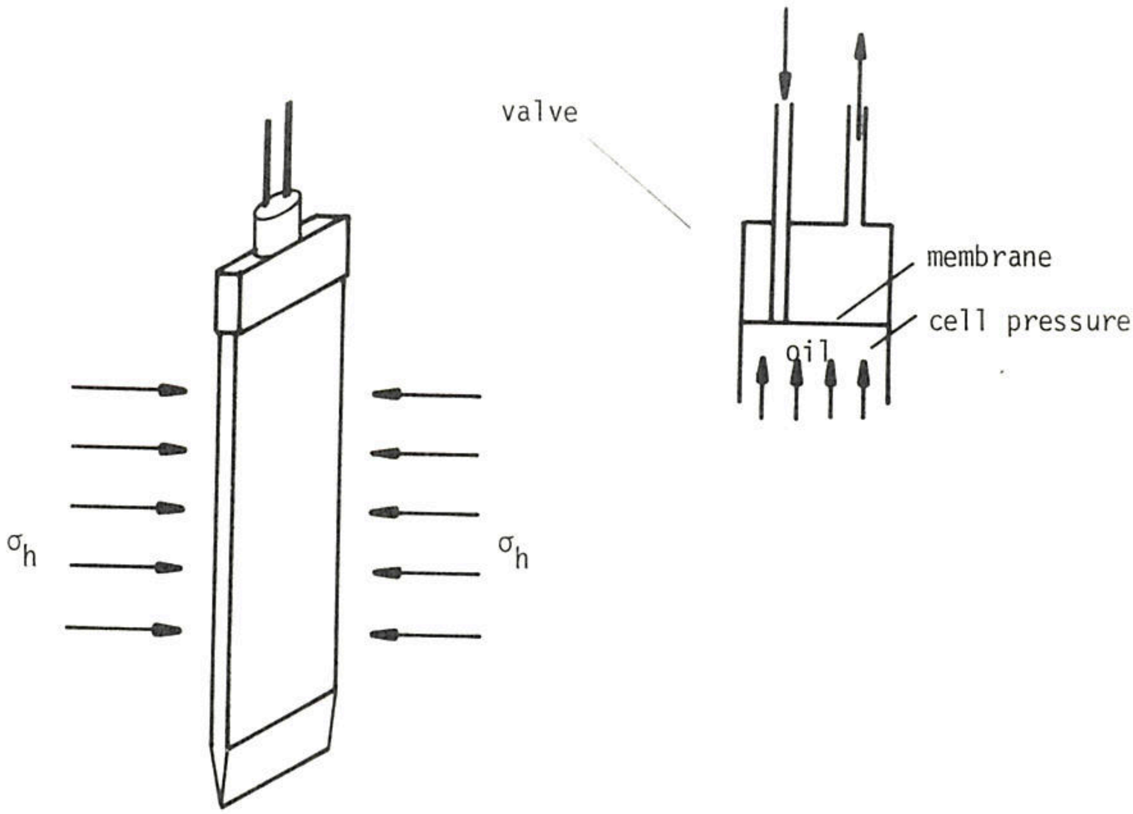
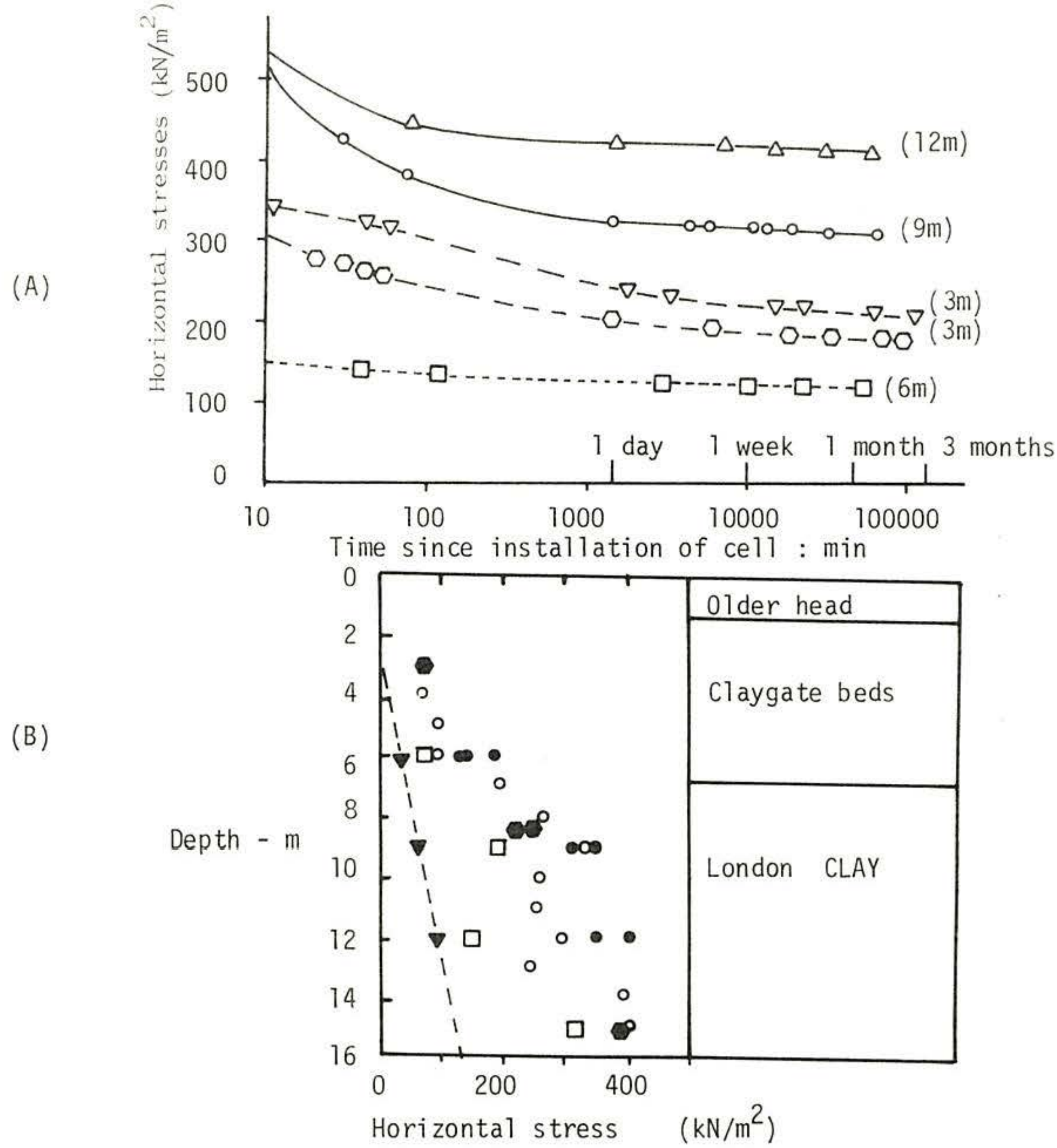


FIGURE 7.7 - THE "DRIVE IN" TOTAL STRESS CELL (MASSARCH, 1974, 1975)



● "Drive in" pressure cells

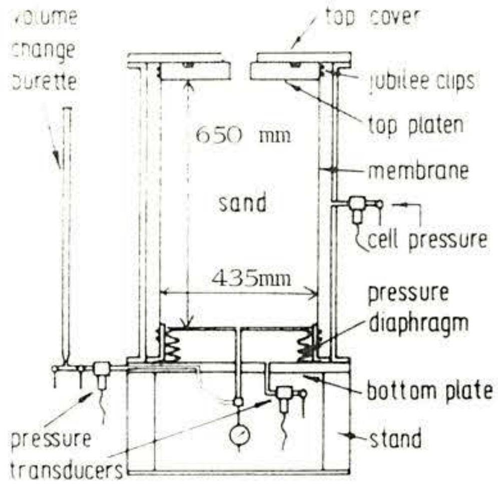
○ Camkometer self-boring pressuremeter

□ Camkometer self-boring load cell

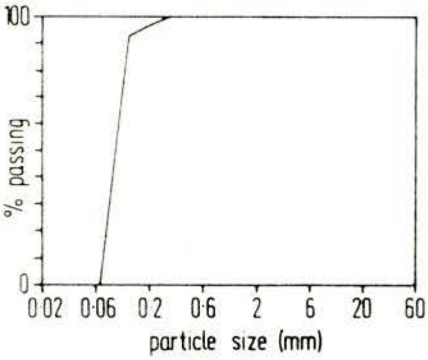
▼ Porewater pressure

⬢ Deduced from suction laboratory tests (Burland and Maswoswe, 1982)

FIGURE 7.8 - THE DRIVE IN PRESSURE CELL
(A) STABILIZATION TIME FOR MEASUREMENTS MADE AT DIFFERENT PLACES, (B) COMPARISON OF VALUES OF HORIZONTAL STRESS OBTAINED BY VARIOUS METHODS FOR THE SAME SITE (TEDD AND CHARLES, 1981)

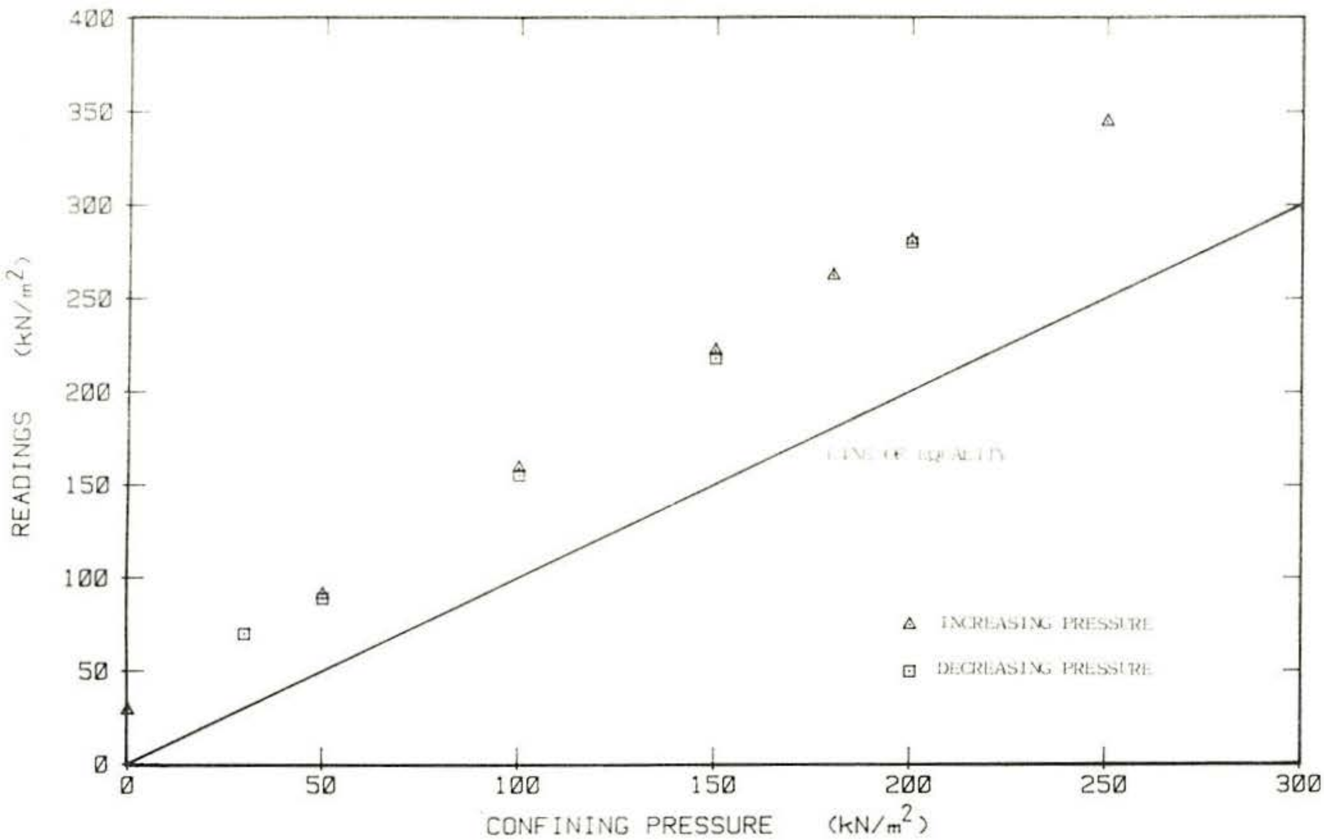


(A)

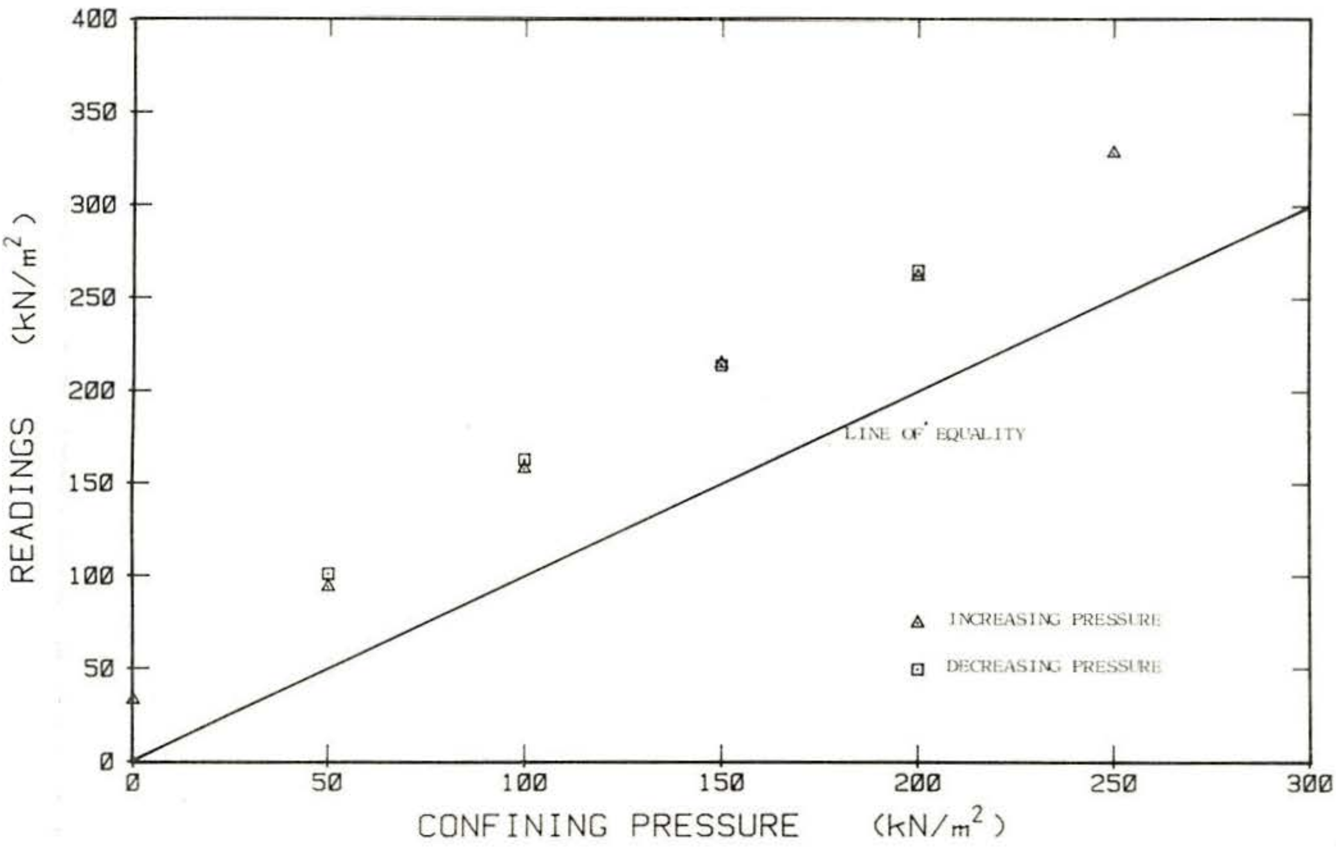


(B)

FIGURE 7.9 - CALIBRATION CHAMBER USED TO TEST THE "NEW PRESSURE CELL" AND THE "DRIVE IN" TOTAL STRESS CELLS
(A) DETAILS OF THE CHAMBER, (B) GRADING OF THE SAND USED ON THE TESTS (FROM CLAYTON AND DIKRAN, 1982)



(A)



(B)

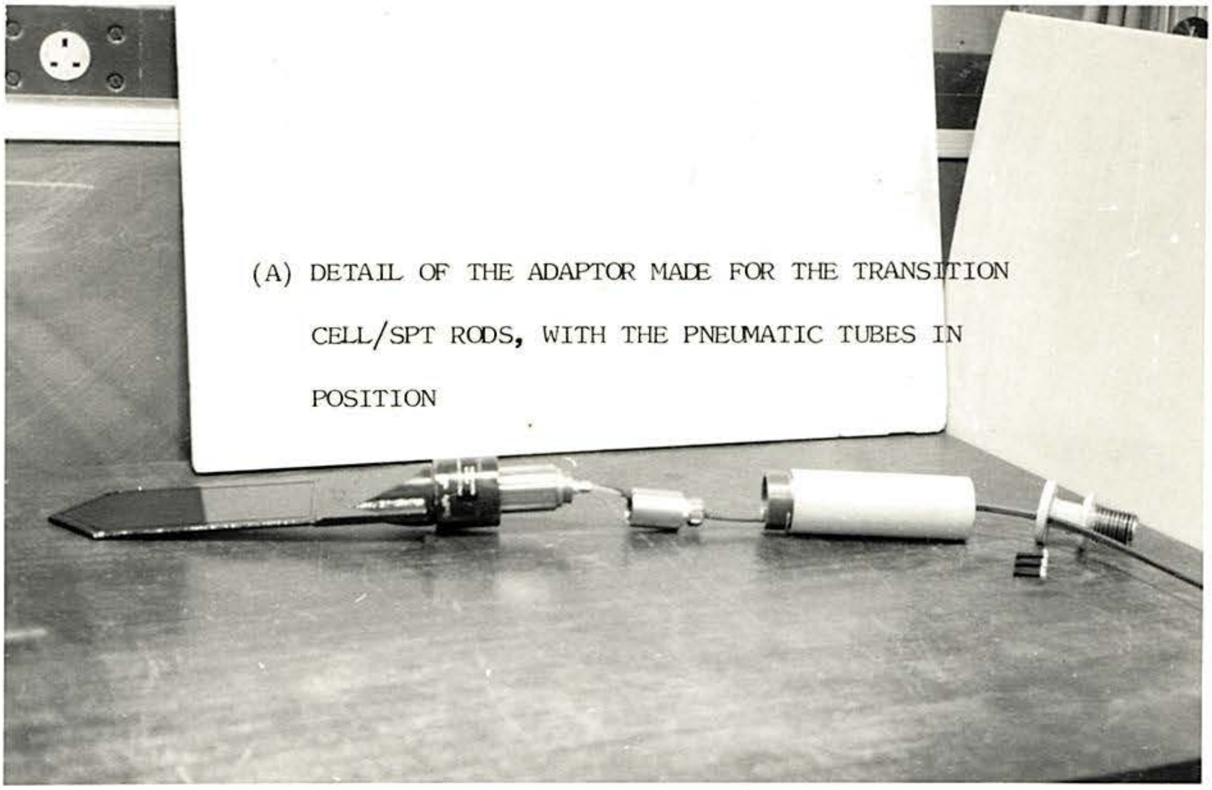
FIGURE 7.10 – CALIBRATION OF THE "DRIVE IN" TOTAL STRESS CELLS, (A) CELL 500, (B) CELL 501

(A) "DRIVE IN" TOTAL STRESS CELL WITH ADAPTOR AND SPT

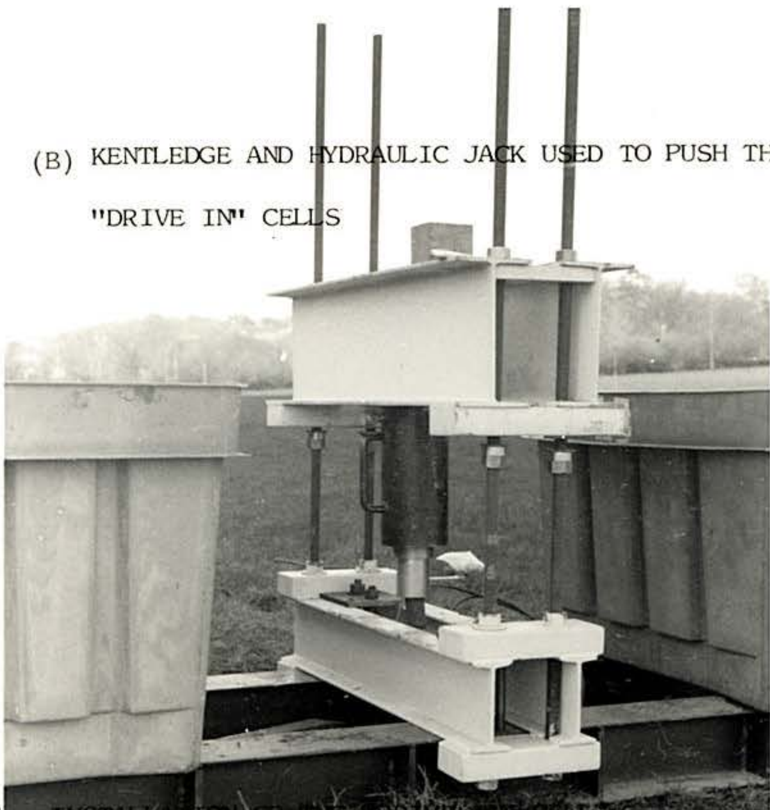
RODS

(B) KENTLEDGE SYSTEM BEING MOUNTED

FIGURE 7.11 - TOTAL STRESS CELL INSTALLATION



(A) DETAIL OF THE ADAPTOR MADE FOR THE TRANSITION CELL/SPT RODS, WITH THE PNEUMATIC TUBES IN POSITION



(B) KENTLEDGE AND HYDRAULIC JACK USED TO PUSH THE "DRIVE IN" CELLS

FIGURE 7.12 - INSTALLATION OF THE "DRIVE IN" CELLS

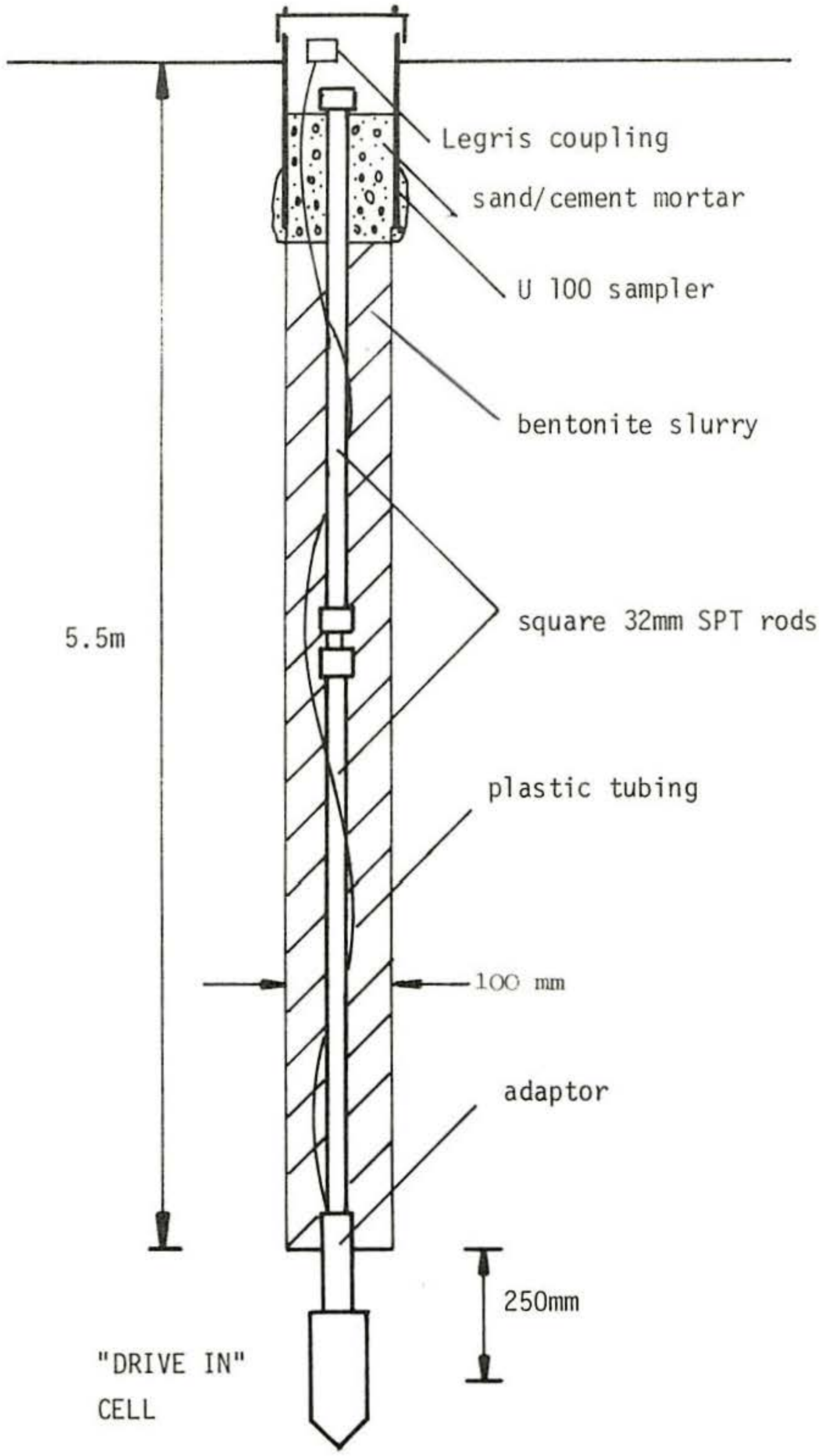


FIGURE 7.13 - DETAILS OF THE FINAL ARRANGEMENT OF THE "DRIVE IN" TOTAL STRESS CELL IN THE GROUND

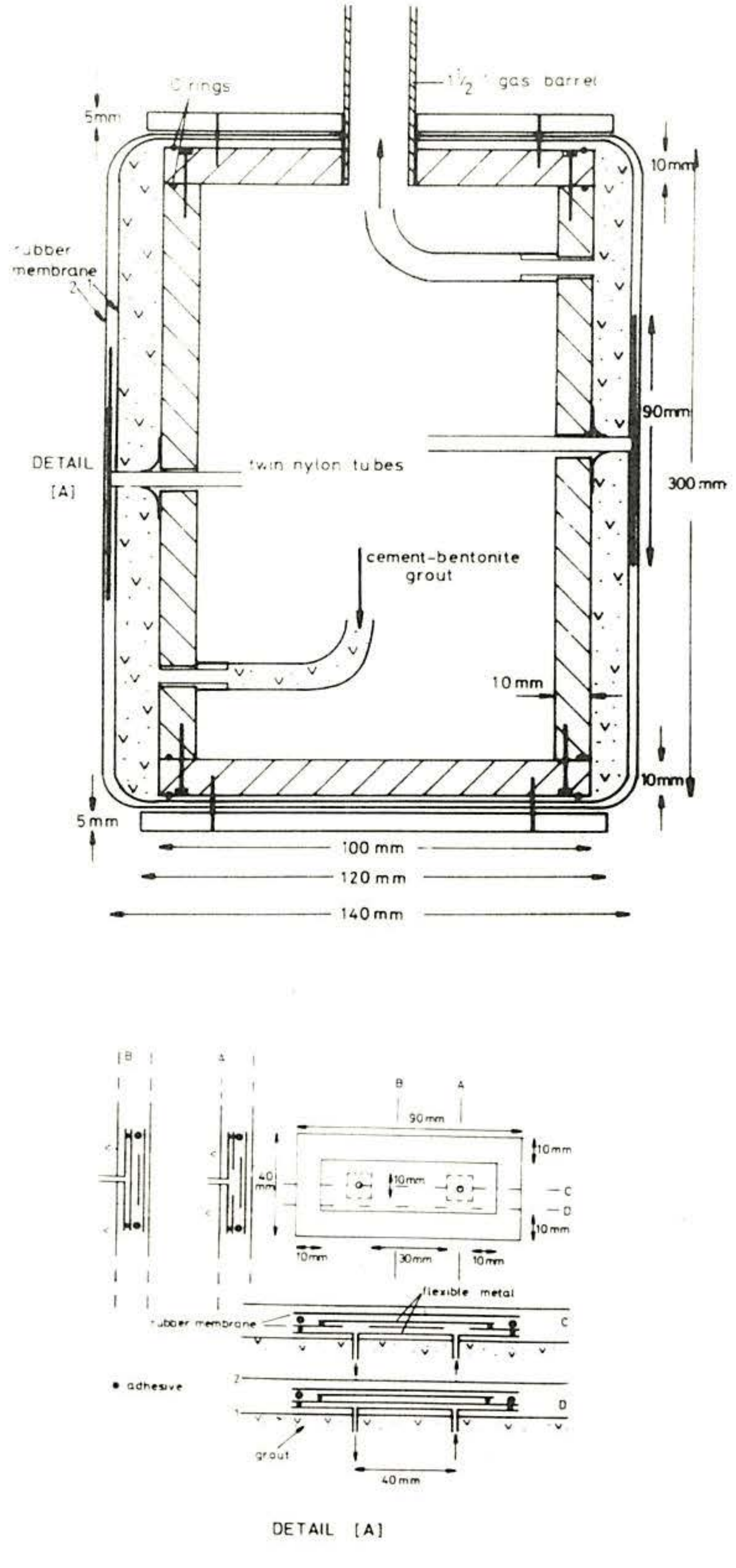


FIGURE 7.14 - DESIGN OF THE "NEW PRESSURE CELL - GP 1"(GROUT PNEUMATIC)

NOT BUILT

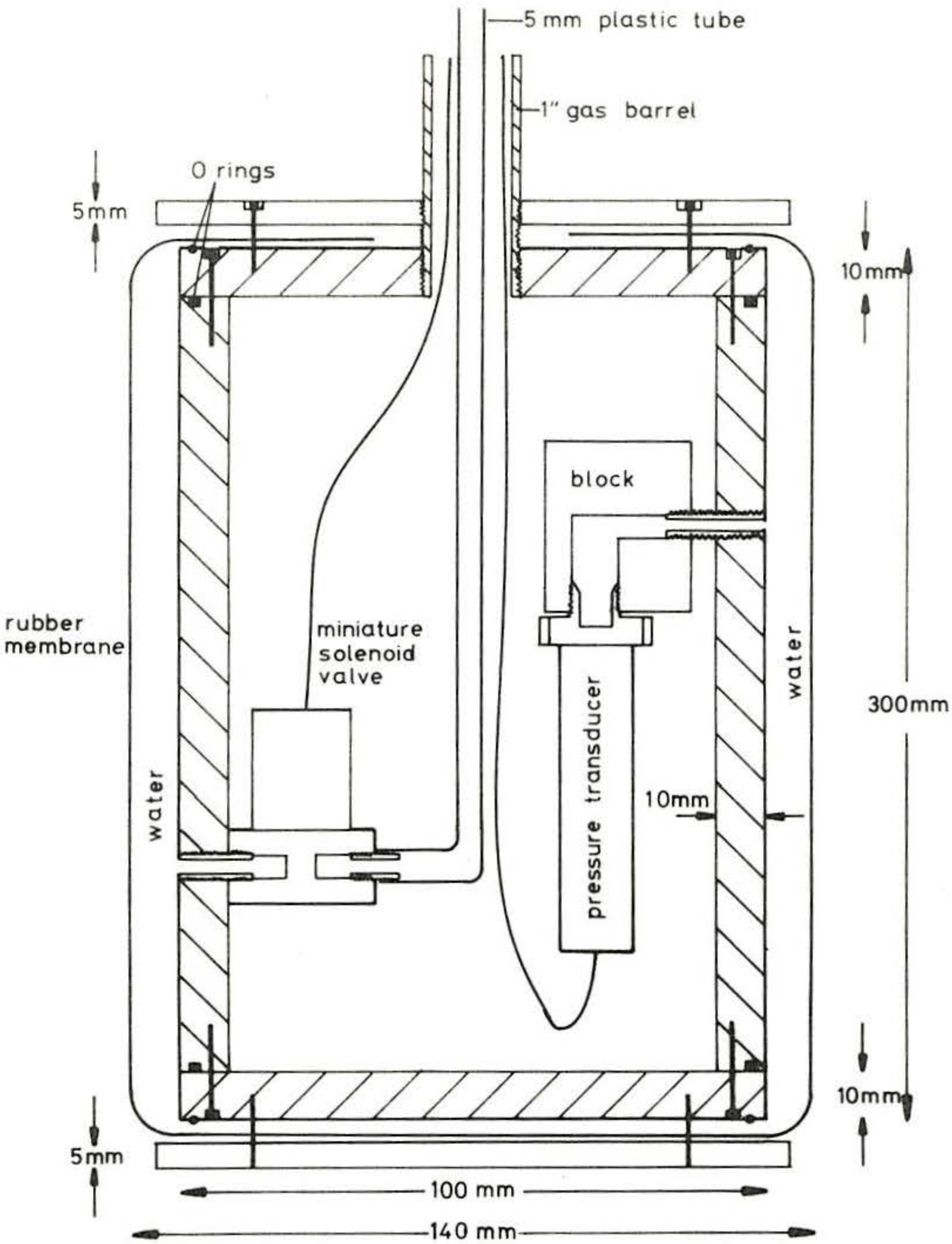
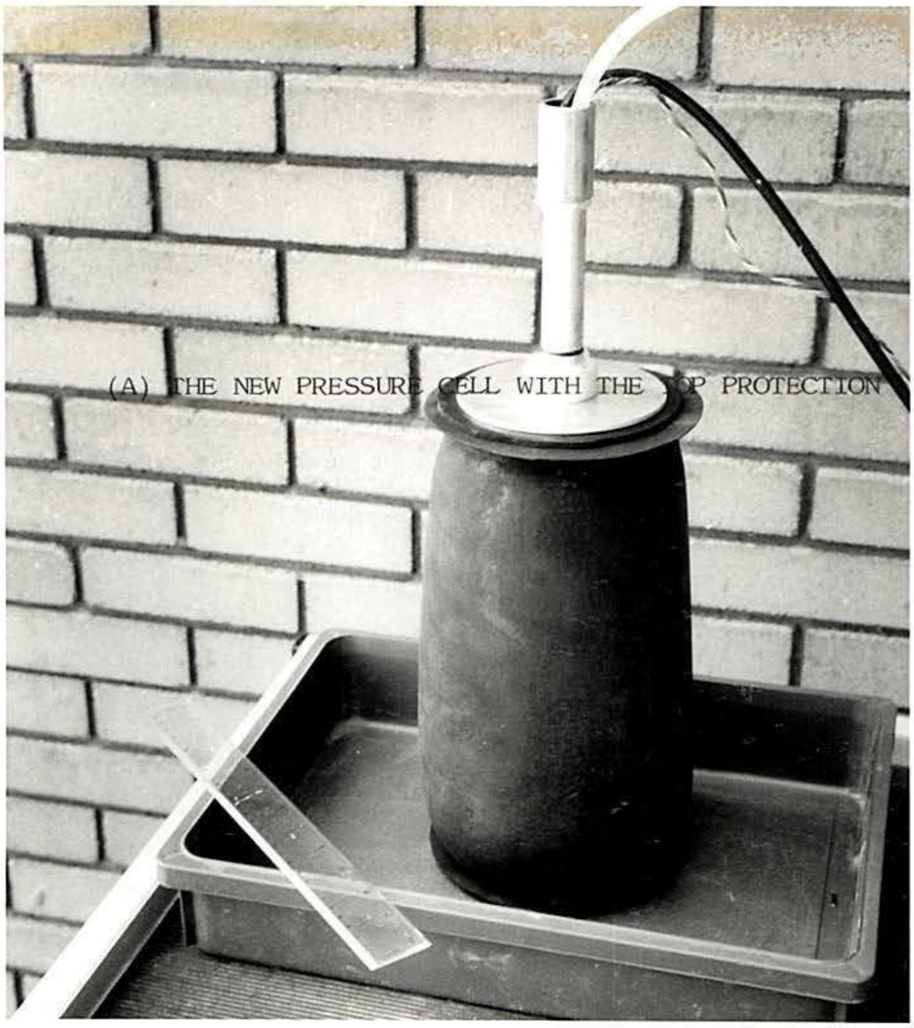


FIGURE 7.15a - THE "NEW PRESSURE CELL" - ST 1 (SOLENOID TRANSDUCER) MODEL

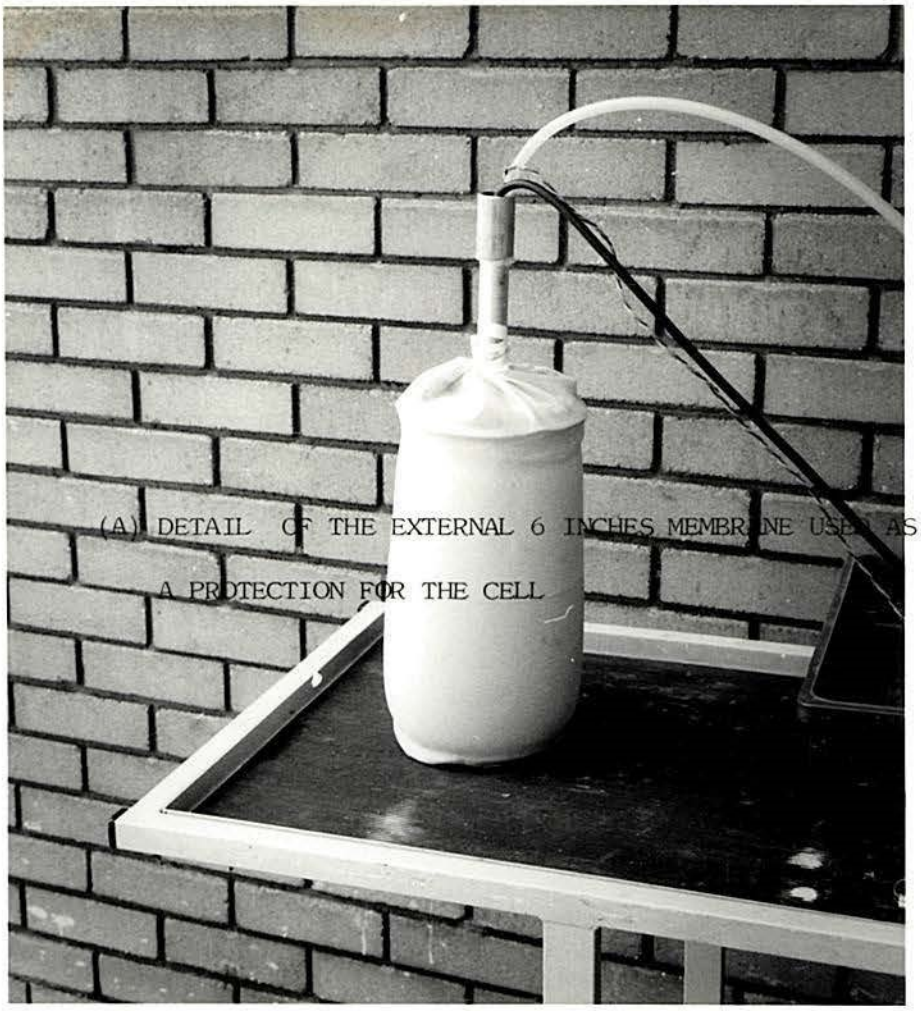
FIGURE 7.15b – BASIC COMPONENTS OF THE "NEW PRESSURE CELL"



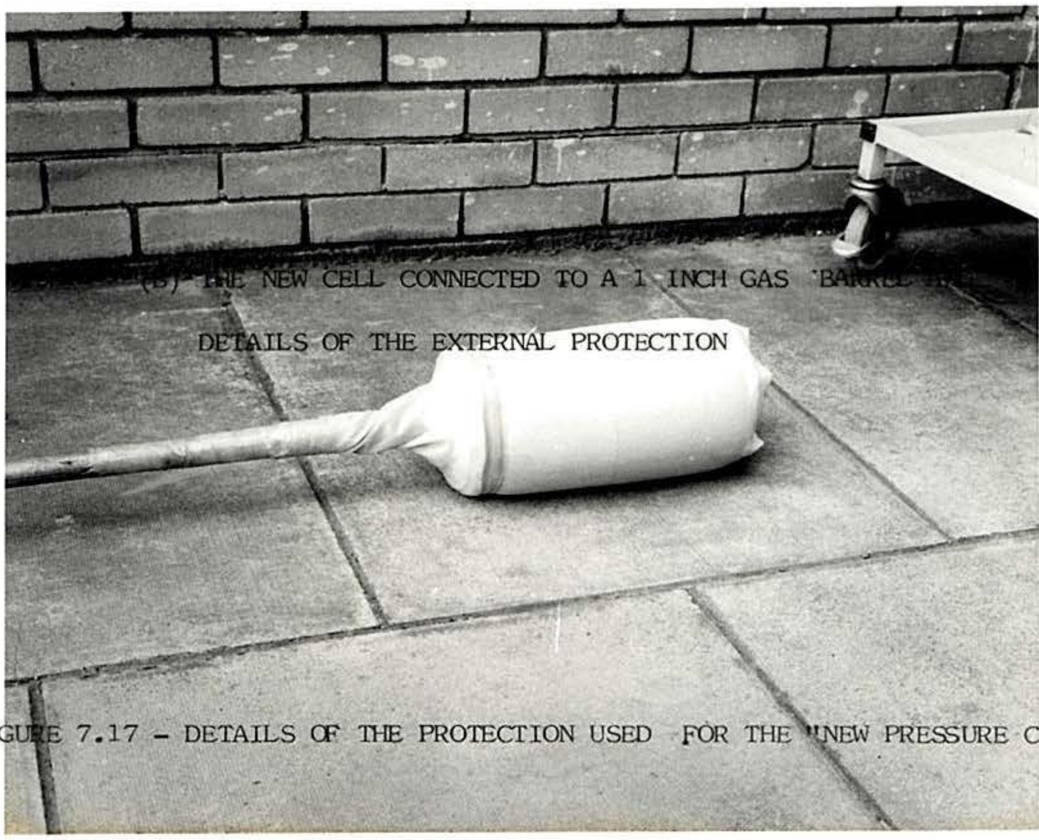
(A) THE NEW PRESSURE CELL WITH THE TOP PROTECTION

(B) DETAILS OF TOP AND BOTTOM PROTECTION

FIGURE 7.16 - DETAILS OF THE "NEW PRESSURE CELL"

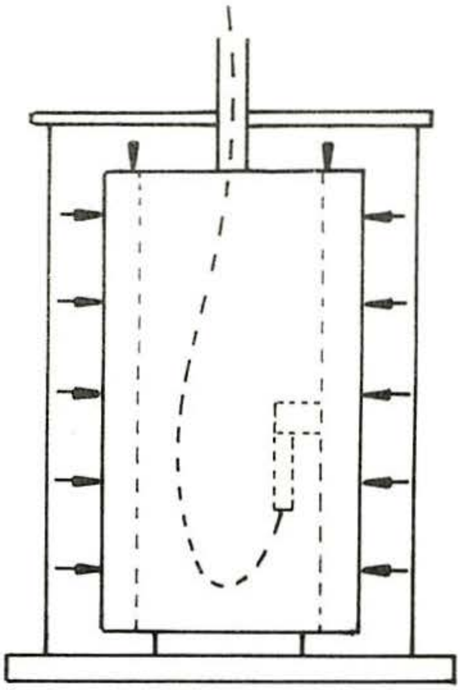


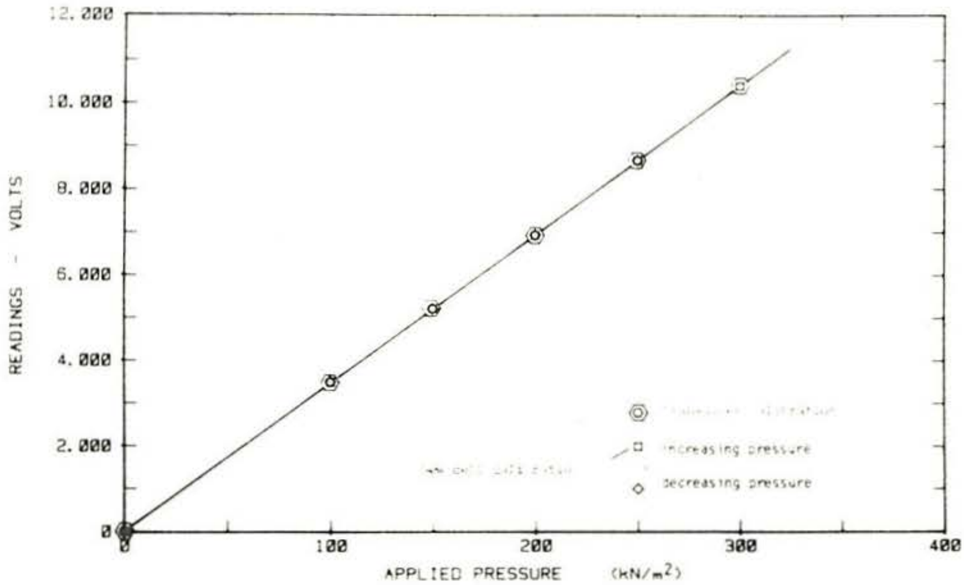
(A) DETAIL OF THE EXTERNAL 6 INCHES MEMBRANE USED AS A PROTECTION FOR THE CELL



(B) THE NEW CELL CONNECTED TO A 1 INCH GAS 'BARREL' AND DETAILS OF THE EXTERNAL PROTECTION

FIGURE 7.17 - DETAILS OF THE PROTECTION USED FOR THE "NEW PRESSURE CELL"





(B)

FIGURE 7.18 - CALIBRATION OF THE "NEW PRESSURE CELL" INSIDE A 6" TRIAXIAL CELL: (A) ASSEMBLY, (B) COMPARISON WITH THE TRANSDUCER CALIBRATION

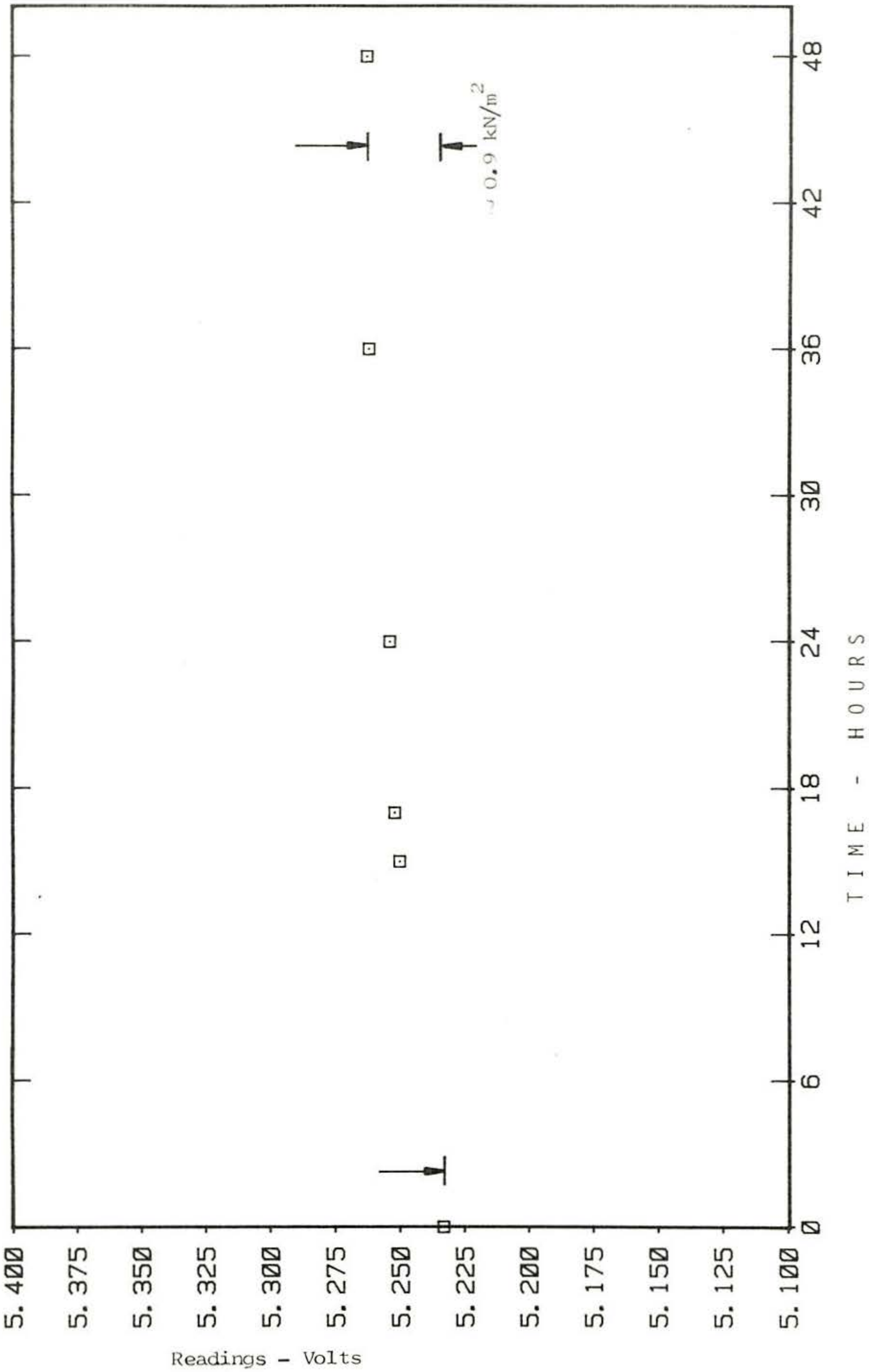
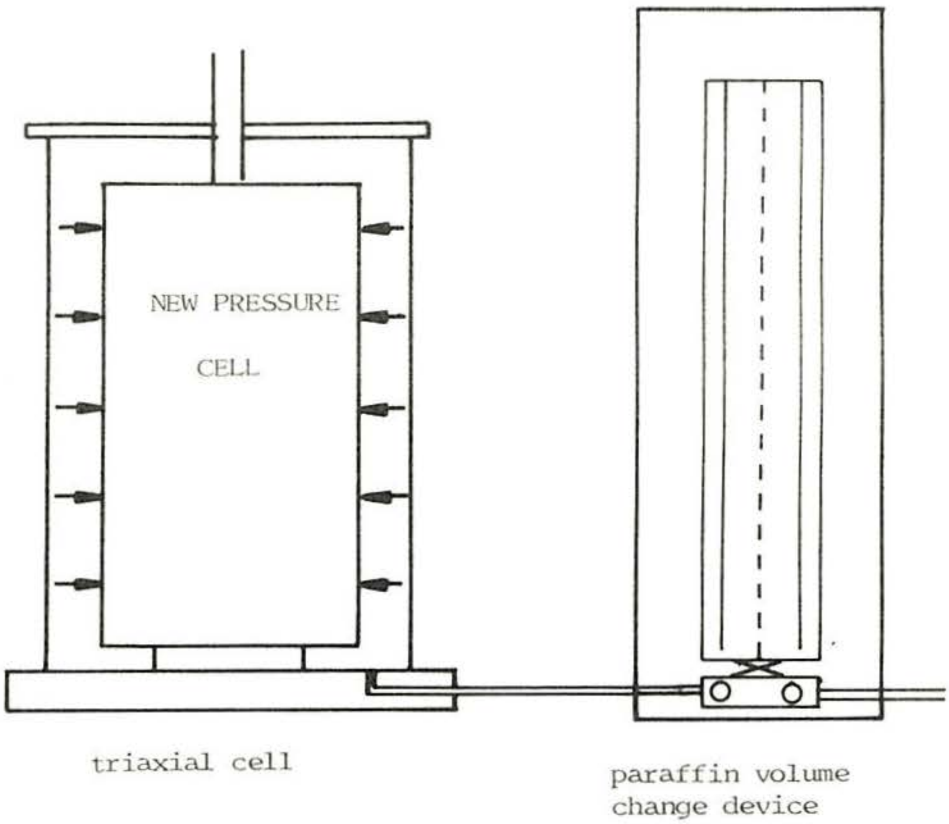


FIGURE 7.18(cont) - (C) "Warming up" effect on the transducer reading



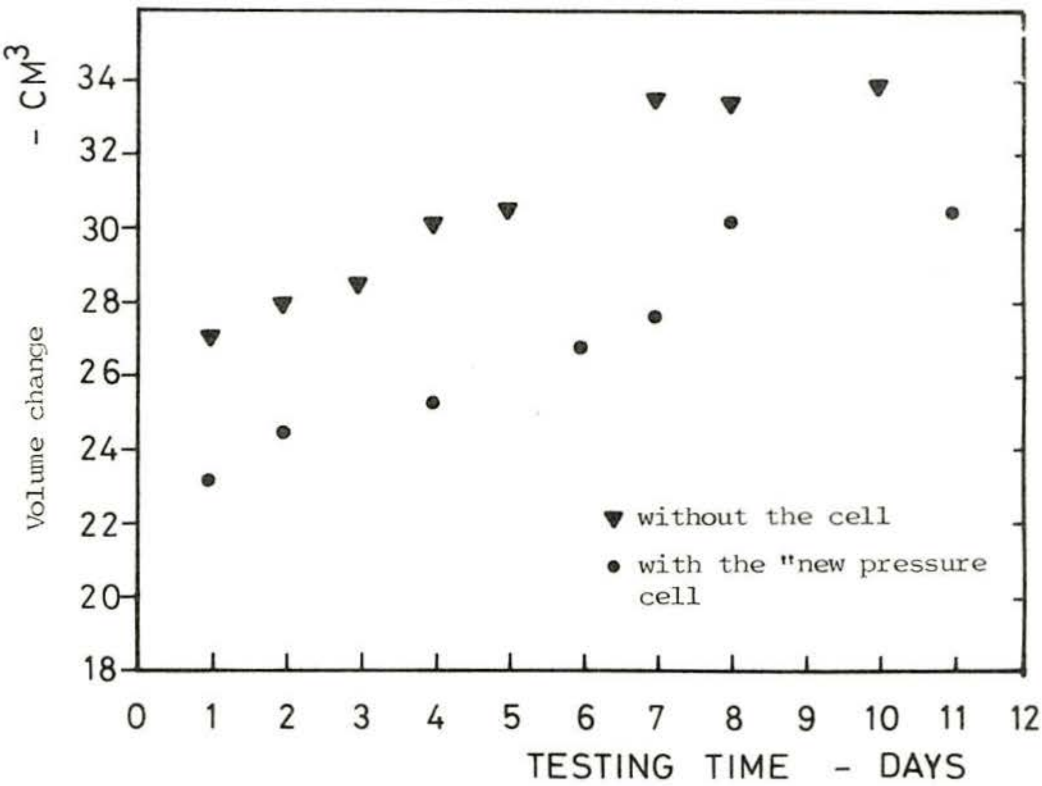


FIGURE 7.19 – VOLUME VARIATION UNDER CONSTANT PRESSURE (250 kN/m^2)

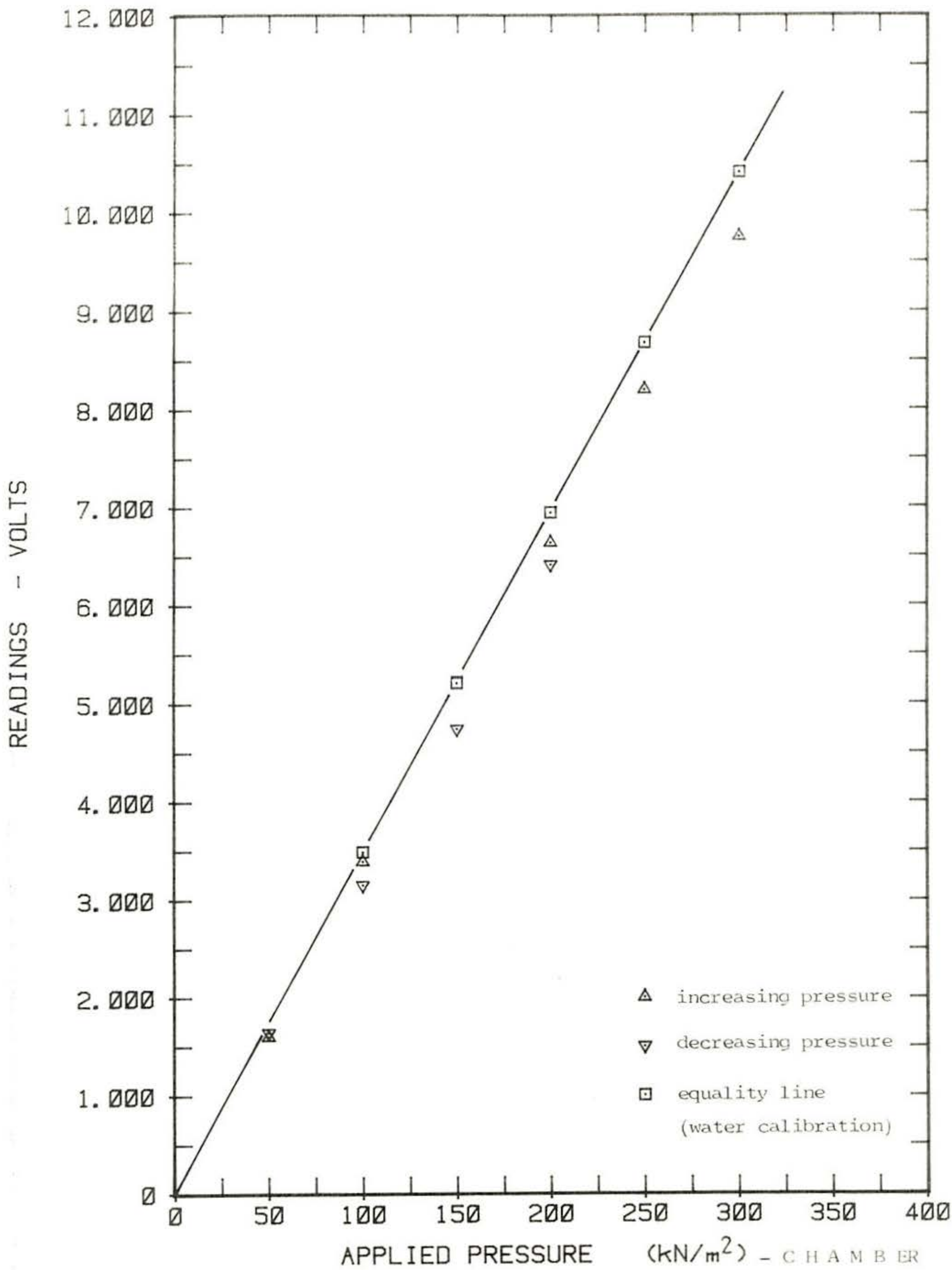


FIGURE 7.20 - RESULTS OBTAINED WITH THE NEW PRESSURE CELL INSIDE THE CALIBRATION CHAMBER

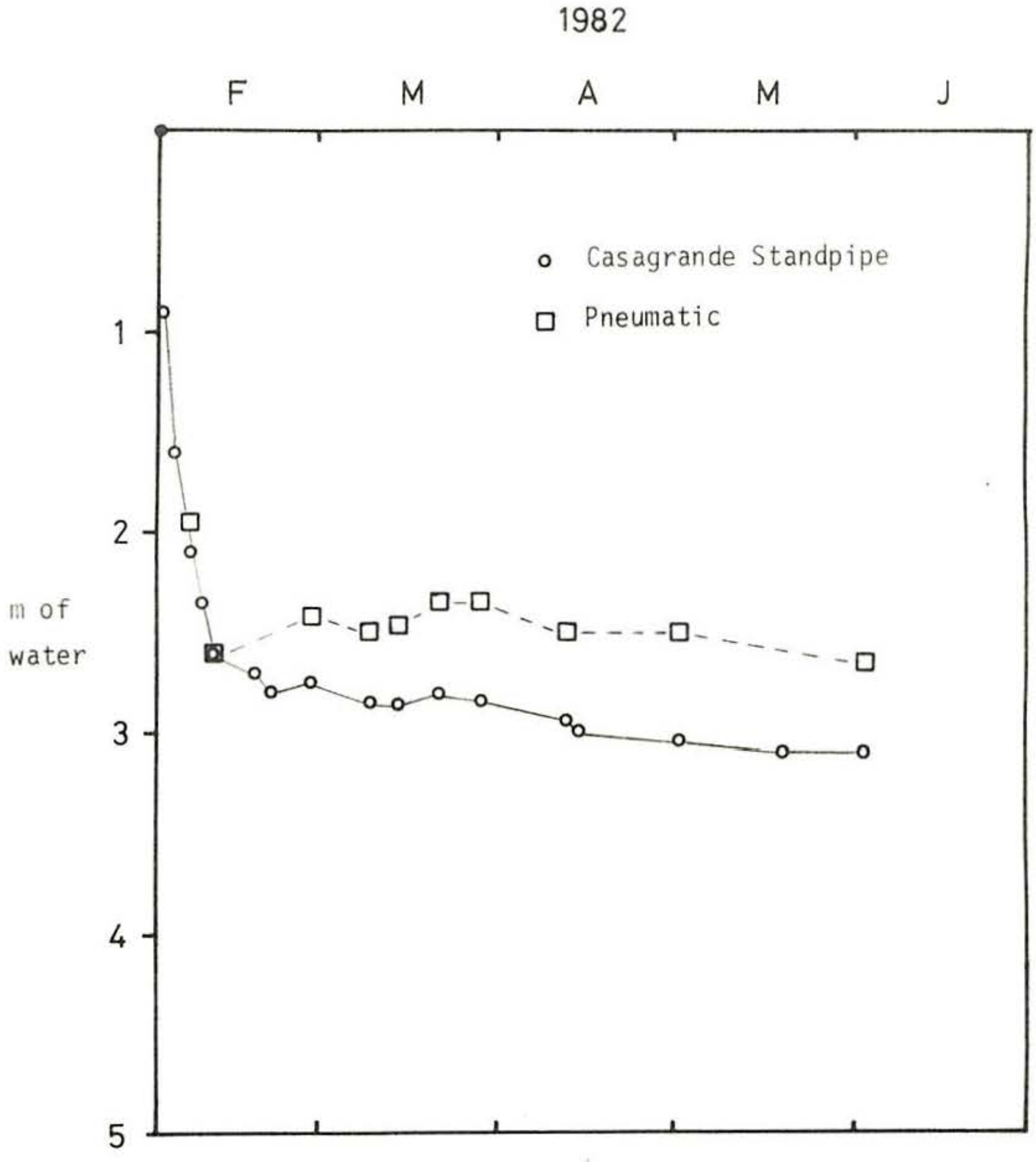


FIGURE 7.21 - FIELD RESULTS OF THE PIEZOMETERS READINGS

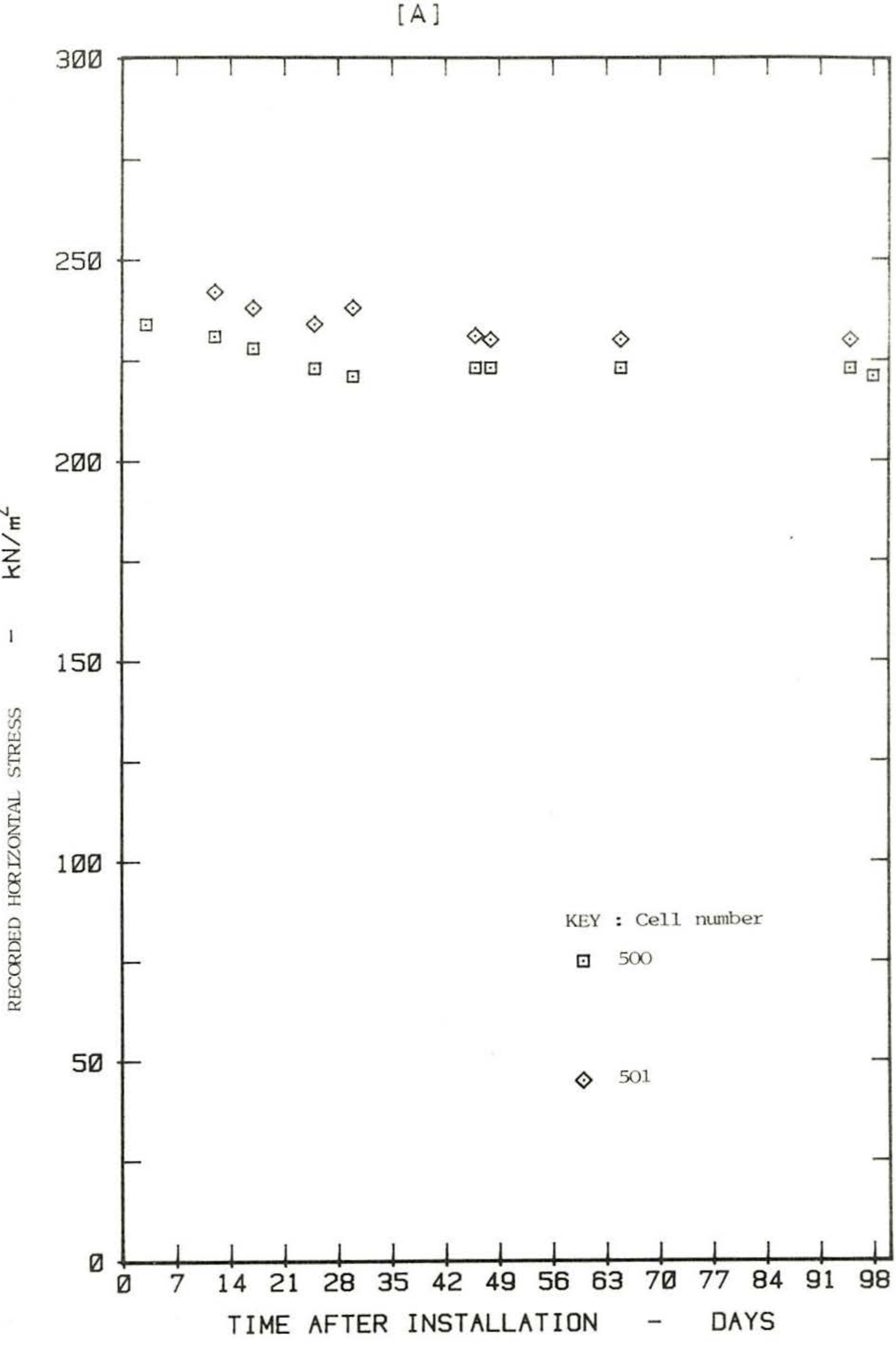


FIGURE 7.22 - RESULTS OBTAINED WITH THE "DRIVE IN" TOTAL STRESS CELLS

(A) RECORDED VALUES

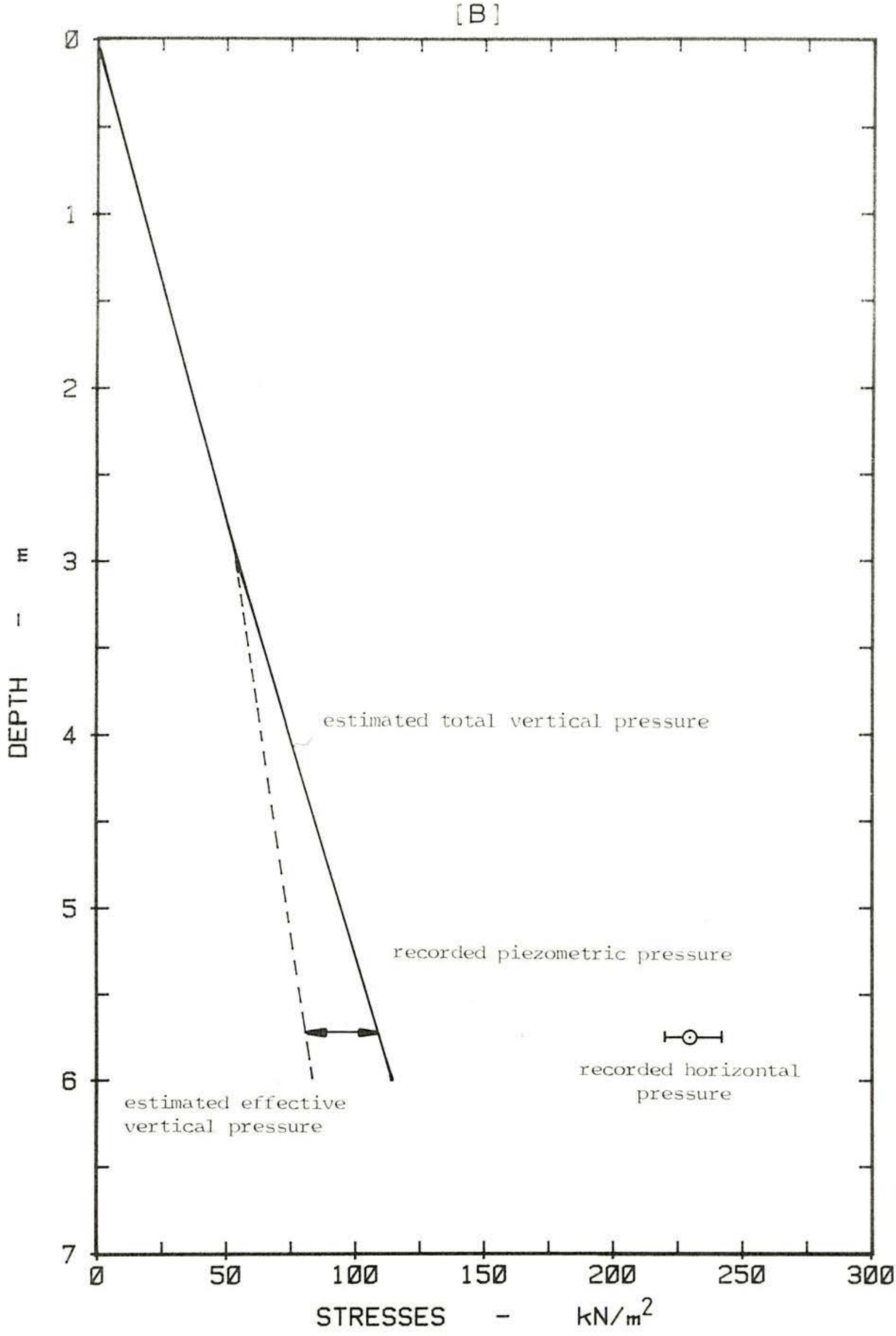


FIGURE 7.22 (cont) - (B) ESTIMATED VERTICAL PRESSURES, PIEZOMETRIC LEVEL AND RECORDED HORIZONTAL PRESSURES

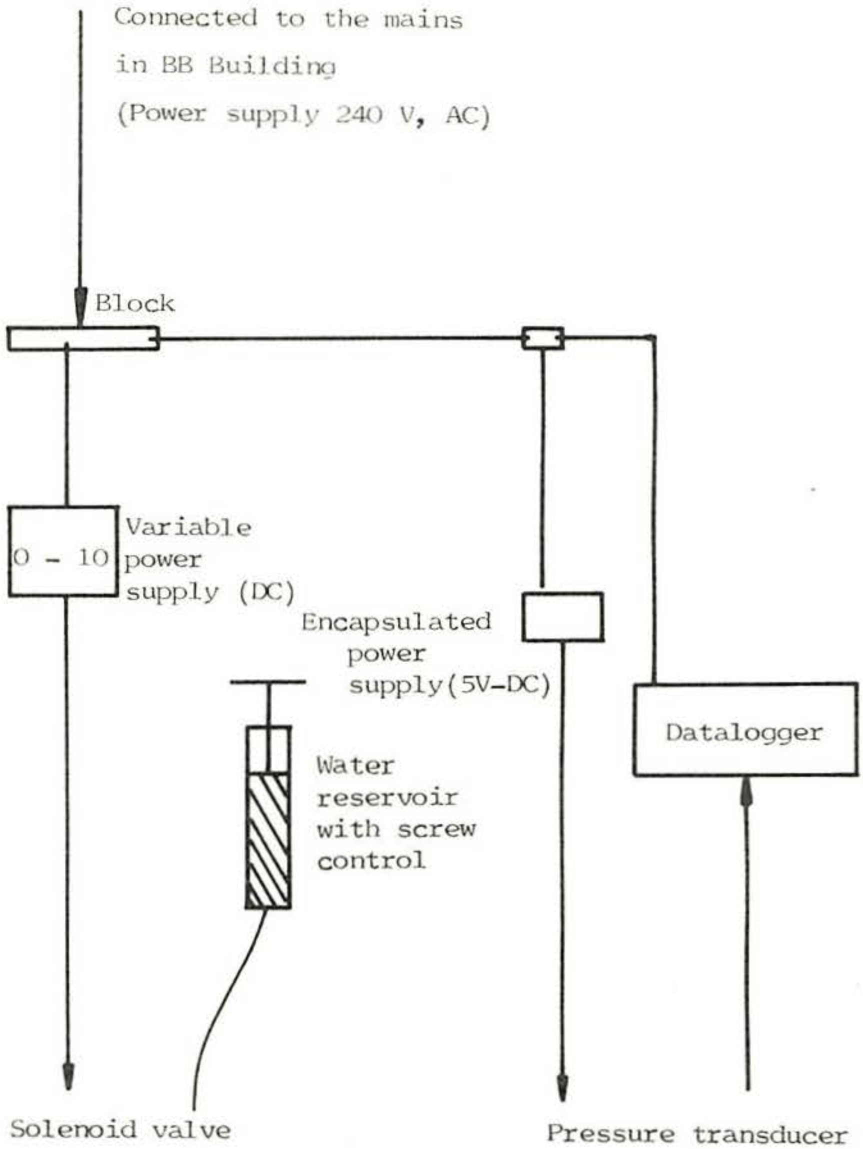


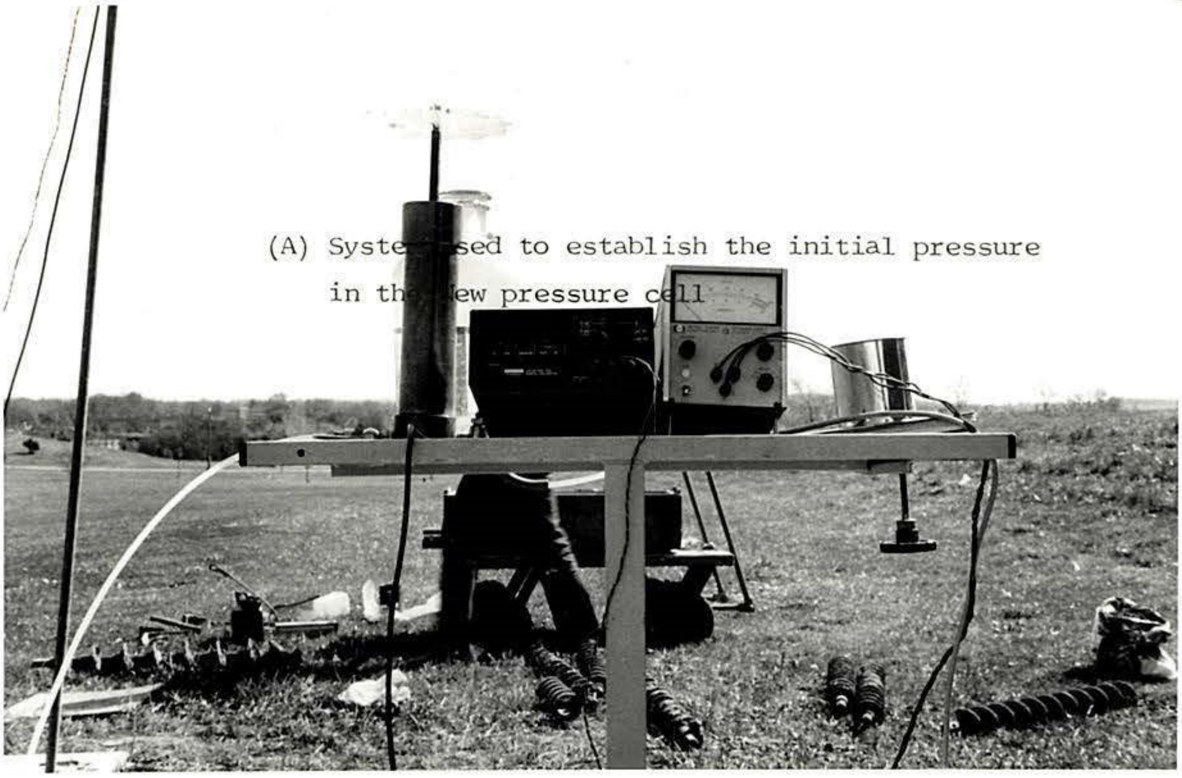
FIGURE 7.23 - SYSTEM USED TO ESTABLISH THE INITIAL PRESSURE LEVEL IN THE NEW PRESSURE CELL AND READOUT

(A) New pressure cell complete, ready for installation

(B) Detail of the model pile installation (New pressure cell in position)

FIGURE 7.24 - NEW PRESSURE CELL COMPLETE FOR INSTALLATION AND DETAIL OF PILE INSTALLATION

(A) System used to establish the initial pressure in the new pressure cell



(B) Installation completed

FIGURE 7.25 - DETAILS OF THE INSTALLATION OF THE MODEL PILE AND NEW PRESSURE CELL

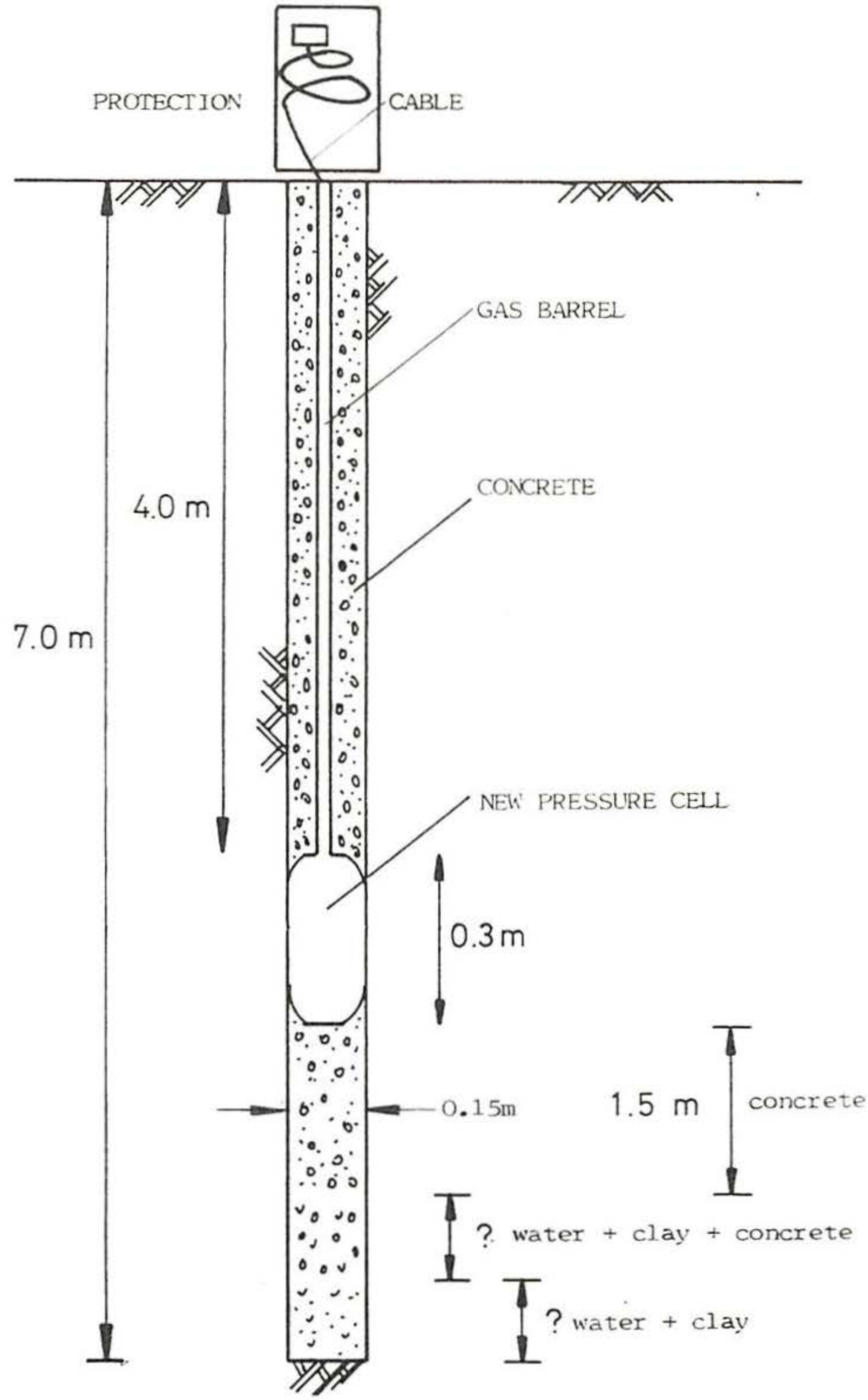
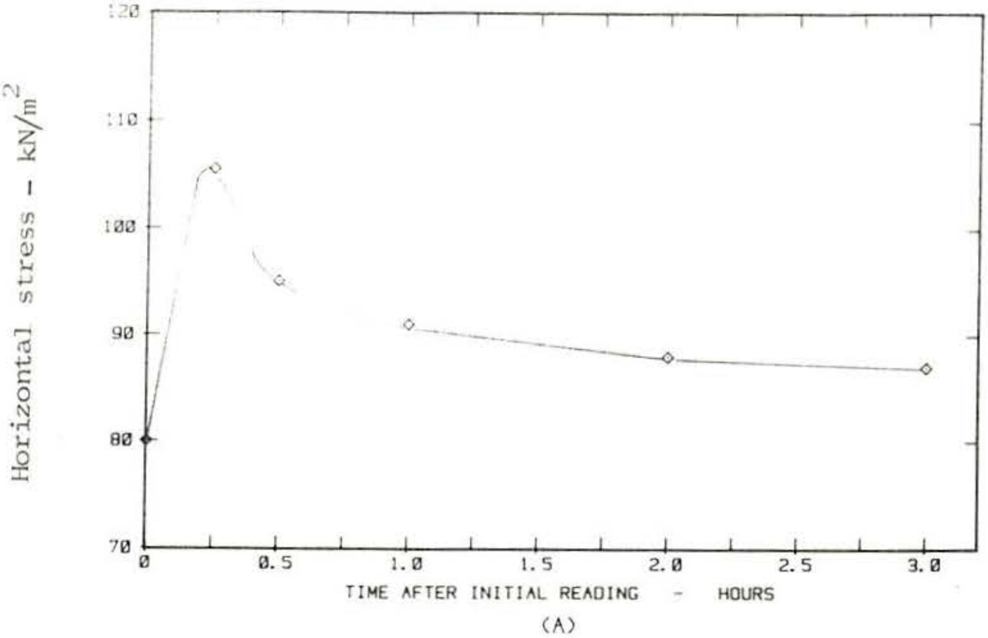


FIGURE 7.26 - FINAL CONFIGURATION OF THE MODEL PILE WITH THE NEW PRESSURE CELL



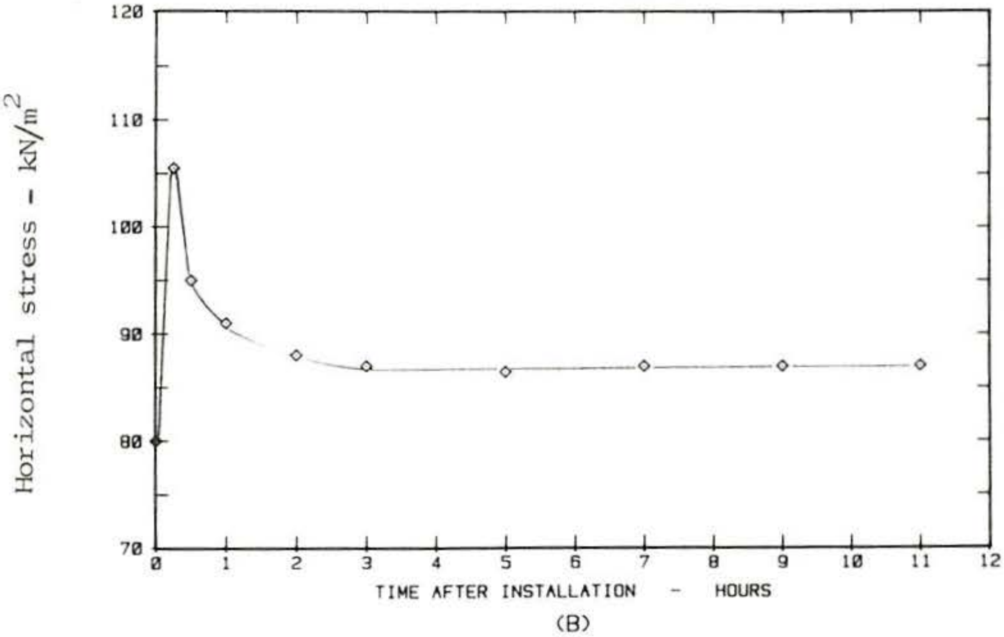
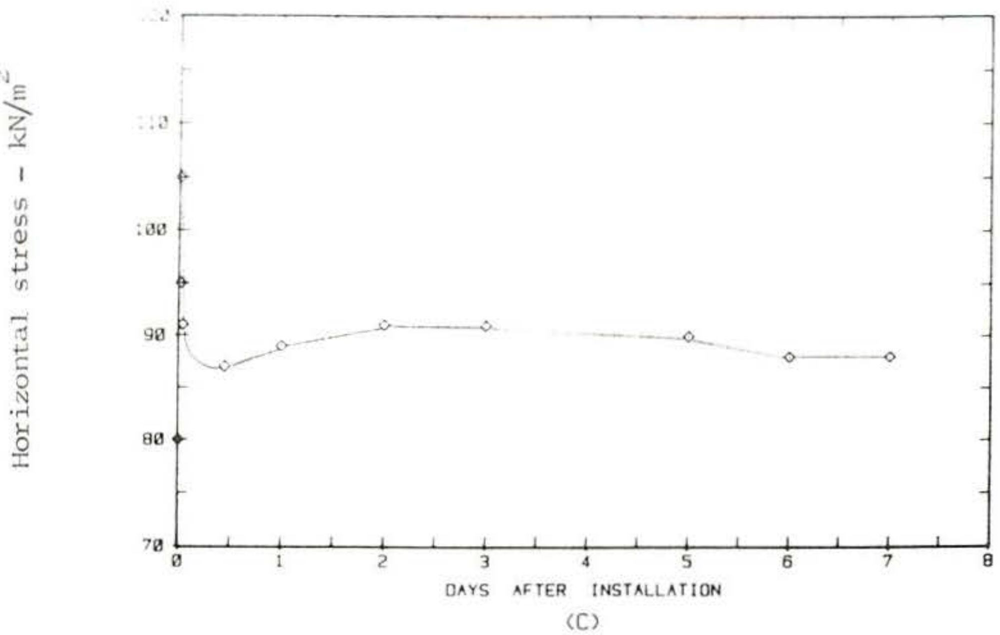


FIGURE 7.27 – RESULTS OBTAINED USING THE NEW PRESSURE CELL

(A) DURING INSTALLATION, UP TO 3 HOURS

(B) FIRST 12 HOURS



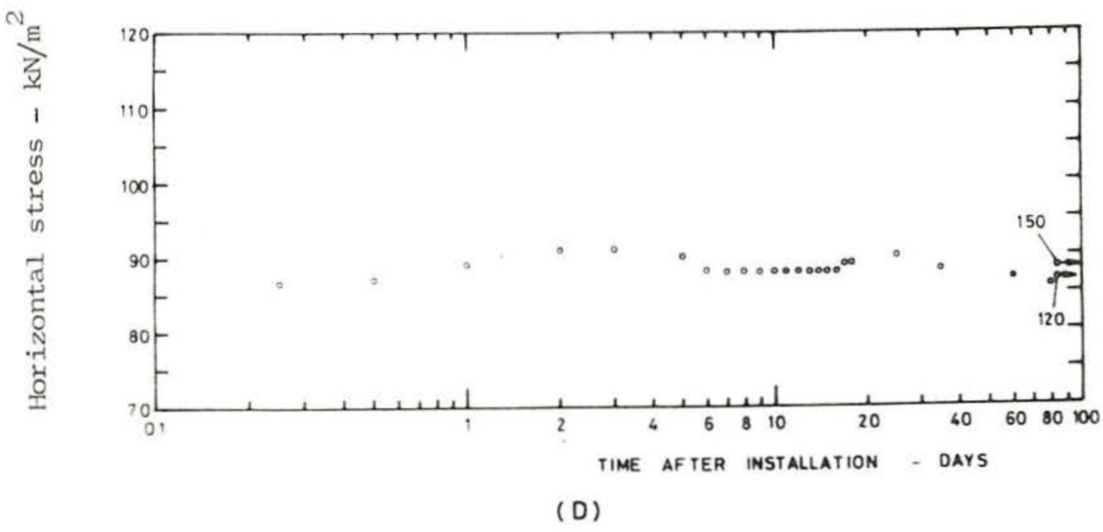


FIGURE 7.27 (cont) - (C) FIRST WEEK

(D) RESULTS UP TO 150 DAYS

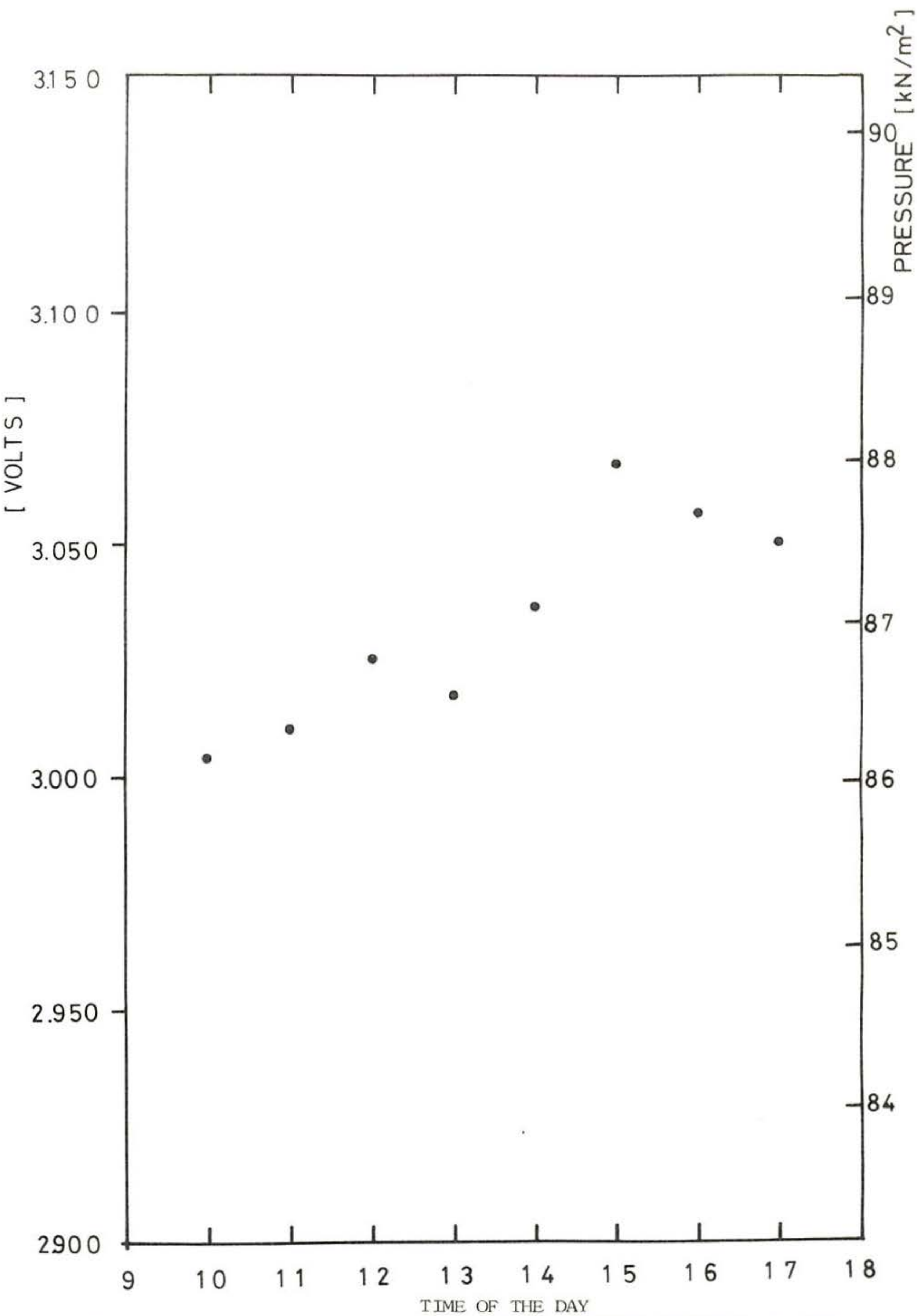


FIGURE 7.27 (cont) – (E) – TYPICAL SET OF READINGS OBTAINED 100 DAYS AFTER INSTALLATION

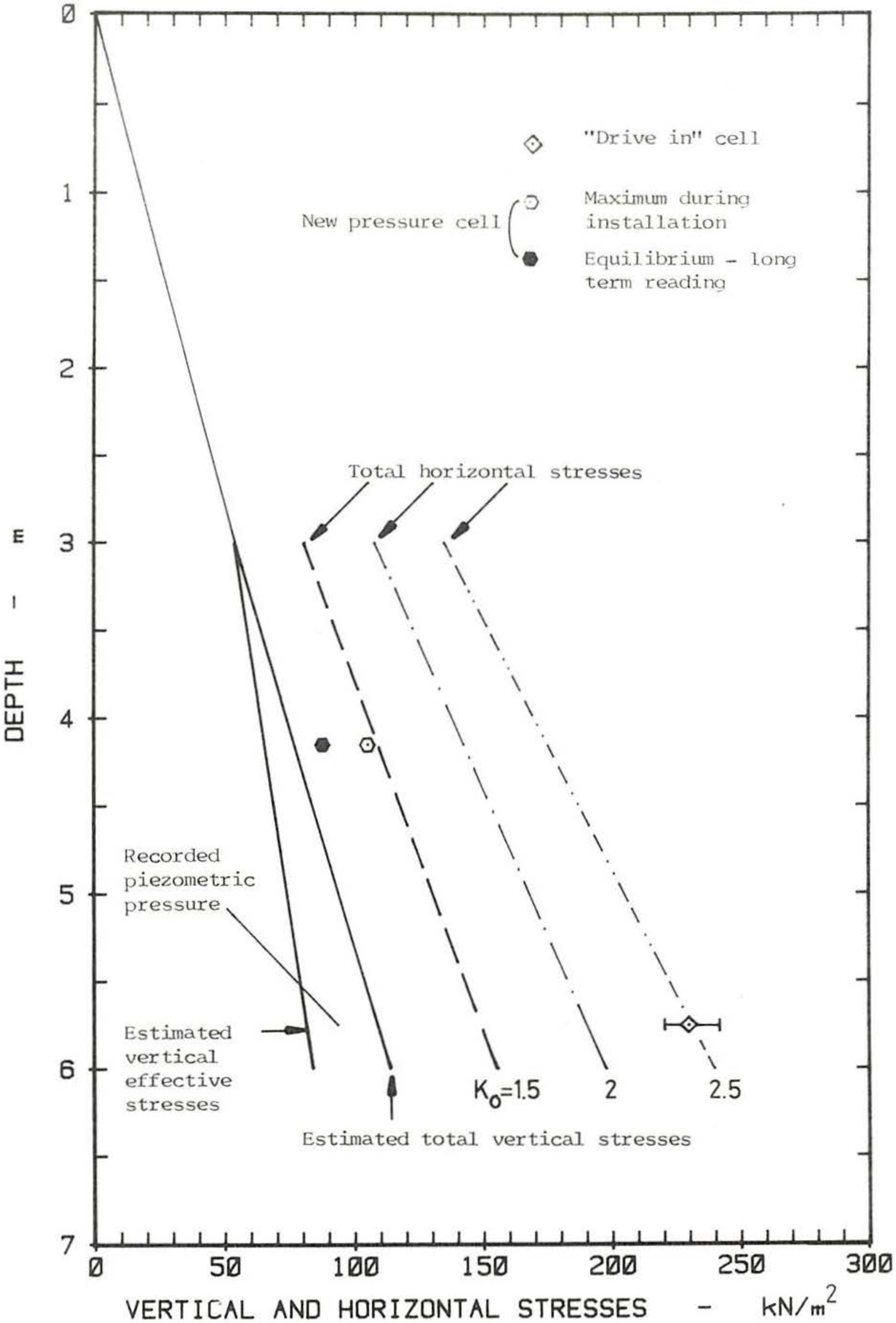


FIGURE 7.28 - "SUMMING UP" OF THE FIELD EXPERIMENTATION RESULTS AND ESTIMATED VERTICAL AND HORIZONTAL "IN SITU" STRESSES

CHAPTER 8 - GENERAL CONCLUSIONS AND RECOMMENDATIONS

CHAPTER 8 - GENERAL CONCLUSIONS AND RECOMMENDATIONS

The development of effective stress approaches to compute the shaft friction of bored piles can provide an improved understanding of the factors that affect shaft friction. There are fundamental advantages of the effective over the total stress approach. The total stress approach is based on the shear strength of the soil available prior to pile installation and loading, and undrained strength is test dependent and obtained in laboratory or field tests which do not represent the loading conditions around the pile shaft. The effective stress approach can be based on conditions before pile loading and uses drained (or effective stress) strength parameters, which are largely test independent.

At present, the effects of pile installation on soil properties and conditions are insufficiently understood to allow them to be simulated by theoretical models or numerical techniques, but theoretical models using the effective stress approach can provide a useful insight into the factors that affect pile capacity, and provide a framework for continued development. Thus the identification of the mechanisms involved in the modification of soil properties and conditions due to pile installation is a subject of major interest for the improvement of the understanding of pile behaviour.

New results have been obtained for moisture content changes in the soil adjacent to the pile due to the presence of fresh concrete, both in laboratory and field experiments. Similar variations in moisture content were found in long term experiments, showing that the variation in the soil's void ratio is permanent. The soil structure is stable under the new stress conditions. Some of the problems of such studies are the lack of accuracy of the traditional method used for the measurement of moisture content (oven drying), the difficulty in establishing the original moisture content before the contact with the fresh concrete, and the fact that one is measuring total instead of variation in moisture content.

A radiochemical method proposed to study the migration of the water from the fresh concrete to the soil proved to be successful. The major advantadges of the proposed method over the usual procedure are: (i) the overal precision of the new method is, at least, one order of magnitude better, (ii) it measures moisture content changes, due to the fact that it counts the radioactivity of the labelled water which comes from the concrete, and (iii) only very small test specimens are necessary for the counting, allowing the study of moisture migration using the same sample at different ages. Results of moisture migration obtained from one to seven days after contact between soil and fresh concrete showed the evolution of moisture content at different points, the results generally being in agreement with those obtained using conventional techniques. The method can be used both in short and long term laboratory experiments, and field tests, to provide more information on the migration pattern at different stages of cement hydration and pile age.

Fall cone tests to measure the variation of undrained shear strength of soil adjacent to the interface with the concrete proved to be a useful approach. Results of laboratory and field tests showed an unexpected similarity, bearing in mind the very different geometry. There is an increase in moisture content near the contact and a reduction in the undrained shear strength. The higher moisture content close to the interface between concrete and soil was not always followed by an equivalent reduction in shear strength, suggesting that the water flowing from the fresh concrete might have some components of the cement in suspension, or dissolved, that act as a stabilizer for the soil near the contact. The use of the fall cone can be widely explored in field experiments and could lead to a possible method of estimation of pile bearing capacity, provided the necessary experience was accumulated. Using the proposed approach, the effects of different installation techniques can be quickly assessed, the sensitivity of soil to the installation of bored piles in new areas or the effect of different concrete mixes can be studied preliminarily under laboratory or field conditions.

As a results of various experimental techniques, this study provides fresh light into the effects of fresh concrete on the surrounding soil, in the form of : (i) identification of the boundary conditions at the interface between the fresh concrete and the surrounding soil (and in particular the variation of the pore water pressure within the concrete), (ii) measurement of the effects of such conditions in the adjacent soil, in terms of pore water pressure variation and possible mechanisms explaining the permanent changes in both moisture content and undrained shear strength. The results obtained in the present work shows that immediately after concrete pouring, the pore water pressure at the boundary between the pile and soil is equal or close to the total stress due to the self weight of the fluid concrete, giving rise to a substantial increase in the pore water pressure in a narrow zone of the surrounding soil. Due to water migration and hydration of cement, the pore pressure reduces in that area and, at a certain stage, finally equalizes with the less affected zone beyond. It appears that the concrete plays a very important role in the installation related changes in the surrounding soil. It is the key to the understanding of the possible mechanisms affecting soil conditions, such as the variation in the stress regime, as a source of moisture, and probably by providing some form of stabilization for the soil very close to the contact between pile and soil.

The establishment of an effective stress approach to the study or design of bored piles is highly dependent on the knowledge of the actual stress regime acting upon the pile shaft. There are no field measurements of such state of stress in the literature, mainly due to technical problems with the instrumentation. The model instrumented pile showed that the proposed new instrumentation for the study of the stress regime during and after a bored pile installation was a successful approach. The equipment designed was highly sensitive, stable, robust and was able to measure the horizontal stresses without interfering with the stress field it was supposed to measure. From the results obtained it was revealed that, when the pile hole remains open for a long period (2 days), no re-establishment of horizontal stresses occurs during the first 150 days after pile installation. Due to the delay in pile installation, the ratio of estimated "in situ" total horizontal stresses and the confining stresses on the model pile shaft was of the order of 0.5. The measured stresses in the pile shaft were of the order of those of the fresh concrete. This may be the lower limit for the case of piles bored into overconsolidated **clays.**

Further efforts towards the identification of the changes in parameters and conditions that occur during a pile installation and the study of the effects of pile loading in the stress field should lead to the establishment of the real conditions of the soil failure in the vicinity of the pile shaft.

Far from providing the answers to the large number of questions related to the topic, this work has identified some basic mechanisms and established a more sound basis for the continuing study of the behaviour of bored piles in clays, apart from providing some new techniques and equipment useful in the improvement of the understanding of pile behaviour.

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APPENDIX 1

LATERAL PRESSURE OF FRESH CONCRETE

A 1.1. - EMPIRICAL STUDIES

A 1.2. - MATHEMATICAL MODELS

APPENDIX 1. - LATERAL PRESSURE OF FRESH CONCRETE.

The basic aspects of the mechanisms and proposed approaches to solving the problem of evaluation of the lateral pressure of fresh concrete are presented in this section. The available studies of formwork pressure can be divided into empirical approaches and mathematical theories.

A 1.1. - EMPIRICAL STUDIES

RODIN, 1952

The first comprehensive review of the previous work on formwork pressure was carried out by Rodin, 1952. According to his ideas, when the concrete is placed between forms, the aggregate settles until the grains are in contact and surrounded by a cement mortar mix. Rodin used the classic Terzaghi, 1934, work on the effect of the movement of the retaining structures in the pressure distribution of granular soils. Rodin claims that arching generally occurs and this, combined with friction between the concrete and form and chemical stiffening of the mortar, is the prime cause of the lateral pressure distribution. Figure A1.1shows Rodin's proposed lateral pressure distribution. For simplicity, Rodin recommends that the shape of the pressure distribution be taken as linear between zero and the maximum pressure. Using his model of pressure, Rodin fitted empirical curves to the experimental results available at the time. Using his judgment, Rodin produced correction factors to allow for different mix proportions, workability and temperature from the basic 1: 2: 4 concrete at 70 $^{\circ}$ F for hand placed concrete used as standard.

for hand placed concrete:

$$P_{max} = 17 \text{ hs} = 19.16\ (3.28\ R)^{\frac{1}{3}}\ (kN/m^2) \qquad \text{Eq. A.1}$$

and for vibrated concrete:

$$P_{max} = 24 \text{ hs} = 25.9\ (3.28\ R)^{\frac{1}{3}}\ (kN/m^2) \qquad \text{Eq. A.2}$$

where:

P_{max} = maximum lateral pressure (kN/m^2)

R = rate of placing (m/h)

hs = head at which the maximum pressure first occurs (m)

Due to lack of data at the time, the importance of setting time, size and shape of the form was acknowledged but not assessed, without any relationship or correction factors proposed.

AMERICAN CONCRETE INSTITUTE (ACI), 1958

The ACI Committee 622 review, 1958, on formwork pressure used Rodin's 1952 survey extensively and also the work of Hoffman, 1943; Schjødt,1955; and the Ontario Hydro-Electric Power Commission, 1957. No new theory on formwork pressure was presented or new experimental work done, but they make use of unpublished data. The current ideas at the time of publication were discussed. From the review, the Committee concluded that rate of placement, concrete temperature and vibration were the most important factors controlling formwork pressure.

The basic general equation proposed was:

$$P_{max} = C\,1\ (1 - \frac{C\,2\,R}{t})\quad (kN/m^2) \qquad \text{Eq. A.3}$$

where:

P_{max} = maximum lateral pressure (kN/m^2)

C 1 = a constant depending on the unit weight of mix

C 2 = a constant depending on the consistency of the concrete

R = rate of placement (m/h)

t = temperature of concrete (^{o}C)

For ordinary Portland cement, a unit weight of 23.6 kN/m^3 and a slump of 100 mm, the equation A.3 becomes:

$$P_{max} = 7.2 + \frac{785\ R}{t+17.8} \quad kN/m^2 \qquad \text{Eq. A.4}$$

The minimum value of 7.2 kN/m^2 was to allow for construction, surcharge and re-vibration loads. Some modifications have been proposed (1962, 1977) for walls, but for columns there has been no change.

Equation A.4 was applied up to a maximum value of 144 kN/m^2. With very high rates of placing, the assumption was made that the whole depth of the column is subjected to vibration and the Committee recommended that the maximum pressure be taken as 144 kN/m^2 at one third the height of the column above the base and varying linearly to zero at the top and the bottom (Figure A1.2). This recommendation was based on unpublished field tests.

The Committee also recommended that the height of the column should not exceed 5.5 m and if it did so, the column should be concreted in 5.5 m lengths with periods of at least two hours between concreting successive lengths. This approach was suggested in order to allow some chemical stiffening to take place and consequently reduce the lateral pressure.

Since first presented in 1958, the ACI method has been widely used not just for formwork design (the small number of changes shows the safety of the method) but as an indication of the actual pressures on large bored piles by some American writers (O'Neill and Reese, 1978; Stewart and Kulhawy, 1981).

ERTINGSHAUSEN, 1965

In 1965, Ertingshausen presented the results of his comprehensive research in laboratory conditions and on real structures and from statistical analysis of the results the following equations were suggested for maximum pressure:

a) rates of placing (R) up to 4 m/hour:

$$P_{max} = 30\ (R)^{\frac{1}{4}} \qquad (kN/m^2) \qquad \text{Eq. A.5}$$

b) rates of placing (R) between 5 and 6 m/hour:

$$P_{max} = 36\ (R)^{\frac{1}{4}} \qquad (kN/m^2) \qquad \text{Eq. A.5a}$$

The suggested design envelope is shown in Figure A1.3. A discussion on the shape of the pressure envelope highlighted two main factors: shrinkage of the concrete and the flexibility of the formwork.

CIVIL ENGINEERING RESEARCH ASSOCIATION - (CERA), 1965

In 1965 a study of formwork pressure was carried out by CERA. Data from site measurements were analysed but no new theory on formwork pressure was produced.

The empirical equation proposed as the best fit for the values obtained during the study was of the form:

$$P = \frac{D.\ R.\ t}{1+C\ \left(\frac{t}{t_{max}}\right)^4} + 1.89\ (2.44-R) \qquad \text{Eq. A.6}$$

where:

P = lateral pressure (kN/m^2)

D = density of the concrete (kN/m^3)

t = time since commencement of placing (hours)

t_{max} = stiffening time of the concrete (hours)

C = factor depending on the workability of the concrete and the continuity of vibration. Typically it ranges from 0.13 to 0.27 for structural concrete

R = rate of placing (m/h)

When $t = t_{max}$:

$$P_{max} = \frac{D \cdot R \cdot t_{max}}{1 + C} \qquad \text{Eq. A.7}$$

Various graphical aids (CERA, 1965, Concrete Society, 1972, CIRIA, 1969, CIRIA, undated) have been produced. Figure A 1.4 shows the pressure design envelope suggested by CERA, 1965.

RITCHIE, 1962a, 1962b, 1963 and RITCHIE AND MCDOWALL, 1969

The work done by Ritchie, 1962 a and b, 1963 and Ritchie and McDowall, 1969, introduced some new techniques and concepts to the subject. Initially the following variables were studied: mix proportions, workability, method of compaction, rate of pour and formwork size. The properties of fresh concrete were studied using the vane test and triaxial tests. Using full sized columns the authors produced data on formwork pressure. The conclusions that are of interest in understanding the mechanisms relevant to large bored piles are that:

(i) - an increase in slump does not necessarily produce an increase in lateral pressure;

(ii) - there were large differences in lateral pressure for equal slump but different cement to aggregate ratios;

(iii) - an increase in angularity of the aggregate , at constant water/cement ratio, reduces the lateral pressure;

(iv) - an increase in the surface area of the cement at constant water/cement ratio makes little difference with rich mixes.

Based on the results of the full investigation the authors presented their concept of formwork pressure, considering the changes in a single lift of concrete which has successive lifts of concrete placed above it. When the lift is first placed (and vibrated) it acts as a fluid of density equal to that of the concrete. When the vibrator is withdrawn, the concrete congeals and regains its viscosity. The cohesion of the concrete balances most of the shearing stresses and the aggregate remains suspended in the cement paste matrix.

When a further slug is placed the vibration and load transmitted to the lower slug will cause it to deform plastically and produce hydrostatic pressure. During this period it is suggested that some particle contact will be established.

Eventually the placing of a new slug of concrete and the transmitted vibration will cause the formation of a continuous intergranular structure involving all the solid constituents. Additional lateral pressure may still be induced by pore fluid. As the shearing stresses are transferred to the intergranular structure, a compression of the volume occupied by the particles occurs resulting in the expulsion of interstitial cement paste and water, and transference of vertical pressure to the walls.

Once maximum density has been attained, further loads cause a dilation of the structure and a volume increase, resulting in the pore fluid being drawn back into the interstitial spaces, reducing both the lateral and vertical pressures. Further lateral pressure reduction are caused by thixotropic regain and the formation of chemical bonds.

Other references concerning the mechanisms involved and a study of the different factors affecting formwork pressure are Adam et al, 1965 and Ore et al, 1968.

A 1.2. - MATHEMATICAL MODELS

The classical theories of earth pressure and silo design have been adapted by researchers on pressure in formwork.

Guerrin, 1950, presented his approach considering different categories of concrete:

- liquid concrete with no internal friction;
- poured concrete having an angle of internal friction of 20°;
- stiff concrete (angle of internal friction of 30°);
- concrete after compaction by vibration ($\phi = 50^{\circ}$);
- concrete after compaction by rodding ($\phi = 30^{\circ}$)

The given angle of friction between the concrete and formwork, ϕ', was 23° for timber and 14° for steel. For wide sections, poured concrete was split into a water component and a solids component and the maximum pressure for a mix with a density of 2.2 was given as :

$$P_{max} = 10\ h_s + 12\ h_s \cdot c \quad (kN/m^2) \qquad \text{Eq. A.8}$$

where:

P_{max} = maximum pressure (kN/m^2)

h_s = product of the rate of placing and the setting time (m)

c = a coefficient given as 0.413 for timber and 0.445 for steel

With vibrated concrete, the material was assumed to act as a fluid to a depth of 0.8 m and the maximum pressure given by the equation:

$$P_{max} = 0.8\ D + 0.13\ D\ (h_s - 0.8) \quad (kN/m^2) \quad \text{Eq. A.9}$$

where:

D = concrete density (kN/m^3)

When considering wide sections where arching was considered to be insignificant, many researchers have used the Rankine coeficient of earth pressure K_a or even Coulomb theory. Different approaches are based on different variations of K with time (Böhm, 1929; Hoffman, 1943; Toussaint, 1950; Olsson, 1953 and Muhs, 1955).

The first attempt to introduce a component of pore water pressure into formwork pressure analysis was carried out by Schjødt, 1955.

In his analysis the following variables have been considered:

setting time

workability of the concrete

weight of the concrete

pore water pressure

rate of placing

depth of vibration

smoothness of the formwork (for arching)

permeability of the formwork (in connection with the pore water pressure)

cross section of the formwork

The concrete model used to develop his formula was that the concrete acts as a fluid during vibration but as soon as it ceases, the material behaves as a solid porous material with pore water pressure. During the fluid stage the pressure curve follows the hydrostatic pressure.

Schjødt derived equations for formwork pressure with and without friction between the concrete and the form. The main problem with the proposed equations, appart from the complexity of some coefficients, is the selection of the material constants.

Witte, 1961, studied the variation of properties of fresh concrete with time using triaxial tests. He tried to reproduce the standard technique to assess K_o for granular soils (adjusting the horizontal stresses to prevent the specimen deforming and referring to the ratio of stresses as λ). At the beginning of the test λ was assumed by the author to be equal to unity. Experimental results lower than expected by the author were attributed to frictional losses in the equipment.

Witte considered that the reduction of λ with time should follow the exponential form and the experimental results from 3 hours onwards fitted reasonably this assumption (Figure A1.5).

Levitsky, 1973, presented an analytical approach for evaluating the formwork pressure. The formulation presented uses a physical model, assuming that the shape of the pressure curve is a result of simultaneous hardening and shrinkage. In 1975 the same author produced an analytical model for form pressure based on a viscoelastic model. He suggested that the relaxation in the form pressure after the maximum was originated by the relaxation on the formwork. Using his model, the author was unsuccessful in correlating experimental data with predicted values.

A new theory of concrete pressure on formwork in wide sections was presented by Harrison, 1979, considering the principle of effective stress . The author considered the problem of variation with time of the pore pressure and the so called coefficient of effective pressure, depending on:

- dispersal of the hydrodynamic excess
- development of suction forces
- changes in the concrete head
- vibration

Figure A1.6 shows the components of vertical and horizontal pressures. Figure A1.7 shows the effect of the consolidation process on concrete pressure.

The mathematical model proposed by Harrison was developed using a modified consolidation theory; a combination of the equation that describes the consolidation process with the development of suction forces (Figure A1.8). No solution was proposed to the equation that defines the rate of change of pore water pressure with time for a constant rate of placing (simplest case), mainly because of the problems of establishing the numerical values of the coefficients presented in such equation.

According to the author, the major advantage of the presented theory is its ability to provide a logical framework that can be used for interpretation existing and new data.

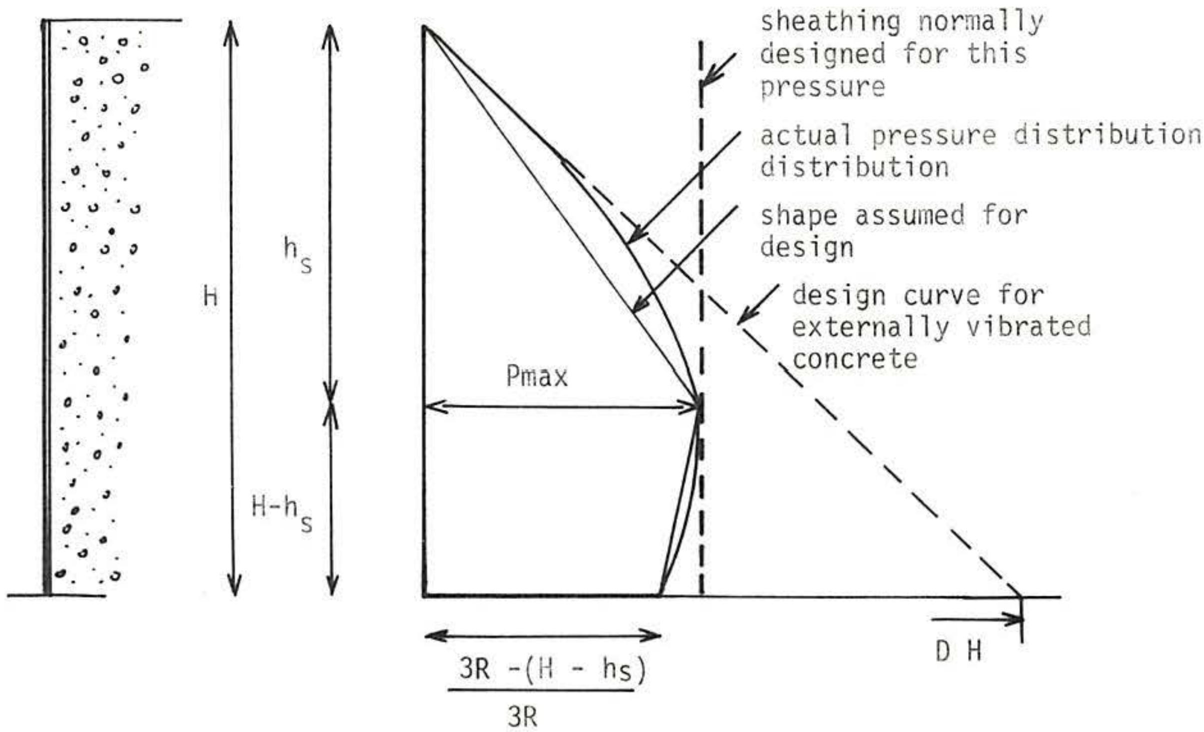


FIGURE A1.1- LATERAL PRESSURE DISTRIBUTION ACCORDING TO RODIN, 1952

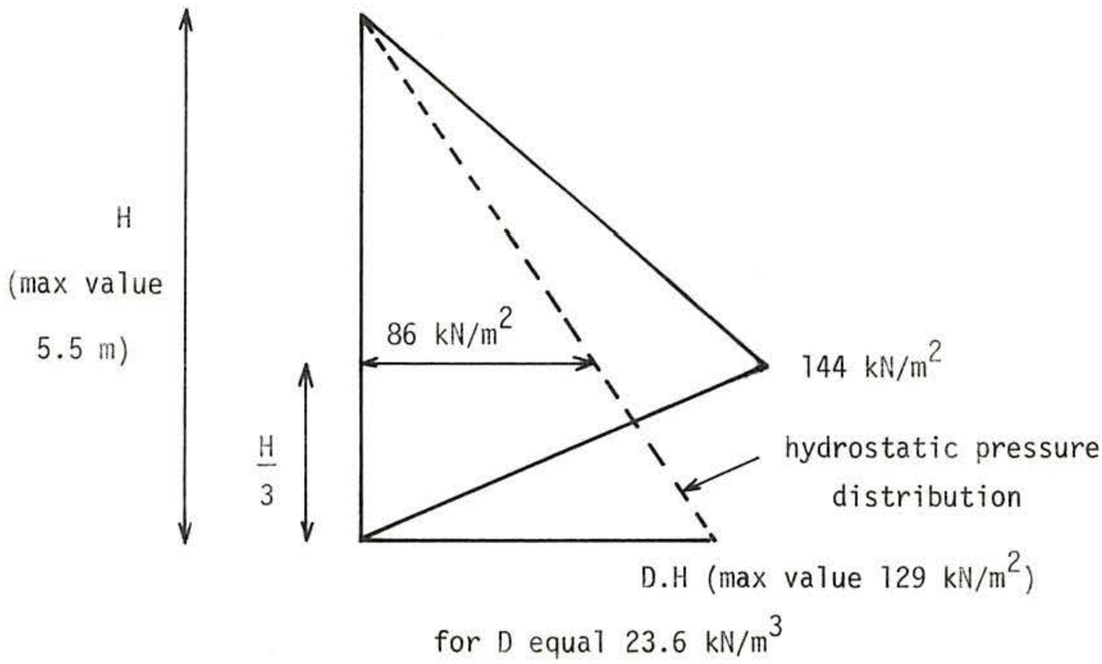


FIGURE A 1.2- AMERICAN CONCRETE INSTITUTE - COMMITTEE 622's PRESSURE DISTRIBUTION FOR UNLIMITED RATES OF POUR IN COLUMNS

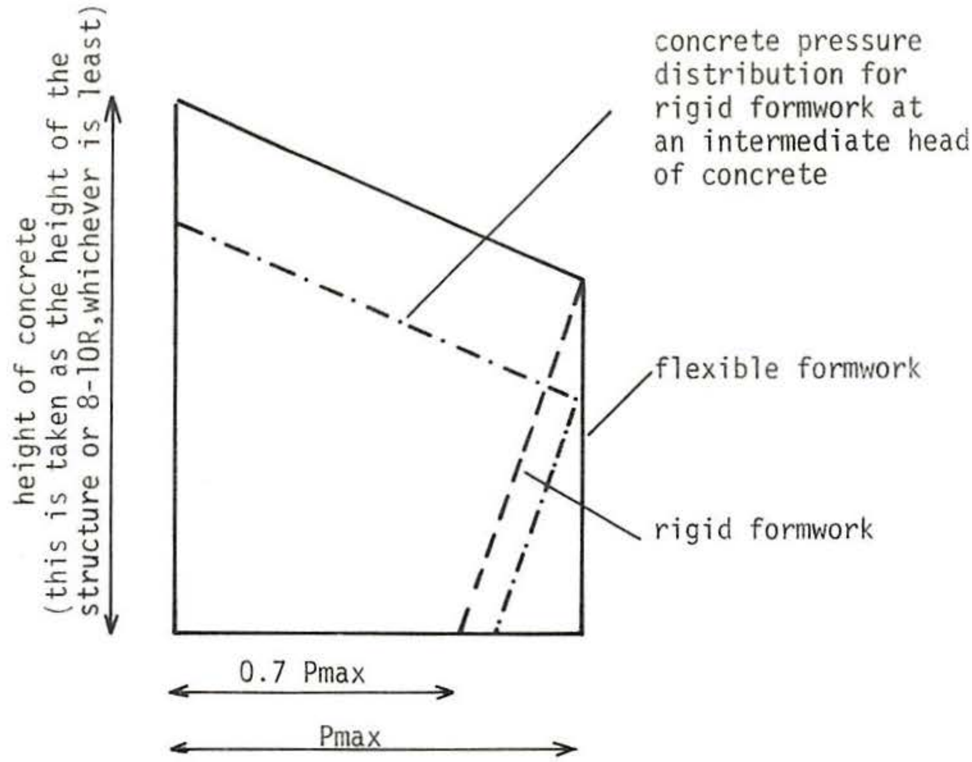


FIGURE A1.3- FORMWORK PRESSURE ENVELOPE, AFTER ERTINGSHAUSEN, 1965

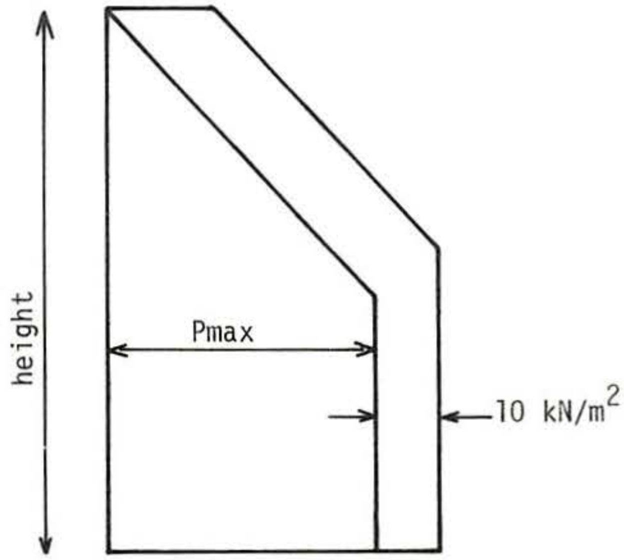


FIGURE A1.4- PRESSURE DESIGN ENVELOPE, CERA, 1965

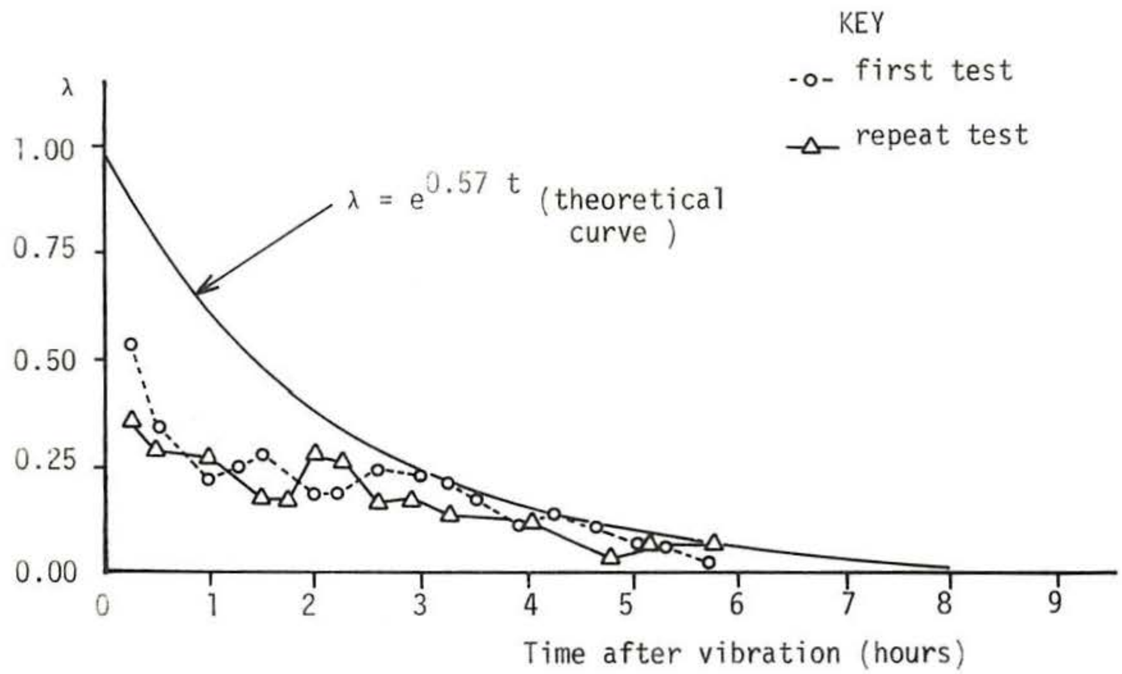


FIGURE A1.5- TWO TRIAXIAL TEST RESULTS FOR VALUES OF λ FOR VIBRATED FRESHLY MIXED CONCRETE AT $20^{o}C$, WITTE, 1961

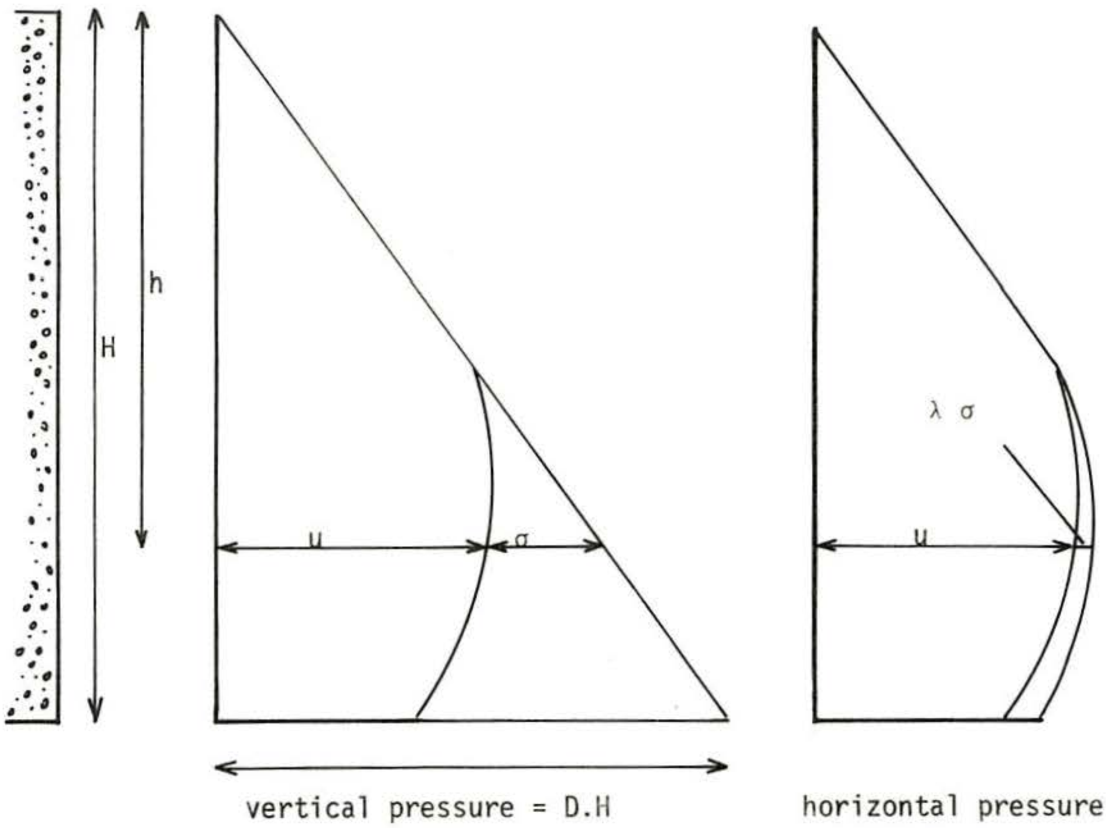
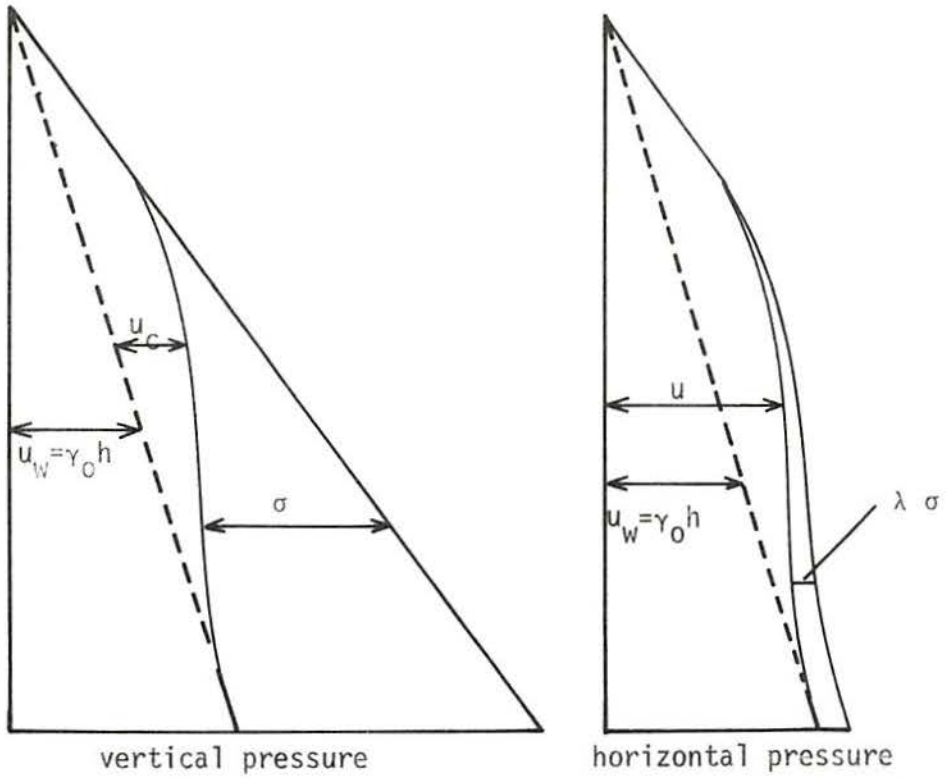


FIGURE A1.6- THE COMPONENTS OF THE VERTICAL AND HORIZONTAL PRESSURE ACCORDING TO HARRISON, 1979



Pore water pressure $u=u_W+u_C$; $u_W=\gamma_O h$ (static water head); $u_C=u-\gamma_0 h$ (hydrodinamic excess)

FIGURE A1.7- THE EFFECT OF THE CONSOLIDATION PROCESS ON CONCRETE PRESSURE (HARRISON, 1979)

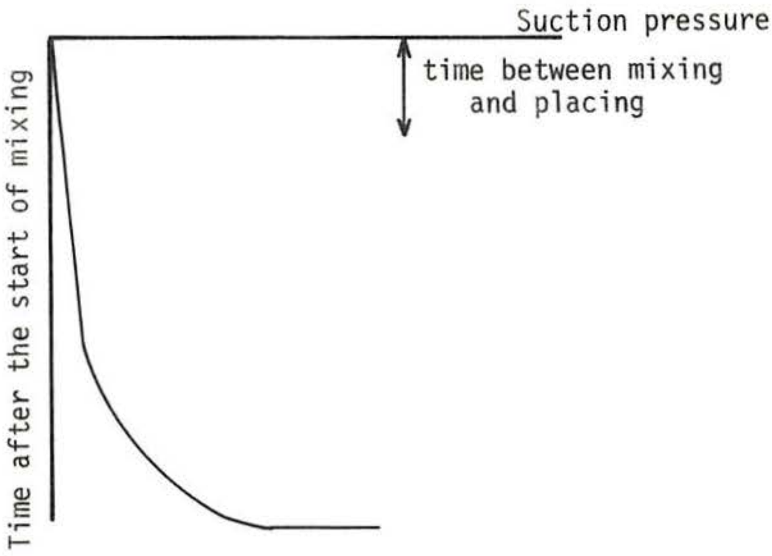


FIGURE A1.8- THE DEVELOPMENT OF SUCTION PRESSURES (HARRISON, 1979)

APPENDIX 2

SPECIAL TRIAXIAL CELL TO TEST MODEL PILES

APPENDIX 2

SPECIAL TRIAXIAL CELL TO TEST MODEL PILES

During the establishment of the different approaches to study the effects of bored pile installation on the properties and conditions of the surrounding soil, it was recognized that, if the changes and final soil condition could be related to a pile load test result, a major improvement in the knowledge of the interaction between pile and soil would result.

In order to make the testing of model piles cast in beds of soil under any stress conditions possible, it was necessary to design an apparatus that would allow the independent application of horizontal and vertical confining stresses on the soil bed. In addition to that characteristic, facilities for measuring the pore water pressure on the top and bottom (or drainage facilities) of the soil bed were required.

It was decided to modify a standard 6 inches diam. triaxial cell.An internal frame, with variable hight was designed and connected to the cell bottom (see Figure A2.2 B).

A diaphragm acting upon the new pedestal provided the independent vertical stresses on the soil bed. The horizontal stresses were applied in the conventional manner, the soil bed being protected by a standard 6 inches triaxial specimen membrane.

Pore water (pressure or drainage) ducts were provided both at the top and bottom (the measurement of pore water pressure should be made at the bottom - copper tube was used to avoid volume change).

The top cover of the frame has a central hole, to permit loading of the model pile to be installed in the soil bed. The load being applied to the pile can be measured by an internal load cell, acting upon the pile, with a Bellofram rolling diaphragm protecting the soil bed from the water on the cell. The pile displacements can be measured by a submersible LVDT monitoring relative movement between the frame's top cover and the load cell bottom.

The possibility of using the technique developed in Chapter 6 (installation of miniature pressure transducers inside a soil specimen) can allow the study of the variation of the pore water pressures at different distances from the model pile during installation (under axi-symetric conditions), up to equalization (in a medium term experiment) and during pile loading.

Different experimental procedures could be suggested for the specimen formation, pile testing and post test studies (various aspects could be studied, such as : effects of different procedures used for pile installation, different concrete mix proportions, importance of stress ratio, stress level, stress history, structural changes in the soil at different times of the model pile history, among others).

The special triaxial was designed and built and its use is left for future work.

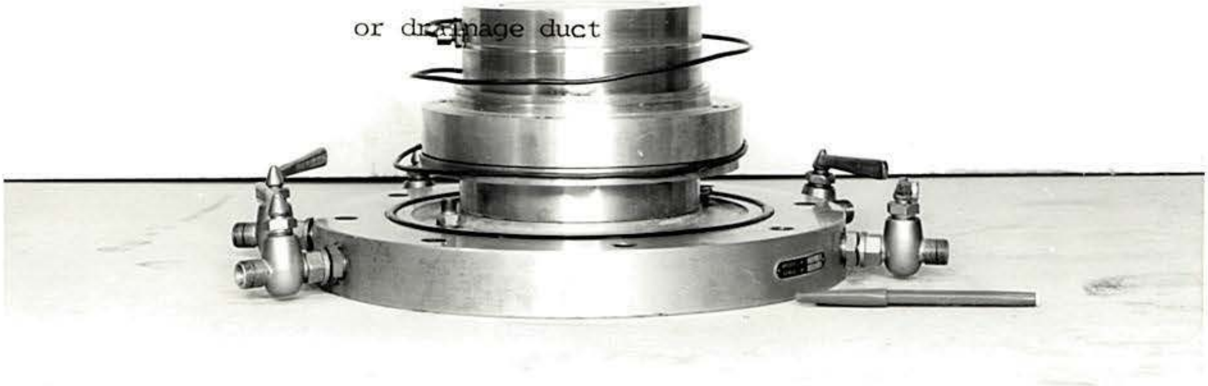
Figures A2.1 to A2.3 show the new apparatus. Figure A2.1 presents details of the basic components and the new pedestal being assembled. Details of mounting the internal frame, top and bottom ducts, and the complete new pedestal are presented in Figure A2.2. The complete new apparatus, inside a standard 6 inches diameter triaxial cell, is presented in Figure A2.3.

(A) Basic components

(B) Pedestal being assembled

FIGURE A 2.4 - Details of the basic components of the special triaxial cell designed to test model-piles.

(A) New pedestal completed, with pore water pressure or drainage duct



(B) Internal frame with variable length in position, top cover, top and bottom ducts

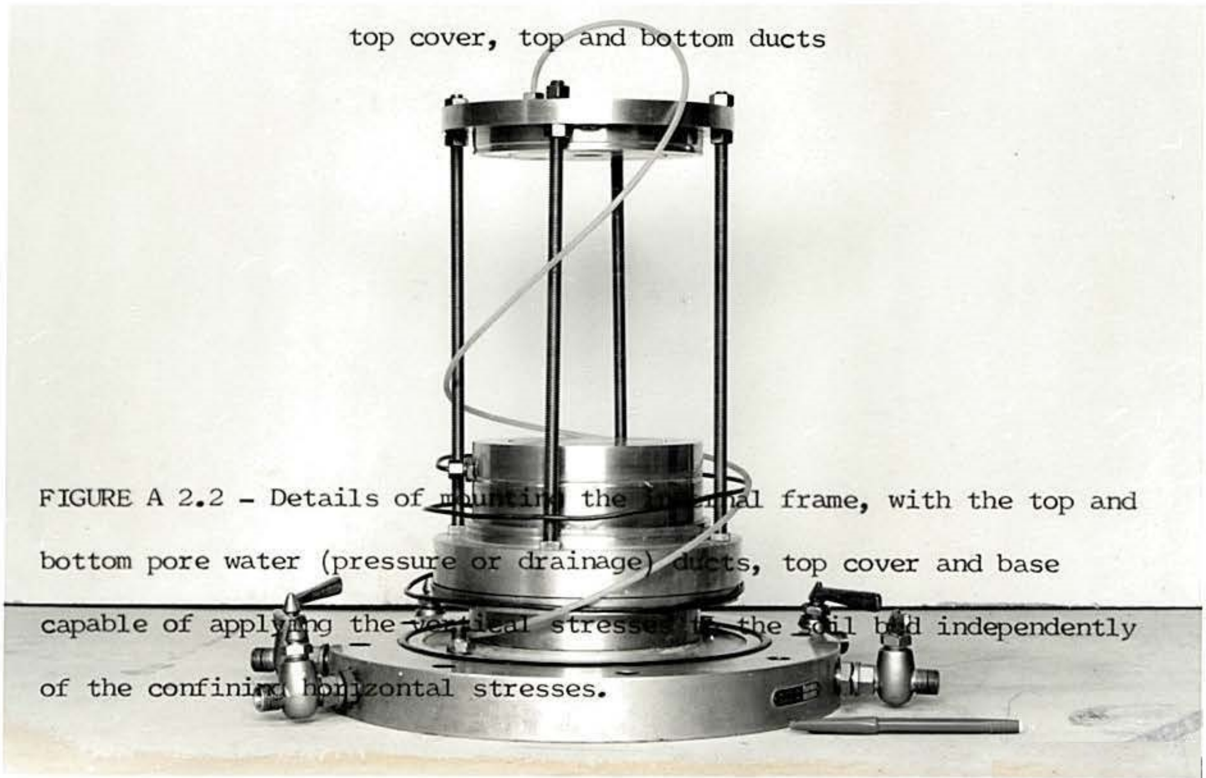


FIGURE A 2.2 - Details of mounting the internal frame, with the top and bottom pore water (pressure or drainage) ducts, top cover and base capable of applying the vertical stresses to the soil bed independently of the confining horizontal stresses.

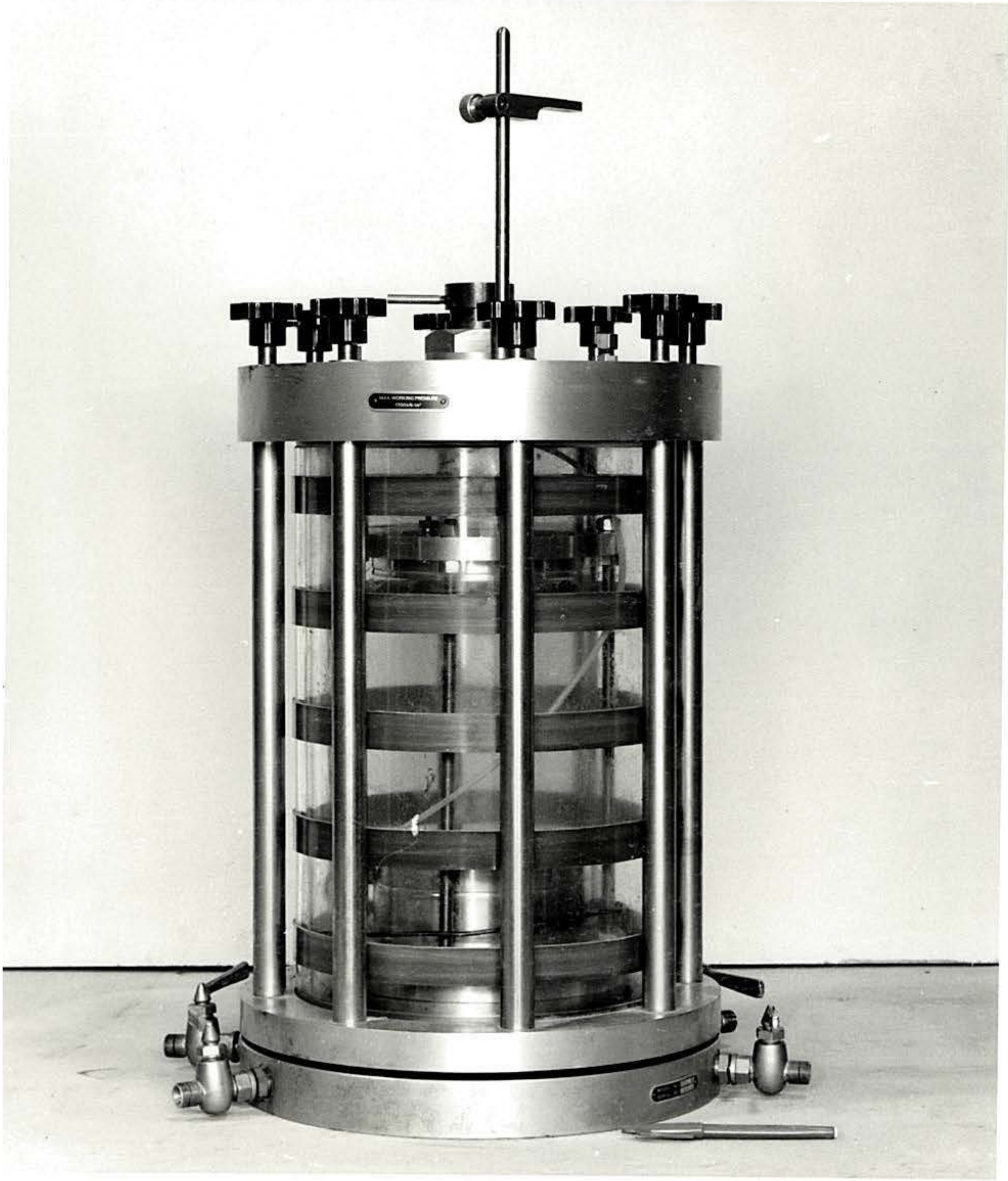


FIGURE A 2.3 - Complete special triaxial cell, with the standard 6 inches diameter triaxial cell used as a container

APPENDIX - 3 - PAPERS PRODUCED

"THE EFFECTS OF BORED PILE INSTALLATION IN CLAYS ON THE PROPERTIES OF THE PILE-SOIL SYSTEM" - 1981

SOLOS E ROCHAS, COPPE, Rio de Janeiro, Vol 4, No 1, pp 15-28,

in Portuguese

MILITITSKY, J. AND CLAYTON, C.R.I.,

"A RADIOCHEMICAL METHOD OF STUDYING THE MOISTURE MOVEMENT BETWEEN FRESH CONCRETE AND CLAY" - 1982

GEOTECHNIQUE, Vol 32, No 3, september, pp 271-275

MILITITSKY, J., JONES, J.R. AND CLAYTON, C.R.I.,

"INSTALLATION EFFECTS AND THE PERFORMANCE OF BORED PILES IN STIFF CLAY" - 1983

GROUND ENGINEERING, accepted for publication (1981)

CLAYTON, C.R.I. AND MILITITSKY, J.,

MILITITSKY, J. AND CLAYTON, C.R.I., 1981

"The Effects of Bored Pile Installation in Clays on the Properties of the Pile-soil System"

Solos e Rochas, COPPE, Rio de Janeiro, Vol 4, No 1, pp 15-28, in Portuguese

EFEITOS DA EXECUÇÃO DE ESTACAS ESCAVADAS EM SOLOS ARGILOSOS NAS PROPRIEDADES DO SISTEMA SOLO-ESTACA

THE EFFECTS OF BORED PILE INSTALLATION IN CLAYS ON THE PROPERTIES OF THE PILE-SOIL SYSTEM

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Recebido em 12.04.81

RESUMO

O presente trabalho analisa os efeitos da execução de estacas escavadas em solos argilosos nas propriedades do sistema solo-estaca, a saber: alívio de tensões devido a escavação, efeito da utilização de lama bentonítica, ação do concreto fresco abordando a alteração no estado de tensões e migração da água livre, efeito da demora na concretagem e reestabelecimento de tensões horizontais. São apresentados resultados de ensaios preliminares realizados em laboratório — com a finalidade de estudar os efeitos da presença do concreto fresco em contato com o solo na resistência não drenada das argilas, bem como resultados de ensaios em amostras coletadas junto a estacas reais, a 12 m de profundidade, mostrando o efeito de execução das estacas na resistência não drenada da argila, 4 meses após a execução. Finalmente comenta-se a importância do conhecimento de tais efeitos na previsão de comportamento das estacas escavadas de grande diâmetro executadas em solos argilosos.

ABSTRACT

The paper describes the effects of bored pile installation on the properties of the pile-clay system. Such effects may be due to relief of stresses as a result of excavation, to the use of slurry, the migration of water from fresh concrete, the delay in concreting or the re-establishment of horizontal stresses. Preliminary test results are presented showing the effect of the fresh concrete in contact with the clay on the undrained shear strength. Results of test· performed on samples taken near real piles, 12 m deep, 4 months after installation are also presented. Finally, comments are presented concerning the relevance of these effects in predicting the behaviour of large bored piles.

INTRODUÇÃO

A execução de estacas escavadas consiste basicamente na perfuração do sub-solo até uma profundidade pré-determinada e na concretagem da escavação. Na prática ocorrem diferentes condições de execução e são utilizados procedimentos diversos em cada caso (Tomlinson, 1977; Weltman and Little, 1977; Reese, 1978). A situação mais simples ocorre em solos argilosos rijos em estacas com profundidade limitada, onde o fuste permanece estável sem utilização de procedimentos especiais de contenção, permitindo a concretagem por queda livre. A execução torna-se mais complexa na medida em que se torna necessária a utilização de revestimento ou lama bentonítica para a manutenção da estabilidade da escavação ou prevenção do

Discussões abertas até 30/12/81

acesso de água no fuste.

Uma série de condições existentes no solo anteriormente à execução de uma estaca são alteradas pelos procedimentos de execução, entre outras: estado inicial de tensões no solo (devido à escavação), teor de umidade do solo adjacente ao fuste (devido à presença de água livre no concreto e alívio de tensões), amolgamento do solo junto às paredes (devido à ação mecânica de escavação) e influência da utilização de lama bentonítica na interface solo-estaca.

A execução de uma estaca escavada não afeta tão marcadamente as condições do solo quanto a de uma estaca cravada (Fig. 1 e detalhes para estacas escavadas nas Figs. 2 a 5), mas certamente os efeitos são importantes no condicionamento do comportamento da estaca sob carregamento. Existe um interesse crescente nos aspectos relacionados com o efeito da execução de fundações profundas no comportamento das mesmas (Endo, 1977; Bustamante et al, 1979; Curtis, 1980).

Passa-se a analisar cada um dos efeitos da execução, separadamente.

ALÍVIO DE TENSÕES DEVIDO À ESCAVAÇÃO

O conceito simplificado do estado de tensões existente no solo anteriormente à execução da estaca, devido somente ao peso próprio do solo, é baseado nas seguintes considerações:

— as tensões verticais são tensões principais e podem ser determinadas a partir do peso próprio do solo (σ_v);
— a pressão neutra é conhecida (u);
— as tensões horizontais são iguais em todas as direções e também são tensões principais (σ_h);
— a relação entre as tensões horizontais e verticais efetivas é definida como o coeficiente de empuxo em repouso (K_0)

Na solução de problemas na prática da Engenharia de Solos, pode-se determinar ou estimar com relativa facilidade as tensões verticais e neutra, porém as tensões horizontais constituem um problema maior (ver, por exemplo, Wroth, 1975).

O processo de escavação do fuste de uma estaca afeta as propriedades do solo argiloso, nas vizinhanças da estaca, de uma forma difícil de quantificar. Durante a escavação do fuste o nível de tensões horizontais próximo à escavação é reduzido, sendo o processo dependente do tipo de execução (uso de lama bentonítica, revestimento ou simples escavação). Considerando a condição mais favorável, onde o revestimento ou a lama bentonítica não são utilizados, uma espessura limitada de solo ao redor

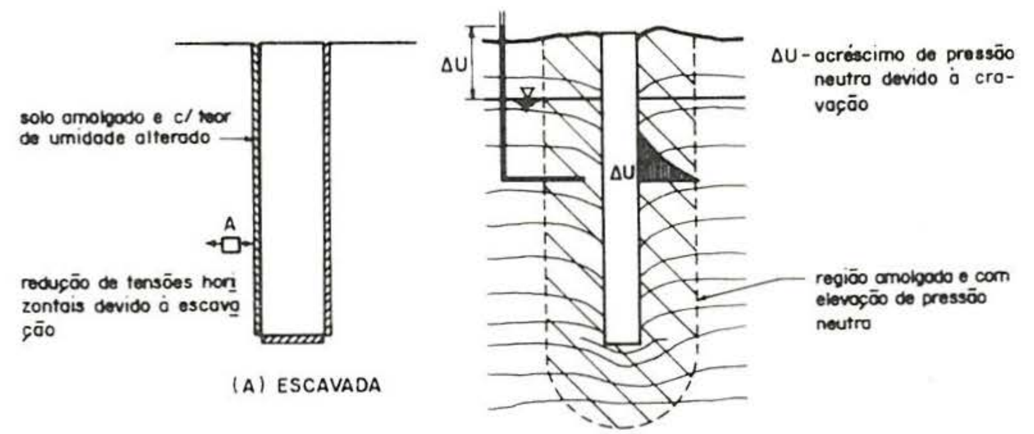

Fig. 1 — Diagrama Esquemático Mostrando os Efeitos da Execução de Estacas no Solo (Detalhes para o Caso de Estacas Escavadas nas Figs. 2 a 5)

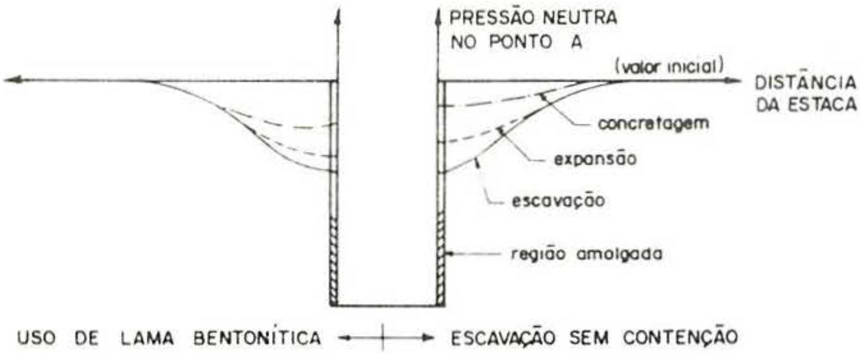

Fig. 2 — Diagrama da Possível Variação da Pressão Neutra Devido a Execução de Estacas Escavadas em Solo Argiloso

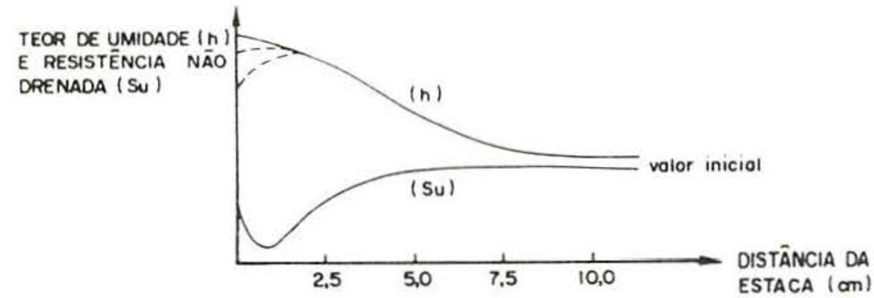

Fig. 3 — Diagrama da Variação do Teor de Umidade e Resistência não Drenada na Interface Estaca-Solo

da escavação sofre um processo de amolgamento devido a retirada do material (sendo dependente do tipo de ferramenta utilizado na escavação); as tensões horizontais radiais totais na superfície do fuste reduzem-se a zero; a eventual água existente na massa de solo migra para a zona com nível de tensões mais reduzido, iniciando um processo de expansão (*swelling*) e redução de resistência (*softening*); em solos fissurados as fissuras podem abrir. Fenômeno similar ocorre na base do fuste, onde as tensões verticais totais caem a zero. A magnitude da expansão e de redução de resistência é certamente influenciada pelo estado de pré-adensamento do solo, coeficiente de empuxo em repouso, densidade do solo, parâmetros de pressão neutra, permeabilidade e pelo tempo em que a escavação permanece aberta até o momento em que se inicia a concretagem (considerando apenas os aspectos referentes ao solo e escavação para conformação do fuste).

Quando é utilizado revestimento para a manutenção da estabilidade da escavação, possivelmente a variação de tensões é menor, a extensão da zona amolgada é maior e a deformação lateral permitida até que o concreto seja colocado é menor.

No caso de utilização de lama bentonítica ocorre amolgamento e certo nível de alívio de tensões horizontais e verticais (possivelmente é um caso intermediário entre os citados anteriormente) até a concretagem do fuste.

Como os parâmetros e condições acima referidos possuem diferentes grandezas na natureza e prática de execução de estacas, o resul-

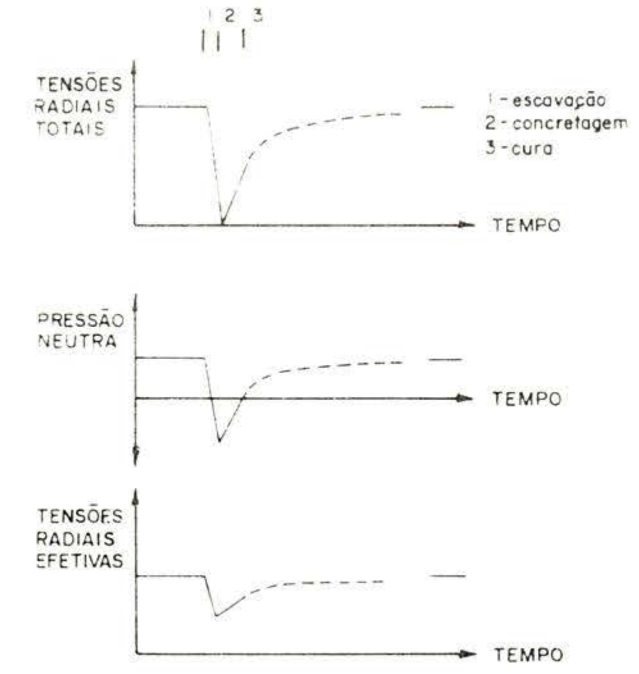

Fig. 4 — Diagrama da Possível Variação de Tensões Radiais com o Tempo Próximo a Estacas Escavadas sem Contenção de Fuste

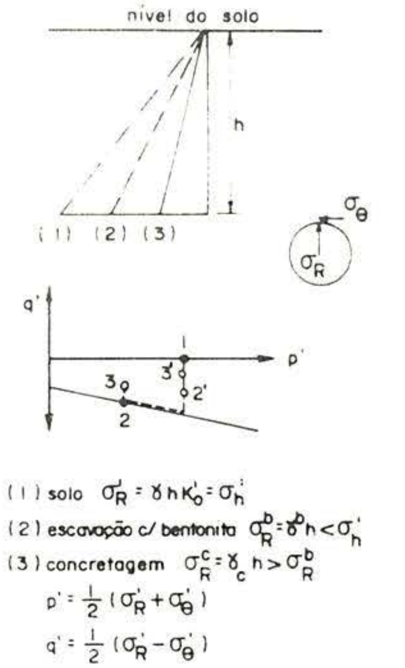

Fig. 5 — Variação no Estado de Tensões Horizontais (Estaca Escavada em Argila Pré-Adensada)

tado é uma ampla gama de variação nas condições do solo relacionada com a execução de estacas, para os diferentes solos e técnicas utilizados.

EFEITO DA UTILIZAÇÃO DE LAMA BENTONÍTICA

Conforme anteriormente citado, a utilização de lama bentonítica afeta o nível de variação de tensões no solo durante a escavação do fuste, influindo também na variação do teor de umidade do solo próximo ao fuste. A lama bentonítica interfere na absorção de água e troca-iônica (concreto fresco-água) e adicionalmente pode influir na carga limite da estaca, interferindo no atrito lateral e resistência de ponta disponíveis, como resultado da formação de película (mud cake) nas paredes e fundo do fuste e remoção incompleta (Sliwinski et al, 1980).

O uso de lama bentonítica envolve outro aspecto a ser considerado: geralmente nesta circunstância a ferramenta utilizada na escavação é diferente (da utilizada na escavação sem

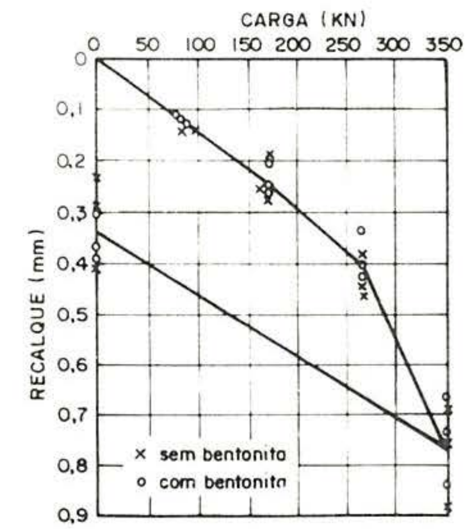

Fig. 6 — Resultados de Provas de Carga Mostrando o Comportamento de Estacas Não Influenciado pelo Uso de Bentonita (Fearenside and Cooke, 1978)

bentonita), o que pode alterar tanto as dimensões finais da estaca quanto as condições da superfície lateral da mesma.

O efeito da utilização da lama bentonítica no atrito lateral das estacas escavadas é tema de discussão e verifica-se a ocorrência de opiniões contraditórias na literatura.

Em solos permeáveis, a lama bentonítica forma uma camada na interface estaca-solo (filter cake), que em alguns casos pode ser removida de forma parcial pela ação do concreto fresco sendo colocado através do tubo tremonha (tremie) (Fleming and Sliwinski, 1977). A possível presença desta camada de bentonita de baixa resistência levanta dúvidas relativas aos valores de atrito a serem obtidos numa estaca construída utilizando tal processo. Atualmente não existe um conhecimento estabelecido sobre o problema, mas a escassa evidência disponível sugere a ocorrência de redução no valor da resistência por atrito, em solos granulares, da ordem de 10 a 30% (Fleming and Sliwinski, 1977).

Em solos de baixa permeabilidade, como as argilas, pode-se esperar apenas a formação de uma camada de pequena espessura recobrindo as paredes da escavação, que eventualmente poderá deixar de ser removida. Este fato é confirmado na prática por observações feitas em estacas e paredes diafragma deixadas a descoberto. Em alguns casos foi constatada a presença de uma camada com poucos milímetros de espessura (Fleming and Sliwinski, 1977) cuja presença e formação é relacionada por certos autores com longas demoras na concretagem, ocorrência de forças elétricas ou eventuais reações químicas. Em outros casos não existe evidência de formação de tal camada na interface (Burland, 1963; Fearenside and Cooke, 1978).

O'Neill and Reese (1972) relatam um caso de sensível redução no atrito lateral causado pela ocorrência de espessa camada de bentonita na interface estaca-solo, devida possivelmente à utilização de revestimento provisório (prática pouco comum).

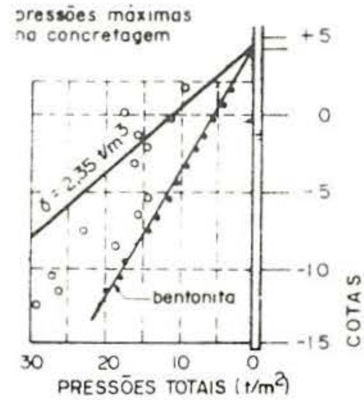

Fig. 7 — Pressões de Concreto Medias durante Concretagem de Painel numa Cortina Diafragma (Dibiagio and Roti, 1972)

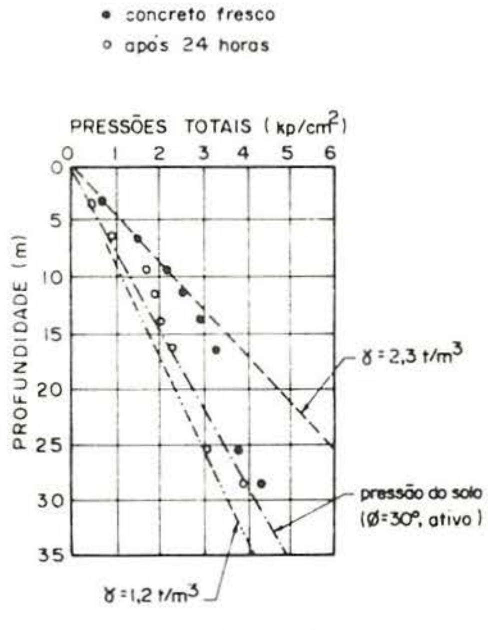

Fig. 8 — Pressões Devidas ao Concreto Medidas em Cortina Diafragma (Uriel and Otero, 1977)

A opinião de que o atrito lateral não é praticamente afetado pelo uso de lama bentonítica é suportada por grande número de provas de carga realizadas em elementos construídos utilizando tal tipo de procedimento (Chadeisson, 1961; Burland, 1963; Fernandez, 1965; Kornick and Wiseman, 1967; Farmer et al, 1970; O'Neill and Reese, 1972; Fleming and Sliwinski, 1977). Num extenso programa de provas de carga em estacas instrumentadas, realizado com

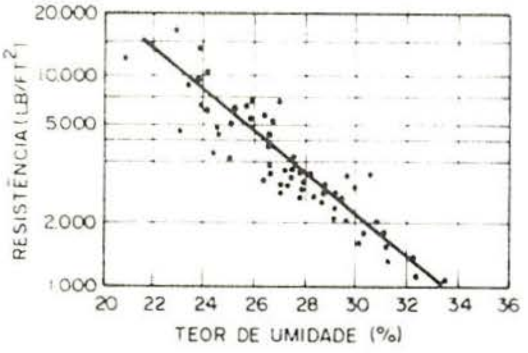

Fig. 9 — Relação entre o Teor de Umidade e a Resistência Não Drenada para a Argila de Londres (Skempton, 1959)

a finalidade específica de estudar o efeito da utilização de lama bentonítica no comportamento das estacas escavadas em solos argilosos. Fearenside and Cooke (1978) concluíram que, apesar da ampla gama de resultados obtidos, não ocorre nenhuma diferença evidente de comportamento devido à utilização deste processo construtivo, Figura 6.

AÇÃO DO CONCRETO FRESCO

Após a fase de escavação, o fuste é preenchido com concreto de alta plasticidade (fluido). Existem dois efeitos distintos a serem considerados: alteração no estado de tensões e migração de água.

Estado de Tensões

Quando a escavação é preenchida com concreto fluido ocorre um aumento nas tensões radiais totais e na pressão neutra. A forma do desenvolvimento do estado de tensões radiais devido à presença do concreto fresco na massa de solo adjacente é um problema complexo. A atual abordagem do problema refere-se ao estudo do caso de pressão do concreto fresco em fôrmas (ACI-1958, CERA-1965, Levitsky, 1973, 1975) e, entre outras, mostra a influência das seguintes variáveis no desenvolvimento das tensões: velocidade de concretagem, consistência do concreto, altura de lançamento, endurecimento do concreto, arqueamento, pêso específico do material, tamanho máximo de agregado, temperatura da mistura, temperatura ambiente, secção da fôrma e procedimentos construtivos. Como as condições de concretagem ocorrente sem estacas escavadas são completamente diferentes daquelas encontradas nas estruturas (alturas de lançamento, *slump*, volume de material envolvido, secção do elemento sendo concretado, somente para citar algumas das condições) as expressões propostas para o caso das fôrmas são inadequadas no caso de estacas escavadas.

O'Neill and Reese (1978) considerando alguns dos problemas referidos, definiram uma profundidade crítica e propuseram um diagrama no qual a pressão lateral do concreto cresce linearmente de zero até 3.000 p/ft³ (143 kN/m³) na profundidade crítica $(z/d)_{cr}$ e permanece constante a partir deste ponto. O fator $(z/d)_{cr}$ é obtido a partir da Equação 1, considerando o concreto fluido submerso para o caso estudado pelos autores (uso de lama bentonítica):

onde

$$(z/d)_{cr} = \frac{3000}{\gamma_c d}$$

onde

γc é a massa específica aparente submersa do concreto (p/ft³)
d é o diâmetro da estaca (ft) e
z é a profundidade (ft)

Referências relativas à pressão do concreto fresco em paredes diafragma (Di Biagio and Roti, 1972; Uriel and Otero, 1977) mostram a ocorrência de crescimento praticamente linear e proporcional ao peso próprio do concreto fluido até certa profundidade (aparentemente função da geometria da peça concretada) e relativa constância a partir deste ponto (Figs. 7 e 8).

Cooke (1979) refere-se a dados experimentais relativos a pressão do concreto fluido em estacas escavadas. O re-exame de dados obtidos em amplo programa de investigação de comportamento de estacas escavadas de grande diâmetro em solos argilosos (Whitaker and Cooke, 1966) mostrou a ocorrência de valores de pressão vertical na base das estacas (indicados nas células de carga) imediatamente após a concretagem, com ordem de grandeza entre 1/2 e 1/3 do valor correspondente a coluna de concreto fluido. Em 11 das 13 estacas ensaiadas estas forças decresceram no período anterior ao início das provas de carga. Segundo Cooke (1979) esta redução deve ser atribuída ao arqueamento entre o concreto e a argila, que pode ter iniciado pela retração do concreto, aliviando a base. Contatos mantidos com o autor (Cooke, 1980) foram infrutíferos na recuperação dos valores numéricos da pressão em cada caso, pois os registros originais não foram encontrados.

Migração de Água

Normalmente a quantidade de água utilizada na preparação do concreto da estaca é maior que a necessária para a hidratação do cimento e após o fim de pega do concreto existe água livre disponível na massa de concreto, o que pode ser uma fonte de abastecimento d'água para solos com avidez. É importante salientar que apenas parte das alterações no teor de umidade que ocorrem nos solos argilosos junto de elementos de fundação escavados é devida à migração de água disponível no concreto; de forma complementar existe a contribuição devida ao alívio de tensões que se segue à escavação, presença do fluido utilizado para manutenção da estabilidade do fuste e água livre no interior da massa de solo fluindo através de fissuras abertas pelas operações de escavação e horizontes permeáveis.

Para ilustrar a marcada influência do teor de umidade na resistência não drenada em solos argilosos, a Figura 9 reproduz os resultados disponíveis (Skempton, 1959) na época da publicação de trabalho clássico sobre estacas escavadas em solos argilosos.

Na literatura existem várias referências à variação do teor de umidade no solo junto às estacas escavados executadas em solos argilosos (Meyerhof and Murdock, 1953; Robin and Tomlinson, 1953; Mohan and Chandra, 1961; Burland, 1963; O'Neill and Reese, 1970; Chandler, 1977; Fearenside and Cooke, 1978). Em laboratório a distribuição do teor de umidade em amostras argilosas em contato com concreto fresco foi estudada por Taylor (1966), Chuang and Reese (1969) e Yong (1979).

Na Figura 10 são apresentados os resultados de ensaios preliminares realizados em laboratório por um dos autores (Milititsky, 1980), onde foi estudado o efeito da colocação de amostra de solo argiloso em contato com concreto fresco, tentando simular as condições de campo. Foi utilizado o ensaio de penetração de cone (utilizado na prática inglesa na determinação do limite de liquidez — BS 1377:1975 Test 2 a) sob a ação de peso de 800 gr com a finalidade de obter uma avaliação qualitativa da variação da resistência não drenada. O referido ensaio apresenta a vantagem de permitir a realização de testes repetidos na mesma amostra, em diferentes posições, caracterizando a variação de resistência a pequenos intervalos e permite a utilização de uma amostra em ensaios visando caracterizar a variação de resistência no tempo.

A Figura 10 mostra a relação direta entre penetração de cone e aumento do teor de umidade (sendo a resistência inversamente proporcional ao quadrado da penetração). Os resultados da amostra G (Fig. 11) indicam o efeito da utilização de diferentes fatores água/cimento. A amostra E (Fig. 11) foi utilizada para estudar o efeito do tempo de contato (idade da estaca) nos resultados de penetração de cone. Utilizando uma amostra controle, a Figura 10 mostra o efeito da evolução do teor de umidade nos valores de penetração (resistência). Dos resultados obtidos conclui-se que: maior fator água/cimento condiciona maior redução de resistência do solo, maior aumento de teor de umidade no contato solo-concreto não caracteriza a região de menor resistência (maior penetração do cone), possivelmente devido a efeitos estabilizadores de componentes do cimento dissolvidos na água e carreados no processo de migração. Os efeitos verificados no atual programa de ensaios coincidem com os resultados obtidos por Chuang and Reese (1969).

Os resultados de ensaios realizados por um dos autores (Milititsky, 1980) em amostras coletadas junto à cortina de estacas justapostas, a 12 m de profundidade, 4 meses após a execução das estacas e imediatamente após a escavação são apresentados na Figuras 12 e 13. A obra referida situa-se em Londres (Monument Street, The City). As estacas foram executadas com a utilização de revestimento na região do sub-solo com horizonte granular e lama bentonítica na profundidade onde as amostras foram coletadas. Observa-se nos resultados obtidos uma tendência bem caracterizada nos resultados de penetração de cone e teores de umidade correspondentes. Nos casos em que o aumento no teor de umidade não é sensível, a penetração do cone não apresenta tendência especial (Fig. 13).

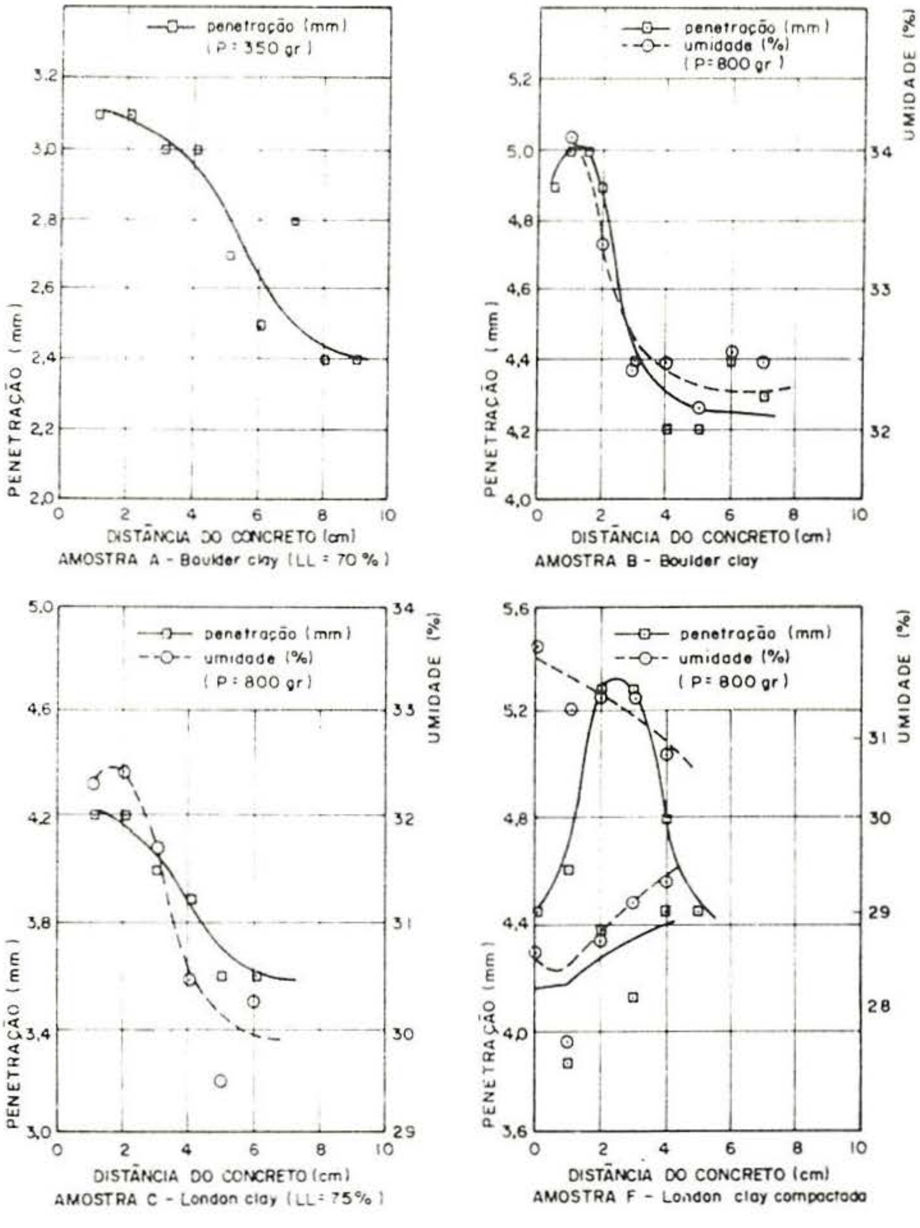

Fig. 10 — Resultados de Ensaios de Laboratório Mostrando o Efeito do Contato entre Concreto Fresco e Argila no Teor de Umidade e Penetração de Cone (Resistência Não Drenada) (Milititsky, 1980)

A medida que o concreto cura, provavelmente ocorrem complexos fenômenos de troca iônica entre o concreto e a argila. A evidência nos ensaios realizados sugere um aumento de resistência na região do solo imediatamente adjacente ao concreto por efeito destes fenômenos.

A utilização de aditivos para melhorar a trabalhabilidade do concreto certamente interfere na inter-relação global entre o concreto e o solo, porém não existem estudos específicos do problema na literatura.

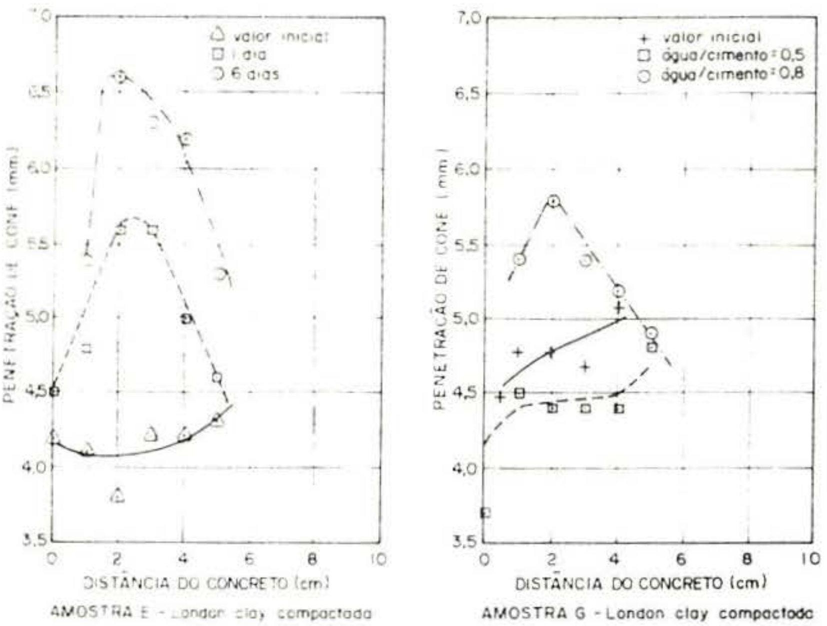

Fig. 11 — Ensaios de Laboratorio, Mostrando o Efeito do Tempo — Amostra E — e Fator Água/Cimento — Amostra G — na Resistência Não Drenada (Penetração de Cone) (Milititsky, 1980)

É importante assinalar que os resultados de qualquer programa de laboratório tentando estudar o efeito da variação do teor de umidade e de resistência do solo devidos à presença do concreto fresco em contato devem ser encarados com cautela. É extremamente complexa a simulação em laboratório das condições de campo e os resultados obtidos devem ser comparados com os de campo afim de apresentar significância.

DEMORA NA CONCRETAGEM

Os efeitos da demora na concretagem podem ser avaliados pelos resultados obtidos em alguns ensaios "in situ". Além do nível de carga e geometria da estaca, o módulo de elasticidade E (ou o módulo de cisalhamento G) do solo é uma variável fundamental, afetando o recalque de uma estaca. De acordo com resultados obtidos por Marsland (1971) a relação E/Su (onde Su é a resistência ao cisalhamento não drenada) determinada através de ensaios de placa em escavações em Londres (London Clay) é dependente do tempo após a escavação do fuste no qual os ensaios são realizados. Para curto espaço de tempo foram obtidos valores de E/Su da ordem de 500, em oposição a valores entre 100 e 200 obtidos em ensaios realizados em períodos maiores que 8 horas. O valor representativo da relação E/Su para a argila de Londres é possivelmente 500, valores inferiores obtidos nos ensaios com diferença de tempo entre a escavação e o ensaio indicam, entre outros, efeitos de amolgamento e expansão, principalmente.

RE-ESTABELECIMENTO DE TENSÕES HORIZONTAIS

Após a execução completa da estaca, as tensões horizontais gradualmente aumentarão de nível e o valor limite que influenciará no resultado de uma prova de carga é dependente do grau de redução de resistência que ocorre no solo ao redor da estaca nos estágios de escavações e concretagem e posterior grau de re-estabelecimento de tensões. Mesmo com perfeitas condições de execução, parece pouco provável que o estado inicial de tensões horizontais em repouso, seja re-estabelecido completamente nas paredes laterais da estaca (Burland, 1973) O re-estabelecimento das tensões hori-

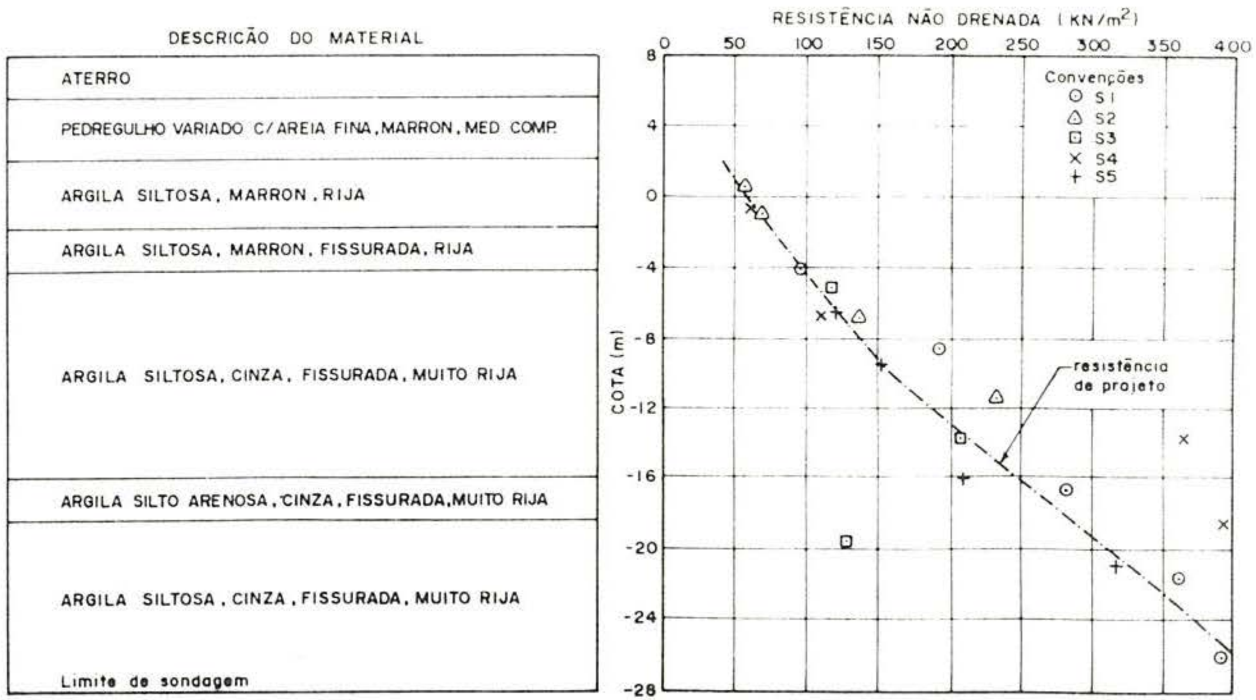

Fig. 12 — Perfil Simplificado do Sub-Solo em Monument Street, The City, Londres, com Valores de Resistência Não Drenada Obtidos no Programa de Investigação do Sub-Solo Realizado para o Projeto de Fundações

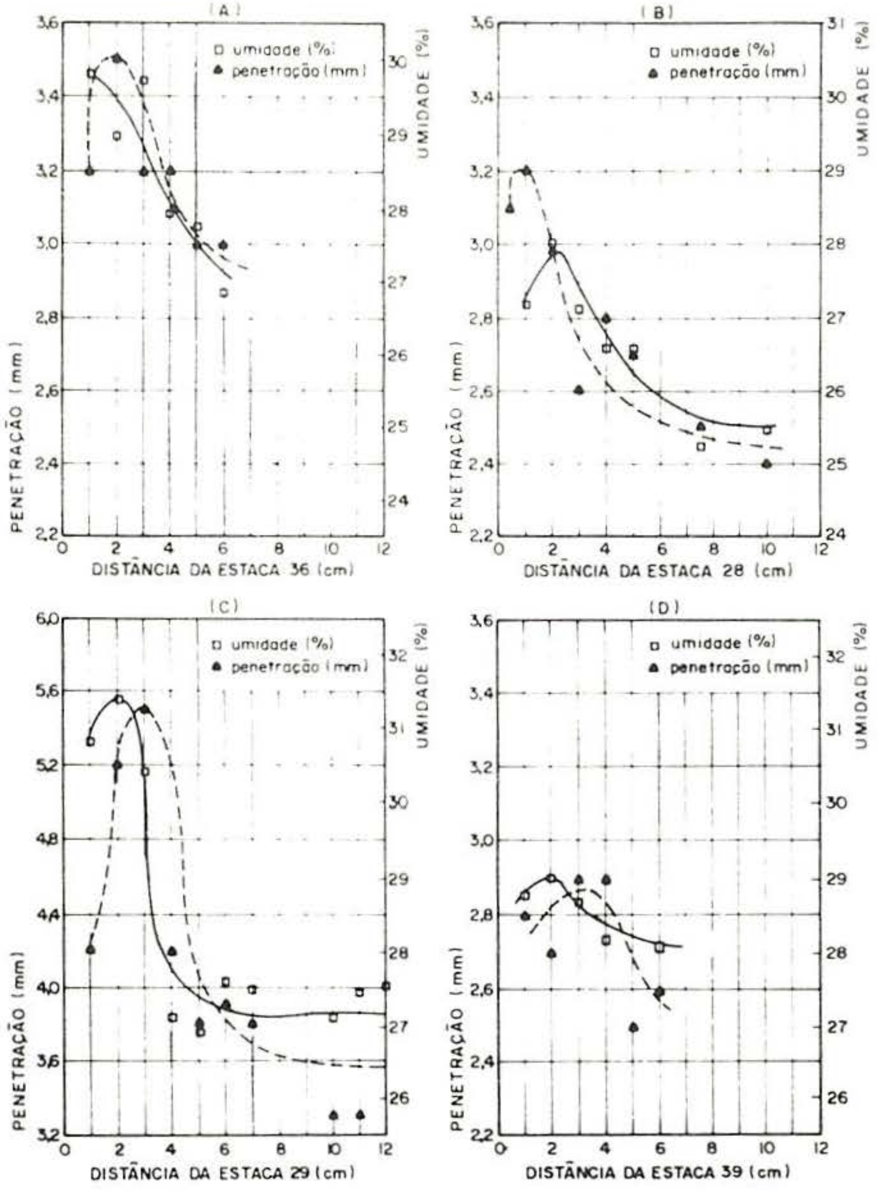

Fig. 13 — Teores de Umidade e Penetração de Cone (P= 800gr) em Amostras Coletadas junto a Estacas Escavadas, 12 m de Profundidade, 4 Meses após a Execução do Estaqueamento, Monument Street, The City, Londres (Milititsky, 1980)

zontais nas vizinhanças da estaca reconsolidará a zona amolgada e em processo inicial de expansão, (não existindo ainda medições das tensões atuantes a longo prazo). A resistência do solo que sofreu um processo inicial de redução devido ao aumento inicial do teor de umidade, amolgamento e expansão, aumentará como resultado da re-consolidação. A Figura 14 apresenta o aumento necessário de tensões horizontais (representado no caso por K_0) para produzir

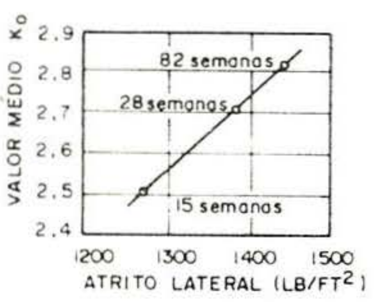

Fig. 14 — Aumento do Atrito Lateral como Conseqüência do Aumento das Tensões Horizontais com o Tempo (Chandler, 1968)

o comportamento obtido em provas de carga a diferentes idades, com a consideração de ser este o único fator variável no problema (dados de Whitaker and Cooke, 1966, segundo Chandler, 1968).

CONCLUSÕES

As vantagens técnicas e econômicas da utilização de estacas escavadas de grande secção resultaram no crescente uso de tal sistema de fundações. Em função das suas características geométricas são comuns elementos isolados transmitindo elevados níveis de carga ao solo (1000 t ou mais não são fatos raros), mas o comportamento das estacas escavadas relacionado com as propriedades do solo ainda não é claramente compreendido.

O efeito da execução das estacas nas propriedades do solo e estado de tensões é tópico de máxima relevância para o conhecimento do comportamento das estacas. O primeiro problema a ser resolvido é a avaliação realista destas condições anteriormente à execução da estaca.

Os efeitos mais significativos da execução de estacas escavadas nos solos argilosos são: o amolgamento do solo, alívio de tensões, variações no teor de umidade e conseqüente redução de resistência, expansão e fenômenos de troca iônica. Na realidade o conhecimento destes fenômenos atualmente é puramente qualitativo e é usualmente considerado como indicação para a adoção de técnicas construtivas ou estabelecimento de fatores empíricos a serem utilizados na previsão de capacidade de carga.

O problema complica-se na medida que as diferentes técnicas de execução de estacas escavadas apresentam influência diversa (o que alerta para o problema de que o resultado de determinada prova de carga, realizada em estaca executada de acordo com uma metodologia, pode ser inteiramente diferente do comportamento a ser obtido no mesmo solo, em estaca com a mesma geometria projetada, quando se alteram as condições de execução da estaca).

1. Condições relativas ao solo:
   - 1.1. Perfil do sub-solo
   - 1.2. Propriedades dos horizontes
   - 1.3. Estado de tensões inicial
   - 1.4. Presença de água

2. Condições relacionadas com a execução da estaca:
   - 2.1. Processo de escavação:
     - 2.1.1. Percussão
     - 2.1.2. Rotação - tipo de ferramenta
   - 2.2. Suporte utilizado:
     - 2.2.1. Escavação não suportada
     - 2.2.2. Revestimento
     - 2.2.3. Lama bentonítica
     - 2.2.4. Hélice contínua
   - 2.3. Limpeza do fundo:
     - 2.3.1. Mecânica
     - 2.3.2. Manual
   - 2.4. Concretagem:
     - 2.4.1. Demora
     - 2.4.2. Fator água/cimento
     - 2.4.3. Propriedades do concreto e integridade
     - 2.4.4. Temperatura
   - 2.5. Geometria:
     - 2.5.1. Comprimento
     - 2.5.2. Secção do fuste
     - 2.5.3. Condição da base - com ou sem alargamento

3. Condições de carregamento:
   - 3.1. Idade da estaca (tempo decorrido desde a execução da estaca até o carregamento)
   - 3.2. Prova de carga - tipo de programa de carregamento
   - 3.3. Sob estrutura real: crescimento da carga permanente
     tipo de carga acidental

Tabela 1 - Fatores Relevantes ao Comportamento de Estaca Escavada Isolada Quando Submetida a Carregamento Vertical de Compressão

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MILITITSKY, J., JONES, J.R. AND CLAYTON, C.R.I., 1982

"A Radiochemical Method of Studying the Moisture Movement Between Fresh Concrete and Clay" Geotechnique, Vol 32, No 3, september, pp 271-275

A radiochemical method of studying the moisture movement between fresh concrete and clay

J. MILITITSKY,* J. R. JONES* and C. R. I. CLAYTON*

INTRODUCTION

Back analyses of field loading tests on bored piles in stiff clays have repeatedly shown that the maximum average shaft adhesion is normally much less than the average undisturbed shear strength of the soil along the pile shaft. Several mechanisms involving stress relief and remoulding have been postulated to explain this shear strength reduction. The fact that a bored cast in situ pile is concreted with a high slump mix containing more water than is necessary for hydration of the cement means that there is a ready source of water for clay swelling.

A number of researchers have tried to assess the effects of wet concrete placement on the adjacent soil, both in the field and in the laboratory (Meyerhof & Murdock, 1953; Skempton, 1959; Mohan & Chandra, 1961; Burland, 1963; Taylor, 1966; Chuang & Reese, 1969; O'Neill & Reese, 1970; Fearenside & Cooke, 1978; Yong, 1979; Milititsky & Clayton, 1981; Clayton & Milititsky, 1982). In the field, the major problem in attempting to assess the magnitude of local moisture content increases adjacent to piles is the variability of the natural moisture content (e.g. see Fearenside & Cooke, 1978). Although this variability may be reduced when using remoulded soils in the laboratory, other problems remain. The moisture content test most commonly used—test 1(a) of BS 1377 (British Standards Institution, 1975)—has limited accuracy compared with the moisture content increase observed. Also a relatively large specimen must be removed and oven-dried, tests may be disturbed and the same soil cannot be sampled at different times during an experiment.

An alternative method, based on the use of radioisotopes, offers considerable potential. Transfer of water from one environment to another can be monitored by means of a labelling procedure. For this purpose, both stable and radioactive isotopes can be used, and in the case of water there are three possibilities

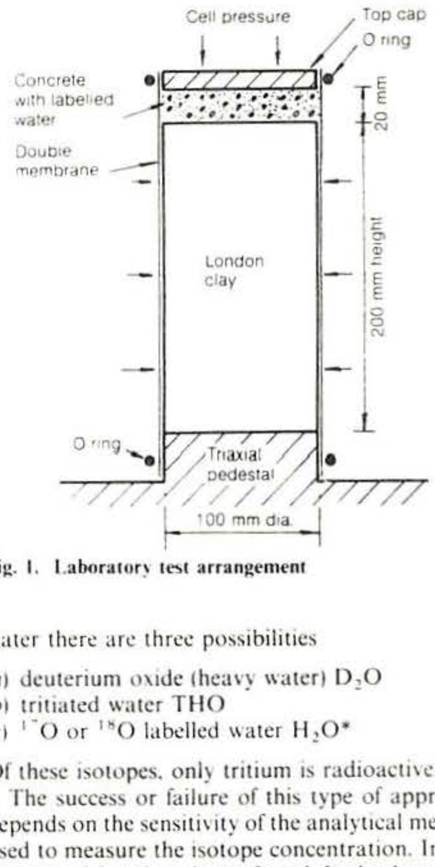

Fig. 1. Laboratory test arrangement

(*a*) deuterium oxide (heavy water) D_2O
(*b*) tritiated water THO
(*c*) ^{17}O or ^{18}O labelled water H_2O^*

Of these isotopes, only tritium is radioactive.

The success or failure of this type of approach depends on the sensitivity of the analytical method used to measure the isotope concentration. In this respect, tritium is to be preferred. It also has other advantages: it is a very weak β emitter ($E_{max} = 18_{keV}$), it has a convenient half-life (12·3 years), it is of low toxicity and it can be supplied at high specific activity (Ci/mmol). Furthermore, the liquid scintillation method of detection (e.g. see Dyer, 1974) is extremely sensitive and lends itself to high sample throughput. Even in experiments where the concentration of tritium decreases by a factor of 10^{10}, there is no difficulty in measuring its concentration. Consequently the use of tritium as a tracer in studies of concrete/soil interaction is attractive. This Note presents details of initial findings.

Discussion on this Technical Note closes 1 December 1982. For further details see inside back cover.
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Fig. 2. **Miniature sampler with a sample of clay**

TECHNIQUE

Test preparation

Figure 1 shows the laboratory test arrangement used to allow clay to swell in contact with fresh concrete. A cylindrical specimen of 'undisturbed' London clay (height 200 mm) obtained with an open drive sampler with 100 mm thick walls was placed on the pedestal of a triaxial cell, sealed with a double membrane and consolidated under a desired effective pressure. When the specimen was fully consolidated, the drainage was closed, cell pressure released, the top cap removed and concrete poured into a former on top of the soil specimen.

The concrete used in the experiments had a water cement ratio of 0·6 and the mix was 1:2·4:1·9 by weight. Tritiated water (10 μl, 5 Ci ml) was added to 0·5 ml of water and this was equilibrated with the 30 ml of water used in the preparation of the concrete. The assembly was sealed again in the double membrane and subjected to the previously applied cell pressure.

Sampling for radioactivity

In order to take a large number of very small samples from the same test specimen, a miniature sampler was designed (Fig. 2). The sampler was attached to the end of a plastic syringe for ease of handling and in an attempt to produce a vacuum behind the sample during withdrawal of the sampler. Initial experiments were made with a sampler with an inside diameter of about 1·5 mm and no inside clearance.

It proved difficult to obtain more than a few millimetres of the very stiff London clay with this sampler, and so modifications were required. Inside clearance was provided by rolling the cutting edge inwards to give the sampler an area ratio of 248%, an inside clearance of 39%, an inside diameter of 1·50 mm and a 24 mm length of drive. There was no difficulty in obtaining samples of the clay with the modified sampler. Samples typically

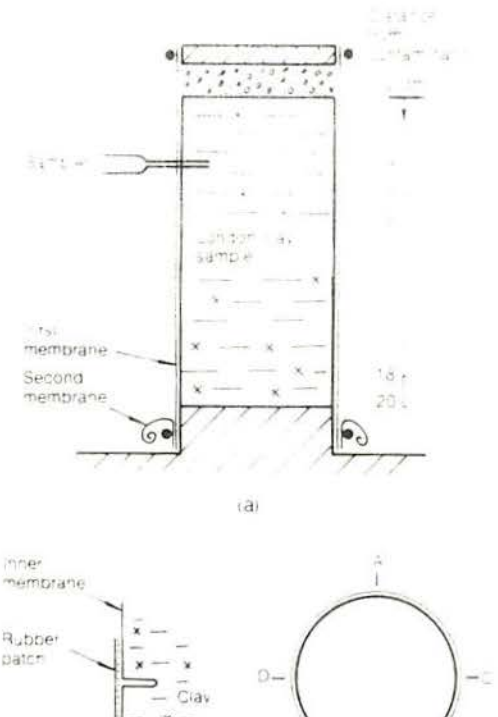

Fig. 3. Details of: (a) test layout; (b) membrane repair; (c) sampling locations

had a volume of about 30 mm³, and were taken from the specimen by rolling back the outer membrane and driving the sampler through the inner membrane for about 20 mm in the required positions (Fig. 3(a)).

After sampling, the hole in the inner membrane was repaired by applying a small patch of latex rubber to the membrane with adhesive (Fig. 3(b)). The second membrane was then replaced in its original position and the cell pressure re-established until more samples were required. Samples were taken at varying distances from the soil–concrete interface, and at different positions around the perimeter of the specimen (Table 1).

Radioactivity determination

The radioactivity of each sample was determined in the following manner.

First the weight of the sample was determined to the nearest 0·0002 g. It was usually in the range 0·06–0·1 g. To induce the isotope exchange reaction

Table 1. Position of sampling

Distance from concrete, cm	Days after preparation		
	[illegible]	[illegible]	[illegible]
1	B	[illegible]	A
2	C	D	B
3	D		C
4		A	
5			D
6	A	B	
7			
8	B	C	A
9			
10	C	D	
11			
12			B
13			
14	B	A	
15			
16			
17		B	C
18			
19	D		
20			

$$(clay)_{H_2O^*} + H_2O \rightleftharpoons (clay)_{H_2O} + H_2O^*$$

1 ml of water was added to each sample.

The labelled water was then removed from the clay solution by freeze-drying using the apparatus shown in Fig. 4. This procedure entailed freezing the water in the clay solution (in flask A) by rotating the flask in a Dewar vessel containing liquid nitrogen. The apparatus was assembled as shown, connected to a vacuum pump and evacuated with the flask immersed in liquid nitrogen. When the apparatus was fully evacuated (to about 0·05 torr) the stopcock B was closed, isolating the system. The condenser tube C was then immersed in liquid nitrogen. The flask was removed from the liquid nitrogen and allowed to attain room temperature. Sublimation of the ice from the flask to the condenser then proceeded. When sublimation was complete the vacuum was released by opening the stopcock. The melted ice in C was then removed and assayed for tritium.

To do this 2 µl of the radioactive (tritiated) water obtained from the condenser was added to 5 ml of a commercially available liquid scintillator (NE250) and the radioactivity was determined using a Beckman LS100 liquid scintillation counter. Enough counts were obtained to give an accuracy better than $\pm 0{\cdot}1\%$.

To convert the counts per minute to disintegrations per minute (i.e. absolute radioactivity units) the efficiency of counting was determined by adding a known amount of an internal standard (^{3}H-hexadecane) to a mixture of

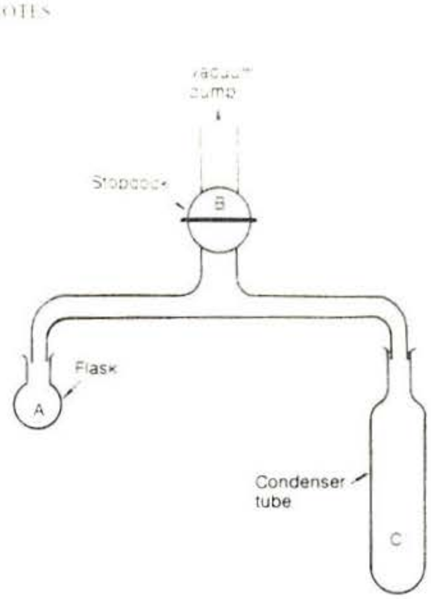

Fig. 4. Freeze-drying apparatus

2 µl of water and 5 ml of NE250 liquid scintillator.

CALCULATIONS

As the process involves controlled labelling, sampling and testing, it is relatively simple to convert the disintegrations per minute (dpm) obtained from the scintillation counter to an increase in tritiated water content. Initially, when tritiated water is added to the concrete mix, three methods present themselves as possibilities for determining the disintegrations per unit time per unit volume of this water

(*a*) use of the specific activity of the tritiated water and the various dilution factors
(*b*) sampling of the completed mix
(*c*) sampling of the diluted but labelled water, before addition to the cement and aggregate.

Method (*a*) is possible only if the radioactivity of the tritiated water, as supplied, is accurately known: normally, the radioactivity of the tritiated water will only be correct to ± 10–20%. Method (*b*) requires a large enough sample to be taken to be representative of all the particle sizes present in the concrete. The labelled water is removed by the addition of unlabelled water, as before. As its radioactivity is higher than can be handled conveniently, it must be diluted.

As soon as the labelled water is added to the concrete, it is to be expected that reactions involving the labelled water will begin. Thus the equilibration process might not remove all of the tritium from the concrete sample, resulting in a low radioactivity and an overestimate of labelled water content in the clay samples. Thus the best method

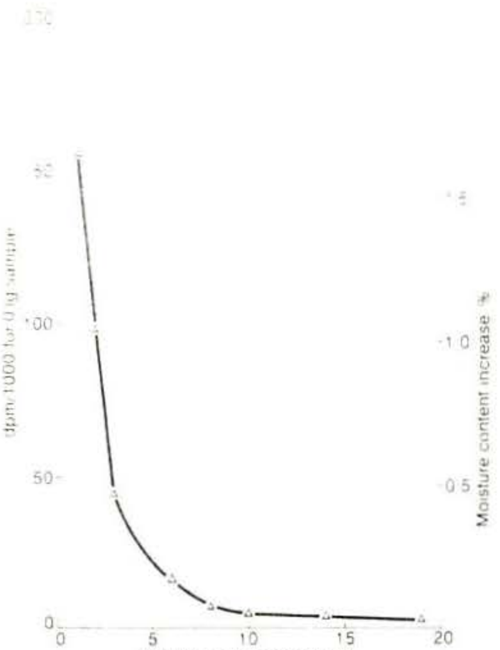

Fig. 5. Clay/concrete contamination (one day)

of determining the radioactivity of the labelled water would appear to be method (c). After sampling and scintillation counting, the radioactivity of the tritiated water can be expressed in dpm per gramme of water. A figure of about 10^{10}–10^{11} dpm/g H_2O* would be expected in the present case.

In order to obtain the labelled water content of the clay it is necessary to have an approximate knowledge of its overall moisture content before the test. Consider a typical set of data for a sample of clay extracted and tested as described

radioactivity of the tritiated water added to concrete	$0{\cdot}1 \times 10^{12}$ dpm/g H_2O*
wet weight of clay sample	0·05 g
radioactivity of extracted water	$0{\cdot}1 \times 10^{6}$ dpm

initial moisture content 30%

During the preparation of the clay sample, to obtain material for scintillation counting the water was removed from the clay by dilution and freeze-drying. The volume of labelled water present in the clay had previously been equilibrated by the addition of 1 ml of unlabelled water. After sublimation a 2 μl sample was withdrawn.

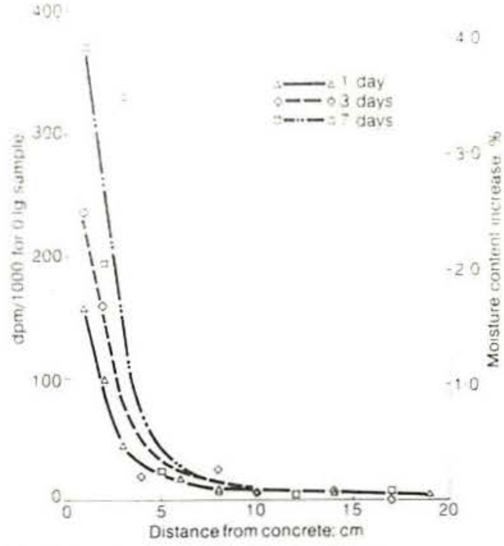

Fig. 6. Clay/concrete contamination: sample 1

Assuming the increase in water content to be small, this represents a reduction in sample size of $0{\cdot}5 \times 10^{3}$. Thus the radioactivity of the full sample would be 50×10^{6} dpm, and the weight of labelled water transferred from the concrete to the sample would be 0·50 mg. As a 0·05 g sample of clay with a moisture content of 30% will have a dry weight of 38·46 mg, this represents an increase in moisture content of 1·30%. An error of 5% in the overall initial water content would lead to an error of about 0·05% in this calculated moisture content increase.

RESULTS

Figures 5 and 6 show results for increases in moisture content in undisturbed London clay when subjected to contamination from fresh concrete for one week. Fig. 5 shows the increase of moisture content of the clay after one day. As expected, the moisture content increase falls off rapidly as the sampling point is moved away from the tritiated concrete. At a distance of 5 cm the radioactivity is only 15% of its value at a distance of 1 cm.

Figure 6 shows the evolution of moisture content with time on the same sample, and presents the same trend observed by other researchers (Chuang & Reese, 1969; O'Neill and Reese, 1970; Yong, 1979) in laboratory work, using conventional techniques.

CONCLUSIONS

The method described involves the use of a radioactive labelling procedure to observe the movement of water in soil. Very small samples can be withdrawn, with the attendant advantages of much closer spacing and greater possible frequency and reduced experimental disruption. Much greater accuracy can therefore be obtained, while variability can still be assessed.

The experimental technique used in this method is relatively simple to perform, requiring only the counting equipment normally found in a radiochemistry laboratory. As sampling is carried out remote from this apparatus, the method can be used on full-scale experiments in the field.

Although the method entails the use of radioactive material, there is little or no hazard as the level of radioactivity is low (μCi) and the average energy of the tritium β radiation is very weak. Consequently the scale of the experiment can easily be increased and transport of samples to a radiochemistry laboratory for counting poses no problems.

In carefully conducted experiments the accuracy of the technique should be about two orders of magnitude better than can be obtained by conventional determination of moisture content by oven-drying.

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CLAYTON, C.R.I. AND MILITITSKY, J., 1983

"Installation Effects and the Performance of Bored Piles in Stiff Clays"

Ground Engineering, accepted for publication

INSTALLATION EFFECTS AND THE PERFORMANCE OF BORED PILES IN STIFF CLAY

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Introduction

For most bored piles shaft friction is the important component of bearing capacity. In the United Kingdom the conventional empirical design approach has been to evaluate the available side shear by applying a reduction factor to an average undrained shear strength profile obtained from laboratory tests on 38mm diameter specimens (Skempton (1959), Whitaker and Cooke (1966), Burland and Cooke (1974)). In the United States of America a similar approach has been used usually taking account of the method of construction, but often the shear strength has been based on correlations with in situ test results (O'Neill and Reese (1970), Reese and O'Neill (1971), Engeling and Reese (1974), Reese and Wright (1977), Wright and Reese (1979)). The fact that undrained shear strength is affected by test method is one of the major shortcomings of this approach.

The possibility of using effective stress analysis for the frictional component of pile bearing capacity has been appreciated for over 20 years (Eide, Hutchinson and Landva (1961)), but this method has not been widely used for design. More recently work by Burland (1973) and Parry and Swain (1976, 1977a, 1977b) has reawakened interest in the effective stress method proposed by Chandler (1966, 1968), but in reality there are many installation effects that must be understood before this type of analysis can be regarded as less empirical than the total stress approach. An advantage of the method is that the effective stress strength is relatively unaffected by test method but the actual practice of performing Constant Rate of Penetration pile tests presents a further problem: the drainage conditions around the pile are not known and the technique was initially introduced as an undrained procedure (Whitaker (1957, 1963), Whitaker and Cooke (1961)

All pile design methods are justified on the basis of experimental data obtained from loading full-scale piles in the field. The variability of pile test results is a significant problem; data presented by Fearenside and Cooke (1978) illustrate the degree to which test results on piles formed by the same technique and placed in close proximity in the field can vary (Figure 1). The ultimate load bearing capacity varied commonly by 20%, with a maximum variation of about 60%.

Stress Relief and the Use of Bentonite

The action of boring a hole in clay leads to a reduction of total stresses in the surrounding soil. Shear stresses are applied to the soil and porewater pressures are depressed in the area immediately surrounding the hole. When the soil is soft or the groundwater table is high then collapse of the hole may occur before concrete is placed. To overcome the problem of collapse the hole may be cased, but the need to insert and extract casing calls for expensive additional plant such as casing vibrators and additional cranage. A more economical approach uses a bentonite slurry in the hole.

The tendency of a hole to collapse is a function of the shear stresses applied to the soil forming the walls and of the seepage forces applied, for example, to blocks of soil bounded by fissures. Bentonite helps in both respects; it applies a total stress to the walls of the hole which normally (because its density is higher than that of water) more than compensates for the static groundwater component of total stress in the soil. In addition it balances the head of water in the soil and eliminates seepage. Because bentonite reapplies some of the total stresses lost during boring the rate of swelling of the clay is reduced and the soil consequently loses less strength than under "dry" conditions.

Bentonite has the property of forming a filter cake on the surface of soil into which it tries to seep (Fleming and Sliwinski (1977)). It has been argued that this is a desirable property, because the swelling of stiff clays can be slowed or even prevented, but in reality when fissured soil is below the ground water table then free water can readily travel from the surrounding soil mass to areas of relatively low pore pressure. Thus the filter cake is in reality a potential weakness in the soil-pile system because:

(a) it will have a relatively low undrained shear strength

(b) it has undesirable effective stress strength parameters.

The fact that concrete is tremied into the base of the pile means that there is at least the possibility that the filter cake is removed by a scouring action. The efficiency of this action depends on the relative Bingham yield stresses of the bentonite and the concrete. Since the yield stress of bentonite is very much lower than that of concrete, Fleming and Sliwinski (1977) have suggested that only very thin bentonite layers will be left in surface irregularities. Measurements of the shear strength of the cake (Veder (1963), Mesri and Olson (1970)) suggest that it will not be removed from the walls of the hole, and observations of the excavated faces of diaphragm walls in pervious soils confirm the fact that part of the bentonite cake remains. In clays the filtering effect on the side of a hole is slight, and only a very thin coating can be expected to form. At times a filter cake of a few millimetres thickness has been found (Fleming and Sliwinski (1977)), but because this is a relatively small thickness compared with the size of surface irregularities formed during drilling there is ample evidence to suggest that the effects of bentonite on the load carrying capacity of bored piles in clays are negligible, (Chadeisson (1961), Burland (1963), Fernandez (1965), Komornik and Wiseman (1967), Farmer et al (1970), O'Neill and Reese (1972), Corbeth et al (1974), Fleming and Sliwinski (1977) and Fearenside and Cooke (1978)).

Fresh Concrete and The Soil

On the basis that smear and/or filter cakes have a negligible effect in reducing water movement between the hole and the soil a number of researchers have tried to assess the effects of wet concrete on the adjacent soil, (Meyerhof and Murdock (1953), Skempton (1959), Mohan and Chandra (1961), Burland (1963), Taylor (1966), Chuang and Reese (1969), Chandler (1977), Fearenside and Cooke (1978)). However rapidly a hole is bored it is ultimately filled with a high slump concrete with a relatively high water/cement ratio (typically 0.6) and there must therefore be considerable opportunity for the soil adjacent to the shaft to swell.

Wet high slump concrete must apply total stresses to the sides of the hole into which it is poured. At high water/cement ratios the B value of the concrete is very close to unity if measured shortly after water is added to the mix, and its strength would be expected to be negligible. Unfortunately, at present, there is little way of knowing the magnitude of the total stresses applied to the edge of the hole. Formwork codes (A.C.I. (1958), C.E.R.A. (1965), C.I.R.I.A. (undate and other forms of analysis (for example see Levitsky (1973, 1975)) are inappropriate because they relate to very different geometries, much shorter lifts, and stiffer mixes. Relevant data can be found in DiBiagio and Roti (1972), Uriel and Otero (1977) and O'Neill and Reese (1978). Work by Uriel and Otero, and DiBiagio and Roti relates to pressures measured during concreting of diaphragm wall panels and is shown in Figure 2. At shallow depths the lateral pressures appear approximately hydrostatic, but at larger depths pressures do not increase so fast. This may be due to arching, or more probably the fact that the concreting of diaphragm wall panels and large bored piles can take a considerable time and that the deepest material begins to stiffen before the upper part of the concrete is placed.

There is a considerable amount of agreement between the observations of swelling made by different researchers. In both the laboratory and the field the results of simple tests such as the determination of moisture content indicate that there is significant swelling for between 2 and 7cm from the soil/concrete interface. Many test results show swelling zones about 5cm thick. Reported moisture content increases vary from nil to 7-10%. An increase of up to 5-6% is not uncommon. Considering the variations in geometry and soil type amongst the reports of soil softening, such agreement is surprising.

Figure 3 shows a simple laboratory test arrangement which can be used to allow clay to swell in contact with fresh mortar or concrete. A cylindrical sample of clay is placed on the base pedestal of a triaxial cell and the mortar or concrete is poured into a former above it. The assembly is sealed in a membrane and subjected to a cell pressure.

Figure 4 shows some results of such testing using sand/cement mortar and remoulded London Clay. After a period of contact with the mortar the clay can be removed and cut up into slices for moisture content determination; if sample preparation is not of the highest quality then problems arise because of the relative inaccuracy of moisture content determination compared with the variability of the specimen. In an attempt to overcome this problem a penetrometer based on the liquid limit apparatus (B.S.1377:1975 test 2 (a)) but with an 800g cone was also used, so that results before and after swelling could be compared.

Although such tests are simple in concept there are many complicating features. Figure 4(a) shows the effect of the mortar on moisture content in the clay. The results are typical of those quoted by other authors, but note the effect of lack of saturation in the mortar on the results. As the cell pressure is raised the pore pressures in the mortar rise less than those in the clay, and migration of water is reduced. These results are not relevant to the field situation, where a high water/cement ratio is necessary. Figure 4(b) shows cone penetration resistance against distance from the mortar/clay interface. It is clear that some considerable time is necessary for moisture migration, but it is also found that after a period of about 1 week further changes are small. In addition, however, it should be noted that the shear strength close to the interface is higher than might be expected from its moisture content. From the test results it seems that the contamination in the water leaving the mortar may penetrate and strengthen soil close by. This process has, perhaps, been observed by Chuang and Reese (1969).

Water/cement ratio can be expected to have a significant effect on the amount of swelling occurring in soils adjacent to mortar or concrete, and Figure 4(c) confirms that this is so. Concretes with a low water/cement ratio can clearly be seen to be generating significant negative excess pore water pressures since decreases in soil moisture content occur, but as Figure 4(d) shows concretes and mortars at virtually all practical water cement ratios will allow an over-consolidated soil to extract water.

The key to the similarity between the results of moisture content increase produced by different workers probably lies with the idea that it is the concrete which dominates the process. Fresh concrete has a high permeability when compared with most clay soils (Figure 5). When placed in contact with soil the effective stresses within a fresh high slump concrete almost certainly approach zero, whilst pore pressures in the surrounding soil have been depressed by stress relief. Swelling of the soil is initiated, but even as it starts the pore pressures in the concrete fall rapidly as hydration of the cement takes place, (Figure 6). Within a few hours pore pressures fall to such a level that swelling on the soil/concrete interface is prevented and may be partially reversed. After hydration and set has taken place any significant flow of water into the concrete is likely to be small because of the relatively low compressibility (in effective stress terms) of the concrete.

In some ways the similarity between field and laboratory observations of swelling is most unexpected. In the laboratory, soil is placed under controlled (i.e. isotropic) stress conditions against a sample of concrete or mortar, and pore pressures can be seen to equalise with time. How, then, do the moisture contents along the specimen vary once equalisation is complete and effective stress levels are supposedly uniform? The stress path followed by each part of the clay specimen would be expected to be the same, leading to identical moisture contents along its length. Presumably swelling can occur adjacent to the mortar while it is wet and pore pressures are rising, but once set takes place, shear stresses would be applied at the mortar/clay interface when pore pressures begin to fall. In the field such a mechanism seems extremely unlikely; all swelling strains would be expected to occur normal to the concrete/soil surface, and the amount of swelling might then be expected to be a function of the compressibility of the wet concrete.

Re-establishment of Stresses and the Mechanism of Load Transfer

Since soil is found adjacent to piles in the field at moisture contents in excess of their initial values (Figure 7), it is clear that the in situ undisturbed lateral effective stresses are not maintained during the installation of bored piles. Even if they were maintained, one of the major problems in the application of effective stress design methods would be the difficulty and expense of determining the in situ coefficient of earth pressure at rest. In reality the situation is more complicated because the softened zone around the pile is partially reconsolidated as soil creep leads to a re-establishment of horizontal effective stresses. There appear to be no direct field measurements of this effect in the literature, although the writers are at present engaged in such a study. Reanalysis of some of the results presented by Whitaker and Cooke (1966), Taylor (1966) and Combarieu (1975) has demonstrated that the shaft friction component can increase over a period of many months (Figure 8), presumably as a result of increasing horizontal effective stresses. This has serious implications for back-analysis, on which virtually all design methods are based, since load tests are often carried out shortly after the piles are formed when strengths are rapidly changing.

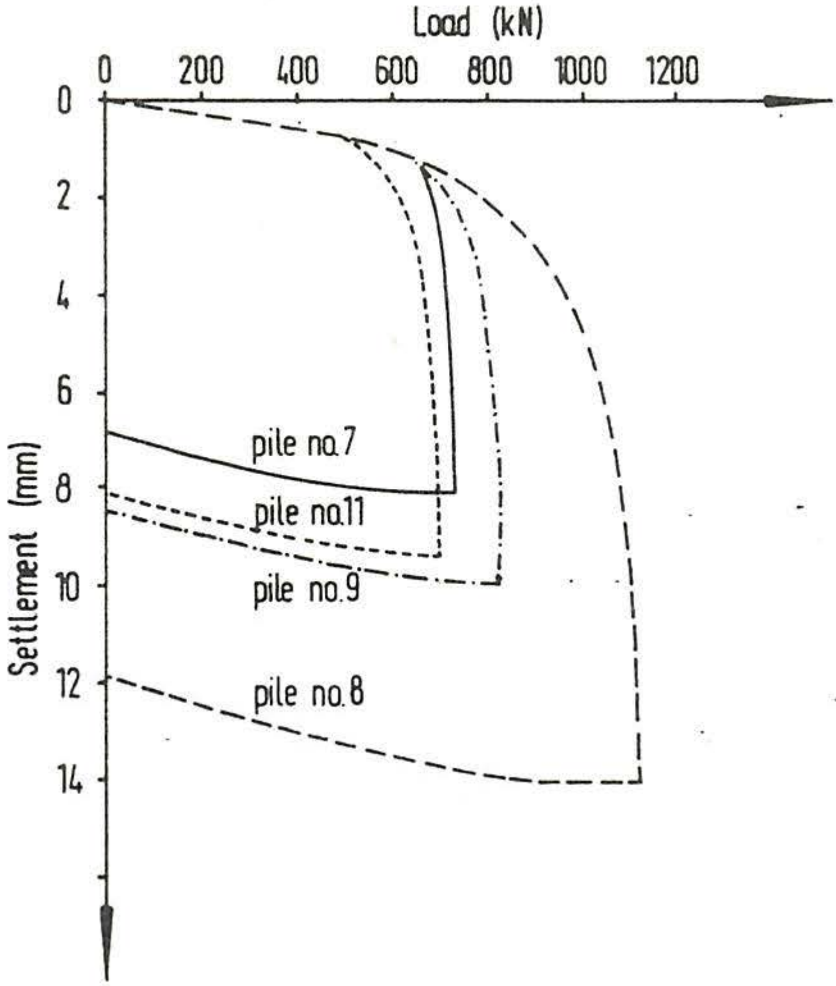
In the final analysis, if horizontal effective stresses are known then the shear strength distribution along the pile shaft should be predictable. Whatever the coefficient of earth pressure acting in the soil around the shaft, the horizontal effective stresses and thus the available shear strength should increase with depth. In reality observations made on instrumented test piles do not show a mobilised shear strength which increases with depth. O'Neill and Reese (1972) carried out tests in the stiff Beaumont Clay in Texas, using piles instrumented for load measurement at various points along the reinforcing cage. As Figure 9 shows, the distribution of shear stress on the shaft is strongly dependent on load level, reaching a parabolic stage at high load level, and bears no relation to either the undrained shear strength or the probable horizontal effective stress levels.

Conclusions

In principle, effective stress analysis of bored piles seems attractive as it offers a chance of carrying out a design which at first sight comes nearer to the fundamental mechanisms at work in the soil/pile system. Close inspection of some of the available data, however, reveals that the real situation is dependent on methods of installation, on creep rates and on a number of other factors not readily calculable.
At present a realistic and simple effective stress analysis of the shaft friction of bored piles in stiff clay seems unlikely; at best these methods are capable of yielding an upper bound solution.

June, 1981

FIGURES



Load (kN)
0
200
400
600
800
1000
1200
Settlement (mm)
0
2
4
6
8
10
12
14
pile no.7
pile no.11
pile no.9
pile no.8

FIGURE 1

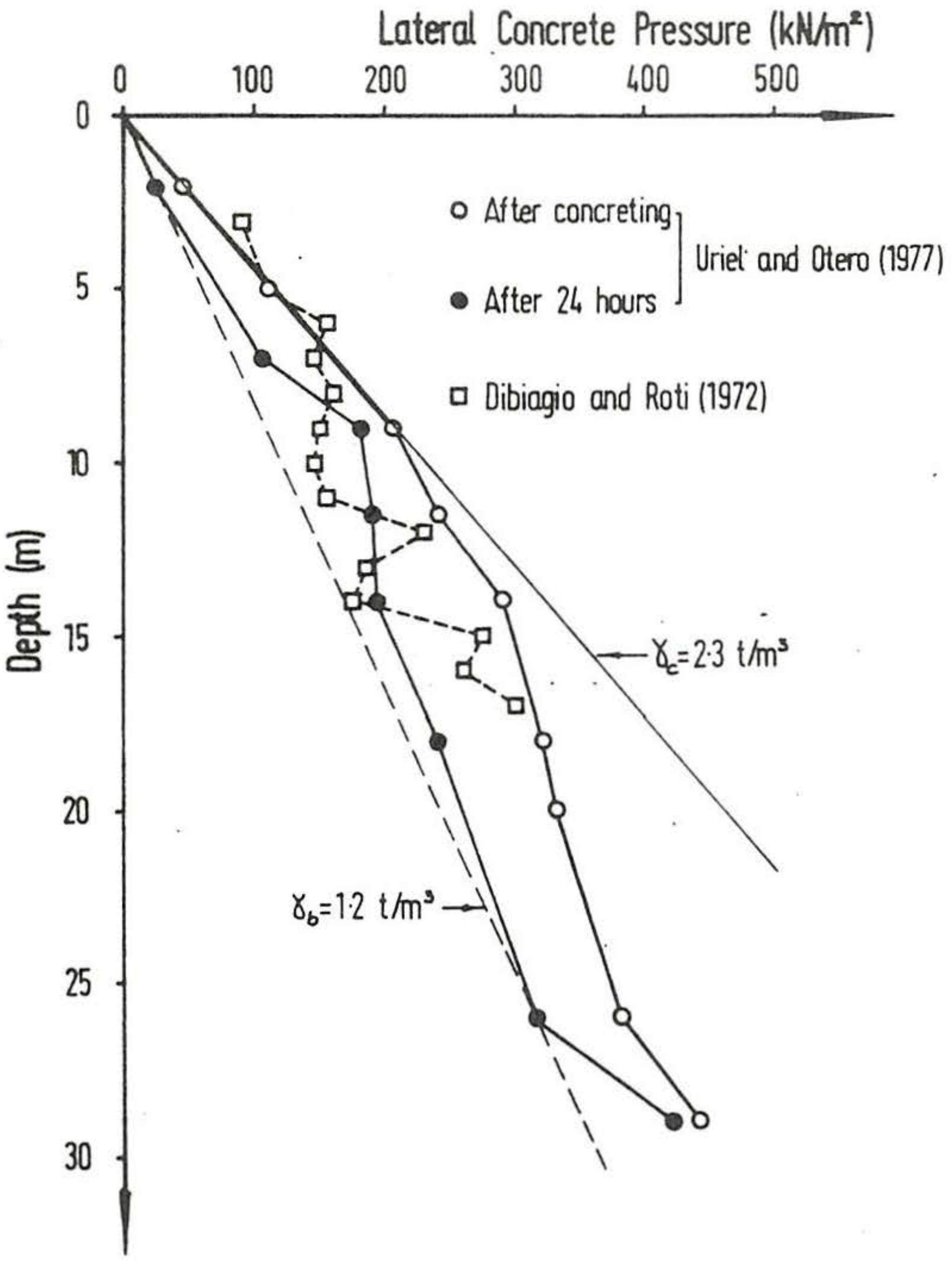

Lateral Concrete Pressure (kN/m²)
0
100
200
300
400
500
Depth (m)
0
5
10
15
20
25
30
After concreting
After 24 hours
Uriel and Otero (1977)
Dibiagio and Roti (1972)
γc=2·3 t/m³
γb=1·2 t/m³

FIGURE 2

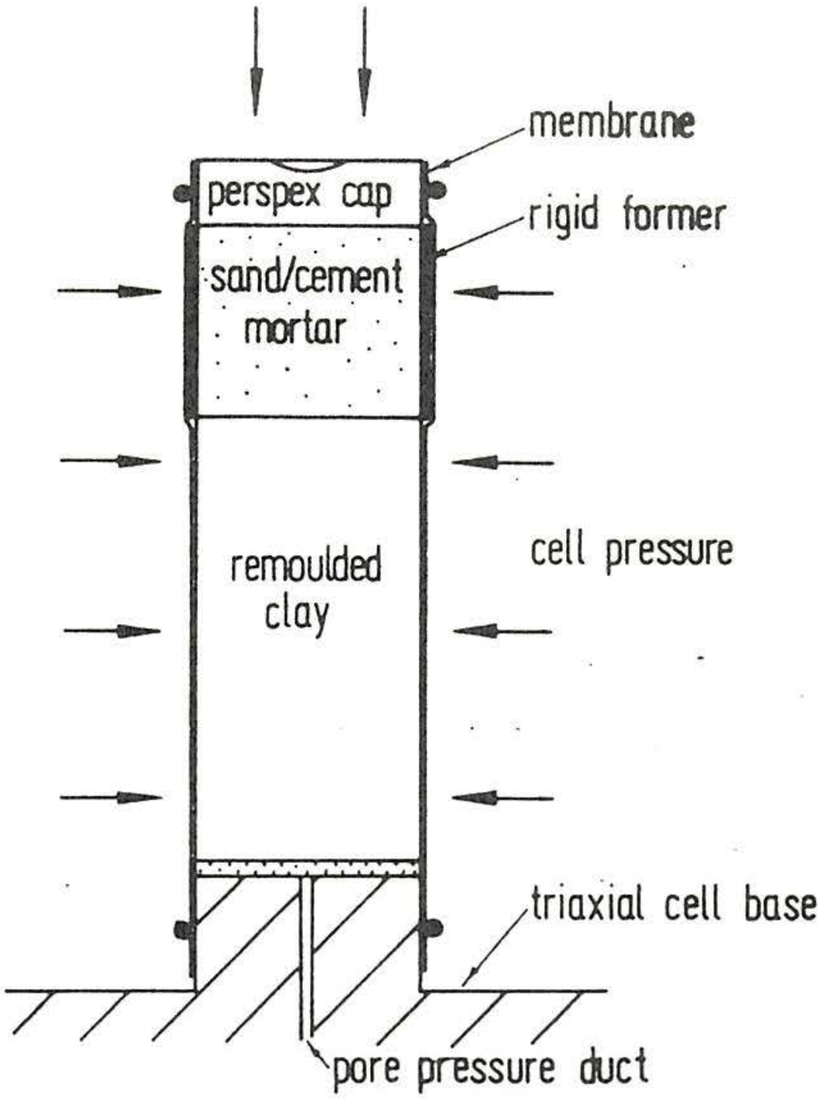

membrane
perspex cap
rigid former
sand/cement mortar
remoulded clay
cell pressure
triaxial cell base
pore pressure duct

FIGURE 3

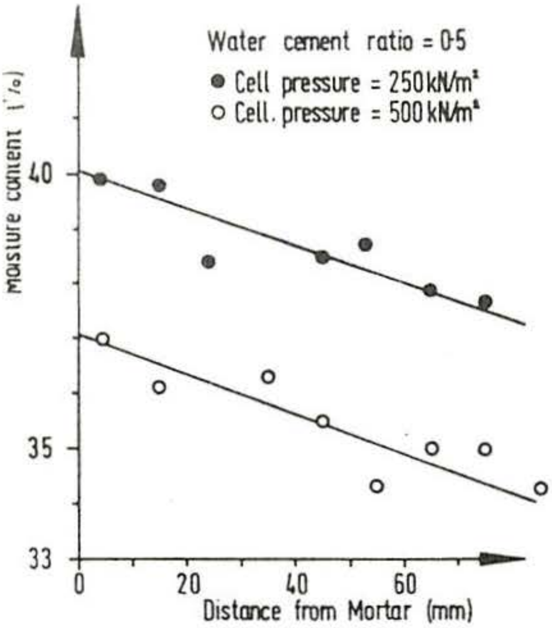


FIGURE 4a

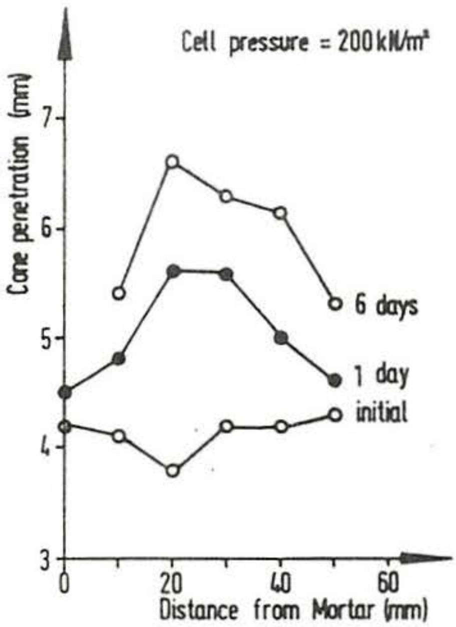


FIGURE 4b

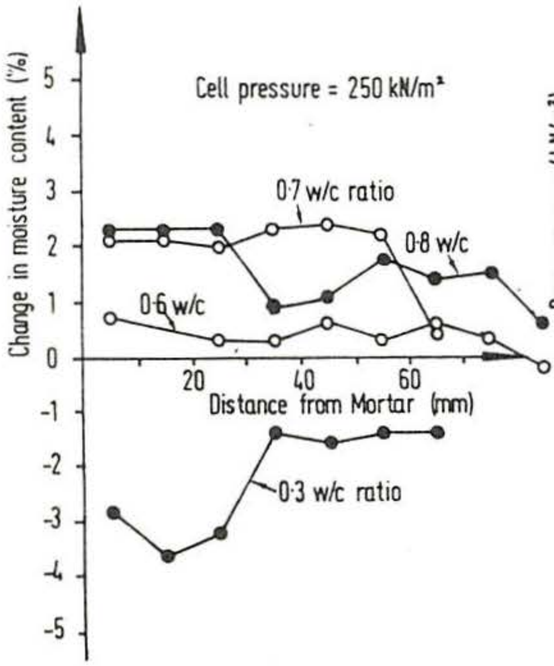


FIGURE 4c

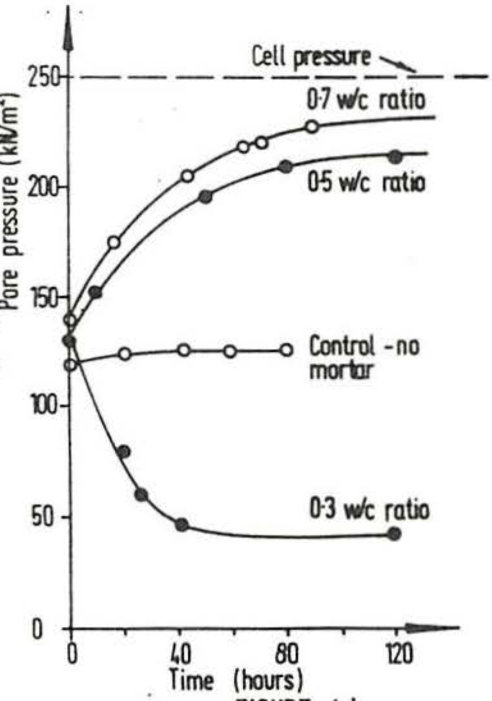


FIGURE 4d

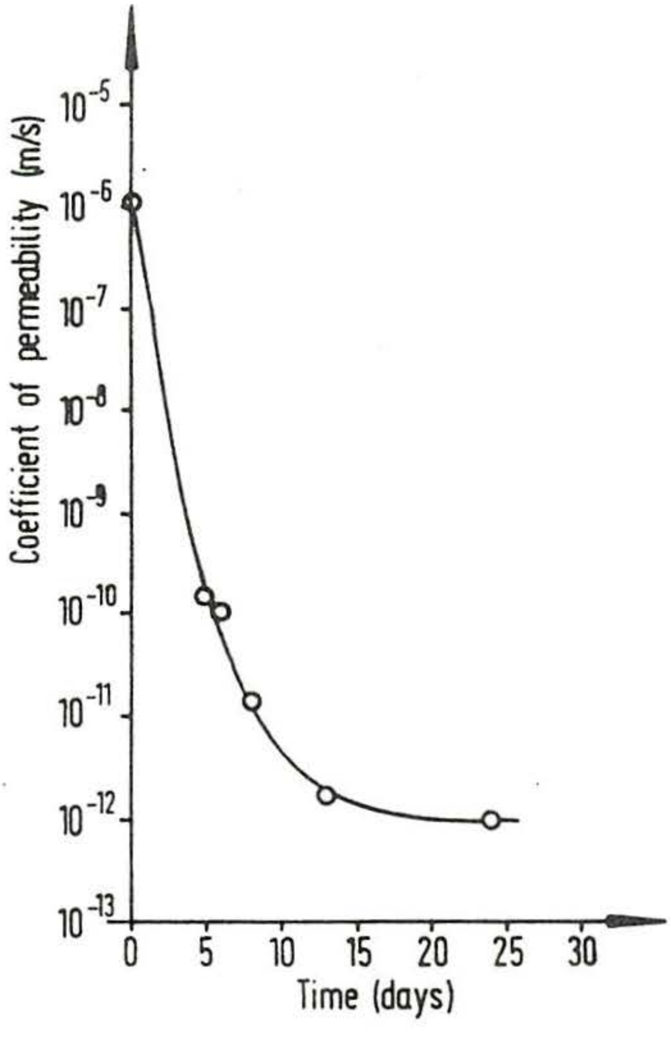

Coefficient of permeability (m/s)
10^{-5}
10^{-6}
10^{-7}
10^{-8}
10^{-9}
10^{-10}
10^{-11}
10^{-12}
10^{-13}
0
5
10
15
20
25
30
Time (days)

FIGURE 5

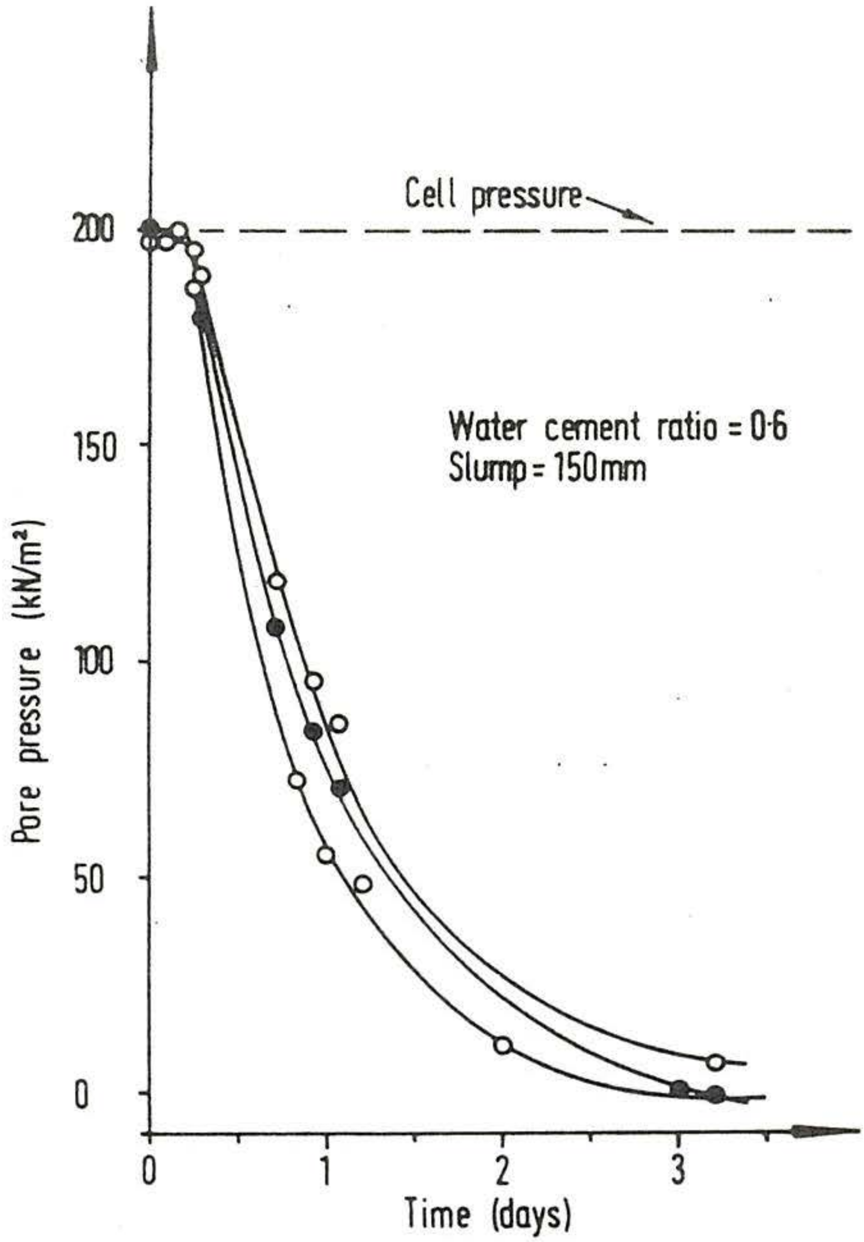

Cell pressure
Water cement ratio = 0·6
Slump = 150mm
Pore pressure (kN/m²)
200
150
100
50
0
0
1
2
3
Time (days)

FIGURE 6

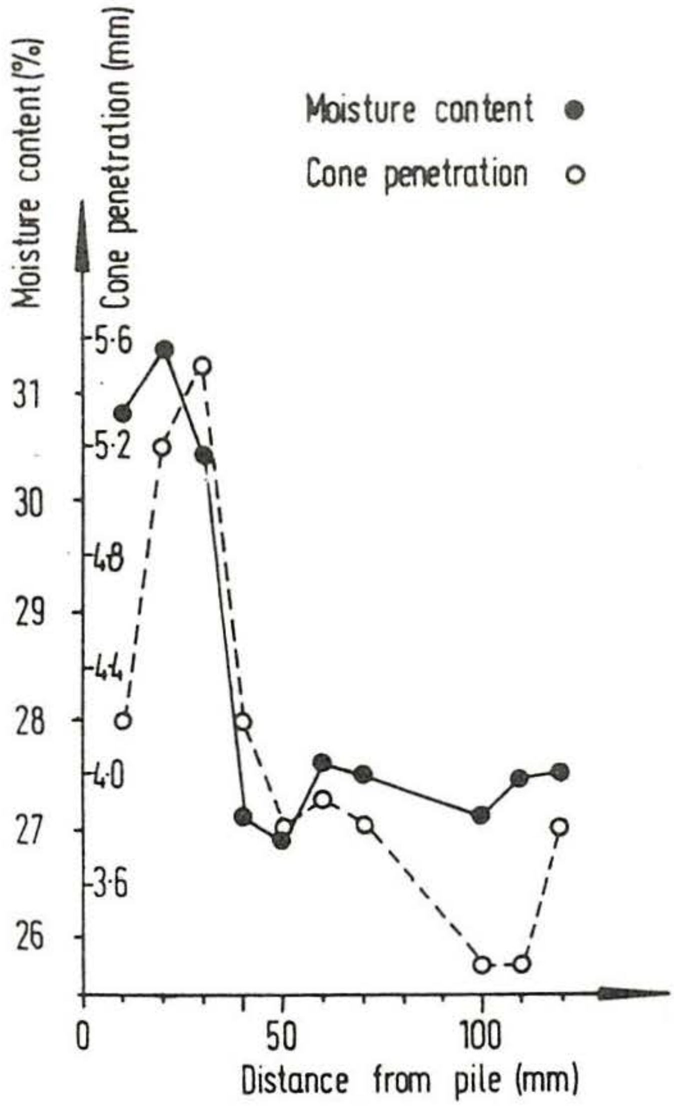

Moisture content (%)
Cone penetration (mm)
Moisture content
Cone penetration
5·6
5·2
4·8
4·4
4·0
3·6
31
30
29
28
27
26
0
50
100
Distance from pile (mm)

FIGURE 7

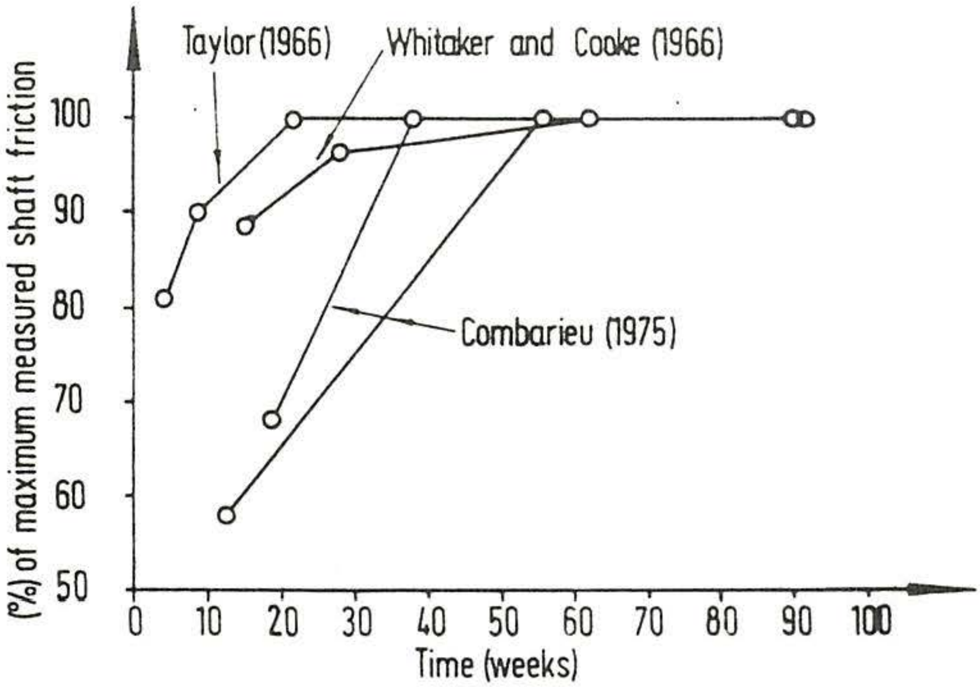

Taylor (1966)
Whitaker and Cooke (1966)
Combarieu (1975)
(%) of maximum measured shaft friction
100
90
80
70
60
50
0
10
20
30
40
50
60
70
80
90
100
Time (weeks)

FIGURE 8

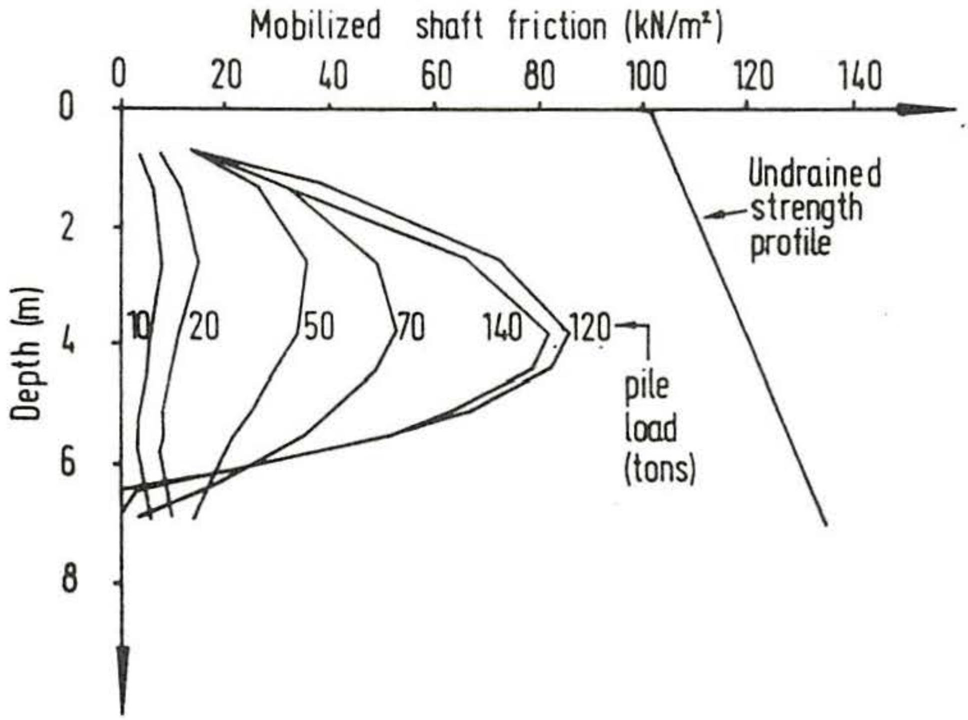

Mobilized shaft friction (kN/m²)
0
20
40
60
80
100
120
140
0
2
4
6
8
Depth (m)
10
20
50
70
140
120
pile
load
(tons)
Undrained
strength
profile

FIGURE 9

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