

GROUND HEAVE AROUND DRIVEN PILES
IN CLAY

by
GUE, SEE SEW

A Thesis submitted for the Degree
of Doctor of Philosophy
at
University of Oxford.

Hertford College

Hilary term, 1984.

ABSTRACT

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Large diameter piles have increasingly been used, especially for offshore foundations. Most of the piles that have been used offshore have been open-ended. There is evidence from previous work that closed-ended piles have merits over open-ended piles; they have a greater ultimate axial capacity and are easier to drive than open-ended piles of the same outer diameter. However, because a greater volume of soil is displaced, closed-ended piles give rise to much greater ground heave. This additional heave may cause unacceptable distortion to the structure as each leg is fixed to the sea-bed in turn. For this reason, this research was commissioned.

Laboratory studies of ground heave around driven open- and closed-ended piles (16 mm o.d.) were carried out. Two different types of specimen reconstituted from Speswhite kaolin were prepared; semi-cylindrical and full cylindrical specimens each 450 mm in diameter and about 450 mm in height. The semi-cylindrical specimens were used to obtain qualitative data about the movements of soil around the driven pile. The full cylindrical specimens were used to take accurate measurements of the ground heave during pile driving and to study the influence of the ground heave on a small surcharged collar through which a pile was driven. A further series of tests investigated the effect of ground heave when a group of piles was driven through a larger surcharged collar which was used to represent the mudmat of an offshore structure.

The lateral movement of soil at around the mid-depth of a driven pile agreed with the cavity expansion model (plane strain analysis) assuming undrained conditions and no vertical movement.

Closed-ended piles were found to generate about four times the magnitude of ground heave compared to open-ended piles with an area ratio of 34%. Ground heave appeared to be unaffected by overconsolidation ratio and rate of pile driving (22 to 150 blows/min.). However, maximum vertical consolidation pressure seemed to have a small effect. A remarkably good agreement was obtained between the field data and the results of this investigation when both were made dimensionless.

The surcharged collar and model mudmat generally settled during pile driving. However, when very low surcharge pressures were used, the collar and mudmat were lifted up. For pile groups, the surcharged mudmat tilted towards the pile being driven. The magnitude of this tilt for the mudmat was greater when closed-ended piles were used.

PREFACE

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Finally, but by no means least, I want to express my thanks to my family for their encouragement, especially my wife for her loving care and understanding.

NOTATION

c_u	Undrained shear strength
c_v	Coefficient of consolidation
C/E	Closed-ended pile
e	Voids ratio
G	Specific gravity of soil
K_o	Stress ratio for one-dimensionally consolidated clay
M	Slope of critical state line (Atkinson and Bransby, 1978)
O/E	Open-ended pile
O.C.R.	Overconsolidation ratio in terms of vertical effective stresses
O.D.	Outer diameter
p'	Mean effective stress $((\sigma'_1 + \sigma'_2 + \sigma'_3)/3)$
q	Deviator stress $(\sigma'_1 - \sigma'_3)$ in triaxial stress space
R	Radial distance from pile axis
R_o	Pile radius
U	Total pore pressure
ΔU	Excess pore pressure
V	Specific volume $(1+e)$
ϵ	Linear strain in vertical direction
κ	Slope of swelling line in $V - \ln(p')$ space
λ	Slope of consolidation line in $V - \ln(p')$ space
ν	Poisson's ratio

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CHAPTER 1

INTRODUCTION

The requirements of certain structures, especially offshore platforms have necessitated the use of increasingly large diameter piles for the foundations. For example, British Petroleum Limited's oil production platform at Magnus field in the North Sea has nine piles driven through each mudmat at the base of the four legs of the platform. The piles are 2134 mm in outer diameter with a wall thickness of 63 mm and have an embedded length of about 100 metres. It is current practice to drive these large diameter piles open-ended.

Theoretical and experimental evidence exists to show that closed-ended piles have merits over open-ended piles when driven into clay. Closed-ended piles :

1. are easier to drive than open-ended piles of the same outer diameter, and
2. have greater ultimate axial capacity.

Rigden, Pettit, St. John, and Poskitt (1979) described the results of some large scale field tests and reported that the axial capacity of a closed-ended pile driven into glacial till is about 21% higher than that of a similar

open-ended pile. The tests also demonstrated that the closed-ended pile is much easier to drive than the open-ended pile. Carter, Randolph and Wroth (1980) have shown in a theoretical study that the final shaft capacity of a closed-ended pile driven into a particular clay deposit may be approximately 25% greater than that of a similar open-ended pile.

Avery and Wilson (1950) reported from their laboratory test results that after reconsolidation the shear strength of medium sensitive and non-sensitive clays increases with the extent of disturbance. They explained that by driving a displacement pile into a fully saturated clay, it causes an increase in pore pressure around the pile. This means that the effective stress in the clay moves from point A to point B, (Figure 1.1a.). In this condition the clay is only partly consolidated. The shear strength is correspondingly reduced from point C to point D. The badly disturbed clay immediately begins to reconsolidate under the new stress conditions that have arisen from pile driving and would reach full consolidation at E. However, if part of the weight of the overlying clay is supported by negative skin friction, between the pile and the clay, the consolidation would stop at an intermediate point F, corresponding to full consolidation under a pressure which may be reduced by this effect. The shear strengths corresponding to points E and F are marked by points G and H, respectively. The same relationships for a non-sensitive clay are shown in

Figure 1.1b but with a much smaller effect than for the medium sensitive clay.

Flaate (1972) also reported that the unconfined compression tests on samples taken more than five years after pile driving show a distinctly higher shear strength for clay nearer to the pile than for clay further away from the pile.

Recently, Martins(1983) has undertaken microfabric studies of the clay around model piles. Most of his 15 mm diameter piles were cast in place. However, he also carried out one test in which the pile was driven into the 102 mm diameter Speswhite kaolin sample and one test in which the pile was jacked. Installation for both cases was completed in half an hour, after which each of the piles was loaded at a constant rate of displacement. He confirmed by microfabric study that a driven pile causes more disturbance than a jacked pile and also showed that the mean shear stress on the shaft of a driven pile is slightly more than 10% greater than on a jacked pile.

Theoretical studies (Wroth, Carter and Randolph, 1979) using a finite element technique together with known Cam-clay parameters for a number of different clays have been undertaken. They have shown that the ultimate strength of the soil close to a closed-ended pile is about 1.5 times the original strength.

If the two advantages mentioned above are confirmed, then considerable savings in both the material cost of the piles

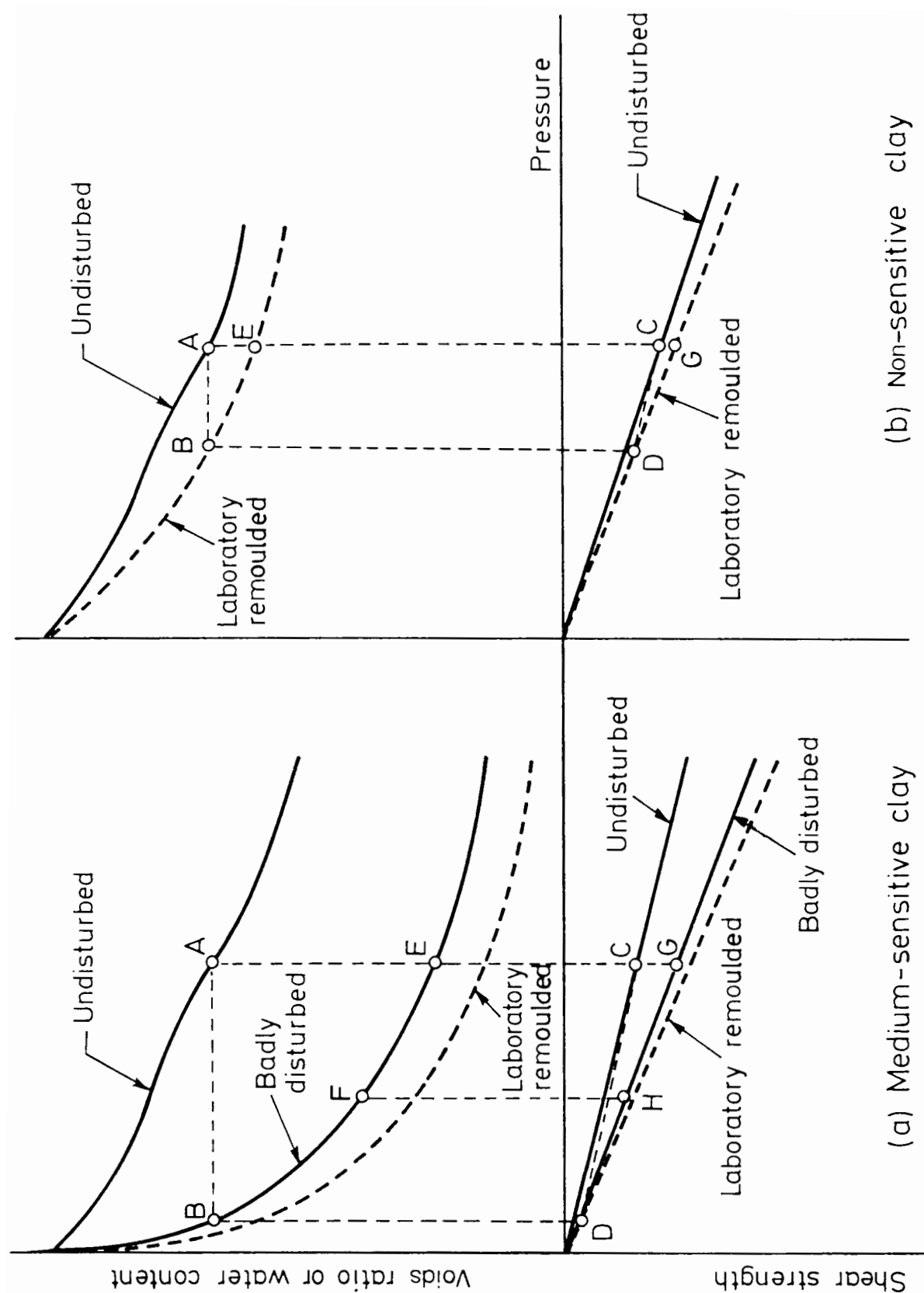


Figure 1.1 Laboratory tests illustrating effect of disturbance on consolidation and strength characteristics of sedimentary clays (after Avery and Wilson, 1950)

and the cost of their installation can be achieved by using closed-ended piles.

One possible disadvantage of using closed-ended piles is the additional heave that might be generated because of the greater volume of soil displaced. If the heave is excessive, the platform might be distorted or overstressed and it is arguable that it might even tilt slightly, particular when the first group of piles are driven through a mudmat. For example, the steel jacket of an oil production platform might become distorted as each leg is fixed in turn to the sea-bed. The lack of knowledge and uncertainty surrounding this problem is a major factor prohibiting the use of closed-ended piles in offshore clay foundation.

There is very little known about the surface ground heave due to pile driving, and there appears to be nothing on its influence when piles are driven through a surcharged foundation. This is because ground heave is not critical for most foundation. The site where piles have been driven can easily be levelled before building the structure. Sometimes piles heave up due to driving adjacent piles. They can then be redriven if necessary. The literature review in the next chapter therefore shows that little data has been published covering this topic. Consequently, this research was commissioned.

The main purpose of this investigation is therefore to study the effect of ground heave around driven piles in clay. Reconstituted kaolin was used as the basic material. It was consolidated one-dimensionally from slurry with both top and bottom drainage.

Two series of model tests were carried out. The first series was aimed at obtaining qualitative data from the pattern of deformation in the clay as a pile is driven. This was achieved by installing a semi-cylindrical pile along the vertical face of a semi-cylindrical clay specimen retained by a perspex sheet. The second series of model tests featured the driving of a model pile into the centre of clay specimen and surface heave was measured accurately using electronic displacement transducers. The surface heave due to pile driving and its effect when suppressed by imposed surface loads were examined.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This section reviews previous work on the effects of pile driving in clay, with particular emphasis on surface or ground heave. Although a great deal of attention has been given to construction problems associated with piled foundations over the last three decades, very little is known about the movements of the soil, particularly near the ground surface and in the immediate vicinity of the pile at greater depth.

2.2 Effects of Pile Driving in Clay

The effects of pile driving in clay fall into two main categories :

1. Deformation of soil surrounding the pile, and
2. Alteration of soil properties.

2.2.1 Deformation of Soil

The three notable effects of deformation due to driving are ground heave, pile heave and lateral movement of ground. These effects are discussed below :

(a) Ground heave

When piles are driven, soil is displaced. Some of this displaced soil moves upwards, since it is unrestrained in this direction. The vertical soil movement at the surface is known as ground heave. This effect has generally been studied in relation to the uplift generated on adjacent piles, and in very few cases has the actual ground heave around the pile been examined.

The movements of soil close to a driven pile in clay have been investigated by Cummings, Kerkhoff and Peck (1950). Zeevaert in his discussion of this paper, has reported a ground heave of 13 ins. (330 mm) in the centre of a group of wooden piles driven to a depth of about 110 feet (33.5 metres) into soft volcanic clay. In this sensitive clay, the heaved soil settled back quickly almost to its original position after about one month. Zeevaert has presented an approximate classification of the state of clay surrounding the pile (Figure 2.1). The material further than one pile diameter from the pile (in zone III) is assumed to remain unaltered during pile driving except for a temporary upward elastic deformation due to the pressure caused by the displaced volume of remoulded clay during pile driving.

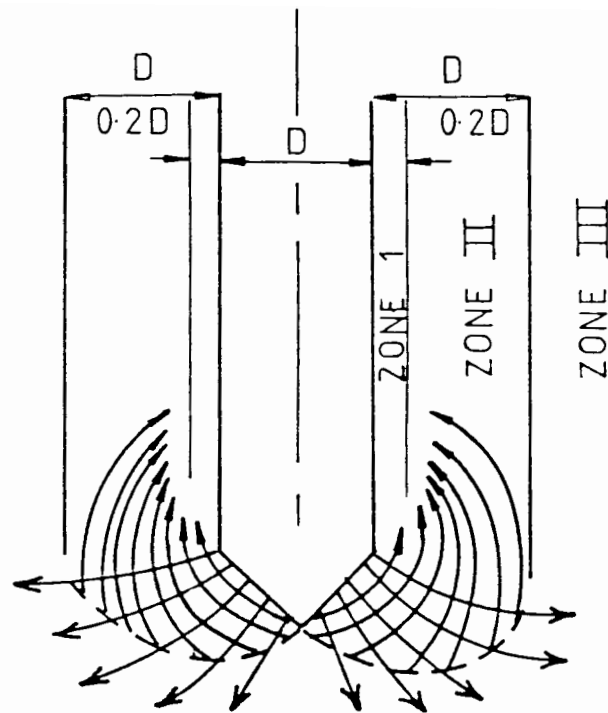


Figure 2.1 Displacement pattern around the tip of a pile driven into cohesive soil (after Zeervaert, 1950)

For about the next two decades, there were no ground heave measurements added to the available literature. Hagerty and Peck (1971) studied the heave caused by pile driving at 16 sites with diverse soil conditions. The average soil heave normalised by the length of the piles was determined for 8 of the 16 sites as a function of the ratio of total volume of the inserted piles to the volume of soil enclosed or surrounded by those piles (volume displacement ratio). This

relationships (Figure 2.2) shows that the volume of soil displaced within the pile group is about equal to one half of the total volume of piles inserted. Surface heave is therefore a function of pile length.

The proportion of the total volume of heave to the total volume of embedded piles which appears as ground heave within the plan area of a foundation has been found to be about 50% by Cummings et al. (1950) for driven cast in-situ concrete piles in soft blue clay. Orrje and Broms (1967) obtained a figure of 40% for precast concrete piles in a soft, sensitive and silty clay. Clearly, the ground heave outside the foundation area has a significant contribution to the total volume of ground heave, but was not included. In contrast, when the ground heave outside a pile group is included, an agreement is obtained. Adams and Hanna (1971) found that for steel H-piles in a firm till, all of the embedded volume appeared as heave. Similarly, Massarsch (1976) reported a figure of 100% in his study.

The movements of soil around a single pile were first studied by Cooke and Price (1973). An instrumented closed-ended pile was jacked into London clay. They reported that most of the ground heave developed during initial pile penetration up to 12 pile radii (about 1 metre). Movements were greatest within 4 pile radii from the pile axis, and decreased radially. Movements are very small but measurable beyond 20 pile radii from the pile axis; at the final penetration, the surface movement beyond

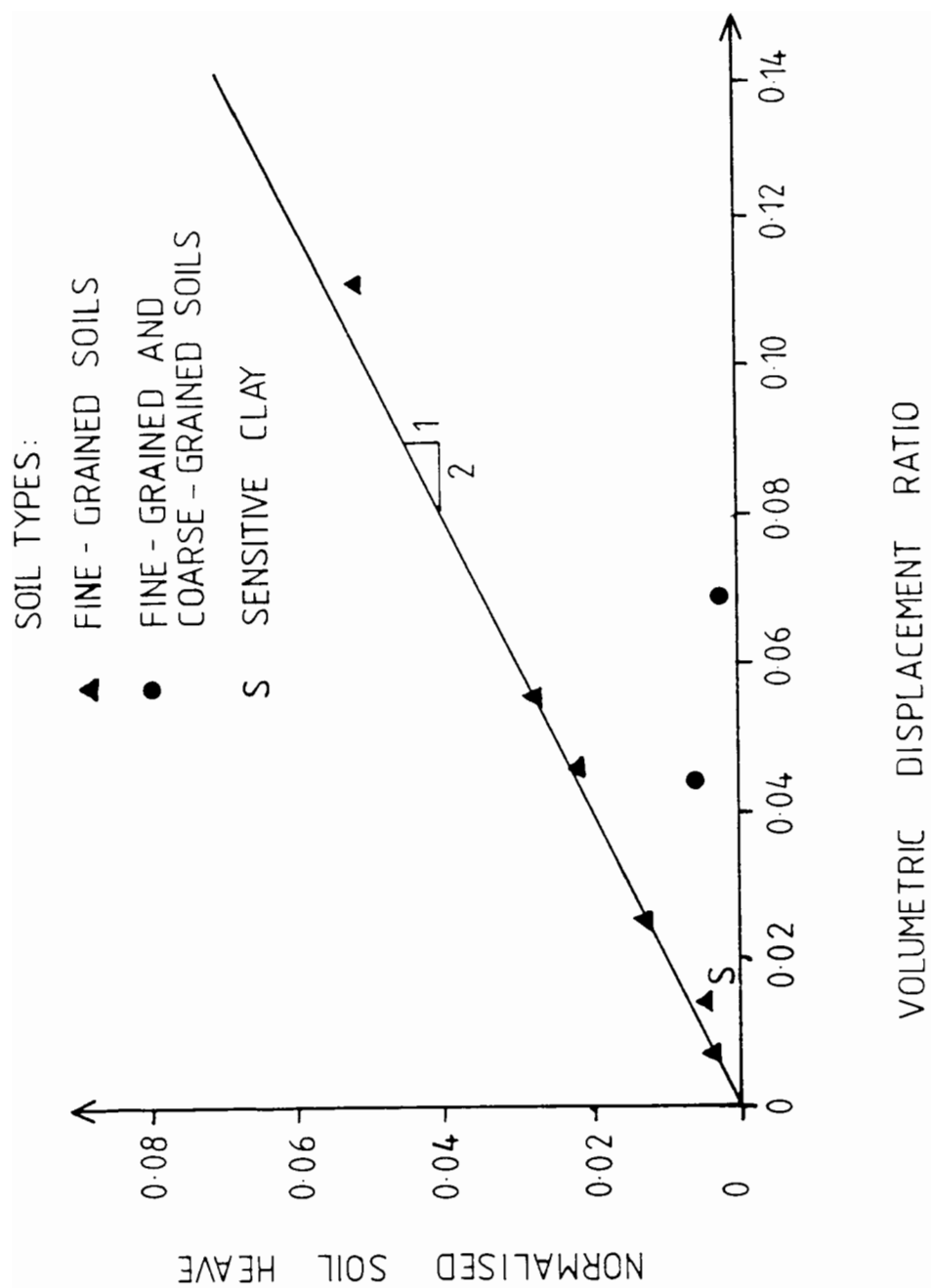


Figure 2.2 Surface heave due to pile driving (after Hagerly and Peck, 1971)

20 pile radii is independent of radial distance. Tomlinson (1957) suggests that these movements are possibly caused by the vibration of the rotary pump used for the hydraulic system.

Subsequent work at the same site has been reported by Cooke, Price and Tarr (1979), the difference being that the vertical movement probes were installed at 0.5 metres below ground level. Their conclusions differed slightly from Cooke and Price (1973). Upward movements of soil continued until the pile reached a penetration of about 24 pile radii (i.e. 2 metres). There was no further significant change in ground heave close to the pile when the penetration of pile continued beyond 24 pile radii. The observed vertical displacement profile was generally similar to those obtained by Cooke and Price (1973).

In contrast, Randolph, Steenfelt and Wroth (1979) presented the results of laboratory model tests which observed the soil deformation patterns around open- and closed-ended piles during jacking. The studies were performed using a cylindrical sample of clay prepared from Speswhite kaolin and one-dimensionally consolidated. The sample was cut in half to give two specimens which were semi-circular in plan view. The flat vertical face of each specimen was marked with grid lines and placed against a lubricated perspex sheet. A semi-cylindrical pile was rapidly jacked into the

clay, flush with the perspex plate. Photographs were taken during pile installation and the movements of soil determined. The results showed no detectable ground heave. This may have been due to the presence of voids between the vertical face of the sample and the perspex plate.

Sharp (1982) provided some ground heave measurements for each of nine piles driven into stiff clay at Cowden, Hull. Open- and closed-ended piles of 200 mm and 300 mm in outer diameter were used. The smaller piles had a wall thickness of 9.5 mm and the larger one was 14.3 mm. Open-ended piles were driven with driving shoes and the area ratio based on the driving shoe area thickness was 34% for both pile sizes. Ground heave measurements to an accuracy of ± 0.1 mm were made possible by precision levelling. The results for closed-ended piles agreed fairly well with the results of Cooke et al. (1979). The following points were established from their investigation :

1. a consistent qualitative ground heave profile due to pile driving was observed. Figure 2.3 shows the profile for a closed-ended pile.
2. The closed-ended piles showed a distinct difference in the amount of ground heave generated from open-ended piles with an area ratio of 34 %. The maximum ground heave generated by the closed-ended pile was about 5 - 6 times that of a similar open-ended pile.

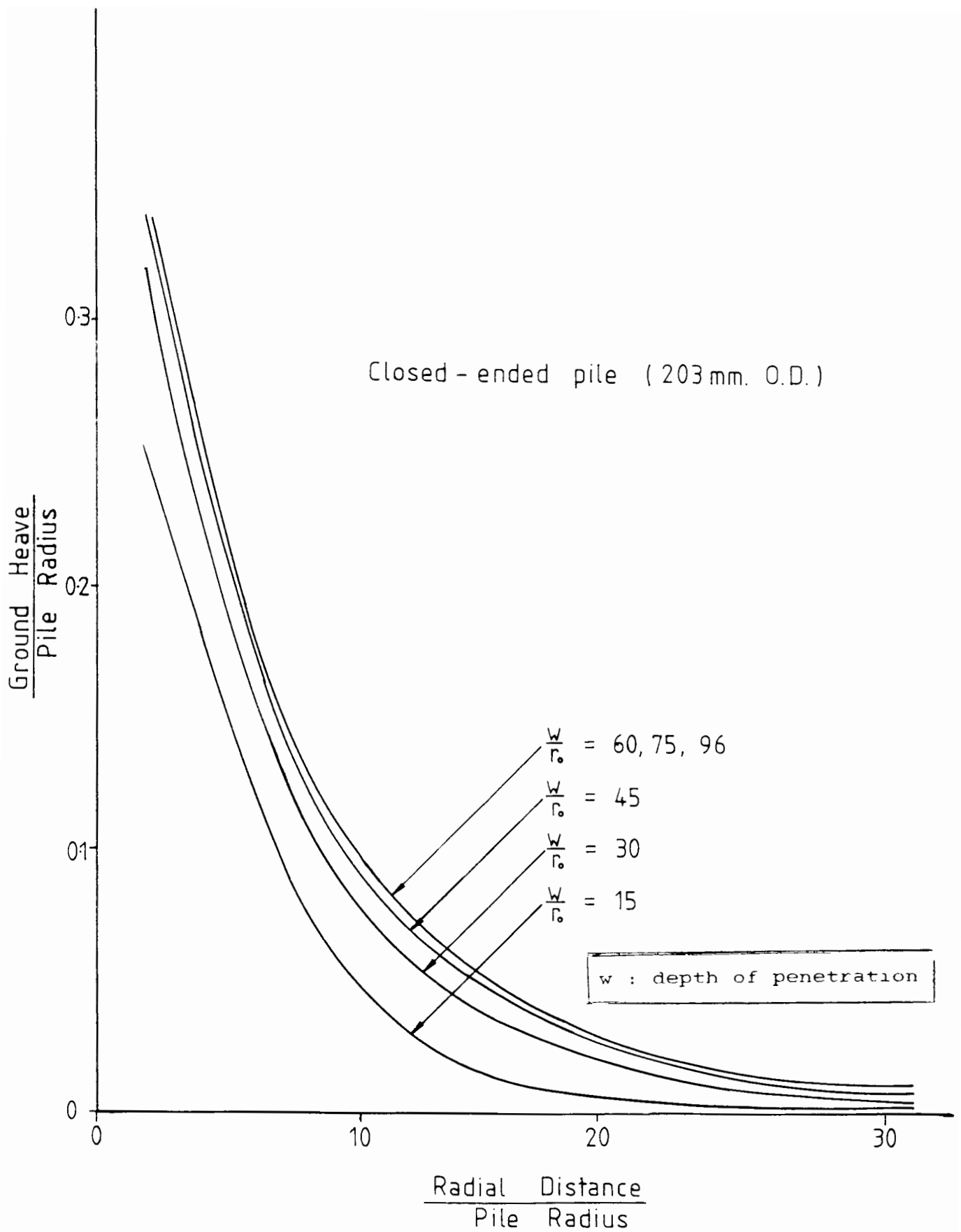


Figure 2.3 Normalised ground heave profile (data from Sharp, 1982)

3. Ground heave was greatest near the pile wall and tapered off at a radial distance of 20 pile radii away from the pile axis.
4. Closed-ended piles of different diameters had a similar relationship of total ground heave against depths of penetration. Most of the ground heave within about 11 pile radii from the pile axis was generated over the initial 30 pile radii of pile penetration. Beyond this depth there was little further heave (Figure 2.4). There seemed to be a critical or limiting depth of penetration by which most of the ground heave close to the pile would have been generated during pile driving.

(b) Pile Heave

It appears that instances of pile heave or uplift of piles due to the subsequent driving of an adjacent pile cited in the literature are all field observations. This phenomenon commonly occurs whenever large numbers of closely spaced piles are driven in clay.

It has been acknowledged that pile heave occurs when other piles are driven close to them : Chellis (1962), Tomlinson (1963) and Terzaghi (1967). A common remedy is to redrive all the piles which have heaved. Klohn (1961) has presented detailed field data with analysis pertaining to pile heave

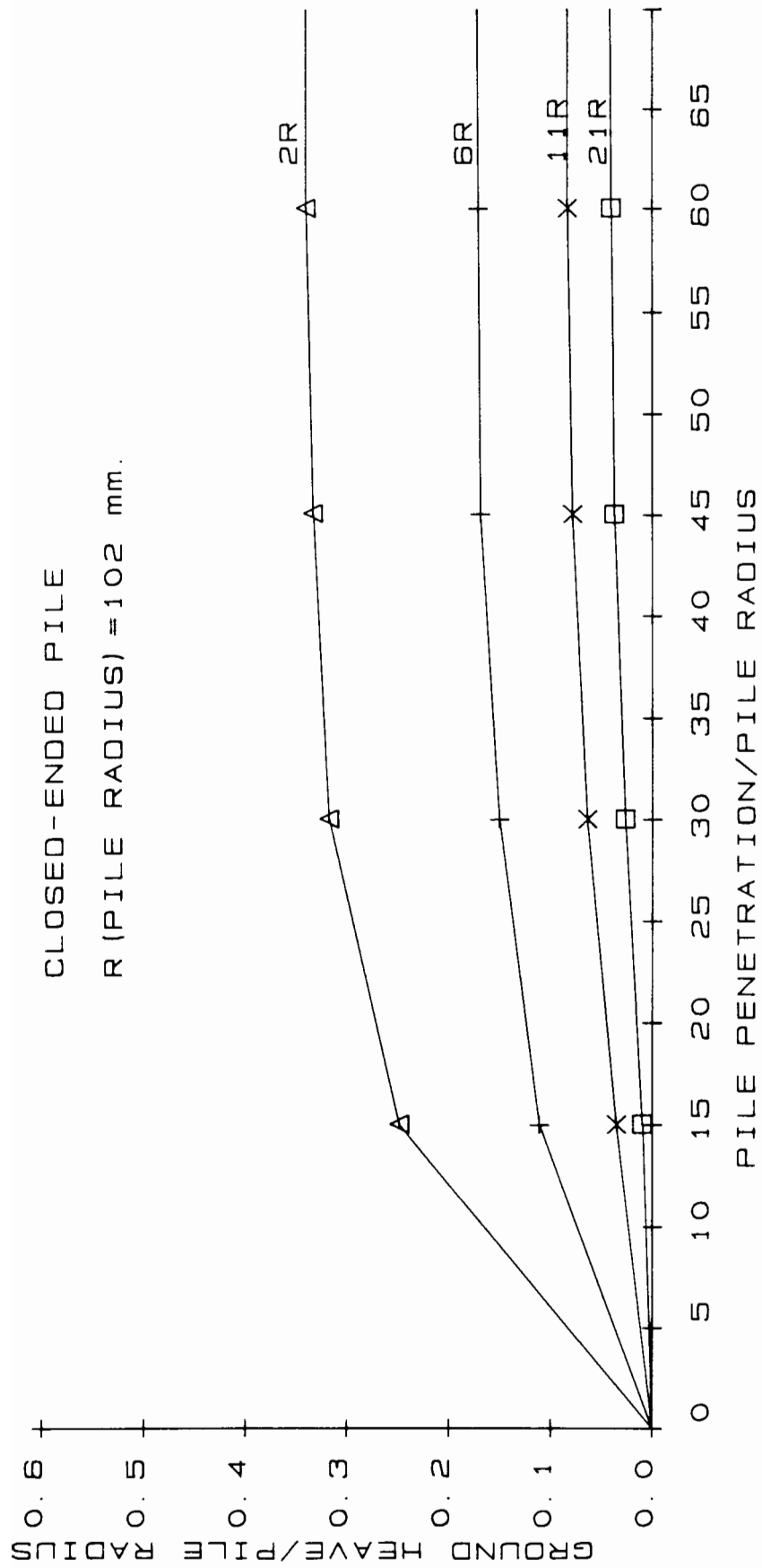


Figure 2.4 Variation of ground heave with pile penetration (data from Sharp, 1982)

problems. He concluded that pile heave may constitute a serious foundation problem for end-bearing piles making redriving of all heaved piles necessary. In the case of friction piles, he found that pile heave had no adverse effects on the capacity of the pile and rendering redriving unnecessary.

Olko (1963) reported a maximum pile heave of 11 ins. (275 mm) which was observed for steel H - piles driven into medium to stiff blue clay at 3 1/3 feet (1 m.) centres. It was observed that the pile heave continued for more than 10 days after the end of driving. However, Koulsoftas (1982) found that the pile heave measurements remained constant during the nine days after completion of driving H-piles into a layer of fine dense and very dense sand overlying a thick layer of silty clay.

From various case histories, Cole (1972) concluded that pile heave appeared to be more a function of pile diameter and pile spacing than soil type or pile length. The final heave of a pile is the cumulative value of all the individual pile heaves caused by driving adjacent piles. An approximate relationship between pile heave and pile spacing was derived using the scattered results from case histories. The pile heave appeared to be greatest for the smallest pile spacing and reduced as pile spacing increased. At a pile spacing of about 20 pile radius heave was found to be negligible. The

results of Clark (1981) indicated that a pile driven at a spacing of 18 pile radii did not affect the previously driven piles. This finding agrees fairly well with Cole (1972). Clark's observation indicated that pile heave occurred mainly during the driving of an adjacent pile to a depth of about 16 to 24 pile radii. Below this depth the pile heave is small. This observation agrees well with Cooke, Price and Tarr (1979) and Sharp (1980).

A simple and theoretically approximate procedure for estimating the heave of an existing pile was proposed by Hagerty and Peck (1971). This procedure was based on the concept that inextensible vertical piles embedded in the clay would be lifted by the relative ground heave along the upper part of the pile, but that below a certain depth, a downward force would act. The depth, d , (Figure 2.5) is estimated by balancing the potential upward and downward adhesive forces on the upper and lower parts of the pile respectively. The pile heave is considered to be related to the heave of the soil, assuming that no heave takes place below the section a-a; that is:

$$\text{Pile heave} = (L - d)(\text{Soil heave})/L$$

The soil heave is to be estimated from Figure 2.2.

Massarsch (1976) attributed pile heave in clay to two different mechanisms. In sensitive clay, the skin friction will be low immediately after the driving. When the buoyant force acting on the pile by the remoulded clay exceeds the

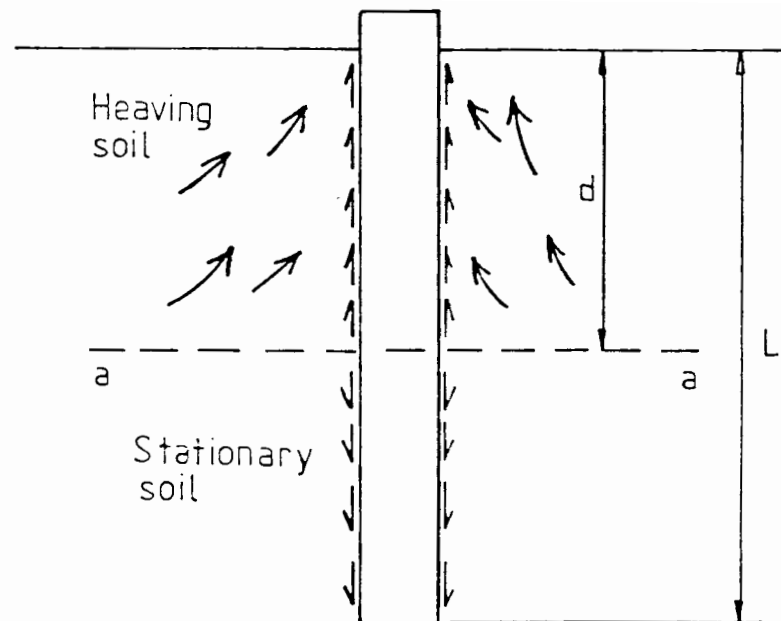


Figure 2.5 Balance of forces along driven pile (Hagerty and Peck, 1971)

weight of the pile, the pile will float and can be squeezed out of the ground during the driving of adjacent piles. Heave caused by the driving of a group of piles has a cumulative effect. This finding concurs with Cole (1972).

(c) Lateral Movement of soil

Lateral movements of the soil and piles occur during pile driving. These movements have been observed by Hagerty and

Peck (1971), and Adams and Hanna (1971) but no particular emphasis was given to this effect. Adams and Hanna (1971) observed very small lateral surface movements which took place beyond a distance of 5 feet from the edge of a group of steel H - piles.

The lateral movement of soil which occurred when an instrumented pile was jacked into London clay was reported by Cooke and Price (1973). Horizontal probes installed at 2 metres below the ground surface were used to measure the movement. Extrapolation from their results suggests that some horizontal movement occurred to a distance of at least 8 pile radii from the pile shaft.

A comparison of measured and theoretical radial soil displacements during pile driving (Figure 2.6) was performed by Randolph, Carter and Wroth (1979). They used the measurements of the radial movement of soil around the model piles of Randolph, Steenfelt and Wroth (1979), together with some field measurements of Cooke and Price (1973). Both sets of data agree well with the theoretical prediction made on the assumption of plane strain and purely radial deformation at constant volume, as stated by Randolph, Steenfelt and Wroth (1979). The radial displacement, v , of a soil particle at an initial radial distance, R , away from the axis of a closed-ended pile before installation is given by :

$$v/R_0 = \left[(R/R_0)^2 + 1 \right]^{1/2} - R/R_0$$

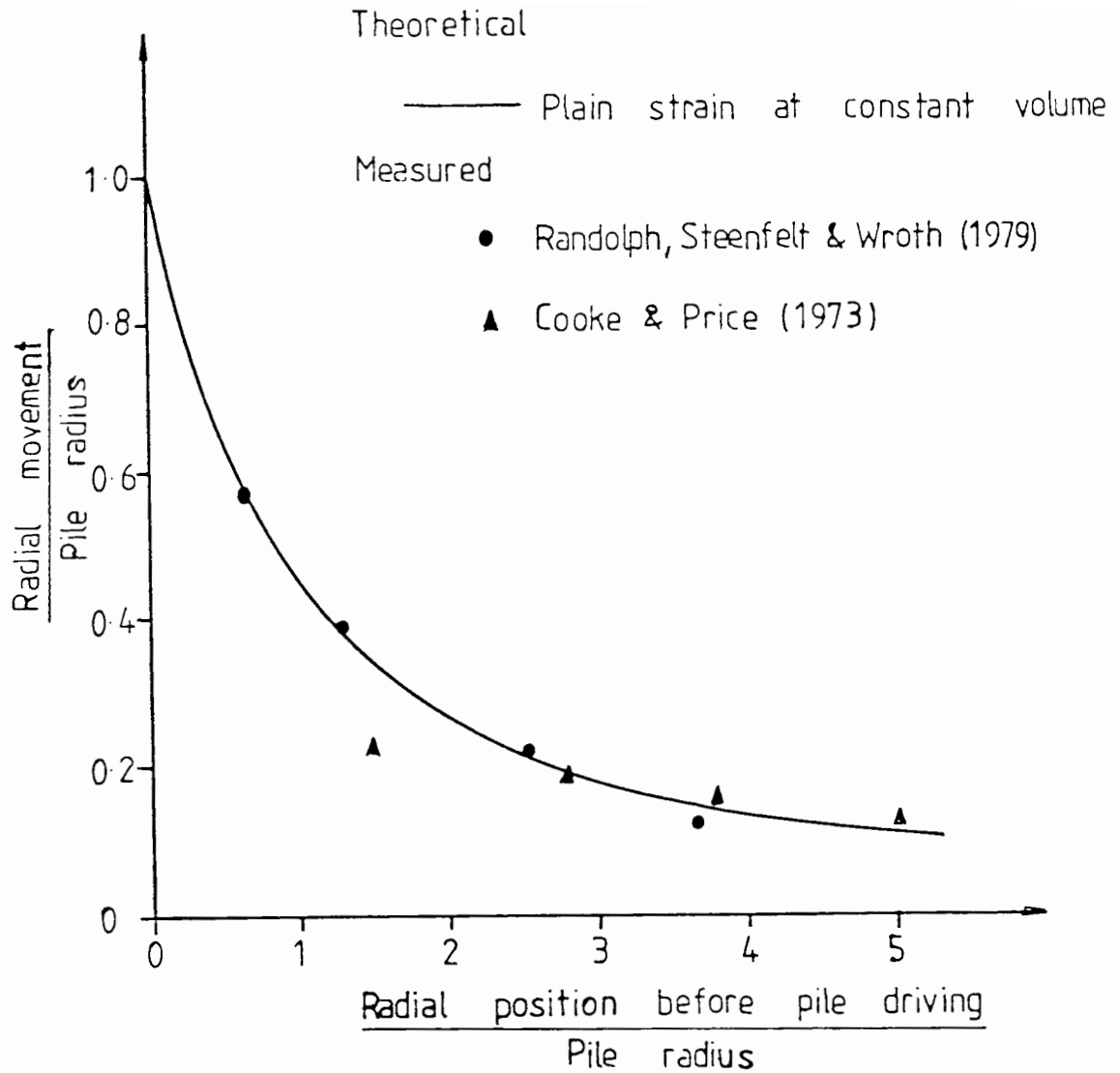


Figure 2.6 Comparison of measured and theoretical radial soil displacements (Randolph, Carter and Wroth, 1979)

where R_o is the outer pile radius. The area ratio, ρ , is defined by Carter, Randolph and Wroth (1980) as $1 - (R_i/R_o)^2$, where R_i is the inner pile radius. The equation above is then altered for open-ended piles with various area ratio as

$$v/R_o = \left[(R/R_o)^2 + \rho \right]^{1/2} - R/R_o$$

The model test piles were closed-ended and therefore the theoretical curve is plotted for $p=1$.

Steenfelt, Randolph and Wroth (1981) attempted some sophisticated laboratory studies using an X-ray technique to measure soil displacements due to pile installation. Samples of clay with different overconsolidation ratios were prepared from speswhite kaolin. They varied from normally consolidated to an overconsolidation ratio of eight. The radial movement due to pile installation was found to extend to about 12 pile radii away from the axis of the instrumented model pile.

2.2.2 Influence on Soil Properties

(a) Soil Shear Strength and Pile Capacity

The driving of piles into clay usually produces an initial reduction in the undrained shear strength of the clay around the piles. This phenomenon has been reported by several investigators. Observation by Orrje and Broms (1967) of concrete piles in a sensitive clay showed that the undrained shear strength was significantly reduced up to 5 pile radii from pile axis. The largest decrease was observed close to the pile surface where the reduction was about 40%. A small reduction of the shear strength was also detected at some distance from the pile but this was of the same magnitude as the accuracy of the measurements. An increase in shear strength was measured 9 months after pile driving but the

clay did not regain its initial shear strength. Cummings et al. (1950) also reported similar results.

Bjerrum and Flodin (1960) observed the bearing capacities of piles which increased rapidly with time, reaching their maximum in less than a month. Tomlinson (1969) reviewed five case histories and concluded that at least 75% of the ultimate bearing capacity was achieved within 30 days of driving. This may explain the common practice of loading test piles a month after driving to take advantage of this time effect.

Figure 2.7 shows the relationship between the pile bearing capacity and the log of time for the cases for which numerical evaluations of the time effect could be made (Soderberg, 1963). The observation of the two previous references (Bjerrum and Flodin, 1960 and Tomlinson, 1969) fit well into this plot.

In contrast, theoretical studies by Wroth, Carter and Randolph (1979) have shown that in any clay, the stress state in the soil close to a driven pile both in the immediate and in the long term, is essentially independent of overconsolidation ratio (OCR). When consolidation is complete, the new strength $C_u(\infty)$, is nearly 60% greater than the original in-situ value, $C_u(0)$ (see Figure 2.8).

(b) Pore pressures developed during pile driving.

The phenomenon of the generation of excess pore water pressures due to pile driving has long been recognised. It

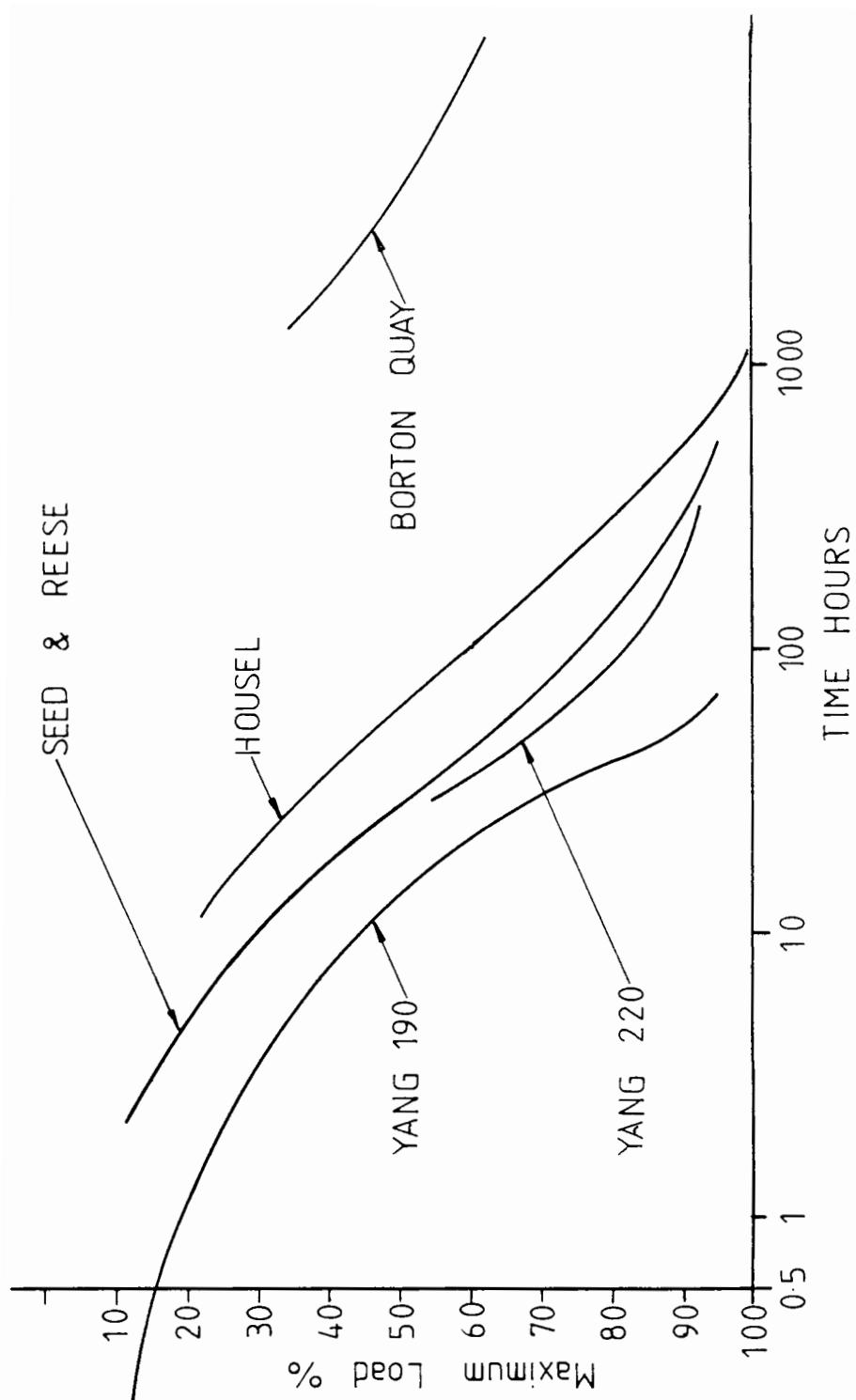


Figure 2.7 Increase of pile load capacity with time (after Soderberg, 1962)

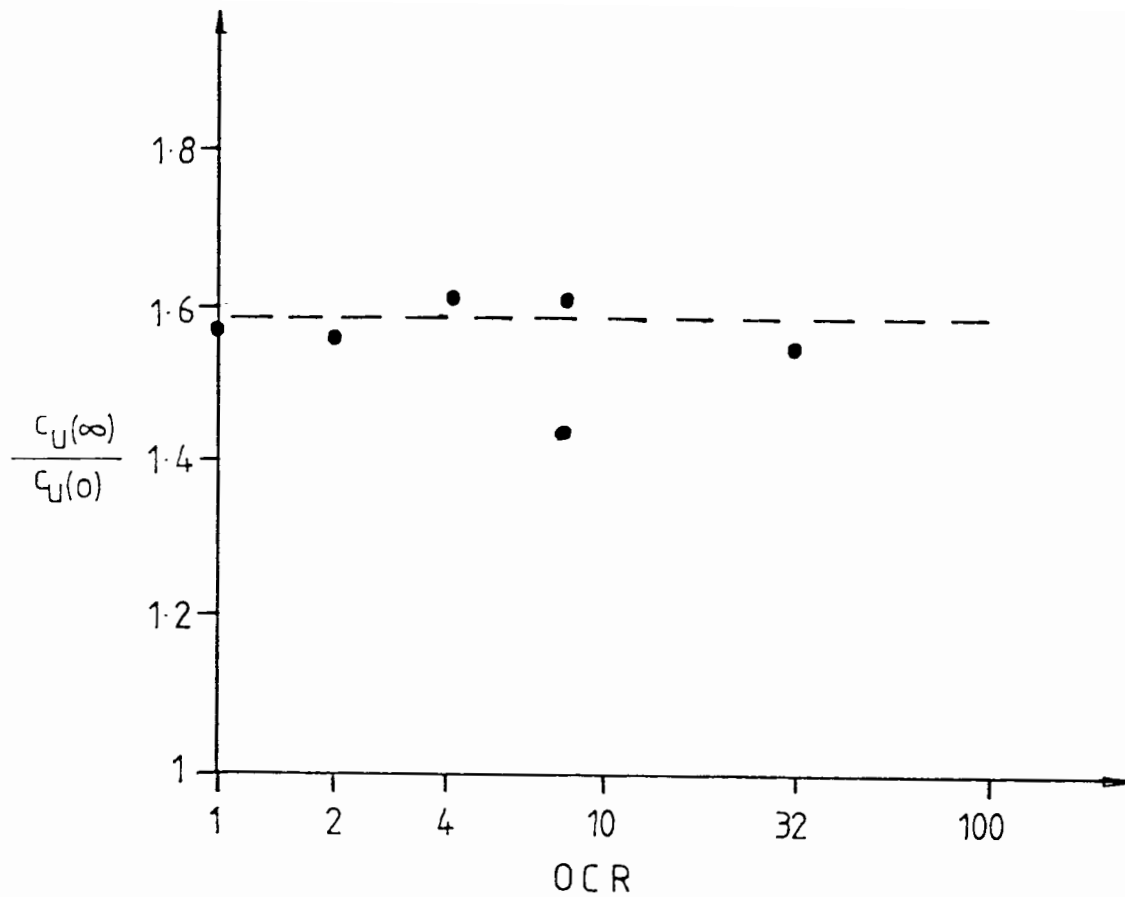


Figure 2.8 Variation of ratio of final to initial undrained shear strength with OCR (after Wroth, Carter and Randolph, 1979)

is known that the dissipation of these pore pressures is a governing factor in the increase of the bearing capacity of piles. Poulos and Davis (1980) have presented a summary of measured excess pore pressure developed due to pile driving from 11 case histories (see Figure 2.9).

In the vicinity of the pile, very high excess pore pressures

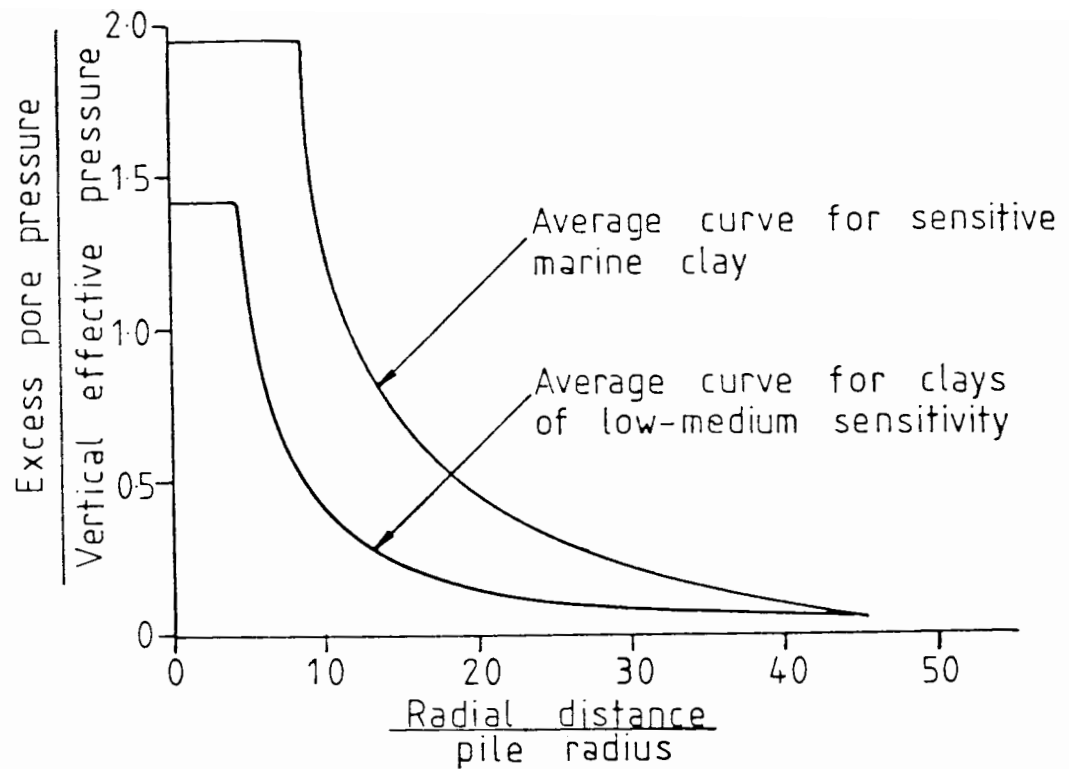


Figure 2.9 Summary of measured pore pressures (after Poulos and Davis, 1980)

develop. In some cases, pressures of 1.5 to 2.0 times the in-situ effective vertical stress have been observed. Figure 2.9 also indicates that for low to medium sensitive (normal) clays beyond a radial distance of 4 pile radii or about 8 pile radii for sensitive clays, there is a rapid decrease in pore pressure and that beyond 30 pile radii, the excess pore pressure is small. For sensitive clay, the excess pore pressure at large radii is smaller than 12% of

its maximum and for low medium sensitive clay it is smaller than 6%.

Wroth, Carter and Randolph (1979) presented a prediction of pore pressure change using a finite element computation. In their studies, the installation of a driven pile was modelled as a cavity expansion under undrained conditions. The modelling details are discussed in Randolph, Steenfelt and Wroth (1979). Samples of London clay were simulated, each with a different one-dimensional consolidation history (different initial OCR). Figure 2.10 shows the values of excess pore pressure generated in the samples close to the pile as a function of OCR. The excess pore pressure is dependent mainly on the initial value of the undrained soil strength and is insensitive to the value of OCR. For an OCR of 8, the excess pore pressure is greatest at the pile soil interface and it decreases quickly to zero at a radial distance of about 10 pile radii.

Francescon (1983) obtained experimental results for an 18.9 mm diameter closed-ended pile jacked into Speswhite kaolin. In contrast to Randolph, Steenfelt and Wroth (1979), he found that the excess pore pressure at the pile wall was dependent on OCR with a maximum of about $5C_u$ for OCR of 1 and decreased almost linearly in the semi-log plot to $2.5C_u$. He also showed for clay with higher OCRs, the excess pore pressure extended further from the pile.

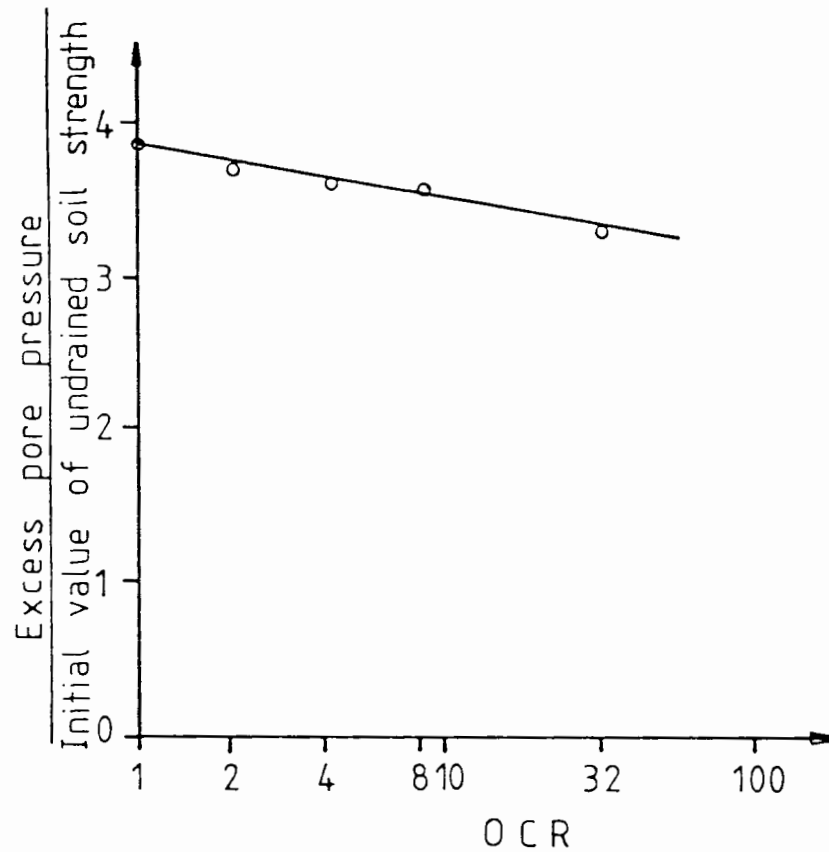


Figure 2.10 Variation with OCR of the excess pore pressure generated next to the pile (Wroth, Carter and Randolph, 1979)

2.3 Conclusions

Ground heave due to pile driving in clay is inevitable. The ground heave extends radially to about 20 pile radii away from the axis of a pile. This conclusion is substantiated by a review of excess pore pressures developed during pile driving. Although disagreement exists in the literature over the extent and magnitude of these excess pore

pressures, it is accepted that no significant excess pore pressures develop beyond a radial distance of about 30 pile radii. Evidence exists to show the dissipation of excess pore pressure generated by pile driving is associated with an increase in the undrained shear strength.

It appears that there is a critical depth of pile penetration at which most of the ground heave will have been generated.

Most of the existing data are based on case histories. Observations of ground and pile heave have largely been made during the driving of groups of piles. There is a need for more good quality model tests to determine the effect of ground heave due to pile driving in clay before any theoretical method can be used to simulate this effect.

No data appear to exist on the effect of ground heave when the ground surface is suppressed by an imposed load. Investigations of this effect would be particularly useful for offshore structures where piles are driven through a mudmat which temporarily carries the structural loads.

CHAPTER 3

DESIGN OF THE APPARATUS

3.1 Philosophy of Soil Modelling

The advantages of using small-scale physical models for studying soil mechanics problems have been discussed in Bassett (1979). They include :

- o models are relatively cheap, easily modified and quick to carry out.
- o soil properties can be chosen with known boundaries.
- o simple controlled variations can be introduced one by one when required.
- o internal movements of soil or rupture lines can be observed.
- o homogeneity of the soil chosen provides a much easier analysis, and
- o tests can be repeated to provide data on the statistical scatter.

Small scale laboratory models have only limited direct

application to prediction for prototype projects. This limitation stems from the complexities of soil such as self weight, stress level, stress path and variable dilation capacity. All these influence the constitutive relationships for soil. The incomplete similarity of a model and the prototype project requires theoretical corrections and computations before the results of model tests can be applied to prototype predictions.

3.2 Choice of Dimensions of Cylinders and Model Piles

All the apparatus described in this chapter are specifically designed for this investigation except where specified standard apparatus, some of which have been modified. Most of the designed apparatus is fabricated outside the department.

The dimensions of the cylinders and model piles were chosen from a compromise between the literature review and modelling philosophy. The radius of the cylinder should be ideally more than 30 times the radius of a model pile; the final height of the test specimen should be about 50 times the pile radius which is approximately equal to twice the depth at which most of the ground heave has been generated (see literature review).

As the clay slurry has to be in the cylinders for several weeks during consolidation and swelling, a non ferrous alloy, duralumin (H15-TB) is used for the cylinders,

closed-ended piles and base plates. The choice of duralumin was made because it has the mechanical strength of mild steel and good corrosion resistance. Since the unit weight of this alloy is about a third that of mild steel, it is easier to handle.

3.3 Reaction Frame

Three sets of specimen preparation assemblies (Figure 3.1) are used. The schematic diagram of each assembly is shown in Figure 3.2. The reaction frame is made up of single size channels (127 mm * 64 mm@ 14.98 kg./m) of grade 43 C mild steel which are welded in position. It is designed to take the reaction during consolidation. The two angles by the sides of the reaction frame provide additional stability and also enable the alignment to be adjusted. Adjustment can be made by means of the bolts fitted into the four corners of the angles. The double acting hydraulic ram is purpose-made to provide a single stroke of 450 mm (the maximum stroke needed for consolidation).

A stiffened top platen made from duralumin plate is fitted to the lower end of the piston of each ram. A flexible rubber hose is used as a seal. When compared to the normal solid 'O' ring type of seal, it provides a highly satisfactory seal with a very low friction for the clearance of 1.5 mm to 3 mm between the perimeter of the top platen and the inner wall of the cylinder. This is because the



Figure 3.1 Three sets of specimen preparation assemblies

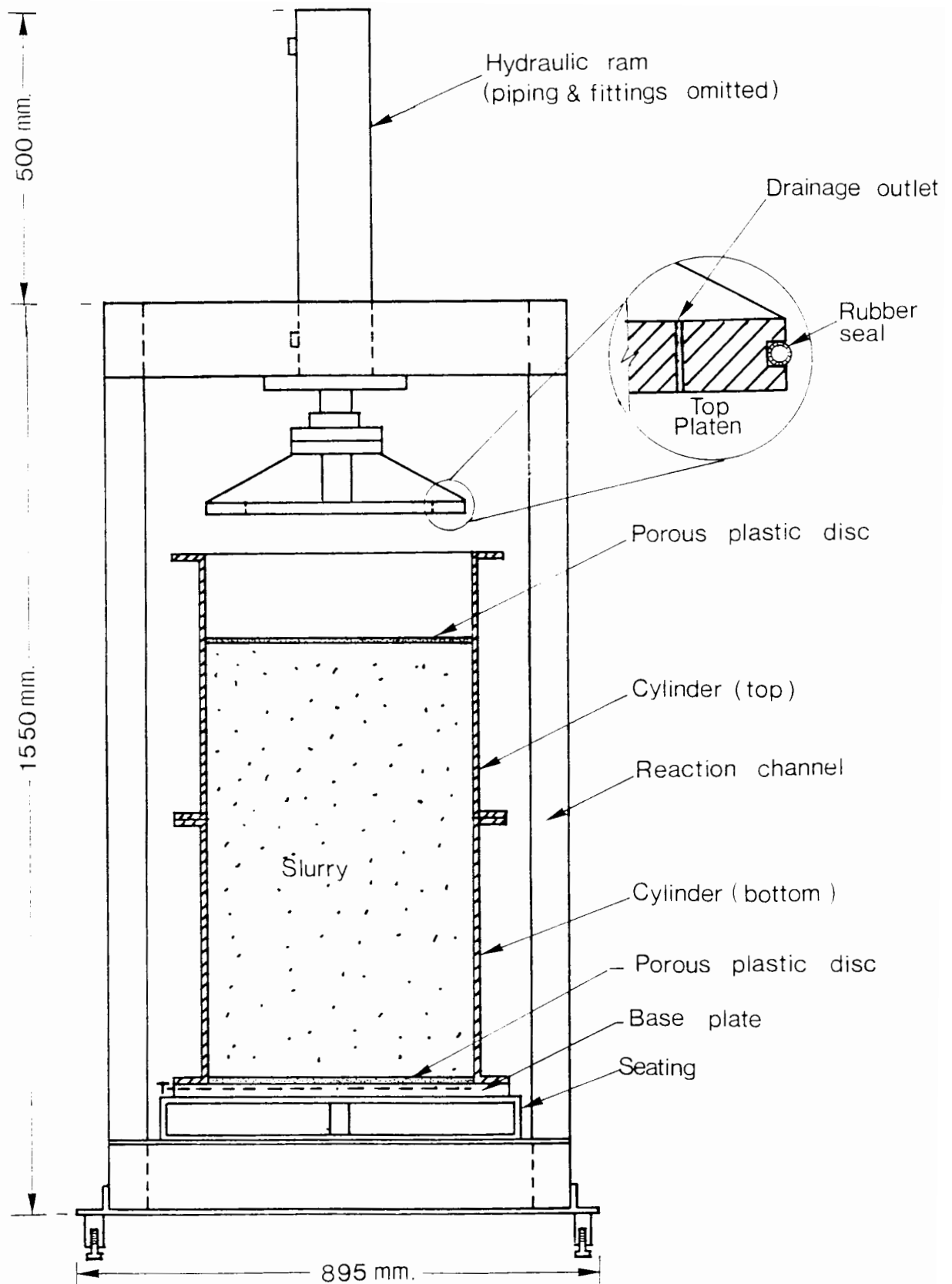


Figure 3.2 Schematic diagram of an assembly

seal is hollow, and can take up the irregularity or tolerance due to the imperfect rolling of the cylinder. The friction can be further reduced by applying a thin coat of silicon grease to the exposed face of the seal. The drainage outlet at the top platen shown is the outlet for the top drainage during consolidation and inlet during swelling of a specimen.

Three assemblies can provide 2 full cylindrical specimens and 2 semi-cylindrical specimens or alternatively 3 full cylindrical specimens in a series.

3.4 Consolidation Cylinders

The top and bottom complete cylinders are identical, and rolled from duralumin sheet. The dimensions of the cylinder (Figure 3.3) are 450 mm in inner diameter, 6 mm in wall thickness and 450 mm in height, with flanges (10 mm thick and 50 mm wide) welded to the top and bottom of the outer perimeter. The eight holes (plan view of Figure 3.3) in the two flanges of a cylinder are provided so that the top and bottom cylinders can be bolted together. The bottom cylinder is connected to the base plate in the similar manner at its bottom flange to form a consolidometer. The base plate has a drainage outlet which serves the same

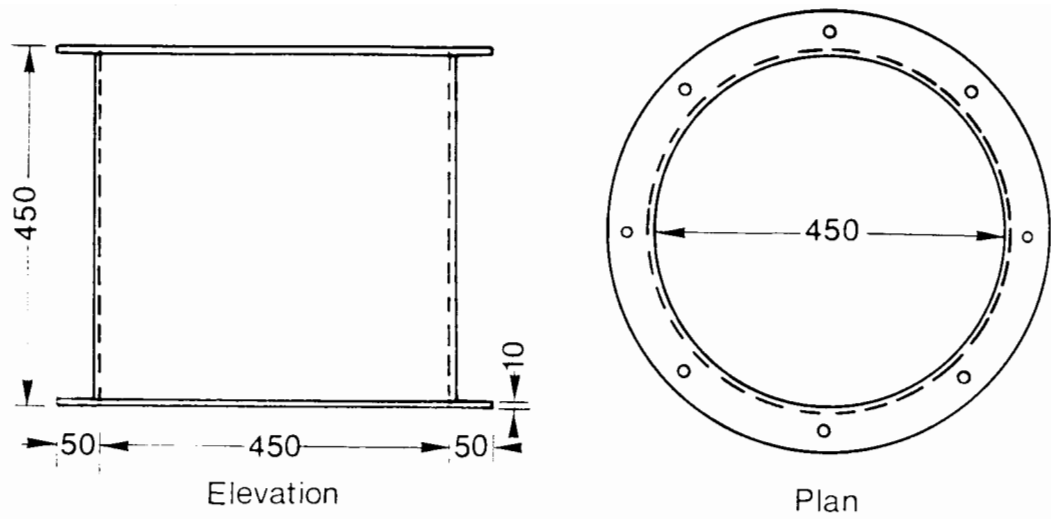


Figure 3.3 Complete cylinder (dimensions in mm.)

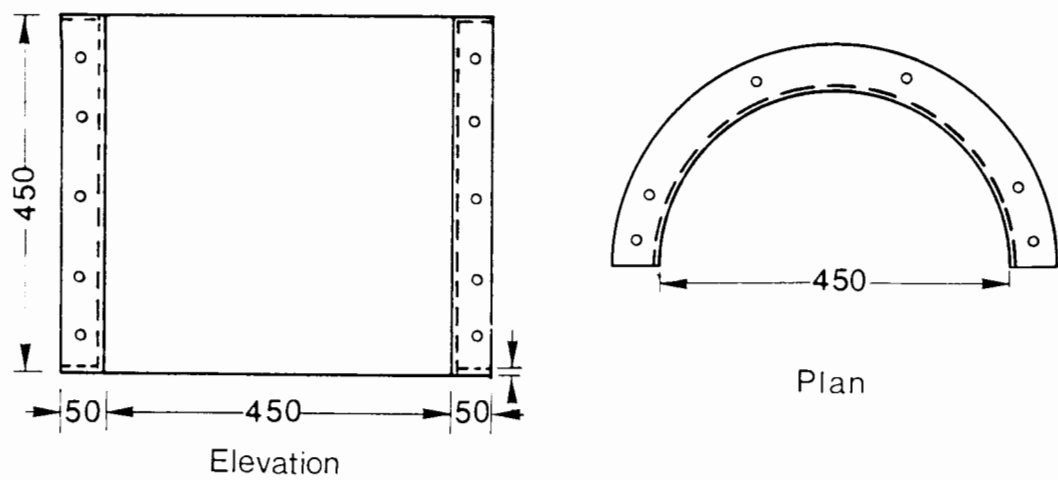


Figure 3.4 Half cylinder (dimensions in mm.)

purpose as the top outlet in the platen. The outlet is connected by a plastic hose leading to the top of the cylinder so that there is no difference in water head between the top and bottom face of the specimen.

The half cylinder (Figure 3.4) has vertical flanges welded on both sides as well as on its top and bottom perimeters (semi-circular). Each half cylinder has a semi-circular base plate with a drainage outlet. The two half cylinders can then be bolted together through their vertical flanges to form a bottom cylinder. A stainless steel plate has to be placed between the two half cylinders so that, at the end of consolidation, two semi-cylindrical specimens can be obtained. The details of this are described in the next chapter.

3.5 Model Piles

Cylindrical model piles (Figure 3.5) of both closed- and open-ended types are used. Both types are 450 mm long with an outer diameter of 16.0 mm with the open-ended piles having a wall thickness of 1.0 mm. All the open-ended piles are driven with a pile driving shoe except those driven in the earlier series and series 8 and 11. The purpose of using a driving shoe is to achieve an unplugged pile during pile driving. This will allow idealised comparison between the results obtained from open- and closed-ended piles.

Dimensions in mm

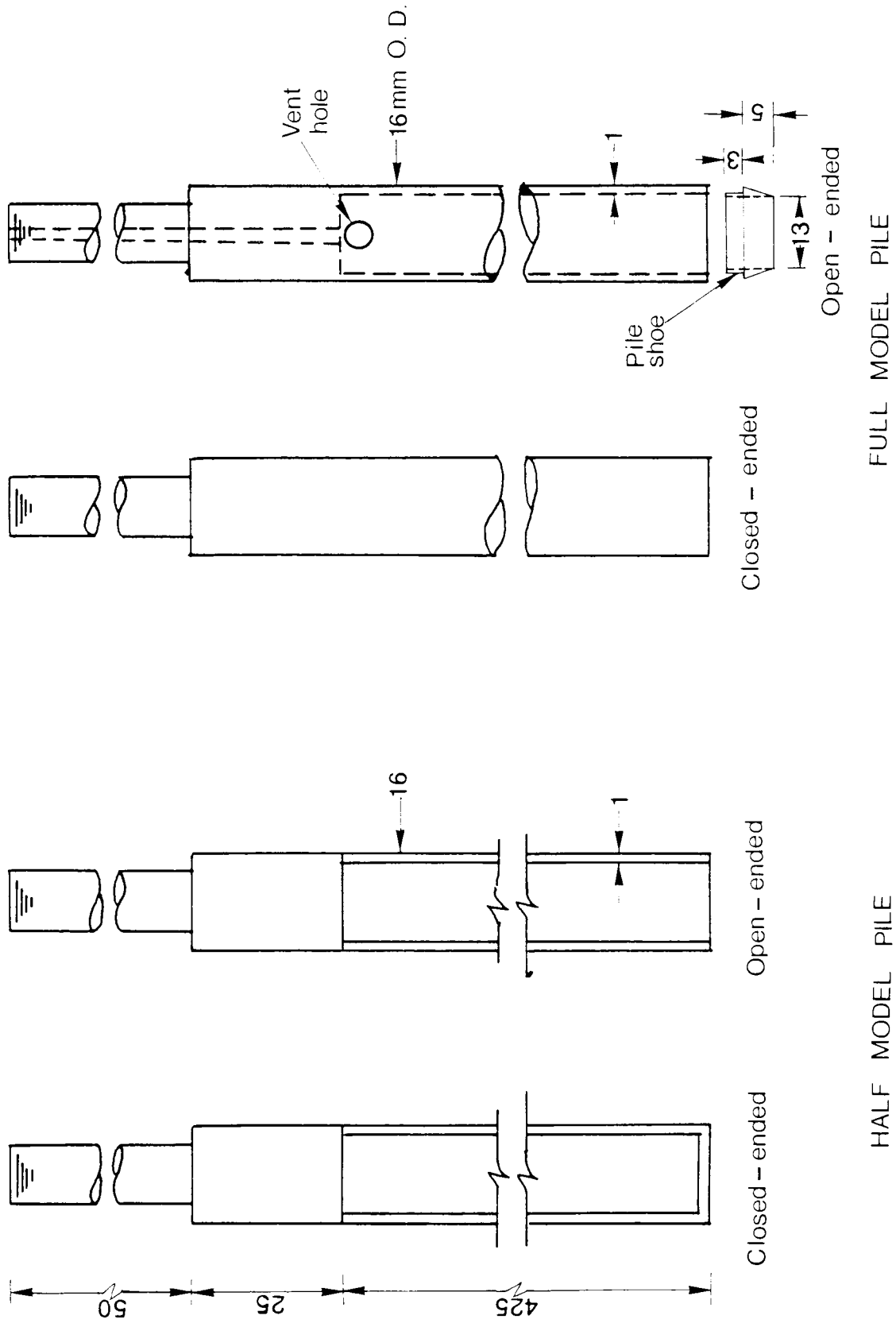


Figure 3.5 Model piles

The closed-ended piles are made from duralumin bar milled to an outer diameter of 16 mm, but the open-ended piles are made from stainless steel pipe with an inner diameter of 14 mm. The outer diameter is milled 16 mm. Two vent holes are provided to prevent the air inside the pile being compressed during driving which may affect the plugging of the soil. The longitudinal hole inside the threaded portion allows indirect measurement of the length of plugged soil to be made. The threaded portion (50 mm) of each model pile is used to connect to the hammer guide (Figure 3.6). The driving shoe is made from stainless steel bar to the dimensions shown, and is fitted tightly into the tip of the open-ended pile before driving. The area ratios with respect to open-ended pile and driving shoe are 23.4% and 34.0% respectively.

The hollow semi-cylindrical model piles are milled to remove half of the section of a stainless steel pipe to achieve a true semi-circle in sectional view. The threaded portion of stainless steel bar is fused to the half cylindrical pile by TIG (Tungsten Inert Gas) welding. The closed-ended pile has a 2 mm thick semi-circular end plate welded on to its tip.

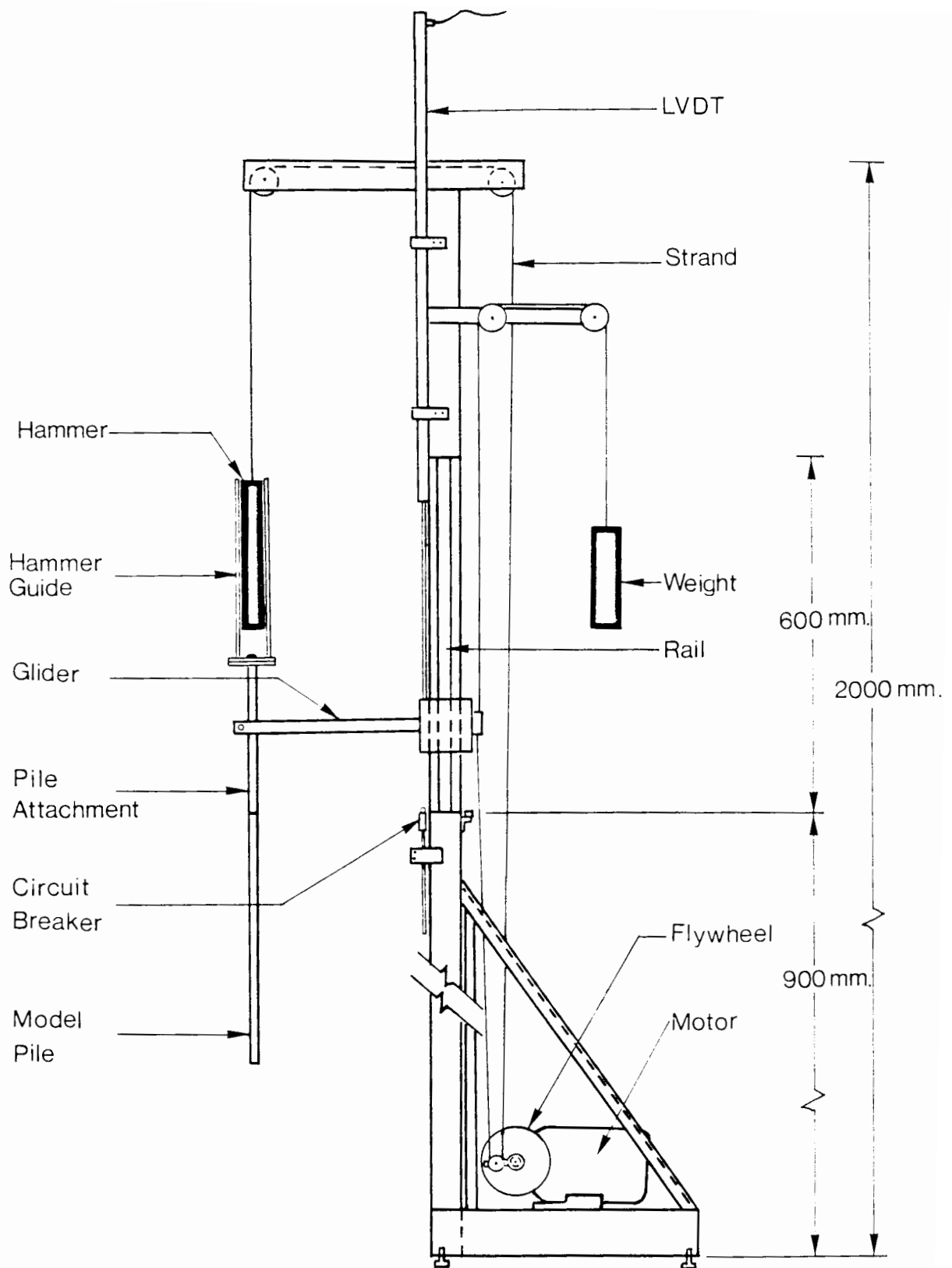


Figure 3.6 Pile driving rig

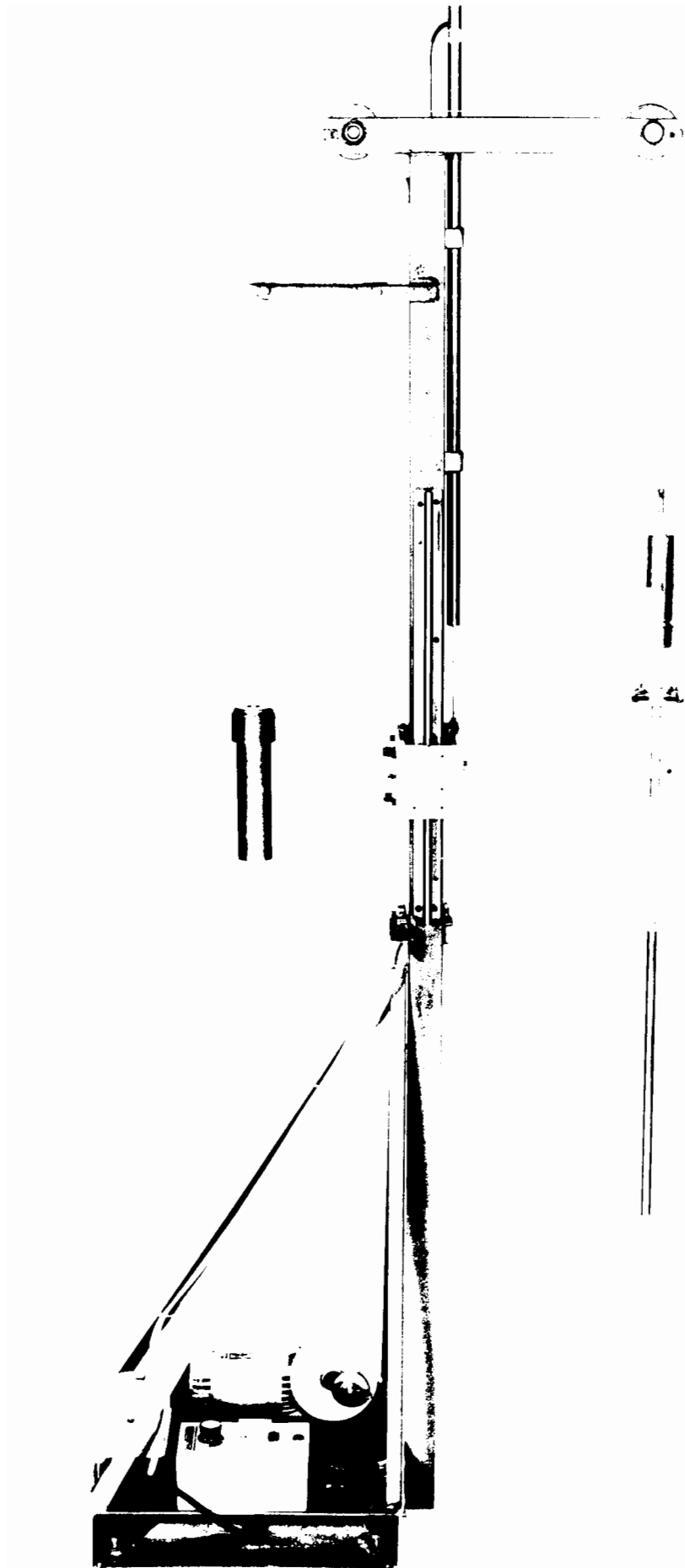


Figure 3.7 Pile driving rig

3.6 Pile Driving Rig

3.6.1 Introduction

There have been a substantial number of model tests on piles, but most piles used in the tests were jacked into the model ground. The main criteria for the design of the pile driving rig are:-

- o to drive a model pile dynamically and vertically into a soil specimen.
- o to provide a constant dynamic energy with depth of pile penetration.
- o to be able to vary the rate of driving (blow/min.) or have the flexibility of using different hammer weights with a fixed rate of driving.

The pile driving rig (Figures 3.6 and 3.7) is designed to drive a model pile vertically into a soil specimen. It provides a constant rate of blows with a fixed drop height of a hammer throughout the pile penetration. Moreover, the rate of blows and drop height can be adjusted for different hammer weights. The rate can be varied from virtually zero to 200 blows per minute by adjusting the thyristor control of the variable speed motor (187 watts). This pile driving

rig has proved to be entirely satisfactory.

3.6.2 Description

The pile driving rig is made mostly from mild steel. It consists of a hollow base box upon which the variable speed motor and its control equipment are attached. A hollow vertical tube of 50mm square cross-section has been welded to the middle of one side of the box and strengthened by the bracing action of two equal angles branching from the tube to the two corners of the base. Two 600 mm long rails have been fixed parallel to two opposite sides of the upper part of the square tube above a height of 900 mm. There are two linear bearings fixed onto each rail. These provide rigidity to the glider so that it is fixed in the horizontal direction and is only able to travel vertically up and down along the rails. The four bolts at the bottom of the base are provided for levelling the rig.

The glider is made from a rectangular duralumin bar, 25mm by 40mm in section. One end of it has a hole for the pile attachment to go through, and is clamped to the glider by a screw at the end of the bar. The other end of the glider houses the four linear bearings that travel vertically along the rails. The counter-balance weight has been incorporated to prevent the glider from falling under its own weight.

The hammer guide is made from a 50 mm inner diameter hollow perspex tube. A circular flange made from duralumin plate

is glued onto the perimeter of the lower end of the tube, and bolted to the upper portion of the pile attachment. The hammer guide has been graduated on the outer face for direct reading of the drop height. The pile attachment is essentially to hold the model pile and to guide the hammer during pile driving. The semi-cylindrical bearing on the pile attachment is to ensure that the fall of the hammer coincides with the axis of the model pile. The lower end of the attachment is used to connect a model pile.

The hammer is made from a solid steel rod which can be varied in weight and drop height to suit a soil specimen. It is connected by a thin strand, which passes over two pulleys (40 mm in diameter) at the top of the rig and over a small pulley located eccentrically at the flywheel of the motor. It is then locked to the lower end of the glider by a fastening screw. This arrangement ensures that the drop height remains constant with pile penetration. The strand is used because of its flexibility and it has a negligible extension due to the weight of the hammer used.

The pulley at the flywheel can be adjusted and moved radially to vary the drop height of the hammer. The drop height is four times the radial distance from the centre of the pulley to the centre of the flywheel. A one-way clutch bearing is sandwiched between the flywheel and the shaft of the motor. This arrangement allows the hammer to fall under its own weight at every rotation of the flywheel. The quarter horse power motor(shunt wound type) with thyristor

control can provide a range of rate of blows.

A long stroke linear variable differential transformer (LVDT) is clamped to the square tube with its armature resting on the glider to measure the depth of pile penetration. The output of the LVDT can either be read through a multimeter and converted to linear depth or can be transmitted to an analog scanner by a length of cable. The lower end of the armature is held down to the glider by plastic adhesive to prevent the armature of the LVDT bouncing up and down during pile driving. At the lower end of the guide rails, a circuit breaker has been installed to stop the motor automatically when the required depth of penetration is reached. It can also be adjusted to a different depth of penetration.

3.6.3 Operation

The pile attachment may be adjusted and is clamped in the position required. A selected weight of hammer can be fixed to one end of the strand by the fastening screw. The pulley at the flywheel is adjusted radially to give the drop height required. The flywheel of the motor is then rotated by hand until the pulley is at its highest position. At this position the hammer should be resting on the pile attachment. The wire can then be tightened near the glider until it is just about to lift the hammer from its resting position, and is then locked by the screw. When the

flywheel is rotated further, and the small pulley is on the downward rotation, the hammer is lifted. The hammer is at its drop height when this pulley is at its lowest level. The one-way clutch bearing is installed so that when the pulley is rotated beyond its lowest level, the hammer falls under gravity, and at this instant the pulley is again at its highest level. This process is repeated by rotation of the variable speed motor. The speed of rotation can be controlled by selecting the appropriate and calibrated setting on the graduated scale (0-9) of the thyristor controller.

A model pile can then be screwed into the lower end of the pile attachment. The rig is then levelled by using the bolts at the bottom of the base box. The glider is lifted up by hand to the start position and the tip of the pile is slowly brought into contact with the surface of the specimen. The circuit breaker can then be set to the required pile penetration, and the system is ready to drive the pile into the specimen.

3.7 Hydraulic System

Figure 3.8 shows the flow diagram of the hydraulic system used for supplying the pressure needed for consolidation. The pressurised gas (oxygen free nitrogen) cylinder (A) supplies the pressure required to the hydraulic rams. The gas can be regulated to the required pressure during

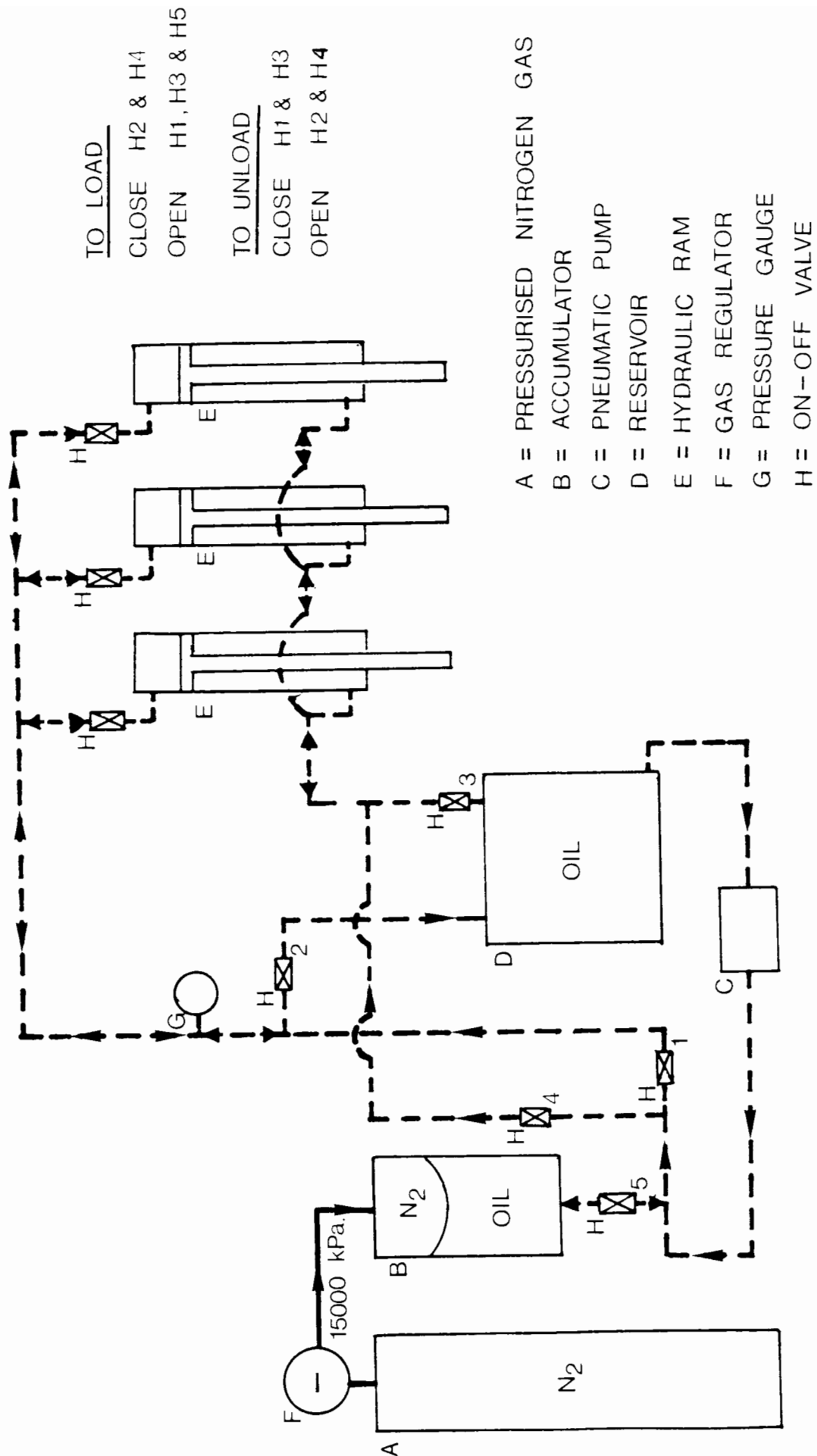
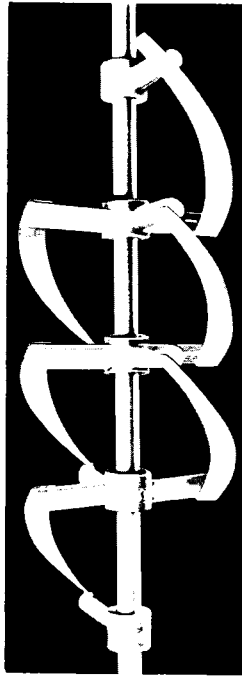
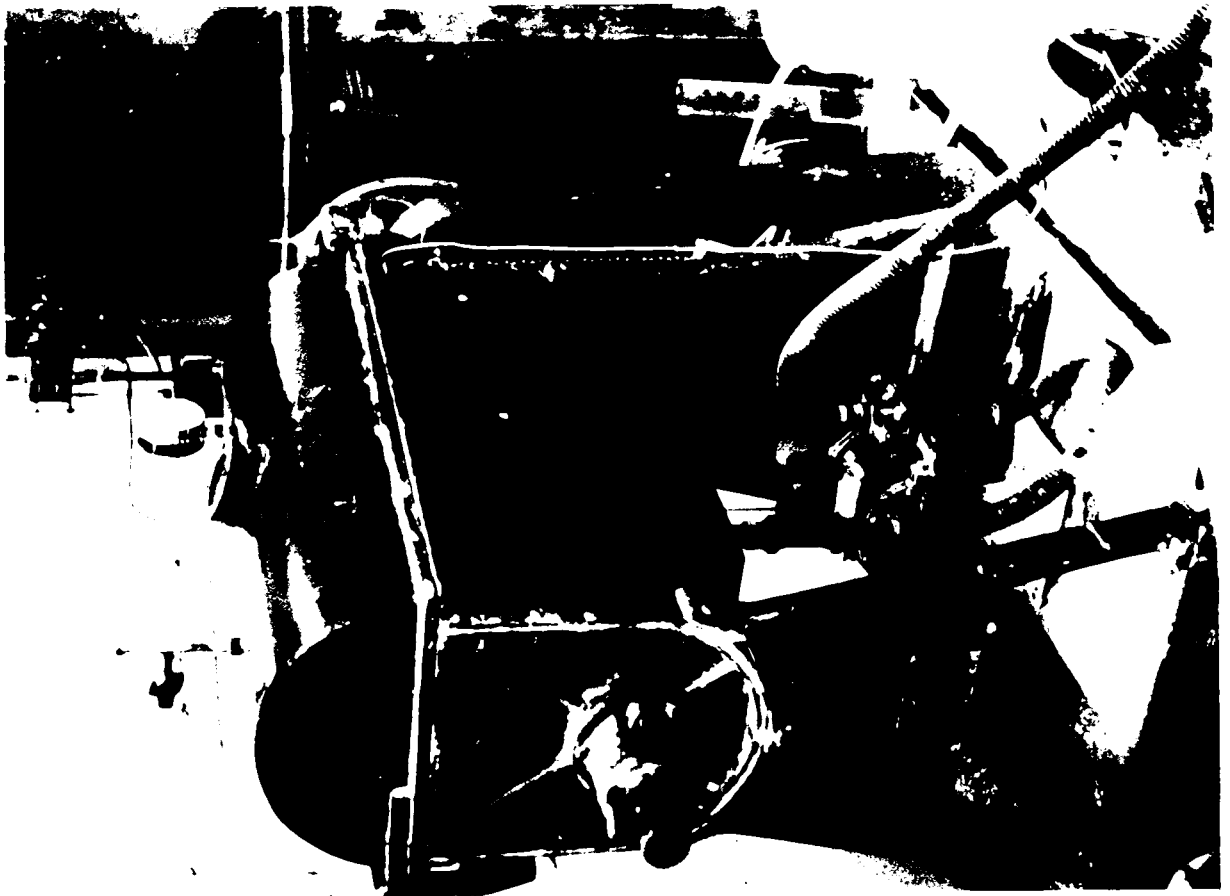


Figure 3.8 Flow diagram of the hydraulic system

consolidation of clay by a regulator (F). The accumulator (B) acts as a gas and hydraulic oil transfer interface and also a safety feature. There are eight on-off valves in the system with one located at each of the three rams. This provides the flexibility of allowing one ram to be raised at a time so one specimen can be raised and tested, while the other two rams are still maintaining their pressure. This is done by simply closing the valve on each of the other two rams. The other five valves are used to control the consolidation and swelling processes. During unloading, the pistons of the three hydraulic rams (E), valves H2 and H4 are closed, and valves H1, H3 and H5 are opened. The pneumatic pump (C) is used for raising the pistons and pumping oil from the reservoir (D) into the circuit, filling up the 75% by volume of the accumulator before the pressure of the nitrogen gas can be regulated for consolidation. When unloading the pistons, valves H1, H3 and H5 are closed and valves H2 and H4 are opened. The pressure gauge (G) indicates the circuit pressure before entering the rams.

The three valves (E) adjacent to the rams are opened throughout the period of consolidation and swelling. In addition, these valves are also used for adjusting the alignment of each reaction frame before the consolidation of specimens begins.



The Ribbon Blades

Figure 3.9 The slurry mixer

3.8 Mixer

The mixer (Figure 3.9) is a standard 110 litres type for operation under coarse vacuum conditions. It is manufactured by Winkworth Machine Limited of Staines, Middlesex. The 'U'-shaped trough is fabricated and rolled stainless steel has flanged bolted end plates. Supporting legs are attached to the bottom of the trough. The lid has a vacuum gauge with its inlet and a viewing window. The standard outlet below the trough has been modified by fixing an on-off valve and is connected to the inlet of the slurry pump by a short length of reinforced hose.

The ribbon blades (see inset) are keyed onto the main blade shaft, and fitted to rotate in the contour of the trough. It is powered by a 2 H.P. Neco 100 rpm. geared motor via 'V' belts to the blade shaft. The final speed of rotation of the blade shaft is 33.3 rpm. The input electrical supply is 415 v./3/50 cycles A.C.

3.9 Instrumentation

The main electronic instrument used for measuring ground heave is standard linear variable differential transformers (LVDT). Five short stroke LVDTs are used to measure the ground heave during swelling of a specimen (after unloading

with the top platen separated from the specimen) and pile driving. An additional long stroke LVDT is used to measure the pile penetration. At the later part of the investigation, two additional short stroke LVDTs are used to monitor the movements (rotation, tilt and settlement) of the mudmat for the group pile tests. Six pore pressure transducers are also incorporated in series 12 to 15 to monitor the change in pore pressures during consolidation, swelling, unloading and pile driving.

3.9.1 Data Logging

Figure 3.10 shows the data logging system and electronic instruments used. Located near the gauging transducers (LVDTs and pore pressure transducers) are two junction boxes, each with its own power supply unit for the transducers. The junction box for the LVDTs contains the signal conditioning modules, one for each LVDT. Each module is basically a self-contained d.c. energised carrier amplifier with a transducer energisation output of 5 volts. r.m.s. at 5 kHz. However, the pore pressure transducers are used without such modules. The d.c. output is proportional to the displacement or pore pressure for each transducer and can be read directly at the junction box using the digital multimeter or be transmitted through the cables to the data logger. During the scans of all measurement channels, the digital readings are temporarily stored in a memory located within the interface. On

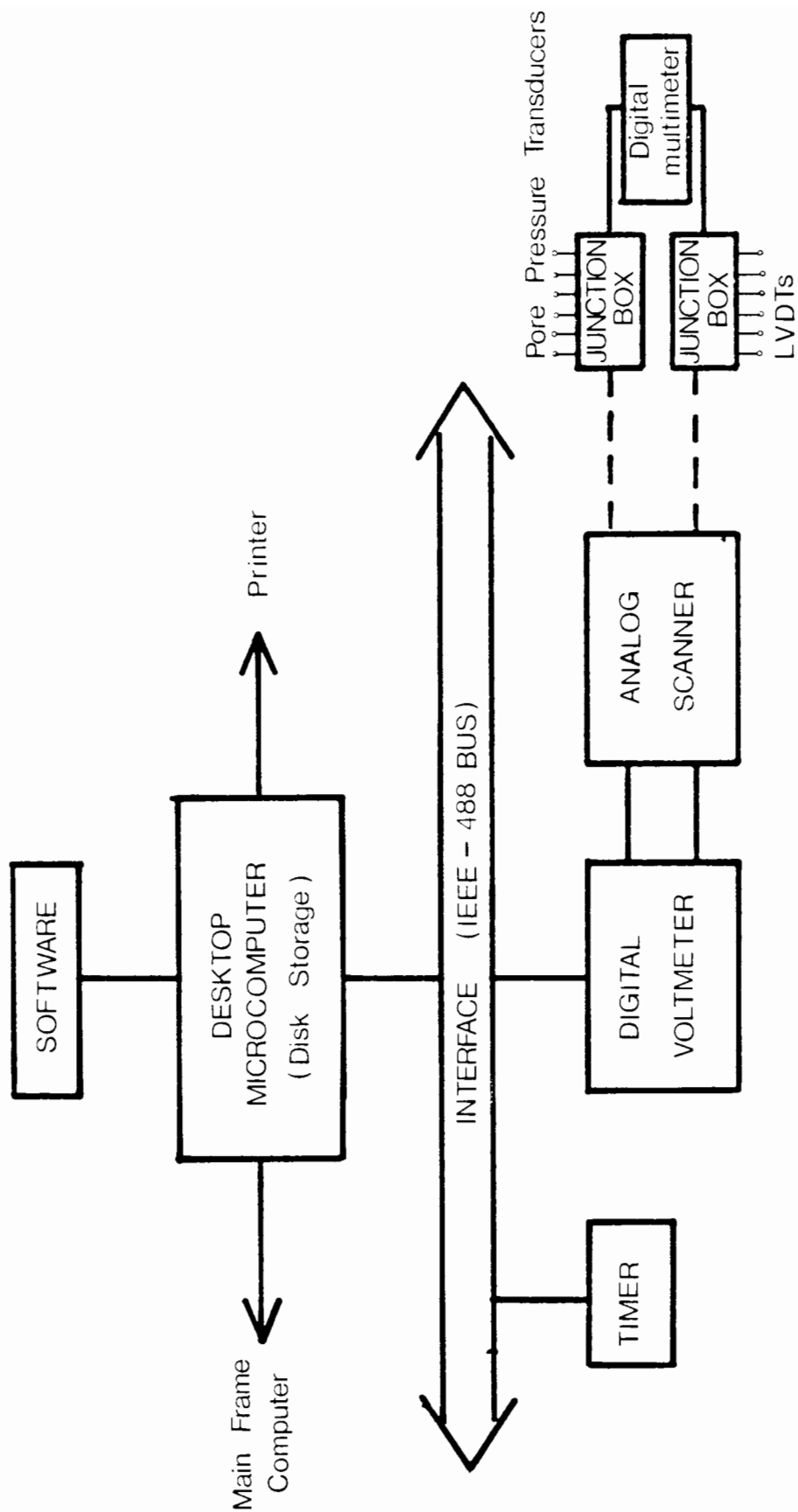


Figure 3.10 Data logging system

completion of the desired scan, the digital outputs are transferred to the disk storage of the desktop microcomputer.

Figure 3.11 shows the flow chart of the program used for the data scanning. When the program is loaded, it first initialises the voltmeter and displays the readings of all the channels. A check of any electrical losses in the readings can be done at this stage by comparing these readings with those obtained near the junction boxes using a multimeter.

Monitoring these readings from the LVDT channels, the pile is positioned in near contact with the soil surface to obtain the start penetration voltage. This together with increment and stop voltages obtained from the LVDT calibration data are then entered. The start voltage entered is always less than the voltage of the LVDT before pile driving so that a scan can be taken just before pile driving and used as a reference. The next and subsequent scans of LVDTs, pore pressure transducers and the elapsed time are logged at intervals of about 10 mm of pile penetration. The scanning process will stop when the pile has penetrated beyond the stop voltage entered. Subsequently, the motor is stopped by the circuit breaker. The digital outputs are then stored on a microcomputer disk.

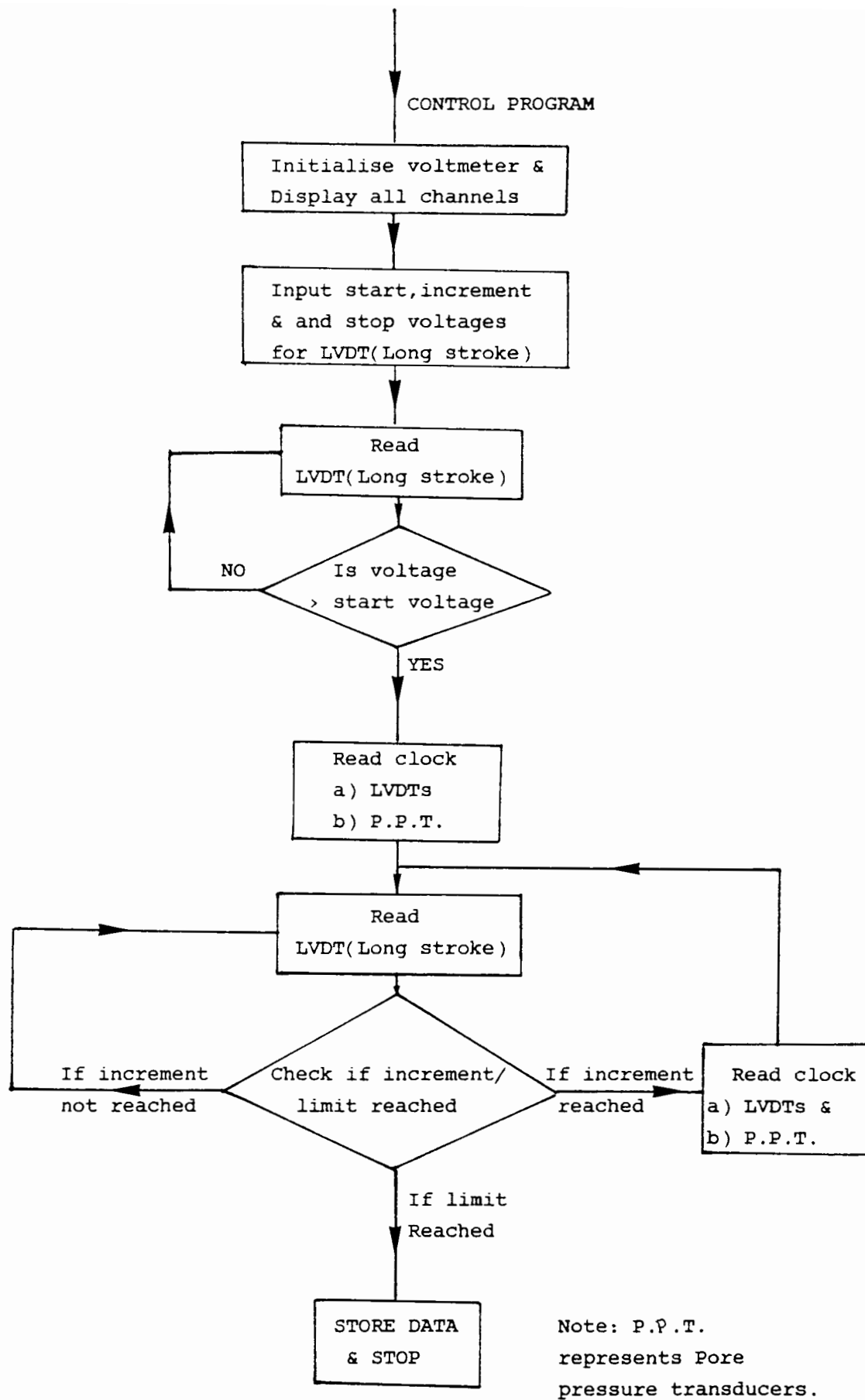


Figure 3.11 Flow chart of the data logging program

The typical scanning time for 6 transducers and a timer is 0.45 seconds for most of the pile driving (80 blows/min.). However, a scanning speed of 0.29 seconds has been used for fast pile driving (150 blows/min.). The scanning speed for 6 LVDTs, 6 pore water transducers and a timer in the later series of tests takes 0.64 seconds.

3.9.2 Displacement Transducers

Each short stroke LVDT (D5/100H; Grade A1) used for measuring ground heave has a stroke of ± 2.5 mm and the long stroke LVDT (D5/8000C; Grade A) has a stroke of ± 200 mm.

The armature of the short stroke LVDT has an enlarged plastic disc (Figure 3.12) aimed to reduce the contact pressure when resting on the clay surface. The assembled armature weighs 1.3 gf., equivalent to a contact pressure of 0.16 kPa. on full contact with the clay surface. The rated accuracies for the short and long stroke LVDTs are 0.1% and 0.2% of the full stroke respectively. Each of these transducers has infinite resolution and the typical sensitivity for the short stroke LVDT is about 7 mV/V/0.1 mm and the long stroke LVDT is about 0.8 mV/V/0.1 mm. Regular calibration before each series of tests with a micrometer gauge has proved that the calibration factors of the LVDTs remained almost unchanged throughout the duration of the investigation. The responses are linear and exhibited very

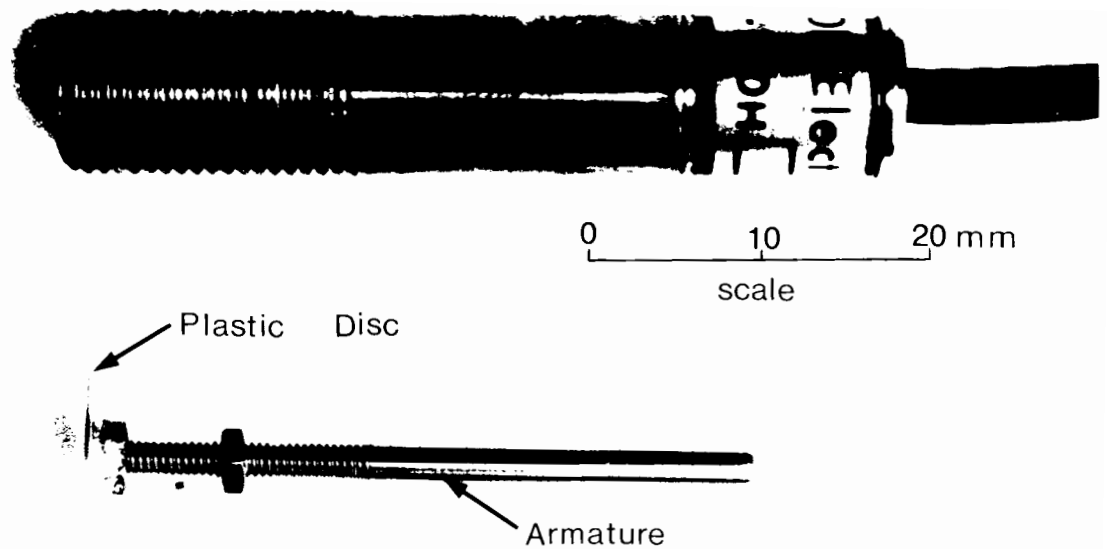


Figure 3.12 Linear variable differential transformer (LVDT)

little scatter. An accuracy of better than 0.001 mm is obtained for the short stroke LVDTs and 0.15 mm for the long stroke LVDTs. The most extreme drift in term of the calibration factor over a period of a year is about 1%.

3.9.3 Pore Pressure Transducers

Six standard miniature pore water transducers (3 Druck PDCR 81 and 3 PDCR 81/S) are later incorporated into each of the specimens in series 12 to 15. Each transducer consists of a single crystal silicon diaphragm with a full strain gauge bridge diffused into the surface. It is energised by a 5

volts d.c. power supply and has a maximum d.c. output voltage of 75 mv. A ceramic porous stone of 5.8 mm diameter is placed in front of this diaphragm to measure the pore pressure. The pressure range of each of the transducers is 700 kPa. with vacuum capability. These transducers have an infinite resolution and a linearity of ± 0.2 BSL. They are calibrated before each series of specimens are prepared.

Three of the transducers (P1, P2 and P3) are fixed to the cylinder wall and the other three (P4, P5 and P6) are inserted into the specimen (Figure 3.13). Each of the transducers located at the cylinder wall has a circular flange (see detail X). When assembled, the exposed face of the porous stone is flush with the inner wall. Detail 'Y' shows the arrangement when a pore pressure transducer (without flange) has been inserted into the specimen. When the cap is tightened, the rubber bung is compressed by the extension collar. This rubber bung acts as a seal and is also flush with the inner wall of the cylinder when the cap is fully tightened.

3.9.4 Deairing the Transducer

It is necessary to bear in mind the warning of Campanella and Robertson (1981) of the effect of entrapped air in the piezometer system. They found that entrapped air seriously affects the maximum pore pressure and dissipation time. Glycerine has been effectively employed by Campanella,

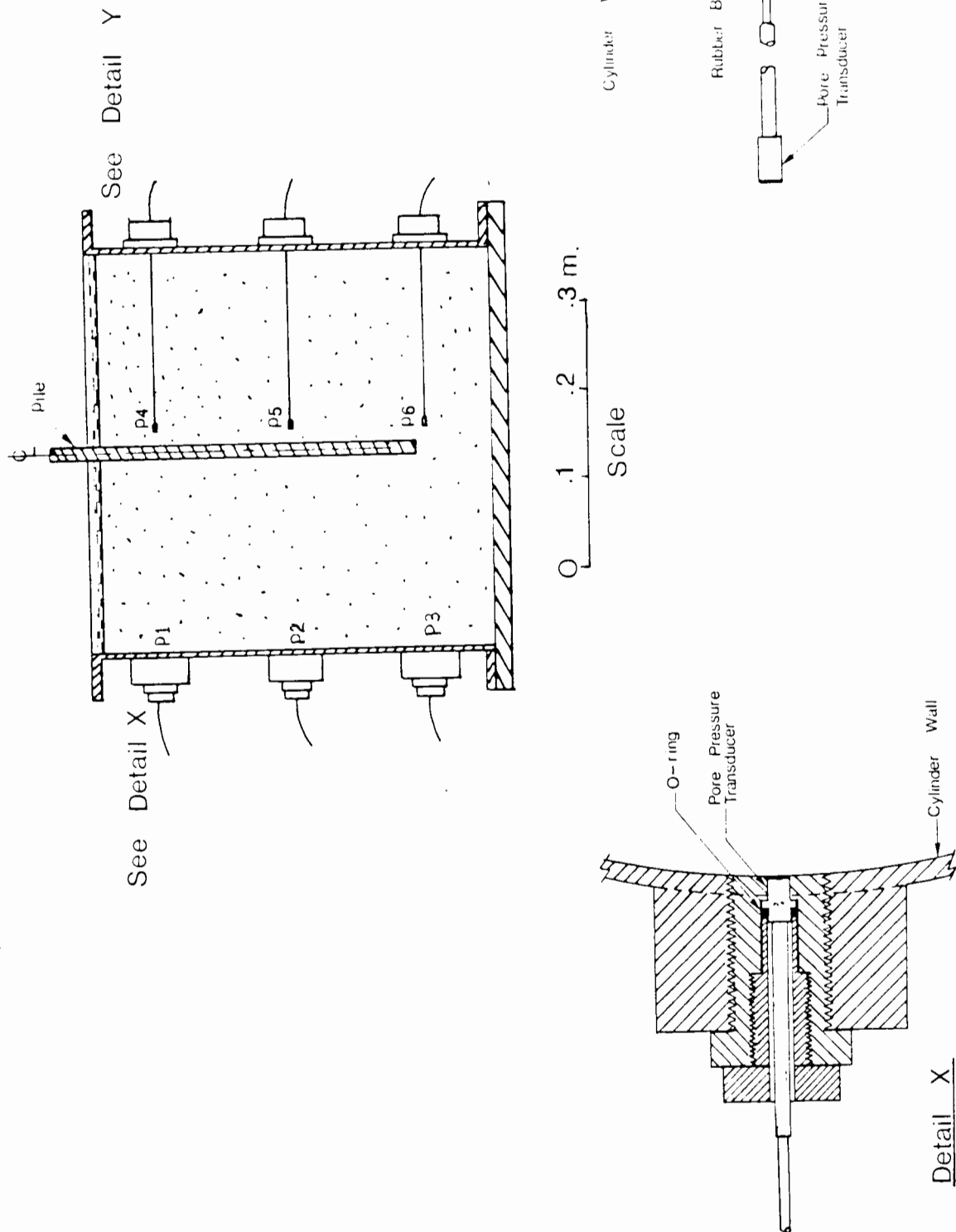


Figure 3.13 Location of pore pressure transducers and their fixing details

Robertson and Gilliespie (1981) in their piezometer system. This is due to the high air entry tension of glycerine that can prevent the loss of saturation when the piezometer is inserted into the clay specimen.

The transducers are found easier to deair in water than in glycerine. Therefore they are first deaired in water by repeated operation in a vacuum system under almost a full coarse vacuum. When they are fully deaired, the vacuum is released. They are then transferred and immersed into a beaker of deaired glycerine for a period of at least a day. This allows the water to diffuse out and be replaced with glycerine; due to the difference in concentration of the two liquids. The saturation of the transducers is confirmed transparent by putting two dummy transducers together with the pore pressure transducers during the process of deairing. Each dummy transducer is made from a short glass tube with one end of it sealed with araldite, while the other end is fitted with an identical ceramic porous stone.

3.9.5 Insertion of a Pore Pressure Transducer

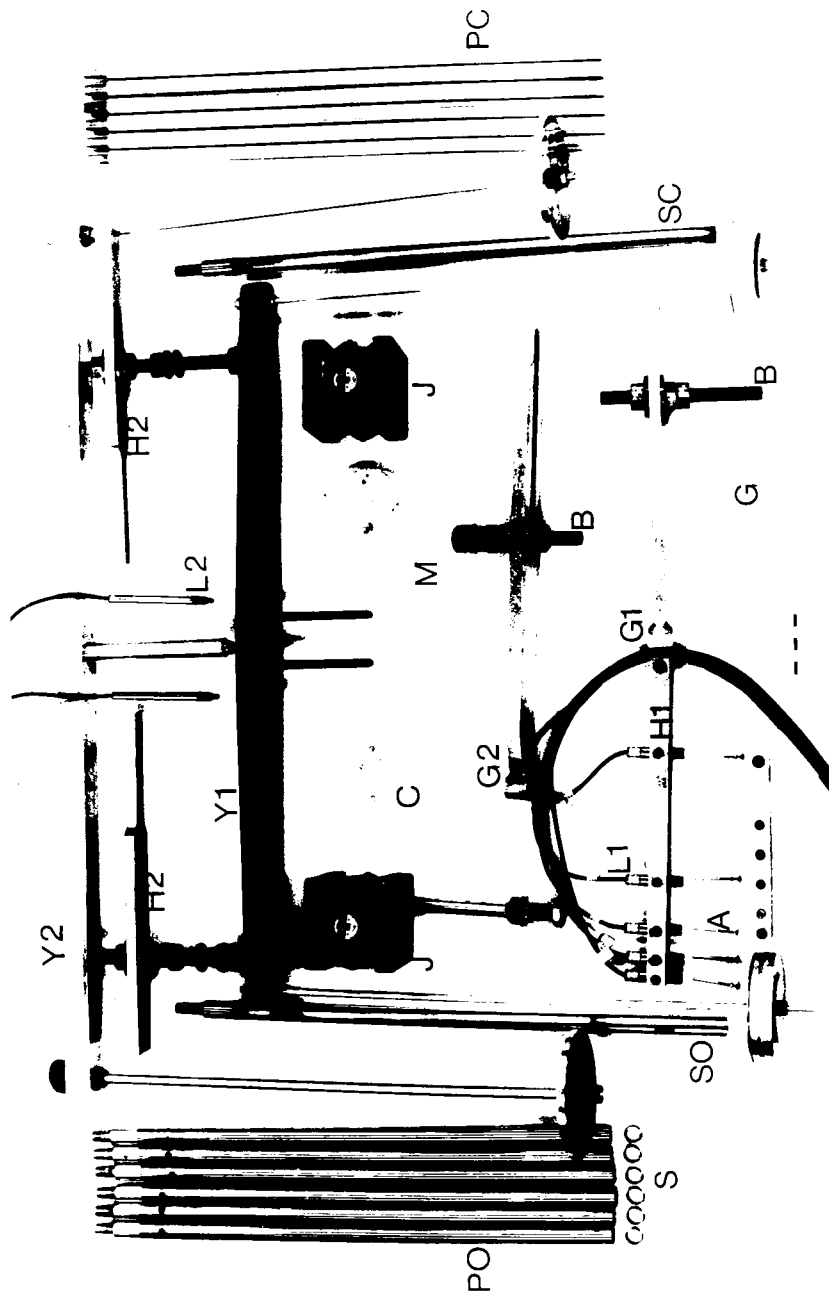
During the initial consolidation stage, only the three transducers located near the wall are used. The rubber bung with a 2 mm diameter hole and a split along its radius (detail Y of Figure 3.13) is replaced by a rubber bung without a split and hole during this initial consolidation.

The other three transducers are later inserted (without unloading) into the specimen at the end of the primary consolidation for the stage just before the final stage of consolidation.

Figure 3.14 shows the apparatus used to insert a pore pressure transducer into a specimen to the required distance from the cylinder wall. The cap is loosened to remove the rubber bung and the extension collar. The cap has to be retightened before the guide of the insertion apparatus can be placed against the cap with its short tube temporarily inserted into the cap. The 2 mm diameter stainless steel rod with its 'U' end pressing against the rear end of the transducer is then pushed into the specimen. The guide has a hollow semi-cylindrical tube to ensure that the transducer can be inserted horizontally to the required distance marked on the rod. The rubber bung (with a split and hole) and the extension collar are later fixed and the cap tightened. The final increment of consolidation pressure can then be added.

The exact position of these three transducers with respect to the pile axis has to be determined after miniature site investigation. They are recovered, cleaned and soaked in water so that the next deairing process can begin.

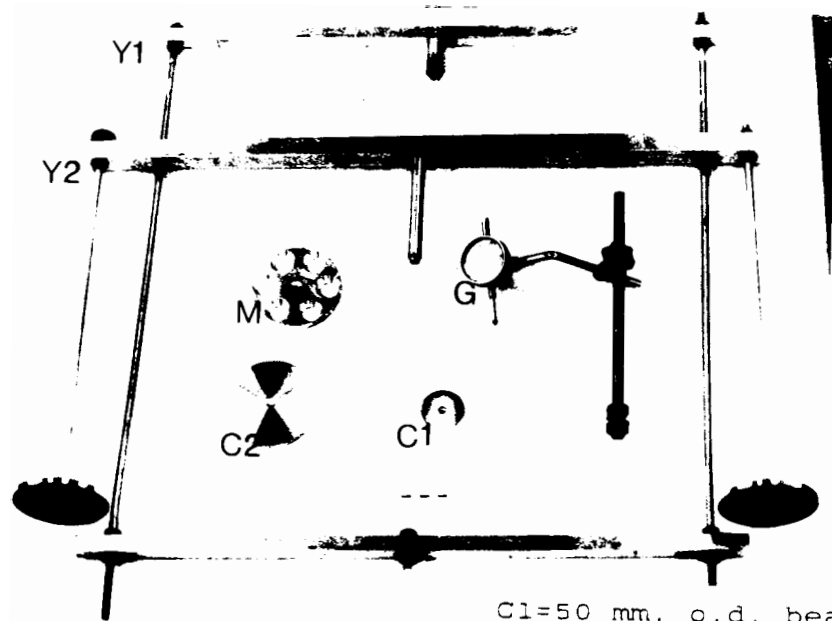
A -Armature of L1
 B =Connecting rod for
 H1 or H2
 C =Collar
 G =Feeler gauge
 G1-Pile guide 1
 G2-Pile guide 2
 H1-L1 holder
 H2-L2 holder
 J =Lab. Jack
 L1-LVDTs(5 Nos.)
 L2-LVDTs(2 Nos.)
 M -Model mudmat
 PC=Closed-ended pile
 PO=Open-ended pile
 SC=Semi-cylindrical pile
 (closed-ended)
 SO=Semi-cylindrical pile
 (open-ended)
 S =Pile driving shoes
 Y1-Yoke for single pile
 Y2-Yoke for group piles



Tests:

1. Single Pile: Include A,B,G,G1,L1,PC and PO.
2. Single pile with surcharged collar: Include A,B,C,G,G1,H1,J,L1,PC, PO,S and Y1.
3. Group piles with surcharged mudmat: Include A,B,G,G1,H1,H2,J,L1,L2,M ,PC,PO,S and Y2.
4. Semi-cylindrical pile: Include A,B,G1,L1,SC and SO.

Figure 3.15 Accessories used for assembling specimens



C1=50 mm. o.d. bearing plate

C2=104 mm. o.d. bearing plate

G =Dial gauge

M =Model mudmat

Y1=Yoke 1

Y2=Yoke 2(as for group pile)

Figure 3.16 Accessories used for plate bearing tests

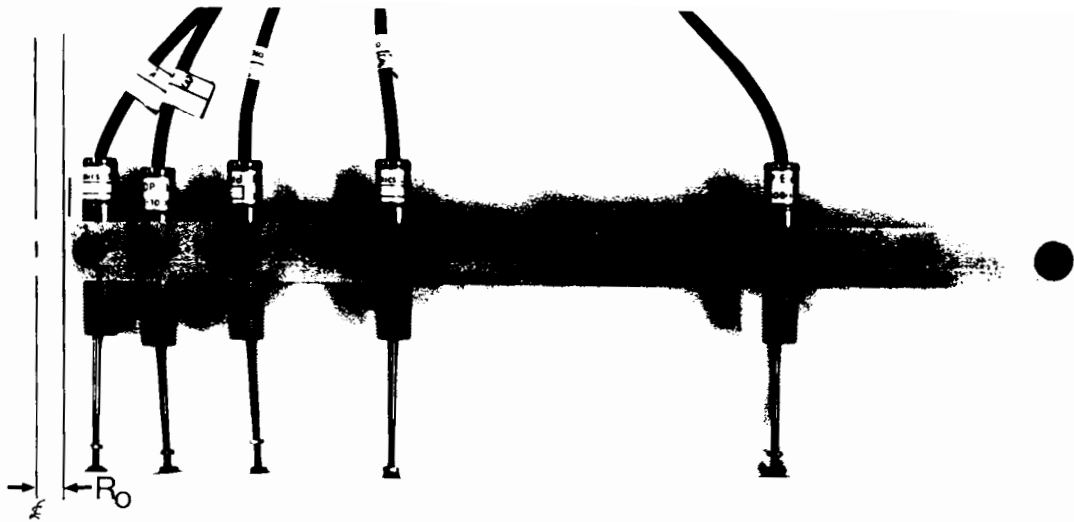


Figure 3.17 Positions of LVDTs

The purpose of the pile guide (see H1 of Figure 3.15) is to ensure that the pile be driven vertically downward into a soil specimen. There is a clearance of 0.25 mm around the pile. On the other end of the pile guide, there is a slot which allows the pile guide to move slightly horizontally towards or away from the pile.

b) Collar and Yoke

The rigid collar has an 18 mm diameter hole at the centre,

which allows a clearance of 1 mm around the pile during driving. The diameter (Figure 3.18) of this hole is intentionally enlarged from 18 mm at the bottom to 28 mm at the top of it. This is to provide space for any clay that may be squeezed out during pile driving. The loading points shown are the points at which the load is to be transferred from the loaded yoke to the collar. There is a 30 mm diameter hole at the mid-section of the wooden yoke for the pile to go through. Each of the two extension rods outside this hole has a ball bearing attached at the end. These will be in contact with the loading points of the collar when loaded. The standard weights (hanger type) are used to provide the load to the wooden yoke.

c) Model Mudmat and Yoke

Figure 3.19 shows the rigid model mudmat. It has a pitch circle radius of $4.5R_0$ (where R_0 is the pile radius) of the model pile. The spacing (centre to centre) of the holes is $4.5R_0$. Each of the six holes in the mudmat is exactly the same as the one described for the collar. Due to the difference in its arrangement, only a single loading point is possible and used. The horizontal arm of the yoke is made from a duralumin bar with an extension rod at the mid-section of the arm. Similarly, a ball bearing is attached to the end of the rod for point loading.

d) Plate bearing accessories

Three types of bearing plates are used (Figure 3.16). The

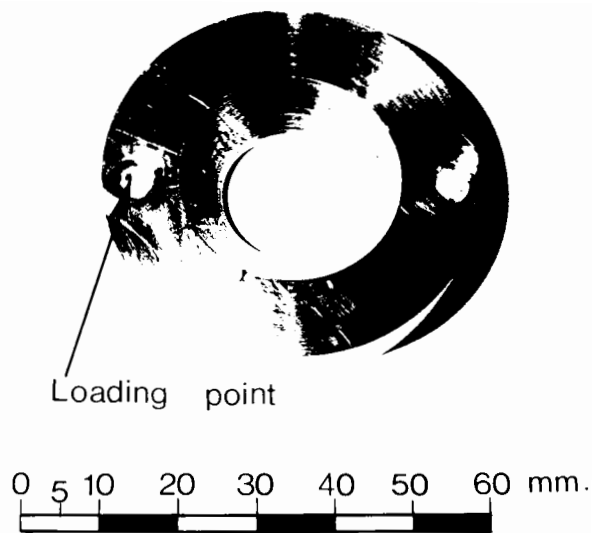


Figure 3.18 The collar

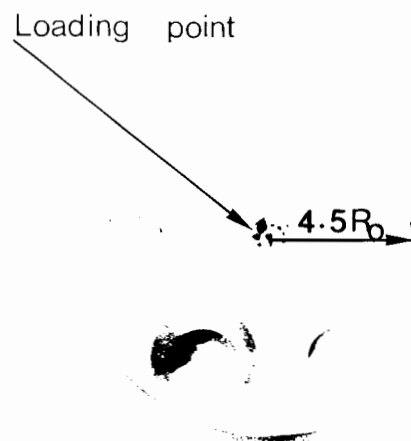


Figure 3.19 The model mudmat

50 mm diameter plate is used for most of the tests. The mudmat and 104 mm diameter plate are also used in a series of tests. The yoke(Y2) used for loading the mudmat in the group pile tests is also used in the plate bearing tests with the two bigger plates. A dial gauge with 0.01 mm/div. graduation is used to measure the settlement of a bearing plate.

CHAPTER 4

EXPERIMENTAL PROCEDURE

4.1 Introduction

This chapter describes the soil properties of the clay used and the experimental procedure of preparing both semi- and full cylindrical test specimens. The detailed descriptions of the various tests and subsequent miniature site investigation and standard laboratory tests are given.

4.2 Soil Properties

The homogeneity and repeatability of specimens were important. Field specimens are non-homogenous and have unknown stress histories. Therefore, reconstituted clays have to be used.

Speswhite kaolin, a China clay, was selected because its properties are well known as a results of many years of experience, particularly at Cambridge. The three principal reasons for this selection are:

1. the clay is readily available in large quantities.
2. the relatively high permeability permits a more rapid specimen preparation.
3. thixotropic effects are negligible.

The properties of Speswhite kaolin are similar to those of the now unavailable Spestone kaolin (see Steenfelt, Randolph and Wroth, 1981). Spestone kaolin has been used by many investigators, notably by Loudon (1967) and Nadarajah (1973).

Speswhite kaolin has also been successfully used by many research workers. Most of these carried out their research at the University of Cambridge. They include: Francescon (1983), Mair (1979), Knoyle (1979), Randolph, Steenfelt and Wroth (1979) and Steenfelt et al. (1981).

The index properties determined by Steenfelt, Randolph and Wroth (1981) and Martins (1983) are given in Table 4.1 while the strength and consolidation properties are given in Table 4.2. Notice that the coefficients of consolidation quoted by the two references have been intentionally omitted in Table 4.2 because this coefficient is not a constant. It varies with the consolidation pressure (see results of the standard laboratory tests in the next chapter). A typical permeability for a specimen with an initial voids ratio of about 1.45 and a vertical pressure increment from 100 kPa to 200 kPa was 2×10^{-5} mm/s.

TABLE 4.1 INDEX PROPERTIES

PROPERTY	SYMBOL	VALUE	
		MARTINS(1983)	STEENFELT(1981)
PLASTICITY INDEX	PI	30	31
PLASTIC LIMIT	PL	32	38
LIQUID LIMIT	LL	62	69
SPECIFIC GRAVITY OF SOLIDS	G	-	2.61

TABLE 4.2 CONSOLIDATION AND STRENGTH PROPERTIES

PROPERTY	DEFINITION	MARTINS(1983)	STEENFELT(1981)
SLOPE OF CRITICAL STATE LINE	M	0.9	0.9
SLOPE OF CONSOLI- DATION LINE	λ	0.25 for $p': 5-50$ kPa. 0.22 for $p': 50-200$ kPa.	0.25
SLOPE OF SWELLING LINE	κ	0.04	0.05
VOIDS RATIO AT $p'=1$ kPa.	$e_{\lambda(1-D)}$	2.52 $p': 50-100$ kPa.	2.58

4.3 Preparation of Specimens

4.3.1 Derivation of the Initial Height of Slurry

The preparation of specimens began with a known weight of kaolin powder and volume of water to produce a consistent slurry of the required water content. The accuracy of the final height of the specimen was important. For the semi-circular specimens, the final height of the specimen had to be slightly higher than the height of the stainless steel plate which was the height of a cylinder. The final height of a full cylindrical specimen had to be about 15 mm lower than the height of the cylinder so that it could retain a layer of 5-8 mm of water on the clay surface (see Figure 4.3).

The initial water content and height of the slurry were calculated from the derived properties of kaolin (Steenfelt, 1981 and Mair, 1979). The following properties were used; $\lambda=0.25$, $e_{(1-D)}=2.58$ at $p'=1$ kPa and $K_o=0.64$ (Nadarajah, 1973). The values were plotted in Figure 4.1 as e , voids ratio against logarithm of vertical consolidation pressure. The water content (w_f) for a particular pressure (w_f) was the ratio of e , voids ratio at that pressure to G , the specific gravity of kaolin.

The initial height of saturated slurry can be expressed as:-

$$H_i = H_s (1 + w_i G)$$

where H_i and H_s are initial height of slurry and soil grains, w_i is the water content of the slurry (120%). The height, H_s can be obtained from the expression :

$$H_s = H_f / (1 + w_f G)$$

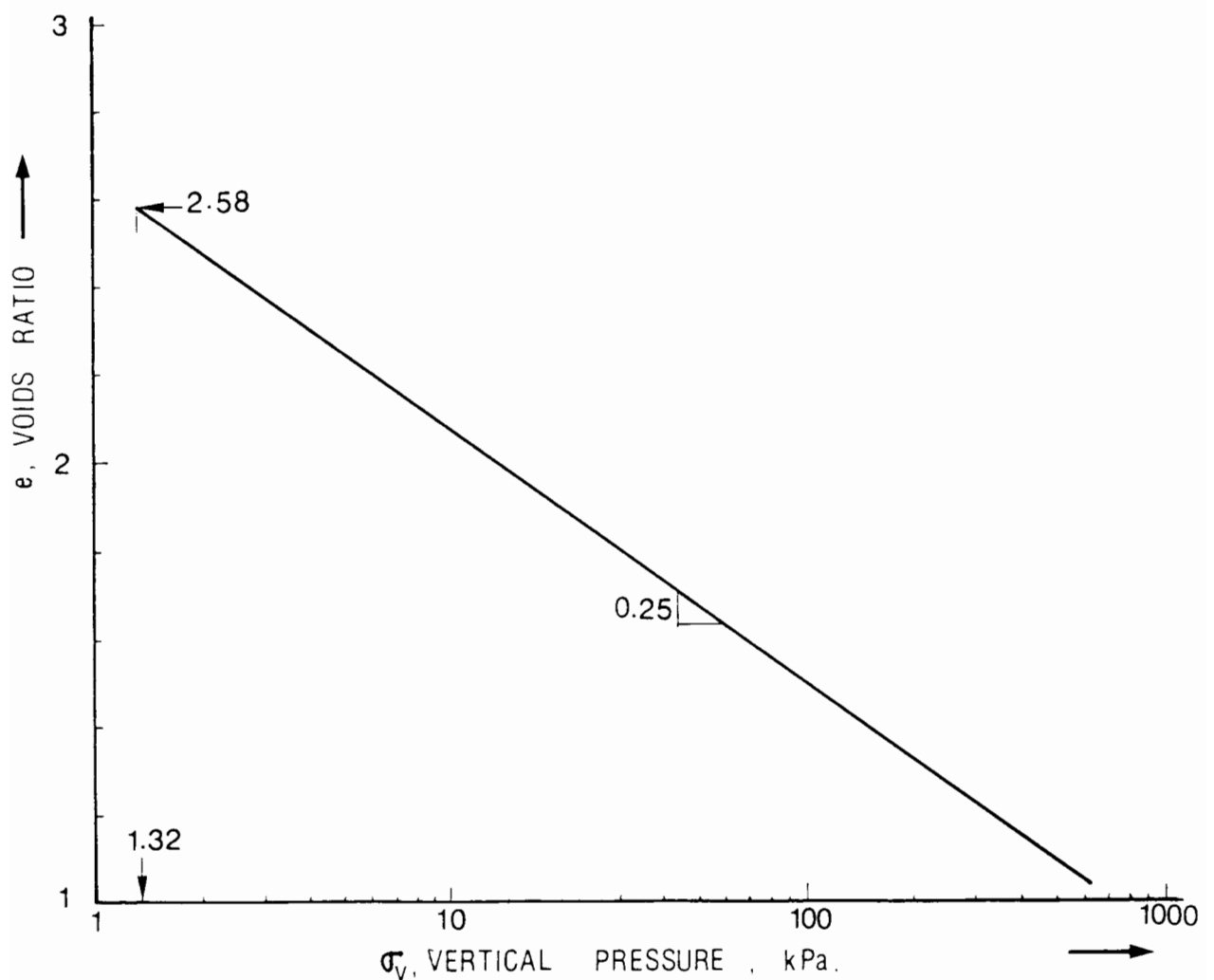


Figure 4.1 Variation of voids ratio with vertical consolidation pressure obtained from experimental results (Steenfelt, Randolph and Wroth, 1981)

where H_f is the final height at the end of consolidation. For overconsolidated specimens, the swelling had to be accounted for. The average amount of swelling from a normally consolidated specimen with a maximum vertical consolidation pressure of 200 kPa to an overconsolidated specimen with an OCR of 6 was about 6 mm. The OCR, (overconsolidation ratio) was defined conventionally as the ratio of the maximum to final vertical consolidation pressure.

4.3.2 Mixing

The main aim of the preparation of specimens was to produce homogeneous specimens. The preparation started by making a consistent and fully saturated slurry.

The trough of the mixer was properly cleaned before mixing. A pneumatic slurry pump situated below the mixer (see Figure 3.9) was used to pump out all the water except for the water inside the inlet hose to the pump. The on-off valve below the trough was closed before pouring in the measured volume of water.

The amount of water added to achieve a water content of 120% in the slurry was calculated from the predetermined moisture content of the kaolin powder (1-2%). The measured volume of water required was poured into the trough (usually about 58.5 litres for two 25kg bags of powder). The powder was then spread onto the surface of the water and allowed to

absorb water and sink. It was important to see that no free powder was floating on the surface of the water before closing the lid. The free powder around the sides of the trough can be scraped by a palette knife into the trough. This prevented the powder from being sucked out by the vacuum when it was applied.

The lid of the trough was closed and fastened with clamps and the motor of the mixer started. The vacuum was applied through the lid to reach its maximum coarse vacuum. A vacuum of 60 torr was achieved for all the mixes. This vacuum pressure at 21⁰ C (controlled laboratory temperature) was slightly below the vapour pressure of water (17 torr). Observation at this stage, through the view window showed that bubbles were generated at a very fast rate.

The slurry was mixed under this vacuum for a total period of 2 hours. In the last 10-15 minutes, the vacuum was slowly released to allow time for the vapour to collapse. It was found that the last quarter of an hour was sufficient time for this to happen. At the end of 2 hours of mixing, the motor was switched off and the lid opened. Moisture content specimens of the slurry were taken, and the on-off valve below the trough was then opened. For the first mix, the pump initially discharged the water that was left in the inlet and outlet hoses before pumping the slurry which was then transferred into the consolidometer.

4.3.3 Preparation of Full Cylindrical Specimens

The inner walls of the top and bottom cylinders were thoroughly cleaned before a layer of silicon grease was applied (about 90 cc/sq m of grease for each full cylinder). This reduced the friction between the clay and walls of the duralumin cylinders during consolidation. The base plate was bolted to the bottom cylinder through its lower peripheral flange, and then also to the top cylinder to form a consolidometer. A hose connected to the bottom drainage outlet at the base plate was led to the top cylinder so that the water that dissipated out from the bottom of the specimen could be collected to maintain the head of water for the bottom outlet (see Figure 3.1). Finally, a porous plastic disc was laid on top of the base plate with its smooth side facing upward. The consolidometer was sealed by installing gaskets between all the peripheral flanges that were either in contact with another flange or the base plate. The gaskets were made from 3 mm natural rubber sheet cut to the required shapes.

The slurry was then pumped through an outlet at the lower end of the trough of the mixer by a slurry pump (pneumatic pump worked on Venturi principle). The slurry from each mix was never completely pumped out from the trough. About 15 litres of slurry were always left in the trough to avoid pumping air into the consolidometer. The on-off valve below the trough must be closed before each mix. The mixer was not cleaned before subsequent mixes until all three

consolidometers had been filled to the required height with the slurry.

When a consolidometer was full, another porous plastic disc that had been chamfered slightly to reduce friction (see Figure 3.2) was laid on top of the slurry with the smooth side in contact with the clay. It fitted loosely so that after unloading, this disc could float freely on the surface of water.

The specimens were consolidated one-dimensionally with top and bottom drainage since some problems have been encountered when radial drainage has been used. Tschebotarioff (1950) presented an explanation based on results of laboratory tests which is illustrated by Figure 4.2. When a vertical pressure was applied to the originally plane surface of clay through a flexible rubber membrane during consolidation, the surface of the clay was found to show a pronounced dip at the center of the specimen. He also found an entirely different condition when a rigid steel plate was used instead of the flexible membrane. At the end of consolidation, the steel plate had been supported by a hollow outer cylinder of consolidated clay with a central shaft of semi-liquid clay of the original consistency (Figure 4.2b).

Before the consolidation began, the top platen was brought into contact with the top porous plastic disc by using the pneumatic pump described in the hydraulic system earlier.

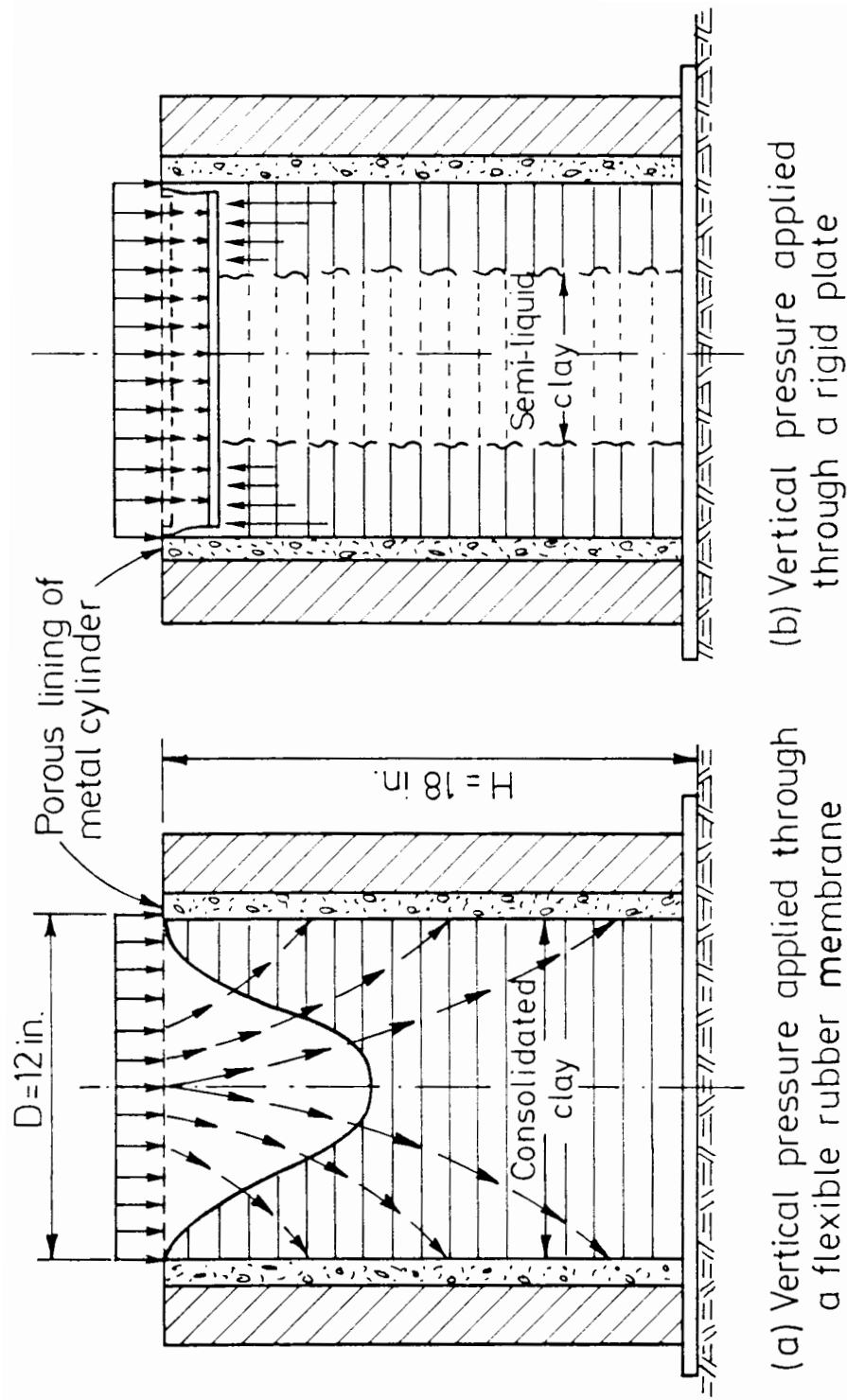


Figure 4.2 The effect of boundary conditions on the surface deformations and on the change of water content of a clay consolidated in the laboratory (after Tschebotarioff, 1950)

The consolidation programme then began, this involved increasing the pressure in stages, initially in small increments but later in larger ones. Each successive increment was equal to the previous pressure so that the total pressure was doubled. The pneumatic pump was used for the initial consolidation up to a vertical pressure of 25 kPa. For greater pressures, pressurised nitrogen was used to continue the consolidation. The consolidation process was monitored by a long stroke LVDT. The output of the LVDT was fed to the chart recorder. In addition, the output of the LVDTs and pore pressure transducers were fed to the analog scanner for series 12 to 15.

Since consolidation of such a large specimen took a long time, no full primary consolidation was reached for all except the final stage. Each initial stage was consolidated for about two days until a rate of settlement of about 1.5 mm per hour was reached. The final stage of consolidation was monitored closely with results plotted as e against $\log \sigma'_v$ throughout the consolidation. This determined the end of primary consolidation and prevented secondary consolidation. However, in series 12 to 15, the last two stages of consolidation were allowed to reach their end of primary consolidation.

Most of the specimens prepared were overconsolidated with an OCR of 6 and a maximum vertical consolidation pressure of

200 kPa. A series of OCR of 1 and 10 each with the maximum vertical pressure of 200 kPa were also prepared to study the influence of OCR on ground heave. To study the influence of the strength of clay on ground heave, two series of specimens with the same OCR of 6, but with the maximum vertical consolidation pressure increased to 400 and 600 kPa respectively were prepared.

For the over-consolidated specimens, the vertical pressure was reduced in stages but each stage did not exceed 100 kPa. This precaution was taken because of the inability of the clay to withstand high negative pore pressure as discussed by Mair (1979). The time taken to prepare a series of specimens with a maximum vertical pressure of 200 kPa and OCR of 6 was about three weeks. Higher stressed specimens took much longer.

4.3.4 Preparation of Semi-Cylindrical Specimens

Semi-cylindrical specimens were prepared similarly to the full cylindrical specimens except that the bottom full cylinder was made of two half-cylinders. Each of the half cylinders described in chapter 3 was bolted to a base plate. A 18 SWG. (about 1.2 mm thick) stainless steel plate was greased with a thin layer of silicon grease on both sides. It was enclosed by a thin polythene sheet and the two exposed sides of the polythene sheet were also greased to reduce friction between the sheet and the clay. The plate

was then placed vertically between the two half cylinders to form a bottom complete cylinder. The plate was fully restrained horizontally between the two semi-circular base plates and on the two vertical sides of the flanges. Similarly, gaskets as described in the preparation of full cylindrical specimens were used to seal the two half cylinders.

4.4 Test Procedure

4.4.1 Introduction

The specimens were tested one at a time. It took three consecutive days to test a series of three specimens. All the specimens in a series were prepared to have the same stress history. The consolidation pressure must be removed from a specimen so that a pile can be driven and the ground heave measured. Removing the pressure in the specimen generated negative pore pressures. To prevent the specimen from sucking in air and becoming unsaturated, each of the full cylindrical specimen had a layer of water on the surface except those in the initial series which were used for preliminary tests. However, the semi-cylindrical specimens were tested without a layer of water on the clay surface.

Miniature site investigation was carried out immediately after pile driving. This investigation included plate bearing tests, laboratory vane tests, moisture content

measurements and sometimes, some 38 mm diameter samples were taken for subsequent triaxial tests. The time taken for each process was recorded so that comparison of site investigation results from different series prepared with the same stress history could be made.

4.4.2 Unloading

When the three specimens were ready for testing, one was unloaded first (only one specimen was tested in a day because of the time involved in preparing a specimen and testing it). Before unloading one of the specimens, the stress condition of the other two specimens must not be altered or affected by the unloading. The process by which this was achieved has been described in the hydraulic system in the previous chapter.

The bottom drainage outlet of the specimen was closed to prevent it from sucking in air during and after unloading. The water was syphoned off leaving about 10 mm of water on top of the top platen. Unloading was done by using the pneumatic pump thus leaving the pressure in the accumulator unchanged when the piston was raised. It took about 5 minutes to unload a specimen and to raise the piston so that its platen was above the top flange of the top cylinder and ready to be lifted out from the reaction frame. On raising the top platen, the water left on the platen could drain onto the top porous disc through the two drainage outlets at

the top plate. The valve of this piston had to be closed before the hydraulic circuit could be in the loading position to maintain the pressure for the other two specimens.

4.5 Testing of a Full Cylindrical Specimen

4.5.1 Single pile

The unloaded specimen was immediately lifted out from the reaction frame by using a forklift (500 kg capacity). The top cylinder was unbolted from the bottom cylinder and removed. The top porous plastic disc could then be seen floating on the water above the specimen. The final height of the specimen was always 430 mm(± 5 mm) with a layer of 5-8 mm of water retained on the surface the clay (Figure 4.3). The specimen was lifted towards the levelled pile driving rig. The specimen was then positioned so that the pile attachment was near the centre of the specimen, and then levelled by adjusting the levelling screws at the four corners of the seating with the help of a spirit level (a metre long construction type).

The holder and bodies of the LVDTs (see H1 of Figure 3.15) but without armatures in them were clamped to a 12.5 mm diameter rod which in turn was bolted to the top flange of the cylinder. A model pile was screwed into the pile attachment and further adjustment of the specimen was made if necessary by fine positioning so that the model pile was

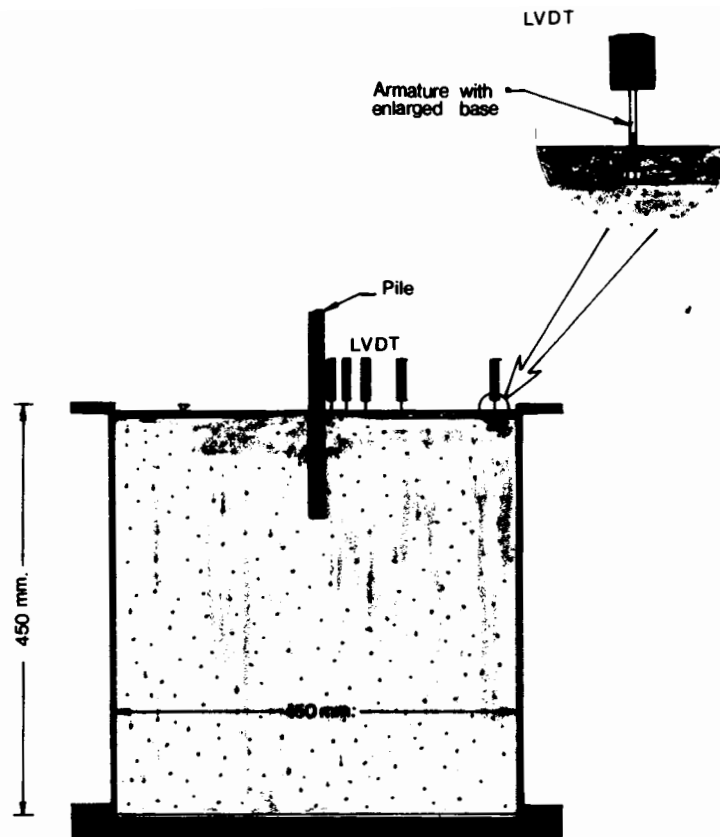


Figure 4.3 Schematic diagram of a single pile test

at the centre of the specimen and LVDTs were at the required distance from the centre of the pile axis. There was a 3 mm gap between the pile wall and the end of the LVDTs' holder, so that the positions of the LVDTs would be at the required radial distance when assembled. A 3 mm thick feeler gauge

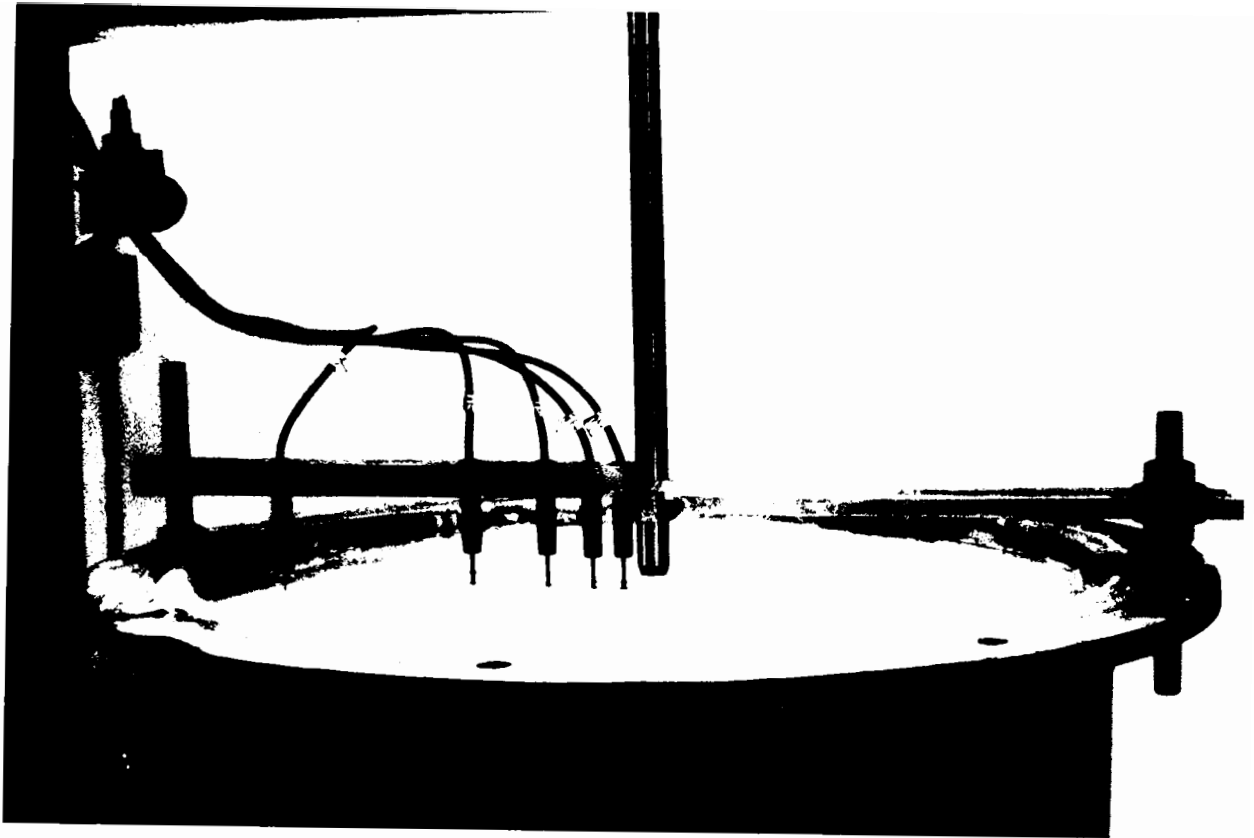


Figure 4.4 Photograph of a single pile test

driving was very small, about 0.01 mm.

After swelling, the armature of each LVDT was readjusted so that the anticipated movement of the armature during pile driving was in the linear range of the LVDT. The flywheel of the motor was manually rotated so that the hammer was always resting on the pile attachment before driving each pile. The speed and drop height of the hammer were checked

was used to set this gap. The armature of each of the LVDTs was later introduced into its housing. This was done by loosening the plastic screw and holding the housing in one hand and moving the housing upward slightly, so that an armature held in the other hand could slowly be slid into its housing. The housing was moved down slightly until about three quarters of the armature was inside its housing. These armatures were carefully adjusted so that they began and ended in the linear range of the LVDTs. A multimeter was used to achieve the initial setting of the armatures.

The pile guide was then installed directly opposite the solder. Figure 4.3 shows a schematic diagram and Figure 4.4 shows the arrangement of the assembly before pile driving. The specimen was then left and allowed to swell for a period of three hours before pile driving. This time was a compromise between the rate of swelling and the time available for testing and carrying out subsequent miniature site investigations.

During swelling, the output of the 5 LVDTs and elapsed time were scanned at intervals of 5 minutes and stored in the microcomputer disk. From the swelling curve (chapter 5) the amount of swelling during pile driving could be estimated with confidence and be deducted from the ground heave recorded during pile driving. Pile driving for most cases took about 90 seconds except for tests where the effect of the ground heave due to the rate of driving were investigated. Therefore, the amount of swelling during pile

before loading the scanning program.

The scanning program was then loaded, the output of all the channels was displayed on the screen of the monitor. A check could be carried out to compare the output of each LVDT at the junction box of the data logging system and the displayed values. The necessary information of the pile position, increment penetration and final depth were entered into the program and the motor started. The additional weight to prevent the glider from falling was removed at the same time as the motor was started. The data were stored on a micro-computer disk at the end of pile driving and preparation for a second pile to be driven could then be carried out.

The model pile was disconnected from the pile attachment leaving the pile in its driven position. The second pile was driven at the locations shown (Figure 4.5). To carry out this test, the specimen had to be positioned on the seating so that the pile could be driven at the required position. Due to the low friction between the seating and the duralumin base plate, positioning the specimen could easily be achieved. A shorter holder (H2 of Figure 3.15) was used together with a much longer pile guide. The pile was driven and scanned as before. As stated in chapter three, this second pile was too close to the specimen boundary. Therefore the results of these tests are not reported. However, they provided information for the selection of variables.

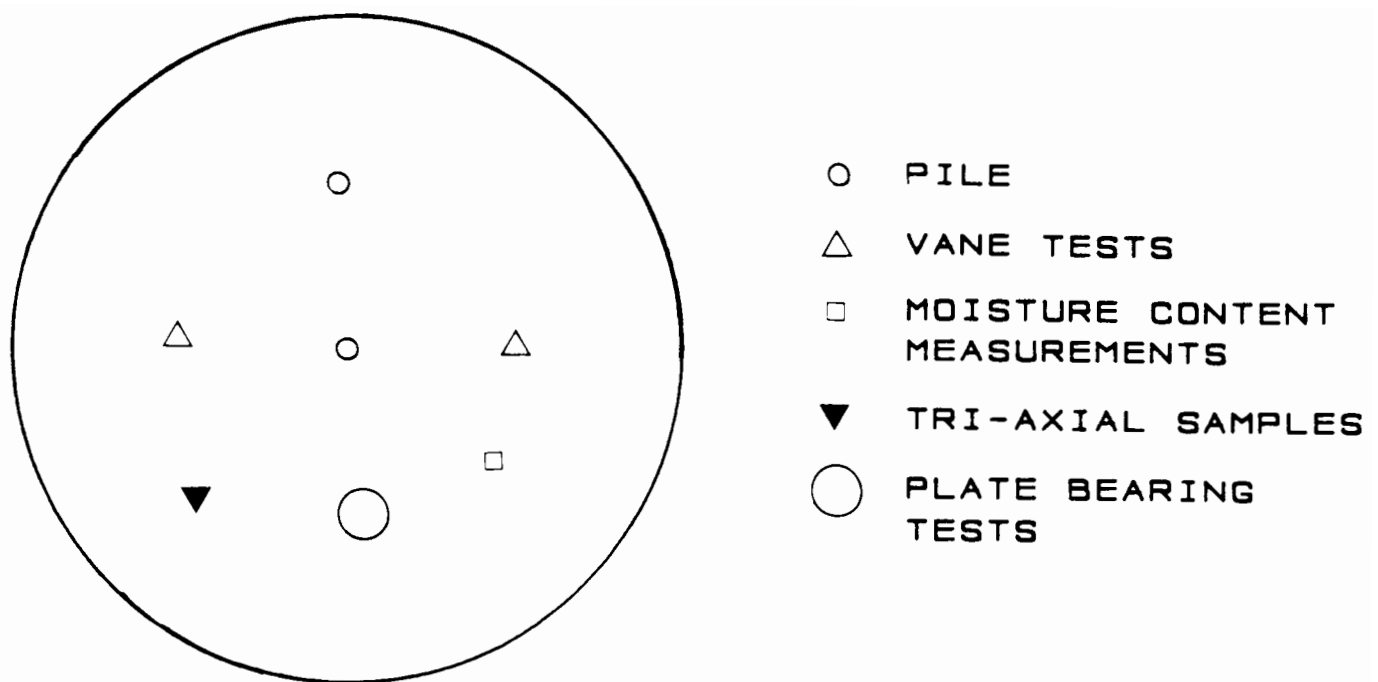


Figure 4.5 Plan view of miniature tests

4.5.2 Pile Driving through a Surcharged Collar

The test procedure was similar to that of single pile described earlier. The arrangement was such that a surcharge pressure could be applied on the surface of a specimen whilst allowing for pile driving. The yoke needed to give the required surcharge pressure on the clay surface was assembled but was supported by the small laboratory screw jacks (see J of Figure 3.15) during the three hours of swelling. After swelling, the collar was positioned so that there was an equal clearance of 0.25 mm, around the collar. One of the LVDT (located at 2 pile radii) was seated on the

collar. The loading arm with the load was slowly lowered so that the two bearings at the end of the two extension rods of the yoke coincided with the loading points of the collar. The loading points were greased lightly to reduce friction when lowering the screw jacks. The purpose of the screw jacks was to position the load on loading points thus avoiding possible uneven loading onto the collar when done by hand. Incorporating the loading arm before swelling reduced the delay time between the end of swelling and pile driving. Similarly, the LVDTs were readjusted so that the anticipated armature movements were again within the linear range of the transducers. The screw jacks were removed or lowered before pile driving.

The motor was started immediately after the program was loaded into the computer to drive a model pile into the specimen. It took less than 5 minutes after swelling to set up and adjust before pile driving. Figure 4.6 shows the schematic diagram with the position of the LVDT resting on the collar shown. Figure 4.7 shows the arrangement at the end of a pile driving.

After driving the first pile in the centre of the specimen, the pile was disconnected from the pile attachment. The loading unit was lifted up and the collar taken out leaving the pile in its driven position. The second pile was driven at the position shown (see Figure 4.5). with the same procedure described for the pile driven at the centre of the specimen.

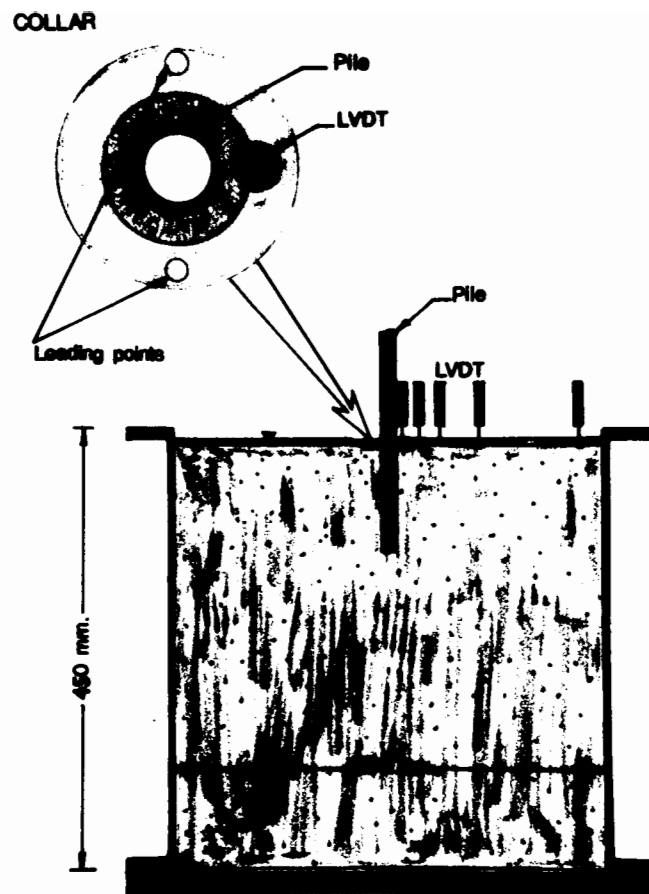


Figure 4.6 Schematic diagram of a single pile driven through a surcharged collar

4.5.3 Group Piles

The details of the locations of the 3 LVDTs on the model mudmat and the sequence of pile driving are shown in Figure 4.8. The ground heave profile for pile no. 1 was monitored as describe for a single pile except that one of the LVDTs (A) was resting on the mudmat. The other two LVDTs (B and C) had to be positioned asymetrically due to

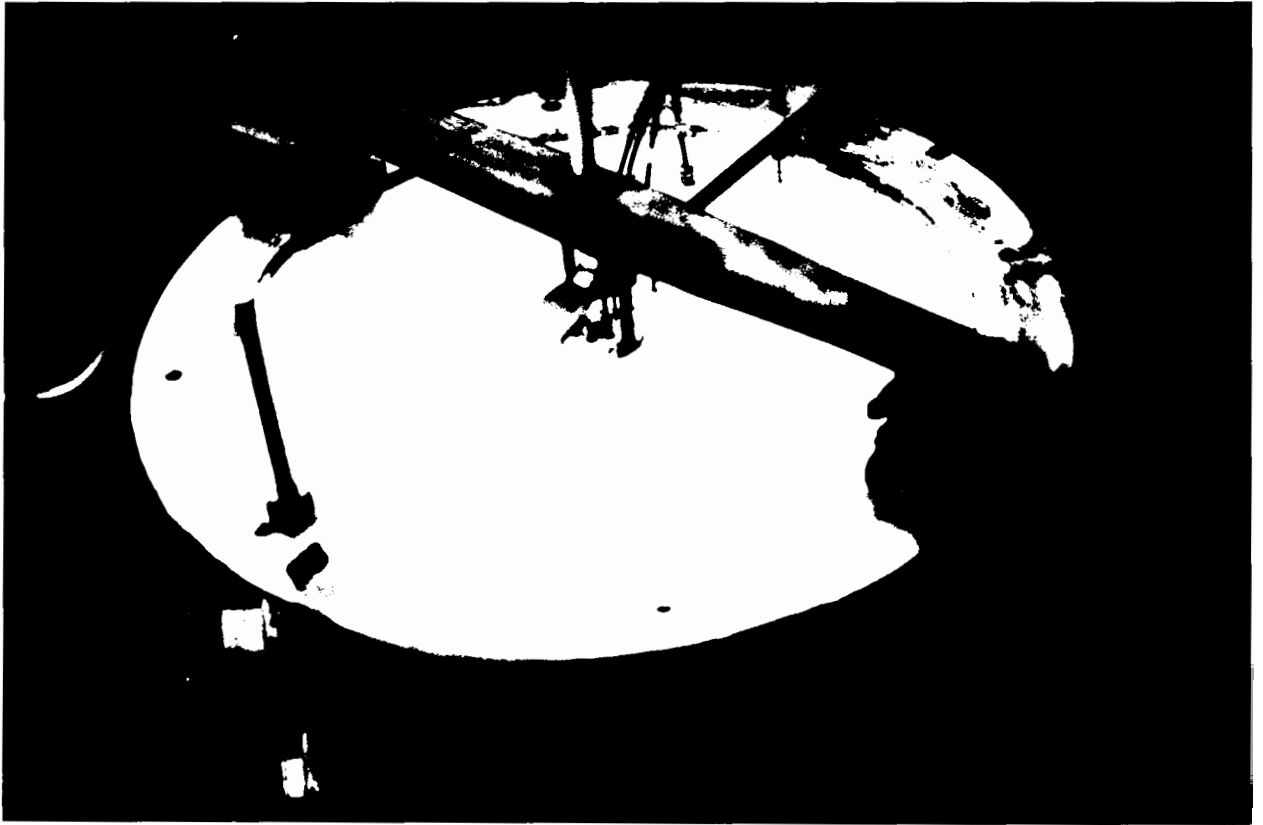


Figure 4.7 Photograph of a single pile driven through a surcharged collar

space restriction (see Figure 4.9). The holes for the piles to be driven through were the same as that in the collar and they served the same purpose.

After three hours of swelling, the model mudmat was positioned on the surface of the specimen. The screw jacks were again used to support the loaded yoke. All the LVDTs were again readjusted as in a) and b) above. Since the two additional LVDTs were added at a later stage after the

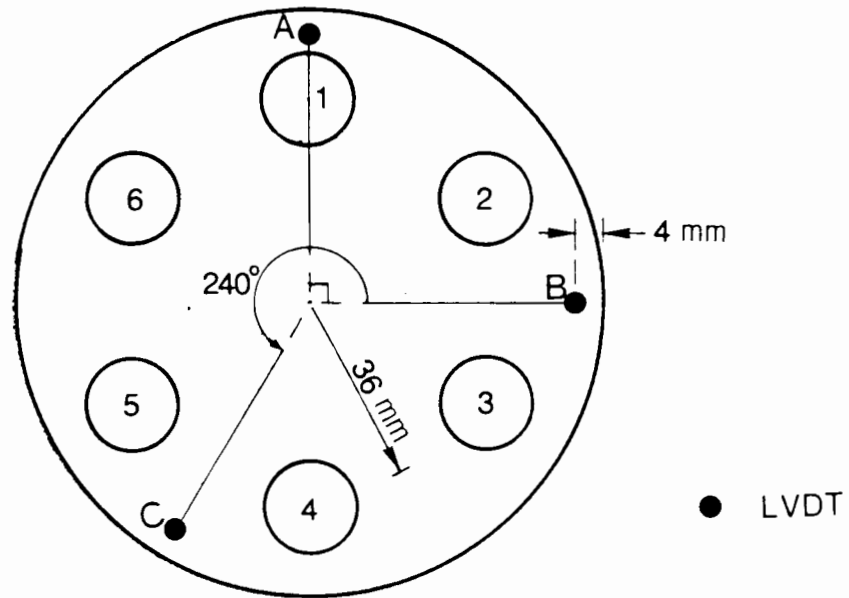


Figure 4.8 Location of LVDTs on the model mudmat

scanning system had been assembled, the voltages of these two LVDTs before and after each pile driving had to be recorded manually. The screw jacks supporting the loaded yoke were carefully lowered so that the bearing at the extension rod coincided with the loading point of the mudmat. A drop of grease was placed onto the loading point. This procedure of loading the mudmat ensured the load was transferred concentrically to the mudmat. The screw jacks were removed before driving the first pile into a specimen.

After driving the first pile, the pile was disconnected from the pile attachment. Another model pile was screwed into

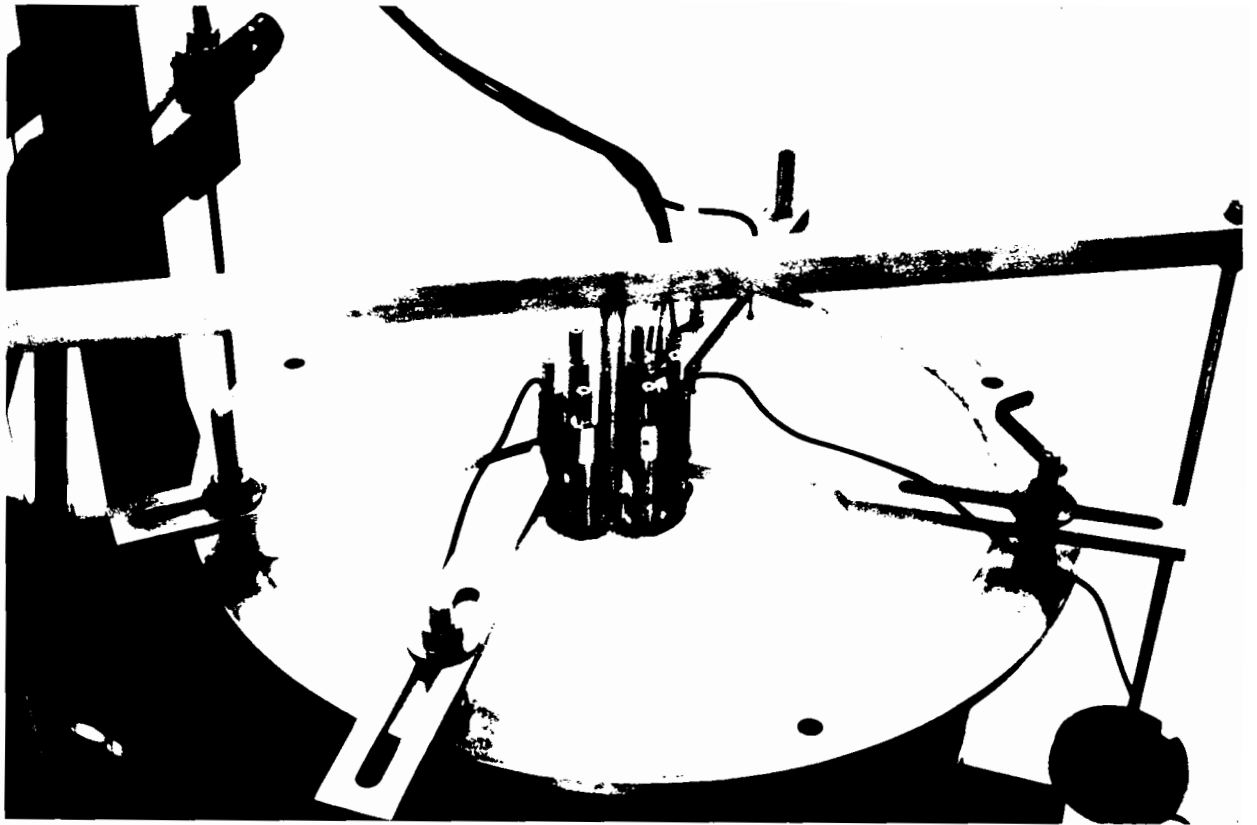


Figure 4.9 Photograph of piles driven through a surcharged model mudmat

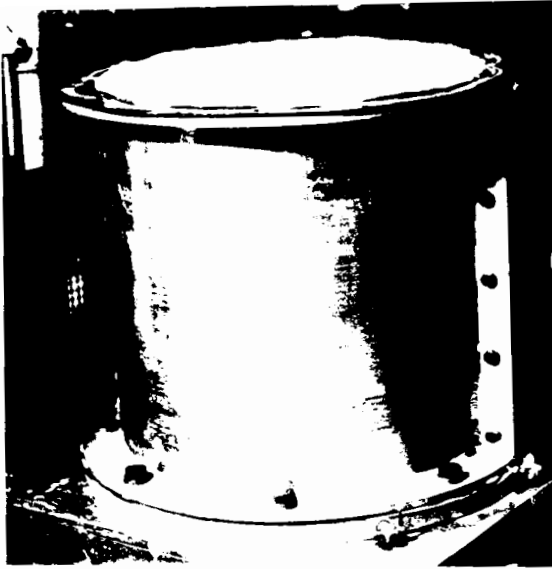
the pile attachment and the specimen was moved carefully with the surcharge still acting on the mudmat. The pile guide had to be relocated and the LVDTs readjusted before next pile driving. This process was repeated for the rest of the four piles. The time taken for each intermediate process from setting its new position, adjustment and pile driving took about 15 minutes. At higher surcharge pressure

the three LVDTs on the mudmat had to be readjusted after almost every pile driving because the surcharged mudmat had settled substantially. Figure 4.9 shows the arrangement of the assembly.

4.6 Testing of a Semi-cylindrical Specimen

After the top platen had been removed, the specimen was always 10 mm (Figure 4.10a) higher than the height of the cylinder (450 mm). The additional soil was trimmed so that the height of the specimen was 450 mm and flush with the top flange. The vertical flanges were unbolted. As there was a stainless steel plate covered with greased polythene sheet, the two half cylinders could be separated easily (see Figure 4.10b). It always ended up with one good semi-cylindrical specimen with the other being slightly less than semi-circular in plan view. If a perspex plate was clamped to the vertical flanges of the latter specimen, a gap of 1-2 mm appeared at the mid-section between the vertical face of the specimen and the perspex for the top half of its height.

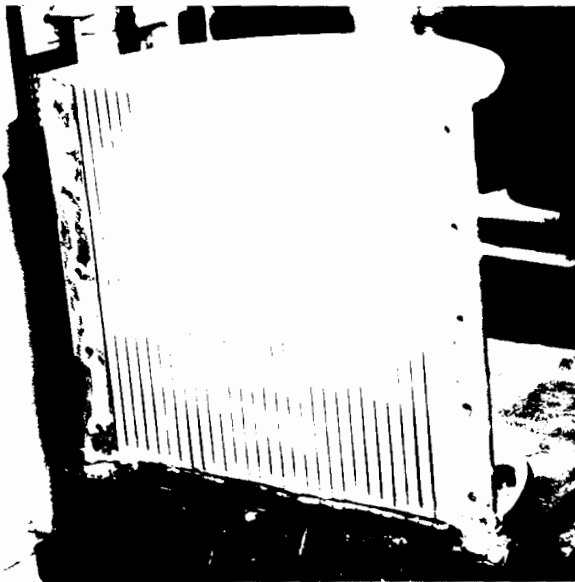
Two techniques of grid marking were used. The first technique involved using a template and matt black quick dry paint. The template was a 2 mm thick perspex plate with 2 mm wide slots drilled out and running parallel at 16 mm intervals. The plate was held against the vertical face of the specimen and the paint sprayed onto the perspex plate. A series of black lines (see Figure 4.10c) resulted. The



[a]



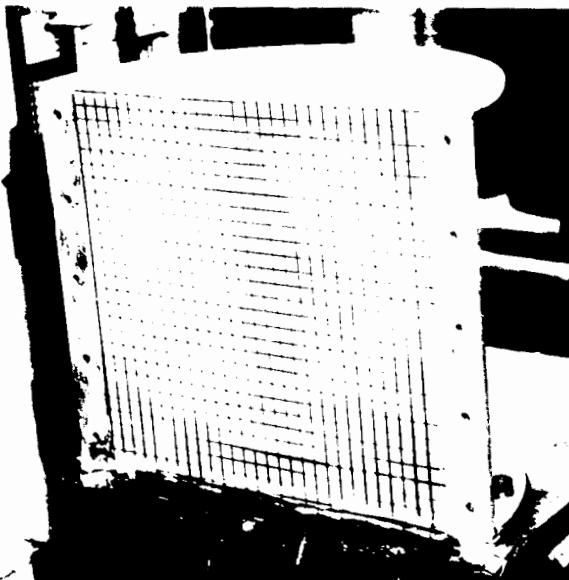
[b]



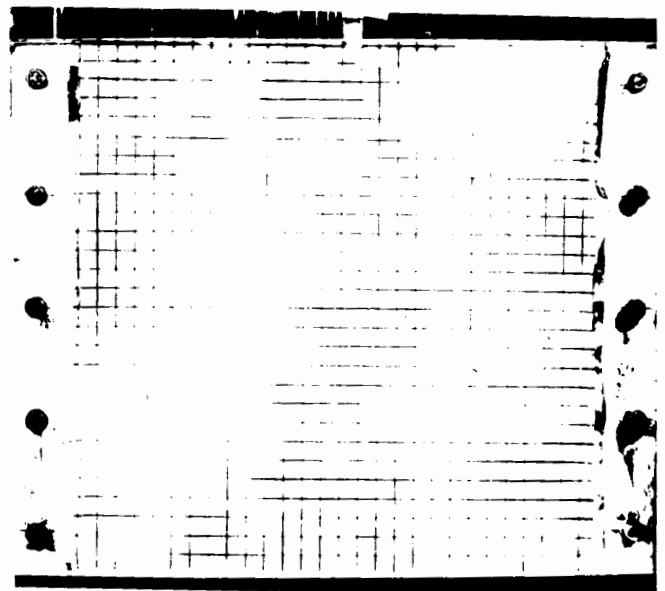
[c]



[d]



[e]



[f]

Figure 4.10 The preparation of semi-cylindrical test specimens

process was repeated after the perspex plate had been rotated through 90 degrees so that a check pattern was produced. (see Figures 4.10d and e). A 20 mm transparent perspex plate was clamped to the specimen through the vertical flanges (Figure 4.10f) which was then ready for the driving of a semi-cylindrical model pile into it. The disadvantage of this technique was the centre of the grids were not well defined enough for accurate digitizing of their coordinates.

The second technique used lead shot with an outer diameter of about 2 mm located at nodal points with spacings between the nodes of 16 mm near the surface and pile wall, and 32 mm at further distances (Figure 4.11).

A piece of graph paper of 550 mm square was used with a check pattern clearly marked. A thin layer of silicon grease was applied to it. This was sufficient to hold the lead shot in place when the paper was lifted vertically. The lead shot were placed at the nodal points of the check pattern. The graph paper was positioned on top of the 20 mm perspex plate and held down temporarily by adhesive tape at the corners of the plate. It was then brought into contact with the vertical face of the specimen. A slight pressure was needed to press the lead shot into the vertical face of the specimen. The paper was then separated from the perspex plate by peeling off slowly. The transparent perspex plate

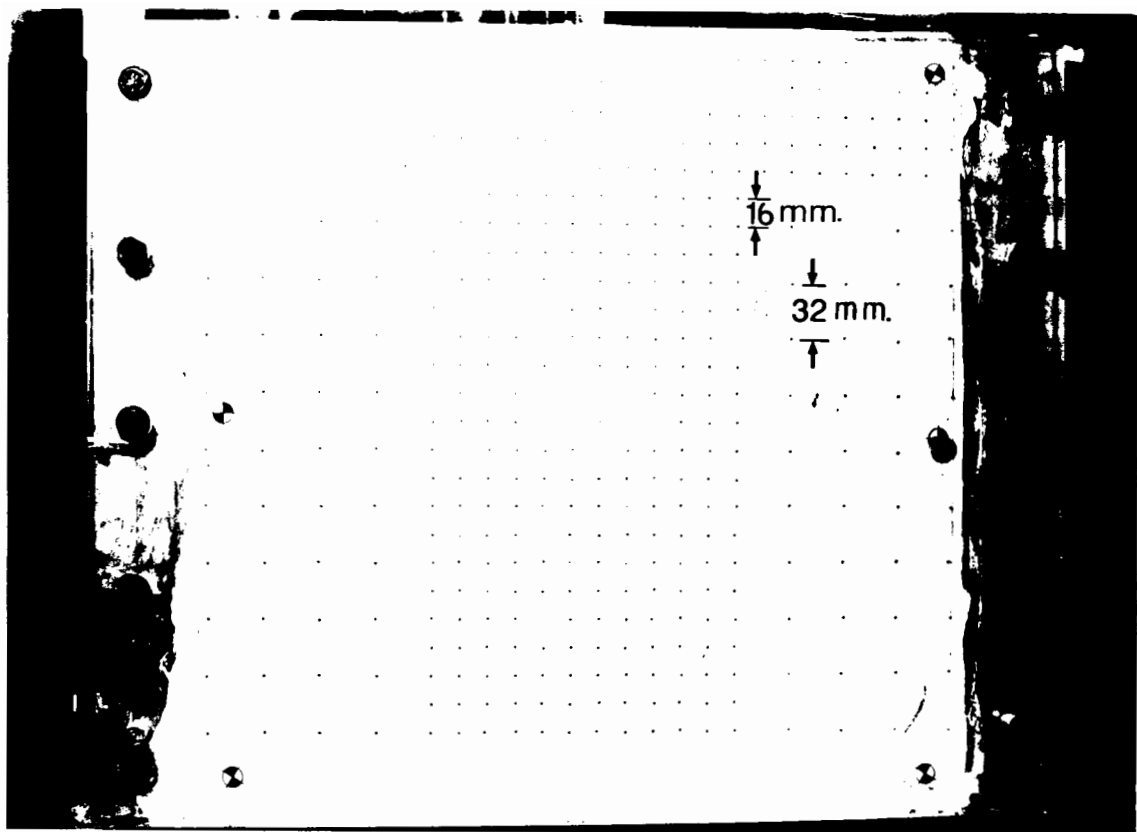


Figure 4.11 Semi-cylindrical test specimen using lead shot technique

with 6 reference markers and known positions was brought into contact with the clay surface and clamped onto the specimen through vertical flanges. About twenty photographs were taken using a SLR standard 50 mm lens camera during pile driving.

The advantages of the second technique were that the lead shot could be positioned onto the paper before the specimen

was unloaded. This allowed the grids to be formed quickly compared with the spraying technique. The lead shot were spherical and all about the same diameter, therefore their centres could easily be pinpointed and digitized accurately. Only the photographs taken from a good specimen were further investigated for the strain and displacements analyses. All the open-ended piles were driven without a driving shoe and their inner wall unpolished and ungreased.

4.7 Miniature Site Investigation

Figure 4.5 shows the plan view of the various tests carried out on each of the specimen after pile driving. Immediately after pile driving various investigations were carried out on a specimen, namely plate bearing test vane tests and moisture content samples taken at different depths.

4.7.1 Plate Bearing Tests

The assembly used for plate bearing tests has been described in the previous chapter. Apart from the normal 50 mm diameter plate, the mudmat and a 104 mm diameter plate (equivalent to the outer diameter of the mudmat) were also used.

Each test was carried out by adding loads incrementally. Initially, each increment was 1 kg but near the anticipated bearing capacity, it was reduced to 0.5 kg and subsequently

increased back to 1 kg at greater settlements. For each increment, the loading was allowed to remain until the plate settled to a rate of 0.01 mm per 10 seconds (equivalent to 0.06 mm per minute). Only one plate bearing test per specimen was carried out because it took about 45 minutes for each bearing test.

4.7.2 Laboratory Vane Tests

The layer of water on the surface of the specimen was removed after a plate bearing test. Standard laboratory vane shear test apparatus manufactured by Pilcon was used. The size of the vane used was a standard 33 mm diameter for all series except for series no. 8 when a 19 mm diameter vane was used because the strength of the specimens in that series exceeded the maximum reading of 600 lbf/ft² (about 29 kPa) for the 33 mm diameter vane. They were carried out at three depths; 75, 210 and 385 mm below the surface. For the two greater depths, a borehole was formed using a soil drill so that there was no friction between the clay and the extension rod to the vane. The vane was rotated at the rate recommended by the supplier i.e. one complete revolution in a minute. The remoulded strength was also taken by rotating the vane quickly for 3 complete revolutions before shearing it at the normal rate.

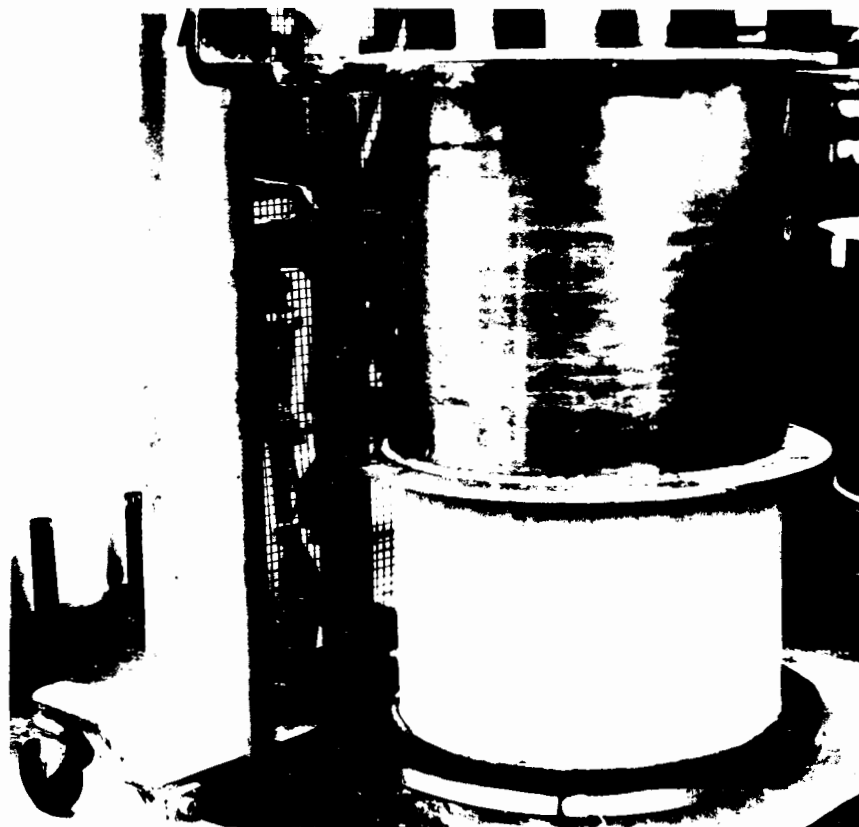
4.7.3 Sampling for Triaxial Tests and Water Content Measurements

The triaxial samplers were thin wall brass tubes specifically made for minimum disturbance. They were tubes with a 38 mm inner diameter and a wall thickness of 1 mm. They were lightly oiled with an ordinary lubricating oil before sampling. The samples were immediately waxed and covered with a layer of cling film. A layer of grease had been sandwiched between the clay and wax. This precaution was taken to prevent the loss of water.

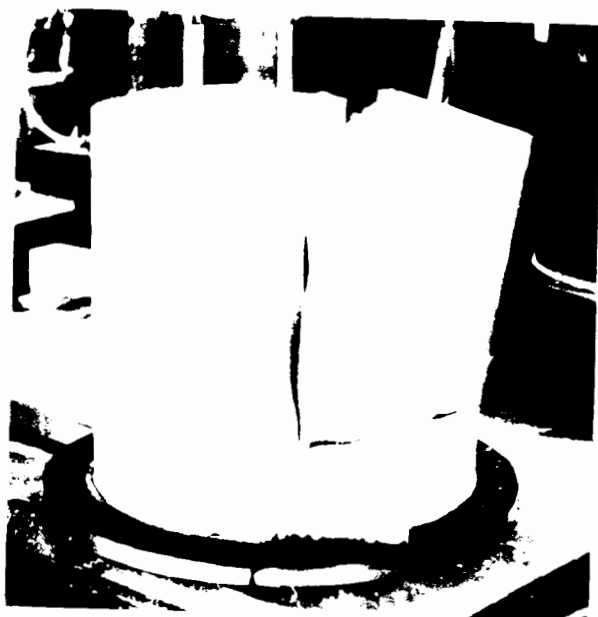
The bolts connecting the bottom cylinder to the base plate were unscrewed after vane tests and the specimen could then be lifted up easily by the top flange of the cylinder with the help of the forklift (Figure 4.12a). The specimen could be sectioned where required using fine wire. This method allowed moisture content samples to be taken quickly. Moisture content samples were taken at 70, 140, 210, 280, 350 and 420 mm below the surface (Figure 4.12c). The water content was determined in accordance with the B.S. 1377. It took less than 2 minutes to take six samples after sectioning.

4.8 Standard Laboratory Tests

The tests included oedometer, undrained triaxial tests with pore pressure measurements and index properties. The index properties tests carried out included liquid limit, plastic



(a)



(b)



(c)

Figure 4-2. Microturbine control pump design

limit and specific gravity tests for the clay used. They were carried out in accordance with B.S. 1377.

4.8.1 Oedometer

The oedometer used was basically the same as the standard consolidation cell with a fixed ring described in B.S. 1377 except the standard cell height had been increased to 90 mm. The consolidation began from a slurry state, with an initial height of slurry of about 51 mm and was consolidated and allowed to swell with top and bottom drainage under a number of successive increments and decrements of vertical pressure. The maximum vertical pressure used was 200 kPa.

A perspex tube of the same inner diameter as the fixed ring was selected for extending to the required height. The inner face of the perspex tube and the fixed ring were greased with a thin layer of silicon grease. An ordinary laboratory filter paper was placed between the top platen and the slurry to prevent the slurry from squeezing out during the initial consolidation. For every stage, the vertical movement of the sample was monitored with a dial gauge. The time intervals were increased logarithmically. The compression of the loading assembly was checked before carrying out oedometer tests to ensure that there was no compression due to load on the loading arrangement and if there was any it could be accounted for.

4.8.2 Triaxial Compression Tests

The undrained triaxial compression tests with pore pressure measurements were carried out for the sample taken after pile driving. Special precautions mentioned by Bishop and Henkel (1962) were heeded. A modern triaxial compression machine was used with a back pressure system which was operated by compressed air via an air water interface. A volume change device with digital output was used together with the pore pressure transducers for both the back/cell and pore pressure measurements.

The samples were tested at a constant rate of displacement of 0.076 mm/min.. The rate was derived assuming the coefficient of consolidation, c_v of the sample was 10 m²/year and the procedure of deriving the rate as recommended by the above reference.

Initially the readings for a test were taken at 1 minute intervals until contact between loading frame and the sample was achieved. Readings were later taken at an interval based on the reading of proving ring (deviator load) of about 10 divisions (approximate 3 kPa) for strain less than 1 %, well within the elastic range. After this stage the deflection of the proving ring was small, therefore the readings were taken at about every 1% strain. However, a closer interval was used near the peak of excess pore pressure.

The results of the miniature site investigation and standard

laboratory tests are discussed in the next chapter.

CHAPTER 5

PROPERTIES OF SPECIMENS

5.1 Introduction

This chapter describes the properties of the clay as determined from the preparation of specimens and the miniature site investigation undertaken after pile driving. Standard laboratory test results are also included.

The main purpose of the standard laboratory tests was to check the properties of the clay derived from the literature review and as a comparison with those obtained from the consolidometer.

5.2 Consolidation and Swelling of Specimens

A total of 17 series of specimens was prepared. In each of the series, the settlement and swelling of a specimen were recorded continuously with a chart recorder. Pore pressure transducers were incorporated in series 12 to 15, and during consolidation and swelling, the outputs from these together with that from the long stroke displacement transducers were recorded on a microcomputer disk.

Figure 5.1 shows a typical graph of the dissipation of

(For the positions of P1 to P6, see Figure 3.13)

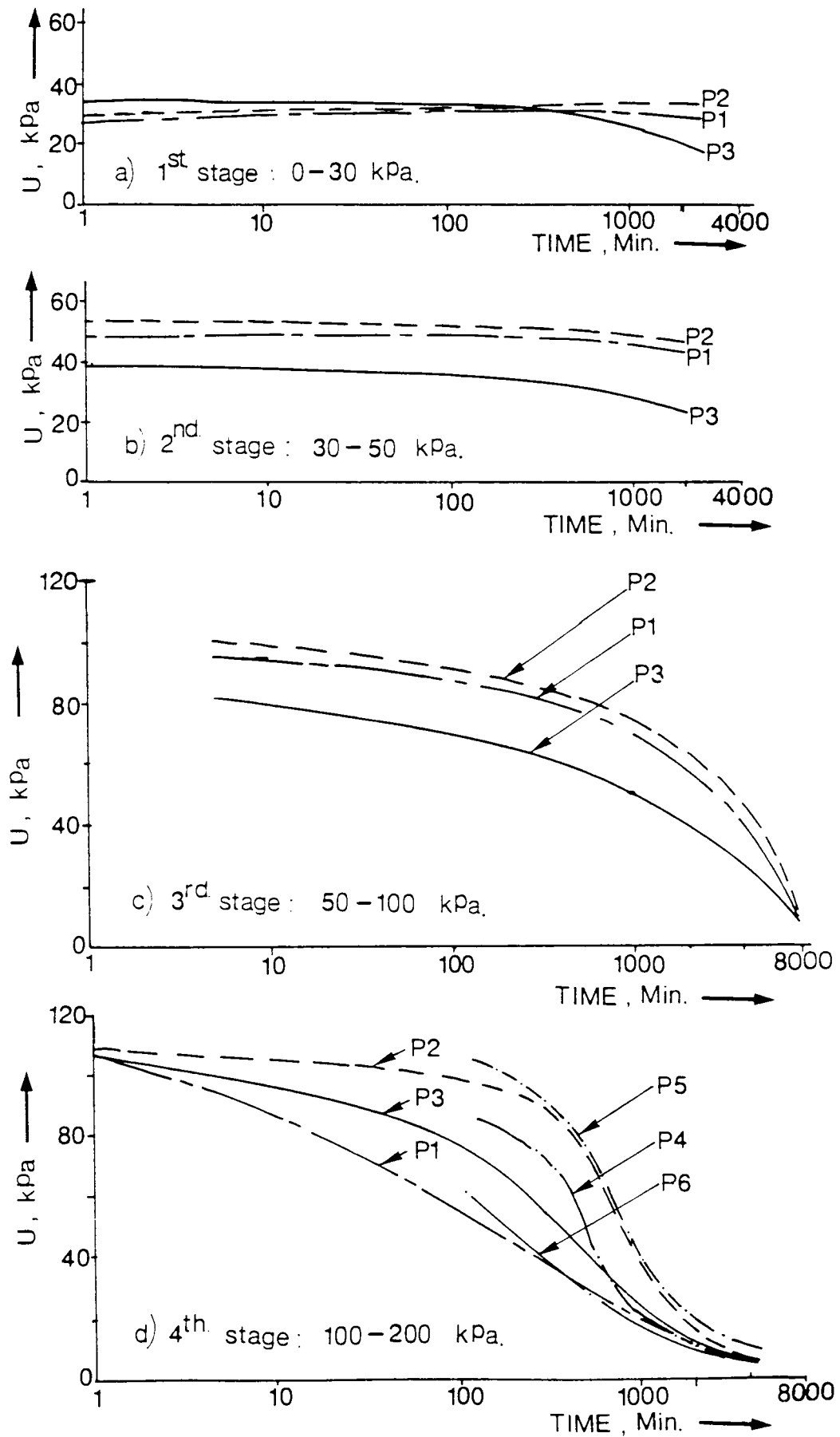


Figure 5.1 Dissipation of pore pressures during consolidation-DC3L

excess pore pressures during consolidation. Similarly Figure 5.2 illustrates the dissipation of excess pore pressures during swelling and after a specimen had been unloaded. The positions of the transducers can be found in Figure 3.13. The specimen underwent a vertical strain of 13.5% during the first stage, but there was generally little dissipation of pore pressures (Figure 5.1a) even after two days except for transducer P3 which was 75 mm away from the porous disc. Figure 5.1b shows an almost similar trend for the second stage i.e. after the specimen had undergone 21% vertical strain. However, in the third and fourth stages (Figures 5.1c and d), there was a distinct dissipation of excess pore pressures immediately after applying the consolidation pressure. It took about six days for the third stage to reach the end of primary consolidation. The fourth and final stage of consolidation exhibited a much quicker response than the third stage. The time to reach the end of primary consolidation was markedly reduced. It appeared that for each stage, the increase in the pore pressure was the same as the increase in the applied pressure. This suggested that the friction losses measured for the hydraulic rams were correct (see appendix A). As expected, pore pressures near the porous plastic disc dissipated quicker than those further away.

To prepare an overconsolidated specimen, the specimen had to be allowed to swell. Figures 5.2a and b show a typical pore

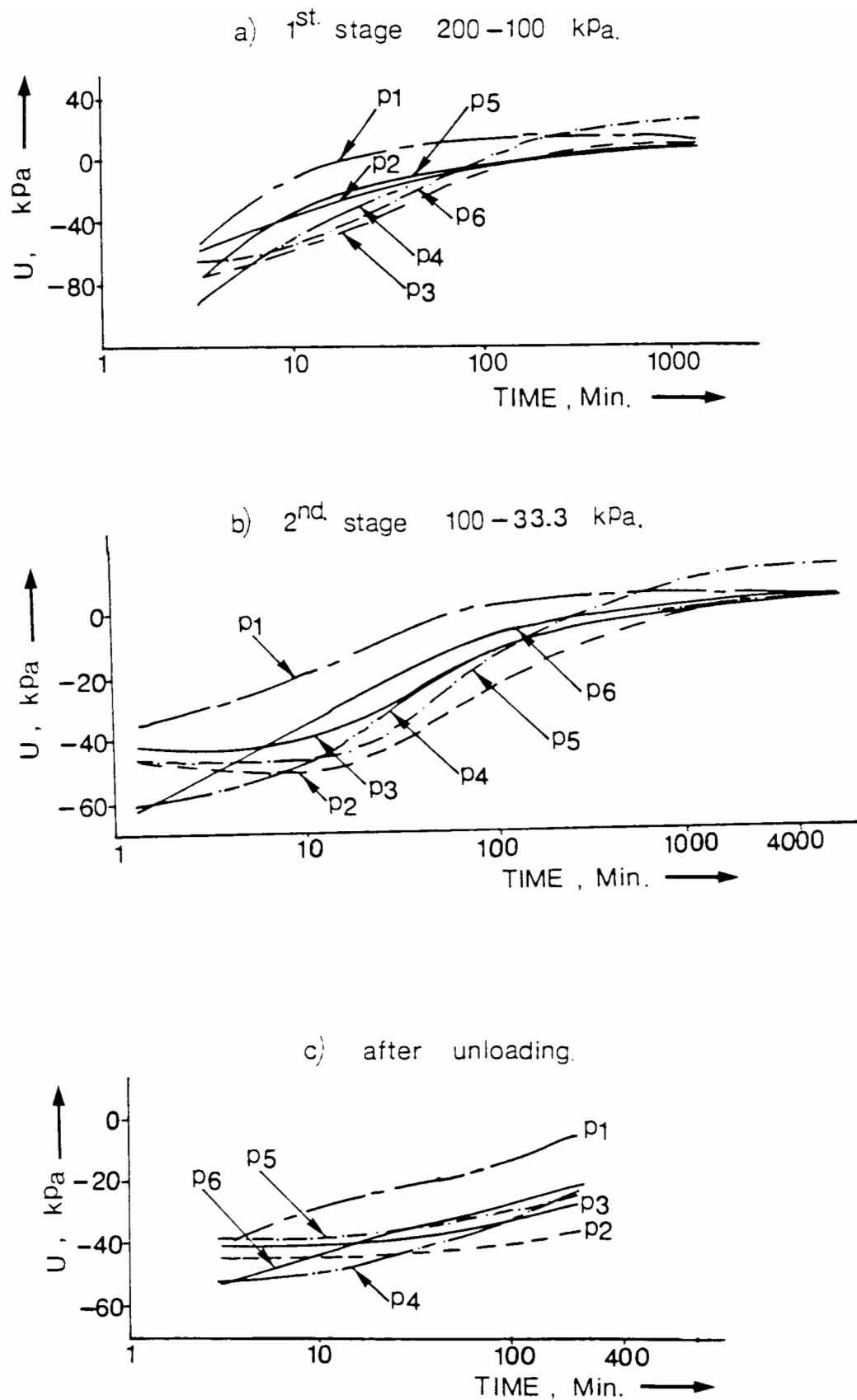


Figure 5.2 Dissipation of pore pressures during swelling and after unloading -DC3L (For the positions of P1 to P6, see Figure 3.13)

water pressure dissipation pattern for the two stages of swelling. The increase in the negative pore pressures corresponded to the decrease in pressure unloaded.

After unloading, each specimen was allowed to swell for a period of three hours with a layer of 5-8 mm of water on the surface. The duration of swelling was a compromise between the time available for testing and the reasonable stable rate of swelling. Figure 5.2c shows that the specimen had negative pore pressures before pile driving and that these negative pore pressures appear to have more scatter at the end of three hours of swelling.

Figure 5.3 shows a typical plot of swelling with time. For clarity, only the results of three transducers are shown. The boundary imposed by the cylinder wall at 28 pile radii had a small effect on the swelling of the specimen, since the specimen swelled less near the wall than at the centre of the specimen. The rates of swelling with time (Figure 5.4) were derived from differentiation of Figure 5.3. The amount of swelling during pile driving could be extrapolated with confidence as the pile was driven within five minutes after the end of swelling. It was found to be about 8×10^{-3} mm/min. before pile driving, giving a total vertical movement of about 0.01 mm whilst one pile was driven. However, the time interval between driving each pile for the study of pile group with the surcharged model

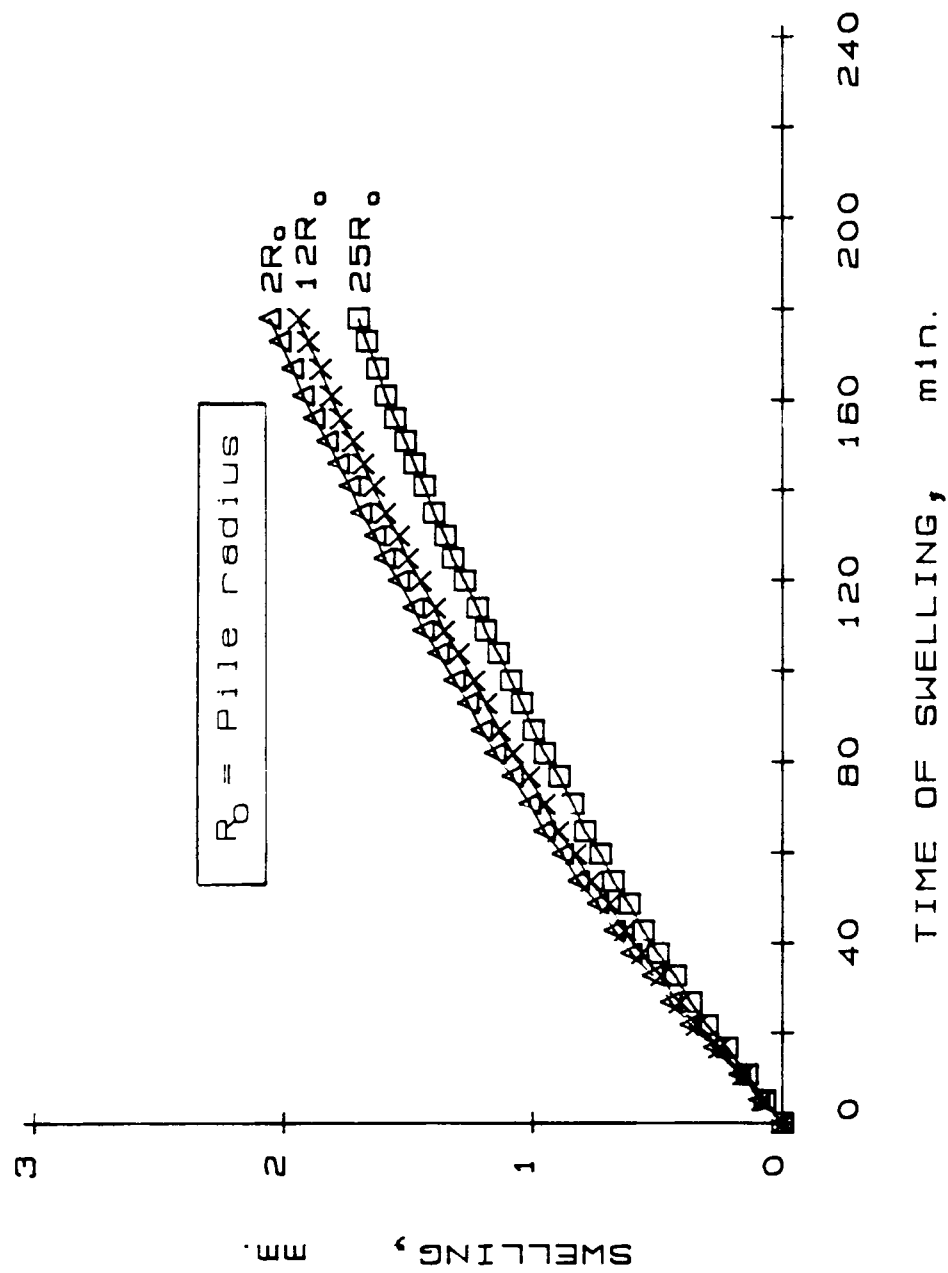


Figure 5.3 Swelling of a specimen before pile driving-C02

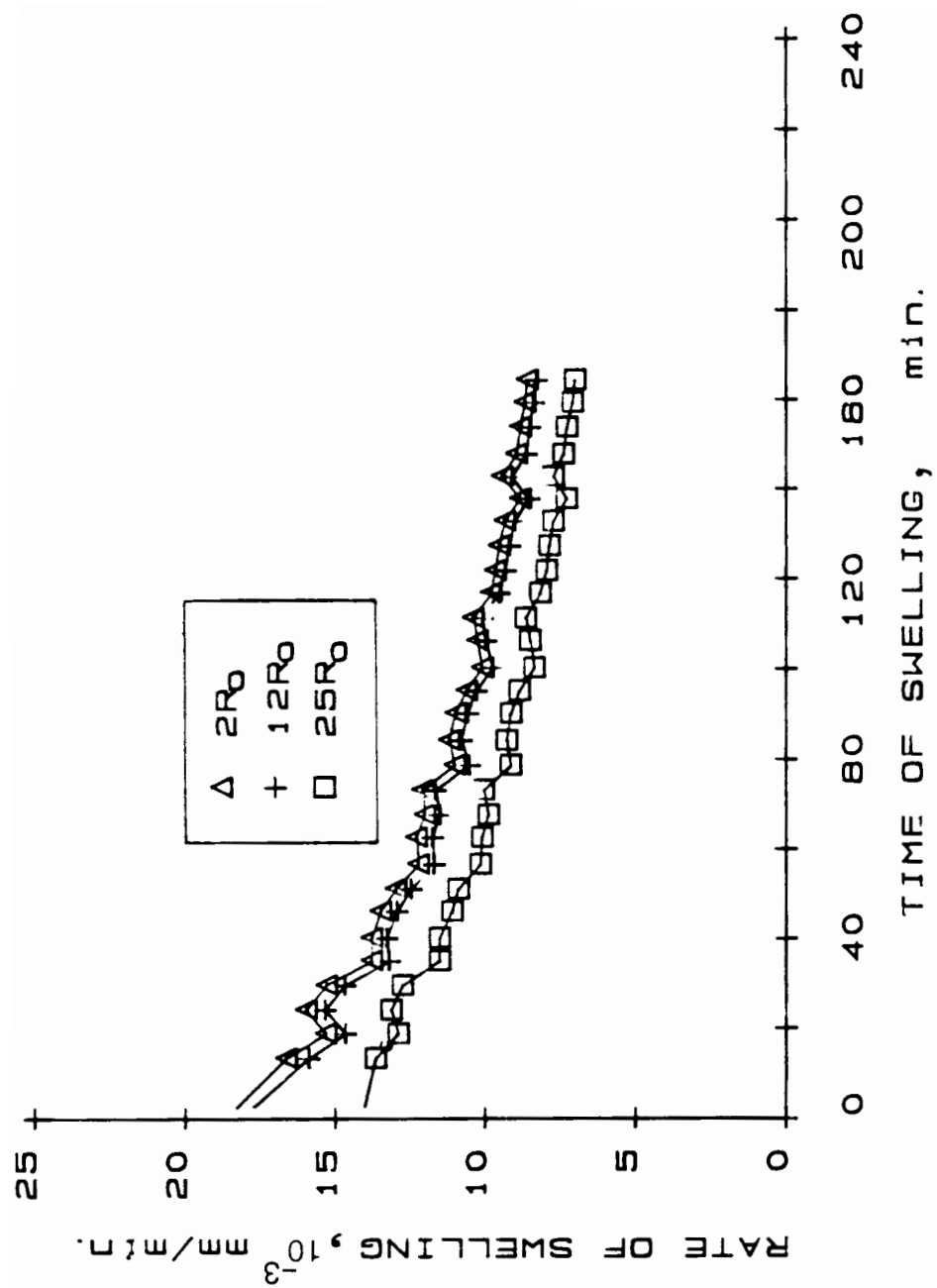


Figure 5.4 Rate of swelling of a specimen before pile driving-7C3

mudmat was about 10-15 minutes. The estimated swelling was deducted from the ground heave measured during driving. Therefore the ground heave presented was that due only to pile driving.

5.3 Miniature Site Investigation

At the end of pile driving, a miniature site investigation was carried out. This included mini plate bearing and laboratory shear vane tests together with moisture content measurements. In some series, samples were also taken for subsequent triaxial tests. The results of the tests are presented below .

5.3.1 Mini Plate Bearing Tests

The purpose of these tests was to find the bearing capacity at the surface of the specimen immediately after pile driving. It was intended to define the surcharge pressure used on the collar and model mudmat as a factor of the bearing capacity. Therefore, the tests were only carried out for those specimens where a surcharged collar or a model mudmat was used. Figure 5.5 shows plate bearing test results for three specimens of one series using a 50 mm diameter plate. There was no clear indication of the

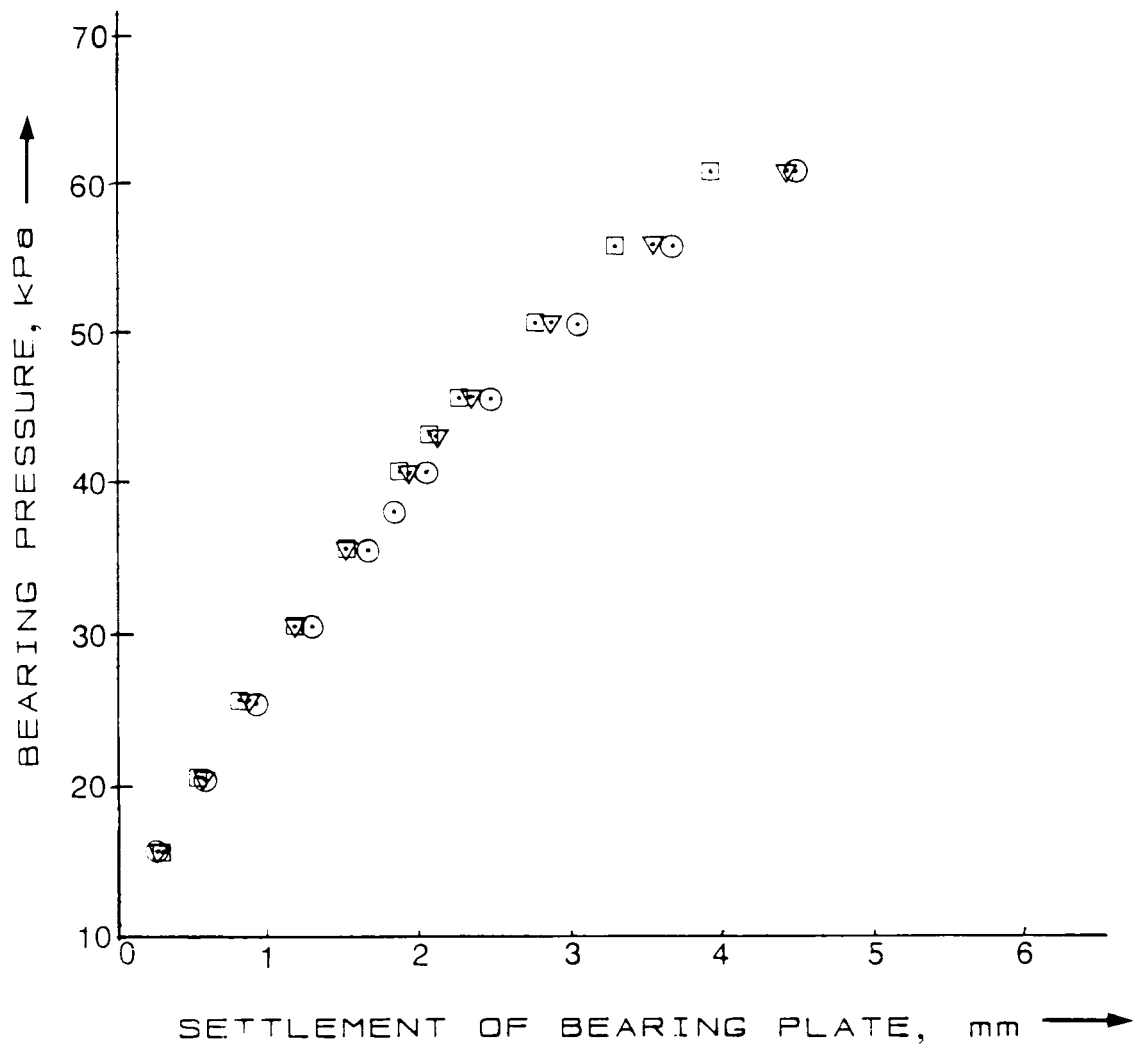


Figure 5.5 The bearing pressure/settlement relationship for three plate bearing tests

bearing capacity as shown by a kink or sudden change in the slope of the curves. The data were replotted on linear-log axes (Figure 5.6), but again there was no conclusive point which indicated the bearing capacity.

For series 15 where three different geometries of plates were used (50 mm and 104 mm diameter bearing plates, and the model mudmat), similar curves were observed even though the

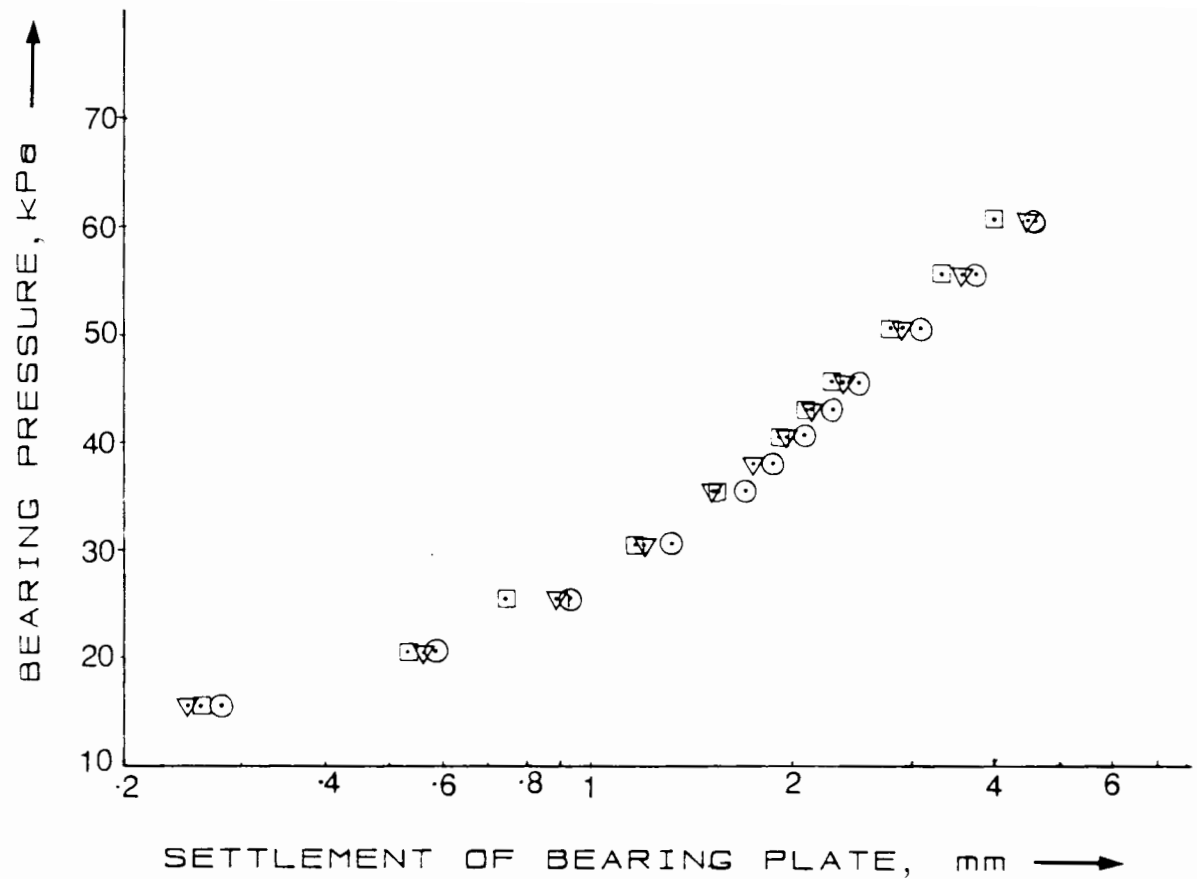


Figure 5.6 The bearing pressure/settlement relationship for three plate bearing tests

shape and size were different (Figure 5.7).

5.3.2 Laboratory Vane Shear Tests

The scatter of undrained shear strengths measured at three depths for each of the three specimens of one series is illustrated in Figure 5.8. The three specimens had been used for three different types of tests: single pile,

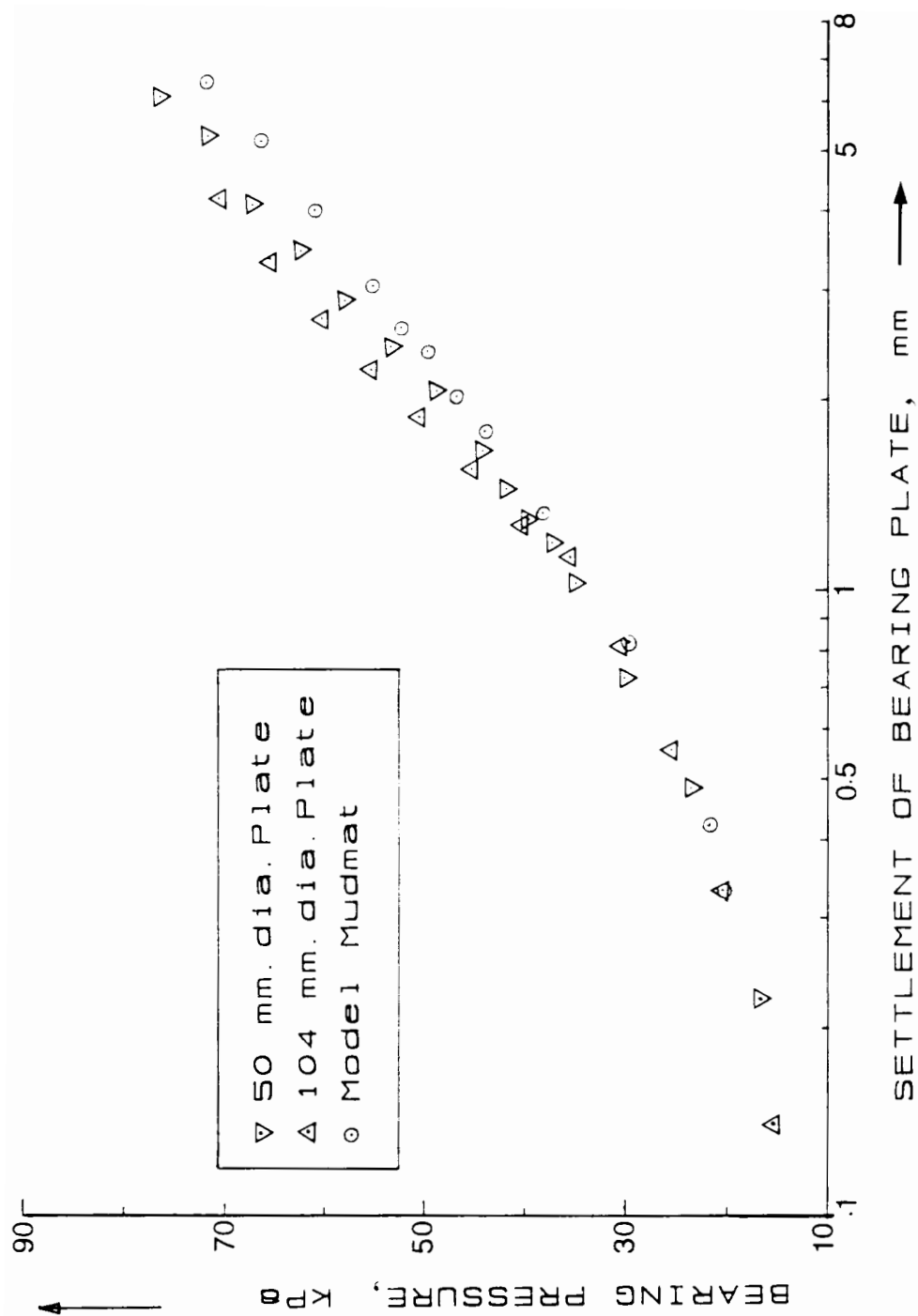


Figure 5.7 The bearing pressure/settlement relationship for three plate bearing tests

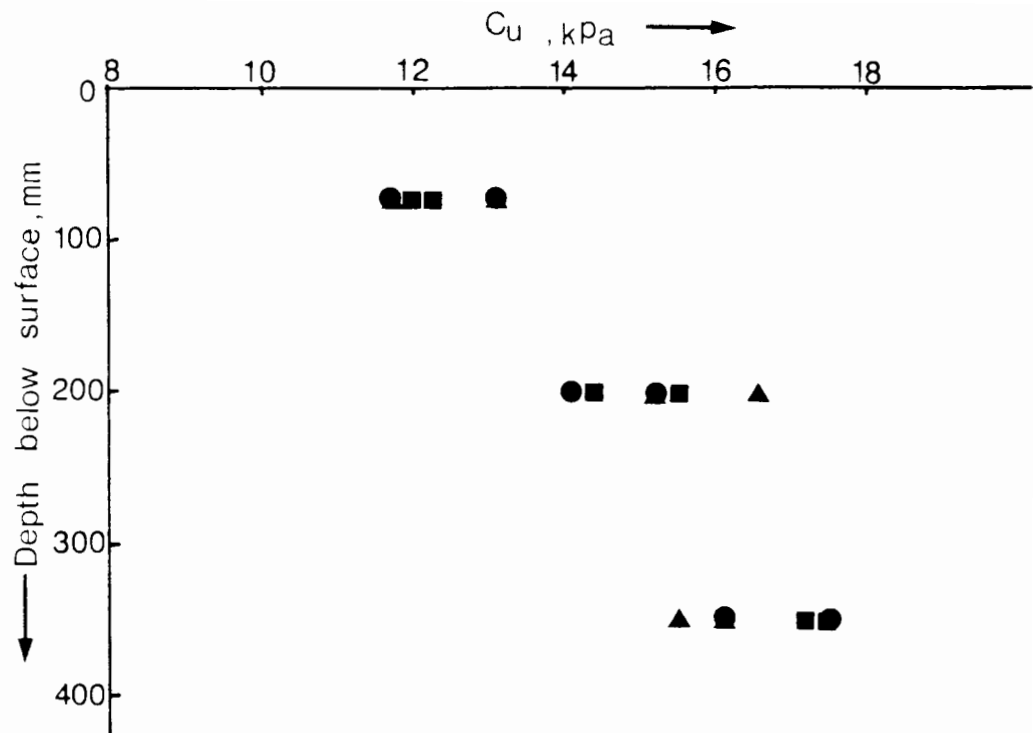


Figure 5.8 Laboratory vane shear test results for Series 13 specimens

single pile with a surcharged collar and pile driving with a surcharged mudmat. The shear strengths increased with depth because the specimens were allowed to swell with an unlimited supply of water at the surface. The undrained shear strength at the surface may be found by extrapolation. The value appears to be about 10 kPa. Therefore, a C_u of 10 kPa will be used as a factor to define the surcharge

pressure on the collar and mudmat. Similar trends but with less scatter were observed for most of the other series for which the same type of tests was carried out on each clay specimen.

The sensitivity obtained from the remoulded shear strengths of the specimens varied between 3.0 to 3.5 and the typical value was 3.2.

5.3.3 Moisture Content of Specimens

The moisture contents at six different depths in the three specimens were determined after placing the moisture content samples in the oven at 107° for 24 hours. A smaller scatter than for the vane shear test was observed (Figure 5.9). There was also a clear pattern, a decrease of moisture content with depth until about 300 mm when it started to increase slightly towards the base of the specimen. This pattern had arisen because the specimens had excess negative pore pressures when they were unloaded and allowed to swell, and so they had sucked in water from the surface and to a lesser extent from the porous disc at the base. Although the bottom drainage was closed immediately before unloading, there was nothing to prevent the specimen from sucking in some water from the disc.

A comparison between the moisture content and shear strength profiles as shown in Figures 5.8 and 5.9 respectively indicates good agreement with the theoretical linear

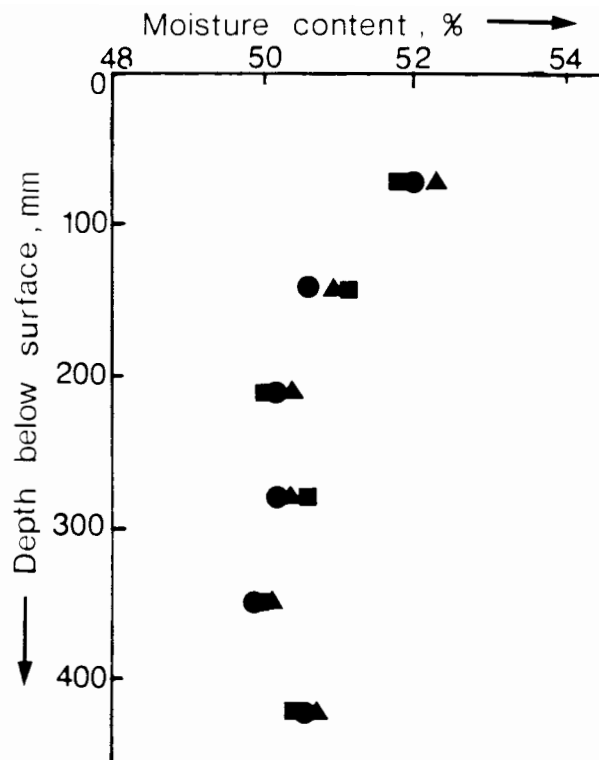


Figure 5.9 Variation of moisture content with depth for Series 13 specimens

relationship between moisture content and the logarithm of shear strength. There was no measurement of shear strength near the base where one might expect the strength to decrease.

5.3.4 Standard Laboratory Tests

Table 5.1 shows the index properties of the clay. They agree well with the values obtained from the literature review. In particular, the specific gravity not only agrees well with the reviewed values but it also corresponds exactly with the supplier's value of 2.60.

TABLE 5.1 INDEX PROPERTIES

PROPERTY	SYMBOL	VALUE
PLASTICITY INDEX	PI	31
PLASTIC LIMIT	PL	34
LIQUID LIMIT	LL	65
SPECIFIC GRAVITY OF SOLIDS	G	2.60

(a) Oedometer tests

The purpose of these tests was to check the values of the clay properties that had been used and also to compare the consolidation properties obtained with those from the consolidometer.

The initial voids ratio of the slurry is the product of the initial moisture content and the specific gravity of the clay. The initial moisture content was obtained with good

accuracy. The percentage error was usually better than 0.25% (expressed as a proportion of the mean moisture content). Therefore the initial voids ratio calculated would be very accurate.

There was some difficulty in measuring the initial height of the slurry accurately, but, the final height of the sample was measured accurately with a Vernier scale. Consequently, the initial height could be deduced from the final height and the known settlement or swelling for each stage. The voids ratio at any other stage of the test was also then calculated from the initial voids ratio and the known settlement or swelling. The final voids ratio was compared with the voids ratio calculated from the final moisture of the sample. The discrepancies for the three tests were 8, 2.6 and 1.1%.

Figure 5.10 shows the consolidation data which agree fairly well with those expected from the reviewed data. The consolidation lines for the three oedometer tests appear to have slightly lower voids ratios for the same pressures. An unexpected behaviour of samples at the initial stages of consolidation was observed. At the initial stages, the state of the samples only began to join the virgin consolidation line when the pressure had increased to about 40 kPa. This behaviour appeared to correspond with the very small dissipation of pore pressures, during the first two stages of consolidation despite a significant amount of vertical strain (see Figures 5.1a and b). The results from

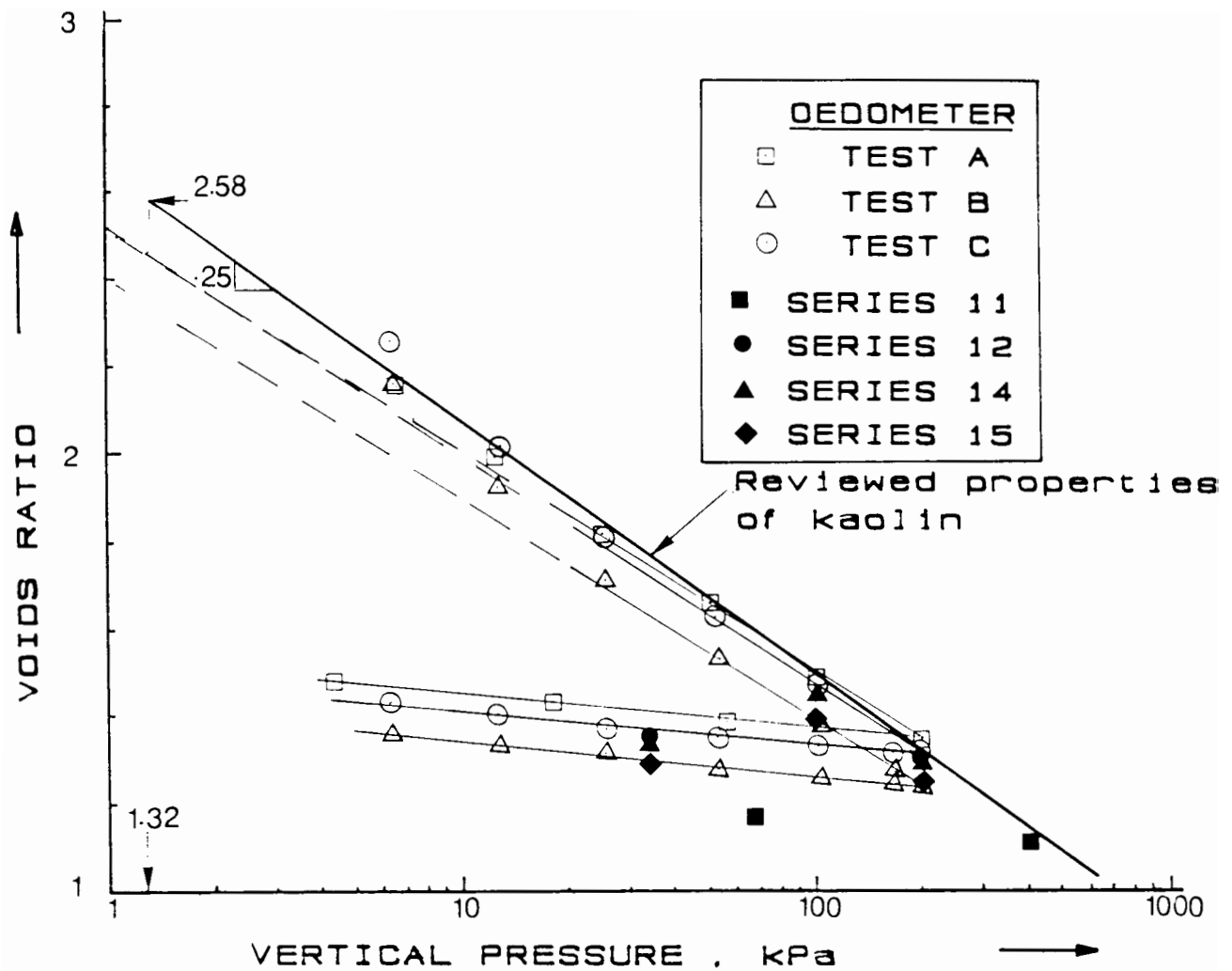


Figure 5.10 Variation of voids ratio with consolidation pressure

the consolidometer (450 mm diameter specimen) were also plotted and they agree well with the results of oedometer tests and the reviewed values.

The coefficient of consolidation, c_v for each stage of consolidation was determined and is shown in Figure 5.11. There appears to be a linear relationship on a linear-log

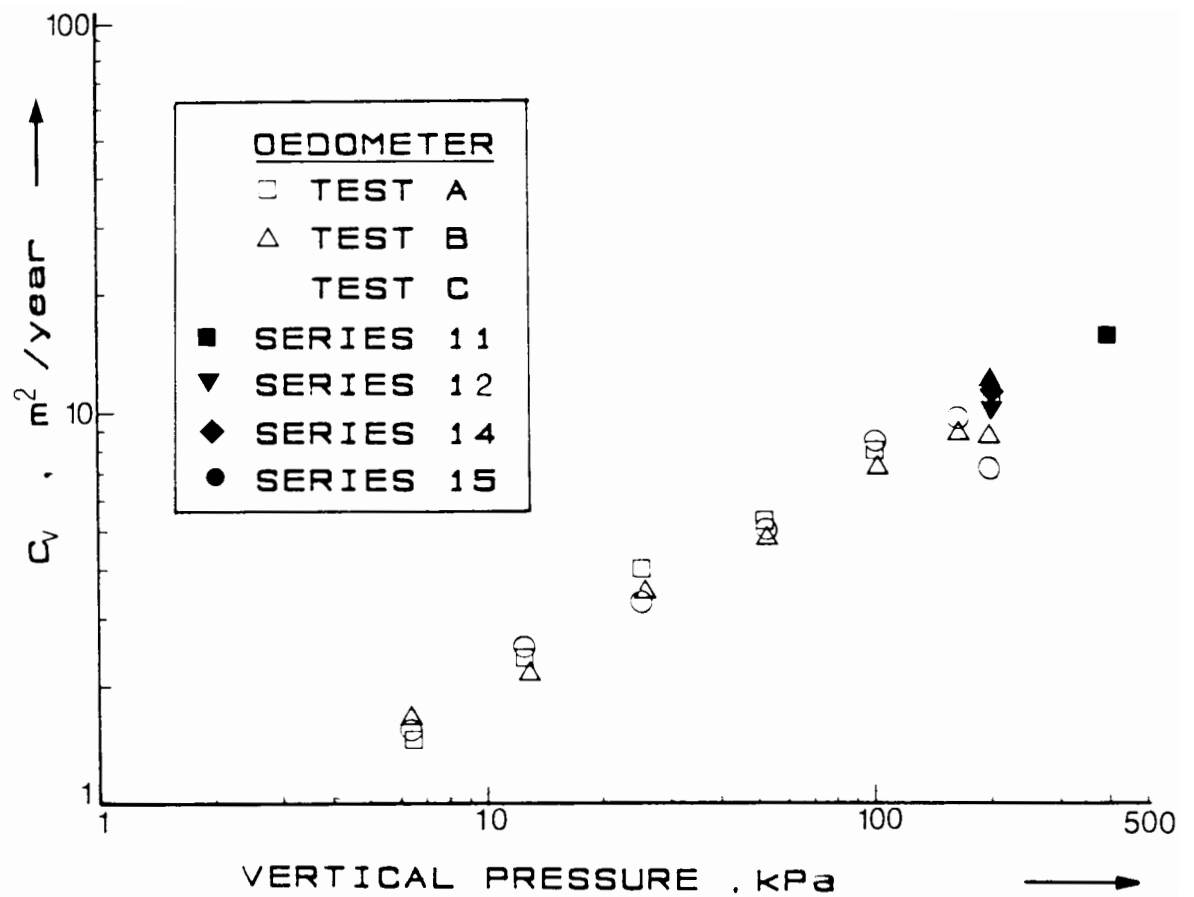


Figure 5.11 Variation of c_v with consolidation pressure

plot with little scatter. The values for the final stage of consolidation in the consolidometer are also plotted and they too agree well.

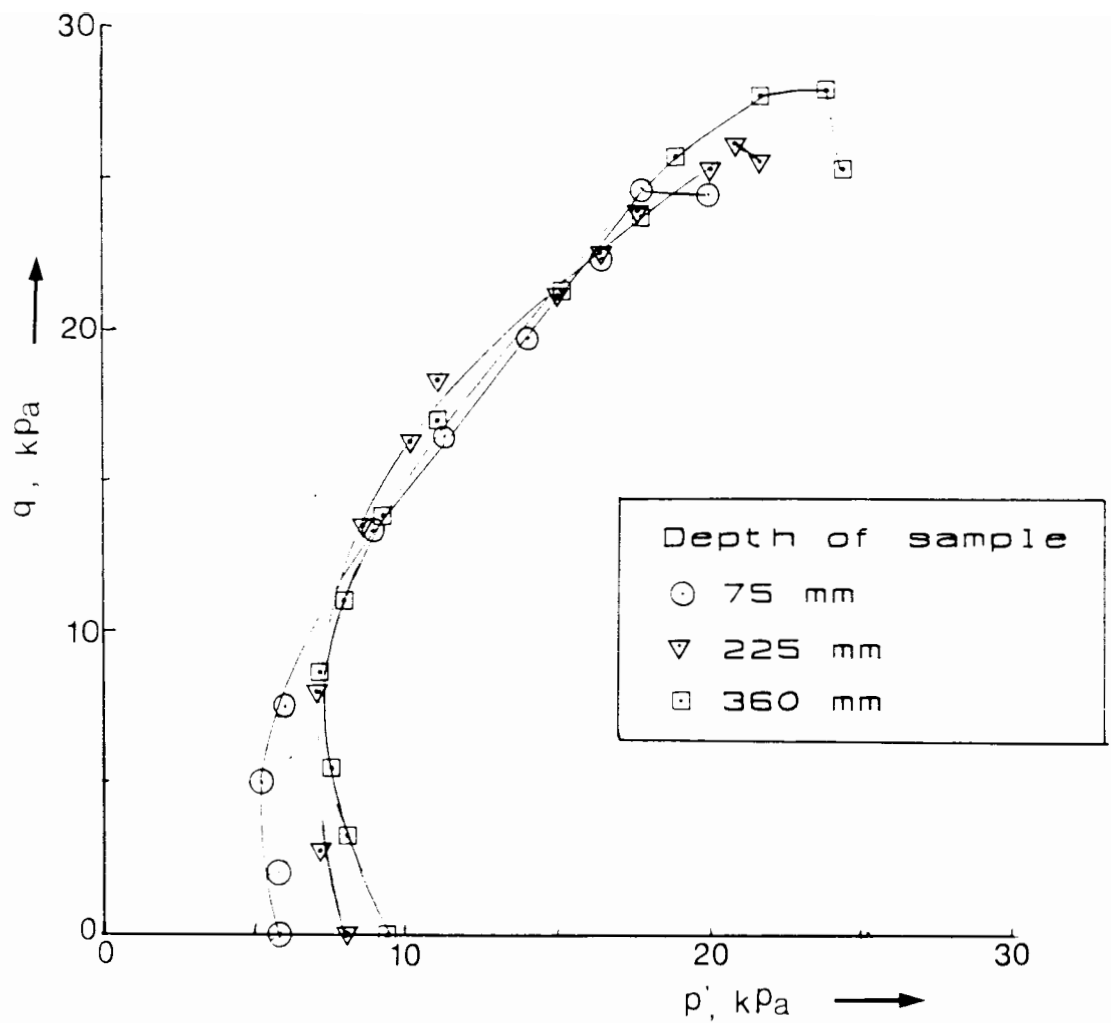


Figure 5.12 Stress paths for three unconsolidated undrained triaxial tests

(b) Triaxial tests

The triaxial test results for the samples taken at three different depths in a specimen have been plotted in Figure 5.12. The stress paths in the deviator and mean effective stress (q and p') space were similar. As mentioned earlier, the specimen had negative pore pressures

before pile driving, and the exact value of p' was not known. The p' before applying any deviator stress was taken as the difference between the cell and pore pressures. By observing the cell and pore pressures whilst the cell pressure was increased by stages to about 300 kPa, it was established that the difference between them was not due to poor saturation of the sample. The difference was constant once the cell pressure had increased beyond about 100 kPa.

Figure 5.12 also shows that the shear strength increased with depth. Figure 5.13 illustrates the variation of excess pore pressure with the vertical strain of the samples. It shows that the maximum positive excess pore pressure appeared to increase with depth. The negative pore pressure also increased with the depth varying between 5.7 kPa for a sample taken at 75 mm below the surface and to 9.3 kPa at 360 mm.

In summary, the properties of the test specimens discussed in this chapter agree well with those determined by other investigators.

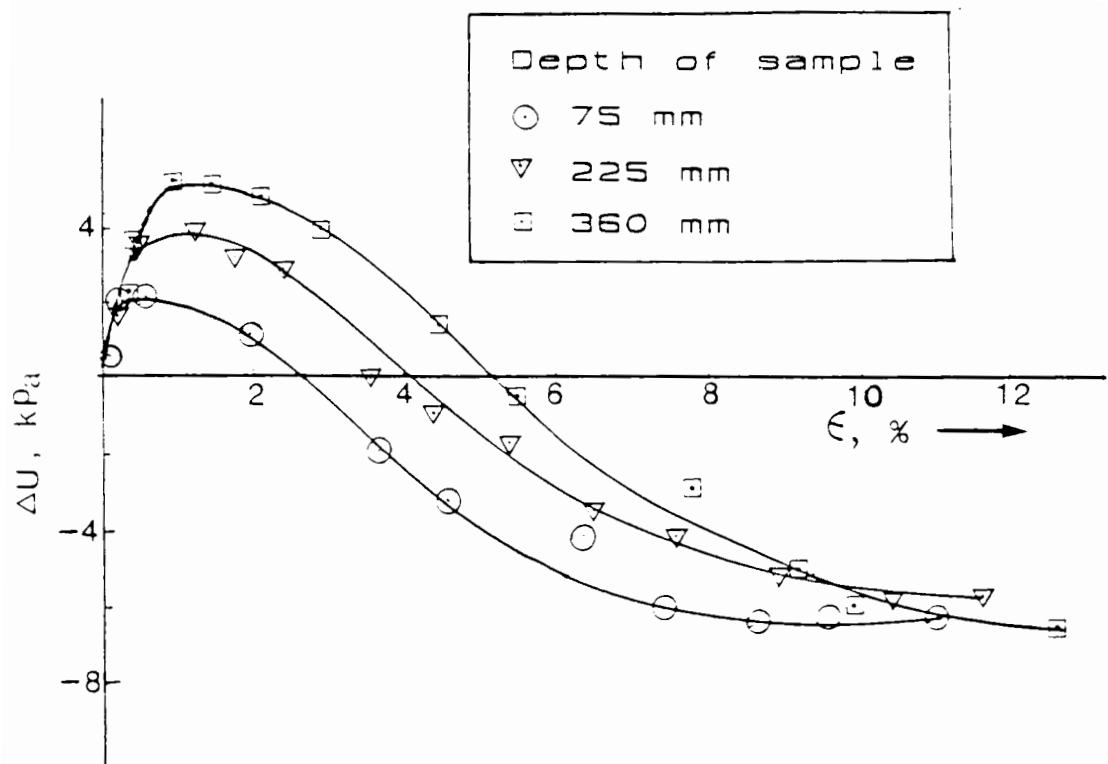


Figure 5.13 Excess pore pressure for three undrained triaxial tests

CHAPTER 6

EXPERIMENTAL RESULTS-SINGLE PILE TESTS

6.1 Introduction

This chapter presents results obtained from the single pile tests with no surcharge pressure, as described in Chapter 4. It consists of two main sections, namely:-

1. Qualitative Semi-cylindrical Pile Tests
2. Quantitative Single Pile Tests

6.2 Qualitative Semi-cylindrical Pile Tests

Table 6.1 shows the summary of the tests. The main purpose of these semi-cylindrical specimens was to examine qualitatively the soil movements around the pile throughout its penetration. The vertical soil movement at the surface could be compared with the ground heave obtained from the full cylindrical specimens, while the radial movement could be compared with other experimental results and a plane strain analysis (Carter, Randolph and Wroth, 1979).

The variation of the length of plugged soil inside an open-ended pile with depth of penetration were also obtained

TABLE 6.1 QUALITATIVE SEMI-CYLINDRICAL PILE TESTS

SERIES	TEST NO.	$\sigma'_{\max}$	OCR	% PLUGGED SOIL	COMMENTS
4	H401	200	7	51	
	H402	200	7	53	not a good sample.
5	H501	200	1	56	
6	H6C1	200	10	-	
8	H8C1	640	6.4	-	
9	H9C1	200	6		
11	HBC1	400	6	-	

NOTE : a) $\sigma'_{\max}$ = maximum vertical consolidation pressure.

b) OCR = Ratio of $\sigma'_{\max}$ to σ'_{final} , final vertical consolidation pressure.

c) Driving :

For all series a 1360 gf hammer, 100 mm drop height and a rate of 80 blows/min. was used except Series 4 for which a 950 gf hammer with the same drop height and rate was used

d) % plugged soil is the proportion of the length of plugged soil to the total length of pile penetration.

from the series of photographs taken during each test. Figure 6.1 shows two photographs taken at an intermediate stage of driving for two tests in Series 4. The length of plugged soil at any stage could be determined to an accuracy of about one half pile radius. Unfortunately, for only shallow penetrations could the location of the pile tip be established to the same degree of accuracy, and for deeper penetrations it could only be located to an accuracy of 1-2 pile radii. Nevertheless, it was possible to determine accurately the final penetration from the scanning record. Figure 6.2 shows the variation of the length of plugged soil with pile penetration. One test series with an OCR=1 has been included on the plot. The theoretical plugged pile line was also plotted, assuming that the pile behaves in a perfectly unplugged manner until the tip resistance (assuming a constant unit area resistance of $9C_u$) equals the internal friction on the pile (assuming unit area skin friction $=\alpha C_u$ and $\alpha = 1.0$). After the pile "plug" is formed there should theoretically be no further increase in the length of the plugged soil. Therefore the theoretical length of plugged soil was 4.5 pile radii. This plot clearly shows that the length of the plugged soil did not reach a constant level but kept increasing with pile penetration.

As the second technique of using lead shot (see chapter 4) was of superior quality to the first, photographs from tests

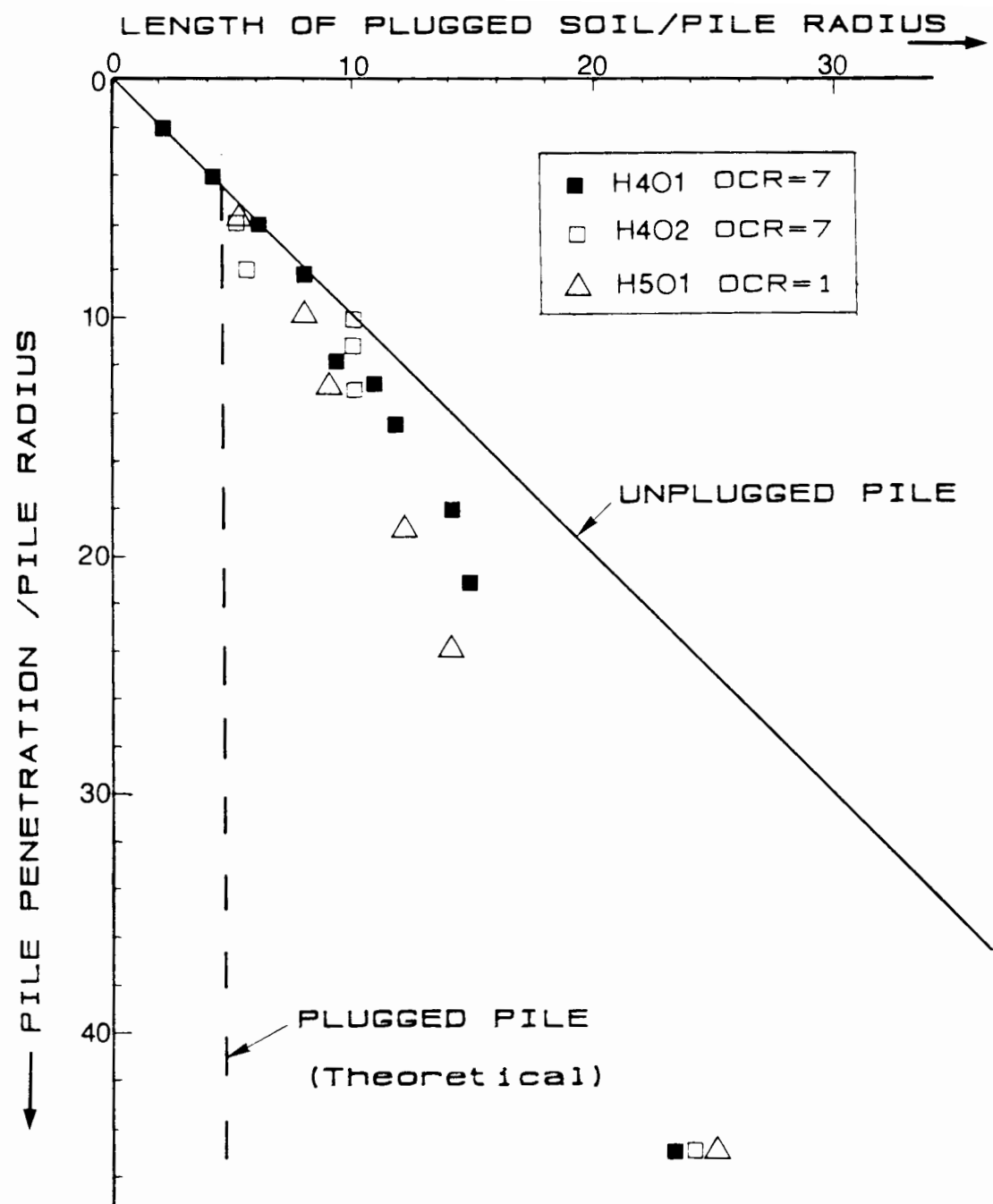


Figure 6.2 Variation of length of plugged soil with pile penetration

using the second technique were used for the analysis of soil movements around the pile. This analysis was performed only for closed-ended piles since the movements generated by open-ended piles were small. A digitizer was used to record the positions of the lead shot and the reference markers either from an enlarged print or from a projected image. A projector fixed above the active area was used for enlarging the film directly onto the digitizer. The active area of the digitizer is 280 mm square with a resolution of 0.1 mm.

The coordinates of the lead shot were converted into digital form and stored on a micro-computer disk. They were later transferred onto the main frame computer for strain and displacement analyses using the program written by Houlsby (1984).

Although a cross-hair sight was fitted to the digitizing cursor to locate accurately the centre of each of the lead shot or the reference markers, there was always some degree of human error. This error was minimised by digitizing the reference markers three times and the lead shot twice. The average values were then used for the analyses. Corrections for the distortion due to photographing, enlarging and printing were taken into account by incorporating the true coordinates of the six reference markers with respect to one of them which was used as an origin. The movement of each

lead shot was small with relative to the grid size. The results were further analysed and the movements were plotted with an exaggerated vertical axis and a reduced horizontal axis.

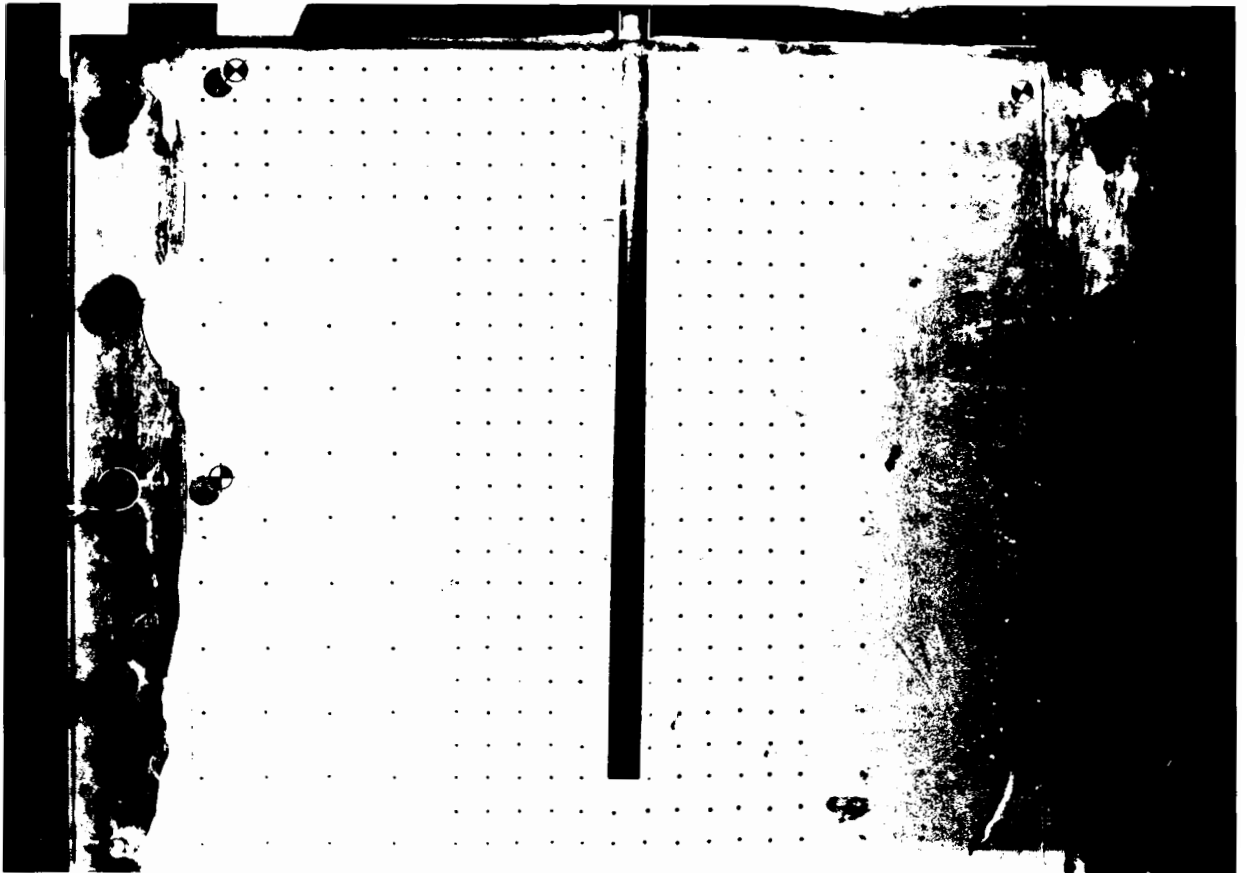


Figure 6.3 Photograph after driving a closed-ended pile

The photographs taken before and after pile driving were investigated and described. Figure 6.3 shows a photograph after driving a closed-ended pile into a semi-cylindrical specimen. The soil movements were examined in three regions

around the pile, as shown in Figure 6.4. The theoretical line based on a plane strain analysis was plotted for comparison with the experimental results. The movement of each of the individual lead shot to the right of the pile on the soil face has been plotted. For clarity, the individual points for the shot to the left of the pile have been omitted. Their movements demonstrated similar scatter and so it has been possible to draw an average line representing the results from these shot. It appears that the radial movements at the mid-depth of the driven pile have the best agreement with the theoretical line. The maximum radial movements near the surface and pile tip regions were markedly reduced; less than half of that at the mid section. However, the radial movements at greater radial distances from the pile appear to be less affected because of the small movements involved.

The vertical movements in the same three regions are plotted in Figure 6.5. The results were compared with the final ground heave profile of Figure 6.13. As above, only an average line for shot to the left of the pile is presented. The upward vertical movements near the surface region agree very well with the results from the full cylindrical specimen. At the mid-section and pile tip, there were downward vertical movements close to the the pile but at greater radial distances from the pile, slight upward movements were observed.

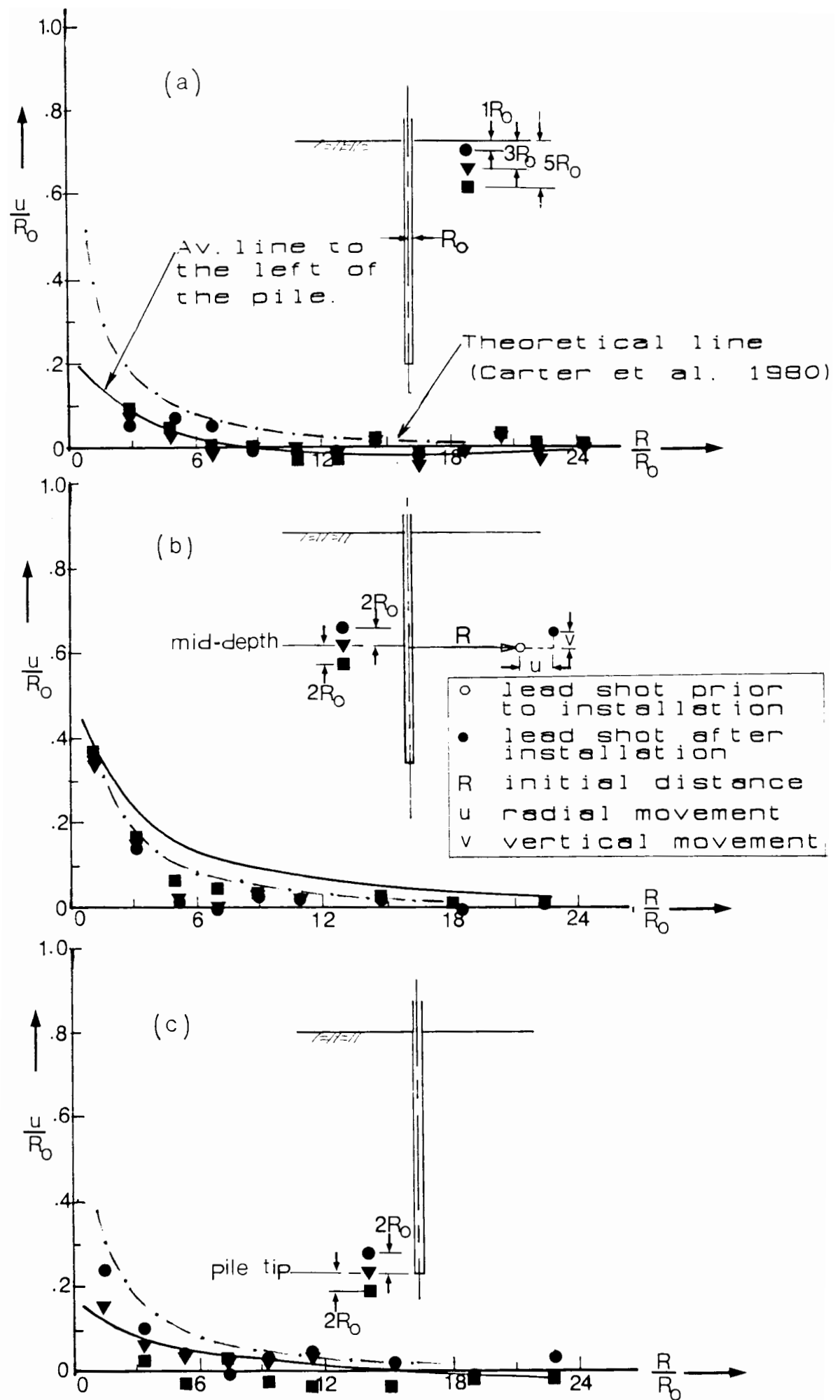


Figure 6.4 Radial soil movements around a driven pile

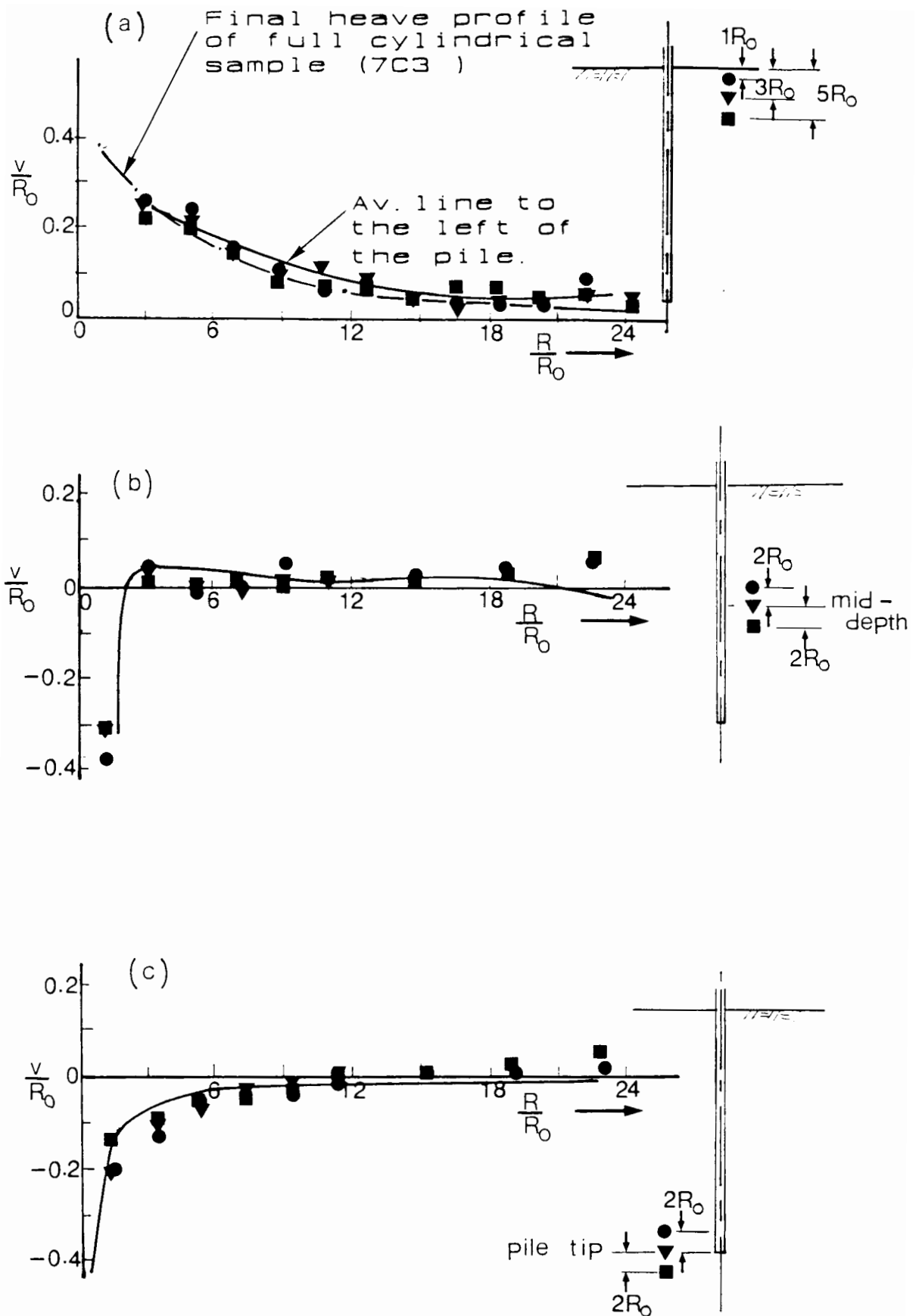


Figure 6.5 Vertical soil movements around a driven pile

6.3 Quantitative Single Pile Tests

A total of 17 series of specimens were prepared and tested. The first three series of specimens were used for a preliminary study and results from them are not presented because of experimental difficulties that were experienced. The summaries of the tests are shown in Table 6.2.

This section is divided into six sub-sections:-

1. Pile driving
2. Ground heave
3. Interpretation of volume of ground heave
4. Effect of driving shoe for open-ended piles on ground heave.
5. Effect of rate of driving on the ground heave
6. Effect of stress history on the ground heave

Only closed-ended piles were used to investigate the effects of the rate of driving, since they generated more ground heave. For all other investigations, both closed- and open-ended piles were used.

TABLE 6.2 QUANTITATIVE SINGLE PILE TESTS

SERIES	TEST NO.	$\sigma'_{\max}$	OCR	% PLUGGED SOIL	COMMENTS
4	4C1	200	7	-	Lighter hammer
	4O2	200	7	51	
5	5C1	200	1	-	
	5C2	200	1	-	
6	6C1	200	10	-	
	6C2	200	10	-	
7	7C1	200	6	-	
	7O2	200	6	69	P & G
	7C3	200	6	-	
8	8C1	640	6.4	-	
	8O2	640	6.4	68	P & G
11	BC1	400	6	-	
	BO2	400	6	70	P & G
15	FO3	200	6	102	P, G & S
16	GC1	200	6	-	22 Blows/min.
17	HC3	200	6	-	150 Blows/min.

NOTE : a) $\sigma'_{\max}$ = maximum vertical consolidation pressure.

b) OCR = Ratio of $\sigma'_{\max}$ to σ'_{final} ,
final vertical consolidation pressure.

c) P, G & S represent the internal conditions of an open-ended piles:-

P polished

G prepared with silicon grease and a light spray of grease

S pile shoe

d) Driving :

i) Series 4, a 950 gf hammer, 100 mm drop height and rate of 80 blows/min. was used.

ii) Series 5 to 15, a 1360 gf hammer, 100 mm drop height and rate of 80 blows/min. was used.

iii) Series 16 and 17 were same as ii) except the rates for series 16 and 17 were 22 and 150 blows /min. respectively.

e) % plugged soil is the proportion of the length of plugged soil to the total length of pile penetration.

6.3.1 Pile Driving

Table 6.2 shows the summary of the tests. All the open-ended piles were driven without driving shoes except for Series 15 where driving shoes were used. The inner walls of the open-ended piles were prepared with a thin layer of silicon grease followed by a spray of ordinary light grease.

In Series 4, the piles were driven by a 950 gf. hammer dropped through a height of 100 mm. However, for all subsequent tests on both closed- and open-ended piles, the hammer weight was increased to 1360 gf.. A driving rate of 80 blows/min. was generally used, except for tests GC1 and HC3 where rates of 22 and 150 blows/min. respectively were used.

Figure 6.6 shows a typical penetration/time plot for a closed-ended pile. There was a delay between when scanning started and switching on the motor for pile driving. The blow count/penetration plot shown in Figure 6.7 was derived from a differentiation of Figure 6.6. Figure 6.8 shows a similar plot for an open-ended pile. Both plots show that the blow count increased almost linearly with pile penetration. It appears that for open-ended pile, the blow count/penetration relationship can be extrapolated back approximately to the origin, whereas for the closed-ended pile there would clearly be an initial blow count at zero

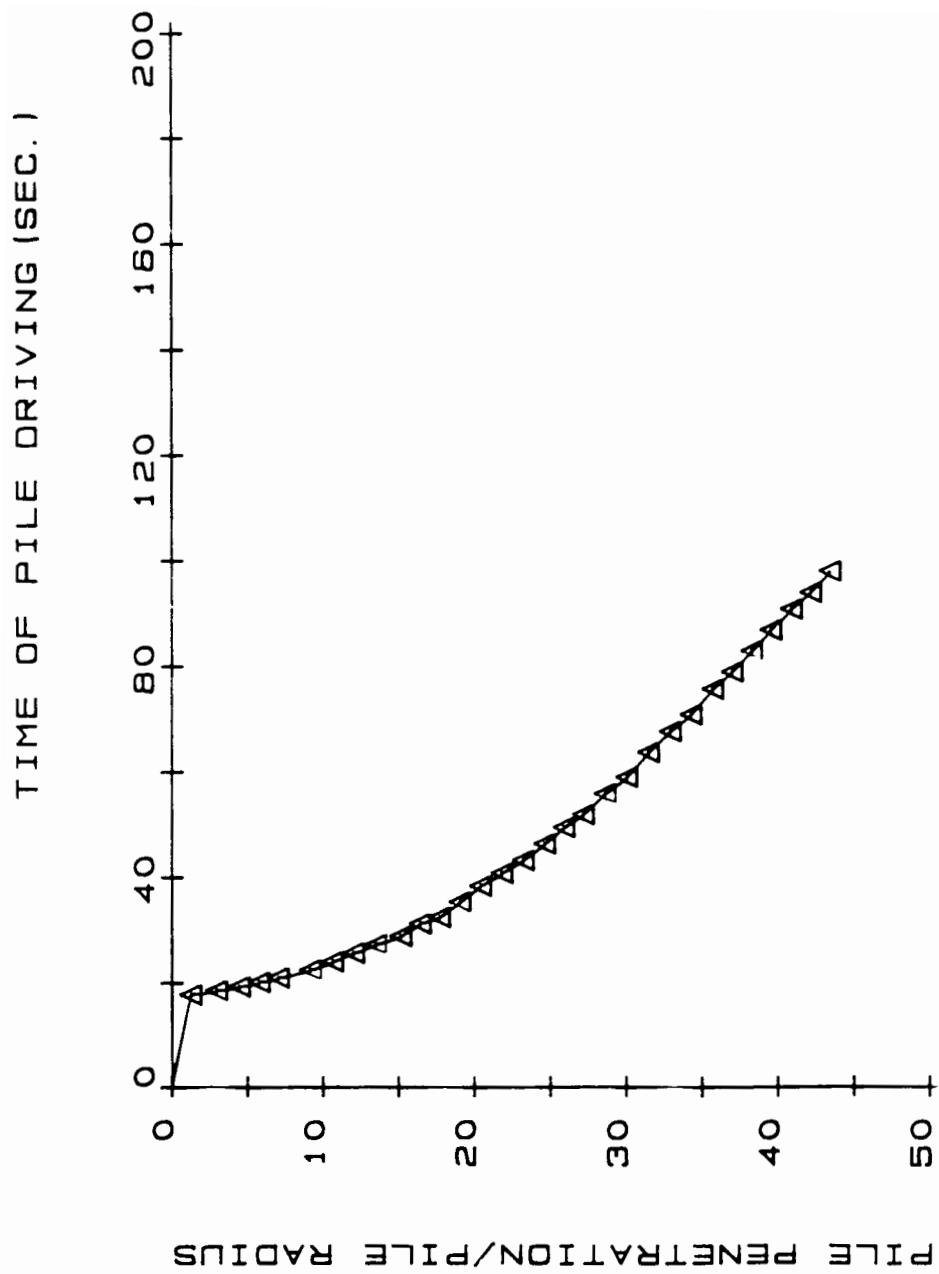


Figure 6.6 Pile penetration/time record-7C3

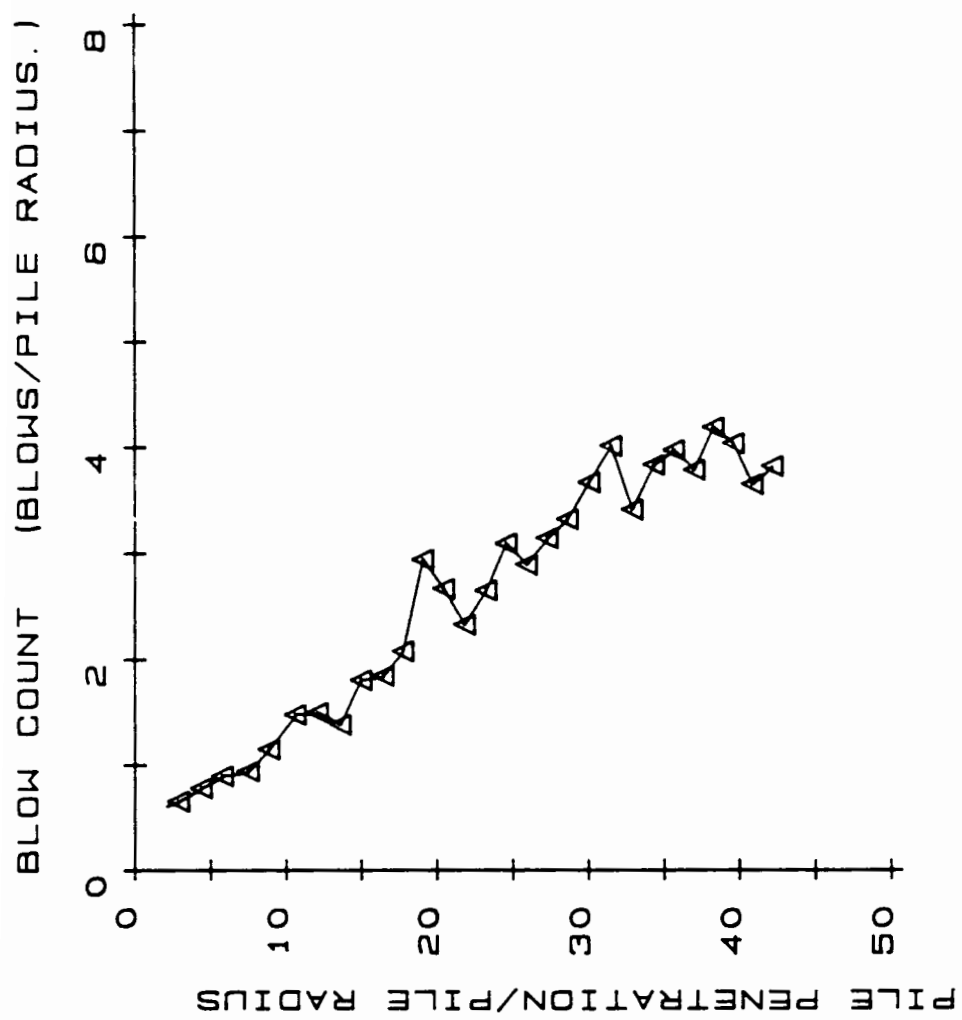


Figure 6.7 Blow count/penetration record of a closed-ended pile-7C3

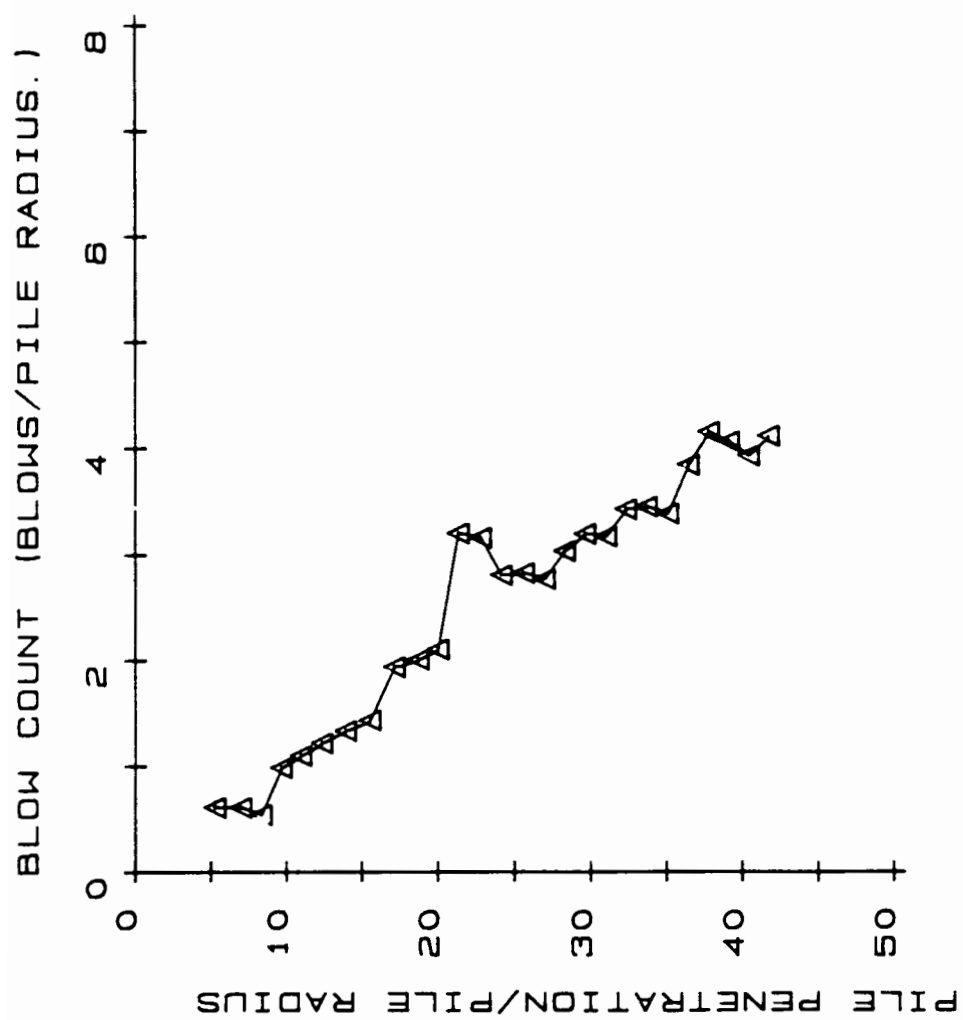


Figure 6.8 Blow count/penetration record of an open-ended pile-F03

penetration. This agreed with the fact that the initial base resistance of a closed-ended pile is significant but that of an open-ended pile is not.

Driving shoes appeared to have no effect on the blow count. However, later in this section it will be shown that driving shoes had a dramatic effect on the volume of ground heave generated.

The blow count/penetration plots for closed- and open-ended piles driven into identical specimens are compared and shown in Figure 6.9. Three pairs of specimens were prepared with differing stress histories (various maximum vertical consolidation pressures). It appears that in this laboratory investigation open-ended piles were slightly easier to drive than closed-ended piles.

One series of the tests used closed-ended piles driven at various rates into specimens prepared with the same history. As expected, the faster the rate of driving, the easier was the driving (Figure 6.10).

Additional information on the excess pore pressures generated by pile driving for closed- and open-ended piles (Figures 6.11 and 6.12) was obtained in tests DC3L and E03L

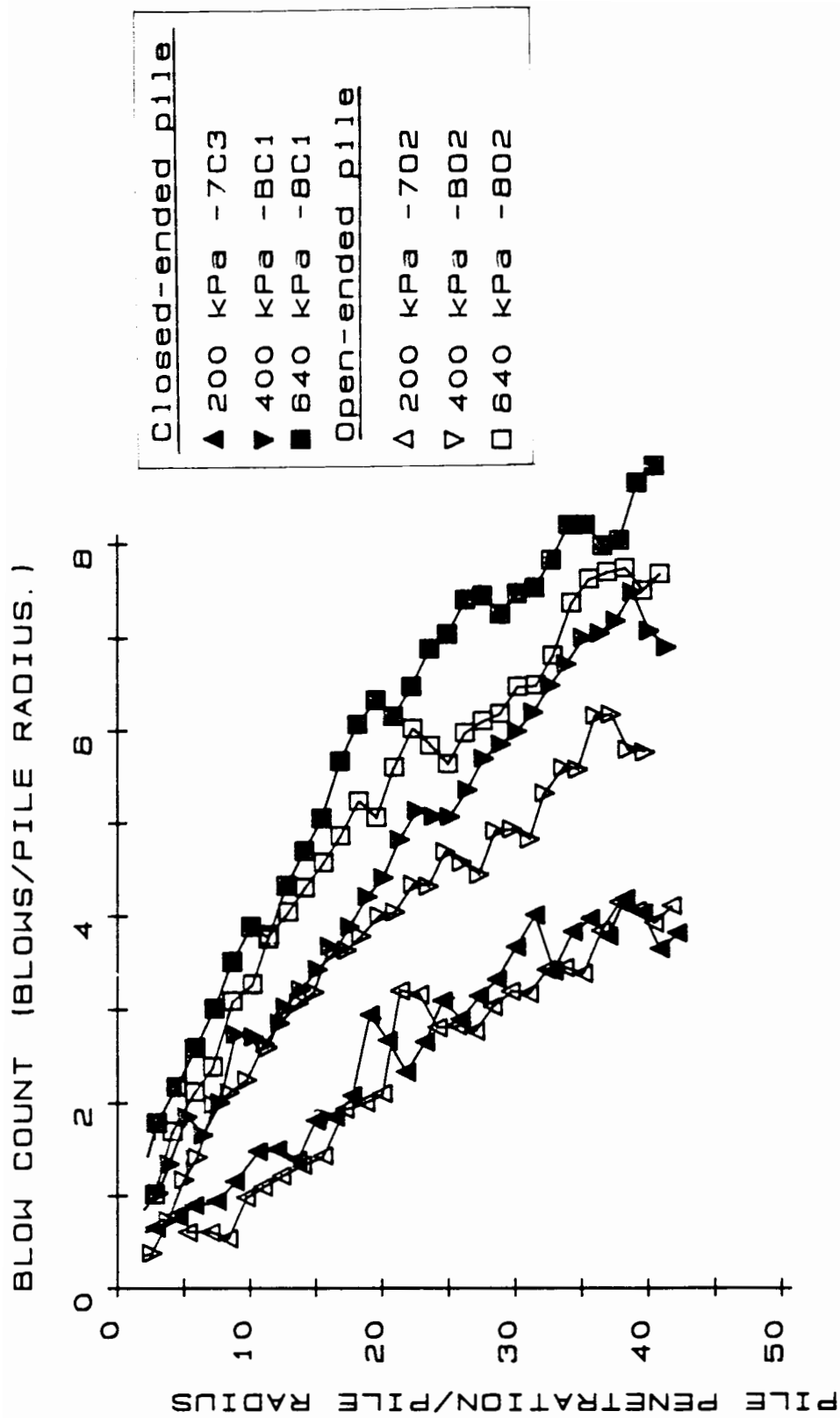


Figure 6.9 Comparison of blow count/penetration records of closed- and open-ended piles for different maximum vertical consolidation pressures

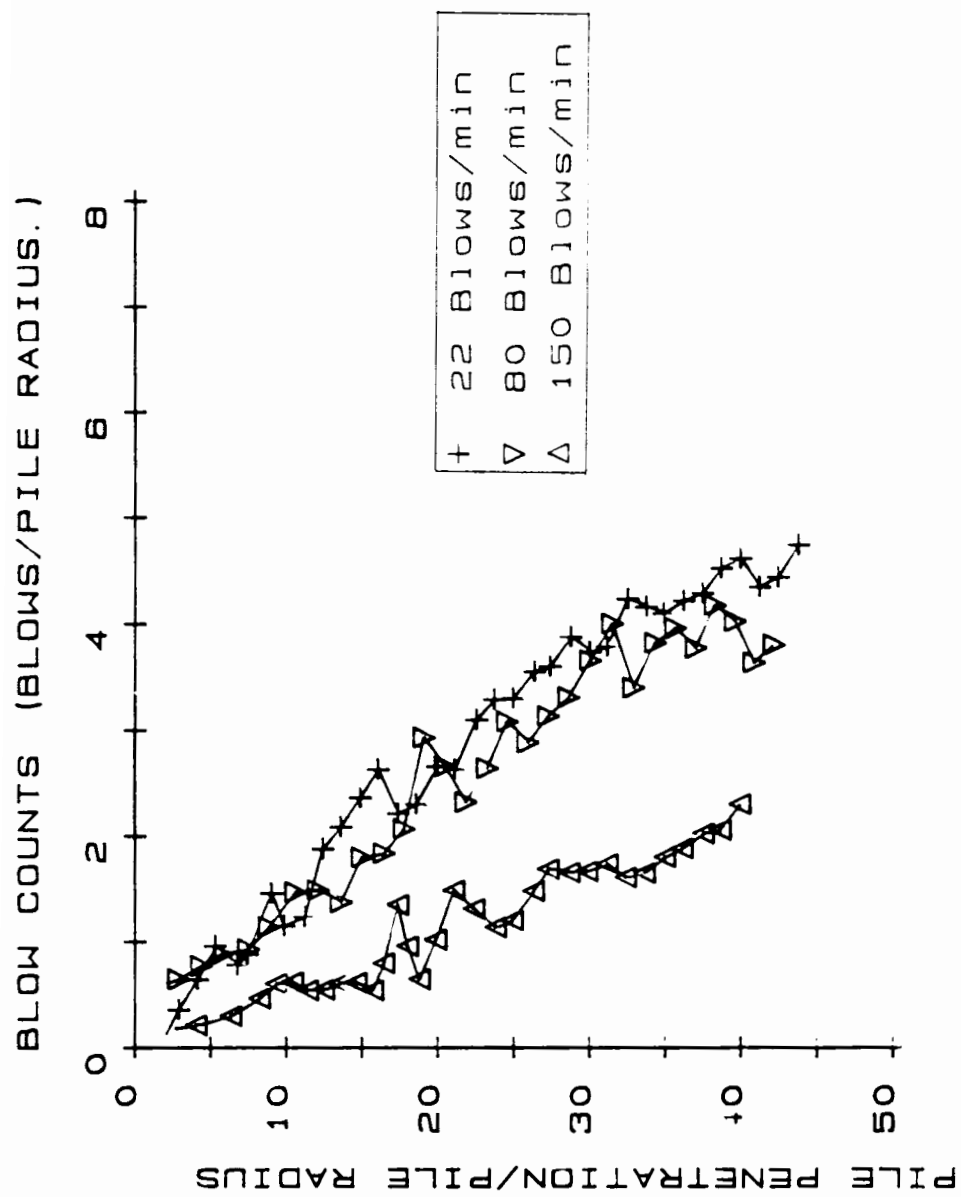


Figure 6.10 Blow count/penetration records for three different rates of driving closed-ended piles.

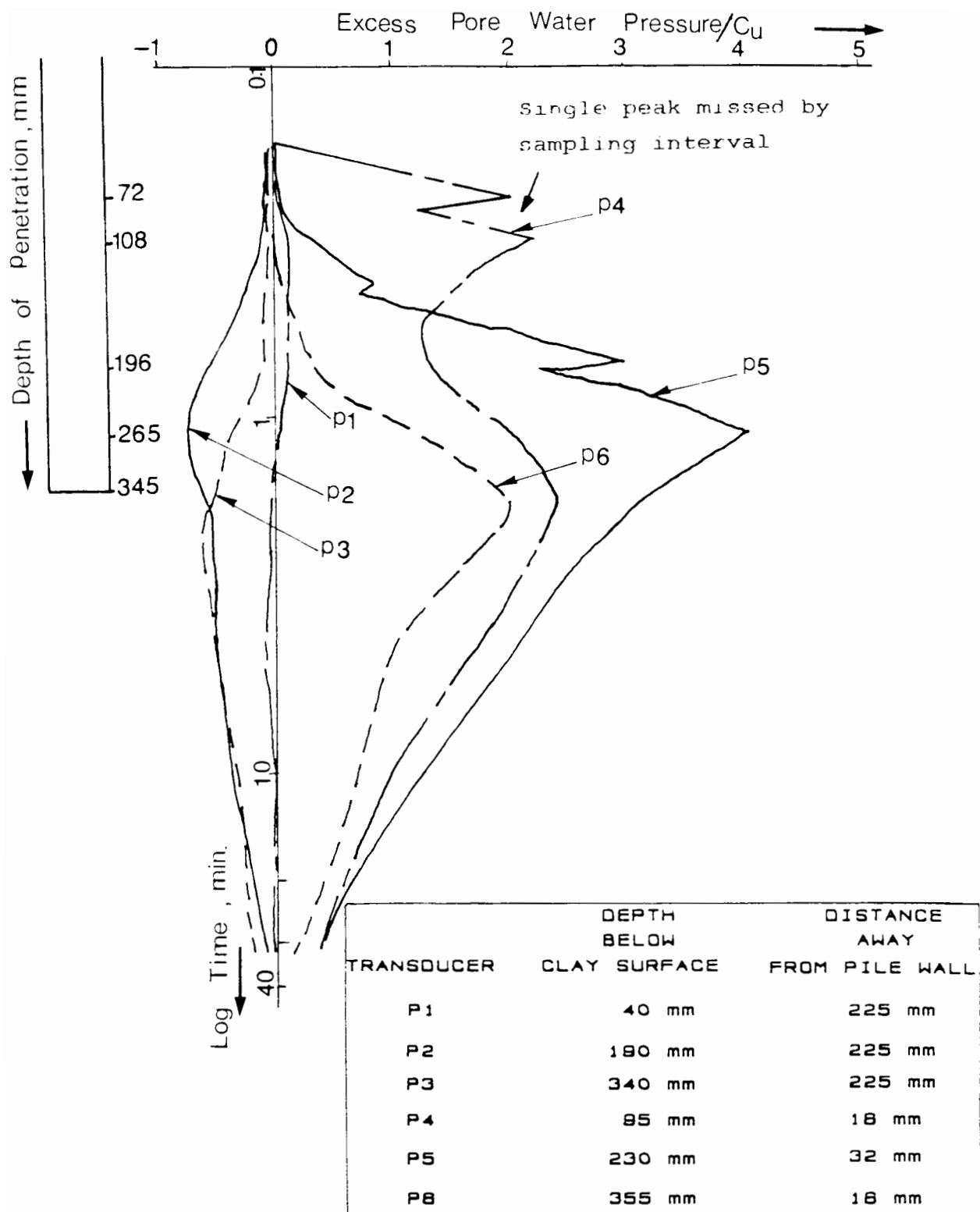


Figure 6.11 Excess pore pressures generated by driving a closed-ended pile-DC3L

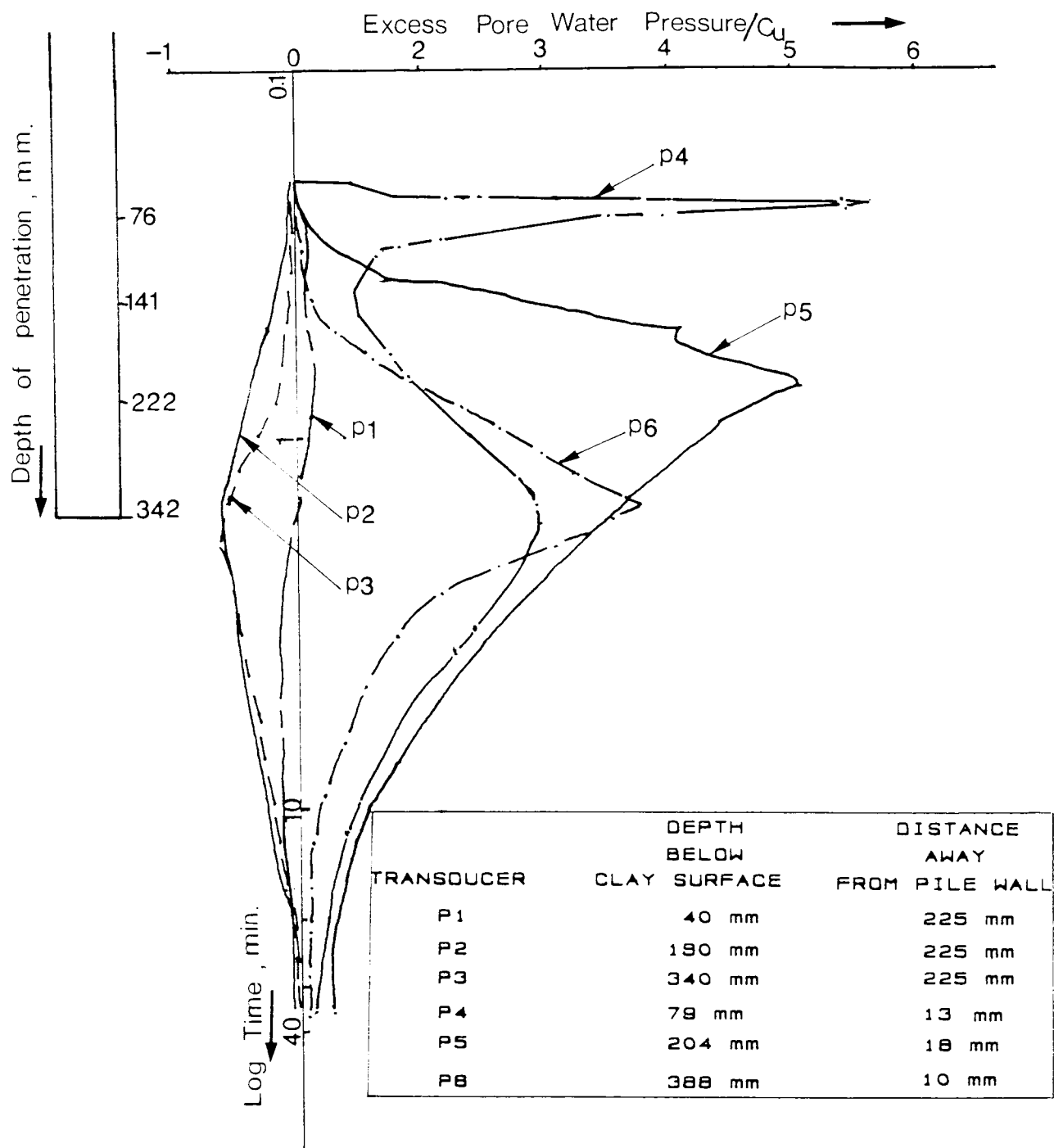


Figure 6.12 Excess pore pressures generated by driving an open-ended pile-E03L

respectively. Excess pore pressure measurements were continued for 35 minutes after the end of pile driving. The depths of penetration with the corresponding time are also plotted on the figures. The positions of these transducers can be found in Figure 3.13. It appeared that the peak pore pressure readings occurred when the pile tip was driven to about the position of each transducer. Both closed- and open-ended piles generated a maximum excess pore pressure of about $4C_u$. For both tests, the excess pore pressure of P4 for both tests appeared to have two pronounced peaks; one immediately after the pile had been driven beyond the transducer and the other one near the full pile penetration. Pore pressure transducers located at the cylinder wall indicated significant negative excess pore pressure of about $1.0C_u$ for the closed-ended pile and more than $0.5C_u$ for the open-ended pile.

6.3.2 Ground Heave

The outputs from the LVDTs were scanned after each 10 mm of penetration until a predetermined depth of 43 pile radii. Therefore each profile in Figure 6.13 represents the

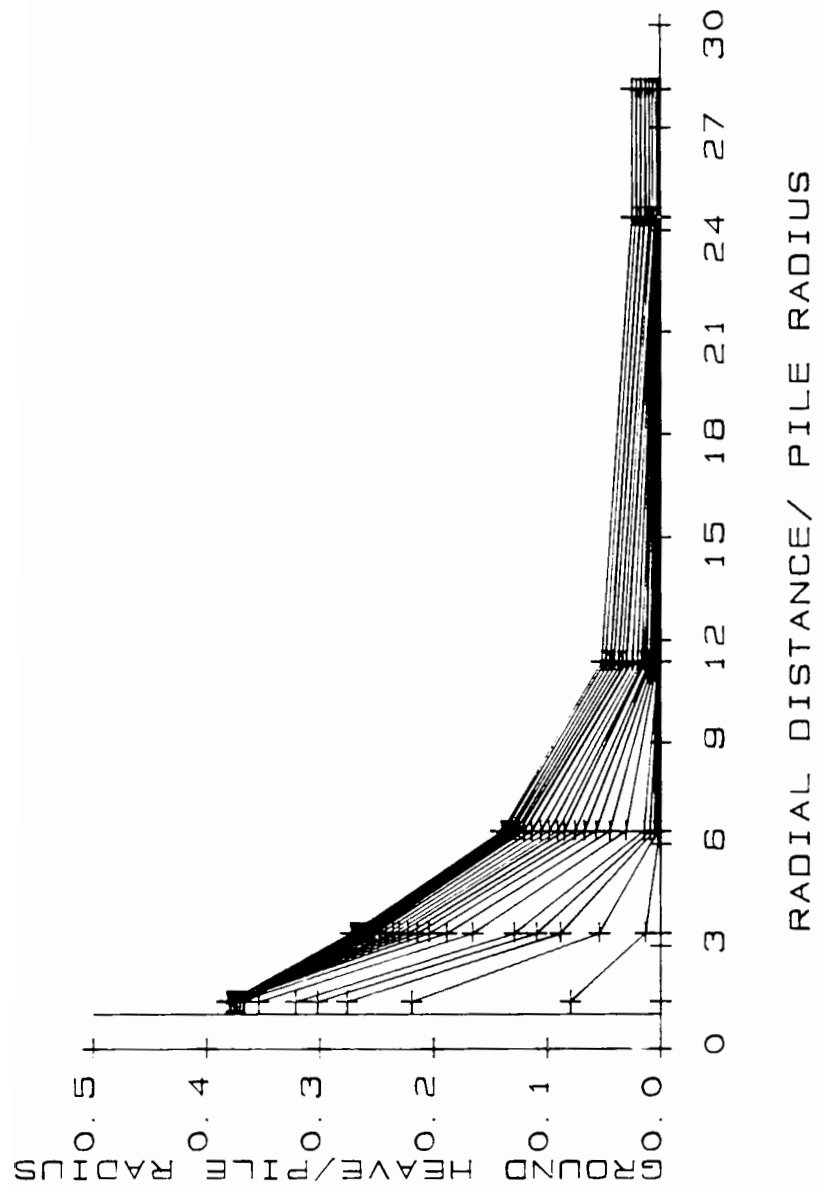


Figure 6.13 Cross-section of ground heave profiles due to driving a closed-ended pile- 7C3

cumulative ground heave at each scan for a closed-ended pile. The vertical scale has been highly exaggerated.

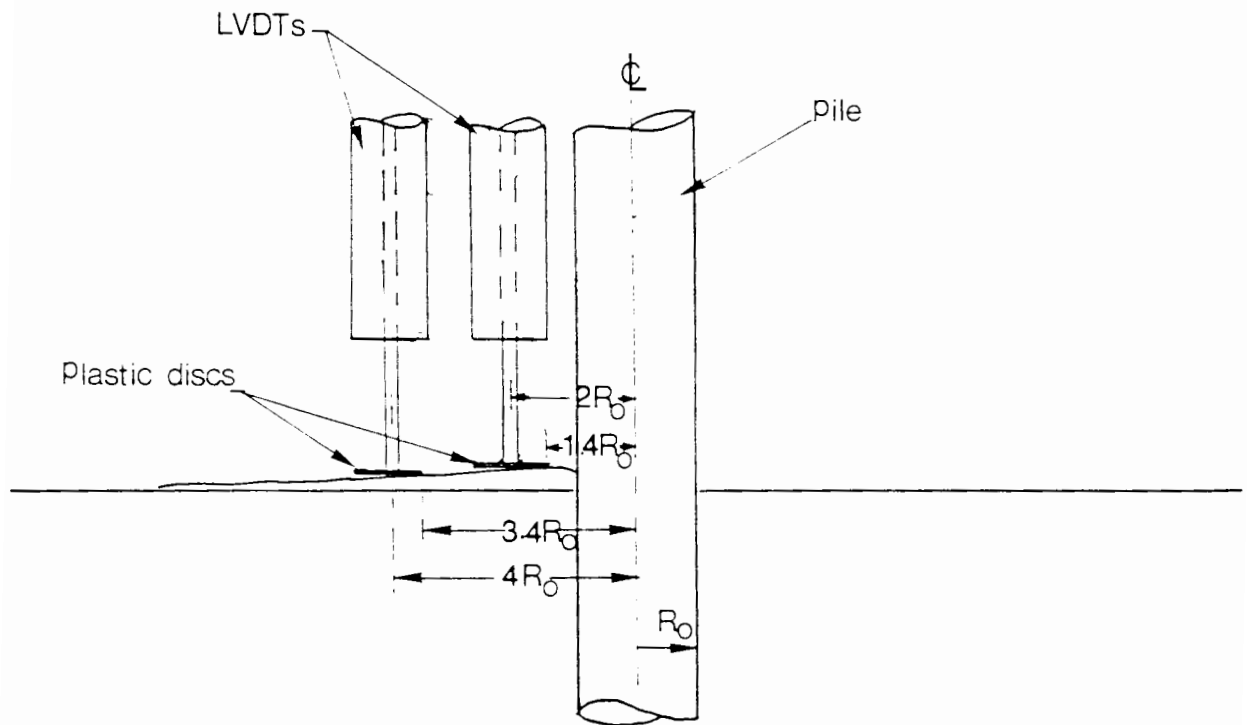


Figure 6.14 Corrections to the positions of the LVDTs

The weight of the LVDT armature and the 10 mm diameter plastic base was about 1.3 gf, which is equivalent to a bearing pressure of only 0.16 kPa assuming the full disc area is in contact with the clay surface. Consequently, as the pile was driven, the ground heave lifted the armatures by their enlarged bases, and so the discs were no longer in full contact with the clay surface (Figure 6.14). The

original positions of LVDTs at 2, 4, 7, 12, and $25R_o$, (R_o , outer pile radius) were therefore corrected to 1.4, 3.4, 6.4, 11.4 and $24.4R_o$.

The data from Figure 6.13 were replotted to show the variation of the accumulated ground heave with pile penetration (Figure 6.15) for a closed-ended pile. The ground heave at $1.4R_o$ was greatest and was mostly generated before the pile had penetrated to a depth of about 12 pile radii. In contrast, at $24.4R_o$, the ground heave was only generated after the pile had been driven beyond about $25R_o$. It seems that, at further radial distances from the pile (greater than 10 pile radii), ground heave only began to be generated after the pile had been driven beyond a penetration equal to the radial distance. Similar behaviour was observed for the open-ended pile except that the magnitude of ground heave was greatly reduced (Figures 6.16 and 6.17).

In two of the tests, one closed- and one open-ended, the scanning of the LVDTs was allowed to continue after the end of driving for a period of 35 minutes; (for the first 5 minutes, they were scanned at intervals of 1 min. and for the following 30 minutes, at intervals of 5 min.). There was no change in the ground heave apart from that due to the swelling of the clay.

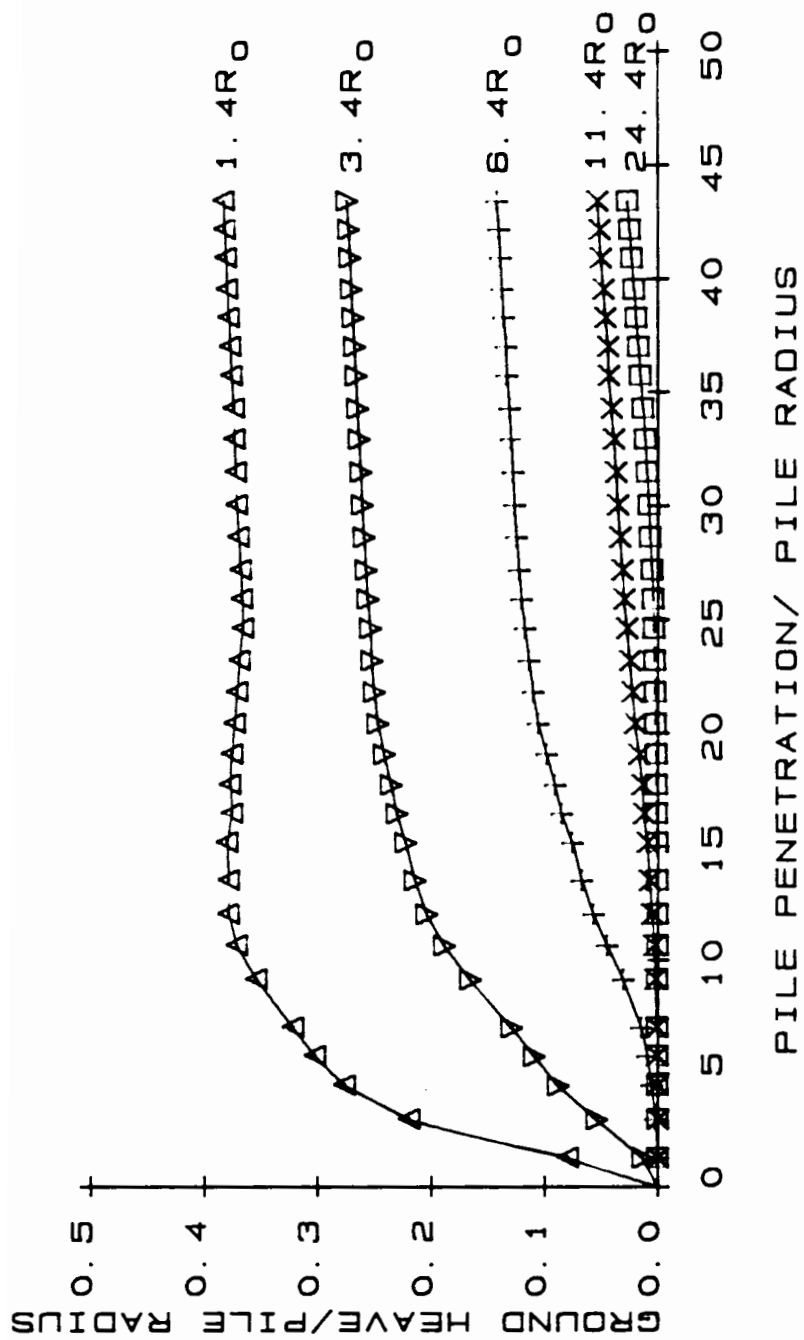


Figure 6.15 Variation of ground heave at various radial distances due to driving a closed-ended pile-7C3

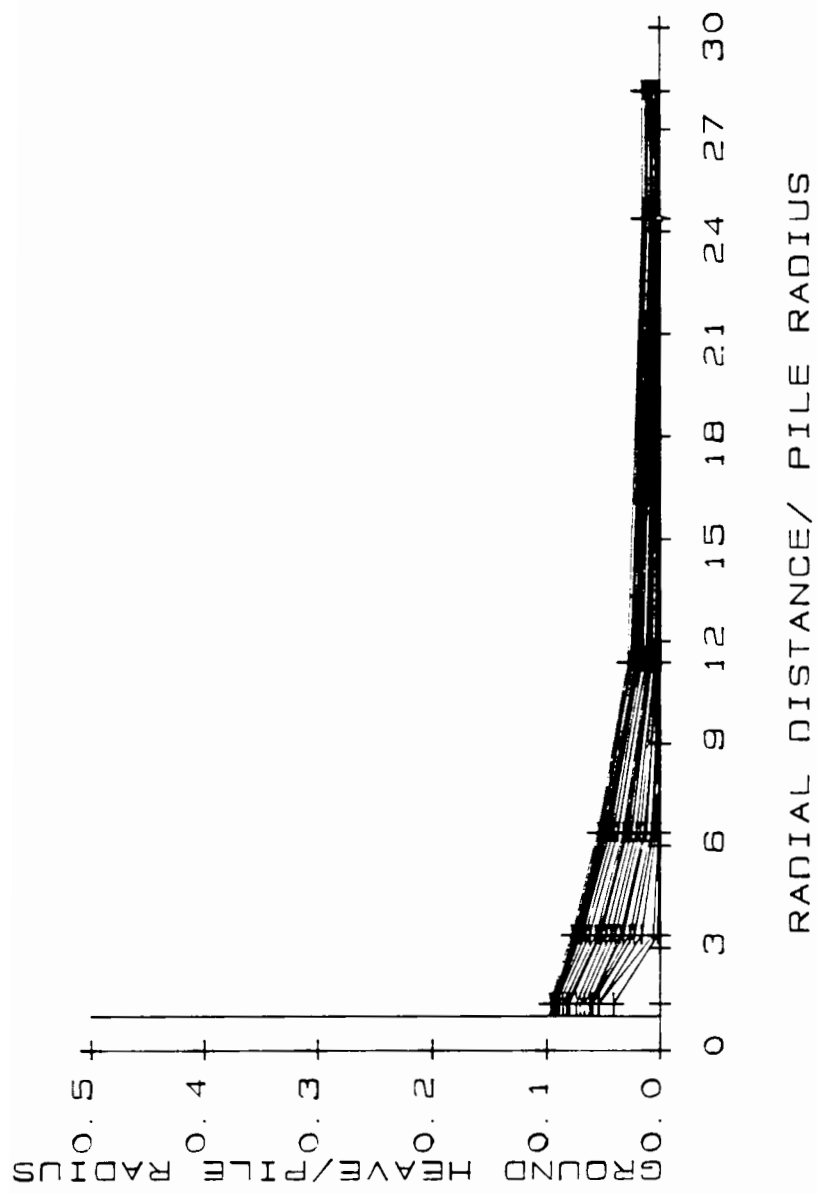


Figure 6.16 Cross-section of ground heave profiles due to driving an open-ended pile- F03

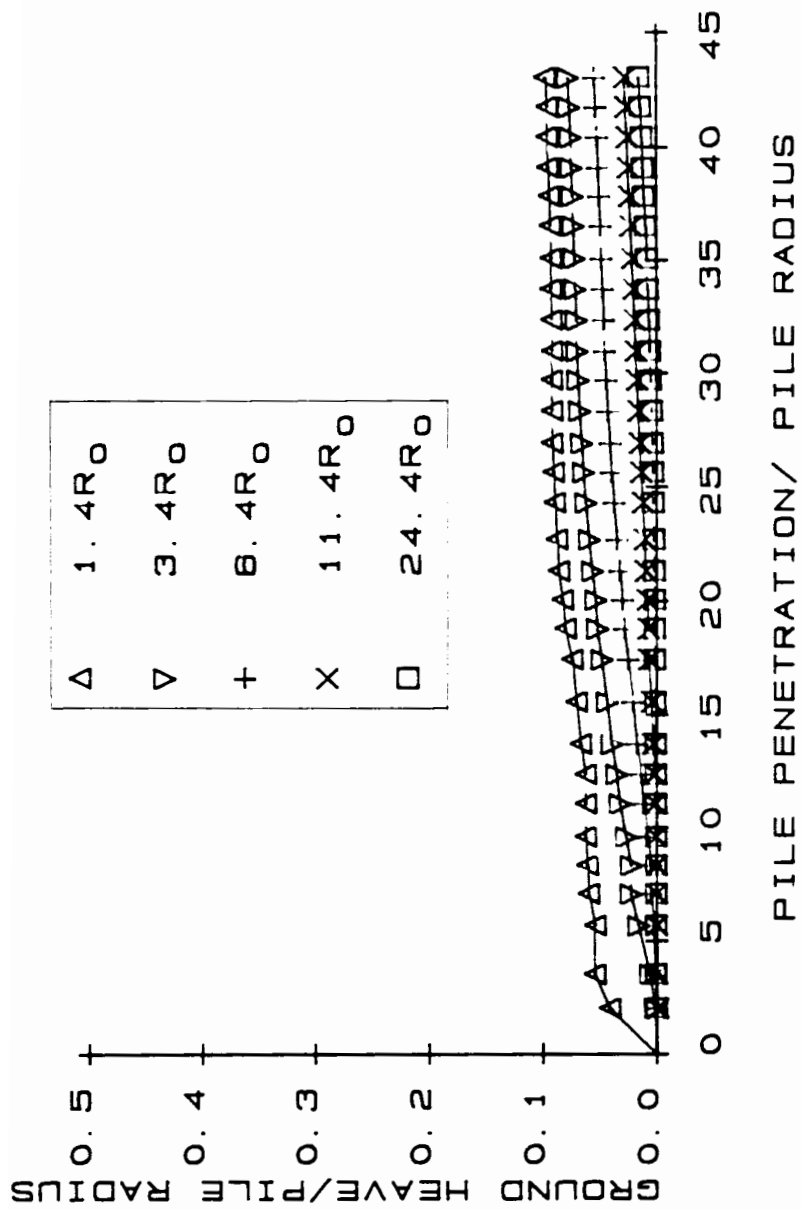


Figure 6.17 Variation of ground heave at various radial distances due to driving an open-ended pile-F03

6.3.3 Interpretation of Volume of Ground Heave

The volume of ground heave was computed by Pappus's Theorem. The theorem defines the volume generated by a plane surface of area A rotating about an axis as :

$$V=2\pi A\bar{y}$$

where $\bar{y}$ is the perpendicular distance of the centroid of the area A from the axis. The magnitudes of ground heave at the pile wall and at the cylinder wall were assumed equal to the ground heave at $1.4R_0$ and $24.4R_0$ respectively (Figure 6.18). These assumptions were based on observation. Although the radial distances on the plots are shown to the nearest one decimal place, more exact corrected radial distances were used for computation. For example at R_1 the radial distance was $2R_0-5.0$ mm. The volume of ground heave, VOLUMH for segment (2) as shown in Figure 6.18 can be expressed by:-

$$\begin{aligned} \text{VOLUMH} = & 2\pi [D_2 (R_2 - R_1) ((R_1 + R_2)/2) \\ & + (D_1 - D_2) ((R_2 - R_1)/2) ((2R_1 + R_2)/3)] \end{aligned}$$

Definitions of the variables are given in Figure 6.18. This

expression may be rewritten as :-

$$\text{VOLUMH} = \pi [D_1 (R_2^2 + R_1 R_2 - 2R_1^2) / 3 + D_2 (2R_2^2 - R_1 R_2 - R_1^2) / 3]$$

The volume of pile inserted (metal) into the soil for a closed-ended pile, VOLUMC and an open-ended pile, VOLUMO are expressed as:-

$$\text{VOLUMC} = \pi D_L (R_o^2)$$

$$\text{and } \text{VOLUMO} = \pi D_L (R_o^2 - R_i^2)$$

where D_L and R_i are depth of pile penetration and internal radius of the pile respectively.

These volumes were made dimensionless by dividing them by πR_o^3 . When this volume is then plotted against normalised pile penetration, a gradient of 1.0 is obtained if the volume of ground heave is equal to the inserted volume of a closed-ended pile.

Figure 6.19 shows a dimensionless plot of the volume of ground heave for both closed- and open-ended piles. The open-ended pile was driven with a driving shoe. This graph shows that almost all the volume of the closed-ended pile installed was displaced as ground heave. Similarly, the open-ended pile displaced a slightly greater volume of soil than the volume of the metal inserted. It therefore appears that the piles were driven into the specimens in undrained conditions, i.e. there was no change in volume of the clay.

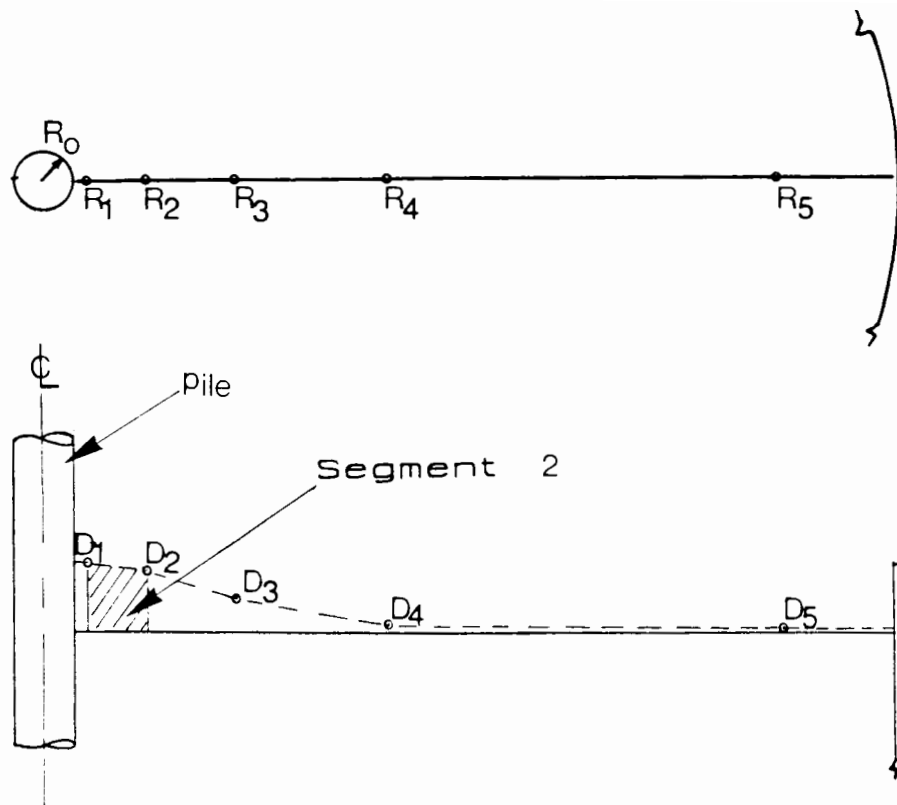


Figure 6.18 Definition sketch for calculation of volume of ground heave

6.3.4 The Effect of Driving Shoe for Open-ended Piles on Ground Heave

As mentioned earlier, the open-ended piles were driven with and without driving shoes. The inner walls of the piles were polished and greased except for Series No. 4 (see Table 6.2). Table 6.2 also shows the proportion of the final length of plugged soil inside the pile to the depth of penetration. When driven without shoe the open-ended pile

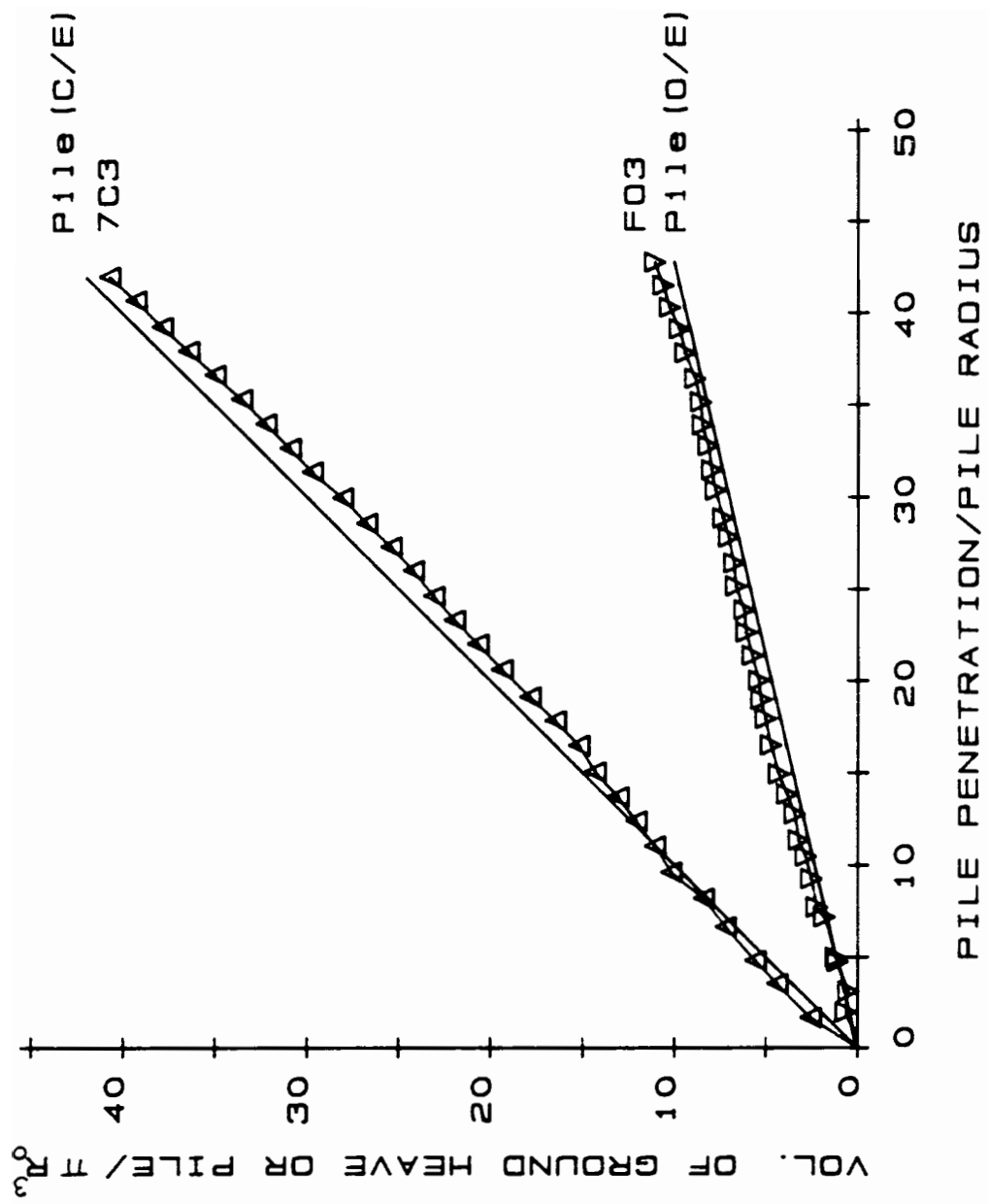


Figure 6.19 Volumes of ground heave for closed- and open-ended piles

plugged. Polishing and greasing the inner wall only slightly increased the length of plugged soil, and the proportion of the length of plugged soil was about 70 %. When a driving shoe was used the length of plugged soil was equal to the pile penetration.

Driving shoes had no significant effect on the profile of ground heave especially the maximum heave near the pile. However, there was a dramatic effect on the volume of ground heave generated (Figure 6.20). It appears that once the pile began to plug, it behaved almost like a closed-ended pile. Consequently, the cumulative volume of ground heave generated began to diverge from the volume of the open-ended pile inserted, when it was driven beyond a penetration of about 13 pile radii. At this stage the proportion of length of plugged soil to penetration was only about 30 % as compared to the 69 % measured at the end of pile driving (see Table 6.2). It is possible that the length of plugged soil might have increased during further driving. The variation of the length of plugged soil found in the qualitative analysis of the semi-cylindrical pile tests confirmed this behaviour.

6.3.5 The Effect of Rate of Driving on Ground Heave

In Series 16 and 17, closed-ended piles were driven at rates of 22 and 150 blows/min. respectively. No significant differences in the ground heave profiles and volume of

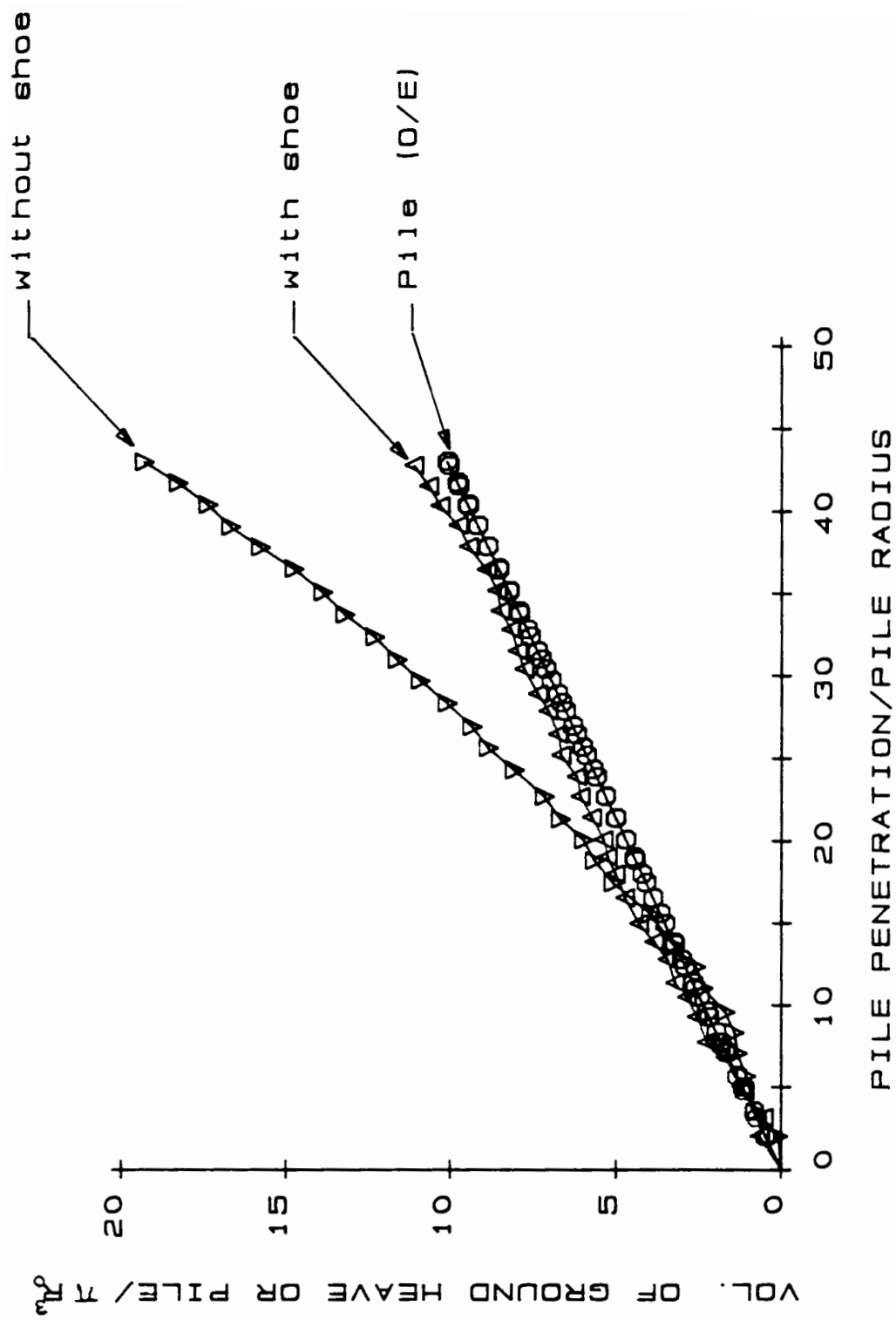


Figure 6.20 Comparison of volumes of ground heave with and without pile driving shoes

ground heave generated were observed (Figure 6.21).

6.3.6 The Effect of Stress History on Ground Heave

The study of this effect began by varying the OCR of three specimens prepared with the same maximum vertical pressure (200 kPa). One of the specimens was normally consolidated and the other two were allowed to swell to OCRs of 6 and 10. Closed-ended piles were driven into each specimen at the same rates of driving. The variation of ground heave was not significantly affected by the OCR (Figure 6.22).

The investigation was pursued further by preparing specimens with three different maximum vertical consolidation pressures (200, 400 and 640 kPa) but all with a same OCR of 6. Both closed- and open-ended piles were driven into these test specimens. The results from closed-ended pile were examined first. Figure 6.23 shows the variation of ground heave at two radial distances. It appeared that the magnitude of ground heave at small radial distances from the pile was slightly reduced for specimens with greater maximum consolidation pressures. However, at greater radial distance there was a very slight increase in heave for greater consolidation pressures. The total volume of ground heave generated was the same in each case. Similar behaviour was observed for open-ended piles.

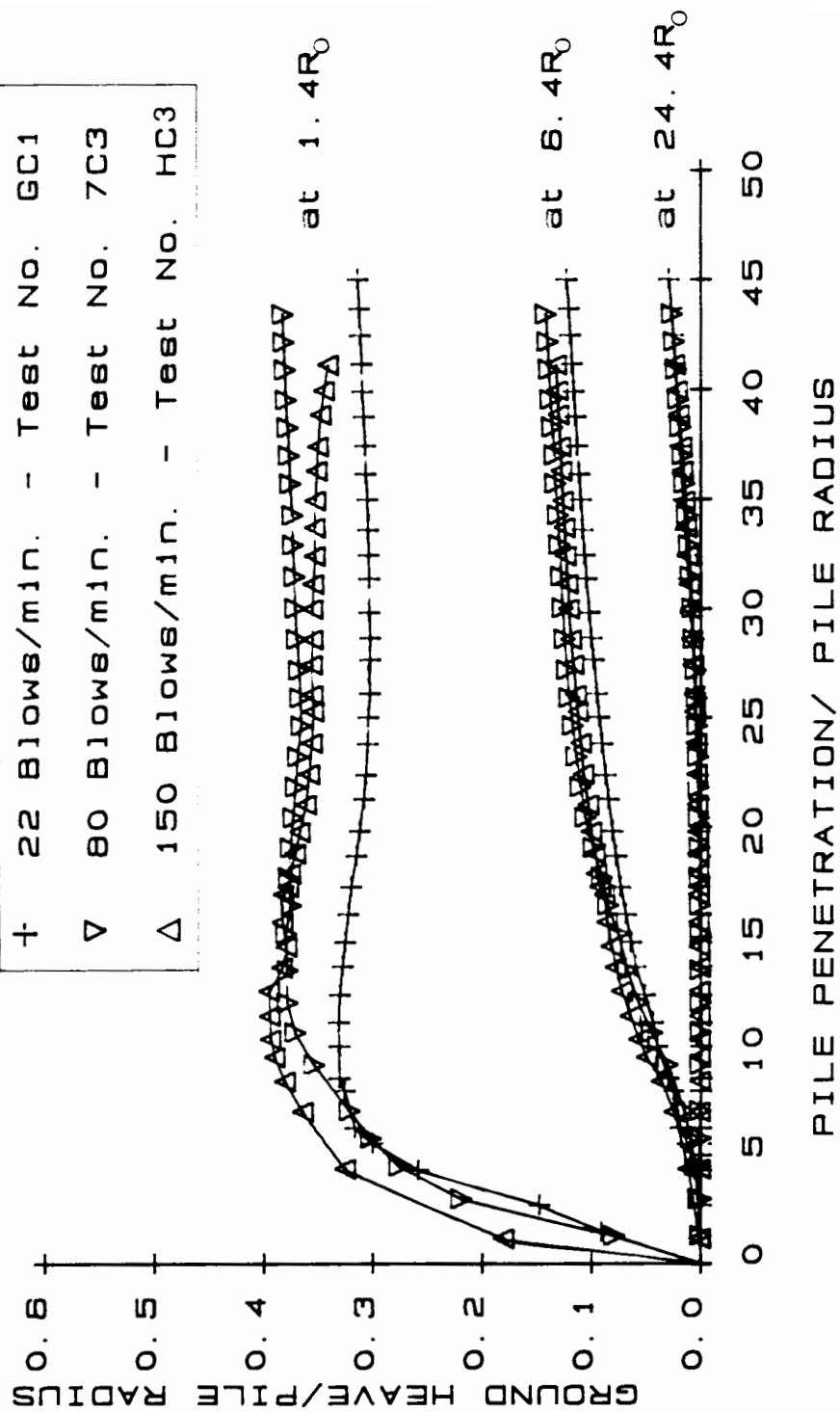


Figure 6.21 Effect of rate of driving on ground heave for closed-ended pile

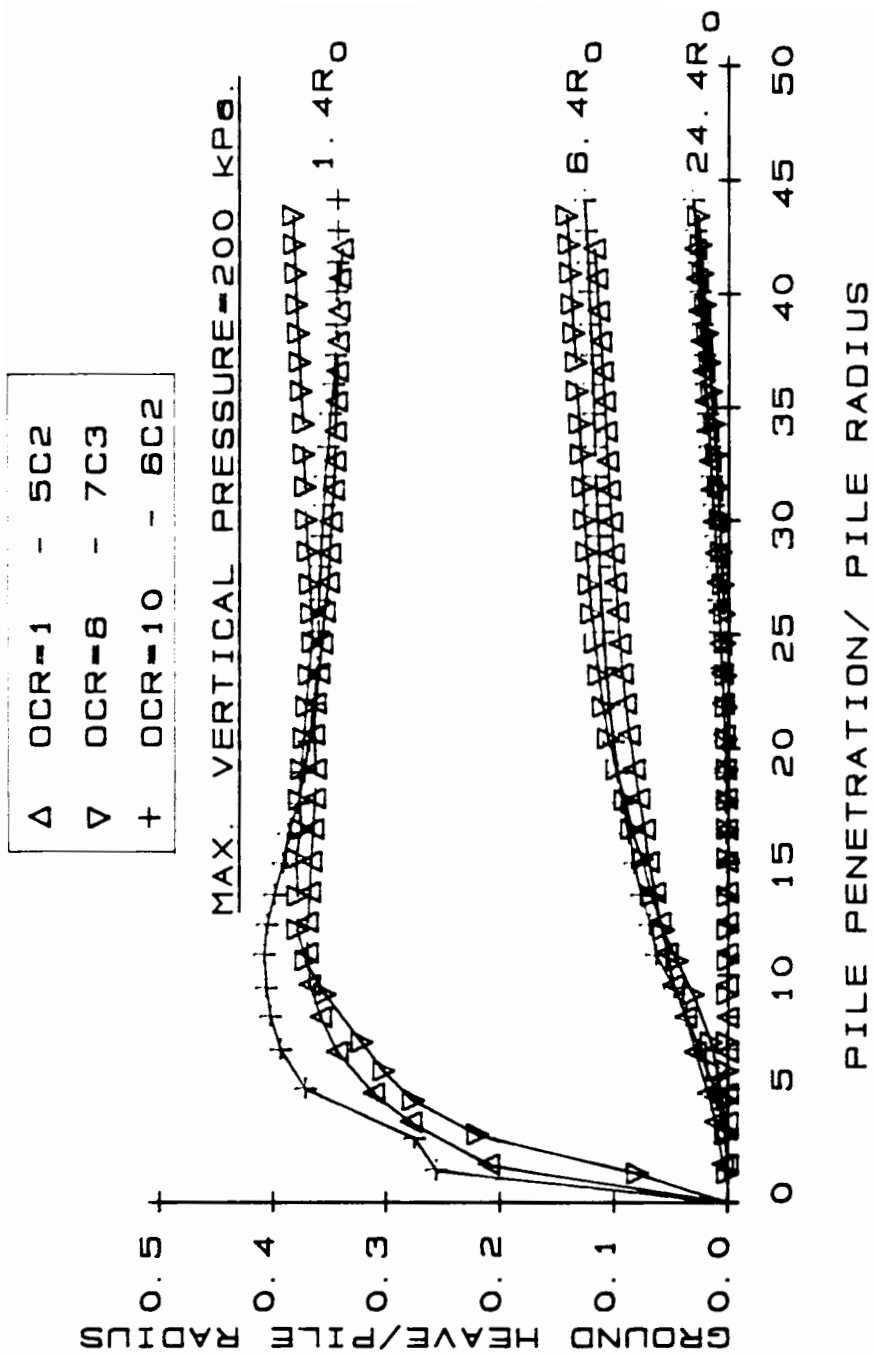


Figure 6.22 Effect of OCR on ground heave for closed-ended piles

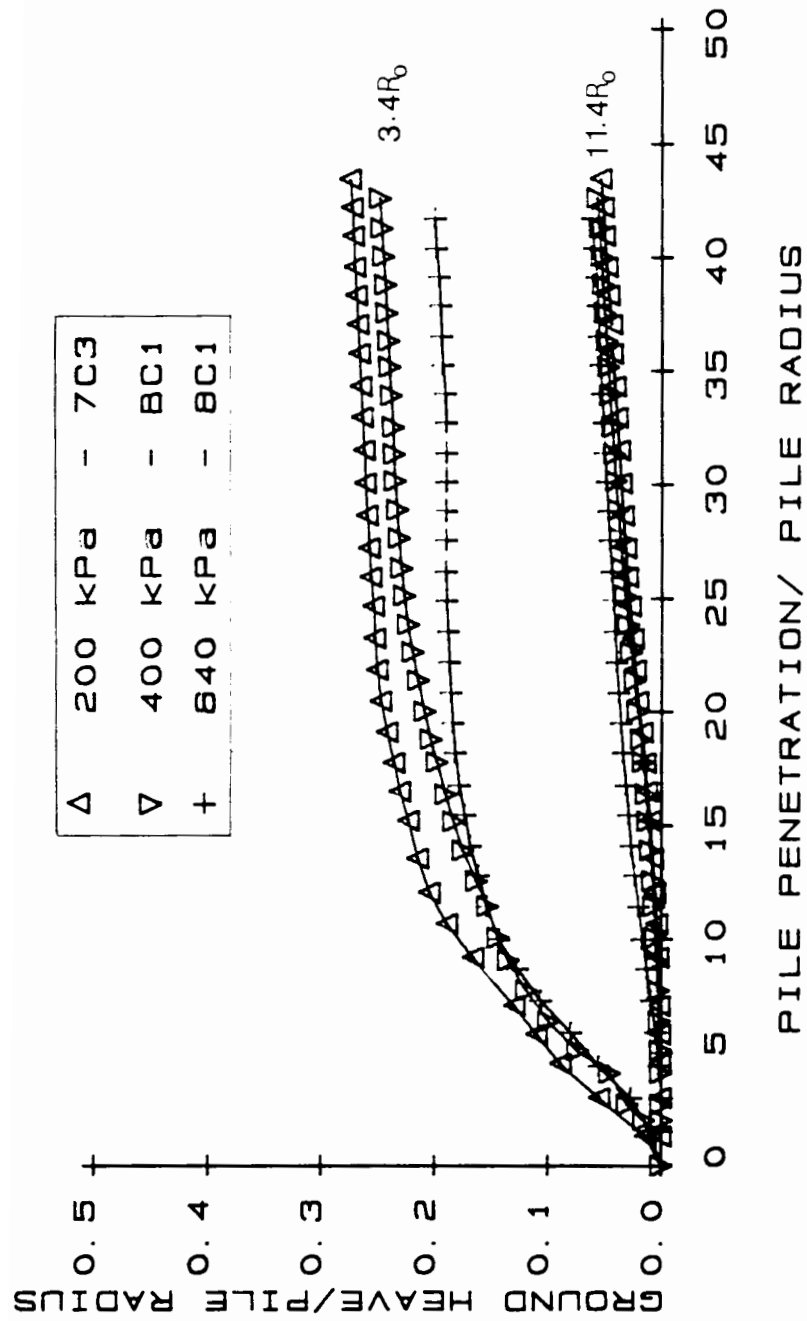


Figure 6.23 Effect of maximum vertical consolidation pressure on ground heave for closed-ended piles

The proportion of the length of the plugged soil to the pile's final penetration (Table 6.2) appeared to be unaffected by the maximum vertical consolidation pressure.

Several tests were repeated, and the results showed good repeatability. Fig 6.24 shows one set of the repeated test on specimens with the same stress history.

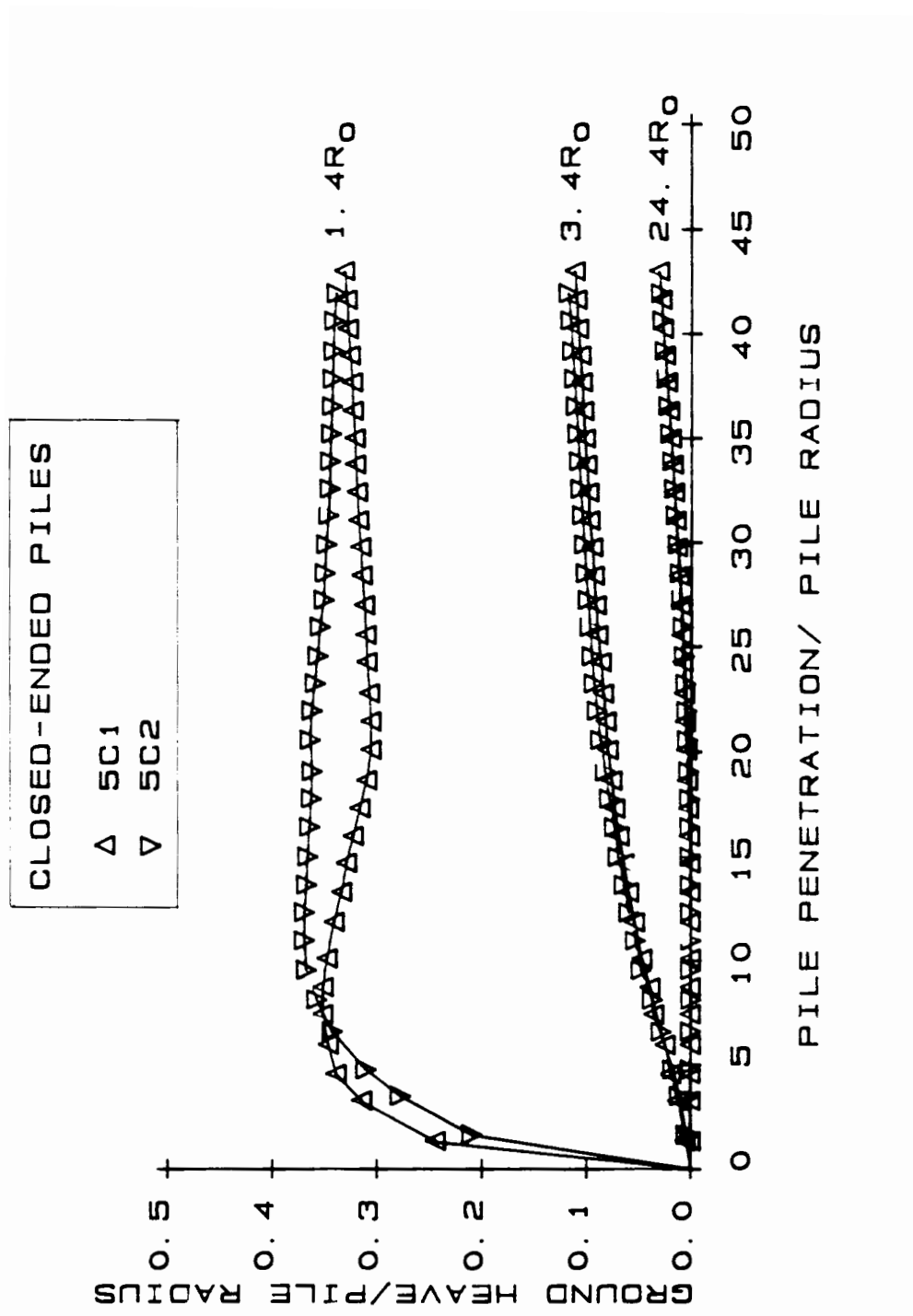


Figure 6.24 Variation of ground heave with pile penetration for two radial distances-5C1 and 5C2

CHAPTER 7

EXPERIMENTAL RESULTS

PILE DRIVING WITH SURCHARGE PRESSURES

7.1 Introduction

This chapter presents the results of tests for pile driving through a surcharged collar and model mudmat. The summaries of the tests are shown in Tables 7.1 and 7.2. The surcharged collar was included as an intermediate stage between the single pile driven with no surcharge pressure surrounding the pile and a group of piles driven through a mudmat.

The effect of OCR on ground heave was found to be insignificant but the maximum vertical consolidation pressure showed a small influence (see chapter 6). Therefore, all the specimens used for the study of the effect of surcharge pressure were over-consolidated with a maximum vertical pressure of 200 kPa and an OCR of 6. This stress history was chosen to minimize the generation of high excess negative pore pressure in a specimen when the top platen was removed for pile driving.

In Series 10, the open-ended piles were polished, greased and driven without a driving shoe. All the open-ended piles

TABLE 7.1 SINGLE PILES WITH SURCHARGED COLLARS

SERIES	TEST NO.	% PLUGGED SOIL	SURCHARGE PRESSURE(kPa.)	COMMENTS
9	9C1L	-	47.2	Not a good test
	9C2L	-	14.4	
10	AC2L	-	33.0	Not a good test
	AO3L	63	32.5	P & G
12	CO2L	96	15.5	P, G & S
13	DC3L	-	33.3	
14	EO2L	96	33.3	P, G & S
15	FC1L	-	4.8	
	FO2L	105	4.8	P, G & S
16	GC2L	-	32.5	Repeat test

TABLE 7.2 PILES DRIVEN WITH A SURCHARGED MODEL MUDMAT

SERIES	TEST NO.	% PLUGGED SOIL	SURCHARGE PRESSURE(kPa.)	COMMENTS
13	DO1G	94-99	16.7	P, G & S
	DC2G	-	16.7	
14	EO1G	67-82	31.6	P, G & S
	EC2G	-	31.6	
16	GC3G	-	31.6	Repeat test
17	HO1G	94-97	4.6	P, G & S
	HC2G	-	4.6	

NOTES: a) All samples were consolidated with a maximum vertical pressure of 200 kPa. and OCR of 6.

b) P, G & S represent the internal conditions of an open-ended piles:-

P polished

G prepared with silicon grease and a light spray of grease

S pile shoe

c) Driving : All piles were driven with 1360 gf. hammer, 100 mm. drop height and driven at 80 blows per minute.

d) % plugged soil is the proportion of the length of plugged soil to the total length of pile penetration.

for the subsequent series were polished and greased as before and driven with shoes. Surcharge pressures ranging from 4.6 to 33.3 kPa were used in the investigation.

7.2 Single Pile Driven through a Surcharged Collar

Figure 7.1 shows a cross-section of ground heave for a closed ended pile driven through a collar with a surcharge pressure of 14.4 kPa. Four of the LVDTs were located outside the collar to measure the ground heave as in the single pile tests described in Chapter 6. The LVDT at $1.4R_o$ was resting on the collar, measuring its vertical displacement. Figure 7.1 was replotted to show the variation of ground heave and the vertical displacement of the surcharged collar with pile penetration, shown in Figure 7.2.

The collar was initially lifted up reaching a maximum at a pile penetration of about 10 pile radii. As the pile was driven further, the collar began to settle and continued to do so until the pile was driven to its full penetration. However, for the open-ended pile (Figure 7.3) with a surcharge pressure of 16.7 kPa, the mudmat was only lifted up slightly during the first scan (10 mm penetration) and subsequently began to settle. The lifting of the collar was less dramatic because there is less soil displaced when open ended piles are driven. In both cases, the ground

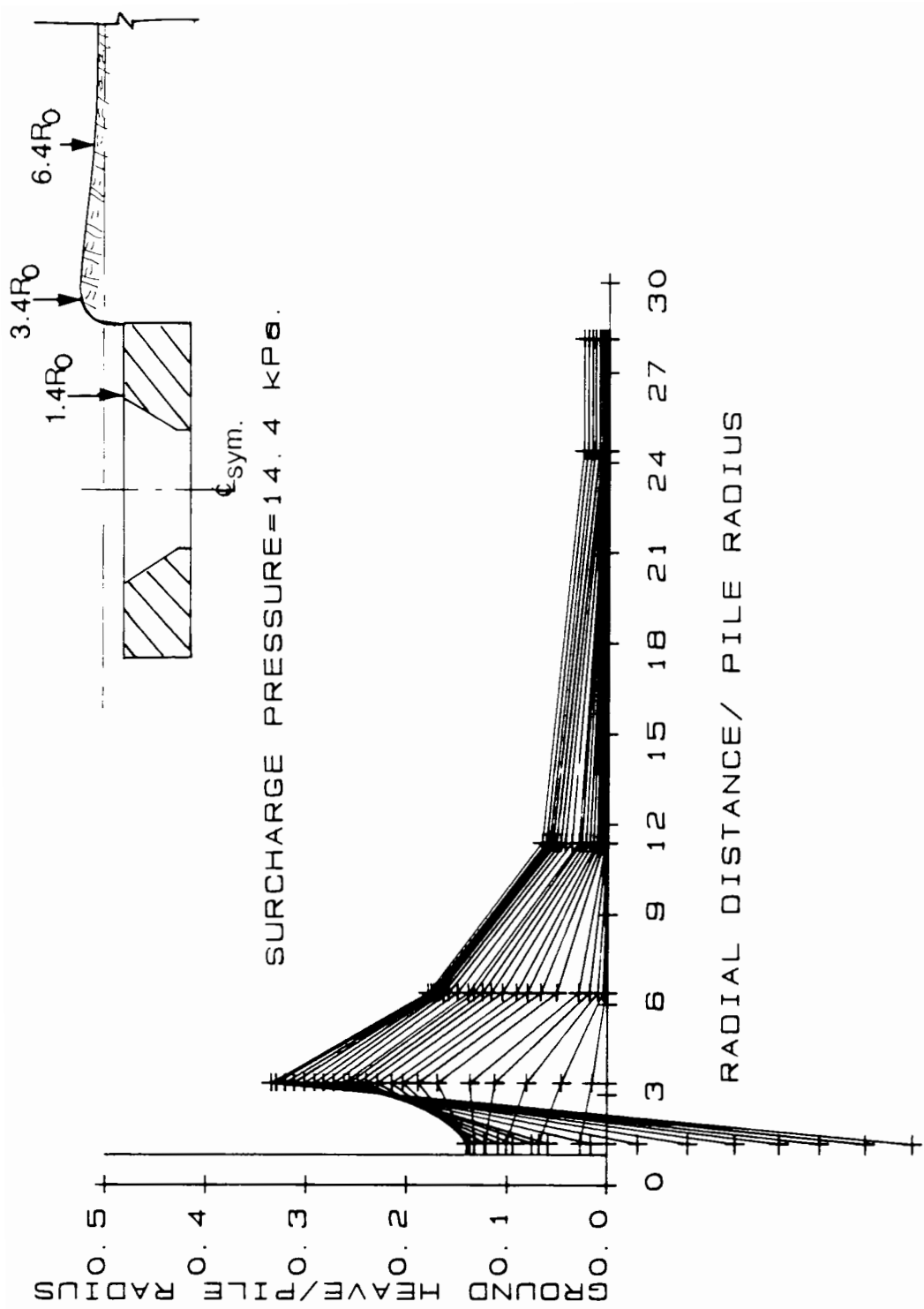


Figure 7.1 Ground heave profiles generated by driving a closed-ended pile through a surcharged collar-9C2L

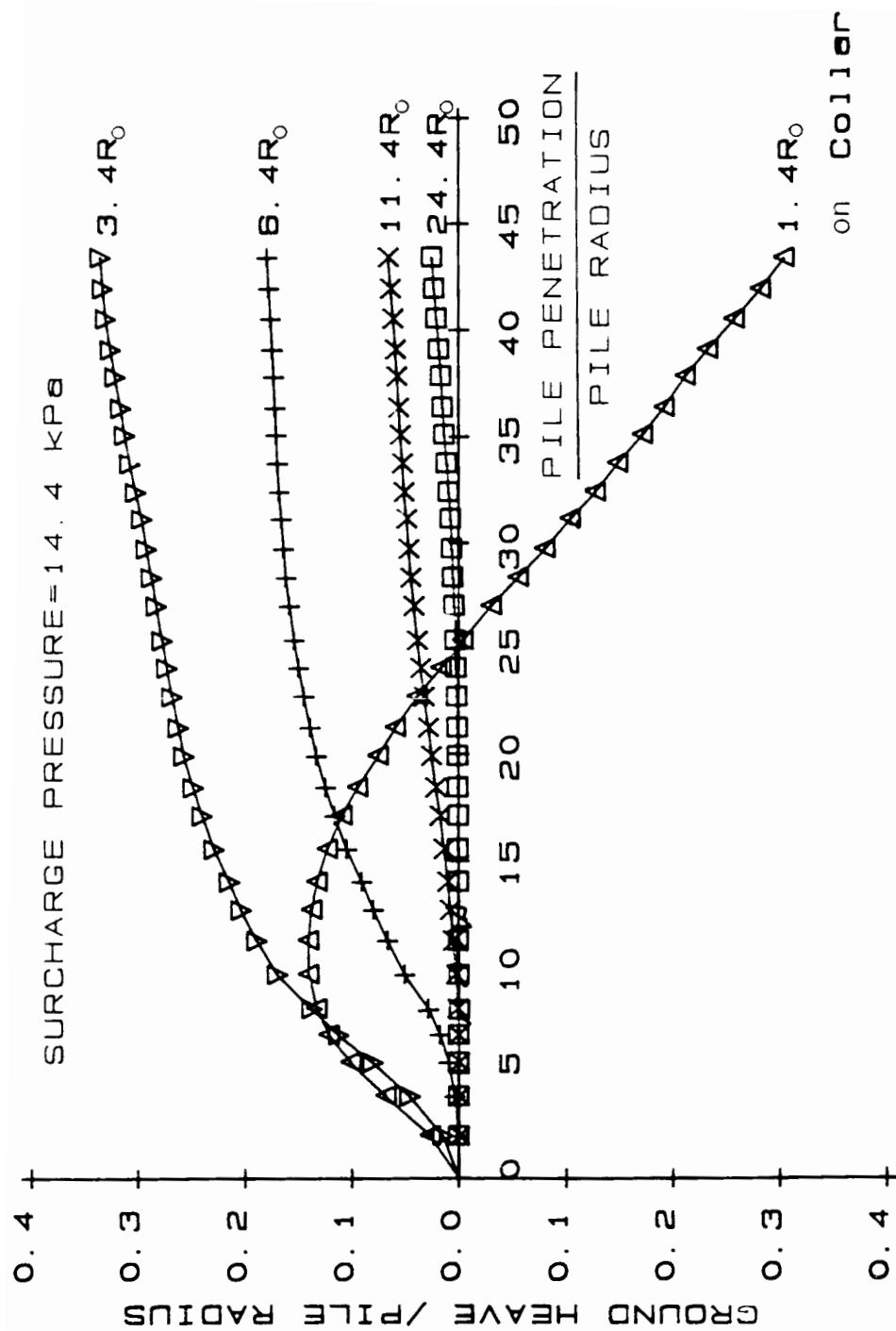


Figure 7.2 Variation of ground heave during driving a closed-ended pile driven through a surcharged collar-9C2L.

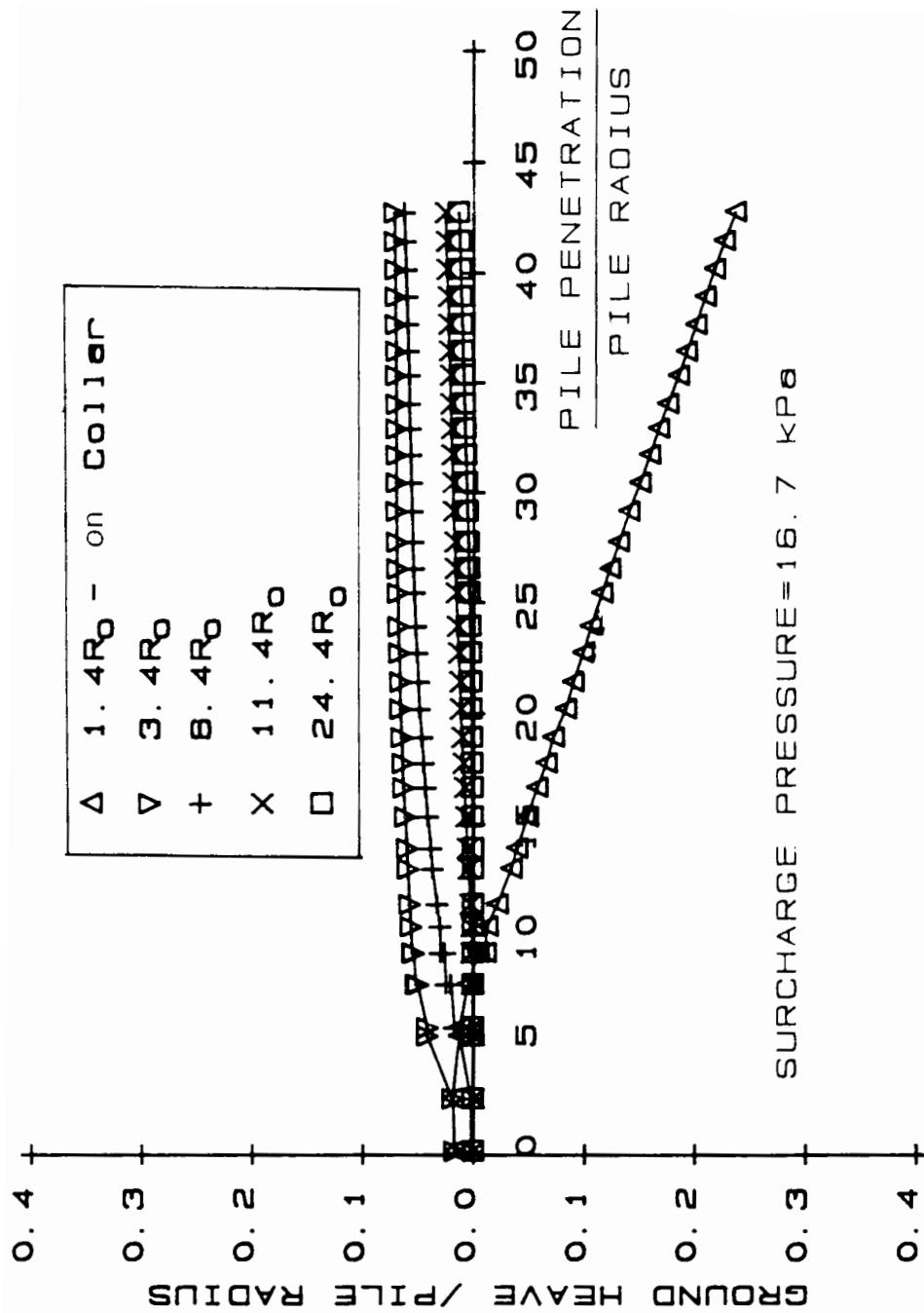


Figure 7.3 Variation of ground heave during driving an open-ended pile driven through a surcharged collar-C02L

heave at 3.4 pile radii was much greater compared to the results for a single pile presented in Chapter 6. The additional ground heave was caused by the soil displaced by the surcharged collar as it settled. This is shown in Figure 7.4 where the volume of ground heave is substantially greater than the volume of the pile inserted into the soil.

Various surcharge pressures were used to investigate the influence of the pressure on the behaviour of the collar (Table 7.1). The open-ended piles were virtually unplugged except for the test No. A03L when the pile was driven without a shoe. The proportion of the length of plugged soil to the embedded pile length was 63%.

Figures 7.5 and 7.6 show the vertical movements of the collar with penetration for three different surcharge pressures for closed and open ended piles respectively. The results for single pile with no collar ($Q = 0$) from previous chapter are included in both figures for comparison. For a very low surcharge pressure of about $0.5C_u$, both collars were lifted up slightly to a peak at a pile penetration of about 10 pile radii but then began to settle slightly as the pile was driven further. The collar used with an open-ended pile appeared to remain stationary

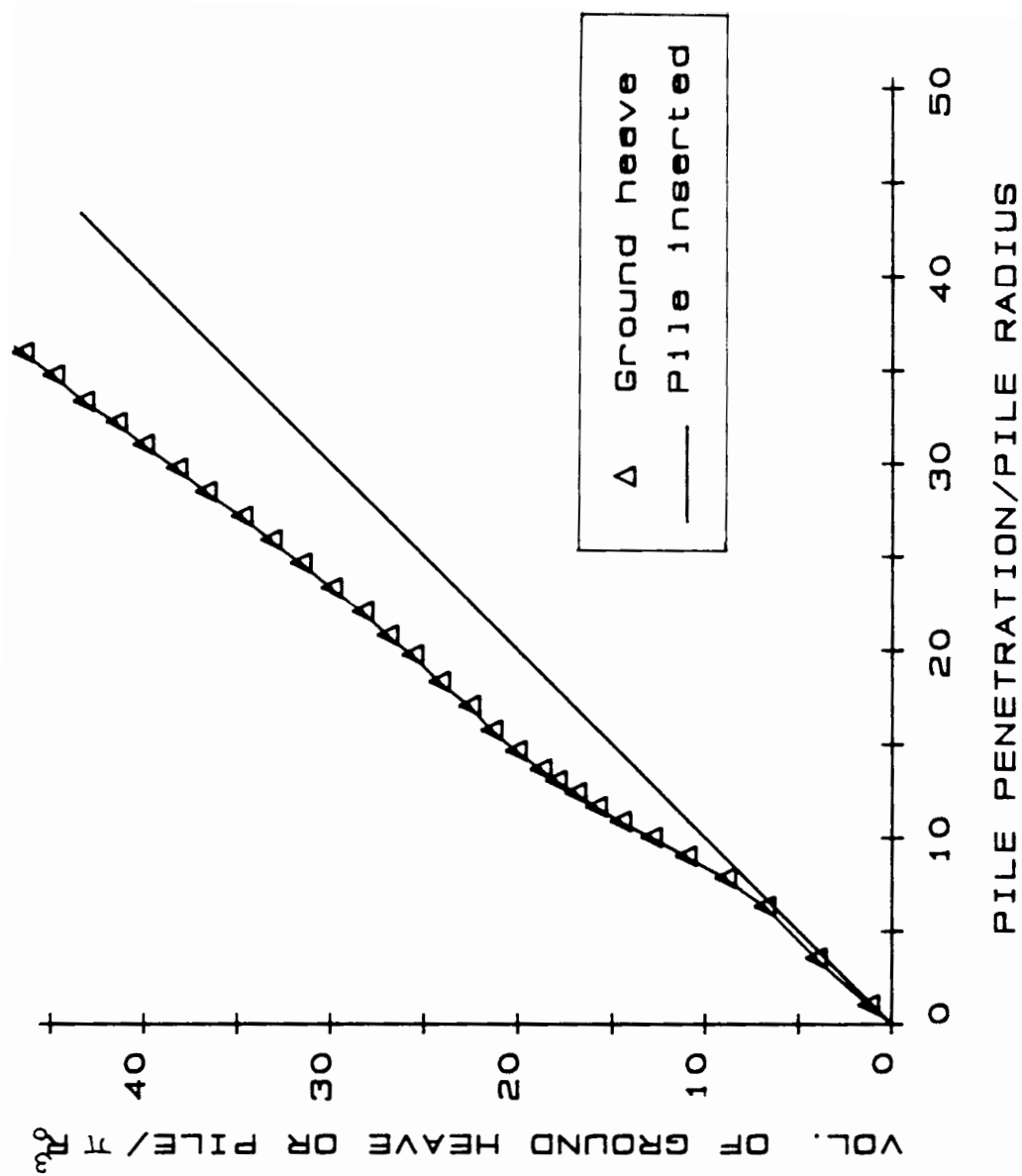


Figure 7.4 Volume of ground heave generated by a closed-ended pile driven through a surcharged collar-DC3L

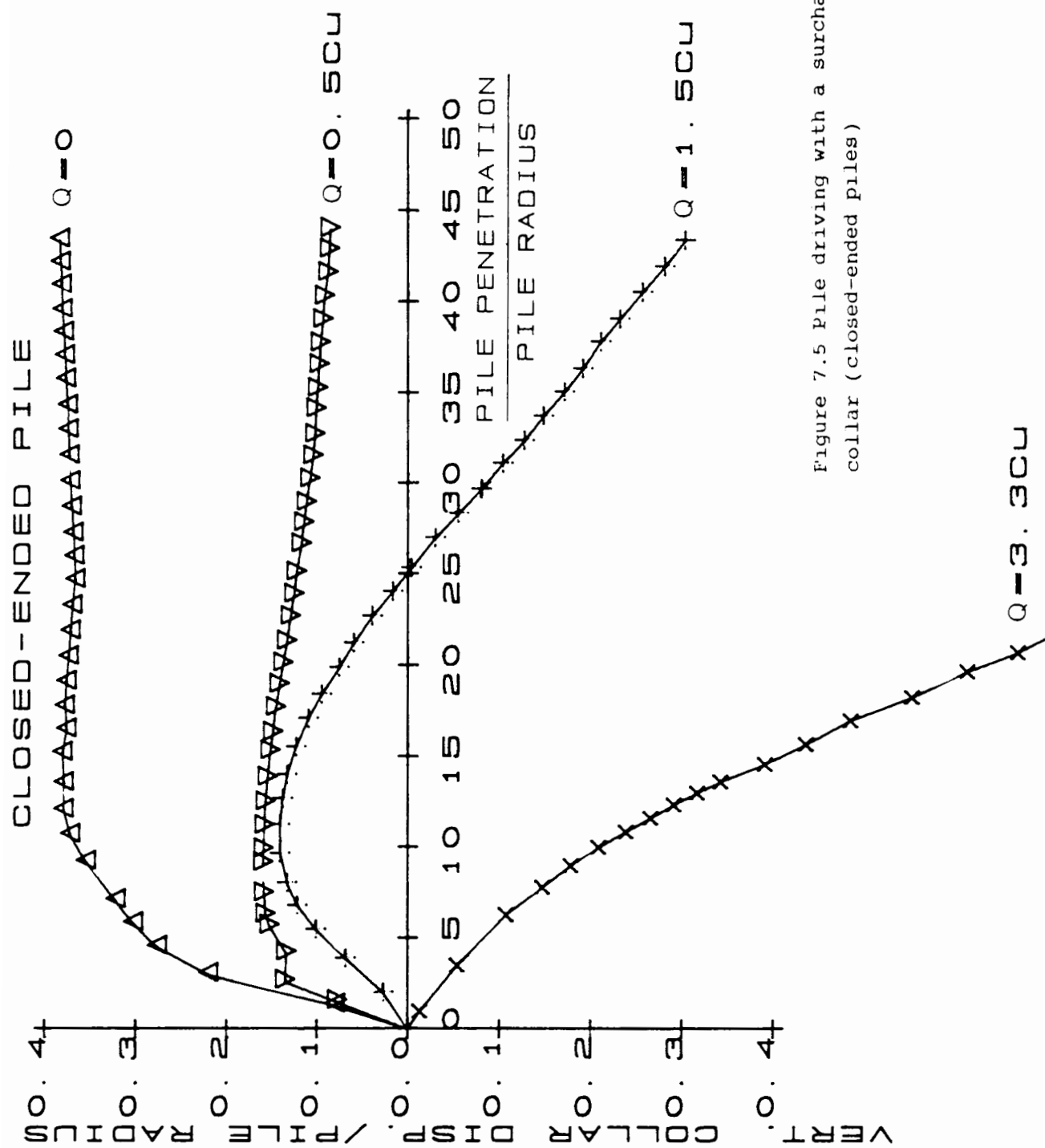


Figure 7.5 Pile driving with a surcharged collar (closed-ended piles)

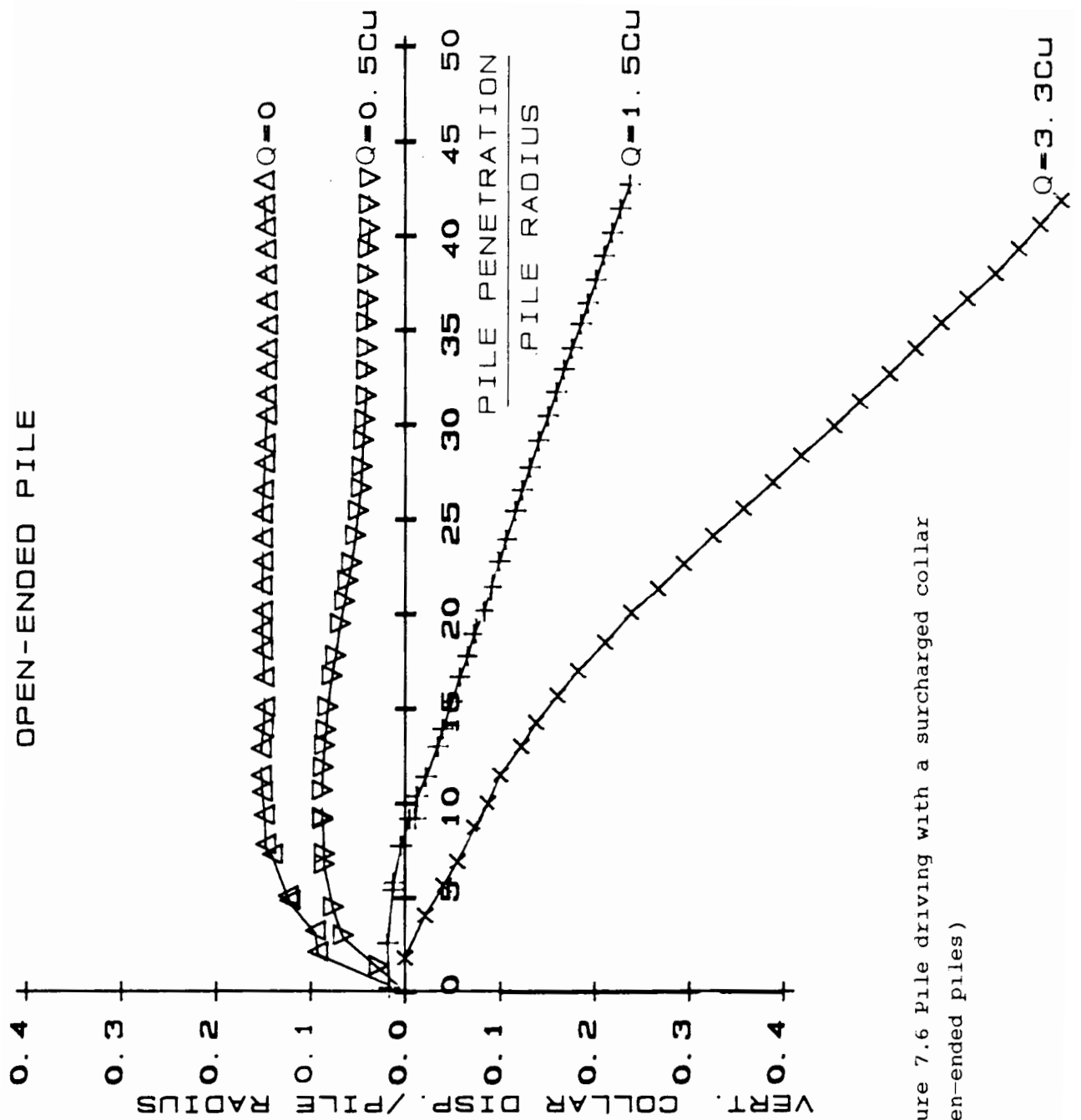


Figure 7.6 Pile driving with a surcharged collar
(open-ended piles)

once the pile had penetrated beyond about 30 pile radii. However, at a higher surcharge pressure of $3.3C_u$, both collars settled immediately on pile driving and reached settlements of 0.75 and 0.99 pile radii at full penetration for the open- and closed-ended respectively.

Figures 7.5 and 7.6 were further examined to highlight the influence of the surcharge pressure on the vertical displacement of the surcharged collar, at four different depths of pile penetration. Figure 7.7 shows dimensionless charts, the coefficient of the surcharge pressure (β) being plotted against the vertical displacement of the collar (δ/R_0). The results from the single pile test (7C3) have been included to represent the $\beta = 0$ case (i.e. no surcharge pressure). Broken lines have been used between $\beta = 0$ and $\beta = 0.5$, since it is doubtful that results from the single pile test are compatible with those from the surcharged collar tests. Both closed- and open-ended piles exhibit the same trend, except that the closed-ended pile induced a much greater settlement of the collar at larger values of β .

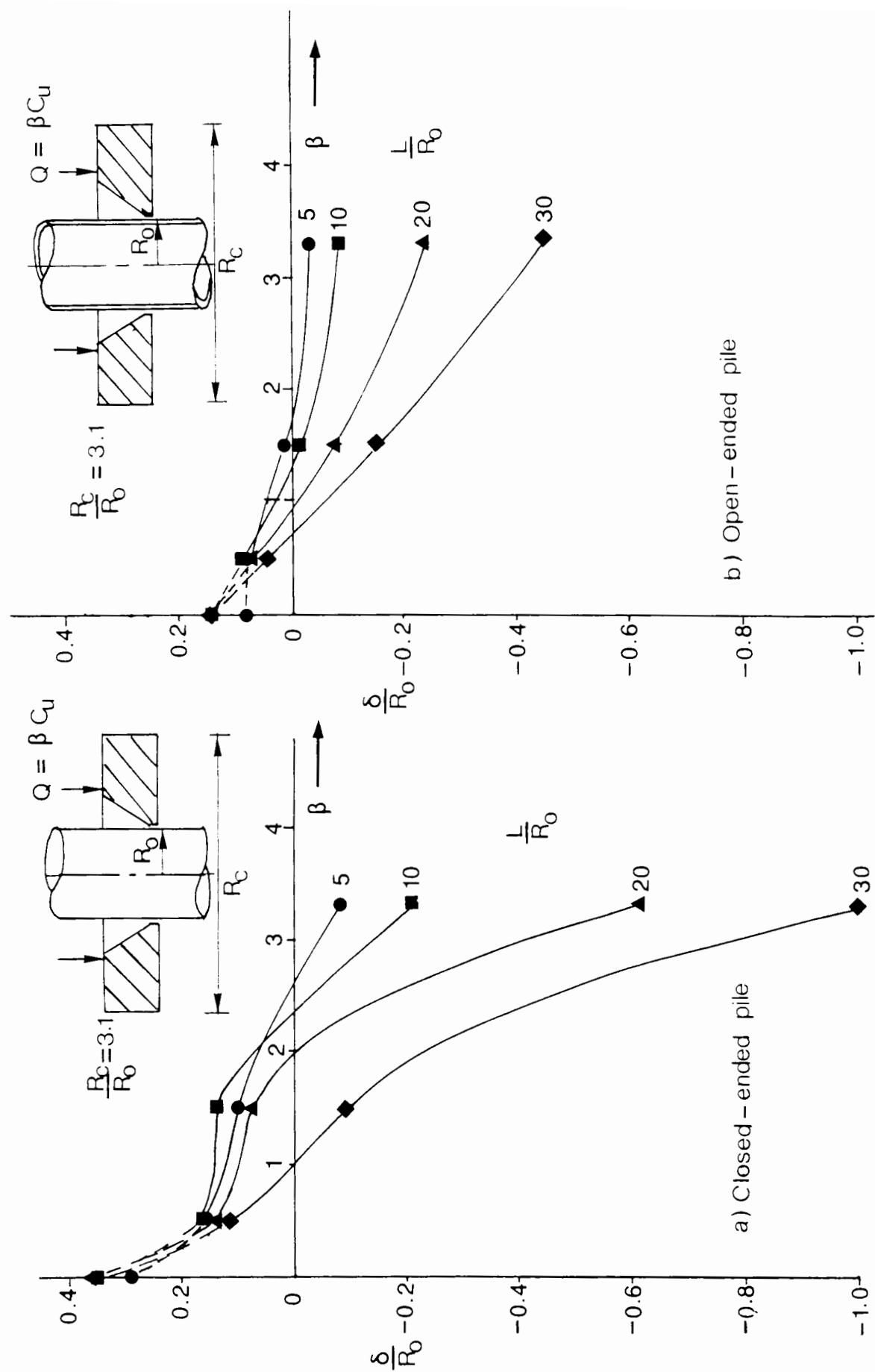


Figure 7.7 The influence of surcharge pressure on collar displacement

7.3 Piles Driven through a Surcharged Model Mudmat

The main purpose of these tests was to investigate the movements of the model mudmat due to driving a group of piles.

Table 7.2 shows the summary of the tests. The inner walls of open-ended piles were polished and greased as before and they were all driven with a driving shoe. The proportion of the length of plugged soil to the pile penetration appeared to be slightly affected by the surcharge pressure. When the pressure was lower than $1.7C_u$, the open-ended piles were almost completely unplugged but at higher surcharge pressures the length of plugged soil decreased.

The piles were driven in the clockwise sequence shown in Figure 4.8. As in the surcharged collar tests, four of the LVDTs were located outside the surcharged model mudmat to measure the ground heave. LVDT A at $1.4R_o$ was resting on the mudmat to measure the displacement at the position of the mudmat as shown in Figure 4.8. Figure 7.8 shows the variation of ground heave and the vertical displacement at LVDT A as the first closed-ended pile was driven through it. The surcharge pressure used for this test was 16.7 kPa. At this surcharge pressure, the ground heaves at $3.4R_o$ and $6.4R_o$ were both dramatically greater than those when no surcharged collar was used because of the additional volume of soil displaced by the mudmat as it settled. The mudmat

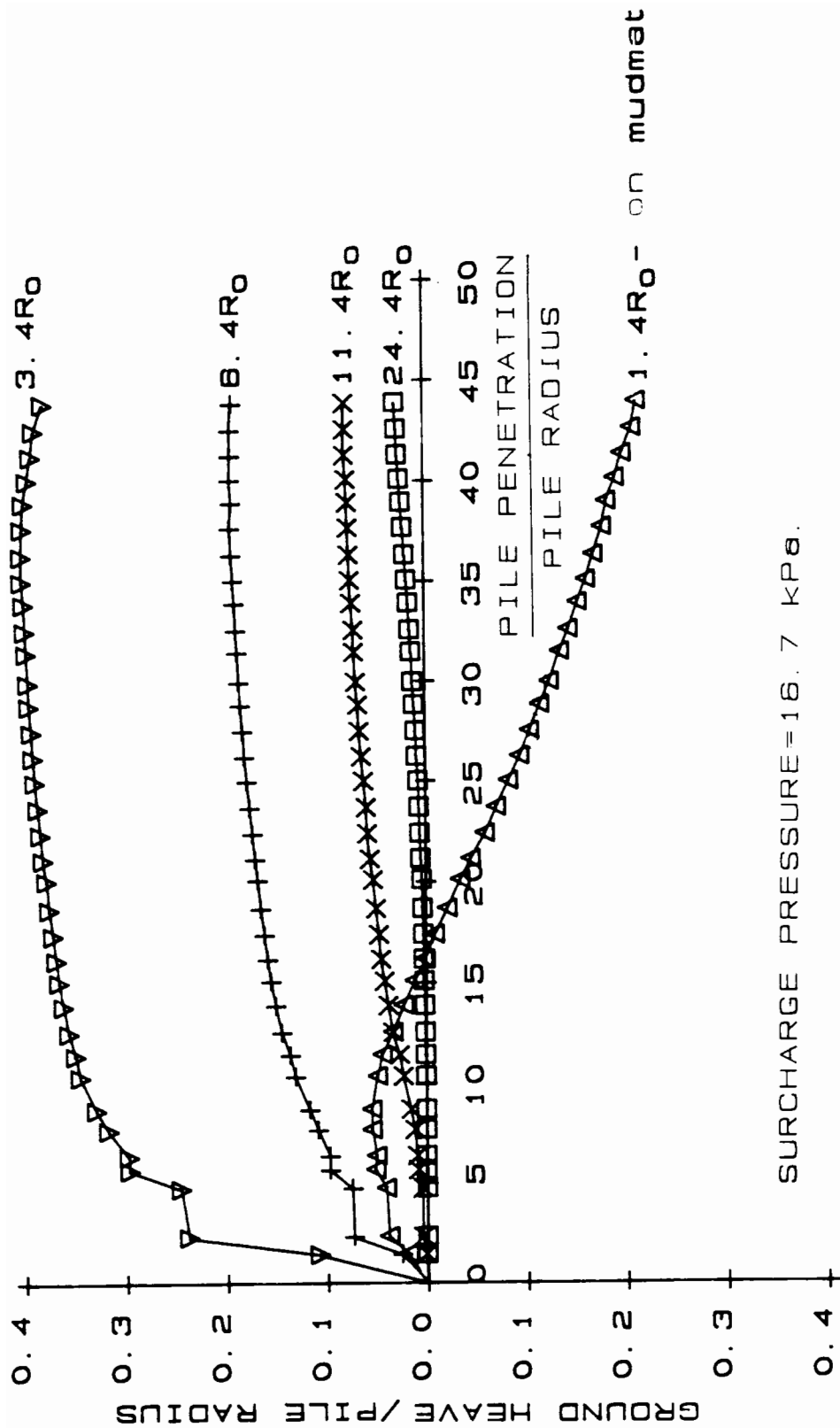


Figure 7.8 Variation of ground heave during driving for the first closed-ended pile driven through a surcharged mudmat-DC2G

at LVDT A was lifted up during the initial pile penetration, and subsequently settled at deeper penetration causing the mudmat to tilt towards this driven pile.

7.3.1 Interpretation of Pile Group Test Results

When a pile was driven through a rigid mudmat, the movements of the mudmat could be expressed by six quantities; three translations and three rotations. It was assumed that three of these are zero (two horizontal translation and a rotation about the axis) leaving three displacements which could be expressed as the settlement of the mudmat centre and its rotations about the two horizontal axes.

A more convenient way of defining the mudmat's movements would be to choose the three quantities as:

- (a) settlement of the mudmat centre.
- (b) magnitude of maximum rotation about any axis in the horizontal plane.
- (c) direction of maximum tilt.

These quantities were monitored by LVTDs measuring vertical displacements at the three points on the mudmat shown in plan view in Figure 4.8. The settlement of a measuring point with horizontal coordinates x, y is given by :

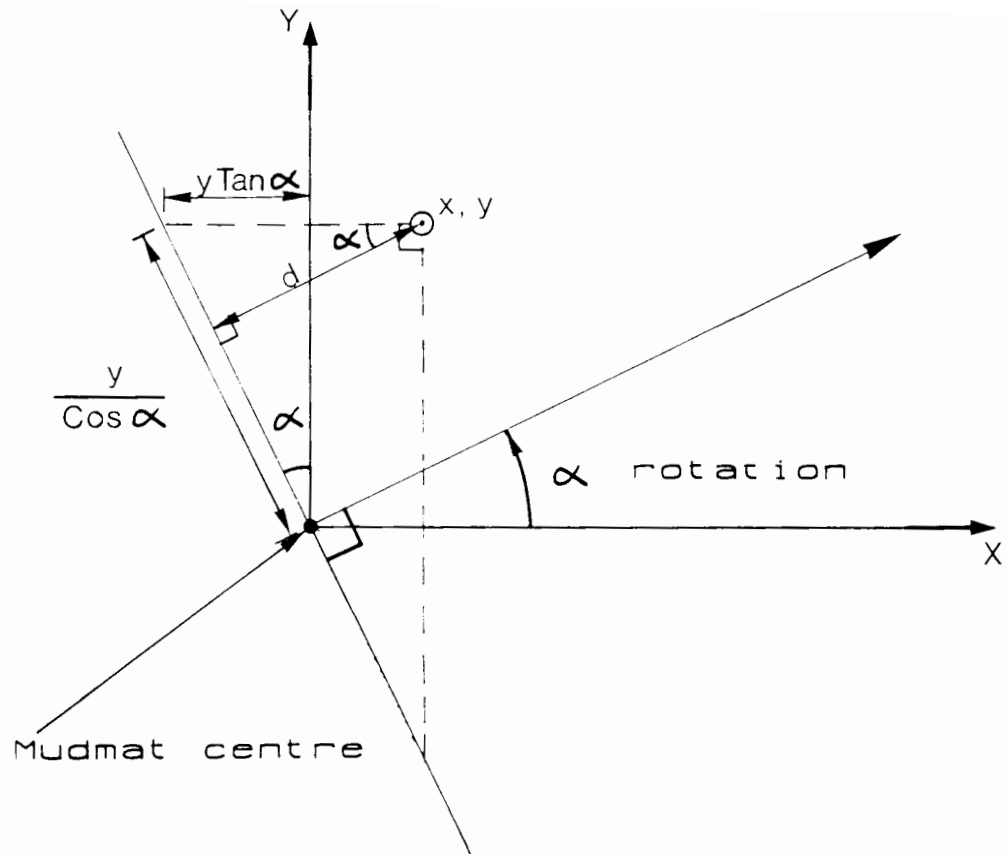


Figure 7.9 Definition of a displacement point on the mudmat (plan view)

$$s = \rho + d\theta$$

where d was the distance as shown in Figure 7.9, and

ρ settlement at the centre of the mudmat

θ magnitude of rotation

$$d = \cos \alpha (y \tan \alpha + x) = y \sin \alpha + x \cos \alpha$$

where α = direction of rotation (maximum tilt)

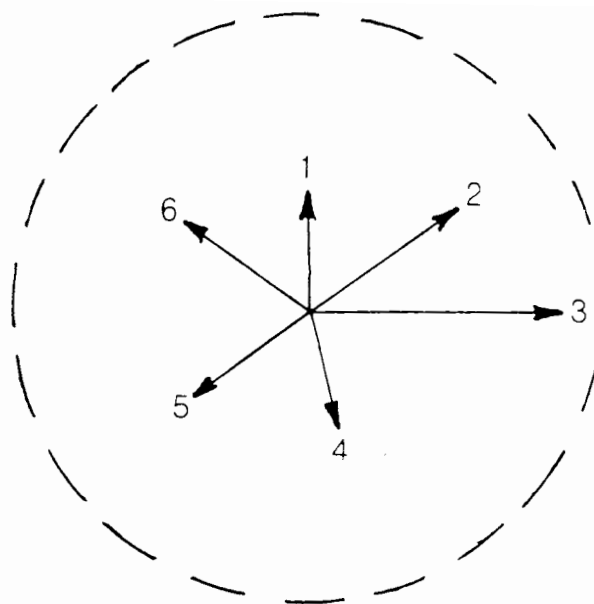
There are three unknowns which are the three assumed parameters, and the unknowns could then be solved.

Figure 7.10a shows the direction and magnitude of the maximum tilt due to the driving of each pile for test D01G. The mudmat tilted towards the direction in which the pile was driven. Figure 7.10b shows the vertical movements of the centre of the mudmat as each pile was driven.

Three different surcharge pressures were investigated for both closed- and open-ended piles. Figures 7.11a, b and c show the cumulative tilt for the three different surcharge pressures when closed-ended piles were used. The corresponding vertical movements of the centre of mudmat are shown in Figure 7.11d. The magnitude of tilt increased with the surcharge pressure. When the surcharge pressure was $0.5C_u$, the mudmat was lifted up by the ground heave generated. This caused it to tilt in the opposite direction to the side on which the first pile was driven. Subsequent pile driving did not tilt the mudmat in any particular direction because the magnitude of tilt was very small. In each case, the mudmat settled or lifted by almost equal increments for each pile driven (Figure 7.11d).

Similar behaviour was observed when open-ended piles were used (Figure 7.12) except that the magnitudes of tilt and vertical movement were significantly less for both low and high surcharge pressures.

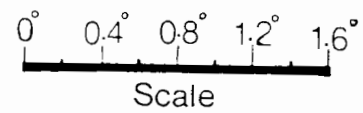
The results were further examined to highlight the influence of surcharge pressure on the mudmat movement at five depths of penetration for the first pile driven. Figure 7.13 shows



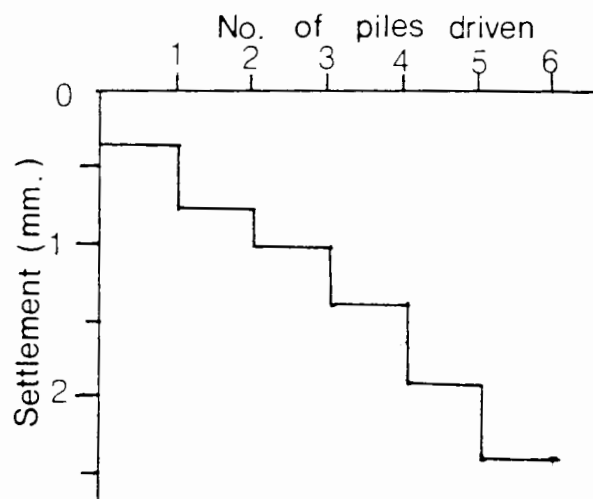
Arrow shows direction of maximum tilt due to driving of each pile.

Length of arrow is proportional to magnitude of tilt.

NOTE: MUDMAT TILTS IN THE DIRECTION OF EACH PILE AS IT IS DRIVEN.



(a) Mudmat rotation due to pile driving



(b) Mudmat central settlement due to pile driving

Figure 7.10 Movements of mudmat-D01G

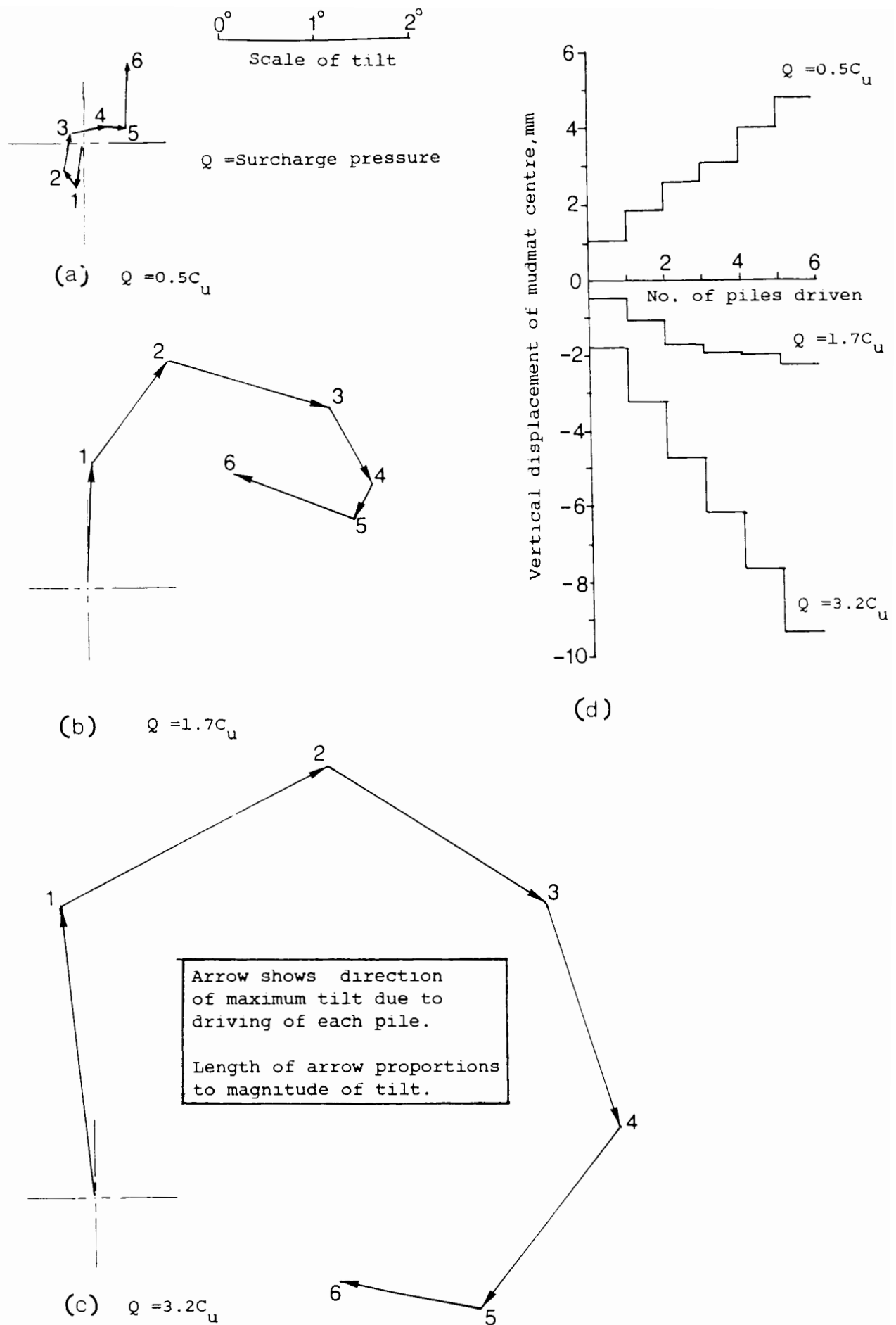


Figure 7.11 Pile driving with surcharged mudmat (closed-ended piles)

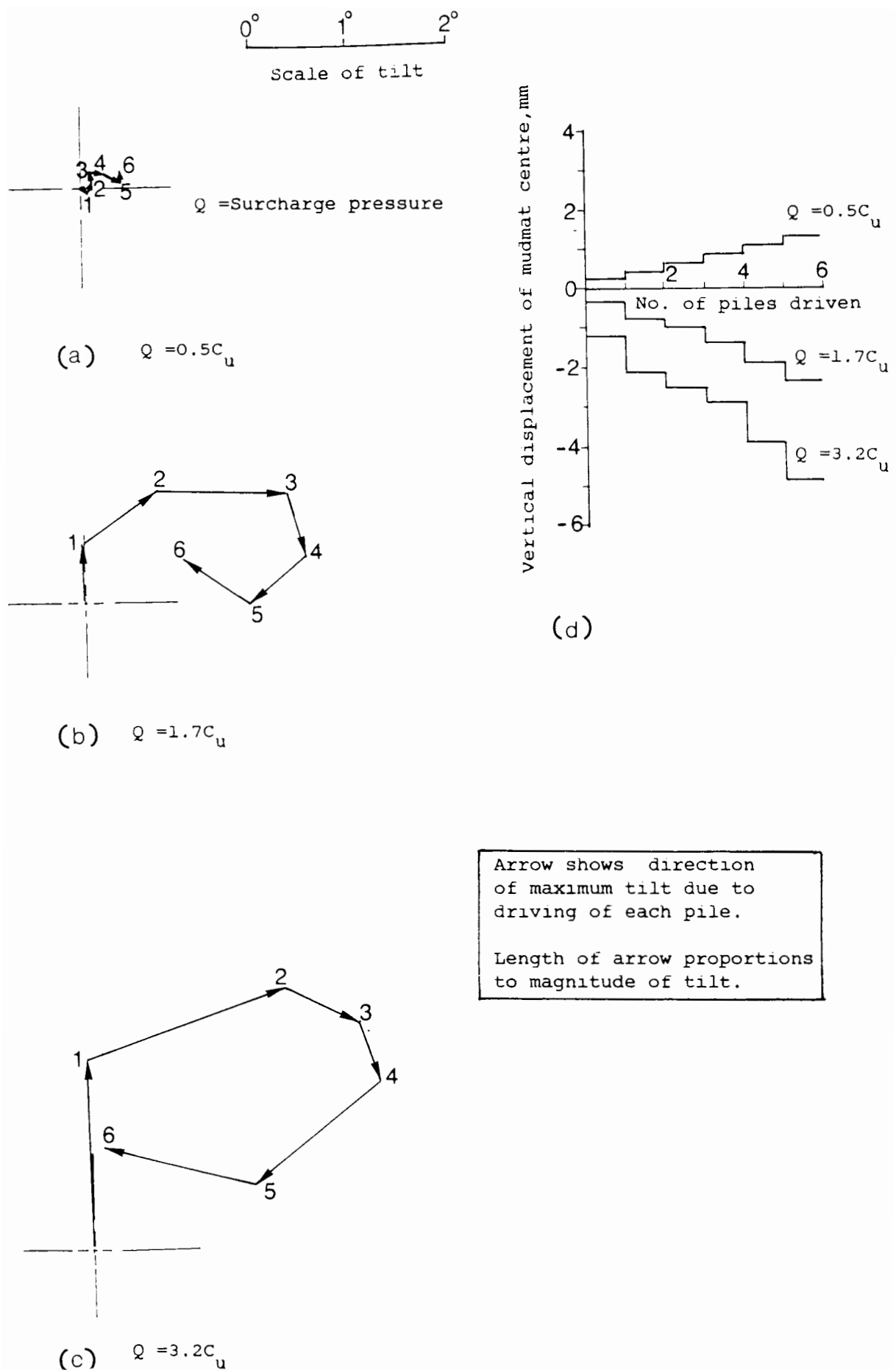


Figure 7.12 Pile driving with surcharged mudmat (open-ended piles)

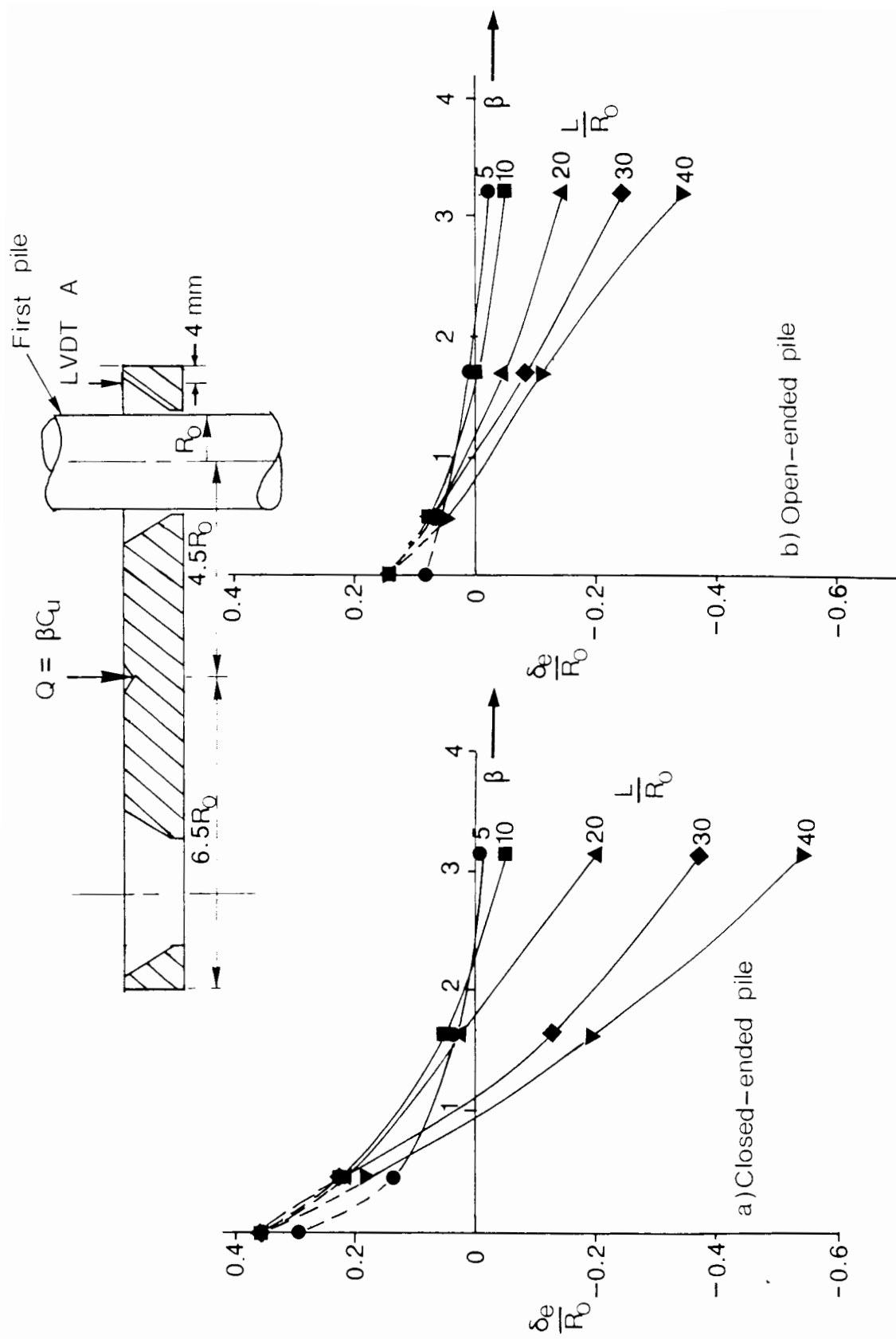


Figure 7.13 The influence of surcharge pressure on mudmat edge displacement for first pile driven

dimensionless charts for closed- and open-ended piles. The results from single pile tests were included to represent the $\beta = 0$ case. Both charts are similar except that the magnitude of settlement is larger for closed-ended piles.

The centre mudmat displacement and maximum tilt after driving all the piles to about $L/R_o = 45$ are shown in Figure 7.14 for closed- and open-ended piles. Both the vertical displacement of the mudmat centre and maximum tilt were larger when closed-ended piles were used.

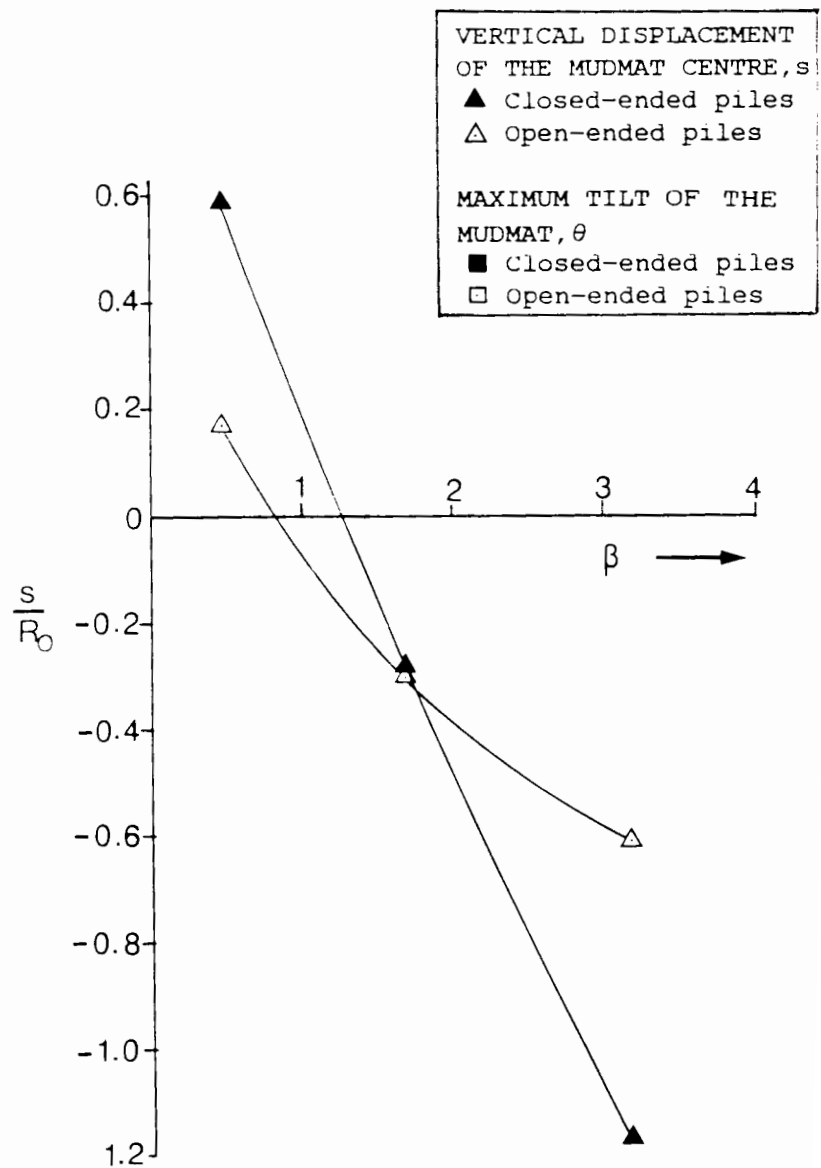
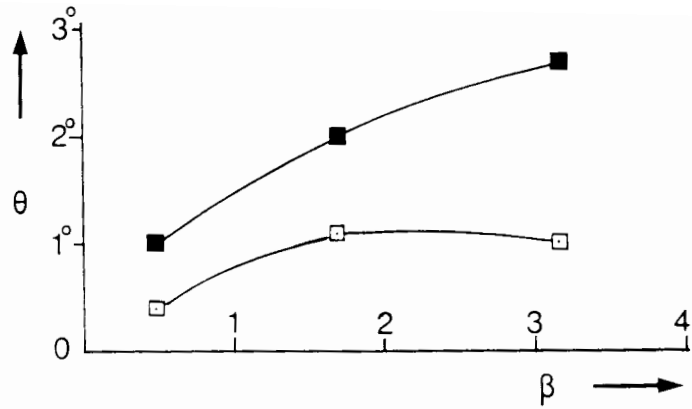


Figure 7.14 The influence of surcharge pressure on mudmat centre displacement and tilt after driving all six piles

CHAPTER 8

DISCUSSION AND CONCLUSIONS

8.1 Soil Properties

The material properties of the kaolin used in this investigation agree well with the properties reviewed in the literature.

8.2 Qualitative Semi-cylindrical Pile Tests

The qualitative results from the semi-cylindrical specimens have proved the usefulness of these tests. In addition, they also provided good agreement with the quantitative results obtained from the full cylindrical specimens (see Figure 6.53).

The movements of the soil as determined by this series of tests disagreed with the qualitative analysis of Zeevaert (1950) in which he assumed that the soil beyond $3R_0$ from the pile axis was unaltered. The results also contradict the theory of Hagerly and Peck (1971) which assumed no heave below $a-z$ (see Figure 2.5).

8.2.1 Vertical Soil Movements

The findings could also be compared with those tests carried out by Francescon (1983) and Steenfelt, Randolph and Wroth (1981). The boundary conditions of their specimens were different from the Author's. Their specimens were tested in a large triaxial chamber, the vertical and radial stresses being controlled through flexible rubber membranes. The piles were jacked into the specimens whilst maintaining the vertical and radial stresses. The soil movements found in this series can be compared with their results. Unfortunately, in both of these previous series of tests, the vertical movement of the soil was only measured up to a radial distance of $5R_0$ from the pile axis and so no upward soil movement was observed. However, Figure 6.5b shows that there are upward soil movements at much greater radial distances. This was also found in the vicinity of the pile tip (Figure 6.5c).

8.2.2 Radial Soil Movements

The radial soil movements observed in Author's tests around the mid-depth region of the pile could be compared with those obtained by Francescon (1983) and Steenfelt, Randolph and Wroth (1981). These researchers only measured radial movements up to a distance of 11 pile radii from the pile axis and have demonstrated good agreement with the plane strain analysis (Carter, Randolph and Wroth, 1980). Similar

results (see Figure 6.4b) for the mid-depth of the pile were obtained in this investigation.

8.3 Quantitative Single Pile Tests

8.3.1 Excess Pore Pressure Measurements

The excess pore pressures generated during driving appeared to be slightly greater than those found by Francescon (1983). Figure 8.1 shows the comparison of his results (for samples with OCRs of 4 and 8) with the results obtained in this investigation. Unfortunately, his results include no measurements of excess pore pressure beyond a radial distance of seven pile radii. Therefore, no comparison could be ^{made} with for the negative pore pressures measured in these tests at the cylinder wall (28 pile radii away from pile axis).

The generation of negative pore pressure is not in accordance with the plane strain analysis which idealised the pile penetration as the expansion of a cylindrical cavity. The measurements of the negative pore pressure were repeatable and were also confirmed by the results of Love (1984) who used specimens of rectangular cross-section.

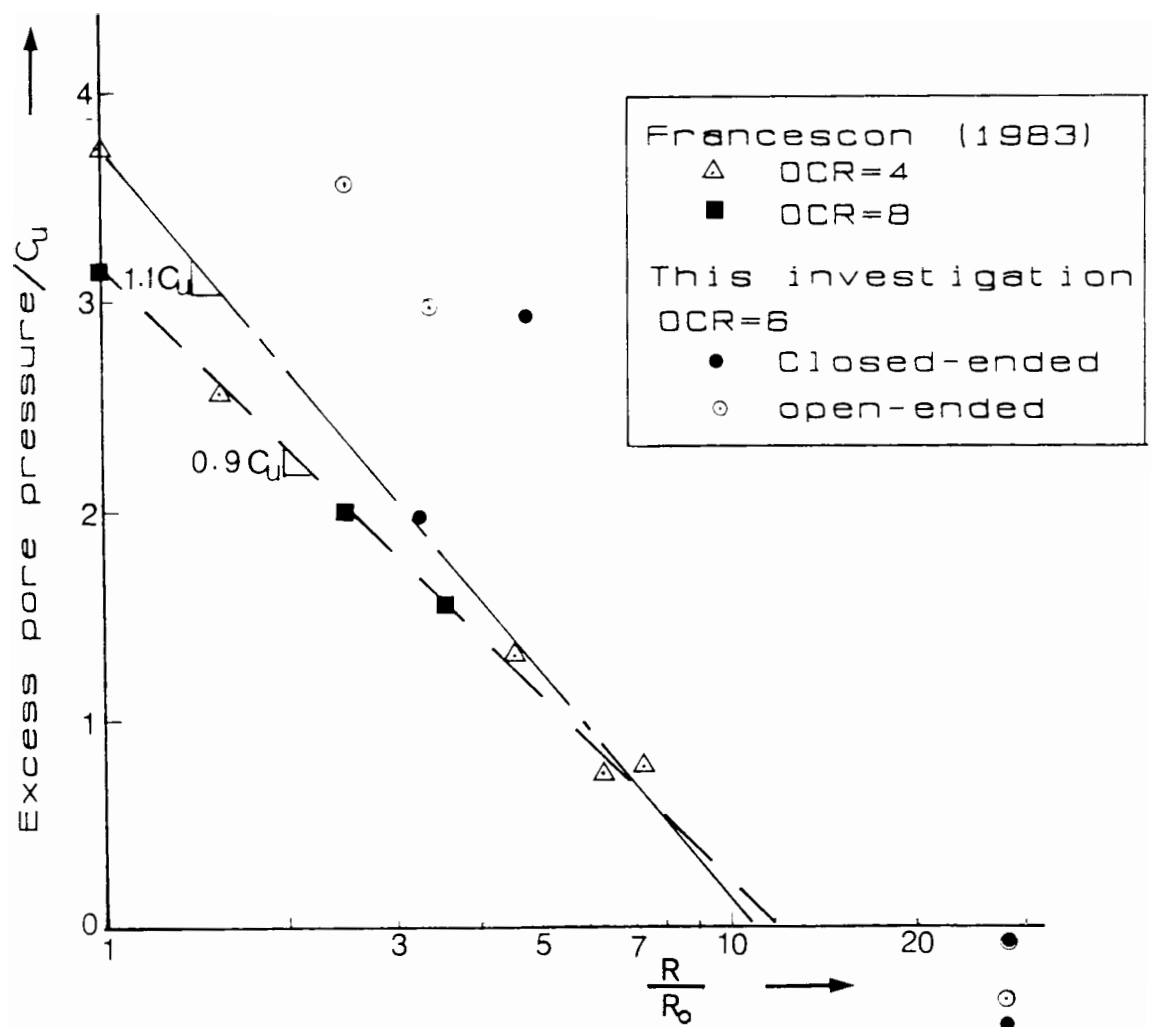


Figure 8.1 Excess pore pressure caused by pile installation

8.3.2 Driving Record

Rigden et al. (1979) found that a closed-ended pile was easier to drive than an open-ended pile. In contrast, the blow count determined in this investigation (see Figure 6.9) showed that model open-ended piles were slightly easier to drive than closed-ended piles. This discrepancy could be due to the difficulty of modelling the dynamic effects of pile driving in the laboratory.

Another possible reason could be the influence of plugging. The weight of the model open-ended pile with the plugged soil was about the same as the solid closed-ended pile. However, the large scale closed-ended pile was not solid, being fabricated from the same tubular sections as the open-ended pile but with a base plate welded to the tip of the pile. The weight of the large scale open-ended pile with the plugged soil was therefore significantly greater than that of the closed-ended pile. This could be the reason why the closed-ended pile was easier to drive than the open-ended pile as found by Rigden et al. (1979).

8.3.3 Ground Heave Measurements

The quantitative results of this investigation show that the ground heave extends to a radial distance of greater than $30R_o$. A possible reason why Cummings et al. (1950) and

Orrje and Broms (1967) obtained only about 50% for the ratio of the total volume of ground heave to the total volume of embedded piles is that the measurements of ground heave were not extended far enough from the pile.

The limitations of soil modelling have already been discussed (see section 3.1), and before any of the dimensionless plots presented in Chapters 6 and 7 are used to predict prototype behaviour, they must be verified by field studies. However there is very little published information on the ground heave generated around driven piles in clay and none on the influence of a surcharged foundation surrounding the piles. The only available good quality ground heave measurements around driven piles in clay were obtained from Sharp (1982). These results were replotted in a dimensionless form so that they could be compared with the experimental results. Figure 8.2 shows the remarkably good agreement that was obtained between the ground heave generated by closed-ended piles of two different diameters with the experimental results.

The laboratory results (see Figure 6.19) showed that the clay was deformed rapidly in an undrained manner, so that the volume of ground heave generated agreed with the volume of pile inserted. However, Figure 8.3 shows that for the field tests, the volume of the ground heave appeared to disagree with the volume of pile inserted once the closed-ended piles had penetrated beyond 30 pile radii. Possible reasons for this may be found by first examining

CLOSED-ENDED PILE

Δ	18 mm dia. pile
+	203 mm dia. pile
X	305 mm dia. pile

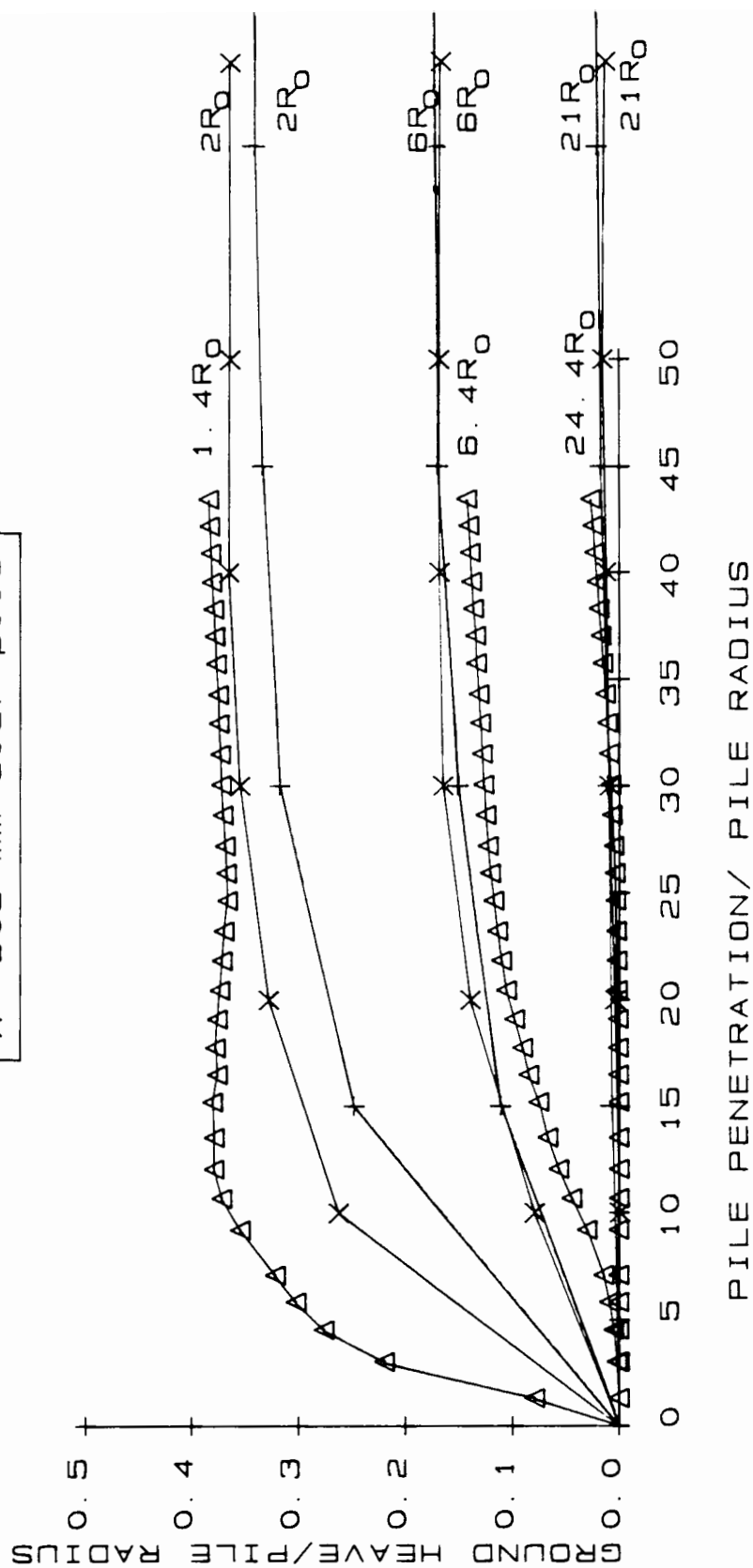


Figure 8.2 Comparison of field tests with experiment results

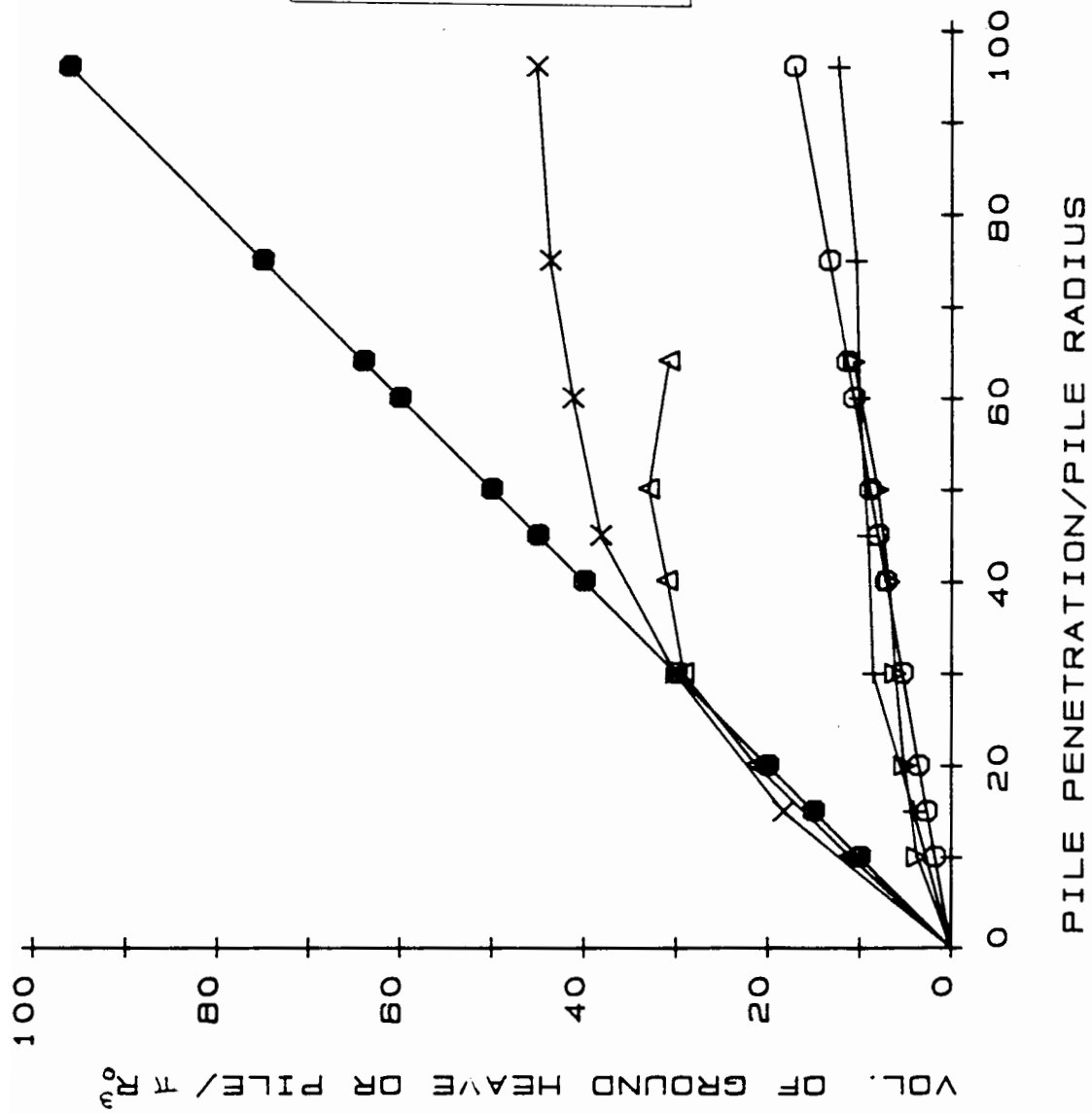


Figure 8.3 Volumes of ground heave from field tests (replotted from Sharp, 1982)

the contribution of the volume of ground heave at each measuring point in the laboratory tests. Pappus's theorem for volume as described in section 6.3.3 shows that the volume of ground heave at any measuring point is a function of ground heave and the radial distance from the pile at that point. Figure 8.4 shows the dimensionless plot of the contribution of volume at each radial distance for a laboratory specimen. The volume contribution at $1.4R_0$ was less than at $3.4R_0$ because the ratio of the magnitude of ground heave at $3.4R_0$ to that at $1.4R_0$ was about 0.7 (see Figure 6.15), but the ratio of the radial distance was 2.4. Although the ground heave at greater radial distances is small, the volume contribution is large. It appears that if the pile penetration increased beyond that investigated, most of the total volume of ground heave would be contributed by the ground heave at large radial distances. This is shown by the ground heave at $24.4R_0$ which increased rapidly at deeper pile penetrations.

The volume contribution at five measuring points from the field test of a closed-ended pile is shown in Figure 8.5. The ground heave at $31R_0$ does not make as significant a contribution as that at $24.4R_0$ in the laboratory tests. This may be because the cylinder wall is at 28 pile radii away from the pile axis which restricts any further radial

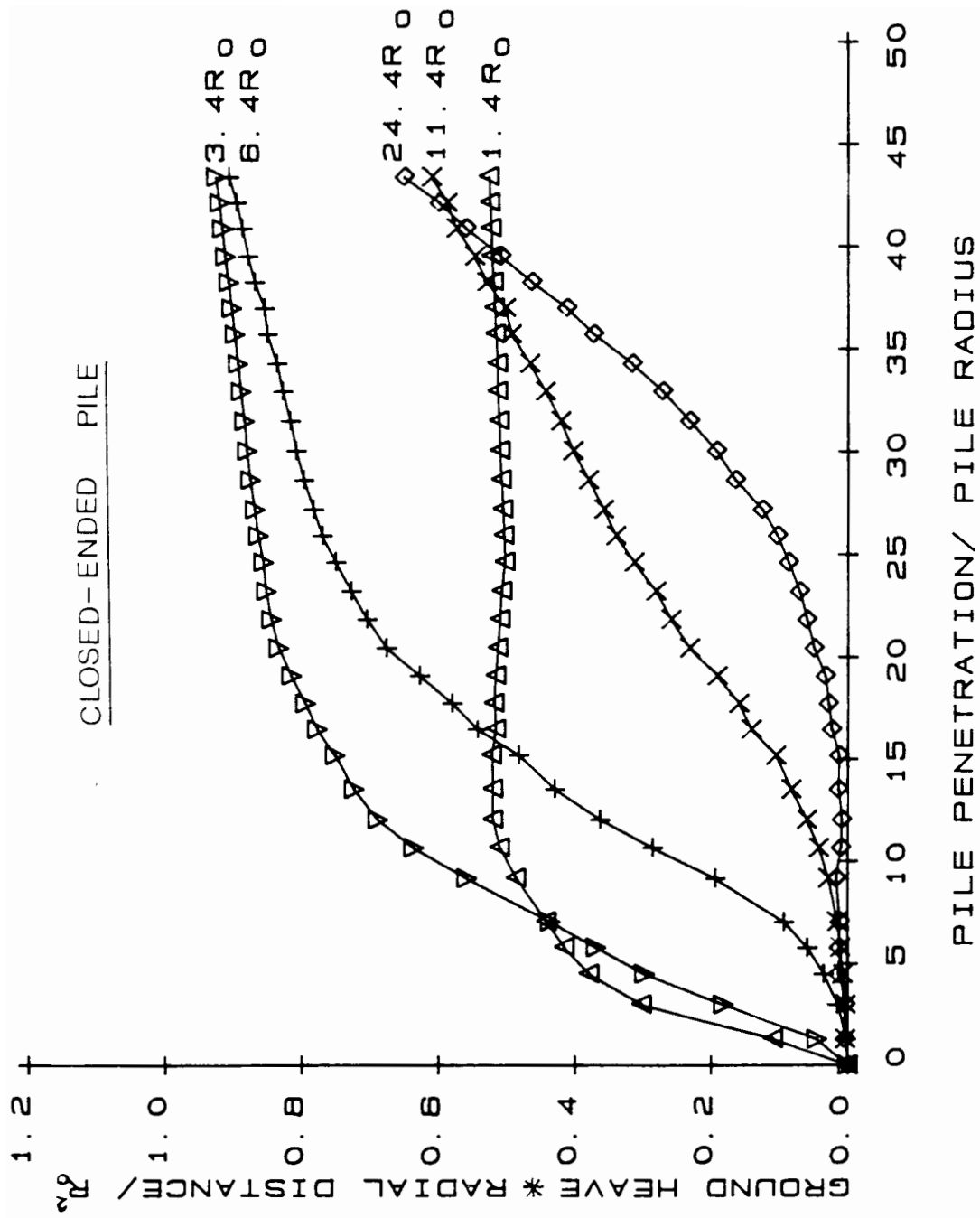


Figure 8.4 Contribution to the total volume at various radial distances-7C3

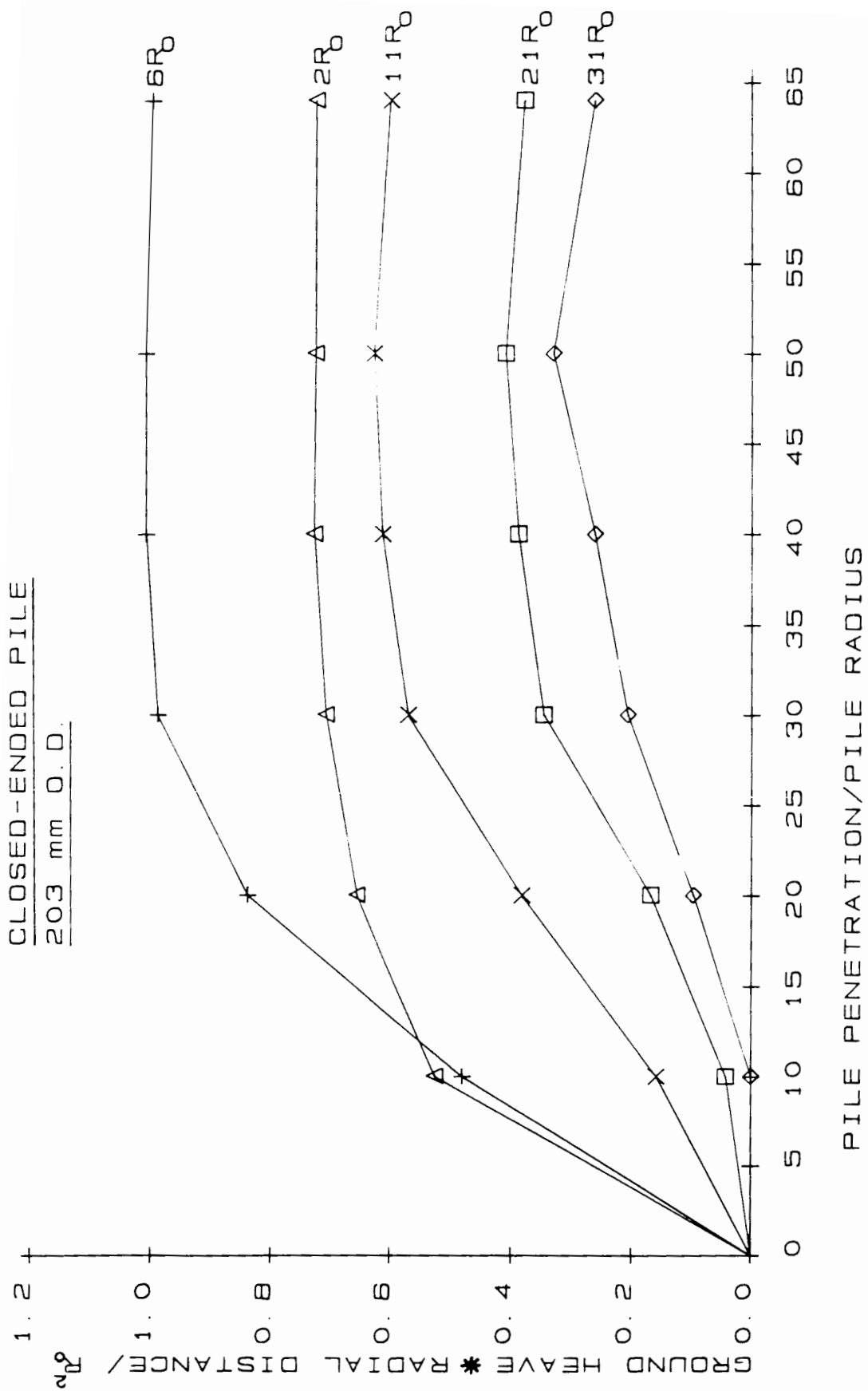


Figure 8.5 Contribution to the total volume for field tests

movement of soil. The volume contributions at $21R_0$ and $31R_0$ for the field tests appear to decrease at deeper pile penetrations. This may be because the measurements of the small magnitude of ground heave at greater radial distance are not accurate enough.

There were no ground heave measurements in the field test beyond a radial distance of $31R_0$, and this coupled with the lack of accuracy mentioned above might be reasons for the disagreement.

The ground heave was largely unaffected by the stress histories and the rate of pile driving. The overriding factor is that as a pile is driven into the ground, soil has to be displaced. There are limited field data for comparison with the experimental results. However, a good agreement between the field data and experimental results was obtained when both were made dimensionless. The dimensionless plots could therefore be used by practising engineers to predict the ground heave due to pile driving.

8.3.4 Comparison of Finite Element Analysis with Experimental Results

An attempt was made to model ground heave behaviour by S.W. Sloan using his finite element program developed at

University of Cambridge (Sloan,1981). The purpose was to attempt to match a numerical model with the experimental results, so as to improve understanding of the problem. If this proved successful, it could be used to scale up the laboratory results to the prototype pile size.

The numerical model assumed the clay as an isotropic, elastic and perfectly plastic material. Tresca yield criteria and associated flow rule were applied. The G/C_u was assumed to be 150 with a Poisson's ratio, $\nu = 0.499$ to simulate undrained conditions using small strain analysis.

It was envisaged that it would be impractical to carry out the numerical analysis to model the pile driving as a continuous shearing process because of the complexity of this computation. Therefore the pile was modelled as a circular footing. The surface vertical displacement (ground heave) was calculated as it was brought to failure. Despite the fact that the program was not specifically designed for this problem, the failure load was predicted with good accuracy. However, the ground heave was very small, and when this was divided by the incremental displacement which was also small, it produced significant inaccuracy as shown in Table 9.1.

The prohibitive computing time required for these analyses and the poor accuracy of the ground heave restricted the extent to which a ground heave profile could be plotted.

TABLE 8.1 COMPARISON OF GRADIENT OF GROUND HEAVE BETWEEN FINITE ELEMENT AND EXPERIMENTAL RESULTS

DEPTH	RADIAL DISTANCE FROM PILE AXIS					
		$1.4R_o$	$3.4R_o$	$6.4R_o$	$11.4R_o$	$24.4R_o$
OR_o	α_{FE}	$0.44 \pm 2 \times 10^{-2}$	$-2 \times 10^{-3} \pm 2 \times 10^{-3}$	$4 \times 10^{-4} \pm 2 \times 10^{-4}$	$2 \times 10^{-4} \pm 2 \times 10^{-4}$	$2 \times 10^{-4} \pm 2 \times 10^{-4}$
	α_{FE}	$0.11 \pm 3 \times 10^{-4}$	$0.02 \pm 3 \times 10^{-4}$	$2.6 \times 10^{-3} \pm 3 \times 10^{-5}$	$3.2 \times 10^{-4} \pm 3 \times 10^{-5}$	$1.3 \times 10^{-4} \pm 3 \times 10^{-5}$
$1R_o$	α_{EX}	$0.074 \pm 5 \times 10^{-5}$	$0.02 \pm 2 \times 10^{-4}$	$8 \times 10^{-4} \pm 2 \times 10^{-4}$	$9 \times 10^{-5} \pm 2 \times 10^{-5}$	$8 \times 10^{-5} \pm 2 \times 10^{-5}$
	α_{FE}	0 ± 0.25	$0 \pm 3 \times 10^{-2}$	$-2.5 \times 10^{-3} \pm 3 \times 10^{-3}$	$2.52 \times 10^{-3} \pm 3 \times 10^{-3}$	$2.5 \times 10^{-2} \pm 3 \times 10^{-2}$
$5R_o$	α_{EX}	$0.02 \pm 1 \times 10^{-3}$	$0.16 \pm 1 \times 10^{-4}$	$2.9 \times 10^{-3} \pm 2 \times 10^{-4}$	$2.8 \times 10^{-4} \pm 1 \times 10^{-5}$	$-3 \times 10^{-5} \pm 1 \times 10^{-4}$
	α_{FE}	$-6.6 \times 10^{-2} \pm 2 \times 10^{-2}$	$-1.7 \times 10^{-2} \pm 2 \times 10^{-2}$	$1.66 \times 10^{-3} \pm 2 \times 10^{-3}$	$1.7 \times 10^{-3} \pm 2 \times 10^{-3}$	$1.7 \times 10^{-3} \pm 2 \times 10^{-3}$
$35R_o$	α_{EX}	$1.35 \times 10^{-3} \pm 1 \times 10^{-4}$	$9.38 \times 10^{-4} \pm 1 \times 10^{-4}$	$1.54 \times 10^{-3} \pm 1 \times 10^{-4}$	$1.7 \times 10^{-3} \pm 1 \times 10^{-5}$	$1.62 \times 10^{-3} \pm 1 \times 10^{-5}$

α = Gradient of ground heave
 $\left[\frac{\text{incremental ground heave}}{\text{incremental pile displacement}} \right]$

α_{FE} = finite element results

α_{EX} = experimental results

8.4 Pile Driving with Surcharge Pressure

The most startling finding in this series of tests was that at high bearing pressures, the surcharged collar and model mudmat settled. When the collar was subjected to a high pressure, the soil beneath the collar was highly stressed. Pile driving through the collar increased the stress on this soil and when the sum of these stresses exceeded the bearing capacity of the soil, the collar settled. The settlement of the surcharged collar caused by the bearing capacity failure is much greater than any ground heave generated.

In the case of the mudmat, when the first pile was driven through it, the stress of the soil beneath the mudmat in the region of the pile increased dramatically. The mudmat therefore settled as the stress approached the soil's bearing capacity which caused it to tilt towards the direction of the driven pile. Similar mechanisms occurred when each subsequent pile was driven. The final tilt of the mudmat when all the piles had been driven was therefore small but the total settlement was large.

There were no field data to compare with the experimental results of this series of tests. Nevertheless, the dimensionless results of single pile tests have shown that they could be scaled up with confidence as discussed in the previous section. Therefore, the dimensionless charts shown in Figures 7.7, 7.13 and 7.14 for the surcharged collar and

model mudmat will be useful to engineers who wish to determine the influence of pile driving next to a surcharged foundation. In particular, offshore geotechnical engineers can use Figure 7.13 to determine the influence of surcharge pressure on the vertical mudmat displacement next to the first piles as it is driven. Figure 7.14 demonstrates the final vertical movement at the centre of the mudmat and the maximum tilt after all six piles had been driven to a depth of about $L/R_o = 45$.

Pile driving through a surcharged mudmat should not give rise to problems due to ground heave, but settlement may be a problem.

8.5 Summary of Findings

The principal findings of this investigation are:

1. The pile driving apparatus developed specifically for this investigation worked entirely satisfactorily.
2. The experiments have proved to be successful; accurate ground heave measurements were obtained with good repeatability.
3. The semi-cylindrical specimens have proved their usefulness in providing qualitative results and also some quantitative results.

4. Qualitative results from the semi-cylindrical specimens showed that the length of plugged soil inside the open-ended piles is not a constant but increased with pile penetration.
5. The dynamic effects of pile driving cannot be modelled in the laboratory.
6. The ground heave appeared to be insensitive to stress histories and the rate of pile driving. During the initial pile penetration, most of the ground heave was generated near the pile. However, at large penetrations, most of the ground heave was generated further away from the pile. The soil specimen was deformed rapidly in an undrained manner with no change in volume.
7. The ground heave generated around the model closed ended piles was about four times that of open-ended piles. This factor represents the ratio of the cross-sectional areas of the two types of pile.
8. Open-ended piles driven with and without driving shoes showed no significant difference in the ground heave generated. However, the volume of ground heave increased dramatically when the pile began to plug. Piles driven with a driving shoe prevented the plugging.

9. The ground heave generated by the closed-ended and open-ended piles did not lift up the surcharged collar or mudmat during pile driving. On the contrary, the surcharged collar and mudmat generally settled during pile driving except for very low surcharge pressures $< 0.5C_u$. The surcharged mudmat generally tilted in the direction of the pile being driven. The maximum tilt when all the six piles were driven was small.
10. There were limited field data to compare with the experimental results. However, the field data supplied by Sharp (1982) agree well with the experimental results both in the magnitude and the shape of the profile of ground heave.

APPENDIX A

CALIBRATIONS OF APPARATUS

This appendix describes the calibrations of the apparatus used in this investigation. These include the hydraulic rams of the test specimen preparation assemblies, displacement transducers, pore pressure transducers and vane shear apparatus.

a) Hydraulic Rams

The hydraulic rams were calibrated using a proving ring before the 2nd. series of test specimen were prepared. The purpose of this calibration was to determine the losses due to the hydraulic fittings and friction in the piston of each ram. Fig. A-1 shows the calibrations for the three rams. The theoretical load shown is the product of the pressure in the hydraulic line and the area of the ram on which the pressure acts. The difference between the calibrated and the theoretical load is a measure of frictional losses.

Each of the hydraulic rams was also calibrated with the top platen (see Fig. 3.2) attached to check the frictional losses of the seal as the platen moves inside the top consolidation cylinder. It was found that the

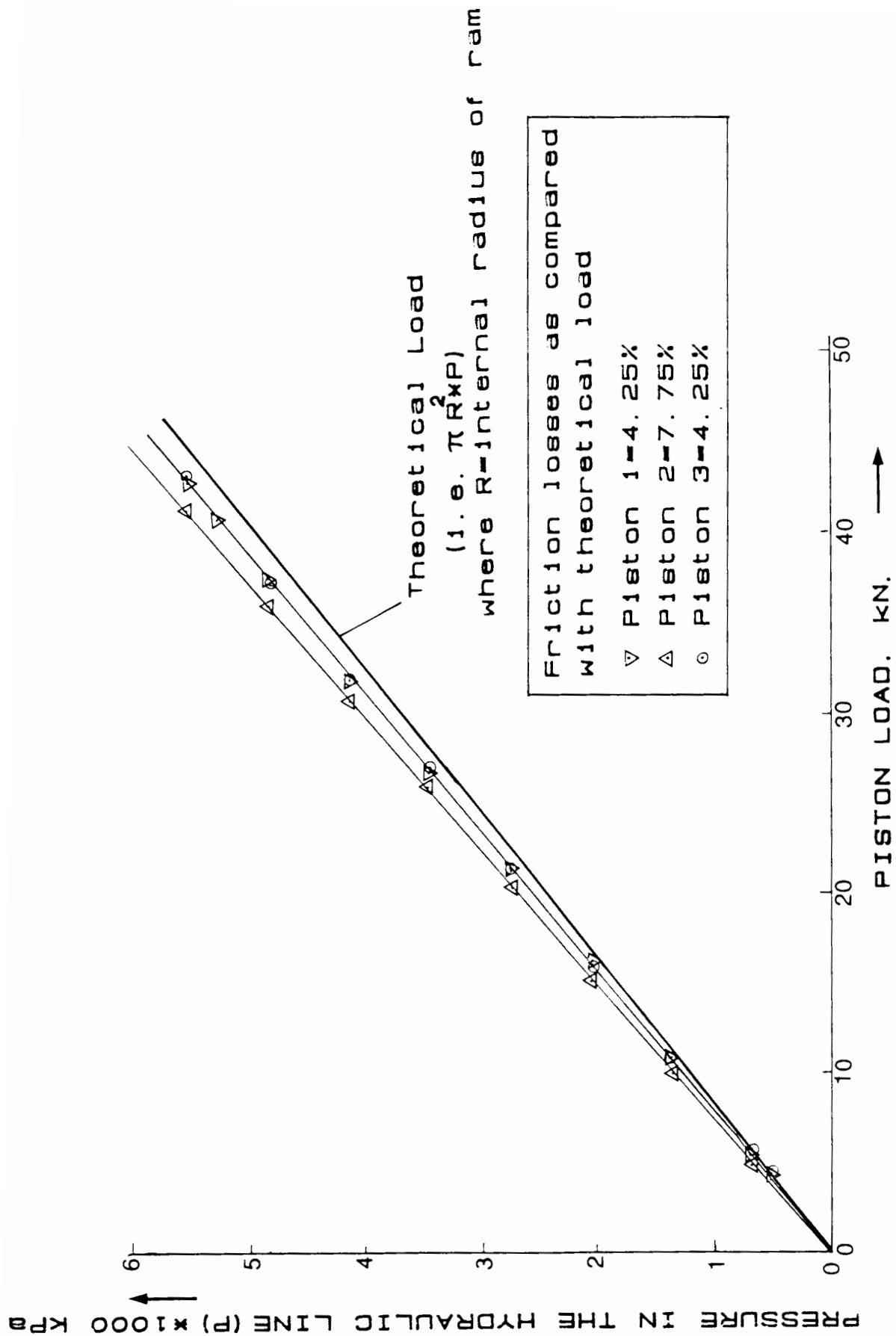


Figure A.1 Calibrations of the hydraulic rams

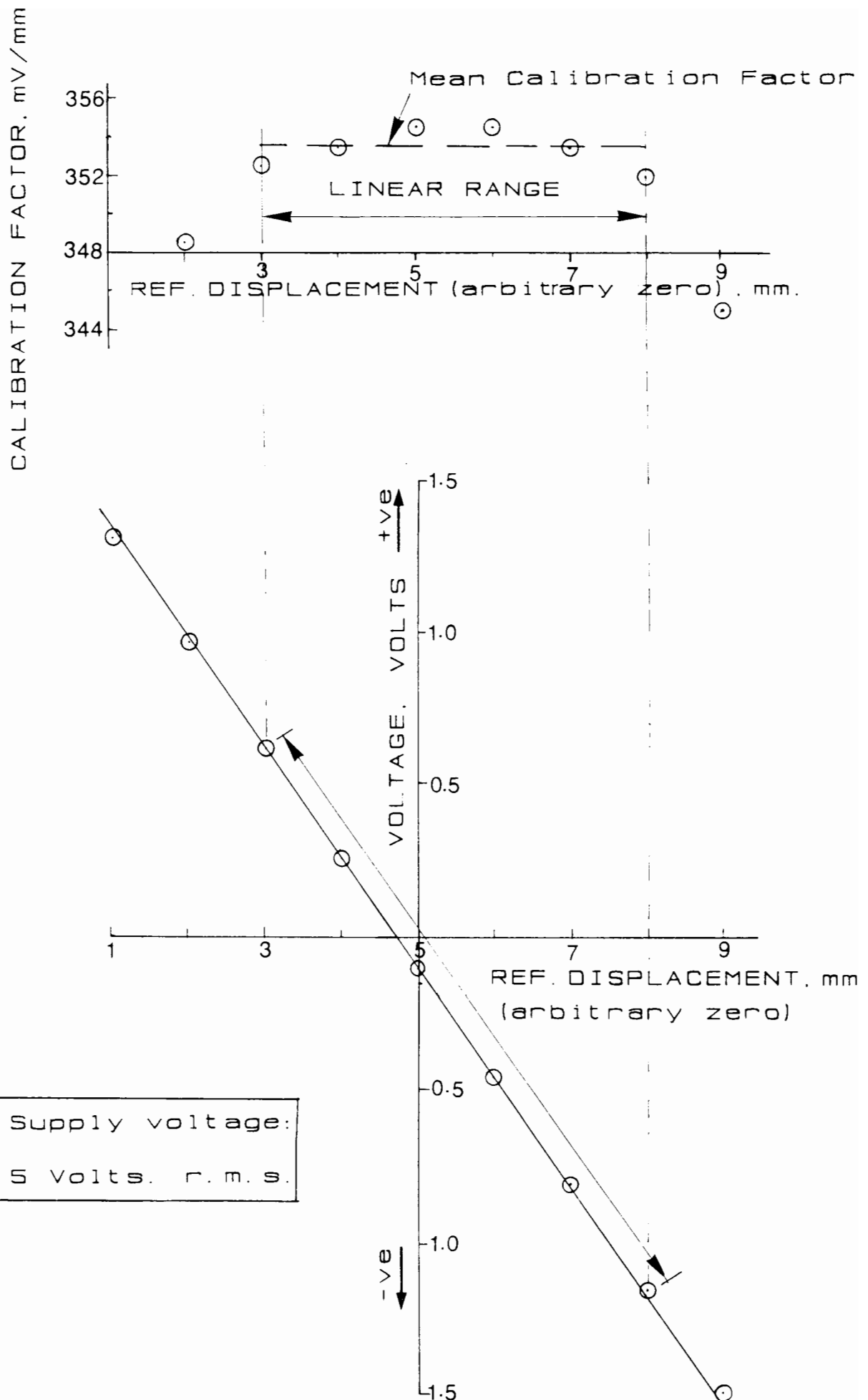


Figure A.2 Calibration of a LVDT (short stroke)

losses due to the seal were negligible and would not be significant when plotted on the Fig. A-1.

b) Displacement Transducers (LVDTs)

Both the long and short stroke LVDTs were calibrated before each series of tests. The short stroke LVDTs were calibrated using a micrometer. Fig. A-2 shows a typical calibration for a short stroke LVDT. The accuracy within the linear range of ± 2.5 mm. is about 0.25%. A better accuracy could be obtained if the linear range was reduced to ± 2.0 mm.

The influence of wetting the armature of a LVDT on the calibration factor was carried out. The armature was dipped into water before introducing it into the LVDT's housing. This process of dipping did not affect the calibration factor.

The long stroke LVDT was calibrated using a metre rule. Fig. A-3 shows calibration, and the accuracy within the range of ± 200.0 mm. is about 1%.

c) Pore Pressure Transducers

The pore pressure transducers were calibrated before the preparation of test specimens. A standard digital pressure indicator which in turn was regularly calibrated by the manufacturer was used for this purpose. Nitrogen

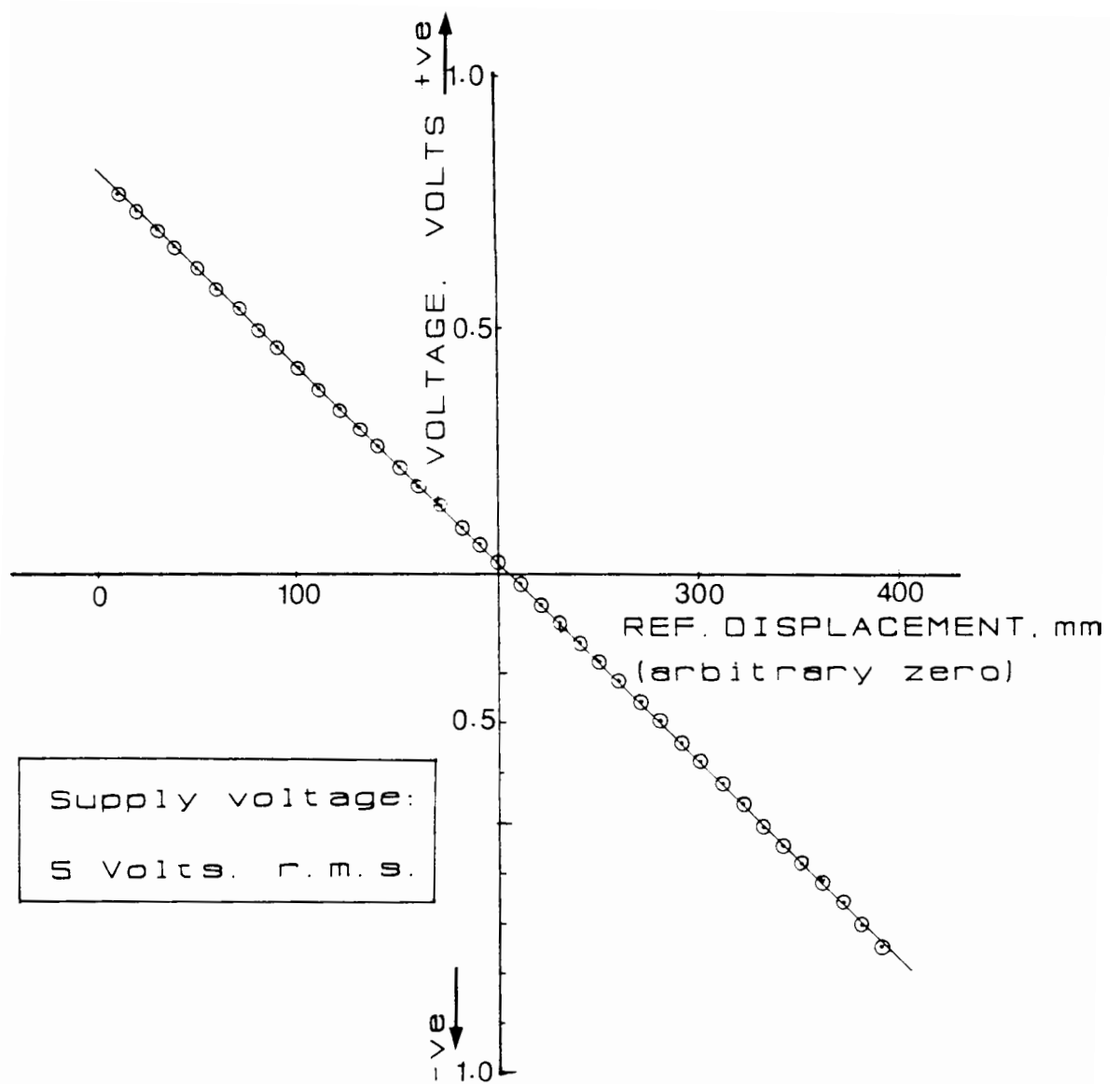


Figure A.3 Calibration of a LVDT (long stroke)

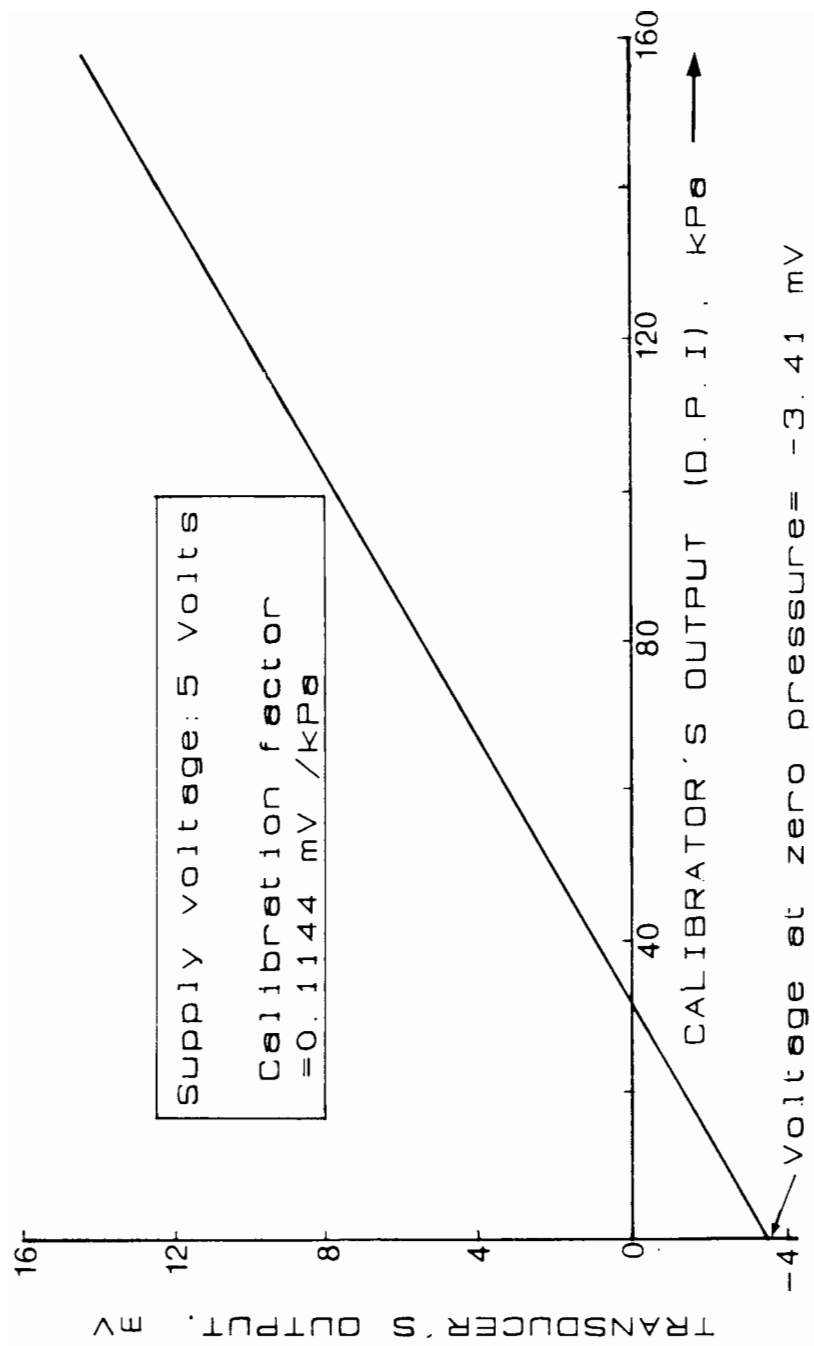


Figure A.4 Calibration of a pore pressure transducer

was used to apply the pressure. A chart recorder was used to monitor the calibration, together with readings taken manually at five different pressures as a check. Fig. A-4 shows the calibration of a transducer.

The transducers were also calibrated under a vacuum of 50 kPa. The calibration factor was the same as for the positive pressure.

d) Laboratory Vane Shear Apparatus

The laboratory vane shear apparatus was calibrated using standard weights hanging through a pulley system to eliminate bending as the torque was applied. Fig. A-5 shows the calibration with the theoretical line derived from B.S. 1377. Taking B.S. 1377 as a reference for comparison the vane shear readings have to be multiplied by a constant of 1.22. Serota and Jangle (1972) calibrated a vane of the same size and found the constant to be 1.145.

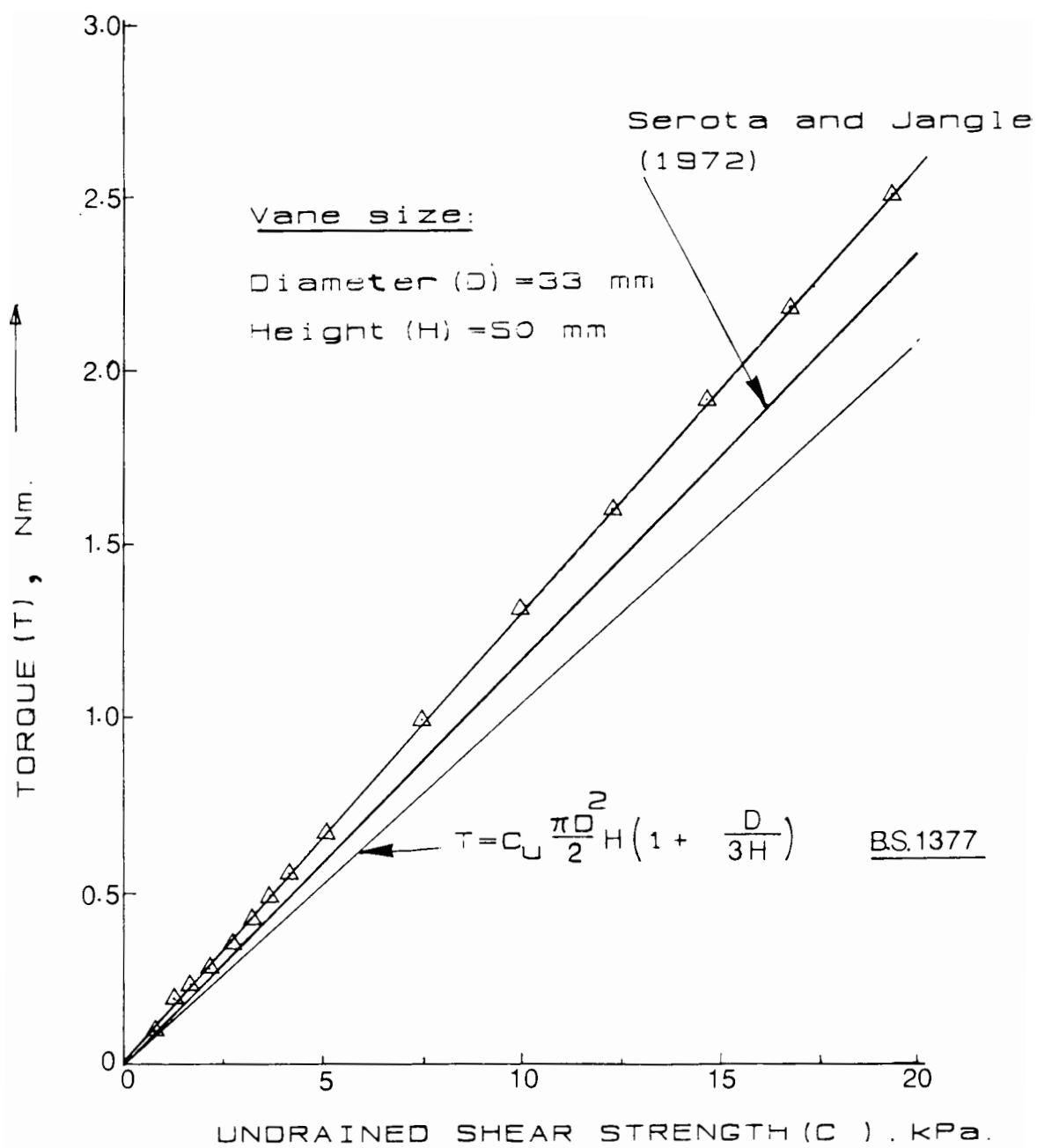


Figure A.5 Calibrations of a vane shear apparatus

APPENDIX B

MANUFACTURERS OF SPECIAL EQUIPMENT AND COMPONENTS USED IN THE INVESTIGATION.

PRODUCT	COMPANY
Accumulator-Air/oil interface	Fawcett Engineering, Bromborough Merseyside, England.
Clutch Bearing (oneway sprat)	Bearing Services Ltd., Oxford, England.
Summagraphics Digitizer BITPAD One (290 mm. square)	T.D. S. Ltd., Blackburn, Lancashire.
Displacement Transducers (LVDTs & Amplifiers)	R.D.P. Electronics, Wolverhampton, England.
Enlarger (De Vere 203)	De Vere Ltd., Beckenham, Kent.
Pressurized Nitrogen Gas (Grade Zero)	B.O.C. Ltd., London, England.
Pressure Gauge	Lucas Fluid Power Ltd., Torrington Avenue, Coventry.
Silicon Grease Molykote 33 Medium	B.D.H. Chemicals, Atherstone, Warwick.
Hydraulics Assembly	Lucas Fluid Power Ltd., Coventry.
Expandable Lab. Jacks (80 mm x 80 mm)	Scientific House, East London.
Speswhite Kaolin	E.C.C. International Ltd., St. Austell, Cornwall.
Variable Speed Motor and Thyristor Controller	Parvalux Electric Motors Ltd., Bournemouth, England.

Porous Plastic (Vyon 'F')
4.75 mm Thick

Porvair Ltd.,
Norfolk, England.

Pneumatic Pump
(Pageboy SFD 15)

M S P Planet Ltd.,
Bicester, Oxon, England.

Slurry Pump

Kecol Pump Ltd.,
Bridgnorth, Shropshire.

Chart Recorder
Servocorder

Environmental Equipment Ltd.,
Environ House, Nantwich, Cheshire.

Best Red Tubing
5/32" i.d. x 1/16"

W. Mannering & Co. Ltd.,
South East London.

Pressure Regulator
(Spectrol XX-87)

B.O.C. Ltd.,
London, England.

1/8" Thick Best Black
Rubber Sheet

W. Mannering & Co. Ltd.,
South East London.

Voltmeter and Scanner

The Solartron Electronic Group
Ltd., Farnborough, Hampshire.

Stainless Steel Tube
17.46 mm x 1.63 mm

Vazgrove Ltd.,
Wooburn Green, Buckinghamshire.

Miniature Pore Water
Transducers.

Druck Ltd.,
Groby, Leicestershire.

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