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Second International Conference
on Behaviour of Off-Shore Structures
Held at: Imperial College, London, England
28 to 31 August 1979

DEVELOPMENTS IN PILING FOR OFFSHORE STRUCTURES

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Summary

This paper reports the results of driving and loading tests on two steel pipe piles driven into glacial till at the BRS Test Site at Cowden. The tests had two primary functions:-

- (i) To field test the prototype of a new hydraulic hammer.
- (ii) To assess theoretical predictions of the driving resistance of similar open ended and closed ended pipe piles and to compare their axial load carrying capacity.

The two piles were 0.45 m o.d. and were driven 9 m into a stiff till. One pile was open ended without a shoe and the other closed ended with a flat plate. Both were comprehensively instrumented with strain gauges, accelerometers, and pore water and total pressure cells.

The two piles were tested to failure under axial compression at a constant rate of penetration about one month after driving.

Wave equation studies of the pile and hammer are presented and comparisons made between measured and predicted axial capacity and stresses during driving.

1.0 INTRODUCTION

The past few years have seen the introduction of hydraulically operated drop hammers for pile driving. The mechanism of such hammers provides automatic control over the impact energy and the stroke rate. This makes them attractive for offshore applications. Over water and under water developments are envisaged. The first objective of the work described here was to field test a new hydraulically activated hammer by driving two test piles.

Theoretical calculations suggest that driving closed ended piles through clay would be easier than driving the same piles open ended, and that the former would have a higher capacity. However this has not been confirmed by full scale field tests. Therefore the two test piles were taken open and closed ended respectively and after driving were load tested to failure.

The final objective of the present work was to develop robust and reliable instrumentation for use offshore in the evaluation of pile installation and in-service behaviour.

2.0 THE HAMMER

The hammer used for driving the test piles was a BSP H.A. type having a ram mass of 3.5 tonnes. This type of hammer presents a driving system which is a new approach to basic pile driving principles. It provides the piling contractor with the simplest and most reliable of piling tools - the drop hammer together with modern technology control.

The drop hammer or driving mass is lifted by a loop that is actuated over a sheave at the top of the leaders, Fig. 1. The rope then passes down to the lower portion of the mast and round a sheave at the bottom of the actuator. The end of the rope is then fixed to the hammer guide carriage. As the hydraulic ram in the actuator forces the sheave downwards the hammer lifts and when the ram returns the hammer drops under free fall.

The system offers many advantages that are self evident such as simple drop mass; isolation of mechanism from impact area; easy visual control of drop; low front end weight and soft driving ability. In addition there are advantages that are embodied in the actuator technology. These include: automatic remote control and the ability to vary the relationship between drop mass, engine horsepower and blow rate. This means that the H.A. user has a piling system that can readily adapt to suit changes of pile contracts.

The energy is related to the drop mass and a range of hammer masses are available. The stroke is infinitely variable between limits of 200 to 1370mm. Generally the hammer works at 40 blows per minute when it produces its maximum energy, but this can be increased to 60 blows per minute when approximately half the maximum energy is produced. The hammer is controlled by a remote electro hydraulic system allowing the stroke to be adjusted. It can be controlled for single blow operation or for a fully automatic sequence.

The system is suitable for soft driving as the loop of rope allows the hammer to follow the pile without restriction. Piles can be driven with rakes up to 30° to the horizontal. Driving can be carried out at refusal for long periods of time since the actuator is isolated from the impact zone. Power supply is from a diesel hydraulic power pack.

3.0 DESCRIPTION OF SITE

The tests were carried out at a site at Cowden on the coast of Holderness, north of Kingston-upon-Hull, which has been used extensively by the Building Research Establishment as a test bed for the evaluation of the properties of glacial till and the performance of site investigation equipment, Marsland (Ref. 1, 2). At this site there is a sequence of clay tills which were laid down during a series of glacial advances, having plasticity indices ranging from 12 to 20 per cent and a clay fraction of 30-40 per cent. Layers and lenses of fine sand are contained within the sequence. Fig. 2 gives the undrained shear strength variation over the top 13 metres as obtained by Marsland (Ref. 2) using 865mm diameter plate bearing tests. These tests were performed in an area about 200 metres distance from the test piles, but numerous CPTs have been carried out over the surrounding area and the general uniformity of the clays' properties established. Four records of cone tip resistance and one of sleeve friction from tests performed in the immediate vicinity of the test piles are shown in Fig. 2.

The water table is close to the surface. A plan of the pile test area is given in Fig. 3 showing the position of the instrumented piles A and B, and the adjacent cone penetrometer tests. The ten piles surrounding the test piles were used for reaction when the static tests were performed. These were cast in-situ in holes left by the extraction of a closed ended vibropile.

4.0 PILE INSTRUMENTATION

Details of test piles A and B are shown in Fig. 4. A is identical to B except that the 25mm thick base plate is omitted. The piles were 457mm o.d. and 19mm w.t..

The behaviour of the hammer during driving was observed by high speed photography. Strain gauges and accelerometers were attached to the outside wall of the pile as shown in Fig. 4.

The high speed camera operated at 200 f.p.s. The position of the hammer relative to a fixed scale painted on the guides was observed. This enabled the displacement-time behaviour to be obtained. The impact velocity of the ram could then be calculated.

The strain gauges were Ailtech SG 129, 120 ohm weldable gauges. They were positioned as shown on Fig. 4. At each level there were two gauges attached diametrically opposite each other. The leads from each gauge to the signal conditioning units were individually screened in order to reduce noise to a negligible level. The signals were then stored on magnetic tape. By summing the signals or differencing them at each level axial and bending stresses could be deduced.

At the pile head two PCB 5000g accelerometers were attached diametrically opposite each other. These were screwed into small metal blocks welded to the pile wall. During the driving of Pile B a mains supply was used for the accelerometers. An earth loop was established as the pile penetrated the soil and the noise derived therefrom masked the signals. The difficulty did not arise when an independent power supply was used. Screened cables were used to connect the accelerometers to the signal conditioning units.

All sensors and associated cables were protected by 38x38mm angles welded along the length of the pile, Fig. 4. Details of the driving head are shown in Fig. 5.

The pressure cells used were of the strain gauged diaphragm type and were initially constructed by Transducers (CEL) Ltd especially for fitting to a 1372mm diameter pile which was driven in the BP Forties field. The pore pressure cells differed from the total pressure cells only in the fact that the 28mm diameter

active face was covered by a porous disc. The pressure cells operated over a range of 0-4137 kN/m².

5.0 DESCRIPTION OF TESTING

5.1 Driving Tests

Pile A was driven on Tuesday, 2nd May over a period of 45 minutes. There were two breaks in driving to prime the pore pressure cells and one break to paint a continuation of the penetration markings. The following morning the pile was re-driven a further 150mm.

Pile B was driven on the following day to 2.1m and then extracted. It was then driven over a period of approximately 30 minutes with two interruptions.

5.2 Loading Tests

The pile loading tests were carried out using BRE's plate loading test equipment which has a maximum capacity of 500 tonnes. Fig. 6 shows the equipment set up in position over test pile B. Reaction was obtained using two spreader beams which were each fixed to three of the tension piles. Head displacement was measured relative to a reference beam which was supported on two concrete A blocks placed outside the region of influence of the piles. Load was measured using a load cell placed between the hydraulically operated loading head and the pile cap. During testing all quantities were recorded visually and remotely.

Each pile was first tested under various levels of maintained load over a period of around three days, left for one day unloaded, and then loaded under a constant rate of penetration, eventually to failure. The rate of loading was 0.33% of the pile diameter per minute. Tests on pile A were completed before the test equipment was moved to pile B. Both piles were left for a period of around one month between driving and testing to failure.

5.3 Pressure-Measurements

The signals from the twelve pressure cells were recorded on a Solatron Compact data logger with tape cassette at a rate of four channels per second during the driving phase. The interval between successive scans was then increased according to the expected rate of change of the signals. During driving efforts were made to maintain the pore pressure cells in a deaired state. This was done by placing the porous discs in position in a water filled trench once each cell had gone just below ground level.

6.0 DRIVING RESULTS AND INTERPRETATIONS

A large volume of test data was collected. Only a representative selection of this is given herein.

6.1 Hammer Impact Velocity

Nine high speed films were made covering a range of penetrations for piles A and B. From these fifteen hammer displacement-time curves were constructed using frame by frame analysis of the negative. The average impact velocity of the hammer for piles A and B was found to be 4.11 m/s and 4.21 m/s respectively.

6.2 Pile A Test Results

590 blows were required to drive Pile A to a final penetration of 9.14m. At intervals of 50 blows throughout the drive two or three consecutive blows have been analysed. From these the variations on a blow by blow basis and during the course

of the whole drive could be assessed. The scatter on all data was of the order of 10%. This is deemed to be acceptable.

Figs. 7 and 8 show typical sets of data corresponding to penetrations of 4.572 and 9.144m. Several checks based upon the theory for propagation of a solitary wave in a pile can be made on the consistency of this data. To help facilitate these, the stresses at the various levels are plotted below each other at distances apart proportional to the levels of the gauges on the actual pile.

Referring to Fig. 7 if a line ab is drawn through the points corresponding to the initiation of the disturbance at each level, then the slope of this line should be equal to that of the speed of sound in steel. From some 21 tests so analysed an average value of 5200 m/s was obtained.

If the hammer is striking uniformly the peak stress and the peak acceleration at the pile head should be almost independent of the soil conditions. For 21 blows the average values of these was found to be 138 MN/m² and 224 g respectively. The initial slope of the acceleration-time curve should also be independent of the soil conditions. An average value of 146 g ms⁻¹ was found for this. The average scatter in all these results was of the order 10%.

The time to zero acceleration and the time to peak stress at the pile head should be nearly equal. Average values of 2.25 ms and 2.4 ms were obtained for these.

Eighteen hours after completing the main drive a redrive test was carried out. The pile was advanced a further 150 mm by means of 21 blows. This is equivalent to 138 blows/m and should be compared with 131 blows/m at the end of the main drive. The set-up is therefore small.

By subtracting the signals from diametrically opposite pairs of strain gauges the bending stress may be obtained. Typical results are shown in Fig. 9. The peak bending stresses were generally of the order of 28 MN/m².

6.3 Pile B Test Results

351 blows were required to drive the pile to a final penetration of 9.144m. The experimental data obtained was checked in the same manner as Pile A. In general the quality of the data matched that of Pile A.

Figs. 10 and 11 show typical sets of data corresponding to penetrations of 4.572 and 9.144m. These results show that a closed end produces noticeably higher toe stresses than an open end. The passage of the peak stress down and up the pile can be more readily seen, i.e. a stronger reflection is observed.

At blow 250 a strain gauge at the toe ceased to function. The reason for this is unknown. Prior to failure it had been operating satisfactorily.

As mentioned earlier the accelerometer data had a noise component superimposed upon it due to an earth loop caused by using a mains power supply. Hence no useable data was obtained.

6.4 Dolley Compression Tests.

On completion of the field work the pile cap was tested in a 6000 kN compression testing machine. Seven cycles of loading and unloading were applied and a typical cycle is shown in Fig. 5.

The stiffness of the cushion during loading and unloading was found by fitting a straight line through the loading and unloading curves respectively. Average values of 1.1x10⁶ kN/m and 1.36x10⁶ kN/m respectively were obtained. The coefficient of restitution is given by square root of the ratio of these two. An

average value of 0.9 was obtained.

6.5 Theoretical Calculations

The driveability studies were carried out by the wave equation method Smith (Ref. 3). For this the damping parameters J' (side) and J (point) and the soil quake Q are needed.

No measurements of J' or J are known for Cowden Clay. Because this material has a clay content of only 45% and has layers and lenses of sand it is thought to be relatively insensitive to rate of deformation. Thus J' will be closer to that of sand than that of pure clay. Accordingly a value of $J' = .328$ sec/m has been used. For clays the work of Litkouhi (Ref. 4) has shown that J is small, accordingly a value of 0.00328 sec/m has been used.

Qualitative information on Q may be deduced from Fig. 16. This indicates Q is in the region 2.54mm.

For modelling the pile/soil adhesion a constant value of 85 kN/m² has been assumed.

At the conclusion of driving Pile A, the plug was found to have risen 40% of the penetration. Hence under all driving conditions a plug height of 40% of penetration has been assumed and the adhesion on the inside wall taken as 85 kN/m².

For Pile B the base pressure was taken as $9 C_u$ x base area.

The results of the driveability calculations are given in Figs. 7, 8 and 10, 11. Agreement between experimental and theoretical curves is on the whole satisfactory.

6.6 Discussion of Driving Results

Prior to entering the field an extensive laboratory investigation to determine the best method of attaching strain gauges and cable leads to the pile wall was undertaken. All instruments and circuits were thoroughly tested in the laboratory.

A rehearsal was held at the main works of BSP to test the procedures which had been devised and modify them in the light of the experience gained.

The importance of this preparatory work in reducing unforeseen difficulties in the main tests cannot be overstated. As a result instrument failure was reduced to an insignificant level, and the time spent on site was minimised.

Throughout the test series the performance of the hammer was found to be very uniform. This was confirmed by the high speed photography and the strain gauge data. A scatter of 10% was observed on the hammer data.

The experimental data and the wave equation calculations confirmed that the mechanism by which the hammer impulse is transmitted down the pile is of the nature of a wave propagation phenomena.

At the completion of each main drive, the residual strains in the pile were observed. They were of the order of 50 microstrain, which indicates a high level of stress relaxation in the soil adjacent to the pile wall. These strains correspond to about 5% of the peak observed during driving.

At final penetration the blow counts of piles A and B were observed to be 131 and 98 bl/m respectively. The ratio of these is 1:0.75. The values calculated by the wave equation were 107 and 78 bl/m which give a ratio of 1:0.73. Thus although the absolute values of the observed and theoretical blow counts differ, due to lack of measured data on J' and J , the ratios are in close agreement. From this it can be concluded that relative movement takes place between the soil plug and the inside pile wall and the hypothesis that under dynamic conditions the pile does

not plug appears to be proved.

7.0 PRESSURE CELL RESULTS AND INTERPRETATION

7.1 Results

Measurements during driving were of limited use because of the slow response of the pressure cells. Several of the cells appear to have suffered from significant zero changes during driving, even before they entered the ground. Additionally it appeared from the results that the majority of the pore pressure cells retained some air. Consequently readings continued to increase after driving.

Fig. 12 shows measurements from pore pressure cell AP2 during driving, together with those from the three levels of total pressure cells on pile A, Fig. 13 shows the readings from the two lower levels of total pressure cells on pile B during driving. The periods during which driving took place are marked on the figures.

Fig. 14 shows the pressure measurements after driving for both piles for a period of around 27 days. Events affecting readings are marked. Pressure changes were also monitored during axial load testing and changes were found to be very small.

7.2 Discussion

It is difficult to draw many quantitative conclusions from the measurements. It can be seen from Fig. 12 that the immediate response to driving is a reduction in both total pressure and pore pressure, followed by a steady increase. The only level at which this was not observed to occur was near the bottom of the closed ended pile (B), where the lateral pressures are likely to be significantly affected by the penetration of the pile tip. It is possible that the pressure drop is caused by the irrecoverable strains in the soil created by the rapid expansion of the pile under the first hammer blow Heerema (Ref. 5). However the magnitude of the change is greater than would be expected.

It is not possible to make a distinction between the values of pressure generated by the two different piles. The summation of the changes that take place during driving gives a maximum total pressure change at the tip of the pile of about 700 kN/m^2 , i.e. around $6 \times C_u$.

It appears from the measurements after driving that pressures have at least stabilised by day 20, although some of the cells are exhibiting inexplicable behavior during the period 15-23 days. Little change is apparent at the uppermost level. This is probably explained by the proximity of a higher permeability layer between 2 and 3 metres. Fig. 15 shows the plot resulting from the subtraction of the pore pressure measurements from the total pressures, which enables the changes in effective stress to be examined. It is apparent that over the time period shown small increases in radial effective stress have been recorded on pile B, at the lower levels but not on pile A.

8.0 PILE LOAD TEST RESULTS AND INTERPRETATION

8.1 Results

The results of the constant rate of penetration tests on piles A and B are presented in Fig. 15 in the form of a plot of applied load against pile deflection at mudline. Before the piles were loaded to failure both were subjected to a load cycle; pile A to 50 tonnes, pile B to 75 tonnes. In both cases the response to loading was identical to that shown in Fig. 15 over the same portion of the initial

loading curve.

The recorded maximum loads were

p max = 143 tonnes for the closed ended pile B
p max = 118 tonnes for the open ended pile A

Pile B which was unloaded after a penetration of around 40mm had been reached was reloaded to failure and maximum load of 137 tonnes was achieved.

The tests were stopped when a penetration of around 8.5% of the pile diameter had been reached.

8.2 Discussion

The load-deflection curves from the two pile tests show some marked differences. The open ended pile, A, very clearly has a bilinear response before failure is reached where the majority of shaft friction has been mobilised by a deflection of 1% of the diameter, and the increase in load is thereafter taken by the plugged pile base. The difference between this and the curve for pile B is caused by the fact that the response is stiffer due to the stresses locked in under the solid base after driving, Cooke (Ref. 6). This is in contrast to pile A, in which the base resistance can only be mobilised once sufficient movement has occurred between the internal column of soil and the shaft to cause the pile to plug.

The piles behaved identically under initial loading. It is possible to deduce the elastic properties of the soil from the initial loading curve using the simple mathematical model suggested by Randolph (Ref. 7) which models the pile as an axially loaded elastic rod which is restrained by elastic tractions along its length and at the base. For the sake of comparing results from large scale plate tests a secant slope of the load deflection curve is taken between 0 and 50 tonnes load. It can be shown that by making sensible assumptions about the relationship between pile movement and mobilised tractions, a value of the shear modulus (G_s) lying between 14 and 18 MN/m² is obtained. In addition, by analysing the response of pile A during the period that base resistance was being mobilised in the same manner as a plate bearing test result would be treated (eg Marsland, Ref. 1), a value of G_s of 14 MN/m² is obtained. This latter value compares very favourably with the values obtained from plate bearing tests on the site.

The effect of closing the end of the pile was to increase its capacity by approximately 21% under axial compressive loading. There was no apparent significant variation in soil properties within the region where the piles were driven. The only factor differing from one pile to the other apart from geometry was the length of time between driving and testing. However, the observed rate of pore pressure dissipation indicates, and measured permeabilities (1 to 10x10⁻⁹ m/sec) would predict that all excess pore pressures generated during driving had dissipated before either of the piles were tested.

The difference between the ultimate capacities of the two piles can only be explained by looking at the effective stress changes that take place in the soil around the pile, and the effect that these have on the shaft resistance. If pile driving is modelled as a cylinder which is expanded into the low permeability soil, and the soil is then allowed to consolidate, it can be demonstrated theoretically, Wroth et al (Ref. 8) that the effective radial stress on the pile surface after consolidation increases as the ratio

$$p = 1 - \left(\frac{r_i}{r_e} \right)^2 \text{ increases}$$

where r_i is the internal radius of the pile
and r_e is the external radius

The effect of an increase in the radial effective stress is to increase both the drained and undrained resistance that can be mobilised.

The ultimate capacities of the two piles have been analysed in terms of some of the common predictive theories.

(i) The α method. If undrained shear strengths (C_u) are taken from the plate bearing test results (Fig. 2), and it is assumed that the base resistance mobilised is given by

$$Q_B = 9 \times C_u \times A_B$$

$$\text{then } \alpha = (\text{Ultimate load } (Q_{ult}) - Q_B) / (C_u)_{ave} A_S$$

where C_u is the value at the base
 $(C_u)_{ave}$ is the average value down the shaft
 A_B is the base area of the pile
 A_S is the total shaft area.

For pile A, $\alpha = 0.64$
 pile B, $\alpha = 0.8$

(ii) The CPT method. Let it be assumed that the total resistance of the pile is given by

$$Q_{ult} = (A_B \times (q_c)_{tip} + A_S \times (f_S)_{ave}) \times \delta$$

where $(q_c)_{tip}$ = Measured cone resistance at pile tip

$(f_S)_{ave}$ = Average measured sleeve friction along the total depth of the pile.

From the results for

pile A $\delta = 0.73$
 pile B $\delta = 0.88$

(iii) The interpretation of the load test results in terms of the effective stress method enables values of the ratio $K = \sigma_r' / \sigma_v'$ to be calculated, where σ_r' is the radial effective stress on the pile at the time of the test, and σ_v' is the initial in-situ vertical effective stress. The values of K range from 4 to 6, with lower values relating to the open ended pile.

9.0 CONCLUSIONS

The tests confirmed that the hydraulically actuated hammer behaved very uniformly while driving the test piles.

The closed ended pile drove more easily than the open ended pile and had a greater axial capacity. This justifies further investigation.

The tests successfully demonstrated that piles can be instrumented to give reliable field observations of stress, acceleration, total pressure and pore pressure.

Further interpretation of the field data obtained is still being carried out.

10.0 ACKNOWLEDGEMENTS

The Authors wish to thank Messrs R. Elliott and D. Keen of BSP, Mr. J. Petty formerly of BSP, and, Mr. V.J.R. Sutton of BP for useful discussions.

Dr. L. Cuthbert of Queen Mary College advised on the instrumentation and Mr. J. Dolwin of Queen Mary College assisted with the interpretation of data. Mr. T. Freeman of BRE assisted with the field work and the interpretation of the site data.

The work carried out by BRE was funded by the Department of Energy. The Authors are indebted to BP Trading Ltd, BSP International Foundations Ltd, and the Director, Building Research Establishment for permission to publish.

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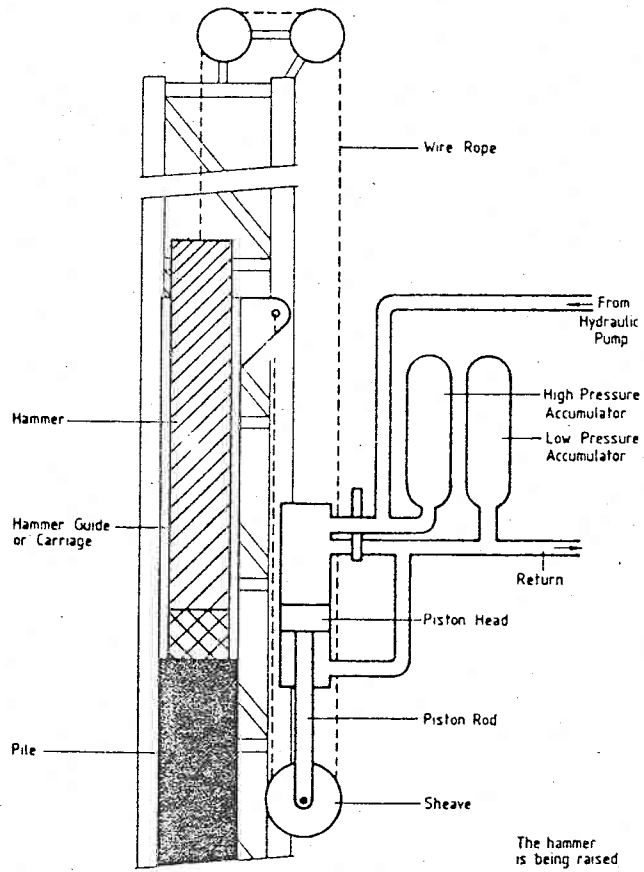


Fig. 1

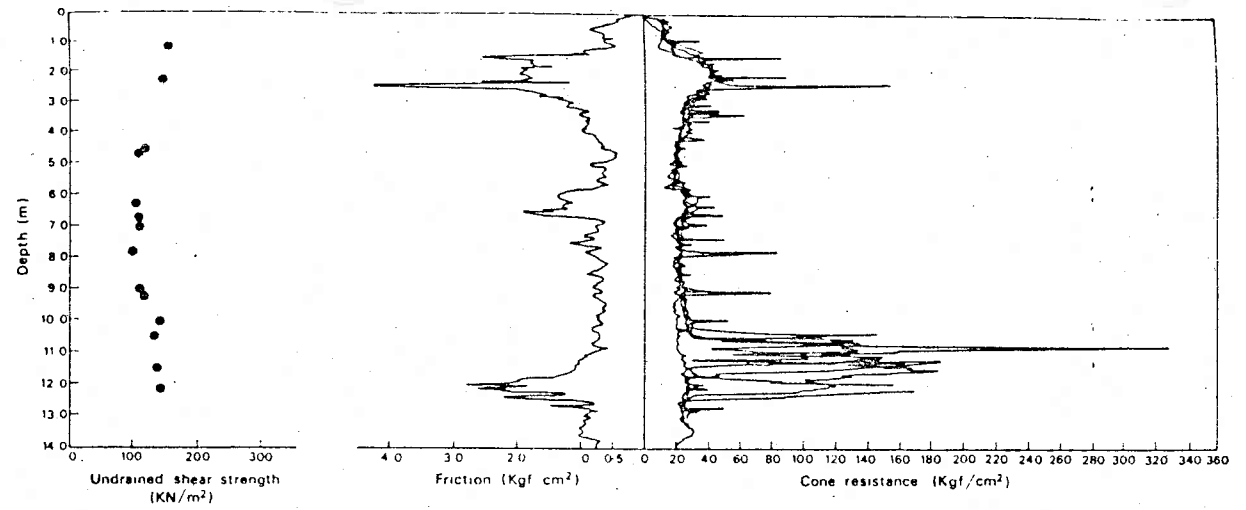


Fig. 2

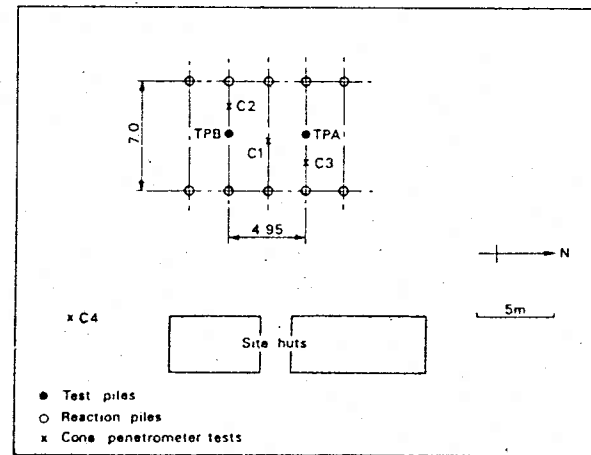


Fig. 3

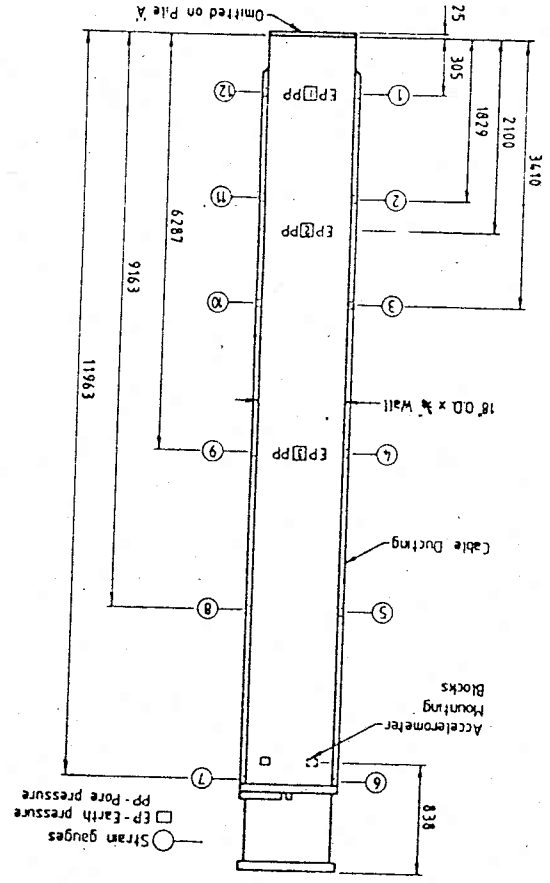


Fig. 4 Test pile B (closed ended)

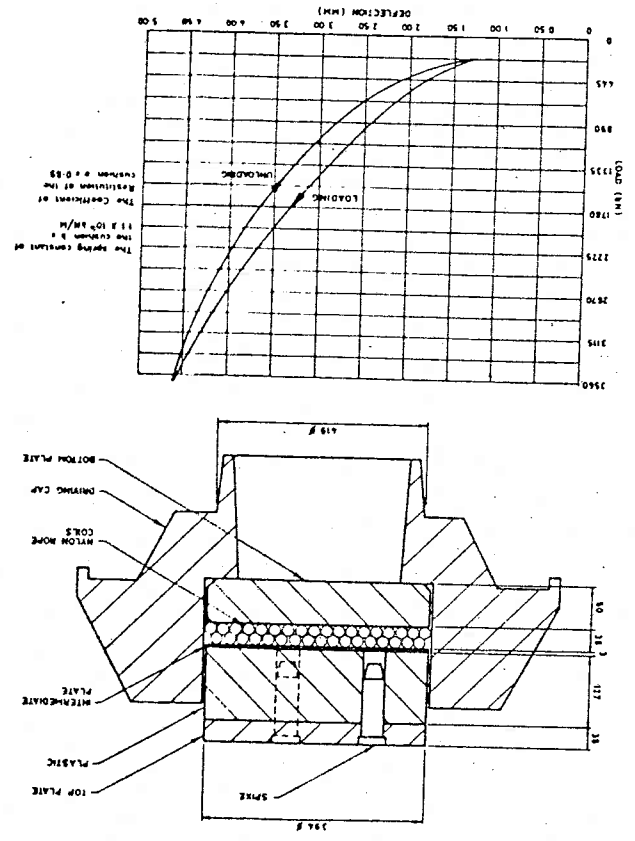


Fig. 5 Typical Dolley compression test

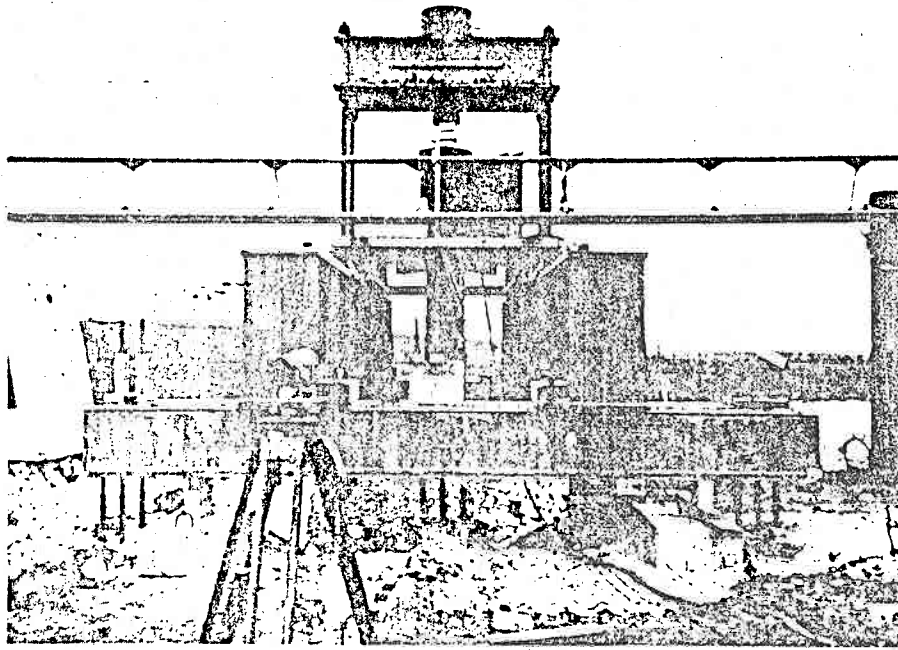


Fig. 6

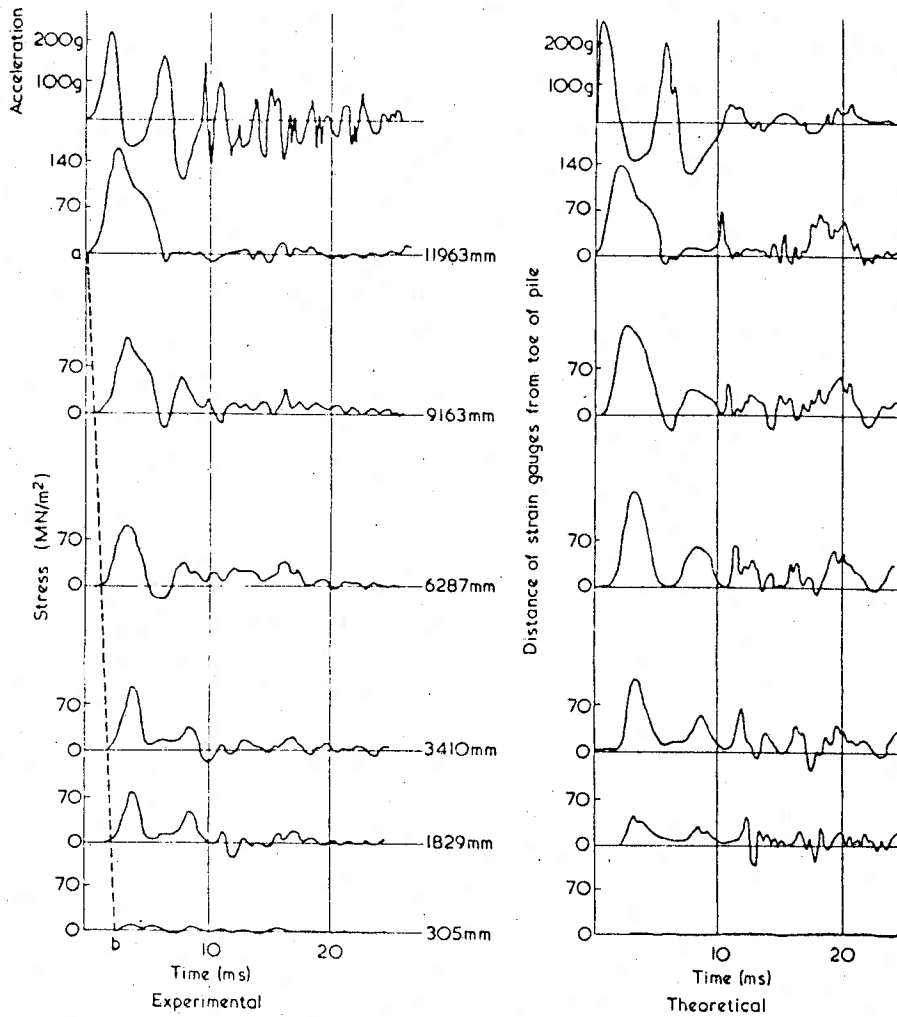


Fig. 7 Pile A penetration 4572 mm

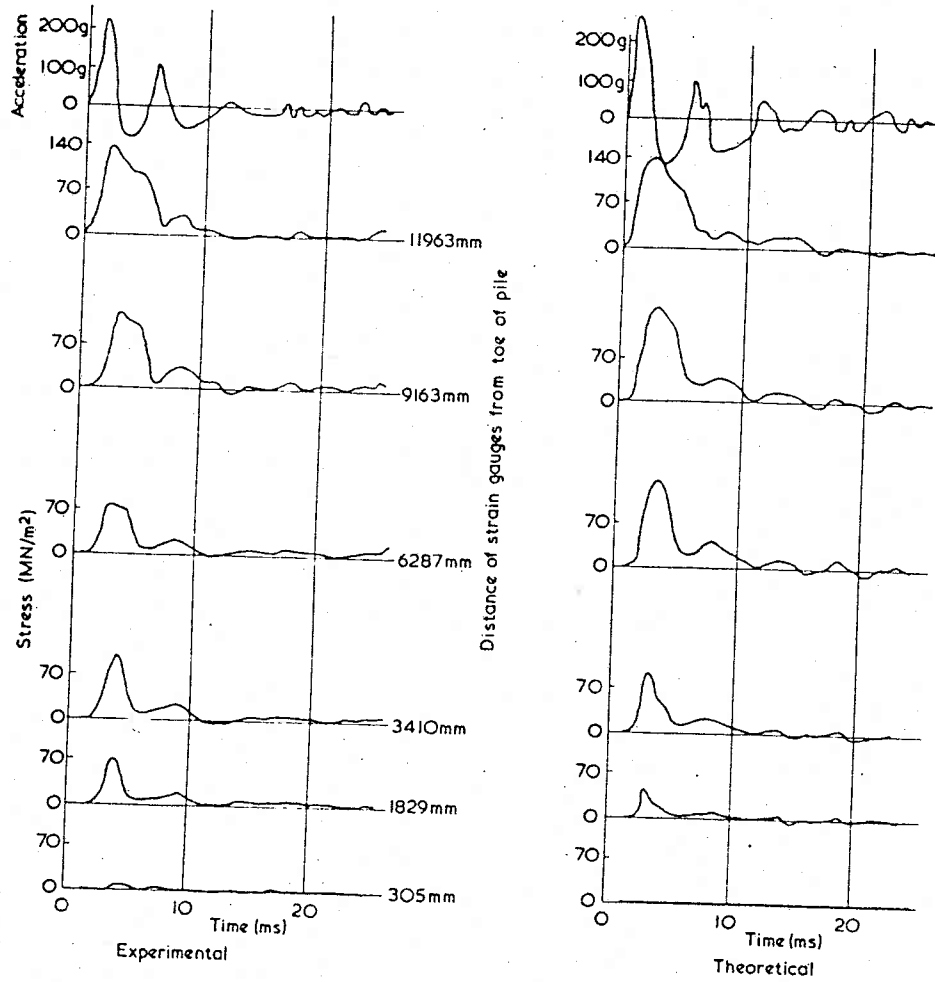


Fig. 8 Pile A penetration 9144 mm

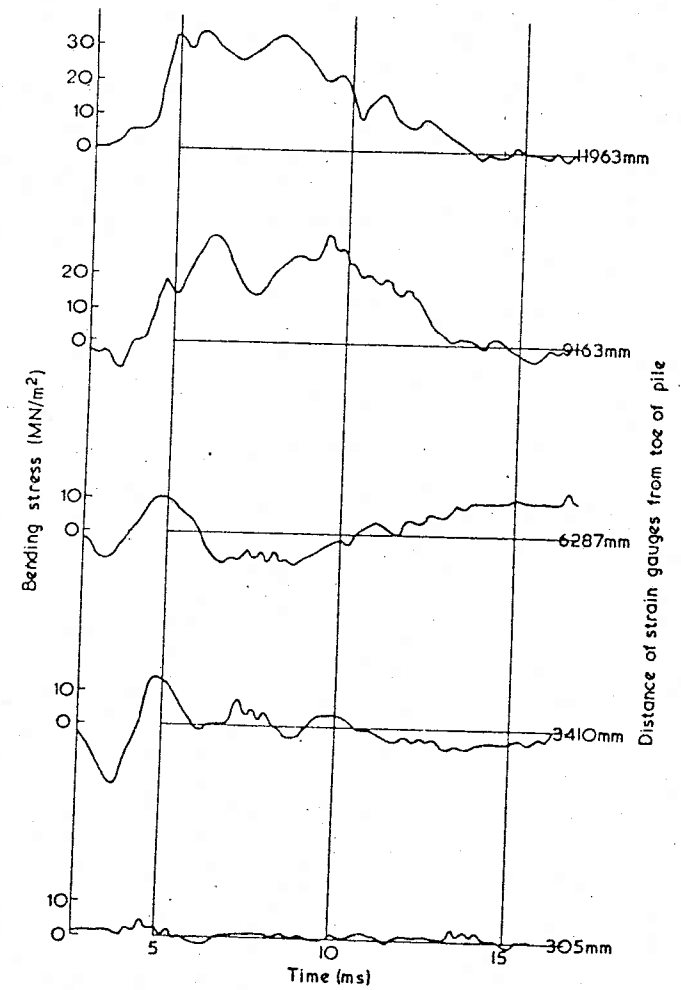


Fig. 9 Measured bending stresses. Pile A penetration 9.144 m

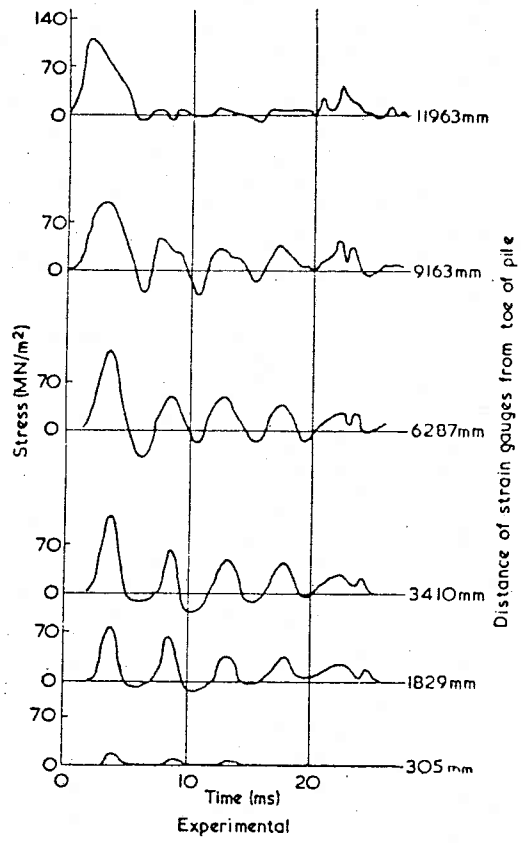


Fig. 10 Pile B penetration 4572 mm

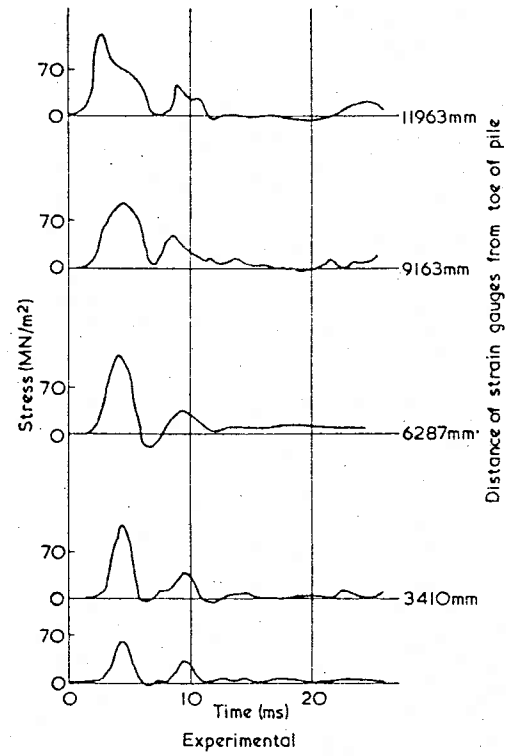
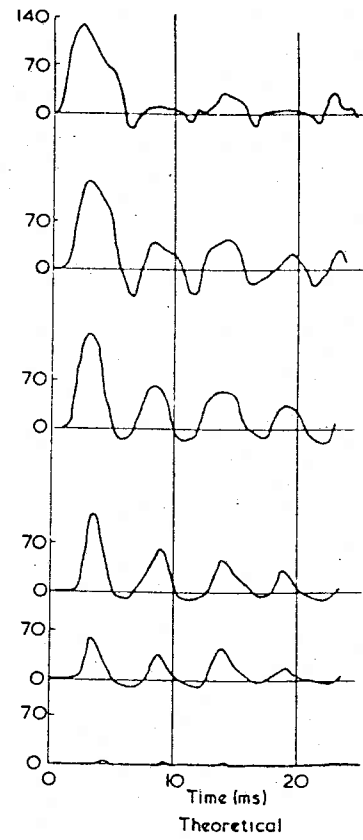
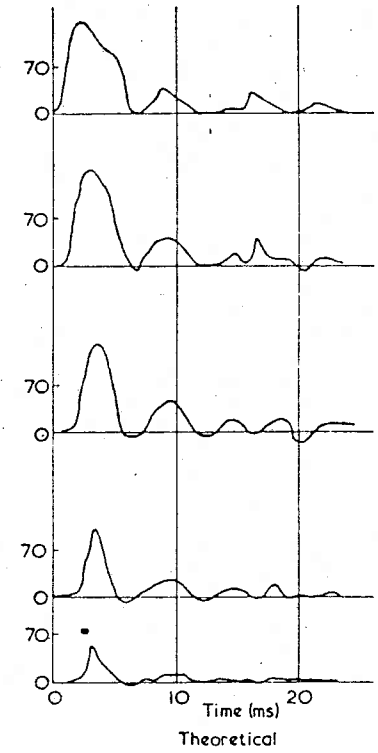


Fig. 11 Pile B penetration 9144 mm



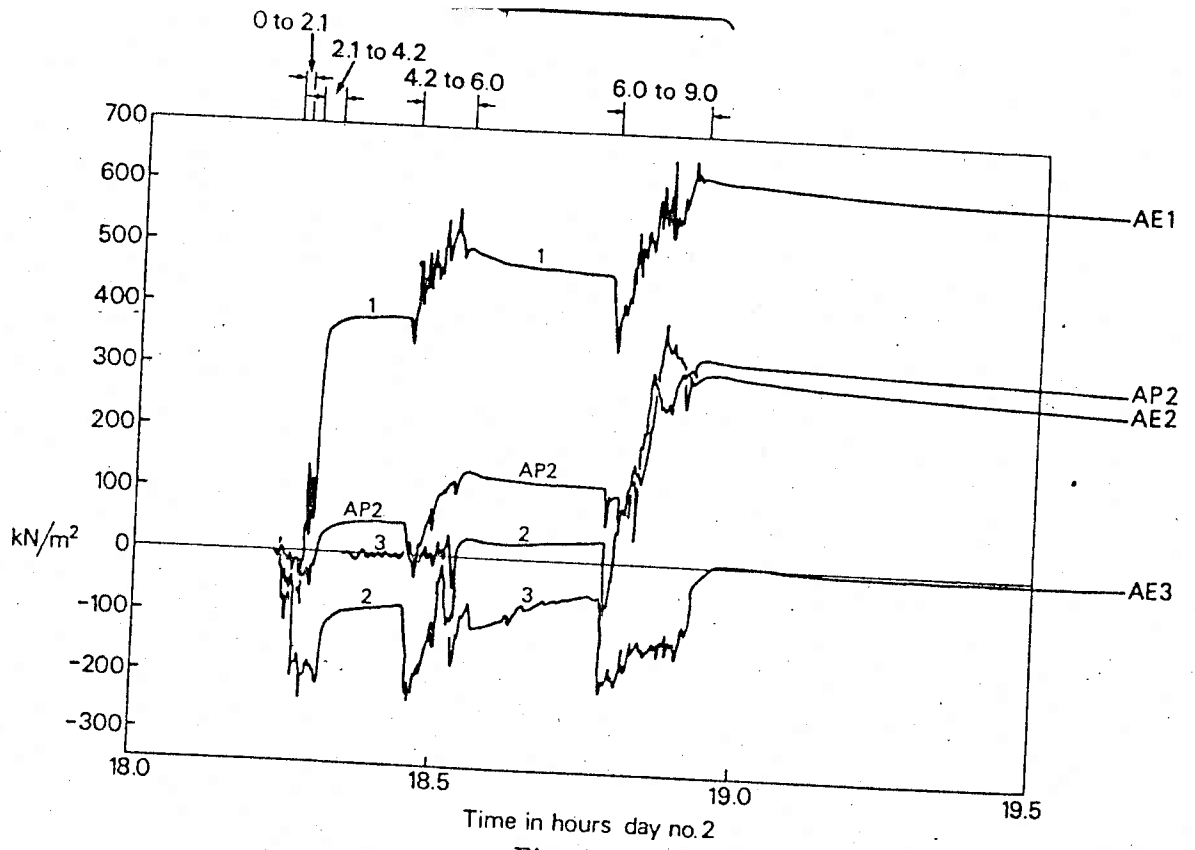


Fig. 12

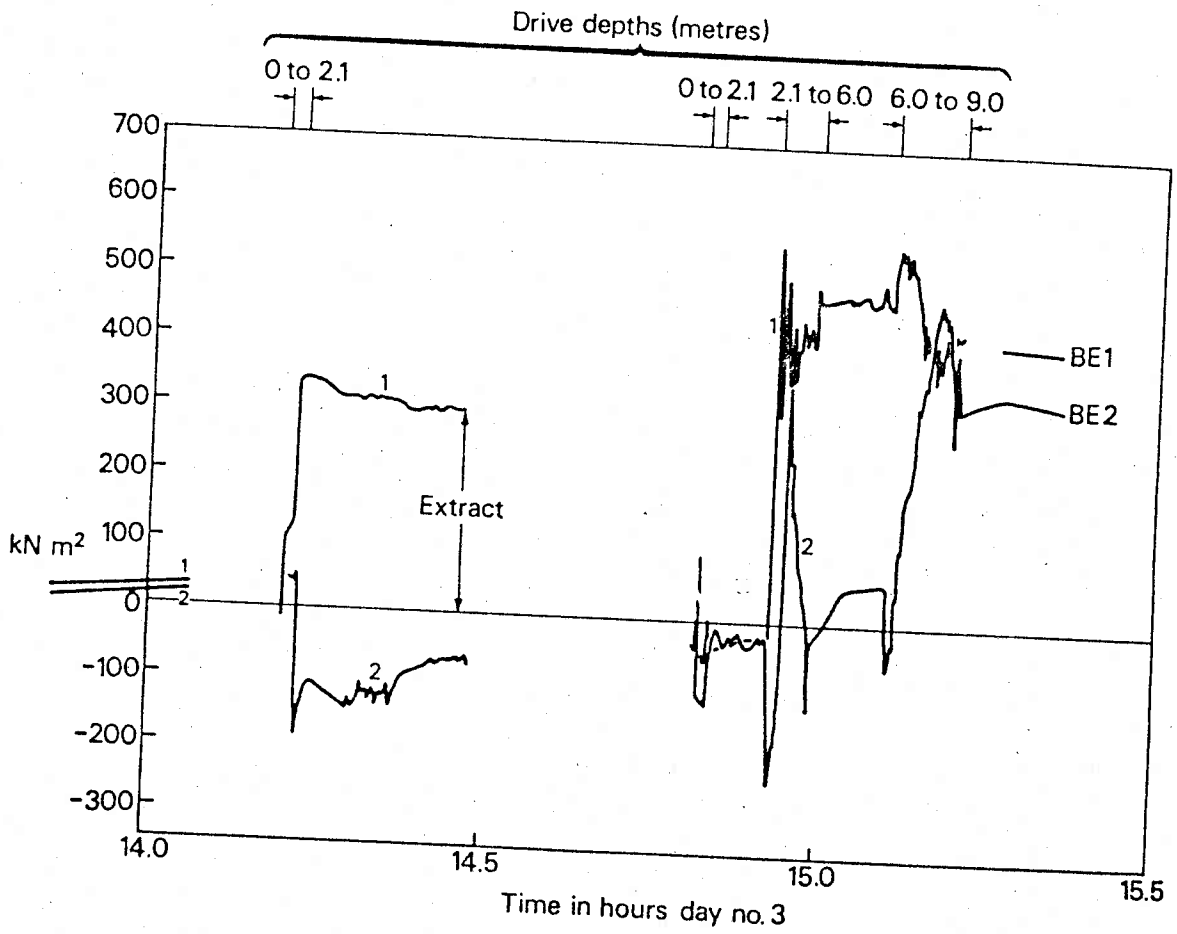


Fig. 13.

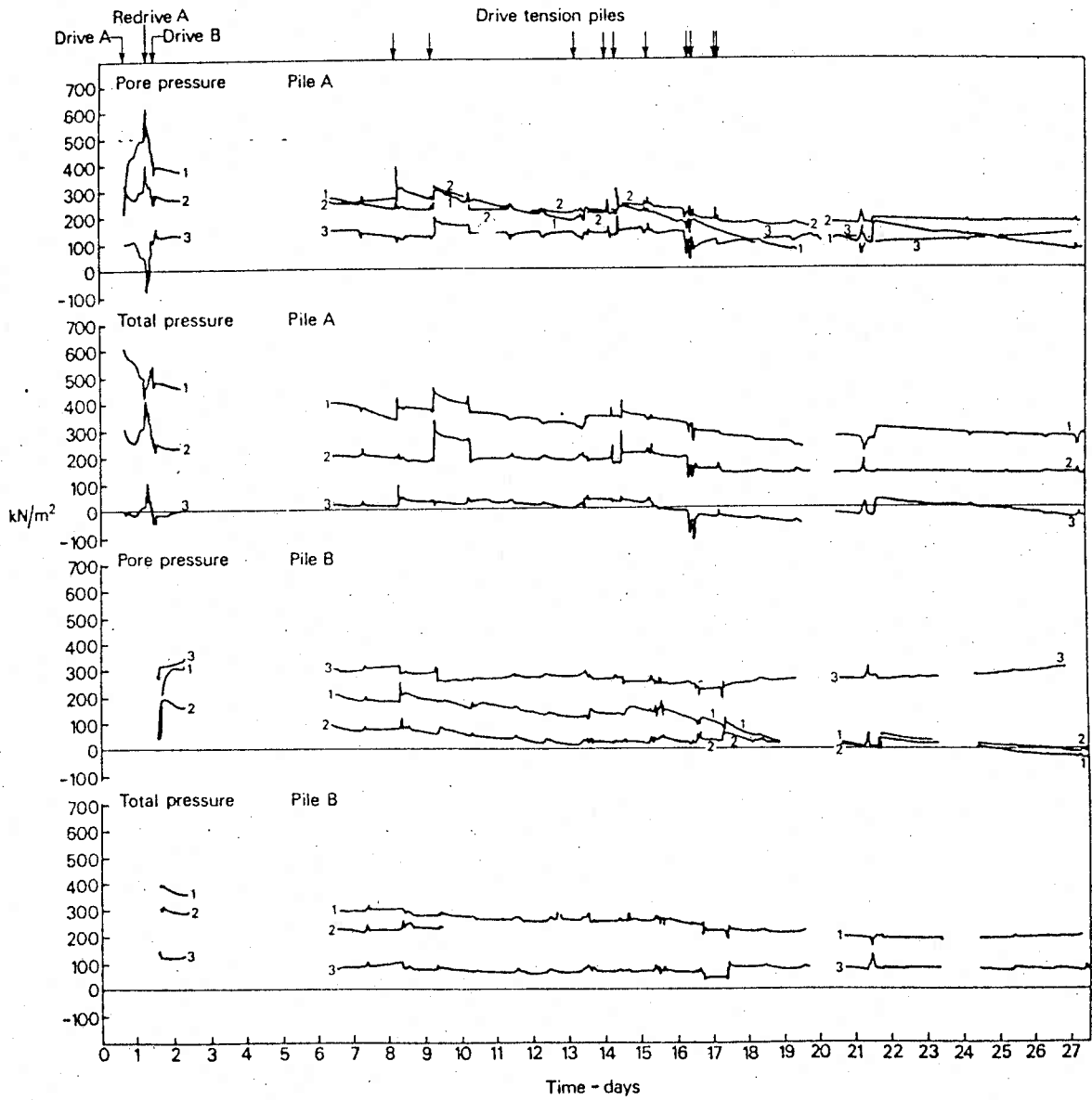


Fig. 14

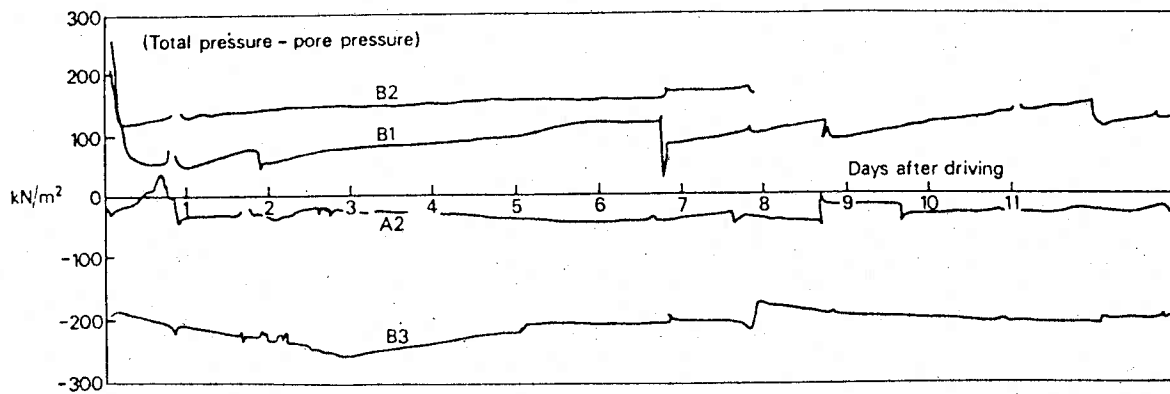


Fig. 15

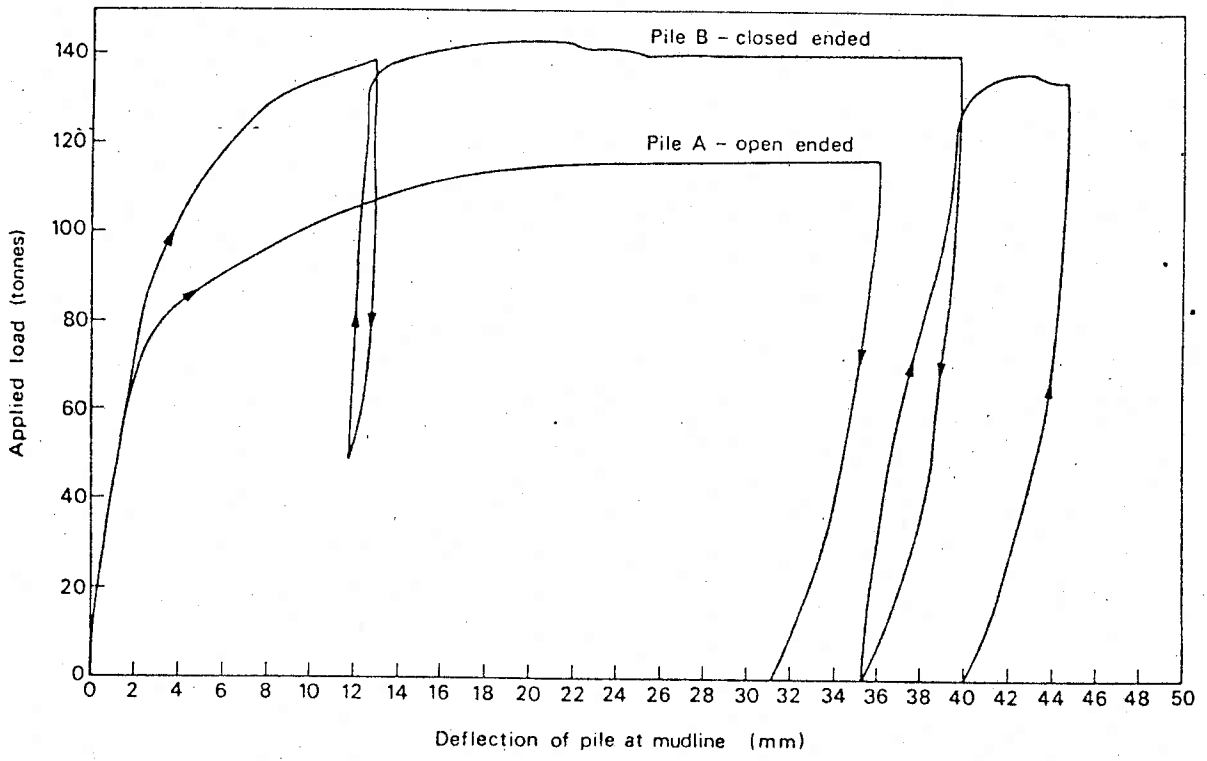


Fig. 16