

THE AXIAL CAPACITY OF DRIVEN PILES IN CLAY

by

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ABSTRACT

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An instrumented model pile was used to investigate the fundamental behaviour in clay soils of driven cylindrical steel piles used for offshore structures. Four test-bed sites were chosen; two in stiff heavily overconsolidated clays, and two in normally/lightly overconsolidated clays.

Data from these sites confirm that a residual shear surface is formed along the pile during installation, the location of which relative to the shaft surface appears to depend on the shaft roughness. Comparisons with other site investigation data and cavity expansion theoretical predictions indicate that stress relief immediately behind the pile tip during driving gives rise to total radial stresses and pore pressures measured on the pile shaft which are lower than predicted. This stress relief is particularly severe in the stiffer clays. The data did however show that the installation total radial stresses and pore pressures are governed by the initial in-situ stresses and undrained shear strength as is predicted by the theory.

During reconsolidation, pore pressures close to the instrument rise initially in all clays, and radial effective stresses drop. The slow recovery in radial effective stress during the later stages of reconsolidation was in some cases insufficient to return it to levels recorded during installation. However, the generation of negative pore pressures during undrained loading increased the radial effective stress and shaft friction at failure. This effect is particularly important in the normally consolidated clays, and is responsible for the set-up of shaft capacity seen in such clays, which might not be observed if the loading were drained. The observed behaviour was therefore quite different from the monotonic increase in radial effective stress during reconsolidation, followed by decrease during undrained loading which was expected from a review of current theory.

PREFACE

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NOMENCLATURE USED IN TEXT

α	factor relating unit shaft friction to undrained shear strength
β	factor relating unit shaft friction to initial vertical effective stress
δ	deflection of the face of a total radial stress transducer
Δp	change in mean normal total stress due to pile installation
$\Delta p'$	change in mean normal effective stress due to pile installation
Δu_r	excess pore pressure generated by pile installation at radius r from pile centreline
Δu_{sh}	excess pore pressure generated on pile shaft by pile installation
Δu_τ	excess pore pressure at non-dimensionalised time instant τ during reconsolidation
κ	gradient of swelling line in $V:\ln p'$ space
λ	gradient of normal consolidation line in $V:\ln p'$ space
M	gradient of critical state line in $p':q$ space
M_{ps}	M for plane strain conditions
ν	Poisson's ratio
ρ	area ratio of open-ended piles
σ'_r	radial effective stress acting on pile shaft
$\sigma'_{r\infty}$	radial effective stress after reconsolidation
σ'_θ	circumferential effective stress adjacent to pile shaft
σ'_z	vertical effective stress adjacent to pile shaft
σ'_{v0}	initial vertical effective stress
σ'_{h0}	initial horizontal effective stress
τ	non-dimensionalised time parameter for reconsolidation
ϕ'	angle of internal friction
ϕ'_{ps}	ϕ' appropriate to conditions of plane strain
ϕ'_{tc}	ϕ' appropriate to conditions of triaxial compression
a	radius of face of total radial stress transducer
c_{uo}	initial undrained shear strength
c_{uops}	c_{uo} appropriate for conditions of plane strain
c_{uotc}	c_{uo} appropriate for conditions of triaxial compression
$c_{u\infty}$	undrained shear strength adjacent to pile following reconsolidation

c_v	coefficient of consolidation
E	drained Young's modulus
G	shear modulus
K_o	initial in-situ earth pressure coefficient
N_k	factor relating cone resistance to c_{uo}
OCR	overconsolidation ratio
P	load acting on face of total radial stress transducer
p'_o	initial mean normal effective stress
p'_f	mean normal effective stress at failure
q_c	CPT end resistance
r	radial distance from pile centreline
r_o	pile radius
R	radial extent of excess pore pressures generated by installation
t	time
u_o	ambient pore pressure
USF	unit shaft friction

NB. Nomenclature shown on diagrams extracted from references may vary with that given above

CHAPTER 1

INTRODUCTION

Traditional methods of pile design have been largely empirical and so are necessarily frequently conservative. In recent years, attempts have been made to investigate the fundamental behaviour of driven cylindrical steel piles, prompted by the potentially large savings that could be made for offshore structures.

Chapter 2 presents a review of the current understanding of pile behaviour. There has been a large amount of research in this field, and the review has therefore focussed on a selection of the more recent work. In an attempt to extend current theory to the actual design of piles, the In-Situ Model Pile was developed at Oxford by Mr.T.J.Freeman under the supervision of Professor C.P.Wroth. The "IMP" was ultimately intended to be used as a site investigation instrument for pile design, but was first used by Freeman in a series of carefully controlled laboratory tests. The next stage in the development was therefore to test the IMP at a selection of well documented onshore sites, and this constituted the principal aim of the research programme described in this dissertation.

Chapter 3 outlines Freeman's design of the IMP, together with modifications made by the author. This is followed by a chapter for each of the four sites visited, describing the IMP data obtained and making detailed comparisons with both theoretical predictions and other available site investigation, particularly pressuremeter data. Two of the sites chosen had stiff heavily overconsolidated clay; Gault clay at Madingley, and London clay at Canons Park. The other two sites at Huntspill and Great Yarmouth had soft normally/lightly overconsolidated alluvial clays. Each was carefully chosen for ease of access and for the availability of other site investigation or pile test data. Most space is

devoted to the Madingley site, since seven of the twelve IMP boreholes were conducted there.

In Chapter 8 some broad conclusions are drawn together on the behaviour of piles in clay as determined by the IMP at these sites.

CHAPTER 2

LITERATURE SURVEY

2.1 INTRODUCTION

The most commonly used method of deriving the shaft capacity of offshore piles driven into clay is the "Total Stress" method presented in the American Petroleum Institute design guidelines (API RP2A). An empirical α -factor relates unit shaft friction, USF, to the initial undrained shear strength of the clay, c_{u0} :-

$$USF = \alpha c_{u0} \quad (2.1)$$

Values of α are derived from a small database of load tests on piles in a variety of clay soils, the piles generally being considerably smaller than those used offshore.

Chandler (1968) and Burland (1973) recognised the need to relate shaft capacity to the effective radial stress acting on the pile. Since the radial effective stress is not readily quantifiable, Burland compared the shaft friction to the initial vertical effective stress, σ'_{v0} :-

$$USF = \beta \sigma'_{v0} \quad (2.2)$$

Numerous other investigations into effective stress methods have been made, notably Parry & Swain (1977a & b), who examined closely the state of stress adjacent to the pile at failure. However, back calculated β -values from load test data such as presented by Kraft, Focht & Amerasinghe (1981) show scatter which is similar to that of α -values. The majority of this chapter is therefore devoted to the recent development of a fundamental approach to determine the state of stress around a pile at failure. Attention is focussed upon three distinct phases during the "life" of a pile:-

- 1) Installation
- 2) Reconsolidation of the soil around the pile
- 3) Loading of the pile to failure

For simplicity most of the theoretical work discussed has investigated the behaviour of full-displacement (closed-ended) piles.

2.2 PILE INSTALLATION

2.2.1 Theoretical Work

Carter, Randolph & Wroth (1979a) idealised the soil movements that occur during the installation of a closed-ended pile as the undrained radial expansion of a cylindrical cavity. Plane strain and axial symmetry were assumed, so that only radial movement of the soil from the pile axis is considered. Soil heave, which may be important at smaller penetrations is therefore ignored, as are local end effects around the pile tip. One further simplification is that shear stresses on the pile/soil interface are ignored, the major principal stress being radial, and the stresses resulting are therefore suggested to be appropriate to the situation immediately after driving.

Randolph & Wroth (1979) suggested that for an elastic, perfectly plastic material, the radial distribution of excess pore pressures arising from pile driving will be given by the following expression:-

$$\Delta u_r = 2c_{u0} \ln(R/r) \quad r_0 \leq r \leq R \quad (2.3)$$

where Δu_r = excess pore pressure at radius r from pile axis

c_{u0} = initial undrained shear strength

r_0 = pile radius

The radial extent of excess pore pressures immediately after installation, R is given by:-

$$R = r_0 \sqrt{(G/c_{u0})} \quad (2.4)$$

where G = shear modulus

However, this soil model neglects pore pressures generated at low stress levels. Carter et al. therefore adopted a finite element technique to solve the problem, with which they were able to use a strain hardening elasto-plastic "modified Cam-Clay" soil model.

Carter et al. chose Cam-Clay parameters to represent a normally consolidated Boston Blue clay. They found that the stresses were proportional to the initial undrained shear strength and were also dependent on the ratio G/c_{u0} , c_{u0} being measured under conditions of plane strain. The stress distribution predicted immediately after installation is shown in Fig 2.1.

The radius to which excess pore pressures extend marks the boundary between the zone of soil near the pile where plastic deformations have occurred and soil further from the pile where deformations have been entirely elastic. Adjacent to the pile the soil is at its critical state, and so the effective stresses within this zone are constant with radial distance from the pile.

The stress paths followed by a soil element adjacent to the pile are shown in Fig 2.2 which neglects ambient pore pressures. The effective stress path quickly reaches the critical state line, and further cavity expansion just leads to an increase in the mean normal total stress. Whilst the effective stress path will be the same for all soil elements within the failed zone, this increase in mean normal total stress will diminish with radial distance from the pile, giving rise to a corresponding decrease in excess pore pressures.

Randolph, Carter & Wroth (1979) extended the work of Carter et al. to look at the influence of the overconsolidation ratio, OCR, on the stresses generated. The excess pore pressures adjacent to the pile derived from their finite element analyses were insensitive to OCR, as shown in Fig 2.3, but this result was found to depend on their choice of shear modulus. The following expression was proposed for the radial distribution of excess pore pressure and gives similar values to those derived numerically:-

$$\Delta u_r = (p'_o - p'_f) + 2c_{uo} \ln(R/r) \quad r_o \leq r \leq R \quad (2.5)$$

from which:-

$$\Delta u_{sh} = (p'_o - p'_f) + c_{uo} \ln(G/c_{uo}) \quad (2.6)$$

where p'_o = initial mean normal effective stress

p'_f = mean normal effective stress at failure

Δu_{sh} = excess pore pressure adjacent to the pile

The second term of the above equation is that proposed by Randolph & Wroth for an elastic perfectly plastic soil (Equation 2.3) and represents the change in mean normal total stress, Δp , during installation. As OCR increases so does G/c_{uo} , and hence the value of this second term. However this is counteracted by a decrease in the value of the first term which represents the change in mean normal effective stress, $\Delta p'$, as the soil is sheared to failure.

The stress paths followed for an element of overconsolidated soil adjacent to the pile are shown in Fig 2.4. The total stress path is similar to that for a normally consolidated soil. The effective stress state within the failed zone is independent of OCR and G , and Randolph et al. present the following equations for these stresses:-

$$\sigma'_r = [(\sqrt{3}/M) + 1]c_{uo} \quad (2.7)$$

$$\sigma'_\theta = [(\sqrt{3}/M) - 1]c_{uo} \quad (2.8)$$

$$\sigma'_z = [\sqrt{3}/M]c_{uo} \quad (2.9)$$

where σ'_r = radial effective stress

σ'_θ = circumferential effective stress

σ'_z = vertical effective stress

M = gradient of critical state line

Wroth, Carter & Randolph (1979) investigated the effect of the Cam-Clay parameters on the stresses obtained, and for overconsolidated London clay the radial stress distribution is shown in Fig 2.5. The excess pore pressures predicted on the pile shaft are similar to those for Boston Blue clay, as Fig 2.3 illustrates.

Since full displacement piles are found to push a cone of soil in front of them during penetration, Levadoux & Baligh (1980) suggest that their work predicting stresses around a cone penetrometer are applicable to piles. Their "strain path" method uses the velocity field of an ideal fluid flowing past a frictionless cone to estimate the strains that might occur in the soil. What they considered an appropriate soil model was then used to compute stresses. Levadoux & Baligh admit that their method will only give approximate solutions since the strains determined from the velocity field of an ideal fluid are unlikely to correspond to those that even the model soil would experience. The calculated stresses also depend on the path of integration used. Figures 2.6 & 2.7 show the distribution of excess pore pressure and radial effective stress predicted for a cone penetrating through normally consolidated Boston Blue clay. Cone angle has a relatively minor influence on the stresses at the cone shaft, and so predictions for closed-ended piles could be expected to be similar. Cylindrical cavity expansion predictions made with their own soil model which are also shown on these diagrams predict a less extensive zone of failure but higher pore pressures and effective radial stresses on the shaft.

Figure 2.8, which has been scaled from Fig 2.6, illustrates more clearly the difference in radial distribution of excess pore pressure predicted by the two methods. The strain path method does not predict a linear decrease of pore pressure with the logarithm of radial distance from the shaft as does cavity expansion.

2.2.2 Experimental Work

A selection of field and laboratory data is presented in this section to provide a comparison with predictions made in the previous section. In the case of cavity expansion, these predictions were made for two particular soils, Boston Blue clay and London clay and values for other

soils will depend on their Cam-Clay parameters. Installation excess pore pressures will also be affected by the initial stresses in the soil as indicated by Equation 2.5. However, excess pore pressures on the pile shaft of $3-4c_{uo}$ might be expected for most insensitive clays, with radial effective stresses in the region of $2-3.5c_{uo}$. For all clays, excess pore pressures generated by installation are predicted to decrease with distance from the shaft with a gradient of $-2c_{uo}$ when plotted against $\ln(r/r_o)$.

Plane strain undrained shear strengths are not generally available for field tests, and so the data presented by the authors of excess pore pressures and radial stresses are normalised using the available shear strengths in their papers. Wroth et al. (1979) show that the Cam-Clay model uses a plane strain undrained shear strength which is about 15% above that measured in triaxial compression.

Figure 2.9 shows a plot of radial distribution of excess pore pressures interpreted from field measurements reported by a number of authors. Broad similarities can be seen with predictions by cavity expansion and strain path methods.

2.2.2.1 Steenfelt, Randolph & Wroth (1981)

Laboratory tests were conducted in which a 400mm long, 19mm diameter model pile was jacked into a 300mm diameter kaolin sample to which vertical and horizontal stresses could be applied independently. X-ray monitoring of lead shot movements showed radial soil displacements 10-20% less than those predicted by cavity expansion. Vertical soil movements were suggested to account for the remaining volume of soil.

Pore pressure transducers in the soil gave reasonable agreement with the radial distribution predicted by cavity expansion, as shown in Fig 2.10, assuming that the excess pore pressures at the shaft might be expected to be about $4c_{uo}$. However, measured excess pore pressures on the

pile shaft were in fact much lower than $4c_{u0}$, but were observed to continue increasing after the end of installation.

2.2.2.2 Martins (1983)

A series of laboratory tests was undertaken with 15mm diameter model piles in 102mm diameter kaolin triaxial samples. The behaviour of the clay adjacent to the pile was observed at various stages by impregnating the clay with Carbowax 6000. This then enabled thin sections to be taken which were observed under plane polarised light to determine the soil structure. Residual shear surfaces were observed developing in the soil during the installation of jacked and driven piles. Upon loading the peak load was therefore much lower than that of "wished-in-place" piles. Despite the residual conditions on this shear surface, a small post-peak reduction in shear stress was still observed. Martins believes that this arose from the fact that ring shear tests have found that the residual angle of friction depends on the rate of shearing. During the relatively rapid installation slightly more turbulent particle movements give rise to a higher residual friction angle than that observed during the slow load test. Upon first loading, the peak load will be representative of the friction angle developed during installation. The shear stress will then decrease to a post-peak level appropriate to the slower rate of shearing during the load test.

2.2.2.3 Francescon (1983)

An 18.9mm diameter closed-ended model pile was jacked into laboratory prepared kaolin samples. Instrumentation along the pile measured shaft friction, radial total stresses and pore pressures. Tests at lower OCRs were conducted in a 250mm diameter chamber with stress controlled boundaries. Less reliable tests were performed where the pile was inserted through holes in the top plate of a consolidometer.

Pile installation excess pore pressures showed greater dependence on OCR than cavity expansion had predicted, as Fig 2.11 shows. Better agreement was obtained if a constant value of 4 was assumed for $\ln(G/c_{u0})$ for all OCRs. Immediately after installation, Francescon measured very low radial effective stresses. Figure 2.12 shows a comparison between total radial stresses at this stage and cavity expansion predictions.

2.2.2.4 Levadoux & Baligh (1980)

Excess pore pressure measurements on the shaft of a piezocone pushed into Boston Blue clay with an OCR of 1.3 yielded values of $\Delta u_{sh}/\sigma'_{v0}$ of 2.0. This does not compare well with the strain path prediction for normally consolidated Boston Blue clay of about 1.0 shown on Fig 2.6. The contours of excess pore pressures predicted by cavity expansion also presented on their diagram are misleading. Figure 2.8 shows that a linear projection of their contours back to the cone shaft gives a value of around 1.8 for $\Delta u_{sh}/\sigma'_{v0}$ which compares well with their field observations. Using their Cam-Clay model of Boston Blue clay, Randolph et al. predicted a value of around 1.3.

Figure 2.13 shows the strain path prediction of excess pore pressure variation along the shaft of a 60° cone. Excess pore pressures are normalised with respect to the value on the shaft. Levadoux and Baligh obtained good agreement with their field measurements. Since Levadoux & Baligh suggest that patterns of normalised pore pressures are insensitive to soil type and OCR, May (1983) has plotted some additional piezocone data on this diagram showing similar trends, as do data extracted from Francescon's work.

2.2.2.5 Roy, Blanchet, Tavenas & La Rochelle (1980)

Six 219mm diameter closed-ended piles were jacked into a soft sensitive silty clay to penetrations of about 7.8m. Pore pressure

transducers were located at the pile tip and at several locations along the shaft. Piezometers were also positioned in the surrounding soil at various depths and radial distances. Undrained shear strengths as measured by in-situ vane, varied from 10kPa at 1.8m to 28kPa at 9m. Excess pore pressures measured on the pile during installation showed a corresponding increase.

Tip excess pore pressures decreased approximately linearly with the logarithm of radial distance from the pile as shown in Fig 2.14. However excess pore pressures measured on the pile shaft are lower than a similar linear radial distribution would imply and Levadoux & Baligh interpret this as evidence of a strain path type of radial distribution.

Tip excess pore pressures were considerably greater than those measured on the pile shaft and their data have been added to Fig 2.13 for comparison with the strain path predictions.

2.2.2.6 O'Neil, Hawkins & Audibert (1982)

Closed-ended, 273mm diameter, test piles were driven 13.1m into a very stiff overconsolidated clay. In addition to the pore pressure measurements shown on Fig 2.9, total radial stresses were measured on the pile shaft. An effective radial stress of about $1.1c_{u0}$ was measured at a depth of 10.4m, and $2.7c_{u0}$ at 12.5m, c_{u0} being measured under triaxial compression.

2.2.2.7 Rigden, Pettit, St.John & Poskitt (1979)

Two similar steel tubular piles were driven 9m into a stiff boulder clay. A mean c_{u0} of around 110kPa was established for this site by a variety of laboratory and field methods with OCRs decreasing from about 6 at 4m to 2 at 10m. One of the piles was open-ended, the other closed. Unfortunately the instrumentation did not function well, but it appears

that the maximum total stress registered during installation on the shafts near the pile tips was about $6c_{uo}$.

A more extensive series of tests on driven and also grouted piles was conducted at this site by the Building Research Establishment (Ove Arup & Partners summary report, 1986). Four of the 10m long driven piles were open-ended and five closed. Pile diameters of 203 & 305mm were used, and the open-ended ones were found to plug part way through installation. Unfortunately, much of the pile instrumentation did not survive driving. The excess pore pressures measured by nearby piezometers, shown on Fig 2.9, indicate lower normalised pore pressures than from other sources.

2.2.2.8 Butterfield & Johnston (1973)

A 100mm diameter closed-ended model pile was jacked into London clay to a penetration of 4m. Measurements of total radial stress and shaft friction were made during installation. The profile of total radial stress with depth strongly reflected variation in c_{uo} as measured under triaxial compression. Total radial stresses were around $6-8 c_{uo}$ on the shaft near the pile tip, decreasing along the shaft to about $4c_{uo}$. These measurements include a component arising from the initial in-situ total stress. The strain path method predicts a decrease in total radial stress along the pile shaft similar to that for pore pressure, since the effective radial stress is fairly constant.

The mobilised friction angles of around 10° suggest residual conditions, as had been predicted by Martins.

2.2.2.9 Koizumi & Ito (1967)

A group of nine closed-ended steel pipe piles of 300mm diameter was inserted into a soft, extremely sensitive, silty clay. The c_{uo} increased with depth from 25kPa at the surface to 40kPa at the pile tip. Two piles were instrumented to measure total radial stresses and pore pressures,

but unfortunately these were some of the later piles driven. Since the pile spacing was only 900mm the readings shown in Fig 2.15 will be influenced by group effects and should be interpreted with caution.

The effective radial stress measured during installation was found to be negligible. This phenomenon may be associated with the sensitivity of the soil. Another unexpected feature of these data is the fact that both total radial stresses and excess pore pressures increase with depth, even when normalised by c_{u0} .

The radial distribution of excess pore pressures interpreted from the data they present is shown in Fig 2.16. Again, there is too great a scatter to judge whether the data best fits a cavity expansion or a strain path type of radial distribution.

2.2.2.10 Kirby & Roussel (1980)

Four 116mm diameter piles were installed in San Francisco Bay Mud to penetrations of 12.2m. Undrained shear strengths derived by various means are very scattered at this site, and beneath the overconsolidated crust they increase from around 5-15kPa at 6m, to about 12-25kPa at 10m. Two of the piles were instrumented, one of which was jacked, the other driven.

Excess pore pressures resulting from pile installation compared well with Cam-Clay cavity expansion predictions as shown by Fig 2.17. Radial effective stresses immediately after installation are much lower than expected. Again normalising the authors quoted readings with respect to their measurements of c_{u0} , values of 0.29-0.59 at 6.1m and 0.56-0.74 at 10.7m are obtained. However, Kirby & Roussel do not believe that their instrumentation is sufficiently accurate to measure radial effective stresses reliably.

2.2.2.11 Jardine (1985)

Johnston's model pile was refurbished and modified to include pore pressure measurement. It was then jacked into London clay at the Canon's Park test bed site of the Building Research Establishment. The results will be discussed at greater length in Chapter 4 where a comparison will be made with data obtained at this site with the IMP.

Pore pressures measured during installation on the pile shaft were generally much lower than those predicted by Wroth et al. (1979), negative values being recorded at some levels. Radial effective stresses near the tip of around $3-4c_{u0}$ compare well with the cavity expansion prediction of 3.4. However, higher on the shaft, Jardine quotes lower radial effective stresses of $1.5-2c_{u0}$. Such a decrease in radial effective stress along the shaft is not predicted by the strain path method.

Jardine calculates mobilised friction angles of $13-22^\circ$ from installation shear stresses. He attributes the fact that this is higher than the residual angles that might be expected to the high speed of installation (500mm/min) applied in short pushes of about 35mm. He compares these results to data from ring shear tests on a London clay/glass interface conducted by Lemos (1985). Slow shearing of 0.03mm/min was found to result in a residual friction angle of 10.2° after very small displacements. Subsequent faster shearing at 133mm/min produced an increase in the friction angle interpreted from total stress measurements to 20° , which only decreased to a similar residual value to that for slow shearing after 300-500mm displacement. Significantly, the shear surface was found to be within the soil mass.

Jardine also discusses work by Kitching (1983) who found a polished shear surface a few millimetres from the shaft of a pile installed at Canon's Park. This might be expected in the light of the ring shear tests and also Martins model pile tests. Upon extraction Jardine's pile was

found to have a 3mm coating of clay, further evidence of shearing within the soil mass.

2.2.2.12 Clegg (1981)

Some 51mm diameter piles were jacked and driven to penetrations of up to 2m into the highly overconsolidated Gault clay at Madingley, near Cambridge. This is another site for which data have been obtained with the IMP.

Of particular interest is one pile which was instrumented with load cells and a pore pressure transducer near the tip. Excess pore pressures measured during the jacked installation of this pile show a large amount of scatter, but are generally very low, fluctuating about zero. As with Jardine's pore pressure readings, only a few peak values are large enough to permit any sensible comparison with radial cavity expansion predictions. Upon retrieval, these piles also had clay adhering to their surface.

2.2.2.13 Azzouz & Lutz (1986)

The research group at Massachusetts Institute of Technology has developed a model pile similar to the IMP. The Piezo-Lateral Stress (PLS) Cell is 38mm in diameter and has a conical end. A single load cell, pore pressure transducer and total radial stress transducer are located 1.8m from the tip. The instrument is similar to the piezocone, being pushed into the ground with cone rods at a rate of 20mm/sec. Unfortunately, the single load cell measures not only the sleeve friction but also tip loads which must therefore be estimated. No provision is made for the fact that the sleeve friction may be non-uniform immediately behind the tip, as has been found by Konrad (1986).

Tests were performed in two zones of clay at the same site. Zone 1, from 35-50m depth, consists of a firm clay with an OCR of 1.7 and c_{uo}

values measured by numerous laboratory and field methods of 20-90kPa. Zone 2 clay, at 62-77m depth has a c_{uo} of 40-140kPa and an OCR of 1.5.

Unfortunately all their data have been normalised with respect to the initial effective vertical stress, making it difficult to interpret. Cavity expansion theory indicates that this method is incorrect since the installation radial effective stress, and hence shaft friction, is related to c_{uo} . Only in a normally consolidated soil could their approach work, as then c_{uo} would be linearly related to σ'_{vo} . The installation radial effective stress normalised by σ'_{vo} is consequently quite different in the two zones; 0.54 in zone 1, and 0.38 in zone 2. Normalised by c_{uo} values of around 1.76 are obtained for both zones, although there is considerable scatter of c_{uo} data. Correspondingly, installation β values of 0.19 in zone 1 and 0.15 in zone 2 show greater discrepancy than α values of 0.62 & 0.69 respectively.

From Equation 2.6, it is clear that excess pore pressures during installation are predicted by cavity expansion to be dependent on the initial stresses as well as c_{uo} at a given site, and the installation excess pore pressures show closer agreement when normalised with respect to σ'_{vo} . Values of $\Delta u_{sh}/\sigma'_{vo}$ of 2.20 and 2.02 are presented for zones 1 & 2, compared to 7.2 and 9.4 respectively when normalised by c_{uo} using the authors data. Excess pore pressures are therefore somewhat higher than predicted by cavity expansion, and radial effective stresses lower.

2.3 RECONSOLIDATION

2.3.1 Theoretical Work

Since the maximum pore pressures are predicted to be adjacent to the pile shaft, reconsolidation takes place by outward radial dissipation, accompanied by inward movement of soil particles. Randolph & Wroth (1979) argued that this is essentially an unloading process. They therefore assumed that deformations could be considered entirely elastically, and

were able to derive an analytical solution. Figure 2.18 shows the dissipation curves they obtained, for which time, t , has been replaced by the non-dimensionalised τ . Reconsolidation times are proportional to the square of the pile radius.

Carter, Randolph & Wroth (1979a) extended their numerical analyses to look at the reconsolidation phase for a strain-hardening elasto-plastic soil model. Soil immediately adjacent to the pile continues to yield and work harden during reconsolidation. However, dissipation times were found to be governed by events in the bulk of the clay, and since deformations are generally elastic, the elastic solution gives dissipation times close to those predicted by the more sophisticated model of Carter et al.

The stress changes Carter et al. found during reconsolidation are illustrated in Fig 2.19. For a strain hardening soil τ has to be redefined (τ^*) since the coefficient of consolidation changes during reconsolidation. As expected the radial effective stress rises during reconsolidation, but is accompanied by a decrease in radial total stress which would not be predicted by an elastic solution. The proportion of initial excess pore pressure which appears as an increase in radial effective stress at the end of reconsolidation was found to be 54-60%.

The stress path followed during consolidation is shown on Fig 2.2. The mean normal effective stress increases as the soil moves away from the critical state line.

Randolph, Carter & Wroth (1979) studied the effect of OCR during the reconsolidation phase, as they had done for installation. It was found that OCR would not affect consolidation times but that the choice of G would. Lower G values would give quicker consolidation due to the less extensive initial pore pressures caused by installation. The state of stress in the soil adjacent to the pile after reconsolidation is independent of OCR. The clay has "forgotten" its stress history and is

essentially normally consolidated under a principal stress which is radial.

At the end of reconsolidation the soil adjacent to the pile has an undrained shear strength, $c_{u\infty}$, predicted to be $1.6c_{u0}$ for Boston Blue clay of any OCR. However, since the radial gradient of shear stress during loading is likely to be steeper than the radial gradient of $c_{u\infty}$, Randolph et al. still expect failure to occur along the pile/soil interface.

The results of an investigation into the sensitivity of cavity expansion predictions to the Cam-Clay parameters made by Wroth, Carter & Randolph (1979) are summarised in Fig 2.20. The radial effective stress after installation is only dependent on the gradient of the critical state line, M , but the value after reconsolidation also depends on the gradient of the normal consolidation line, λ . The final undrained shear strength adjacent to the pile is influenced additionally by the gradient of the soil's swelling line, κ .

2.3.2 Experimental Work

Francescon (1983) observed a reduction in radial total stress during reconsolidation of 17% for normally consolidated clay as shown on Fig 2.12. Reconsolidation times also compared well with cavity expansion predictions. However, as a result of his low measurements of radial effective stress, he did not believe that at the end of reconsolidation the soil was normally consolidated under the radial stress.

In their field tests Koizumi and Ito (1967) found a much greater decrease of radial total stress of around 55% during reconsolidation. Radial effective stresses were found to increase from insignificant levels at the end of installation to the values shown in Fig 2.15.

Kirby & Roussel (1980) back-calculated values of the coefficient of consolidation for Bay Mud using dissipation curves for pore pressures

measured at the pile shaft such as those illustrated in Fig 2.21. The elastic solution of Randolph & Wroth was assumed. However, values of c_v measured in the lab were only a tenth of those calculated. Such deviation between lab and field measurements is not uncommon, particularly since dissipation of pore pressure around a pile is more dependent on horizontal permeability which is usually larger than the vertical.

Pile shaft measurements of radial total stresses also on Fig 2.21 show that until the transducers malfunctioned, a larger decrease than could be expected from cavity expansion was observed as a comparison with Fig 2.19 will show. Another interesting feature of Kirby & Roussel's data is the initial rise in pore pressures seen at 1-10mins together with an associated decrease in radial effective stress. Whilst this may be associated with the unloading of the pile after installation, it could indicate that maximum pore pressures during installation are not on the pile shaft, contrary to cavity expansion and strain path predictions. The pore pressure rise could not be explained by the propagation along the shaft of higher excess pore pressures near the tip, since it occurs too quickly, and simultaneously at various elevations.

Jardine's reconsolidation data have been replotted on a logarithmic time scale in Fig 2.22. They have also been scaled by a factor equal to the square of the ratio of the IMP radius to that of his pile. This is to allow direct comparison with the IMP data presented for the same site in Chapter 5. These data also indicate a complex pore pressure distribution around the pile since pore pressures fluctuate dramatically and the anticipated smooth dissipation curves are not seen. Meanwhile, a steep drop in the radial total stress results in a fall in effective radial stress during the first 600 hours to levels which are generally below in-situ horizontal effective stresses. It will be recalled that cavity expansion predicted a monotonic increase in radial effective stress. Unfortunately the total stress transducers then failed.

During reconsolidation, the PLS cell of Azzouz & Lutz measured an increase of radial effective stress by a factor of around 2.6, to about $1.54\sigma'_{v0}$ in zone 1 and $0.90\sigma'_{v0}$ in zone 2. The authors did not know why there should be this discrepancy in the final radial effective stress in the two zones. However, normalisation with respect to c_{u0} reveals that there is in fact little discrepancy, with values of around 5.0 in zone 1 and 4.2 in zone 2, which are slightly lower than cavity expansion predictions by Wroth et al. shown in Fig 2.20.

The PLS Cell reconsolidation data are shown in Fig 2.23, and it is interesting to note the initial decrease in the effective radial stress measured in one test. In this case the decrease arises from a large and rapid decrease in the radial total stress and not from initial pore pressure increases. A 44% decrease in the radial total stress was measured in the zone 1 test.

2.4 PILE LOADING

2.4.1 Theoretical Work

Randolph & Wroth (1981) suggested that undrained failure of the soil adjacent to the pile would be similar to that observed in a simple shear test. Rupture in the simple shear test initially occurs on vertical planes, as shown in Fig 2.24b. This occurs in preference to failure along horizontal planes shown in Fig 2.24a, since σ'_x is less than σ'_y . The apparent angle of friction at failure will therefore be less than ϕ' . In the case of a pile, the major principal stress at the start of loading was predicted in the previous section to be radial. Consequently, the radial effective stress is equivalent to σ'_y , and initial failure planes would be horizontal. The stress ratio at initial failure would be given by the following equation:-

$$\text{USF}/\sigma'_r = \sin\phi' \cos\phi' / (1 + \sin^2\phi') \quad (2.10)$$

This would give an apparent mobilised friction angle on the shaft of 17.3° for a soil with a ϕ' of 23° .

Kinematic constraints dictate that the ultimate failure must be along failure planes parallel to the pile, so the state of stress adjacent to the pile must change to that shown in Fig 2.24a. This would be accompanied by a large reduction in the radial effective stress, and under undrained conditions, large pore pressures would result.

Field measurements generally show only small pore pressure changes during loading. Esrig & Kirby (1979) performed finite element analyses which showed only small changes in the mean normal total stress during pile loading. Since they assume that failure will occur when the soil at the pile face reaches its critical state they conclude that the state of stress in the soil adjacent to the pile after reconsolidation must be close to the critical state line. Martins' alternative explanation is that since failure of the pile is along a residual surface created during installation, any stress changes during loading would be curtailed.

2.4.2 Experimental Work

In his undrained load tests after full consolidation, Francescon (1983) observed a post-peak decrease of shaft capacity of about 30%. This decrease would not be expected if failure was along a residual shear surface. He also found that stress controlled cyclic loading had little effect on capacity, but that a significant degradation did occur for large displacement cycling.

Francescon calculated peak values of mobilised friction angle of around $15-17^{\circ}$, the higher angles being for a pile with sand stuck to its surface. As residual conditions were approached the mobilised friction angle decreased to around 13° . Only small excess pore pressures of about $0.2c_{u0}$ and a decrease of radial effective stress of $0.5c_{u0}$ were observed.

Puech & Jezequel (1980) drove a 273mm diameter closed-ended steel pipe pile 13m through a mixed soil profile, the lower part of the pile ending up in a silty clay. The undrained shear strength of this soil can be estimated from the site investigation data they present to be in the region of 25-45kPa. During undrained loading, measurements were made of the total radial stress and pore pressures and these are shown on Fig. 2.25. Towards failure the effective radial stress was observed to decrease significantly, as pore pressures reach levels approaching the undrained shear strength.

Roy et al. (1980) have also measured two fairly high excess pore pressures during undrained loading of $1.7c_{u0}$ and $0.6c_{u0}$. No excess pore pressures were measured at a radial distance of $2.7r_0$, confirming that pore pressures arising from loading are more localised than those during installation.

In contrast, Jardine measured no significant pore pressure changes up to 85% of peak load upon tensile undrained loading of his pile. Even then large pore pressure changes did not occur until after the pile had been unloaded and then pore pressures continued to fluctuate for several hours. No consistency of pore pressure change was seen, movements in both positive and negative directions being observed. Profiles of shaft friction developed during the load test seemed to indicate progressive failure, the compressibility of the pile allowing soil adjacent to the upper part of the pile to start strain softening before peak values were obtained on the lower part.

Price & Wardle (1982) have also conducted full-displacement pile tests at Canon's Park. Three 168mm diameter 7m long piles were installed, one each by jacking, driving and boring. In line with Martins' findings, the undrained load tests showed a peak in the axial load deflection curve of the driven pile which was not seen for the jacked. However, the driven

pile capacity was found to be less than the jacked, whereas Martins suggested that the residual capacities would be the same.

The BRE pile test series showed no significant differences in undrained shaft capacity between their driven piles and a jacked pile installed at the same site by Taylor Woodrow (Garas & McAnoy, 1980). However, their data are confused by the fact that pile failure seemed to occur on the shiny mill varnish left adhering to some piles. In addition, the first set of load tests were conducted before reconsolidation was complete. However, the data do indicate similar shaft friction in tension and compression, and that the initial peak load was similar whether it was achieved under undrained loading, sustained or even cyclic tensile loading. Large reductions in capacity were only seen when cycling took the pile into compression as well as tension.

In his undrained load tests at the Madingley site, Clegg reports little difference in the behaviour of jacked and driven piles. He did however report tensile shaft capacities greater than compressive, which he believes may result from the high overconsolidation of the soil at shallow penetrations. The instrumented pile indicated progressive failure similar to that experienced by Jardine.

Kirby & Roussel's (1980) undrained load tests also showed very small excess pore pressures at the pile shaft of between 0.7-3.2kPa. No significant total radial stress changes were seen. Tensile shaft frictional capacities were found to be at least equal to compressive. Post-peak degradation of capacity seemed to be associated only with displacement as Francescon had found, and not repeated loading.

Following complete reconsolidation, Azzouz & Lutz performed undrained pullout tests on the PLS Cell. The β values they present for first loading of 0.39 in zone 1 and 0.24 in zone 2 are consistent with the apparent discrepancy they highlighted in the radial effective stress after full reconsolidation when normalised by σ'_{v0} . Unfortunately, no data

on radial stress changes during the load tests are presented, but it is clear that a substantial decrease in the radial effective stress must have occurred during loading if the mobilised friction angle at failure was the same as that during installation.

Azzouz (1986) extends the PLS Cell work to make comparisons with a 350mm diameter open-ended pile driven at the same site into the zone 1 clay. The pile was sleeved by a conductor from the surface to the top of zone 1. First undrained loading after full reconsolidation revealed a lower β value than for the PLS Cell of around 0.272-0.328. The uncertainty arises from the unknown end bearing loads. Other complications are the fact that the pile had been prematurely loaded to failure several times during reconsolidation and that no measurements of the length of the soil plug inside the pile were possible. If failure takes place on a residual plane, then the loading time of 2.5 hours may not have been sufficiently rapid to ensure undrained conditions. A decrease in β value was observed for successive load tests, but the time between tests was inadequate to ensure complete reconsolidation.

Azzouz suggested that the installation β value of the PLS Cell could be used to design piles in the Empire clays he investigated, since it was conservatively below the minimum β indicated by the full scale pile. However, the PLS Cell data did not conclusively model the observed reduction in capacity for successive loadings to failure, suggesting some difference in the behaviour of the instrument to the pile. Extrapolation of β values to prototype piles is not the best use of high quality field model pile data, and should not be done without a full understanding of such discrepancies in the behaviour of the two and also the influence of such factors as pile whip during driving and axial flexibility during loading.

2.5 FURTHER CONSIDERATIONS

2.5.1 Open-Ended Piles

The research so far described has exclusively dealt with closed-ended piles. The behaviour of open-ended piles has been investigated by Carter, Randolph & Wroth (1979b) who again use their finite element technique with the Cam-Clay parameters of Boston Blue clay. They recognised that the area ratio of the pile, ρ , would be a governing factor. This is the ratio of the cross-sectional area of the pile wall to the gross cross-sectional area. Carter et al. made the important assumption that the soil inside the pile remains undisturbed and stationary as the pile is driven around it. All the soil displaced by the volume of the pile wall is therefore assumed to move radially outwards from the pile, which is unlikely to be the case in practice.

The effect of ρ is illustrated in Fig 2.26. Installation excess pore pressures will be lower and also be less extensive for open-ended piles, and faster reconsolidation times could be expected.

Figure 2.26 also shows that the final undrained shear strength of the clay adjacent to the pile may be over 25% greater for a closed-ended pile, and its capacity will be correspondingly greater. Rigden et al. (1979) have observed this phenomenon. However, the more extensive BRE tests at the same site indicated no significant differences in the drivability and capacity of open and closed-ended piles.

2.5.2 Sensitive Soils

Randolph et al. (1979) investigated the behaviour of a sensitive soil during pile installation, and Fig 2.27 shows the radial distribution of stresses immediately after driving that they obtained for a soil with a sensitivity of 2.86 and a peak plane strain shear strength of 17.9kPa. The state of effective stress in the failed zone is not constant as it is

for an insensitive clay. Low effective stresses are predicted on the pile shaft.

From their field tests in a very sensitive clay Koizumi & Ito found radial effective stresses too low to measure reliably.

2.6 SUMMARY

Existing field and laboratory data show little consistency in their findings largely because of the unreliability of measurements and the variability of soil conditions. The cavity expansion model seems to predict excess pore pressure during installation which are broadly in agreement with most field data, but may overpredict radial effective stresses. However, in highly overconsolidated clays, measured excess pore pressures in field tests appear very much lower than those predicted.

The model of reconsolidation used by Carter et al. does not explain initial decreases in radial effective stress observed by some field tests. The predicted increase in radial effective stress may therefore be an overestimate.

Existing load test data show no clear trend for stress changes, although it does appear that residual shear surfaces developed during installation may be of importance.

CHAPTER 3

EQUIPMENT AND PROCEDURE

3.1 INTRODUCTION

The In-Situ Model Pile (IMP) together with its extension tube were originally designed by Freeman and constructed at Oxford. The first section of this chapter deals with the design of the IMP, modifications which have been made to it by the author during the course of the field testing programme and the ancillary equipment built by the author for field use. This is followed by a description of the field testing procedure including details of the preparation and calibration of the instrument beforehand.

The philosophy behind the design of the equipment was to maximise portability, ease of operation, ruggedness, and self-containment. A site-investigation unit was created which operated at sites where no services were available and also under hostile weather conditions. A Land-Rover and trailer were purchased to provide transport and to house the equipment whilst on site.

IMP site investigations were undertaken by the author with one technician, and under commercial conditions similar personnel would be required. For research purposes however, a second technician was frequently included to allow greater flexibility to cope with any problems arising from the continual introduction of new techniques and equipment.

3.2 EQUIPMENT

3.2.1 The In-Situ Model Pile

3.2.1.1 General Design

Figures 3.1 & 3.2 illustrate typical layouts of the IMP. It comprises two concentric cylinders attached to a common pile head. The inner

stainless steel cylinder is rigidly connected to the cutting shoe assembly and so end bearing and friction forces acting on the tip section of the IMP are transmitted straight to the head of the IMP, without being measured. A small gap between the tip section and the remainder of the outer cylinder ensures isolation of the outer cylinder from end bearing forces.

The 80mm diameter outer cylinder is made up of interchangeable brass segments, some of which house instrumentation. Figure 3.3 shows a typical segment containing a total pressure transducer which has its covering cap removed. Grooves are provided along the inside of the cylinder to conduct electrical wiring to the IMP head. The segments are secured to each other by grub screws, a seal being maintained by an O-ring. A similar seal is provided between the inner and outer cylinders near the IMP tip. Water ingress is thus prevented and back pressure may be applied to the pressure transducers. In the field, nitrogen pressure was supplied from the surface through two hoses which also housed the screened instrumentation cabling. Through one hose cables conducted the power supply to the IMP head and through the other the cables returned the transducer signals to the surface, an arrangement which minimised interference.

The IMP is hollow so that it may be used open-ended to investigate the differences in the behaviour of open-ended and closed-ended piles. The length of the inner cavity is 940mm and it has an area ratio of 49%. To encourage soil into the IMP and so reduce the apparent area ratio, a cutting shoe with an inward taper is provided, above which is the 2mm step of a cutting shoe, provided to reduce inner shaft friction. A plunger inside the IMP is attached to a rod which is pushed through the IMP head as the sample chamber fills with soil. The soil may then be extruded after extraction from the borehole by jacking against this rod,

as shown in Fig 3.4. A vent hole is provided through the plunger to allow easy escape for water trapped under the IMP at the start of a sounding.

For field use the plunger arrangement was modified by providing the restraining mechanism shown in Fig 3.5. Serrated wedges gripped the rod, holding the plunger stationary and so enabling the IMP to be inserted closed-ended. The plunger rod was connected to the surface by an inner drill string of 1in galvanised conduit sections as shown in Fig 3.5. This conduit also provided protection for the gas hoses which ran through it. Upon reaching a depth at which open-ended penetration was desired, the wedges were removed, releasing the plunger and allowing soil into the IMP. This was achieved by first raising the IMP slightly, say 20mm, to remove end bearing forces. The hammering head also shown in Fig 3.5 allowed a few sharp blows to be applied to the conduit string without damaging the gas hoses. This forced the plunger down against soil which will have filled the cutting shoe section, beneath the plunger, and allowed the wedges to be pulled clear by means of nylon lines running through the conduit to the surface. As penetration then proceeded, the quantity of soil entering the IMP could be judged by the upward movement of the conduit at the surface.

From the consideration of portability it was not practical to use a constant diameter pile from its tip to the surface since the shaft friction generated in a stiff clay would require a very large jacking and reaction system. Drill rods of smaller diameter than the IMP were therefore used to jack it into the soil, and NWY size with an outer diameter of about 67mm were chosen. The inner conduit sections passed through the middle of these rods.

A 76.3mm diameter steel tube, the extension tube, connected the drill string to the IMP. This had an inner diameter sufficient to house both gas hoses and the plunger rod during open-ended penetration.

The specific arrangement of instrumentation used for the first two Huntspill boreholes (A & B) is similar to that shown in Fig 3.1 and is shown schematically in Fig 3.6. This arrangement is similar to that used by Freeman in his series of laboratory tests conducted at Oxford. Minor modifications included the replacement of the tip PDCR-81 transducer and the provision of an additional similar transducer in the shaft. Freeman's original 4 tonne load cell was modified to 1.5 tonne capacity and re-straining gauged. This load cell was positioned at mid-height on the IMP, and the other load cell of 2 tonne capacity was placed at the IMP head. It was hoped that this arrangement would give independent measurements of shaft friction on the upper and lower halves of the shaft. However as will be shown later, results from these first two preliminary boreholes showed that significant axial loads were transmitted to the outer cylinder from the inner cylinder via the O-ring seal between them. Further axial load errors in the lower friction sleeve were incurred from soil and pore pressures acting in the gap between the end of the outer cylinder and the tip section. To measure and so eradicate this error, an additional load cell was fabricated and positioned at the base of the outer cylinder.

Data from these two boreholes also showed inconsistency in the radial stresses measured. Consequently the total and pore pressure instrumentation was replaced by the arrangement of new instrumentation shown in Fig 3.7. Two independent measurements of radial total stress and pore pressure were made, one near the IMP tip and one near its head. Two total pressure and two pore pressure transducers were provided at each location, similar transducers being mounted diametrically opposite each other to check for any non-uniformity of stresses around the IMP. Both inner and outer cylinders were modified at this stage to provide extra slots to house the additional instrumentation wiring. Wiring considerations within the IMP necessitated the positioning of the

uppermost shaft friction load cell below the uppermost set of pressure transducers. The photograph in Fig 3.2 shows the IMP with the new pressure instrumentation and bottom load cell in place.

The arrangement shown in Fig 3.7 was used for Boreholes C-F. For Boreholes G-K the two sets of pressure transducers were interchanged to check if the location of instrumentation in any way influenced the readings obtained. The revised arrangement is shown in Fig 3.8. Minor modifications were made to the instrumentation throughout the testing programme, as and when problems arose. These are noted in the following chapters which present the data obtained with the IMP.

The final borehole (L) in this programme was used to investigate the effect of surface roughness. The upper friction sleeve was knurled whilst the lower was left smooth. The modified rough-walled IMP is that shown in Figs 3.12 & 3.20. To avoid the possibility of damaging the IMP instrumentation at such a late stage in the programme, the part of the stainless steel body of the uppermost load cell which forms a small section of the top friction sleeve was left un-knurled.

In the following sections are discussed details of the design and fabrication of each of the transducers. The new instrumentation described was designed by the author but are essentially modified versions of Freeman's instrumentation. The new transducers were machined and gauged in the Engineering Department's workshops, then being mounted and wired into the IMP by the author.

3.2.1.2 Pore Pressure Transducers

In the tip section of the IMP a Druck PDCR-81 differential pore pressure transducer is mounted axially behind a filter, as shown in Fig 3.9. A porous plastic filter was used for Boreholes A & B, which was then replaced for one fabricated from epoxy resin and sand. The other PDCR-81,

mounted similarly in a shaft segment used a sand and epoxy filter throughout.

The second type of pore pressure transducer used on the IMP shaft for Boreholes A & B was adapted by Freeman from a design by Cambridge In-situ for use with the self-boring pressuremeter. Figure 3.10 shows its arrangement. Behind the porous filter is a cavity, the back face of which is strain-gauged with a diaphragm type of gauge which is self-temperature compensating. This cell is designed to take sintered bronze filters, but as these tended to clog easily and are difficult to shape to the correct curvature of the IMP, sand and epoxy filters were used initially. Freeman had sealed this type of transducer into the circular cavity in the IMP with epoxy resin. However laboratory tests showed that a reliable pressure seal could not be obtained, and so for the field testing programme the circular annulus around the transducer was filled with a synthetic rubber. This provided a successful pressure seal and also helped to reduce cross-sensitivity of the transducer to axial loads in the IMP. To maintain radial rigidity of the transducer, a thin layer of epoxy was still used to affix the base of the transducer into the IMP.

The three replacement pore pressure transducers fabricated after the the preliminary site investigation were essentially copies of this original. Figure 3.11 shows a photograph of the diaphragm base of one of them with its strain gauge in place. Four screws had been added which screwed into the the transducer mounting in the IMP shaft, thereby improving radial rigidity and aiding accurate transducer placement. Figure 3.12 shows a close-up shot of a pore pressure transducer mounted in the IMP, complete with filter holder and filter. Also shown are the adjacent total pressure transducer and shaft friction load cell.

Epoxy and sand filters were used with this type of transducer until the rearrangement of the IMP following Borehole F. A porous plastic filter was then fitted to pore pressure transducer No.3, now relocated

near the IMP head as shown in Fig 3.8, and a sintered bronze filter was fitted to No.5 now located in the lower friction sleeve. These filter types were then maintained until the end of the testing programme to investigate what effect filter type might have.

3.2.1.3 Total Radial Stress Transducers

The original total pressure transducer, used for Boreholes A & B was also fabricated at Oxford to a design by Freeman. Figure 3.3 shows a photograph of it with its top cap removed revealing the strain gauges below. Pressure acting on the piston deflects the strain-gauged beam upon which the piston sits. When wired so that maximum sensitivity to radial loads is obtained, the strain gauge arrangement shown in Fig 3.13 gives low cross-sensitivity to shear loads on the cap, axial loads in the IMP and to eccentricity of radial load. Gauges which were self temperature compensating were again used, and additional temperature insensitivity was gained by the use of a full bridge circuit. An O-ring seal around the piston prevented water ingress, and the whole cell was again glued with Araldite into its housing in the IMP, the surrounding annulus being sealed with synthetic rubber, as for the strain-gauged pore pressure transducers.

The four replacement total pressure transducers, the design of which is shown in Fig 3.13 were of similar construction, except that the beam was reduced from 1.5mm to 1.2mm thick to gain greater sensitivity. The O-ring seal around the top cap was housed in the main body of the transducer rather than in the cap itself which greatly facilitated installation of the top cap. To improve radial rigidity of the new transducers, the top caps were glued onto their columns, as well as being held down with a screw. As for the new pore pressure transducers, four screws were used in addition to epoxy to hold the transducers firmly in the IMP.

Strain gauges were mounted below the beam in the new transducers, which is a much easier gauging operation. One of the new transducers used the same gauge layout as the original, but the other three used the simpler four gauge layout also shown in Fig 3.13. Accurate shear compensation in this circuit relies on the inner two gauges experiencing similar magnitudes of strains under shear as the outer two. This is a less sophisticated arrangement than that of the eight gauge circuit for which perfect shear compensation is dependent only on gauges in the same arm of the circuit having exactly the same location, but on opposite sides of the beam. However, no significant differences in the performance of the two types were observed.

One drawback of this transducer design is that since the piston moves away from the soil to register a pressure, the pressure acting upon it will decrease, leading to an error in the reading. During IMP installation, this compliance is not a problem, since the transducer is continually moving into fresh soil. Whilst fluctuations during penetration may be subject to a small error, the mean reading will be correct. However, during the reconsolidation phase the IMP is stationary relative to the soil and so the error may become significant, particularly in stiff clays. To overcome this problem, a back pressure system was introduced for Borehole C onwards. Regulation of this back pressure enabled the total stress transducer face to be held effectively stationary during reconsolidation. The back pressure system was used to conduct down-hole tests investigating the effect of compliance of both total and pore pressure transducers.

It was to reduce compliance that the measures described were taken to reduce the radial flexibility of the total and pore pressure transducers in their mountings.

3.2.1.4 Load Cells

As previously outlined, the two axial load cells used throughout the testing programme are of 1.5 tonne capacity (LC2) and 2 tonne capacity (LC3). As shown in Figs 3.14 & 3.15, the load cells consist of four strain-gauged columns. These measure the axial load in the outer cylinder of the IMP and the shaft friction is calculated from the difference between adjacent load cell readings. This system in which end bearing forces are not measured allows sensitive measurement of shaft friction since there is no danger of load cells being overstressed if the IMP tip should strike an obstacle such as a mudstone layer or boulder.

Each column of LC3 was gauged by Freeman with a full bridge circuit, again using gauges which were self temperature compensating. This arrangement was copied when LC2 was regauged before the start of field work. The four bridges are wired in parallel to compensate for eccentric loading. The stainless steel load cell is protected by a brass sleeve which is flush with the outer cylinder of the IMP. This sleeve is secured at the base of the load cell with grub screws. The two screws used in Freeman's design were increased to eight to provide firm attachment without distorting the shape of the sleeve. The load cell therefore measures the accumulated shaft friction up to the top of its sleeve.

Figure 3.15 shows the uppermost load cell (LC3) removed from the head of the IMP. The inner, stainless steel cylinder of the IMP can be seen, and by the side of the load cell is its sleeve. This is a rough walled sleeve which was only used in Borehole L, as previously mentioned. A smooth sleeve was fabricated for use in the other boreholes.

As previously described, after the first site visit, a new load cell (LC1) of only 0.6 tonne capacity was fabricated. Its purpose was to measure forces being transmitted through the base of the outer cylinder, and incorporated in the base of the cell is the O-ring pressure seal required at this location. It is a more compact cell than the other two,

replacing just one 60mm long IMP segment together with the existing O-ring housing. The discarded segment was that containing the original total pressure transducer shown in Fig 3.3, which was no longer required. Another difference in the design is that the sleeve of LC1 is secured at the top of the cell, so as not to decrease the area of the IMP over which shaft friction is measured.

A more effective strain gauge arrangement, also shown in Fig 3.14 was chosen for LC1. Instead of using passive dummy gauges for temperature compensation, use was made of these arms of the bridge by placing the gauges on the columns with the grids aligned horizontally, to pick up strains resulting from the Poisson's ratio effect. The output is thereby increased whilst maintaining temperature insensitivity. To decrease power consumption and hence self heating effects which were found to be a problem with the existing design, the resistance of the transducer was increased by using only one full bridge circuit with two gauges in each arm. The sensitivity is not affected by this rearrangement and compensation for eccentric loading is now achieved by having gauges from opposite columns in the same arm of the bridge. This new strain gauge arrangement was used again when LC3 had to be regauged following the tests in Borehole I.

3.2.2 Data Acquisition System

A photograph of the system is shown in Fig 3.16, and it is illustrated schematically in Fig 3.17. The system was constructed by the author under the guidance of the Electronic Services Group in the Engineering Science Department. It was completely self contained, and was powered by four car batteries. For convenience, heavy duty Land Rover batteries were used, and the system was capable of running for several days before these required recharging. For safety, these batteries were housed in two 1/2in plywood boxes, one box providing a 12V supply the

other 24V. As shown, all the electronic equipment was housed in protective cases.

The 24V supply powered a Dome Electronics D305-2 DC/DC converter which provided the stabilised $\pm 15V, 0V$ & $+5V$ supplies required by the signal conditioning/transducer power supply unit. The converter was housed in the galvanised case on the front of the battery box, being kept well away from the signal conditioning to prevent radiation interference. Down line interference was minimised with a series of filter circuits.

The signal conditioning/transducer power supply unit was based on a modified Sangamo CA4 system, which provided the cheapest solution to powering a large amount of instrumentation at the end of 30m of cable. A Sangamo C56 oscillator card provided one single AC power supply to all the transducers via screened cables running through one of the gas hoses. Feedback wires also running through the same hose allowed the voltage at the head of the IMP to be monitored enabling the power supply to regulate it to a stable 4V. Monitoring the voltage near the instrumentation in this way was essential to avoid voltage changes as the long cables changed temperature.

The transducer signals were conducted back up along screened cabling through the other gas hose to the surface. The two hoses are shown in Fig 3.16 together with the gas exchanger originally built by Freeman to conduct the electrical wires out of the gas pressure circuit. The modified CA4 series Sangamo signal conditioning cards amplified and rectified the signals allowing a choice of gain and also permitting zero offsets to be balanced. All signals were amplified so as to give a full scale amplified output of 2VDC which was convenient for the data logger. The signal conditioning unit was earthed via the cable screening to the IMP which was of course buried in the ground.

Data acquisition was performed by a Mess & System Technik MDP-8230 intelligent data logger powered by the 12V battery supply. This was

programmed and interrogated via an RS232 link by an Epson PX8 micro-computer and disk drive, both of which ran off their own internal batteries, which could be replenished from the 12V supply. To reduce the effect of any noise, the logger was programmed to take the mean of eight successive readings over about 2ms for each reading of each channel. The time taken to log all fifteen channels of about 0.03s is negligible relative to the shortest logging interval time used which was 0.6s. The data could be stored on the PX8's microcassette, its 120kB RAM or more frequently it was stored directly onto 3.5in floppy disks. Although the PX8 was programmed to give on-site graphical display of the data, full data reduction and analysis was performed on return from the field on the Department's Vax 11-780 computer.

The back pressure applied to the IMP was controlled by the Druck DPI600TR combined regulator and pressure indicator, also shown in Fig 3.16. Nitrogen was supplied from a tank via a coarse regulator at around 800kPa. The pressure indicator gives both a digital display and an analogue output to be logged by the MDP.

Also shown in Fig 3.16 is the displacement transducer used to measure IMP penetration. It is a Celesco rotary potentiometer type of transducer which has a drawstring attached to a spring loaded spindle. The transducer was fixed to the crossheads of the jacks and the drawstring attached to the jack's base. A Sangamo CA4 card provided the AC power supply and conditioned the signal, which was again logged by the MDP. Penetration was computed from the stroke of the jacks, with occasional hand measurements of the drill string protrusion above ground surface acting as a check.

For the first, preliminary site investigation the smaller amount of instrumentation on the IMP could be powered by the power supply/signal conditioning unit built by Cambridge In-Situ for the self-boring pressuremeter, which had been used by Freeman in his laboratory tests.

This proved unsuitable for field use with the IMP since the long cables combined with the low resistance of the transducers made the system temperature sensitive, as did the less sophisticated method it employs to balance bridge circuits. A linear variable differential transformer (LVDT) was initially used to measure penetration but this was found to be insufficiently rugged for field use. The back pressure system was not used on this first site investigation.

3.2.3 Jacking Equipment

The hydraulic jacks used by Freeman in his laboratory tests were adapted for field use. These are a pair of double acting 18in stroke cylinders. Two pairs of friction jaws between the crossheads transmitted the load to the drill string. Reaction was provided by four 1.2m long ground anchors, a 50mm diameter solid steel bar being threaded through the eyes of each pair across the base of the jacks. Initially, Pilcon anchors with a 200mm diameter blade were purchased, but these were found to provide insufficient reaction for deep penetration in stiff clays, and for Borehole L a new set of 250mm diameter was fabricated by English Drilling.

A photograph of the jacking system is presented as Fig 3.18. Hydraulic pressure was provided by the portable power pack shown, designed and constructed by the author and technicians of the soil mechanics laboratory. A schematic of its layout is shown in Fig 3.19. An 8hp Briggs & Stratton petrol motor drove a Plessey gear pump, supplying oil to the jacks. The rate of penetration could be controlled, and dump valves could be used for pressure controlled load tests. The system was capable of pushing the rams through their stroke in under 30s, but a nominal penetration rate of 2mins per stroke was chosen for field use. Hydraulic pressures of up to 3000psi enabled up to 20tonnes to be exerted, given sufficient reaction. However, the friction jaws were only

capable of transmitting about 8 tonnes. The low pressure dump valve allowed a minimum load of only about 70kg to be applied to the drill string, an important feature for load tests in weak clays. Pressure gauges on the power pack were calibrated against the jacking force and could be used to give an estimate of the total load applied to the drill string.

3.3 TEST PROCEDURE

3.3.1 Calibration

3.3.1.1 Introduction

This section deals with the primary calibration and the measurement of cross-sensitivities for each of the transducers. The IMP instrumentation was recalibrated before, and if possible also after each site visit. Calibration factors and cross-sensitivities were found to vary slightly when the IMP was stripped and reassembled. One reason for this was that lead wire lengths between the transducer and the constant voltage at the IMP head sometimes changed. The sensitivity of pressure transducers to axial loads in the IMP was particularly affected by the re-orientation of the IMP segments, since this altered the axial stress distribution through the outer cylinder. Calibration factors and the principal cross-sensitivities were therefore derived for each transducer for each site visit.

The number of transducers and calibrations of them prohibits the presentation of all the calibration data. Typical calibration curves and cross-sensitivity factors are therefore shown for each type of transducer. Since the load cells are all different they are treated individually. Primary calibration factors are given for the unamplified output of the transducers, since the amplified output was set at about 2V full scale. Cross-sensitivities are presented in terms of the apparent load or pressure they induce in the transducer. The calibration factors

used in data reduction have always been calculated over the pressure/load range experienced by the transducer in the field.

Table 3.1 shows a summary of typical calibration factors and cross-sensitivities. Each of the cross-sensitivities shown were accounted for in the data reduction and so should not be regarded as errors. Data reduction also took account of any slight deviation of the power supply voltage from the nominal value of 4V.

3.3.1.2 Load Cells

An axial load calibration rig, shown in Fig 3.20 was constructed to calibrate the IMP load cells whilst in position on the IMP. Dural rings were fixed to the shaft by bolts screwed into the grub screw holes between the IMP segments. Load was applied by turn of nut on threaded bars either side of the IMP, the load being transmitted through thrust races. Proving rings were used to measure the load in each bar.

Unamplified calibration factors are presented in Table 3.1, the difference between the three load cells reflecting their relative capacities. Very little difference was detected between the tensile and compressive behaviour. A typical amplified output calibration curve is shown in Fig 3.21. The hysteresis loop of about 0.4kN is largely accounted for by forces absorbed by the O-ring at the base of the outer shaft as it is taken through a tension/compression cycle. A compression only calibration of this load cell demonstrates no visible loop. In field use LC1 and hence also this O-ring is always in compression, even during extraction, and so the hysteresis loop will not be experienced. The calculation of shaft friction by subtraction of adjacent load cell readings means that such a hysteresis error would in any case cancel. Similarly, the slight sensitivity of load cells to back pressure and also the pore pressure in the soil cancels since the effect on each load cell

is virtually the same. The error arising from this assumption was found to be too small for the system to measure.

The effect of temperature was investigated by immersing the IMP and calibration rig in water baths of various temperatures. Although the sensitivities are small, they are still accounted for in the data reduction. The only significant sensitivity is that of the zero of LC1 to temperature which results from poor strain-gauging. Later chapters will show that this may be of concern for shaft friction measurements in very weak clays.

The load cells were found to be insensitive to any reasonable eccentricity of loading.

3.3.1.3 Pore Pressure Transducers

For transducer saturation and calibration, the de-airing cylinder shown in Fig 3.22 was constructed. It was sealed onto the IMP shaft by means of O-rings at either end. It also protected the IMP and maintained saturation during transportation. Calibration could be undertaken by applying air or nitrogen pressure directly to the cylinder before de-airing, or via an air/water interface afterwards. No difference in calibration factors derived by these two methods was observed.

The Druck PDCR-81 transducers use semi-conductors and consequently give a much higher output than the strain-gauged transducers as Table 3.1 shows. The linearity of the calibration curves of amplified readings from the Druck transducers was found to be excellent as shown by Fig 3.23. This particular calibration involved loading to 400kPa, then unloading by means of increasing the back pressure to 500kPa, followed by unloading of the back pressure and finally unloading of the front pressure. Readings were generally taken at 50kPa intervals throughout. As can be seen, there is no visible difference between the sensitivity to back and front

pressures. The same was found to be true for both the strain-gauged type of pore pressure transducer and the total radial stress transducers.

Figure 3.24 shows the corresponding calibration curve for one of the strain-gauged pore pressure transducers. This has a hysteresis loop of about 10kPa. However, this calibration goes far beyond the pressure range it experiences in the field and so the error in field data arising from hysteresis will be very much smaller.

Both types of pore pressure transducer showed a slight sensitivity of their calibration factors to temperature, and the strain-gauged type showed a serious cross-sensitivity to axial load in the IMP. If this were not calibrated for, the typical sensitivity of 2kPa/kN would give rise to an error of perhaps 30kPa for a transducer located near the head of the IMP whilst it was penetrating through stiff clay.

The strain-gauged pore pressure transducers were also found to be slightly sensitive to the net total radial stress acting on the IMP, ie. the total radial stress acting on it less the back pressure. This was calibrated for by blocking the transducer filters before calibration. The typical value shown would give rise to an error in the vicinity of 5kPa in the pore pressure reading in a stiff clay if it were not calibrated for.

3.3.1.4 Total Radial Stress Transducers

The amplified signal calibration curve of a typical total pressure transducer shown in Fig 3.25 demonstrates good linearity. The calibration factor was insensitive to temperature, but zero readings were significantly affected. These transducers had a similar cross-sensitivity to axial load to the strain-gauged pore pressure transducers.

Sensitivity of the total pressure transducers to shear acting on their face was found to be too small to measure accurately. Shear stresses in a stiff clay would be likely to alter the measured total

radial stress by somewhere in the region of 0.5kPa. Sensitivity to eccentric loading would be similarly insignificant.

3.3.1.5 Discussion on Instrumentation Accuracy

Zero readings were taken for each transducer before and after each borehole. Table 3.1 shows typical differences that might be observed at a stiff clay site between the two sets of zeros. The larger errors seen by the total pressure transducers are probably related to the temperature sensitivity of their zeros. Much less difference between the two sets of zero readings was seen at the softer clay sites visited, particularly for the load cells. Unless something is known to have gone wrong with a particular transducer a mean of the two zero readings was used in data reduction. The plots of borehole data shown in the following chapters will therefore show disagreement between the zeros before and after a borehole for a particular transducer by the data plot not commencing on the zero line, or the hydrostatic line in the case of pressure transducers.

As previously stated, the cross-sensitivities of transducers do not represent errors, since they were accounted for in the data reduction. However, they do indicate where the inaccuracies of the transducers lie, since in general it was found that cross-sensitivity calibrations showed poorer repeatability than the primary calibration of a transducer. This is partly a function of the lower readings involved.

Temperature correction was achieved by monitoring laboratory temperatures during calibration and borehole temperatures in the field. However, for most boreholes only one or two temperatures have been taken, at a depth of perhaps only 2-3m. It is likely that the temperature at depth might be up to perhaps 1°C different, representing an error of about 5kPa for a typical total pressure transducer.

From the above considerations, it is likely that the PDCR-81 pore pressure transducers are accurate to $\pm 5\text{kPa}$ at the stiff clay sites, the bulk of this error arising from the difficulty in obtaining consistent zero readings before and after boreholes. Strain-gauged pore pressure transducer measurements are probably within $\pm 10\text{kPa}$, total pressures within $\pm 20\text{kPa}$ and load cell readings within $\pm 0.3\text{kN}$. Likely errors at the two soft clay sites would be about half these values.

As will be seen in later chapters this relative inaccuracy of the IMP instrumentation may lead to significant unreliability of the calculated radial effective stresses at soft clay sites, although the high redundancy of the instrumentation helps to counteract this.

The performance of the IMP instrumentation which was poorer than intended arose from the fact that the system was designed for the very high total and pore pressures in stiff clays that are predicted by cavity expansion. At the time no better information was available, and consequently, making use of the back pressure system, the IMP can cope with pore pressures up to 1100kPa and total pressures up to 1300kPa . As will be discussed in the following chapters, much lower pressures were actually encountered at these sites and the IMP pressure transducers were therefore working well below their anticipated range.

Bearing in mind these problems, the data presented show that when the stress regime around the IMP might be expected to be stable, such as during reconsolidation, remarkable consistency of readings has been achieved.

3.3.2 Preparation of IMP

Before each field trip the IMP was reassembled in the desired configuration from its interchangeable segments, and the transducer circuits connected up to the cables running through the gas hoses. It was then tested for leaks under back pressure in a water bath.

The surface of the IMP was prepared by rubbing down with a fine (grade 600) emery cloth, the dust created being washed off. From Borehole J onwards, the four black permanent marker pen lines shown in Figs 3.12 & 3.20 were drawn longitudinally along the IMP, each about 2mm wide. The purpose of these was to investigate the abrasion that the IMP experiences underground. Similarly, 15mm wide hoops of emulsion paint were applied at 150mm intervals to the extension tube and drill rods following Borehole E.

To minimise resistance to soil entering the IMP, the plunger and sample cylinder were thoroughly oiled and greased. The IMP was then inserted into its de-airing cylinder, and an air/water interface chamber attached to one of its pressure inlets. Saturation was achieved by applying a vacuum to the cylinder via the interface for 45mins and then flooding it with de-aired water through the pressure inlet on the opposite side of the IMP. The vacuum was then maintained for a further 15mins. Three or four more cycles of vacuum of at least 30mins were then applied again via the interface chamber which was now partly flooded. If possible, these were applied over several days. The IMP, which was horizontal during de-airing was rotated through 180° at least once with the vacuum always being pulled through an interface attached to the top of the cylinder.

To check the efficiency of de-airing, perspex simulated transducers were mounted in a model IMP shaft inserted in a perspex de-airing cylinder. The same de-airing sequence was found to give perfect saturation, and further tests showed that upon removal from the de-airing cylinder, the only means by which the transducers become unsaturated is by evaporation from the filter face. Prior to these model tests, glycerine had been used to replace the de-aired water in the cylinder after the final vacuum cycle. Students at Oxford, such as Gue (1984) had found that since glycerine and water are miscible, continued soaking in

glycerine of transducers saturated with water allowed some glycerine to enter the transducers and so help maintain saturation whilst they were installed in kaolin samples.

The IMP was de-aired separately for each borehole, with the exceptions of Boreholes B & F. For these two boreholes the filters were covered with wet mud for the time of around ten minutes taken to transfer the IMP from the previous borehole.

3.3.3 Field Testing

Upon arrival at a site, the first task was to auger a short starter borehole by hand. Following doubts over the axiality of jacking in Boreholes A & B, the tripod augering guide shown in Fig 3.26 was constructed. This could be levelled and helped ensure verticality of the borehole. Augering was terminated when the IMP could rest completely submerged below the water table.

The four ground anchors were then screwed in, and their eyes levelled so that during penetration when the jacks reacted against them the jacks would also be level and perpendicular to the borehole. After positioning the jacks over the borehole and filling it with water, the IMP was pushed directly from its de-airing cylinder into the borehole water. The IMP was then left for several hours, and overnight if possible to assume ground temperature. The instrumentation took about 1.5 hours to stabilise before testing and zero readings were taken under hydrostatic pressure in the starter borehole immediately before penetration.

The IMP boreholes were semi-continuous soundings, the IMP being jacked through its entire length from the base of the starter borehole and breaks only occurring to add on extra drill rods or to allow a reconsolidation phase. Typically, three strokes of the jacks taking around 7mins in total, could be made before a new drill rod was required or lack of disk storage space dictated a break. Ten minutes was a typical

time for a drill rod change, slightly longer being taken if the mechanism was operated to proceed open-ended. Each jack stroke was fixed at about two minutes, giving a nominal penetration rate of around 200mm/min which combined with the logging interval of 0.6s, gives one dataset every 2mm. When used open-ended, the IMP was generally found to plug before the sample chamber was full, particularly in the stiff clays. Further penetration beyond this point was therefore once again closed-ended.

If penetration was halted to allow reconsolidation, the logging interval time was increased from 0.6s to 5s after a few minutes, and then 100s after say 30mins. After several hours had elapsed, readings were taken by hand. The back pressure was altered by hand during reconsolidation, trying to maintain a constant differential reading on one selected total pressure transducer. Generally, all total pressure transducers showed similar consolidation behaviour, so the error incurred on the other three transducers not traced with the back pressure is minimal.

Following the full reconsolidation which was allowed in some boreholes, undrained load tests were conducted. The hydraulic pressure in the jacks was increased over a period of about a minute until failure was obtained, motion of the IMP being halted after 30mm. The stabilisation of pore pressures was then observed for up to 2hours before conducting the next test. The precise programme varied from site to site, but generally two or three monotonic compressive tests were followed by a tensile test and then a further compressive test, the latter investigating behaviour with little end bearing on the IMP. These were followed by a series of pressure controlled and then displacement controlled cyclic load tests. For greater accuracy during the load tests, the displacement transducer was referenced to the ground surface rather than measuring the jack stroke.

After the reconsolidation and load tests, the IMP sounding would be continued to its limit before the IMP was extracted. The limit was determined at the stiff clay sites by the capacity of the load cells or the ground anchors. Since extraction of the IMP from the borehole was not of primary interest, this was done as quickly as possible, with a more rapid jack upstroke of around 600mm/min and a logging interval time typically of 1s. A second set of zero readings was obtained on extraction of the IMP from the sounding.

In several boreholes, back pressure tests were conducted to investigate the effect of transducer compliance. Data-logging at 1s intervals, a large, sudden back pressure change of for instance 100kPa was imposed, and the effect on the total and pore pressures measured was monitored.

During all tests performed with the IMP, data-logging was always sufficiently rapid so that in the presentation of the data in penetration profiles or reconsolidation curves, straight lines could be used to join data points, and no curve fitting has been employed.

CHAPTER 4

FIELD TESTS AT MADINGLEY

4.1 INTRODUCTION

The Madingley site is located near the geotechnical centrifuge of the Cambridge University Engineering Department, on the Madingley road just outside the city. This site was chosen since model pile tests had already been conducted here by Clegg (1981) and extensive site investigation data are available. In recent years it has been used as the test-bed site for several new site investigation instruments. Table 4.1 summarises the IMP site investigation work undertaken at Madingley. Following a discussion of the site conditions, data from the IMP investigation will be presented in the same order as was used in the literature survey: 1) installation 2) reconsolidation and 3) loading.

4.2 SOIL CHARACTERISTICS

Figure 4.1 shows the location of the IMP boreholes relative to the other site investigation work. The Fugro investigation of 1979 shows that the Gault clay extends to at least 30m. This is a stiff, fissured silty clay, as can be seen from Fig 4.2 which shows Fugro's sample descriptions together with a graphical representation of their Atterberg limit data. Figure 4.3 shows c_{uo} data from a number of sources. In drawing the interpreted profile, most weight has been given to the undisturbed triaxial compression tests conducted by Fugro and by Clegg, and to the self-boring pressuremeter tests by PM In-Situ (1986). Remoulded shear strengths by Clegg and by Fugro have shown the clay to be insensitive.

In-situ values of G are available from the pressuremeter tests, the data from which are also shown on Fig 4.3.

Estimates of in-situ stresses are shown in Fig 4.4. The in-situ pore pressures were measured from standpipe piezometers installed by Fugro. These show that there is slight downward drainage into the underlying

Lower Greensand. The effective vertical stress has been calculated using Fugro's bulk densities, and for clarity only one profile based on mean pore pressures is presented. None of the analyses presented later is sufficiently sensitive that they require any better estimate. The profile of horizontal effective stress has been derived from pressuremeter data. Since pressuremeter values of in-situ pore pressures showed a large scatter, the standpipe values measured by Clegg at the time of the pressuremeter tests have been used.

Figure 4.5 presents estimates of OCR. Values have been calculated from in-situ stresses by means of the relationship proposed by Mayne & Kulhawy (1982), selecting a ϕ' of 24° from Clegg's data:-

$$K_o = (1 - \sin \phi') OCR^{\sin \phi'} \quad (4.1)$$

Very much lower OCRs have been calculated from Fugro's oedometer data and by Powell & Uglow (1986) from dilatometer tests. The discrepancy may arise from the fact that Equation 4.1 applies to primary unloading only.

4.3 IMP INSTALLATION

4.3.1 Raw Data Profiles

4.3.1.1 Presentation of Data

Figures 4.6-4.10 show IMP transducer readings during penetration in each of the Madingley boreholes, with the exception of Borehole L which will be discussed in a later section. Borehole D has also been omitted since it was abandoned after only a couple of metres penetration. The data presented in these diagrams are the raw pressures and loads measured by the transducers, and there is therefore a small component of the pressure transducer readings due to initial in-situ pore pressures. Reference should be made to Figs 3.7 & 3.8 which identify the colours used for individual transducers on these plots and give their locations on the IMP.

Each plot is constructed from around 10-20,000 data points, and it is therefore sufficient to join these with straight lines. With such a vast quantity of data, emphasis has been placed on the clarity of presentation at the expense of some detail. Transducers mounted opposite each other on the IMP are shown with the same colour as indicated on the key. In each case it is the penetration of the transducer which is plotted, so the various readings shown at a particular depth will have been made at different times as the IMP penetrated past that point.

A study of the IMP data from all sites has shown that transducer TP2 tends to register the higher total radial stress if there is a significant difference between it and its twin, TP1. Similarly, PP3 tends to register the higher pore pressure if there is any difference compared to PP2. No other consistent discrepancies between transducer pairs were observed. Filter type was not found to influence consistently pore pressure data. The discrepancies between both these and the total pressure transducers are therefore not easily explained, since the transducers are of similar types. The answer may lie in the accuracy with which the transducers were installed in the IMP shaft. Their design only allowed them to be placed to within around $\pm 0.1-0.2\text{mm}$ of being flush with the shaft. The fact that TP1 and PP2 are slightly recessed by comparison with TP2 and PP3 may explain the lower pressures registered, although it is difficult to understand why agreement should be better in some boreholes than others.

The colours used for each transducer data profile are specific to the transducer location, so when the two sets of pressure instrumentation were swapped following Borehole F, the transducers represented by each colour change.

For ease of comparison shaft friction load cell data are presented alongside that from the pressure transducers. For these the penetration plotted for all three is that of the base of LC1, ie. the bottom of the

friction sleeve, from which point the shaft friction measured by each of the load cells is cumulative. With this method of presentation, features such as the start of the sounding are easily picked out. However, it does mean that the readings are not representative of the shaft friction at the penetration plotted, but represent the accumulated shaft friction over the friction sleeve, the tip of which is at that depth.

Also for clarity, the same penetration scale is used for all boreholes at each site, even though this means that only small plots result from some of the shorter boreholes.

4.3.1.2 General Description of IMP Data Profiles

Borehole E was a semi-continuous sounding with no prolonged breaks, and is therefore examined in detail as representing typical Madingley data. Considering first the load cell data, the most obvious feature is the start of the sounding as the IMP was pushed through the base of the pre-augered hole. At first all three load cells registered the same reading, until the base of the friction sleeve entered the soil at which point LC2 and LC3 diverged from LC1. Similarly, when the top of LC2 passed into the soil, the reading it registered levelled out, whereas that on LC3 continued to increase until the top of that cell entered the soil. As previously explained, LC1 measures only the "error" in axial load passing through the base of the friction sleeve. In the early boreholes at Madingley the loads measured by LC1 were unusually high because the gap between it and the tip section of the IMP was part filled with silicon sealant to help maintain the pressure seal. Shaft friction values will still be correct since the same increase will be measured by the other two load cells. This silicon was removed when it was realised that it was transmitting significant load and was in any case unnecessary. The spikes and gaps which interrupt the load cell profiles mark breaks between jack strokes.

The pressure transducer traces start at hydrostatic pore pressures in the pre-augered hole. As the sounding starts, the total radial stress transducers picked up the base of the hole very clearly. A comparison of boreholes starting at different penetrations shows that the soil is softened immediately below the pre-augered hole such that it is around 0.4m before the total radial stress readings reach a consistent profile. Total radial stress profiles show a significant gradient until 5-6m, then tending to be more constant with depth, which corresponds well with the undrained shear strength profile.

Large positive pore pressures are frequently seen in the softened zone at the start of the sounding. Pore pressure profiles generally show considerable scatter, very different readings being registered both along the length of the IMP and around its circumference. Some profiles are smooth and featureless, which was initially thought could result from poor saturation of those transducers. This is highly unlikely since all transducers were de-aired simultaneously. A closer examination shows that poor saturation is certainly not the reason, since transducers showing smooth profiles are capable of registering crisp responses if sudden changes are there to be measured. For example, both lower friction sleeve pore pressure transducers (brown) register remarkably uneventful profiles in Borehole E until at about 8m both see a very sudden drop in reading.

Violently oscillating pore pressure responses are generally only seen by the tip pore pressure transducer or sometimes by those in the lower friction sleeve. This might indicate that the smooth profiles result from the fact that pore pressures at a given depth in the soil have become stable by the time that transducers reach it.

Changes in the nature of pore pressure profiles are frequently seen, such as smooth profiles becoming more oscillatory and vice versa, or as already mentioned, violent changes in otherwise uneventful profiles. These may result from changes in the nature of the soil, but it is

possible that changes in the pattern of soil movement around the penetrating IMP are responsible. It is also interesting to note the asymmetry around the IMP, both in the magnitude of pore pressure response and the type: smooth or oscillatory.

During longer breaks in jacking, for instance to change drill rods, large increases in pore pressure are frequently observed, but upon restarting, pore pressures generally return quickly to their former values. This is most clearly shown in Borehole E where both the tip (black) pore pressure profile, and one from the upper transducer set (blue) show spikes between jack strokes.

4.3.1.3 Comments on Specific Profiles

Borehole C was the first done at Madingley, and after initial open-ended penetration was closed-ended for the remainder of its length. Following the first jack stroke, the IMP was extracted, before repenetrating, giving rise to the loss in shaft friction seen for the second jack stroke. Large differences of around 100kPa are observed for pairs of opposing total radial stress transducers, and there is a reduction in reading of about 200kPa in the readings of the upper pair of transducers compared to the lower. Tip pore pressures oscillate around zero. Those measured on the shaft also show large differences between the two pairs of transducers, but there is still a clear trend for pore pressures to increase with depth from values around zero immediately below the zone of softened clay. Since Fig 4.4 shows that the in-situ pore pressure gradient is small by comparison, it is likely that this increase reflects the change in c_{u0} .

As the Madingley programme continued, there was a tendency for the profiles of total and pore pressure from the various transducers to show better agreement, both around the circumference and between the upper and lower sets of instrumentation. It is possible that this may have been

influenced by the progressively greater care taken to ensure axiality of jacking and the verticality of the borehole. Data from the earlier boreholes could also be influenced by the fact that at this time the IMP sections were secured to each other with grub screws which in trying to maintain axial rigidity were rather too enthusiastically tightened, leading to distortion of the shaft. As has already been highlighted, maintenance of a perfectly flush surface could be very important.

On the next visit to Madingley Boreholes E & F were conducted. Borehole E was a continuous closed-ended sounding. For Borehole F the IMP penetrated open-ended from tip penetrations of 3.76m to 4.17m, at which depth full reconsolidation was allowed. Gaps in the profiles appear at the depth of each of the transducers during reconsolidation, and represent the section of borehole taken up by load tests which followed reconsolidation.

Tip pore pressures (black) are very low in these two boreholes, generally below 200kPa and frequently negative. Pore pressures measured on the lower friction sleeve (brown) give higher readings than those seen at the tip, but there is then a decrease along the shaft to readings near the IMP head (blue) which are similar to those at the tip. The distribution of pore pressures is therefore quite different from that predicted by Levadoux & Baligh for normally consolidated clays.

There is a corresponding decrease in total radial stress along the shaft, but it is interesting to note that just above the tip penetration during reconsolidation (4.17m) there is a short section of borehole where the two sets of total radial stress transducers register similar readings. These readings were made as the transducers passed through soil that was adjacent to the IMP during reconsolidation. Load tests had little effect on the total radial stress, and the lower readings in this section are similar to those at the end of reconsolidation. The fact that the two sets of readings are much closer is confirmation that total

radial stresses are seeing the same drop along the shaft of the IMP as pore pressures during continuous penetration, and that the effective radial stress is similar at the location of both pressure transducer sets.

Some difference between the measured stresses is evident in Boreholes E & F. During continuous penetration both show similar total radial stresses, but higher pore pressures were measured in Borehole E than in Borehole F.

Boreholes G & H were undertaken on the third site visit to Madingley. Borehole G was essentially a repeat of F, with open-ended penetration commencing at a tip penetration of 5.04m, and full reconsolidation being allowed at 5.79m. The remainder of the borehole was closed-ended. Unfortunately the data between tip penetrations 3.61-4.90m were marred by a fault in the transducer power supply system. No recurrence of this problem was seen, and these data have been omitted for clarity.

Borehole H was continuous apart from a break of 40mins at 5.25m to conduct back pressure tests. Penetration then continued open-ended until the IMP plugged at around 5.8m. Data from tip penetrations 4.49-5.25m were lost as a result of a faulty floppy disk.

The continuous penetration parts of the two boreholes show very similar total radial stress measurements made on the lower friction sleeve compared to the previous boreholes. However, there appears to be a much smaller decrease of total stress between the readings from the lower transducer set and those of the upper for a given soil depth. There is a correspondingly smaller variation of pore pressure measurements made by the two principal transducer sets, and pore pressures measured on the lower friction sleeve (brown) are much lower for instance than in Borehole E. Consequently it will be shown that rather higher radial effective stresses act on the IMP in Boreholes G & H, which are again fairly similar at a given depth for the two transducer sets. The load

cell profiles show that correspondingly greater shaft friction was developed.

4.3.1.4 Comparison With Other Data

Lunne, Eidsmoen, Powell and Quarterman (1986) have published piezocone data for the Madingley site. The Fugro type of cone with the filter in the cone face showed very high pore pressures (about 1MPa at 5m depth) with a similar profile shape to and also the characteristically oscillatory nature of the cone resistance profile. However, the Delft Soil Mechanics Laboratory cone, which has the element fitted behind the shoulder, produced the profile shown in Fig 4.11. There is some similarity with IMP measurements, and it is interesting to note the smooth nature of the data.

Figure 4.12 shows a comparison between IMP data from all boreholes with pressuremeter data from the site. The PM In-Situ self boring pressuremeter tests show terminal pressures at the maximum cavity strain of about 15% which are about 100kPa higher than the total radial stresses measured by the IMP. The inflation curves indicate that limit pressures had not been reached and this is confirmed by data from the prototype cone pressuremeter tested by Fugro on this site for which true limit pressures are reached. This instrument has a pressuremeter mounted behind a piezocone. Since there is a zero initial cavity size, inflation curves from this instrument quickly reach a limiting pressure, and values plotted on Fig 4.12 are very much higher than self-boring pressuremeter terminal pressures and total radial stresses measured by the IMP. Since the installation of the instrument is similar to that of the IMP, it is not surprising that the lift off pressures do show good agreement with total radial stresses on the IMP during installation. If the installation of such an instrument were correctly modelled by cylindrical cavity

expansion, then it is clear that no further increase in pressure would be seen after lift off.

Total radial stresses measured on the IMP are therefore very much lower than limiting cavity expansion pressures, and it is likely that as Levadoux & Baligh predicted for normally consolidated clay, there is a large reduction in total radial stress on the shaft immediately behind the shoulder of the instrument. This might be particularly severe in such a stiff overconsolidated clay.

4.3.2 Data Analysis

Figures 4.13-17 show interpreted profiles of unit shaft friction, USF, effective radial stress, σ'_r , and the ratio of the two. For ease of comparison, data from both friction sleeves are presented on the same plot. The USF and σ'_r profiles have also been combined on one graph to highlight the strong similarities between them.

The USF was calculated from the difference between LC2 and LC1 readings for the lower friction sleeve, and between LC3 and LC2 for the upper. The penetration used for the calculated values was that of the centre of each friction sleeve, or at the start of the borehole, the centre of the embedded portion of the sleeve.

Effective radial stresses have been calculated at the location of the two main sets of transducers using mean values of total and pore pressures from the pairs of transducers at each location. Since the shaft friction should be related to the effective radial stress integrated along its length, each point on the σ'_r profile was calculated by taking an average effective radial stress over a length of borehole equal to the relevant friction sleeve length, centered on that point. This method of comparison assumes that the effective radial stress acting at a given position on the friction sleeve is equal to that measured by the pressures transducers when they pass through that same element of soil.

This assumption appears valid since there is generally little change in σ'_r calculated at a given depth for the two sets of transducers.

Another assumption made in these calculations is that penetration is continuous. Consequently a gap has been left in the σ'_r profiles either side of the penetration at which the particular set of transducers stopped for reconsolidation. This gap represents the section of borehole for which there are insufficient continuous data to calculate an average σ'_r over the length of the sleeve. Similarly, if significant changes in effective radial stress should occur during breaks in jacking such as drill rod changes, then the calculated σ'_r profile will have the effect of averaging stresses before and after the break. The resulting smoothed out σ'_r profile will therefore not be correctly representative of stresses actually acting on the IMP immediately before the break or upon restarting penetration. Typically, the additional time taken to add on a drill rod might increase the time (s) between the two sets of transducers reaching a given point from $\log_{10} t = 2.5$ to 2.8. Reconsolidation data at the normally consolidated clay sites show only small stress changes in this time. However, as has already been seen, in this stiff, overconsolidated clay large increases in pore pressure are frequently seen during breaks in jacking, although upon restarting there is a tendency for readings to regain quickly their former values.

Bearing in mind these limitations, the profiles produced are still able to illustrate some important points. As had been expected from the raw transducer data profiles, most boreholes show little variation of σ'_r calculated from the two sets of transducers. Where differences do occur, there is no consistent pattern. Similarly, USFs calculated for the two friction sleeves are generally very close, and the shape of the friction profiles are always remarkably similar to those of σ'_r .

For the lower friction sleeve the ratio USF/σ'_r is simply calculated using data from the set of transducers at the centre of the sleeve.

However, the upper pressure transducer set is located above the upper sleeve, and the σ'_r used for the upper sleeve is therefore interpolated between the values calculated for each of the transducer sets.

No consistent difference in the ratio USF/σ'_r is observed for the two friction sleeves. As was clear from the raw data profiles, there is considerable variation in the stresses and friction measured in the various boreholes. As an example, radial effective stresses calculated for Borehole H are around 50% higher than those for Borehole E. Whether this reflects actual variation in soil conditions or as previously discussed, improved quality of IMP assembly, is not known. However, it is encouraging that both boreholes show very similar USF/σ'_r ratios.

A considerable increase of σ'_r with depth is usually seen, presumably resulting from the c_{u0} increase, but again the USF/σ'_r ratio is fairly constant. A striking example of this can be seen in Borehole C, where the IMP was extracted and then reinserted after the first jack stroke. Consequently, very low effective radial stresses of around 70kPa are initially registered on the lower friction sleeve (brown), compared to over 400kPa for the remainder of the borehole. The ratio USF/σ'_r ratio is, however, similar.

Figure 4.18 shows all the data from Figs 4.13-17 superimposed on the same plot. The USF/σ'_r ratio appears to show a slight increase at the start of the boreholes. At 2.5m depth, the mean is around 0.14, representing a mobilised friction angle on the IMP shaft of about 8° . From 4m onwards the ratio is constant at about 0.16 ($\approx 9^\circ$). There is considerable scatter, and in places, friction angles as low as 5.3° are confirmed by both friction sleeves. Relative to their magnitude however, the calculated ratios show very much less scatter than the individual plots of USF and σ'_r .

Clearly the low friction angles indicate residual failure conditions. Lupini (1981) collected together data from ring shear tests from numerous

sources. Data he presented from Gault clay sites in other parts of the country do indicate low drained residual friction angles, with some values less than 7° . However, as Lemos (1985) demonstrated, the interface friction angle measured by the IMP will be influenced by the smoothness and hardness of its surface, the type of soil and the speed of penetration.

Sleeve friction measurements made by the DSML cone shown in Fig 4.11 fall near the upper bound of IMP values, providing some confirmation of IMP measurements.

Figure 4.19 shows the installation shaft friction normalised by the initial undrained shear strength and also by the initial vertical effective stress. Neither provides a satisfactory correlation, and the installation α values show a pronounced increase with depth. This would not be expected from cavity expansion predictions, since in a given soil, for which M_{ps} would be expected to be fairly constant with depth, the installation USF should be a constant multiple of c_{uo} and it is installation β values that would be expected to vary with depth.

Hydraulic oil pressure data during penetration are difficult to analyse in terms of the shaft friction on the IMP because of the unknown end bearing force. If it is assumed that the unit end bearing is equal to the cone resistance from one of Fugro's tests, then it is clear that significant shaft friction was developed on the drill string. An examination of cone pressuremeter records showed that it was not likely that the soil would contact the drill rods during continuous penetration. If all of this shaft friction is therefore developed on the extension tube, the values of unit shaft friction shown on Fig 4.18 are calculated. Uniform friction over the whole length of the IMP was assumed, equal to that on the friction sleeve. The shaft friction on the extension tube is surprisingly high considering that there is a 5% reduction in cavity strain from the IMP shaft to the extension tube.

4.3.3 Comparison With Cavity Expansion Predictions

Cavity expansion estimates for the pore pressure on the IMP shaft during installation have been predicted by the following equation proposed by Wroth et al.:-

$$\Delta u_{sh} = (p'_o - p'_f) + c_{uops} \ln(G/c_{uops}) \quad (4.2)$$

The soil strengths, stiffnesses and in-situ stresses used were those shown on Figs 4.3 & 4.4. Wroth et al. suggested the following equation for the mean effective stress at the critical state, p'_f :-

$$p'_f = (\sqrt{3}/M) c_{uops} \quad (4.3)$$

The slope of the critical state line used should also be one appropriate to plane strain conditions, which Wroth et al. suggested could be calculated in the following way:-

$$M_{ps} = (\sqrt{3}) \sin \phi'_{ps} \quad (4.4)$$

As Wroth (1984) proposed, ϕ'_{ps} has been estimated as $(9/8)\phi'_{tc}$ giving a M_{ps} of 0.786 for a ϕ'_{tc} of 24° .

Since the triaxial and pressuremeter c_{uo} data are inseparable at this site, two sets of calculations have assumed that the selected profile is either representative of plane strain or of triaxial compression. Use has been made of the following relationship, proposed by Wroth et al. to relate triaxial compression shear strengths to the plane strain values which should be used in cavity expansion:-

$$c_{uops} = (2/\sqrt{3}) c_{uotc} \quad (4.5)$$

The initial in-situ pore pressure, u_o is added to the increase predicted by Equation 4.2 (Δu_{sh}) to obtain the total pore pressure which may be compared with IMP readings during installation. Wroth et al. found that the above method provided pore pressure estimates within 10% of those from more sophisticated numerical analyses.

The pore pressures predicted for installation by the equation of Wroth et al. are illustrated in Fig 4.12, the shaded band representing the range between the two extreme assumptions for the c_{uo} values.

The radial effective stresses were estimated from the following equation also proposed by Wroth et al., which assumes that the soil is at its critical state with a radial major principal stress:-

$$\sigma'_r = ((\sqrt{3}/M) + 1) c_{uops} \quad (4.6)$$

The effective radial stresses calculated for the two assumptions about c_{u0} are shown in Fig 4.12, as are the total radial stresses calculated by adding them to the pore pressures predicted above.

The above cavity expansion predictions assume the initial soil state to be isotropic, and as Wroth et al. suggest, are also only appropriate to stresses acting on the pile immediately after installation, since the major principal stress is assumed to be radial, thereby ignoring the shear stresses on the pile. Sagaseta (1984) has extended the closed form solution for cavity expansion of Gibson & Anderson (1961) to account for the effects of initial anisotropic stresses and the friction generated at the pile/soil interface during installation. The soil is assumed to be elastic, perfectly plastic, but this assumption has been found not to influence significantly predictions or conclusions drawn from them. The frictional stresses used between the IMP and soil for these calculations were the actual USF measurements made by the IMP. For comparison with field measurements the in-situ horizontal total stress is added to the cavity expansion stress predicted by his program. As shown on Fig 4.12 the total radial stresses are in close agreement with those which arise from the method of Wroth et al. This emphasises the insensitivity of cavity expansion predictions to the effects of initial in-situ stresses and friction on the interface, since the two methods would give identical answers for a frictionless pile in an isotropic soil. Installation effective radial stresses and pore pressures will be similarly insensitive to these factors.

Figure 4.12 shows that radial effective stress predictions fall within the scatter of IMP measurements during installation and show an

encouragingly similarly shaped profile. They are however around 20% higher than the mean of IMP measurements. Radial total stress and pore pressure predictions also show similarity of profile shape with IMP measurements, indicating that the stresses generated are strongly dependent on c_{uo} . However, cavity expansion predictions are very much higher, as had been suspected from the cone pressuremeter data, and measured total stresses on the lower sleeve are as little as half the predicted values. Limit pressure data from the cone pressuremeter do give better agreement with the total radial stresses predicted by cavity expansion.

4.3.4 IMP Extraction

Figure 4.20 shows data analysed from the extraction of the IMP from Borehole G. Effective radial stresses and shaft frictions are generally only slightly lower than during installation despite the fact that full reconsolidation had been allowed at 5.79m during installation. Values of USF/σ'_r tend to be on the low side of the scatter for this site, a surprising result since in his tests in London clay, Jardine (1985) had found that the speed of installation significantly affected the shaft friction developed. IMP installation speeds were generally in the range 200-400 mm/min, whereas during extraction, the average speed was 1300mm/min. If Jardine's findings were applicable to Gault clay, an increase of shaft friction of around 20% would be expected.

Raw transducer profiles for extraction are shown for Borehole E in Fig 4.21, which was a continuous sounding followed by immediate extraction. Tensile load cell forces are shown as positive on this diagram. High pore pressures, low radial effective stress and hence low USF are seen on extraction. This is particularly evident in the soil which was above the top of the extension tube at the deepest penetration, again perhaps indicating that the soil was in firm contact with the

extension tube, but not with the drill rods. The data from this extraction give similar USF/σ'_T values (not shown) although with greater scatter partly arising from the longer scanning interval used (5s). It is interesting to note that during extraction the total radial stresses measured near the IMP head are closer to those near the tip, but not greater, as might have been expected from the fact that higher pore pressures are registered near the head.

Upon retrieval from the boreholes, the IMP was found to be covered only by loose, watery clay, which when washed off revealed a highly polished IMP surface, as shown in Fig 4.22. This photograph also shows the cone formed under the IMP tip during closed-ended penetration. The extension tube was also found to have been abraded, all rust having been removed and only a few remnants of the emulsion paint hoops remaining, as illustrated by Fig 4.23. By comparison the hoops on the drill rods were unscathed, confirming that little shaft friction was picked up by them.

4.3.5 Rough Walled IMP Tests

As described in Chapter 3, Borehole L was conducted with a knurled upper friction sleeve. Figure 4.24 shows the coating of firm clay found to be adhering to the rough section on extraction. The thin soft mud on the smooth lower sleeve has dried out in the few minutes before the photograph was taken. A groove has been drawn through the coating demonstrating its thickness to be around 4-5mm. The coating commences below the start of the roughened section, and tapers away similarly above it, the stainless steel section of the upper friction sleeve which was not knurled giving rise to a neck in the clay cladding.

Upon removal of the clay, marker pen lines shown in Fig 4.25, which originally extended the length of the IMP were intact beneath the coating even where it extended onto the smooth parts of the IMP shaft. Clearly this coating has travelled down with the IMP during penetration. The

majority of displacement taking place on a surface some distance from the IMP shaft.

This test provides strong support for the findings of researchers at Imperial College (Martins (1983), Kitching (1983) & Jardine (1985)), who have suggested that during installation a residual failure surface forms in the soil and not on the shaft. The lack of a continuous brass surface forces a soil/soil failure for the rough walled IMP section. However, cavity expansion predicts constant effective stresses within the zone of soil at critical state and so failure would still be expected adjacent to the IMP where shear stresses are greatest. If the residual failure surface is to form elsewhere, effective stresses cannot be constant in the failed zone, although once a residual failure surface has formed it is a stable feature since peak ϕ' must be developed on any other potential failure surface. In this case, it would not matter if the highest shear stress were adjacent to the IMP.

These observations are particularly interesting in the light of reconsolidation data to be discussed later, which show that the greatest pore pressures may not be developed adjacent to the IMP shaft. However, with the smooth IMP, lower friction angles on the shaft might keep failure on this surface, even though effective stresses were lower elsewhere.

The raw transducer data and analysed data for Borehole L are presented in Figs 4.26 & 4.27. The tip pore pressures (black) and lower friction sleeve total radial stresses (red) are much as in other boreholes. Pore pressures on the lower sleeve (brown) are lower than previously measured, perhaps being influenced by the proximity of the transducers to start of the clay coating.

The size of the coating on extraction was probably representative of its thickness at maximum penetration. A crude calculation, scaling from the deflation curve of a cone pressuremeter test at 9m, and assuming that

full limit pressure is developed on the outside of the coating, predicts that the total radial stress measured above a 4.5mm coating would be about 200kPa. Figure 4.28 shows a sketch of an idealised pressuremeter inflation/deflation curve illustrating this calculation. The assumption is also made that strains calculated from this deflation curve are applicable to the larger radial dimension of the IMP. Data from the upper set of total radial stress transducers (green) show a trend to such a low figure only towards the end of the borehole, and the smaller reduction seen for most of the borehole would imply a clay coating thickness of only about 1mm. It is not clear why the clay thickness should have increased at 7-8m penetration, particularly since as already stated, the residual failure surface is such a stable feature. Perhaps the partial reconsolidation allowed at this depth influenced the failure mechanism.

Owing to the high loads picked up by LC3, it was progressively overstressed from about 5m onwards. The resulting zero changes have been estimated by laboratory simulation of the test, but even so the readings from this load cell are probably only accurate to $\pm 1\text{kN}$ when the data are discontinued. The frequent spikes in load cell data between 6-7m arise from a slipping fan belt on the hydraulic power pack leading to momentary losses in pressure.

The extension of the soil coating down onto LC2 may have transferred some load onto the upper friction sleeve, which would help explain the low USF/σ'_r ratios for the lower sleeve. Similarly, loads for the upper sleeve would be increased. End bearing on the clay coating, and the assumption of an 80mm diameter in the calculation of USF will also increase calculated values, although if the coating were only 1mm thick at 4m penetration, the latter two factors would have a combined effect on USF of only 8kPa. It is clear therefore that the USF/σ'_r ratio is much higher for the upper friction sleeve than in other boreholes, even allowing for these factors. Whilst this may correctly reflect a higher

friction angle for the soil/soil friction compared to the soil/shaft failure suspected for other tests, it must be strongly emphasised that the stresses are not now being measured on the failure surface. Not only are the pressure transducers not located in the rough sections of the IMP, but also the failure surface is no longer on the IMP shaft.

Tests in Borehole L have clearly demonstrated that it is possible during installation for displacement to be concentrated some distance from the IMP surface. All evidence so far considered has indicated that this is not the case with the conventional smooth walled IMP. However there is some evidence to the contrary. Again scaling from the unloading curves of the cone pressuremeter, it is calculated that the effect of a 0.1mm increase or decrease in diameter is to change radial stress by around 40kPa. This sensitivity is less severe than if the IMP radial stresses had been closer to the limit pressure, where the unloading curve is steeper, as can be seen from Fig 4.28. However, it is difficult to understand how total radial stresses on opposing transducers are so close to each other in the later boreholes, considering that the transducers are only mounted to within 0.1-0.2mm of being flush with the IMP shaft. Transducer readings would also be influenced by the shape of the IMP between the transducer and the IMP tip. Whilst progressively greater care was taken to remove any high spots on the shaft as the programme continued, the segmented nature of the IMP, together with the grub screw type of connection meant that the surface tolerance is of similar size. Whilst infilling with clay might smooth out local recesses, it is still slightly surprising that readings so close together can be achieved. If failure occurred within the soil, imperfections in the IMP surface would be masked, and close readings would be expected. A further implication of the similarity of transducer readings generally seen on opposite sides of the IMP is that installation must have been perfectly axial with no consistent tilt or wobbling.

Oil pressure data from Borehole L, analysed in a similar manner to that for Borehole E produces similar estimates of unit shaft friction on the extension tube, as shown on Fig 4.18. For this calculation, the USF on the smooth part of the IMP above LC3 was based on the σ'_r measured at this location together with an estimate of friction angle. The results are surprising considering the additional radial stress relief that must follow the clay coating. Figure 4.29 illustrates that in this case, abrasion of the extension tube had been sufficient to remove the rust but not the painted hoops from the tube, again perhaps contradictory to the calculated USF values.

4.3.6 Open-Ended Penetration

Figure 4.30 shows values of the apparent area ratio calculated from measurements of the IMP plunger movement plotted against penetration for the open-ended borehole sections. In this case, the apparent area ratio represents the proportion of soil beneath the IMP tip being radially displaced and not entering the sample chamber. In Boreholes G & L, frequent measurements have been made, and the scatter probably reflects the crude measuring technique. In other boreholes, such as F & H, infrequent measurements mean that data points represent average readings over a greater length of borehole. The load tests which followed reconsolidation in Boreholes F & G all had apparent area ratios over 80% and can be considered essentially closed-ended.

Figure 4.31 shows the predictions of Carter et al. (1979b) of the effect of area ratio on Boston Blue clay with an OCR of 2. Data scaled from their graph are replotted on a scale which extends to unity. They suggested that similar effects would be seen for other OCRs.

The depths at which open-ended penetration commenced in each borehole are shown on the raw data profiles (Figs 4.6-10 & 4.26) and on the analysed data profiles (Figs 4.13-17 & 4.27). Also marked are estimates

of the depth at which the apparent area ratio exceeded 80%, this being arbitrarily taken as representing the return to closed-ended penetration.

As described in Chapter 3, in order to operate the mechanism allowing open-ended penetration, the IMP was lifted by 30mm. The effect of this was to relieve the stresses on the soil immediately below the IMP tip, so that upon repenetration a dip in the total radial stresses is registered by all transducers as they pass this point. In Borehole G, problems of operating the system meant that three attempts were made, and so three spikes of low pressure appear on the total radial stress transducer profiles, each showing successively greater stress relief. Similar features are seen at the tip penetrations during reconsolidation in Boreholes F & G, because of the effects of tensile load tests. A further consequence of the uplift of the IMP is that the first reading of apparent area ratio is artificially high, since the IMP initially penetrates through 30mm of soil it has already penetrated closed-ended. A minimum of area ratio is therefore seen at around 100mm, followed by a steep increase, reaching values of 80% at around 400-500mm. The design of the IMP was therefore not particularly successful from the point of view of examining open-ended behaviour, since the apparent area ratios are high and variable, and the instrument plugs long before the sample chamber is full.

Taking an average area ratio of 50% as being representative of data from 50-250mm open-ended penetration, the predicted effect on the final radial effective stress after reconsolidation is negligible. Assuming that the effect on installation pore pressures will be the same proportion of c_{u0} as for the predictions of Carter et al., a decrease in pore pressure and hence total radial stress of 27kPa is predicted for Borehole C, the shallowest open-ended penetration, and 57kPa for the deepest, Borehole H. The radial effective stress, and hence shaft friction during installation should be unaffected as the soil adjacent to

the IMP is predicted to be at critical state whether penetration is closed or open-ended.

Looking first at continuous penetration boreholes, L provides the best quality data. Examining Figs 4.26 & 4.27, no significant effect is registered on shaft friction, effective radial stress or indeed total radial stresses and pore pressures in the open-ended section for the lower set of transducers. The small dip seen in the total radial stresses (red) on the lower sleeve, towards the end of the open-ended section does not correspond to the isolated low measurements of area ratio shown on Fig 4.30. Small effects on the total and pore pressures, as predicted above would be obscured by the scatter of the data. Similarly, no detectable effect is seen in Borehole H (Figs 4.10 & 4.17). The dip in total radial stress part way through the open-ended section is more likely to result from variation within the soil. The low effective radial stress and shaft friction in the open-ended section of Borehole C result from the extraction and reinsertion of the IMP.

In Boreholes F & G, reconsolidation was allowed after open-ended penetration. The lower set of pressure transducers stopped mid-way through the open-ended section of Borehole F for reconsolidation. The tip pore pressure transducer (black) and the total and pore pressure transducers in the lower sleeve (red & brown) generally show large reductions in readings upon entering the zone of softened clay at the start of the open-ended section. Total radial stresses measured by the lower transducers (red) immediately before reconsolidation appear to be around 40kPa lower than might have been expected from a continuation of the closed-ended penetration, but this is again well within the scatter of readings.

The upper set of pressure transducers passed through the open-ended section of Borehole F after reconsolidation and load tests, and it is interesting to note that total radial stresses (green) drop as the

transducers pass into the open-ended section and remain low throughout its length. Perhaps the effective radial stress after consolidation is more sensitive to area ratio than predicted.

In Borehole G the lower set of pressure transducers passed through the open-ended section before reconsolidation, the upper set after. No clear effects on the stress measurements due to open-ended penetration are discernable.

4.4 RECONSOLIDATION

4.4.1 Observations From Penetration Data.

The effects of reconsolidation can be seen on the raw transducer data profiles for penetration in Boreholes F & G (Figs 4.8 & 4.9). At the location of each transducer during reconsolidation, there is a gap in the raw data profile representing the section of borehole taken up by load tests. When transducer profiles restart after this gap they are penetrating through soil that was adjacent to the IMP during reconsolidation and load tests, until they pass the tip penetration during reconsolidation. Readings in this zone of soil for Borehole G show total radial stresses close together as in the rest of the borehole, indicating a similar large decrease in stress during reconsolidation for both sets of transducers. For Borehole F, however, the continuous penetration parts of the borehole showed total pressures measured at the IMP head are significantly lower than those nearer the tip. After reconsolidation and load tests, however they show much closer readings until the transducers pass into fresh soil. This convergence of total radial stress readings during reconsolidation might be expected, since the radial effective stress during installation is usually very similar for the two sets of transducers. Differences between the two sets of transducers in their readings of total stress and pore pressures are

clearly features of continuous penetration only and would not be expected to be observed after reconsolidation.

4.4.2 Reconsolidation Data

Figures 4.32 & 4.33 show the raw transducer readings and interpreted data from the full reconsolidation stages in Boreholes F & G at 4.17m and 5.79m tip penetrations respectively. Data collected during the partial reconsolidation in Borehole L are shown in Fig 4.34. As for all data plots, pressure transducer readings include a component due to ambient pore pressure.

The first thing to be observed from the data is its good consistency and lack of problems such as zero drift, despite the sixteen days spent underground in the case of Borehole G. In Borehole F, one of the lower sleeve pore pressure transducers (brown) shows pressures about 25kPa too high by comparison with the other measurements. In the same borehole, the more reliable Druck PDCR-81 type of transducer, mounted near the IMP head (blue), showed a sudden zero change during the first night after the start of reconsolidation at $\log_{10} t = 4.5$. Since the change was consistent with the zero difference upon extraction of the IMP from the borehole, it has been accounted for in the data reduction. It is virtually certain that it arose in the course of repairs made to the signal conditioning unit at this time.

As demonstrated in Fig 4.12, the total radial stress and pore pressures at the start of reconsolidation are much lower than cavity expansion predictions. By comparison, radial effective stresses are only slightly lower than predictions. Soon after halting driving, large positive pore pressures build up along the length of the IMP, and there follows a period of complex pore pressure stabilisation, culminating after about 5-6 hours with a uniform distribution around the instrument, both circumferentially and axially. The maximum pore pressures registered

during reconsolidation are still well below cavity expansion predictions for installation as shown on Fig 4.33.

The source of these pore pressures changes is uncertain. However, they certainly do not result from high pore pressures around the tip spreading back along the shaft since in Borehole G the tip pore pressure transducer (black) is the last to see the increase. It is possible that the increase in pore pressure could be associated with the relaxation of friction on the failure surface. However, the release of the drill string at the surface, after 5mins reconsolidation in Borehole G reduces USF by 40kPa with no noticeable immediate influence on stress readings. Some effects are seen for the corresponding release in Borehole F but they are in inconsistent directions. It is not known why these did occur, since axial load sensitivity of transducers could not account for such changes.

It has also been suggested that the increase in pore pressure results from poor saturation. It has already been shown that this is not the case, but if it were, there is no reason why a transducer should register low pore pressures during perhaps an hour of driving and then suddenly register higher readings within a few minutes of halting. One factor which might influence the rapidity of response to this increase is transducer compliance. However, there are many cases when the more compliant strain-gauged type of transducers register more rapid responses than the Druck PDCR-81's.

It is possible that the initial increase in pore pressure results from pore pressure maxima during penetration which are radially remote from the IMP shaft. This is supported by the results of the rough walled IMP tests in Borehole L which showed that it is possible for the failure surface to be away from the shaft. Pore pressure dissipation in this case would take place initially both inwards and outwards. Whatever the mechanism which causes the initial increase in pore pressures, it is

encouraging that Jardine (1985) has made similar observations in London clay.

Meanwhile, total radial stresses show a slight decline, leading to a very large reduction in the calculated radial effective stress during this initial phase. A minimum of radial effective stress is reached after around an hour, for which a minimum of pile capacity might also be expected. This behaviour is completely contrary to the predicted stress changes during reconsolidation put forward by Carter et al. (1979), who predicted a monotonic decrease in pore pressure and increase in radial effective stress.

When pore pressures do start to decay, the rate of decrease of total radial stresses increases. Consequently the recovery of radial effective stress is insufficient even to take it back to installation values. Scaling from the predictions of Wroth et al. shown in Fig 2.20, and assuming a value of 0.1 for the slope of the normal consolidation line, based on the data of Clegg and Fugro, a final radial effective stress in the range 875-1098kPa would be expected in Borehole G. This range reflects the different transducer depths and the two assumptions on the c_{u0} profile. IMP measurements are only 35-45% of these predictions, the decrease in radial total stress being around double that predicted.

It is encouraging in Borehole G how closely pore pressure measurements come to estimated ambient values from piezometer readings. In Borehole F, pore pressures were still around 30kPa above static values when the load tests were conducted. Whilst pore pressures are clearly asymptotic to ambient values, total radial stresses show a continued decline through secondary consolidation which results in a downward curve at the end of the graph of radial effective stress. It is interesting to note that radial effective stresses converge to identical readings for each transducer set.

Scaling from the curves for the dissipation of pore pressure predicted elastically by Wroth et al. the curves shown in Fig 4.33 have been obtained. This represents the dissipation behaviour that might have been expected if the initial pore pressure distribution had conformed to expectations from cavity expansion. The shaded band for this curve and also other predictions and estimates on this diagram represents the various transducer locations on the IMP together with the two assumptions of c_{uo} . A Poisson's ratio of 0.2 has been used with a permeability of 2.7×10^{-10} m/s. The latter value is based on laboratory values of around 0.6×10^{-10} m/s calculated from Fugro's oedometer data multiplied by a factor of 4.5 estimated from an assessment of the difference between field consolidation coefficients measured with the screw plate by Kay & Parry (1982) and laboratory values.

The purpose of this plot is to demonstrate that although low pore pressures are registered on the IMP shaft during penetration, large and extensive pressures do exist in the soil surrounding the IMP. Indeed, they are so extensive that they take an order of magnitude longer to dissipate than anticipated. For the times to full reconsolidation to be correctly predicted by the elastic theory, field permeabilities lower than those in the laboratory would be implied, which is very unlikely. However, the theoretical predictions could not estimate dissipation times accurately since it appears that they are based on an incorrect radial distribution of initial pore pressures.

There are numerous reasons why measured dissipation times would in fact be expected to be quicker than those for an ideal instrument, the most important of which is that it is not infinitely long and at an infinite depth, and vertical drainage paths therefore exist. Additionally, drainage may be influenced by paths through the soil inside the chamber, along the extension tube or through the threaded connection between the IMP and extension tube. Since these appear to have little

effect, soil must be in firm contact with the extension tube and be tightly packed in the sample chamber, despite the 2mm cutting shoe. Lubricating grease in the connection threads probably provided a pressure seal.

Approaching the reconsolidation depth, the IMP penetrated open-ended. Even though apparent area ratios were high, an examination of the predictions of Carter et al. does indicate that pore pressures may be less extensive and so dissipate quicker. They predicted a decrease in the dissipation time for an area ratio of 0.2 by nearly a factor of ten as compared to that for a closed ended pile.

As suspected from the penetration data, the total radial stresses measured on the two halves of the instrument converge during reconsolidation. Total pressures measured by the upper set of transducers (green) show less decay if they were significantly lower than those nearer the tip (red) during penetration. This is particularly evident in Borehole L where total pressures measured at the head of the IMP (green) in the wake of the soil coating actually increase. This increase is underestimated by the IMP measurements since the back pressure was following the decrease seen by the lower total radial stress transducers. Back pressure tests at this site indicate that in this extreme case where total radial stress transducer readings are moving in opposite directions, the error might be as high as 50kPa. There is also a pronounced trend for the initial decrease in radial effective stress to become more severe at greater depths, even if normalised with respect to the radial effective stress during installation.

The influence of compliance on the total radial stress measurements is illustrated by the dip of around 20kPa seen by each of the transducers at around $\log_{10} t = 5.3$ in Borehole G. This arose from a failure in the back pressure system which resulted in a back pressure decrease of about 130kPa for around 12 hours, until the system could be repaired.

Measurements of shaft friction during reconsolidation frequently show negative values as the shaft provides reaction to end bearing forces. As these relax, shaft friction becomes more positive. It is not known why after reading very similar values, unit shaft frictions on the two halves of the IMP tend to diverge, although the segmented nature of the IMP means that it has sufficient axial flexibility to allow this to occur. The reduction in USF seen towards the end of reconsolidation in Borehole G may well result from the swelling of the surface soil under prolonged heavy rainfall at this stage. The drill string was fixed at the surface during reconsolidation.

The partial reconsolidation in Borehole L was used to see if a decrease in shaft friction was observed if the IMP were redriven when the radial effective stress reached its minimum. As Fig 4.27 shows, a decrease of about 34% was measured by the lower friction sleeve, not as much as might have been expected from the 45% decrease of radial effective stress seen on Fig 4.34. The radial effective stress curve shown on this figure interferes with the plot above, but the advantage of presenting all data at the same scale was felt to outweigh this inconvenience. The reason for this discrepancy is clarified in Fig 4.35, which shows pore pressure measurements at the start of redriving. Whilst no significant total radial stress changes were measured until the IMP started to penetrate, pore pressures show a sudden drop even before any displacement was registered at the surface. The two pore pressure transducers in the lower friction sleeve show very different readings, but if an average is taken, the calculated effect on radial effective stress is such that at failure it is only 30% lower than when driving was halted, which is closer to the observed drop in USF. This is evidence that shearing on the failure surface causes a local decrease of pore pressure. Similar rapid decreases in pore pressure had been seen when the

IMP started to penetrate after shorter breaks during installation in most boreholes.

During installation the radial distribution of pore pressure may therefore consist of extensive high pore pressures resulting from the radial straining of the soil, upon which is superimposed a localised decrease in pore pressure generated by shearing on the residual failure surface adjacent to the IMP. The pore pressure registered on the IMP shaft increases at the start of reconsolidation as the more extensive higher pore pressures further from the shaft initially dissipate both towards and away from the IMP.

Figure 4.35 highlights a problem with the IMP instrumentation, which is that since pore pressure transducers are saturated with water they are not capable of registering pore pressures lower than -100kPa for prolonged times and transducers tend to level off at this value, presumably as cavitation occurs, although there is rarely evidence that transducer response in the remainder of the borehole is affected. Restricted drainage in the soil will allow lower pore pressures to develop, and the limitations of the transducers will give rise to an error in the calculated stress. However there are few occasions when it appears that significantly lower pore pressures would have been registered if the instrumentation had allowed.

4.5 LOAD TESTS

4.5.1 Monotonic Tests

Load tests were conducted in Boreholes F & G, but since they showed similar results, only those from G will be discussed. Figures 4.36 & 4.37 show the USF calculated and the stresses measured during the first load test after full reconsolidation. The test was intended to be undrained, failure being achieved in about 90s. It should again be emphasised that

all raw data presented for pressure transducers include a component due to ambient pore pressure.

Residual USF acting on the IMP at the end of reconsolidation was much higher on the lower friction sleeve than the upper. This difference was maintained throughout a substantial portion of the loading curve, the two values of shaft friction only converging near failure. Upon unloading, the lower sleeve again registered the higher residual friction.

Up until 87s when substantial movement was first registered at the surface, no significant radial stress changes were registered by the IMP, with the exception of the tip pore pressure transducer (black) which showed strongly negative values. This might indicate a tensile failure around the tip and was not seen for tensile tests. The radial effective stress acting on the IMP as it started to fail was practically identical to that at the end of reconsolidation, about 382kPa. The USF registered by the two sleeves at this stage was around 56kPa, giving a ratio of 0.147 and a mobilised friction angle on the IMP shaft of 8.4° , values which fall well within the scatter of data for installation.

The assumption that stresses measured by these discrete transducers are representative of stresses on the whole shaft is not unreasonable, since at the end of reconsolidation stresses were very similar for the two pressure transducer sets. However, the top 50mm of the upper friction sleeve was in ground softened by the uplift of the IMP before open-ended penetration. This small part of the shaft may therefore have been subjected to lower stresses and it is surprising that this was not reflected by slightly lower USF calculated for the upper sleeve.

As penetration proceeded, the USF continued to increase, until a definite shoulder was seen at 8mm displacement (91s). This increase can be accounted for by the stress changes that occurred on the shaft as the IMP started to move. Sharp decreases in pore pressure were registered, but total radial stress changes were in no consistent direction.

Behaviour was therefore similar to that seen when the IMP started to repenetrate after partial reconsolidation in Borehole L, although in that case the pore pressure decrease was registered before movement was detected at the surface. This discrepancy may only indicate the relative inaccuracy of the displacement measuring system. If these stress changes are assumed to be representative of those taking place over the length of the IMP shaft, the calculated radial effective stress for both sleeves is around 417kPa, which with the USF at this stage of 64kPa gives a ratio of 0.152 and a mobilised friction angle of 8.6° which is similar to that when the IMP just started to move.

Unit shaft friction continued to increase to about 66kPa when penetration was halted. At the time when installation was stopped to allow reconsolidation, USFs of 64 & 69kPa were being registered by the two friction sleeves. It is therefore very clear that in this stiff, overconsolidated clay there is no increase in pile capacity, or set-up, associated with reconsolidation. This arises from the fact that radial effective stresses did not show the predicted monotonic rise during reconsolidation. In fact, if loading should be carried out before full reconsolidation, a decrease in capacity would be observed.

When normalised by c_{u0} , the USF measured during these tests yields an α factor of 0.43, slightly below the value of 0.5 commonly used in such clays.

Towards the end of the load test, large stress changes were registered by the upper pair of total radial stress transducers (green). The penetration data (Fig 4.9) shows that this is attributable to the transducers passing into the soil softened by IMP uplift before open-ended penetration. Subsequent load tests showed similar large changes, in either direction, as the transducers moved into and out of the three distinct softened zones. Data from this set of transducers are therefore

not representative of stress changes on the IMP shaft during the later load tests.

Figure 4.38 shows data from the first three load tests for comparative purposes, together with data collected in the intervening periods of reconsolidation. The second compressive load test showed a continued slight increase in USF with displacement. Test No.3 showed that tensile USF is not significantly different from that measured under compression.

One of the pore pressure transducers in the lower friction sleeve (brown) consistently showed a large increase in pore pressure after an initial decrease, reaching up to 150kPa and seen for both compressive and tensile tests. Upon release of the load, this decreased back to former values within 20mins. Other transducers sometimes measured similar effects, but only during reconsolidation between tests. The negative tip pore pressures (black) dissipated over a period of around an hour after relaxation of the load, and if left longer, increased beyond the ambient pore pressure.

These load tests clearly show that the failure of the IMP is along the residual shear surface established during installation, with an increase of effective radial stress during loading giving rise to increased USF as the IMP starts to repenetrate. The drop in pore pressures as failure proceeds would not be expected if loading were drained and so drained capacities would be lower. This is in contrast to previous understanding of pile loading for which consolidation leads to increased capacity as the soil follows a drained stress path to a critical state type of failure.

No tendency for USF to show strain softening has been observed. However, since the penetration rate of the IMP as it fails during the load test is of similar magnitude to that during installation, Martins (1983) shows that no such behaviour would be expected because of the

failure on the residual failure surface. It is probable that the IMP is in any case too flexible axially to ensure that any peak in USF was picked up.

Two further compressive load tests not presented showed similar findings.

4.5.2 Cyclic Load Tests

Figure 4.39 shows the results of a series of cyclic load tests. Initial oil pressure controlled cycles showed small displacements on each cycle but no continuous failure. It is possible that this too reflects the axial flexibility of the IMP. During this stage, peak USFs are frequently seen greater than those during continued failure.

At 300s the oil pressure cycle was increased, and owing to over compensation for end bearing loads, continued failure on each compressive cycle was observed. A further increase in the oil pressures at 600s gave continued failure in both tension and compression. Throughout the cyclic load tests, the lower friction sleeve (blue) showed lower USF than the upper (green) in compression and sometimes higher in tension. However, mean values of around 67 kPa in both directions for the latter stage are very similar to those during the monotonic tests.

At 780s large displacement cycling was started. Readings from the lower sleeve under compression on the graph are obscured by those from the upper. They are in fact consistently about 18kPa lower. Again taking mean values, tensile USF immediately increased by 52% and compressive by 37% upon starting large displacement cycling. As displacement cycling continued, USF decreased, tending towards similar values to those measured during the monotonic tests and pressure controlled cycling. A further series of displacement controlled cycles, not shown, which were carried out 19mins later showed only about a 5kpa initial increase in USF followed by continued decrease.

It is probable that the initial increase in USF on displacement cycling is attributable to the increase in penetration rate from around 110mm/min during the last cycles of the pressure controlled tests to typically 830mm/min during the displacement controlled tests. Initial increases in friction angle have been observed by Lupini (1981) in ring shear tests. Prolonged shearing at the higher velocity soon reduces the observed USF to similar values to those for slower rates of shearing, again showing similarity with Lupini's work. Similar findings have already been noted from the extraction data which showed similar friction angles to those during penetration, despite the difference in velocities. Reversal in shear direction does not appear to influence the friction angle, since the trend during cyclic loading is towards similar values to those during slower, one directional movement.

At the start of the cyclic test series, pore pressures were slightly above ambient. As testing continued, they showed no clear trends. As for the monotonic tests the Druck PDRC-81 mounted in the shaft (brown) gave generally high readings. This transducer appears to be affected by changes in the axial load in the IMP, giving a positive pressure when the load suddenly changed from tension to compression, and a negative spike for the opposite change. This arises from the fact that the transducer filter was mounted directly in the IMP shaft and was not isolated from axial load as the other transducers were. A sudden change in the axial load momentarily trapped some water until it could dissipate through the soil, so registering a spike in pressure. A more compliant type of transducer would not suffer so severely from this problem, but will not register sudden pore pressure changes correctly, as will be shown in Chapter 5. Both types of transducer therefore have their disadvantages.

Total radial stresses showed little clear behaviour. There was a slight increase registered over the duration of the cyclic tests by the

lower (red) pair, but the upper (green) pair showed great variation as they move through the previously mentioned softened zone of soil.

CHAPTER 5

FIELD TESTS AT CANONS PARK

5.1 INTRODUCTION

The Building Research Establishment's test bed site at Canons Park, North-West London, has been extensively used for pile testing, notably by Price & Wardle (1982) and Jardine (1985). IMP Borehole I was undertaken at this site on 27th February 1986.

5.2 SOIL CHARACTERISTICS

The location of Borehole I relative to other work at this site is given on Fig 5.1. The soil profile and Atterberg limits data interpreted by Jardine are illustrated on Fig 5.2. Below 2.5m of gravel and head is the London clay. The uppermost clay is disturbed, and at 4.2m the Brown London clay shows a strata change to firmer material. This shows up clearly on the G and c_{u0} profiles given in Fig 5.3. The data for these profiles have been taken from Jardine's triaxial tests and self-boring pressuremeter boreholes carried out for Imperial College by P.M.In-Situ. Jardine's tests on reconstructed samples showed the clay to be insensitive.

Estimates of in-situ stresses are shown on Fig 5.4. Horizontal effective stresses are estimated from pressuremeter data, and vertical effective stresses from Jardine's measurements of bulk density. Jardine's piezometer data showed some downward drainage. Since standing water levels measured at the time of the IMP test coincided with his data, Jardine's in-situ pore pressure profile has been used.

Overconsolidation ratios have been plotted on Fig 5.5. OCRs have been estimated from in-situ stresses using Mayne & Kulhawy's method (Equation 4.1), assuming a ϕ' of 23.5° as suggested by Jardine. Once again, this method seems to greatly overpredict OCR, as illustrated by Jardine's interpreted OCRs from oedometer data.

A comparison with the characteristics of the Gault clay at Madingley shows similar OCRs, ϕ' , in-situ stresses and G/c_{uo} ratios. The c_{uo} profile has a stepped shape, immediately on the upper side of which c_{uo} is around 45% lower than at similar depths in the Gault clay, and on the lower side about 9% less. It might be expected that installation shear and radial stresses for the London clay would be similarly lower than those at Madingley.

5.3 IMP DATA: BOREHOLE I

For ease of comparison with the Madingley data, Figs 5.6-5.7 showing that obtained at Canons Park are presented at the same scale. Figure 3.8 gives details of the transducer locations on the IMP for Borehole I. Raw pressure transducer data are the actual measurements made by the IMP and therefore include a component due to the ambient pore pressure.

Penetration was closed-ended to a depth of 4.34m. The apparent area ratio for the subsequent open-ended penetration was less than 10%. However, after only 100mm the IMP suddenly plugged, probably as the tip encountered firmer clay at the strata change. With a large scatter of data at this point in the borehole, no discernable effects of open-ended penetration are evident, except the dip in the total radial stress measurements at the location of IMP uplift.

Partial reconsolidation was allowed at 4.95m, the data from which are reproduced in Fig 5.8. Only a small additional penetration was gained upon redriving before the ground anchors pulled out, perhaps as the IMP approached the Mudstone layer which caused Jardine's pile to be overloaded.

Total radial stress measurements show up the 4.3m strata change particularly well. They are considerably higher than was expected, readings on the lower sleeve being around 15% lower than Madingley immediately before the strata change, and about 18% higher immediately

after. Total radial stresses measured by the upper set of transducers (green) are only slightly lower at a given depth than those on the lower (red), as had been seen in the previous two boreholes at Madingley (G & H).

Pore pressures are generally lower than measured at Madingley, and smooth responses are again seen further back on the shaft during penetration. In contrast to the Gault clay, higher pore pressure measurements are made on the upper part of the IMP (blue) than the lower (brown). As Fig 5.7 shows, the resulting radial effective stresses are therefore quite different for the two sets of transducers in the London clay. Effective stresses were calculated using the same method as for the Madingley data.

Unit shaft frictions for continuous penetration shown in Fig 5.7 are up to double Gault clay values, and LC3 was consequently overstressed beyond about 4m. As for Borehole L, laboratory simulation of the loads encountered has enabled zero changes to be estimated, and it is unlikely that the error in the data presented is more than 0.5kN at the highest loads shown.

Unit shaft frictions are very similar on the two friction sleeves, and the difference in radial effective stress therefore gives rise to a higher mobilised friction angle on the upper sleeve. The ratio USF/σ'_r is initially considerably higher in the London clay than the Gault, at around 0.2 at the start of the borehole. The decrease with depth might result from higher friction angles being mobilised at the start of the borehole due to downdrag of overlying sand and gravel, which was confirmed by the deep scratching of the IMP shaft found on retrieval.

5.4 COMPARISONS WITH OTHER DATA

5.4.1 Radial Total Stresses and Pore Pressures

Piezocene data are not available for this site, but data are presented by Lunne et al. for London clay at Brent Cross. Here the DSML cone shows a smooth pore pressure response at around -50kPa for similar penetrations, resembling the measurements made by transducers on the lower sleeve.

Only self-boring pressuremeter data are available for this site. Once again, terminal pressures at around 15% strain are slightly in excess of total radial stresses measured by the IMP as shown on Fig 5.9. Since the inflation curves show no limit to have been reached at the end of the tests, it is clear that the stresses acting on the IMP are not cavity limit pressures.

As discussed in Chapter 2, good quality model pile work has been undertaken at this site by Jardine (1985). Figure 5.10 shows his radial total stress and pore pressure measurements during installation, replotted in a similar format to the IMP data. Pore pressures below 4m are similar to IMP measurements, but the peaks at about 3.5m are in excess of similar peaks seen by the IMP at 3m. Total radial stresses are around 25% lower than those measured by the IMP. As has been previously discussed, the degree to which measurements made on the IMP shaft show stress relaxation relative to cavity expansion predictions, may be affected by the tolerances of fabrication/assembly of the shaft. It is interesting to note therefore, that Jardine's model pile is rather less accurately constructed than the IMP, with total pressure transducers up to 1mm out of flush. If, as Jardine (1985) and Kitching (1983) contend, principal displacements occur on a failure surface within the soil, the effect of such fabrication inaccuracies would be masked. However a ring shear test between a roughened glass interface and London clay carried out by Lemos (1985), showed a failure surface only 20-50 μ m from the

interface. Under such conditions, the condition of the shaft could be of crucial importance.

Jardine's consolidation data have already been presented on Fig 2.22. His data have been replotted on a $\log_{10}$ scale and times have also been reduced by the square of the ratio of the radius of his pile to that of the IMP. This correction should allow direct comparison with Fig 5.8. IMP measurements of pore pressure show similar behaviour to that observed by Jardine and also that at Madingley with initially fluctuating readings. However, when the reconsolidation phase was terminated, the pore pressures still showed little inclination towards uniformity along the IMP as they had done at Madingley. Dissipation is perhaps complicated by the fact that the IMP is astride the strata change, and by the fissured nature of the clay. Jardine's data showed no smooth dissipation curves, and pore pressures which did not return to expected ambient values.

As in the Gault clay, the initial trend during reconsolidation is for effective radial stresses to fall. Total radial stress transducers (green) located in the disturbed London clay show a smaller decrease than observed by Jardine, with a correspondingly smaller drop in effective radial stress up to the end of this partial reconsolidation phase. A more dramatic loss in effective radial stress is calculated from the data from transducers in the underlying Brown London clay.

5.4.2 Unit Shaft Friction and Friction Angles

Figure 5.7 also shows a comparison between unit shaft frictions measured by the IMP during installation with those measured by Jardine's pile. Price & Wardle (1982) have also jacked a pile into this site. Data are shown from the lowermost section of their 168mm diameter pile during installation. Shaft friction measurements further up their pile show considerably lower readings.

The IMP data fall directly between these two comparisons. Jardine explained the difference in measurements with reference to installation speeds. During each push of the jacks, he had used penetrations speeds of about 500mm/min, compared to only 20mm/min for Price & Wardle's pile. IMP installation speeds ranged from about 220-540mm/min during each stroke, conveniently between the two other tests.

As a result of his low measurements of effective radial stress and high shaft frictions, Jardine calculates ratios of USF/σ'_r between 0.23-0.4, representing mobilised friction angles in the range $12-22^\circ$, which are rather higher than the values of about $9-13^\circ$ measured by the IMP.

As previously mentioned, Lemos (1985) has carried out a ring shear interface test between London clay from this site and sand blasted glass. His results are also shown on Fig 5.7. Unfortunately, the clay he used was the Blue London clay found beneath the Brown London clay in which all pile tests have been conducted. Upon increasing the speed of shearing, Lemos saw an immediate increase in the apparent friction angle calculated from total stresses as shown in Fig 5.11, and it is with this that Jardine compares his friction angles and to which he attributes the differences in shaft friction on the various piles. However, since no pore pressure measurements were made at the interface during this test, there effect during this fast stage is uncertain.

Following the peak at the start of fast shearing, Lemos' work shows that there is a decline to a lower value. His tests on other soils show that if fast shearing is temporarily halted, like for instance the breaks between strokes of the jacks during IMP installation, there may not be a large peak on reshearing. The comparison should therefore be made between the pile/soil friction angle and the fast residual value beyond the initial peak. The latter is almost identical to the slow residual in Lemos' test, and this is shown on Fig 5.7 together with the slow residual soil/soil angle determined by Lupini for Blue London clay from another

site. The IMP friction angles compare well, although this may be fortuitous, since Lemos has shown that in addition to shearing speed, other possible influences on the residual friction angle include the smoothness and hardness of the interface. However, it is interesting to note that IMP friction angles in the Madingley Gault clay also compared well with drained residual values.

Since the lower friction sleeve starts at only 108mm from the IMP tip, Lemos' work might suggest that higher friction angles than on the upper sleeve might be expected, since the decay to residual may be slower for the faster shearing. The reverse has been found.

Observations from the Madingley data show some similarity with Lemos' findings. Upon increasing velocity of shearing, as in for instance the cyclic load tests, there appeared to be an immediate increase in the mobilised friction angle. Prolonged faster shearing, such as during extraction from the borehole, did not appear to influence the friction angle, and significant peaks were not generally seen when shearing was restarted after a break.

Following his test Lemos found a failure surface at around 20-50 μ m from the interface. As was seen with the rough walled test at Madingley, he found that rougher interfaces could produce clear failures within the soil. His findings for smooth interfaces, all in soils other than London clay are less clear cut, as is the evidence regarding the nature of failure around the smooth IMP. Lemos discovered that smooth surfaces can lead to a reduction in friction angle, but that even with smooth glass interfaces no polished shear surface adjacent to the interface had been discovered, although changes in interface roughness indicated that abrasion of the surface had occurred, as has been seen with the IMP. Failure is therefore more complex than shearing on a polished surface either at the interface or within the soil.

5.5 COMPARISON WITH CAVITY EXPANSION PREDICTIONS

Cavity expansion predictions of installation radial stresses and pore pressures have been made in the same way as for Madingley, again adding initial in-situ stresses to cavity expansion predictions to allow direct comparison with IMP measurements shown on Fig 5.9. The G and c_{uo} profiles used are shown on Fig 5.3 and the in-situ stresses on Fig 5.4. Since the triaxial and pressuremeter data are again inseparable, two sets of predictions again assume that the profile chosen represents plane strain or triaxial compression. Sagaseta's solution shows that the combined influence of shaft friction and anisotropic initial stresses is to reduce predicted total radial stresses by 10-20%. This is a much more noticeable difference than at Madingley, and arises from the fact that installation α values in the London clay are as high as 1.5.

Total radial stress measurements show a good correlation with the chosen shear strength profile in that they are constant up to the strata change. In contrast to the Madingley data, they are also in reasonable agreement with the cavity expansion predictions, measurements on the lower sleeve (red) being only about 10% low in some parts of the borehole. However, as has already been discussed, pressuremeter data indicate much higher limit pressures than those measured on the IMP.

Measured pore pressures are again very much lower than predictions from the method of Wroth et al., with average values in the broad vicinity of zero.

Radial effective stresses were predicted as before (Equation 4.6) assuming a M_{ps} of 0.771 based on Jardine's data. Because of the high radial total stress measurements and low pore pressures, the calculated radial effective stresses are up to double those predicted. This is in contrast to Jardine's data where the radial effective stresses were lower than cavity expansion predictions. The only obvious factor which could explain both this and the likely underestimate of the cavity limit

pressure is if in-situ shear strengths were being drastically underestimated. The close agreement of shear strength data from two sources indicates that this is not likely.

5.6 BACK PRESSURE TESTS

5.6.1 Introduction

Figure 5.12 shows data from the back pressure tests in Borehole I, Canons Park. These were carried out at a tip penetration of 4.94m, towards the end of the partial reconsolidation stage, when it was hoped that stresses around the IMP would be fairly stable for the test duration.

Numerous back pressure tests in the Gault clay showed similar results, but lack of space prevents their inclusion. For this particular test the back pressure was suddenly increased from 200 to 303kPa at 960s after the arbitrary zero time. The reverse back pressure change was made at 1660s. As for all other IMP data presented, the transducer responses take account of the back pressure change, and so an ideal transducer would show a constant reading, apart from a small spike as the change is applied. This peak arises from the fact that the back pressure is measured at the surface. During the 15s it takes to stabilise, the back pressure measured will therefore not correctly reflect that inside the IMP.

5.6.2 Total Radial Stress Transducers

The total pressure transducer response which was selected as being typical was located at 4.72m during the test ie. within the stiffer Brown London clay. Upon increasing the back pressure the transducer is displaced into the soil by about 1.6×10^{-3} mm. This movement leads to an increase of the total radial stress acting upon it. Similarly, when the back pressure is reduced, the transducer face moves away from the soil

and so the pressure it registers is reduced. In both cases, there is a continued relaxation of readings after the back pressure is stable. This is accounted for by the dissipation of localised pore pressures generated by the test.

Similar results are obtained if the back pressure is first reduced and then increased. This test is therefore further evidence that the total radial stresses acting on the IMP after installation are at a state of unloading from the limit pressure. However since the influence of localised effects such as arching at the edge of the transducer are unknown, this evidence is only speculative.

The measured change in the pressure reading can be used to estimate the shear modulus of the soil by means of a solution for the settlement of a rigid circular footing on a semi-infinite elastic half-space by Timoshenko & Goodier (1951).

$$\delta = P(1 - \nu^2) / 2aE$$

where

δ = transducer face deflection

P = total load on transducer face

a = transducer face radius

E = drained Young's modulus = $2(1 + \nu)G$

ν = Poisson's ratio

The deflection is assumed to be drained and the error is therefore measured after the dissipation of pore pressures. A drained Poisson's ratio of 0.2 has been used. The resulting estimate of G is 100MPa. This is very approximate, since without resorting to complex numerical analysis or sophisticated laboratory measurement, the transducer stiffness can only be estimated. The cavity strain for this test is about 0.004%, and so much higher shear moduli than from the pressuremeter would be expected. Jardine has measured shear moduli of about 41-52MPa in triaxial tests on soil from this depth. The difference could be accounted for by the fact that his axial strain levels were around 0.01%, by the

inaccuracy of the calculated value from the IMP data, or by the unknown effect of localised arching.

5.6.3 Pore Pressure Transducers

Pore pressure transducers should not be affected by back pressure changes in the same way as total radial stress transducers, since the sensitive face of the transducer is not in contact with the soil. Consequently the Druck PDCR-81 transducer shows a perfect response to the back pressure test, with no noticeable change in reading once the back pressure is stable.

In contrast, the strain-gauged type of pore pressure transducer is very much more flexible, and as the back pressure is increased, the movement of the diaphragm requires about 0.2mm^3 of water to pass through the filter. Because of the relative impermeability of the clay, pore pressures are locally increased, and on the typical response curve shown in Fig 5.12, the pressure registered has increased by about 25kPa when the back pressure has stabilised. The pore pressures measured by this transducer increase during the testing period, but it is clear that after each back pressure change the pore pressures registered become asymptotic to the reading it would have registered had the test not been conducted. This feature arises from the dissipation of the localised pore pressures generated by the transducer compliance. It is around 10mins before the error is below 5%. However, it must be emphasised that applying sudden, large back pressure changes with the transducer stationary in the soil is a particularly severe test. During IMP penetration, for instance, the transducer continually moves into fresh soil and so has a constantly changing reservoir of pore water with which to interact. Sudden spikes or troughs will be underregistered by this type of transducer, but their compliance cannot account for the completely smooth profiles frequently observed. Mean pore pressures during penetration will not be affected.

CHAPTER 6

FIELD TESTS AT GREAT YARMOUTH

6.1 INTRODUCTION

The Great Yarmouth site was chosen as one of the few sites in the country where a direct comparison with self-boring pressuremeter tests in normally consolidated clay is possible. An extensive site investigation was carried out for the now completed A12, Great Yarmouth Western Bypass. This was supervised by C.H.Dobbie & Partners (1986) for the Department of Environment and Transport, Eastern Road Construction Unit. IMP Borehole J was carried out at this site on 23rd April 1986 on land owned by Great Yarmouth Borough Council, adjacent to the southern approach embankment to the Yare bridge. Figure 6.1 shows its location, and that of other site investigation data.

6.2 SOIL CHARACTERISTICS

The soils at this site are essentially normally consolidated estuarine deposits, with a lightly overconsolidated crust. Figure 6.2 shows sample descriptions and Atterberg limit data from nearby boreholes undertaken by Foundation Engineering Ltd.. This illustrates the variable nature of the soil profile, but the sample descriptions do not agree well with adjacent in-situ test data which do not pick up the sand at shallow depths. When hand augering the starter borehole to 2m, no substantial sand was seen, the soil generally being the soft, very silty clay or silt as it is described on the logs. There is therefore a considerable variation of soil profiles across the site, and this is reflected in the scatter of c_{u0} values shown in Fig 6.3. In the immediate vicinity of the IMP borehole, c_{u0} data are available from self-boring pressuremeter tests conducted by PM In-Situ (1986) and from field vane tests by Foundation Engineering Ltd..

Wroth (1984) showed that undrained shear strengths are dependent on the mode of failure. A ϕ'_{tc} of 32° has been selected from a study of Atterberg limits data, with reference to the correlation between plasticity index and ϕ' by Kenney (1959). Foundation Engineering also conducted a few drained triaxial compression tests. For this value of ϕ'_{tc} , the field vane is expected to give results about 40% lower than those from the pressuremeter in a normally consolidated clay. The field vane data shown on Fig 6.3 have therefore been adjusted by this factor so that they may be used alongside the pressuremeter values in establishing the chosen shear strength profile. After applying this factor, reasonable agreement was obtained between the two sets of data.

Also shown on Fig 6.3 are some unconsolidated undrained triaxial compression data. These have not been included in the calculation of the mean values used, since they are from boreholes further from the IMP site, and are for samples taken from beneath the pre-existing embankment.

The c_{uo} data and the pressuremeter shear modulus data also shown on Fig 6.3 highlight strata changes at around 4m and 7.5m. The soil between these two depths is particularly variable, and shows large scatter of strengths and moduli. Mean c_{uo} values have been used in the upper two strata, but in the lowermost, the clear increase of strength with depth has been fitted by linear regression. Shear moduli have similarly been fitted by mean values or regression lines.

Remoulded vane shear strengths show this to be a sensitive clay. The profile of sensitivity interpreted from these data is shown in Fig 6.4.

Estimated in-situ stresses are shown in Fig 6.5. The flat, low lying land was completely waterlogged, and in-situ pore pressures are assumed hydrostatic from a water table 0.1m below ground level. This is based on a measurement of the standing water level made 16 hours after the IMP was extracted from the borehole. Since the borehole was still open to 5m, the assumption of hydrostatic pressures cannot be far wrong.

Vertical effective stresses are based on bulk densities from Foundation Engineering Ltd. boreholes. The horizontal effective stress line was fitted by linear regression through the pressuremeter data, which show considerable scatter at these low stress levels. The assumption is made that the in-situ pore pressures were the same at the time of the pressuremeter tests as that for the IMP borehole. This is likely considering the poor drainage of the site, and the fact that both tests were conducted at a similar time of year.

Estimates of OCR shown on Fig 6.5, have been made from the in-situ stresses using Mayne & Kulhawy's method (Equation 4.1). Apart from a slightly overconsolidated crust, the soil is essentially normally consolidated, although this approximate method gives OCRs tending to a value of 1.6 with depth.

6.3 INFLUENCE OF NEARBY ROAD EMBANKMENT

Borehole J was located 36m from the toe of the new 6.5m high road embankment. Crude elastic analyses showed that the increase in effective horizontal stress normal to the embankment may be of the order of 10kPa at the base of the borehole. Clearly this will give rise to an error when comparing IMP predictions with site investigation data obtained before construction. However this will be largely offset by the fact that the pressuremeter and vane tests were conducted about half the distance from a pre-existing 3m embankment, which was subsequently widened and heightened.

Unfortunately it was not possible to check the IMP orientation at this site to look for the expected slight anisotropy of stresses normal and parallel to the embankment.

6.4 IMP DATA

6.4.1 Installation

The transducer raw data profiles are shown in Fig 6.6, with the interpreted USF and σ'_r profiles in Fig 6.7. Reference should be made to Fig 3.8 which gives the transducer locations on the IMP for this borehole and the colours used to represent them on the raw data profiles. The same calculation technique was used as for the Madingley site, with radial effective stresses represented by a running average, taken over a length of borehole equal to the length of the relevant friction sleeve.

It is important to emphasise the expanded scales used on both data plots. Stress levels are very much lower than at the stiff clay sites, so that only the USF/ σ'_r ratio data can be presented at the same scale. This magnification accounts for some of the increased activity seen on the pore pressure plots. One of the pore pressure transducers had a blocked filter and its data have been omitted. The IMP was closed-ended throughout the borehole. The raw pressure transducer data again include a component due to ambient pore pressure, this being a much larger proportion of the measured stresses than in the stiff clays discussed in the previous two chapters.

Once again the use of the IMP as a profiling tool is well demonstrated, strata changes being picked out with remarkable clarity. It is interesting to note that transducer pairs register much closer readings than at the stiff clay sites, perhaps indicating less sensitivity to shaft surface tolerances in a soil with a much lower shear modulus. As at the stiff clay sites the total radial stress registered by the upper transducer set is less than that on the lower, by around 20-50kPa in this case.

The start of the borehole is less well defined than in the stiff clays. Successive transducers register the start progressively earlier as soil is squeezed up into the pre-augered hole.

Down to about 3.5m, the soil is extremely soft, with USFs in the range 2-3kPa, giving installation α values as low as 0.1. Similar observations were made qualitatively from the ground anchors, which after being screwed into place could be turned by hand. Effective radial stresses are correspondingly low. Unfortunately negative effective radial stresses are calculated for the lower transducers at the start of the borehole, an anomaly which arises from the different points of measurement for total stresses and pore pressures, and which is not helped by the loss of one of the pore pressure channels.

At around 3.5-4m, pore pressures measured at the head of the instrument (blue) fall to negative values whilst all other pressure readings remain unchanged. The resultant increase in σ'_r on the upper friction sleeve is strongly reflected by an increase in USF, with USFs on the lower sleeve remaining unchanged. It is thought that this behaviour might result from sandier soils where pore pressures might dissipate in between two sets of transducers reaching a given point, or perhaps dilation might give rise to negative readings.

The strata change at 4.5m is marked by sharp increases in total radial stress and smaller increases in pore pressures. Whereas in the upper stratum pore pressures nearer the tip (black & brown) are frequently greater than total radial stresses further back along the shaft (blue), in this stratum pore and total pressures are more clearly separated. Pore pressures near the IMP head are again low, perhaps indicating that this stiffer material is also sandier. This was confirmed by the observation of sandier soil adhering to the drill string upon extraction. Higher USFs again correspond nicely with the higher radial effective stresses.

The middle stratum is punctuated at 5.8m and 6.4m by thin layers of probably siltier material similar to that at the start of the borehole. These features may give rise to some of the variation in c_{u0} in this

stratum. Total radial stresses and pore pressures show sharp dips at these points as do radial effective stresses. Pore pressures near the IMP head (blue) converge to similar values to those nearer the tip. Load cell data also show dips, but these are out of phase since the transducer penetration used for all load cells is that of the base of LC1. Unit shaft friction plots show these layers more clearly since these are correctly plotted at the location of a particular soil horizon.

The lowermost of the three strata again appears to be less sandy, with little difference in the pore pressures measured by the two main transducer sets. However total radial stresses measured by the lower set (red) diverge from those measured by the upper, increasing the calculated effective radial stress on the lower sleeve. However, this does not correlate with shaft friction measurements which are very low on the lower sleeve. Figure 6.6 even shows that in places load cell No.2 registers lower readings than No.1. Changes in the load cell zeros upon extraction were small and could not explain this anomaly. This therefore demonstrates how the very low stresses at this site are beyond the limit of accurate resolution by the instrumentation. However, considering that at this site effective radial stresses and USFs are calculated by subtracting small numbers from each other, the agreement is generally remarkably good, and ratios of USF/σ'_r frequently show very good consistency between the two sets of transducers.

Mobilised friction angles on the IMP shaft again indicate residual conditions, and despite the very different stress levels, they are broadly similar values to those in the stiffer clays. Readings of around 12° are calculated in the silty material at 3m, decreasing to a minimum of around $5-6^\circ$ before increasing again, probably as the soil becomes sandier. At the centre of the middle stratum, friction angles reach a maximum of 17° then dropping to 14° before the 7m strata change. These higher angles would again suggest sandier material. The siltier horizon

at 6.8-6.9m shows up very clearly, having very low friction angles as does the lowermost stratum, below 7m. However, readings are too scattered and the data of too poor quality in these layers to accurately estimate friction angles.

In Chapter 2 it was shown how Levadoux & Baligh predicted pore pressures becoming asymptotic to a constant value along the shaft of a penetrating instrument. IMP data appear in broad agreement with this in the silty-clay strata, particularly below 7m where measurements for a given depth at the head (blue) are only slightly lower than those made by the transducer on the lower friction sleeve (brown), but tip readings (black) are significantly higher. However in the sandier strata, very low pore pressures are measured at the IMP head (blue), and those measured at the tip (black) are frequently lower than those by the transducer in the lower sleeve (brown). There is clearly not a monotonic decrease of pore pressure along the shaft and no trend towards a constant value.

Levadoux & Baligh predicted that in normally consolidated clay the radial effective stress would be practically constant along a cone shaft, some distance from the shoulder. Measurements made by the IMP show that similar σ'_r values are rarely registered by the two sets of transducers, and that the difference in σ'_r values calculated for the two transducer sets at a given depth may depend on soil type.

6.4.2 Reconsolidation

A partial reconsolidation phase of about 1.75 hours was allowed with the IMP in the lowermost stratum, at a tip penetration of 8.29m. Sudden decreases can be seen in the pressure transducer data profiles on Fig 6.6 at the reconsolidation depth of each transducer, followed by slow recovery as the IMP was redriven. The reconsolidation data are shown on Fig 6.8. The fact that very rapid dissipation is not seen confirms that this is not one of the sandier strata.

The data show significant similarity to that obtained in San Francisco Bay Mud by Kirby & Roussel (1980) which were discussed in Chapter 2. As at the stiff clay site, the pore pressures do not immediately start to dissipate, but increases of up to 30kPa are seen, larger increases occurring further from the tip. These increases are much less dramatic than those at the stiffer clay sites since pore pressures start much closer to the total radial stresses. Pore pressures converge to uniformity along the instrument after about an hour, and then start to dissipate, exactly as in the Gault clay. Meanwhile total radial stresses fall gradually, steepening when pore pressures start to dissipate. Once again total radial stresses appear to converge. Those measured at the head of the IMP (green) start at lower values but show less inclination to drop. Radial effective stresses therefore fall sharply from values which were already low at the start of reconsolidation. Small negative values are calculated for the upper set of transducers, again an anomaly which arises out of measuring total and pore pressures at different locations. Just as in the overconsolidated clays, there appears to be a recovery in radial effective stress as pore pressures dissipate.

6.5 IMP CONDITION ON EXTRACTION

Figure 6.9 shows a photograph of the IMP after retrieval and is typical of its state after boreholes in both soft and stiff clays. Beneath a thin smearing of watery clay the IMP shaft is highly polished, indicating abrasion of the shaft. Four 2mm wide permanent marker pen lines drawn longitudinally along the IMP have been completely removed.

6.6 COMPARISON WITH OTHER DATA

Terminal pressures at about 11% strain from self-boring pressuremeter tests are compared with IMP data on Fig 6.10. Apart from two readings in the middle stratum showing smaller readings which might have arisen from

soil variability, terminal pressures are greater than total radial stresses measured on the IMP lower sleeve. As for the stiffer clays, total radial stresses measured by the IMP must be well below cavity expansion limit pressures since none of the pressuremeter inflation curves show any evidence of a limit being reached.

6.7 COMPARISON WITH CAVITY EXPANSION PREDICTIONS

Cavity expansion predictions of installation radial stresses and pore pressures have again been calculated by the same method as used for Madingley again adding initial in-situ stresses to cavity expansion predictions to allow direct comparison with the IMP raw data from installation. The input G and c_{u0} profiles are shown on Fig 6.3, the latter being assumed to represent plane strain. A ϕ'_{tc} of 32° was used to give a M_{ps} of 1.018. Radial effective stress predictions made by means of Equation 4.6 are insensitive to the choice of ϕ' for which there is a little uncertainty, changing by only 2% for a 1° change in ϕ' . The total radial stresses which result from Sagaseta's program are again very similar to those obtained directly from the approximate method proposed by Wroth et al. as shown by Fig 6.10.

Sagaseta's solution assumes an elastic, perfectly plastic soil, but vane tests on this site show significant sensitivity. Houlsby (1984) has derived another closed form Gibson & Anderson type of solution to undrained cavity expansion which does not allow for the initial stress state in the soil or friction on the pile/soil interface, but can take into account the shear stress-strain behaviour of the soil. This has been used to derive the lower limit to the total radial stress shown on Fig 6.10. This limit is based on a stress/strain curve which is linear up to yield, with a slope equal to the pressuremeter measured shear modulus. The soil is then assumed to immediately strain soften, reaching vane test remoulded strengths at only twice the yield strain. This is therefore a

very pessimistic prediction. The selected sensitivity profile is that shown on Fig 6.4. No effective stresses or pore pressures have been predicted for this case.

In making comparisons with the data obtained during IMP installation, the large scatter in soil strength data should be kept in mind. However it is clear from Fig 6.11 that radial effective stress predictions are very much higher than IMP measurements in the uppermost and lowermost strata. In the middle stratum, reasonable agreement is obtained between predictions and measurements by the upper transducer set, but σ'_r values for the lower set of transducers are only about half of these. Randolph et al. (1979) have shown that low effective radial stresses would result from soil sensitivity, and perhaps higher values are recorded by the upper set of transducers because of pore pressure dissipation.

Pore pressure predictions are higher than measurements made by the IMP, throughout the borehole, and total radial stresses are sometimes as low as half the elastic, perfectly plastic predictions.

The total radial stresses for the rapidly strain softening soil model should provide a lower limit and the elastic, perfectly plastic an upper limit to IMP measurements. In the middle stratum this is the case, except for the low readings in the softer layer at 5.8m. However, throughout much of the rest of the borehole, total radial stress measurements are lower even than the expected lower limit.

Analyses of pressuremeter inflation curves by the method of Palmer (1972) have shown that the elastic, perfectly plastic model is reasonable for this soil and that there is no tendency to strain soften up to strains of 11%. Terminal pressures from the tests also indicate that the true limit pressure must be well above the pessimistic, strain softening prediction. It is therefore likely that the elastic, perfectly plastic solution gives the closer estimate of cavity expansion limit pressures. This implies that cavity expansion overestimates the radial total and

effective stresses on the IMP particularly in the upper and lower strata. If peak cavity expansion stresses were reached around the IMP tip, those measured on the shaft show that as in the overconsolidated clay, there has been considerable stress relief.

At a site with so much variability of soil conditions, the above conclusions must be tentative. To obtain good agreement between predictions and measurements would, for example require shear strengths of about 10 kPa in the upper stratum. The scatter shown on Fig 6.2 shows that this is not completely impossible.

6.8 BACK PRESSURE TESTS

Figure 6.12 shows typical transducer responses for back pressure tests conducted after the partial reconsolidation at 8.25m. As at Canons Park, the back pressure was first increased, but this time by only 42kPa. After 15mins the back pressure was reduced by 56kPa.

As in the London clay, all total radial stress transducers register an increase as they are pushed outwards into the soil, perhaps further evidence that the radial total stresses measured on the IMP shaft are not limit pressures. As expected, in this softer soil, this effect is much less severe, but the mean shear modulus calculated from the two transducers shown is still 25MPa, nearly a factor of ten greater than the large strain pressuremeter measurements. This result appears high, but there are no other small strain data for comparison.

As in the London clay tests, the Druck PDCR-81 pore pressure transducers showed perfect responses. However the strain gauged pore pressure transducers again showed slow responses due to their compliance. The typical response illustrated takes 1.5mins for the error to reduce below 5% of the pressure change, compared to 10mins in London clay. The faster stabilisation arises from the greater permeability of the clay. The fact that the pore pressure transducers do return to the same reading

after a back pressure change is down-hole confirmation of their calibration factors.

Back pressure tests at all four sites have shown pore pressure compliance errors to be a function of the transducers flexibility and the permeability of the soil. Filter permeability has no influence since it is so much greater than that of the clay.

CHAPTER 7

FIELD TESTS AT HUNTSPELL

7.1 INTRODUCTION

The Huntspill site is located on the Somerset Levels about two miles south of Burnham-On-Sea. The land, which is adjacent to the River Parret, is owned by Somerset County Council and was extensively investigated for use as a waste disposal site, under the direction of Aspinwall & Co. who produced two summary reports including all the work done at this site referred to in this chapter (1978 & 1981a). The soils are broadly similar to those at Great Yarmouth, being geologically recent estuarine sediments. Better drainage for agricultural purposes has led to a more noticeably overconsolidated crust.

This site was initially used for the preliminary IMP field tests (Boreholes A & B) on 11th-13th December 1984. A second visit was made with the rebuilt IMP on 2nd-5th May 1986, when Borehole K was conducted. The borehole locations are shown on the site plan, Fig 7.1.

7.2 SOIL CHARACTERISTICS

Figure 7.2 shows sample descriptions and Atterberg limit data from Somerset County Highways Laboratory's closest borehole to the IMP work. Whilst soil types vary quite considerably across the site, locally, around the IMP soundings, there is fairly uniform soft silty clay. The site variation is highlighted by the scatter of c_{u0} data shown from the whole site on Fig 7.3. By comparison, the c_{u0} profile derived from the cone resistance values of a Fugro cone penetration test, adjacent to the IMP boreholes shows little scatter. The unconsolidated undrained triaxial tests shown on Fig 7.3 were conducted by the Highways Lab, and the consolidated undrained by Bath University. For the latter, the consolidation pressure used was the effective overburden stress. Shear strengths have therefore been corrected by the ratio of the in-situ mean

effective stress to that during consolidation. This will be correct at 2m depth where the ratio is unity, and provide a slight underestimate of c_{uo} towards the end of the plot where the soil is normally consolidated. For the mildly overconsolidated clay in between, c_{uo} will be overestimated.

Shear strengths were derived from cone resistance using an N_k factor of 15 in the following equation:-

$$q_c = N_k c_{uo} + \sigma_{vo} \quad (7.1)$$

q_c = cone end resistance

Fugro recommended values between 10-17. A correlation of cone resistances with triaxial or field vane shear strengths in adjacent boreholes, shown in Fig 7.4, yields a value of 16 from linear regression, but 15 was selected, giving greater weight to the triaxial data. The resultant c_{uo} profile is illustrated on Fig 7.3, and it is this that has been used in data analyses, assuming that it is representative of triaxial compression. Additional uncertainty arises in this profile because of scaling off very small Fugro CPT plots where cone resistance is represented by just a few millimetres.

Figure 7.5 shows estimates of the in-situ stresses. Pore pressures are based on standing water levels in the boreholes, but take into account some downward drainage which Aspinwall (1981b) showed to exist below about 4m all year round. Vertical effective stresses are based on unit weights presented by Somerset County Council and Bath University. The assumed values drop from 20kN/m^3 at the surface to 17kN/m^3 at 4m. The water table has been assumed to be at 0.5m for the calculation of initial vertical stresses. Also shown in Fig 7.5 are estimates of OCRs from oedometer data given in these two references. These agree well with the trend reported by Cook & Roy (1984) for Somerset alluvium, and so the latter has been used to derive estimates of the in-situ horizontal effective stress by means of Mayne & Kulhawy's method (Equation 4.1). A

value of ϕ'_{tc} of 27.5° was calculated from plasticity index data using the relationship proposed by Kenney (1959).

Soil properties, in-situ stresses and overconsolidation are therefore similar to those at Great Yarmouth.

7.3 IMP DATA

7.3.1 Installation

7.3.1.1 Preliminary Tests

Data from Borehole B are shown on Fig 7.6. The same colours represent the same transducer locations as on all other plots of raw data, purple data lines being used for transducers at other locations. Figure 3.6 gives details of colours used for transducers in this borehole and their locations on the IMP. It is again emphasised that all pressure transducer data presented includes a component arising from ambient pore pressures. At a tip penetration of 4.3m, the IMP was withdrawn, resulting in a loss of friction upon reinsertion. The particularly interesting feature of this data is the overnight reconsolidation allowed at a tip penetration of 5.1m. The large reductions in both pore pressure and total radial stress are clear, and upon redriving, a 130% increase in shaft friction was observed by the load cells. It was thought that this confirmed that significant set-up occurred in normally consolidated clays.

Data from Borehole A (not presented) showed total radial stress measurements up to 60 kPa greater than for Borehole B, and it was this that prompted the complete re-instrumentation of the IMP.

7.3.1.2 Borehole K

Penetration data are shown on Fig 7.7 with calculated USFs and σ'_r profiles on Fig 7.8. Reference should be made to Fig 3.8 which shows the locations of individual transducers on the IMP for this borehole. Unfortunately load cell 1 drifted very badly during the reconsolidation

phase. Despite attempts to correct for this by means of the zero measured upon extraction, readings from this transducer (red) and hence the lower friction sleeve USFs (brown) are inconsistent and should be disregarded below the reconsolidation depth of 5.51m tip penetration. The data have been included only to demonstrate that similar profile shapes are seen to those from the upper sleeve, only with a zero offset.

The data strongly resemble that from Great Yarmouth, and are presented on the same scale. Below about 4m the clay seems very similar to the firmer silty-clay below about 7.5m at Great Yarmouth. Tip pore pressures (black) are generally about 50-70kPa higher than those measured by other pore pressure transducers. Pore pressure variation further back on the IMP shaft is much smaller with generally slightly higher measurements at that IMP head (blue) than in the lower friction sleeve (brown). As below 7.5m at Great Yarmouth, the tip pore pressure (black) sometimes exceeds total radial stresses on the lower friction sleeve (red). Total radial stresses are found to decrease by around 40-50kPa between the lower and upper sets of transducers reaching a given depth at both sites. Since this drop in total pressure is accompanied by an opposite change in pore pressure, there is a dramatic drop in effective radial stress at a given depth between that calculated for the lower transducer set and that for the upper. This too was observed below 7.5m in Borehole J.

In the top half of the Huntspill borehole, before reconsolidation, the two friction sleeves registered virtually identical USFs, and so calculated values of USF/σ'_r for the upper sleeve are double those for the lower. This is an unusual result, although the ring shear tests of Lemos (1985) did show that for some interface/soil combinations, residual friction angles do increase with displacement along the failure surface. Sleeve friction measurements from an adjacent Fugro CPT are about 50% higher than IMP readings.

The IMP sounding starts towards the end of the overconsolidated crust where shear strengths are reducing rapidly with depth. It is thought that the higher overconsolidation gives rise to the different response in this zone. The site investigation data available do not show any other reason why different behaviour might be expected, and no soils which might account for this behaviour, such as sand were seen during the hand augering of the starter borehole. Negative readings are recorded by the tip pore pressure transducer (black) and also those in the lower sleeve (brown). The two pore pressure transducers near the IMP head (blue) show diverse and fluctuating responses. In this zone very high total radial stresses were recorded giving calculated effective radial stresses as high as 250kPa on the lower transducer set. This is reflected by similarly very high calculated USFs, although the resulting friction angle does drop slightly in the first few metres of the borehole. At about 2.5m total radial stresses drop rapidly, whilst pore pressures increase and so shaft friction falls dramatically. From about 3m onwards, the profiles seem to indicate normally consolidated soil, showing slow linear increases with depth. The shape of the raw data profiles is therefore very similar to that of c_{u0} , and similar trends were observed in Borehole B. Data from this site confirm that stresses during driving are much more strongly dependent on OCR than predicted by cavity expansion, which would agree with the observed poor correlation between measurements and cavity expansion predictions for the stiff highly overconsolidated clays.

Mobilised friction angles on the IMP shaft are far more consistent in this more uniform clay than at Great Yarmouth, but show similar values to those in the silty-clay strata at that site. Upper sleeve values decrease to around 6.3° at about 3.5m and are then fairly constant to the end of the borehole. Up to the reconsolidation depth the lower sleeve generally registered much lower values, with a minimum calculated angle of 2.9° .

In contrast to almost constant friction angles with depth, α values for installation drop dramatically with depth from about 0.88 at 2.5m to 0.20 at 8m which would not be expected from cavity expansion theory for which the radial effective stress and hence installation USF is proportional to c_{u0} for a soil with a given value of M .

Figures 7.9 & 7.10 show an attempt to compare the data with the normalised distribution of excess pore pressure around a cone predicted by Levadoux & Baligh for normally consolidated clay. For the tip and lower friction sleeve transducers, pore pressures are normalised as follows:-

$$\text{PRESSURE RATIO} = (\Delta u_{(\text{tip/ls})} - \Delta u_{\text{hd}}) / \Delta u_{\text{hd}} \quad (7.2)$$

where Δu = excess pore pressure above initial in-situ

ls: transducer in lower sleeve

hd: transducer near IMP head

Levadoux & Baligh predicted constant radial effective stress along the shaft, and so a similar normalisation has been performed with the total radial stress data:-

$$\text{PRESSURE RATIO} = \{(\sigma'_r)_{\text{ls}} - (\sigma'_r)_{\text{hd}}\} / \Delta u_{\text{hd}} \quad (7.3)$$

In the more normally consolidated clay below 5m, all three ratios are constant as would be expected. However, as has already been highlighted, pore pressures on the lower sleeve (brown) are generally lower than those near the IMP head (blue), and so the ratio is negative. Towards the top of the borehole the ratio is also negative for the tip pore pressure transducer (black), whilst it increases for the total radial stress transducers. This demonstrates how the stresses around such a driven instrument are highly sensitive to OCR. Data from Great Yarmouth showed how sensitive they also are to soil type.

Mean values for the ratios below 5m are superimposed on Levadoux & Baligh's predictions (Fig 7.10), where the agreement is poor. Their prediction of constant σ'_r along the shaft has been shown not to be

correct for this soil, hence the pressure ratio calculated from the total radial stress data is out of step with the other two calculated from pore pressure data.

7.3.1.3 Open-Ended Data

The IMP was driven open-ended from 4.91m to the reconsolidation depth of 5.51m. An 80% apparent area ratio was reached at 5.41m, and so all load tests and subsequent driving were essentially closed-ended.

Immediately before jacking was halted for reconsolidation, readings from both total (red) and pore pressures (brown) in the lower sleeve were about 35-40kPa lower than would have been expected from a continuation of the closed-ended data. The penetration of the transducers at this point was about 5.2m. The assumption of Carter et al. (1979b) that the radial effective stress would be unchanged for open-ended installation is therefore verified, but from their predictions the 54% reduction in excess pore pressure would only be produced if the area ratio dropped to 14%, as shown by Fig 4.31. Measurements shown on Fig 7.11 show that in this at this depth, area ratio was increasing rapidly from a minimum of about 25-30% as the IMP plugged.

Total stress transducers near the IMP head (green) pass through this soil after reconsolidation and load tests. Their readings do not show any significant difference from soil which was penetrated closed-ended. This is some evidence in support of the prediction of Carter et al. that the final radial effective stress after reconsolidation would be little affected.

The above comparisons must be tentative since it is possible that the observed features could arise from soil variation.

7.3.1.4 Condition of the IMP on Extraction

Figure 7.12 shows the condition of the IMP immediately upon retrieval. The clay adhering to it was perhaps slightly firmer than at other sites. After washing the instrument down, Figs 7.13 & 7.14 show that the abrasion had once again polished the IMP and extension tube, but had not removed the painted rings from the latter. Some remnants of the marker pen lines on the IMP were also observed. These lines take around a dozen firm wipes with a damp cloth to erase. It is surprising that they have survived, and this therefore gives support to the argument that principal displacements were not along the shaft surface. Lemos (1985) has shown that failure can take place in soil very close to an interface whilst still causing abrasion of its surface.

7.3.1.5 Comparison with Cavity Expansion Predictions

The theoretical predictions for installation radial stresses and pore pressures shown on Figs 7.15 & 7.16 were derived in the same way as for the Madingley site (Chapter 4). As for all other cavity expansion predictions, initial in-situ stresses have been added, where appropriate, to allow direct comparison with the raw data obtained from the IMP during installation. A value of M_{ps} of 0.888 was estimated from the selected ϕ'_{tc} of 27.5° . In the absence of pressuremeter data, a $G/(c_{uo})_{ps}$ of 93 has been used based on the average measured at Great Yarmouth, where the soils were similar. A factor of two on the G value used would only effect total radial stress predictions by around 8%. Once again the two cavity expansion prediction methods give very similar total radial stresses.

Below about 3m, radial effective stresses calculated for the lower friction sleeve transducers agree remarkably well with the predictions. However, that near the IMP head is only about half that predicted. In the

overconsolidated clay, near the surface, the cavity expansion prediction underestimates σ'_r .

Pore pressure predictions form an approximate upper bound to measurements with the exception of tip pore pressures (not shown on Fig 7.15) which exceed predictions below about 4.5m.

Total radial stresses from Sagaseta's program are consistently 30-40kPa greater than measurements on the lower friction sleeve. However the gradient of total stress increase with depth is correctly predicted, which implies that the correct Nk factor has been chosen to derive shear strengths, but that total stress measurements made on the shaft are subject to stress relief compared perhaps to full cavity expansion pressures at the tip. Back pressure tests again showed total radial stress increases on inflation.

The above comments must also be made with some degree of caution because of the uncertainty of soil properties and in-situ stresses, but such stress relief behind the tip in normally consolidated clays was predicted by Levadoux & Baligh. However, they did not predict the large drop in both total and effective radial stress along the remainder of the shaft. In stiff clays decreases in total stress along the shaft may arise from an imperfectly flush instrument. In the softer clays this must be a very much less important factor as shown by the fact that pairs of transducers give virtually identical readings.

Field vane test data are quoted for this site which suggest sensitivities in excess of 6. Unfortunately no raw data from remoulded vane or triaxial tests are contained in the currently available site investigation reports. As at Great Yarmouth it would appear that vane tests greatly exaggerate the importance of sensitivity, but it could certainly account for the observed differences between predicted and measured behaviour.

7.3.2 Reconsolidation

Figure 7.17 shows the data collected during the reconsolidation at 5.51m. As at Madingley, final pore pressures correlate well with estimates of ambient values, although most transducers show terminal readings about 5kPa lower than expected, the exception being the tip transducer. The drift of load cell No. 1 starts soon after reconsolidation commences and is shown by the diverging USF calculated for the lower sleeve (blue).

As at Great Yarmouth, pore pressures rise initially, although less spectacularly than in the stiffer clays. With a small drop in total radial stress, effective radial stress falls rapidly during this phase, particularly at the pile head where it approaches zero. The source of the pore pressure increase is again clearly not from higher pressures around the tip during penetration.

As in the stiffer clays, when pore pressures begin to decrease, the drop in total radial stress steepens, with the result that after full reconsolidation, they have fallen by nearly 50% at the IMP head (green). Recovery in effective radial stress is therefore limited and final radial effective stresses are below those during installation. Unlike the overconsolidated clay, there is no tendency for the radial effective stresses measured by the two sets of transducers to converge, and the final radial effective stress near the IMP head remains below the initial in-situ σ'_{ho} .

Figure 7.17 shows that the dissipation curves predicted from the elastic solution of Wroth et al. compare better than they did for overconsolidated clays. The predicted curve was calculated in the same way as that for Madingley (Chapter 4) and represents the predicted dissipation had the initial pore pressure distribution been correctly predicted by cavity expansion. The shaded band for this curve and also other predictions and estimates on this diagram covers the different

predictions for the various transducer locations during reconsolidation. Somerset County Council measured the in-situ permeability in an adjacent borehole to be 2×10^{-9} m/s. A shear modulus of 24MPa has been assumed, again using $93c_{uops}$. For the same reasons as outlined for Madingley, quicker reconsolidation than predicted might have been expected. In fact, dissipation is initially delayed, but when it does start, a steeper gradient than predicted is observed. Overall reconsolidation times are therefore similar. However, a factor of two on either the permeability or G , which is not unlikely, would shift the $\log_{10} t$ scale by 0.3.

Estimates of the final radial effective stress have been made from Figure 2.20 of Wroth et al., using a normal consolidation line gradient of 0.127 based on the available oedometer data. As for the Gault clay, the final radial effective stress is drastically overestimated, since measured values actually drop during reconsolidation at this site. This is a surprising result since both Boreholes B & K showed considerable set-up of axial capacity during reconsolidation, a feature expected for normally consolidated clays.

7.3.3 Load Tests

Load tests were conducted in the same manner as at Madingley. As previously explained, USFs on the lower sleeve (blue) are not correct. A zero offset means that compressive friction is underregistered.

Figure 7.18 shows the USFs calculated from the load cell data for the first compressive load test, and Fig 7.19 the pressure transducer responses which, as always, include a component arising from ambient pore pressures. Immediately the IMP starts to move, the maximum load of around 11kPa is reached, which remains constant up to about 30mm displacement, with no post-peak decline. This represents a 160% increase on the USF at the end of penetration. The calculated α is 0.50, well below the API RP2A recommended value of 1.0 for clays of this strength.

Pressure transducers show little change up to the onset of failure. This implies a mobilised friction angle on the IMP shaft of 14.4° , which is much higher than measurements during penetration. However, showing some similarity with the behaviour observed in the Gault clay, as the failure continues, large drops in pore pressure are initially registered. If a mean radial effective stress is calculated just before failure was halted, it is found to have increased to 73kPa from 43kPa at the start of the test. With still the same USF of 10.9kPa, the mobilised friction angle is now calculated to be 8.4° , much closer to installation values. This calculation is inaccurate since it assumes that the mean of very different transducer readings will correctly represent the stresses acting on the whole IMP shaft. Towards the end of failure, and immediately afterwards, some pore pressure transducers registered increases in pore pressure.

Lupini only documents ring shear tests on two very different alluvia, the lower of the two slow residual friction angles being 13° . Without similar tests on the clays at the IMP sites, no firm conclusions can be drawn. However it does appear that the IMP friction angles at both normally consolidated clay sites may be being influenced by its smooth surface, which would also account for the low α values for the Huntspill load tests.

Lupini, Skinner & Vaughan (1981) suggested that in materials such as this where residual behaviour could be expected to be transitional between the pure sliding and turbulent types, lower residual friction angles than for soil/soil shearing might be measured against smooth interfaces due to local sliding on platy particles adjacent to the interface.

The load test data therefore show that the increases of shaft friction on piles in normally consolidated clays result from stress changes during the load test itself, not from large radial effective

stress increases during reconsolidation. The apparent delay in the pressure transducer responses and the insensitivity of the shaft friction to large radial effective stress changes on the shaft implies that failure may be occurring in the soil away from the shaft. This would also explain why consistent friction angles are only measured during continuous penetration, when the state of stress around the instrument is more stable.

Figure 7.20 shows data from all the monotonic load tests. Unit shaft friction on the upper sleeve decreases by around 10% over the three tests, although there appears to be little difference between tensile and compressive capacities.

Total radial stresses increase during the first two tests, but this results only from the transducers moving into different soil, since a decrease is seen during the tensile test.

The tip pore pressure (black) builds up during the two compressive tests, approaching values similar to those during continuous penetration. During the tensile test, a drop in the tip pore pressure is registered as the end bearing load is released.

Allowing for the offset in the USF readings on the lower sleeve by assuming that the USF would be equal in tension and compression, values around 10% higher than for the upper sleeve are obtained. However the estimated radial effective stress on the lower sleeve at the end of reconsolidation is around 50% higher than that for the upper sleeve calculated by interpolation between the two pressure transducer sets.

The last tensile monotonic test gave a USF of about 10kPa on the upper sleeve. Subsequent compression only cycling, shown in Fig 7.21 initially gave a failure USF of only about 9kPa. This was followed by large displacement cycling. As at Madingley, an initial increase in USF was seen, but only as far as values similar to those during the first monotonic test. As cycling continues, USF decays, but pressure

transducers show no clear trends. This indicates that the reduction is probably a function of increased displacement on the failure surface. Reversal of direction does not appear to prevent this.

Towards the end of cycling, USF tends to a constant value, and was about 7.5kPa on the final cycle. There was then a break of 1.5 hours whilst back pressure tests were undertaken, and upon redriving the USF had recovered to nearly 12kPa, which would not have been expected if USF during the cyclic tests was a function of displacement. It would suggest dissipation of positive pore pressures, not clearly measured by the IMP, built up on the failure surface during cycling.

As redriving continues, shaft friction quickly decreases even before the friction sleeve enters fresh soil. In contrast, the top set of pressure transducers only see significant changes as they pass the tip location during reconsolidation. Such contradictory evidence from the IMP means that the mechanisms of failure cannot be firmly identified.

CHAPTER 8

CONCLUSIONS

8.1 INTRODUCTION

The ultimate intention is to develop the IMP as a site investigation instrument to aid in the design of piles, particularly those used offshore. However, this goal has yet to be achieved because the present work with the IMP has had to concentrate on the investigation of fundamental pile behaviour, which has been found to be quite dissimilar to that expected. The following sections highlight the behaviour of piles which has been observed with the IMP, together with recommendations for future research.

Apart from being a research instrument for the behaviour of cylindrical piles, the IMP has also shown itself to be a particularly useful general site investigation tool, and it would be possible to develop this type of instrument so that it could give continuous borehole profiles of many soil strength, deformation and consolidation parameters. The final section therefore covers the implications of the IMP work for general site investigation.

8.2 THE FUNDAMENTAL BEHAVIOUR OF PILES

The IMP work has confirmed that in clays a residual shear surface is formed during installation, as had been suggested by researchers at Imperial College. For the rough walled test, this surface was clearly a few millimetres from the shaft, but for the smooth surface used for most of the tests, its location was uncertain. Residual friction angles on this surface appear to be the same for installation, extraction and both compressive and tensile load tests. They can only be temporarily increased by a sudden increase in the rate of shearing.

In the Gault clay, no consistent difference was seen in the friction angles developed on the two independent friction sleeves, indicating that

the displacement to mobilise this angle must be smaller than the length of about 112mm from the IMP tip to the start of the friction sleeve. At other sites, both increases and decreases of friction angle are seen between that calculated for the lower sleeve passing a given depth and that for the upper. At none of the other sites are there sufficient data to ensure that these differences are consistent. In the two stiff clays, mobilised friction angles compare well with laboratory drained residual values, but those at the two alluvial sites are lower than might have been expected, perhaps indicating the influence of the smooth shaft of the IMP.

The observed failure mechanism which is not accounted for by cavity expansion means that this method could not be successful in predicting installation stresses. In addition, pressuremeter data and IMP measurements indicate that stress relief behind the tip of the instrument, and to some extent further back along the shaft, results in total radial stresses and pore pressures on the shaft during installation being lower than cavity expansion predictions. This effect is most severe in the stiffer clays, and it appears that the prediction of reasonable total radial stresses during installation in London clay by cavity expansion was due to the underprediction of the theoretical cavity limit pressure. Real piles would experience considerably greater stress relief along their shafts due to the poorer fabrication tolerances and pile whip during installation. However, it may still be possible to estimate driving resistance by cavity expansion methods, since the IMP data show total radial stresses and pore pressures to be similarly affected and that there is therefore frequently still a reasonable agreement between the predicted effective radial stress and IMP measurements. Any agreement may to some extent be fortuitous in the light of the failure mechanisms during installation being quite different to those assumed by the theory. At some sites, effective radial stresses are too high, at others,

particularly the normally consolidated ones, they are too low. In the London clay, and also the two alluvia, large variations of σ'_r at a given depth for the two pressure transducer sets during installation, contrary to the predictions of Levadoux & Baligh, give rise to a corresponding variation of agreement with the cavity expansion predictions. As for the friction angles, no consistent trend for the difference in σ'_r at a given depth between the lower and upper transducer sets of the IMP is seen for the three sites where differences do occur. However these are again the sites where there are insufficient data to be certain that the observed trends are consistent for each particular clay.

Although the agreement of measurements with cavity expansion predictions is sometimes poor, the IMP data do strongly indicate that in a given clay the installation σ'_r is a function of c_{u0} and that the total radial stress and pore pressures are governed by both c_{u0} and the initial in-situ stress. In normally consolidated clays the two components of the total radial stress are of similar orders of magnitude, and the fact that at Huntspill the rate of increase of the total radial stress and pore pressures with depth were well predicted for installation confirms that the relative importance of the two factors is correctly assessed by cavity expansion theory. It also indicates that with modifications a cavity expansion approach could be made to work.

In the shallow heavily overconsolidated clays investigated by the IMP, the initial in-situ stresses are relatively less important, and hence the IMP measurements of total radial stress and pore pressure during installation strongly resemble the c_{u0} profiles for these sites. The normalisation of installation and also post-reconsolidation stresses with respect to the initial effective vertical stress used by some researchers is therefore a misleading method of presenting data.

Large sensitivities measured by in-situ vanes for the two alluvial sites do not appear to be a major cause of discrepancy between IMP

measurements and cavity expansion predictions. If sensitivity is to be accounted for, then that measured by the pressuremeter is more appropriate, since the radial straining of a large mass of soil is similar to soil movements at the IMP tip during installation. In contrast, the vane imparts very large strains to thin sections of soil. At Great Yarmouth zero sensitivity was calculated from back-analysis of self boring pressuremeter data, compared to values of around 3-5 measured by the vane.

Reconsolidation behaviour was quite different to the theoretical predictions reviewed in Chapter 2. Instead of a monotonic rise in radial effective stress, an initial steep drop was observed in all clays, followed by a slow rise. Final radial effective stresses after reconsolidation were sometimes lower, sometimes about the same as during installation, but always very much lower than those predicted. In the two clays where full reconsolidation was allowed, the times to full reconsolidation were very much greater than elastic predictions of the dissipation of pore pressures resulting from cavity expansion. It is likely that many field test piles are load tested prematurely.

Subsequent undrained loading showed confused pore pressure data, but seemed to indicate that shearing on the residual failure surface results in a decrease in pore pressure adjacent to the pile, whilst the total radial stress is unaffected. The reduction in pore pressure seems to be associated with displacement on this surface, and so only appears at failure. Some pore pressure transducers indicate increases in pore pressure in the later stages of the load tests. These might arise in soil further from the shaft, outside the residual failure surface, which although not at failure is still loaded in an undrained simple shear mode. It is likely that pore pressure decreases at the residual failure surface are very much more localized than those in the surrounding soil. They will also only be generated once, at the initial failure,

dissipating quickly thereafter with continued shearing. As the load test proceeds, pore pressures on the shaft may then increase, which in the absence of any changes in total radial stress should lead to a post-peak reduction in USF if the tests are continued long enough, which the IMP tests were not. A further consequence is that for load tests which are nominally undrained, the pore pressure at the shaft, and hence the USF measured are likely to be highly sensitive to loading rate. This may be the case for installation as well as loading and explains the confusion in pile test data presented in the literature.

This decrease of pore pressure adjacent to the shaft for undrained loading is of a similar order of magnitude in soft normally consolidated and stiff overconsolidated clays and so is of much greater importance in the normally consolidated, relative to the effective radial stress after reconsolidation. It is this rise in radial effective stress at failure that gives rise to the observed set-up in normally consolidated clays. This behaviour is therefore quite different to the expected increase in radial effective stress during reconsolidation followed by a decrease during loading as the soil fails at its critical state.

If the decrease in pore pressure during undrained loading is independent of radial stress level, as is suggested by the similar data in the normally and overconsolidated clays, then its influence will decrease in significance with depth, particularly in normally consolidated clays. Correspondingly, there should be a decrease with depth of empirical factors relating pile capacity with c_{uo} and/or σ'_{vo} . This has been observed by many researchers and is currently explained with reference only to progressive failure of longer piles. Again the inadequacy of normalising pile shaft capacity with these parameters is clearly shown.

During installation, the negative excess pore pressures generated on the residual surface are superimposed on positive excess pressures

arising from the radial straining of the clay. When driving is halted the low pore pressures dissipate quickly as they are localised, leading to the observed initial sharp rise in pore pressures and decrease in radial effective stress. Reconsolidation then continues by the outward dissipation of positive pore pressures generated by cavity expansion. Stresses measured on the IMP at the start of reconsolidation therefore include a component resulting from undrained shearing on the residual surface. Those observed later do not, and in this sense the IMP radial stress data during reconsolidation are misleading as a guide to changing pile capacity.

Data for the stiff overconsolidated clay sites indicate that the very large observed drop in radial effective stress at the start of reconsolidation may not solely be due to the above mechanism, as indicated by the reduction in shaft friction when a load test was performed upon reaching the minimum radial effective stress. Reductions in pile capacity from that immediately after installation might not be observed for full scale piles for the following reasons:-

- 1) Full sized piles driven open-ended would have reconsolidation times similar in magnitude to the IMP. Such effects might be lost during the greater installation time.
- 2) Data from the stiffer clays indicate that the initial decrease in radial effective stress is less severe where there has been greater radial stress relief along the shaft during installation. As previously mentioned, the stress relief along the shaft may be much greater for full scale driven piles than for the IMP.

8.3 IMPLICATIONS FOR PILE DESIGN AND RECOMMENDATIONS FOR FUTURE RESEARCH

The IMP work has confirmed that axial pile capacity is governed by residual shearing behaviour of the clay, as was suggested by researchers at Imperial College. This clearly demonstrates why correlations of

capacity against c_{u0} and σ'_{v0} or any combination of the two meet with so little success. Installation radial stresses, and so perhaps those at failure have been shown to be related to c_{u0} and σ'_{h0} at a given site, but in heavily overconsolidated clays at shallow depths the influence of initial in-situ stresses is small. However, the residual friction angle is a fundamental property of the soil, and such empirical and semi-empirical methods of pile design could only succeed by taking account of it.

Of current fundamental models of pile behaviour, cavity expansion is the only one which allows a reasonable estimate of installation stresses at a specific site to be readily obtained. The IMP work, and other pile research have shown the predictions of this simplified model to be incorrect in detail. This results from the fact that it incorrectly models the installation process and is not a function of the specific soil model chosen, be it modified Cam-Clay or simply elastic-perfectly plastic. It is now time for a more sophisticated model of pile behaviour to be developed, based on observations, from which more reliable methods of pile design should evolve.

The IMP research described has clearly demonstrated the use of such an instrument to investigate the shaft frictional behaviour of an element of pile shaft. With the similar PLS cell, Azzouz (1986) recommends the use of installation shaft friction from the instrument for the design of piles in the Empire Clays he investigated. Whilst the author recognises the need for a simple pile design method until the complexities of their behaviour are resolved, the IMP data shows that if generally used, such a method would be unconservative, since at all sites the radial effective stress acting on the shaft following reconsolidation was less than or at best equal to that during installation. In addition, before extrapolating to full scale pile design, an investigation is required of how the behaviour of such piles would differ from that of research instruments

such as the IMP, particularly in that they are driven and not jacked. Unfortunately, wave equation analyses showed that the IMP could not be driven without damaging the instrumentation. Other factors which require research are the effect of pile flexibility during driving on the radial stresses and the effect of axial flexibility during loading. However, before proceeding to investigate the complexities introduced by these factors the behaviour of a simple jacked pile like the IMP should be completely understood. Future research instruments should be axially stiffer than the IMP to provide a better simulation of a short section of pile.

Future research should also investigate the effects of shaft surface roughness and the effects of shaft fabrication tolerances. The IMP data showed that the latter, particularly with regard to the flushness of instrumentation, may influence radial stresses and/or the measurement of them.

Since residual friction angles are governed by the particle size distribution and shape, data must not be extrapolated from one site to another without accounting for these factors. The heavily overconsolidated North Sea clays are generally poorly sorted boulder clays. These have residual friction angles much closer to peak values than the older sedimentary overconsolidated clays used for many onshore pile tests, such as the IMP investigations in Gault and London clays. In contrast, normally consolidated North Sea sediments are likely to be more uniformly and finer graded than the normally consolidated estuarine alluvium available for onshore test sites. In view of the low residual friction angles likely to be measured in deep normally consolidated offshore clays, present design methods may not be as conservative in such clays as had been thought from cavity expansion considerations.

Jardine's work at Canons Park were the preliminary tests of an ongoing project. It is understood that future research at Imperial

College intends to investigate the effect of pile surface condition, which in a crude way, the IMP has shown to be important. It would be of particular interest to see what effect surface roughness would have in the alluvial clays.

If the focus of future research is on piles for the North Sea, then tests should be conducted in clays showing greater similarity with those found offshore, as the Building Research Establishment have done with their work in boulder clay at Cowden, Humberside. The mechanisms of pile behaviour in such clays which have residual friction angles much closer to critical state values might not be identical to those observed, for instance, in London clay.

The IMP work has highlighted several practical problems of pile research in normally consolidated clays at shallow depths. Even in fairly uniform clays, such as at Huntspill, the inadequacies of present testing methods, both in-situ and laboratory, give rise to unacceptable scatter at the low levels of strength and in-situ stresses being measured. The IMP work also shows that a more sensitive instrument is required at such sites to separate the radial effective stress accurately, since this is calculated as the relatively small difference between two much larger measurements: pore pressure and total radial stress. To gain the best data from an instrument like the IMP, designed for use in stiffer clays, tests should be carried out in much deeper normally consolidated clays as Azzouz & Lutz (1986) have done with their similar instrument.

Future pile load tests should investigate fully drained loading as well as undrained. As many other researchers have found, the IMP data from undrained load tests are inconclusive and highlight difficulties in making localised stress measurements on the shaft which are representative of complex and rapid changes around the pile. The increase in radial effective stress during undrained loading which the IMP data appeared to indicate would suggest that drained loading would give rise

to no change in the radial effective stress at the end of reconsolidation, and so lower capacities. Any such difference in drained and undrained capacities would be most obvious in shallow normally consolidated clays for which the stress changes during undrained loading are of greater importance relative to the radial effective stress after reconsolidation. In overconsolidated clays, or in deeper normally consolidated clays, changes in radial effective stress during undrained loading may be of lesser importance. Theories which associate pile failure with the soil adjacent to the shaft at the critical state would predict larger drained loading capacities as the soil is allowed to consolidate and gain strength during loading.

Attempts to investigate the behaviour of open-ended piles with the push-in mechanism of the IMP have been relatively unsuccessful, serving only to show that small reductions in area ratio do not result in significant stress differences compared to closed-ended piles. Another method of simulating open-ended behaviour is required, capable of achieving lower area ratios for greater lengths of borehole.

8.4 COMMENTS ON THE DEVELOPMENT OF SITE INVESTIGATION INSTRUMENTS

Clearly an instrument like the IMP will be an invaluable aid to pile design once the fundamental behaviour of piles is fully understood. However the IMP does also suggest ways in which general site investigation tools could be improved.

A piezocone with the filter element mounted behind the shoulder would give similar pore pressure data at the IMP sites to that from the IMP tip pore pressure transducer, shown in black on the data plots. However it is obvious at all the sites that this is only a fraction of the information available from a penetrating instrument. In particular, the simple addition of a total radial stress transducer would give far better soil profiling information as was shown by the clarity with which these

transducers picked up strata changes in the IMP boreholes. Data would be more easily back-analysed to give soil strength and stress history parameters than by using only pore pressures as some researchers have done.

More consistent data might be obtained if the shaft of penetration instruments were provided with a slight taper, so that the soil on the shaft was returned to the limit pressure following stress relief at the shoulder. Data might then be more easily interpreted by cavity expansion methods to give soil strength, deformation and stress history information.

The results from Borehole L suggest a further way in which penetration instruments might be modified. A small reduction in the diameter of the instrument, part way up the shaft, with total radial stress transducers on both larger and smaller diameters would give a continuous profile of deformation modulus data during penetration.

Modifications to the IMP incorporating these features were designed and part constructed, although lack of time prevented their completion and testing. Total radial stress transducers were built which could be ground flush with the instrument shaft to overcome problems encountered in stress measurement due to the inaccuracy of their placement. These transducers have a similar sensitivity to the existing design but are sufficiently stiff not to require back pressure.

Down-hole back pressure tests with the IMP have highlighted the need to maintain rigidity of total stress transducer design, and have also indicated how the inflation of such devices like a stiff pressuremeter could give useful low strain shear modulus data.

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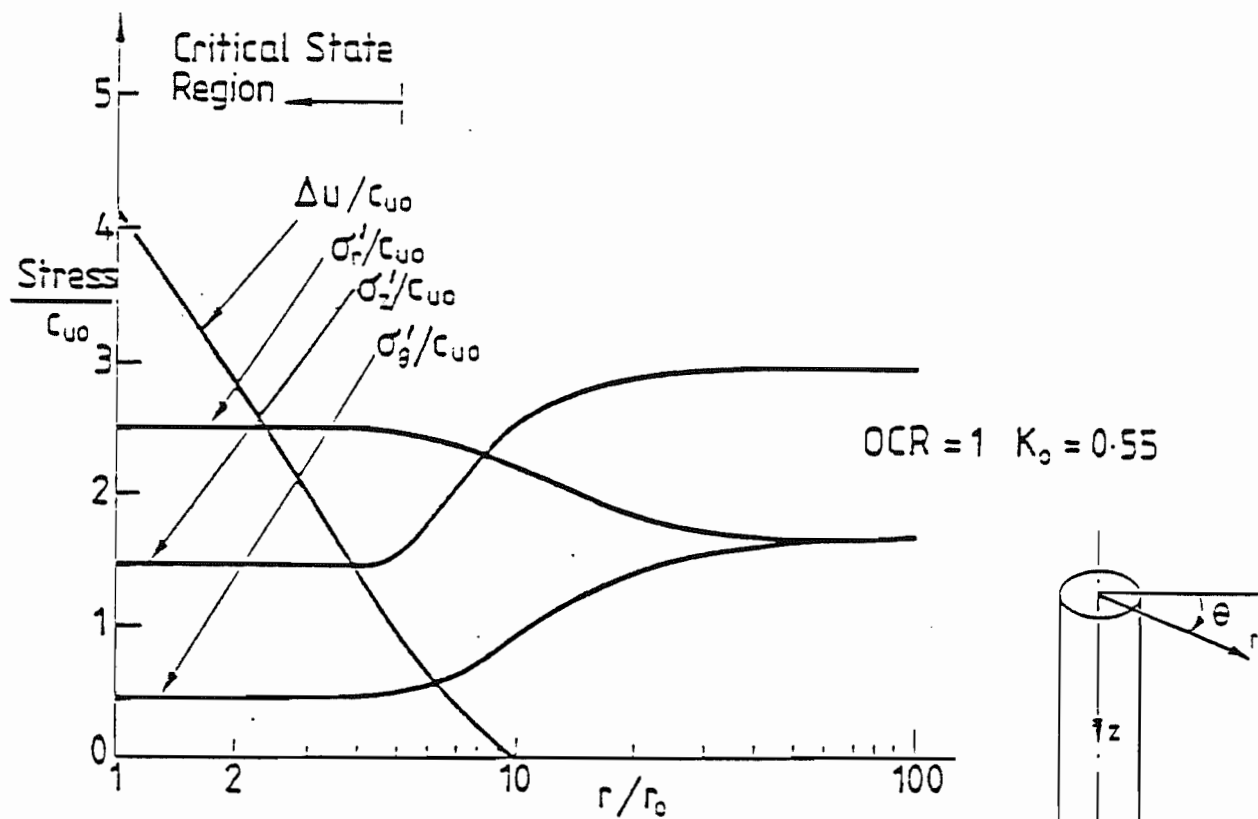


Figure 2.1 Stress Distribution Around a Pile Immediately After Installation (After Randolph & Wroth 1982)

A = initial state
 B, B' = immed. after installation
 C = after full reconsolidation

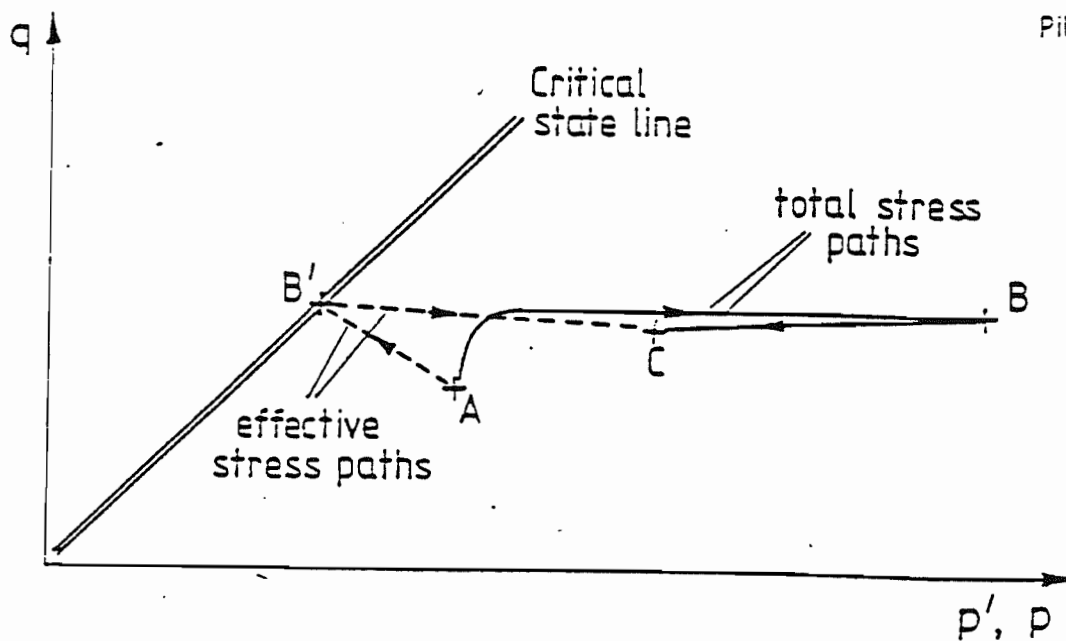


Figure 2.2 Cavity Expansion in a Normally Consolidated Clay: Stress Paths for Soil Adjacent to the Pile. (After Randolph & Wroth 1982)

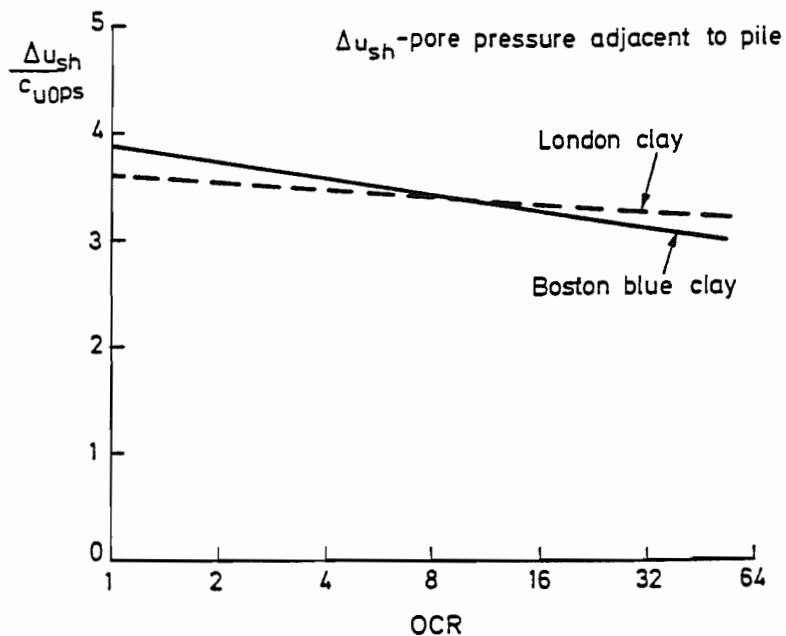


Figure 2.3 Effect of OCR on Installation Excess Pore Pressures.
(redrawn from data in Randolph et al. 1979 and
Wroth et al. 1979)

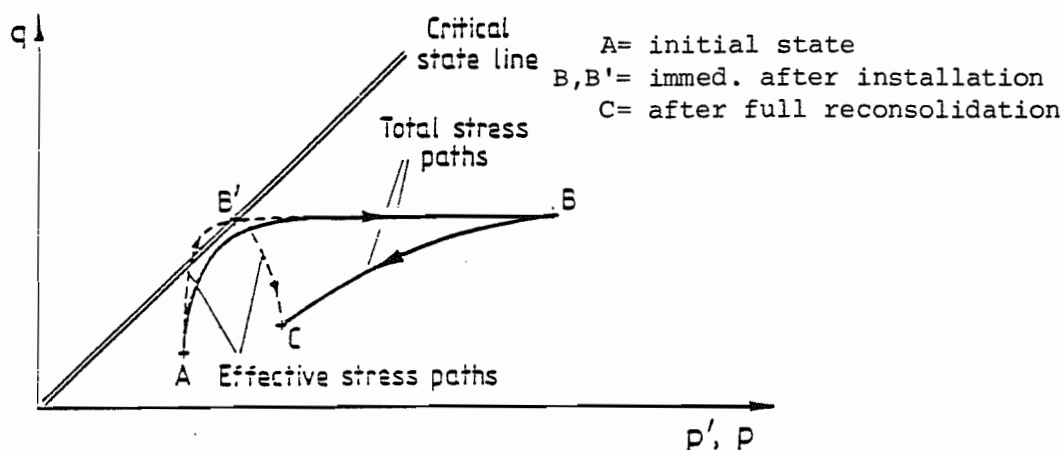


Figure 2.4 Cavity Expansion in an Overconsolidated Clay:
Stress Paths for Soil Adjacent to the Pile.
(After Randolph & Wroth 1982)

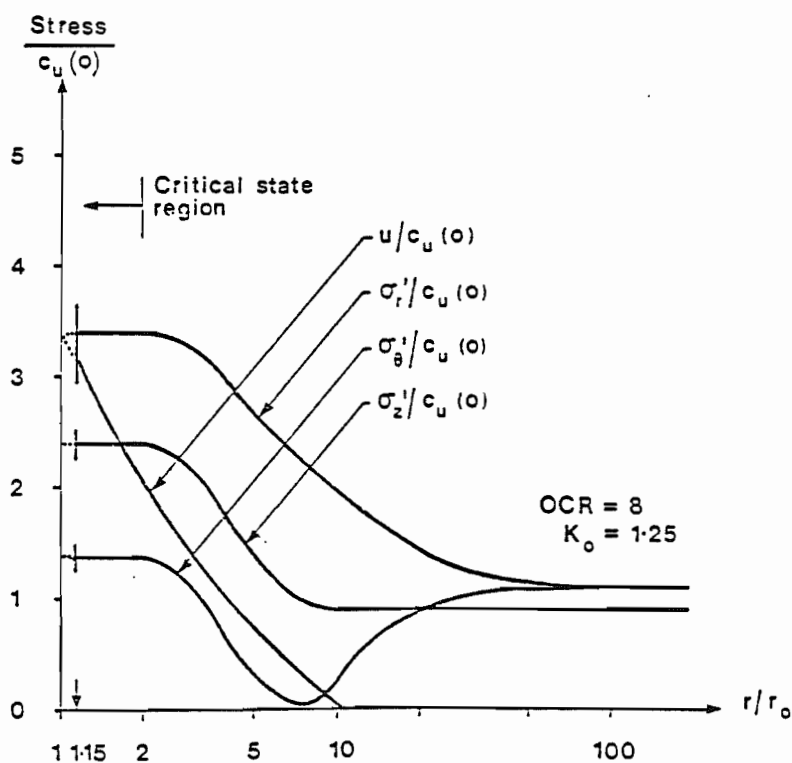
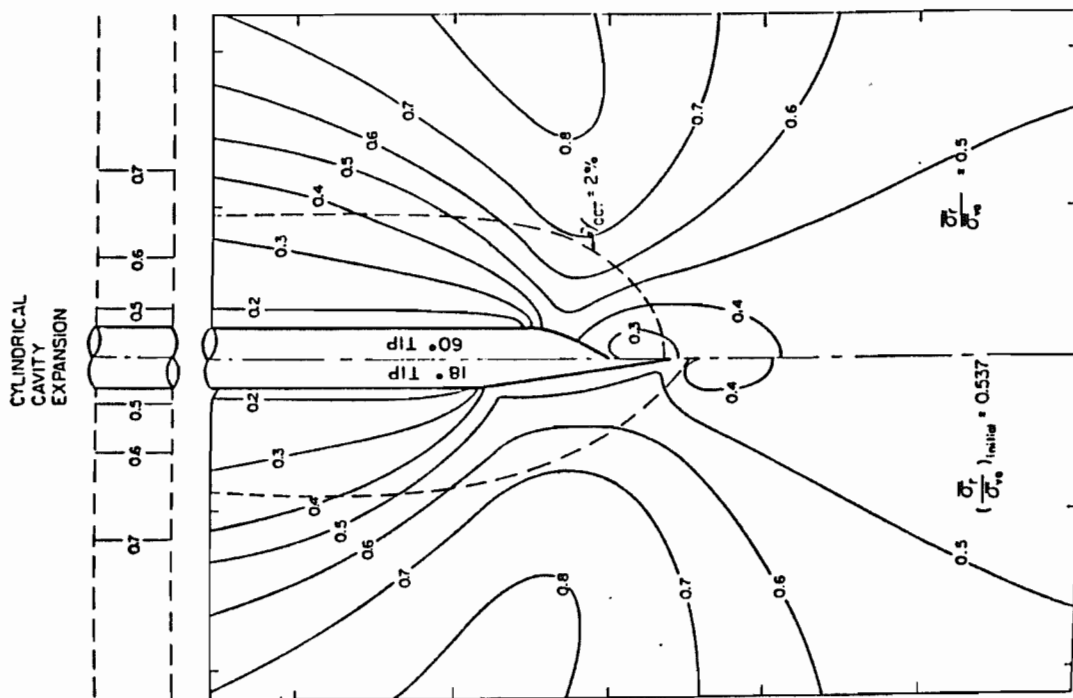
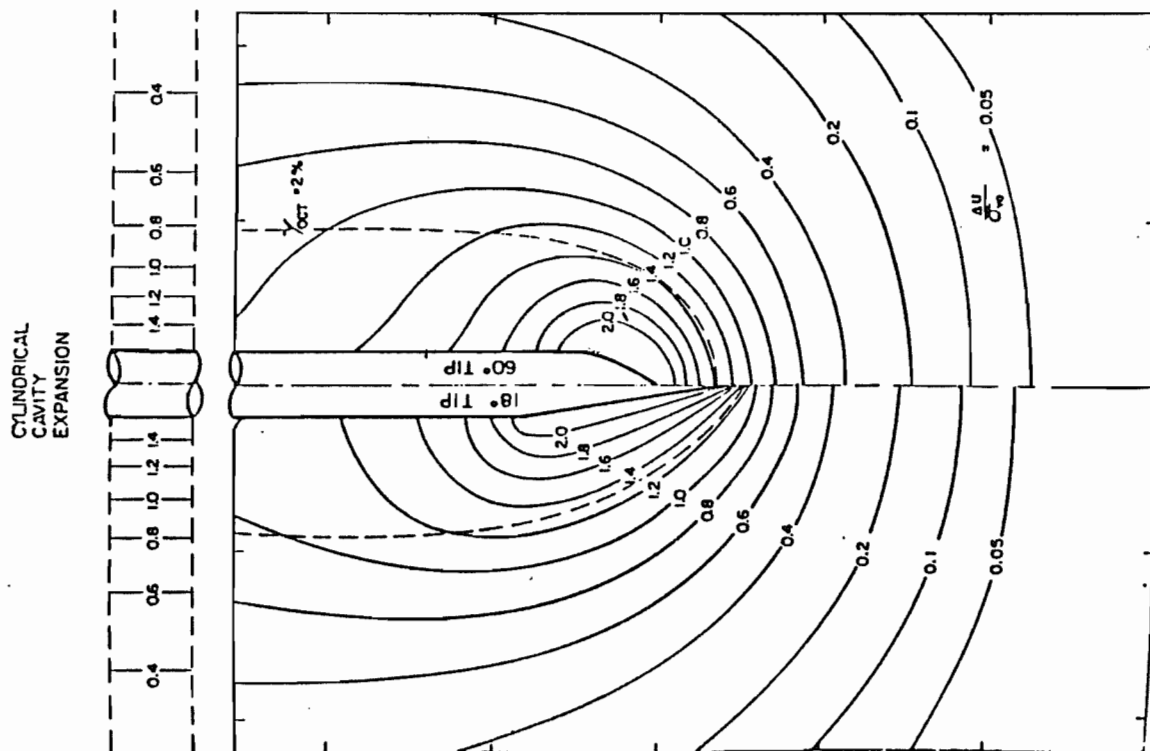


Figure 2.5 Radial Stress Distribution Immediately After Installation
in London clay (After Wroth et al. 1978).



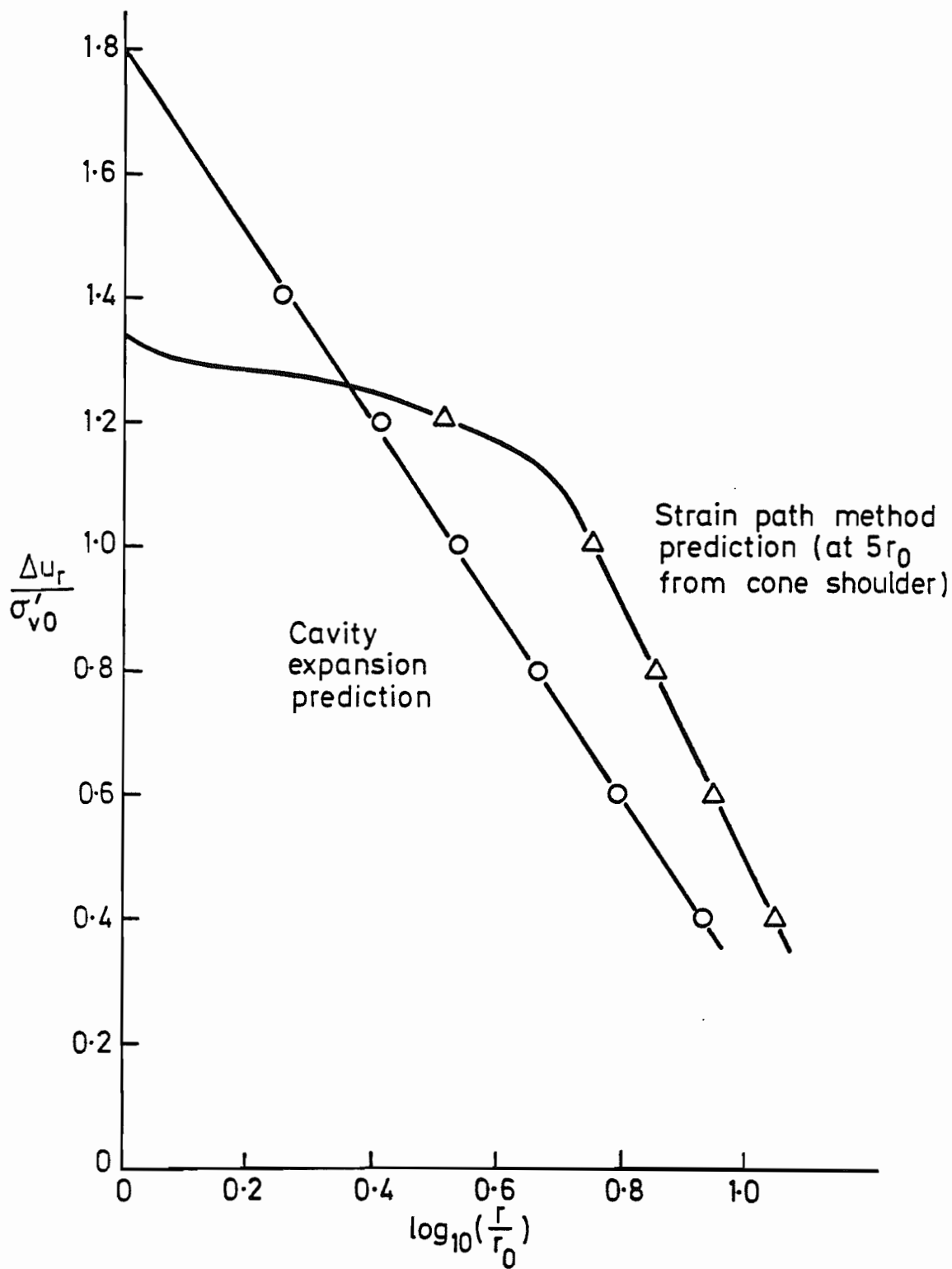


Figure 2.8 Predicted Radial Distribution of Excess Pore Pressures for Installation Scaled From Data Presented by Levadoux & Baligh (1980).

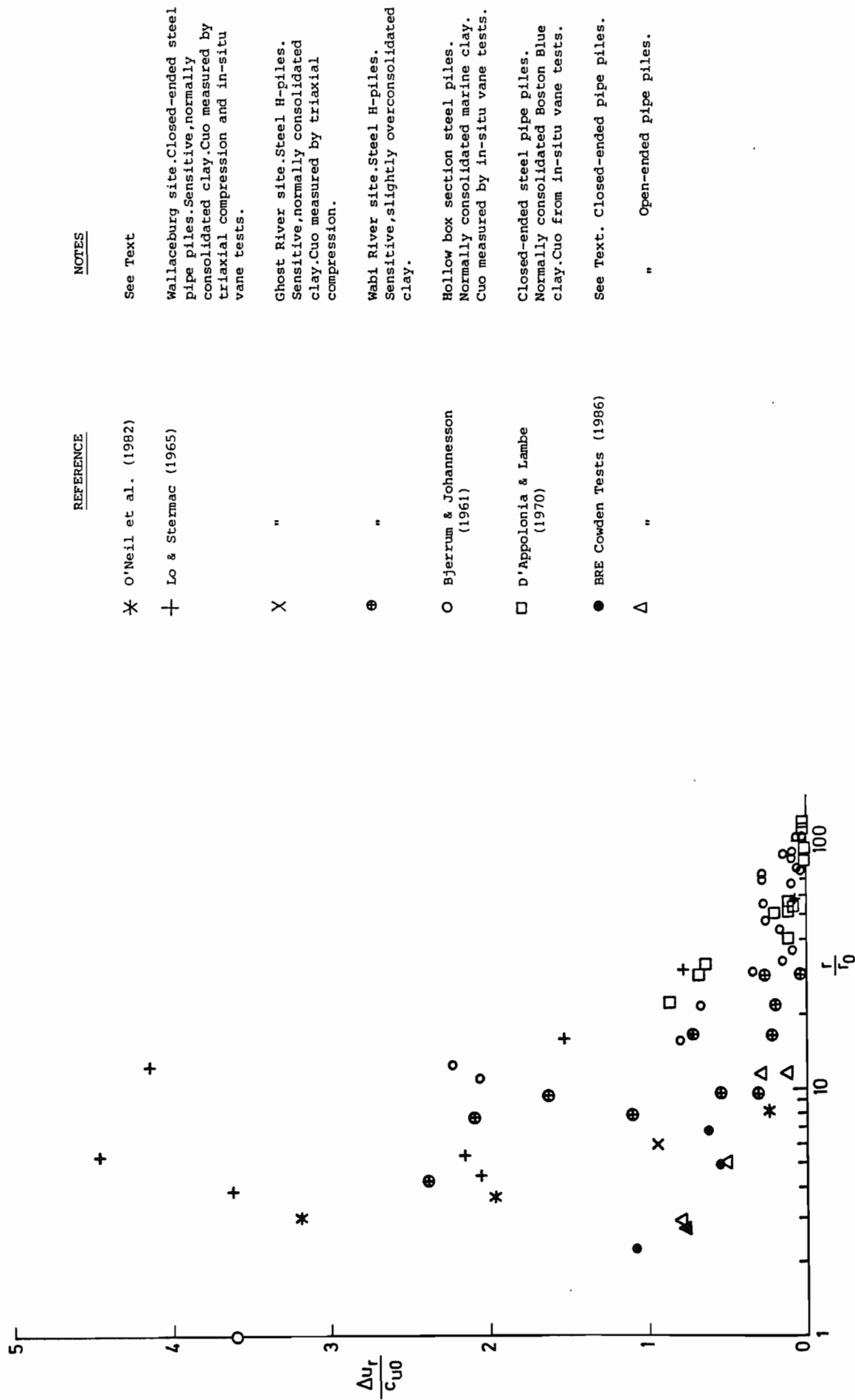


Figure 2.9 Field Measurements of Installation Excess Pore Pressures Around Piles.
(Datapoints Interpreted from the References Indicated)

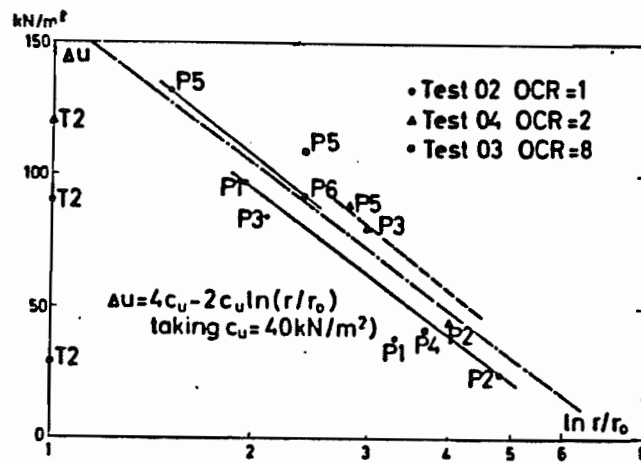


Figure 2.10 Radial Distribution of Excess Pore Pressures for the Installation of a Model Pile (After Steenfelt et al. 1981)

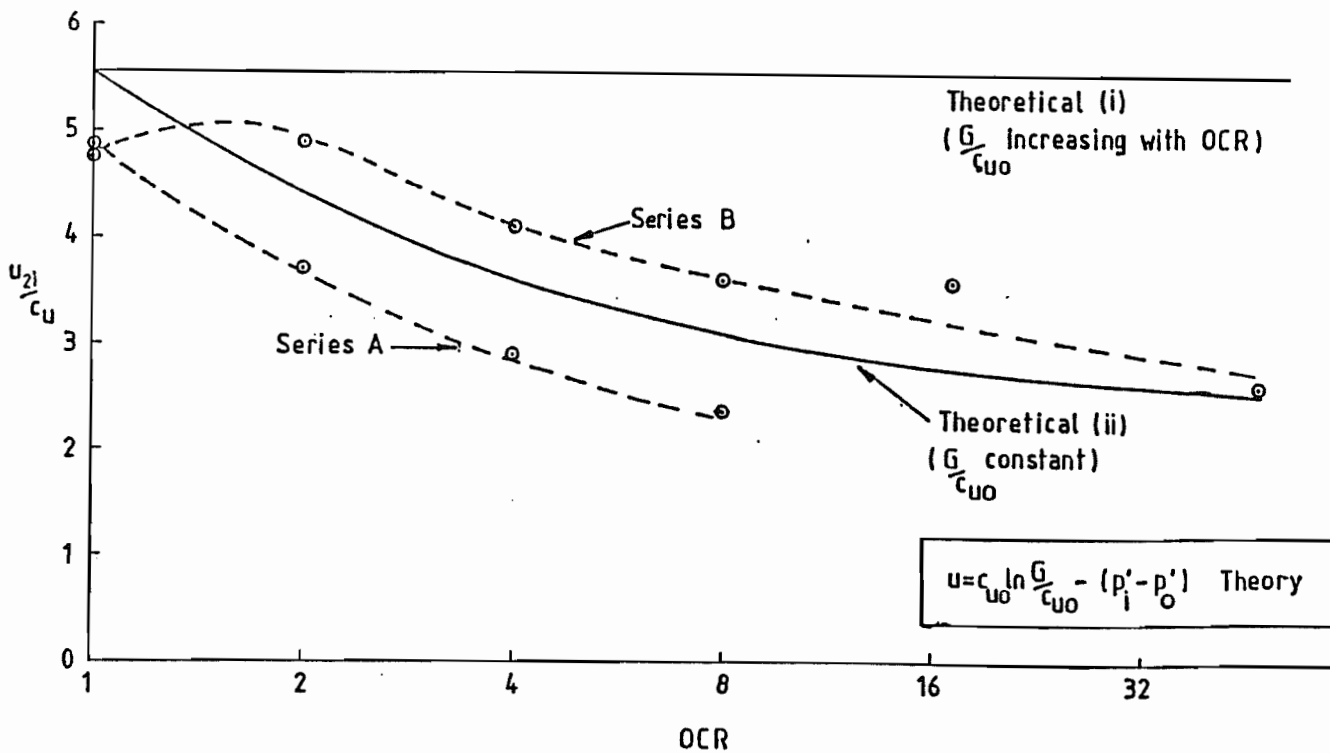


Figure 2.11 Excess Pore Pressures on the Shaft of a Model Pile During Installation (After Francescon 1983) -series A= stress controlled chamber tests, series B = tests in consolidometer

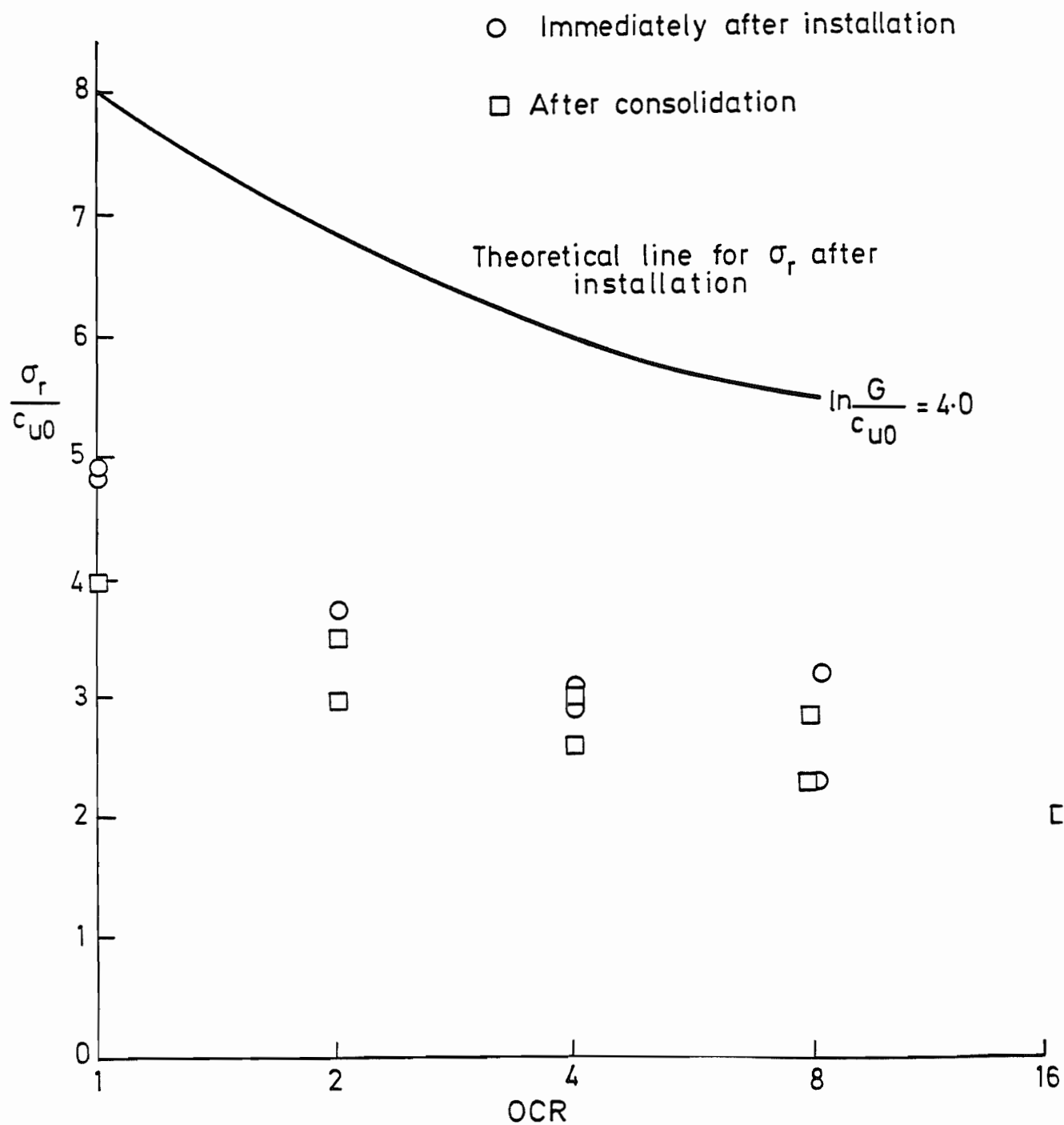


Figure 2.12 Total Radial Stress Measurements on the Shaft of a Model Pile
Made by Francescon (1983) (-redrawn from his data).

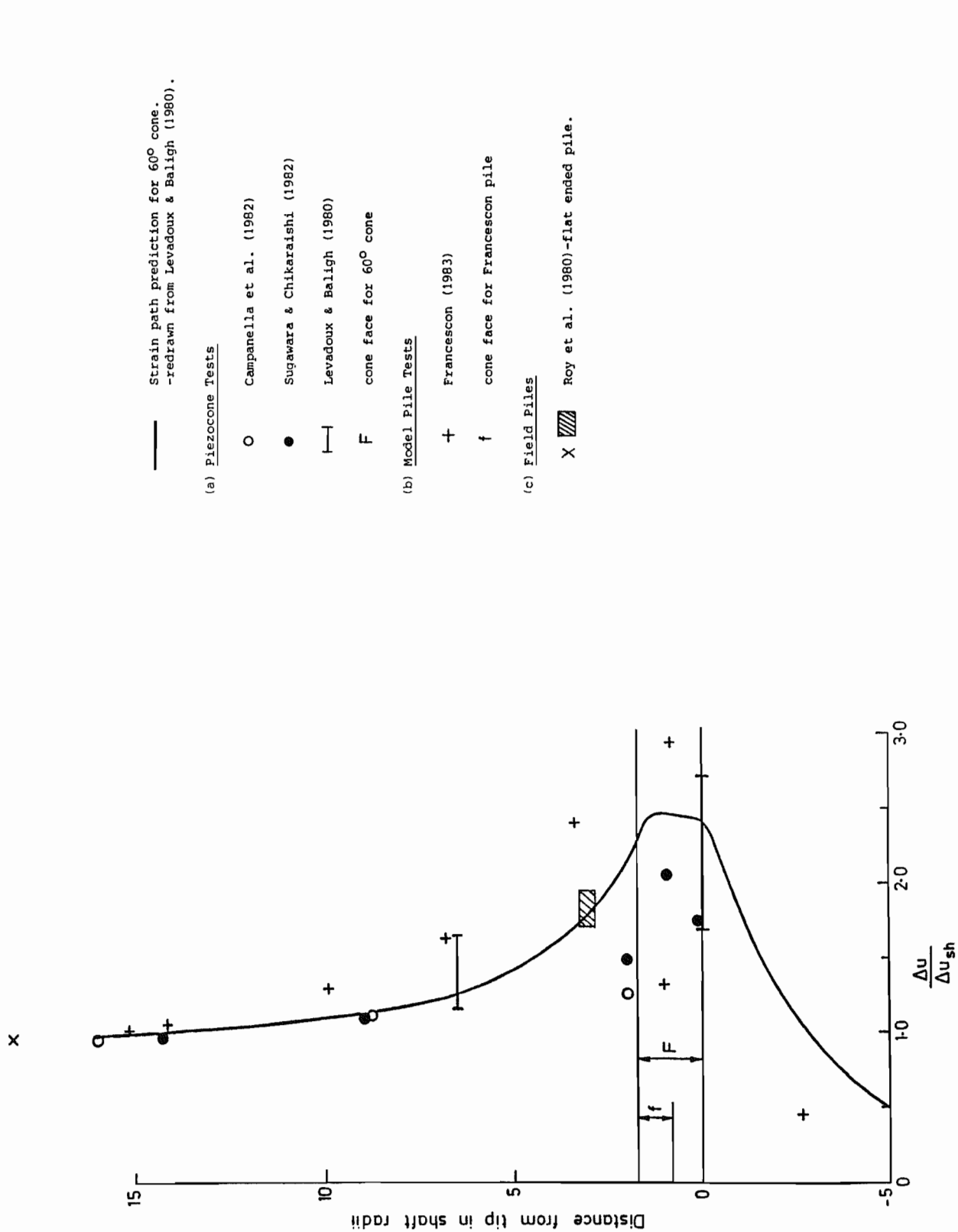


Figure 2.13 Variation of Installation Excess Pore Pressures Along the Shaft of Piles and Penetrometers.
 (Redrawn with some additions from May 1983)

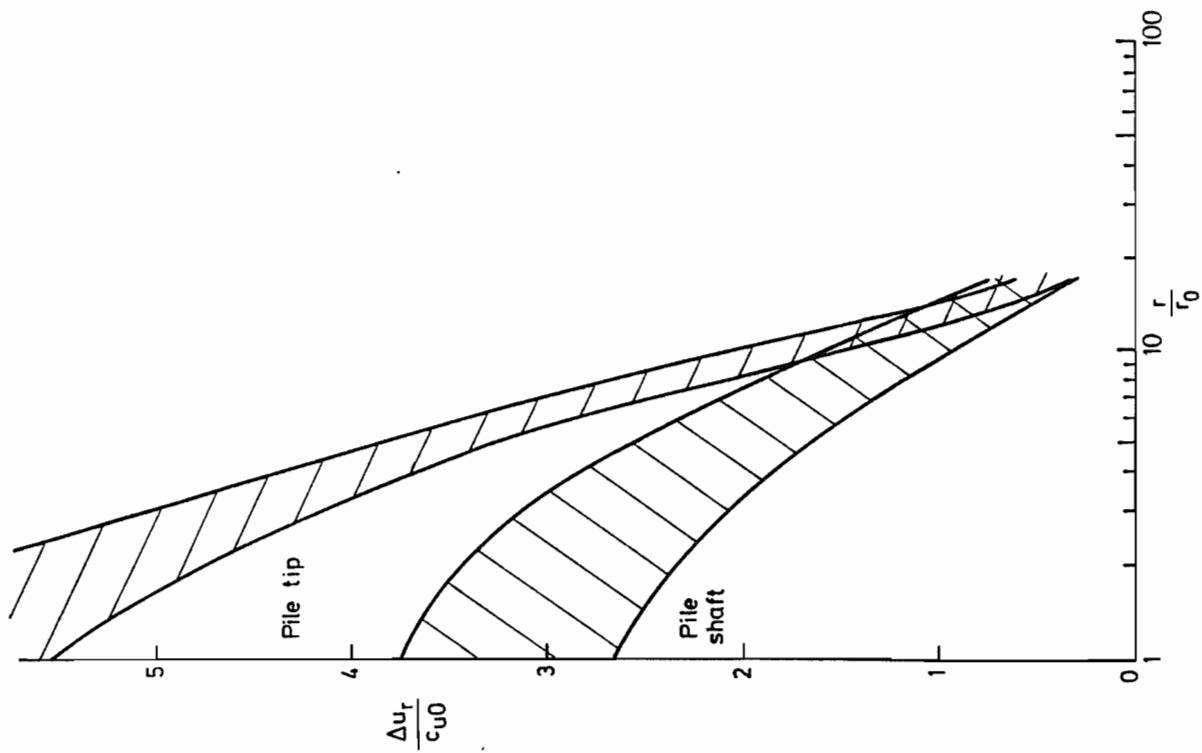


Figure 2.14 Radial Distribution of Installation Excess Pore Pressures Measured by Roy et al. (1967).
-data replotted on log scale.

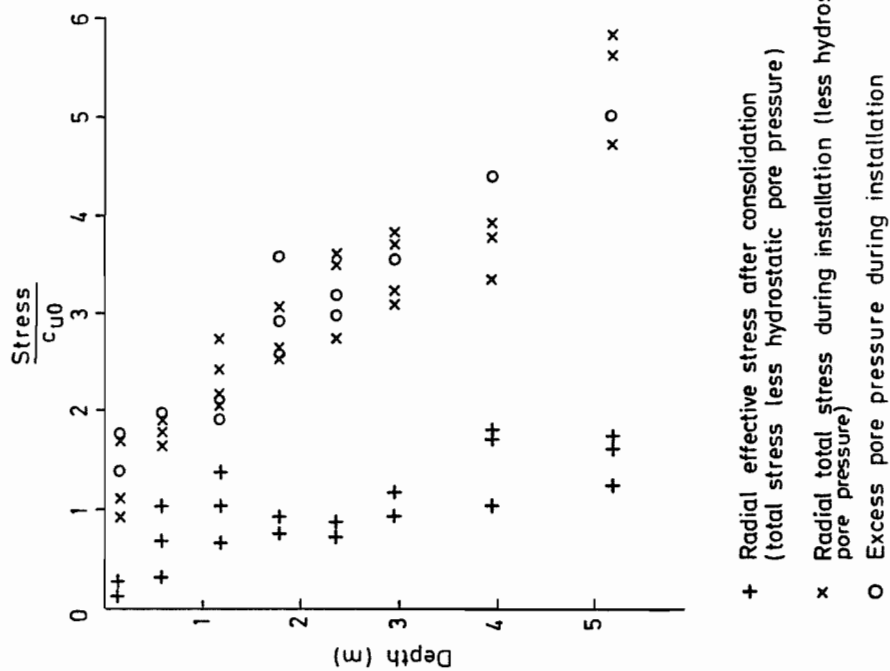


Figure 2.15 Normalised Stresses Acting on a Driven Pile Interpreted from Data presented by Koizumi & Ito (1967).

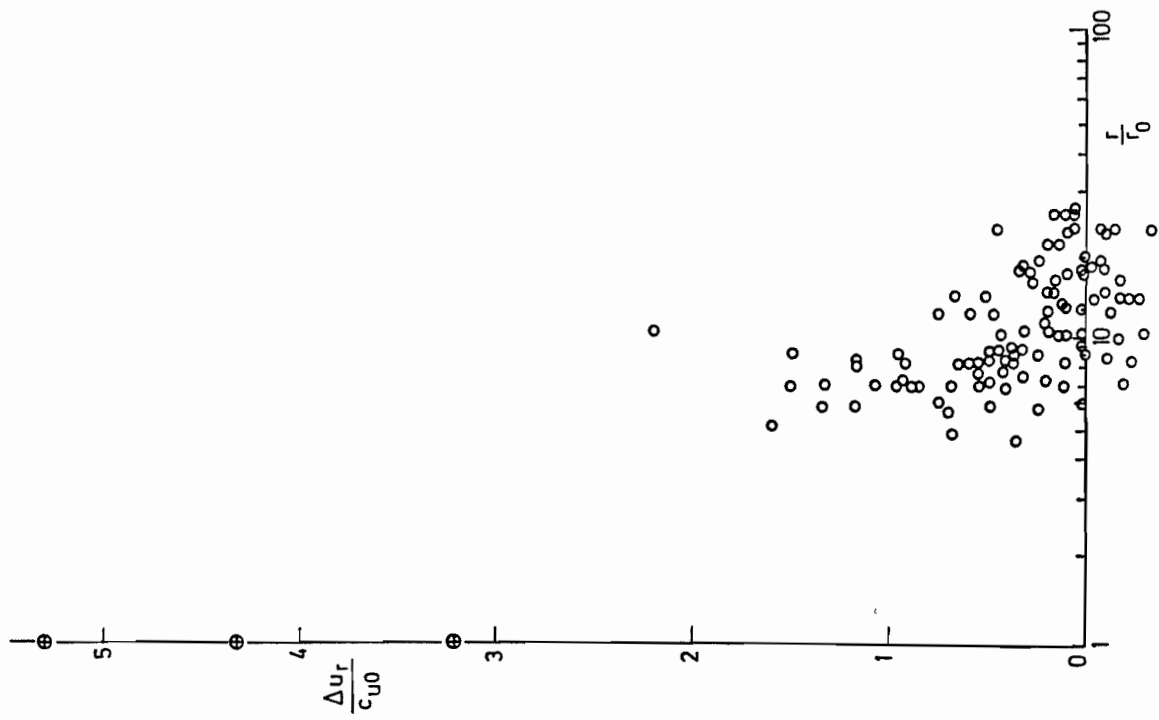


Figure 2.16 Radial Distribution of Installation Excess Pore Pressures Interpreted from Data Presented by Koizumi & Ito (1967).

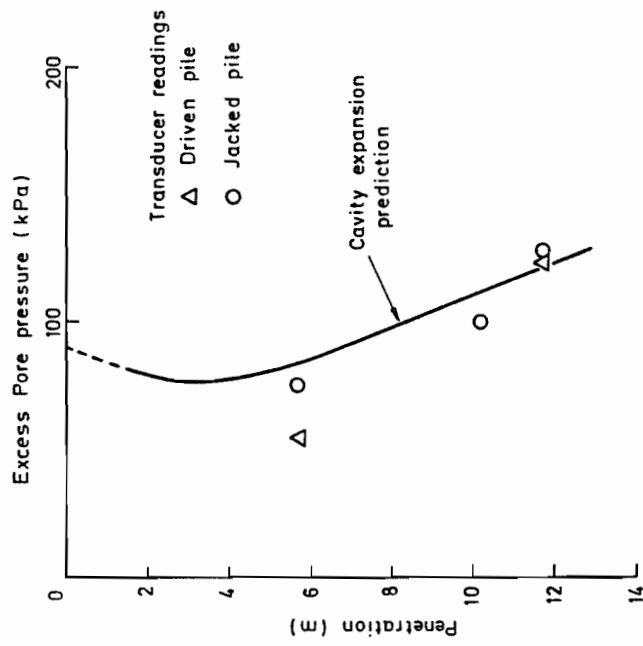


Figure 2.17 Installation Excess Pore Pressures Measured on Pile Shaft by Kirby & Roussel (1980) -redrawn from their data

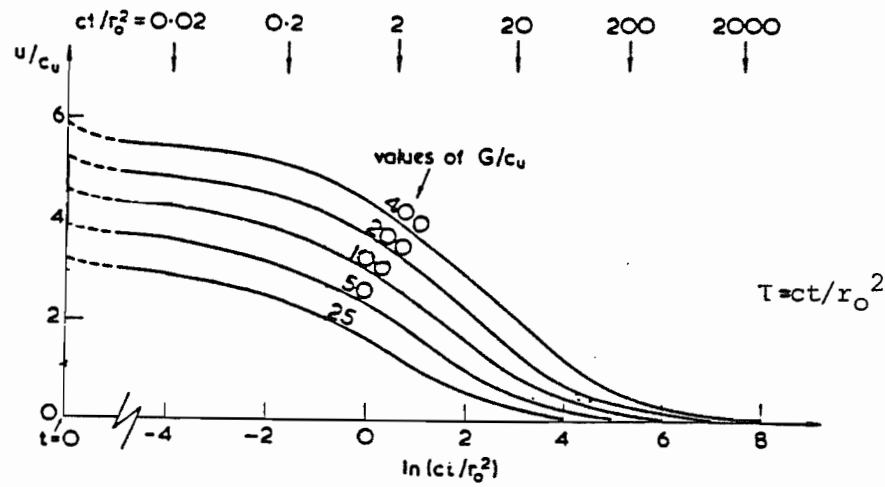


Figure 2.18 Dissipation of Excess Pore Pressures During Reconsolidation (After Randolph & Wroth 1979).

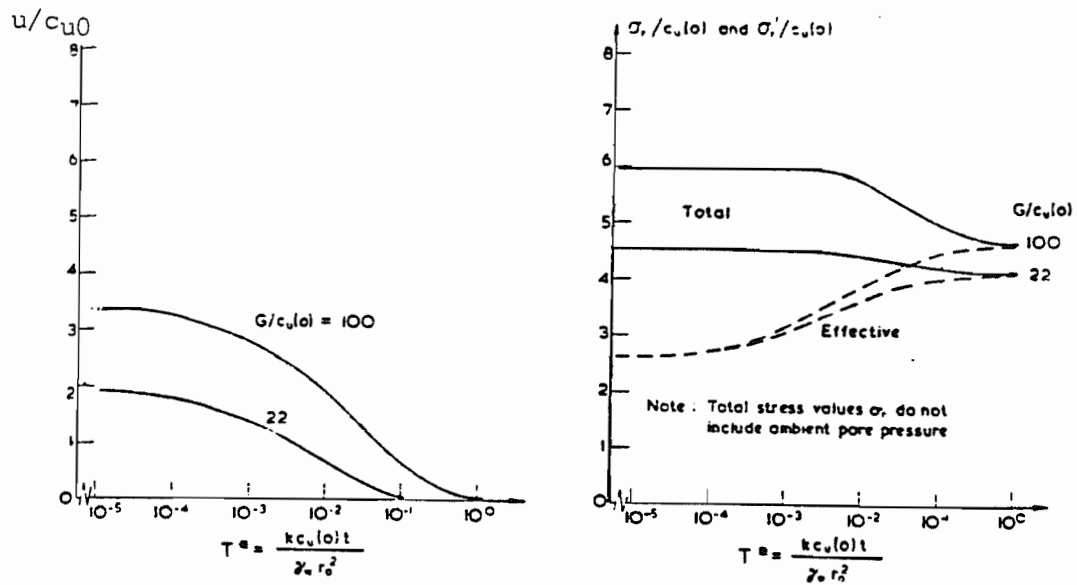


Figure 2.19 Change in Radial Stresses and Excess Pore Pressures Predicted During Reconsolidation (After Randolph et al. 1979)

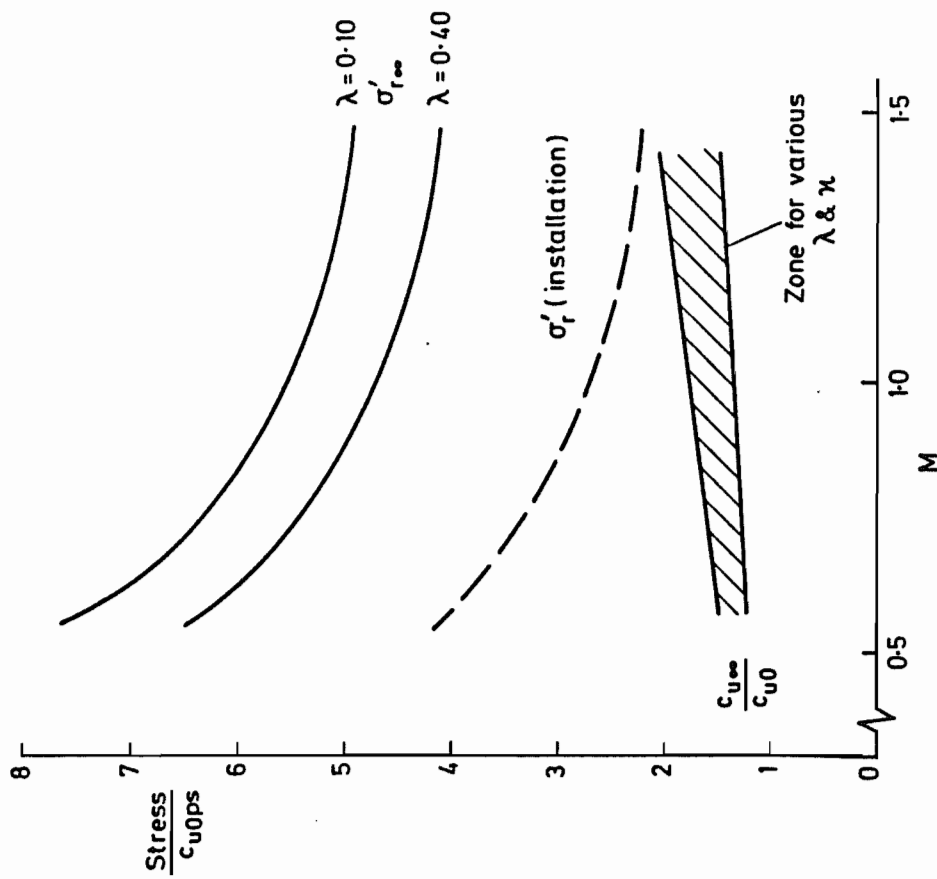


Figure 2.20 Effect of Cam-Clay Parameters on Cavity Expansion Predictions (Redrawn From Wroth et al. 1979).

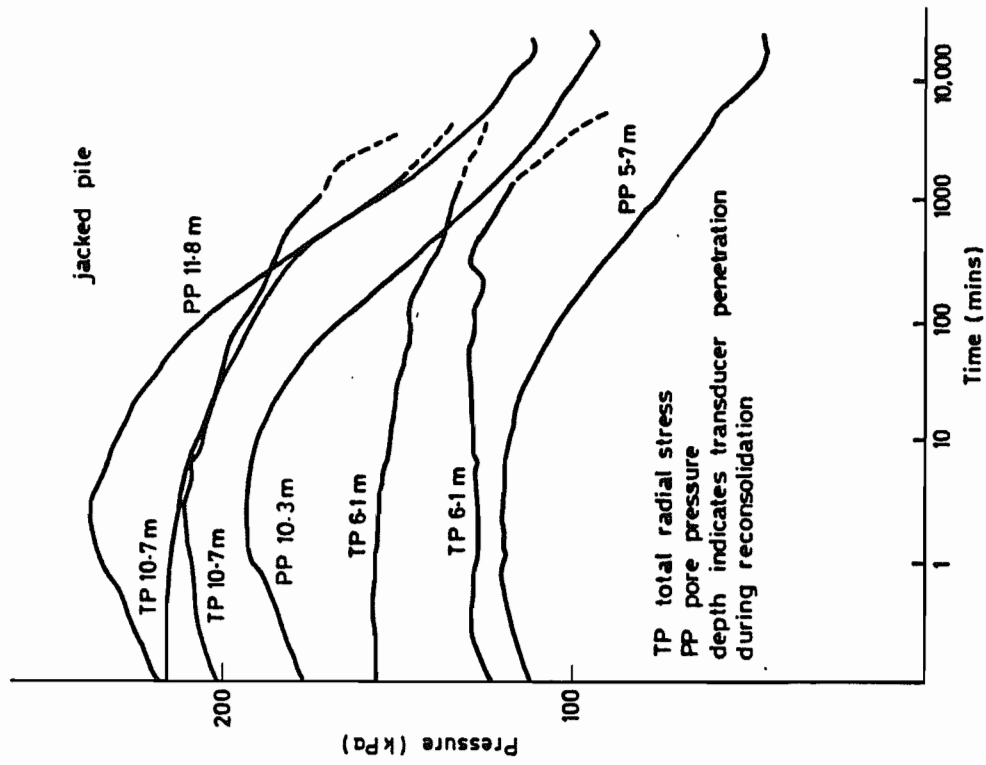


Figure 2.21 Stress Changes Around a Pile During Reconsolidation Measured by Kirby & Roussel (1980) -redrawn from their data

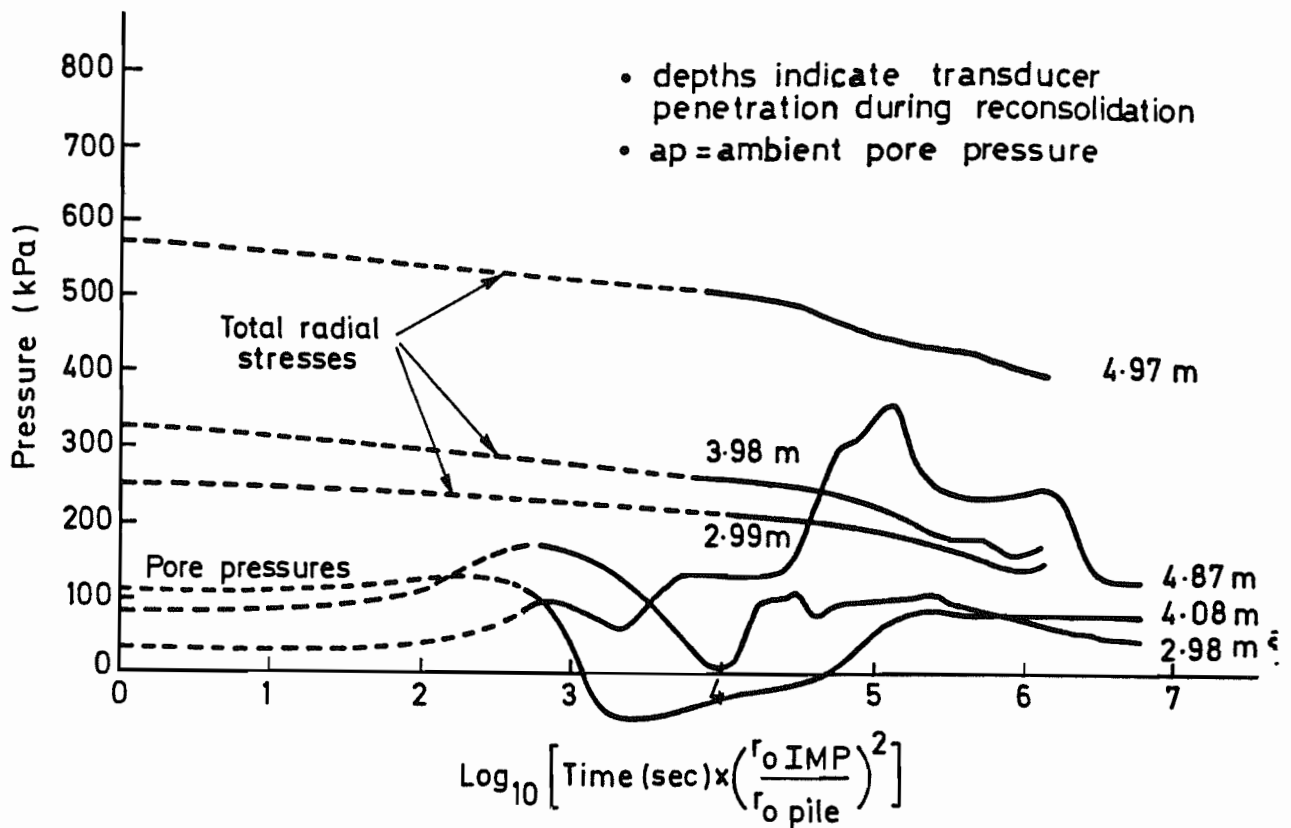


Figure 2.22 Reconsolidation Data from Jardine's Model Pile
(Data from his thesis (1985) scaled for comparison with IMP data and replotted on Log scale.)

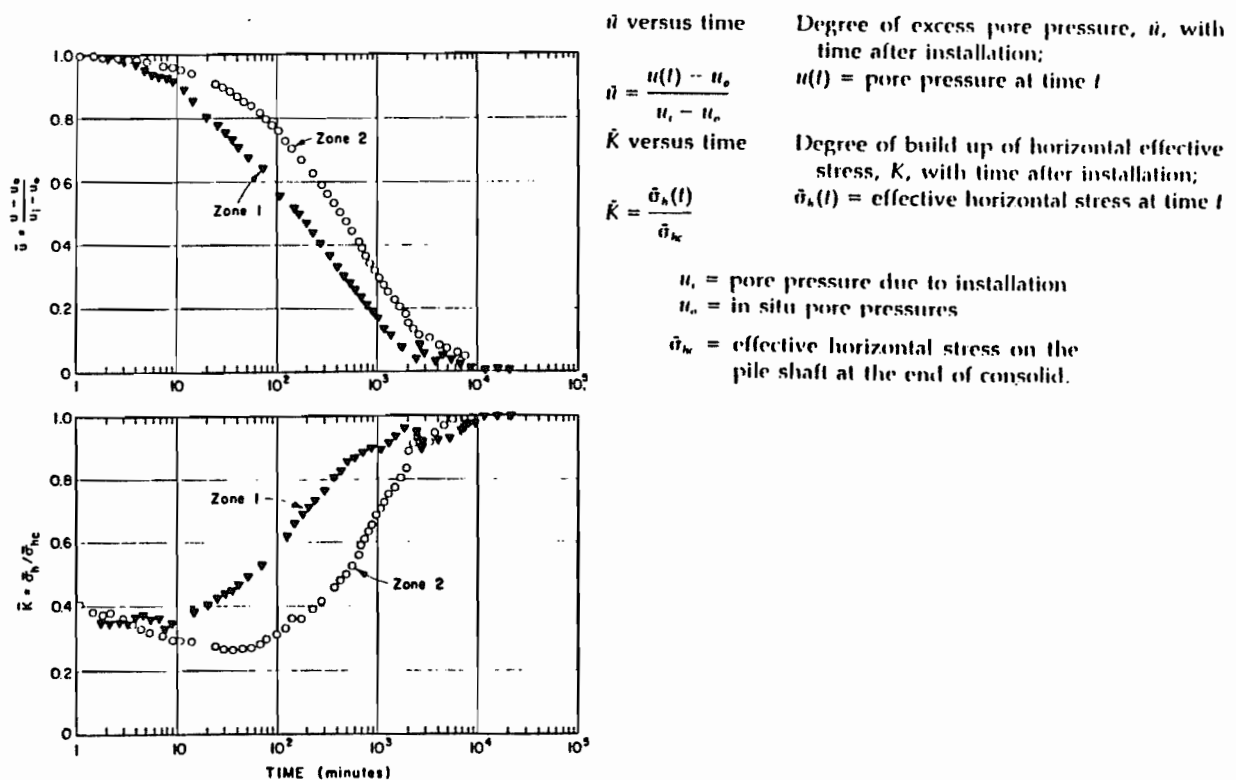


Figure 2.23 Reconsolidation Data from the PLS Cell
(After Azzouz & Lutz 1986)

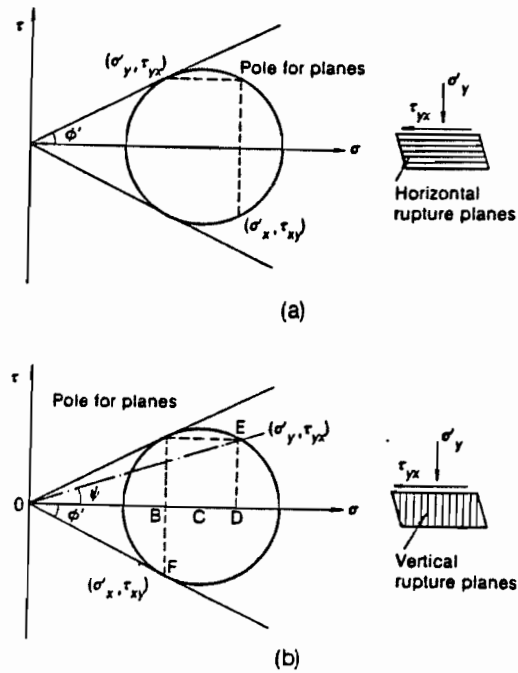


Figure 2.24 Modes of Failure in Undrained Simple Shear.
(After Randolph & Wroth 1981)

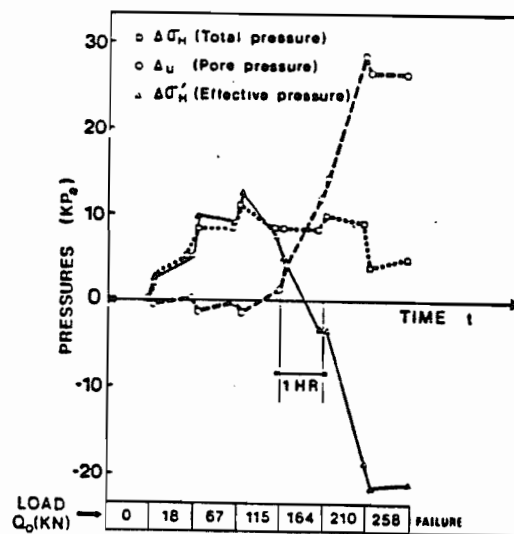


Figure 2.25 Field Measurements of Stress Changes During Undrained Pile Loading (After Puech & Jezequel 1980).

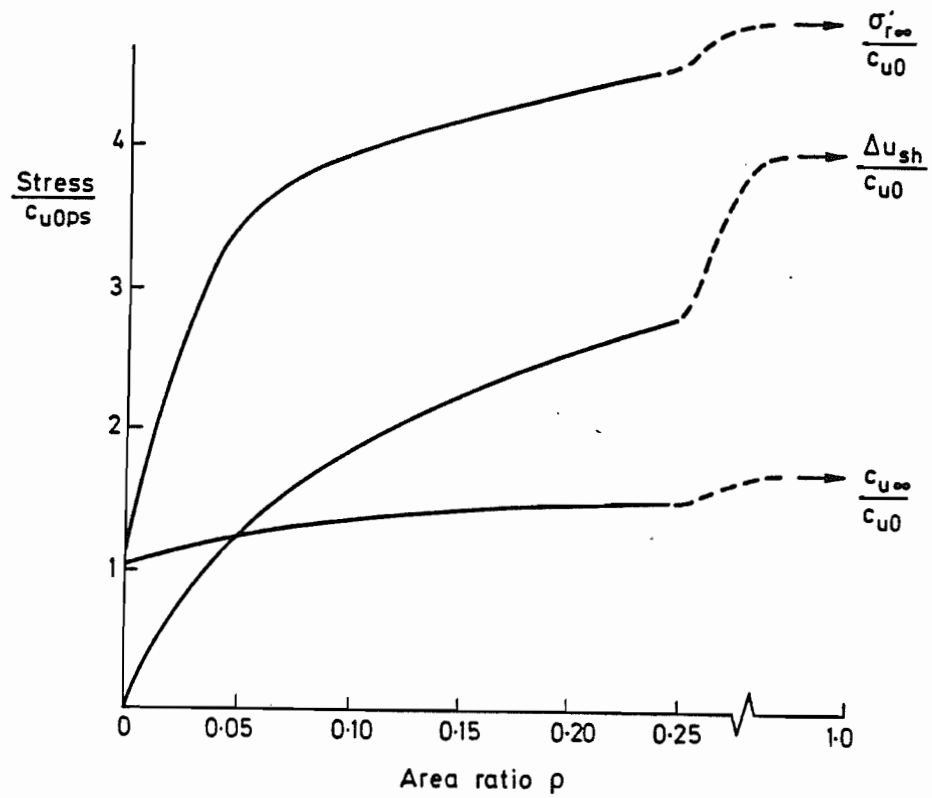


Figure 2.26 Effect of Open-Ended Installation on Cavity Expansion Predictions (Redrawn from Carter et al. 1979b)

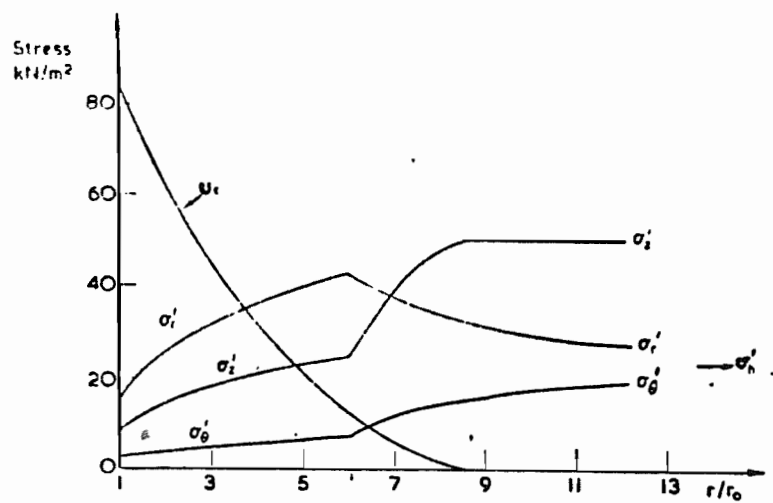


Figure 2.27 Predicted Stress Distribution Immediately After Installation for a Sensitive Soil (After Randolph et al. 1979).

	PDCR-81 PORE PRESSURE TRANSDUC.	STRAIN- GAUGED PORE PRES TRANSDUC.	TOTAL RADIAL STRESS TRANSDUC.	AXIAL LOAD CELLS		
				LC1	LC2	LC3*
UNAMPLIFIED CALIBRATION FACTOR AT 4 V.AC	0.088 mv/kPa	0.005 mv/kPa	0.004 mv/kPa	0.47 mv/kN	0.22 mv/kN	0.19 mv/kN
SENSITIVITY OF CALIBRATION FACTOR TO TEMPERATURE	0.25 %/°C	0.43 %/°C	0.04 %/°C	0.07 %/°C	0.01 %/°C	0.02 %/°C
SENSITIVITY OF ZERO READING TO TEMPERATURE	0.7 kPa/°C	0.9 kPa/°C	5.1 kPa/°C	0.154 kN/°C	0.005 kN/°C	0.002 kN/°C
SENSITIVITY TO AXIAL LOAD	0.0 kPa/kN	2.0 kPa/kN	1.8 kPa/kN	-	-	-
SENSITIVITY TO NET TOTAL RADIAL STRESS	0.0 kPa/kPa	0.016 kPa/kPa	-	-	-	-
SENSITIVITY TO SHEAR LOADS	-	-	0.005 kPa/kPa	-	-	-
TYPICAL ZERO ERRORS ON EXTRACTION	3.7 kPa	3.5 kPa	7.8 kPa	0.1 kN	0.1 kN	0.3 kN

* Figures shown for re-gauged version

TABLE 3.1 Typical Calibration Factors and Sensitivities of IMP Transducers.

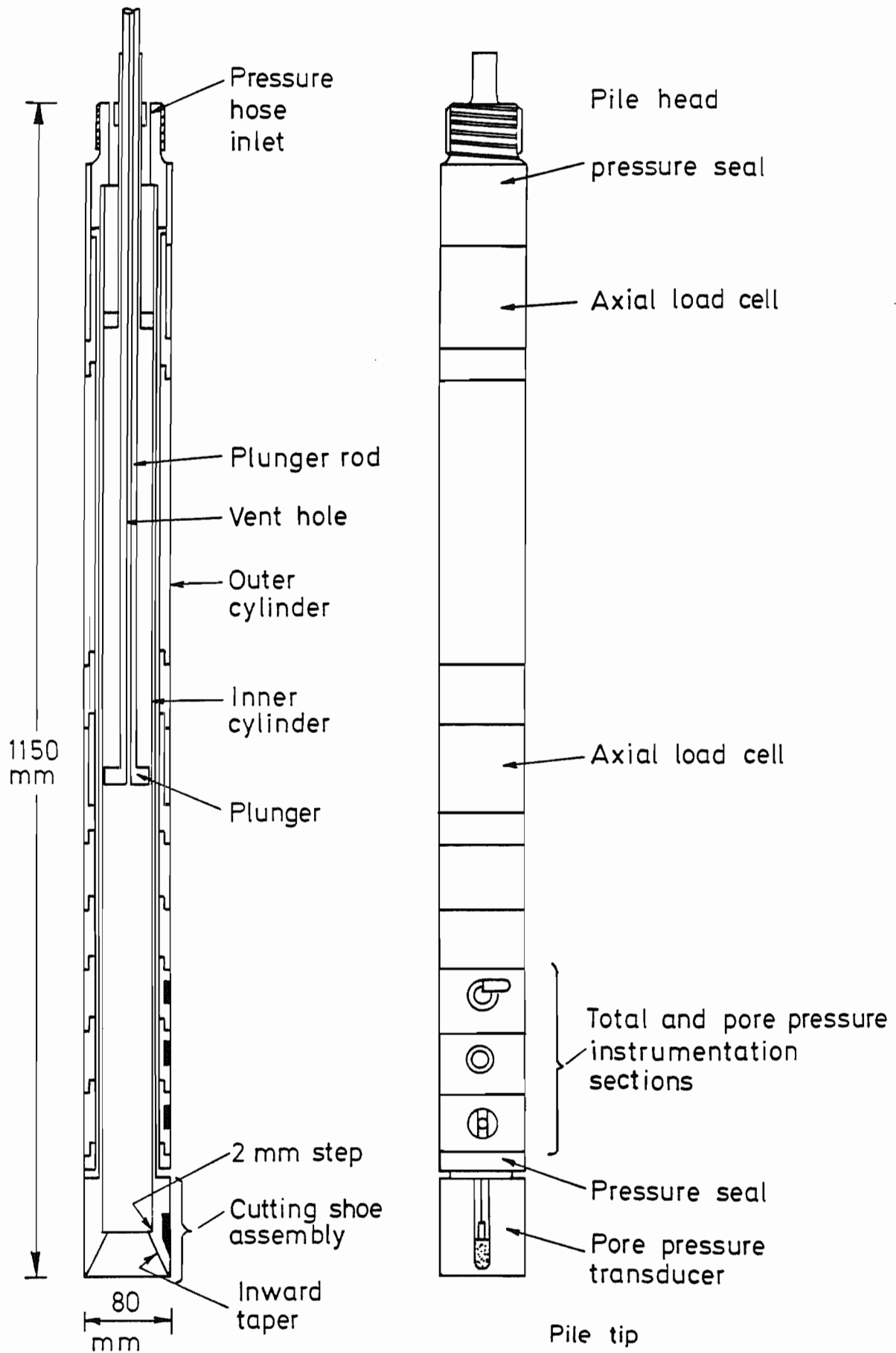


Figure 3.1 Sketch of the In-Situ Model Pile (-instrumentation configuration similar to that for Boreholes A&B.)

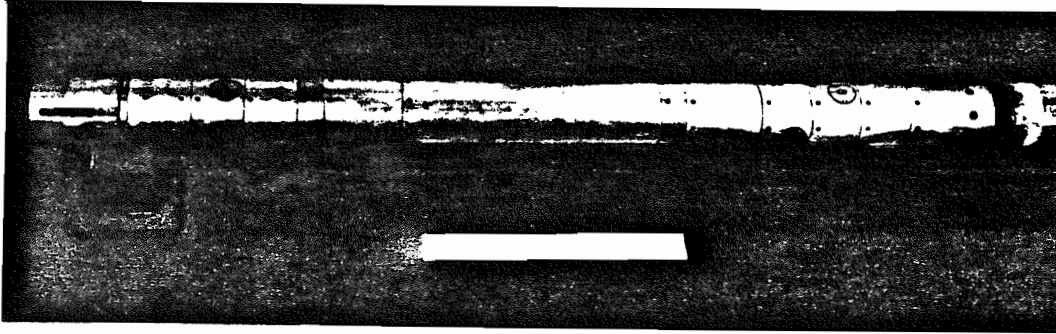


Figure 3.2 The In-Situ Model Pile. (Instrumentation as Used for Boreholes G-L)

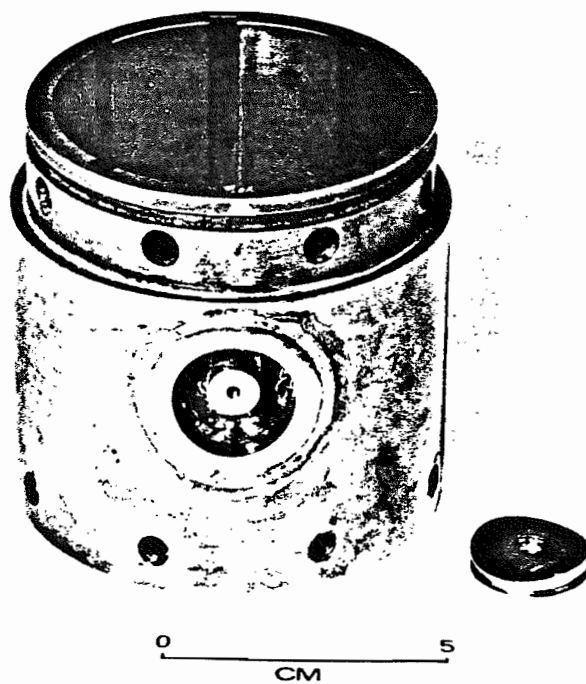


Figure 3.3 A Segment of the IMP Shaft Showing a Total Radial Stress Transducer.

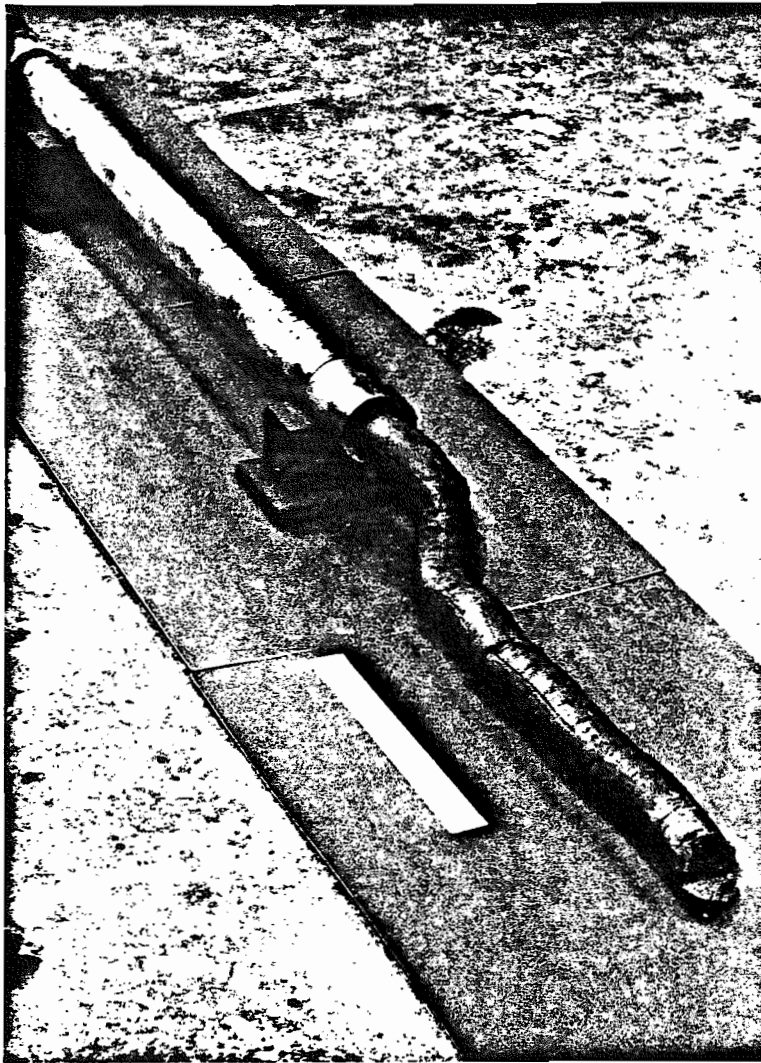


Figure 3.4 Extrusion of Clay from the IMP After Borehole K.

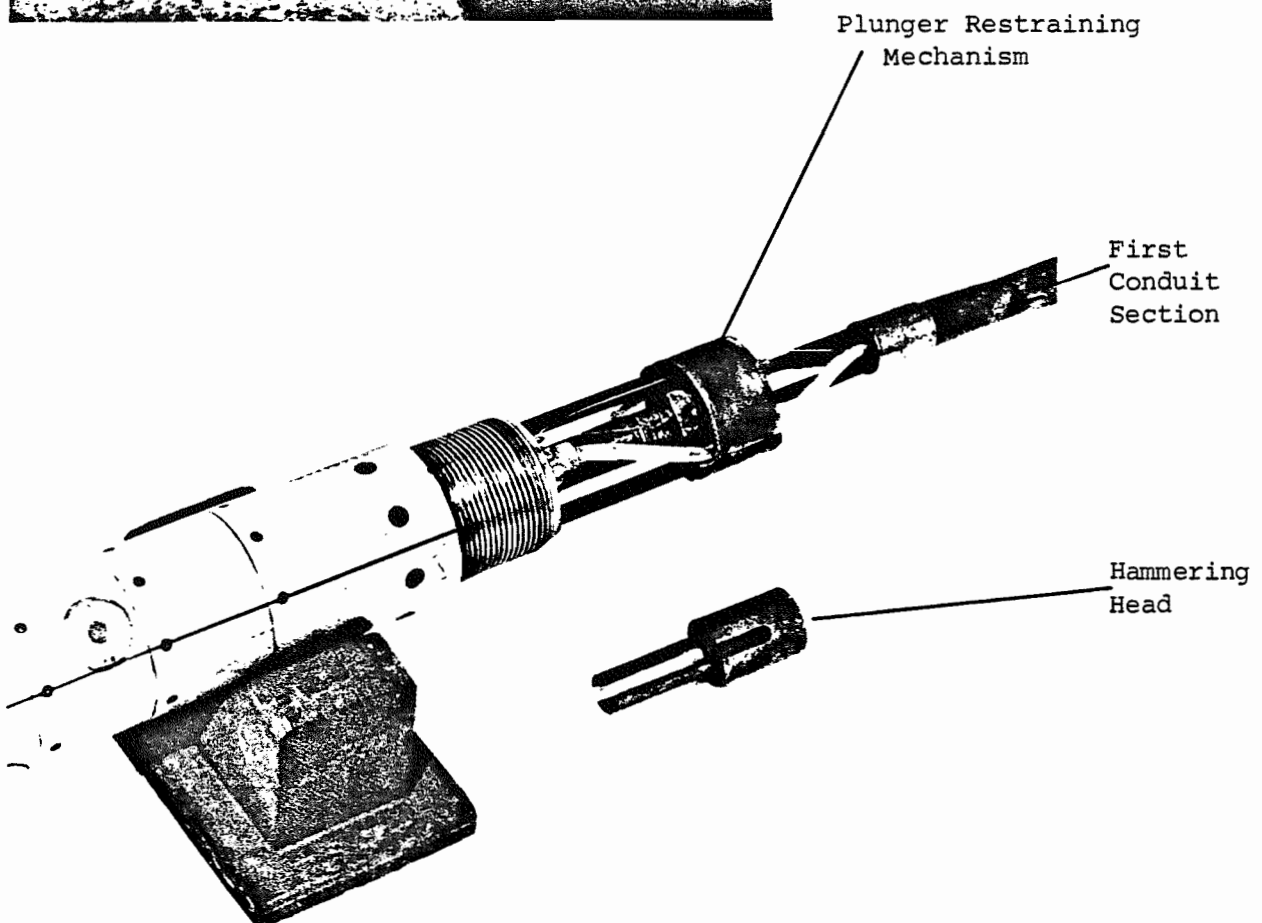


Figure 3.5 The IMP Head Arrangement.

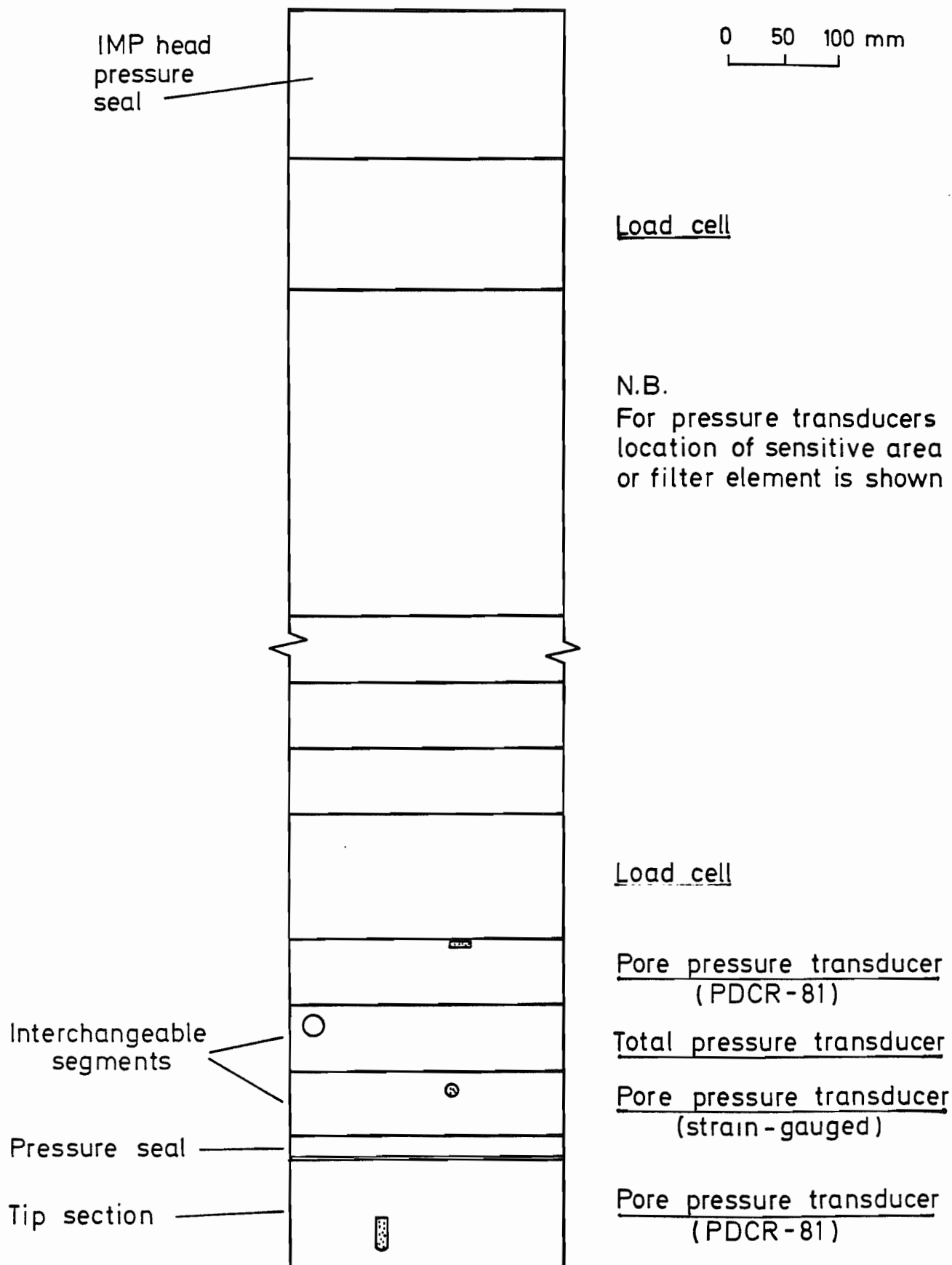


Figure 3.6 Opened-Out Plan of IMP Shaft Showing Transducer Locations for Boreholes A & B -colours indicate those used on plots of raw data

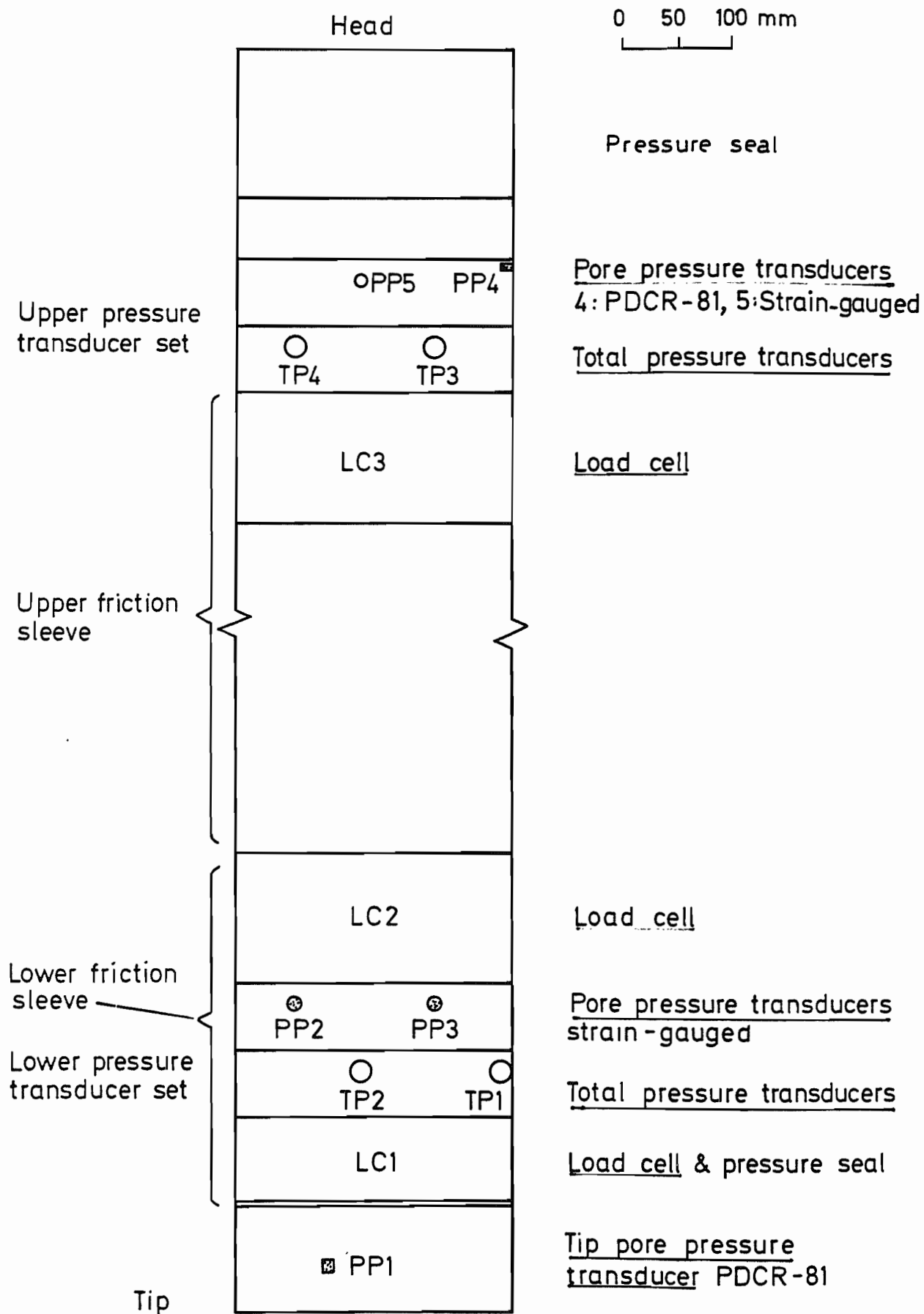


Figure 3.7 Opened-Out Plan of IMP Shaft Showing Transducer Locations for Boreholes C-F .
-colours are those used on raw data plots.

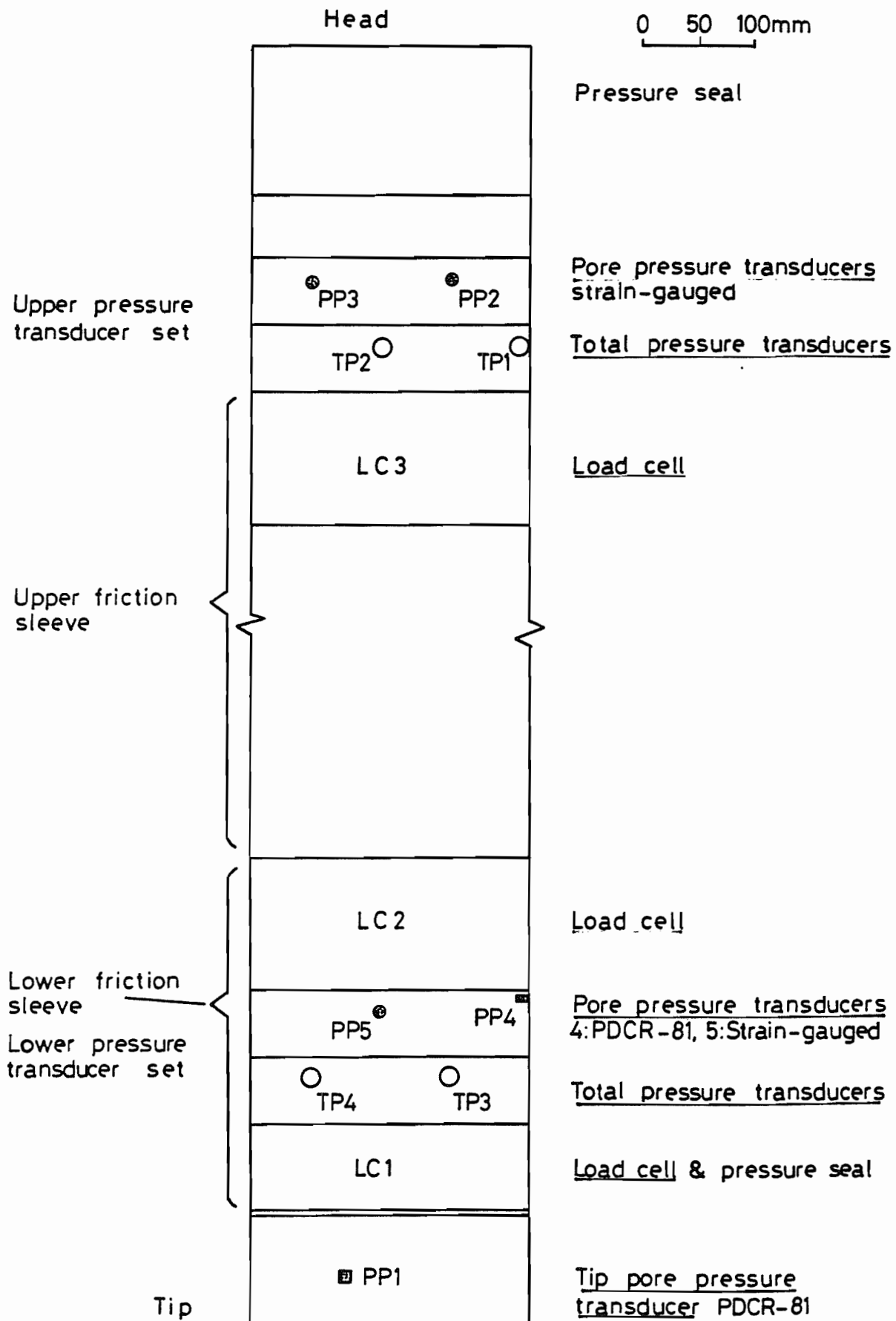


Figure 3.8 Opened-Out Plan of IMP Shaft Showing Transducer Locations for Boreholes G-L.
-colours are those used on raw data plots.

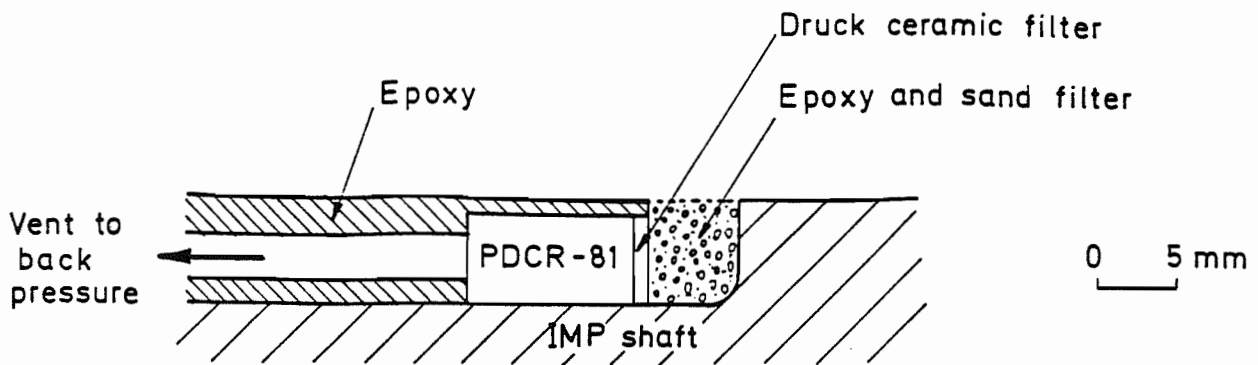


Figure 3.9 Sketch of a Druck PDCR-81 Pore Pressure Transducer Mounted in the IMP Shaft.

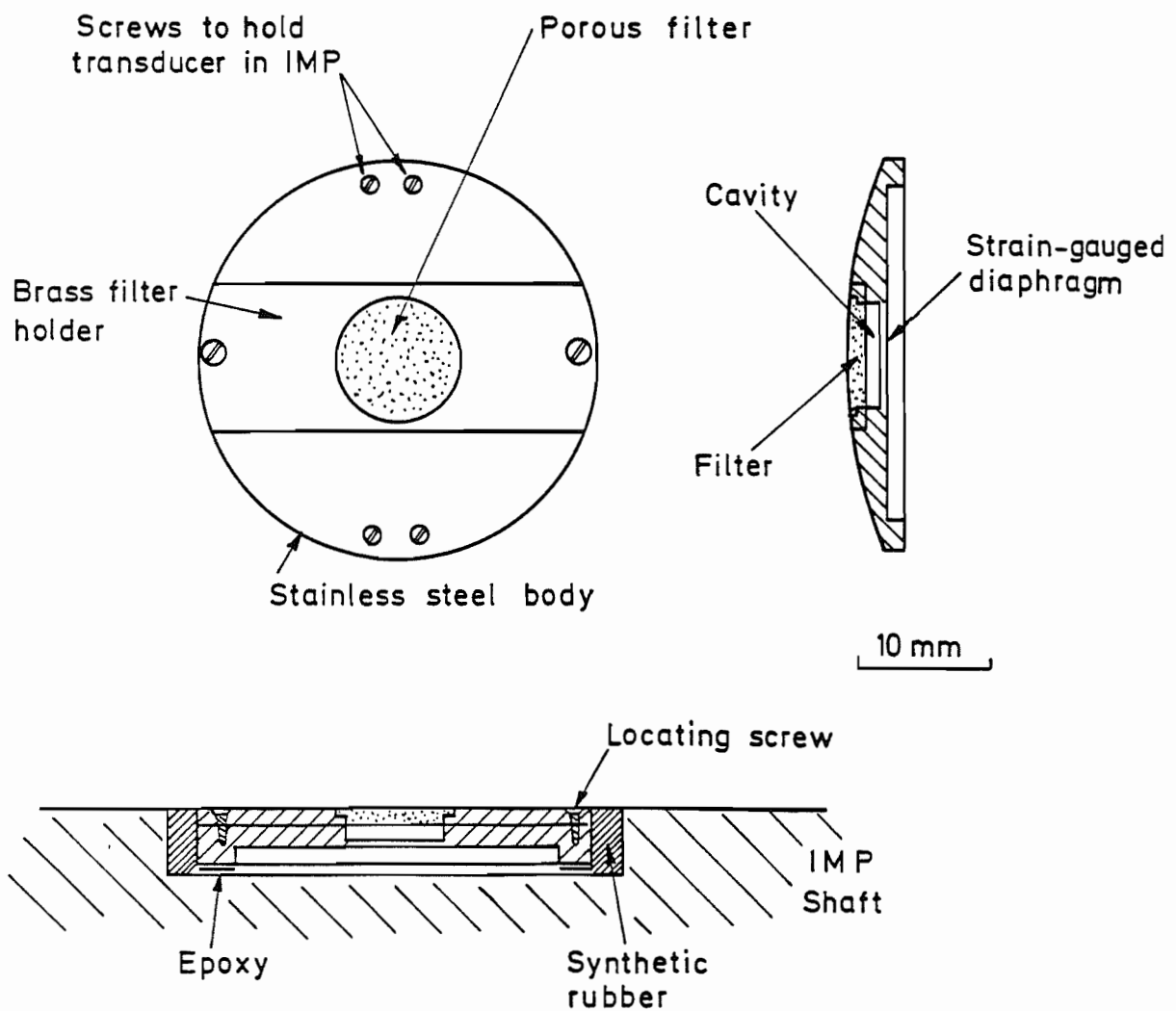


Figure 3.10 Strain-Gauged Pore Pressure Transducer.

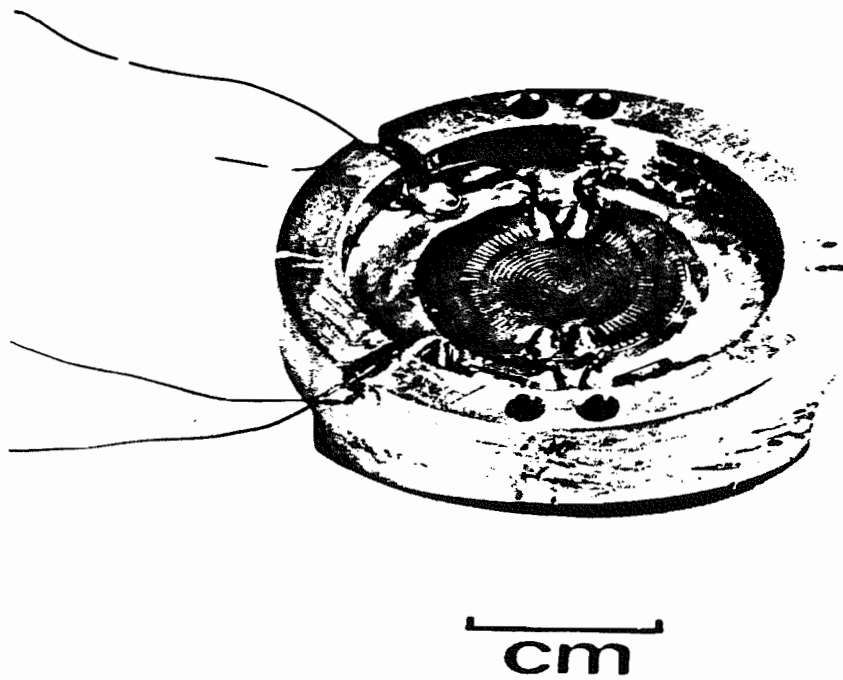
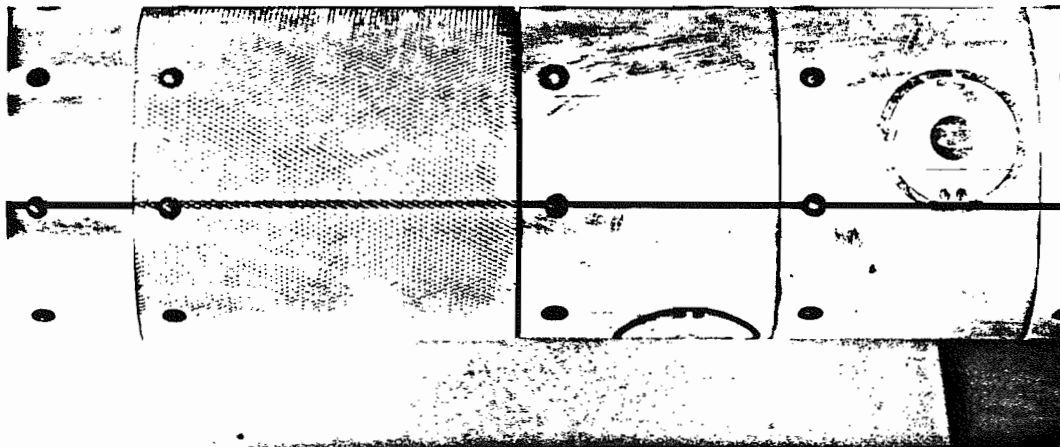


Figure 3.11 The Base of a Strain-Gauged Pore Pressure Transducer.



Load Cell (with
Rough Walled Sleeve)

Total Radial Stress
Transducer

Pore Pressure
Transducer

Figure 3.12 Instrumentation Mounted in the IMP Shaft.

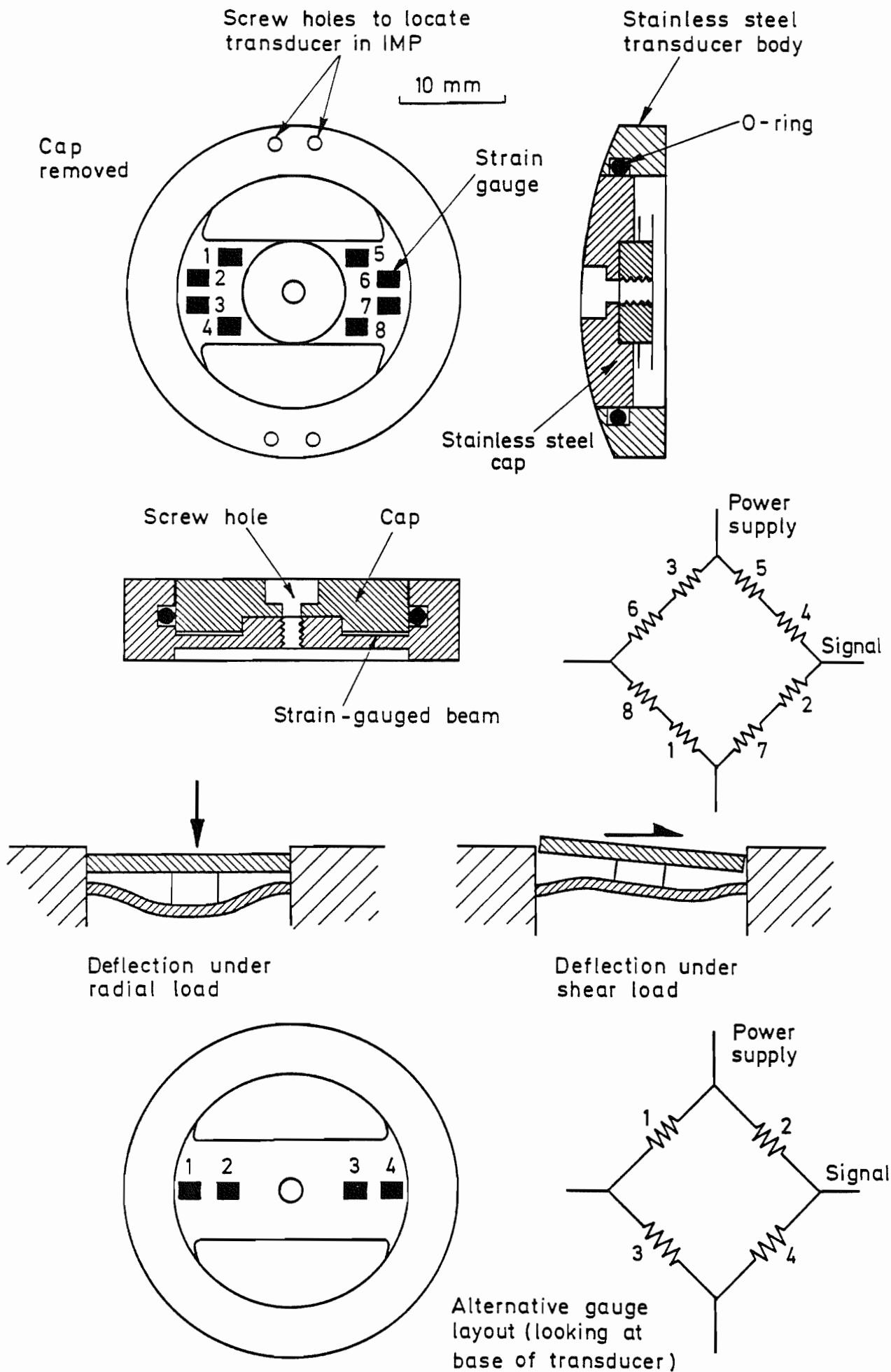


Figure 3.13 IMP Total Radial Stress Transducer.

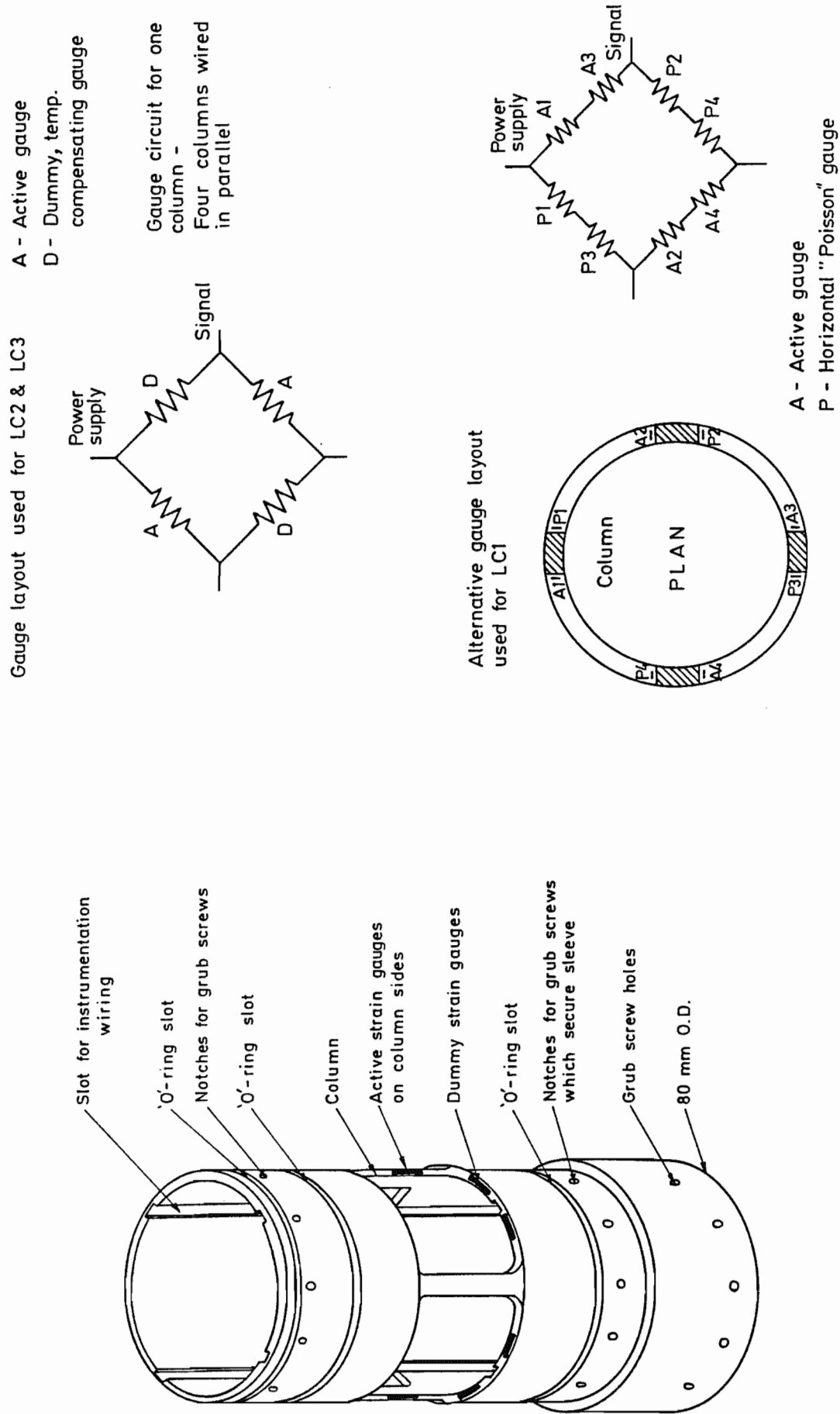


Figure 3.14 Sketch of a Typical IMP Load Cell Illustrating Gauge Layouts.

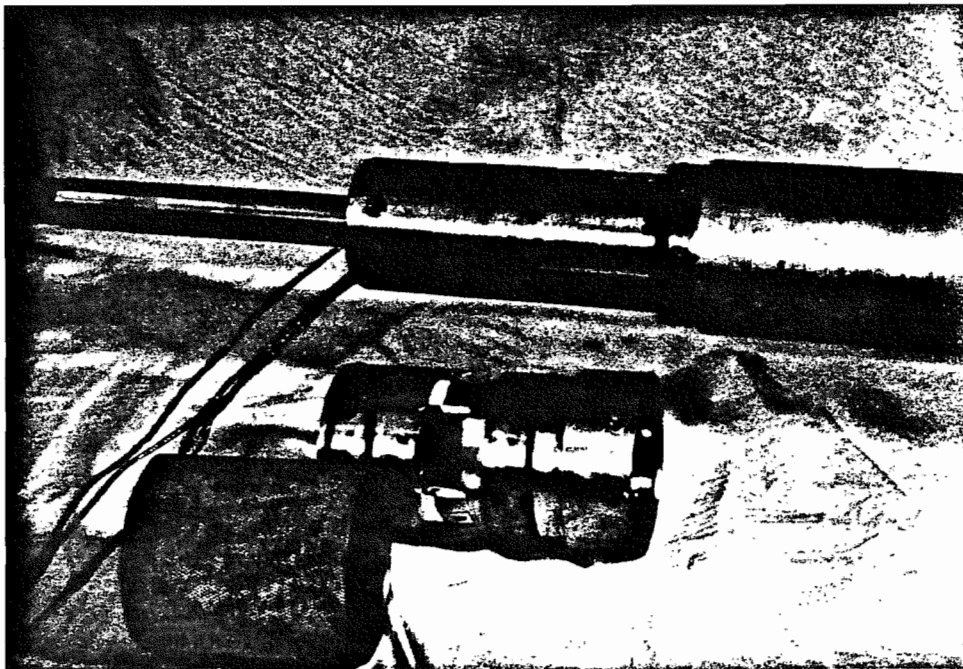


Figure 3.15 An IMP Load Cell.

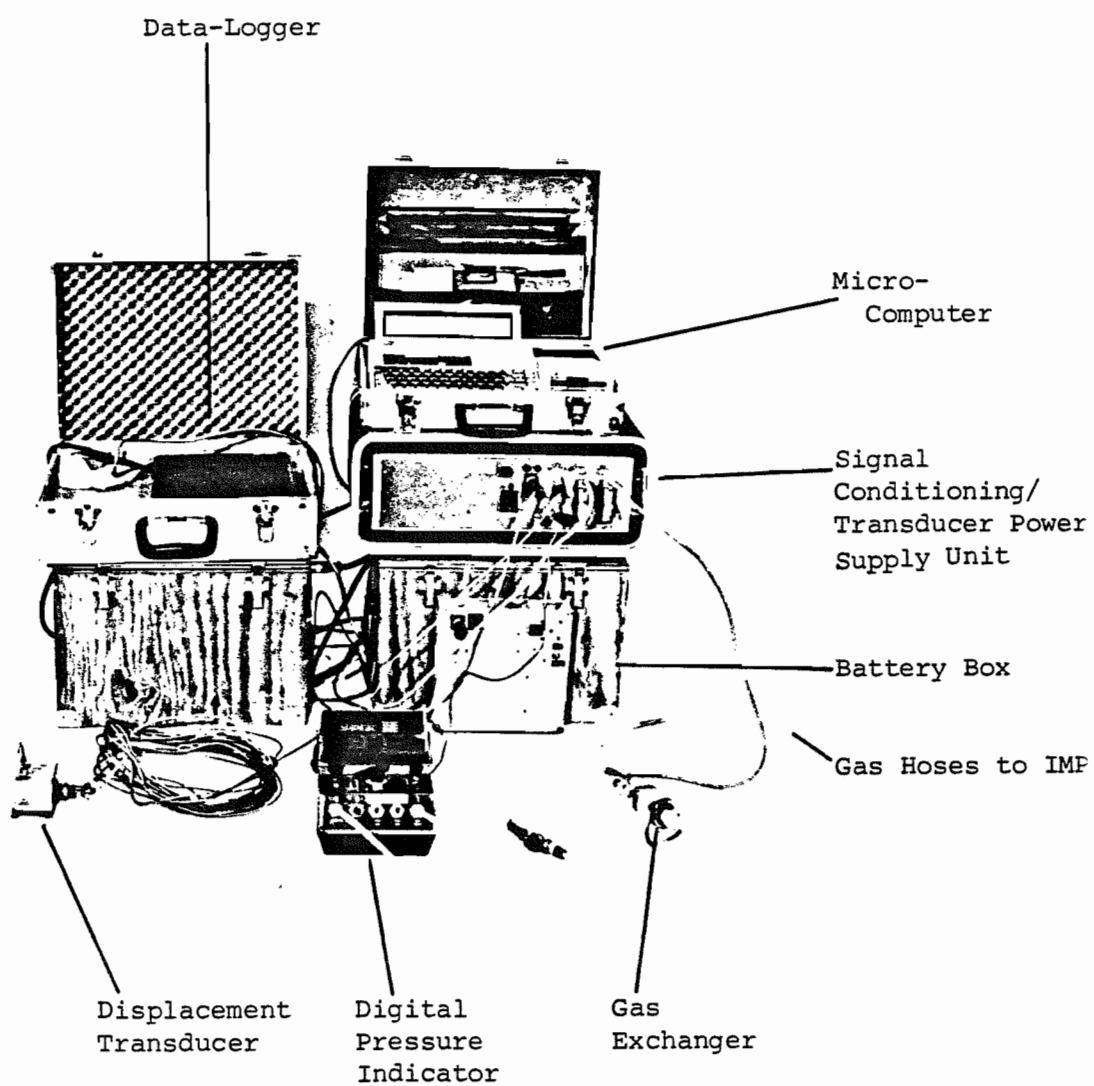


Figure 3.16 Data Aquisition System.

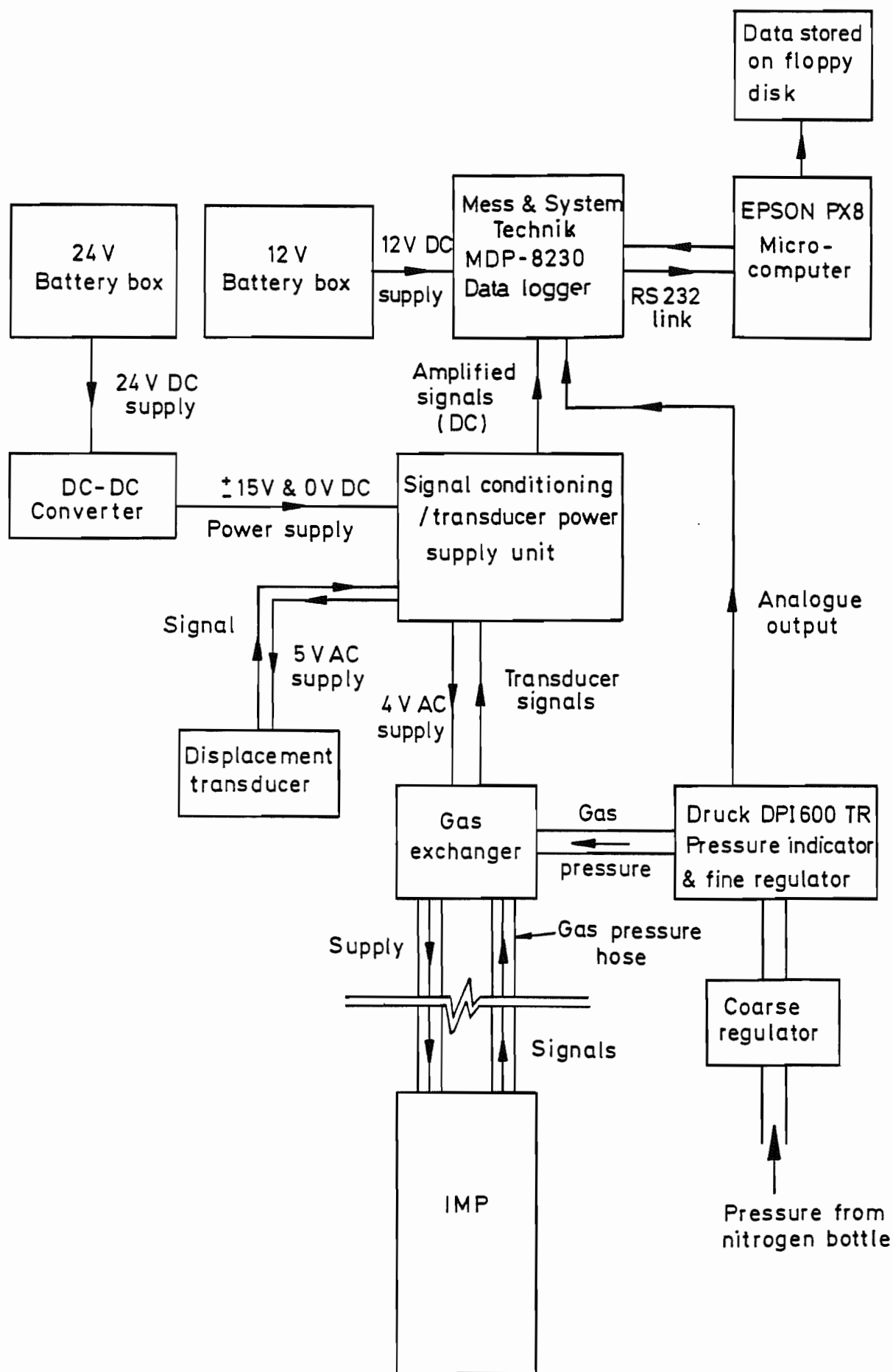


Figure 3.17 Schematic of Data Aquisition System.

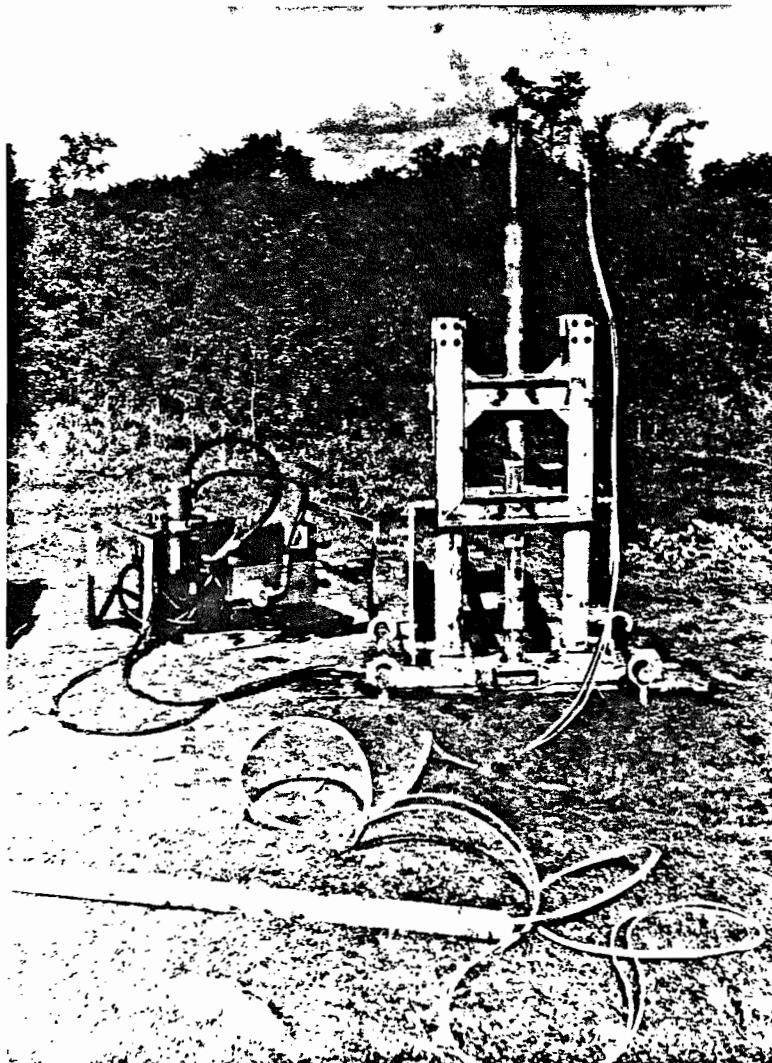


Figure 3.18 Jacking System Used in the Field.

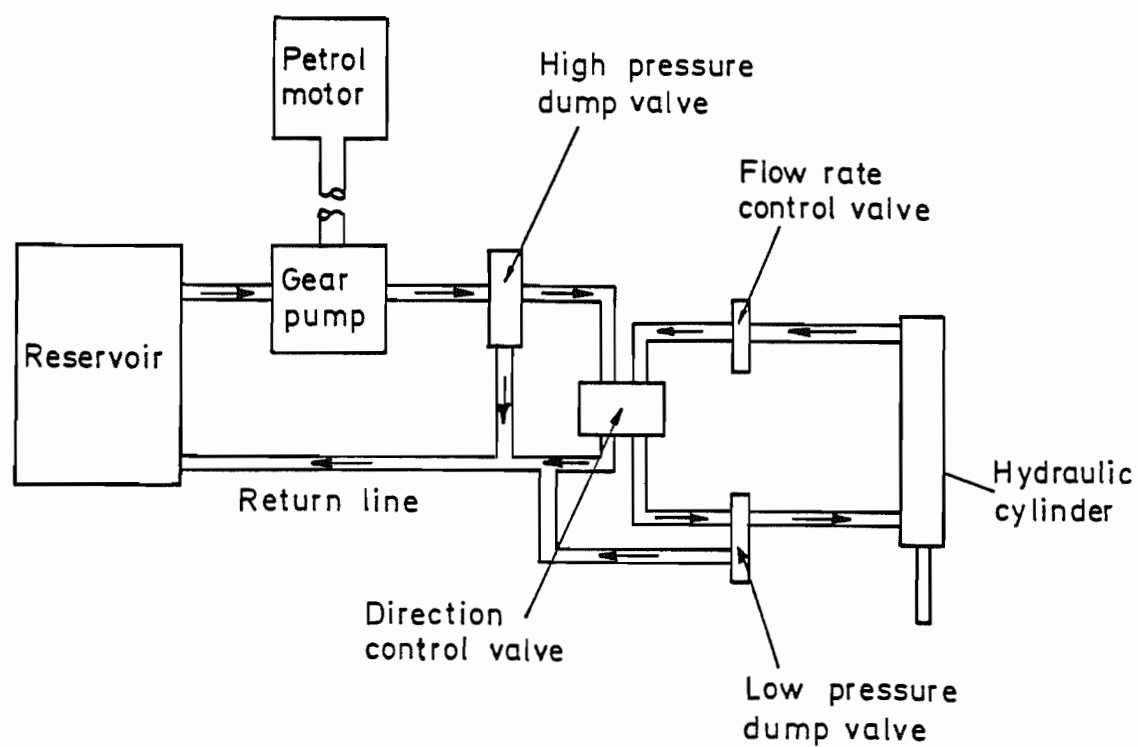


Figure 3.19 Schematic Layout of Hydraulic Power Pack.

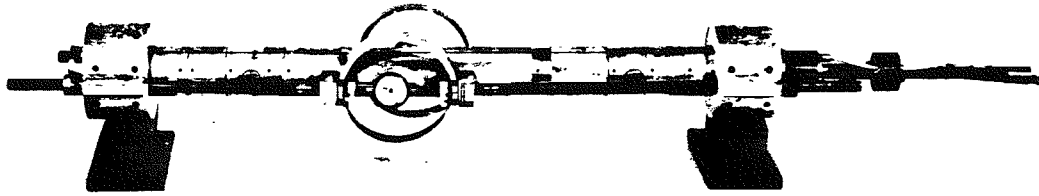


Figure 3.20 Axial Load Calibration Rig.

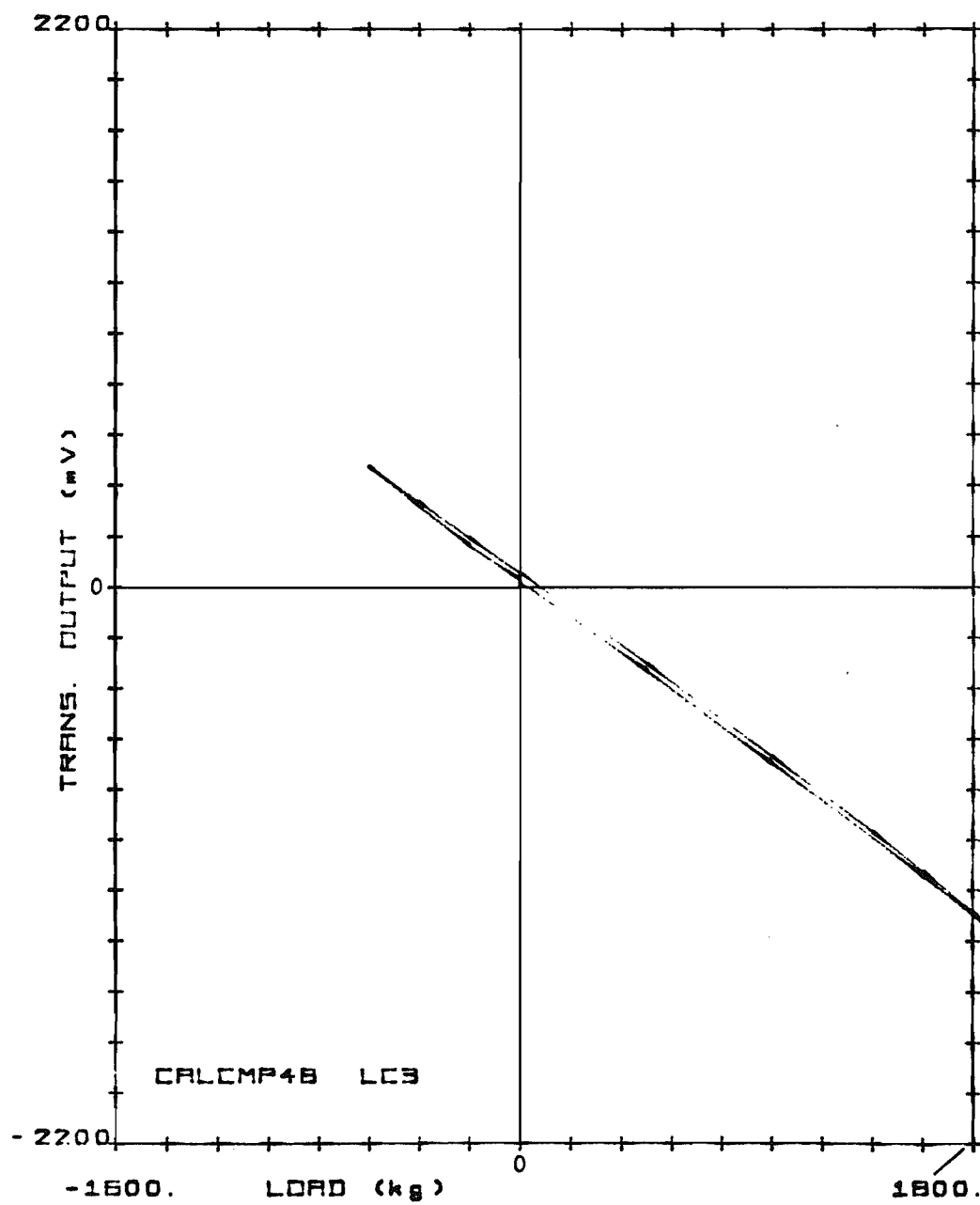


Figure 3.21 An Axial Load Cell Calibration Curve (Amplified Output)

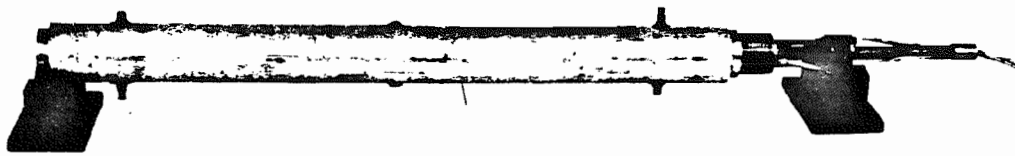


Figure 3.22 The IMP in its De-Airing/Calibration Cylinder.

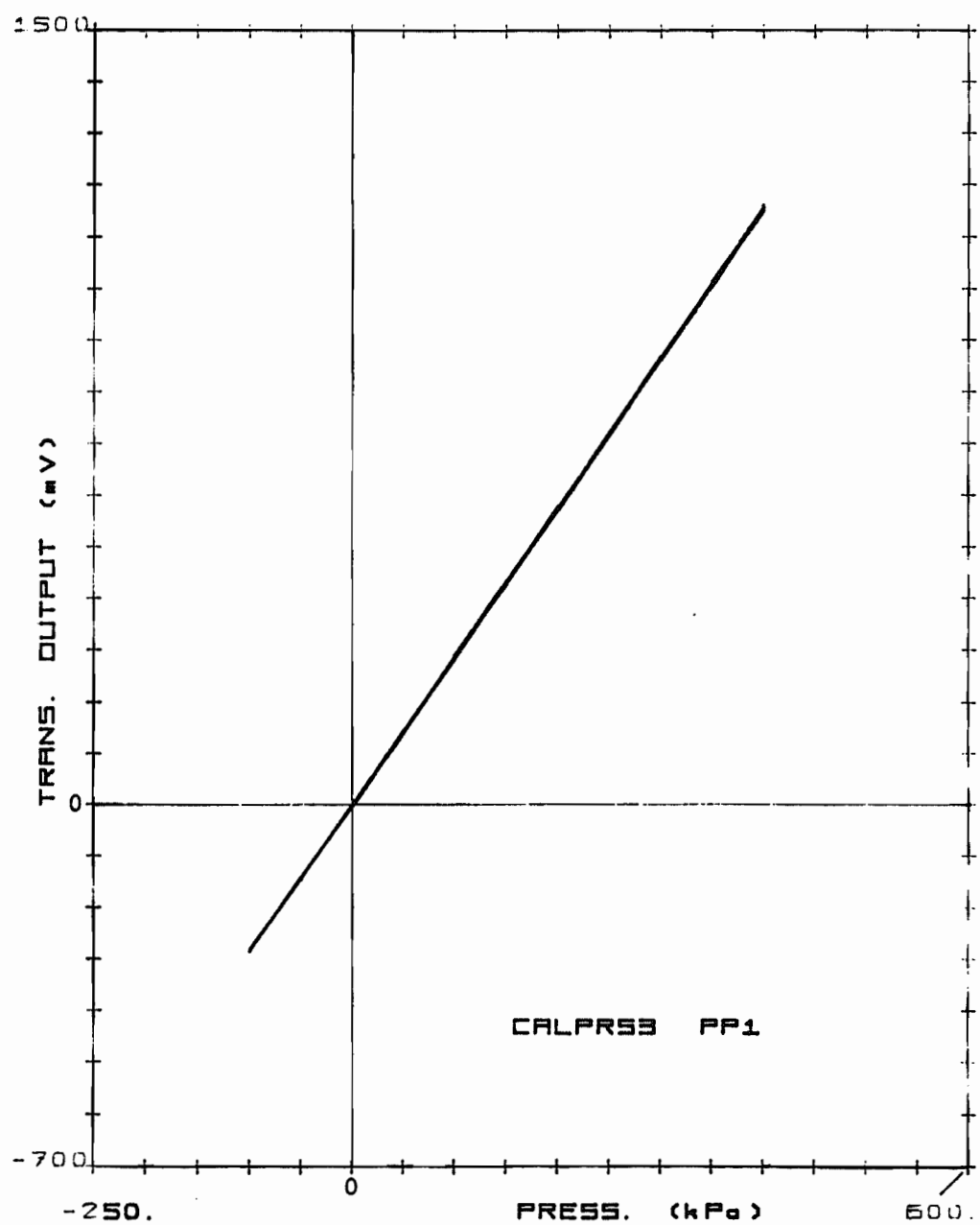


Figure 3.23 Druck PDCR-81 Calibration Curve (Amplified Output).

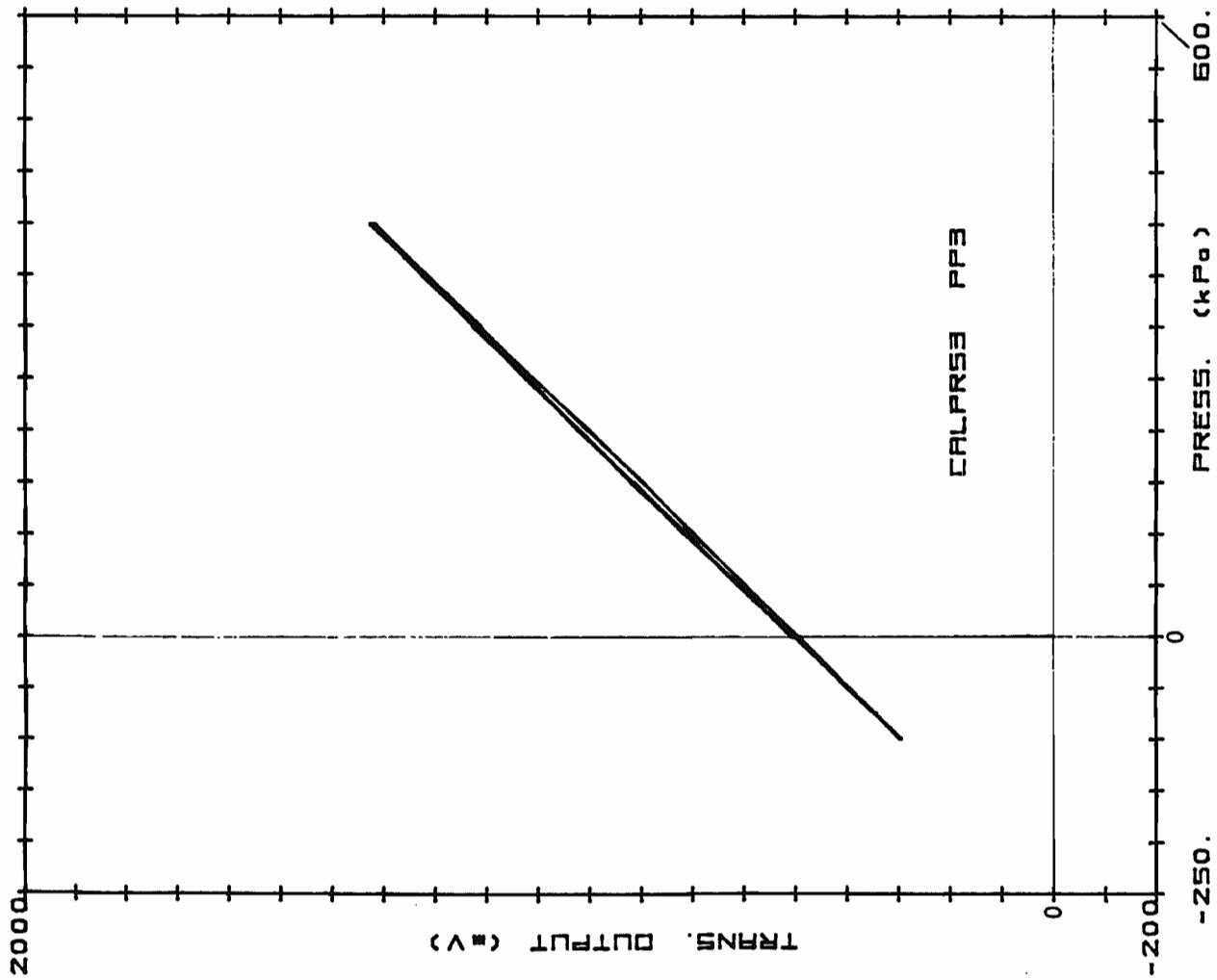


Figure 3.24 Strain-Gauged Pore Pressure Transducer Calibration Curve (Amplified Output).

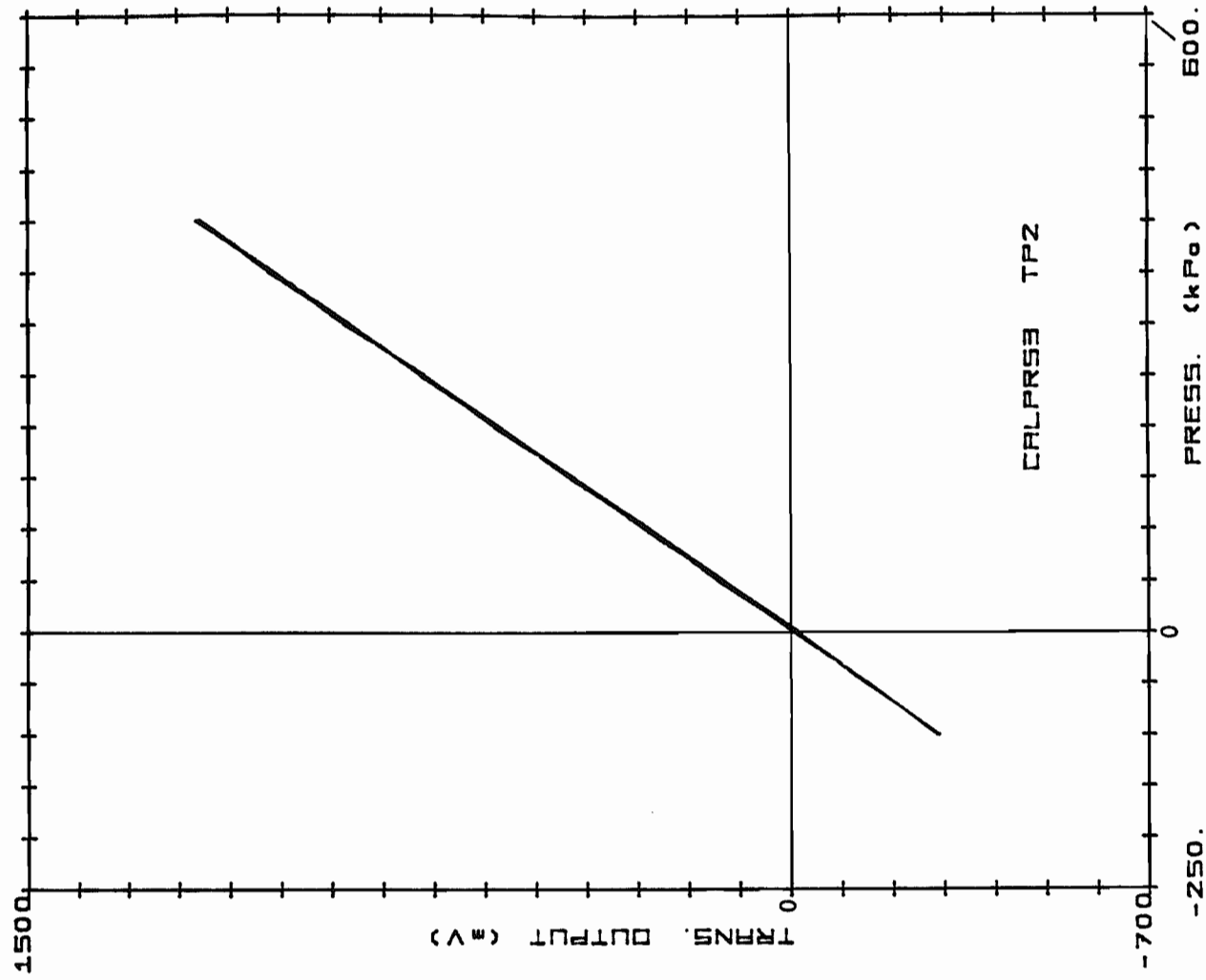
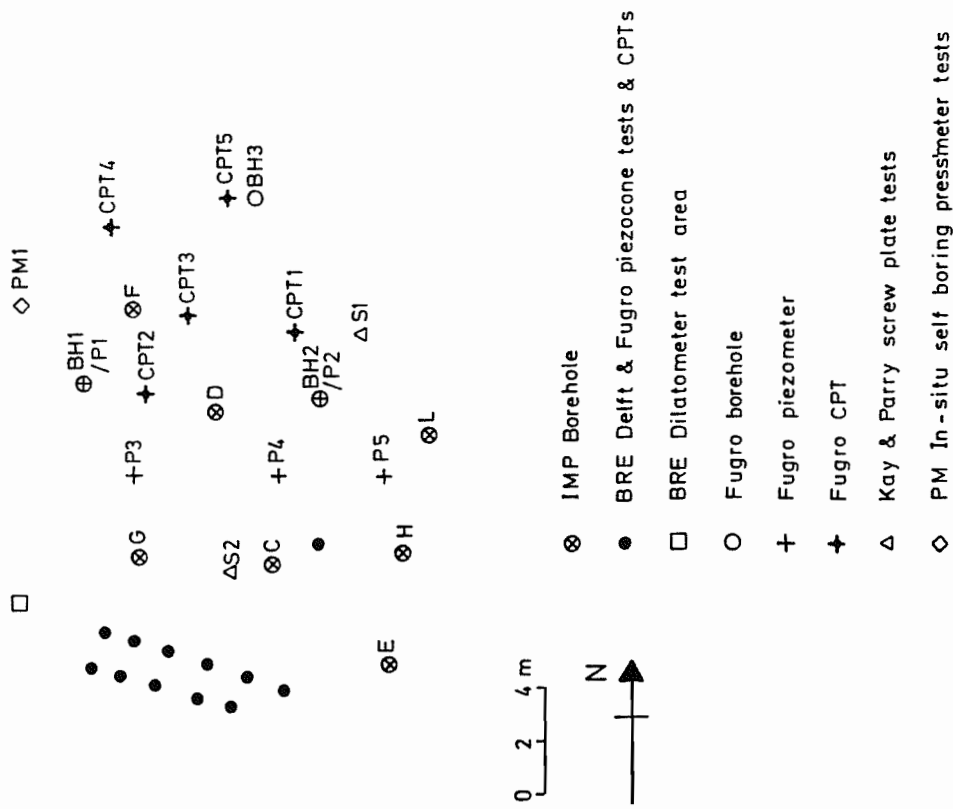


Figure 3.25 Total Radial Stress Transducer Calibration Curve (Amplified Output).



Figure 3.26 Augering the Starter Borehole at Madingley.

approx. 45m to Fugro
cone pressuremeter
tests & 60m to Clegg's
model pile tests



BOREHOLE	DATE	COMMENTS	PROBLEMS
C	31.7.85		LC3 overstressed at end of borehole -zero shift on extraction
D	2.8.85		Abandoned after approx. 1.5m Data not presented
E	3.9.85		LC3 overstressed at end of borehole -zero shift on extraction
F	4-6.9.85	Full reconsolidation at 4.17m tip penetration	PP4 zero shift during reconsolidation (see text) PP5 poor zero on extraction - blocked filter? TP3 zero shift due to overstraining near end of borehole
G	25.11- 12.12.85	Full reconsolidation at 5.79m tip penetration	LC3 overstressed at end of borehole -zero shift on extraction
H	13-14.12.85		
L	29.5.86	Partial reconsolidation at 8.11m tip penetration	LC3 progressively overstressed throughout borehole PP4 sudden zero change upon extraction from borehole

Table 4.1 Summary of IMP Site Investigation at Madingley

Figure 4.1 Plan of the Madingley Site.

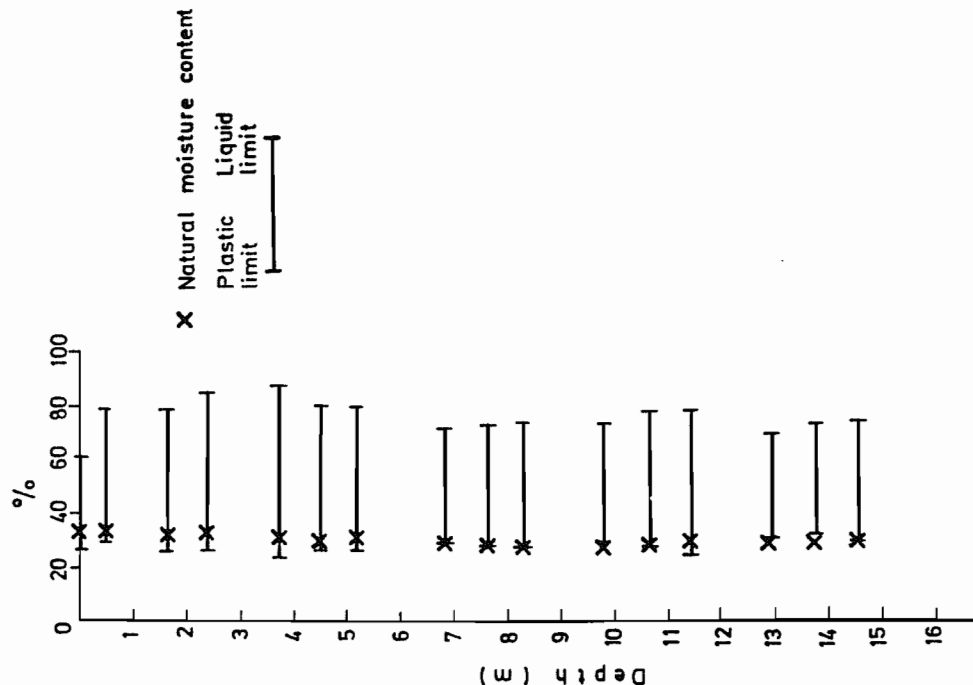
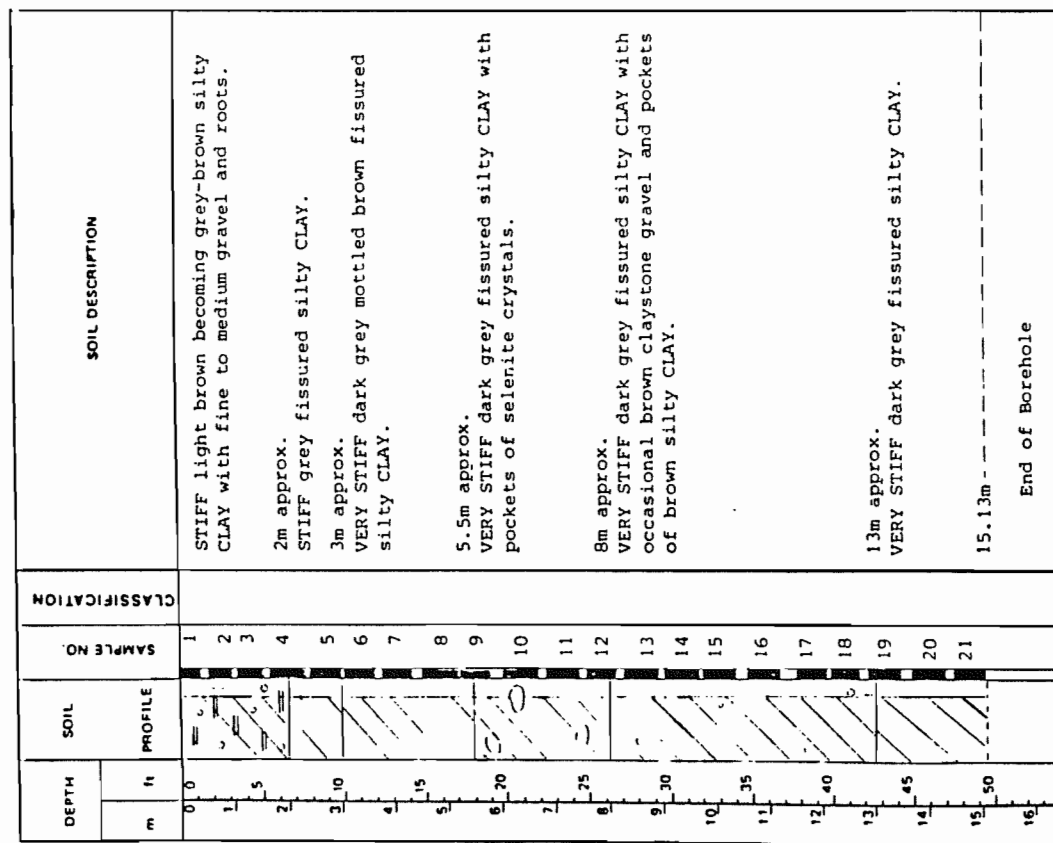


Figure 4.2 Sample Descriptions for Madingley (after Fugro 1979) and Atterberg Limits (redrawn from Fugro data).

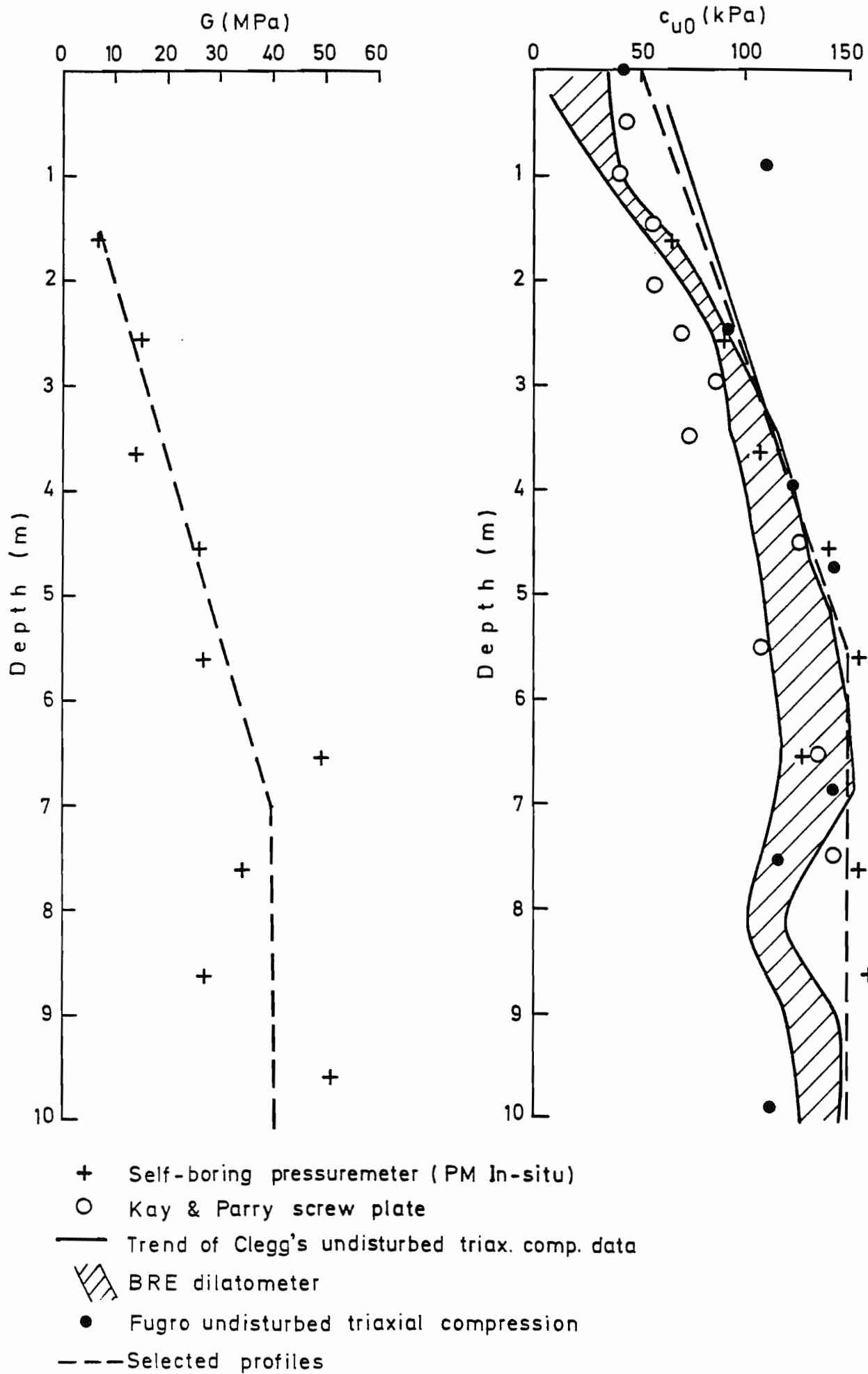


Figure 4.3 Madingley Soil Strength and Shear Modulus Data.

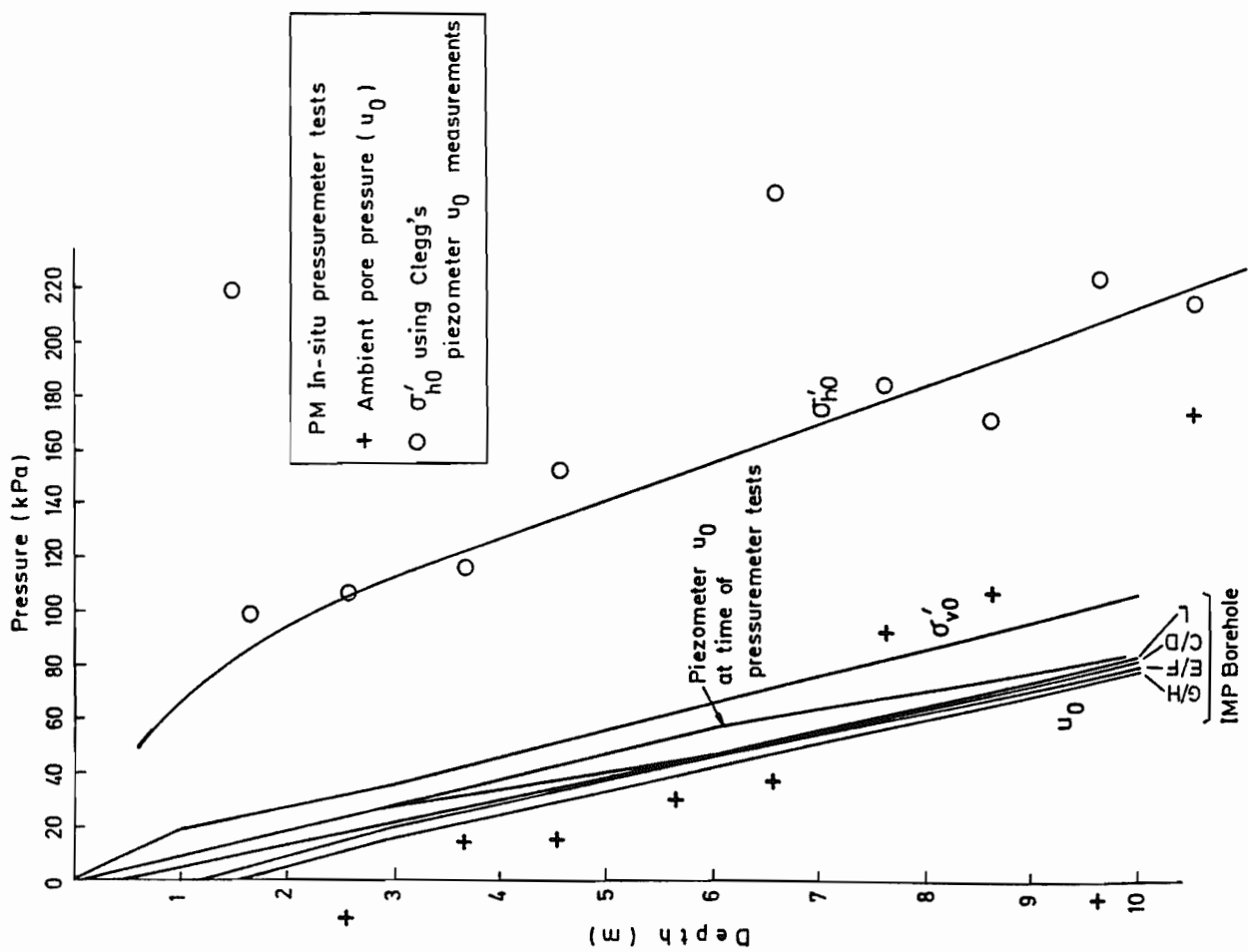


Figure 1.4 In-Situ Stresses at Madinlay

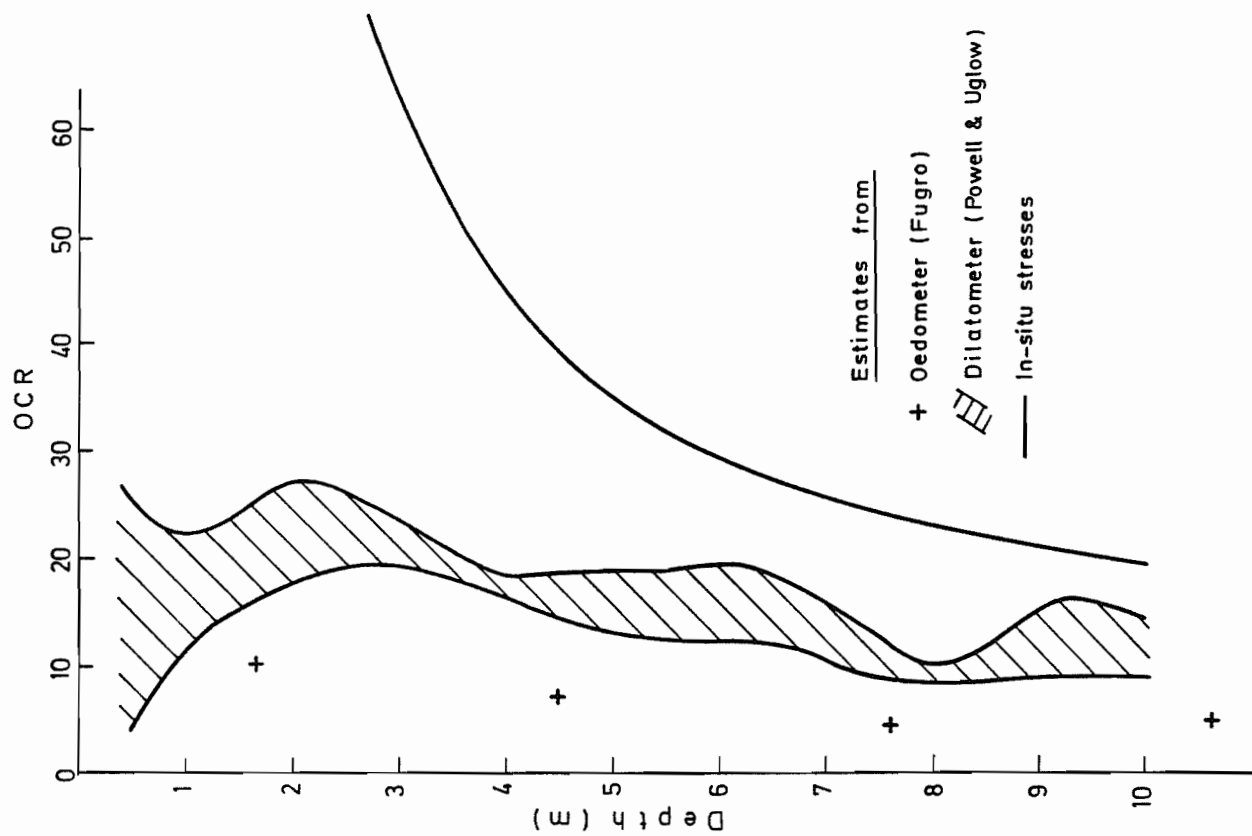


Figure 1.5 Estimates of OCR at Madinlay

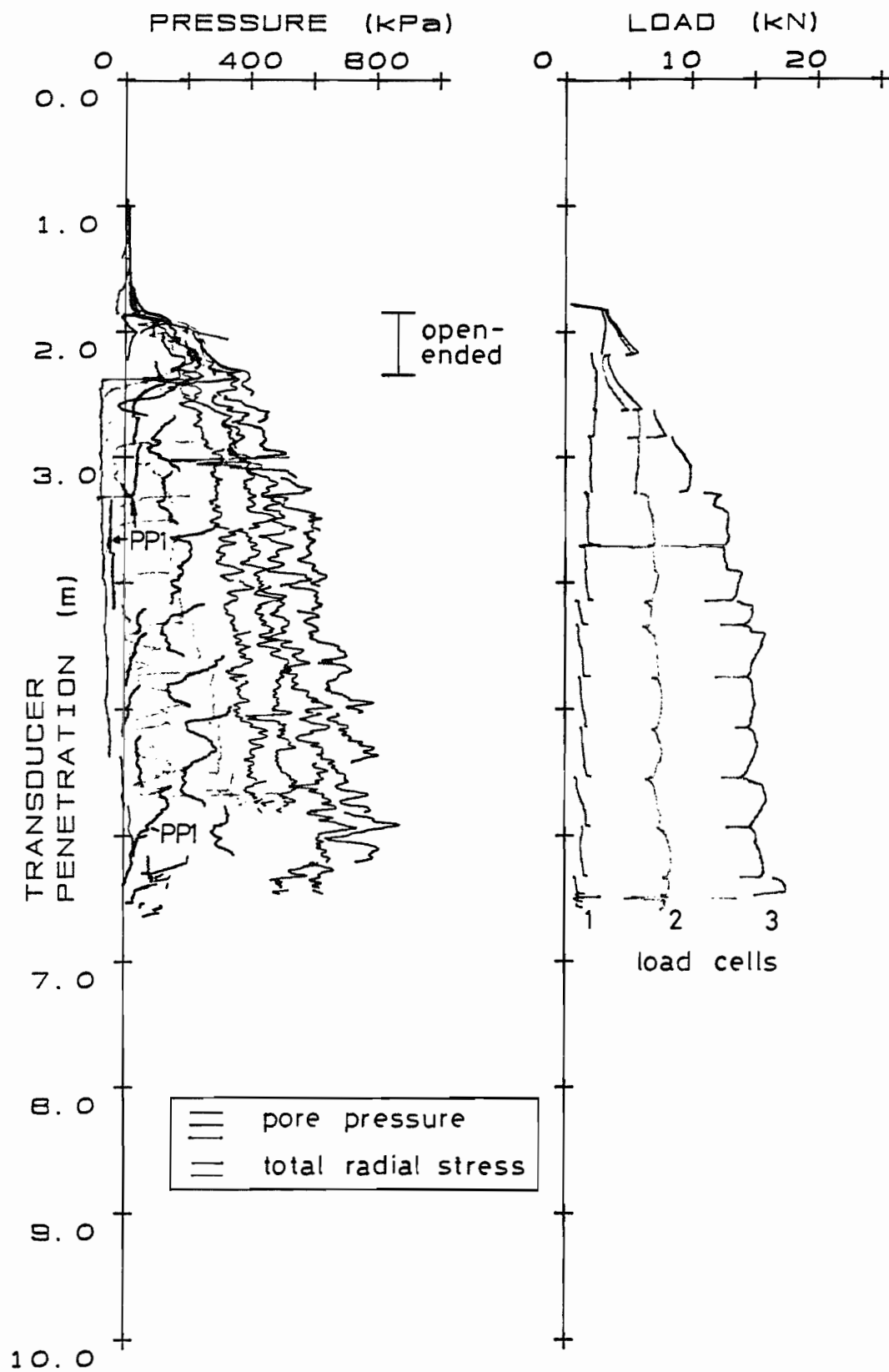


Figure 4.6 Raw Data Obtained During IMP Penetration in Borehole C, Madingley. (Refer to Fig.3.7 for key)

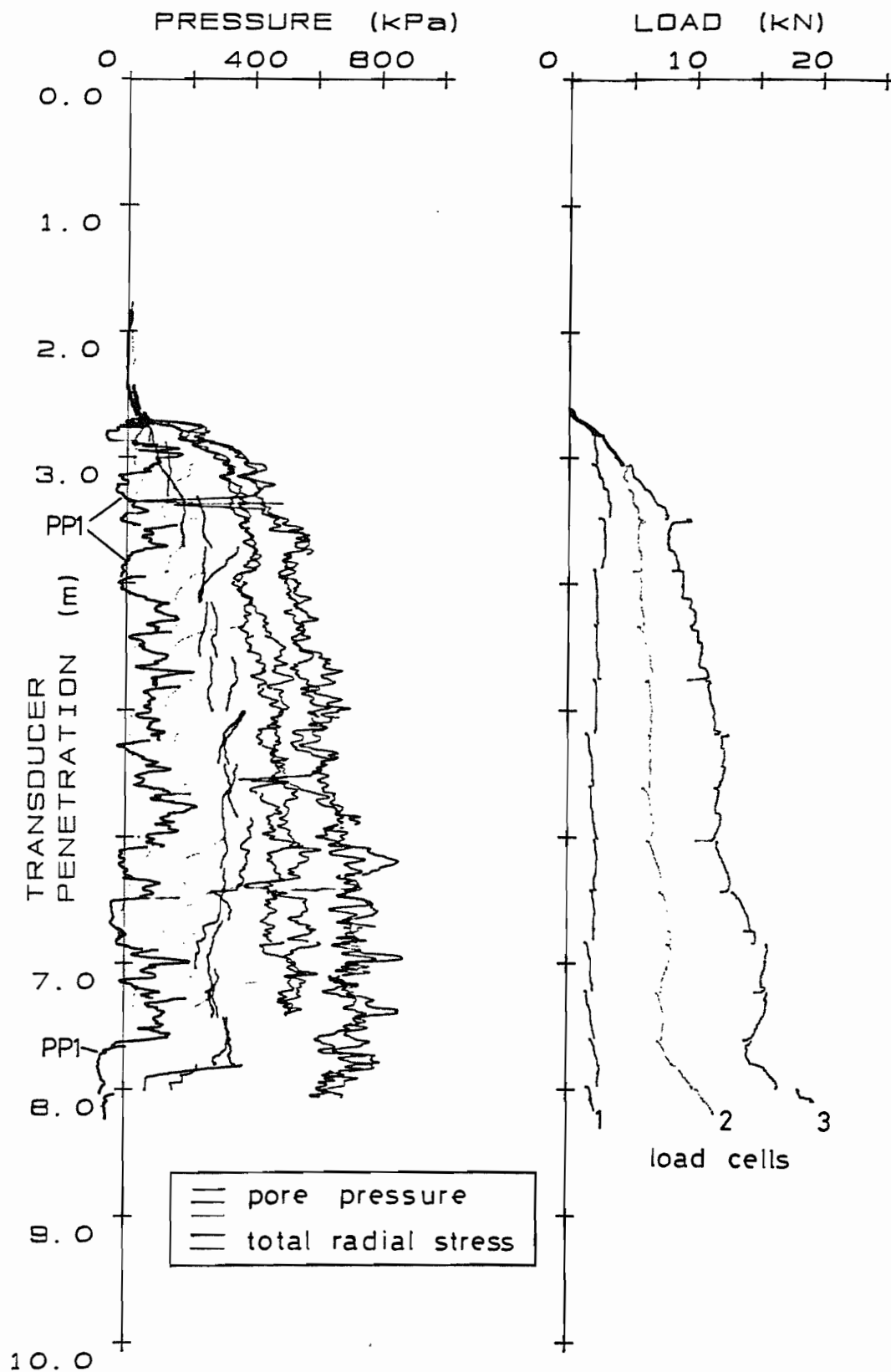


Figure 4.7 Raw Data Obtained During IMP Penetration in Borehole E, Madingley. (Refer to Fig.3.7 for key)

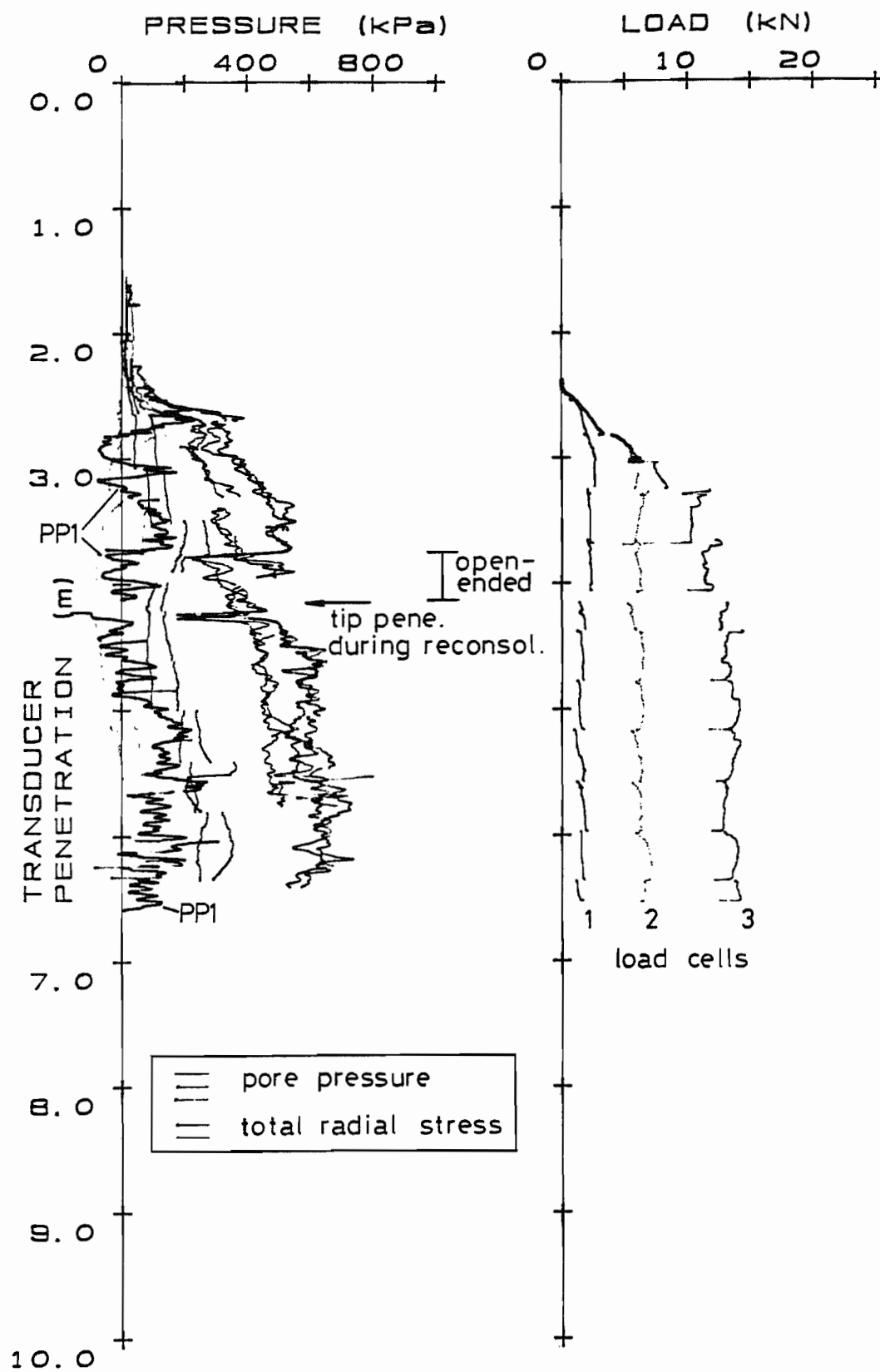


Figure 4.8 Raw Data Obtained During IMP Penetration in Borehole F, Madingley. (Refer to Fig.3.7 for key)

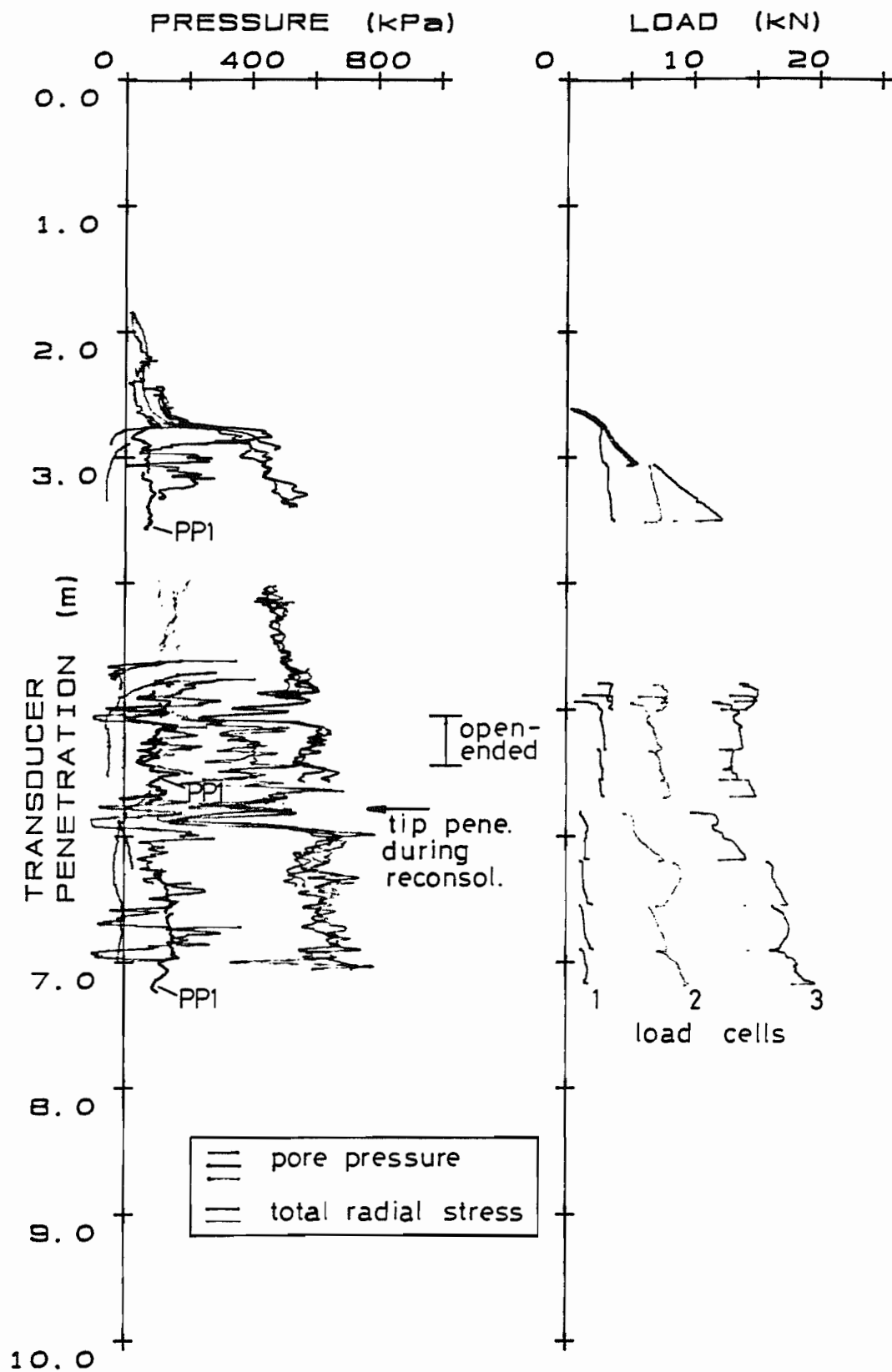


Figure 4.9 Raw Data Obtained During IMP Penetration in Borehole G, Madingley. (Refer to Fig.3.8 for key)

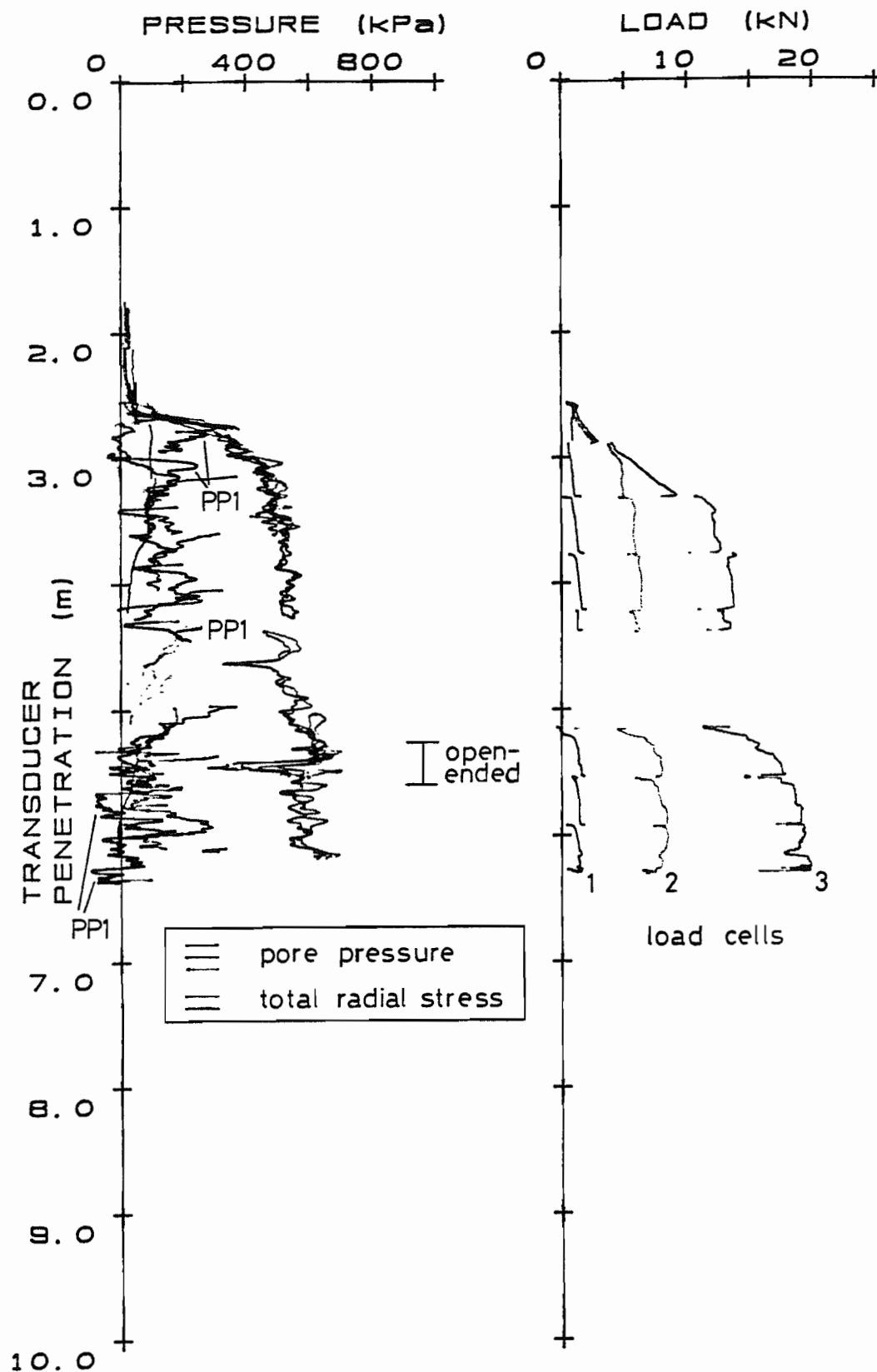


Figure 4.10 Raw Data Obtained During IMP Penetration in Borehole H, Madingley. (Refer to Fig.3.8 for key)

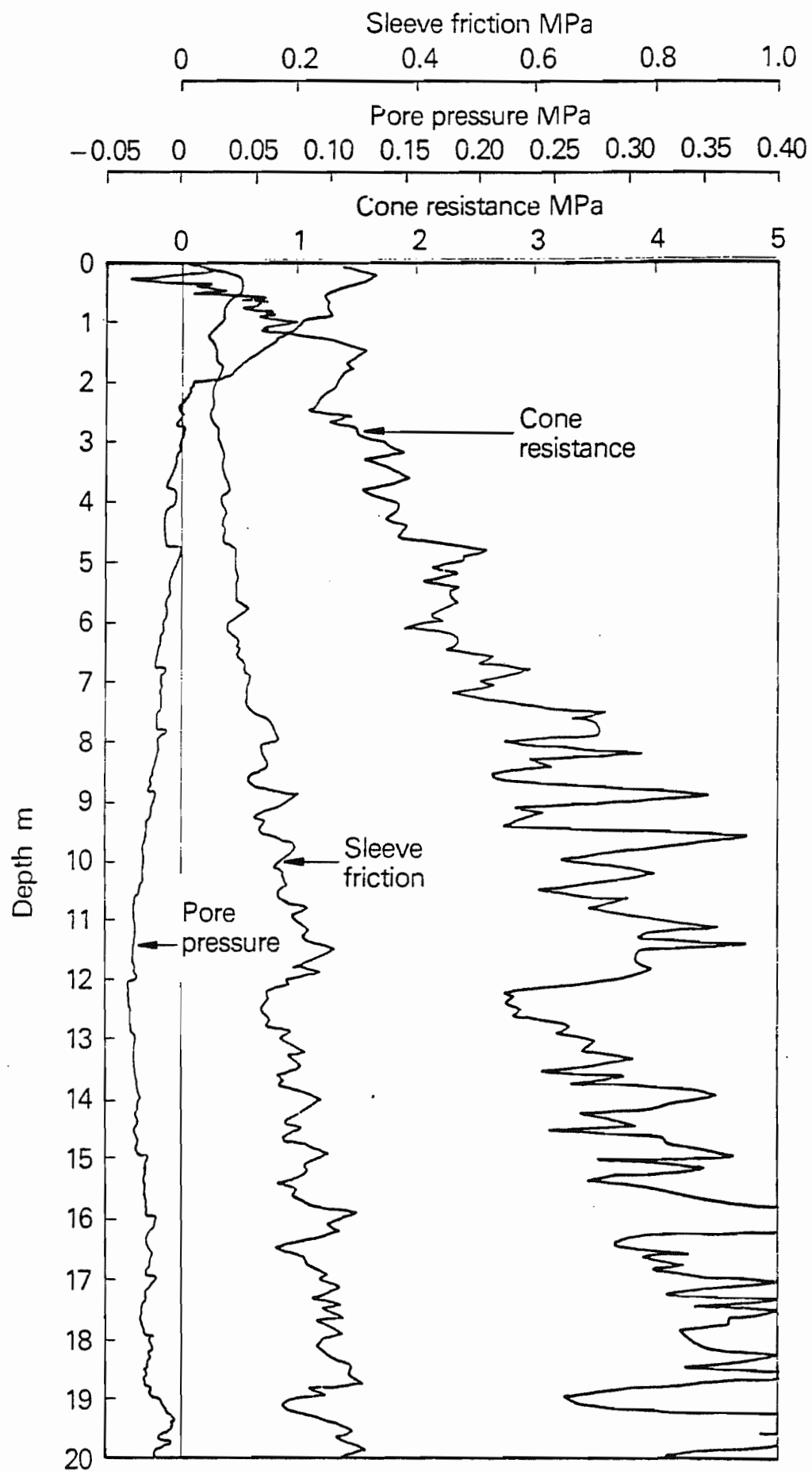


Figure 4.11 DSML Piezocone Data from Madingley (after Lunne et al.)

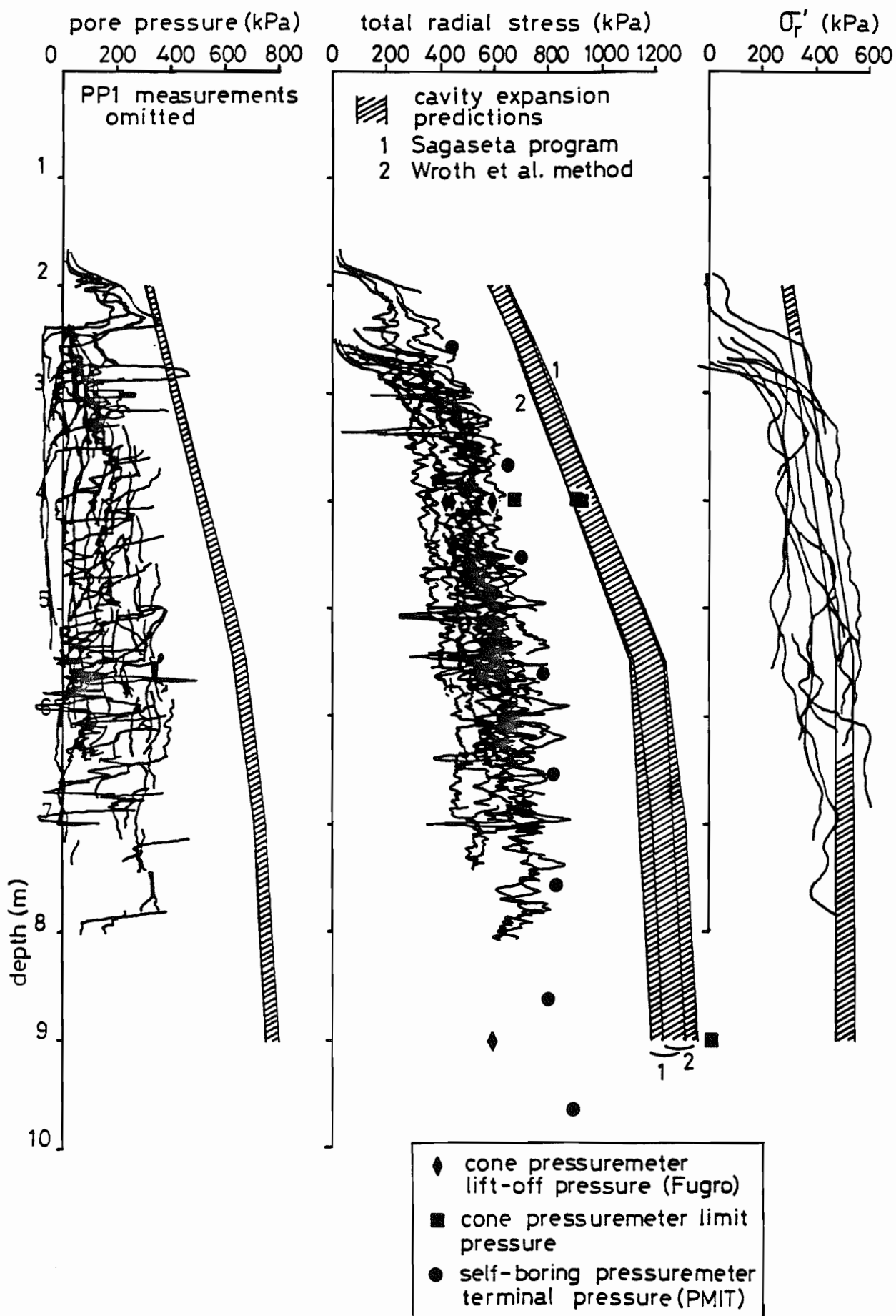


Figure 4.12 Comparison of Madingley IMP Penetration Data with Cylindrical Cavity Expansion Predictions and other Site Investigation Data.

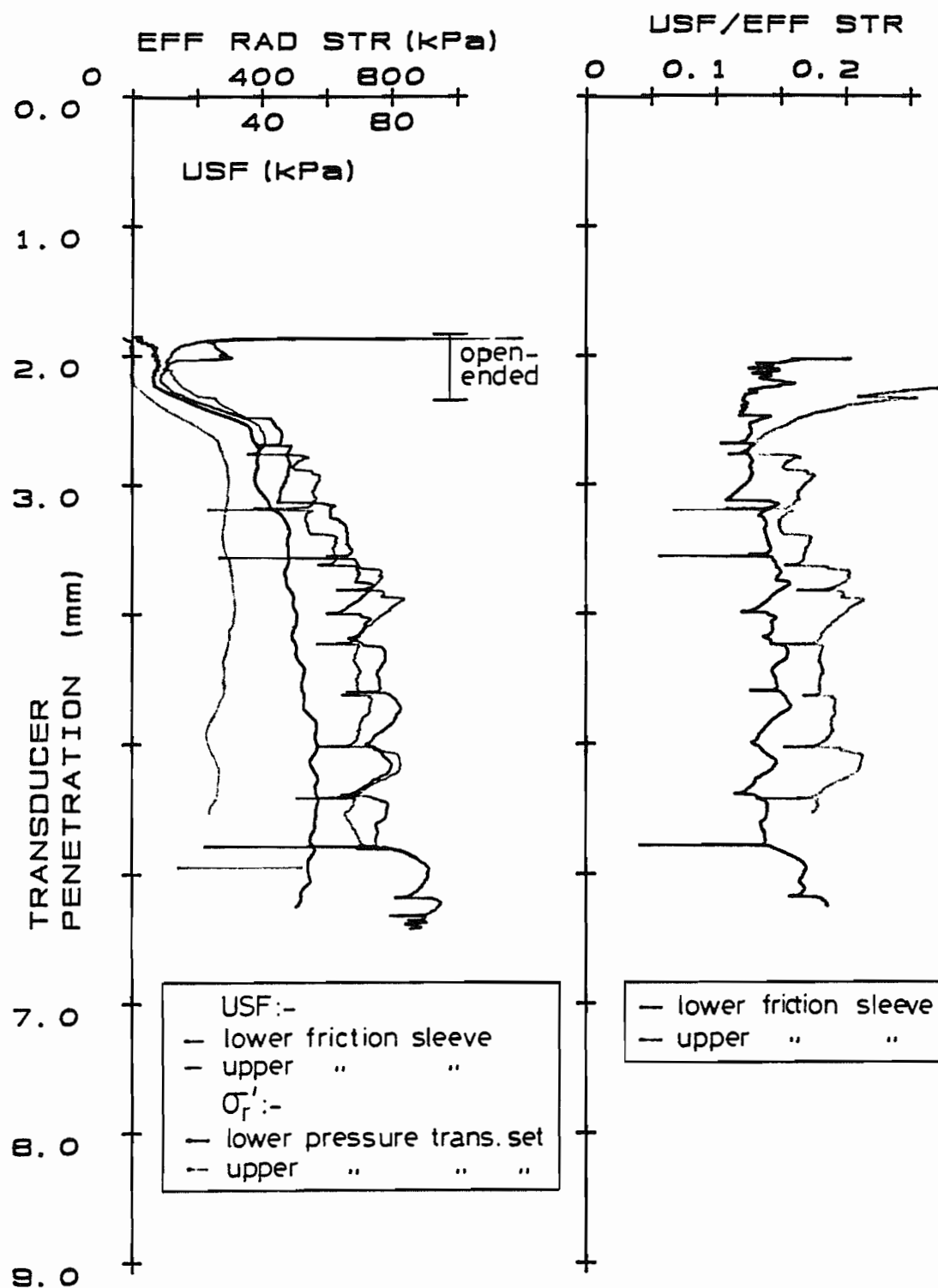


Figure 4.13 Interpreted Data from IMP Penetration in Borehole C, Madingley.

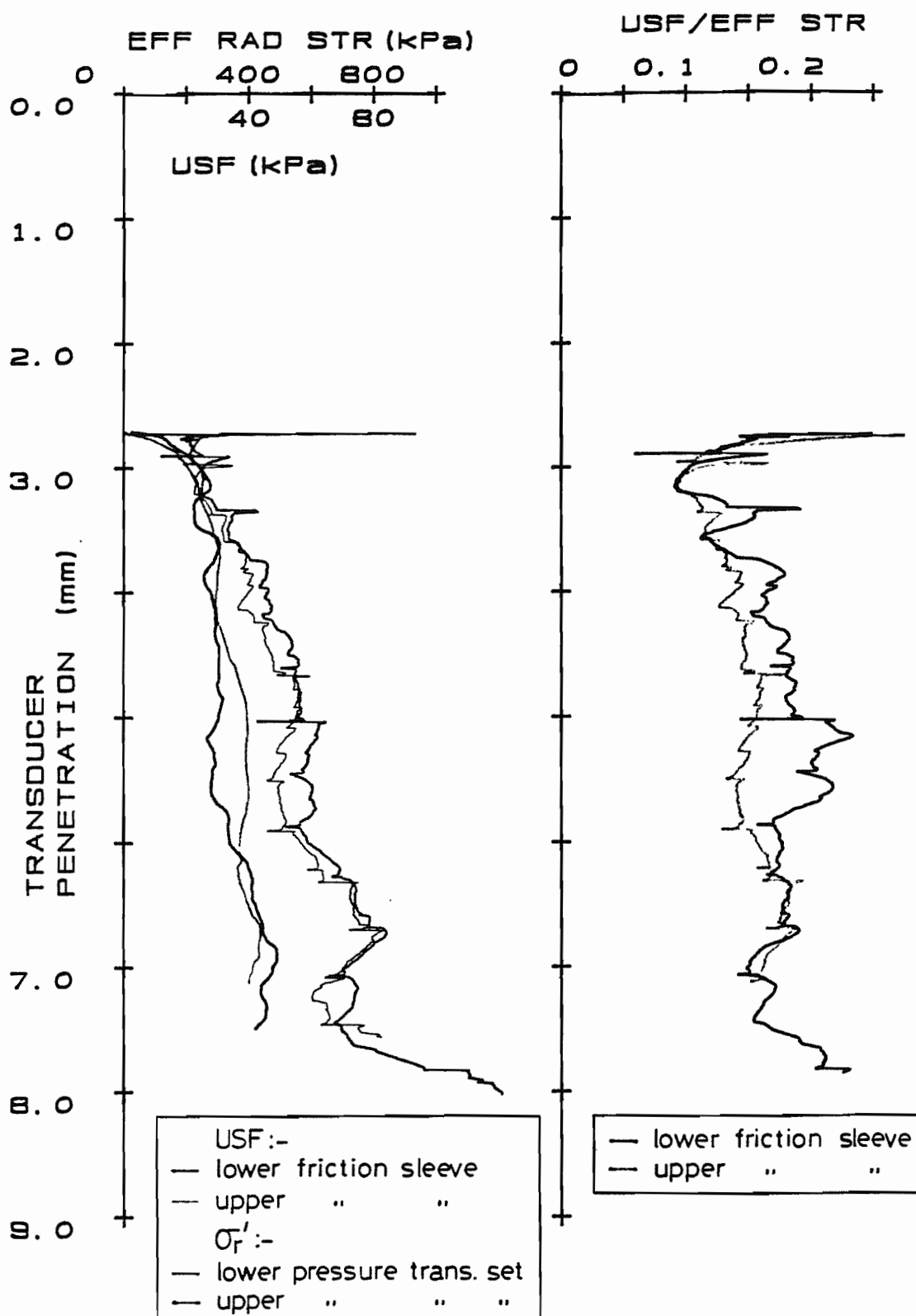


Figure 4.14 Interpreted Data from IMP Penetration in Borehole E, Madingley.

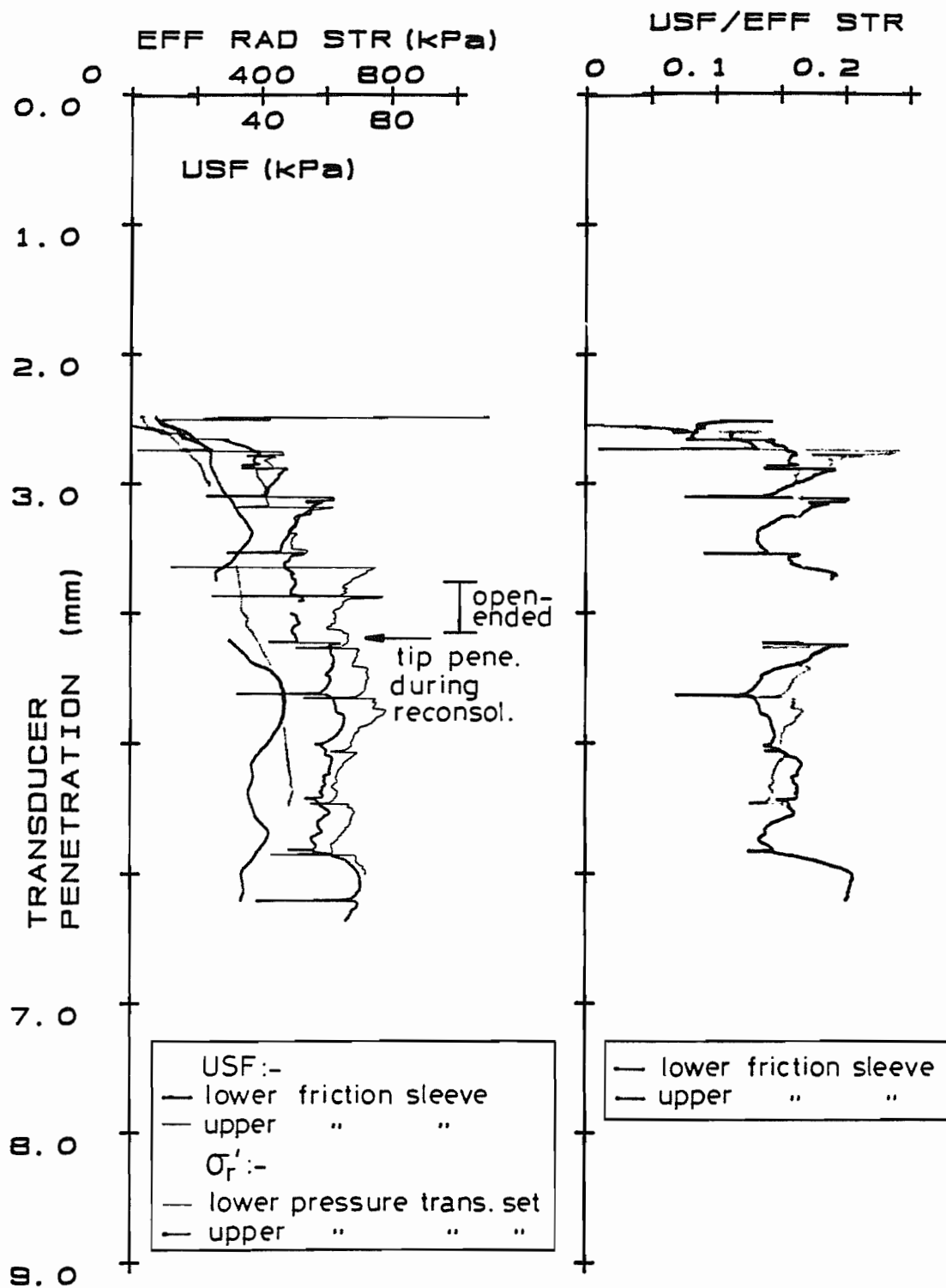


Figure 4.15 Interpreted Data from IMP Penetration in Borehole F, Madingley.

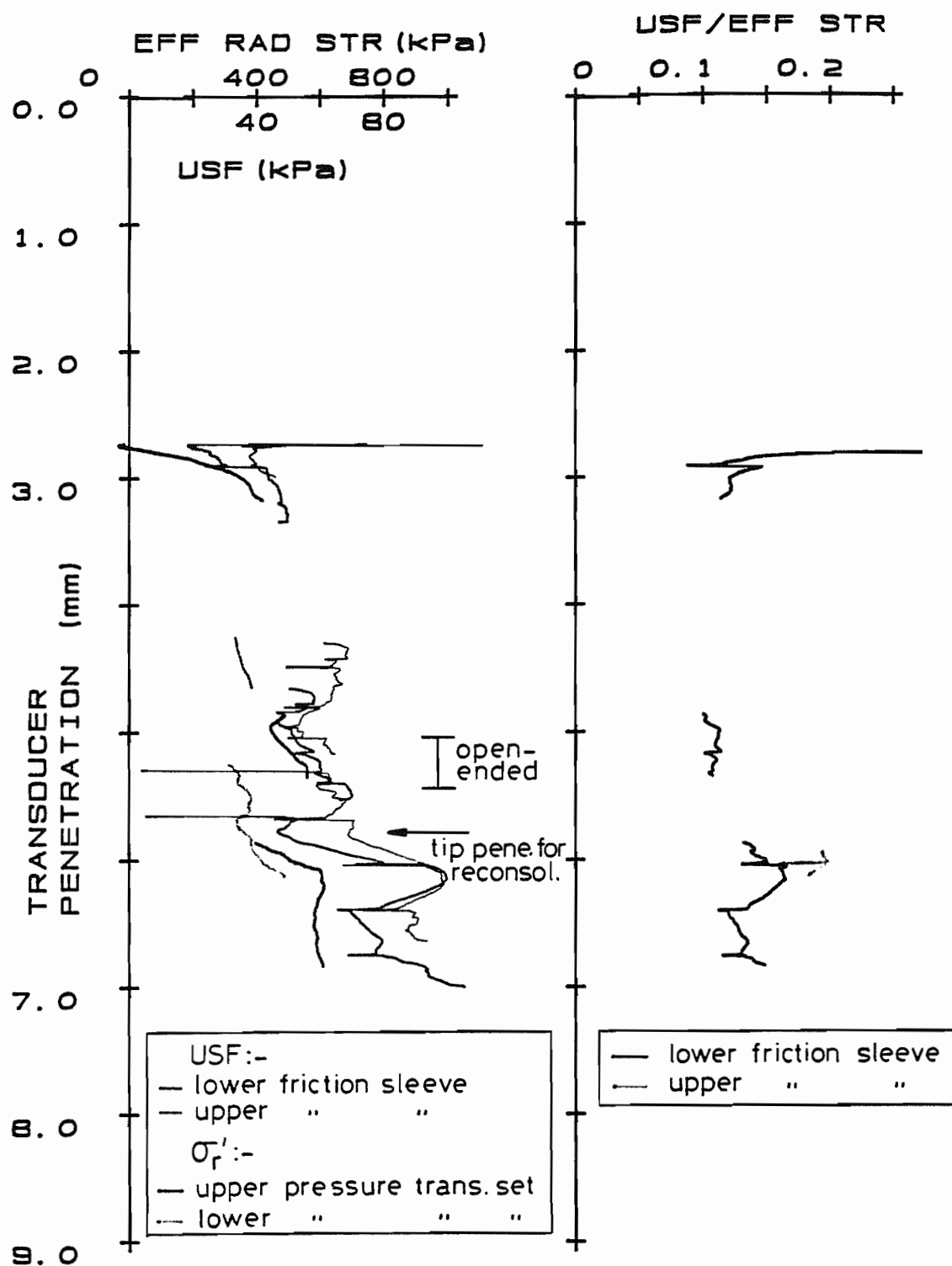


Figure 4.16 Interpreted Data from IMP Penetration in Borehole G, Madingley.

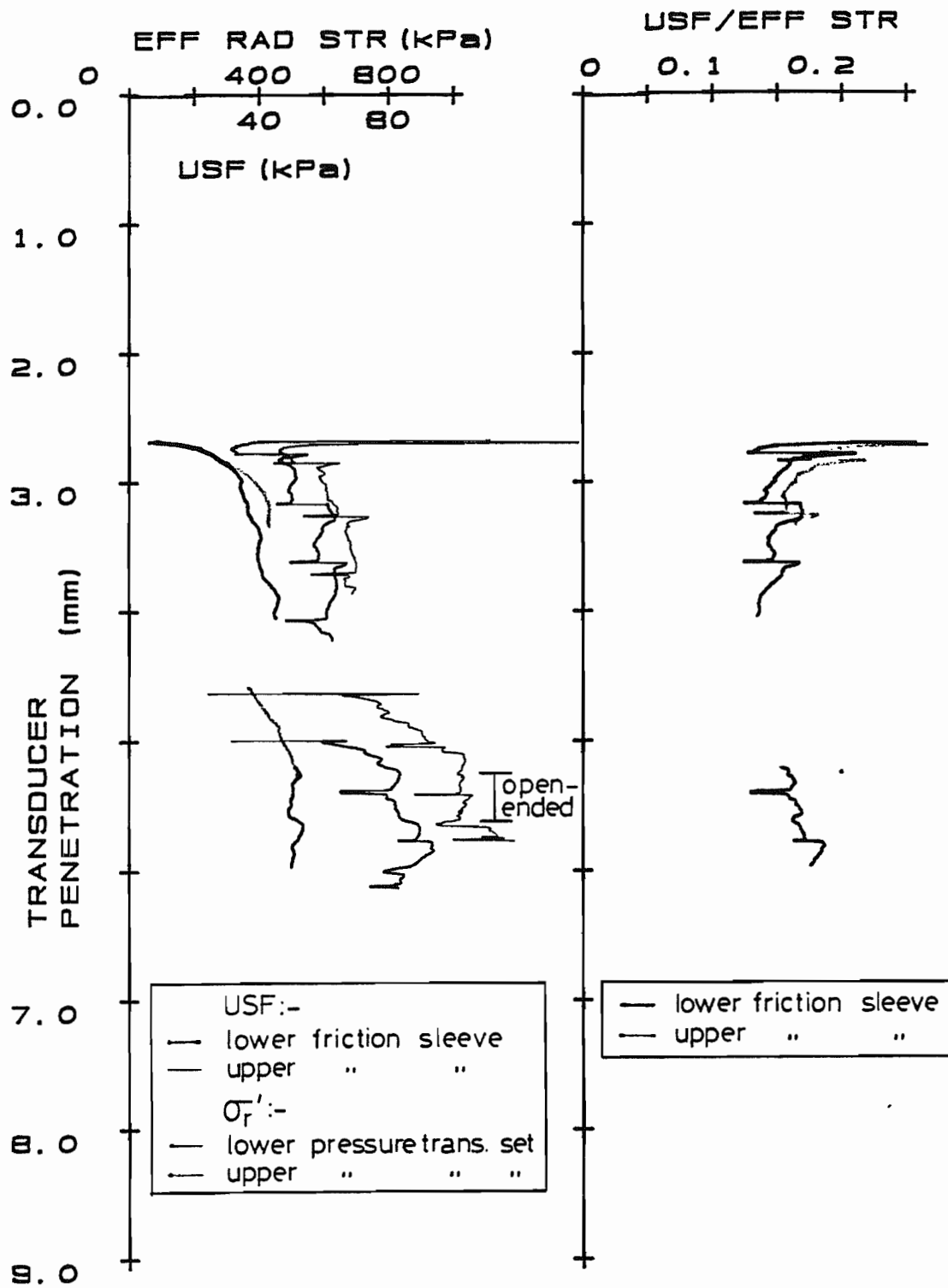


Figure 4.17 Interpreted Data from IMP Penetration in Borehole H, Madingley.

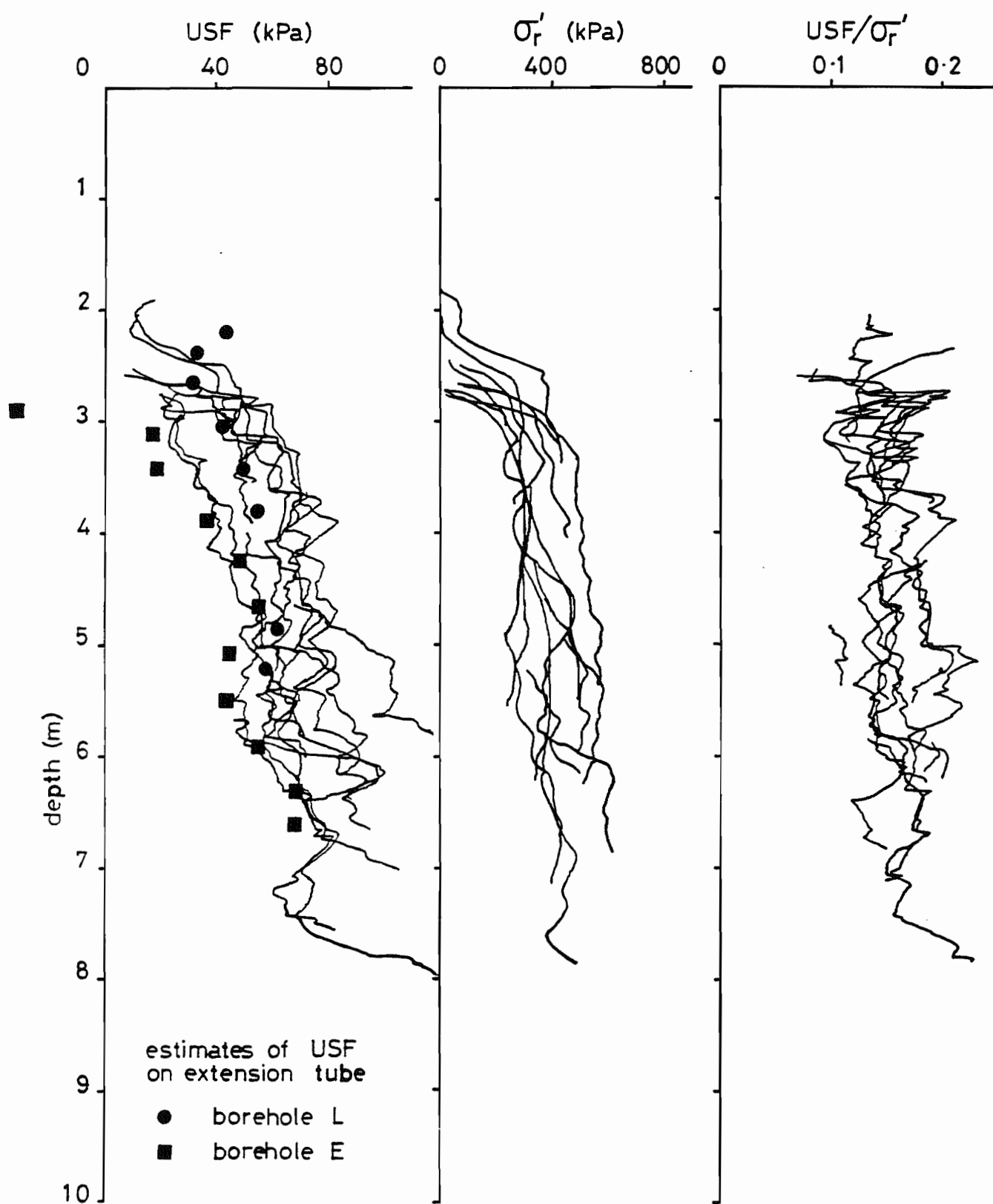


Figure 4.18 Summary of Interpreted IMP Data from Boreholes
C-H Madingley.

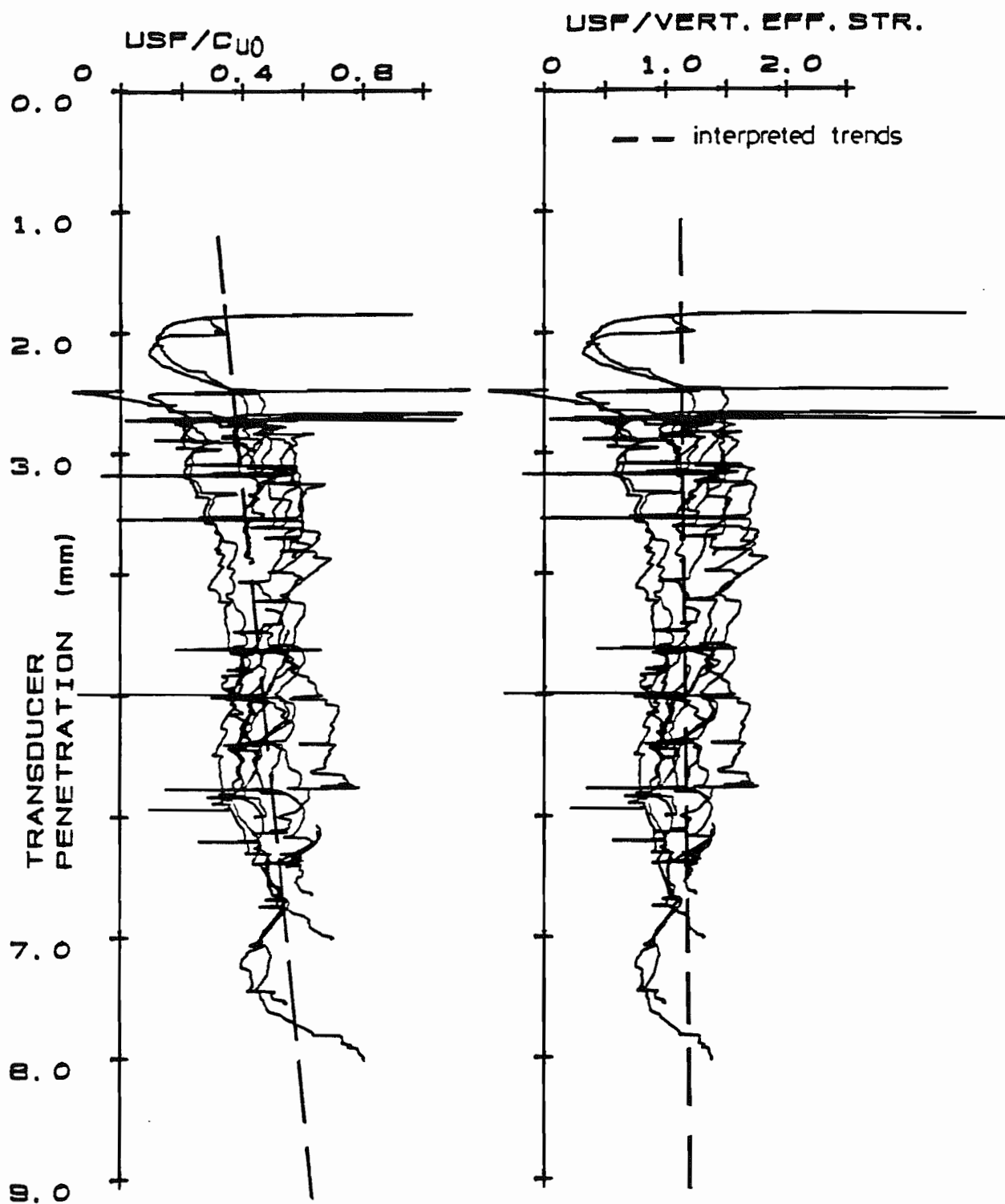


Figure 4.19 IMP Installation "α" and "β" Values for Madingley.

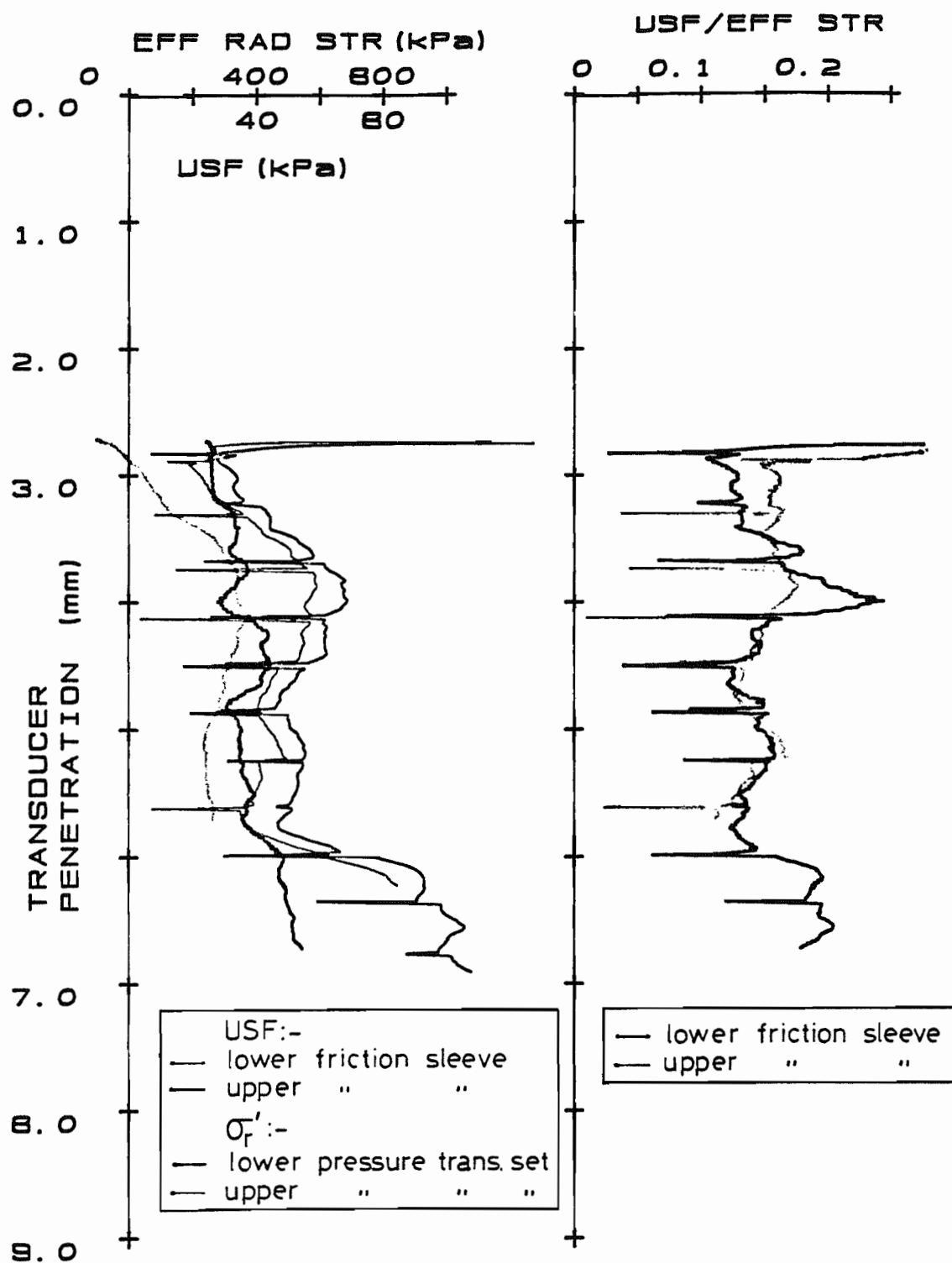


Figure 4.20 Interpreted Data for Extraction of IMP from Borehole G, Madingley.

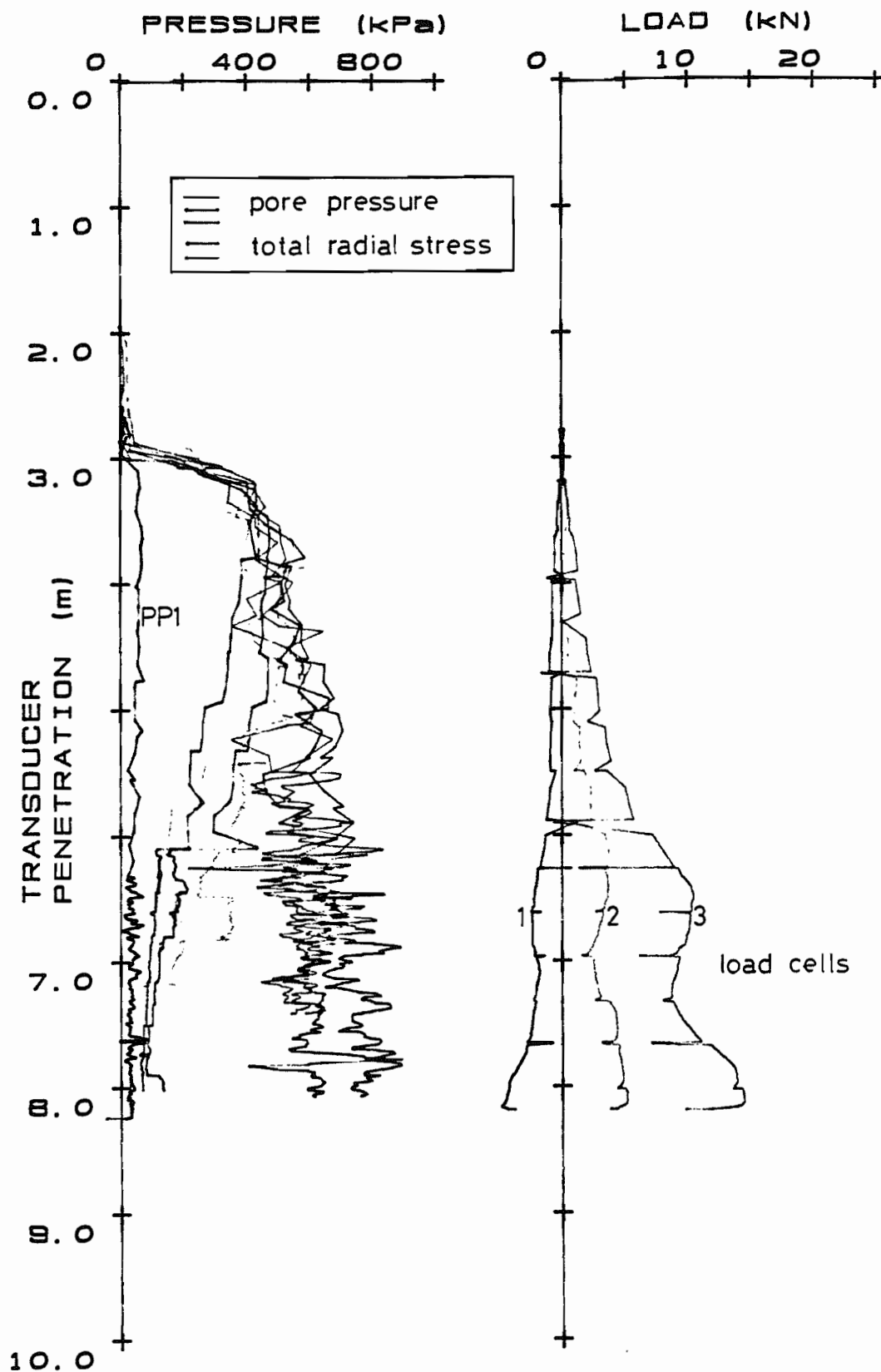


Figure 4.21 Raw Data Obtained During IMP Extraction from Borehole E, Madingley. (Refer to Fig.3.7 for Key)

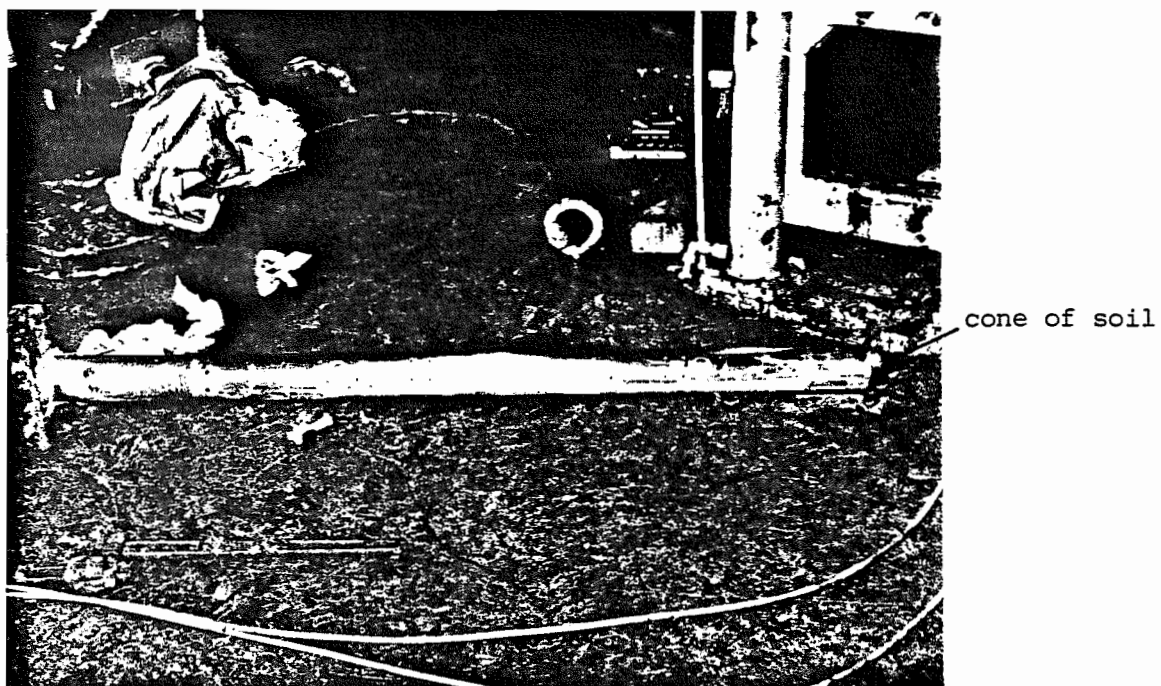


Figure 4.22 The IMP After Extraction from Borehole G.



Figure 4.23 The Extension Tube After Extraction from Borehole G.



upper, knurled sleeve
has clay coating
approx. 5mm thick.

marker pen lines

lower sleeve covered
only with thin mud.

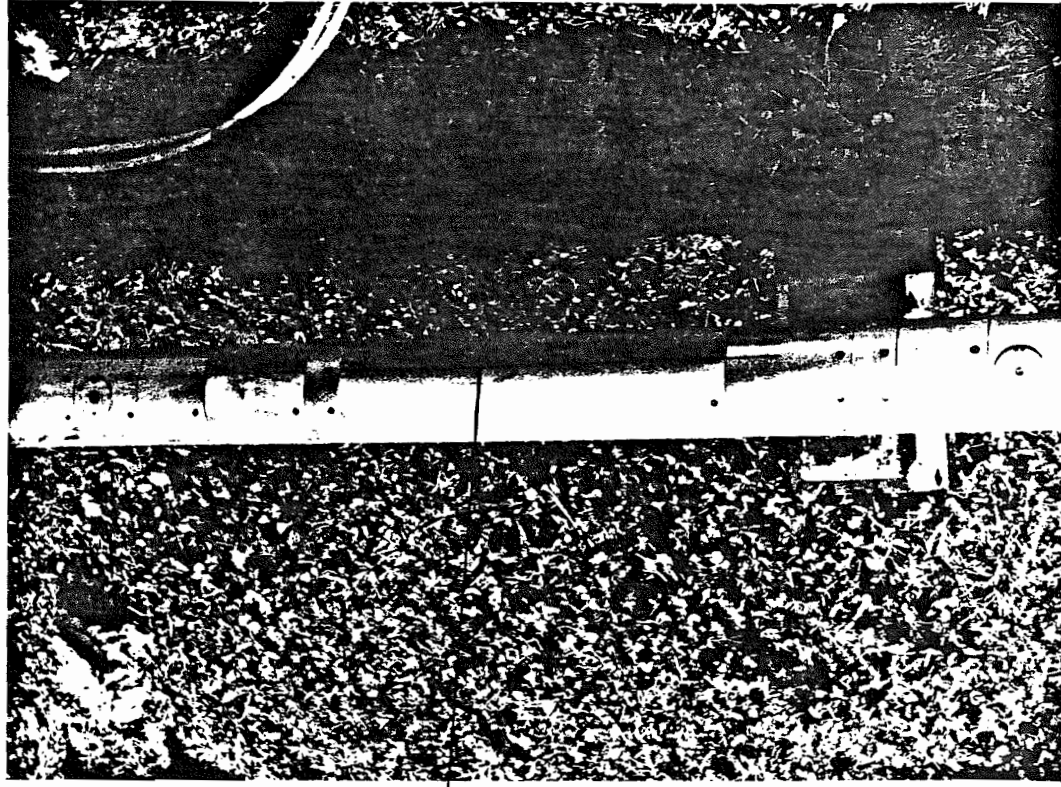


Figure 4.24 The IMP Immediately After Retrieval from Borehole L.

Figure 4.25 The IMP After Borehole L with Clay Removed.

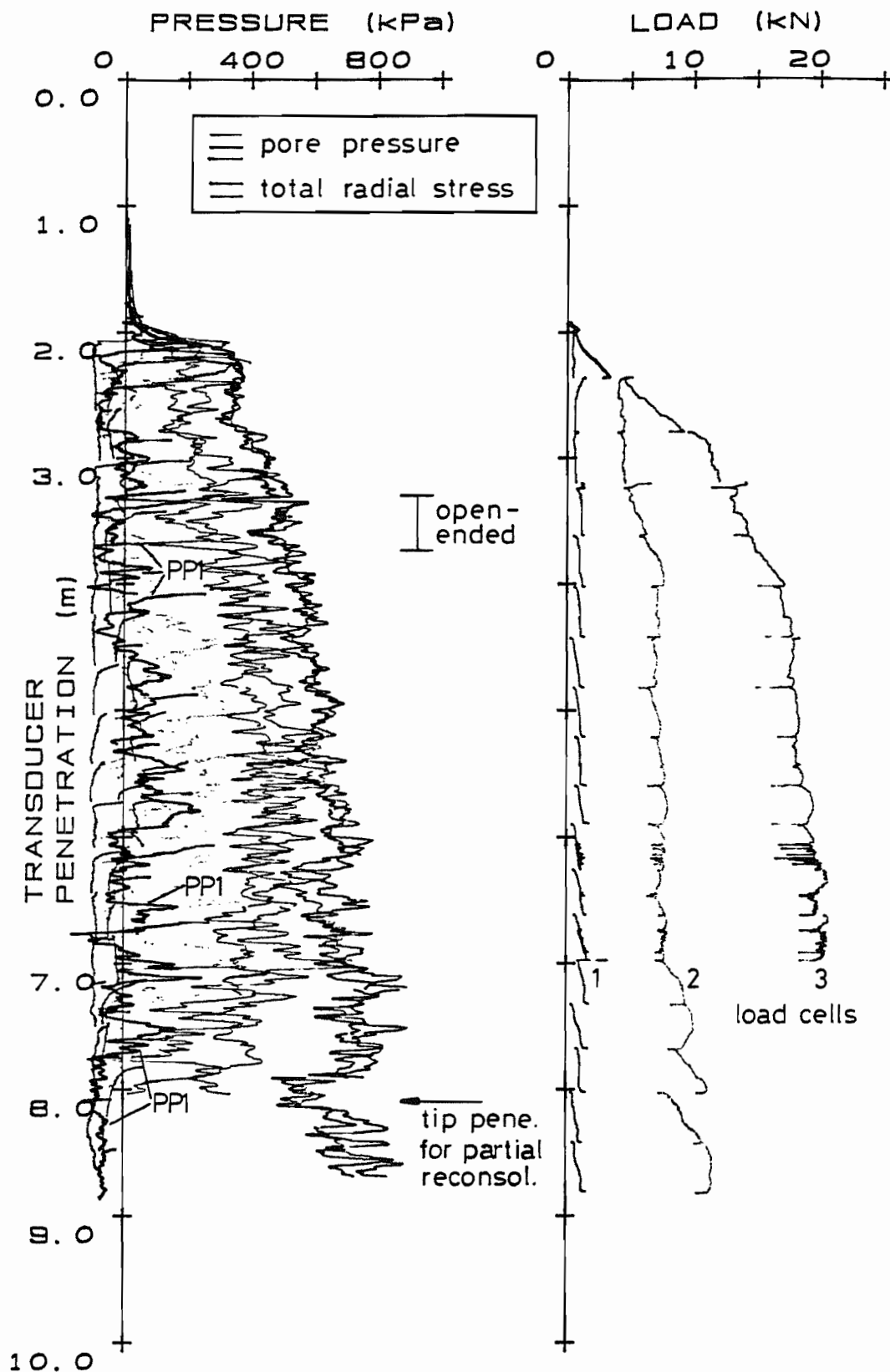


Figure 4.26 Raw Data Obtained During IMP Penetration in Borehole L, Madingley. (Refer to Fig.3.8 for Key)

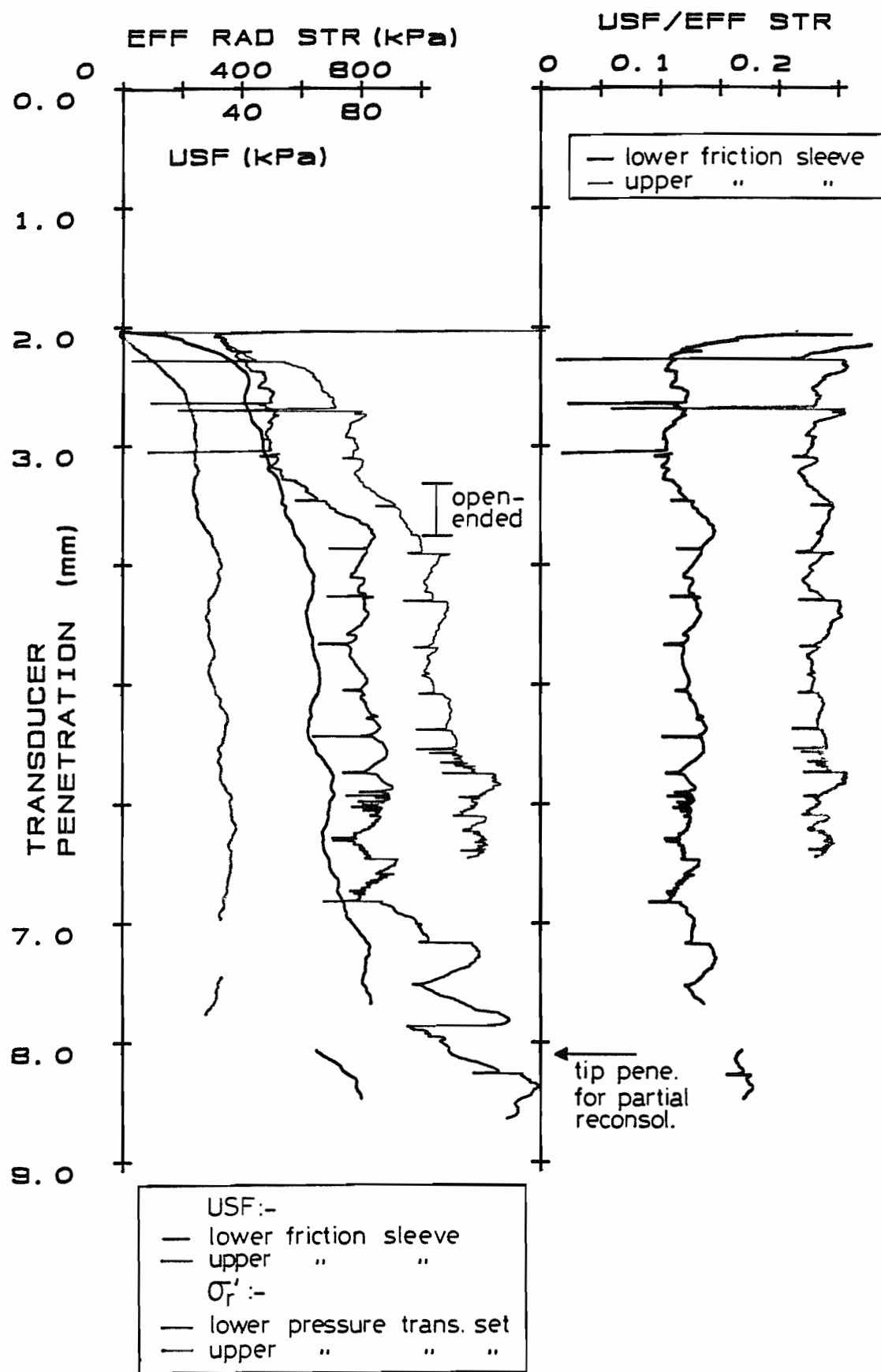


Figure 4.27 Interpreted Data from IMP Penetration in Borehole L, Madingley.

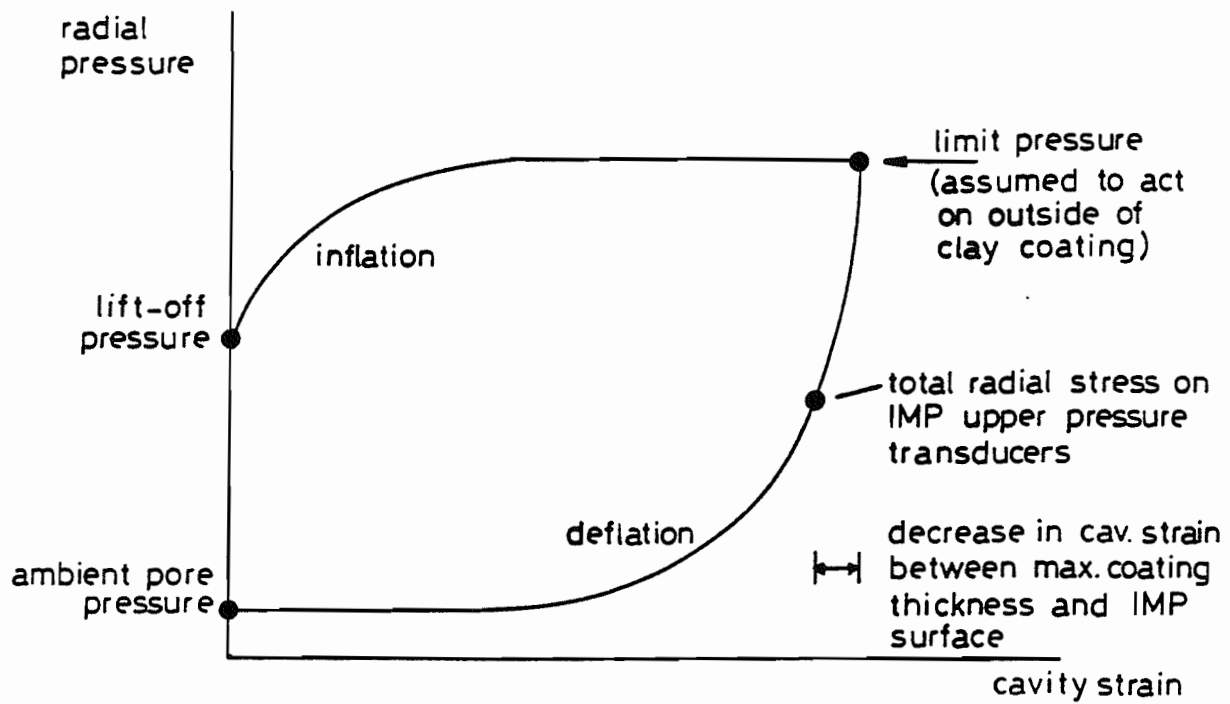


Figure 4.28 Idealised Sketch of a Pressuremeter Inflation Curve.



Figure 4.29 IMP and Extension Tube After Extraction from Borehole L (mud removed)

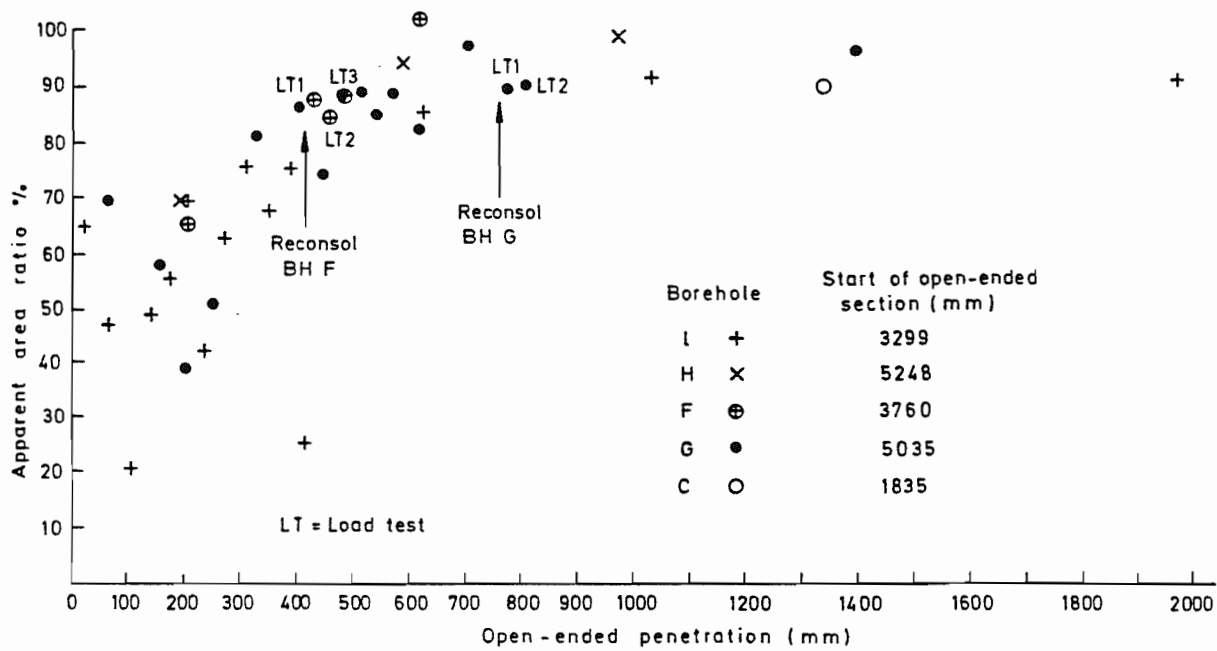


Figure 4.30 Apparent Area Ratios for Open-Ended Penetration, Madingley.

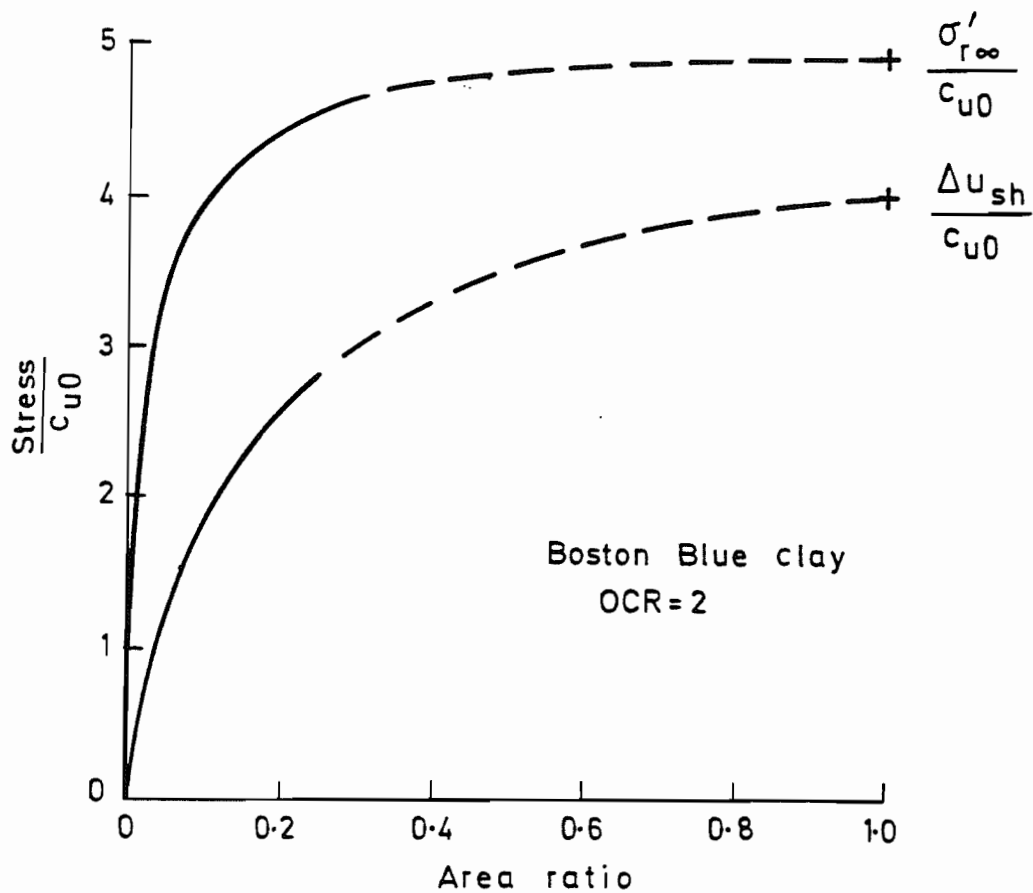


Figure 4.31 Predictions of the Effect of Area Ratio from Carter et al. (1979b) -redrawn from their data.

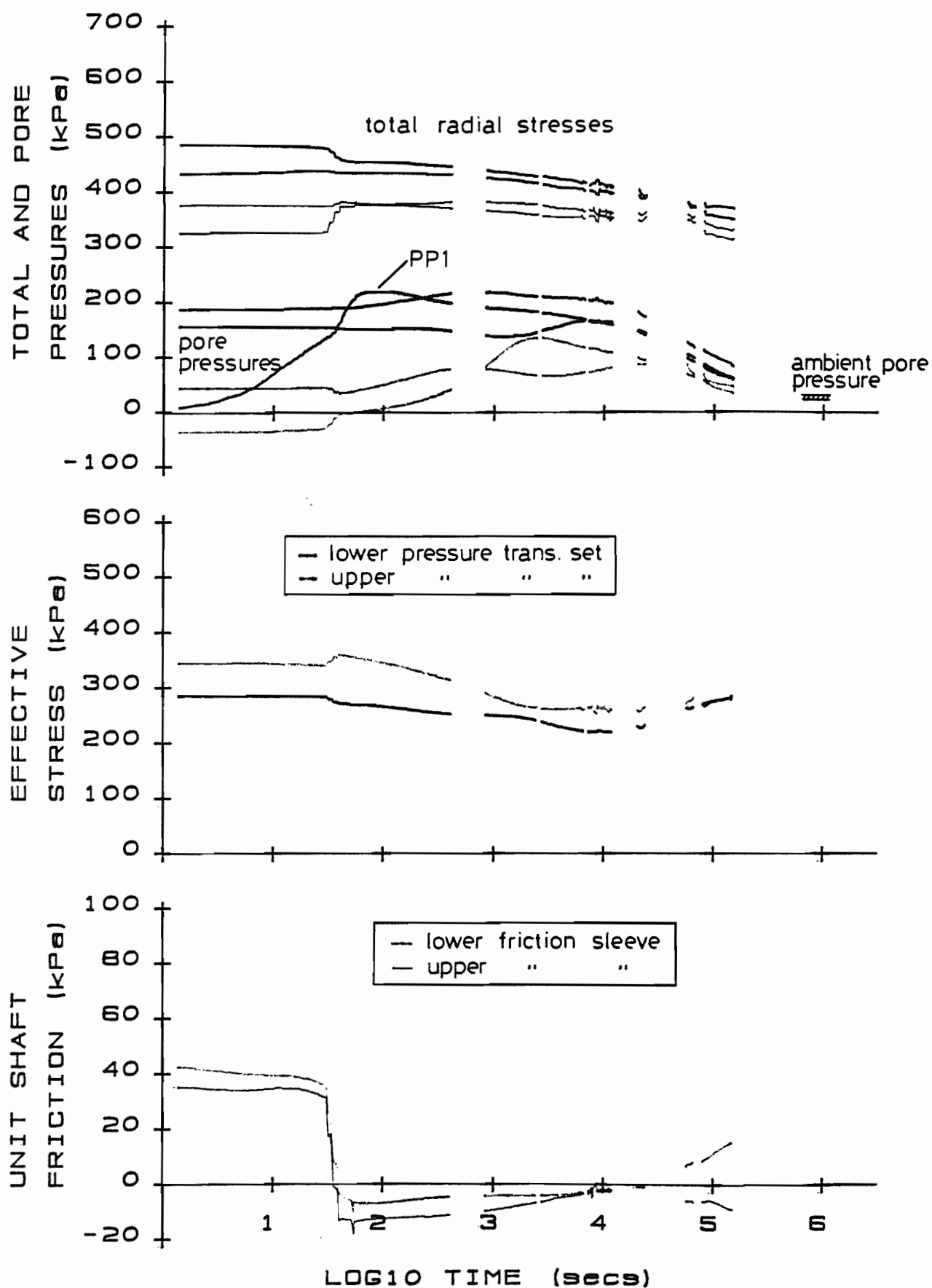


Figure 4.32 Reconsolidation Data from Borehole F, Madingley.
 Tip Penetration = 4.17m, Refer to Fig.3.7 for Key.

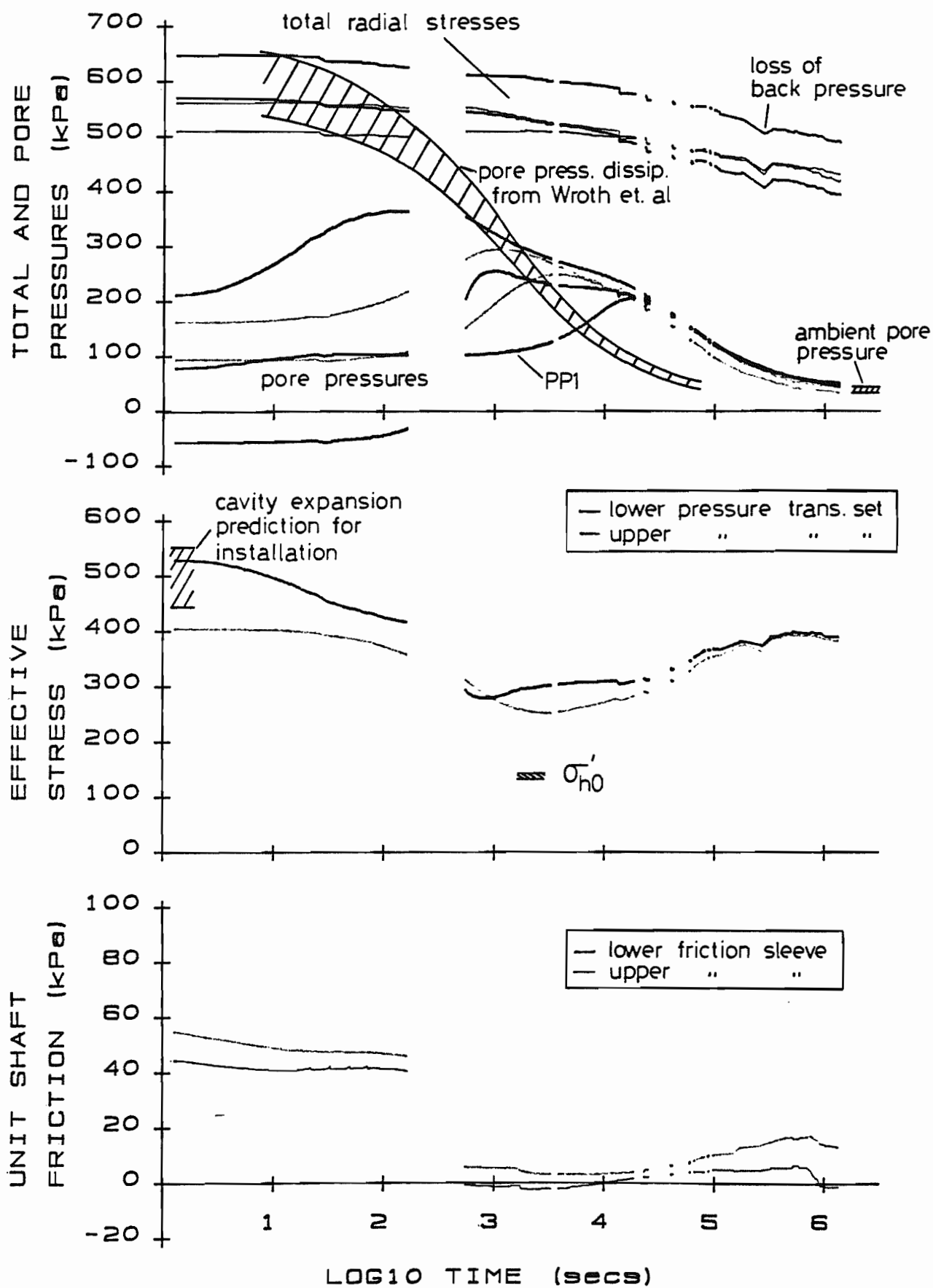


Figure 4.33 Reconsolidation Data from Borehole G, Madingley.
 Tip Penetration =5.79m, Refer to Fig.3.8 for Key.

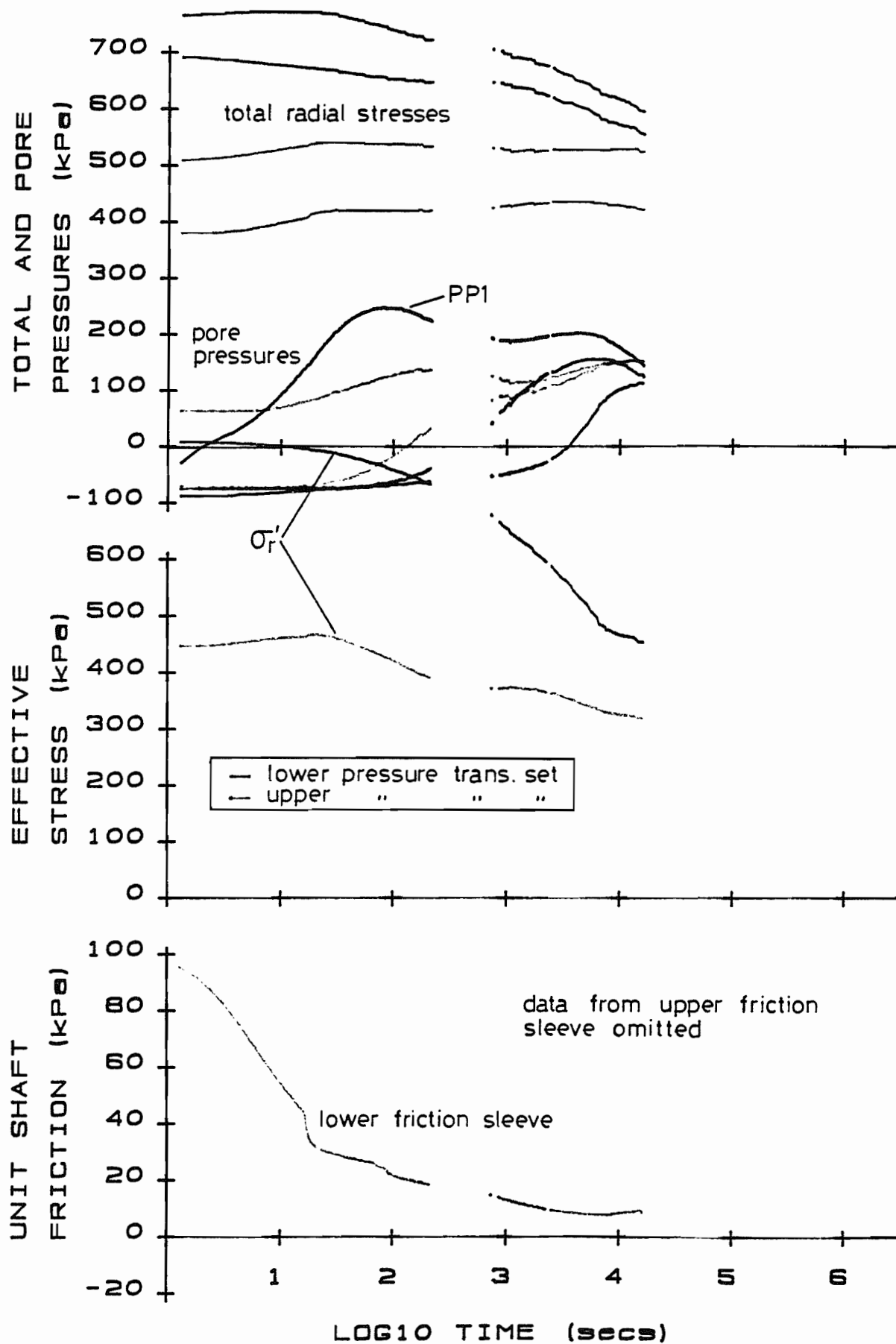


Figure 4.34 Reconsolidation Data from Borehole L, Madingley.
 Tip Penetration =8.11m, Refer to Fig.3.8 for Key.

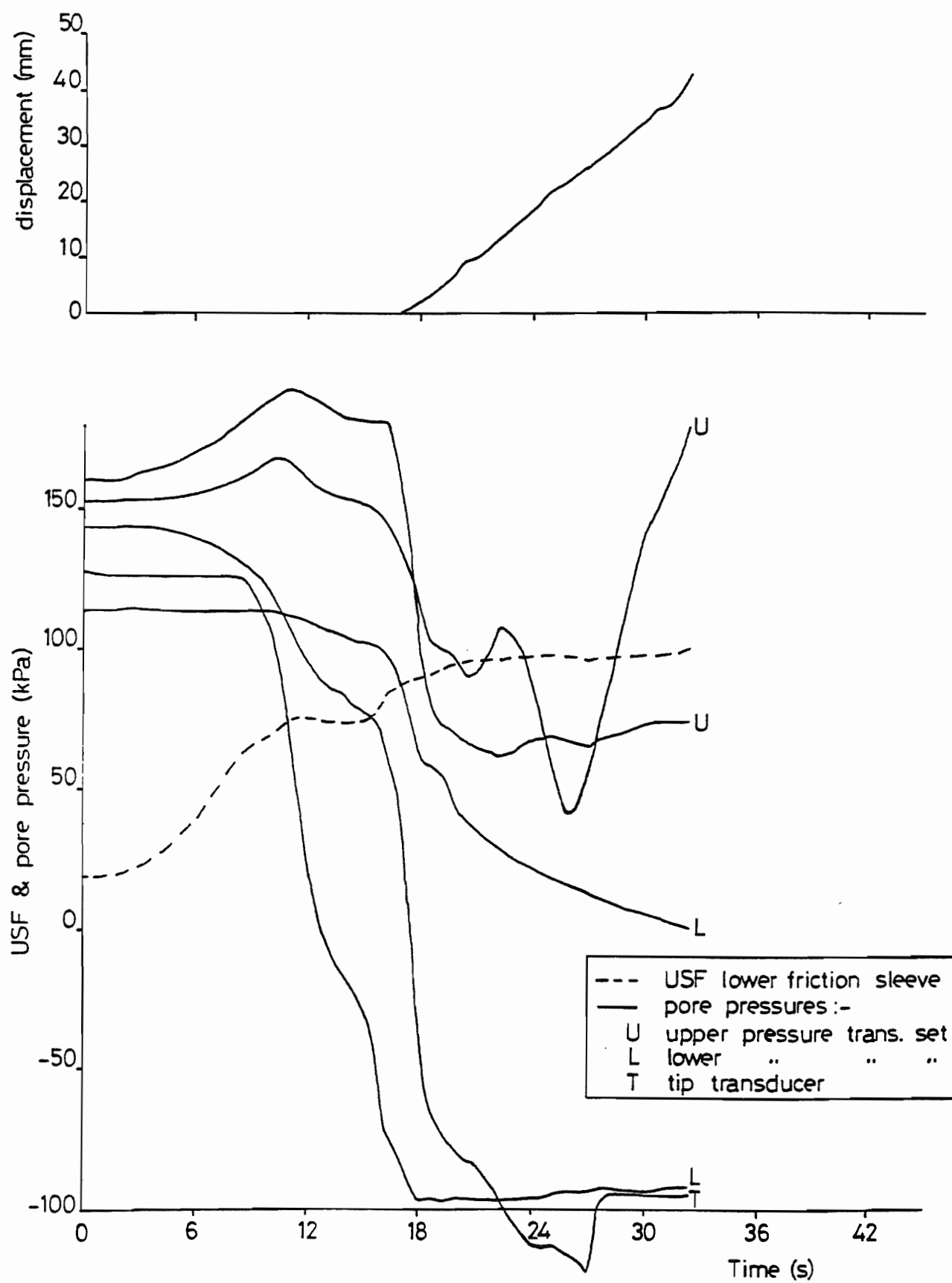


Figure 4.35 Pore Pressure Changes Upon Restarting Penetration
After Partial Reconsolidation in Borehole L.

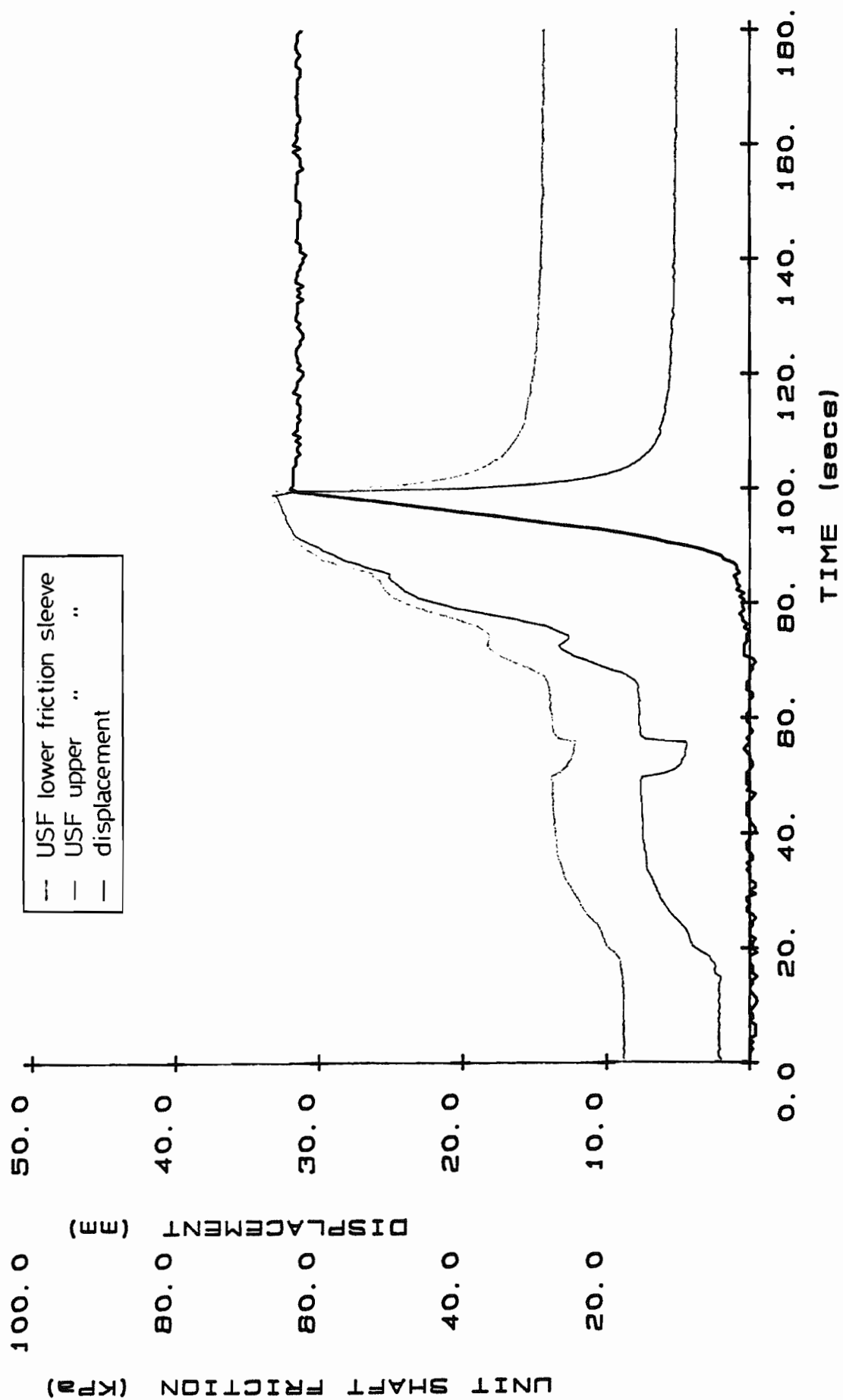


Figure 4.36 First Compressive Load Test, Borehole G: Unit Shaft Friction.

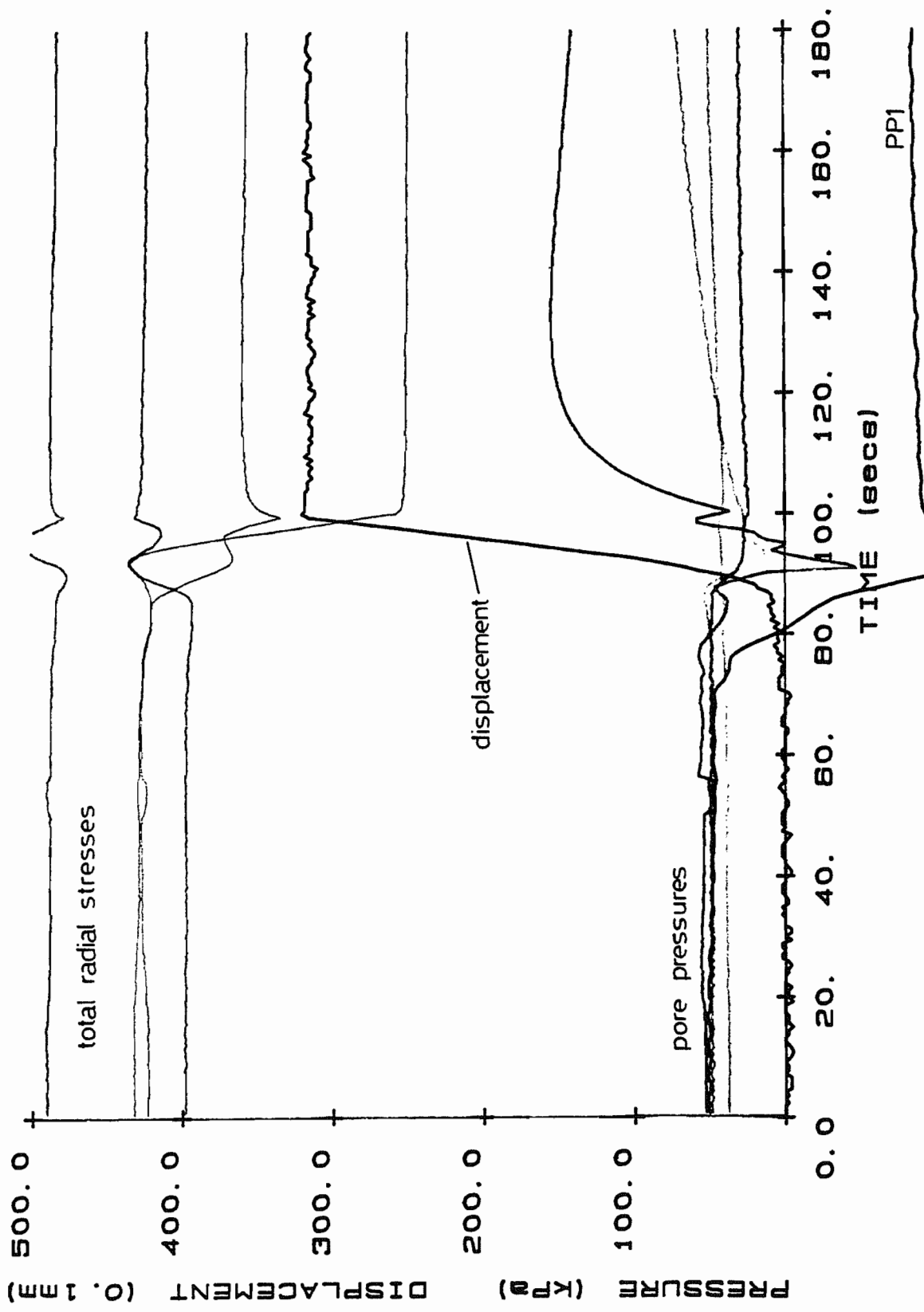


Figure 4.37 First Compressive Load Test, Borehole G: Pressure Transducer Responses.

(Refer to Fig.3.8 for Key)

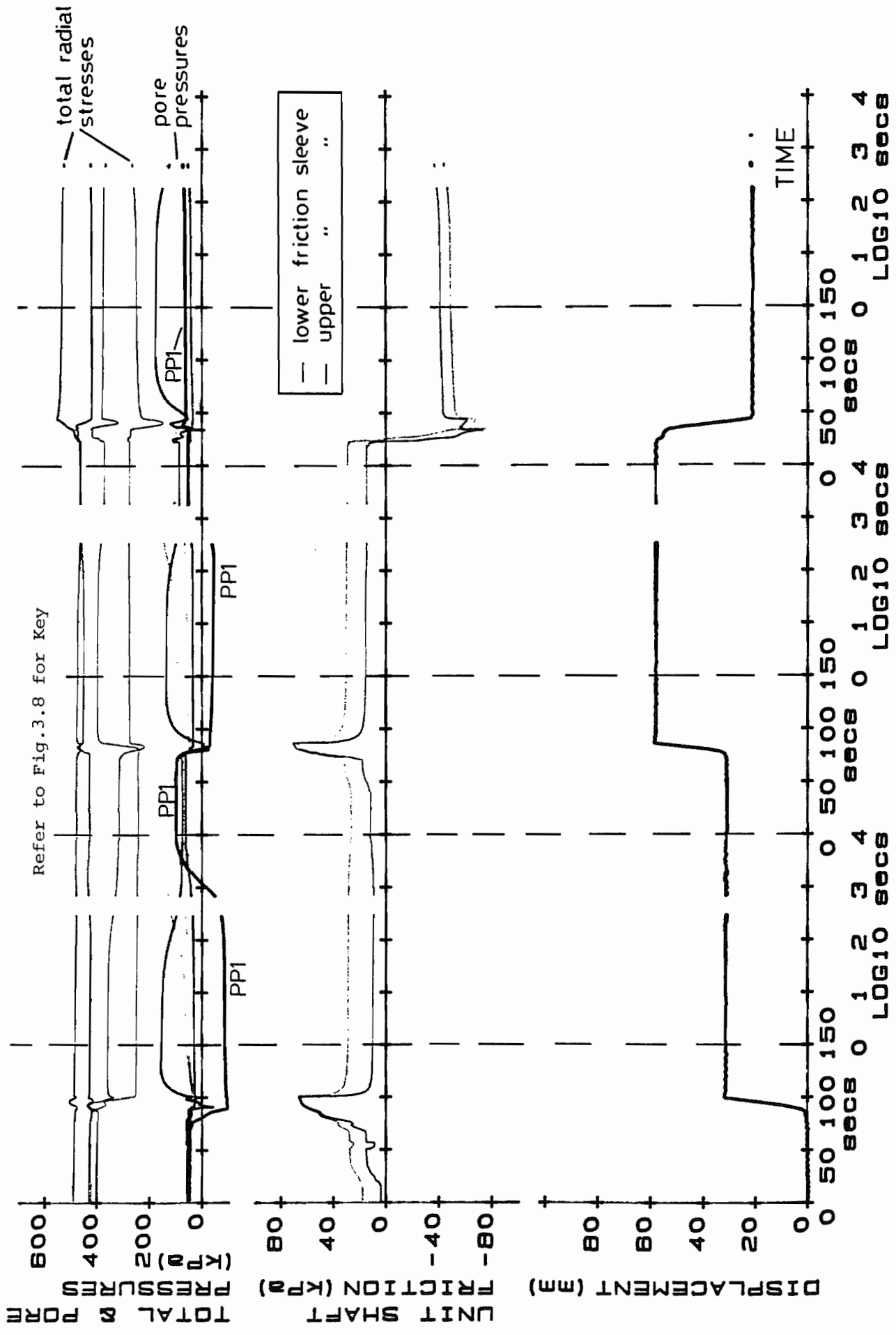


Figure 4.38 Data from the First Three Monotonic Load Tests, Borehole G.

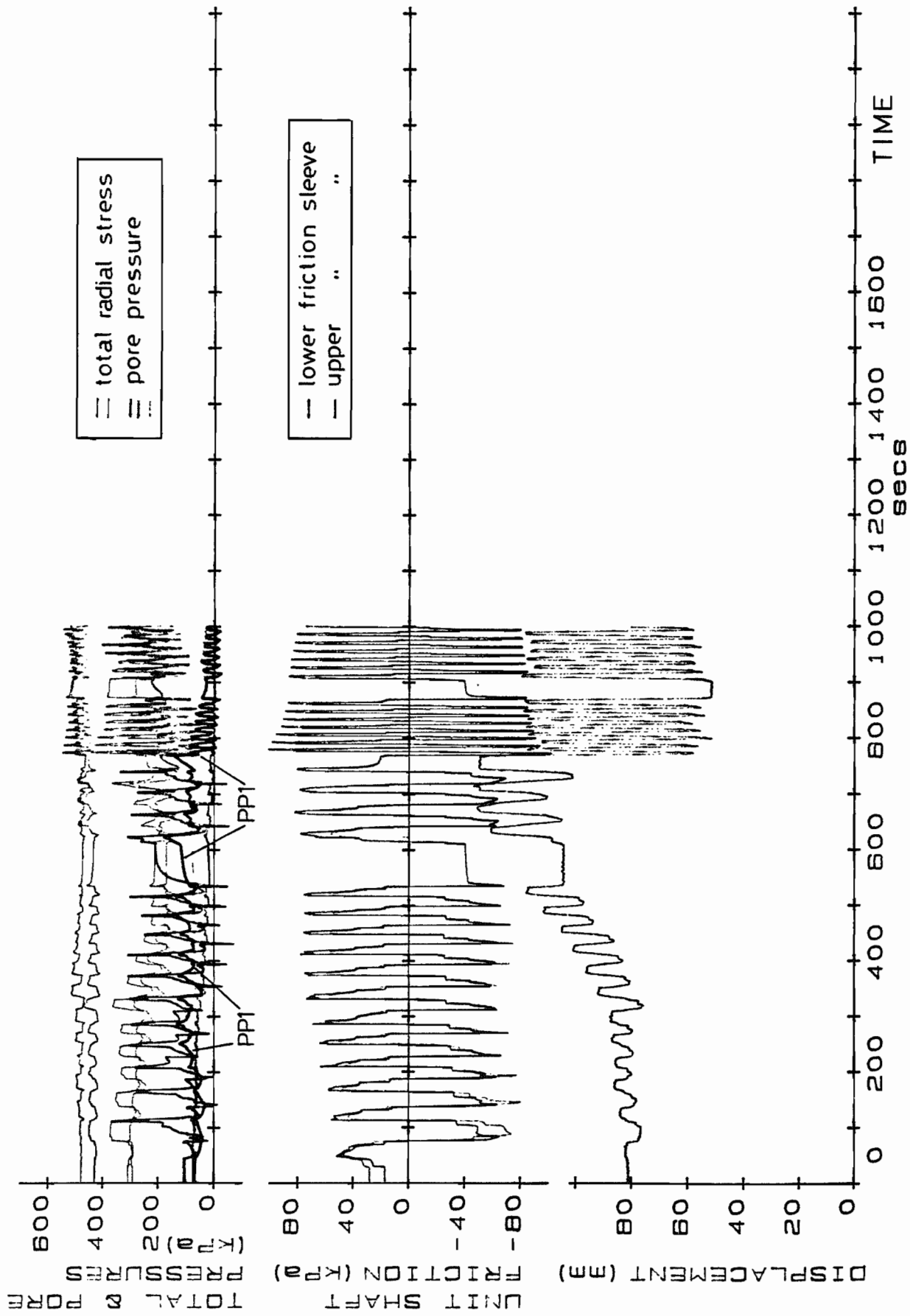


Figure 4.39 Cyclic Load Test, Borehole G, Madingley
Refer to Fig.3.8 for Key.

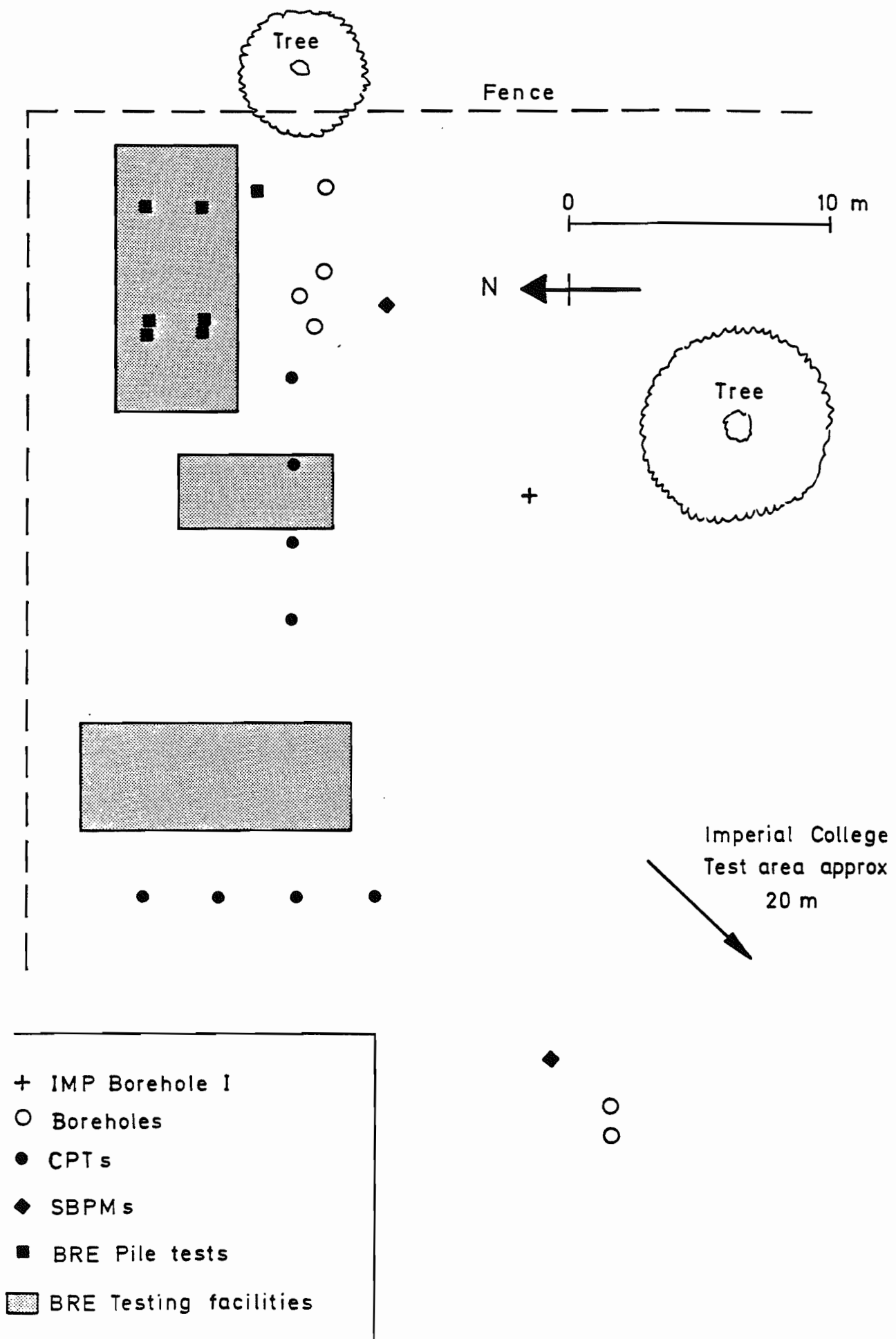


Figure 5.1 Site Plan for Canons Park. (Data Supplied by BRE and from Jardine 1985)

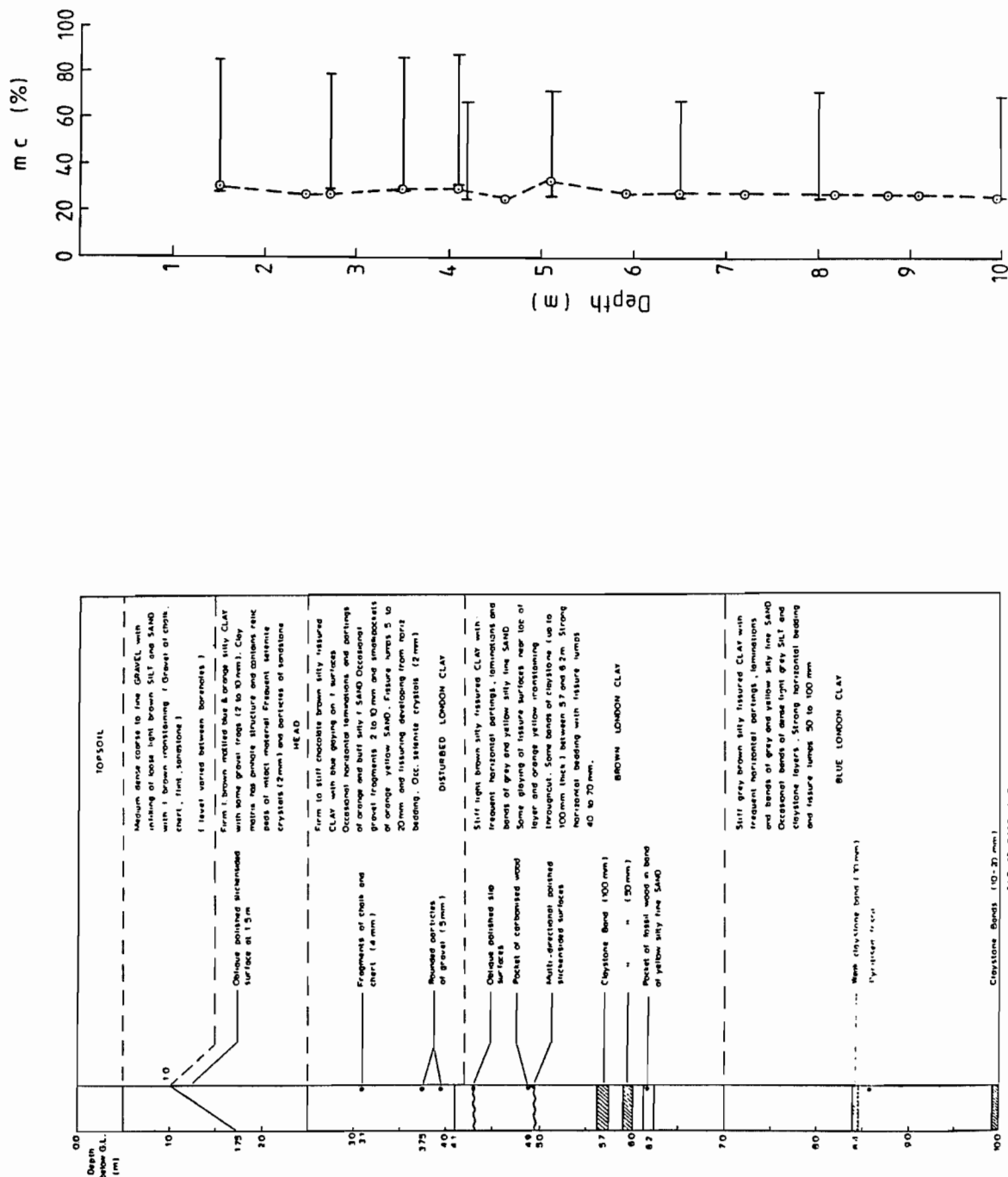


Figure 5.2 Borehole Log and Index Properties for Canons Park (After Jardine 1985).

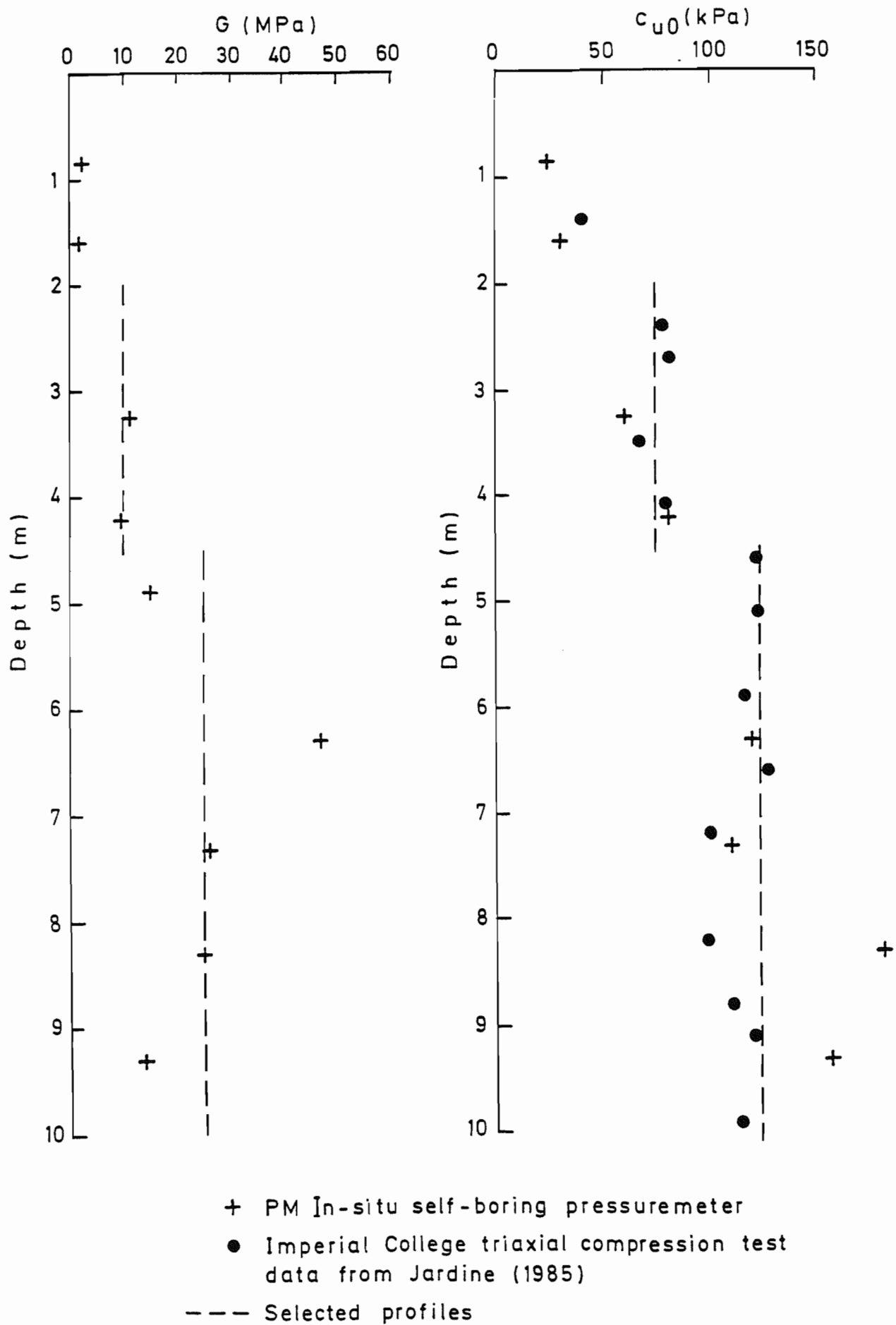


Figure 5.3 Shear Strength and Shear Modulus Data for Canons Park.

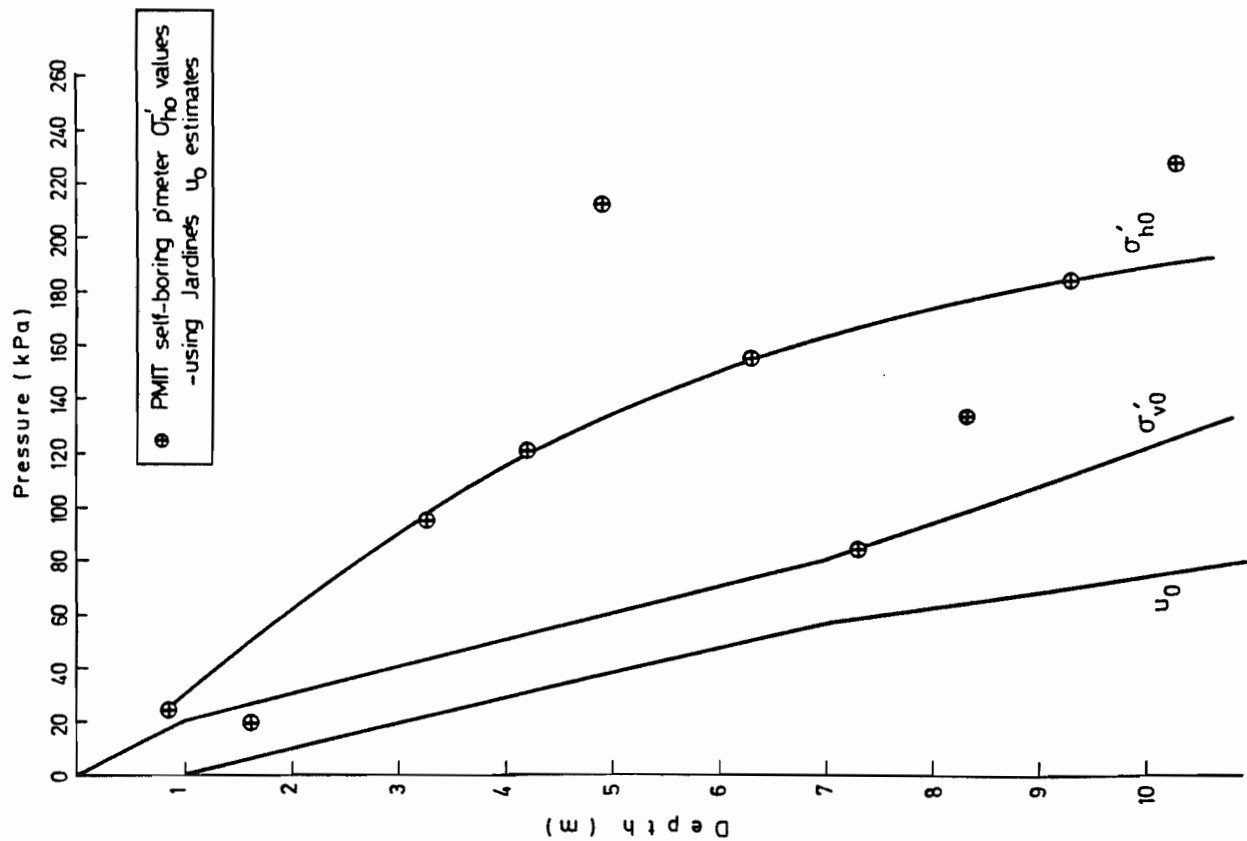


Figure 5.4 Estimates of In-Situ Stresses, Canons Park

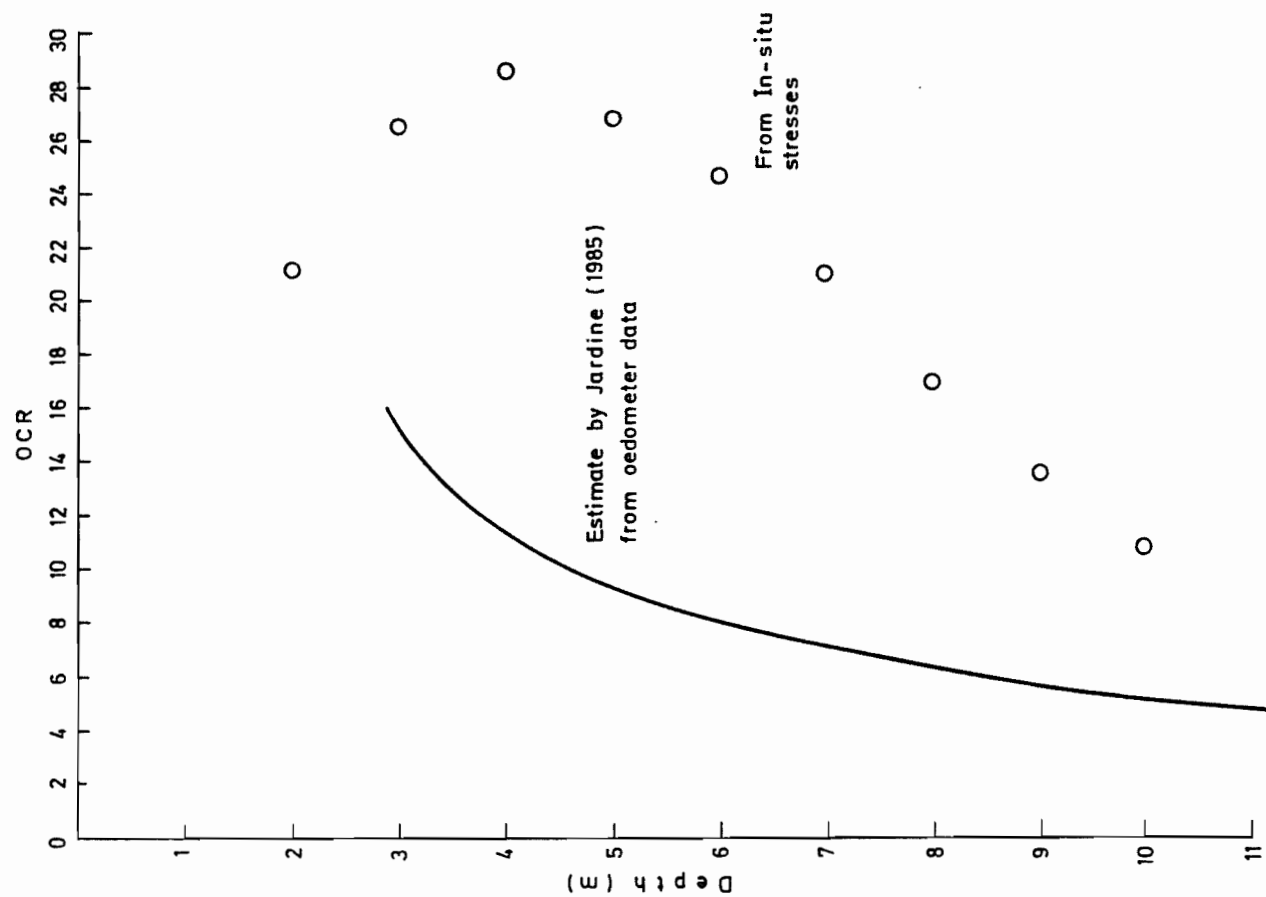


Figure 5.5 Estimates of OCR, Canons Park.

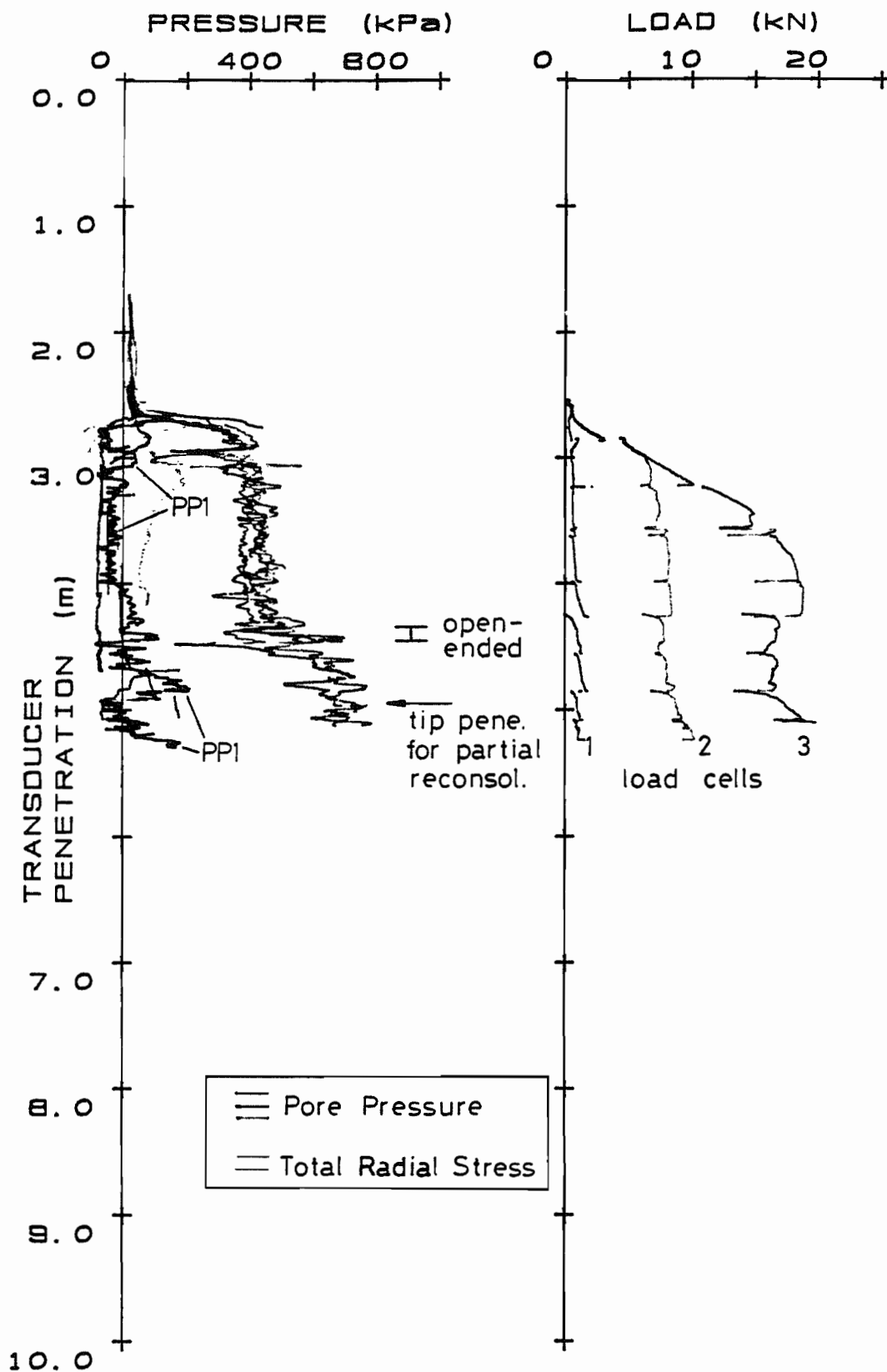


Figure 5.6 Raw Data Obtained During IMP Penetration in Borehole I, Canons Park.

(Refer to Fig.3.8 for Key)

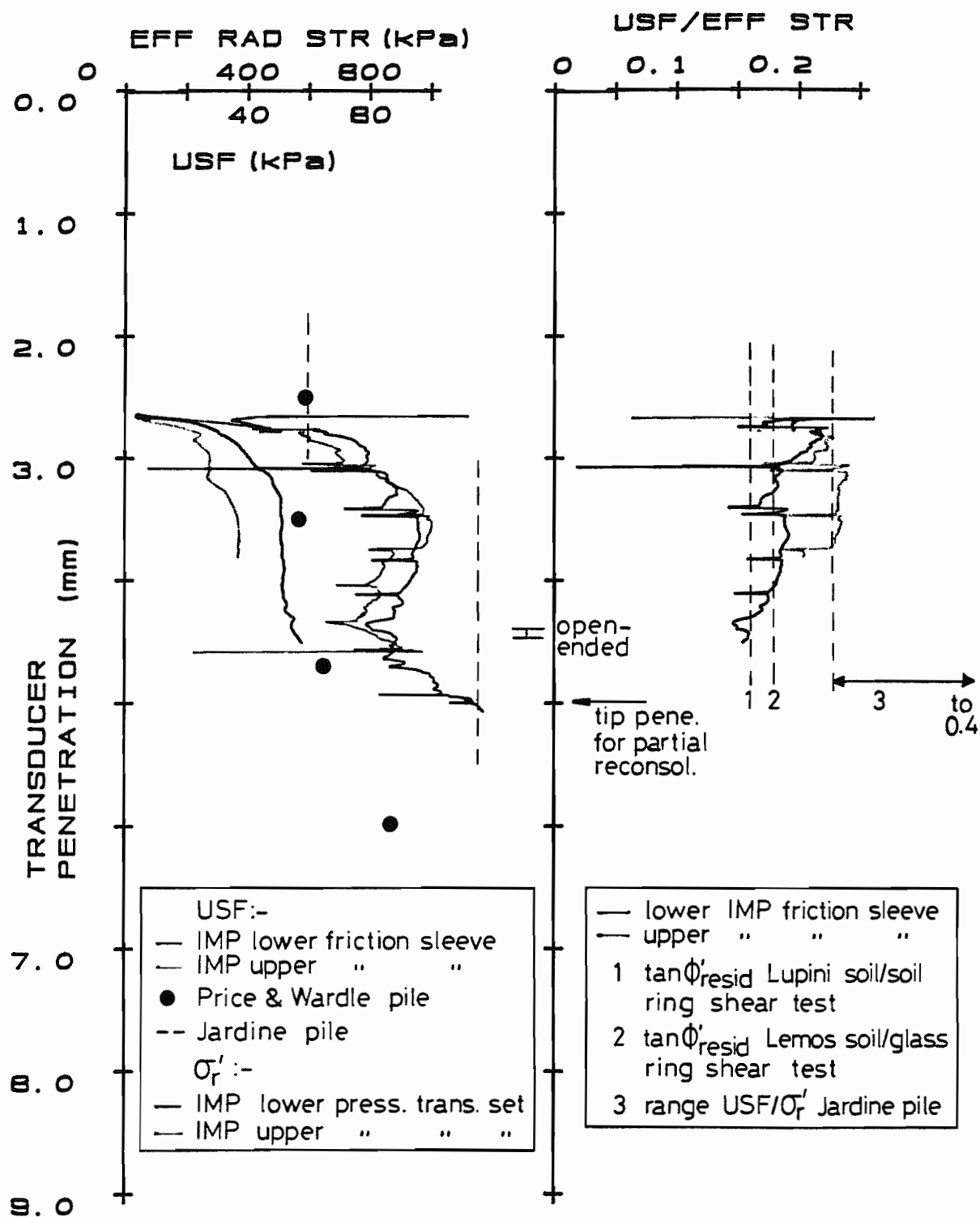


Figure 5.7 Interpreted Data from IMP Penetration in Borehole I, Canons Park.

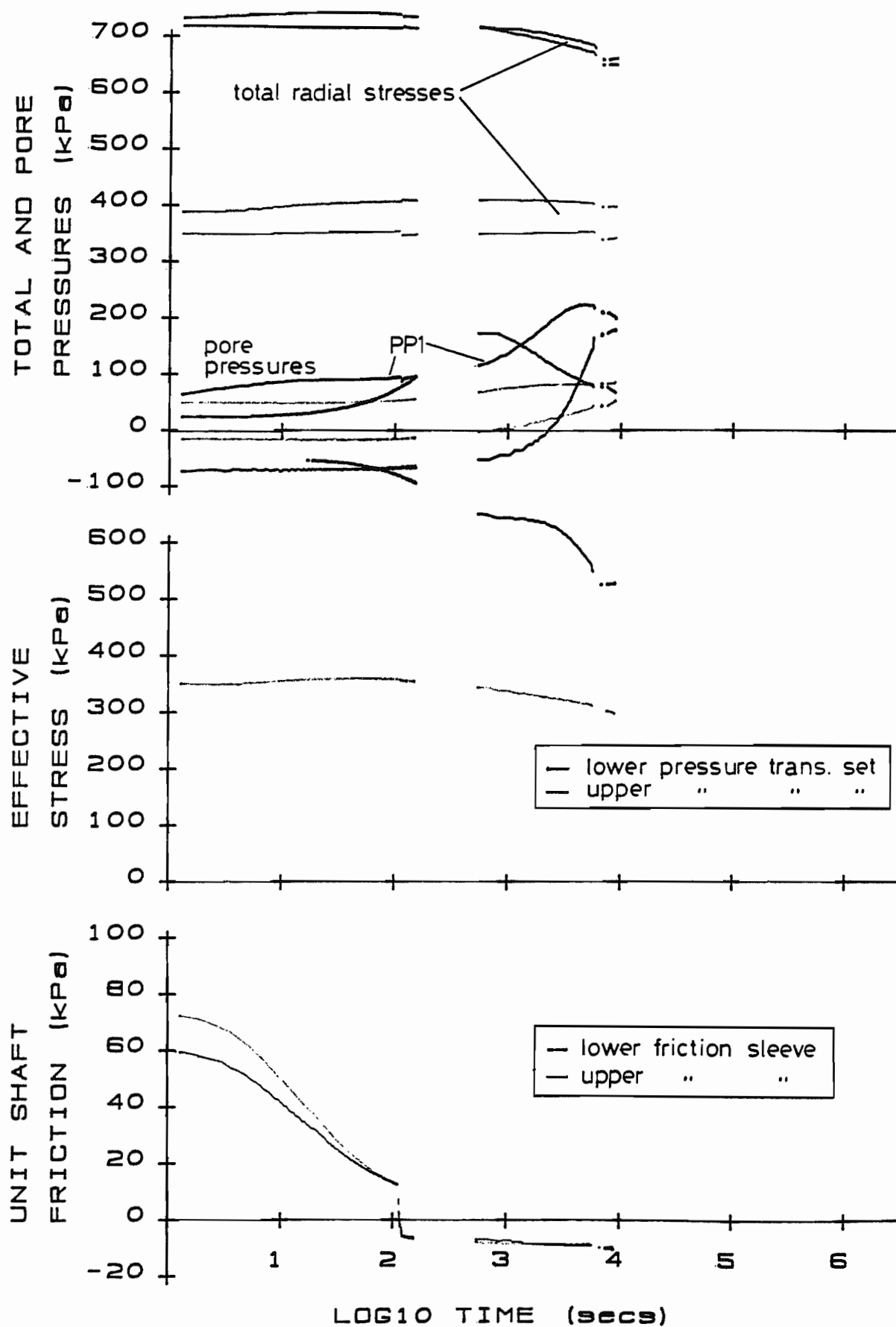


Figure 5.8 Partial Reconsolidation Data from Borehole I,
 Canons Park. Refer to Fig.3.8 for Key.
 (Tip Penetration =4.95m)

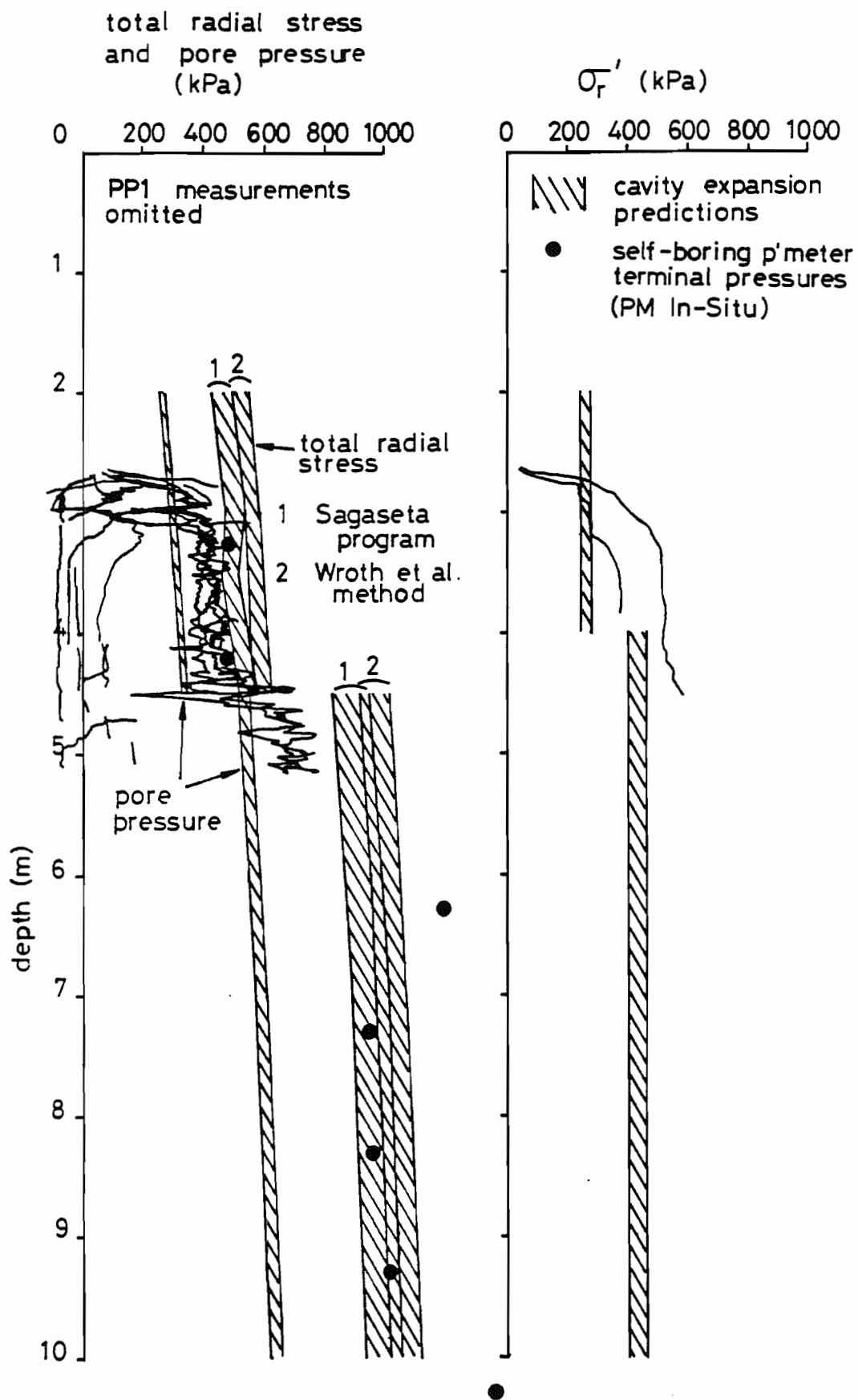


Figure 5.9 Comparison of IMP Penetration Data for Canons Park with Cavity Expansion and Self-Boring Pressuremeter Data.

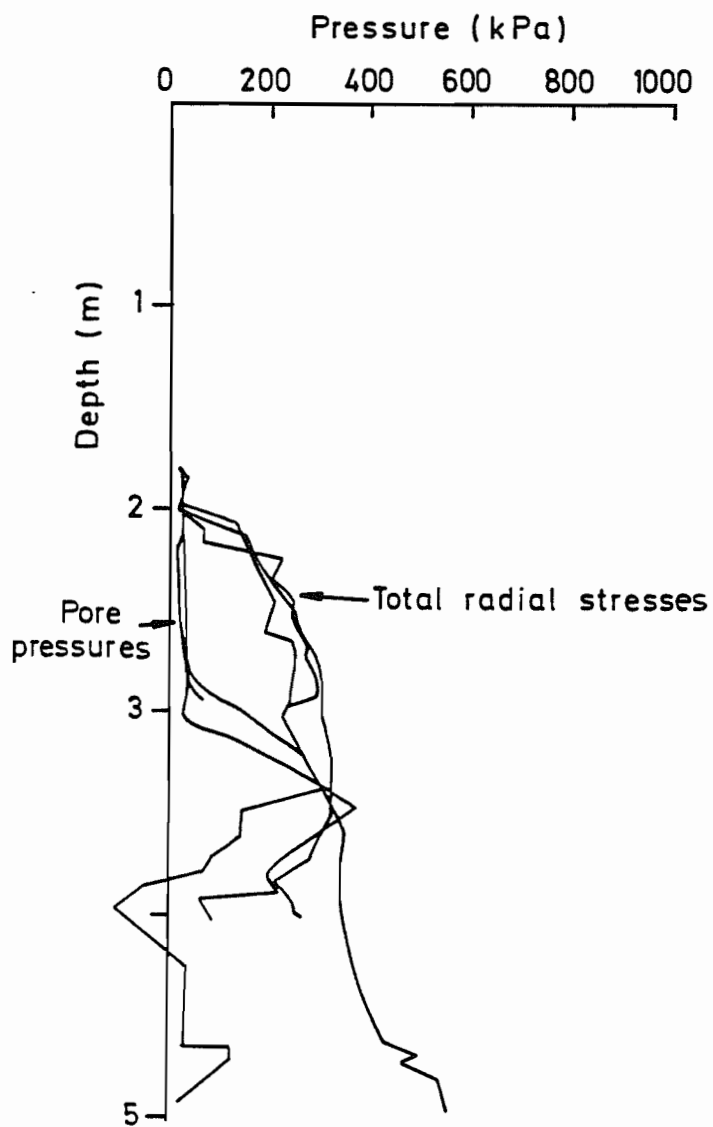


Figure 5.10 Data Obtained from Jardine's Pile During Installation.
(Data from His Thesis Replotted at a Different Scale)

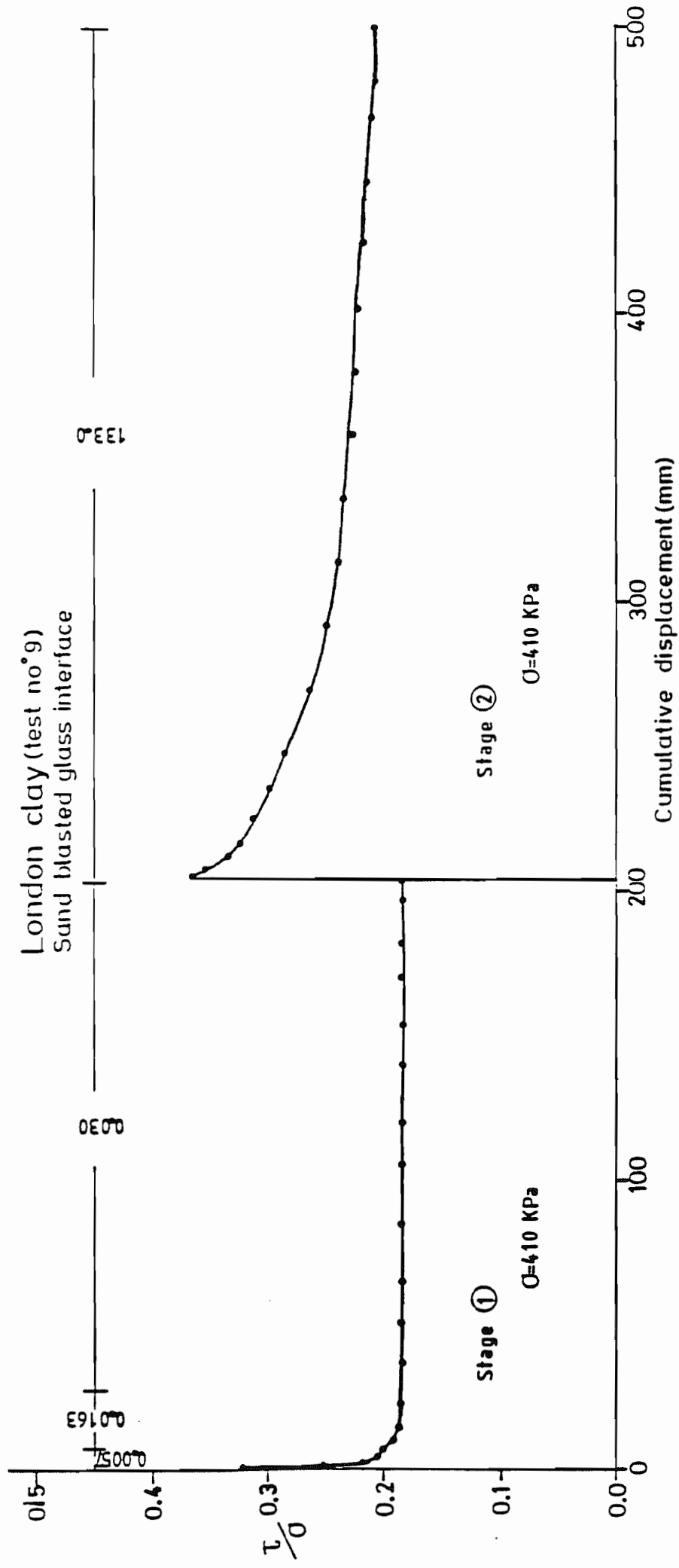


Figure 5.11 Lemos' Interface Ring Shear Data for London clay (After Lemos 1985) .

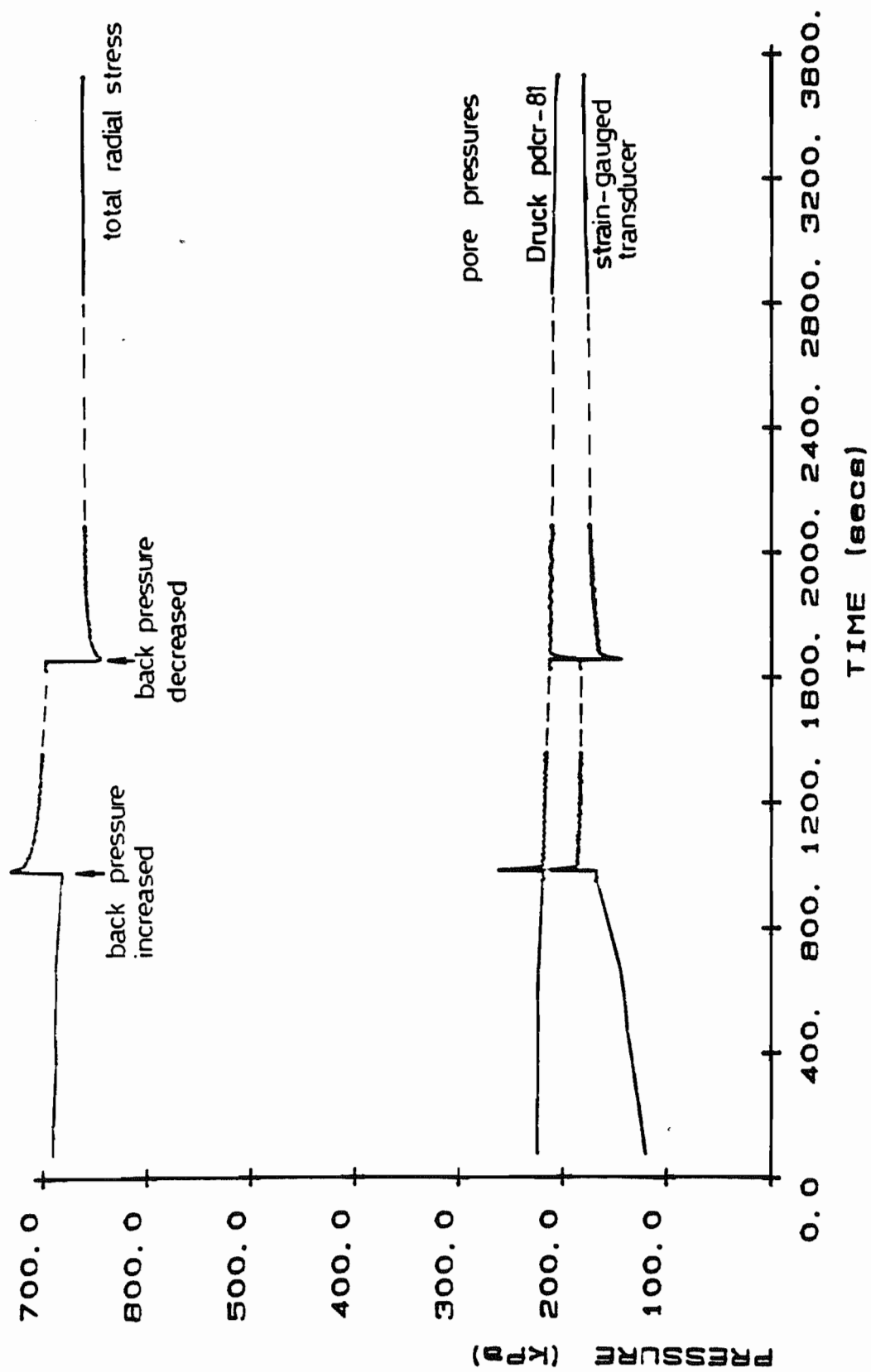


Figure 5.12 IMP Back Pressure Test Data from Canons Park.

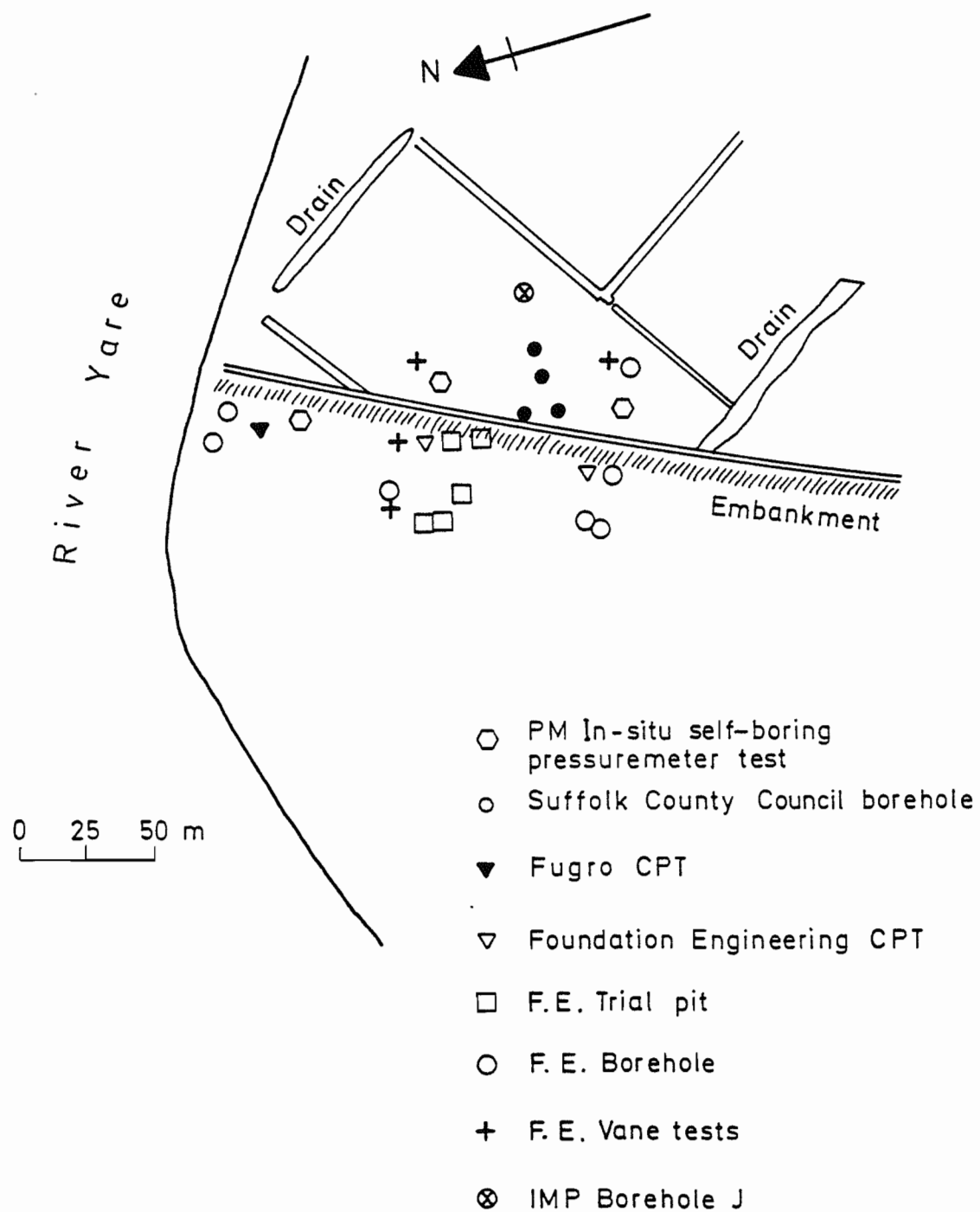
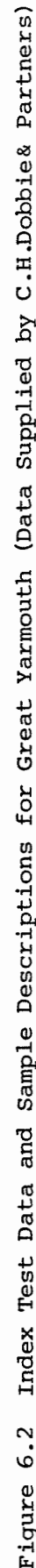
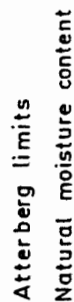


Figure 6.1 Plan of the Great Yarmouth Site.

(Data Supplied by C.H.Dobbie & Partners)



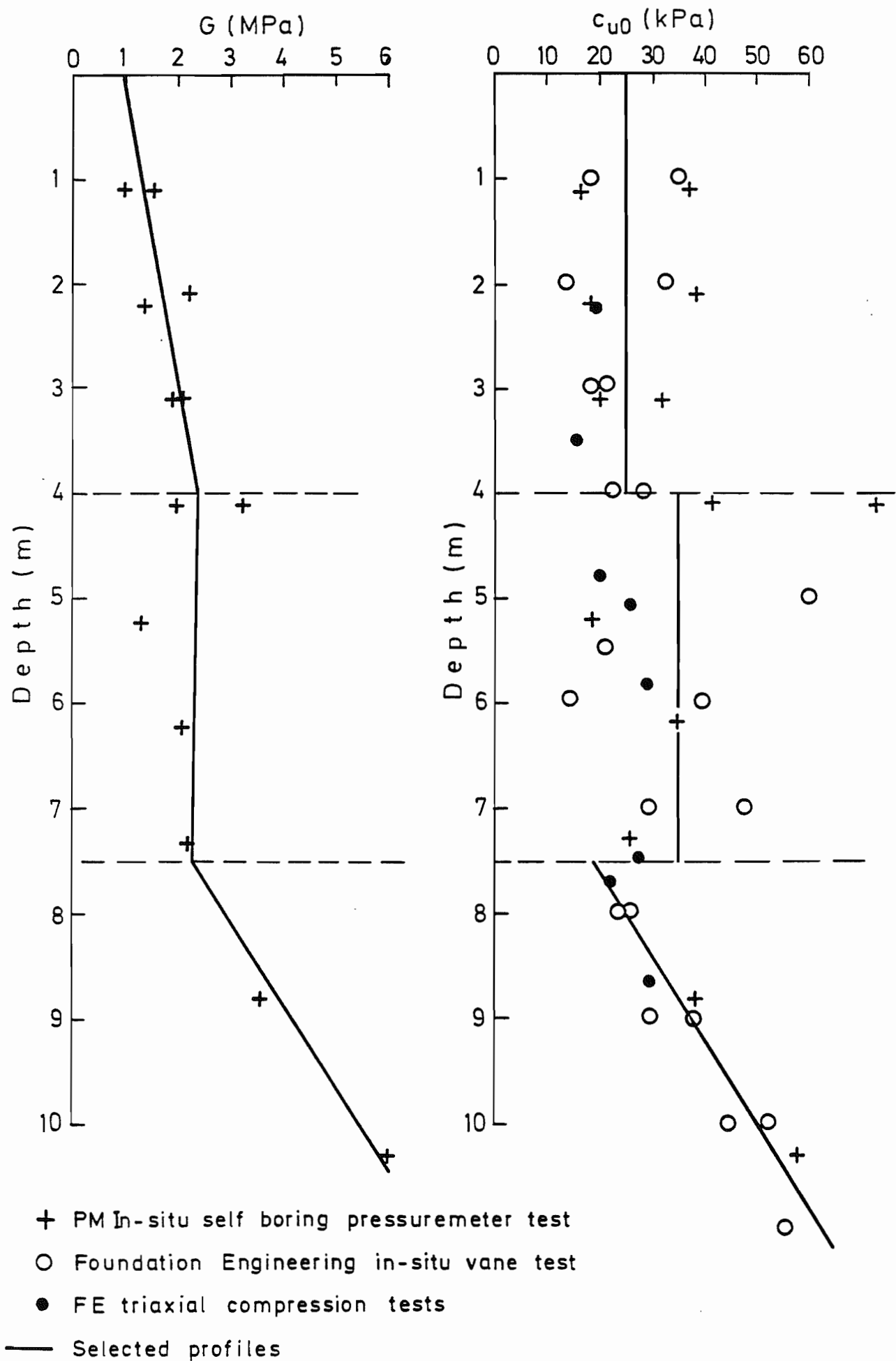


Figure 6.3 Undrained Shear Strengths and Moduli for Great Yarmouth.

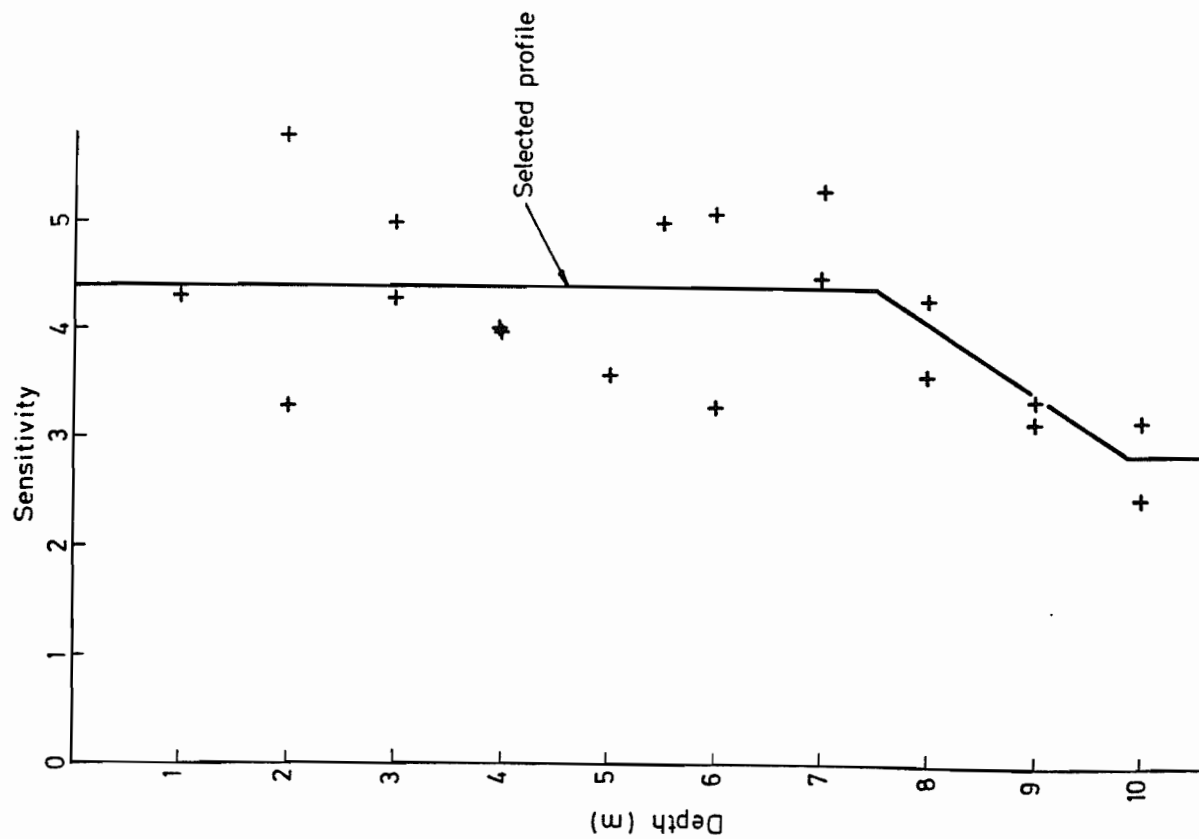


Figure 6.4 Great Yarmouth, Soil Sensitivity.
(Foundation Eng. Data Supplied by
C H Nabbie and Partners)

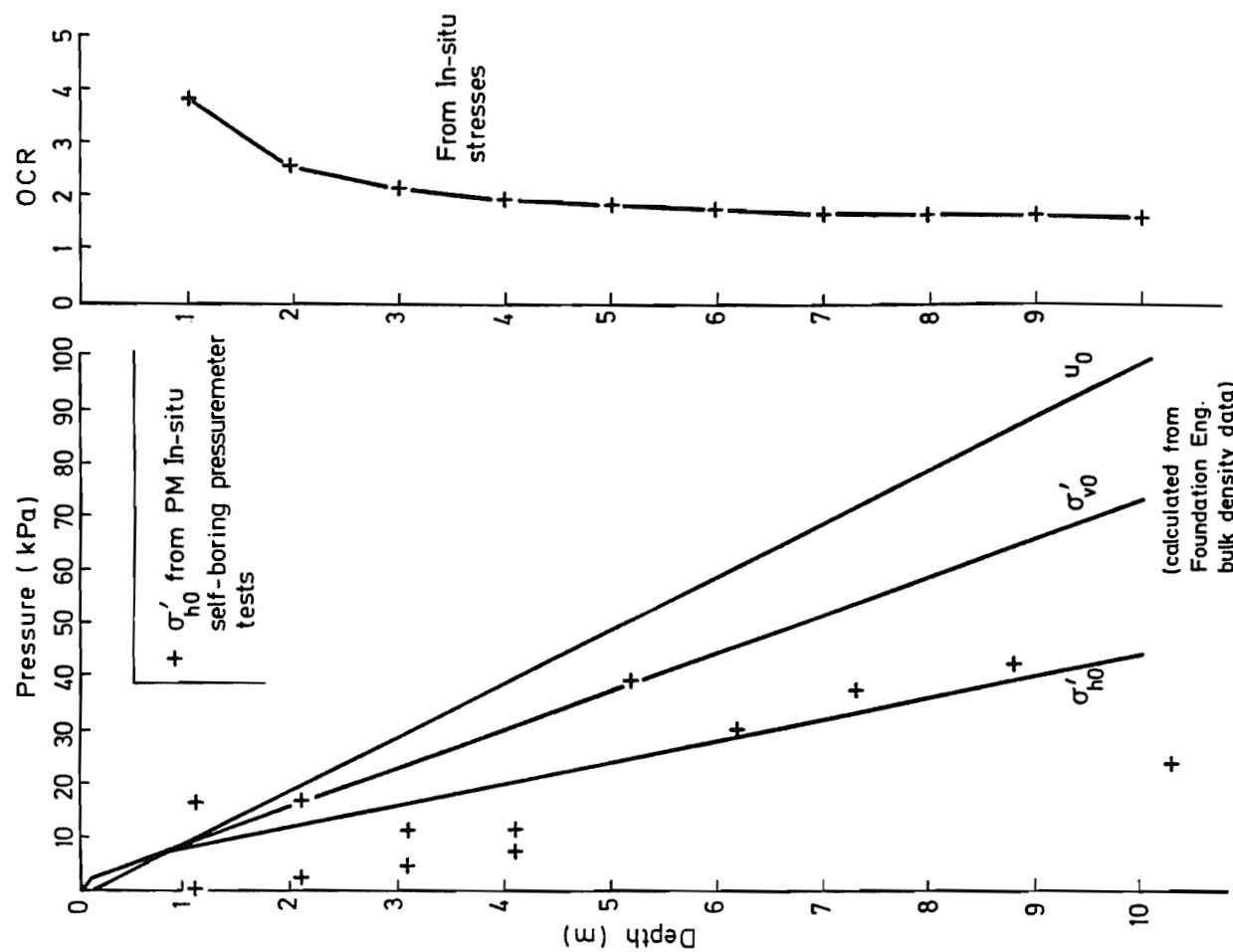


Figure 6.5 In-Situ Stresses and OCRs for Great Yarmouth.

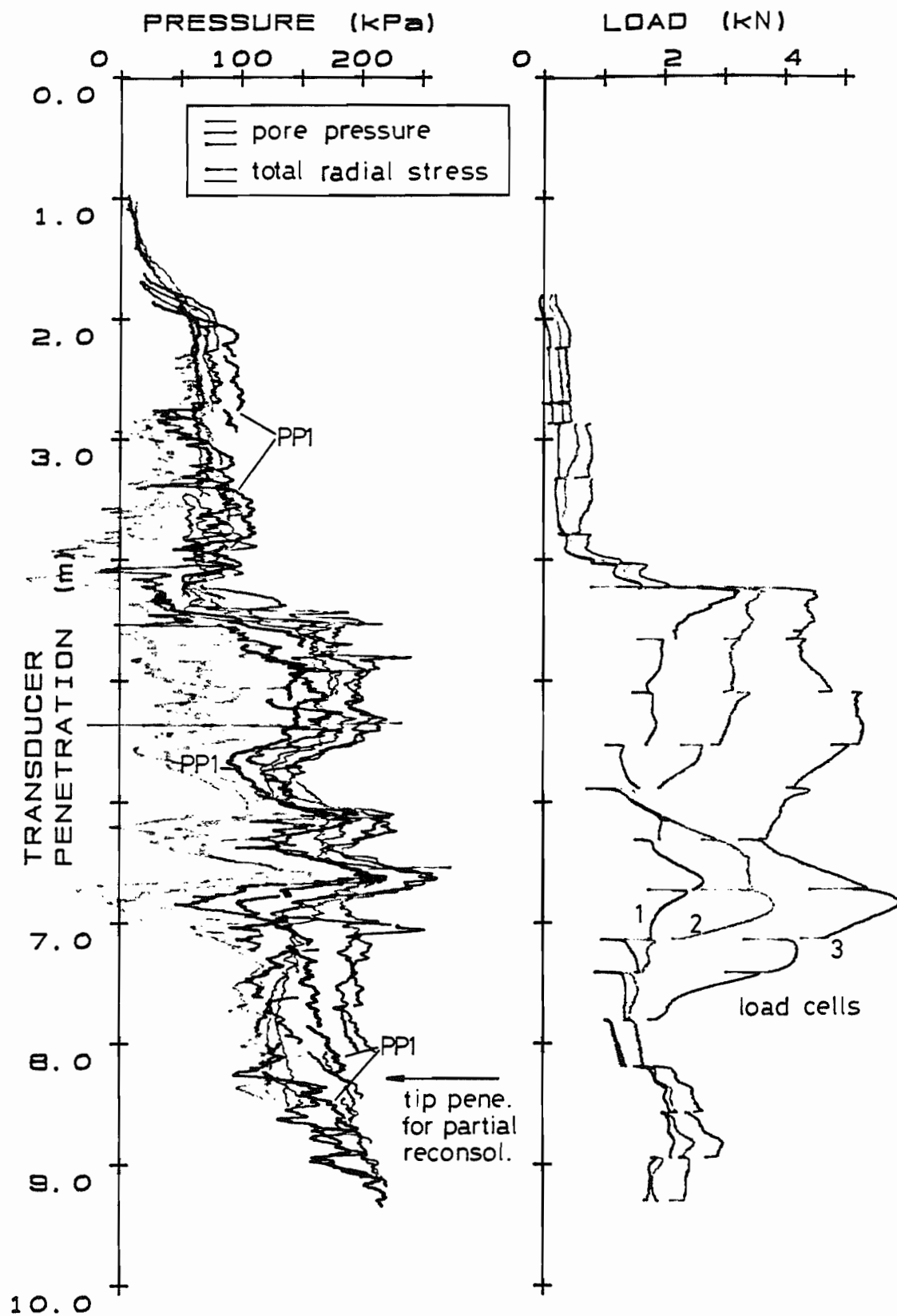


Figure 6.6 Raw Data Obtained During IMP Penetration in Borehole J, Great Yarmouth. Refer to Fig.3.8 for Key.

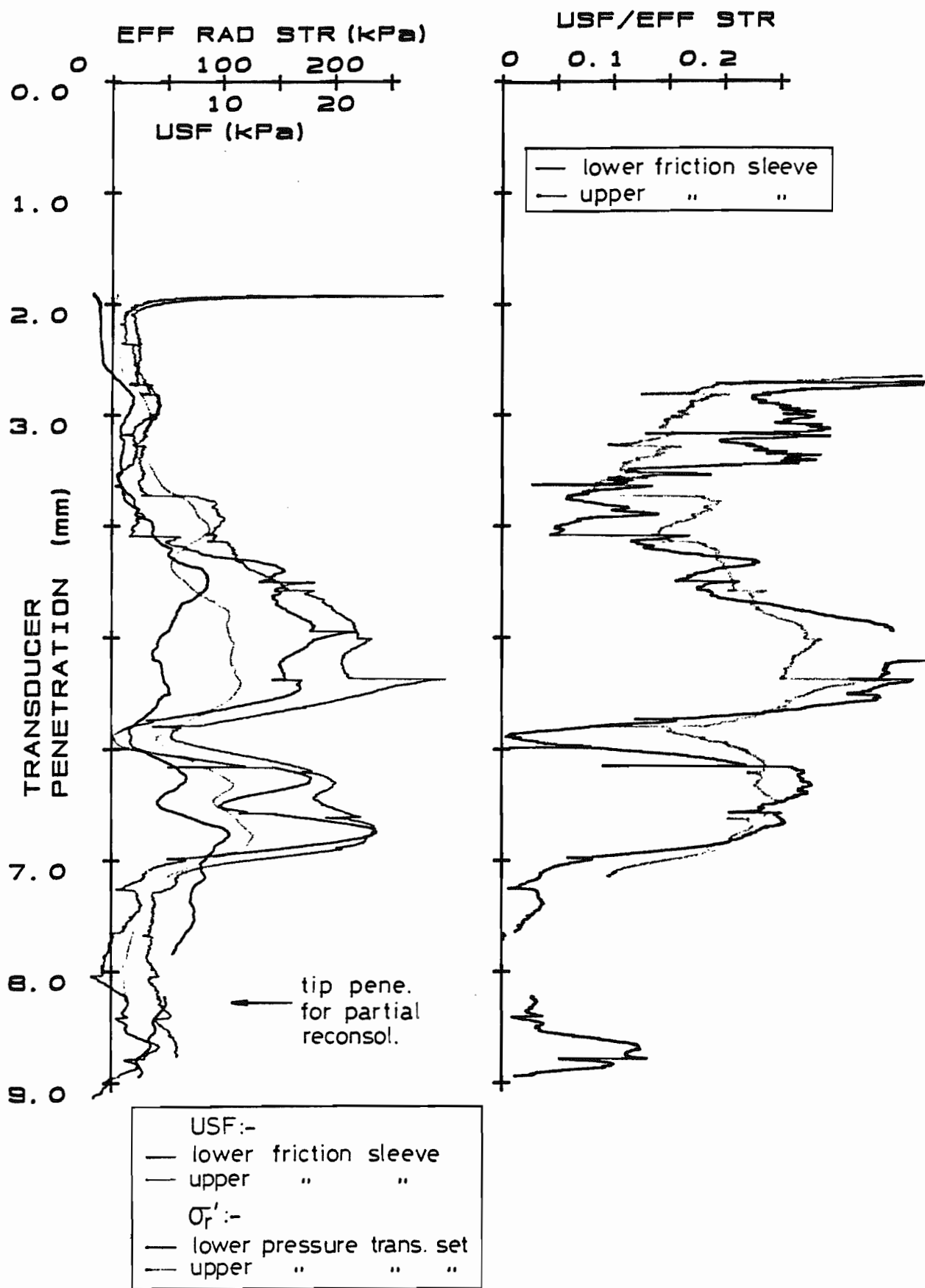


Figure 6.7 Interpreted Data from IMP Penetration in Borehole J, Great Yarmouth.

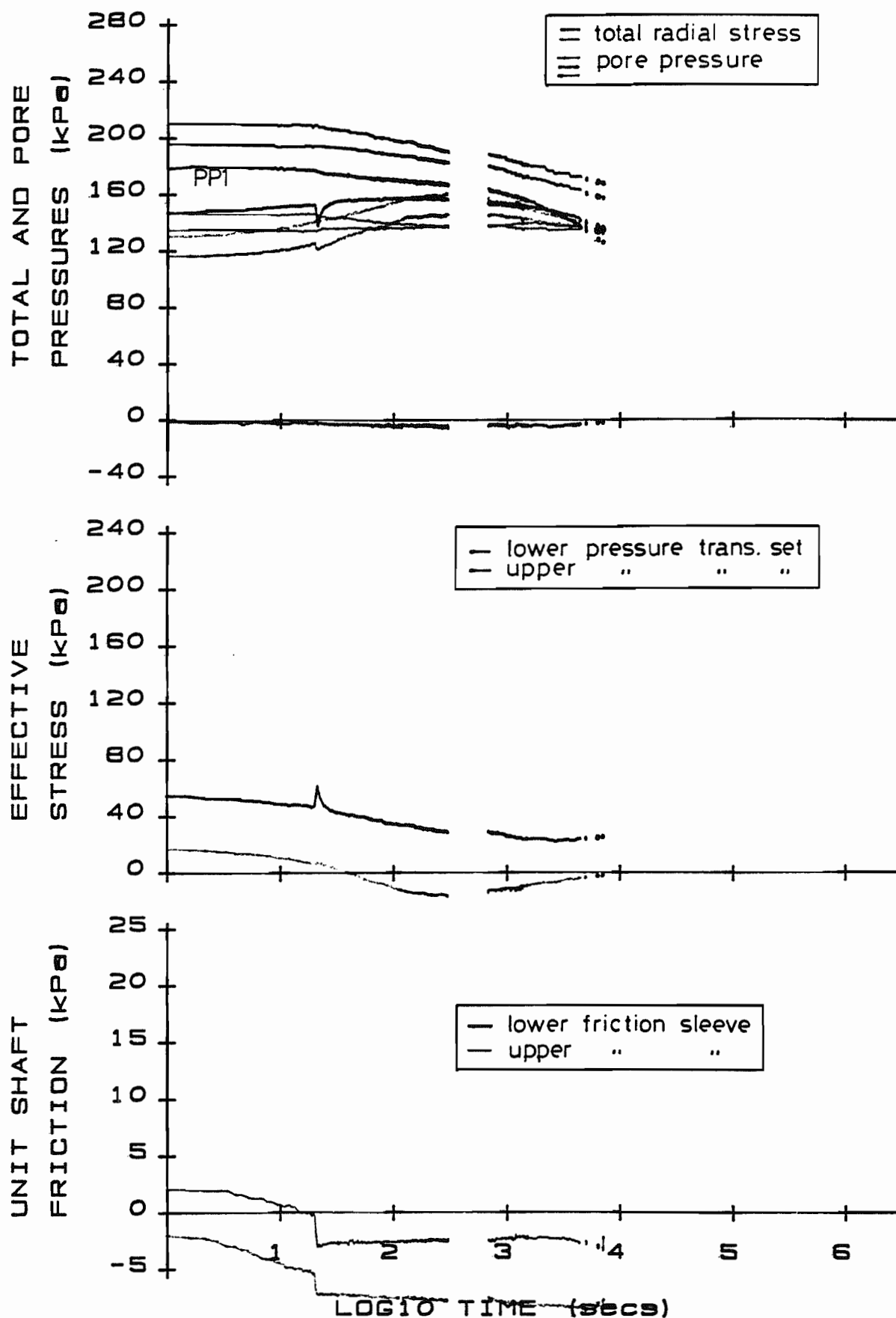


Figure 6.8 Partial Reconsolidation Data from Borehole J, Great Yarmouth.

Tip Penetration = 8.29m, Refer to Fig.3.8 for Key.



Figure 6.9 The IMP After Extraction from Borehole J.

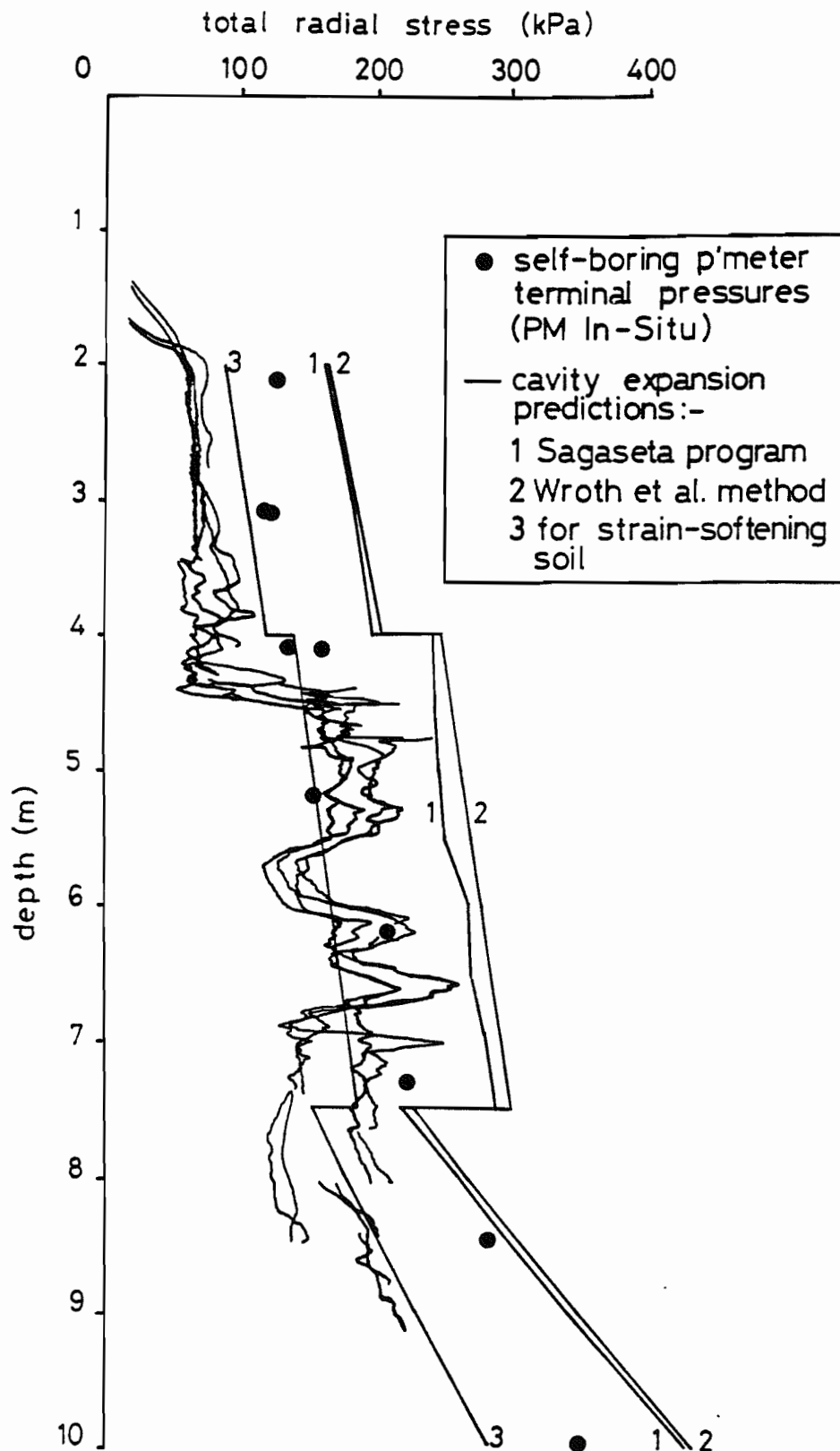


Figure 6.10 Comparison of Total Radial Stress Measurements on IMP During Penetration at Great Yarmouth with Cavity Expansion Predictions and Pressuremeter Data.

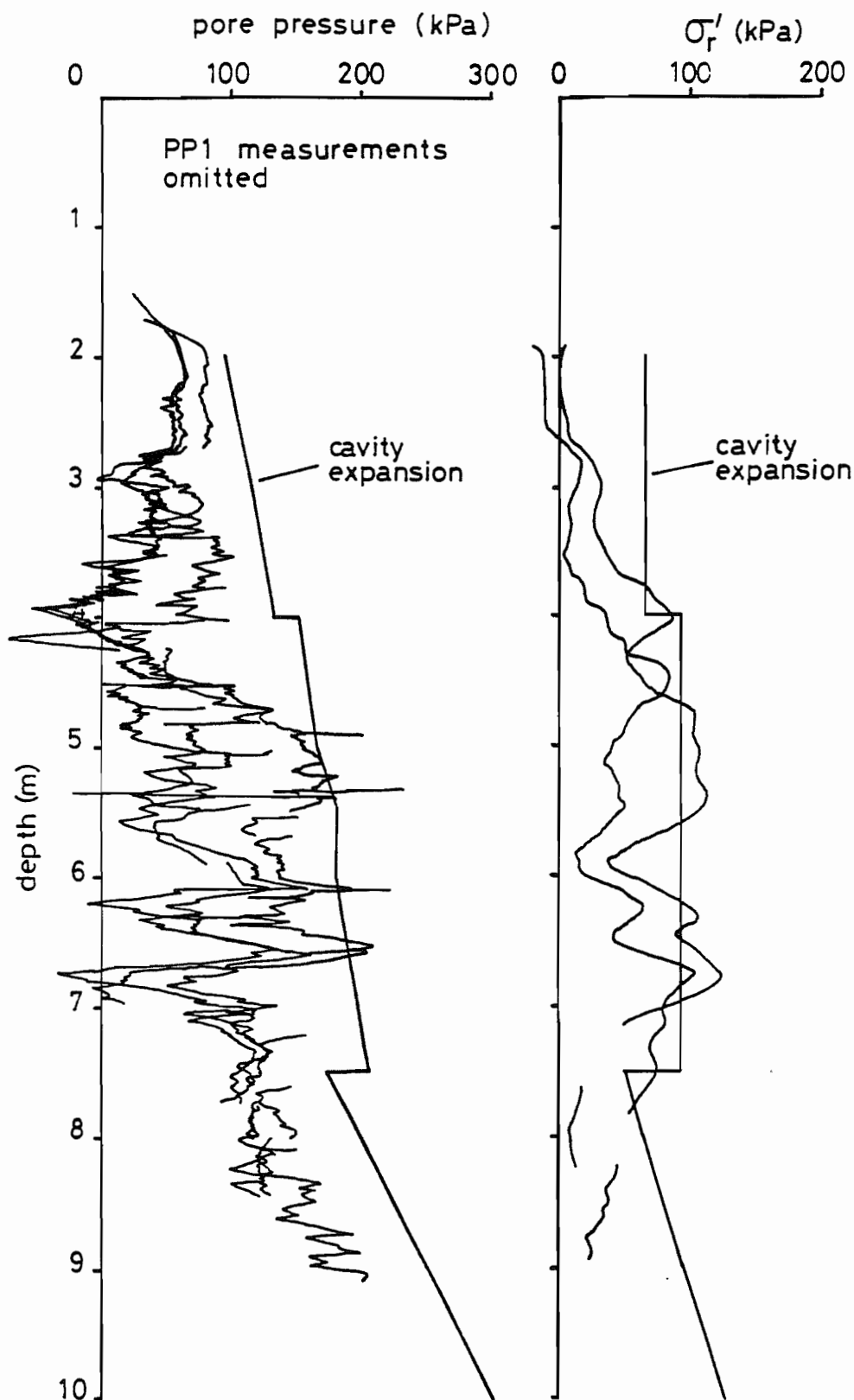


Figure 6.11 Comparison of IMP Pore Pressure and Effective Radial Stress Data for Penetration at Great Yarmouth with Cavity Expansion Predictions.

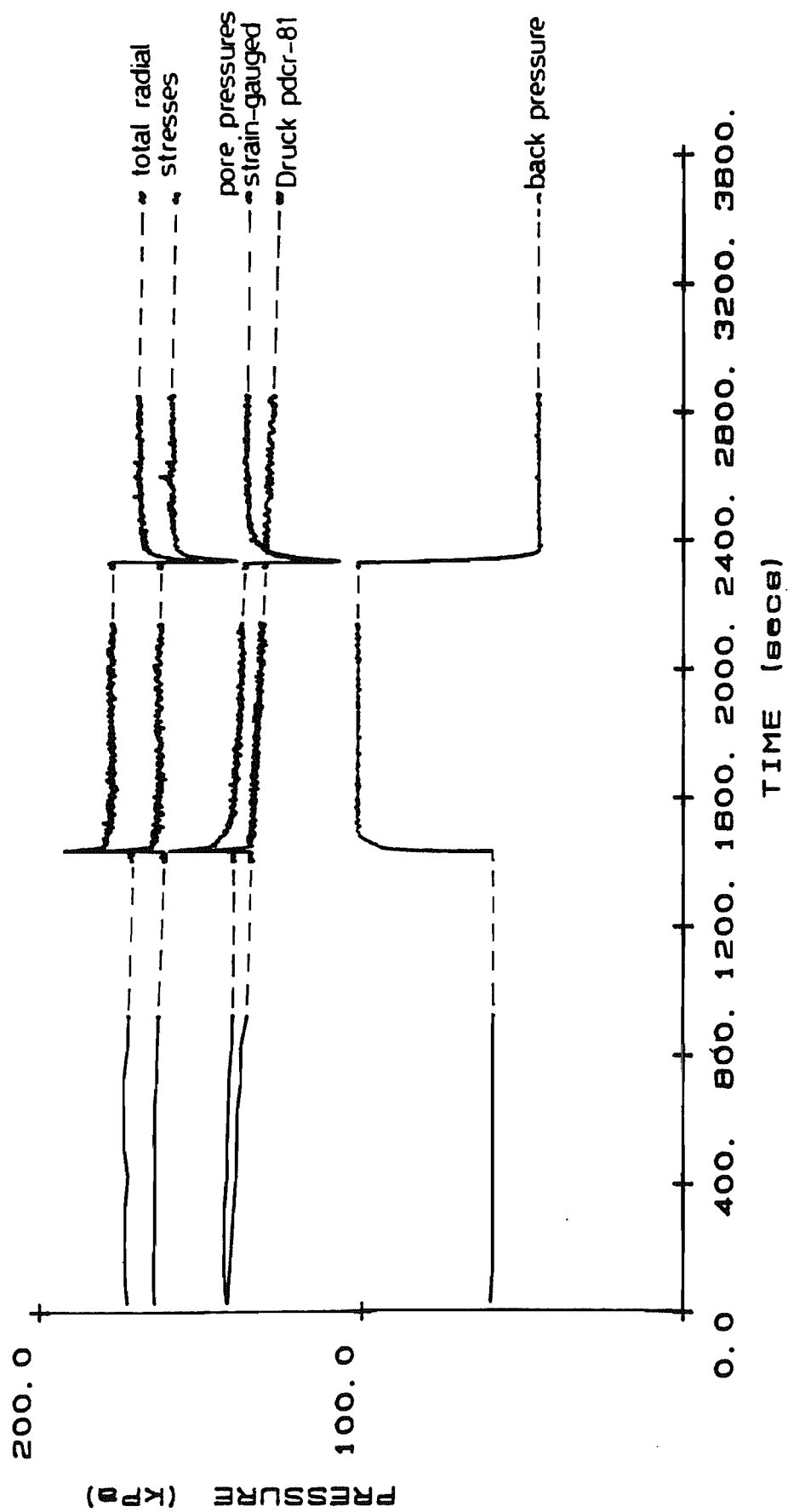


Figure 6.12 IMP Back Pressure Test Data, Great Yarmouth.

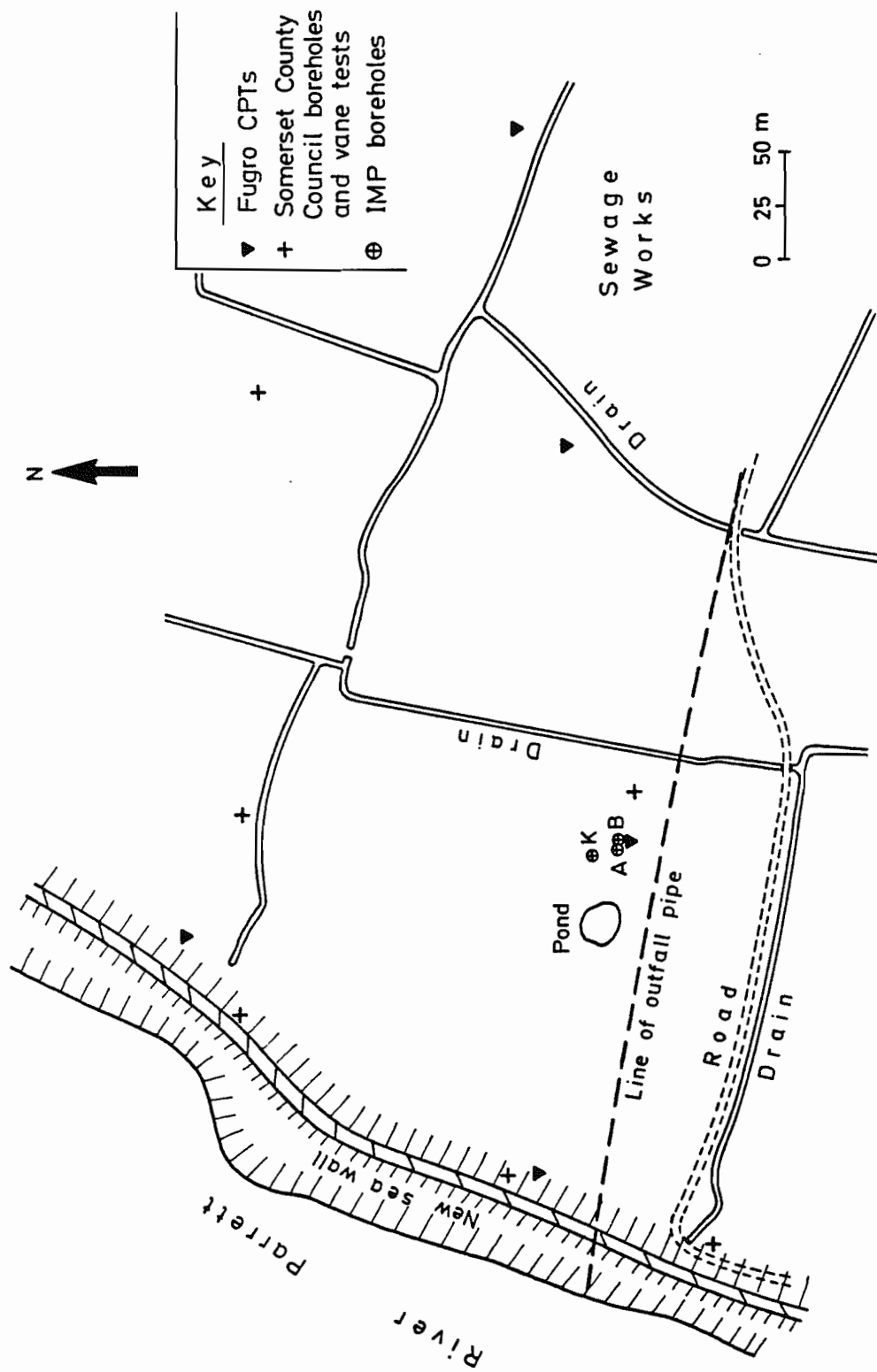


Figure 7.1 Sketch of the Huntspill Site (Part Only Shown -Data from Aspinwall & Co. 1981a)

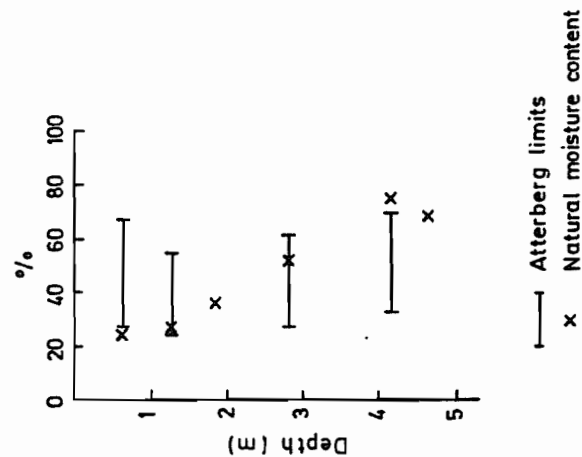
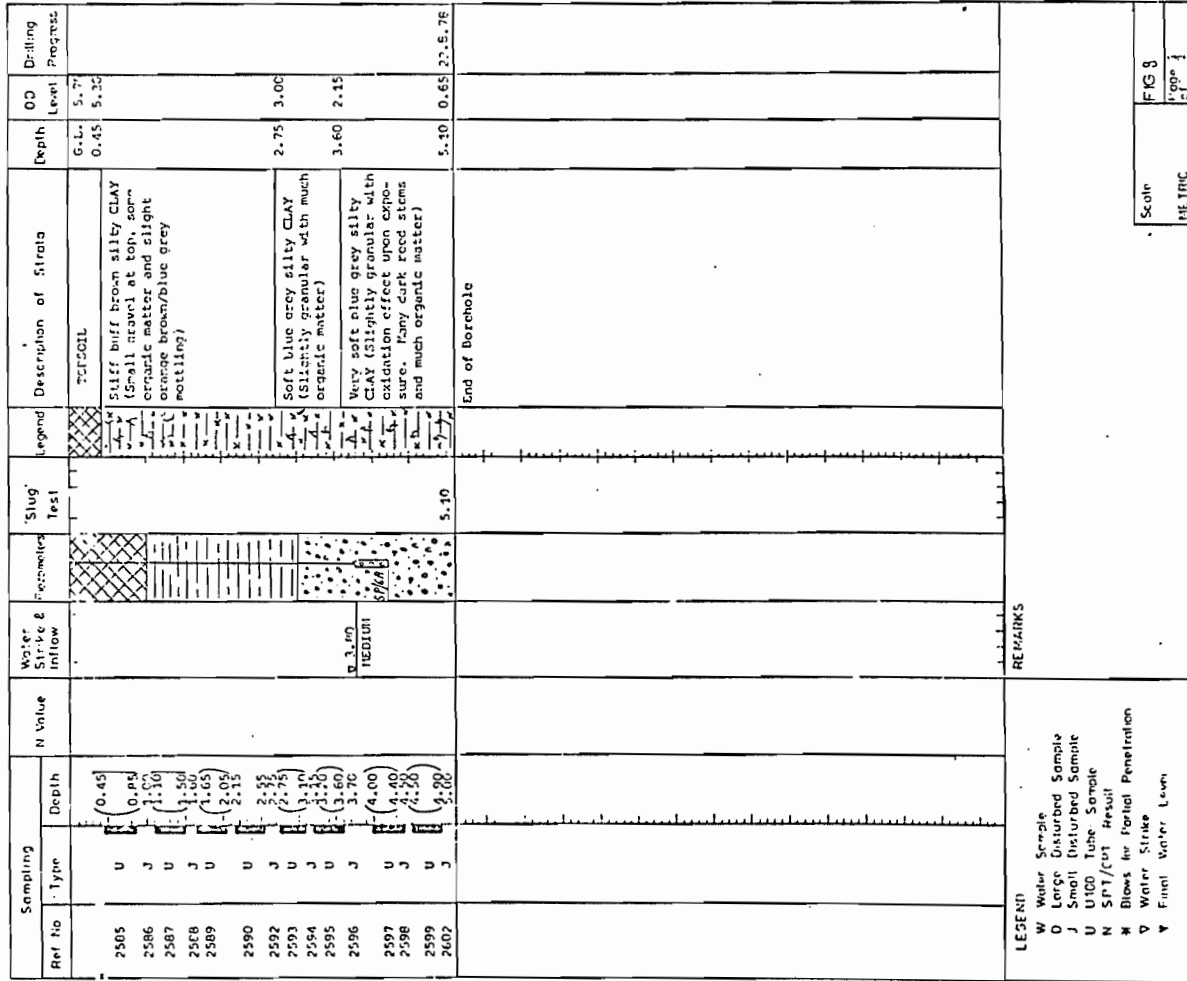


Figure 7.2 Borehole Profile (After Aspinwall & Co. 1981a) and Index Test Data (Replotted by Author) for Huntspill.

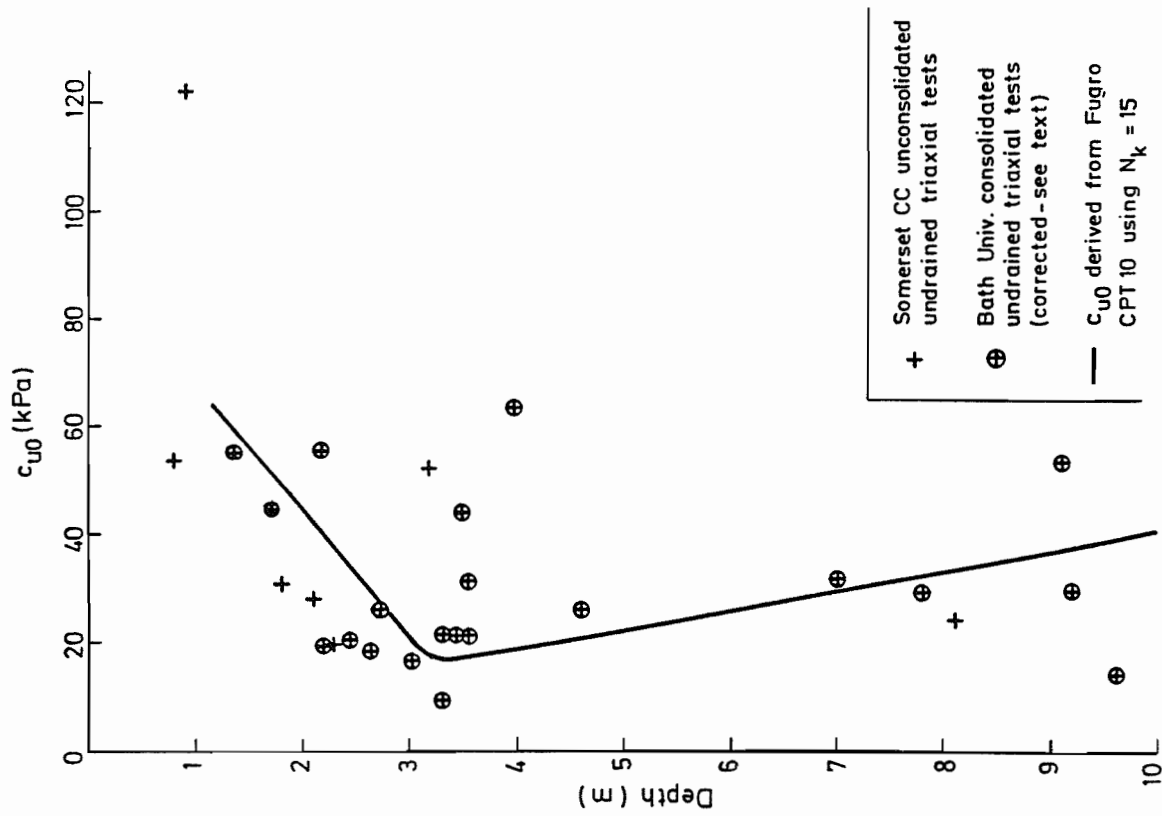


Figure 7.3 Undrained Shear Strength Data for Huntspill.

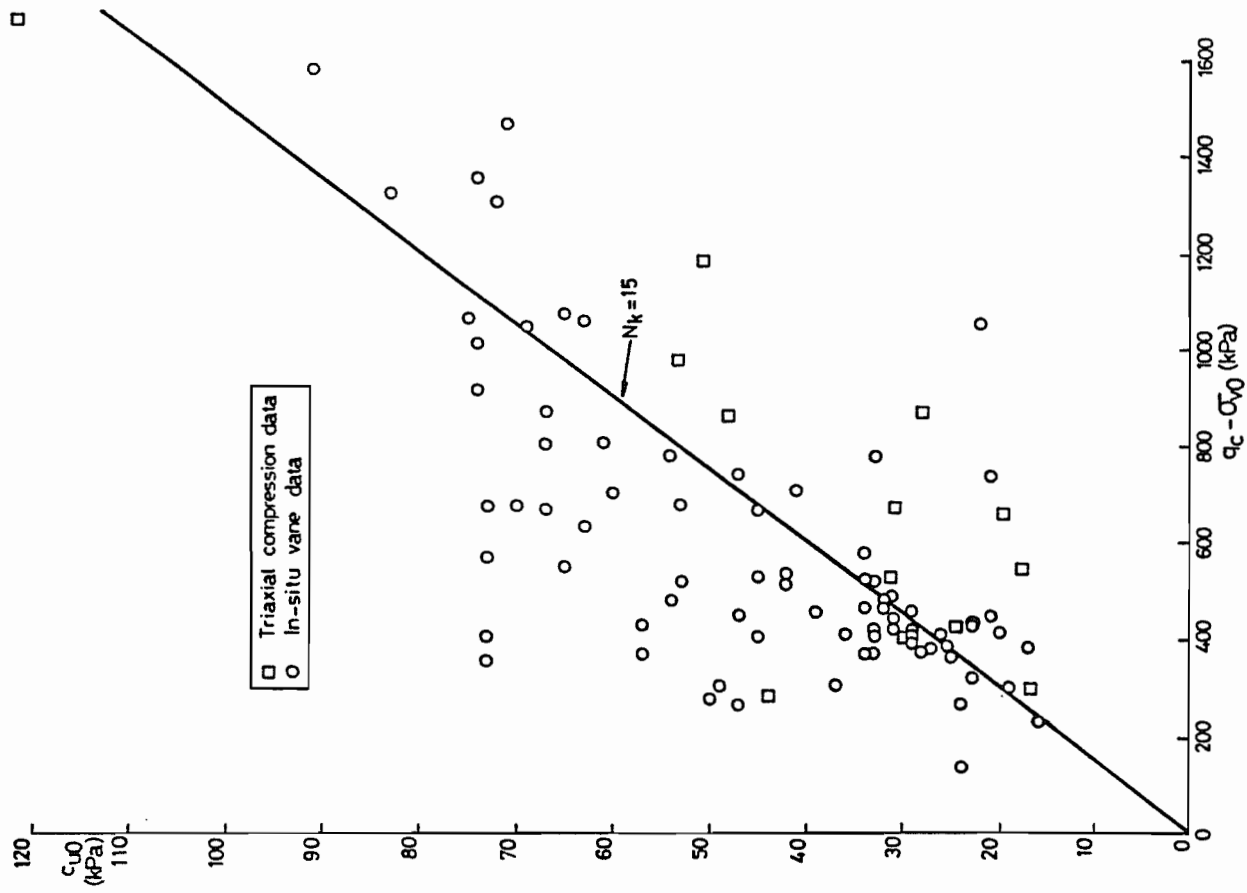


Figure 7.4 Cone Penetrometer N_k Factor Derivation, Huntspill

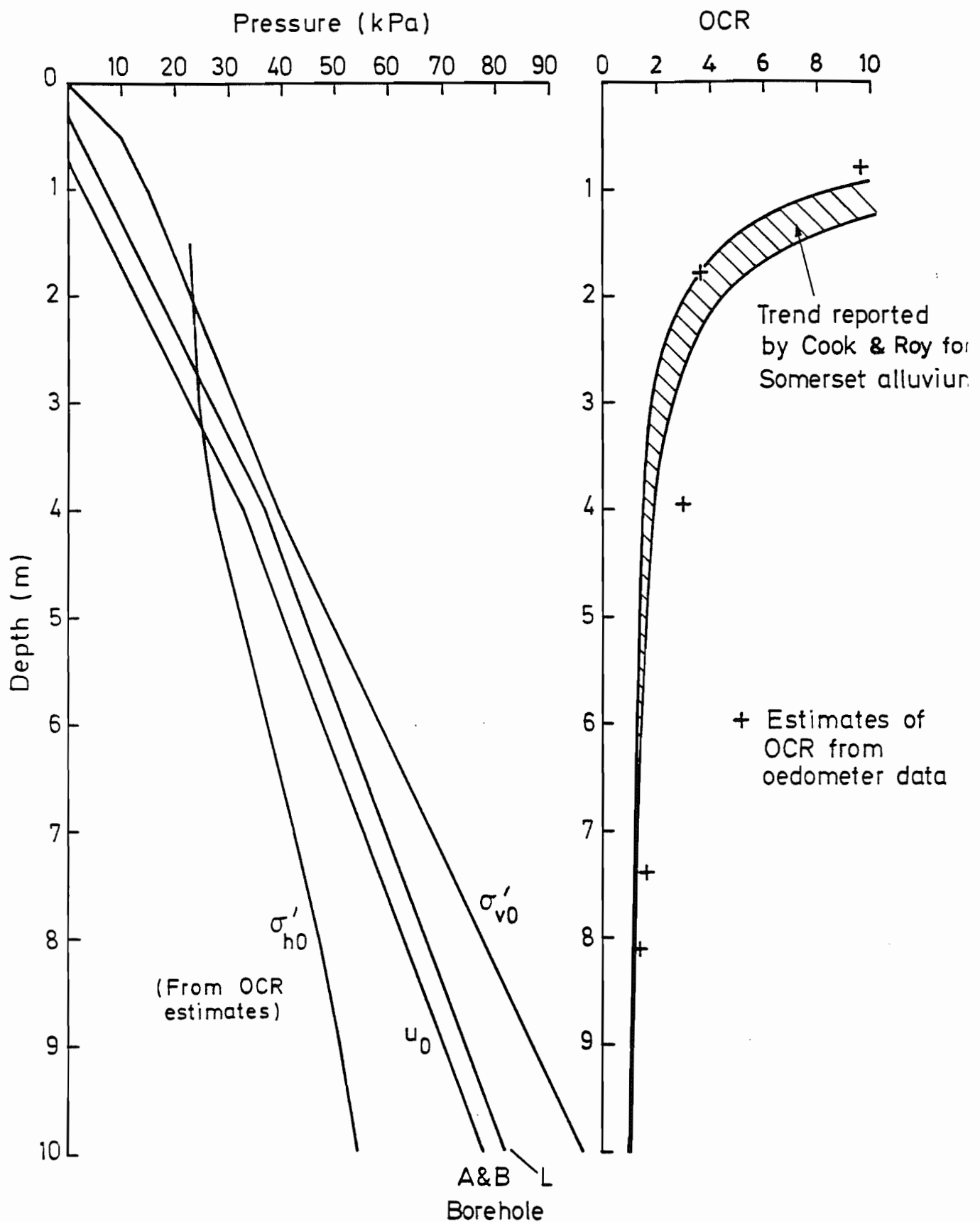


Figure 7.5 In-Situ Stresses and OCRs, Huntspill.
(Based on Data from Aspinwall 1981a)

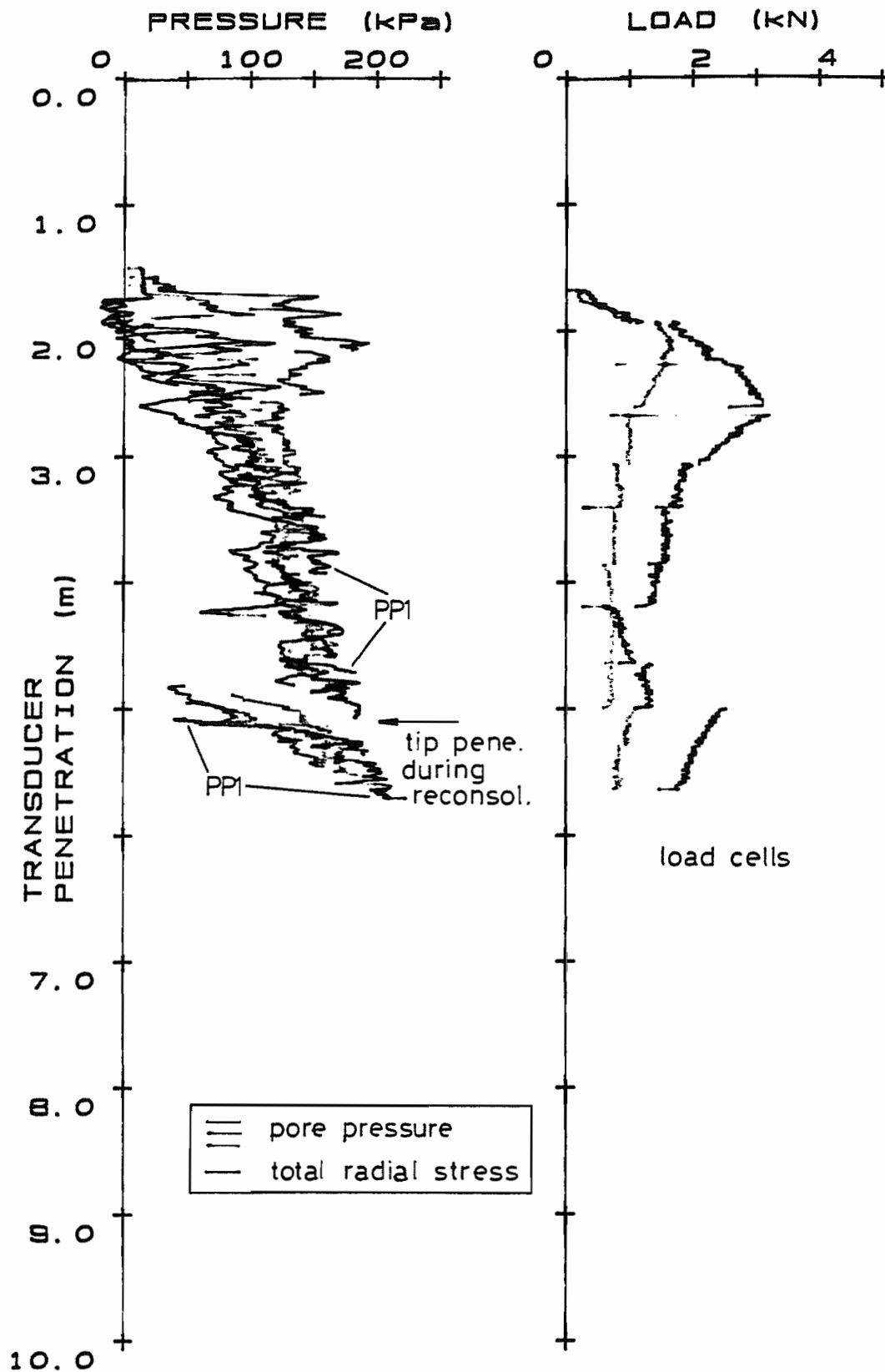


Figure 7.6 Raw Data Obtained During IMP Penetration in Borehole B, Huntspill.

(Refer to Fig.3.6 for Key)

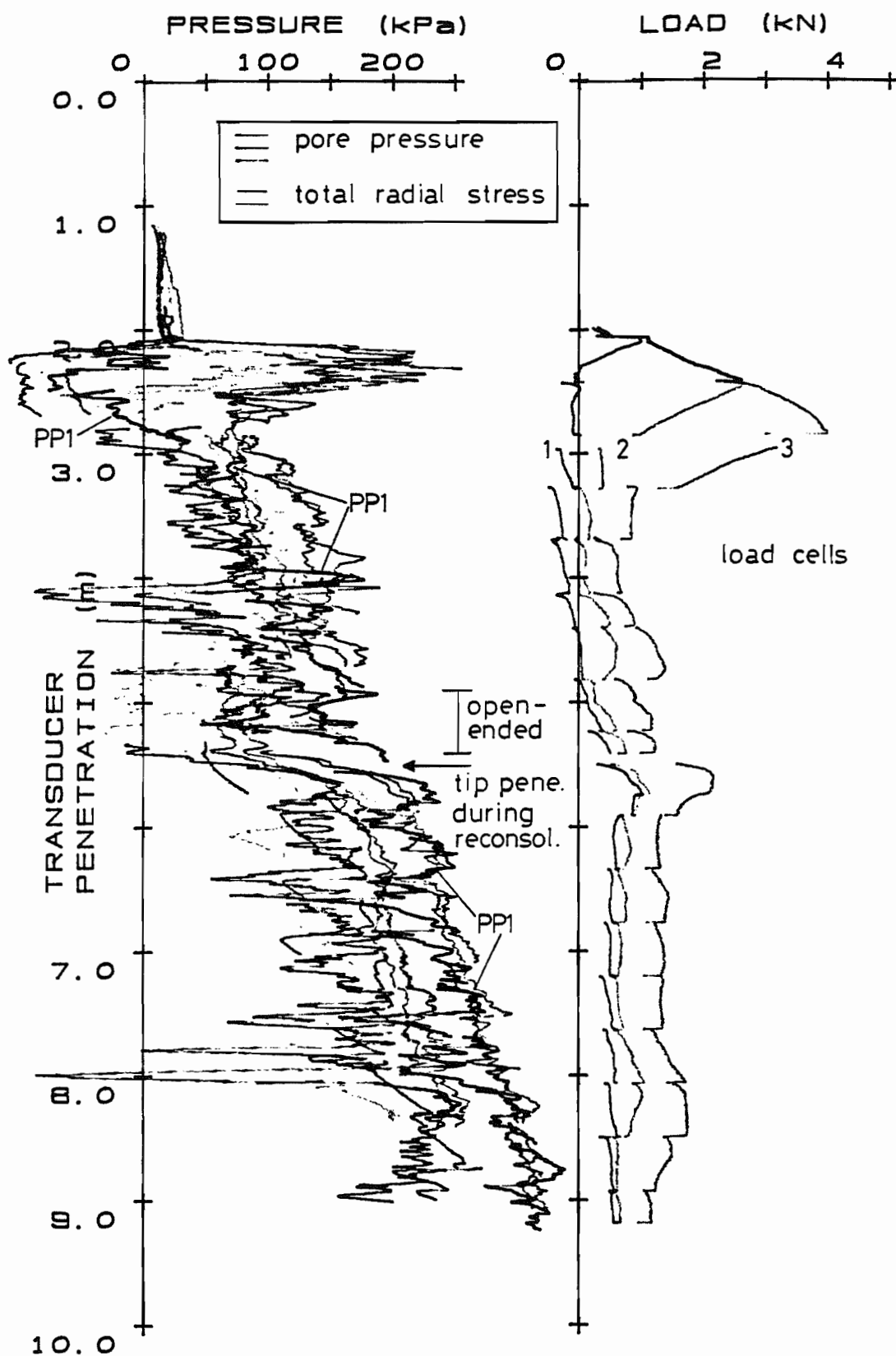


Figure 7.7 Raw Data Obtained During IMP Penetration in Borehole K, Huntspill.

(Refer to Fig.3.8 for Key)

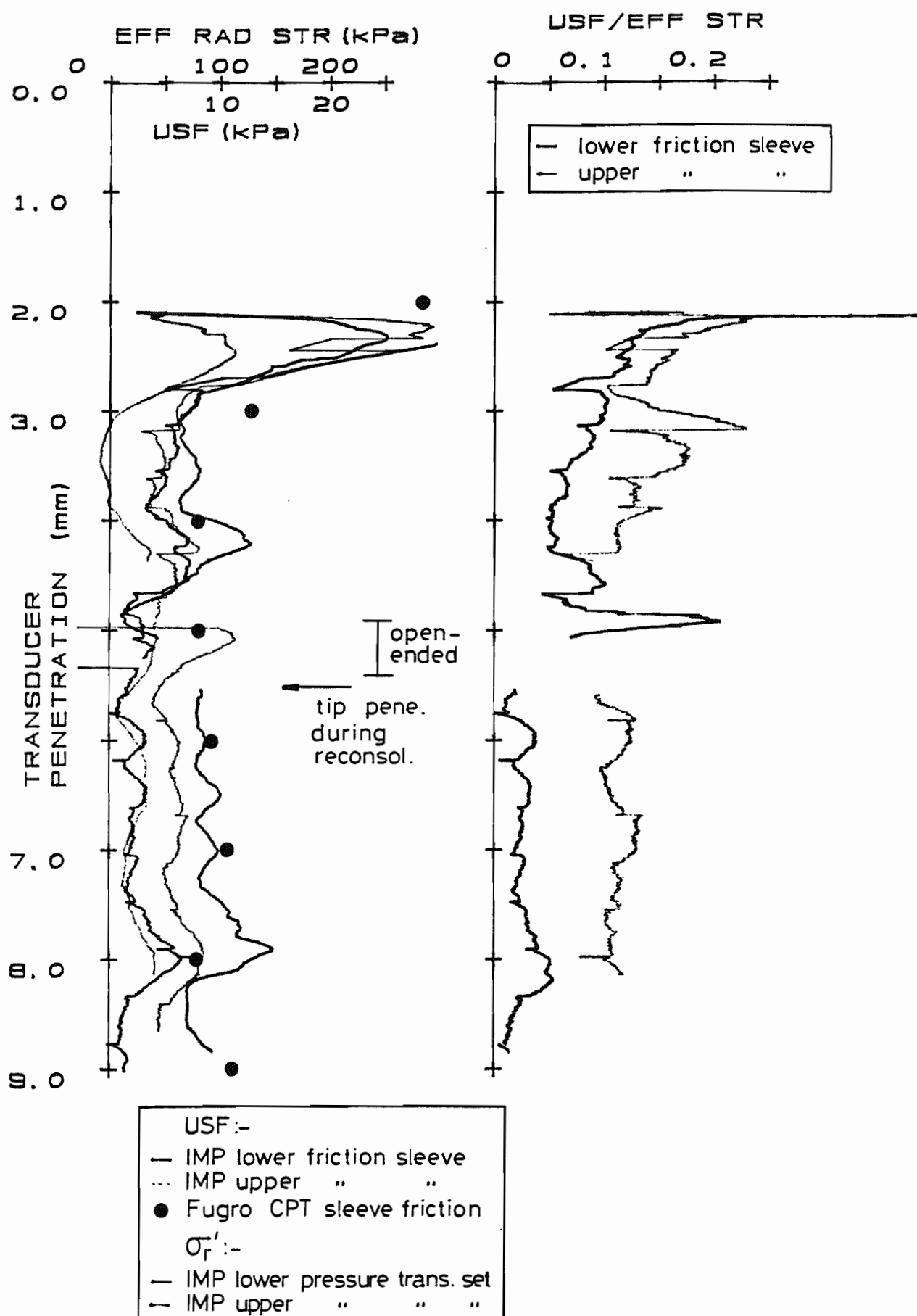


Figure 7.8 Interpreted Data from IMP Penetration in Borehole K, Huntspill.

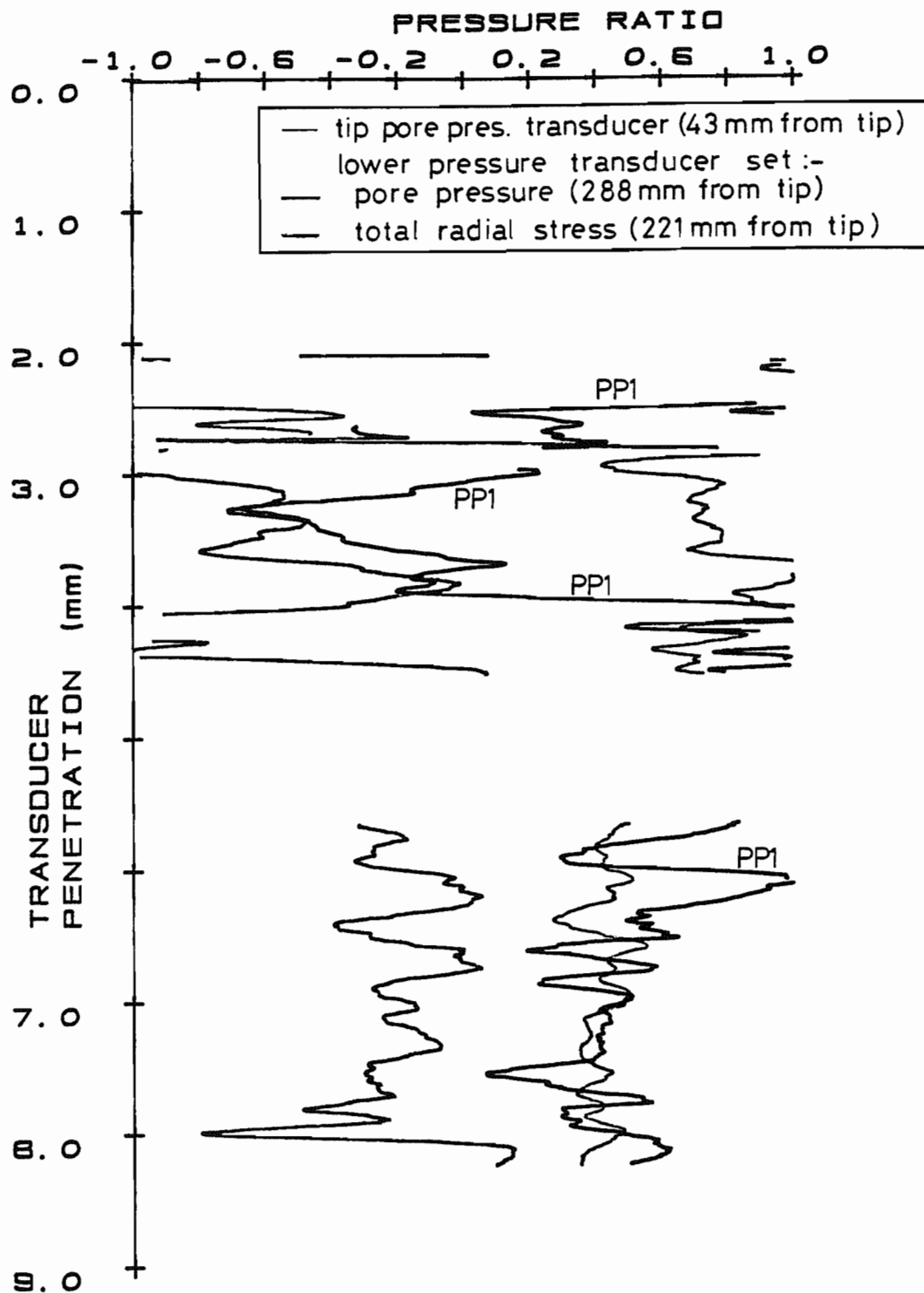


Figure 7.9 Borehole K, Huntspill: Variation of Stresses Along the IMP Shaft (see text for definition of pressure ratio).

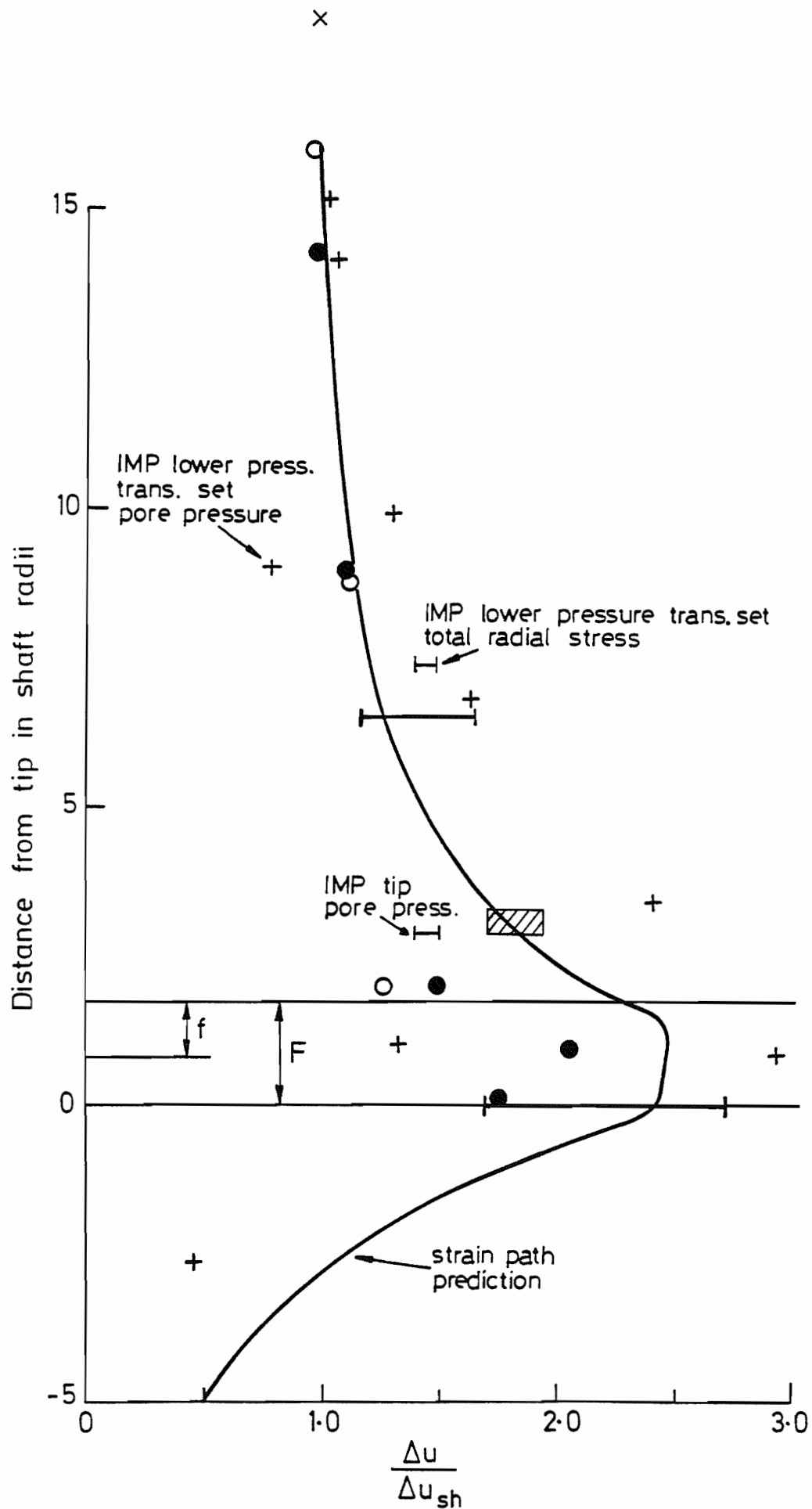


Figure 7.10 Comparison Between IMP Measurements and Levadoux & Baligh Predictions for Variation of Pore Pressure Along the Shaft of a Penetrating Instrument.
(See Fig 2.13 for Key)

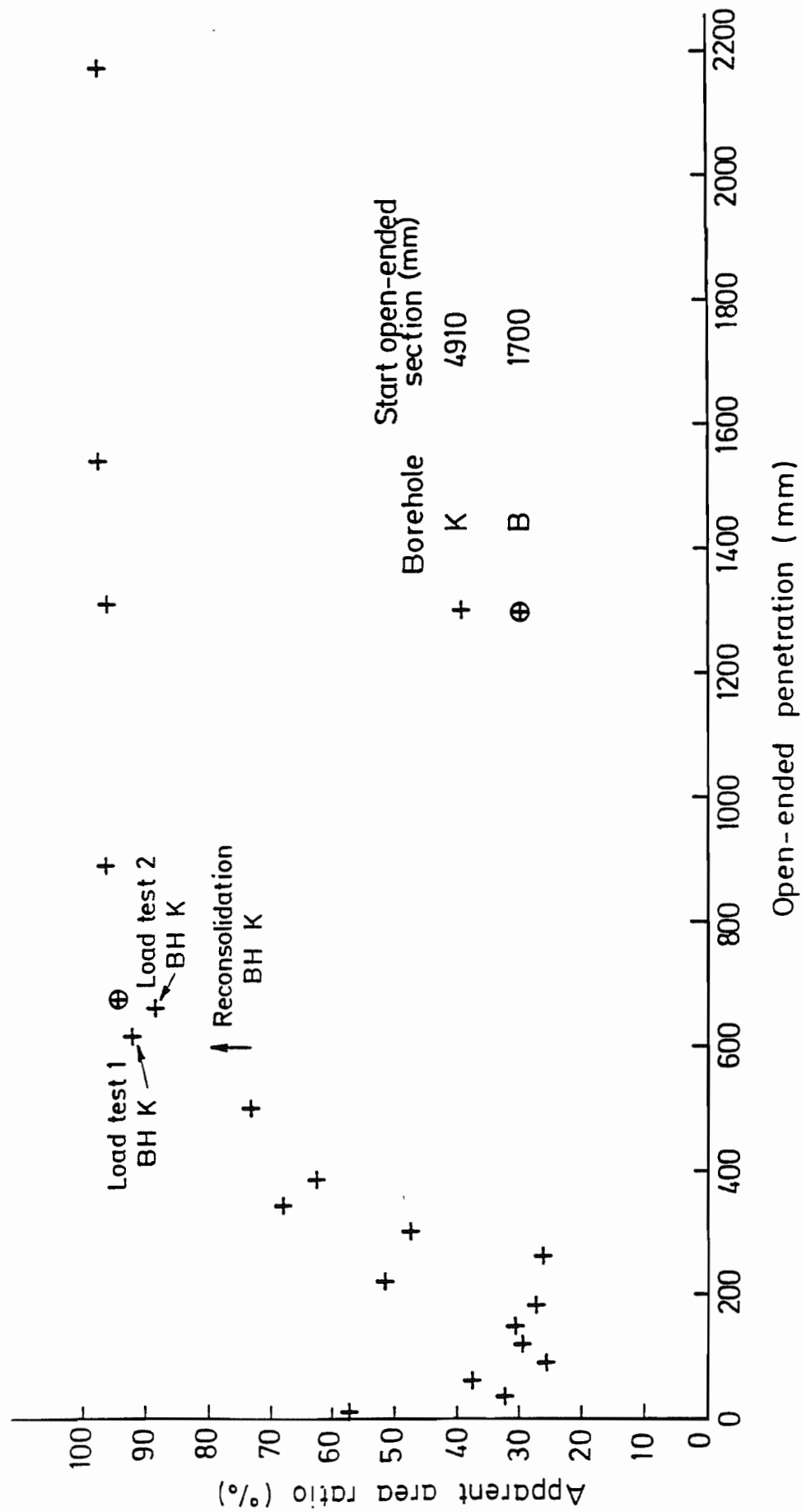


Figure 7.11 Apparent Area Ratios for IMP Open-Ended Penetration, Huntspill.

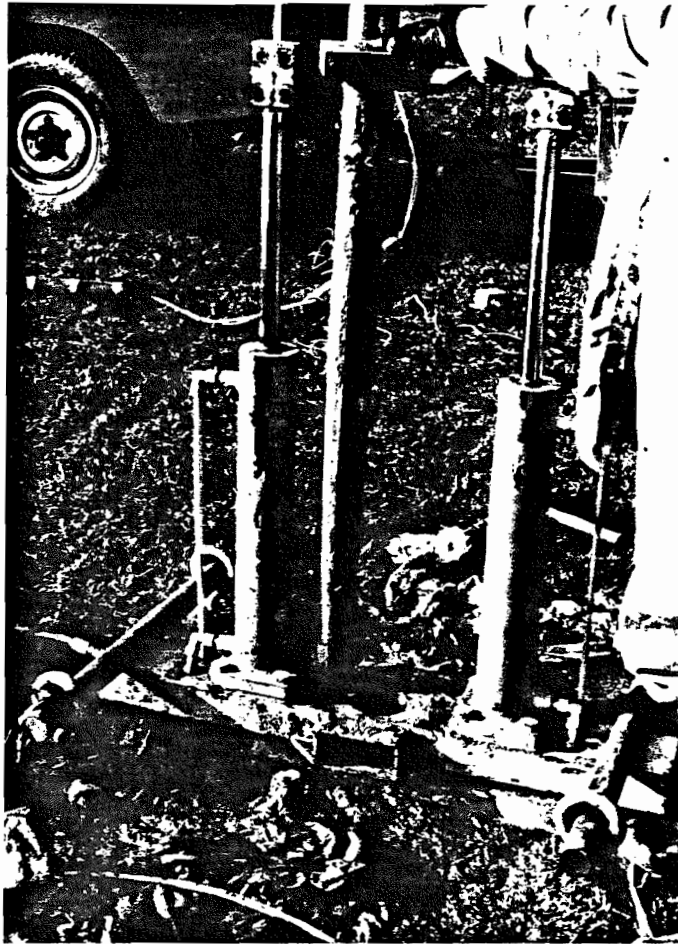


Figure 7.12 The IMP Immediately on Retrieval from Borehole K.

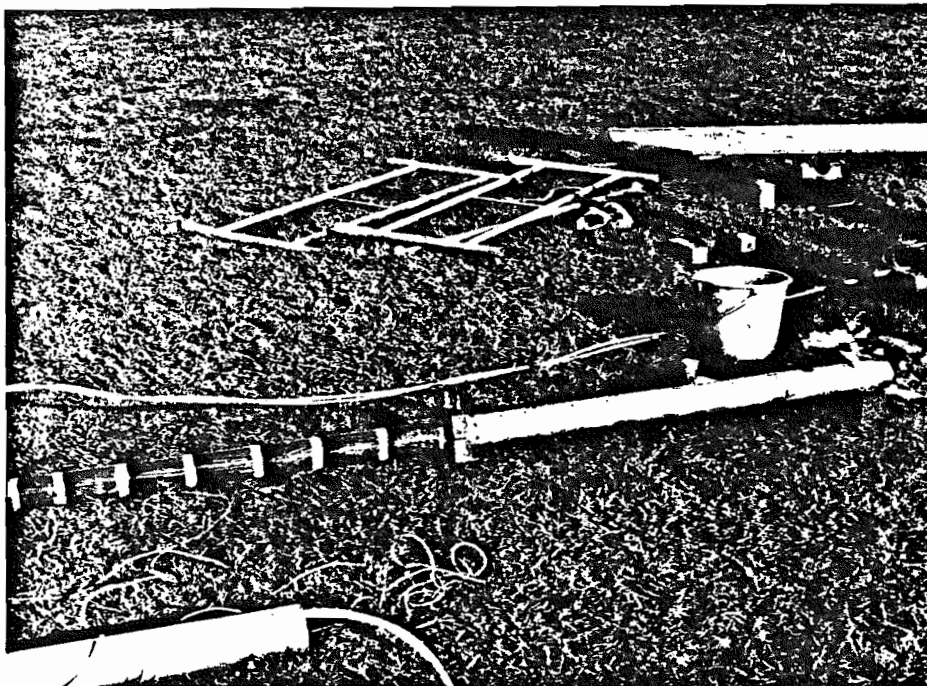


Figure 7.13 The IMP with Mud Removed After Borehole K.

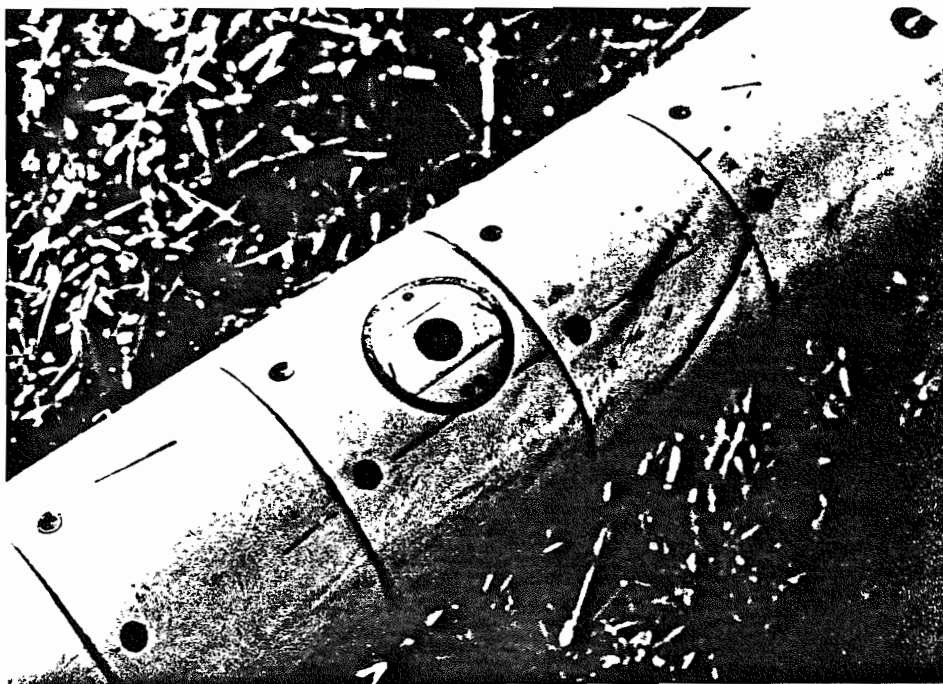


Figure 7.14 Remnants of Marker Pen Lines on IMP After Borehole K.

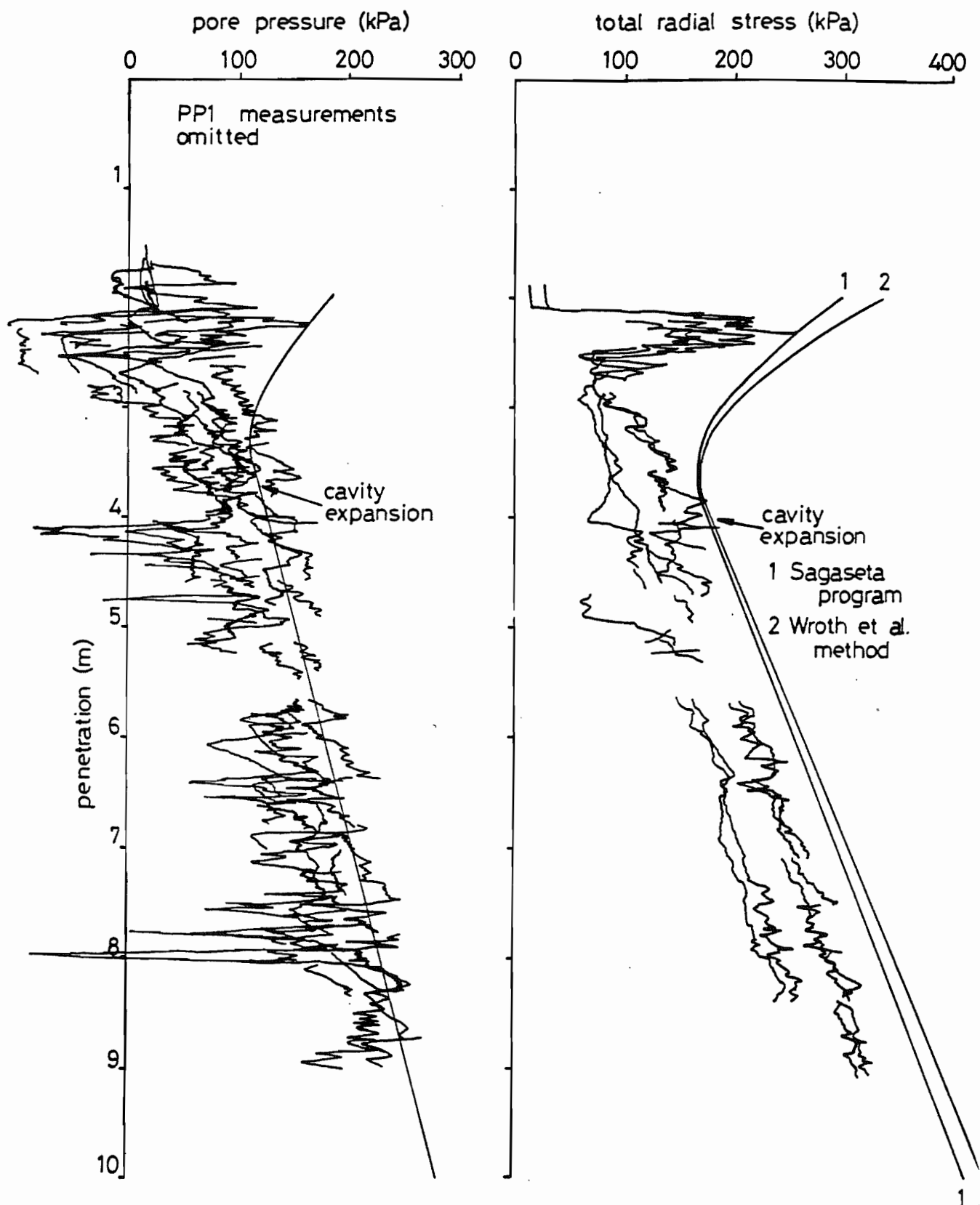


Figure 7.15 Cavity Expansion Predictions Compared to IMP Measurements During Penetration at Huntspill.

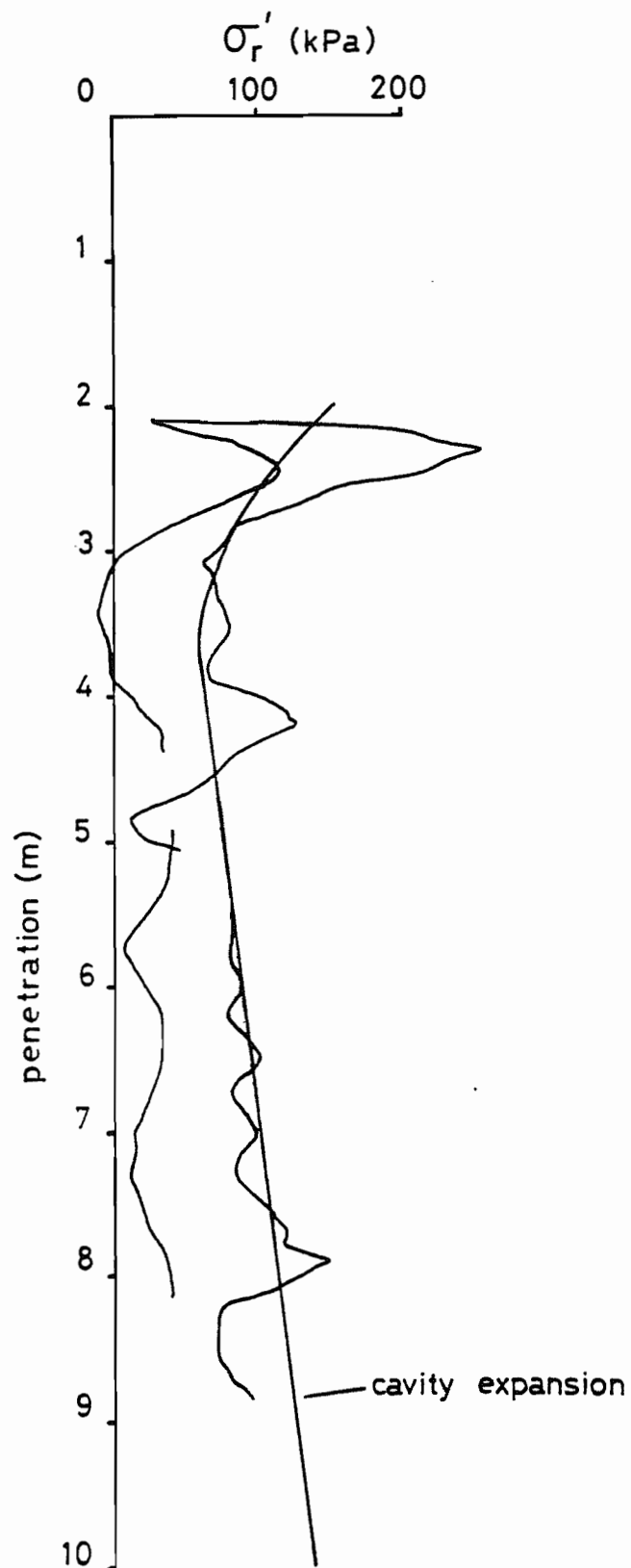


Figure 7.16 Comparison Between Radial Effective Stresses Calculated from IMP Penetration Data at Huntspill with Cavity Expansion Predictions.

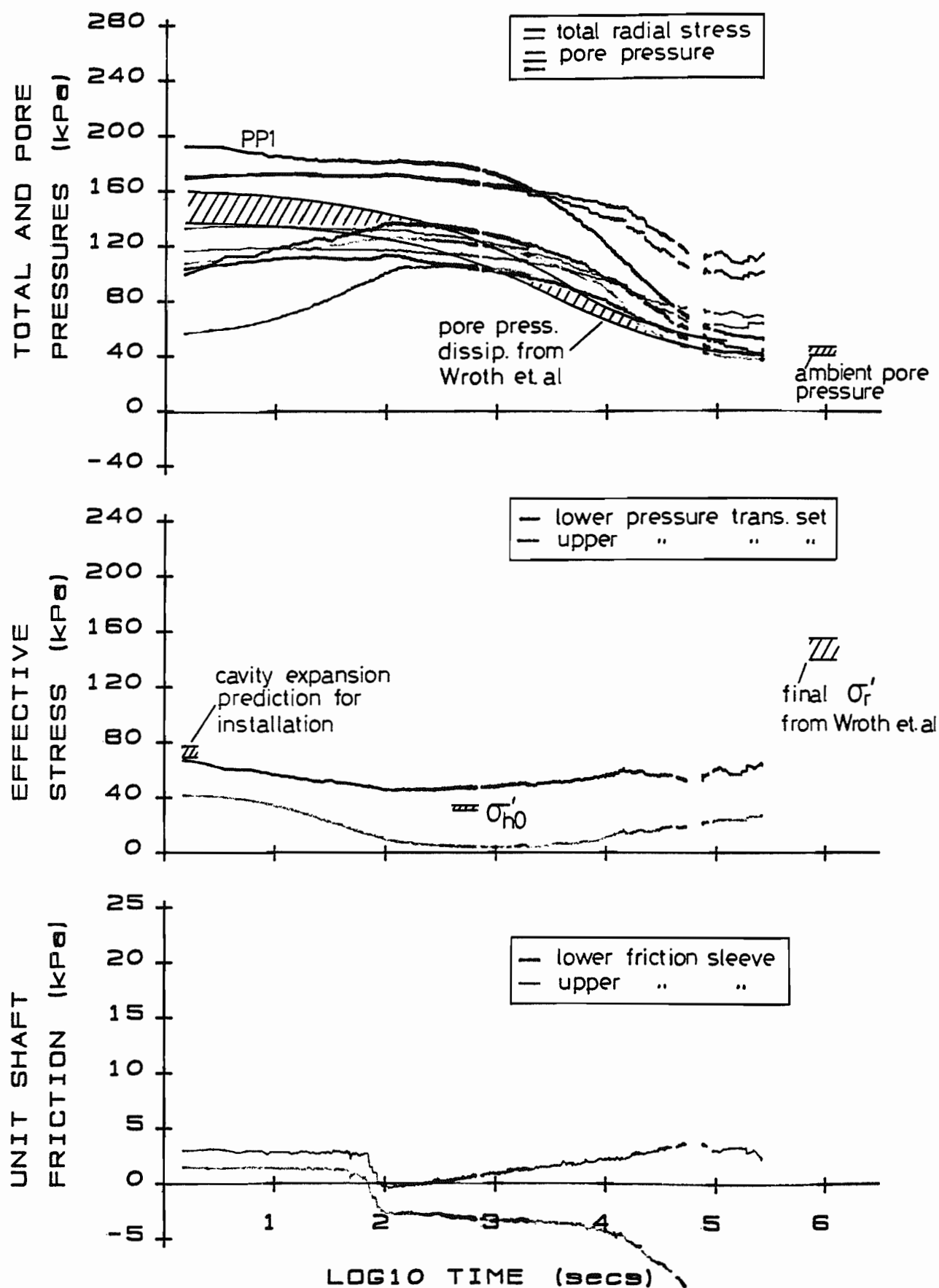


Figure 7.17 Reconsolidation Data from Borehole K, Huntspill.
Tip Penetration = 5.51m, Refer to Fig.3.8 for Key.

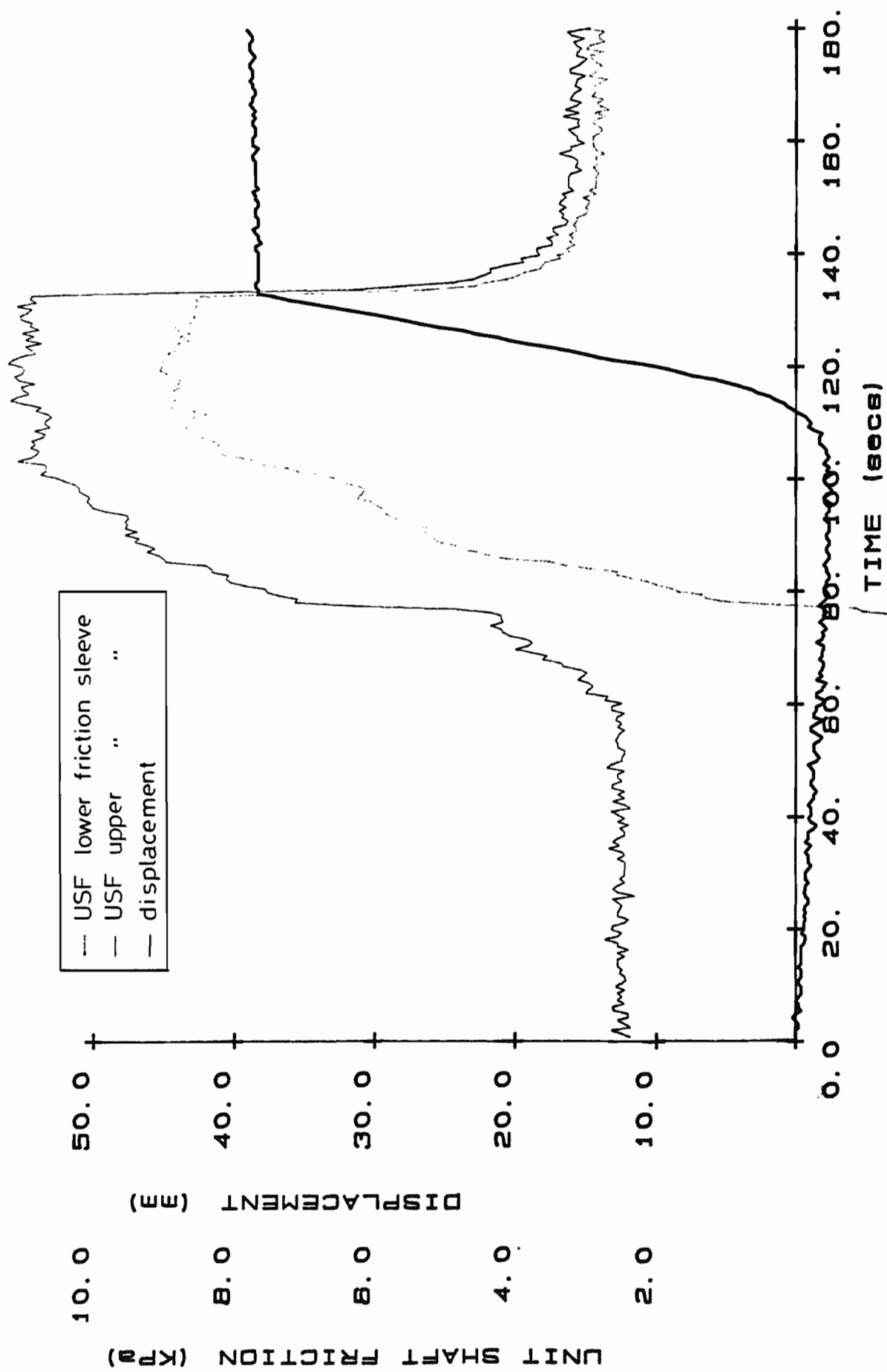


Figure 7.18 First Compressive Load Test, Borehole K, Huntspill: Unit Shaft Friction.

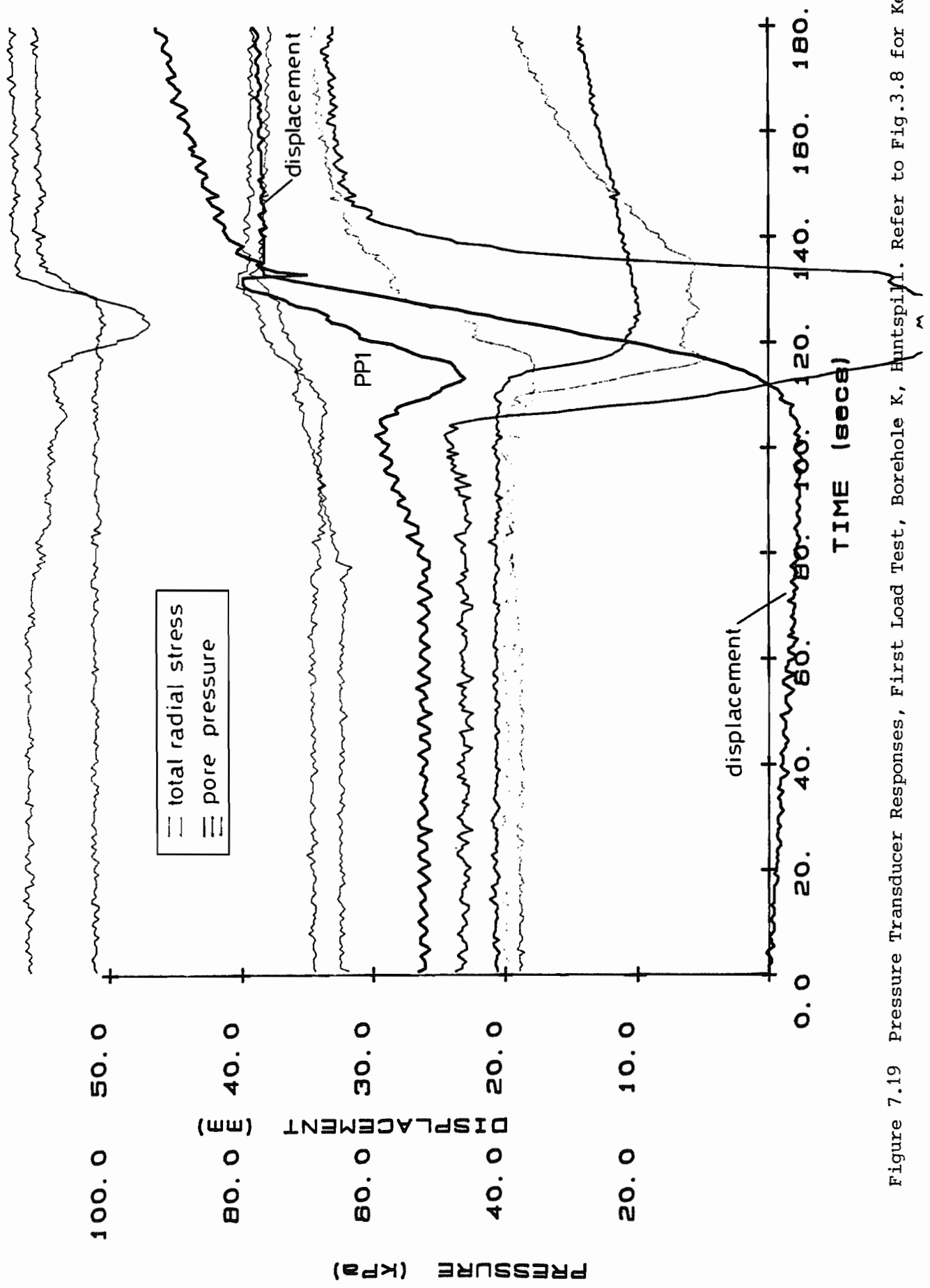


Figure 7.19 Pressure Transducer Responses, First Load Test, Borehole K, Huntspill. Refer to Fig.3.8 for Key.

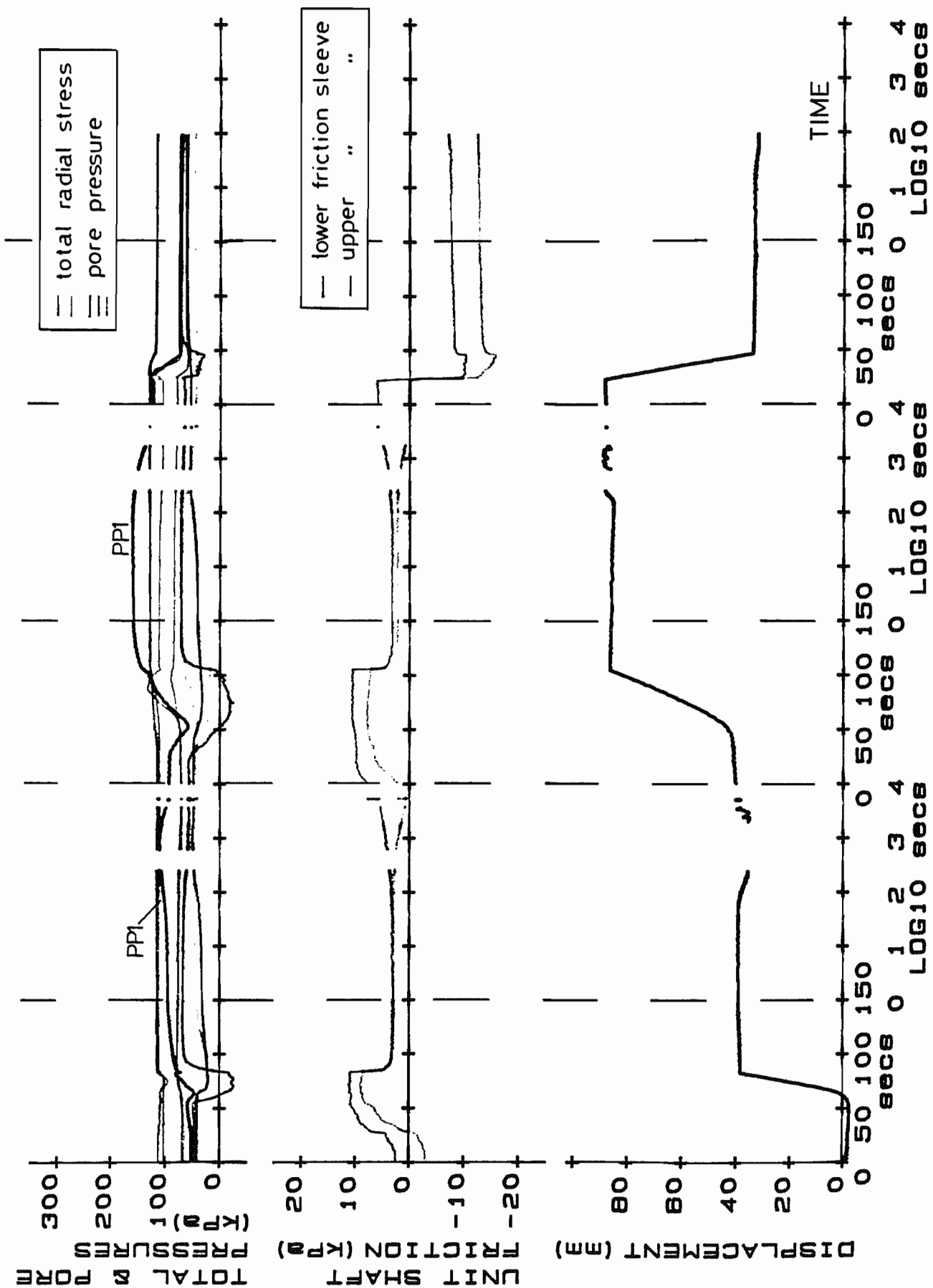


Figure 7.20 First Three Monotonic Load Tests, Borehole K, Huntspill. Refer to Fig.3.8 for Key.

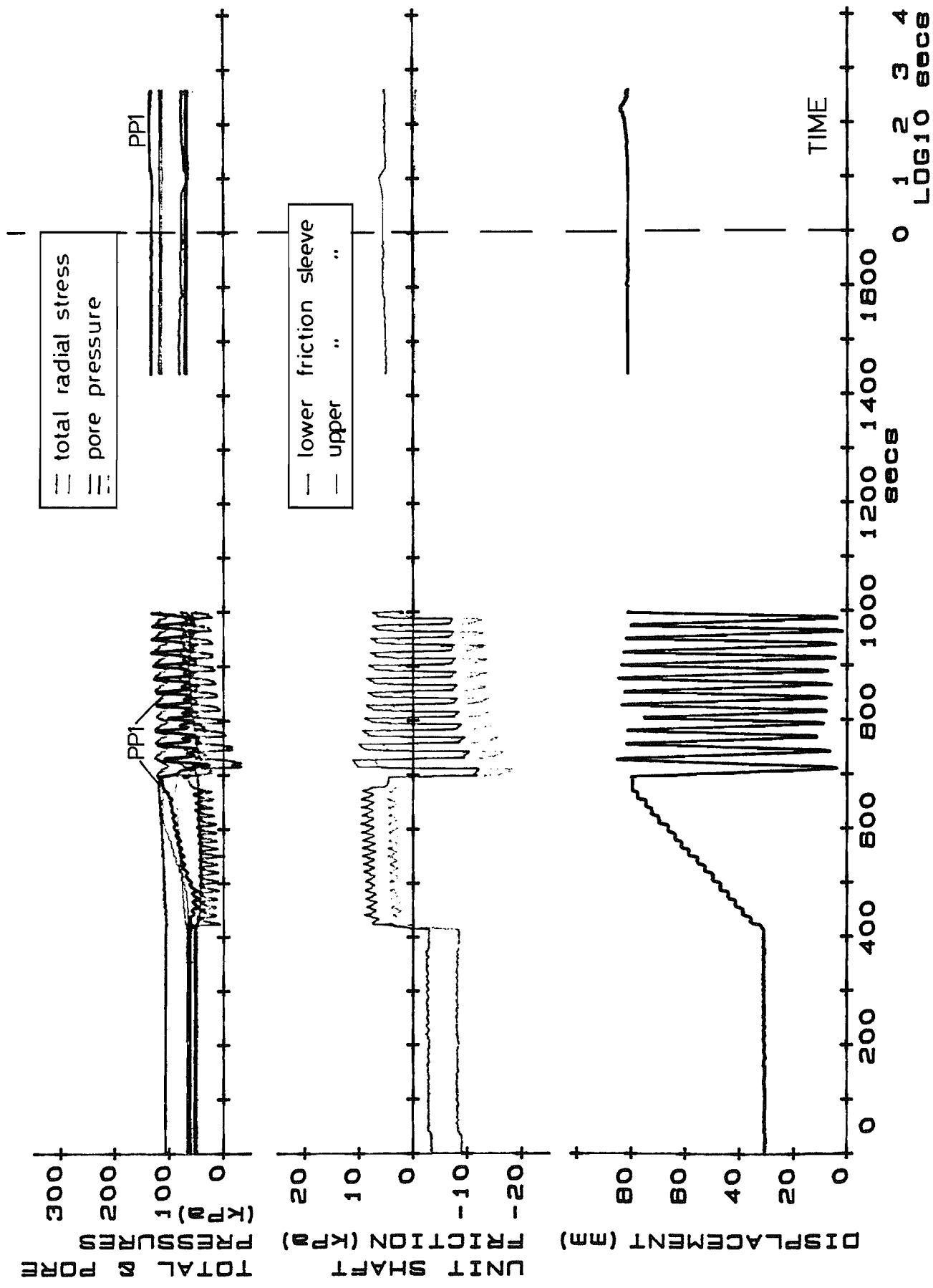


Figure 7.21 Cyclic Load Test, Borehole K, Huntspill. Refer to Fig.3.8 for Key.