The behaviour of pile groups containing raked piles in cohesionless soil when subjected to vertical and horizontal forces .

A Thesis

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by

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Synopsis

The purpose of this investigation was to examine the behaviour of inclined piles , both singly and in groups , embedded in cohesionless soil .

Model tests have been carried out on single piles and on pile groups driven into carefully prepared beds of uniform sand . Freestanding groups of piles with different configurations , with and without raked piles have been tested under both vertical and horizontal loads . The behaviour of the groups has been compared .

Various methods of analysis have examined , and a method has been developed to predict the behaviour of pile groups contaning raked piles from the observed behaviour of a single test pile . Two computer programmes are presented to analyse single piles and pile groups .There was good agreement between the computer predictions and the observed behaviour of the models .

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To My Mother

BIOGRAPHICAL NOTE

The author obtained the degree of Bachelor of Science in Civil Engineering at the Faculty of Engineering , University of Basrah in July 1968 .

In January 1969 , he worked for the Civil Engineering Department , University of Basrah , for a period of seven months as research assistant in the Concrete Laboratory .

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In February 1972 , he was awarded a grant from the Iraqi Ministry of Oil to do a post graduate research in piling .

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List of Symbols

Symbols are defined when they first appear and are listed here for convenience .

a_m and p_u	coefficients governing the linear and non linear behaviour of the soil respectively .
B	pile width .
C_c	compression index .
E I	flexural stiffness of the pile .
k_a	coefficient of active earth pressure .
k_H	modulus of subgrade reaction .
k_o	coefficient of earth pressure at rest .
k_p	coefficient of passive earth pressure .
n_H	constant of horizontal of subgrade reaction .
M	bending moment .
P	force .
p_o	effective overburden pressure .
q	transverse soil pressure on the pile .
q_c	unit cone resistance .
q_{max}	maximum lateral pressure .
R	relative stiffness factor .
RD	relative density .
T	relative stiffness factor .
y	transverse displacement of the pile .
Z	depth .

α an angle defining the size of the sliding wedge in front of the pile .

$\beta = 45^{o} + \phi / 2$

ϕ angle of shearing resistance .

ρ settlement .

γ effective unit weight of soil .

Θ rotation of the pile head .

δ displacement of the pile head .

Chapter 1

Introduction

1.1 Historical review

Piled foundations have been used from the earliest times . Vitruvius gives detailed instructions for the use of piles for bridge foundations and for the support of load - bearing walls in poor soil . The art of using piles however was not supplemented with scientific knowledge until the eighteenth century . The earliest recorded observations of the driving behaviour of a pile were made by Perronet , a famous eighteenth century French engineer . Sandeman (1880) performed the first recorded lateral load tests on piles . In many cases piles appear to have been used by early engineers with the object of compacting soft ground , for which purpose they would frequently have been ineffective .

1.2 Purpose of piled foundations

Although piles are occasionally used for compacting suitable soils , they are commonly used in modern practice as a means of transmitting surface structural loads through weak or compressible soils to stronger or stiffer soils below . They may also be used to resist uplift forces or lateral loads . These lateral loads , if small , may be resisted solely by the lateral resistance of vertical piles , but where they are larger they are commonly resisted by incorporating raked piles in the foundation .

Although piled foundations capable of resisting vertical and lateral forces have been frequently used in the past , the behaviour of laterally loaded piles and pile groups is still in some respects not well understood . However , in recent years the increasing quantity and size of civil engineering works and the need to build on weak and compressible soils has induced economic pressure for a more precise knowledge of pile behaviour .

1.3 Requirements for satisfactory design

The choice of the type of pile will be determined by the ground conditions , the effects which the installation of the piles may have upon nearby structures and other property and , if more than one type is suitable in the particular conditions , by the relative cost .

Whatever type is chosen however , every design will have to satisfy two conditions :

a - The factor of safety against ultimate failure , both of the fabric of the foundation and of the supporting soil will need to be adequate .

b - The settlement and displacement of the foundation (and in particular the differential settlement under working load) will have to be no more than can be tolerated by the completed structure . A complete analysis of the behaviour of a piled foundation is possible only if the load deformation characteristics of each pile in the group , both vertical and raked , can be precisely assessed .

1.4 Method of load transfer

Piles may be classified into three major categories based on the manner in which they function (Terzaghi and Peck (1948)) .

a - Friction piles in coarse grained soils :

These piles transfer most of the load to the soil by skin friction and they are usually " driven " piles . Sometimes a cluster of these piles is driven specifically to reduce the porosity of the granular material , in which case they are usually referred to as compaction piles .

b - Friction piles in fine grained soil :

These piles also transfer their load to the soil primarily by skin friction , but without compacting the soil appreciably .

c - End bearing piles :

These piles transfer their load directly to a firm stratum located at a considerable depth below overlaying soft strata through which they are normally driven .

A piled foundation may be subjected to various loading conditions , static , dynamic and repetitive . The orientation of loading is also a variable factor .

1.5 Effect of the pile cap

Pile groups may be classified into two major categories based on the position of the cap :

a - Freestanding pile groups , as in a jetty where there is no direct contact between the pile cap and the soil .

b - Pile groups with caps in contact with or embedded in the soil . In these structures a significant part of the vertical and horizontal loads may be transfered directly from the cap to underlying and surrounding soil .

1.6 Methods of analysis

Several analyses have been made of the behaviour of single vertical and raked piles in different types of soil and on groups of vertical piles under axial and lateral forces . In most of these analyses the soil is assumed to be either a Winkler medium or a homogeneous isotropic elastic solid .

Analyses of the behaviour of pile groups subjected to lateral loads - and , in particular , of pile groups containing raked piles subject to vertical and lateral loads - has received little attention compared with the space devoted in the literature to single piles . In the present investigation , the various methods are fully discussed , and a method derived from the work of Reese and others is developed to predict the behaviour of pile groups containing raked piles from the observed behaviour of a single pile .

1.7 Model tests

The best information about pile group behaviour would clearly be obtained from the results of full scale tests . However such tests are slow , very expensive , and frequently difficult to control . Much valuable information may be obtained from model tests carried out under carefully controlled conditions .

In the present investigation , model tests have been carried out on single piles and on pile groups driven into carefully prepared beds of uniform sand . Groups of piles with different configurations , with and without raked piles , have been tested under both vertical and horizontal loads and the behaviour of the groups has been compared .

1.8 Conclusions

The investigation reported herein attempts to correlate the behaviour of a single pile with that of a pile group when they are subjected to vertical and lateral loads .

The information given in this thesis may be used to solve the following problems :

a - To predict the moment and displacement distributions along a single vertical or raked pile on the basis of it's own load - displacement data at the cap , and to predict the rotation of the cap .

b - To predict the displacement and rotation of the cap of a pile group , and to predict the distribution of bending moment along the piles in the group , on the basis of the load - displacement record of a single test pile .

On the basis of all the work performed as part of this investigation , conclusions are drawn about vertically and laterally loaded pile groups , and suggestions are made for research that could advance the knowledge of pile behaviour .

Chapter 2

Previous studies of the behaviour of piles and pile groups in cohesionless soil

2.1 Introduction

This chapter presents a review of the previously published work on piles and pile groups in cohesionless soil .

In the past , many important factors affecting pile behaviour have not been well understood , and as a result a very large number of model and field tests have been made and reported . The greater part of this previous work has been confined to studies of single piles , but in recent years more attention has been paid to the behaviour of pile groups . However , because of the very large forces required to load a full sized pile group , the reported tests on such groups have been almost invariably on a model scale . Moreover , very few of those tests have included raked piles .

The literature on this subject is very extensive , and in many cases the same points have been made by a number of investigators . It has therefore been neither desirable nor indeed possible to refer in detail to every published reference in the discussion which follows . Only the more important and easily accessible authorities have been cited in the text of this chapter . A full list of all the publications consulted in its preparation will be found in the references . The principal references for this chapter are Fleming (1958) , West (1964) , Kezdi (1960) , Broms (1972 , 1976) and Franke (1976) .

2.2 Observed behaviour of an axially loaded single pile

The ultimate resistance to axial load is generally found to depend on :

a - The rate of penetration .

b - The roughness of the pile shaft .

c - The depth of penetration .

d - The cross section of the pile .

e - The relative density (RD) of the soil .

The point resistance increases rapidly with increasing depth of penetration , although this rate of increase is smaller in loose soil than in dense soil . The shaft resistance probably represents between 10% and 50% of the total ultimate resistance , depending on the soil density and the length / diameter ratio and roughness of the pile shaft .

The ratio of shaft resistance to total resistance is higher in loose soil than in dençe soil Fleming (1958) .

The shaft resistance is greater if the shaft is rough .

The majority of the shaft resistance is generally mobilized at much smaller settlements than the point resistance (Fleming (1958) , West (1964) , Hindi (1964)) . This has two important consequences . Firstly , where resistance forms a large part of the ultimate resistance , the working load (say 40% of the ultimate resistance) may be carried almost entirely by shaft friction . Secondly , the settlement under the working load is commonly very much smaller than the settlement as the ultimate resistance is approached .

The pull - out resistance of a pile (unless it has an enlarged base) depends only on the shaft resistance and this is significantly less than the shaft resistance during penetration (Hindi (1964)) .

It is worth noting in this context, that negatively battered piles in the rear row of a group subjected to horizontal load only are often in tension, and hence offer poor resistance to axial movement.

2.3 Observed behaviour of a laterally loaded single pile

a - Deflections

The deflections under a given lateral load are decreased if the embedded length is increased. However, beyond a certain point, increasing the length has only an insignificant effect on the deflection, and the pile may be assumed for the purposes of analysis to be of infinite length. This critical length depends on the relative stiffness of pile and soil which, for the purposes of analysis using a coefficient of subgrade reaction, may be defined as follows :

(i) Where the modulus of subgrade reaction (k_H) is constant with depth

$$R = \sqrt[4]{\frac{E\,I}{k_H}} \qquad (2.1)$$

where R = the relative stiffness factor

EI = the flexural stiffness of the pile

(ii) Where the modulus of subgrade reaction (k_H) increases linearly with depth (Z)

$$k_H = n_H\ Z \qquad (2.2)$$

where n_H = the constant of horizontal subgrade reaction

and $$T = \sqrt[5]{\frac{E\,I}{n_H}} \qquad (2.3)$$

where T = the relative stiffness factor.

Any pile whose length is greater than 5R or 5T , respectively , may be assumed to be infinitely long .

The value of the coefficient of subgrade reaction depends on :

a - The stiffness of the pile .

b - The size of the pile .

c - The soil properties .

d - The nature of the applied load .

e - The compaction of the soil .

The value of the coefficient of subgrade reaction decreases as :

a - Pile deflection increases .

b - Pile width increases , and

c - Flexural resistance of the pile increases .

If piles of different flexural stiffness are embedded in the same soil so deeply as to be considered infinitely long , the deflection for a given load is smallest for the stiffest pile .

A pile whose head is restrained against rotation will deflect under a given transverse load only about one half or one third as much as a pile not so restrained .

b - <u>Maximum bending moment</u>

Under lateral load , the maximum bending moment in the pile occurs at a relatively shallow depth below ground surface (in practice usually not more than five or six pile diameters) . However , a pile whose head is restrained against rotation by a pile cap may have its maximum moment at the cap .

c - Creep

Pile deflections under sustained lateral load are nearly constant with time . However , there is evidence , at least on a model scale , for some creep in sands for a short period after loading . For this reason it is usual in model tests to apply the loads slowly and in small increments .

d - Maximum lateral soil resistance

Near the ground surface , the ultimate soil resistance is governed by the resistance of a wedge of soil moving upwards in front of the pile . At some depth below the ground surface there is a transition to a different mode of failure in which the soil flows around the pile (Parker and Reese (1971)) . The maximum pressure that can be developed at any depth excedes Rankine's two dimensional passive pressure at that depth . Parker and Reese give the following theoretical expressions for the maximum lateral pressure :

(i) Near the ground surface

$$q_{max} = \gamma Z \{ K_p - K_a + \frac{Z}{B} \tan\beta \left[K_p \tan\alpha + K_o (\tan\phi - \tan\alpha) \right] \}$$

where $\beta = 45^o + \phi / 2$

α is an angle defining the size of the wedge and is assumed (by Parker and Reese) to be $\phi / 2$.

(ii) At greater depths

$$q_{max} = \gamma Z \{ K_p^3 + 2 K_o \tan\phi (K_p^2 + 1) - K_a \}$$

where , γ = the effective unit weight of soil

Z = the depth

B = the pile width

K_a = the coefficient of active earth pressure ($\tan^2 (45^\circ - \phi / 2)$)

K_p = the coefficient of passive earth pressure ($\tan^2 (45^\circ + \phi / 2)$)

K_o = the earth pressure coefficient at rest which is assumed to be 0.5 .

e - <u>Inelastic behaviour</u>

Upon removal of the lateral load , the recovery of the pile is seldom complete . Usually some residual deflections and the corresponding moments are locked into the soil - pile system . The primary causes of this behaviour are :

i - Irrecoverable compaction of the soil in front of the pile , and

ii - The tendency for a void to form behind the pile and to become filled with loose material .

As a result of this , the response of the soil - pile system to lateral loads is generally non - linear and inelastic even under small loads .

f - <u>Cyclic loading</u>

Reese (1973) , discussing piles for oil platforms subject to wave action , says that " .... it is believed that the effect of cyclic loading on the behaviour of sand around a laterally loaded pile would be small . " However , it is clear that this is not generally true . Where the load oscillates symmetrically about a zero value , the alternate application of the two effects described above will cause a general tightening of the soil on both sides of the pile and a corresponding reduction in the deflection for a given load . Where the load is applied in one direction only , there is an increasing permanent deflection . The increase of this deflection however is at a decreasing rate with respect to the number of load cycles . Eventually an equilibrium deflection is reached (Prakash and Chandrasekaran (1970)) .

2.4 Theoretical analyses of single piles loaded axially and transversely

The many published theoretical investigations of single piles fall into two main groups :

a - Methods based on linear elastic theory .

b - Methods based on subgrade reaction theory .

The former methods are generally based on the assumption that both pile and soil are linear elastic solids . They have been more generally applied to the study of pile groups and are considered fully in section 2.8 below .

In the latter methods , the pile is assumed to behave as a vertical or inclined beam and the soil as a Winkler medium - that is , the soil reaction at any point on the pile is proportional to the transverse displacement of the pile at that point . The methods are only applicable to analysis of behaviour under transverse loading .

Then , the ordinary differential equation for a beam in bending may be written ,

$$E\,I \frac{d^4y}{dZ^4} = -\,k_H\, y\, B \qquad (2.4)$$

where , $E\,I$ - is the flexural stiffness of the pile .

y - is the transverse deflection of the pile .

Z - is the distance coordinate measured along the pile .

k_H - is the modulus of transverse subgrade reaction of the soil .

By solving equation (2.4) , the transverse deflections , and hence the bending moments , shear forces and lateral pressures applied to the pile may be calculated .

The advantages of these methods are as follows :

a - The computation is relatively simple . Although the use of a digital computer is a help , the computation time required is trivial compared with that required for any of the methods based on linear elastic theory .

b - It is possible to make allowance for variations in soil stiffness with depth and to allow for the non - linear response of the soil - pile system .

The disadvantage of these methods is that the modulus of subgrade reaction (k_H) depends on the pile dimensions and properties as well as on the soil characteristics . It is therefore not possible to determine (k_H) directly from conventional soil tests . Nor can the results of a lateral load test on one pile be directly applied to the analysis of another with a different cross - section .

2.5 Ultimate resistance of vertically loaded groups of driven piles

The ultimate resistance and the load / settlement characteristics depend to a large extend on the order of driving , the pile roughness , the spacing , the compactness of the sand , the group size , the depth and the shape of the piles (conical or straight - sided) .

a - Effect of pile driving

A pile driven into cohesionless soil of loose or medium density causes compaction of the surrounding soil . Meyerhof (1959) and many others since have indicated significant compaction of the soil to a distance of three to four diameters from the pile face . When therefore the spacing of the piles in a group is less than about seven diameters , centre to centre of the piles , driving the later piles in the group compacts the soil around and between the piles already driven , increasing both shaft friction and point resistance . Where however the spacing excedes about seven diameters , the effect of this compaction is insignificant (Walker (1964) , Tcheng and Panet (1973)) .

On the other hand , where the soil is initially dense , driving the later piles in a group may actually reduce the density around piles already driven . (West (1964) , Kishida and Meyerhof (1965)) . Thus the average capacity of piles in a group may be expected to increase with increasing number of piles when the initial relative density of the soil is low or medium , but may decrease if the soil is initially dense .

b - Effect of surface roughness

Stuart , Hanna and Naylor (1960) , and several others since , have found that the roughness of the pile surface has a large influence on the bearing capacity of the pile group . For groups of piles with rough surfaces in both loose and dençe soil the ultimate bearing capacity is larger than the capacity of the same number of single piles . For piles with smooth surfaces , the ultimate resistance of the group is less than for the single piles in dense sand but larger in loose sand .

c - Optimum spacing and group efficiency

In loose soil , the efficiency of a pile group (that is , the ratio of the group bearing capacity to that of the individual piles) increases as the spacing is reduced until an optimum spacing is reached . Any further decrease in spacing reduces the group bearing capacity . Schiff (1961) reports an optimum spacing of about five or six pile diameters , but most reported results indicate a value of about three pile diameters (Cambefort (1953) , Kezdi (1960) , Stuart et al (1960) , Stuart and Hanna (1961)) .

For a small group of short piles in loose sand , Broms (1976) suggested , that the efficiency is typically about 1.5 at three diameters spacing , reducing to 1.0 at five diameters . For a large group of long piles the efficiency may be 2.0 at three diameters spacing . This will reduce to 1.0 at about seven diameters .

d - Pull - out resistance of pile groups

Pulling tests on pile groups by Fleming (1958) have shown that the resistance to uplift depends on the sizes of the piles and of the groups , but the pulling force is always less than that required for an equal number of isolated piles . For any given group , the efficiency falls rapidly with decreasing spacing .

e - Eccentric and inclined loads

For small eccentricity , the ultimate bearing capacity will not be significantly affected , for larger eccentricities the total bearing capacity decreases rapidly with eccentricity .

The ultimate strength of pile groups containing vertical piles only under inclined loads , decreases with increasing inclination of the applied load .

f - Distribution of load between piles

Test results show that the load on a pile group is not evenly distributed among the piles , and the corner piles carry a smaller part than those in the centre (Fleming (1958) , Stuart et al (1960) , Hindi (1964) , Beredugo (1966)) .

Since the driving resistance increases with increasing number of piles in the group , the order of driving has an important influence on the distribution when the group is loaded for the first time (Fleming (1958)) . At failure , the corner piles fail first (offering least resistance) , followed by the midside piles and finally by the centre piles .

Hindi (1964) reports that when the cap was removed from a group after application and removal of the load , the pile heads rebounded in the form of a dome , with the maximum rebound for the centre pile . This indicates that , after relief of loads but before removal of the cap , there were considerable axial forces locked into the soil - pile system .

2.6 Settlement of axially loaded pile groups

a - Settlement ratio

Time dependent settlement of a pile group in cohesionless soil is generally insignificant and almost all settlement is complete at the end of loading . The settlement of the group is affected by the driving process which compacts the soil and increases the bearing capacity .

The settlement of a pile is generally considerably greater than that of an equivalent number of isolated piles under the same load (West (1964) , Hindi (1964)) . However there are reported cases where the reverse has been observed (Leonards (1972)) .

The group settlement ratio (that is , the ratio of the group settlement to that of an individual pile under the same average load) is generally larger for piles which mainly carry the load by shaft friction (Kezdi (1960) , Beresantsev et al (1961)) . Piles with a fairly large shaft resistance frequently carry almost all the working load by shaft friction (section 2.2 above) . Then the settlement ratio can be expected to be considerably larger for a group driven into a loose bed of sand than for a similar group driven through alluvium and founded in dense gravel below .

Kezdi (1957) and Stuart et al (1960) both found that the settlement of a pile group decreased with decreasing spacing , presumably as a result of the increasing compaction of the surrounding soil .

b - Empirical methods for predicting the group settlement ratio

Of the considerable number of empirical methods which have been proposed , Broms (1976) recommends the use of those suggested by Skempton et al (1953) , by Beresantzev et al (1961) , and by Vesic (1967) .

Skempton et al (1953) , on the basis of a rather small number of test results suggested the following relationship for piles in sand ,

$$\text{Group settlement ratio} = \left(\frac{4\,B + 3}{B + 4} \right)^2$$

where , B is the breadth of the pile group in meters .

Beresantzev et al (1961) , carried out load tests and , from an analysis of these they suggested that :

i - The settlement is nearly proportional to the equivalent width as shown in Fig. 2.1 , and

ii - The settlement is nearly independent of the number of piles in the group .

This method of calculation is said to be widely used in the USSR .

Vesic re-examined the test data reported by Beresantzev et al and suggested that the settlement is nearly proportional to B / D where B is the width of the group and D is the pile diameter .

Table 2.1 shows the group settlement ratio computed by these three methods for a square (3 x 3) group of nine piles of 400 mm diameter for spacings of three and five diameters and for lengths of 30 diameters and 50 diameters .

From the considerable variation in the values it may be seen that the methods are far from exact .

	$\frac{Z}{D} = 30$		$\frac{Z}{D} = 50$	
	$\frac{S}{D} = 3$	$\frac{S}{D} = 5$	$\frac{S}{D} = 3$	$\frac{S}{D} = 5$
Skempton et al (1958)	4.35	6.01	4.35	6.01
Beresantzev et al (1961)	1.71	2.19	1.45	1.75
Vesic (1967)	2.6	3.3	2.6	3.3

Table 2.1 Group settlement ratios computed by various methods

c - Prediction of settlements by static penetration tests

Settlements of pile groups have also been predicted from the results of static cone penetration tests using the methods proposed by De Beer and Martens (1957) , for analysis of bridge foundations . The piled foundation is assumed to be equivalent to a raft of the same dimensions founded at one third of the embedded length above the bases (see Fig. 2.2) . Soil below this level is divided into horizontal layers and for each layer a compressibility index C_c is determined from the empirical expression ,

$$C_c = 1.5 \frac{q_c}{p_o'}$$

where , q_c is the unit cone resistance

p_o is the effective overburden pressure at the centre of the layer .

The increment of vertical stress $\Delta \sigma_z$ is computed either by using Boussinesq's analysis or by assuming the simple 2 : 1 spread of the load as shown in Fig. 2.2 .

Then for each layer the change of thickness is ,

$$\Delta H = \frac{H}{C_c} \log_e \frac{p_o' + \Delta \sigma_z'}{p_o'}$$

and the total settlement is given by the sum of the settlement in all the layers ,

$$\rho = \Sigma \Delta H$$

The method is intended to give an upper limit to the settlement and predicted values are commonly between one and three times those actually observed .

Settlements in overconsolidated sands are likely to be considerably less than predicted by this method which was intended for normally consolidated soils (De Beer (1965)) .

2.7 Pile groups subjected to lateral load

The various factors governing the lateral deflection and the ultimate strength of laterally loaded pile groups are :

a - The dimensions of the pile

b - The properties of the pile material

c - The type of soil

d - The batter angle

e - The arrangements of piles in the group

f - The spacing of piles in the group

g - The batter ratio (ie , the number of battered piles to total number of piles in the group)

h - The rotational restraint and pile end conditions .

The piles in the group tend to react as less than the sum of the individual piles if the pile spacing is decreased to less than eight pile diameters in the direction of the load and one or three pile diameters normal to the load .

The moments and deflections of a laterally loaded pile increase , for a given load , as the relative density of the sand decreases . Pile driving tends to increase the density of the soil within and around the pile group , so that deflections and moments under given load are thereby reduced (Parkash (1962)) .

Cyclic loading tends to increase the deflections and moments at a decreasing rate . After about 50 to 100 cycles , an equilibrium position is reached . An increase in the load level will start the process again . The moments tend to reach a constant level before the deflections (Parkash

(1962) , Parkash and Chandrasekaran (1970) , Oteo (1973)) .

The soil becomes progressively more elastic with the increase of number of cycles and a nearly perfect elastic state (defined by reversible load - deflection characteristics) is attained after a certain number of repetitions (Parkash and Chandrasekaran (1970)) .

Battered piles in the group offer larger resistance to sustained lateral loading . The performance of a negatively battered pile is generally poor as compared with that of positively battered piles (Parkash and Chandrasekaran (1970)) .

The effect of the pile cap on the horizontal bearing capacity of a vertical pile , founded in sand is very small . The load carried by the cap ranged from 0 to 7% of the bearing capacity of similar free standing pile group .

2.8 Theoretical analyses for pile groups

From the preceding sections of this chapter , it is clear that an ideal method of analysis would need to take account of the following :

a - Any three dimensional combination of forces and moments applied to the pile cap .

b - The geometry of the pile group , including the number , diameter , length , stiffness , spacing , configuration , and batter of the piles .

c - The rigidity of the pile cap and the effect of contact (if any) between the cap and the soil .

d - The effect of interaction between closely spaced piles under all combinations of axial and transverse load .

e - The variation of the soil behaviour with depth .

f - The non - linearity of the stress / strain behaviour of the soil .

g - The effect of pile driving on the behaviour of soil surrounding each pile .

h - The effect of cyclic loading .

The analysis should yield values for the displacement and rotation of the pile cap and for the axial load , shear force and bending moment in each pile . It should not make excessive demands on a digital computer .

No analytical method exists which will meet all these criteria , but with the exception of (g) and (h) above , all the factors can be allowed for in one or more of the methods described below . The final choice will depend on the relative importance of the factors in the particular case being considered .

The following is an evalution ~~for~~ of the different theoretical methods which can deal with the settlement and load distribution problem for pile groups when subjected to general loading :

1 - Three - dimensional frame analysis .

2 - Methods based on a modulus of transverse subgrade reaction .

(Three - dimensional stiffness method)

3 - Linear elastic methods .

(The pile - soil interation method)

a - Integral equation method .

b - Finite element method .

1 - Three - dimensional frame analysis

This method assumes that :

a - The piles behave as columns subjected to axial force and moment .

b - The presence of the soil is ignored .

c - The pile heads are rigidly connected to a rigid cap .

d - The pile bases are fixed or pinned in a three - dimensional space .

This method has been found to lead to inaccurate estimates of displacements and moments even for very soft soils or for end bearing piles .

2 - Methods based on a modulus of transverse subgrade reaction

This method has been developed by Kishida and Meyerhof (1965) , Reese (1971) , and Bowles (1974) , is essentially the same as the frame analysis method , except that the presence of soil in the direction normal to the pile axis is included by treating the soil as an ideally elastic Winkler medium . The behaviour of the medium at any point is defined by a single elastic constant , the modulus of subgrade reaction (k_H) . (Terzaghi (1955))

This method can be applied to any end bearing pile groups but not to floating pile groups because a Winkler medium cannot transmit shear from the piles to the surrounding soil in these situations . It is possible to extend this method to deal with such problems by using two sets of spring constants , k_{H1} to transmit the loading intensity acting normal to the pile axis , and k_{H2} to transmit the shear stress along the pile shaft surface .

In this method the main disadvantage lies in the difficulty in assessing the value of the modulus of subgrade reaction (k_H) .

The value of the modulus of subgrade reaction (k_H) that can be used to analyse the behaviour of pile groups depends upon :

1 - The properties of the soil .

2 - The length to diameter ratio of the piles .

3 - The diameter of the piles .

4 - The compressibility (EA) and the flexibility (EI) of the piles .

5 - The height of the cap above ground .

6 - The spacing of the piles .

7 - The angle of rake , including the sign .

8 - The number of piles in the group .

9 - The relative magnitudes of the ratio of the vertical loads to that of the horizontal loads .

10 - Whether the piles are end - bearing or floating within the soil layer .

11 - The method of construction employed (boring or driving) .

12 - The order in which piles are being driven .

13 - The separation of the soil from the pile near the ground level .

14 - The compaction of the soil .

The use of a test pile to determine suitable values for the coefficients of subgrade reaction is discussed in chapter 3 .

3 - <u>Linear elastic methods</u>

a - <u>Integral equation method</u>

This method has been developed by Poulus (1968 , 1971 , 1972) , and by Butterfield and Banerjee (1970 , 1971) , and is based on the assumption that the piles themselves are linearly elastic and are embedded in another linearly elastic three - dimensional solid .

It uses an integral equation method , for representing the pile soil interfaces , which is similar to Mindlin's solution for a point load within a semi - infinite , elastic solid . Such an idealisation differs in many respects from real soil .

This method has the following disadvantages :

1 - The coefficient of soil elasticity (E_s) will not only vary from one point to another in the soil mass , but at a given point it will be a function of stress condition at that point .

2 - Near failure , Mindlin's equation , which has been used for calculating displacement at any point in the elastic soil mass , can not be used , therefore this method is limited to working load only .

3 - The accuracy of the solutions depends upon the number of elements into which the pile is divided , but fine subdivision of the pile makes large demands on computer store and time .

b - The finite element method

The finite element method may be regarded as an extension of the stiffness method used in structural analysis to continuum problems or to combined problems .

This extension is effected by representing the piles and the soil as an assembly of small elastic finite elements connected together at node points .

The advantages of the method in it's flexibility . It is possible to model complicated geometry . Variations in elastic properties use simply provided for , as the elastic constants may be different in each element . The method may be adopted to allow for non - linear material properties .

The major disadvantage of the method is the very large demand made on the computer . This has hitherto prevented the use of the method except in very simple cases .

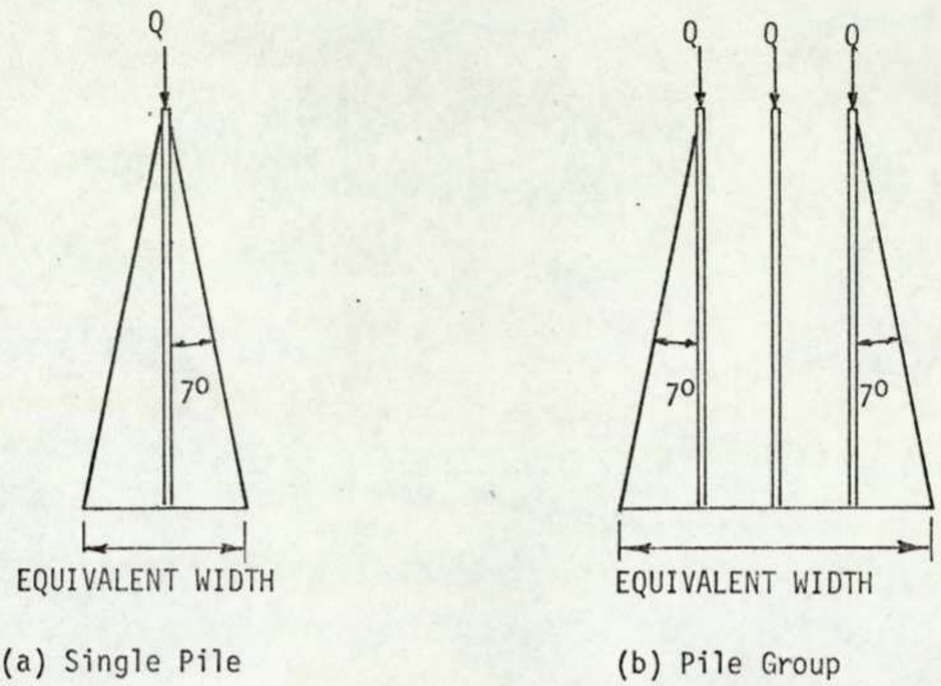


Fig. 2.1 Settlement calculations for a pile group in cohesionless soil (after Beresantzev et al, 1961)

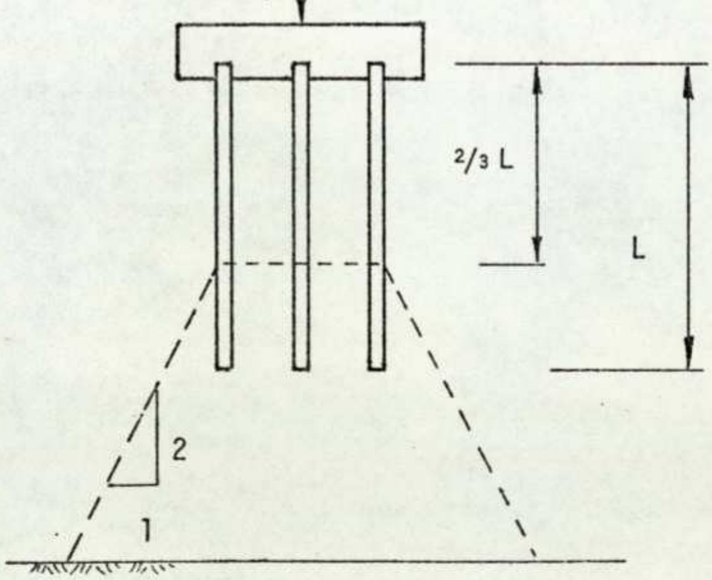


Fig. 2.2 Calculation of settlement of pile groups

Chapter 3

Analytical method and computer programme for a single laterally loaded pile

3.1 Introduction

This chapter presents an analysis based on a modulus of lateral subgrade reaction . The method uses a numerical solution (based on finite differences) to determine the transverse displacement , and hence the bending moment , at points along the pile . The computer programme uses an iterative procedure to allow for the non - linear response of the soil to lateral pressure .

The programme is first used to compute the elastic and plastic soil constants which give the best fit to the observed behaviour of a single test pile under transverse loading . It is then used to compute the moments and forces at the pile head resulting from unit transverse displacement and unit transverse rotation , assuming the soil response is elastic . This information is required for use in the programme PILEGROUP described in the next chapter .

3.2 The modulus of lateral subgrade reaction

The soil reaction at any point on the pile may be expressed in the form

$$q = y\,k_H \qquad (3.1)$$

where , q is the transverse soil pressure on the pile

y is the transverse displacement of the pile at this point

k_H is a modulus of subgrade reaction .

The modulus k_H also increases with depth in sand and , for very small horizontal displacements may be assumed to have the form

$$k_H = a_m \left(\frac{Z}{B} \right)^n \qquad (3.2)$$

where , Z is the depth below ground level

a_m and n are constant coefficients .

Terzaghi (1955) proposed taking n = 1.0 for sands . A preliminary comparison between predicted and observed behaviour of a single pile (Fig. 3.1) shows good agreement for the magnitude and position of the maximum bending moment , if n is taken to be 1.0 . Further down the pile the observed moment is rather larger than predicted and agreement here might have been somewhat improved by taking a smaller value for n . However this is of secondary importance and it is assumed that

$$k_H = \frac{a_m Z}{B} \qquad (3.3)$$

for small displacements .

Terzaghi also suggested values for a_m but these are known to give too large a value for horizontal displacement and in all the work which follows the values of a_m have been determined by back analysis of the observed behaviour of a single laterally loaded test pile .

Sands behave nonlinearly even under quite small loads , q eventually reaching a maximum value as the soil fails . Parker and Reese (1971) have computed the following theoretical maximum values for q .

Near the ground surface

$$q_{max} = \gamma Z \{ K_p - K_a + \frac{Z}{B} \tan \beta \left[K_p \tan \alpha + K_o (\tan \phi - \tan \alpha) \right] \} \qquad (3.4)$$

where , $\beta = 45^{o} + \phi / 2$

α is an angle defining the size of the sliding wedge in front of the pile and is assumed (by Parker and Reese) to be equal to $\phi / 2$

At greater depth

$$q_{max} = \gamma Z \{ K_p^3 + 2 K_o \tan \phi (K_p^2 + 1) - K_a \} \qquad (3.5)$$

Thus the ultimate resistance q_{max} may be expected to vary with depth as shown in Fig. 3.2 .

A preliminary investigation indicates that the effect of the reduced ultimate resistance near the ground surface is small and it is therefore assumed that

$$q_{max} = p_u Z \qquad (3.6)$$

where , p_u is a constant determined by back analysis of a test on a single laterally loaded pile .

At stresses lower than the ultimate , Parker and Reese suggested that the lateral pressure may be expressed (see Fig. 3.3) in the form

$$q = q_{max} \tanh \{ \frac{k_H y}{q_{max}} \} \qquad (3.7)$$

$$= p_u Z \tanh \{ \frac{a_m y}{p_u B} \} \qquad (3.8)$$

A comparison of predicted and observed displacements of a pile head under increasing transverse load shows good agreement except at small loads (Fig. 3.4) . Here the predicted displacement is somewhat larger than that observed , but this again is of secondary importance , and it is therefore assumed that

$$q = p_u \, Z \tanh \left\{ \frac{a_m \, y}{p_u \, B} \right\}$$

where , a_m and p_u are constants determined by back analysis of a test on a single laterally loaded pile .

3.3 General method of analysis using finite differences

Consider the pile as a vertical or inclined beam of breadth B and length Z . Then , the general expressions for bending are :

$$E\,I \frac{d^2y}{dZ^2} = M \tag{3.9}$$

$$E\,I \frac{d^4y}{dZ^4} = (p - q) \, B \tag{3.10}$$

where , M is the bending moment

p is the intensity of external applied load (per unit area of the pile face)

q is the intensity of soil reaction (per unit area of pile face)

y is the lateral displacement

Z is the longitudinal dimension

E I is the flexural rigidity of the pile .

Now consider the pile divided into m equal parts by m+1 node points , spaced $h = \frac{Z}{m}$ apart (Fig. 3.5) .

For any node point (i) , not near the ends , the expressions (3.9) and (3.10) may be written in finite difference form as follows :

$$y_{i-1} - 2\,y_i + y_{i+1} = \frac{M_i\,h^2}{E\,I} \qquad (3.11)$$

$$y_{i-2} - 4\,y_{i-1} + 6\,y_i - 4\,y_{i+1} + y_{i+2} = (\,p_i - q_i\,)\,\frac{B\,h^4}{E\,I} \qquad (3.12)$$

3.3.1 Boundary conditions at the pile head

Let M_1 , P_1 , and Θ_1 be the moment , applied force and rotation at the pile head (node 1) (see Fig. 3.5)

Then between nodes (1) and (2) , at any depth Z below (1) ,

$$\frac{d^2y}{dZ^2} = \frac{M}{E\,I} = \frac{M_1}{E\,I} + \frac{P_1\,Z}{E\,I}$$

$$\frac{dy}{dZ} = \frac{M_1\,Z}{E\,I} + \frac{P_1\,Z^2}{2\,E\,I} - \Theta_1$$

$$(\text{ since } \frac{dy}{dZ} = -\,\Theta_1 \quad \text{if } Z = 0\,)$$

Then ,

$$y = \frac{M_1\,Z^2}{2\,E\,I} + \frac{P_1\,Z^3}{6\,E\,I} - \Theta_1 Z + y_1$$

$$(\text{ since } y = y_1 \;,\; \text{if } Z = 0\,)$$

The equations above may be rewritten as follows :

Moment $\quad M = M_1 + P_1\,Z$

Rotation $\quad \Theta = \Theta_1 - \frac{M_1\,Z}{E\,I} - \frac{P_1\,Z}{2\,E\,I}$

Displacement $\quad y = y_1 - \Theta_1\,Z + \frac{M_1\,Z^2}{2\,E\,I} + \frac{P_1\,Z^3}{6\,E\,I}$

Then at node (2)

$$\Theta\,h - y_1 + y_2 = \frac{M_1\,h^2}{2\,E\,I} + \frac{P_1\,h^3}{6\,E\,I} \qquad (3.13)$$

Also at node (2) (using eq. 3.11)

$$y_1 - 2\,y_2 + y_3 = \frac{M_2\,h^2}{E\,I} = \frac{M_1\,h^2}{E\,I} + \frac{P_1\,h^3}{E\,I} \qquad (3.14)$$

At node (3) (using eq. 3.11)

$$y_2 - 2\,y_3 + y_4 = \frac{M_3\,h^2}{E\,I} = \frac{M_1\,h^2}{E\,I} + \frac{2P_1\,h^3}{E\,I} - q_2\,\frac{B\,h^4}{E\,I} \qquad (3.15)$$

Also at node (3) (using eq. 3.12)

$$y_1 - 4\,y_2 + 6\,y_3 - 4\,y_4 + y_5 = -\,q_3\,\frac{B\,h^4}{E\,I} \qquad (3.16)$$

Alternatively , if M_1 and P_1 are unknown while y_1 and Θ_1 are known , equations (3.13) to (3.16) may be rewritten in the form :

$$\frac{M_1\,h^2}{2\,E\,I} + \frac{P_1\,h^3}{6\,E\,I} - y_2 = \Theta_1\,h - y_1 \qquad (3.17)$$

$$\frac{M_1\,h^2}{E\,I} + \frac{P_1\,h^3}{E\,I} + 2\,y_2 - y_3 = y_1 \qquad (3.18)$$

$$\frac{M_1\,h^2}{E\,I} + \frac{2\,P_1\,h^3}{E\,I} - y_2 + 2\,y_3 - y_4 = q_2\,\frac{B\,h^4}{E\,I} \qquad (3.19)$$

$$-\,4\,y_2 + 6\,y_3 - 4\,y_4 + y_5 = -\,y_1 - q_3\,\frac{B\,h^4}{E\,I} \qquad (3.20)$$

3.3.2 Boundary conditions at the toe of the pile

These are that the moment and shear force at the toe are both zero .

$$M_{m+1} = V_{m+1} = 0$$

By a similar process to that described above , we obtain ,

$$y_{m-2} - 4\, y_{m-1} + 5\, y_m - 2\, y_{m+1} = - q_m \frac{B\, h^4}{E\, I} \qquad (3.21)$$

$$2\, y_{m-1} - 4\, y_m + 2\, y_{m+1} = - q_{m+1} \frac{B\, h^4}{E\, I} \qquad (3.22)$$

3.3.3 Finite difference equations in matrix form

The equations therefore have one or other of the forms given on the next two pages .

In the computer programmes PILE and PILEGROUP , the appropriate form of matrix R is set up by subroutine FORMR .

Subroutine RHS sets up the appropriate form of matrix $\{ P \}$

Then ,

$$\left[R \right] \{ y \} = \{ P \} - \{ Q \} \qquad (3.23)$$

Case A (Θ_1 and y_1 are unknown , M_1 and P_1 are known)

$$
\begin{bmatrix}
+1 & -1 & +1 & & & & & & & & & \\
0 & +1 & -2 & +1 & & & & & & & & \\
0 & 0 & +1 & -2 & +1 & & & & & & & \\
0 & +1 & -4 & +6 & -4 & +1 & & & & & & \\
0 & 0 & +1 & -4 & +6 & -4 & +1 & & & & & \\
 & & & & & & & & & & & \\
 & & & & & & & +1 & -4 & +6 & -4 & +1 \\
 & & & & & & & & +1 & -4 & +5 & -2 \\
 & & & & & & & & & +2 & -4 & +2
\end{bmatrix}
\left\{ \begin{array}{c} \Theta_1 h \\ y_1 \\ y_2 \\ y_3 \\ y_4 \\ : \\ : \\ : \\ y_{m-1} \\ y_m \\ y_{m+1} \end{array} \right\}
=
\left\{ \begin{array}{r} M_1 h^2 / 2EI + P_1 h^3 / 6EI \\ M_1 h^2 / EI + P_1 h^3 / EI \\ M_1 h^2 / EI + 2 P_1 h^3 / EI - q_2 B h^4 / EI \\ - q_3 B h^4 / EI \\ - q_4 B h^4 / EI \\ \\ \\ \\ -q_{m-1} B h^4 / EI \\ - q_m B h^4 / EI \\ - q_{m+1} B h^4 / EI \end{array} \right\}
$$

$$[R]\{y\} = \{P\} + \{Q\}$$

Case B (M_1 and P_1 are unknown , Θ_1 and y_1 are known)

$$
\begin{bmatrix}
+\frac{1}{2} & +\frac{1}{6} & -1 & & & & & & \\
+1 & +1 & +2 & -1 & & & & & \\
+1 & +2 & -1 & +2 & -1 & & & & \\
0 & 0 & -4 & +6 & -4 & +1 & & & \\
0 & 0 & +1 & -4 & +6 & -4 & +1 & & \\
 & & & & & & & & \\
 & & & & & & & & \\
 & & & & +1 & -4 & +6 & -4 & +1 \\
 & & & & & +1 & -4 & +5 & -2 \\
 & & & & & & +2 & -4 & +2
\end{bmatrix}
\begin{Bmatrix}
M_1 h^2 / EI \\
P_1 h^3 / EI \\
y_2 \\
y_3 \\
y_4 \\
: \\
: \\
y_{m-1} \\
y_m \\
y_{m+1}
\end{Bmatrix}
=
\begin{Bmatrix}
\Theta_1 h - y_1 \\
y_1 \\
0 \\
-y_1 \\
0 \\
 \\
 \\
0 \\
0 \\
0
\end{Bmatrix}
-
\begin{Bmatrix}
0 \\
0 \\
- q_2 B h^4 / EI \\
q_3 B h^4 / EI \\
q_4 B h^4 / EI \\
 \\
 \\
q_{m-1} B h^4 / EI \\
q_m B h^4 / EI \\
q_{m+1} B h^4 / EI
\end{Bmatrix}
$$

$$[R]\{y\} = \{P\} - \{Q\}$$

3.3.4 Soil reaction terms (matrix { Q })

The intensity of soil reaction at any node (i) is assumed to be :

$$q_i = p_u Z_i \tanh \left\{ \frac{a_m y_i}{p_u B} \right\} \qquad (3.24)$$

where , Z_i is the depth of node (i) below the ground surface

a_m , p_u are coefficients governing the linear and non linear behaviour of the soil respectively .

Then we may write ,

$$q_i \frac{B h^4}{E I} = K_i y_i \qquad (3.25)$$

where ,

$$K_i = p_u Z_i \tanh \left\{ \frac{a_m y_i}{p_u B} \right\} \frac{B h^4}{E I y_i} \qquad (3.26)$$

The programme proceeded iteratively by successive improvement . The existing values of y_i at each stage are used to compute values of K_i . These are in turn used to compute improved values of y_i (see Fig. 3.6) , then

$$\{ Q \} = \left[K \right] \{ y \} \qquad (3.27)$$

where , $\left[K \right]$ is a diagonal matrix of K_i terms .

But

$$\left[R \right] \{ y \} = \{ P \} - \{ Q \} \qquad (3.28)$$

Then we may write ,

$$\left[S \right] = \left[R \right] + \left[K \right] \qquad (3.29)$$

and

$$\left[S \right] \{ y \} = \{ P \} \qquad (3.30)$$

Matrix $\left[S \right]$ is formed by subroutine SOIL .

3.3.5 Solution of the equations (subroutine SOLN)

Matrix $\left[S \right]$ has a band width of only 5 , and each solution is required once only . The best method is therefore a simple Gauss elimination routine which may be expressed in matrix form as shown below ,

The set of (m + 1) equations are partitioned as shown below ,

$$\left[\begin{array}{c|c} S_{11} & S_{12} \\ \hline S_{21} & S_{22} \end{array}\right] \left[\begin{array}{c} y_1 \\ \hline y_2 \end{array}\right] = \left[\begin{array}{c} P_1 \\ \hline P_2 \end{array}\right]$$

where , S_{11} is a 1 x 1 matrix

S_{12} is a 1 x m matrix

S_{21} is a m x 1 matrix

S_{22} is a m x m matrix

Then this may be reduced to a set of m equations of the form

$$\left[\bar{S} \right] \left[\bar{y} \right] = \left[\bar{P} \right]$$

where ,

$$\left[\bar{S} \right] = \left[S_{22} \right] - \left[S_{21} \right] \left[S_{11} \right]^{-1} \left[S_{12} \right]$$

$$\left[\bar{P} \right] = \left[P_2 \right] - \left[S_{21} \right] \left[S_{11} \right]^{-1} \left[P_1 \right]$$

The procedure is then repeated until $\left[\bar{S} \right]$ is reduced to a single element when direct solution is possible for y_{m+1} .

The remaning values of y may be obtained by back substitution of the form ,

$$y_1 = \left[S_{11} \right]^{-1} \left[P_1 \right] - \left[S_{11} \right]^{-1} \left[S_{12} \right] \left[y_2 \right]$$

$\left[S_{11} \right]^{-1}$ is written over $\left[S_{11} \right]$, while $\left[S_{12} \right]$ and $\left[S_{21} \right]$ are retained at each stage so that the whole solution is performed within the original band width of the matrix .

3.4 Programme PILE

General This programme is designed :

a - To determine the values of a_m and p_u by back analysis of observed displacements of the pile head under transverse load .

b - To compute values of displacement , rotation and bending moment for each stage of loading , for comparsion with observed values .

c - To compute elastic coefficients of pile head stiffness for use in matrix [B] of programme PILEGROUP .

Data These consist of :

a - Pile geometry

b - Loads applied to the pile head at each stage of the test

c - First approximations to a_m and p_u

d - Two values of displacement $y_{1(e)}$ and $y_{1(p)}$ determined from the test results as shown in Fig. 3.7 .

3.4.1 Procedure

(see flow chart attached)

Stage 1

a - Form matrix $[R]$ for case A (y_1 , Θ_1 , unknown)

b - Compute matrix $[S]$ for $y_i = 0$ only (so that $K_i = a_m Z_i \dfrac{h^4}{EI}$ is independent of p_u)

c - Using this value of $[S]$ compute y_1 and compare it with $y_{1(e)}$

d - If $y_1 - y_{1(e)} \nless y_{1(e)} / 100$ correct the value of a_m and repeat .

Stage 2

As above but procede iteratively by successive improvement until the correct values y_i have been used to compute K_i .

Compare the last value of y_i with the given value of $y_{1(p)}$.

If $y - y_{1(p)} \nless y_{1(p)} / 100$, correct the value of p_u and repeat .

Stage 3

Using the values of a_m and p_u computed above , calculate the displacements and moments for each stage of loading used in the test and print these for comparsion with the observed values .

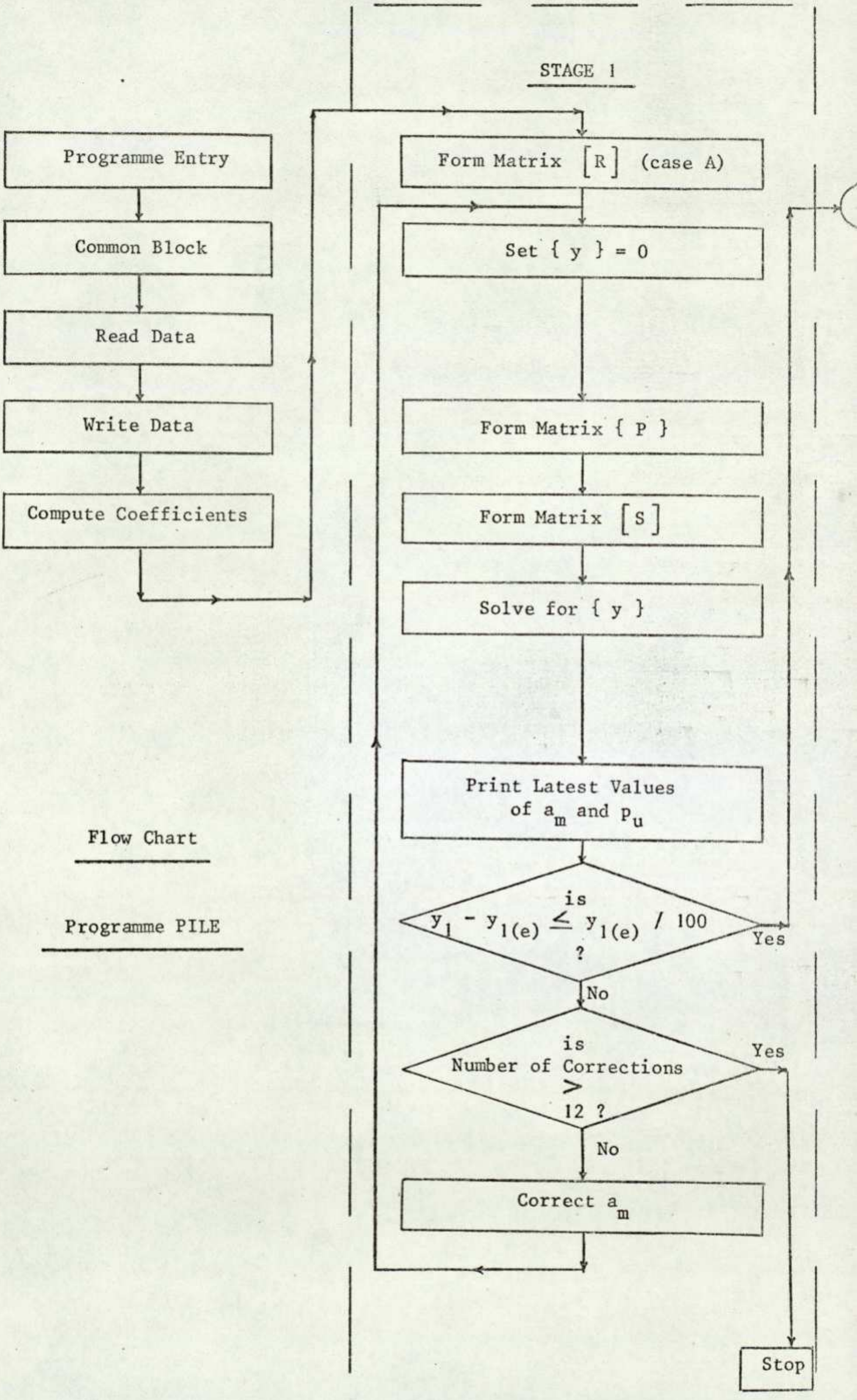
Stage 4 and 5

a - Form matrix $\left[R \right]$ for case B (M_1 and P_1 unknown)

b - Compute matrix $\left[S \right]$ for $y_i = 0$ only (as in stage 1 above)

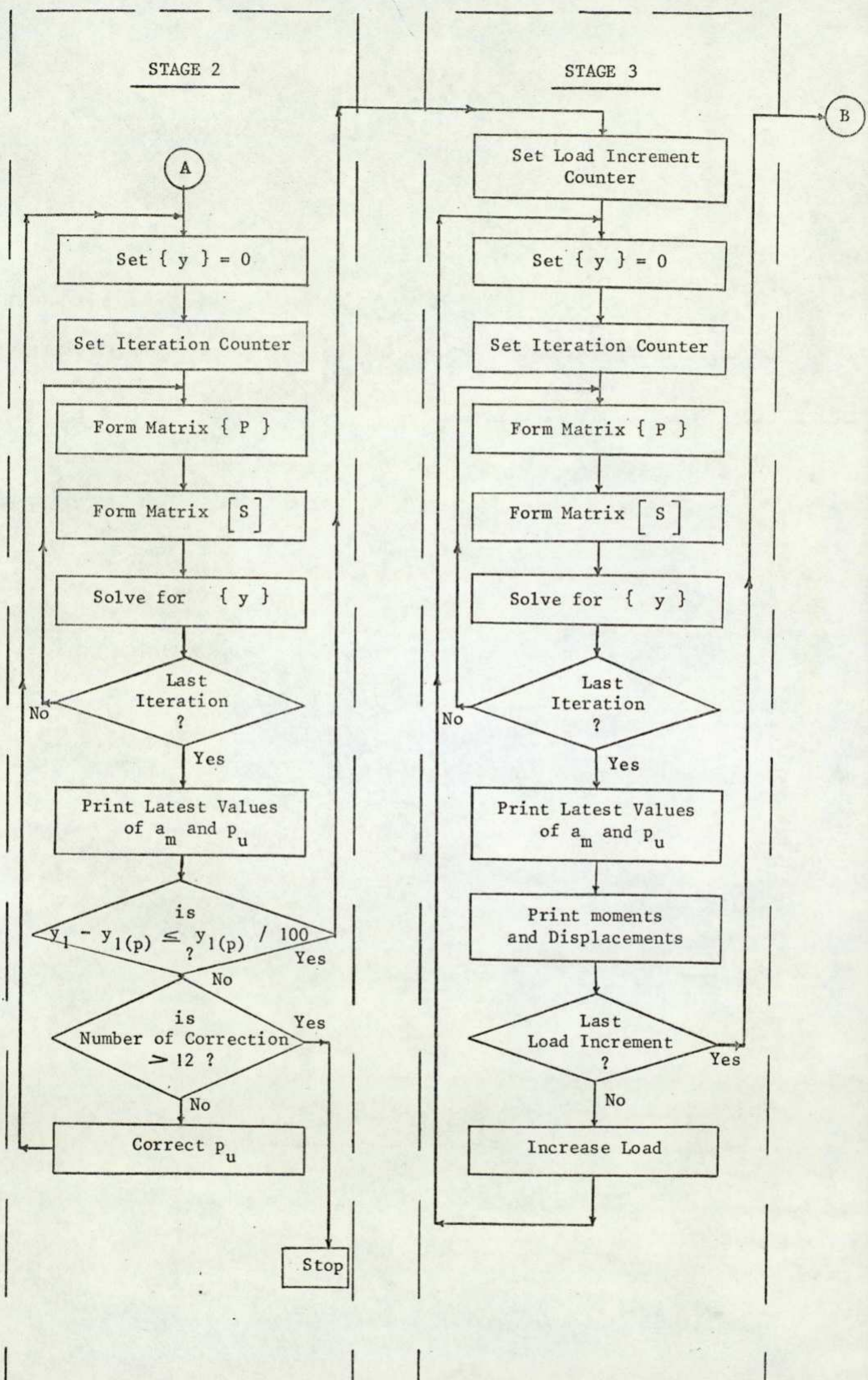
c - For $y_1 = 1.0$ and $\Theta_1 = 0$, compute M_1 and P_1 (stage 4)

d - For $y_1 = 0$ and $\Theta_1 = 1.0$, compute M_1 and P_1 (stage 5)

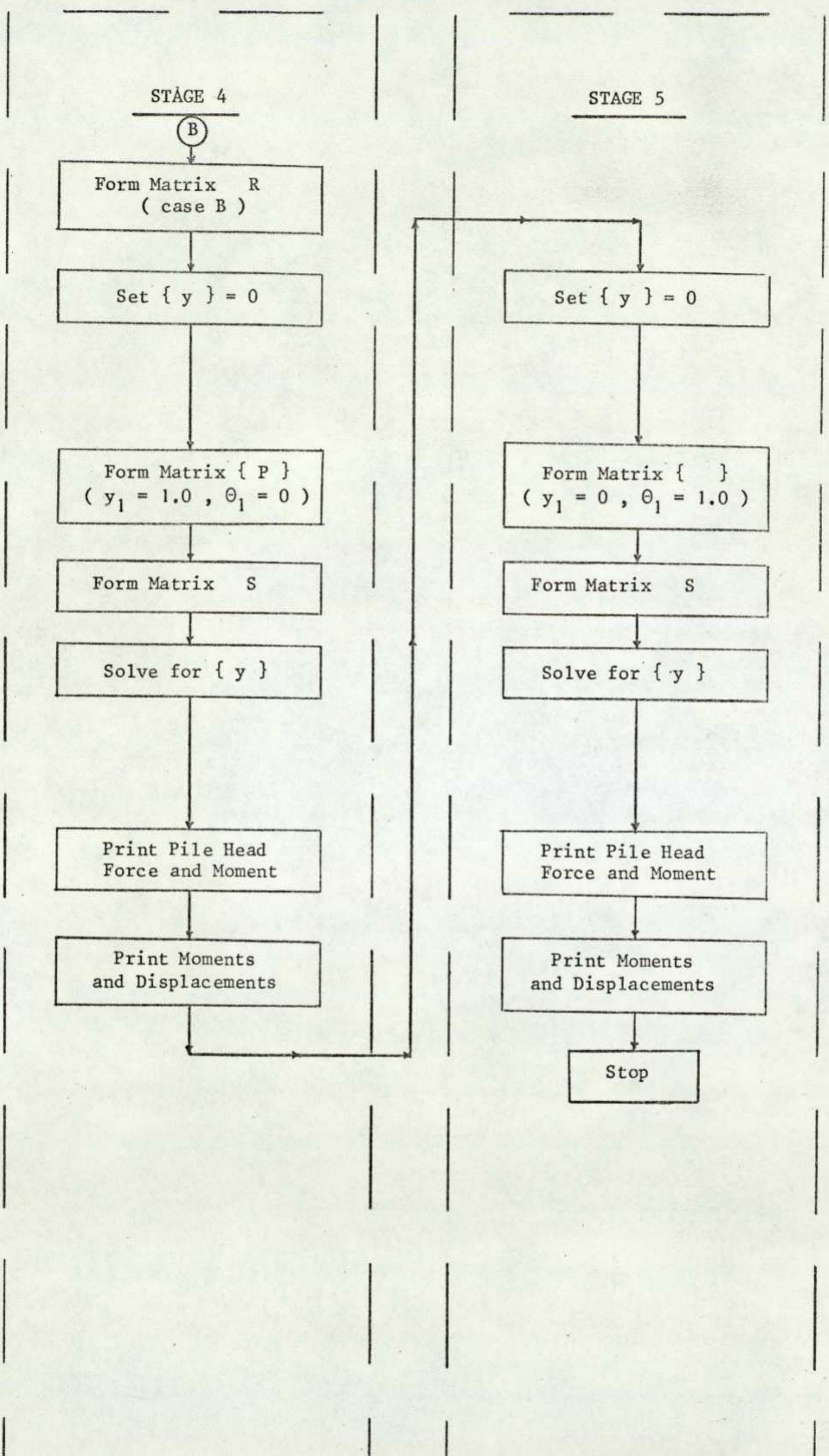


Flow Chart

Programme PILE



STAGE 2
A
Set { y } = 0
Set Iteration Counter
Form Matrix { P }
Form Matrix [S]
Solve for { y }
Last Iteration ?
No
Yes
Print Latest Values of a_m and p_u
is $y_1 - y_{1(p)} \leq y_{1(p)} / 100$?
Yes
No
is Number of Correction > 12 ?
Yes
No
Correct p_u
Stop
STAGE 3
B
Set Load Increment Counter
Set { y } = 0
Set Iteration Counter
Form Matrix { P }
Form Matrix [S]
Solve for { y }
Last Iteration ?
No
Yes
Print Latest Values of a_m and p_u
Print moments and Displacements
Last Load Increment ?
Yes
No
Increase Load


STAGE 4
B
Form Matrix R
(case B)
Set { y } = 0
Form Matrix { P }
(y_1 = 1.0 , Θ_1 = 0)
Form Matrix S
Solve for { y }
Print Pile Head
Force and Moment
Print Moments
and Displacements
STAGE 5
Set { y } = 0
Form Matrix { }
(y_1 = 0 , Θ_1 = 1.0)
Form Matrix S
Solve for { y }
Print Pile Head
Force and Moment
Print Moments
and Displacements
Stop

3.4.2 Data specification

1 - Heading card (10A8)

variable	entry
TITLE	Enter heading information to be printed with out put .

2 - Pile data (3 F 0.0 , 2 I 0)

variable	entry
Z	Length of the pile
B	Breadth of the pile
E I	Stiffness of the pile
M	Number of parts (m)
K	Number of load stages.

3 - Load data - One card per load stage (2 F 0.0)

variable	entry	
P1 (I)	Applied force at pile head	for load
BM1 (I)	Applied moment at pile head	stage I

4 - Soil data (2 F 0.0 , 2 I 0)

variable	entry
AM	First trial value of linear soil coefficient a_m
PU	First trial value of non - linear soil coefficient p_u
MG	Number of node at ground level
L	Number of iterations .

5 - Measured displacement data (4 F 0.0)

varible	entry
Y1E	Measured value of $y_{1(e)}$
Y1P	Measured value of $y_{1(p)}$
AN	Coefficient for correction of a_m
PN	Coefficient for correction of p_u

Test No. 13A

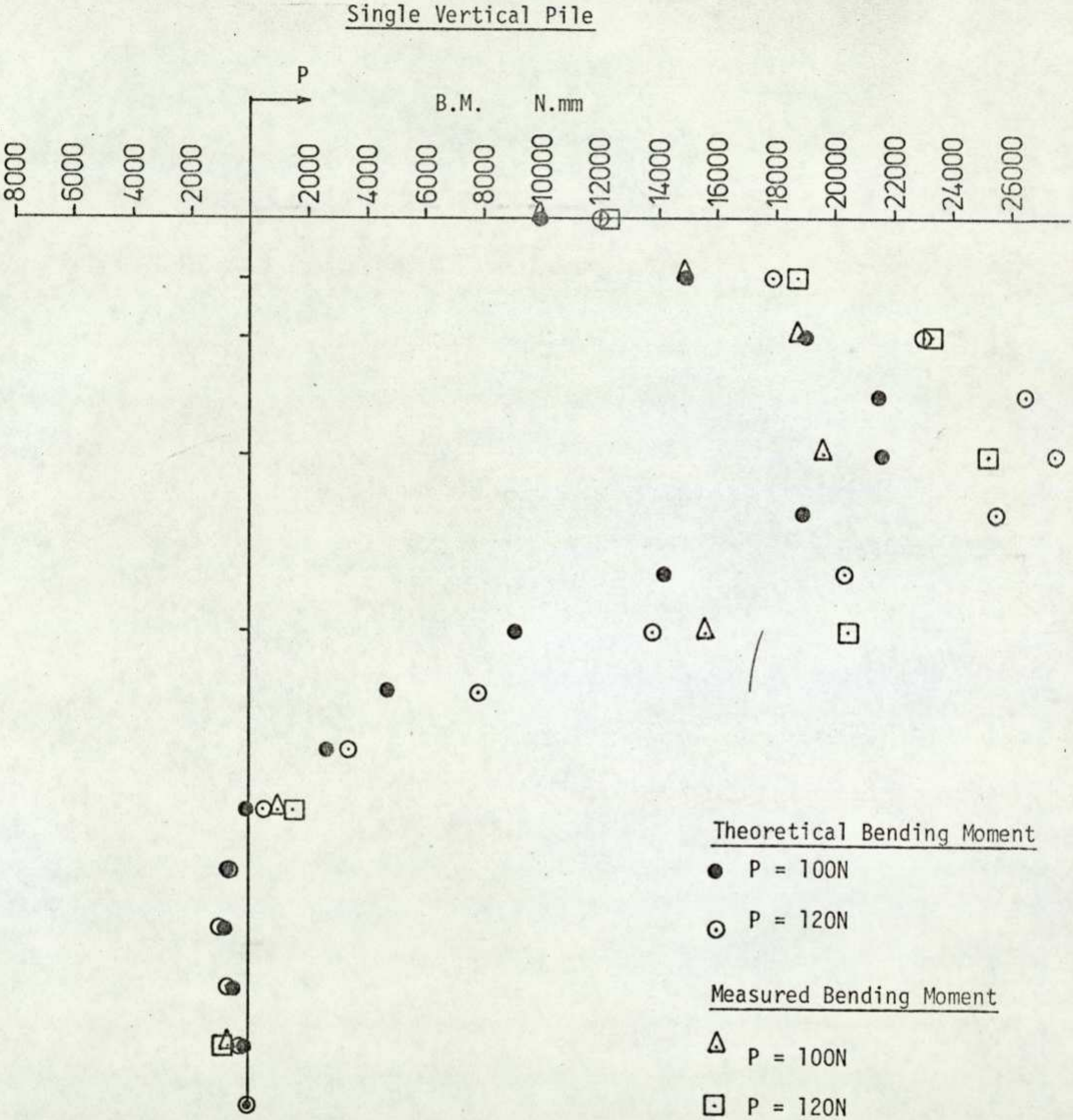
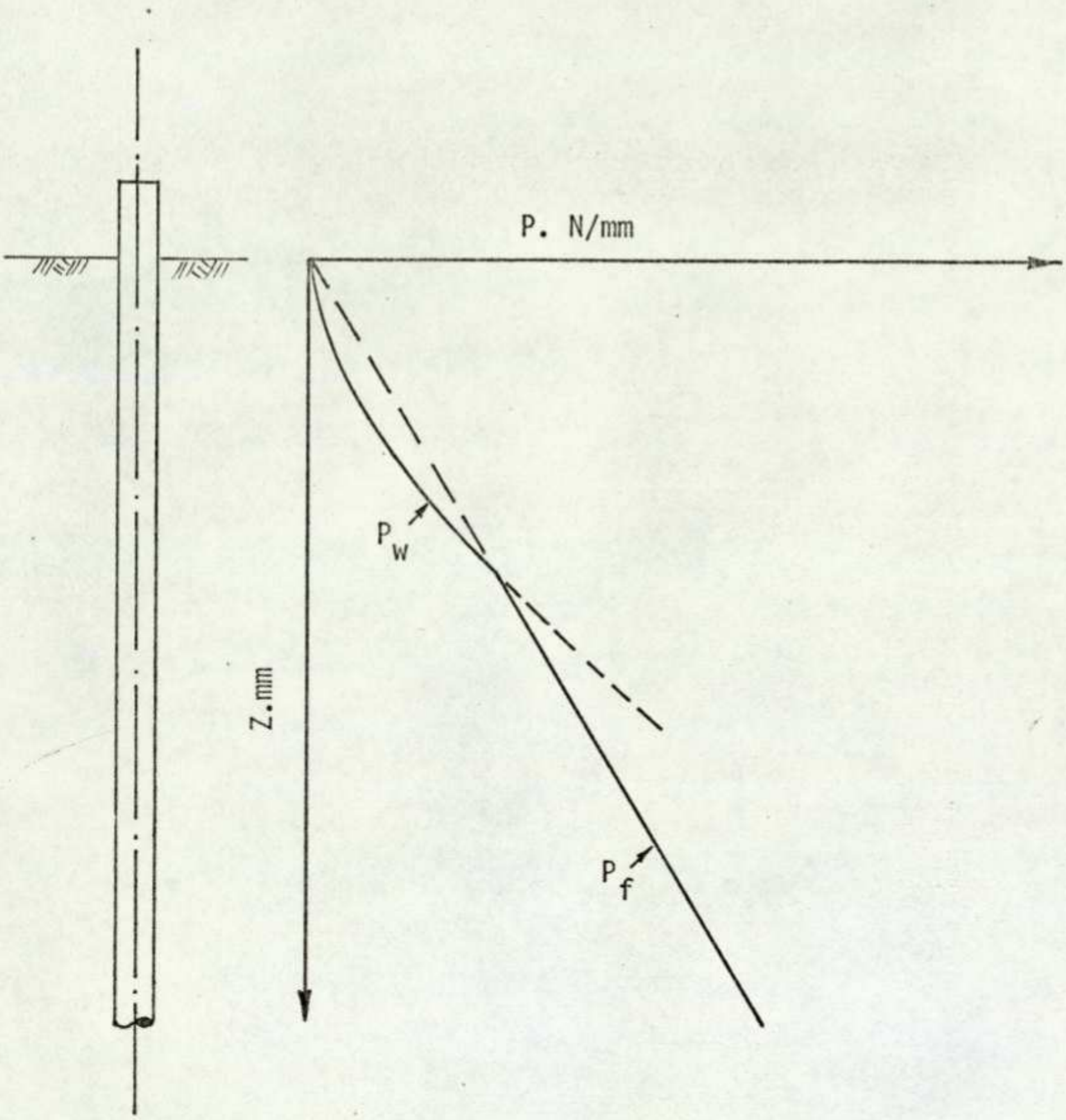


Fig. 3.1 Theoretical and experimental bending moment Vs Depth



P_f - is the ultimate soil resistance per unit length of pile in N/mm

P_w - is the ultimate lateral soil resistance by wedge type failure.

$$q_{max} = \frac{P_w}{B}$$ Near the ground surface.

$$q_{max} = \frac{P_f}{B}$$ At greater depth.

Fig. 3.2 Ultimate lateral soil resistance for sand (after Reese, 1970)

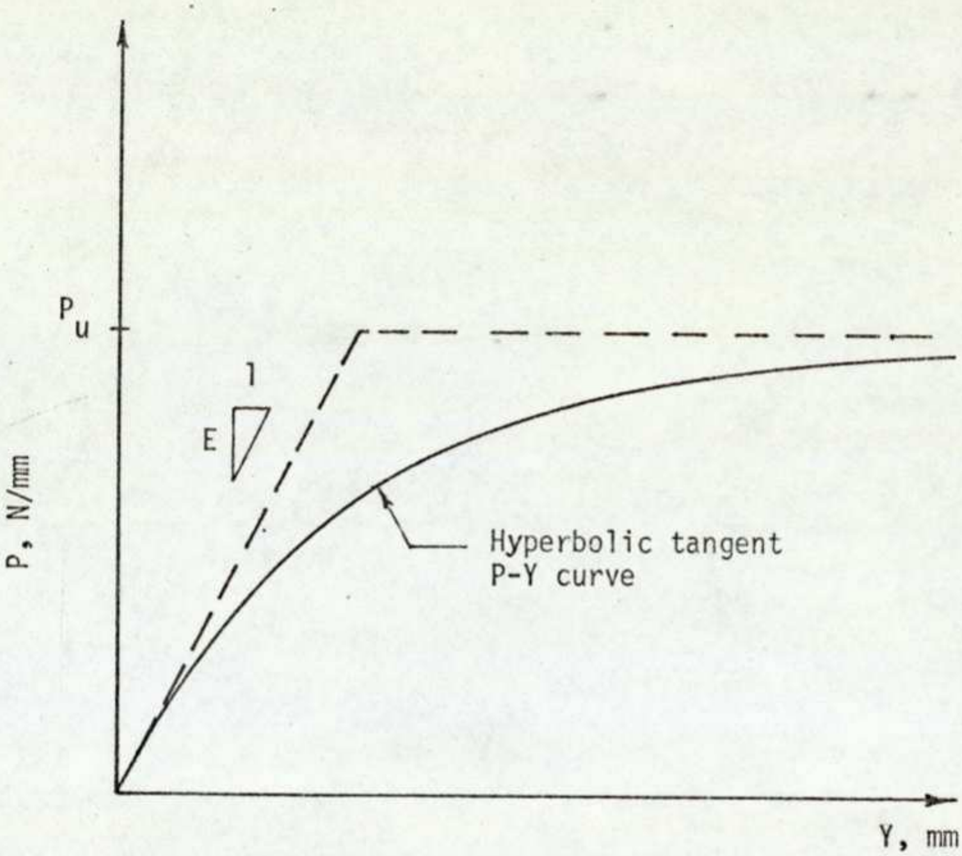


Fig. 3.3 Form of P-Y curves for sand (after Reese, 1970)

Test No. 13A

Single Vertical Pile

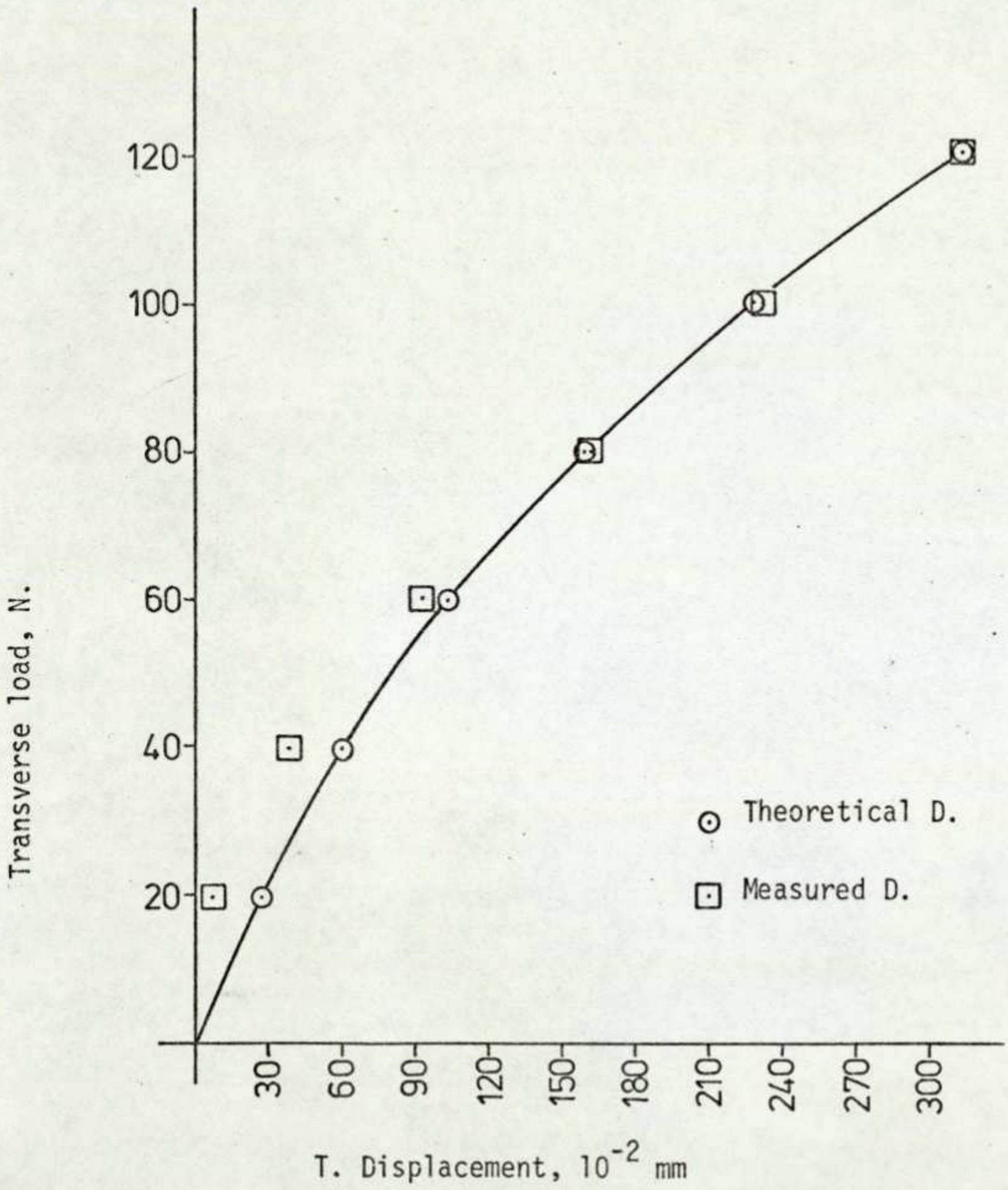
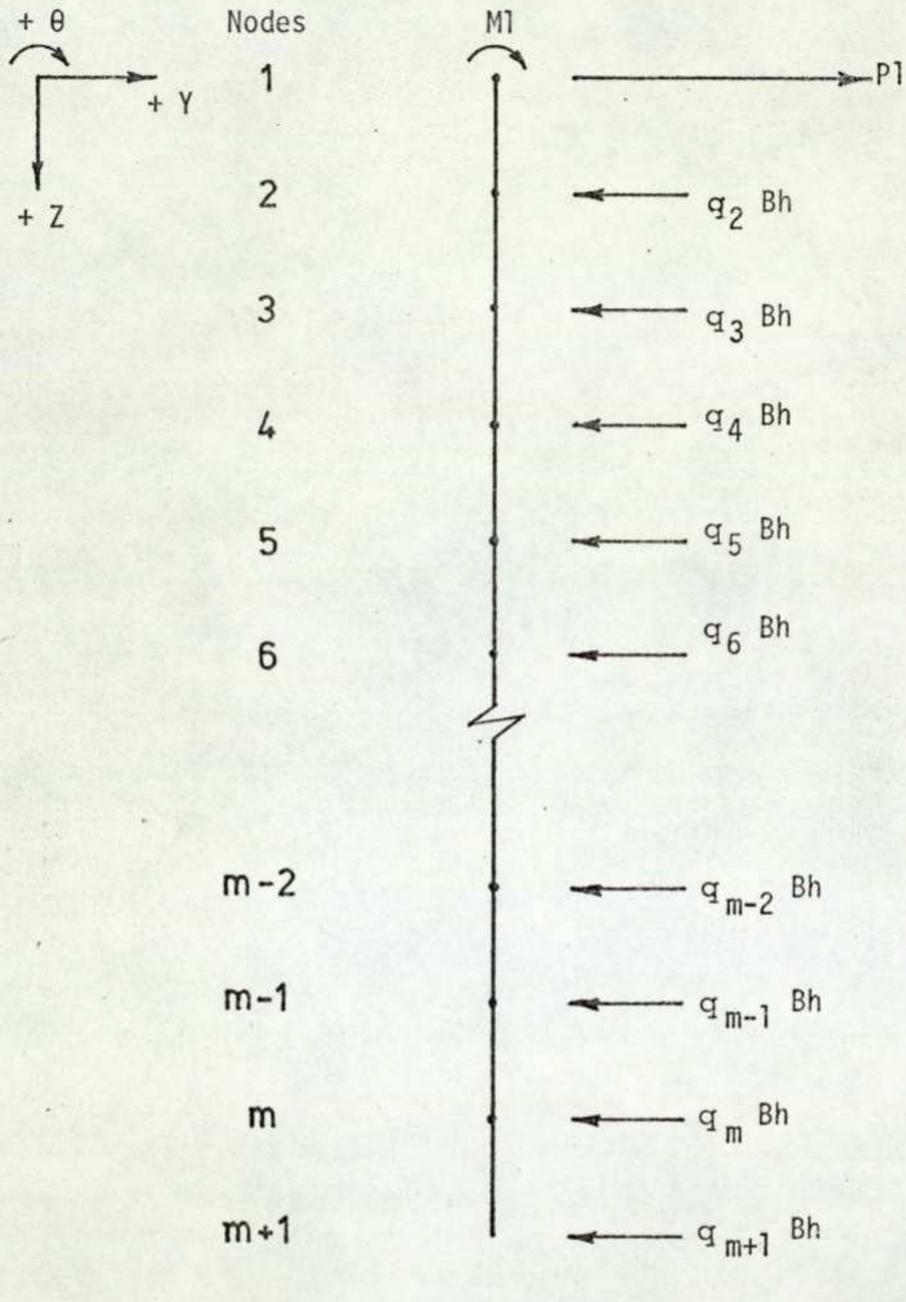


Fig. 3.4 T. Load Vs Theoretical and experimental T. Displacement



Displacements, rotations, forces and moments are taken to be positive in the directions shown.

Fig. 3.5 Representation of pile

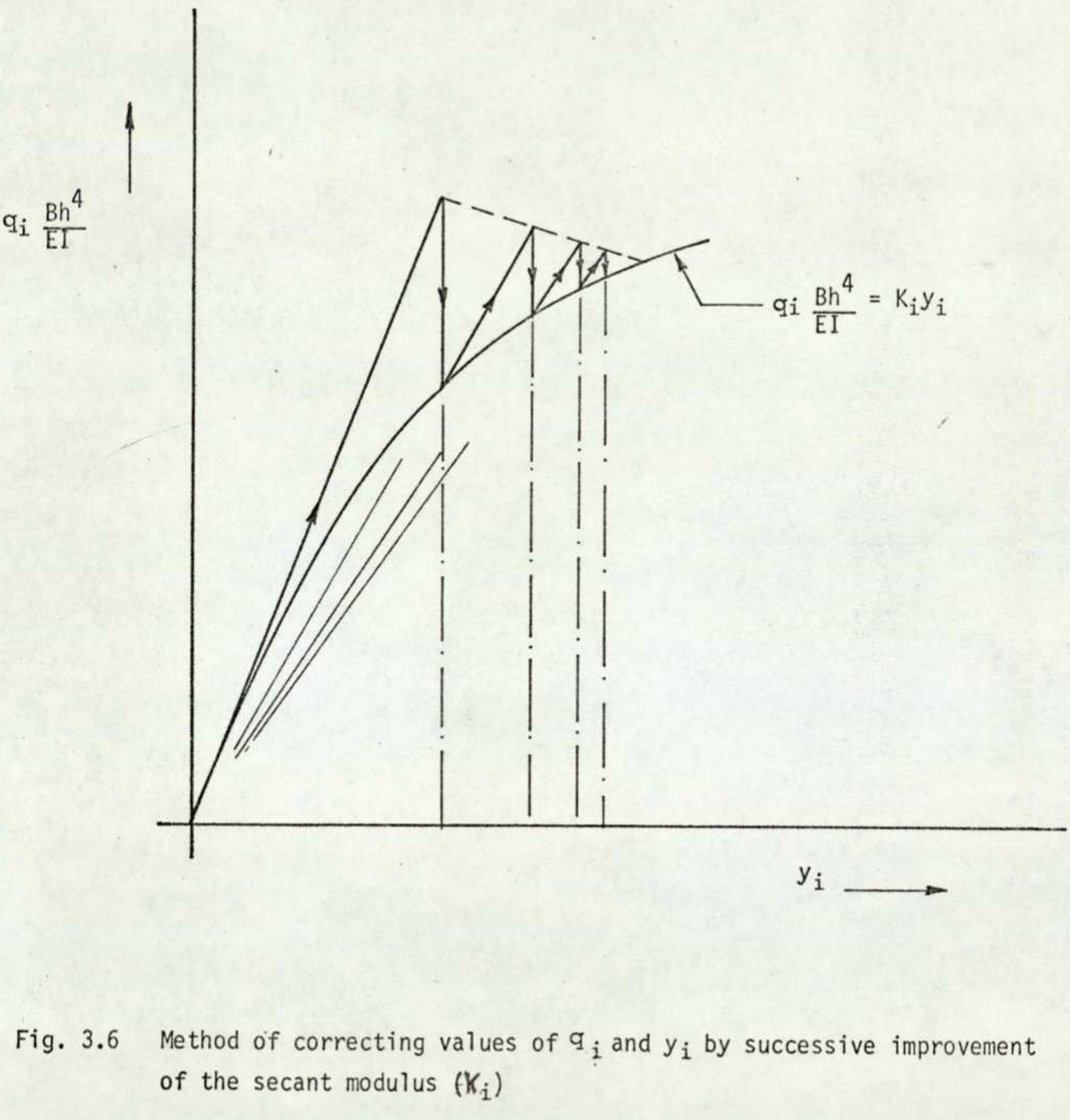


Fig. 3.6 Method of correcting values of q_i and y_i by successive improvement of the secant modulus (K_i)

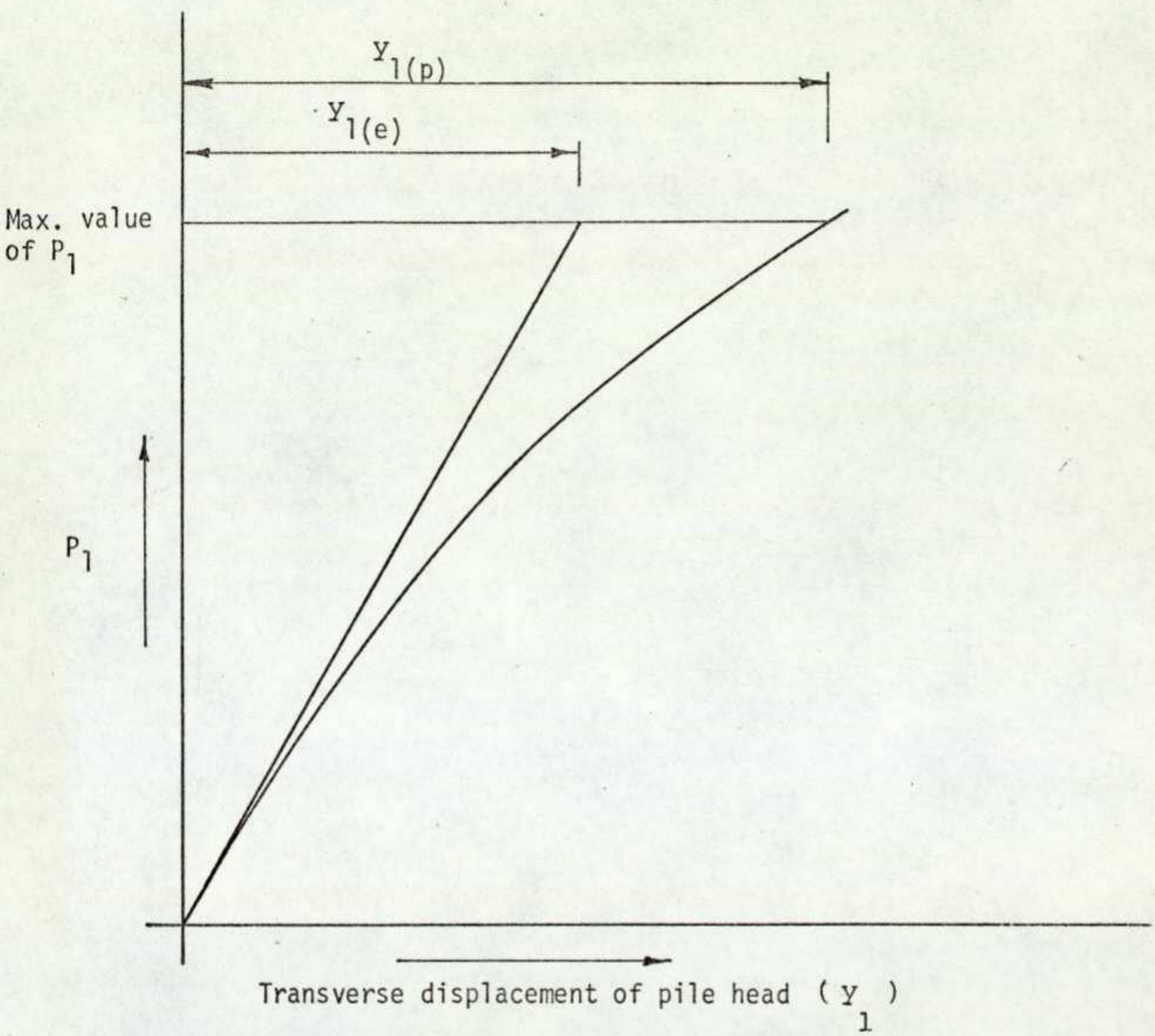


Fig. 3.7 Load Vs Elastic and plastic displacement of pile head in sand

Chapter 4

The analysis and computer programme for the pile group

4.1 Introduction

When several piles are clustered , it is reasonable to expect that the soil pressures developed in the soil as resistance will overlap . With sufficient overlap , either the soil will fail or the pile group will settle excessively .

The variables affecting the ultimate load and displacement performance of pile groups are : depth , spacing , soil properties , placing technique , roughness and shape of the pile surface , group shape and number of piles in the group , and effect of repeated loading .

A designer is interested in the following aspects of the behaviour of a pile group ,

1 - Evaluation of collapse load of such foundations .

2 - Calculation of immediate and long term displacement so that he can select a suitable factor of safety in his design .

3 - The distribution of the loads and moments in piles so that he can provide adequate strength in the piles .

Evaluations of the collapse loads of pile groups are usually of secondary importance because relatively large factors of safety are needed to keep displacements within acceptable limits .

Analytical methods which can deal with the displacements and the load distribution problems are :

1 - Three - dimensional frame analysis .

2 - Modulus of subgrade reaction methods .

3 - Linear elastic methods .

These methods were discussed in section 2.8 above .

The linear elastic method is used in the comparison in section 8.4 .

This chapter presents an analysis based on a modulus of subgrade reaction method . The method of solution is in principle similar to that used by Reese et al (1970) .

4.2 Analysis of pile groups

To formulate a three - dimensional matrix solution for a group of piles , assume that the piles are interconnected at their heads to a rigid pile cap . (this implies that relative movement of the pile cap between adjacent pile - head connections is negligible) .

Consider a pile - cap having six degrees of freedom (translations and rotations) , three displacements in the X and Y directions (horizontally) and in the Z direction (vertically downwards) , and three rotations about X , Y and Z axes .

4.2.1 Linear elastic analysis

The forces and moments applied to a pile group (expressed in the X Y Z system) are related to the pile head forces (expressed in a local axial and transverse coordinate system A T S on the pile head) as follows (see Fig. 4.1) :

$$\{ G \} = \Sigma ([A] \{ F \})$$

where ,

$$\{ G \} = \left\{ \begin{array}{c} P_x \\ Py \\ Pz \\ M_x \\ M_y \\ M_z \end{array} \right\} \begin{array}{l} \left.\begin{array}{l} \\ \\ \\ \end{array}\right\} \text{Forces on the cap in the X , Y and Z directions .} \\ \left.\begin{array}{l} \\ \\ \\ \end{array}\right\} \text{Moments about the X , Y and Z axes .} \end{array}$$

and

$$\{F\} = \begin{Bmatrix} P_A \\ P_T \\ P_S \\ M_A \\ M_T \\ M_S \end{Bmatrix} \quad \begin{array}{l} \text{Axial force on pile head .} \\ \text{Transverse forces in the} \\ \text{pile head .} \\ \\ \text{Moments at the pile head .} \\ \\ \end{array}$$

and $\left[\ A\ \right]$ is a 6×6 transformation matrix for each pile . Consider a pile having a batter of β in a direction α from the X axis (see Fig. 4.1) . An axial load P_A in this pile would apply (among others) a force in the X direction of ,

$$P_A \cos\alpha \ \sin\beta$$

then $$a_{11} = \cos\alpha \ \sin\beta$$

The other terms of matrix $\left[\ A\ \right]$ are shown on the fo~~llowing~~ page~~s~~ 57, (Saul (1968)) .

Similarly , the displacements in the A T S system are related to the displacements in the X Y Z system by the expression

$$\{U\} = \left[A\right]^T \{H\}$$

where ,

$$\{U\} = \begin{Bmatrix} \delta_A \\ \delta_T \\ \delta_S \\ \Theta_A \\ \Theta_T \\ \Theta_S \end{Bmatrix} \quad \begin{array}{l} \text{Displacements of the} \\ \text{pile head .} \\ \\ \text{Rotations of the pile} \\ \text{head .} \\ \\ \end{array}$$

and

$$\{H\} = \left\{ \begin{array}{c} \delta_x \\ \delta_y \\ \delta_z \\ \Theta_x \\ \Theta_y \\ \Theta_z \end{array} \right\} \begin{array}{l} \left.\begin{array}{c} \\ \\ \\ \end{array}\right\} \text{Displacements of the cap .} \\ \left.\begin{array}{c} \\ \\ \\ \end{array}\right\} \text{Rotations of the cap .} \end{array}$$

and $\left[A \right]^T$ is the transpose of matrix $\left[A \right]$.

The pile head forces may be expressed in terms of the pile head displacements in the form

$$\{ F \} = \left[B \right] \{ U \}$$

where , $\left[B \right]$ is a 6 x 6 stiffness matrix for the pile head . Matrix $\left[B \right]$ is shown on the f~~ollowin~~g page~~s~~ 56.

Note that most of the terms in matrix $\left[B \right]$ are zero . For example , b_{11} is the axial force P_A required to give a unit axial displacement δ_A .

Then ,

$$\{ G \} = \Sigma \left(\left[A \right] \left[B \right] \left[A \right]^T \right) \{ H \}$$

$$= \left[C \right] \{ H \}$$

where , C is the stiffness matrix for the whole group .

Then ,

$$\{ H \} = \left[C \right]^{-1} \{ G \}$$

$$= \left[D \right] \{ G \}$$

where , $\left[D \right]$ is the inverse of the stiffness matrix $\left[C \right]$.

Matrix A

	P_A	P_T	P_S	M_A	M_T	M_S
P_x	$\cos\alpha\sin\beta$	$\cos\alpha\cos\beta$	$-\sin\alpha$	0	0	0
P_y	$\sin\alpha\sin\beta$	$\sin\alpha\cos\beta$	$\cos\alpha$	0	0	0
P_z	$\cos\beta$	$-\sin\beta$	0	0	0	0
M_x	$(Y\cos\beta - Z\sin\alpha\sin\beta)$	$-(Y\sin\beta + Z\sin\alpha\cos\beta)$	$-Z\cos\alpha$	$\cos\alpha\sin\beta$	$\cos\alpha\cos\beta$	$-\sin\alpha$
M_y	$(Z\cos\alpha\sin\beta - X\cos\beta)$	$(Z\cos\alpha\cos\beta + X\sin\beta)$	$-Z\sin\beta$	$\sin\alpha\sin\beta$	$\sin\alpha\cos\beta$	$\cos\alpha$
M_z	$(X\sin\alpha\sin\beta - Y\cos\alpha\sin\beta)$	$(X\sin\alpha\cos\beta - Y\cos\alpha\cos\beta)$	$(X\cos\alpha + Y\sin\alpha)$	$\cos\beta$	$-\sin\beta$	0

Matrix B

$$\begin{bmatrix} B_1 & 0 & 0 & 0 & 0 & 0 \\ 0 & B_2 & 0 & 0 & 0 & B_6 \\ 0 & 0 & B_3 & 0 & -B_6 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & -B_6 & 0 & B_4 & 0 \\ 0 & B_6 & 0 & 0 & 0 & B_5 \end{bmatrix}$$

where ,

B_1 is the force in the A direction required for unit displacement in the A direction .

B_2 is the force in the T direction required for unit displacement in the T direction .

B_3 is the force in the S direction required for unit displacement in the S direction .

B_4 is the moment about the T axis required for unit rotation about the T axis .

B_5 is the moment about the S axis required for unit rotation about the S axis .

B_6 is the force in the T direction required for unit rotation about the S axis and is assumed to be equal to the force in the S direction required for unit negative rotation about the T axis .These must also be equal to the moments required for unit displacement in each case .

The moment about the A axis to cause unit rotation of the pile is small compared with the moment required to cause rotation of the whole group , and it has been assumed to be zero .

4.2.2 Procedure for nonlinear analysis

a - For any given set of forces $\{ G \}$ applied to the pile cap, determine the linear elastic displacement $\{ H \}$ of the cap,

$$\{ H \} = \left[D \right] \{ G \}$$

b - For each pile in turn determine the displacements $\{ U \}$ and the corresponding elastic forces $\{ F \}$

$$\{ U \} = \left[A \right]^T \{ H \}$$

$$\{ F \} = \left[B \right] \{ U \}$$

c - Use the procedure of programme PILE to compute modified values of the pile head forces $\{ F' \}$, allowing for the effect of soil plasticity. (all elements of $\{ F' \}$ will be less than or equal to $\{ F \}$)

d - Compute the total force transmitted to the pile cap by the piles.

$$\{ G1 \} = \Sigma \left(\left[A \right] \{ F' \} \right)$$

e - Compute the residual forces

$$\{ RG \} = \{ G \} - \{ G1 \}$$

f - If $\{ RG \}$ is not less than some prescribed limiting value (say $\{ G \} / 100$), reapply $\{ RG \}$ as a new force as in (a) above.

This procedure is shown diagrammatically in Fig. 4.2.

4.3 The computer programme PILEGROUP

The computer programme PILEGROUP (Appendix II) computes the displacements and rotations of the pile group either for first time loading or after cyclic loading .

The pile reactions are assumed to be linear except for displacements in the Υ direction (transverse ~~horizontal~~ direction of loading) . In this direction the reactions are computed using the procedure of programme PILE .

The programme first computes the displacements and rotations of the pile cap under the applied load using trial values of stiffness for δ_x and Θ_y .

The pile head forces and moments for these displacements and rotations are then computed using the method of programme PILE .

The total pile reactions are then compared with the applied load and any residual load is reapplied in the same manner . This iterative procedure continues until the residual force and moment are both sufficiently small (assumed to be when both P_x and M_y are less than 1% of the applied value) .

A flow chart is attached .

4.4 Some practical limitations

The method of analysis employed in programme PILEGROUP cannot take account of the following practical aspects which are known to influence the behaviour of pile groups :

a - The stiffness of the cap

b - The method of construction

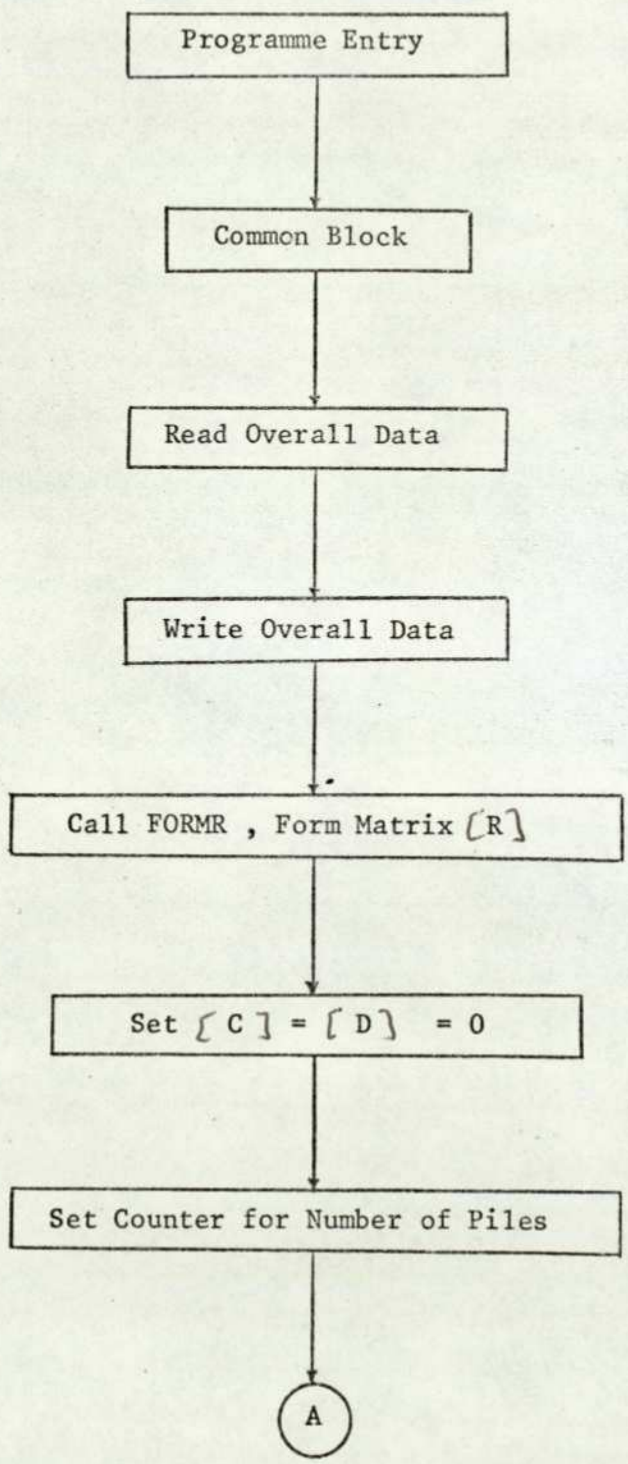
c - The order of driving

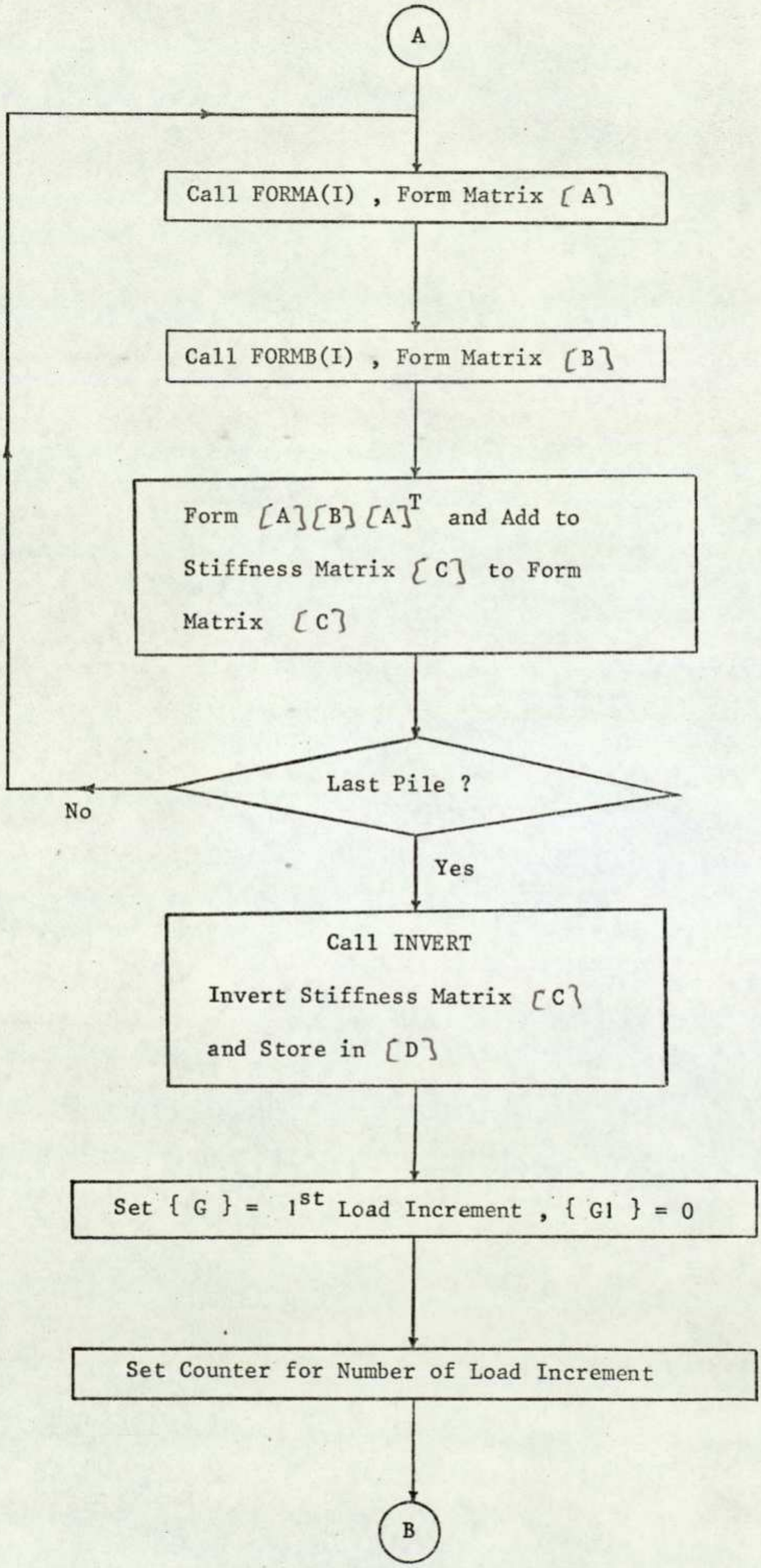
d - The compaction of loose sand and the loosening of dense sand caused by pile driving .

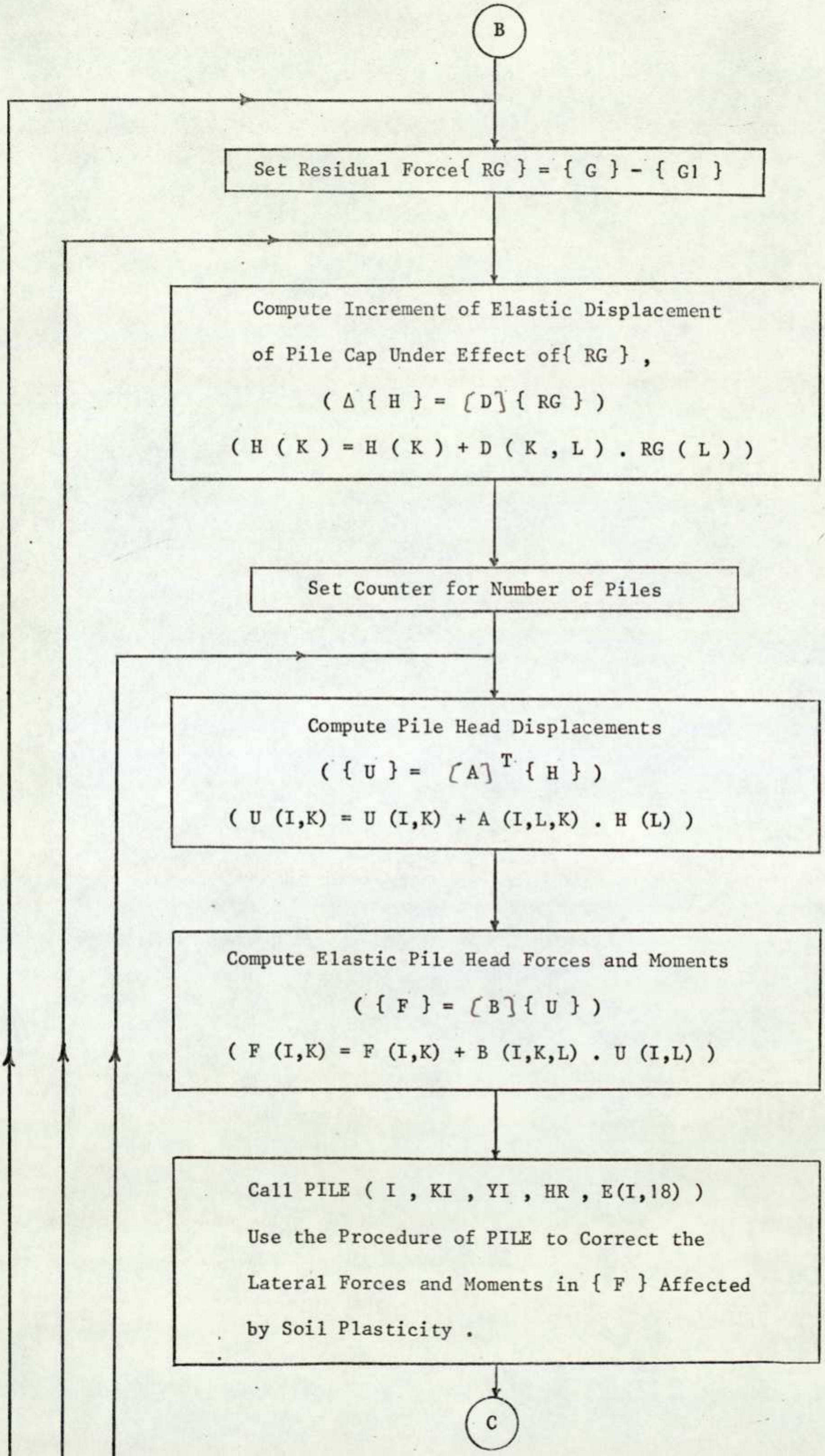
The piles ~~has~~ have been assumed to carry all loads , therefore the programme can be used for pile groups in which the cap rests on relatively soft overlying strata , or , more appropriately , where the cap is not in contact with the ground .

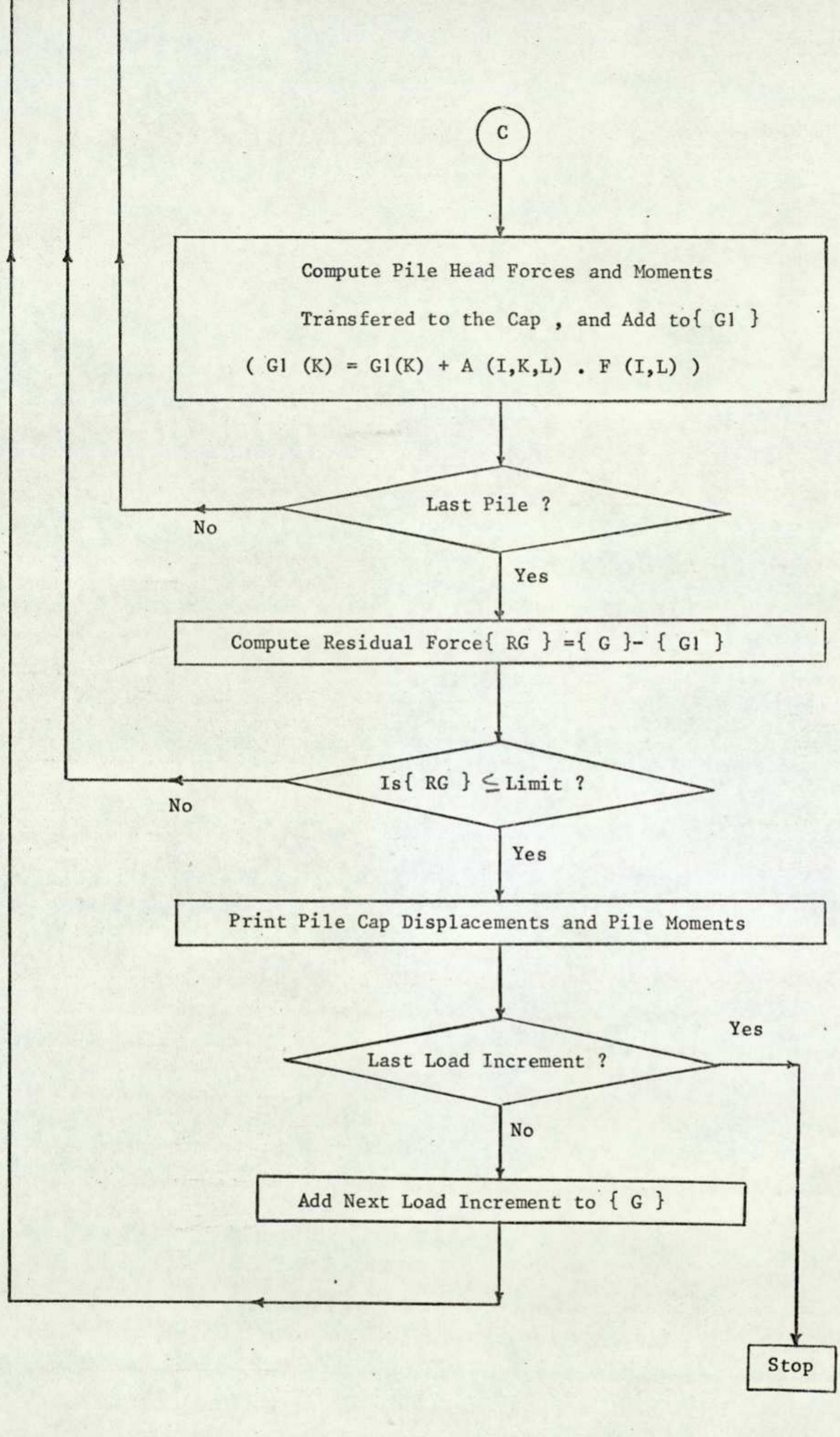
Programme structure for pile group analysis

Programme PILEGROUP




A
Call FORMA(I) , Form Matrix [A]
Call FORMB(I) , Form Matrix [B]
Form [A][B][A]^T and Add to Stiffness Matrix [C] to Form Matrix [C]
Last Pile ?
No
Yes
Call INVERT
Invert Stiffness Matrix [C] and Store in [D]
Set { G } = 1st Load Increment , { G1 } = 0
Set Counter for Number of Load Increment
B


B
Set Residual Force{ RG } = { G } - { G1 }
Compute Increment of Elastic Displacement
of Pile Cap Under Effect of{ RG } ,
(Δ { H } = [D]{ RG })
(H (K) = H (K) + D (K , L) . RG (L))
Set Counter for Number of Piles
Compute Pile Head Displacements
({ U } = [A]^T { H })
(U (I,K) = U (I,K) + A (I,L,K) . H (L))
Compute Elastic Pile Head Forces and Moments
({ F } = [B]{ U })
(F (I,K) = F (I,K) + B (I,K,L) . U (I,L))
Call PILE (I , KI , YI , HR , E(I,18))
Use the Procedure of PILE to Correct the
Lateral Forces and Moments in { F } Affected
by Soil Plasticity .
C


C
Compute Pile Head Forces and Moments
Transfered to the Cap , and Add to{ G1 }
(G1 (K) = G1(K) + A (I,K,L) . F (I,L))
Last Pile ?
No
Yes
Compute Residual Force{ RG } ={ G }- { G1 }
Is{ RG } ≤ Limit ?
No
Yes
Print Pile Cap Displacements and Pile Moments
Last Load Increment ?
Yes
No
Add Next Load Increment to { G }
Stop

4.5 Programme PILEGROUP

Data specification

(All input is in free format)

1 - Heading card (10A8)

variable	entry
TITLE(12)	Enter heading information to be printed with output .

2 - Master control card (5I0 , F 0.0)

variable	entry
NP	Number of piles
NL	Number of load stages
NM	Number of parts (m) per pile
NI	Number of iterations in subroutine PILE
	EQ. 1 for linear analysis
	GE. 2 for non - linear analysis
LIST	Output control parameter
	EQ. 0 only maximum +ve and -ve moments are printed
	EQ. 1 moment and displacement at each node is printed
Z	Depth of ground surface below pile heads .

3 - Pile data - Two cards per pile

1^{st} card (8F0.0)

variable	entry	
E(I,1)	Pile head coordinates	X
E(I,2)		Y
E(I,3)		Z

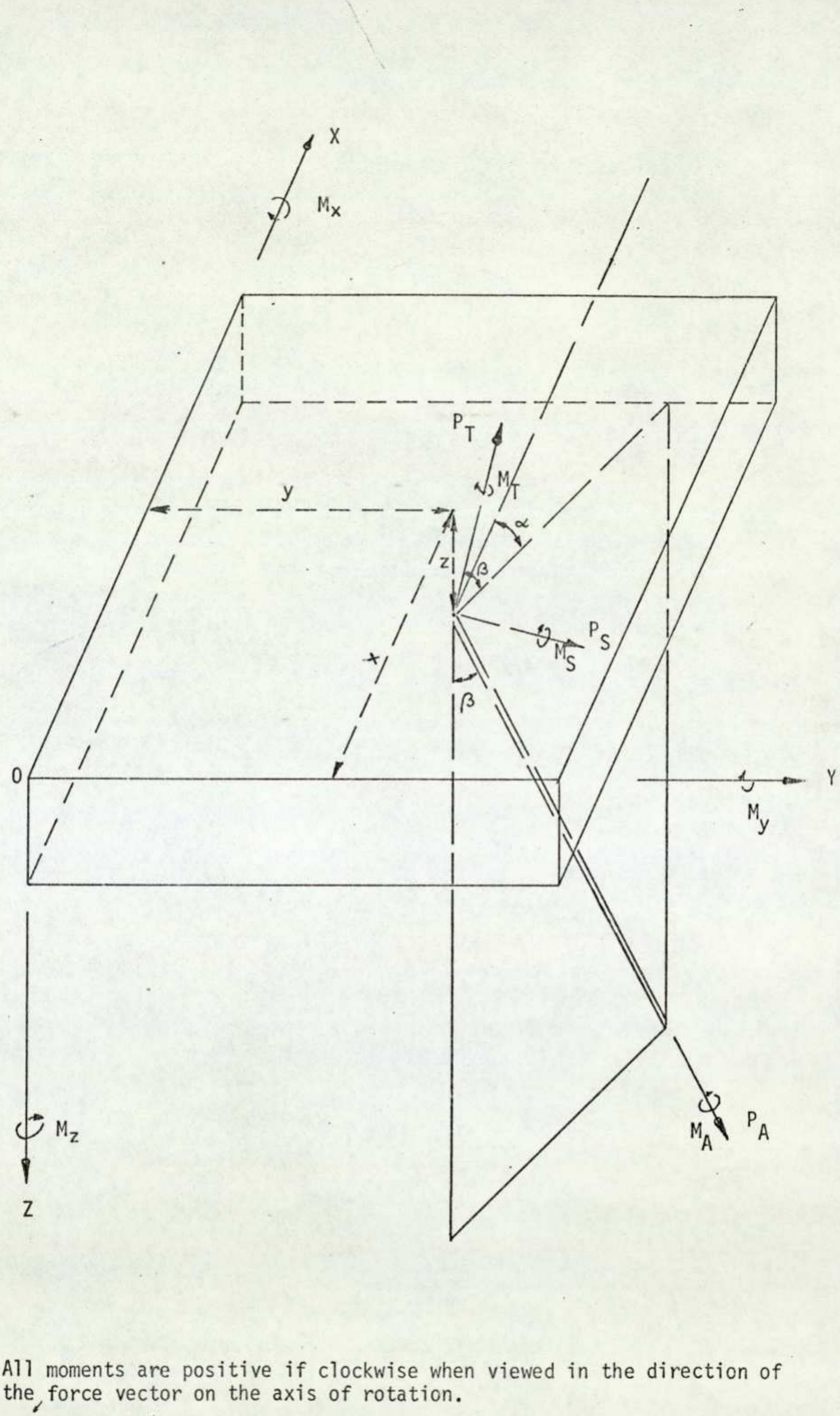
E(I,4)	Pile direction angle α^o
E(I,5)	Pile batter angle β^o
E(I,6)	Pile length
E(I,7)	Pile breadth
E(I,8)	Pile stiffness (E I)

2^{nd} card (8F0.0)

variable	entry	
E(I,10)	Linear soil constant (elastic) a_m	
E(I,11)	Non - linear soil constant (plastic) p_u	
E(I,12)	Linear stiffness coefficients	B_1
E(I,13)		B_2
E(I,14)		B_3
E(I,15)		B_4
E(I,16)		B_5
E(I,17)		B_6

4 - Load data - One card per load stage (6F0.0)

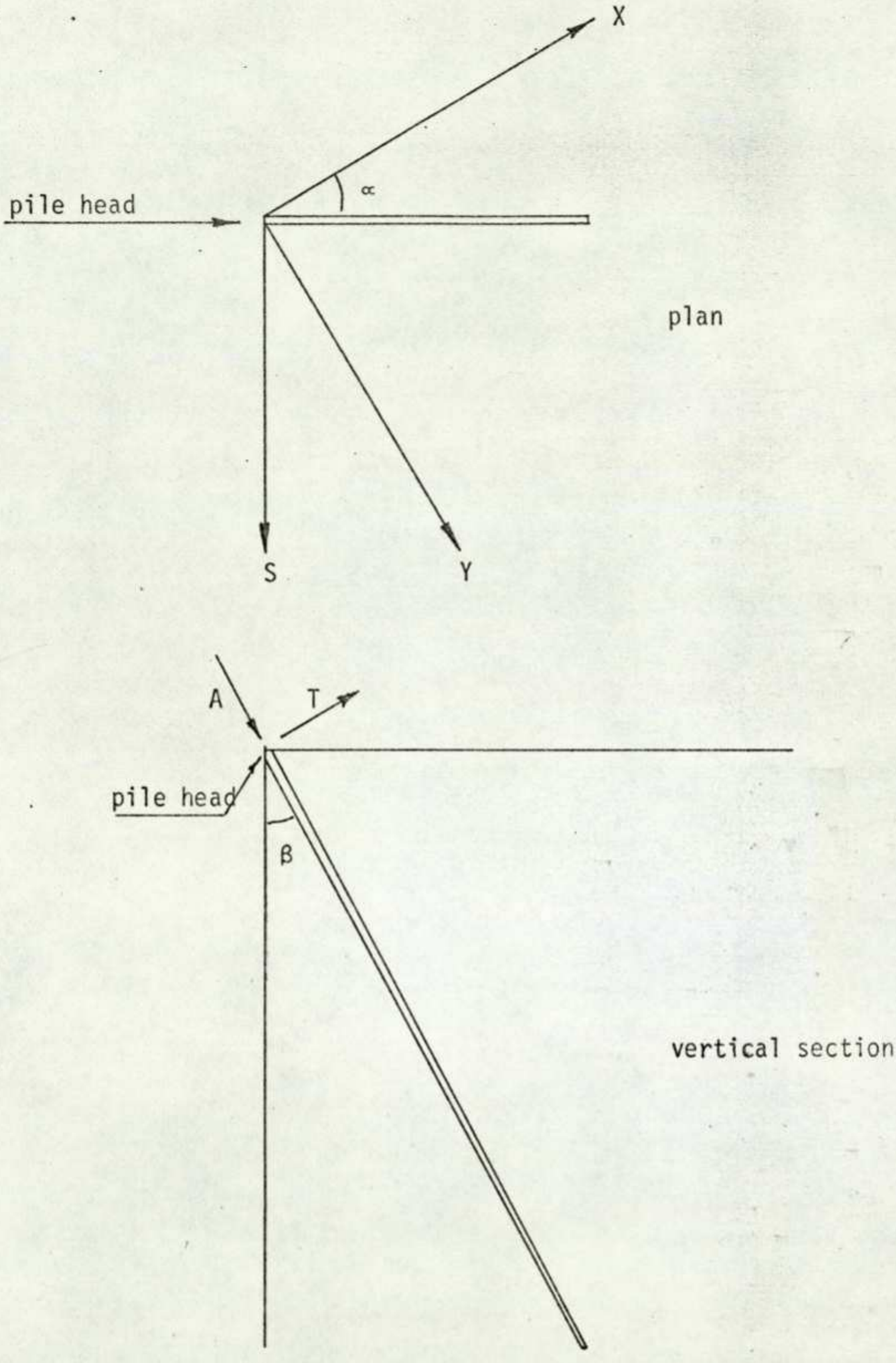
variable	entry	
G(I,1)	Applied forces	P_x
G(I,2)		P_y
G(I,3)		P_z
G(I,4)	Applied Moments	M_x
G(I,5)		M_y
G(I,6)		M_z



All moments are positive if clockwise when viewed in the direction of the force vector on the axis of rotation.

+ve

Fig. 4.1 Pile Group with foundation forces and individual pile forces



Relationship between XYZ and ATS systems of co-ordinates.

Fig. 4.1. A

Method of obtaining non-linear relationship between {G} and {H}

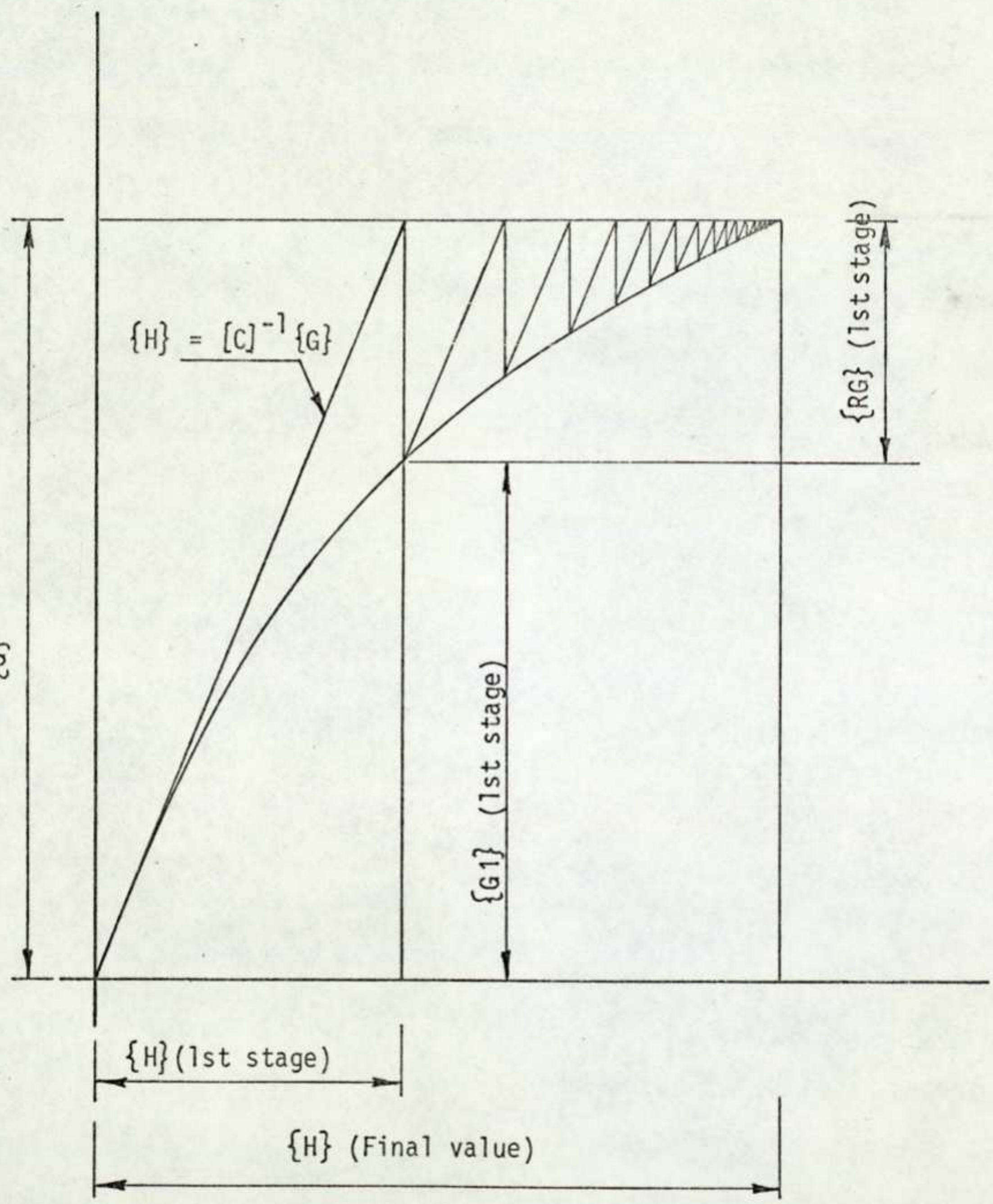


Fig. 4.2

Chapter 5

Design of test apparatus and preparation of the sand beds

5.1 Introduction

Properly conducted full - scale tests in the field will generally provide the most reliable experimental evidence of pile group behaviour . However the scope of such tests is usually restricted by the high cost and time required . Moreover , it is often difficult to control all the variables affecting the pile group behaviour , so that the relevance of the test results to groups under different conditions is often difficult to establish .

Significant quantitative results may be obtained from model studies , provided that the requirements for similarity are adequately accounted for . Useful results are generally more easily obtained from model studies of problems where , as in the present case , reasonable theoretical solutions may be developed .

The number of variables affecting the behaviour of a pile groups is large , and it was therefore necessary to limit those which could be examined . The present study is primarily concerned with the effect of varying the batter angle in groups of different configuration . The apparatus and test programme have therefore been designed to maintain the following constant so far as is possible :

a - Soil type and density of packing .

b - Pile diameter and stiffness .

c - Ratio of embedded length to pile diameter .

d - Pile spacing at cap level .

e - Pile cap stiffness .

f - Method of installation of the piles and pile caps .

This chapter presents the design of the apparatus , and preparation of the sand beds , with these ojects in view .

5.2 Choice of soil type

The overriding consideration here is that the granular soil chosen should be easily compacted to a standard density . For this reason a washed Leighton Buzzard sand , screened nominally between 0.4 mm and 1.00 mm , was used . Figure 5.1 shows a grading curve of this material , as received . Two further sieve analyses were carried out on this material after use , but no significant difference in the grading was observed although there was a very small increase in the finest fractions (see table 5.1) . The maximum and minimum densities of this sand were found to be 1.780 Mg/m^3 and 1.534 Mg/m^3 respectively , and the specific gravity G_s = 2.65 . After placing in the tank by the method described below , the mean density was found to be 1.732 Mg/m^3 representing a Relative Density (RD) of 0.82 .

The particles of this sand are vary round in shape , and in this respect this is a somewhat unnatural material . However , very satisfactory uniformity of density was obtained , and the particles , although fairly large , are sufficiently small in relation to the pile diameter for the soil to be considered as a continuum .

5.3 The sand tank

All the model tests were carried out in a sand placed in a steel tank 1.82 m square internally and 1.37 m deep . A detachable stiff steel frame or " upper structure " was bolted to the top of the tank to carry a double acting hydraulic jack for driving the piles and also to provide vertical load to the pile groups under test . The design allowed for the jack to be moved in three dimensions and to be tilted at any angle to the vertical (see Fig. 5.2 and 5.3 , and Plate 6) .

A concrete bin was provided , capable of holding the whole charge of tank , and a hopper (described below) was used to transfer the sand between bin and tank .

Two trap doors were provided in the side of the tank to facilitate emptying . These doors could be replaced by plate glass screens to allow visual inspection and photography of the sand during placing .

5.4 The hopper

A steel hopper was constructed to transfer the sand between the bin and the tank . It would obviously have been desirable for the hopper to have been large enough to carry the whole charge of the tank , but the limited headroom in the laboratory and the limited capacity of the crane available (2 tonnes) required the charge to be transfered in several stages .

The hopper was the same size as the tank in plan and 450 mm deep . The bottom was formed of two steel plates , both of which were perforated with holes 12.75 mm in diameter at 38 mm centres in both directions . The lower plate could be moved relative to the upper by means of two hand - operated screw jacks fixed to the side of the hopper , thus opening or closing the holes . Thus the sand could be discharged into the tank as a steady rain evenly spread over the plan area . (Plates 6 and 8)

5.5 The sand diffuser

It had been hoped that the sand grains , in passing through the holes in the hopper bottom , would have aquired sufficient horizontal velocity to ensure a uniform spread , provided that the hopper was kept at a sufficient height above the sand surface in the tank .

However , visual inspection and photographs taken through the side of the tank (Plates 8 and 9) showed that this was not happening and that the sand grains were falling nearly vertically from each hole . On landing , the grains appeared to rebound and scatter , and at the end of each charge there was no surface indication that the particles were not uniformly distributed . However it seemed likely that the constant impact of grains falling the full height would result in a column of more densely compacted sand immediately under each hole . A timber diffuser was therefore constructed to scatter the sand grains immediately after they passed through the hopper bottom .

This diffuser consisted of four layers of hardwood dowels 19 mm in diameter spaced at 38 mm cetre to centre in each layer . The layers were placed alternately in a longitudinal and transvers direction and the lower layer of longitudinal dowels was placed immediately below the spaces in the upper layer (see Fig . 5.4) .

Thus there was no direct passage through the diffuser and each grain on rebounding from the surface of a dowel would aquire a horizontal velocity sufficient to scatter it .

Subsequent visual inspection showed that the scattering appeared to be comlete and the distribution of the sand grains on first impact appeared to be uniform .

5.6 Formation of the sand beds

Seven charges of the hopper were required to fill the tank (see Fig . 5.5) . At each stage the height of the hopper and diffuser was adjusted to give , as far as possible , the same height above the sand surface in the tank . The procedure for each stage was as follows :

a - A plywood skirt of appropriate height was attached to the top of the tank .

i - to retain the sand in the tank , and

ii - to support the diffuser .

b - Four vertical steel bars of appropriate length were bolted to the top corners of the tank to locate the hopper at the correct height .

c - The hopper was closed and filled from the bin to a depth of 200 mm , the surface being carefully levelled .

d - The hopper was placed on the locating bars and the bottom was opened , discharging the sand through the diffuser into the tank .

At each stage , containers were placed in the tank to measure the sand density . After each stage the hopper was rotated through 180° to eliminate the small effects of any misalignment of the holes in the two bottom plates . After the final (seventh) stage the sand was everywhere slightly higher than the required level in the tank . The surplus was carefully removed by hand with the minimum disturbance using a sharp edged steel plate and a scoop .

5.7 Uniformity of the sand beds

The uniformity of the sand beds was checked by two methods :

a - By measuring the dry density of the sand in the tank .

The dry density was measured by placing cylindrical containers (98.5 mm in diameter and 127.5 mm deep internally) inside the tank before each hopper discharge . The containers were placed at positions marked A and B in Table 5.2 so as not to interfere with the driving of the piles . However , there is no reason to suppose that the measured densities are not typical of the whole tank . The measured dry density increased very slightly with depth (due to increased overburden pressure) , with an average of 1.73 Mg/m^3 for all the tests carried out . Details are given in Table 5.2 . The maximum difference in the mean dry densities from one bed to another was less than 2% which would have no significant effect on the behaviour of the piles driven into it .

b - <u>By coparing the horizontal displacements of single vertical piles under horizontal load .</u>

Four tests (6A , 5A , 13A , 13B) were made on single vertical piles and the measured displacements of the pile head are shown in Table 5.3 . Tests 6A and 13A were in the centre of the tank but in different beds and the displacement are nearly identical . Tests 13A and 13B were made in the same bed , but 5A and 13B (in different beds) were within 400 mm of the tank side . This boundary effect probably accounts for the slightly increased soil stiffness in these cases . No other test was carried out so close to the wall .

5.8 Advantages of the technique used in forming the sand bed

The following are the main advantages of the technique used in forming the sand bed :

a - The rate of deposition of the sand can be controlled by controlling the size of the holes in the hopper base .

b - By keeping the height of the hopper above the sand bed nearly constant , the mean velocity of the sand particles on landing is nearly the same at all stages of loading the tank .

c - Since the holes in the hopper are all opened to the same extent , the sand is deposited uniformly and to the same level over the whole plan area of the tank .

5.9 Model piles - dimensions and materials

Three preliminary tests on instrumented single vertical piles were carried out in sand as follows :

Test	Material	Outside diameter mm	Wall thickness mm	Total length mm	Embedded length mm
A	Aluminium tube	25.4	1.26 (Gauge 18)	900	750
B	Brass tube	25.4	1.626 (gauge 16)	900	750
C	Brass tube	25.4	1.26 (Gauge 18)	900	750

From the load - displacement behaviour (see Fig . 5.6) and the bending moment behaviour , the dimensions and material of the third test (C) was chosen for the model piles .

So , the model piles were made of Brass alloy tubing , 25.4 mm outside diameter and 1.26 mm wall thickness . The overall length of the pile was 900 mm of which 750 mm was embedded in the sand .

The mean flexural stiffness (EI) was found to be 0.8×10^9 N.mm^2 .

The model piles were instrumented with electrical resistance strain gauges at different depths .

5.10 Instrumentation of the model piles

The piles were instrumented with 120 ohm electrical resistance foil strain gauges mounted as follows :

a - Fourteen gauges were mounted in pairs on the inside of the embedded portion of each pile , to measure bending strains . Each pair of gauges was mounted on opposite sides of the pile in the direction of horizontal loading and at the depths shown in Fig . 5.7 .

b - One pair of gauges was mounted transversely to the direction of horizontal loading , inside and near the top of the pile , to measure the axial load .

The technique developed to mount the gauges was as follows :

a - The lead wires were soldered to the wire connections of each gauge using the minimum of heat and a suitable heat sink . The connections and the back of the gauge were sealed with I S 12 adhesive , and the connections were insulated with rubber sleeves . Continuity of the gauges was checked at this stage .

b - The inside of the brass tube was thoroughly cleaned .

c - Seven gauges were attached with double sided " Sellotape " to small pads of plastic foam and these were in turn secured at the required spacing to a wooden dowel 10 mm in diameter and 1.1 m long . Care was taken to ensure that the gauges were all in the same longitudinal alignment on the dowel .

d - The face of each gauge was coated with a thin layer of Araldite adhesive .

e - The wooden dowel , with the strain gauges attached was carefully inserted in the brass tube . The tube was then pressed firmly onto the wooden dowel , the ends of the dowel were placed on a pair of horizontal supports , and a load of 5 Kg was placed at the centre of the tube to keep it firmly in contact with the dowel .

f - After 24 hours , the load was removed and the dowel was sharply twisted to remove the plastic pads from the Sellotape , leaving the gauges adhering to the inside of the tube .

g - The same procedure was used to mount the seven gauges on the opposite side of the tube , and the additional pair of gauges near the top (for measuring axial load) .

h - The leads from the gauges were brought out through holes cut near the top of the tube , and were attached to lengths of miniature tag strip secured to the top , and thence to the Modulog Data Logger .

Temperature compensation for each pair was provided by two dummy gauges attached to an unloaded length of brass tube embedded in the sand in the tank .

After mounting , the gauges were tested for continuity , insulation and bond . No discontinuity or electrical leakage was detected in any of the circuits . Bond was checked by placing the ends of the pile horizontally on a pair of knife - edges , and loading it at the centre . In every case the sensitivity , linearity and stability of the response was found to be good .

A total of fourteen piles were instrumented in this way .

This method of instrumentation has the following advantages :

a - The piles were made from one continuous length of drawn tube .

b - The wall could be thin enough for reliable sensitivity , and yet the pile diameter was large enough for the gauges to be mounted inside .

c - The flexural stiffness EI of the pile was constant throughout the length .

d - There was no loss of effective pile surface area or difference in surface roughness , as would have been the case if the gauges had been mounted on the outside .

e - The gauges and wires were completely protected against both mechanical damage and humidity .

5.11 Data logging equipment

Strain measurements were made using a " Modulog " automatic data logger (Plate 1 and 2) , the essential features of which were :

a - A constant d.c. voltage supply for energising the strain gauge bridges .

b - A commutating unit to select the channels to be read .

c - Trim potentiometers for the initial balancing of each channel .

d - A digital voltmeter to record the output of the selected bridge , and to provide a visual display .

e - A printer to provide an immediate hard copy of the digital voltmeter reading .

The instrument has a maximum sensitivity of 0.1 microstrain and a maximum accuracy under the present conditions of about 0.3 microstrain .

A full bridge circuit was provided for each pair of gauges (see Fig . 5.8) , the four arms of the bridge being the two active gauges and two dummy gauges for temperature compensation mounted on an unloaded length of brass tube embedded in the sand in the tank .

5.12 Model pile caps

The pile caps used throughout the test programme were free standing - that is , there was no contact between the cap and the soil to avoid uncertainties about the contact pressure of the cap on the soil .

The cap was required :

a - To secure the pile head with the minimum relative rotation between the cap and the pile .

b - To have a flexural stiffness much greater than that of the piles .

c - To be capable of being fitted after the piles had been driven without imposing lateral forces or moments on the piles .

d - To be easily removed after testing without risk of damaging the piles .

e - To provide guides for driving the piles .

Two types of cap were used in the present study :

a - Bolted mild steel caps .

b - Reinforced plaster caps .

The steel caps were used as guides when driving the piles and for tests where no vertical loads were applied . Details of these caps are shown in Figs . 5.9 to 5.12 and Plates 3 and 4 .

The plaster caps were used for tests on pile groups where vertical loads were applied . The caps were 76 mm thick and 228 mm by 355 mm in plan . They were made of " Kaffa D " hard dental plaster having the following properties :

Water content	100 parts plaster 30 parts water	by weight
Initial set approximately		10 minutes
Compressive strength	at 2 hours	23.43 N/mm^2
	when dry	51.69 N/mm^2

5.13 Loading system

Vertical loads were applied to the models using the hydraulic jack and a proving ring to record the load applied .

Horizontal loads were applied by attaching weights to a flexible cable passing over a pulley and attached to the pile cap .

Both vertical and horizontal loads were applied on the centre line of the cap (see Plate 7) .

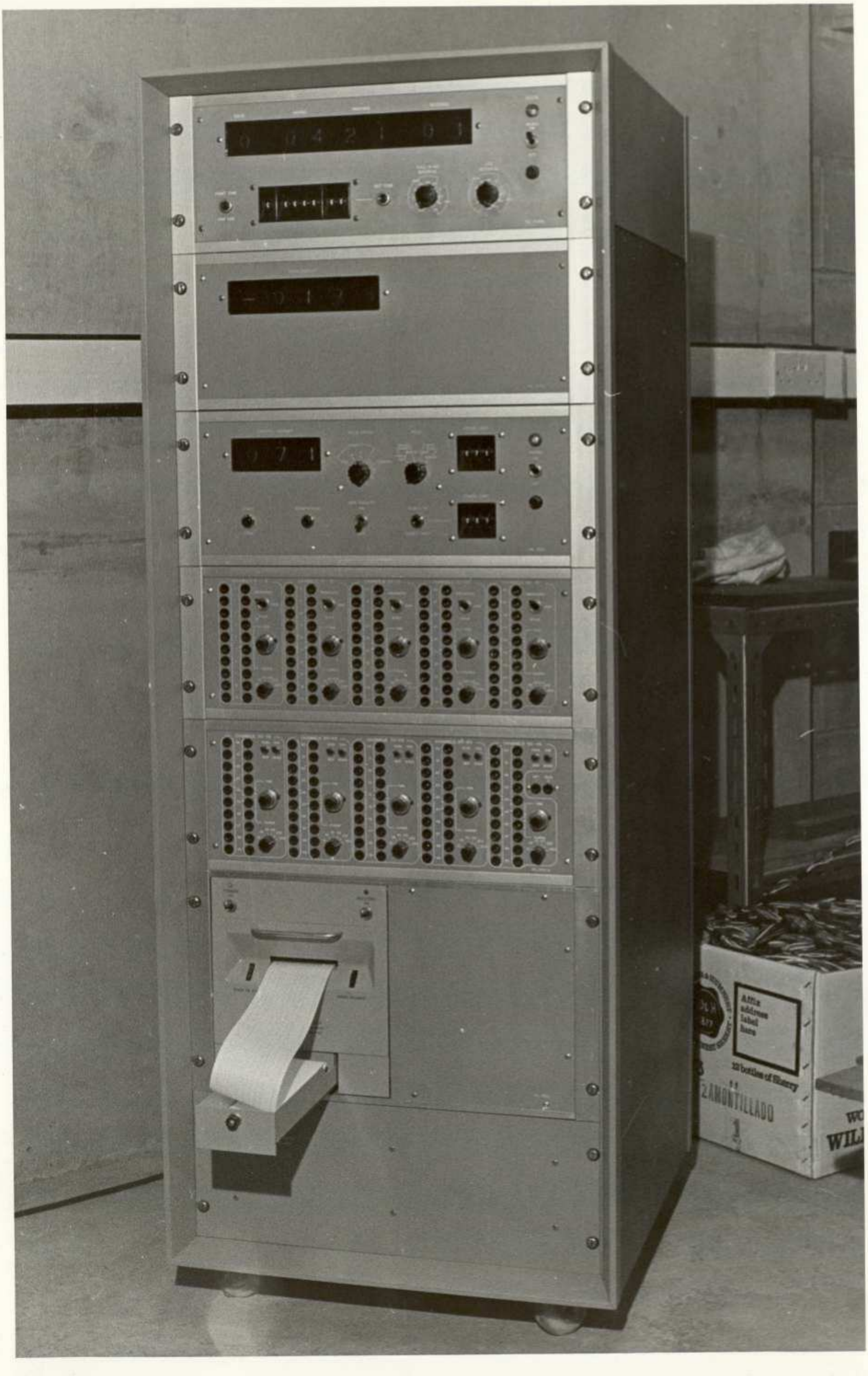


PLATE 1

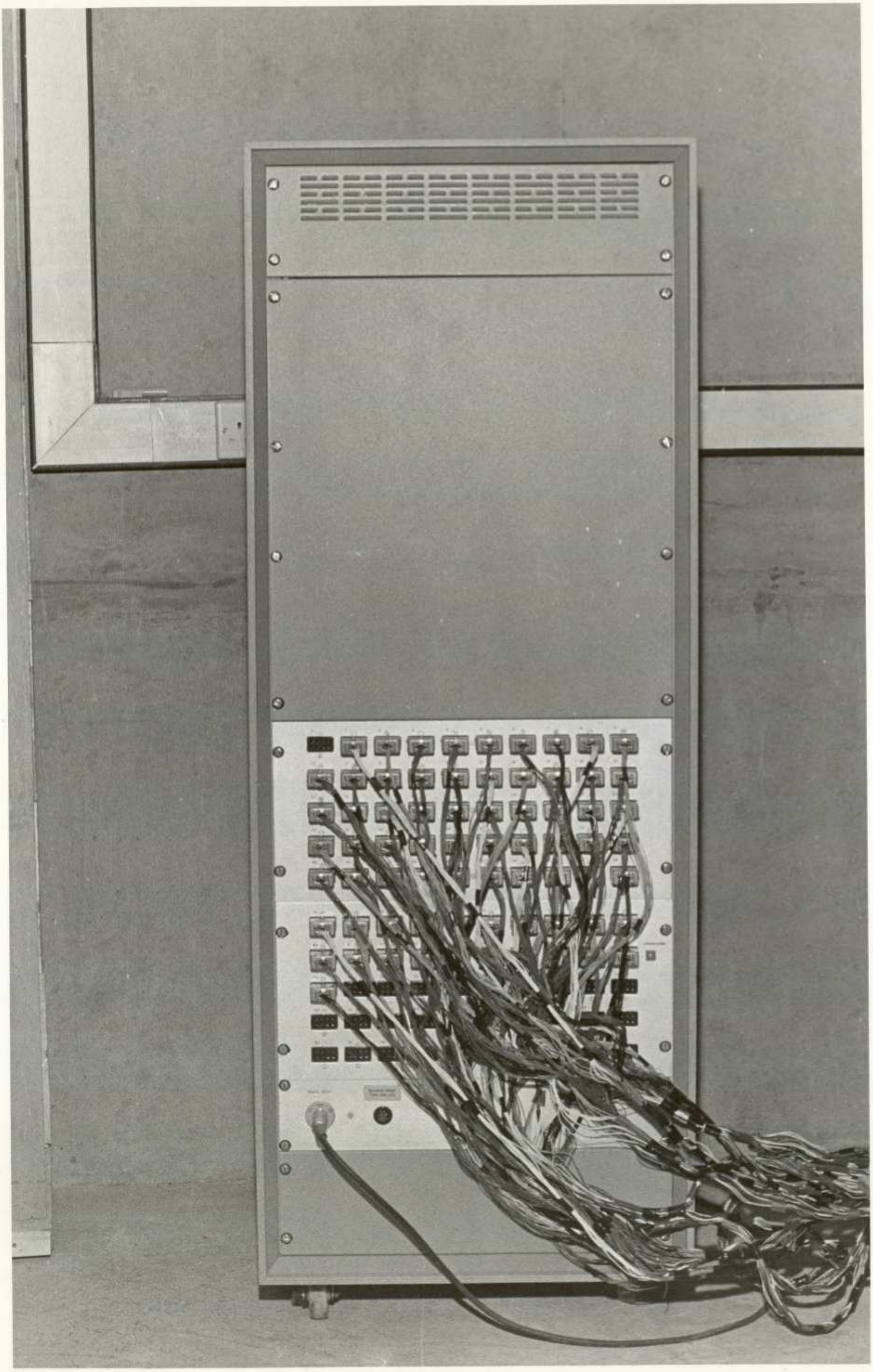
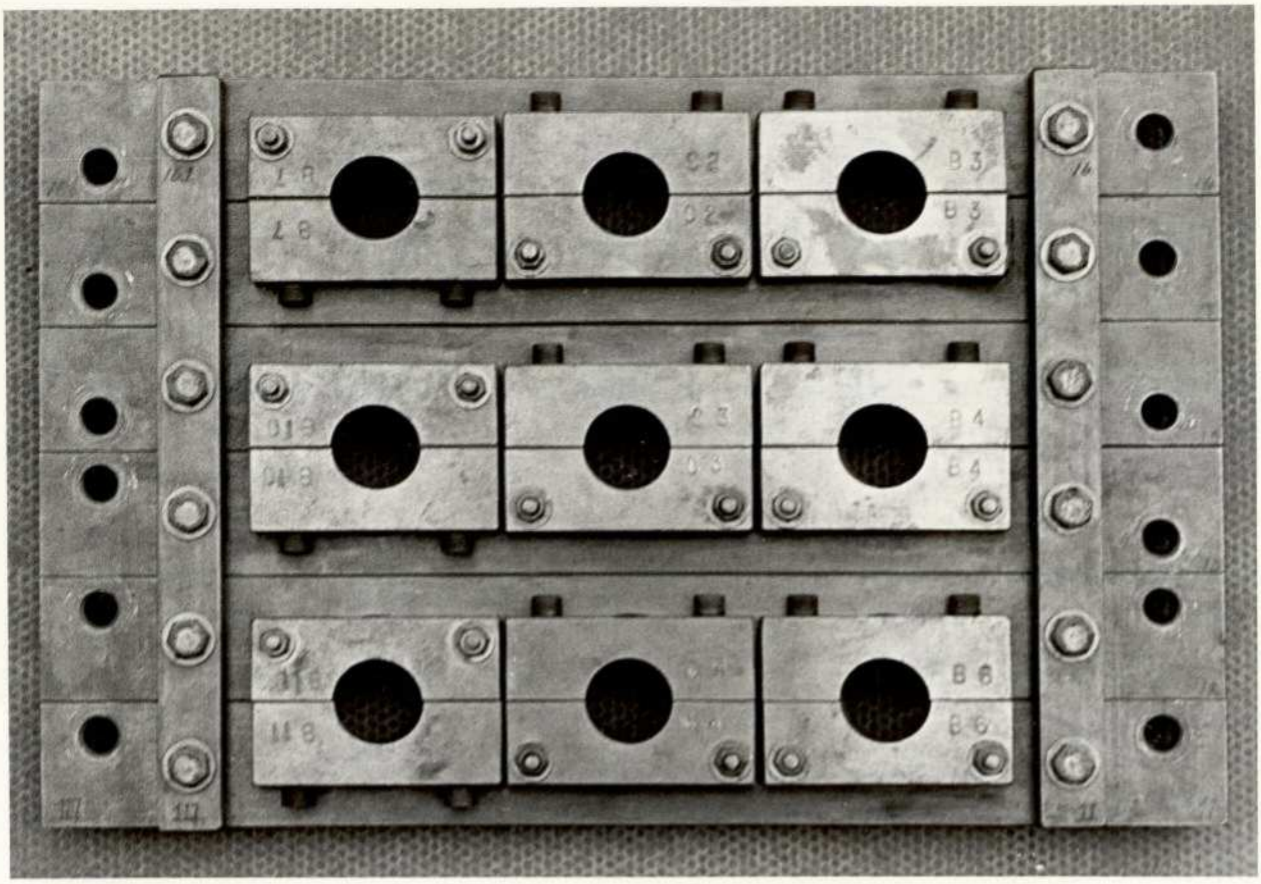


PLATE 2



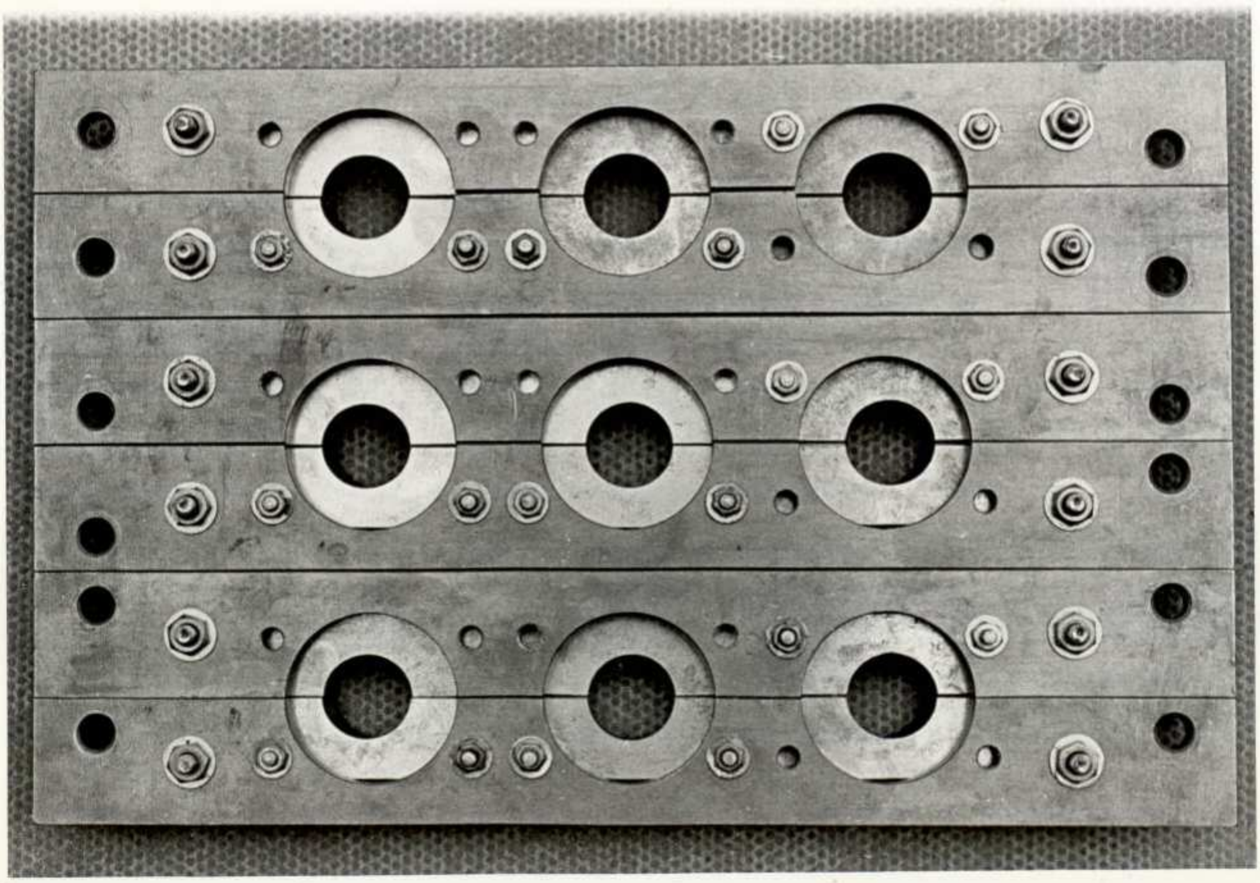


PLATE 3

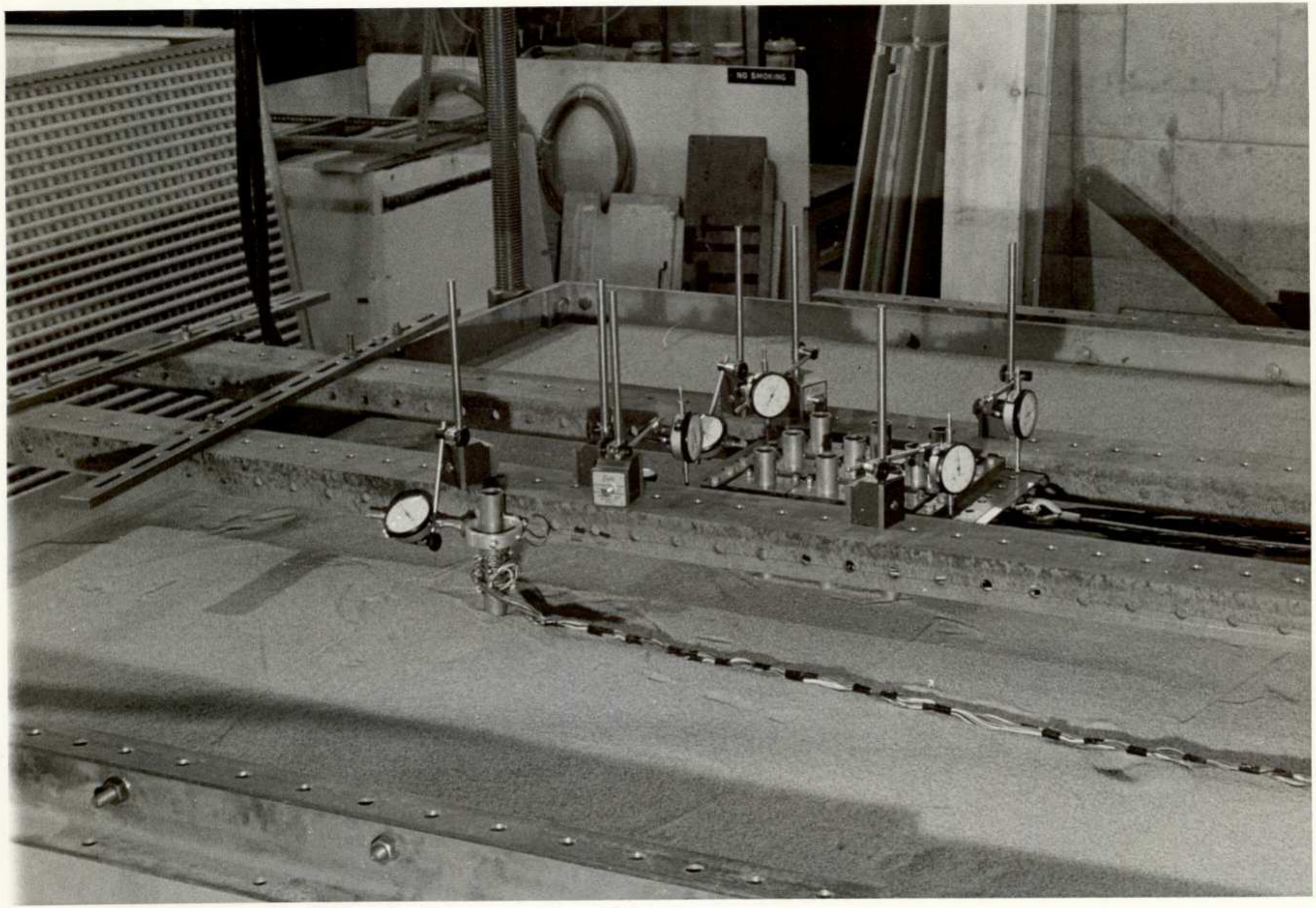


PLATE 4

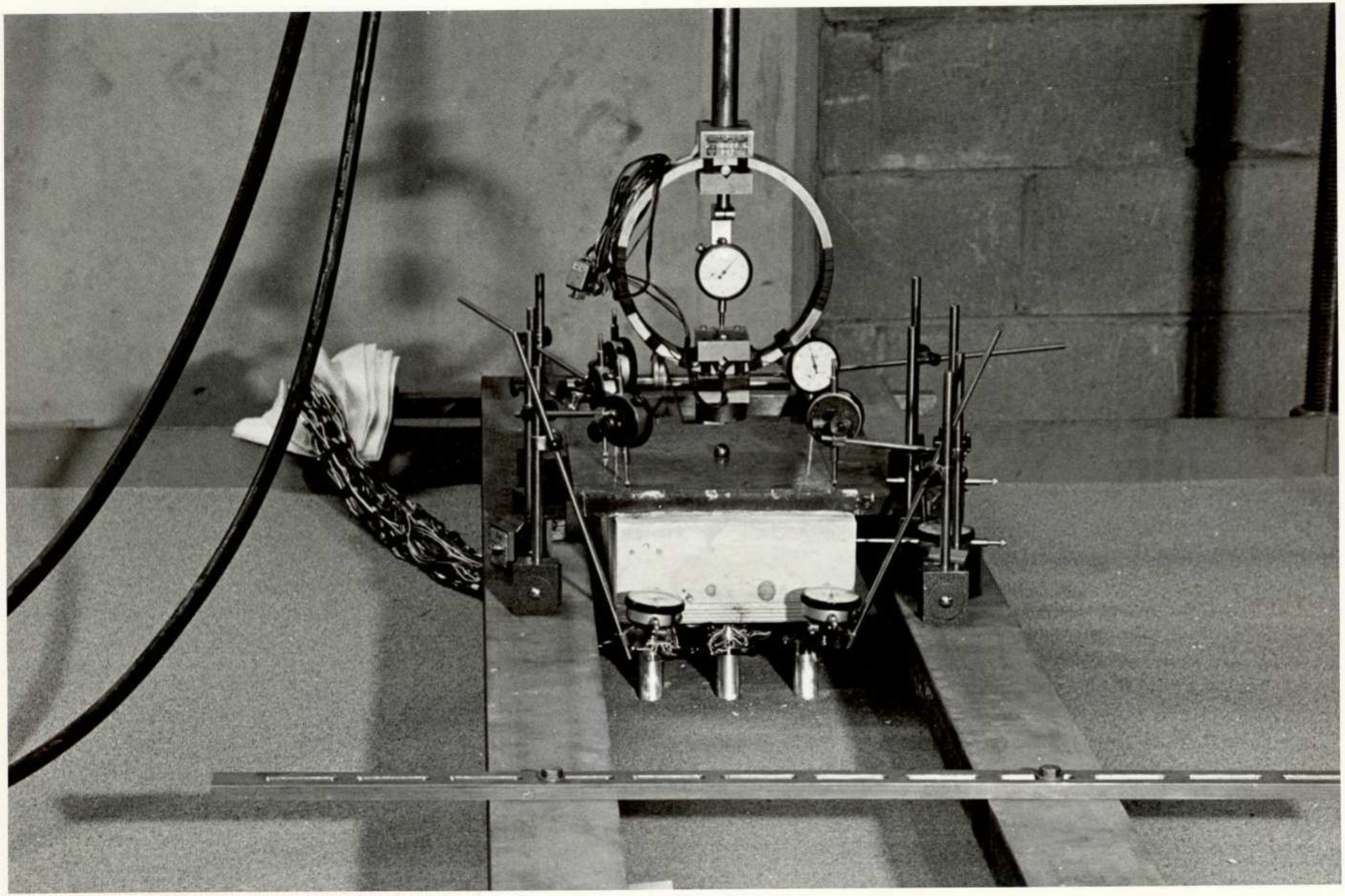


PLATE 5

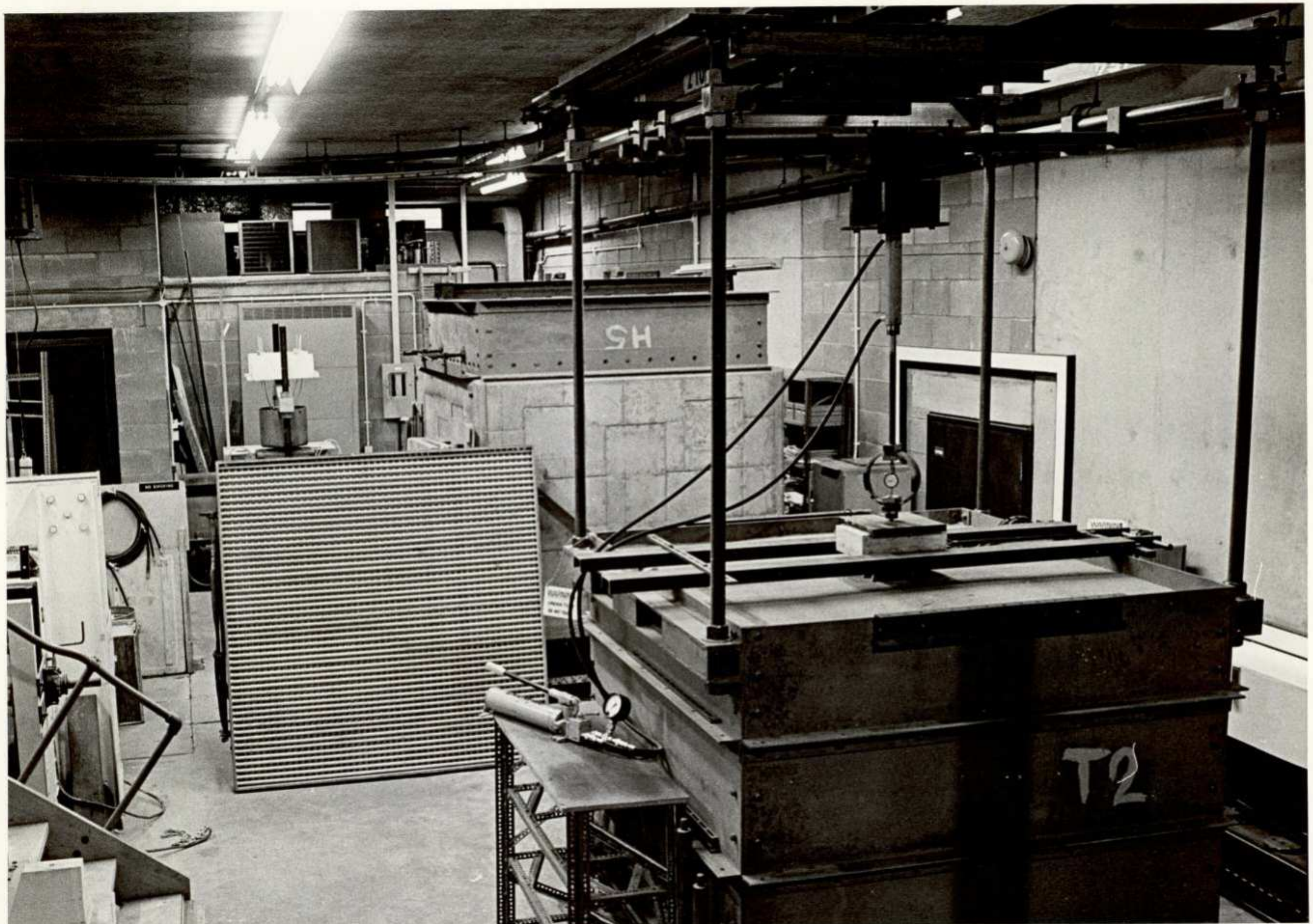


PLATE 6

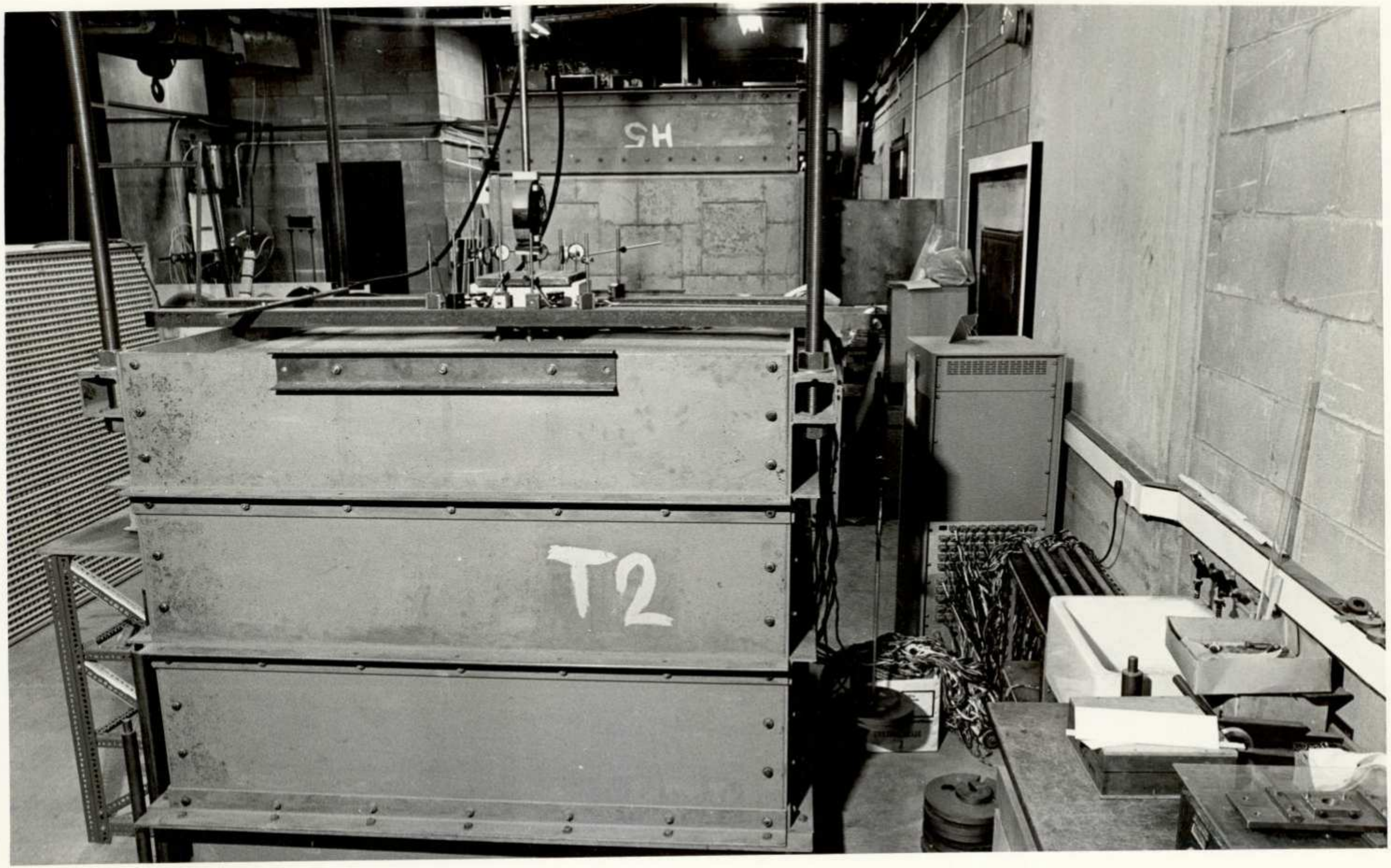


PLATE 7

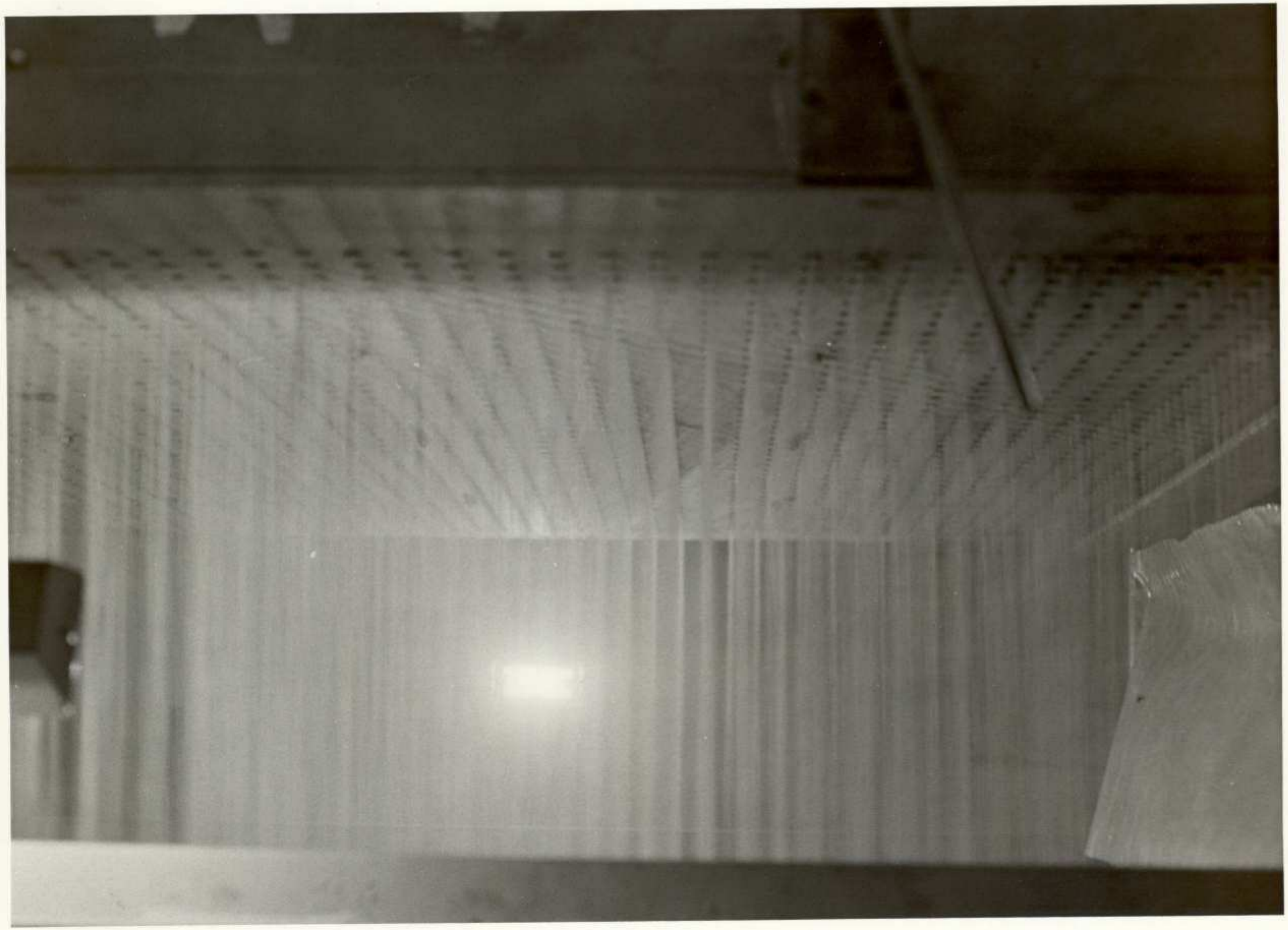


PLATE 8

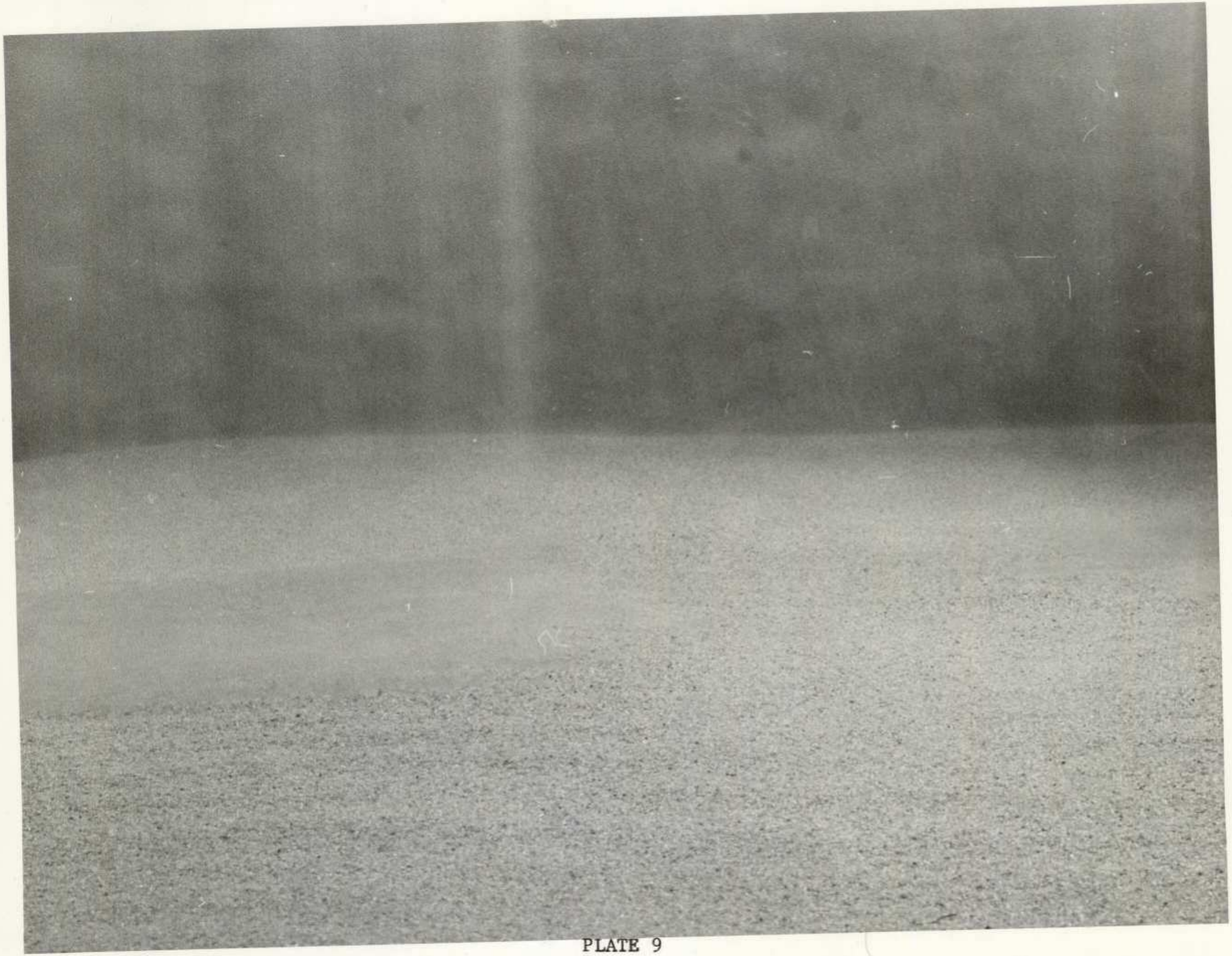


PLATE 9

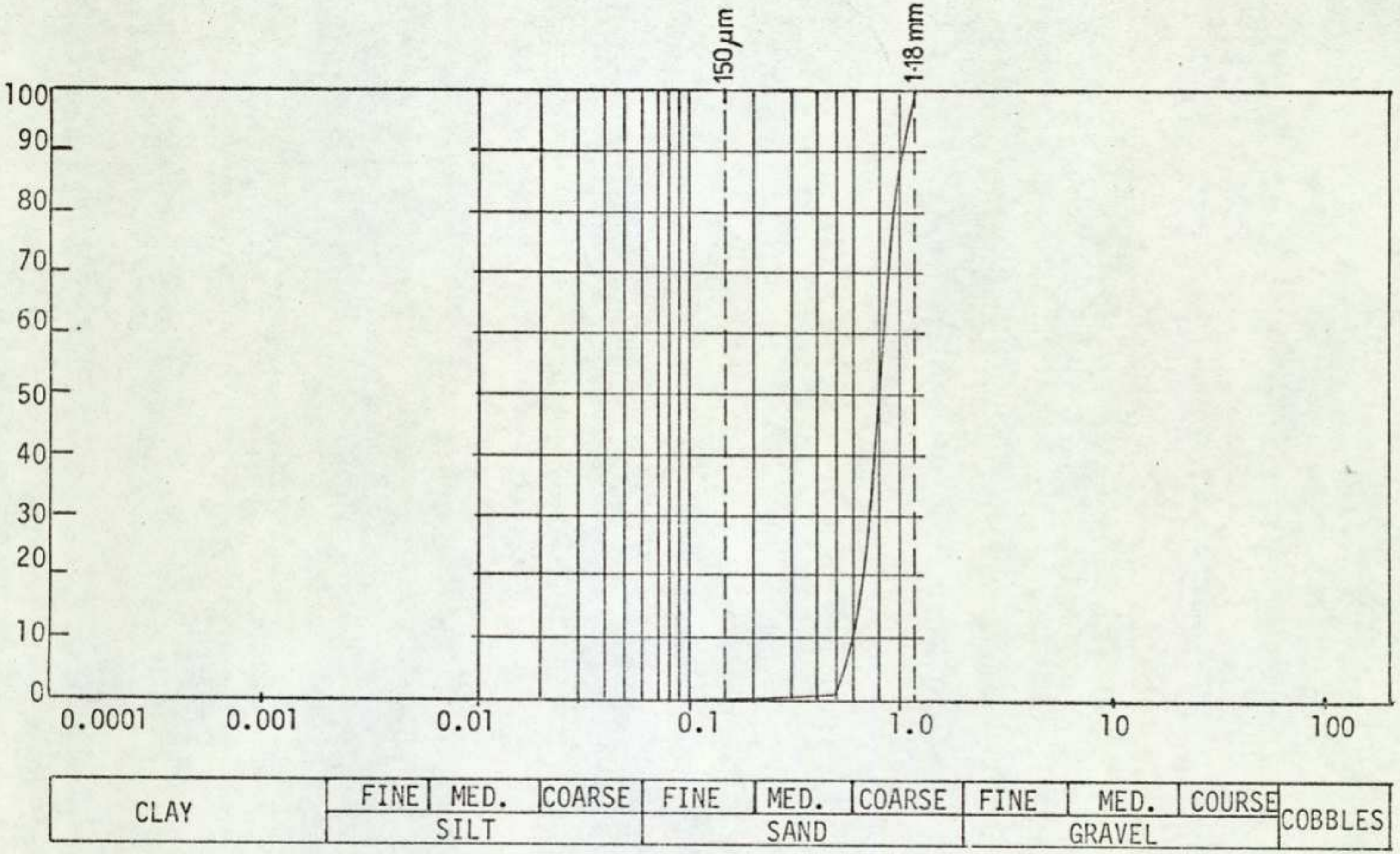


Fig. 5.1 British Standard test sieves

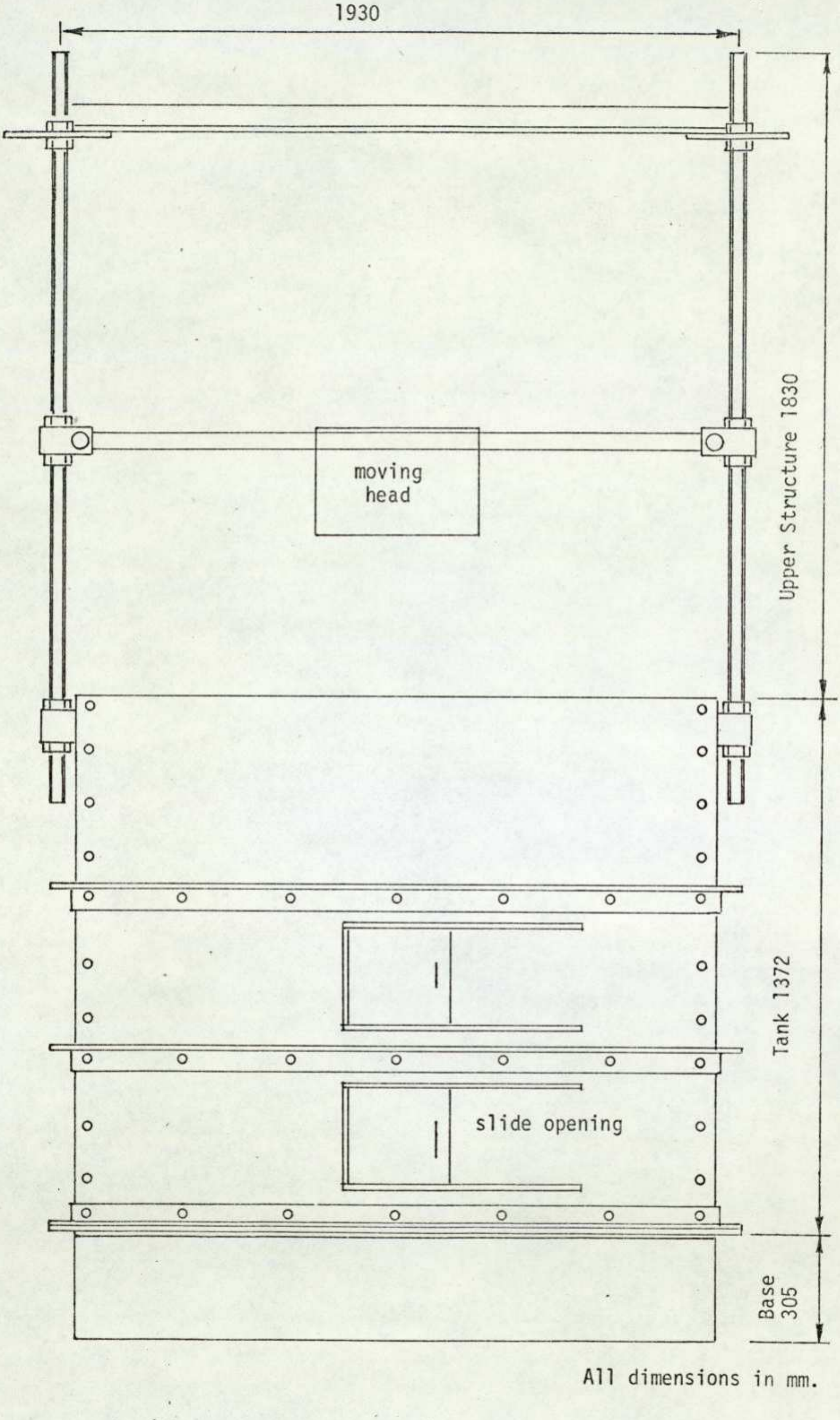


Fig. 5.2 Elevation of Apparatus

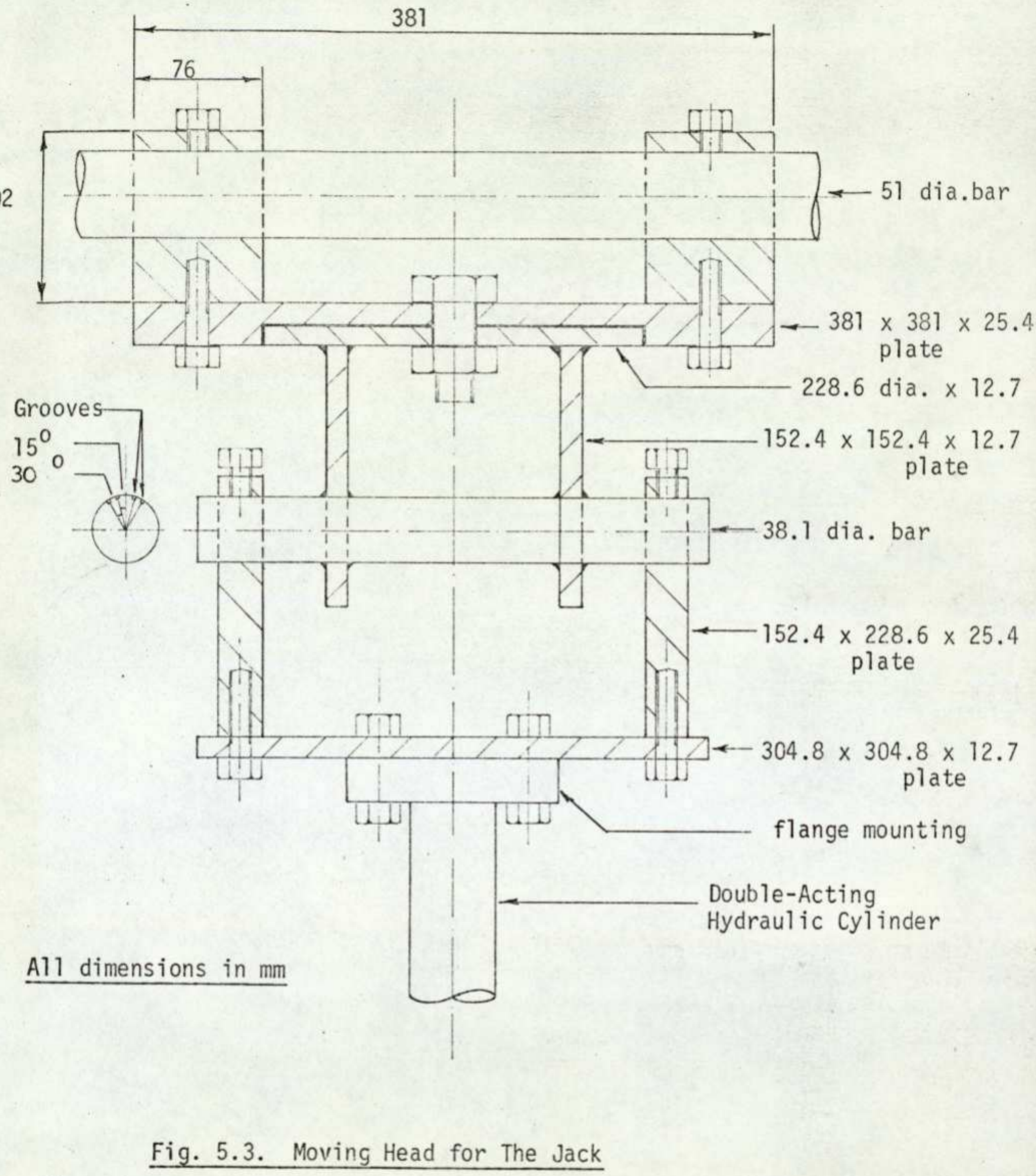


Fig. 5.3. Moving Head for The Jack

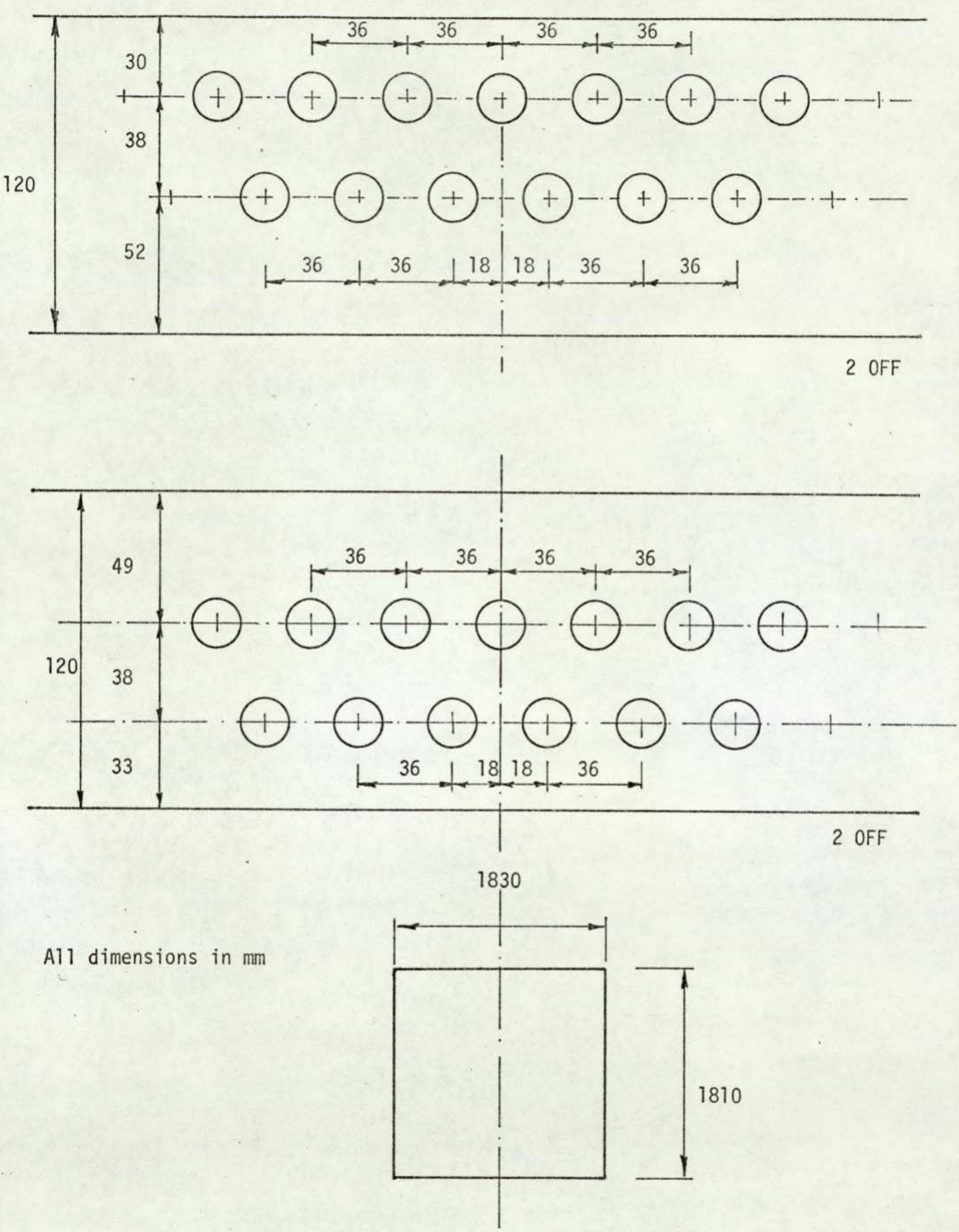
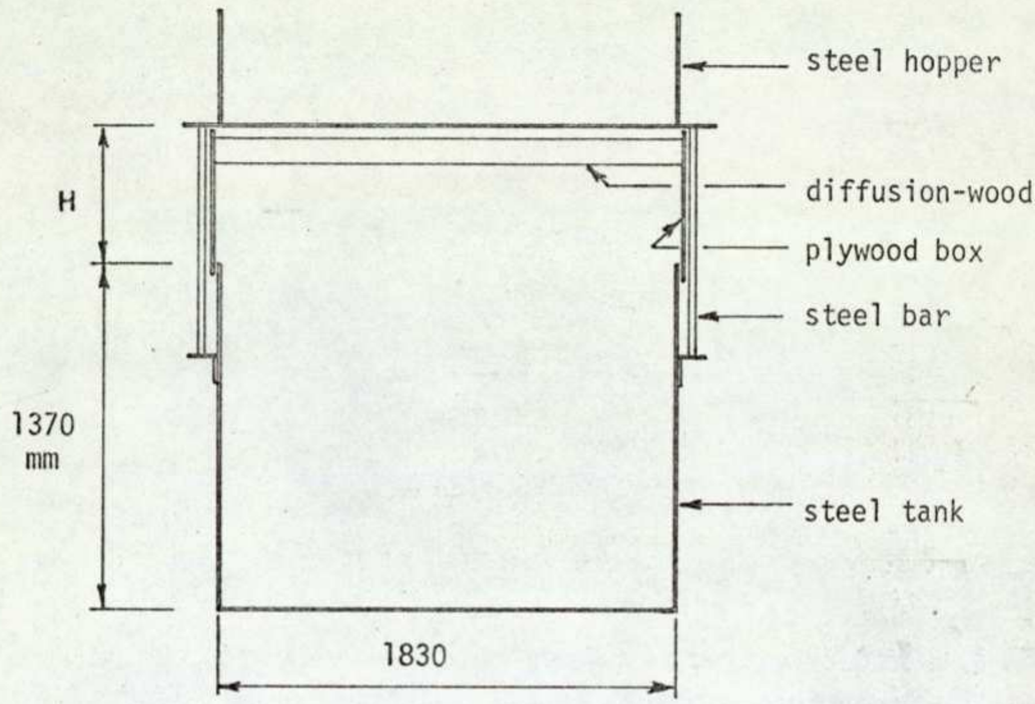


Fig. 5.4 Arrangement of the wood dowel in the scatter.



Stage 1 & 2 H = 210mm
Stage 3 & 4 H = 420mm
Stage 5,6 & 7 H = 570mm

Fig.5.5 The stages used in formation of the sand beds.

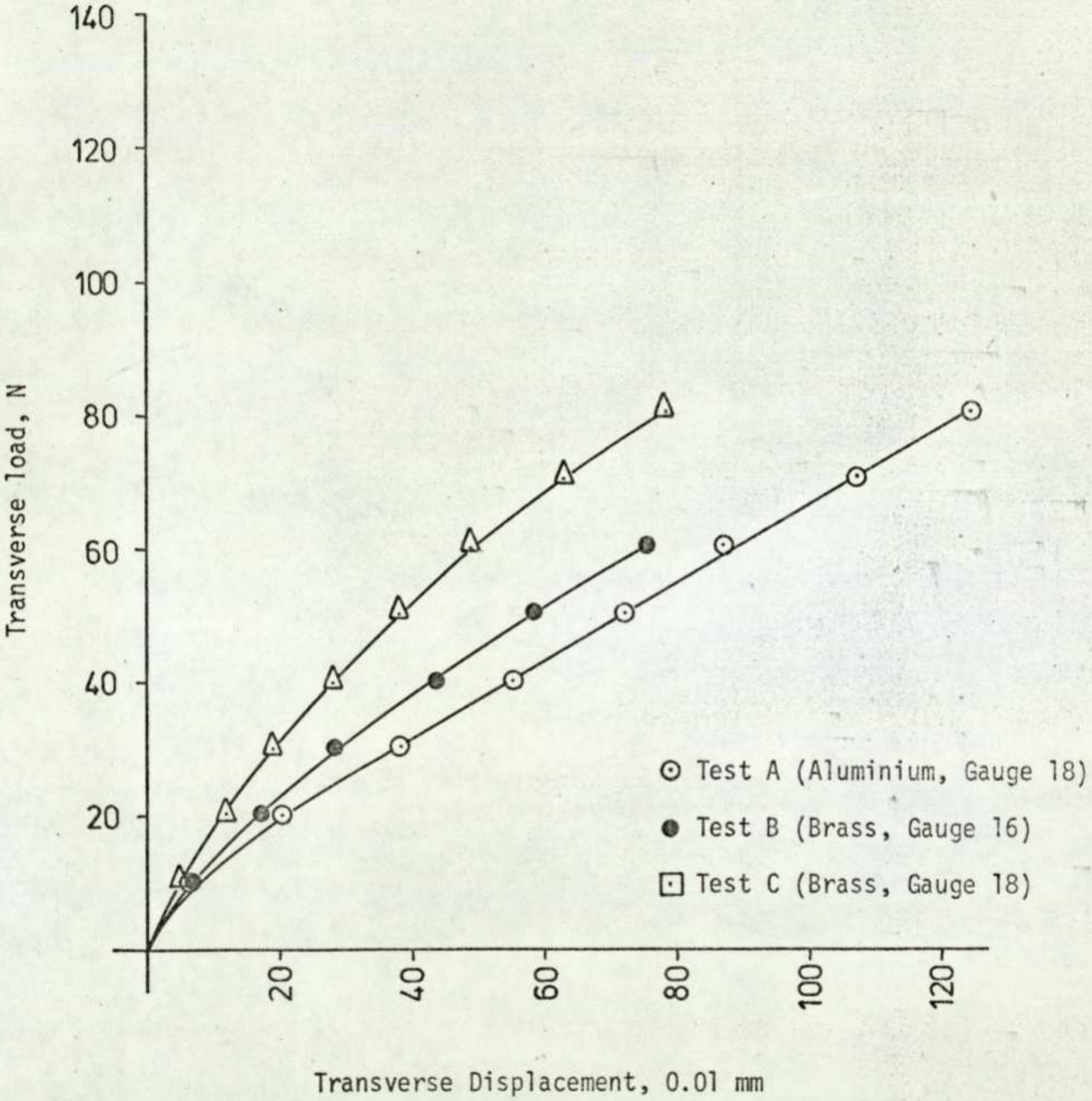


Fig. 5.6 T. Load Vs T. Displacement for test Nos. A, B & C

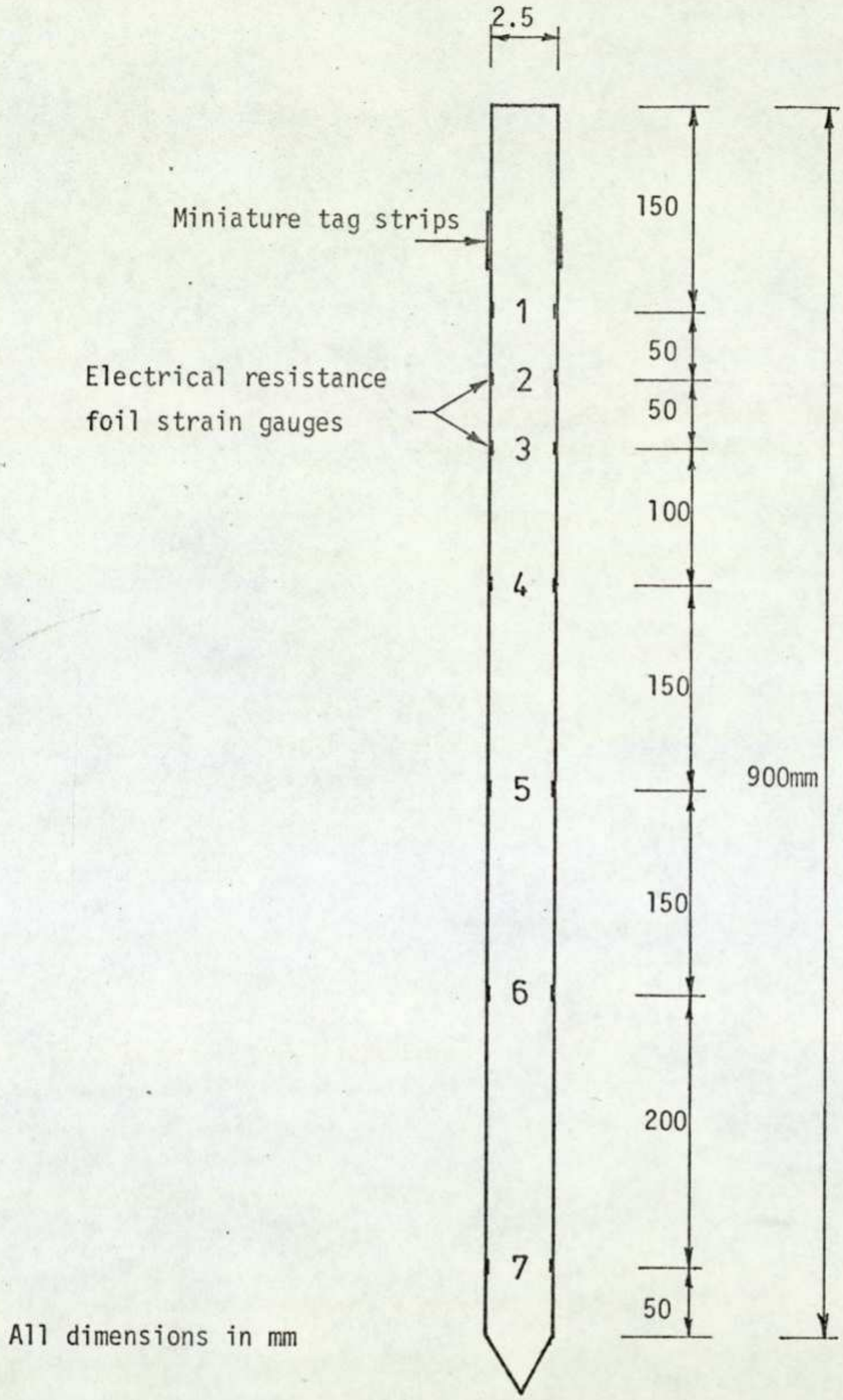
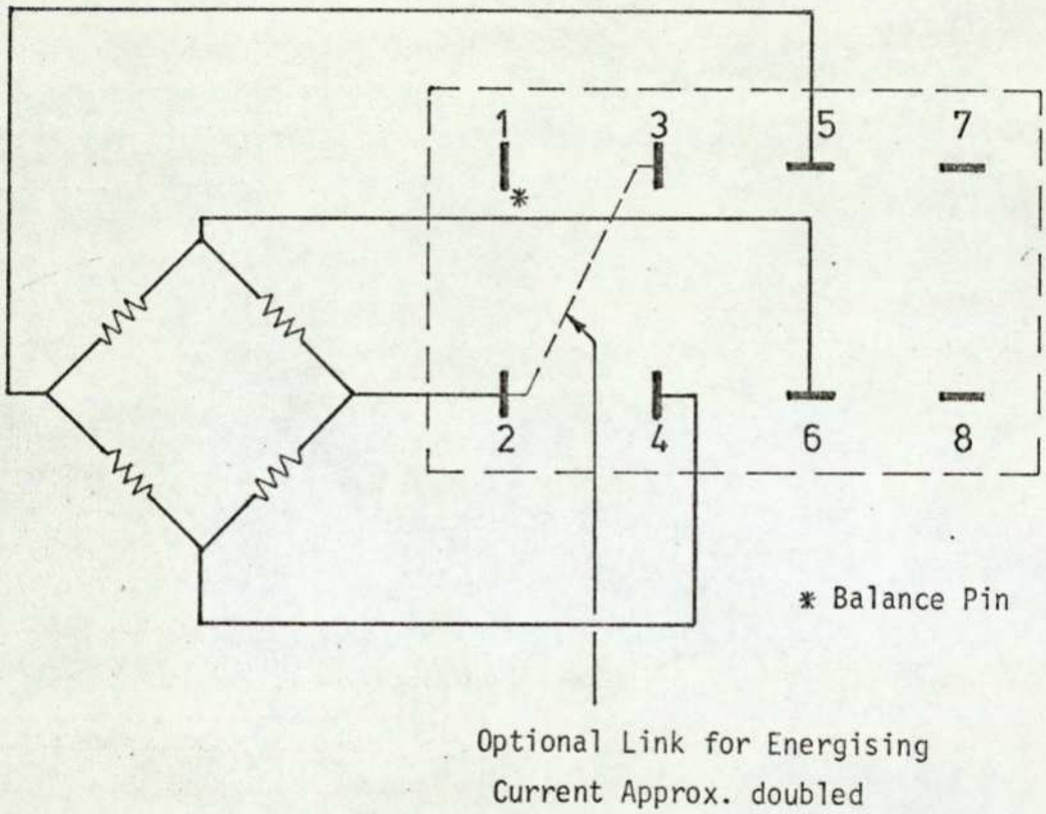
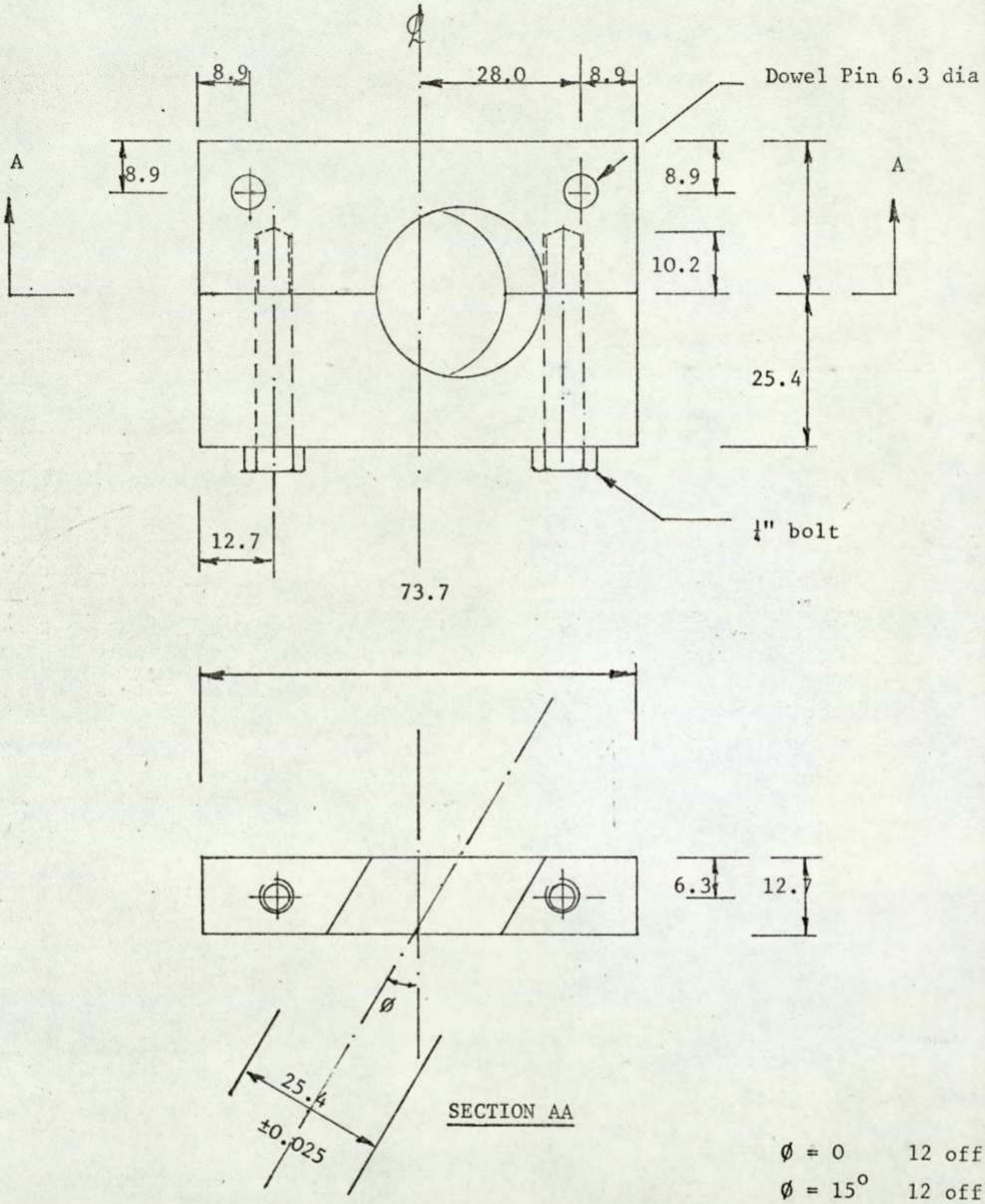


Fig. 5.7 Position of the electrical resistance strain gauges



$50 < R_{(Apex)} < 3000\Omega$

Fig. 5.8 Full Bridge System



ø = 0 12 off

ø = 15° 12 off

ø = 30° 12 off

All dimensions in mm

Fig. 5.9

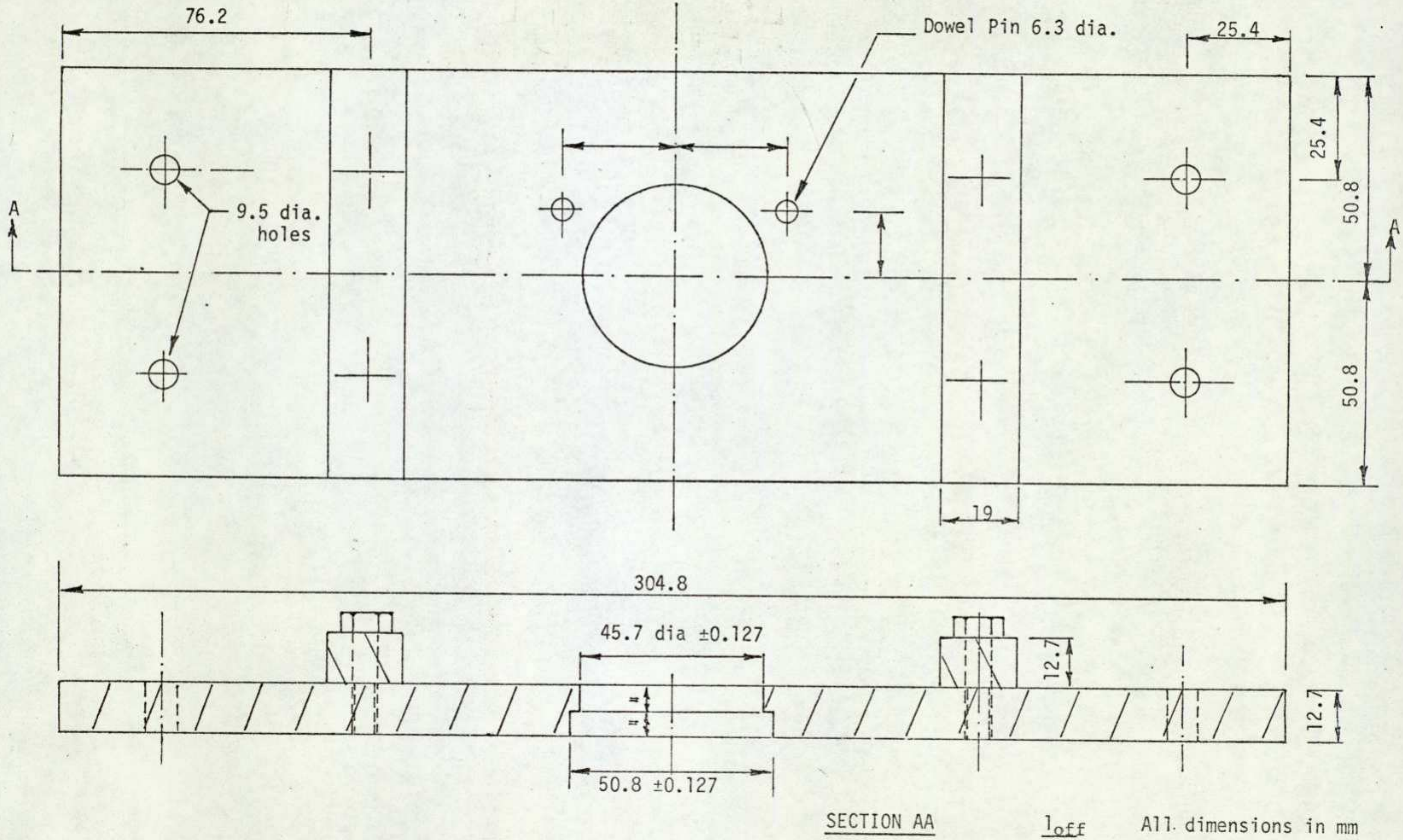
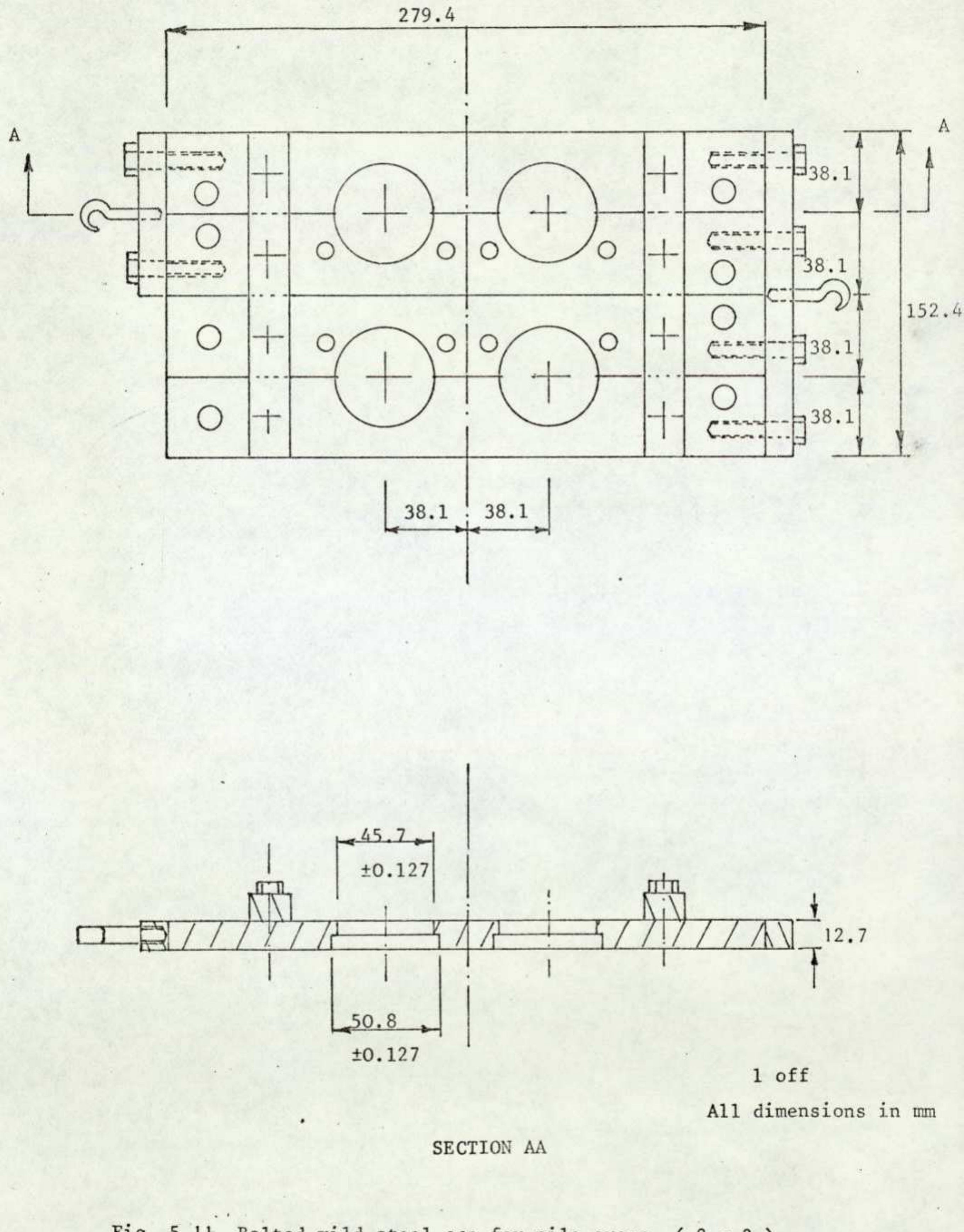


Fig. 5.10 Bolted mild steel cap for single pile

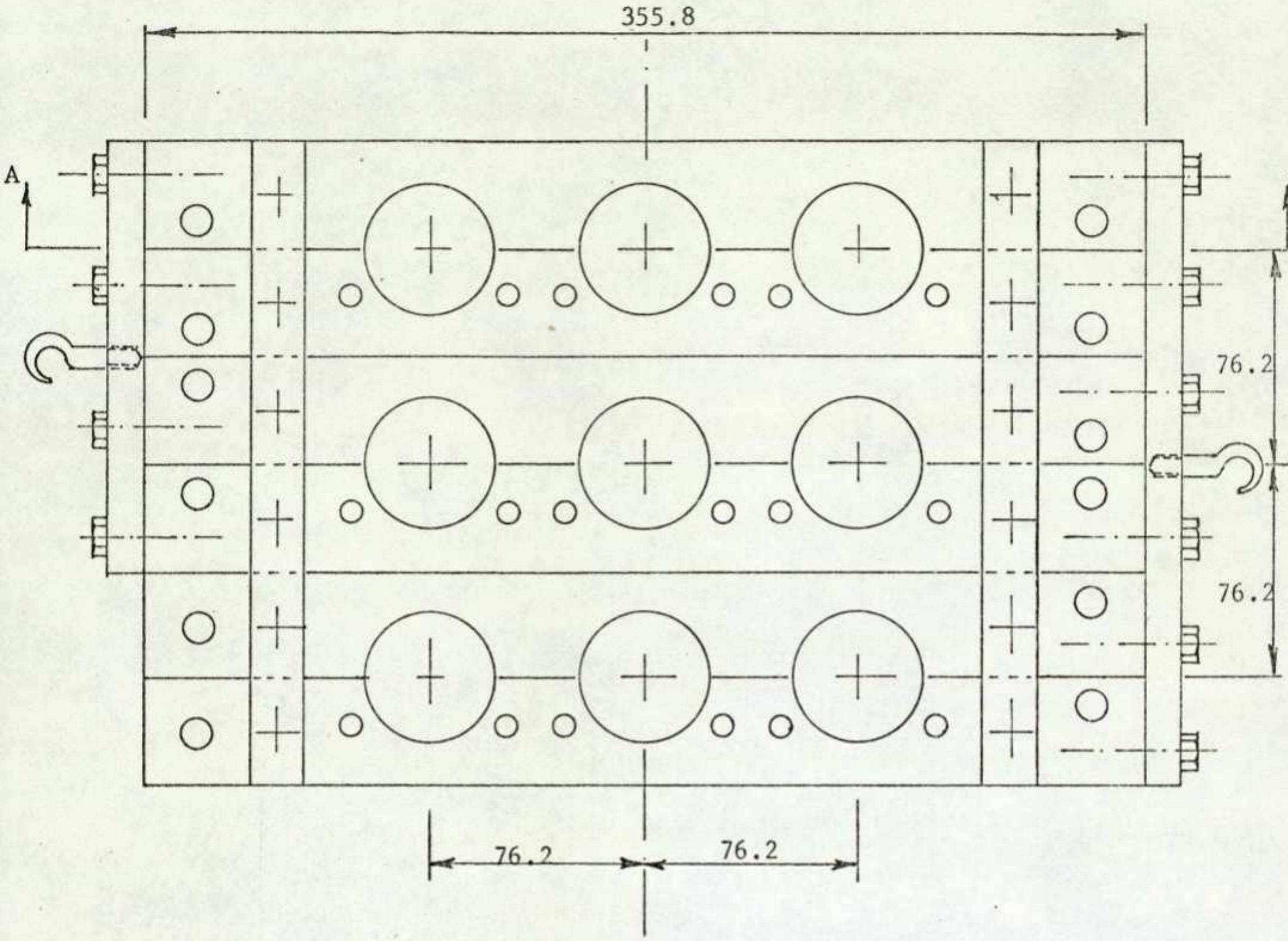


1 off

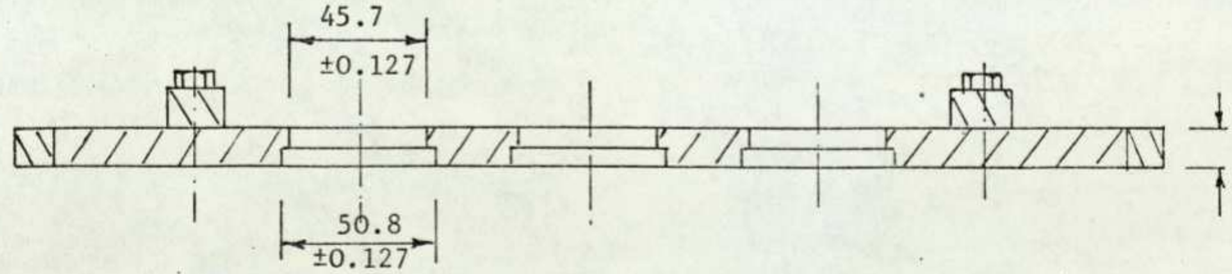
All dimensions in mm

SECTION AA

Fig. 5.11 Bolted mild steel cap for pile group (2 x 2)



All dimensions in mm



1 off

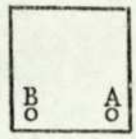
Fig. 5.12 Bolted mild steel cap for pile group (3 x 3)

Table 5.1

Sieve analysis test

Sieve	Mass retained , gm Sample 1	Sample 2	Sample 3
1.7 mm	-	-	-
1.4 mm	-	-	-
1.18mm	0.38	0.368	0.32
1.0 mm	52.564	52.815	51.32
850 μm	179.968	179.5	178.92
710 μm	136.468	135.5	136.05
600 μm	80.99	80.461	79.862
425 μm	44.268	45.235	47.052
300 μm	4.291	4.033	4.331
212 μm	0.704	0.74	0.873
150 μm	0.102	0.176	0.256
63 μm	0.083	0.256	0.353
Pan	0.018	0.051	0.108
Total	499.836	499.135	499.445

Table 5.2



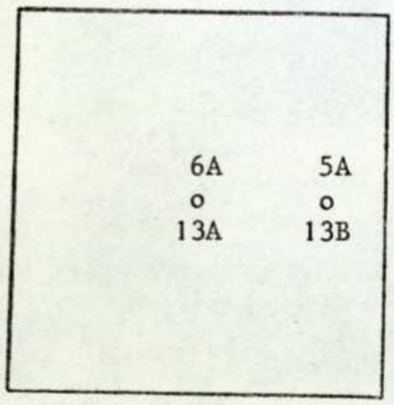
Container	Depth from sand surface Cm	Dry Density gm/~~Cm~~^{3} cc Test No. 3,4,5	6,7,8,9	10,11,12	13,14	15
1A	30	1.7070	1.7430	1.7374	1.72	-
2A	45	1.7127	1.7472	1.7425	1.7204	1.7240
3A	60	1.7415	1.7508	-	-	1.6808
4A	75	1.7343	1.7353	1.7549	1.7127	1.7148
5A	105	1.7564	1.7585	1.7513	1.7106	1.7405
1B	30	-	-	1.7477	1.7204	-
2B	45	-	1.7395	1.7436	1.7080	1.7302
3B	60	1.7271	-	-	-	1.7261
4B	75	1.7256	1.7456	1.7400	1.6920	1.7353
5B	105	1.7395	1.7636	1.7436	1.7280	1.7292
Average A		1.7304	1.7470	1.7465	1.7160	1.7150
Average B		1.7307	1.7490	1.7437	1.7120	1.7302
Average		1.73055	1.7480	1.7451	1.7140	1.7226

Average = 1.732 gm/Cm^3

Table 5.3

Horizontal load N	Horizontal displacements of single vertical piles , mm Test No. 6A	13A	13B	5A
60	1.34	0.93	1.16	1.11
80	1.91	1.61	1.64	1.65
100	2.49	2.34	2.25	2.21
120	3.15	3.15	2.85	2.85

Tests No. 6A and 13A were made in the centre of the tank but in different beds , while tests 13B and 5A were made within 400 mm of the tank side (in different beds) .



Chapter 6

Testing procedures and the test programme

6.1 Introduction

This chapter describes the experimental procedure for installing and testing the piles and pile groups . This includes :

a - The calibration of the instrumented piles for bending moment and axial load .

b - The determination of the flexural stiffness of the piles .

c - The method of driving the piles and forming the pile groups .

d - The method of testing the single piles and pile groups .

e - The test programme .

All material and equipment used for these tests are described in chapter 5 above . The results of the tests on single piles and pile groups are fully discussed in chapter 7 below .

6.2 Calibration of the model piles in bending

The instrumentation of the model piles was calibrated against known bending moments before the test programme commenced . This calibration was repeated three times during the test programme but , with the exception of three gauges which were apparently damaged , only insignificant changes in the calibration were observed . The calibration constants used in calculation were the mean of those observed in all three tests and are given in Table 6.1 .

The procedure used was as follows :

a - Each pile was supported horizontally on a pair of knife - edges as a simply supported beam .

b - A load of 100 N was applied in five equal increments to the centre of the pile and the digital voltmeter reading was taken after each increment had been in position for one minute .

c - The calculated bending moments and observed voltmeter readings for each gauge were plotted and the mean calibration constant was determined . In all cases , the relationship between bending moment and voltmeter reading was found to be linear within 1% over the range tested .

6.3 Calibration of the model piles under axial load

Each pile was set up vertically in the tank . The base of the pile was secured to the guide frame channels to prevent vertical movement , and the pile was supported laterally at two levels to prevent buckling . A vertical load was applied in increments using the hydraulic jack and proving ring , and the voltmeter reading was recorded for each increment .

Axial load and voltmeter reading were plotted to obtain the mean calibration constant .

6.4 Flexural rigidity of the pile

During calibration of the piles in bending , the deflection of the centre of the pile was measured . The flexural stiffness (EI) was computed from the expression

$$EI = \frac{P\,L^3}{48\,\delta}$$

where ,

P is the applied load

L is the length between supports

δ is the measured deflection at the centre

The measured values were nearly identical for all the piles , the mean value being

$EI = 0.80 \times 10^9$ N.mm^2

This value has been used in all calculations .

6.5 Preparing the model

The following procedure was adopted in driving the piles and preparing the model for testing :

a - The upper - structure was mounted on the tank , with a clearance of 1.0 m between the sand surface and the head of the jack in its fully retracted position .

b - A temporary guide frame was bolted to the top of the tank to hold the steel pile cap in position while the piles were driven . The cap was bolted to this frame , with the clamps on the cap loosened to allow the pile to pass freely through (see Fig. 6.1) .

c - The order of driving the piles for the model pile groups was standardized and is shown in Fig. 6.2 . This order is commonly used in the field .

d - Each pile in turn was placed in the appropriate hole in the cap and was aligned with a temporary guide 400 mm above the cap .

e - The hydraulic jack was aligned with the centre line of the pile and was used to drive the pile at a constant rate of penetration . An approximate measure of the driving resistance was obtained by recording the jack pressure .

f - After each pile had been driven , a dial gauge was placed on the head to record the vertical movements as subsequent piles were driven . In addition , during tests 15 and 16 , the strain gauges in the piles were read during the driving of subsequent piles .

g - After all piles had been driven , the appropriate cap was fixed securely to the heads of the piles , and the guide frame was removed .

h - Dial gauges were attached to measure the displacements and rotation of the cap , and the loading equipment was attached to the cap .

The model was then ready for testing .

6.6 Test procedure

The following procedure was adopted in testing each of the models :

a - An initial reading of the strain gauges and dial gauges was taken .

b - When both vertical and horizontal loads were applied to the model , the vertical load was first applied in increments . When the full vertical load had been applied , this was maintained constant and the horizontal load was applied , also in increments .

c - As each increment was applied , reading of all gauges were taken after the increment had been applied for one minute .

d - After the maximum load had been applied , this was maintained constant for 60 minutes , readings being taken every 20 minutes to observe any time dependent effects on the model .

e - The load was then removed from the model in stages , first the horizontal load and then the vertical load . After all the load had been removed , the model was left unloaded for 60 minutes and further readings were taken to observe any time dependent effects .

f - This loading cycle was repeated ten times for each model to determine the effect of cyclic loading on the pile behaviour .

6.7 Test data

During the driving and test loading process the data which were recorded for each test were :

a - The total applied load .

b - The bending moments at points along the pile shaft . The voltmeter output observed at each electrical strain gauge station was converted to moment by multiplying by the appropriate calibration constant .

c - The vertical and horizontal displacements of the pile cap .

d - The rotations of the pile cap .

e - The movements of the existing piles during driving of another pile .

All the data were analysed , and the full analysis is discussed in chapter 7 below .

6.8 Test programme

All tests were carried out in a uniform bed of sand and the caps were free standing - that is , there was no contact between the cap and the soil surface .

The test programme (see Table 6.2) was divided as follows :

a - Single piles

i - Three single piles were tested under horizontal loads only , with batters of :

$+ 15^{\circ}$ (Test No. 8A)

0° (Test No. 6A)

$- 15^{\circ}$ (Test No. 9A)

ii - Five single piles were tested under combinations of axial and transverse load with following batters :

$+ 30^{o}$	(Test No. 14C)
$+ 15^{o}$	(Test No. 14A)
0^{o}	(Test No. 13A)
$- 15^{o}$	(Test No. 14B)
$- 30^{o}$	(Test No. 14D)

b - Four - pile groups (2 x 2)

Five model groups were tested under horizontal load , using steel pile caps .

No. of positive battered piles	No. of vertical piles	No. of negative battered piles	Degree of inclination	Test No.
-	4	-	0^{o}	9
2	2	-	15^{o}	2
-	2	2	15^{o}	1
2	-	2	15^{o}	8
2	2	-	30^{o}	12

c - Nine - pile groups (3 x 3)

Seven model groups were tested under horizontal load , using steel pile caps .

No. of positive battered piles	No. of vertical piles	No. of negative battered piles	Degree of inclination	Test No.
-	9	-	0^{o}	3
3	6	-	15^{o}	4
6	3	-	15^{o}	5
3	3	3	15^{o}	6
6	-	3	15^{o}	7
3	3	3	30^{o}	10
3	6	-	30^{o}	11

d - Nine - pile groups (3 x 3)

Two model groups were tested under vertical and horizontal loads , using reinforced plaster caps .

No. of positive battered piles	No. of vertical piles	No. of negative battered piles	Degree of inclination	Test No.
-	9	-	0^{o}	15
3	3	3	15^{o}	16

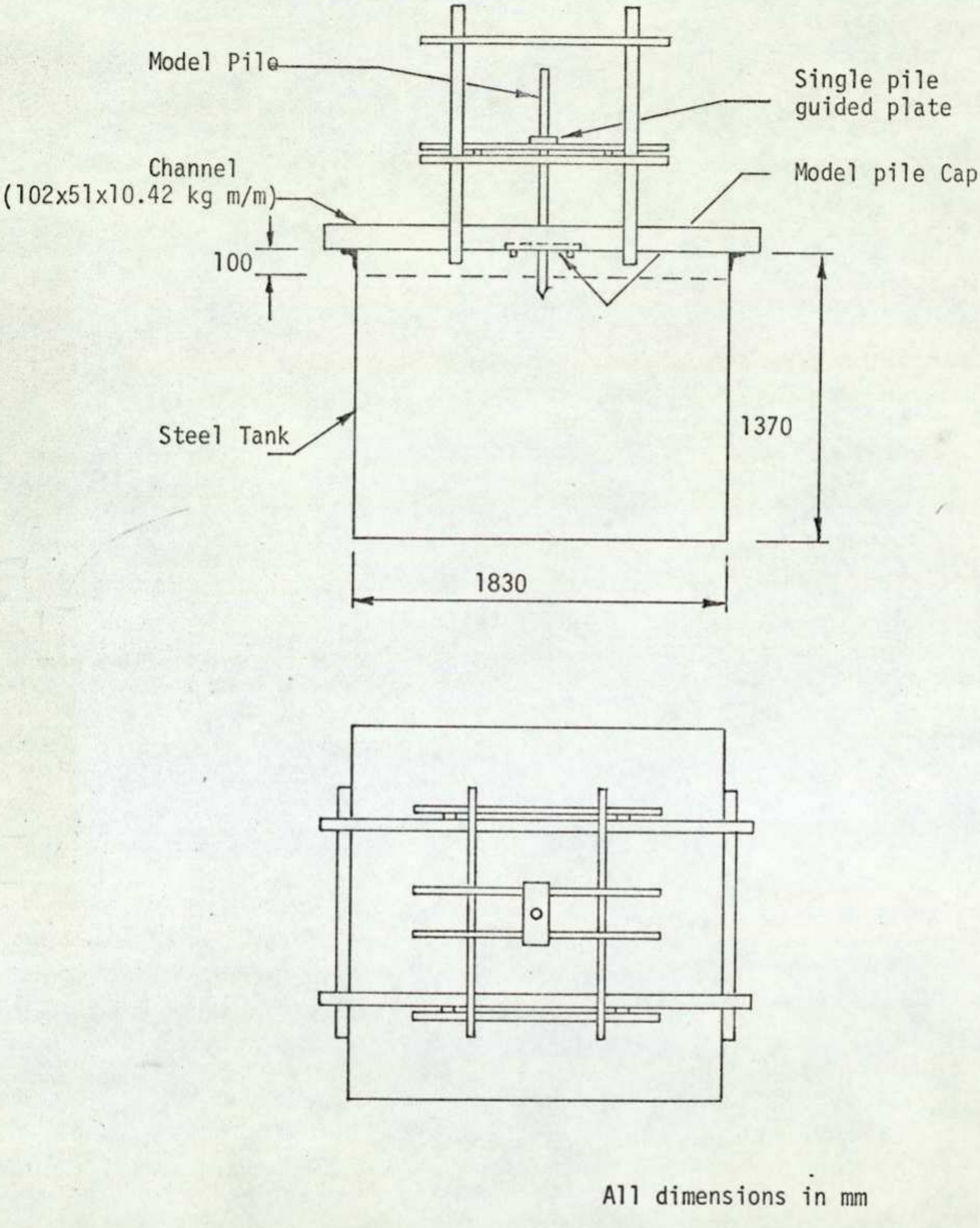
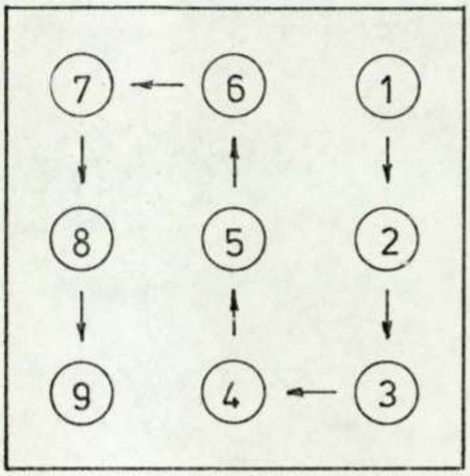
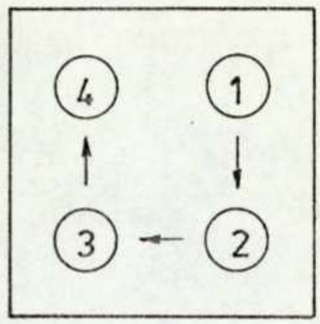


Fig. 6.1 The arrangement of the pile guidance



3 x 3

Pile group



2 x 2

Pile group

Fig. 6.2 Method of driving the piles in the group

Table 6.1

Mean gauge constant

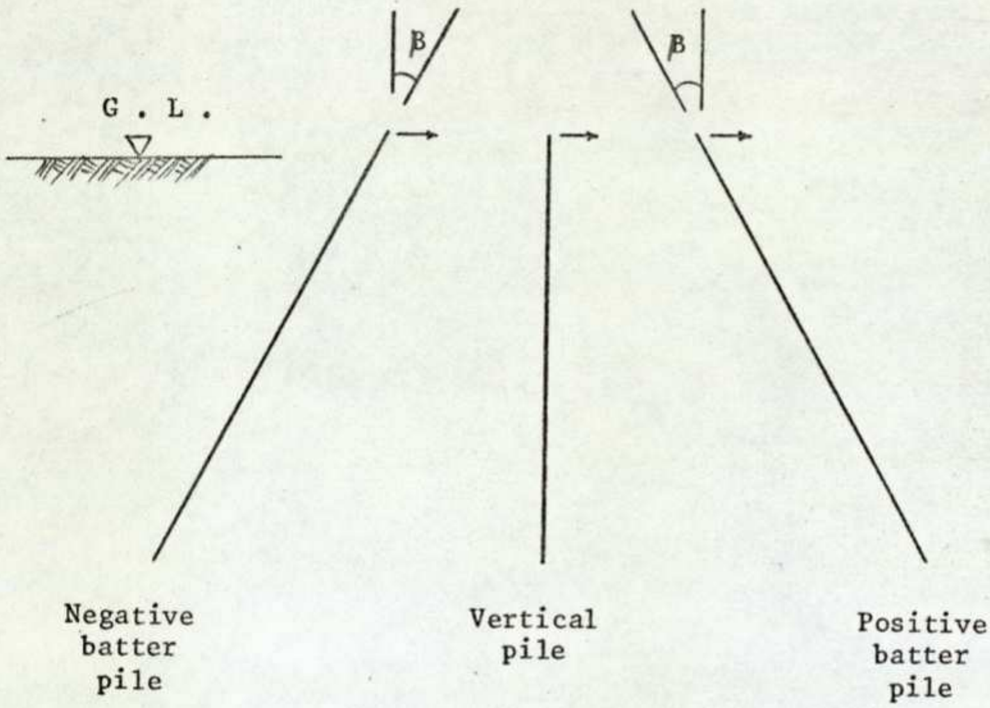
Station No.	Mean gauge constant N . mm / μv Piles A	B	C	D	E	F	G
1	-127.7	-125.0	-121.3	-128.4	-128.5	-132.0	-124.0
2	-125.2	-127.0	-134.2	-128.4	-127.0	-126.7	-125.0
3	-125.5	-131.6	-137.2	-133.3	-131.8	-155.0	-128.0
4	-132.0	-328.6	-798.5	-131.0	-139.0	-144.6	-135.6
5	-128.0	-137.5	-	-128.2	-	-155.7	-123.4
6	-125.0	-140.0	-228.5	-122.5	-129.0	-146.7	-125.8
7	-125.0	-123.8	-129.0	-115.5	-121.8	-119.7	-112.9

	H	J	K	L	M	N
1	-113.8	-132.4	-107.6	111.0	130.0	112.0
2	-112.8	-222.2	-108.2	116.5	130.0	-
3	-169.6	-117.6	-122.9	124.0	123.5	113.5
4	-430.6	-240.2	-217.3	120.0	130.0	-109.5
5	-329.7	-225.4	-268.2	123.0	122.0	-300.0
6	-167.7	-234.1	-500.0	122.0	127.0	254.5
7	-223.1	-178.4	-123.2	118.0	137.0	114.5

Table 6.2

Test Programme Diagram

Table 6.2



Sign convention for battered piles

○ Vertical pile

● Negative batter pile

◑ Positive batter pile

Table 6.2 - continued

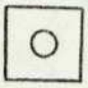
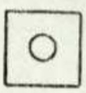
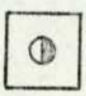
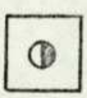
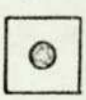
Test No.	Schematic Diagram Plan	Elevation	Type of piles	Degree of Inclination	Notes
6A			V	-	Single piles under sustained lateral load
8A			+ B	15^o	
9A			- B	15^o	
13A			V	0^o	Single piles under axial and transverse loads
14A			+ B	15^o	
14B			- B	15^o	
14C			+ B	30^o	
14D			- B	30^o	

Table 6.2 - continued

Four - pile groups (2 x 2)

Pile groups under horizontal load and using steel pile caps

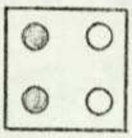
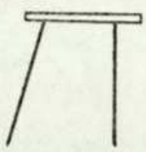
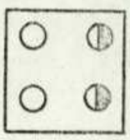
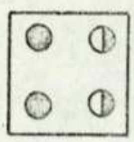
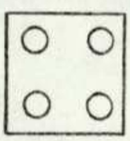
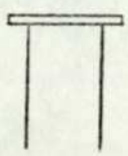
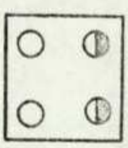
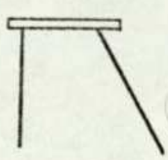
Test No.	Schematic Diagram Plan	Elevation	No. of Vertical Piles	No. of Battered Piles	Degree of Inclination
1			2	- 2	15°
2			2	+ 2	15°
8			-	-2,+2	15°
9			4	-	0°
12			2	+ 2	30°

Table 6.2 - continued

Nine - pile groups (3 x 3)

Pile groups under horizontal load and using steel pile caps

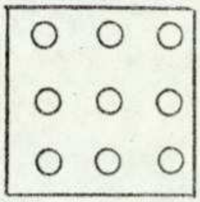
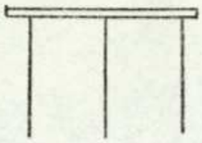
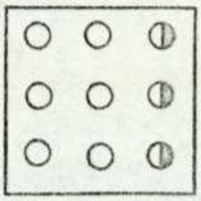
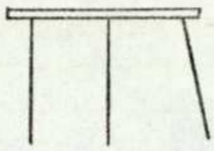
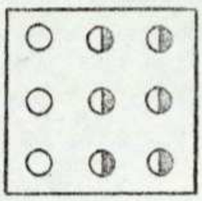
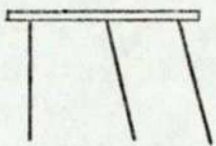
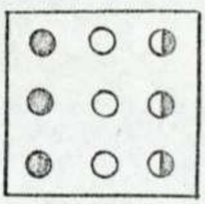
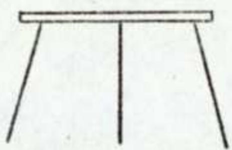
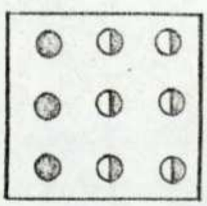
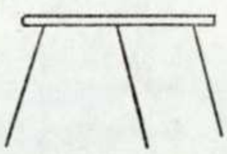
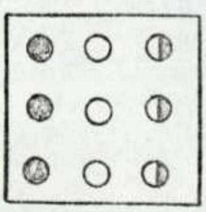
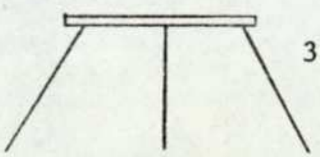
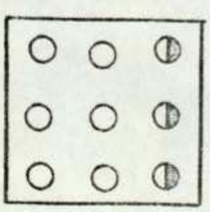
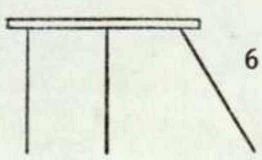
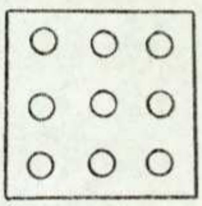
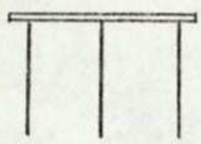
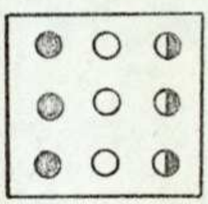
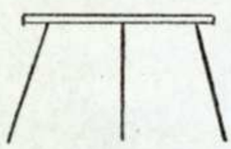
Test No.	Schematic Diagram Plan	Elevation	No. of Vertical piles	No. of Battered piles	Degree of Inclination
3			9	-	0°
4			6	+ 3	15°
5			3	+ 6	15°
6			3	+3,-3	15°
7			-	+6,-3	15°
10			3	+3,-3	30°
11			6	+ 3	30°

Table 6.2 - continued

Nine - pile groups (3 x 3)

Pile groups using reinforced plaster caps and under vertical and horizontal loads

Test No.	Schematic Diagram Plan	Schematic Diagram Elevation	No of Vertical Piles	No.of Battered Piles	Degree of Inclinatio
15			9	-	0°
16			3	+3 , -3	15°

Chapter 7

Model pile and pile group test results

7.1 Introduction

The purpose of this chapter is to present and discuss the results of tests on single model piles and model pile groups carried out according to the test procedure presented in section 6.8 above .

The results of 20 tests are reported , and the following are presented in each case :

a - The displacements and rotations of the pile cap .

b - The variation of bending moment along the length of each pile .

c - The effect of time and cyclic loading on these displacements , rotations and bending moments .

d - The residual displacements , rotations and bending moments after unloading .

e - The effect of pile installation on the axial stiffness of piles previously and subsequently driven , and on the movement of piles already installed .

Comparisons between the theoretical and experimental results are discussed in chapter 8 below .

7.2 Analysis of tests on single piles

The results of eight tests are discussed in this section . In five of these tests (13A , 14A , 14B , 14C and 14D) the piles were subjected to a combination of axial and transverse loads . In the remaining three tests (6A , 8A and 9A) the piles were subjected to horizontal load only .

7.2.1 Displacements and rotations of the pile caps

The measured displacements and rotations of the pile caps are shown in Fig. 7.1 to 7.18 and are summarised in Table 7.11 to 7.13 .

In test 13A , a single vertical pile was first loaded vertically with 1335 N (about 35% of the driving resistance) and then subjected to a horizontal load of 120 N applied in six equal increments of 20 N . This load cycle was repeated 10 times , and the measured transverse displacements and rotations are given in Figs. 7.1 and 7.2 . A further four load cycles were then applied in which the axial load was increased by equal increments to 3114 N (about 80% of the driving resistance) . The measured transverse displacements and rotations are given in Figs. 7.3 to 7.10 .

Four further tests (14A to 14D) were then carried out on piles driven at various inclinations to the vertical . The measured values of transverse displacements and rotations are given in Figs. 7.11 to 7.18 .

The following general conclusions may be drawn from the results of all these tests :

a - The transverse displacements of the pile cap on first loading were reduced with increasing inclination to the vertical . This reduction was significantly greater for piles with a positive batter , as shown in Table 7.1 below . The rotations of the pile cap were similarly reduced with increasing inclination , and these reductions were again most significant for piles with a positive batter .

Test No.	Inclination	Displacement	Rotation
14D	-30°	41%	86%
14B	-15°	74%	89%
13A	0°	100%	100%
14A	$+15^{\circ}$	52%	69%
14C	$+30^{\circ}$	20%	53%

Table 7.1 Transverse displacements and rotations on first loading expressed as % of the displacement and rotation of a vertical pile .

b - The transverse and rotational stiffness after cyclic loading were increased for vertical piles and piles with a negative batter but were almost unchanged for piles with a positive batter . This is shown in Table 7.2 below .

Test No.	Inclination	Displacement 5^{th} loading	Displacement 10^{th} loading	Rotation 5^{th} loading	Rotation 10^{th} loading
14D	-30°	43%	41%	50%	48%
14B	-15°	62%	59%	73%	69%
13A	0°	70%	64%	78%	72%
14A	$+15^{\circ}$	100%	94%	101%	93%
14C	$+30^{\circ}$	103%	100%	97%	94%

Table 7.2 Transverse displacements and rotations for 5^{th} and 10^{th} loadingsexpressed as % of displacements and rotations on 1^{st} loading .

It is also clear from Table 7.2 that the bulk of the increase in stiffness occurs in the first few load cycles .

c - Cyclic loading caused a significant permanent displacement of the pile head . In the case of vertical piles and those with a negative batter , this displacement was in the direction of transverse load . However , in the case of positively battered piles under both axial and transverse loading , the permanent displacement was in the contrary direction - that is , a positive transverse load was associated with a negative permanent displacement . Examination of Figs. 7.11 to 7.18 indicates that this permanent displacement is the result of two effects superimposed :

i - A permanent displacement associated with cycling of the transverse load . This is always in the direction of transverse loading .

ii - A permanent displacement associated with cycling of the axial load . This is small in the case of the vertical pile (Test 13A) . In the case of the battered piles , this displacement is always contrary to the direction of batter and increases with increasing inclination of the pile .

The probable explanation of this behaviour is as follows . While the piles for tests 15 and 16 were being driven , measurements were made of the bending moments in the piles . In all cases there was a significant bending moment locked into the raked piles after driving . This moment was in all cases positive for the positively raked piles and negative for those with a negative rake , and perhaps related to the increasing stiffness of the soil with depth . As explained above , examination of Figs. 7.11 to 7.18 indicates that much of the inelastic movement during cyclic loading takes place during the application and removal of the axial load . These movements are negative for the piles with positive rake and positive for those with a negative rake . It would seem therefore that the large cumulative movements of the raked piles under cyclic loading are related to the release of this locked - in bending moment by the changes of axial load , and the consequent straightening of the pile .

Similarly , the greater stiffness of the raked piles on first loading is perhaps related to the prestressing effect on pile and soil of this locked - in moment .

d - Table 7.3 compares the transverse displacements of piles on first loading , with and without axial loads . From this it can be seen that the ratio of maximum axial to transverse loads does not significantly affect the transverse stiffness of the pile head . This is confirmed by the behaviour of the pile in test 13A during the four final load cycles . (Figs. 7.3 to 7.10) . Thus , in determining these stiffnesses for use in the pile group analysis , they may be treated as independent quantities and superposition may be used in computing the forces applied to the piles .

Test No.	Inclination	Axial load N	Transverse load N	Transverse displacement mm
9A	-15^{o}	0	116	3.01
14B	-15^{o}	1335	116	2.29
6A	0^{o}	0	120	3.15
13A	0^{o}	1335	120	3.115
8A	$+15^{o}$	0	116	1.58
14A	$+15^{o}$	1335	116	1.63

Table 7.3 Comparison of transverse displacements of piles with and without axial load .

7.2.2 Distribution of bending moment due to transverse load (on first loading)

The distribution of bending moment along the single piles is shown in Figs. 7.19 to 7.21 , Figs. 7.23 to 7.25 , and in Tables 7.14 and 7.15 .

From these results , the following conclusions may be drawn :

a - The position of the maximum moment was at about one quarter of the embedded length below the soil surface in the case of the vertical pile . The maximum moment was slightly higher for raked piles , particularly those with a positive rake .

b - The maximum bending moment in the negatively inclined piles was nearly equal to that in the vertical pile . A positive inclination however reduces the maximum moment significantly . This is shown in Table 7.4 below .

Comparison with the three tests without axial load confirms again that this load has little effect on transverse behaviour .

Test No.	Inclination	Axial load N	Maximum moment
14D	-30°	1335	101%
14B	-15°	1335	103%
13A	0°	1335	100%
14A	$+15^{\circ}$	1335	76%
14C	$+30^{\circ}$	1335	64%
9A	-15°	0	109%
6A	0°	0	106%
8A	$+15^{\circ}$	0	74%

Table 7.4 Maximum bending moment due to transverse force expressed as % of value for a vertical pile with axial load .

c - In the case of the piles with a positive batter , the maximum moments were reduced very sharply below the point of maximum moment .

7.3 Analysis of the pile group tests

The results of 14 tests are presented and discussed below as follows :

a - F ive tests on four - pile groups (2 x 2) , under horizontal load only (Tests 1 , 2 , 8 , 9 and 12) . The results of these tests are presented in F igs. 7.26 to 7.46 and in Tables 7.16 to 7.18 .

b - Seven tests on nine - pile groups (3 x 3) , under horizontal load only (Tests 3 , 4 , 5 , 6 , 7 , 10 and 11) . The results of these tests are presented in F igs. 7.47 to 7.95 and Tables 7.19 to 7.13 .

c - Two tests on nine - pile groups (3 x 3) , subjected to a combination of vertical and horizontal loads (Tests 15 and 16) . The results of these tests are presented in F igs. 7.96 to 7.112 and in Tables 7.24 to 7.28 .

7.3.1 Displacements and rotations of the pile caps - F our - pile groups

F rom the results of these tests , the following general conclusions may be drawn :

a - Where the groups contained raked piles , the horizontal displacements of the pile caps on first loading were considerably smaller than in the case of a group of vertical piles . This was especially marked where both front and rear rows were raked . Thus , the horizontal displacements were reduced by about 70% if either the front or rear row was raked and by about 85% if both front and rear rows were raked .

b - Where the groups contained raked piles , the rotations of the caps were similarly much smaller than the rotation of the group of vertical piles under the same load . Also , the rotation was in the reverse direction -

that is , a positive horizontal force caused a negative rotation of the pile cap . This is to be expected since the axial stiffness of the raked piles is considerably greater than the transverse stiffness .

c - There was a significant increase in the transverse and rotational stiffness of the pile cap as a result of cyclic loading . In all cases , the greater part of this increase in stiffness occured within the first few load cycles .

d - Cyclic loading caused a significant permanent horizontal displacement and rotation of the pile cap . In all cases this displacement was in the direction of horizontal loading . Thus , in the groups containing raked piles , the rotation under each load was negative but the cumulative permanent displacement was positive . These permanent displacements and rotations were considerably smaller where the group contained raked piles , and much of it occured within the first few load cycles .

7.3.2 Displacements and rotations of the pile caps - Nine pile groups under horizontal load only

a - As in the case of the four - pile groups , the horizontal displacements on first loading were considerably smaller where the group contained raked piles . The effect of the different group configurations is shown in Table 7.5 below .

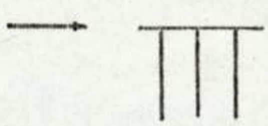
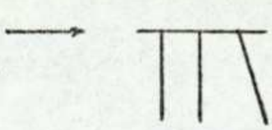
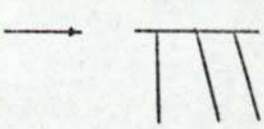
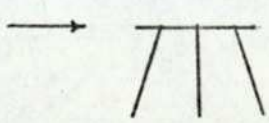
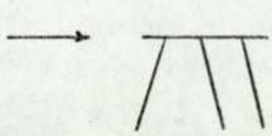
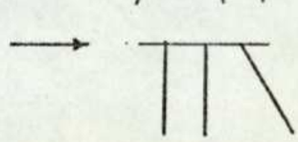
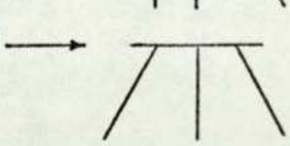
Test No.	Diagram	Inclination of raked piles	Horizontal pile cap displacement
3		-	100%
4		+ 15°	56%
5		+ 15°	50%
6		± 15°	29%
7		± 15°	27%
11		+ 30°	26%
10		± 30°	11%

Table 7.5 Horizontal pile cap displacement expressed as % of the displacement of a group of vertical piles .

b - The pile cap rotations were also considerably reduced where the groups contained raked piles . As in the four - pile groups , the rotations of groups containing raked piles were in the reverse direction - that is , positive horizontal forces caused negative rotations . The effect of the different group configurations is shown in Table 7.6 below .

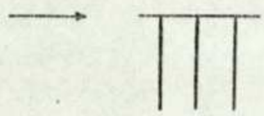
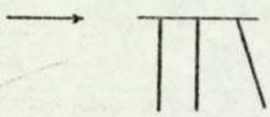
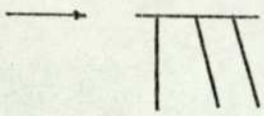
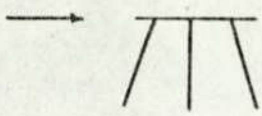
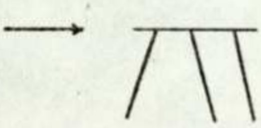
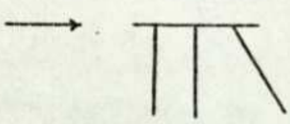
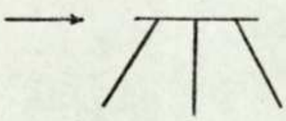
Test No.	Diagram	Inclination of raked piles	pile cap rotation
3		-	100%
4		+ 15°	- 51%
5		+ 15°	- 46%
6		± 15°	- 84%
7		± 15°	- 53%
11		+ 30°	- 91%
10		± 30°	- 69%

Table 7.6 Pile cap rotations expressed as % of the rotation of a group of vertical piles .

c - There was a significant increase in the horizontal and rotational stiffness of the groups as a result of cyclic loading . Most of this increase in stiffness occured during the first few cycles of loading (see Table 7.7) .

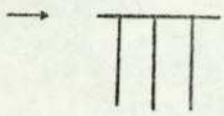
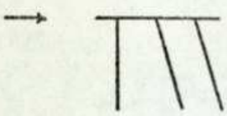
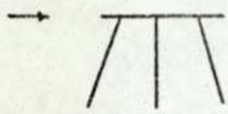
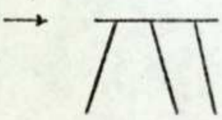
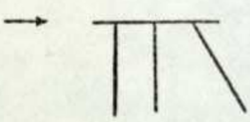
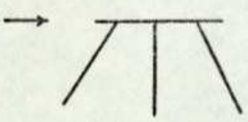
Test No.	Diagram	Inclination of raked piles	Displacement 5^{th} loading	Displacement 10^{th} loading	Rotation 5^{th} loading	Rotation 10^{th} loading
3		-	63%	58%	50%	47%
4		$+ 15^{o}$	47%	47%	71%	74%
5		$+ 15^{o}$	56%	53%	85%	72%
6		$\pm 15^{o}$	64%	67%	86%	86%
7		$\pm 15^{o}$	63%	63%	86%	81%
11		$+ 30^{o}$	58%	56%	76%	78%
10		$\pm 30^{o}$	73%	71%	85%	84%

Table 7.7 Displacements and rotations for 5^{th} and 10^{th} loading expressed as % of displacements and rotations for 1^{st} loading .

7.3.3 Displacements and rotations of the pile cap - Nine - pile groups under vertical and horizontal loads

Two tests only were carried out in this series - one test in which all piles were vertical (Test 15) and one test in which the front and rear rows were inclined at 15° to the vertical (Test 16) . The results of these tests are presented in Figs. 7.97 to 7.100 and Figs. 7.106 to 7.108 and in Tables 7.24 to 7.27 . The following general observations were made :

a - The vertical displacement under vertical load on first loading in Test 16 was only 47% of that for Test 15 in which all the piles were vertical .

b - There was some small rotation of the cap under central vertical load as a result of the variation in axial stiffness of the piles produced by the driving sequence . Raking the piles , as in Test 16 , reduced this rotation by 80% .

c - Comparison of Tests 3 and 6 with Tests 15 and 16 show that the existance of the vertical load had insignificant effect on the horizontal stiffness of the pile head . Thus , superposition may be used in computing the forces applied to the piles (see Table 7.8) .

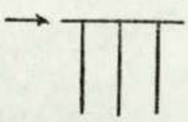
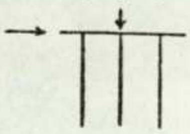
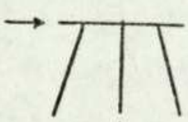
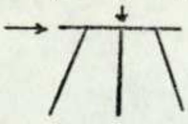
Test No.	Diagram	Inclination of raked piles	Horizontal displacement	Rotation
3		0°	100%	100%
15		0°	104%	146%
6		$\pm 15^{\circ}$	29%	- 84%
16		$\pm 15^{\circ}$	20%	- 62%

Table 7.8 Horizontal displacements and rotations on first loading

expressed as % of the displacement and rotation of the pile group in Test 3 .

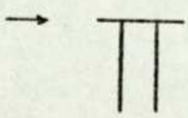
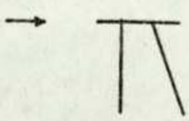
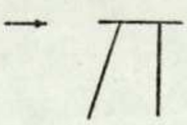
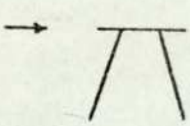
7.3.4 Bending moments in the piles - four - pile groups

The distribution of the bending moments along the piles in these groups are shown in Figs. 7.27 , 7.28 , 7.31 , 7.32 , 7.32a , 7.36 , 7.37 , 7.41 , 7.42 , 7.45 and 7.46 and in Table 7.18 .

The following general conclusions may be drawn :

a - The position of the maximum positive bending moment was at about one quarter of the embedded length for vertical piles . In the case of raked piles the position of the maximum bending moment was significantly lower , particularly in the case of piles with a negative rake .

b - The mean value of the negative bending moment at the soil surface was reduced where the rear row of piles was raked , but was not greatly affected by raking the front row . Also , there was a considerable reduction in the mean maximum positive bending moment where any of the piles were raked . This is shown in Table 7.9 below .

Test No.	Diagram	Inclination of raked piles	Mean negative moment at the soil surface	Mean maximum positive moment
9		0^{o}	100%	100%
2		$+ 15^{o}$	100%	30%
1		$- 15^{o}$	75%	35%
8		$\pm 15^{o}$	100%	30%

(cont.)

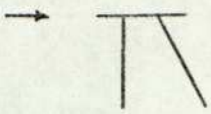
12		$+30^{o}$	110%	26%

Table 7.9 Mean negative moment at the soil surface and mean maximum positive moment expressed as % of the value in Test 9 .

c - Within any pile group , the maximum positive moments in the front row were larger than those in the rear row . Within each row , the moment in the first pile driven was smaller than that in the second .

d - In most of the piles , a small negative moment occured near the tip .

7.3.5 Bending moments in the piles - nine - pile groups

The distribution of the bending moments along the piles in these groups are shown in the following figures .

Figs. 7.50 to 7.53	(Test 3)
Figs. 7.57 to 7.60	(Test 4)
Figs. 7.64 to 7.67	(Test 5)
Figs. 7.71 to 7.74	(Test 6)
Figs. 7.78 to 7.81	(Test 7)
Figs. 7.85 to 7.88	(Test 10)
Figs. 7.92 to 7.95	(Test 11)
Figs. 7.101 to 7.104	(Test 15)
Figs. 7.109 to 7.112	(Test 16)

The following general conclusions may be drawn :

a - The position of the maximum positive bending moment was at about one fifth of the embedded length for vertical piles . The position of maximum positive moment was significantly lower where the piles were raked . This was particularly noticeable in the case of piles in the centre and rear rows .

b - The mean negative moment at the soil surface , and the mean of the maximum positive moments were smaller where the group contained raked piles , than where the piles were all vertical . This is shown in Table 7.10 below .

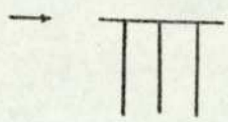
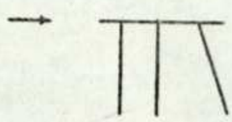
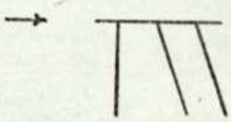
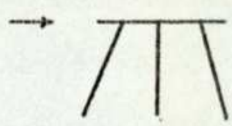
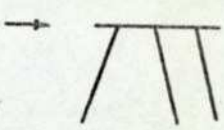
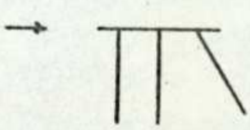
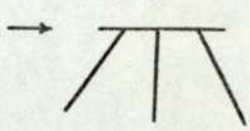
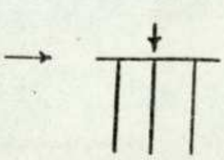
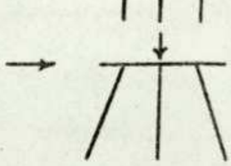
Test No.	Diagram	Inclination of raked piles	Mean negative moment at the soil surface	Mean maximum positive moment
3		0°	100%	100%
4		$+ 15^{\circ}$	84%	62%
5		$+ 15^{\circ}$	86%	62%
6		$\pm 15^{\circ}$	75%	44%
7		$\pm 15^{\circ}$	60%	39%
11		$+ 30^{\circ}$	65%	30%
10		$\pm 30^{\circ}$	55%	25%
15		0°	85%	99%
16		$\pm 15^{\circ}$	67%	40%

Table 7.10 Mean negative moment at the soil surface and mean maximum positive moment expressed as % of the values in Test 3 .

c - The negative bending moments at the soil surface in the piles of each group had nearly the same value . However the maximum positive moments depended on the position of the pile in the group , and on the inclination in the case of a raked pile . For example , in Test 15 , the greatest positive bending moment were those for the four corner piles , which had nearly the same value . Similarly , the four piles at the centre of each side of the group carried nearly the same moment , while the smallest moment was carried by the centre pile .

d - The mean value of the maximum positive bending moments in Test 3 was 10719 N.mm and in Test 15 it was 10607 N.mm a difference of less than 1% . Similarly , the mean values in Tests 6 and 16 were 4753 N.mm and 4283 N.mm respectively - a difference of less than 10% . Thus the vertical load does not significantly affect the bending moments caused by horizontal loads .

7.4 Driving and redriving resistance of the piles

In driving the piles for all of the groups except those for Tests 15 and 16 , an approximate measurement of the driving resistance was obtained by noting the jack pressure . These driving resistance are shown in Figs. 7.33 , 7.38 , 7.47 , 7.54 , 7.61 , 7.68 , 7.75 , 7.82 , 7.89 . No attempt was made to redrive any of these piles .

When driving the piles for Tests 15 and 16 , an accurate measurement of the driving resistance was made and the piles were initially driven to within 5 mm of there correct final position . When all the piles were in position , they were redriven , in the same order as before , to the correct final level . The redriving resistance was measured and is shown in Figs. 7.96 and 7.105 .

It was observed that :

a - For pile groups containing vertical piles only , the driving resistances were increased with increasing number of piles in the group , with the largest value for the latest driven pile .

For Test 15 , the redriving resistance was greater than the driving resistance for the same pile in the group by an ammount ranging between 10% to 25% depending on the position of the pile in the group .

These increases in the driving and redriving resistance were related to the increase of soil density during driving .

b - For pile groups containing raked piles , the driving resistance within each row increased with increasing number in the row (the largest value was always for the latest driven pile) . The mean resistance in the second driven row ~~were~~ was always greater than in the first , but the increase in resistance was small where the inclination of the rows were different so that the tips of the piles were far apart .

7.5 Upward movement of the existing piles due to pile installation

The upwards movement of the existing piles due to pile installation was measured when installing the nine - pile groups , the movements are given in Figs. 7.47 , 7.54 , 7.61 and 7.68 .

The upward movement of an existing pile during installation of another pile was affected by the position of the pile in the group , the degree of inclination , and the number of raked piles in the group .

The maximum value was noticed for the pile group containing vertical piles only and for the pile installed first .

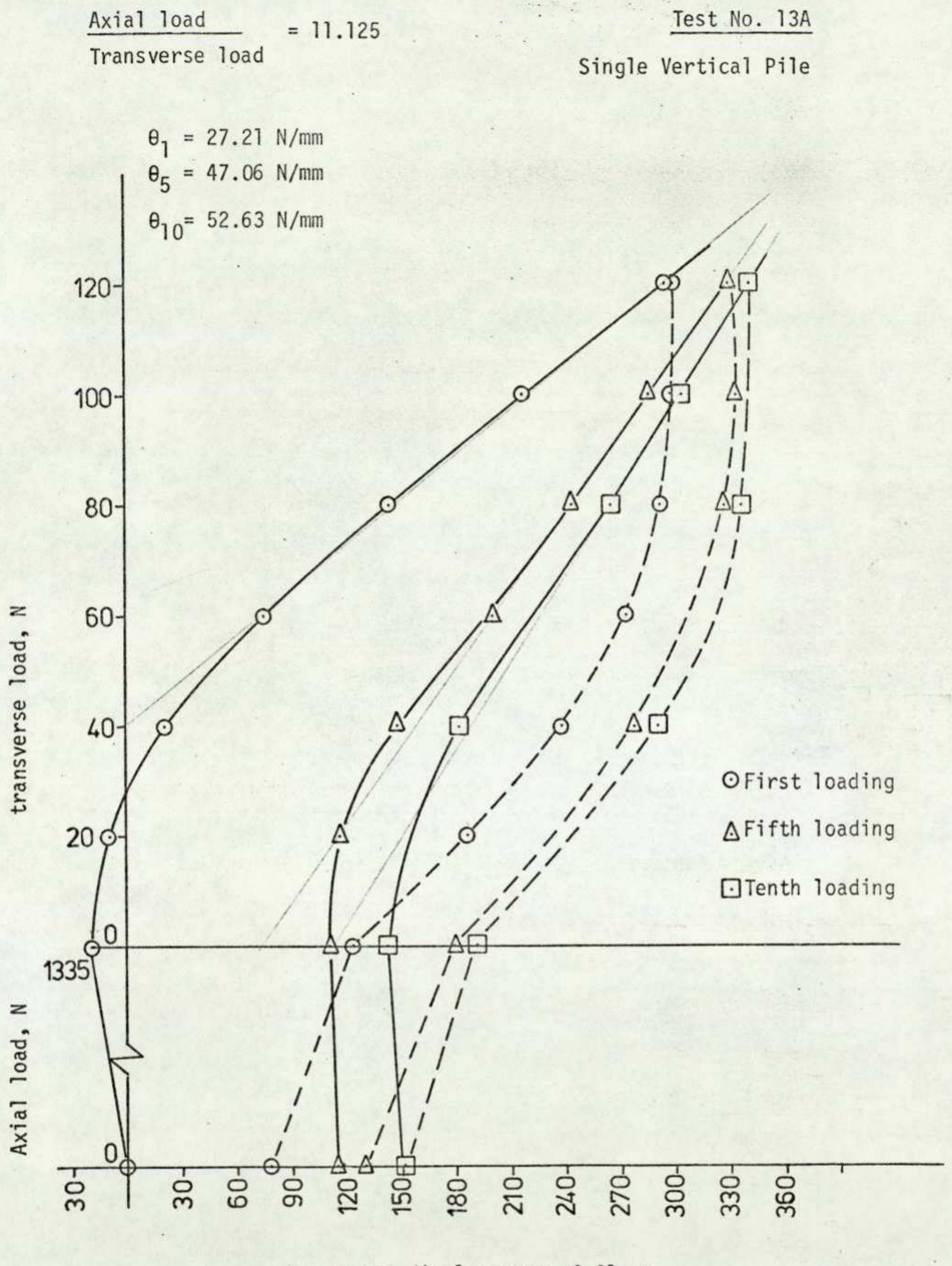


Fig. 7.1 Axial and Transverse loads Vs Transverse Displacement

Test No. 13A

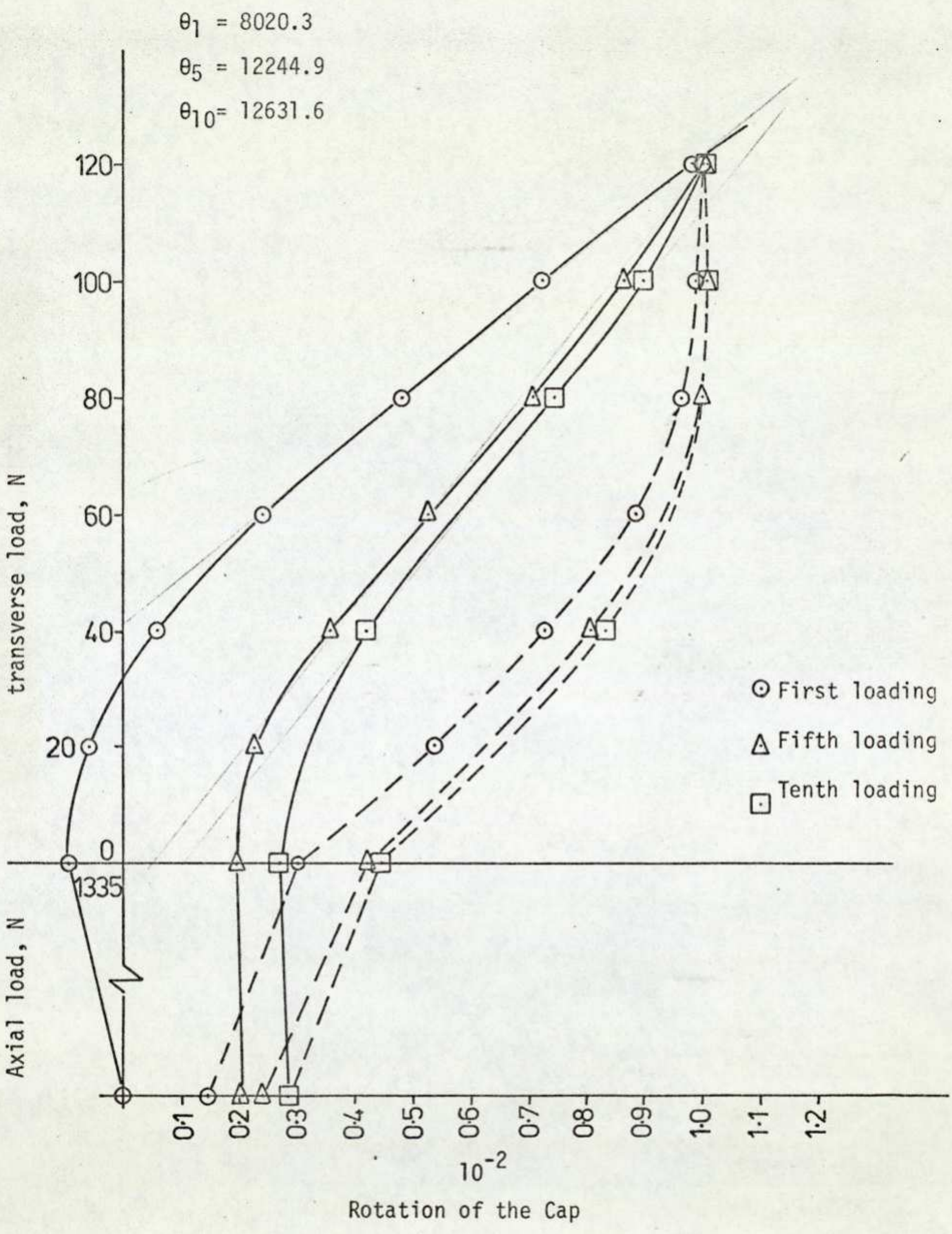


Fig. 7.2 Load Vs Rotation of the Cap

Axial load / Transverse load = 14.825

Test No. 13A

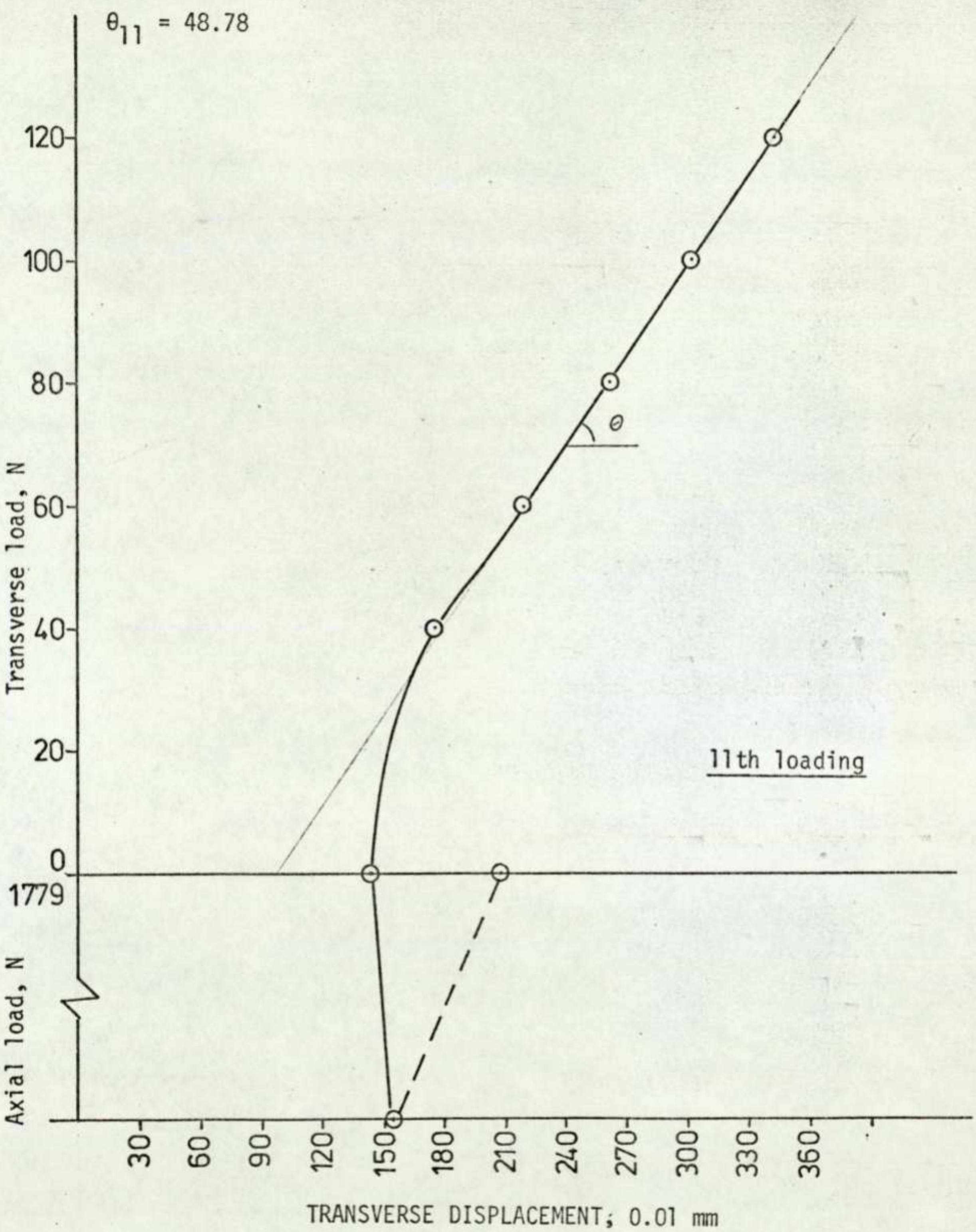


Fig. 7.3 Axial and Transverse loads Vs Transverse Displacement

Test No. 13A

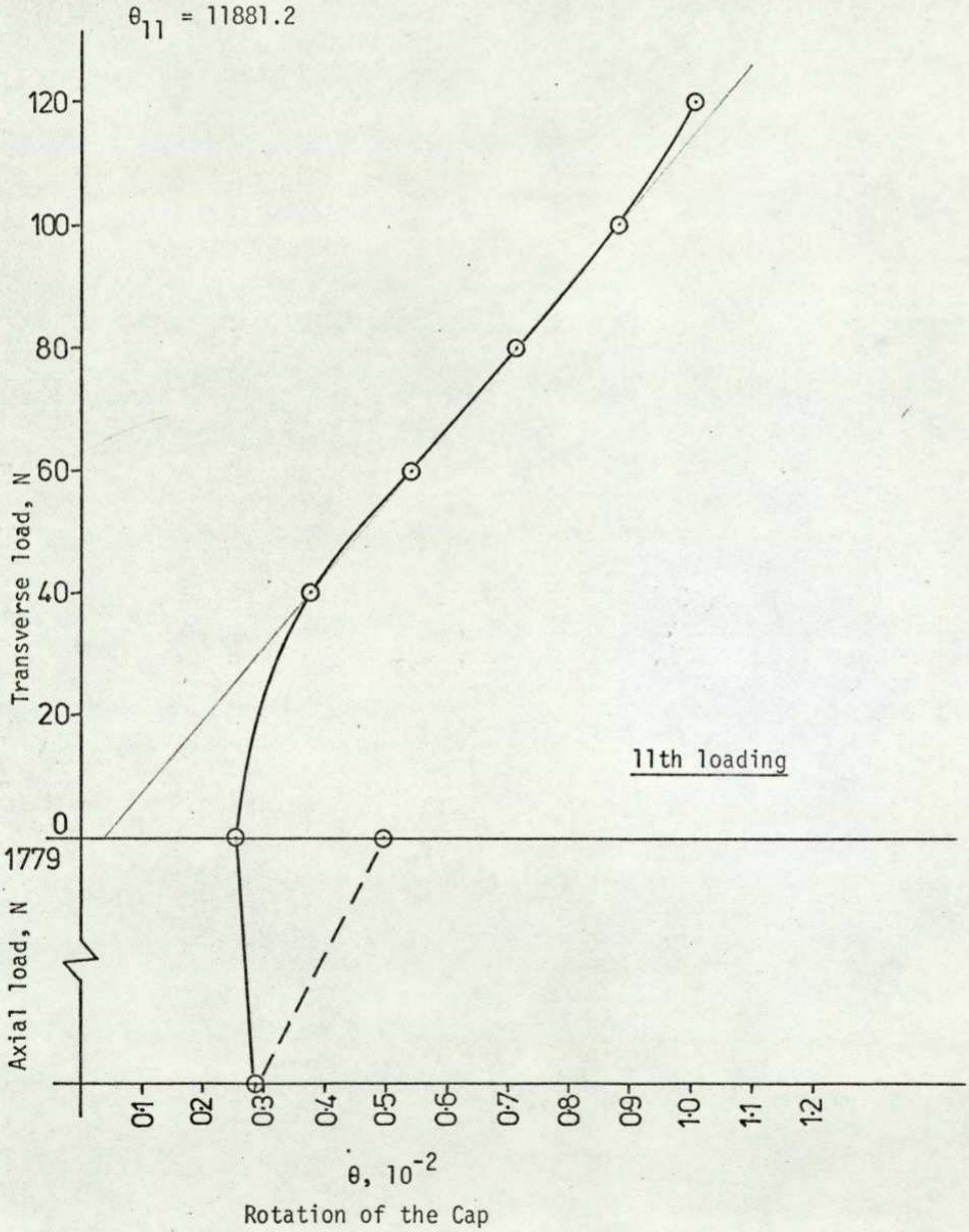


Fig. 7.4 Load Vs Rotation of the Cap

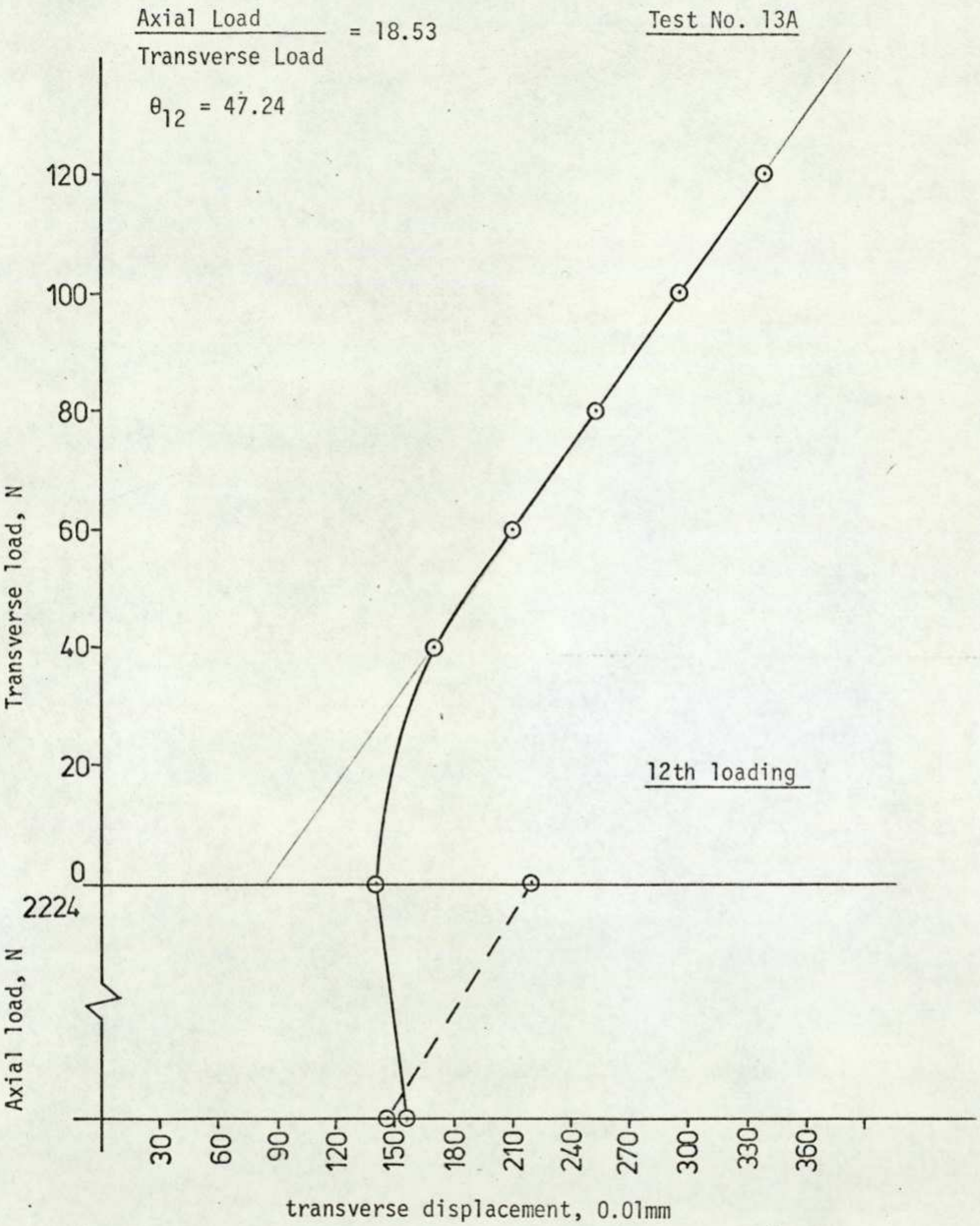


Fig. 7.5 Axial and Transverse loads Vs Transverse displacement

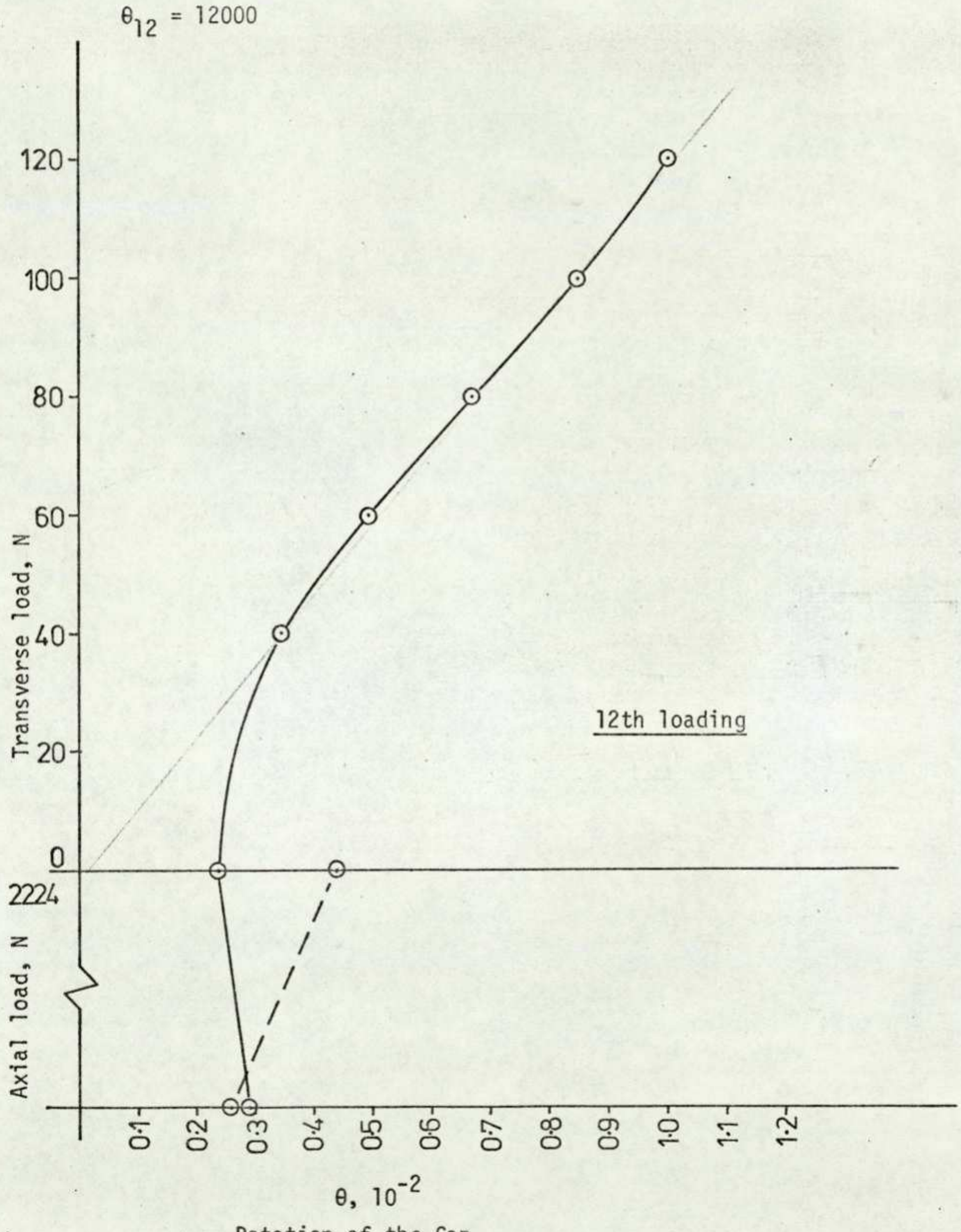


Fig. 7.6. Load Vs Rotation of the Cap

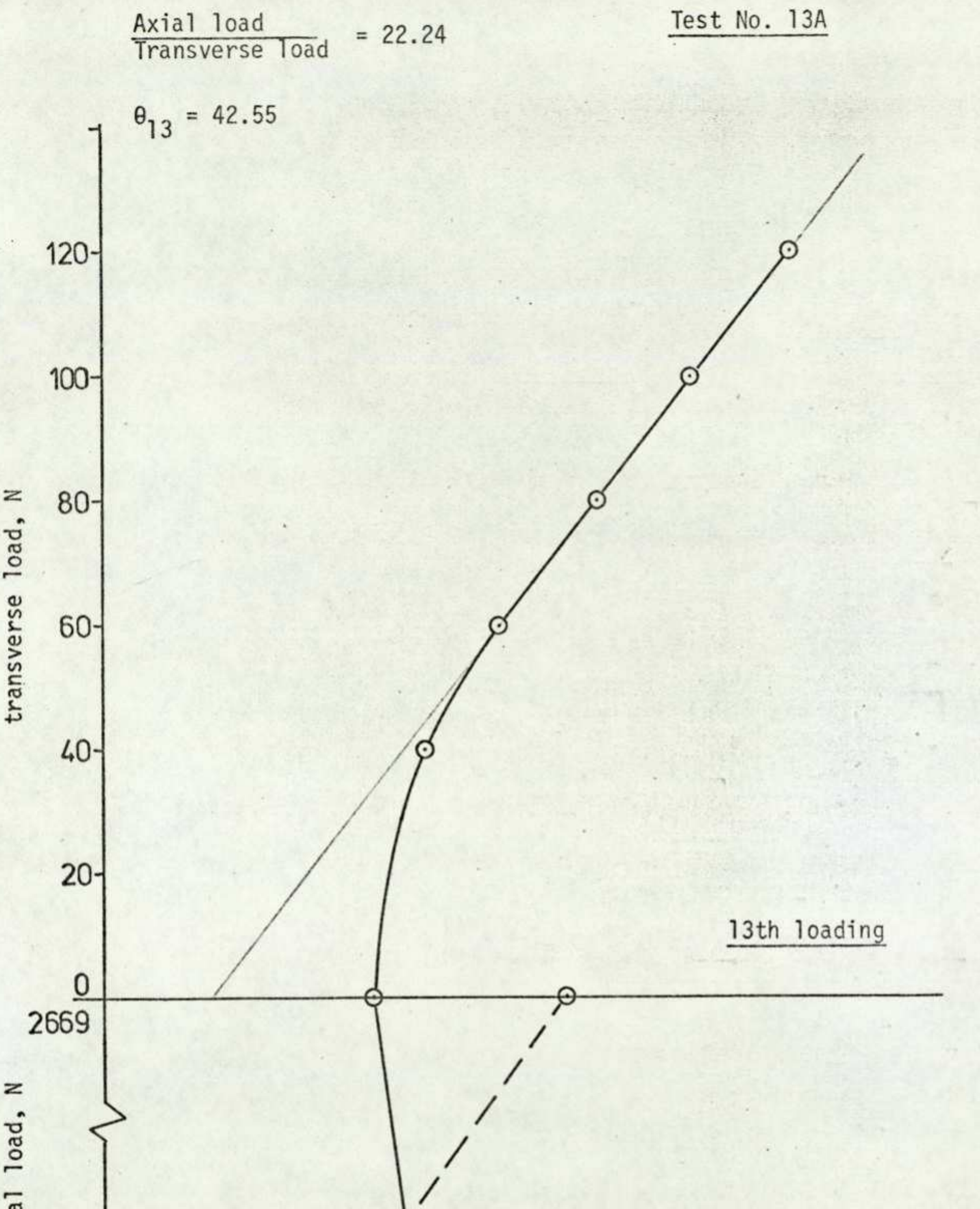


Fig. 7.7 Axial and Transverse loads Vs Transverse Displacement

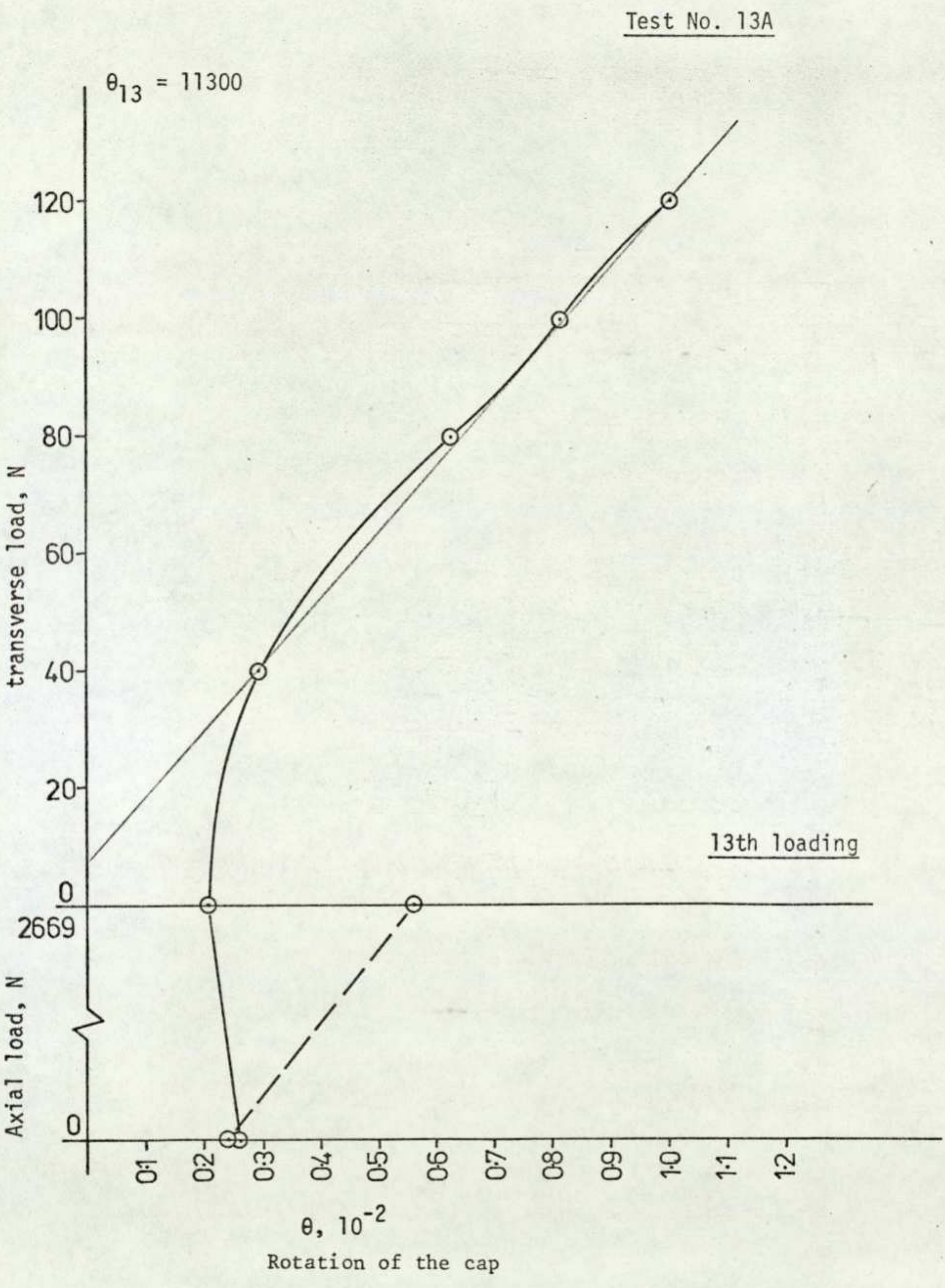


Fig. 7.8 load Vs Rotation of the Cap

Test No. 13A

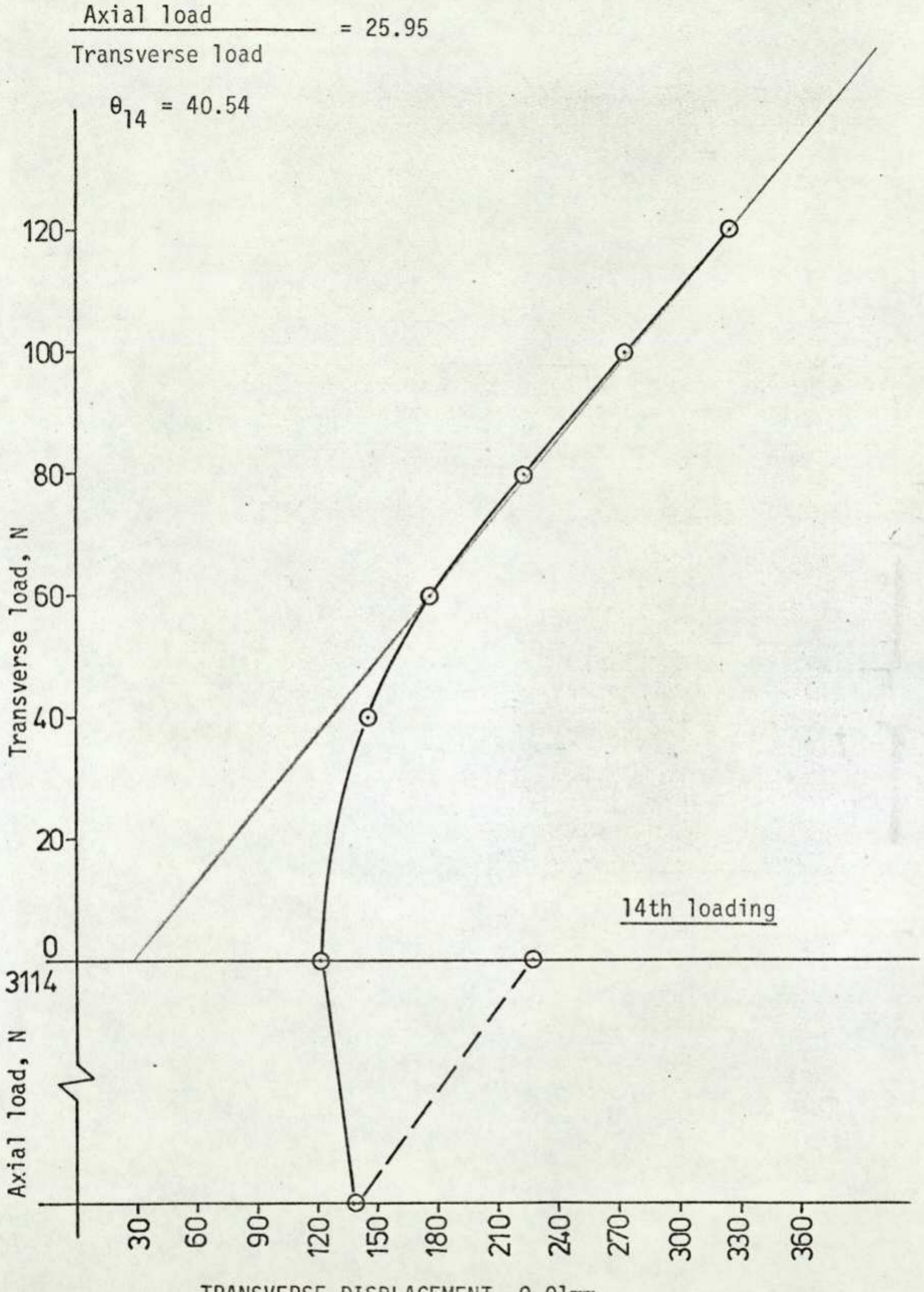


Fig. 7.9. Axial and Transverse Loads Vs Transverse displacement

Test No. 13A

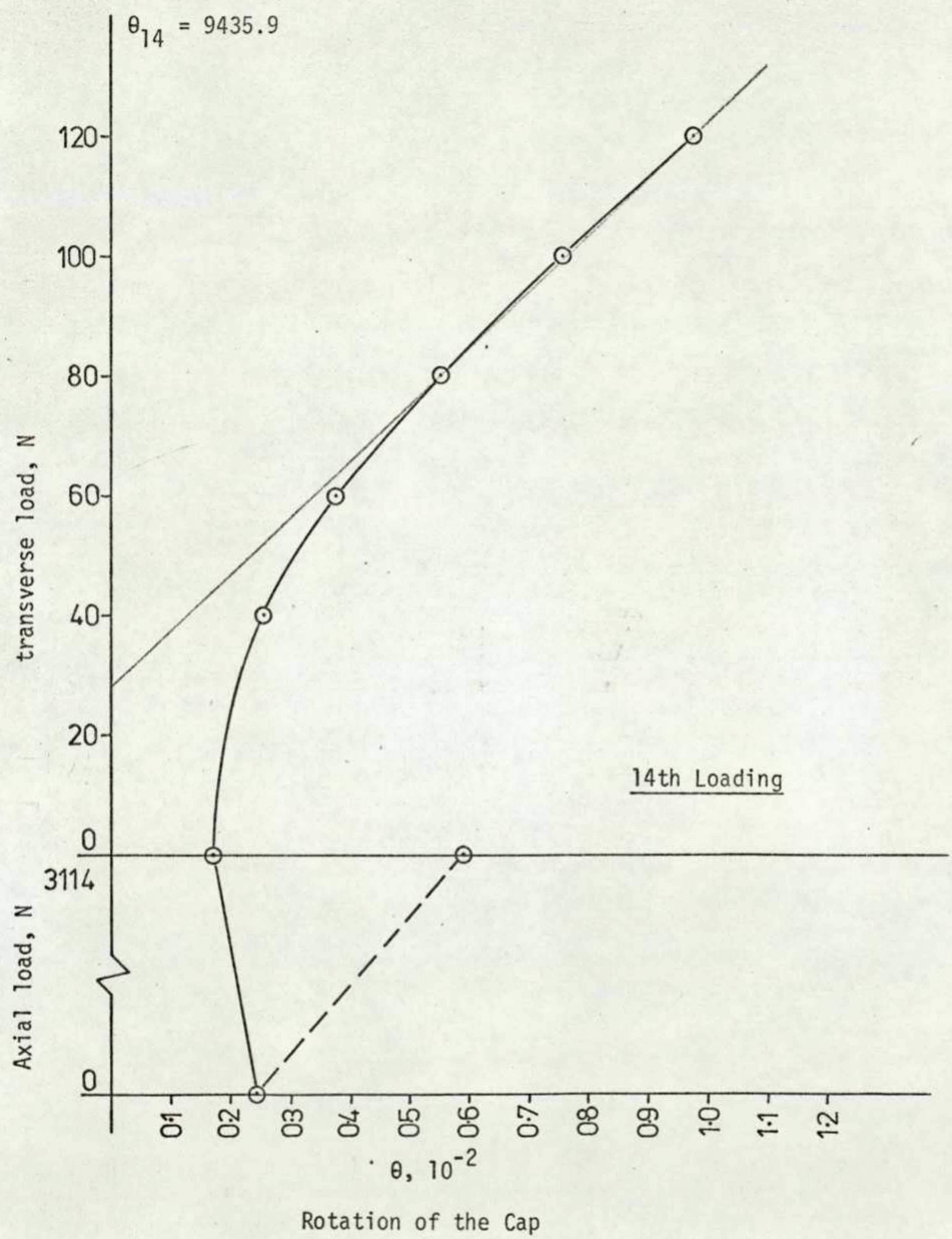


Fig. 7.10 load Vs Rotation of the Cap

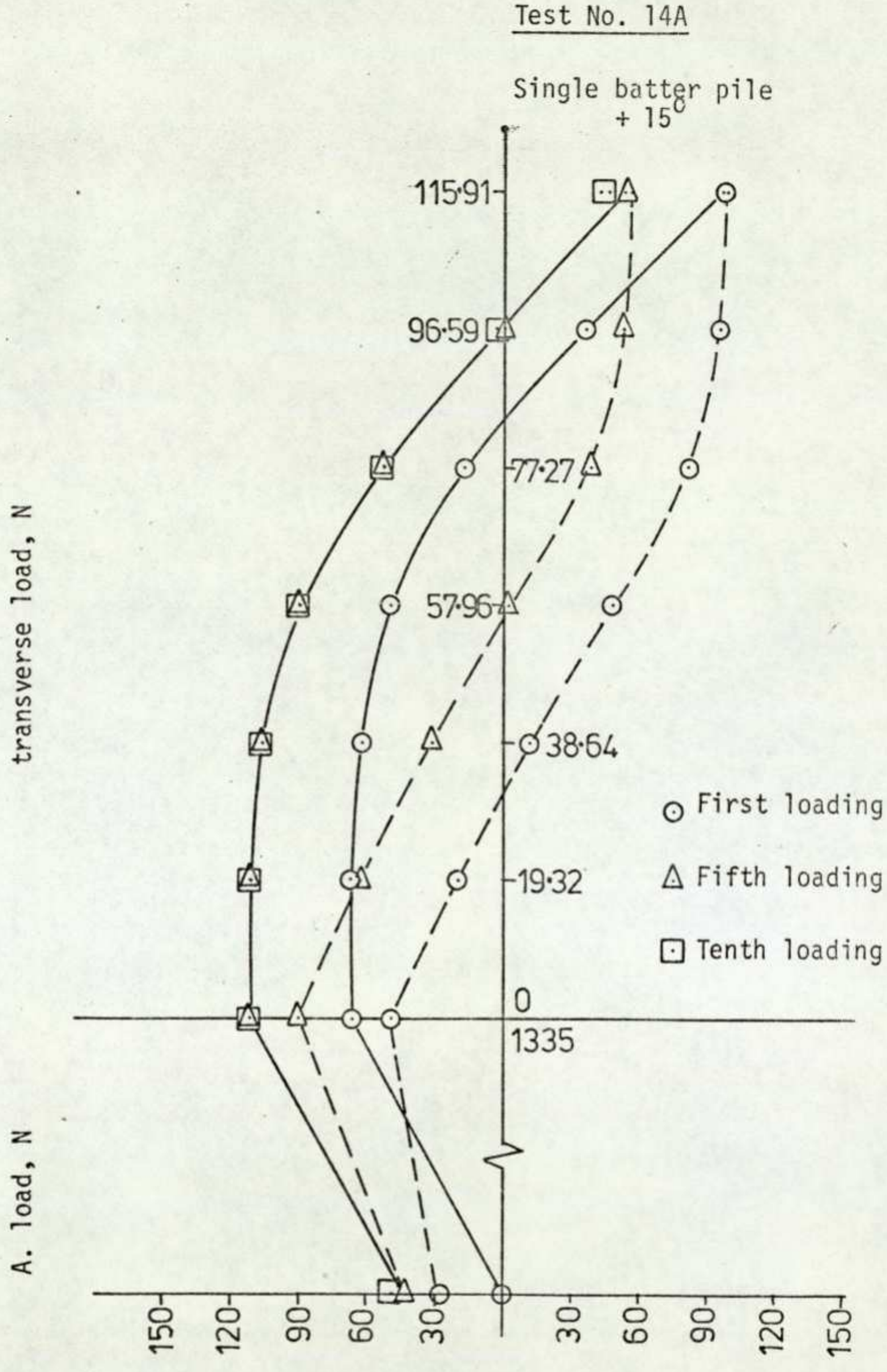


Fig. 7.11 A. & T. loads Vs Transverse displacements

Test No. 14A

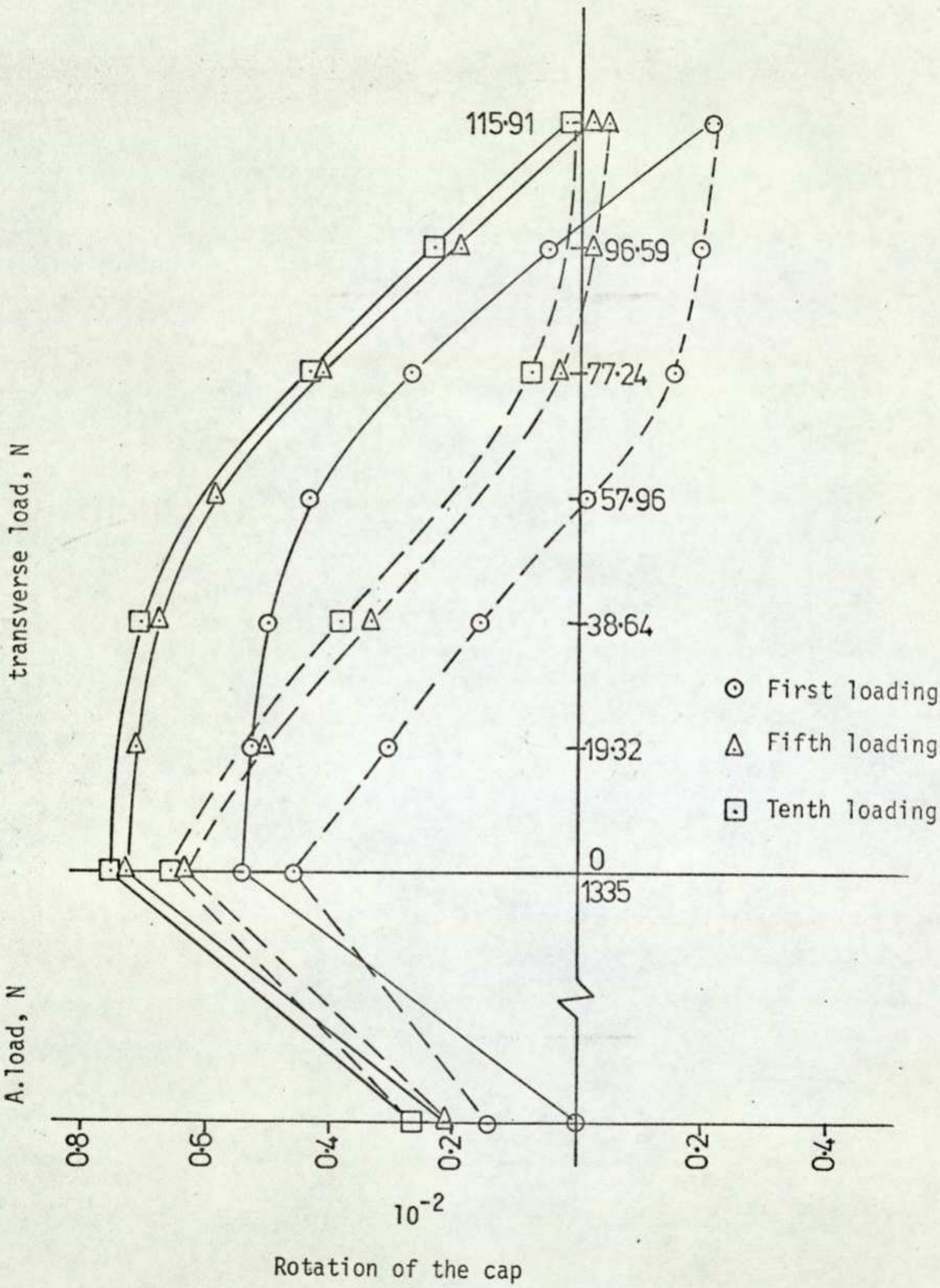


Fig. 7.12 Load Vs Rotation of the Cap

Test No. 14B

Single batter pile

-15^0

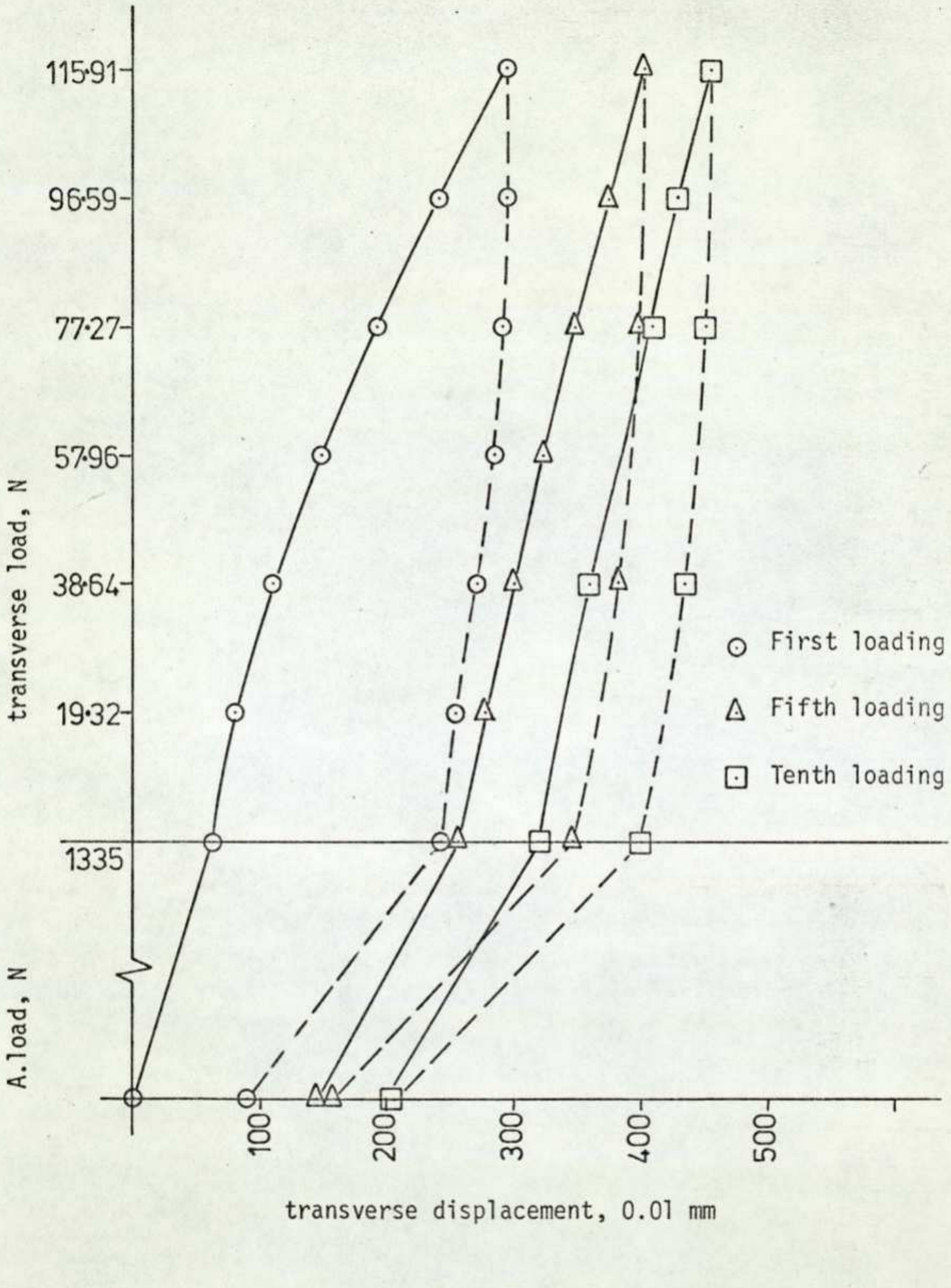


Figure 7.13 A. & T. loads Vs Transverse displacement

Test No. 14B

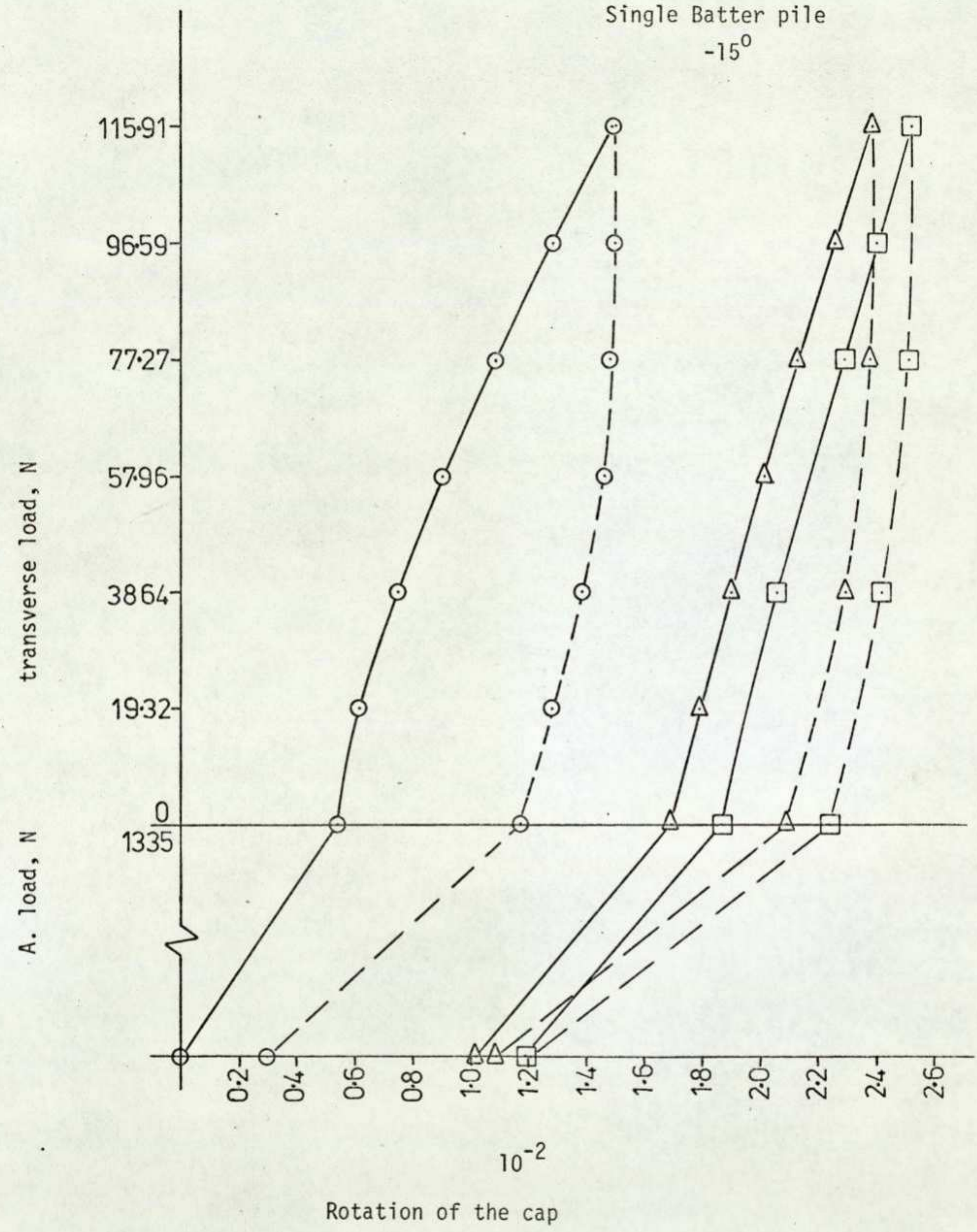


Fig. 7.14 load Vs Rotation of the Cap

Test No. 14C

Single batter pile

+ 30^0

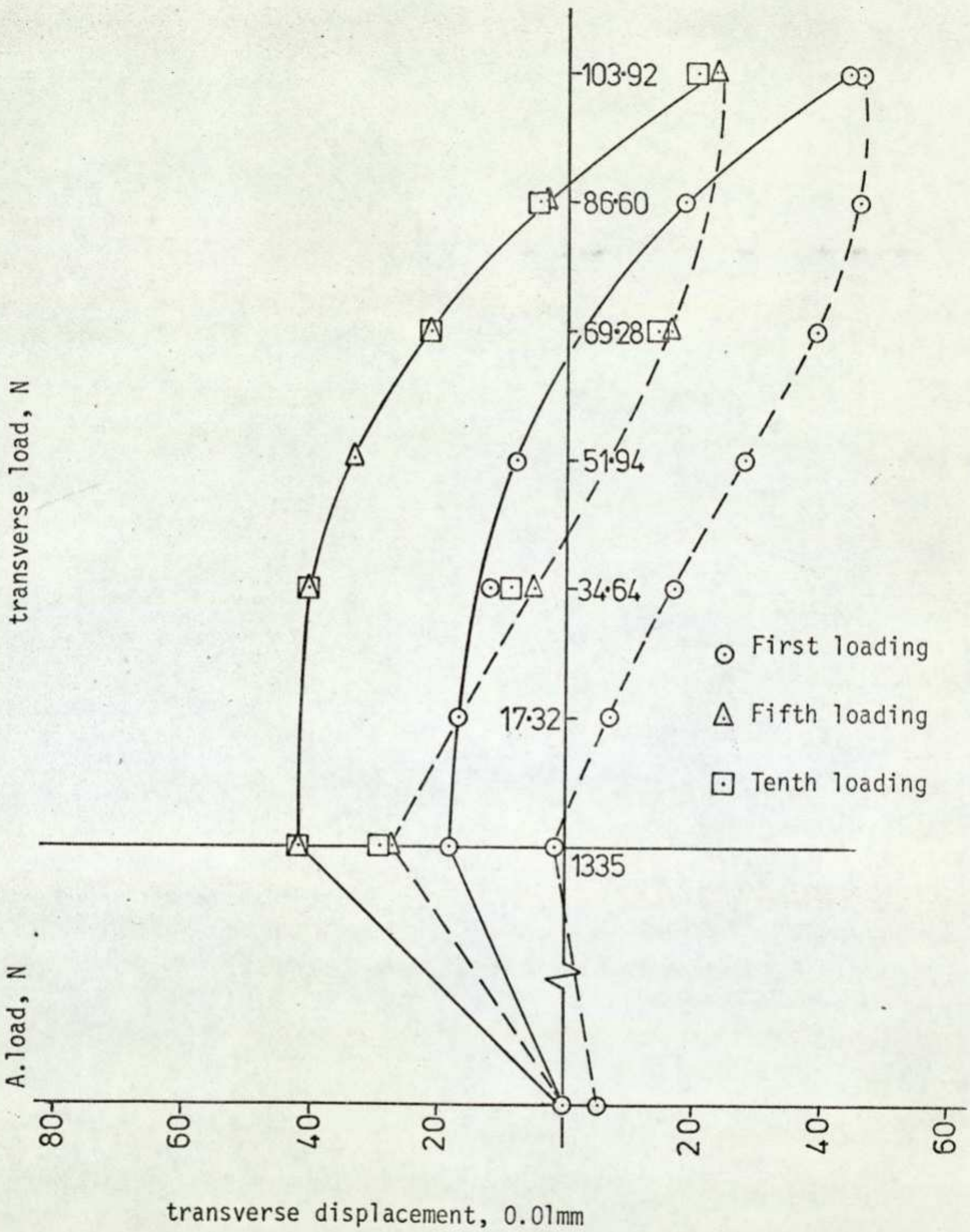


Fig. 7.15 A. & T. Loads Vs Transverse displacement

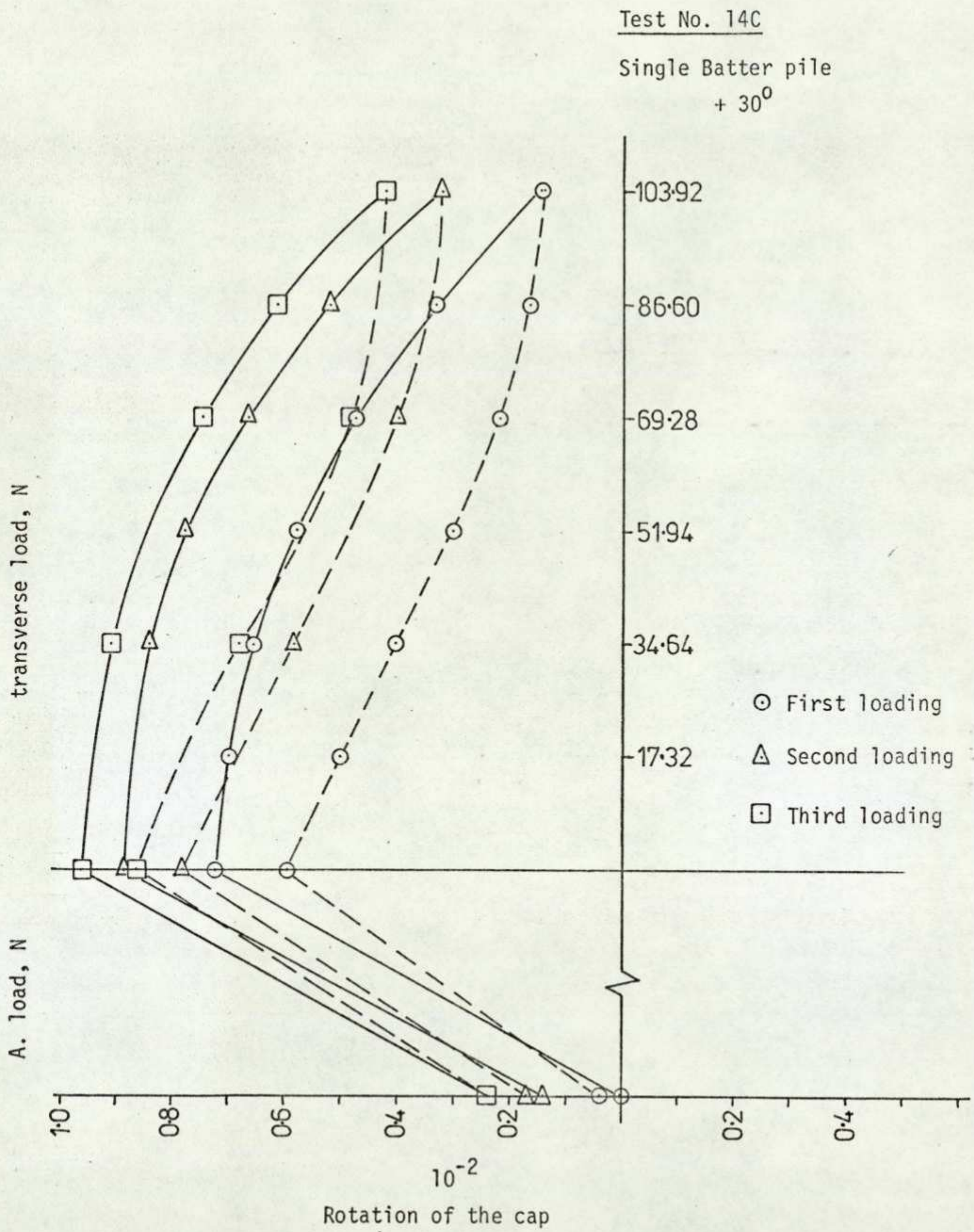


Fig. 7.16 load Vs Rotation of the Cap

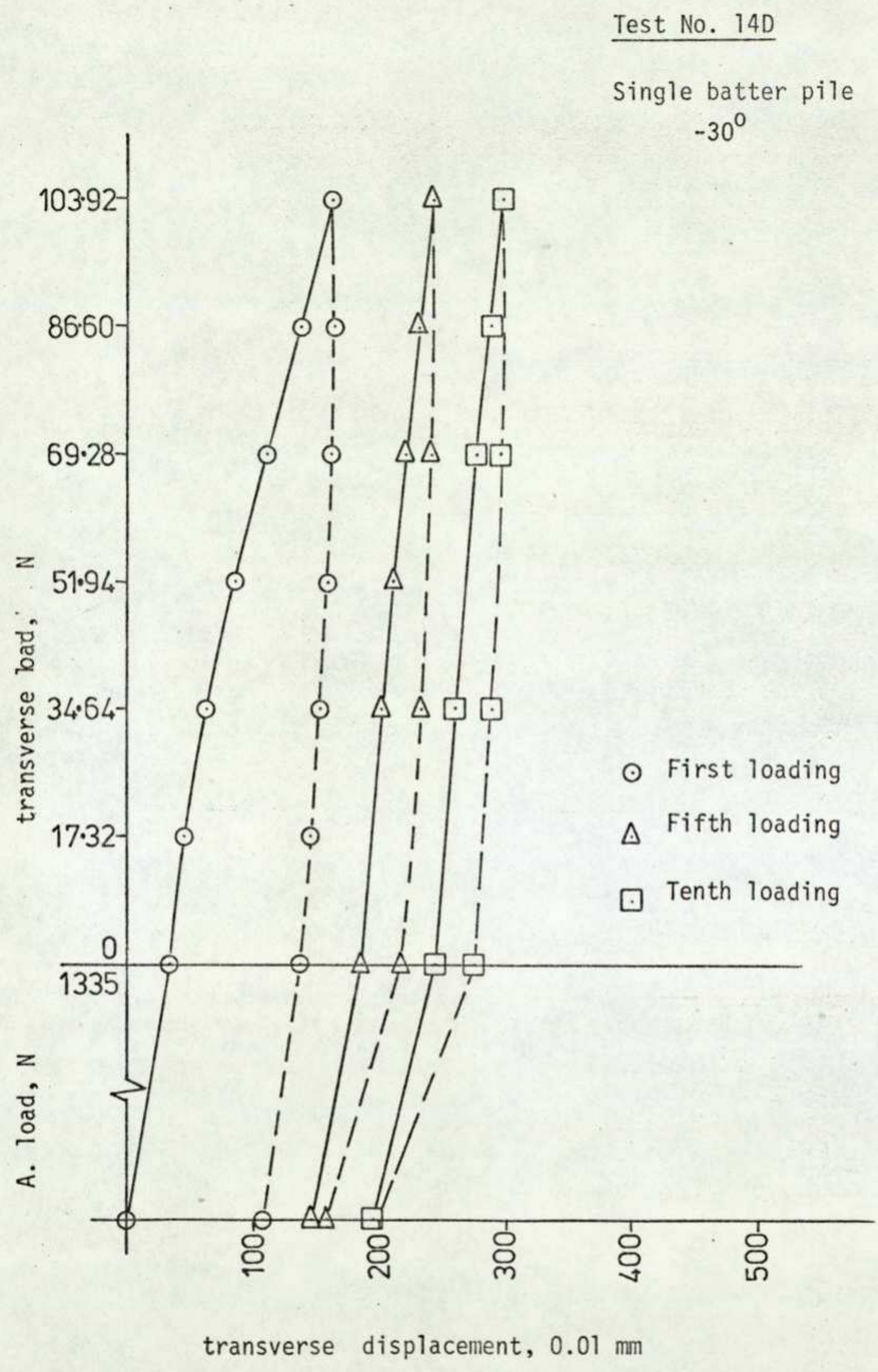


Fig. 7.17 A & T loads Vs Transverse displacement

Test No. 14D

Single Batter pile
-30^0

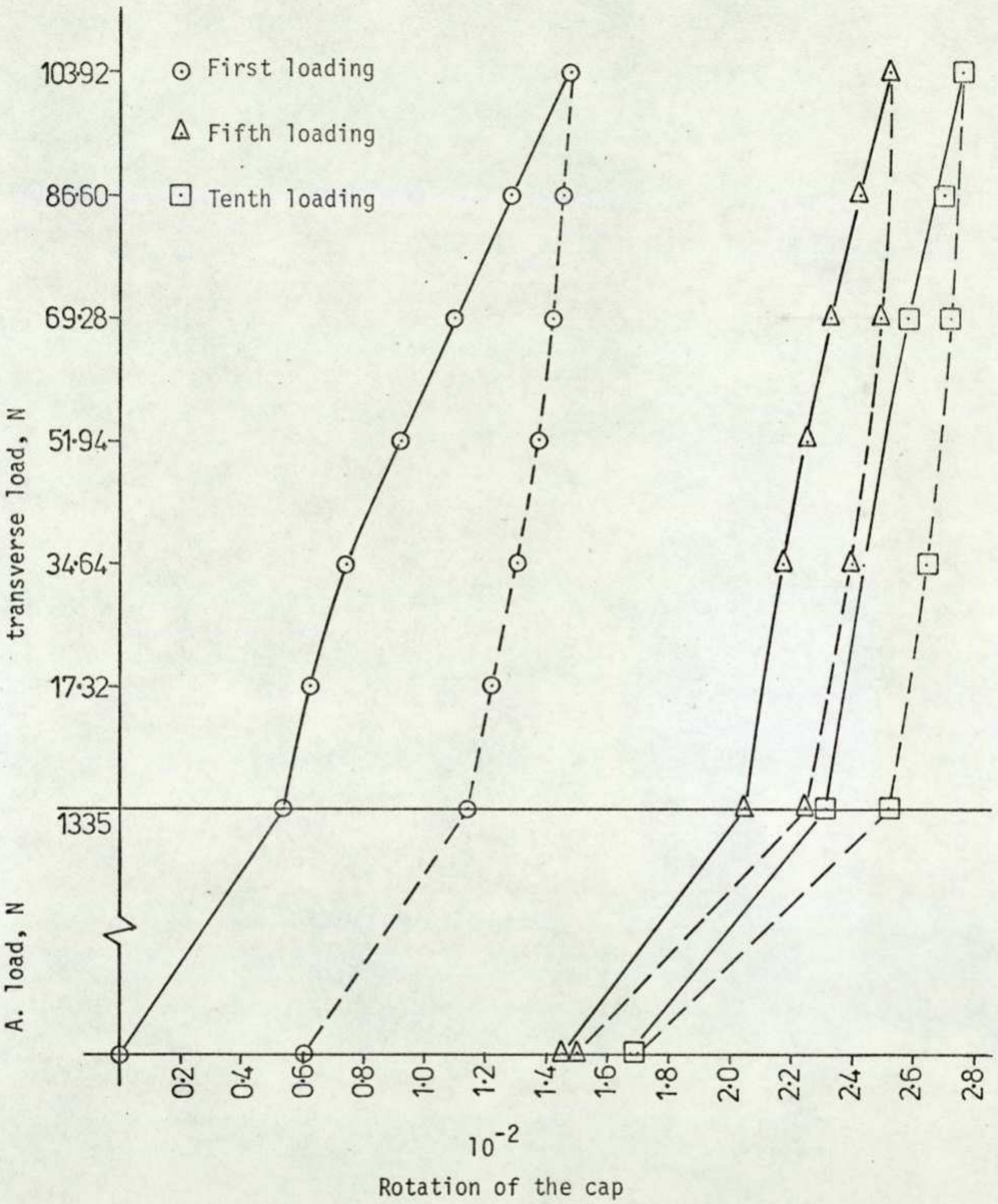


Fig. 7.18 load Vs Rotation of the cap

Single Vertical pile

T.Load 100N

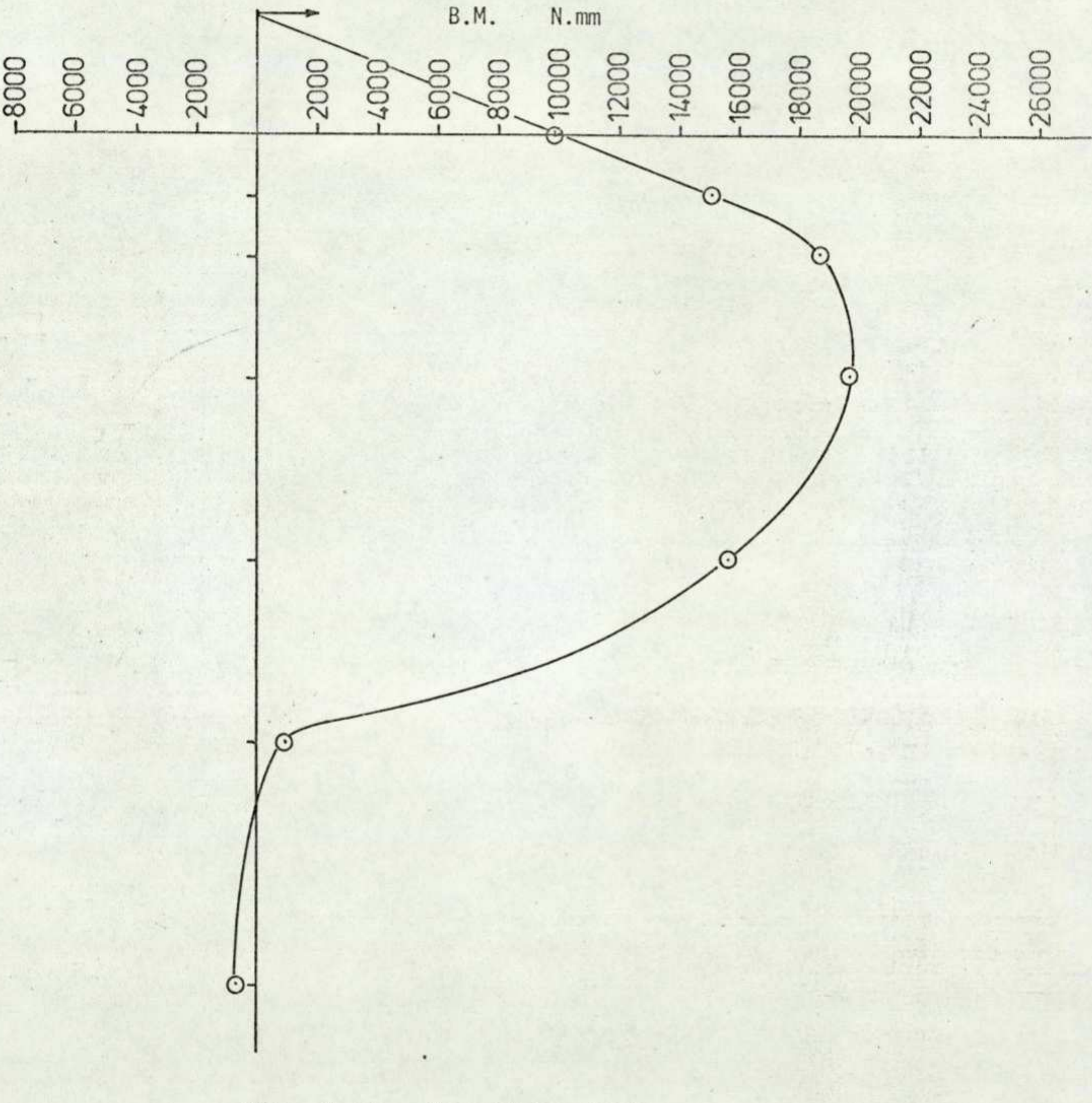


Fig. 7.19. Moment Vs Depth

Test No. 13A

Single Batter pile $\mp 15^{\circ}$

T.Load 96.59 N

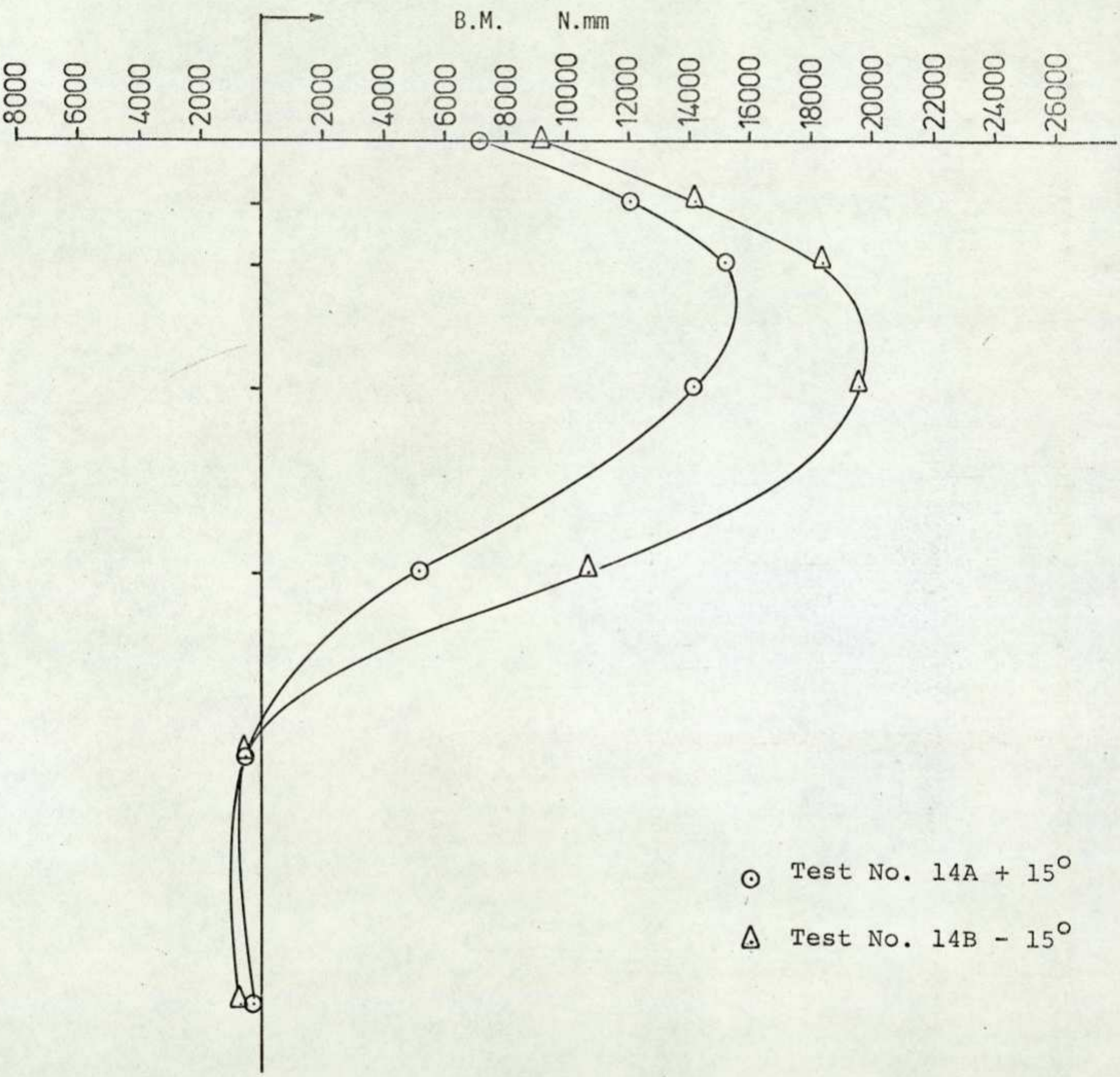


Fig. 7.20. Moment Vs Depth.

Test No. 14A, 14B

Single Batter pile ∓ 30°

T.Load 86.60 N

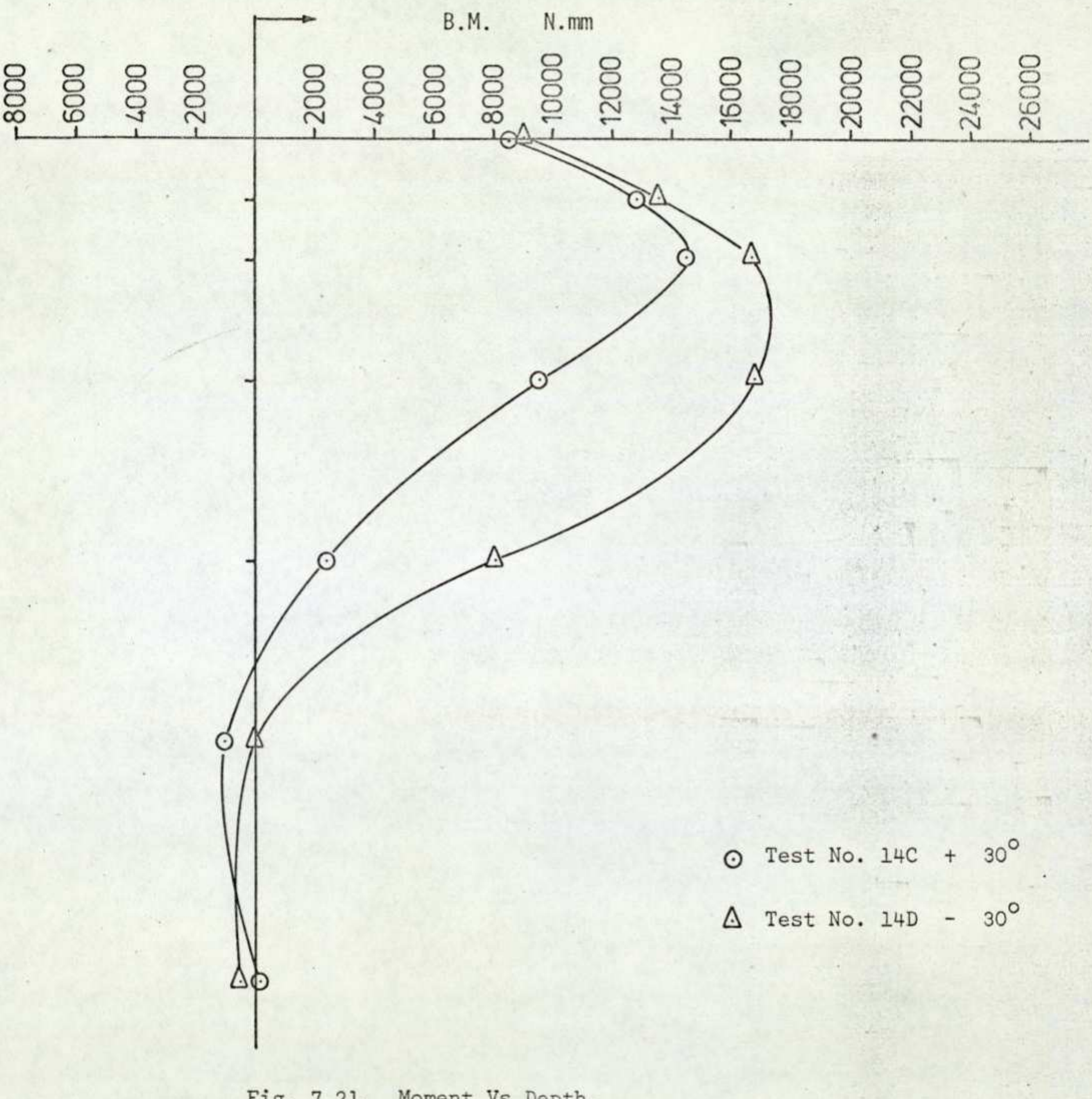


Fig. 7.21. Moment Vs Depth.

Test No. 14C, 14D

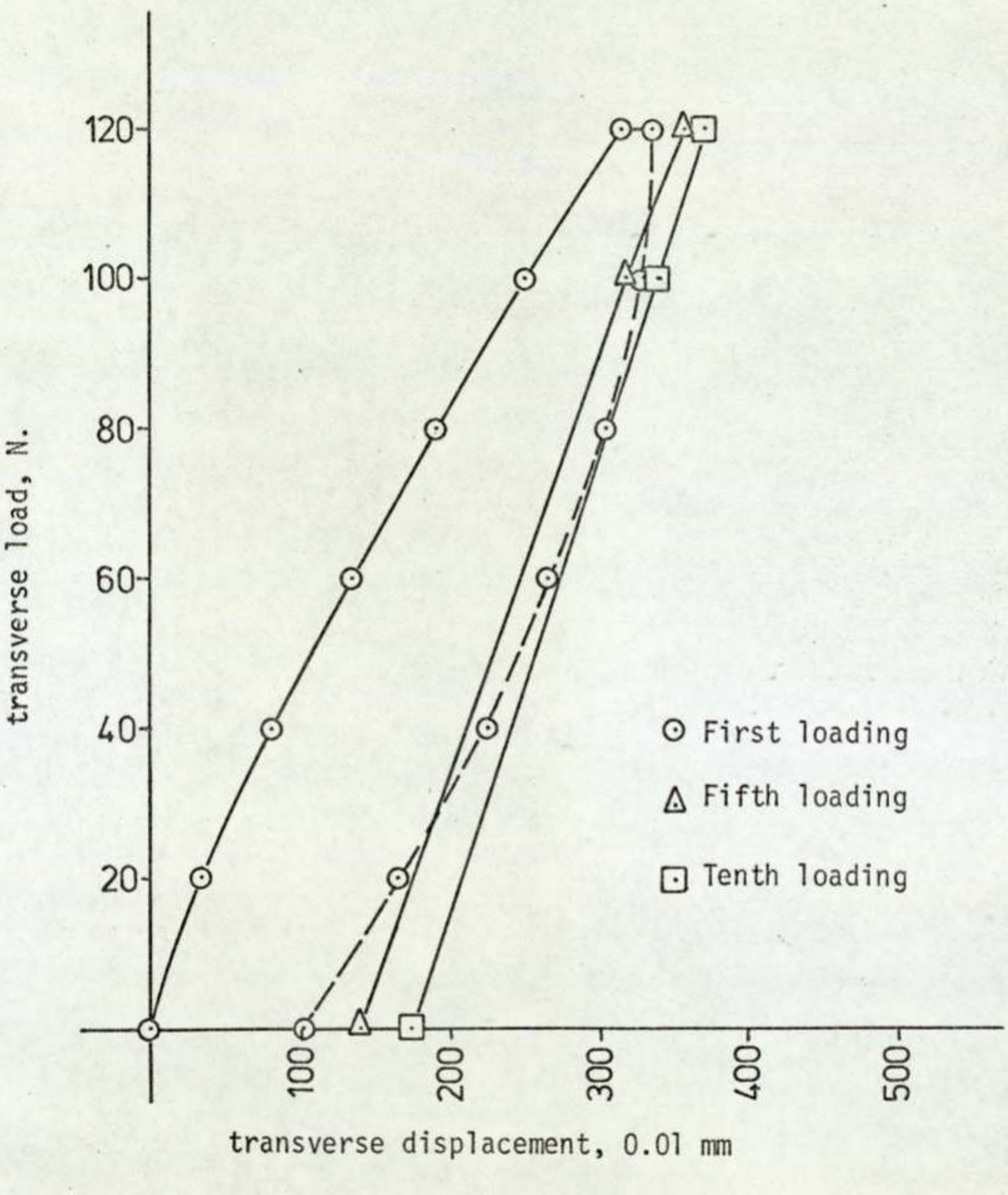


Fig. 7.22 Transverse load Vs Transverse displacement

Test No. 6A

Single Vertical Pile

Single Vertical pile

T.Load 100N

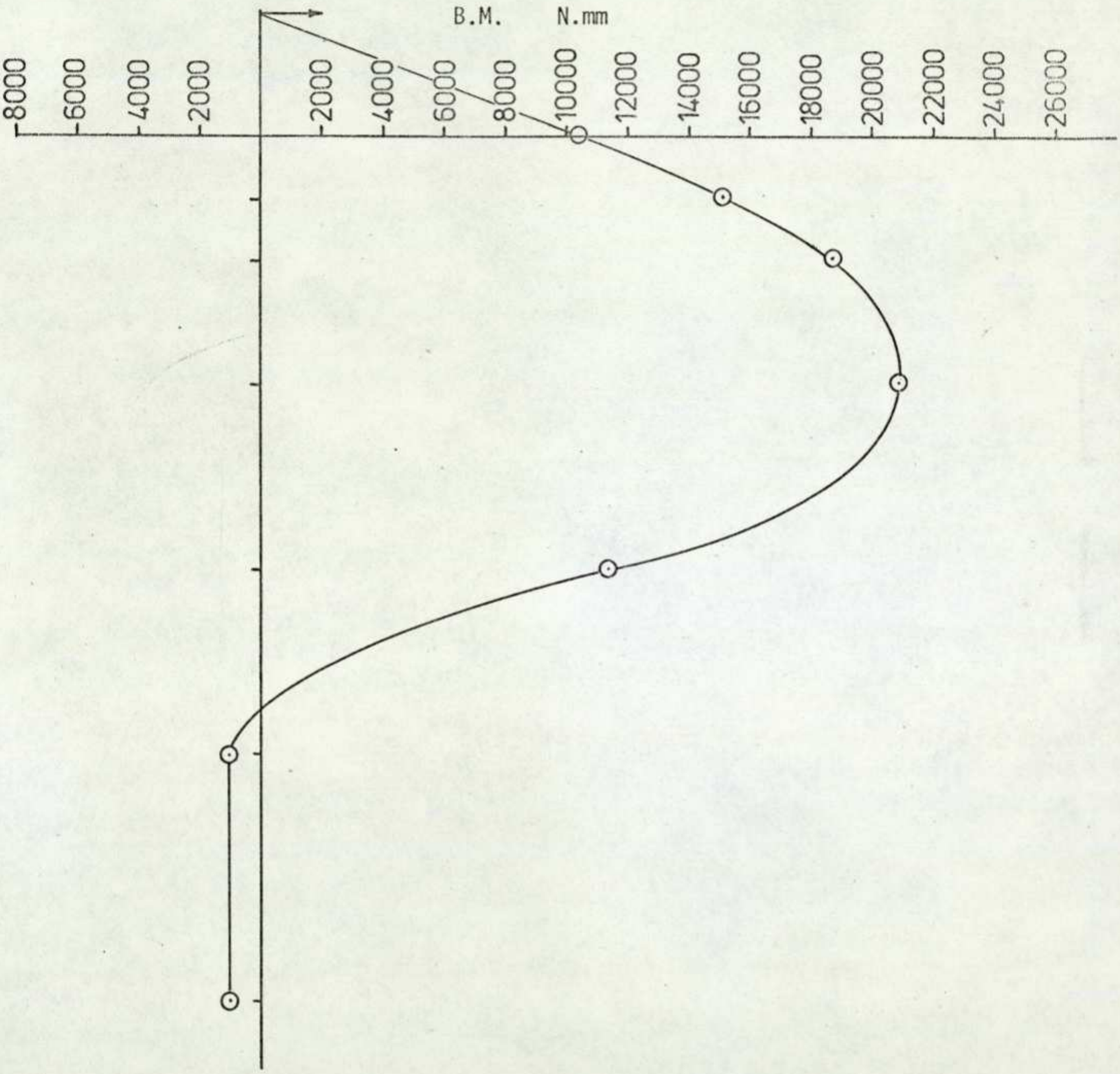


Fig. 7.23. Moment Vs Depth.

Test No. 6A

Single pile

T.Load 96.59 N + B 15°

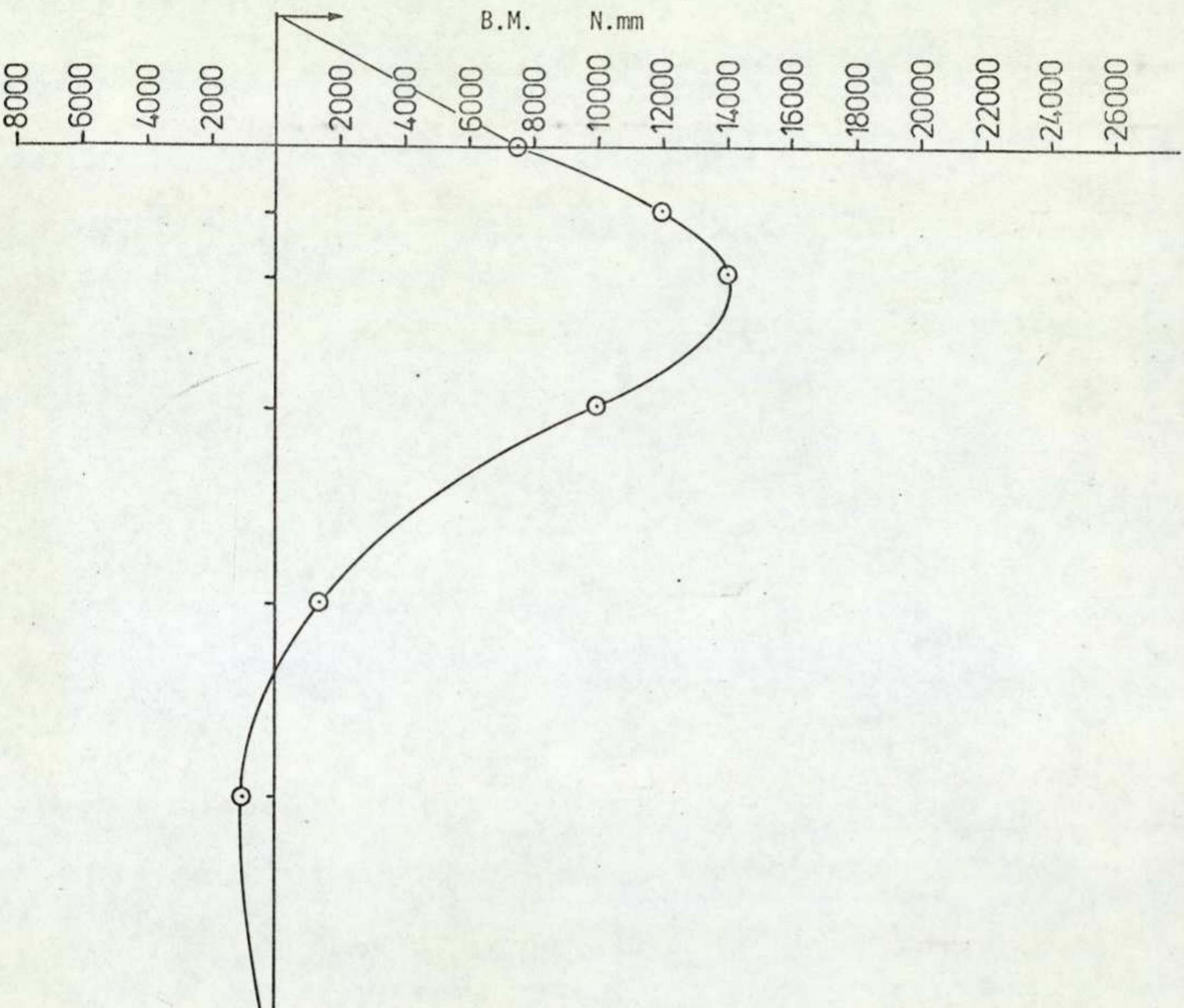


Fig. 7.24. Moment Vs Depth.

Test No. 8A

Single pile

T.Load 96.59 N - B 15°

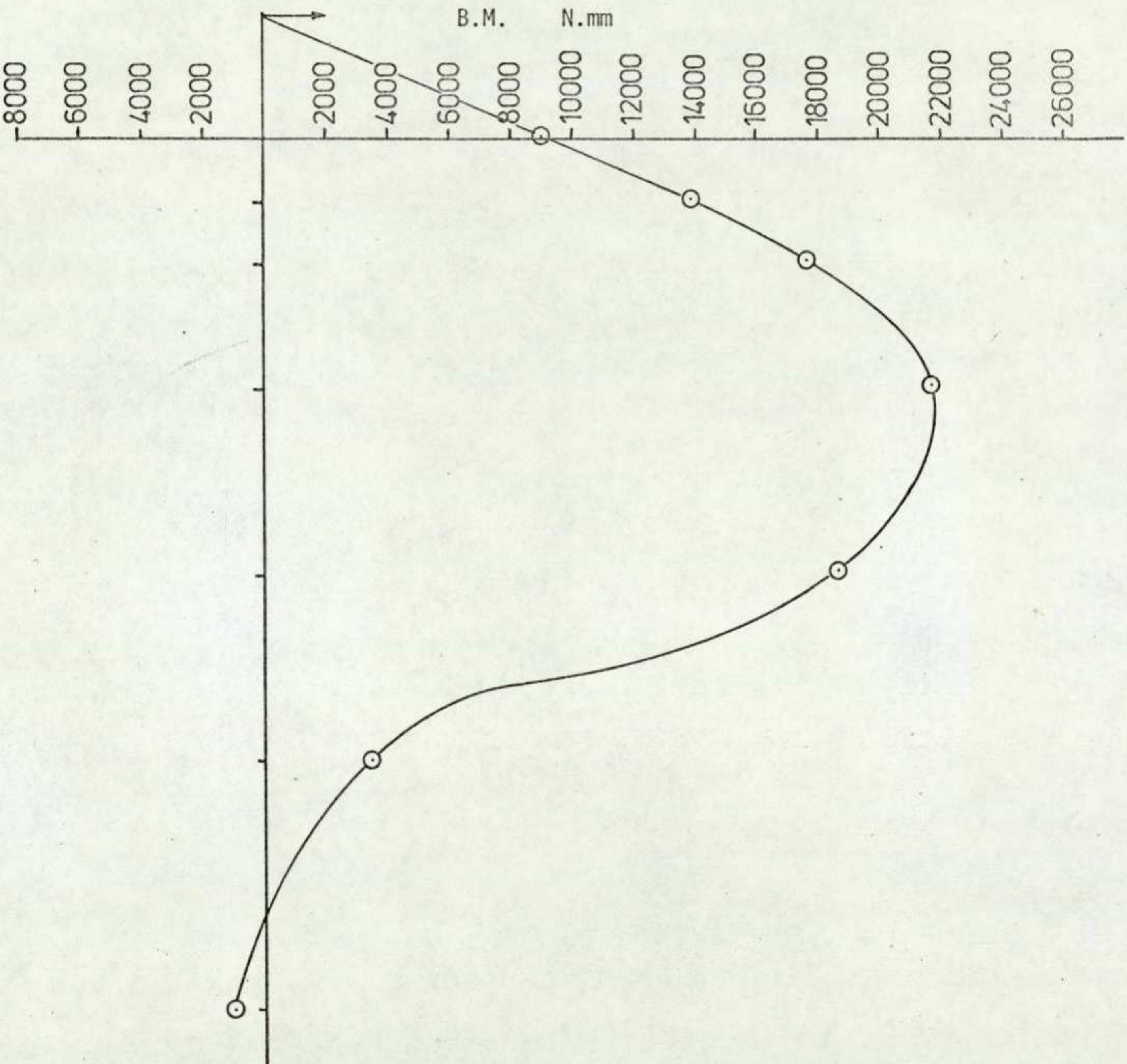


Fig. 7.25. Moment Vs Depth.

Test No. 9A

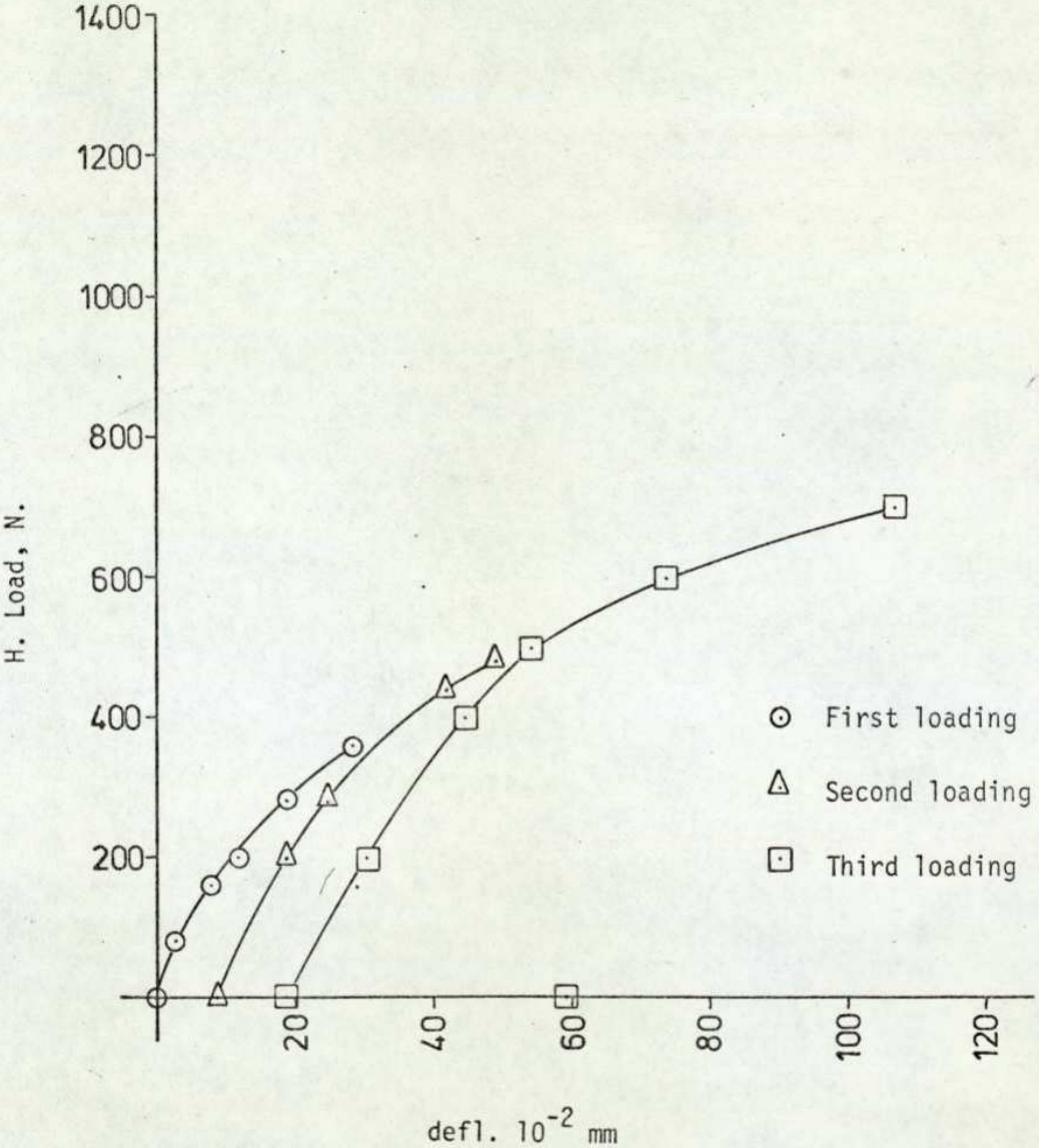


Fig. 7.26 load Vs defl. - pile group

Test No. 1

Pile Group (A,B,C,D)

Third loading

load 500N

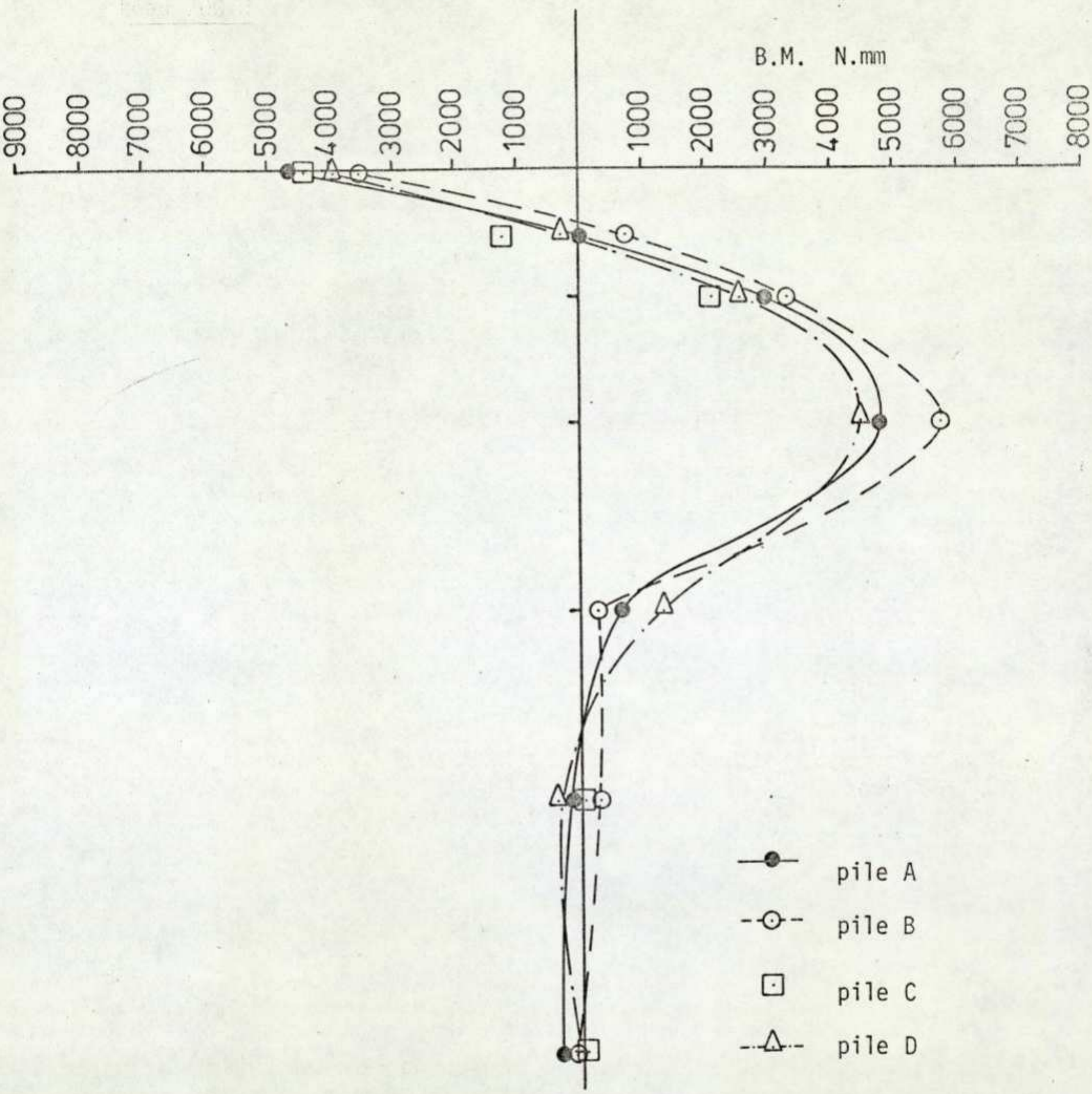


Fig. 7.27 Moment Vs Depth. - Test No. 1

Pile Group (A, B, C, D)

Third loading

H. Load, 700N

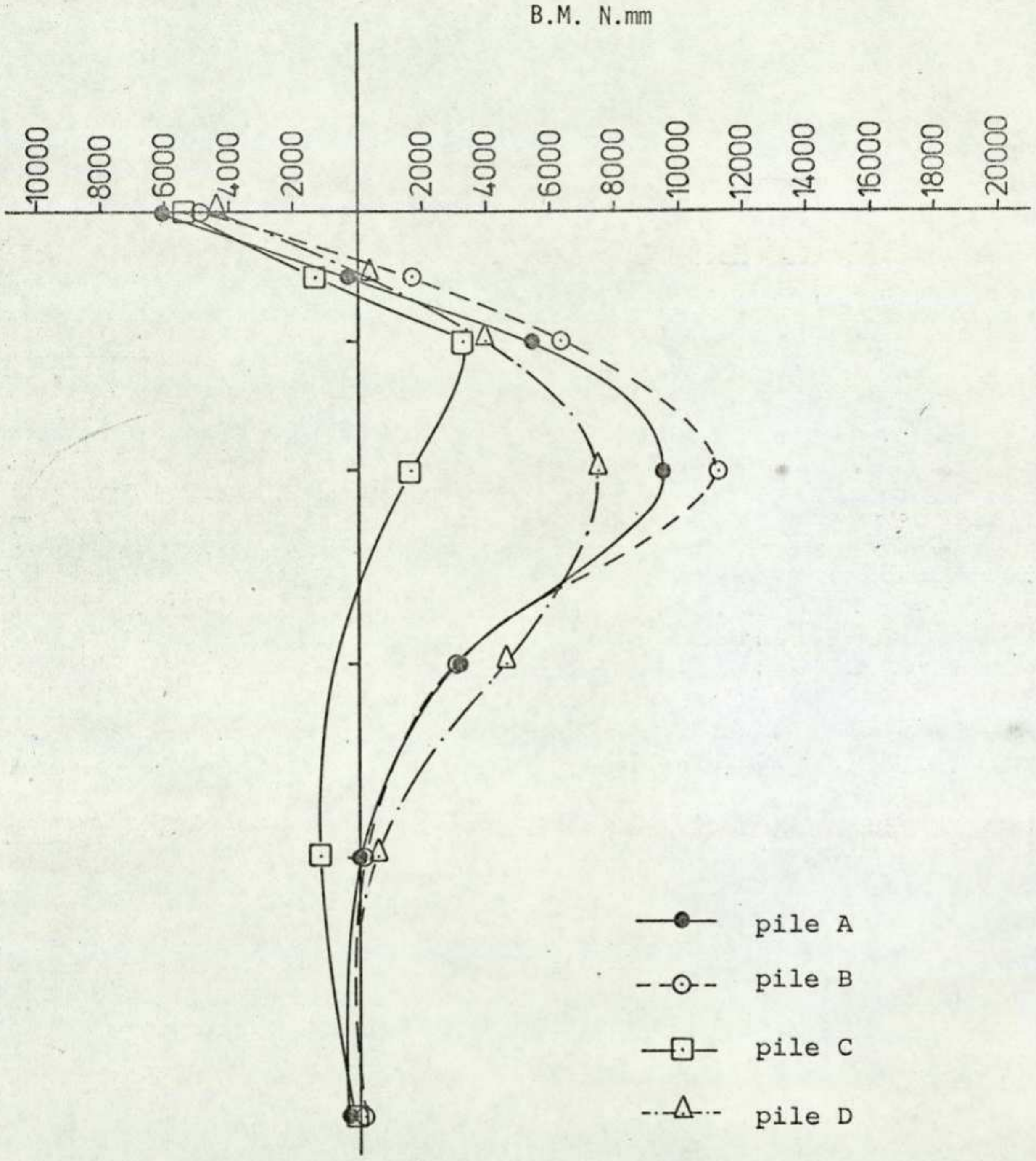


Fig. 7.28. Moment Vs Depth - TestNo. 1

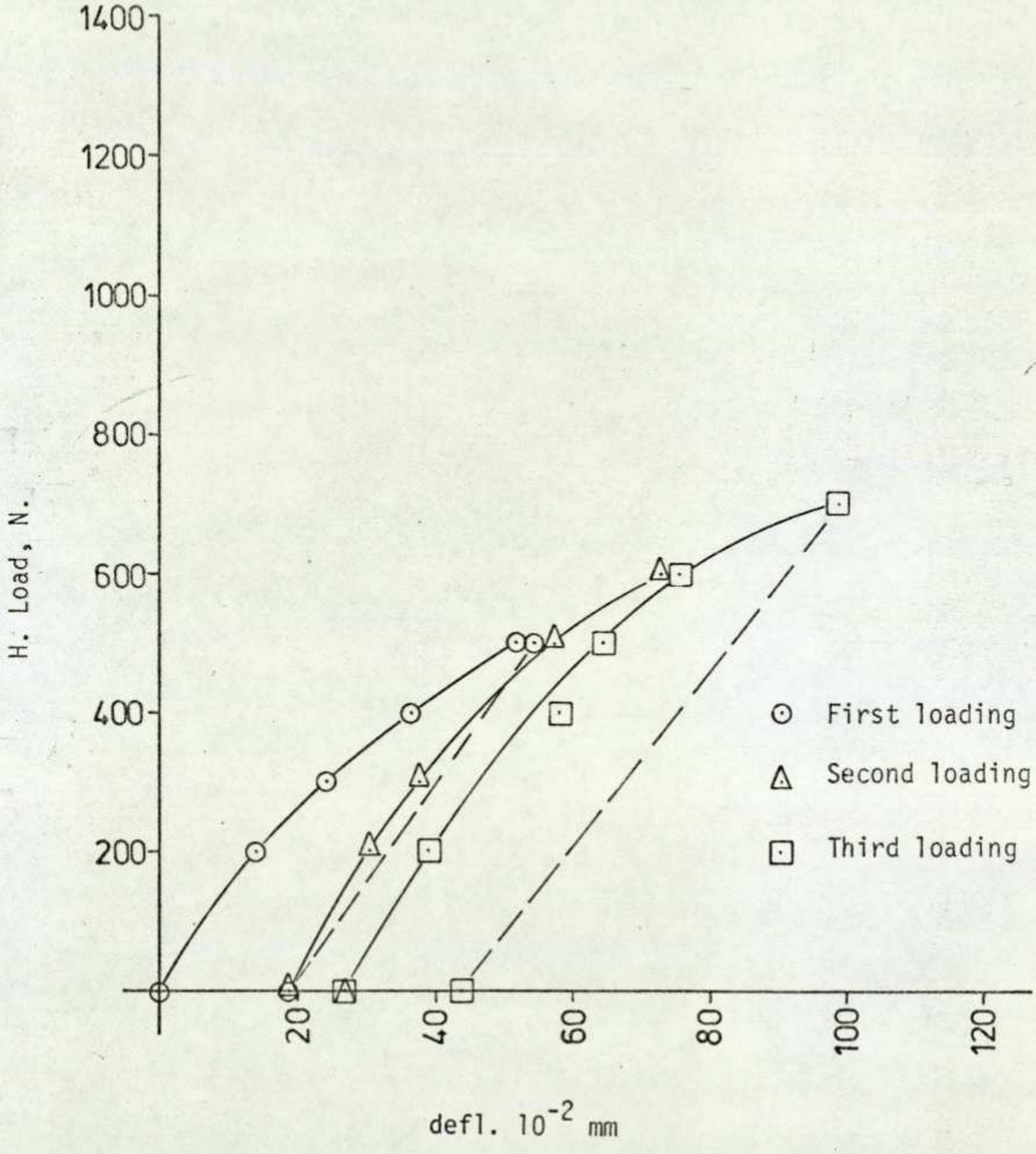


Fig. 7.29 Load Vs Defl. - pile group

Test No. 2

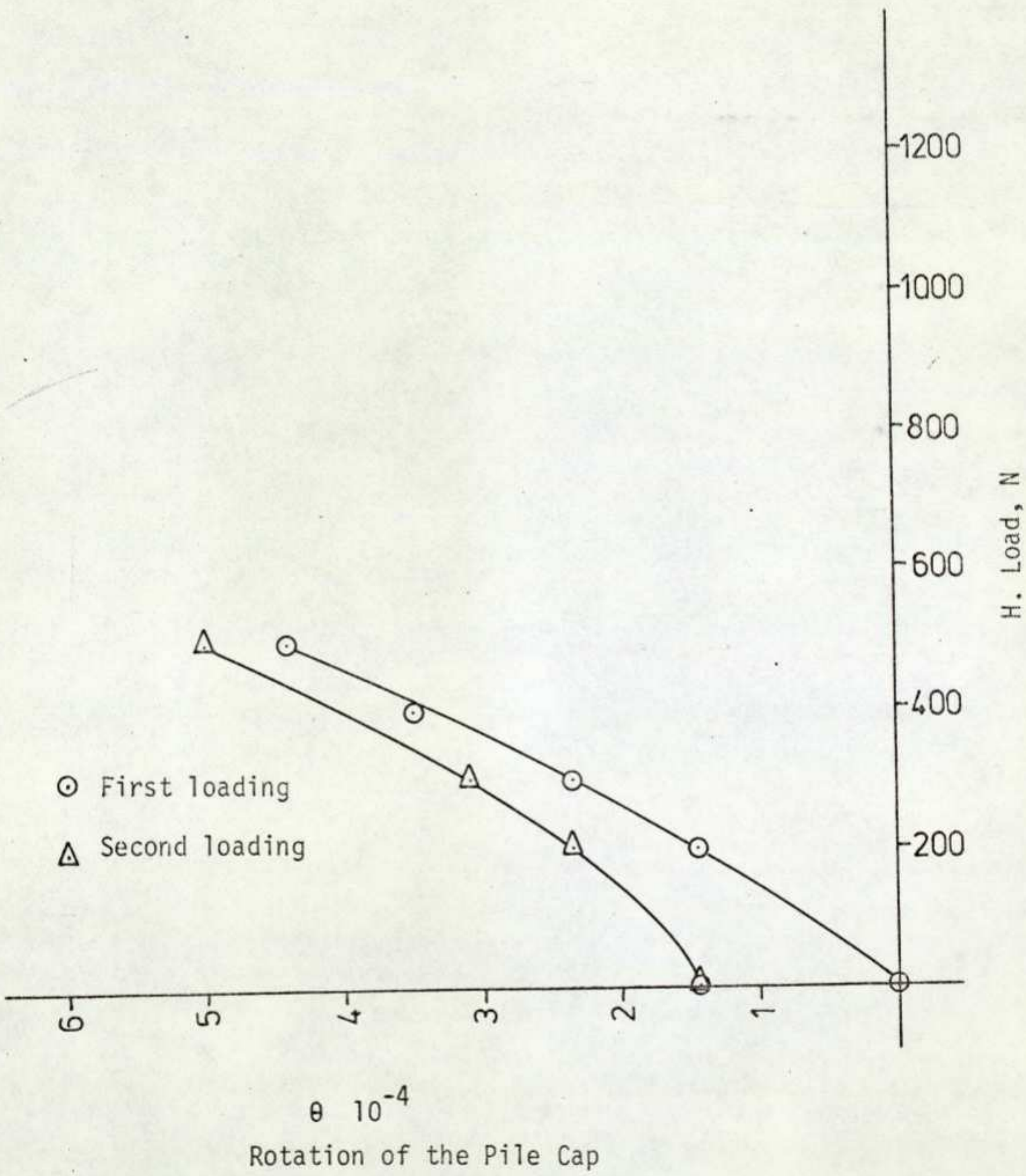


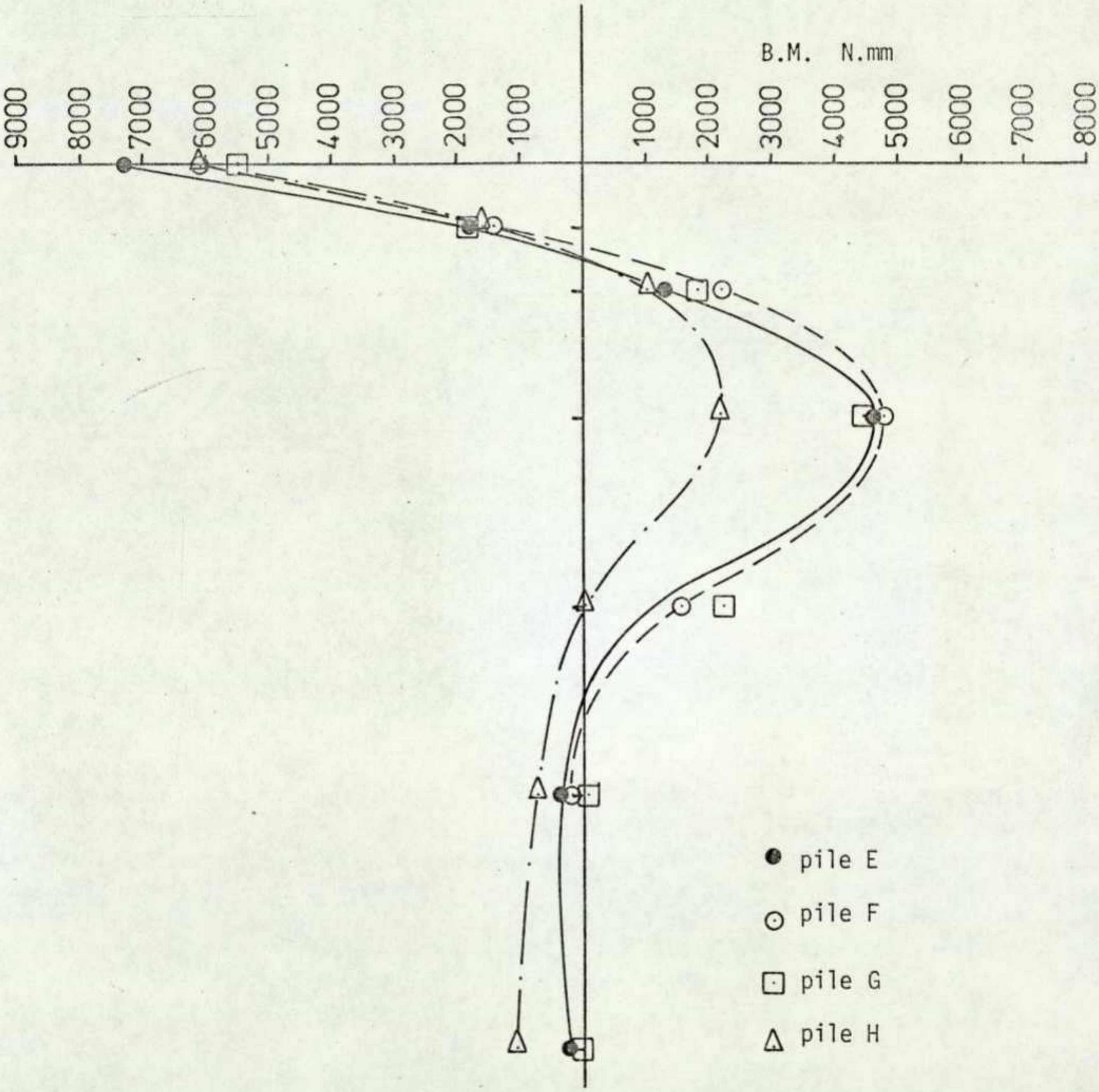
Fig. 7.30 Load Vs Rotation of the Pile Cap

Test No. 2

Pile Group (E,F,G,H)

load 500N

Third loading



Test No. 2

Fig. 7.31 Moment Vs Depth

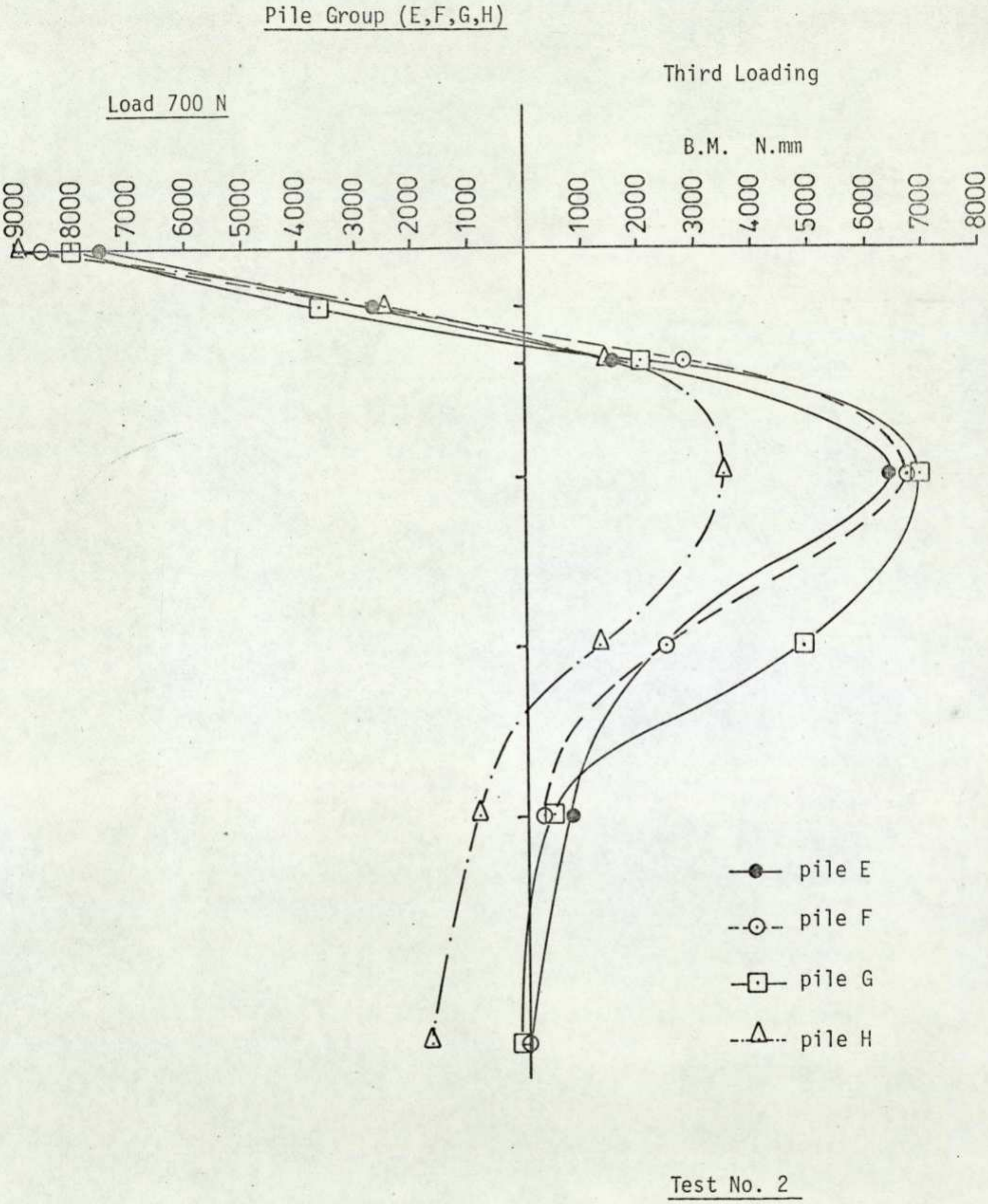


Fig. 7.32 Moment Vs Depth

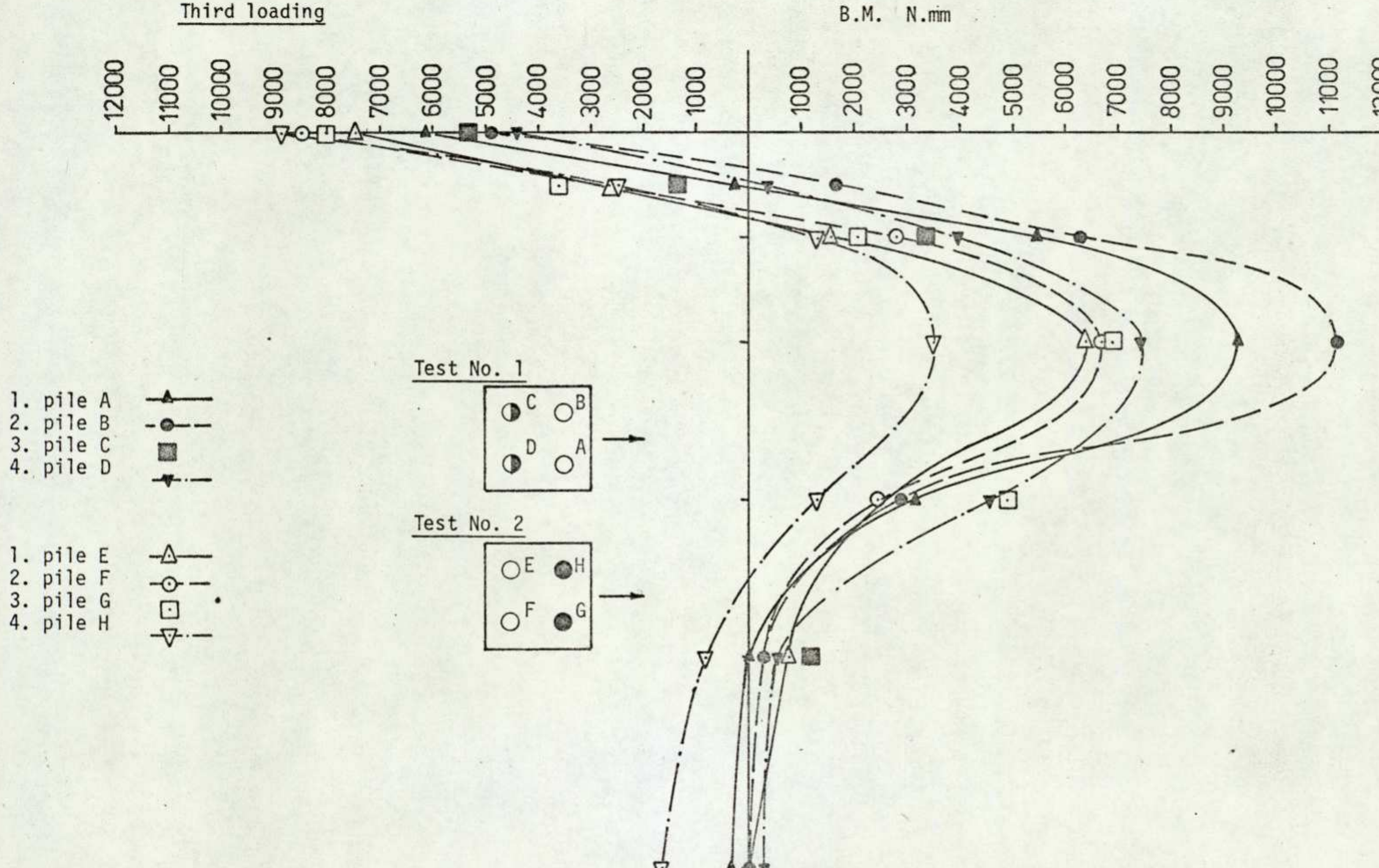


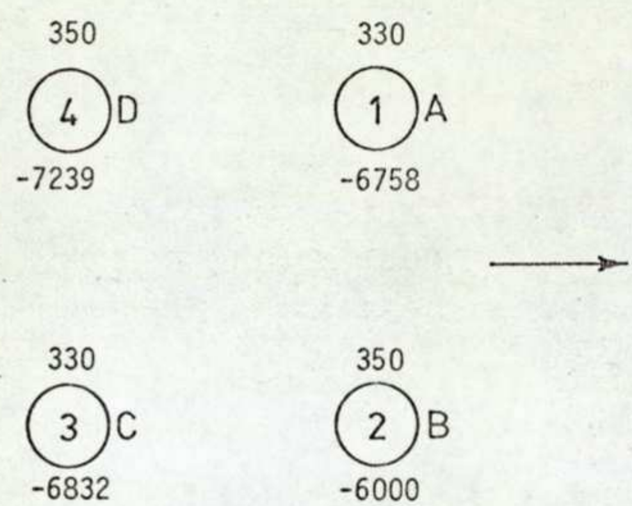
Fig. 7.32A Moment VS Depth

Test No. 1 Test No. 2

Test No. 8

2 x 2

+2B, -2B 15°



Driving Resistance, kg

Maximum negative bending moment, N.mm

A, B	+B	15°
C, D	-B	15°

Fig. 7.33 Driving resistance and maximum negative bending moment

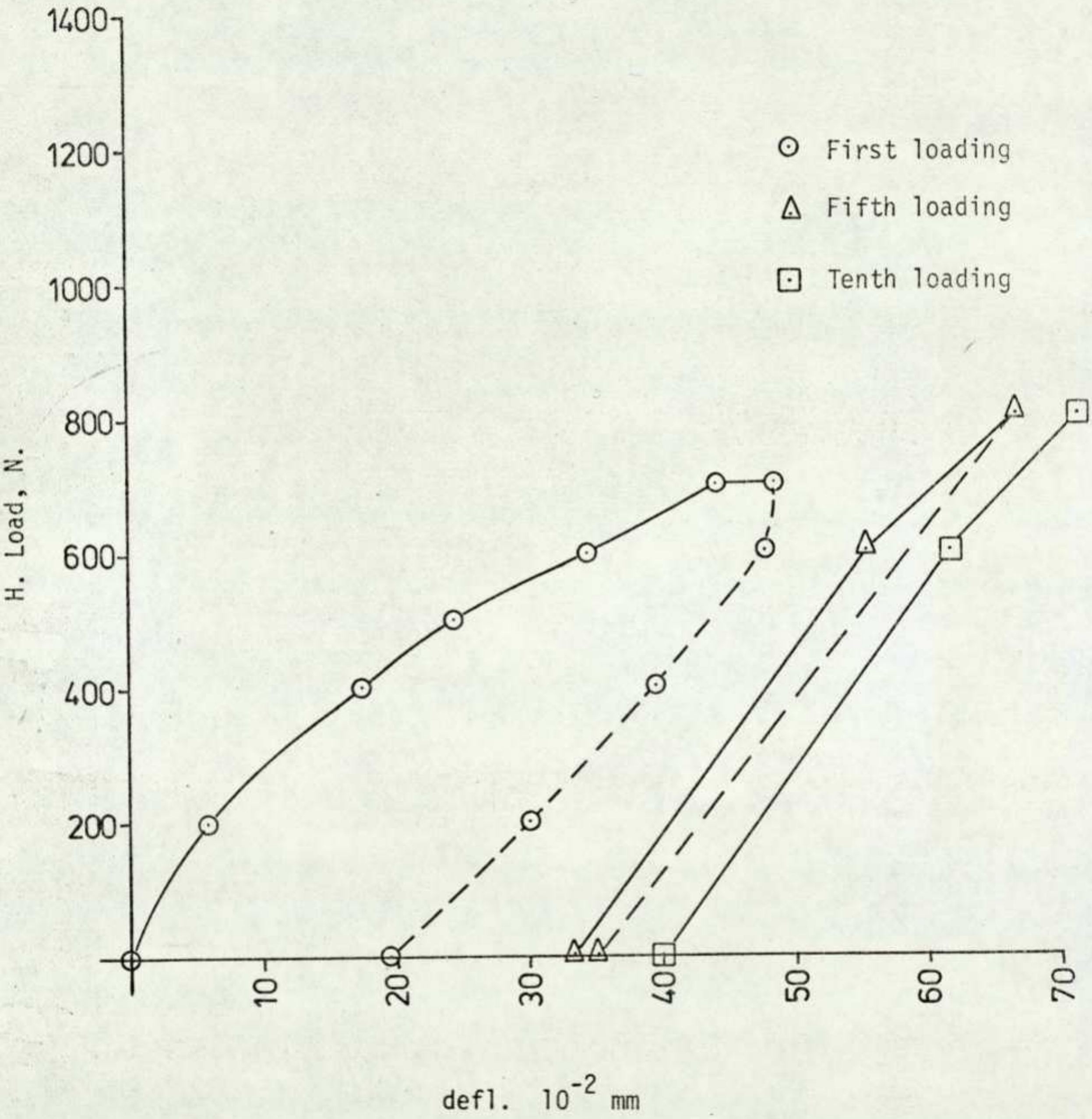


Fig. 7.34 Load Vs Defl. - Pile Group

Test No. 8

2 x 2
+2B 15^{o}
-2B 15o

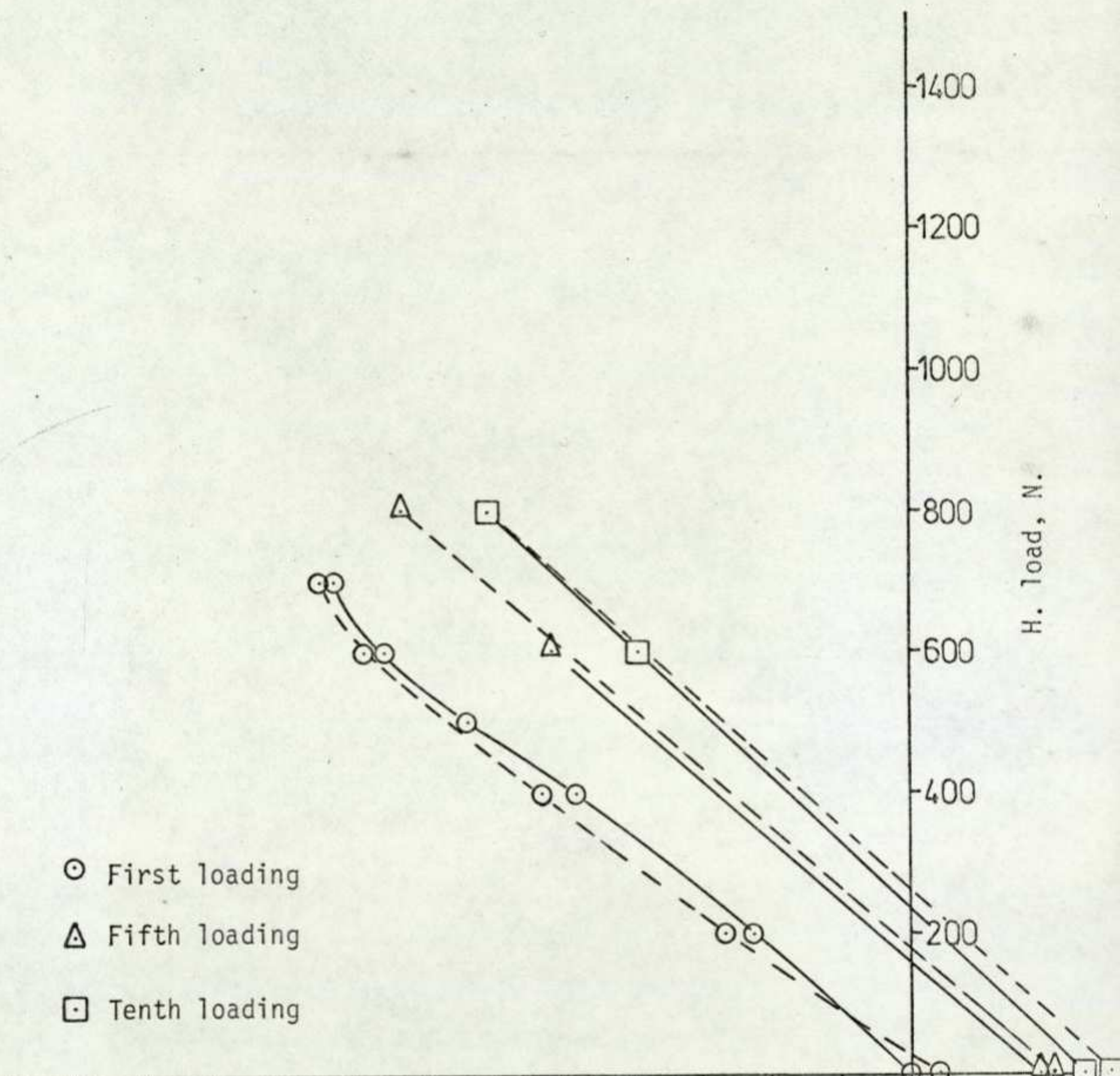


Fig. 7.35 Load Vs Rotation of the Cap

Pile Group

2 x 2

Test No. 8

+2B 15^0

-2B 15^0

Pile Group

B = 15°

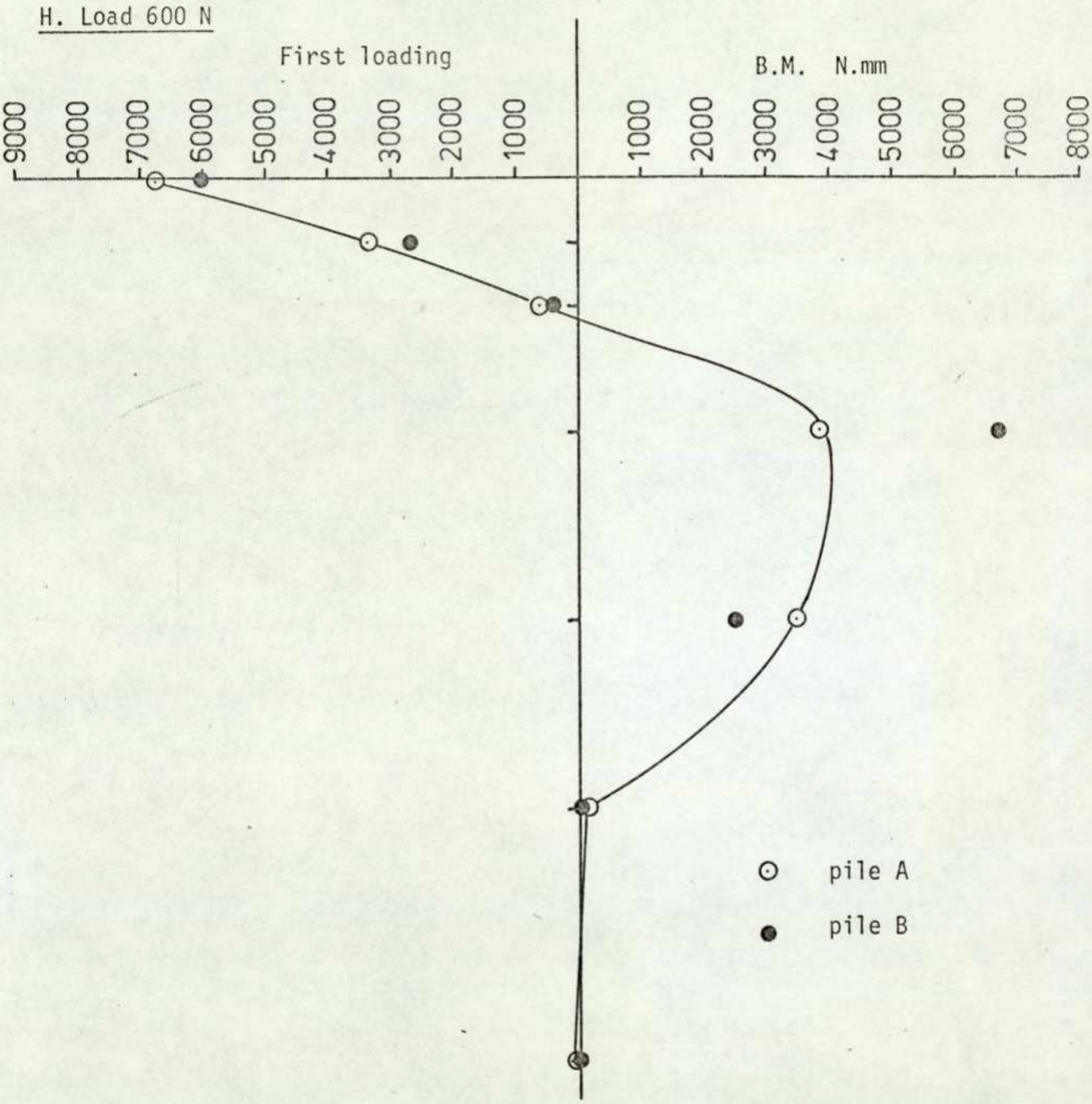


Fig. 7.36 Moment Vs Depth

Test No. 8

2 x 2

+2B 15°

-2B 15°

Pile Group

$B = 15^0$

H. Load 600 N

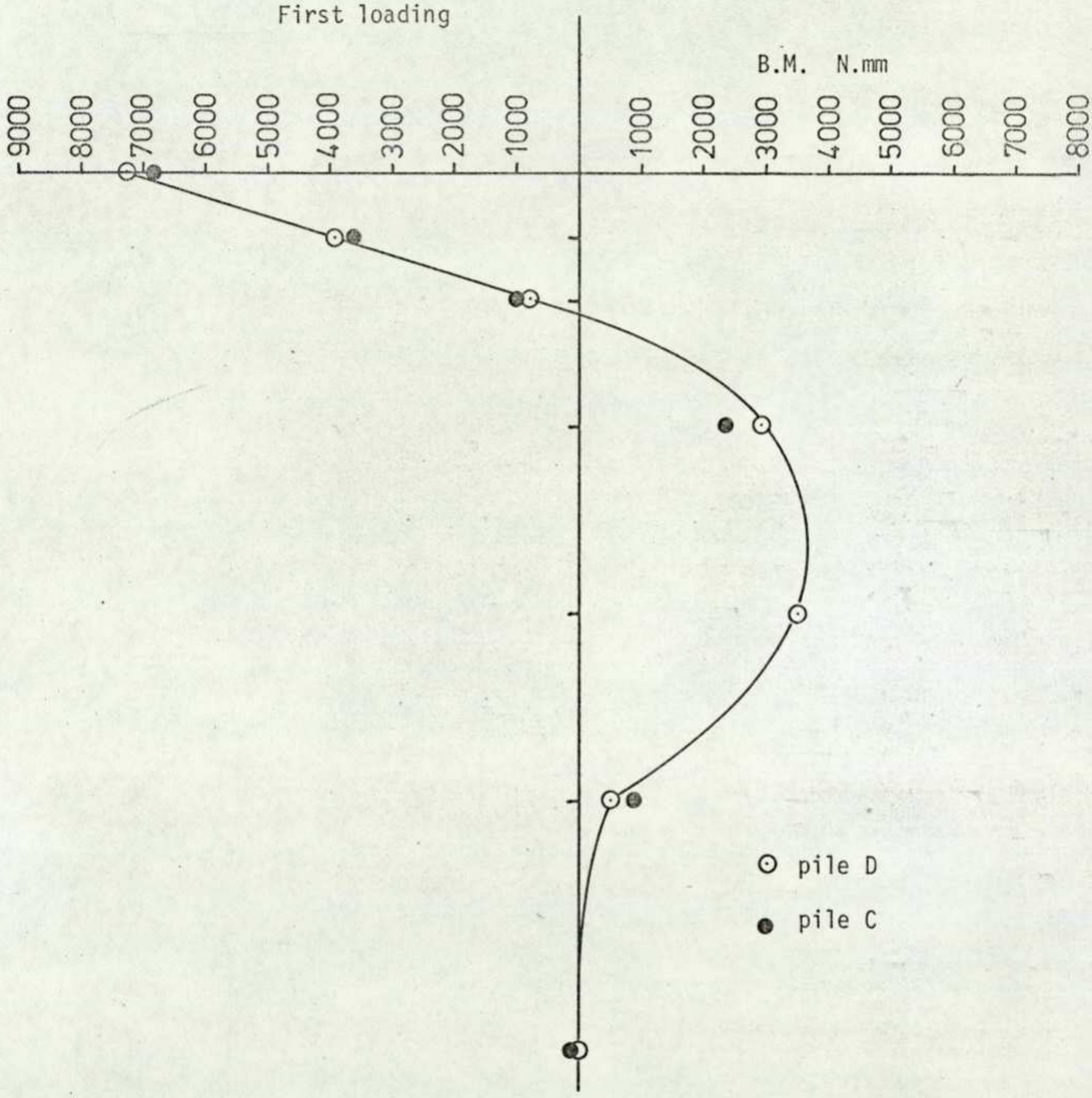


Fig. 7.37 Moment Vs Depth

Test No. 8

2 x 2
+2B 15^0
-2B 15^0

Test No. 9

2 x 2
4

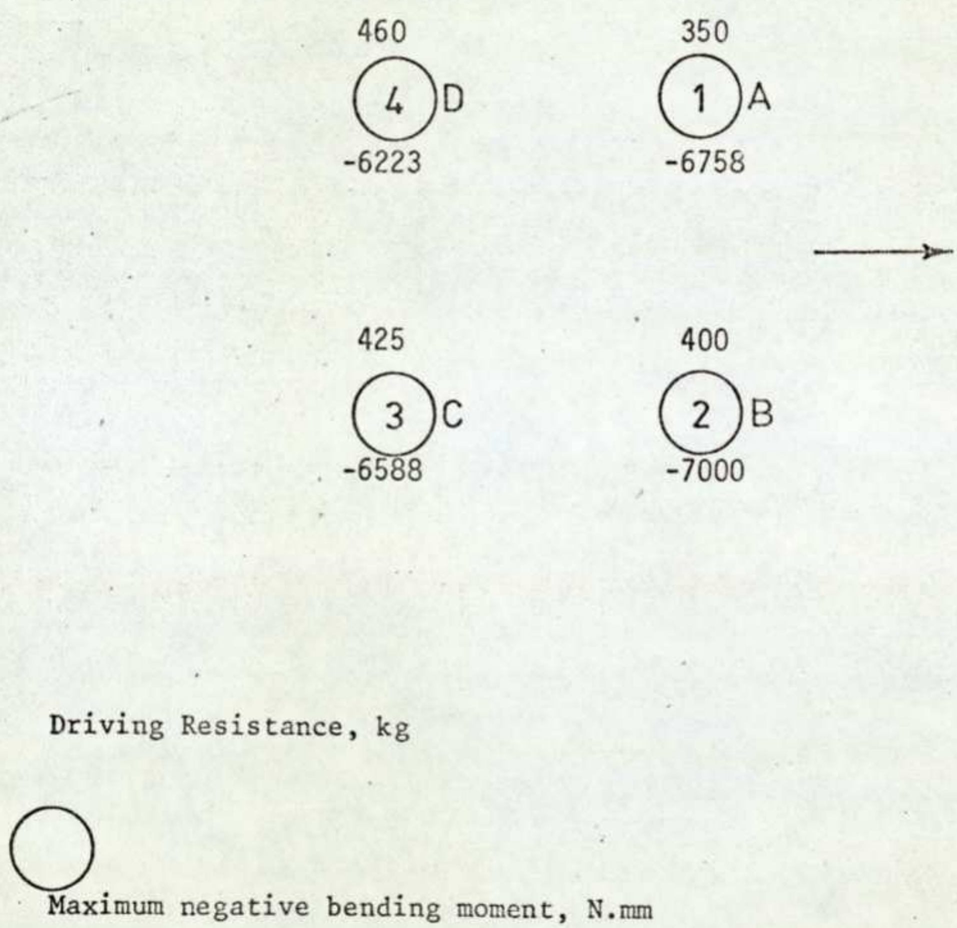


Fig. 7.38 Driving resistance and maximum negative bending moment.

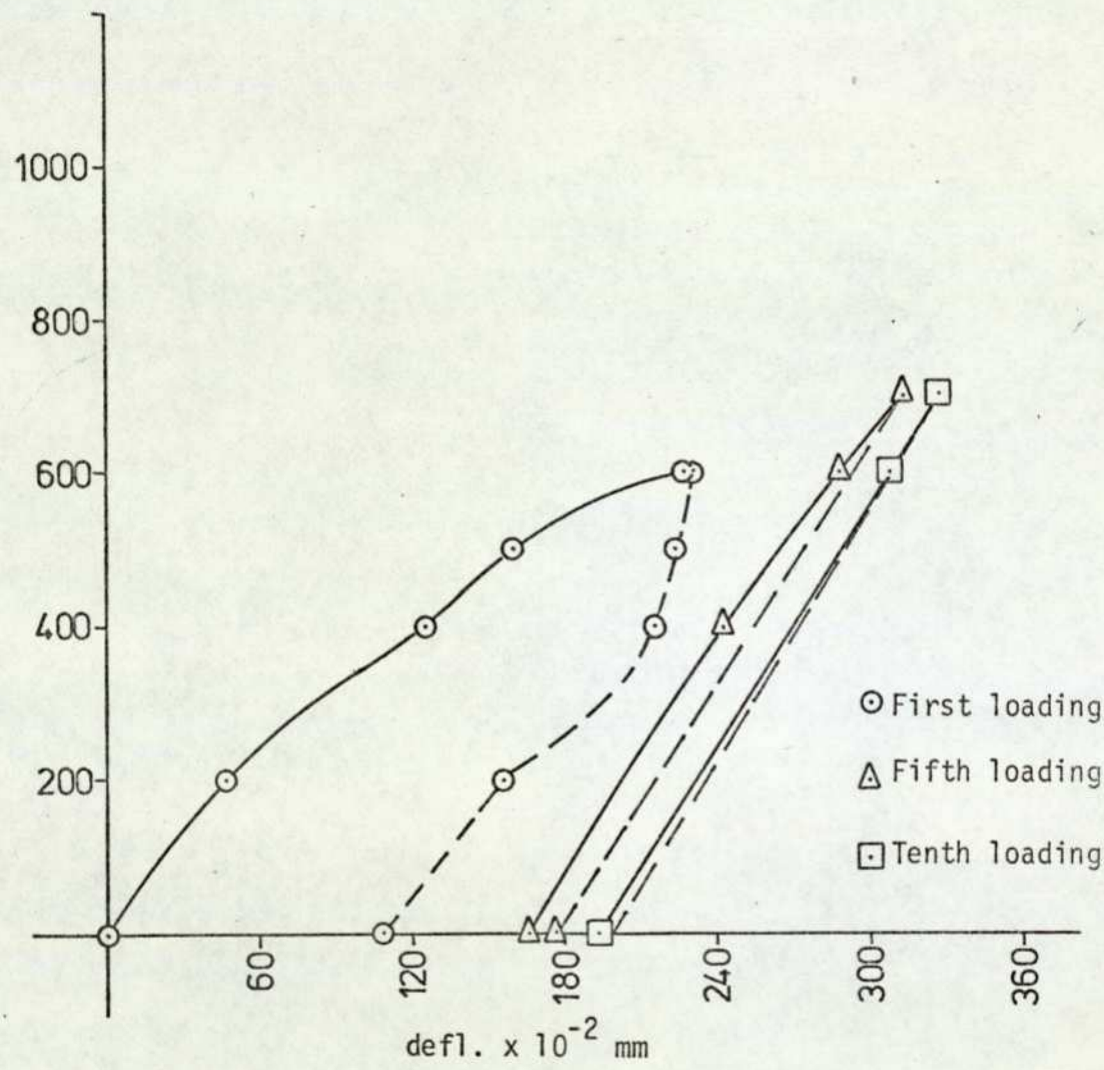


Fig. 7.39 Load Vs defl. - pile Group

Test No. 9

2 x 2
4V

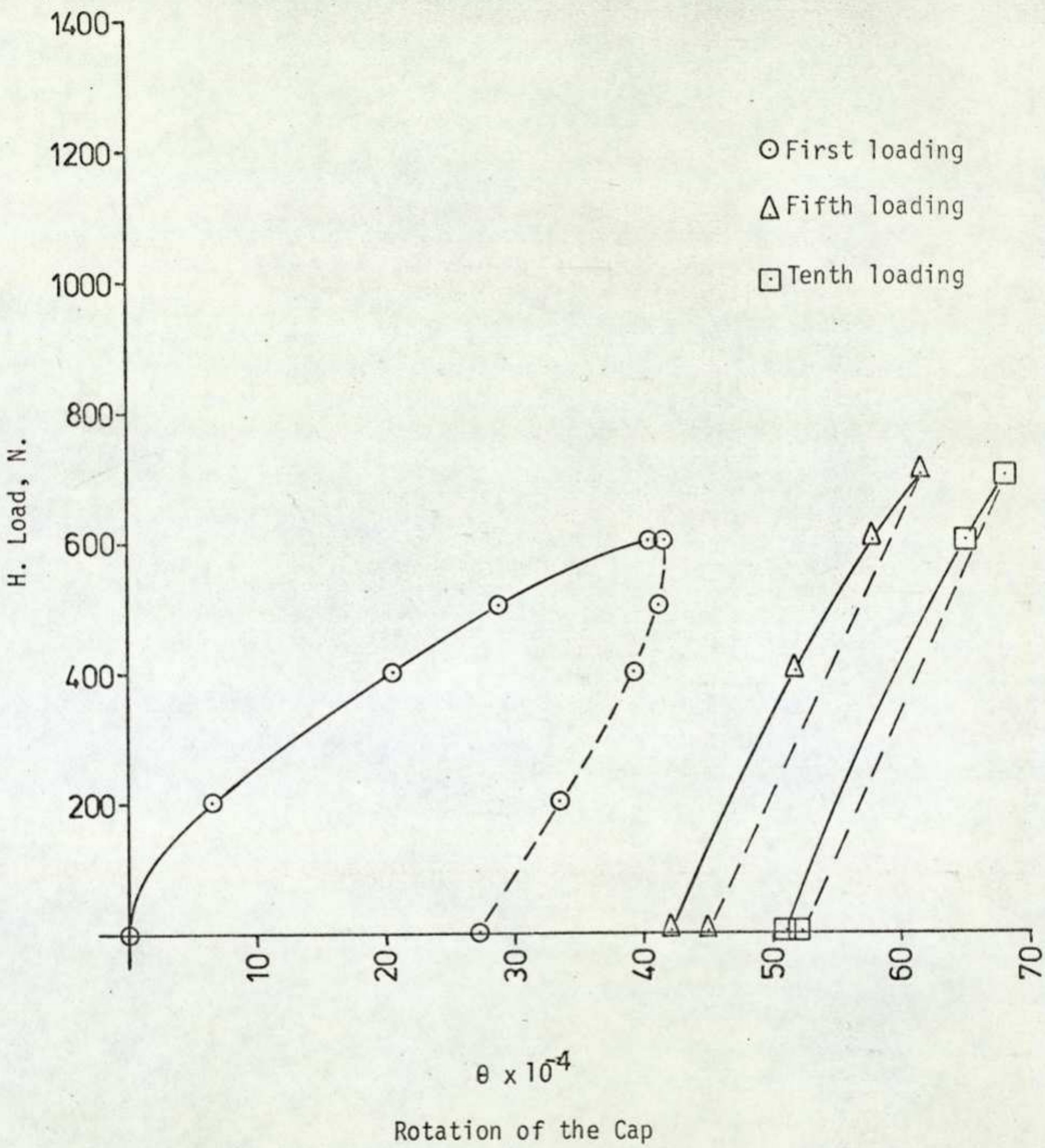


Fig. 7.40 Load Vs Rotation of the Cap

Pile Group 2 x 2

4

Test No. 9

Pile Group

H. Load, 600N

First loading

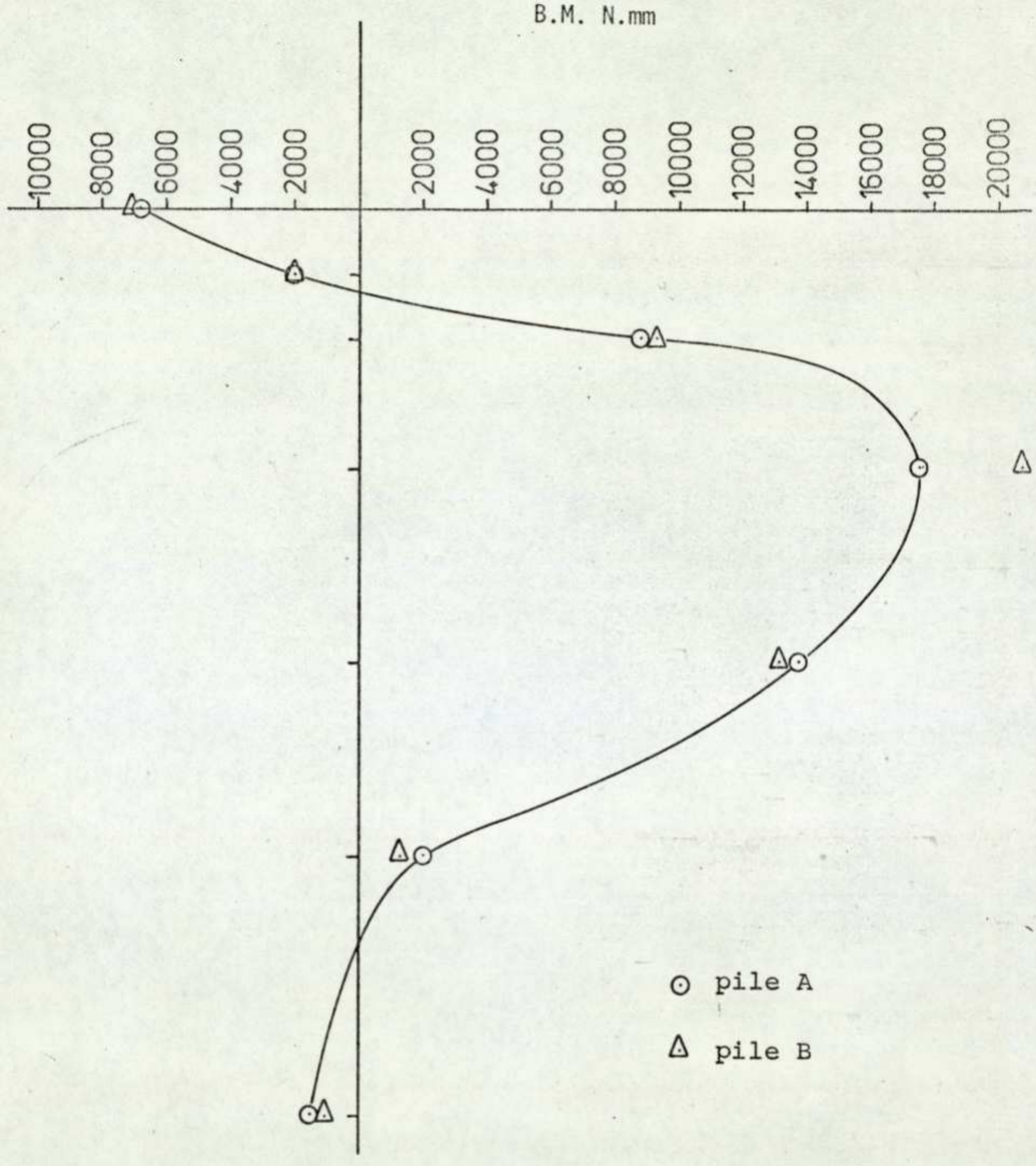


Fig. 7.41. Moment Vs Depth

Test No. 9

2 x 2
4v

Pile Group

H. Load, 600N

First loading

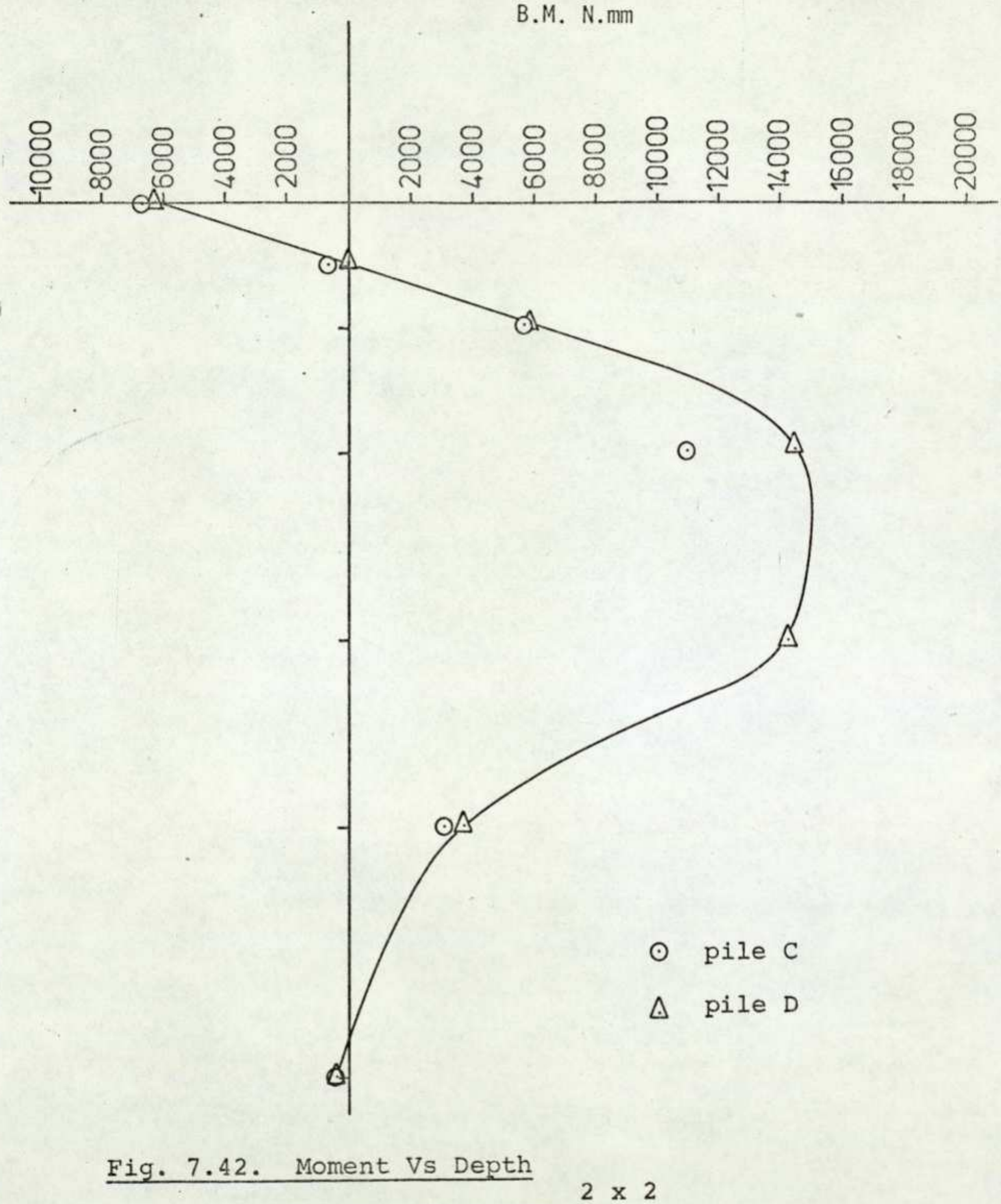


Fig. 7.42. Moment Vs Depth

Test No. 9

2 x 2
4v

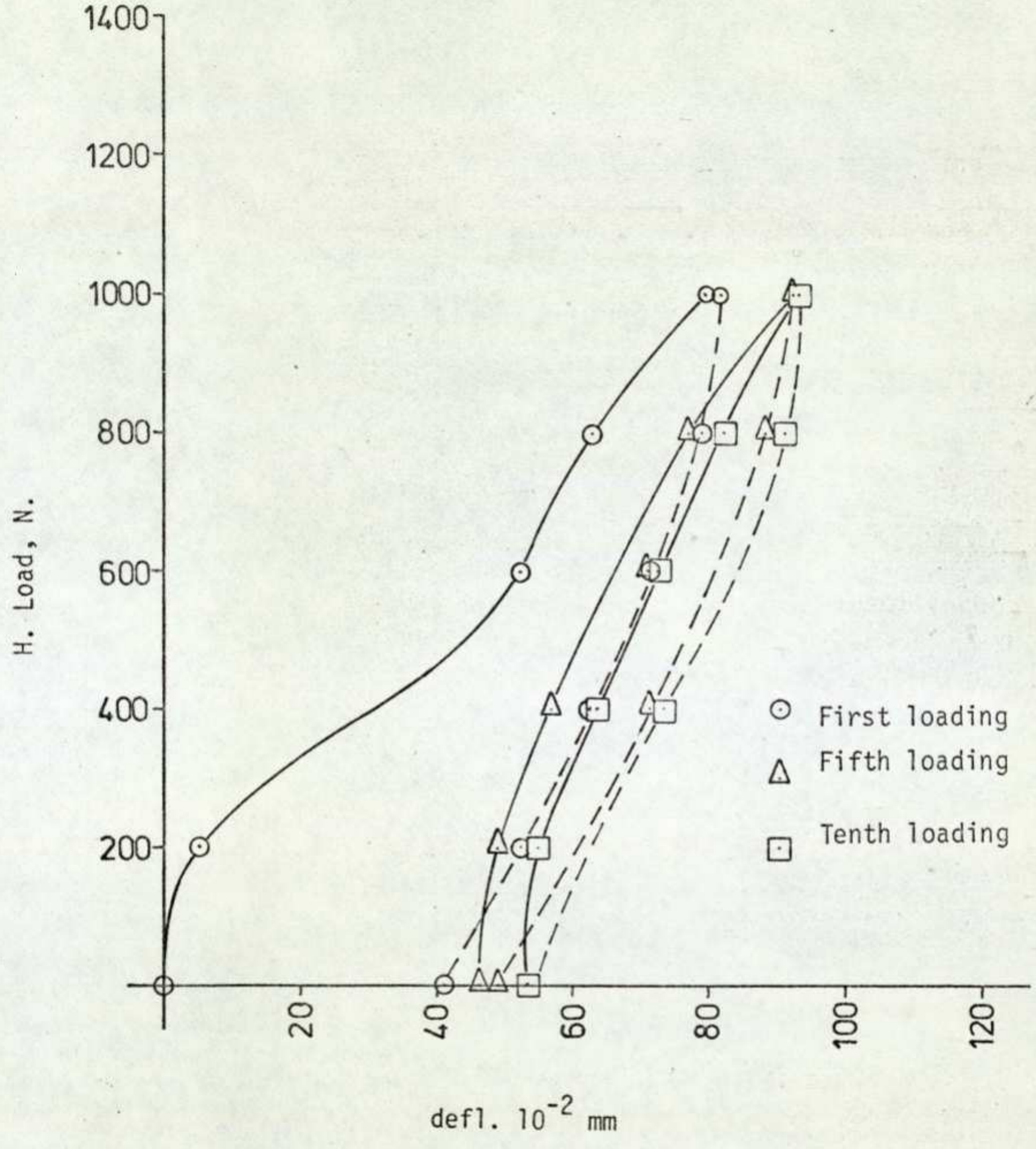


Fig. 7.43 load vs defl. - pile group

Test No. 12

2 x 2
+2B 30^0
2V

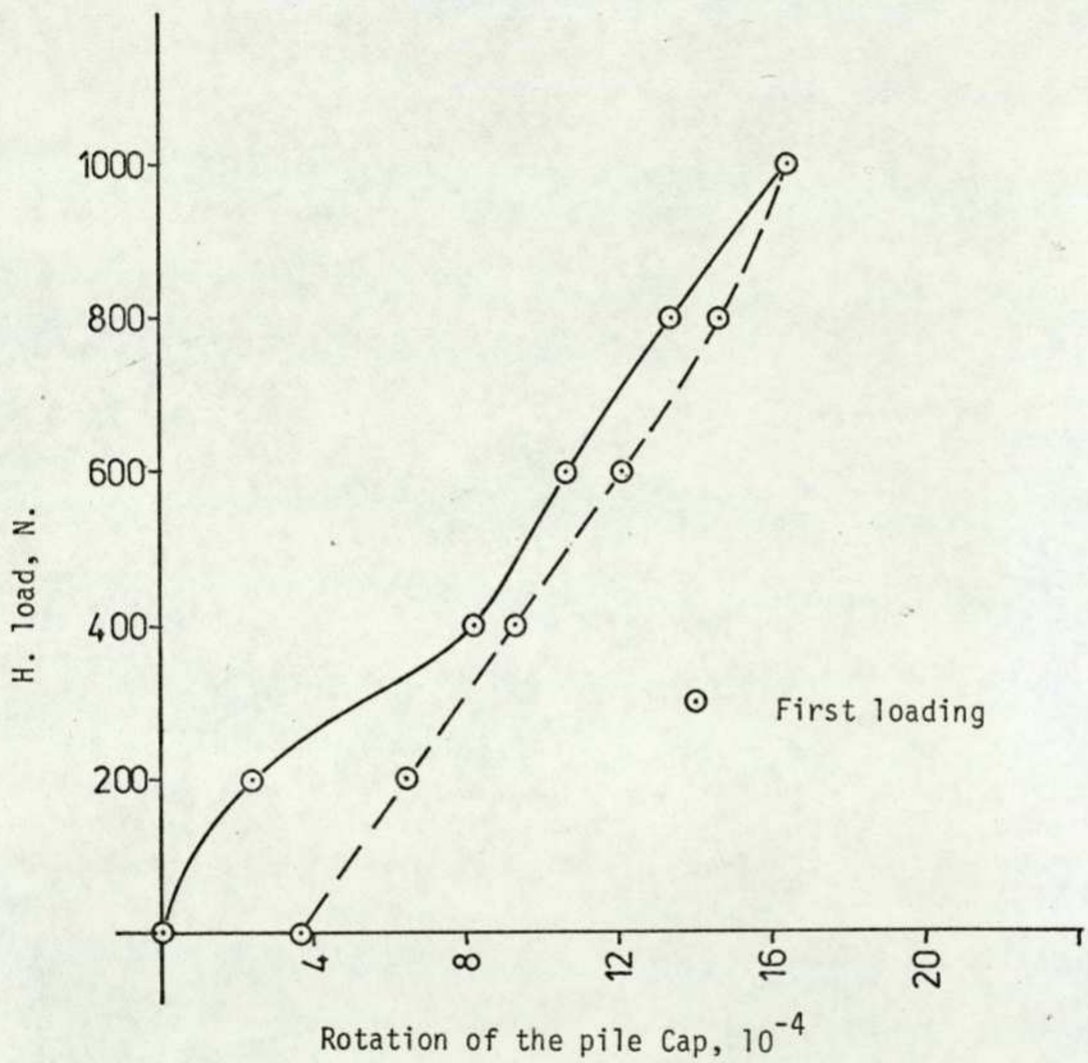


Fig. 7.44 load Vs Rotation of the pile cap

Test No. 12

H. Load, 600N

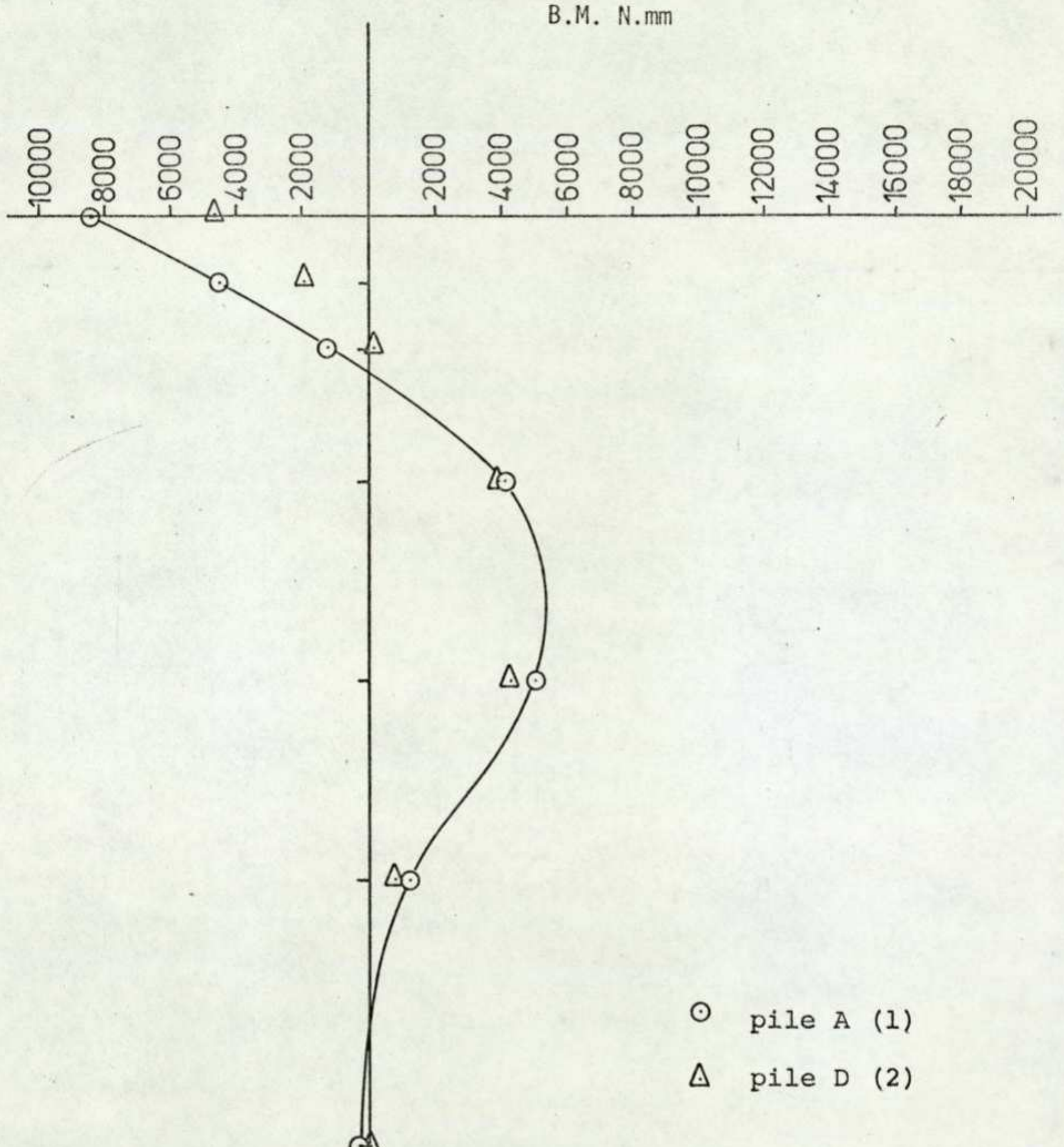


Fig. 7.45 Moment Vs Depth

Test No. 12

H. Load, 600N

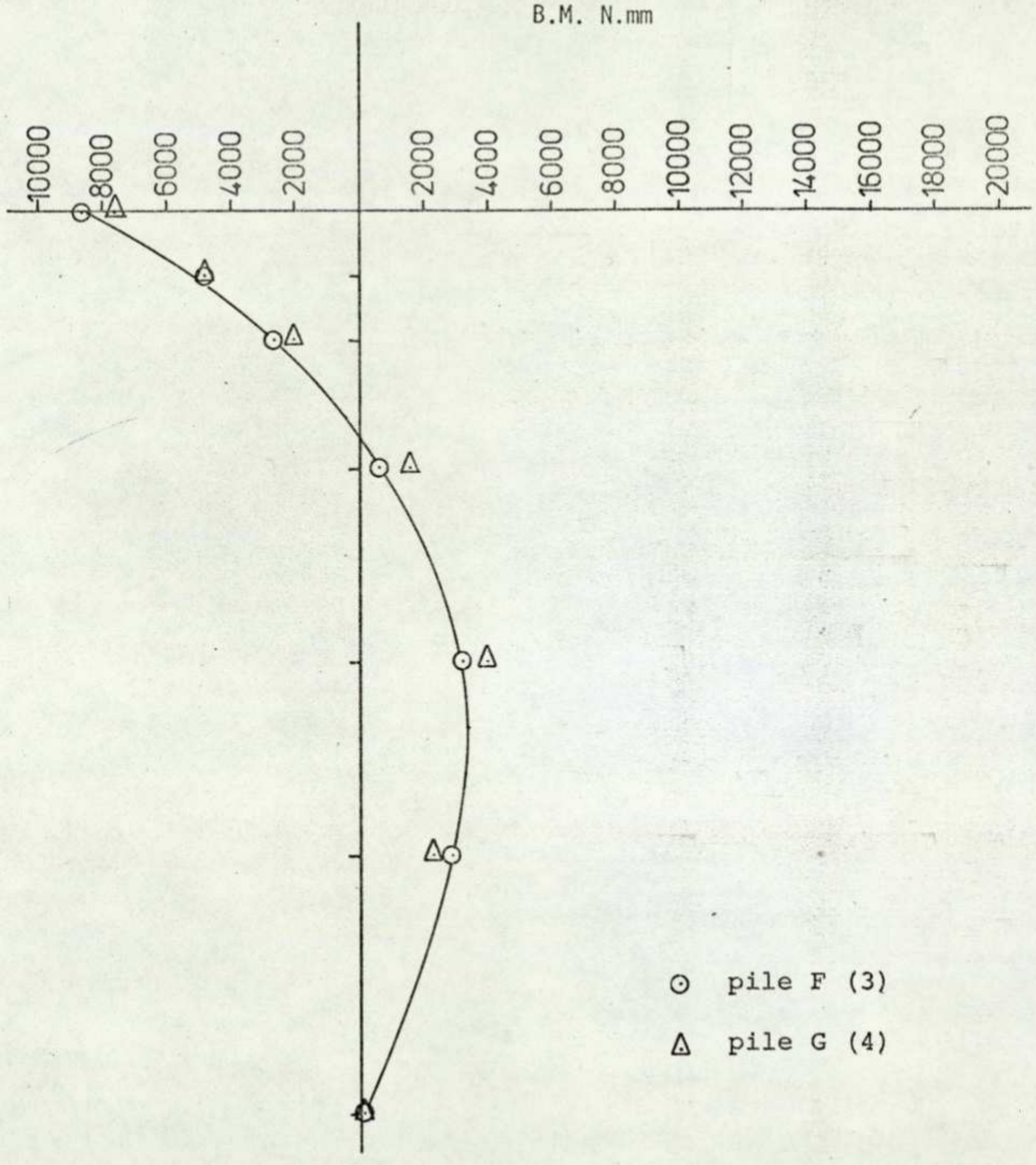


Fig. 7.46. Moment Vs Depth

Test No. 12

Test No. 3

3 x 3

9V

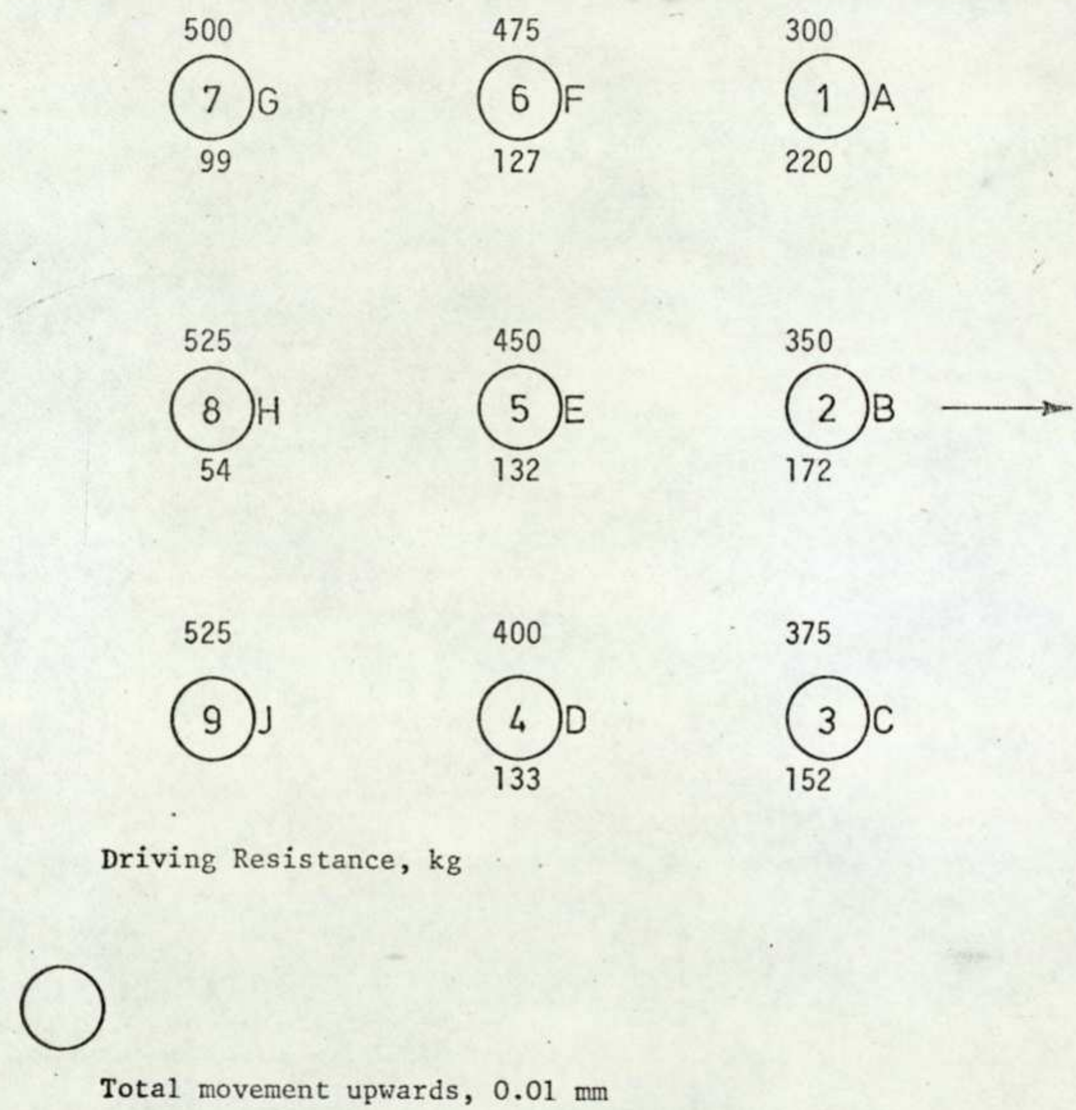


Fig. 7.47 Driving resistance and upwards movement in the piles during driving

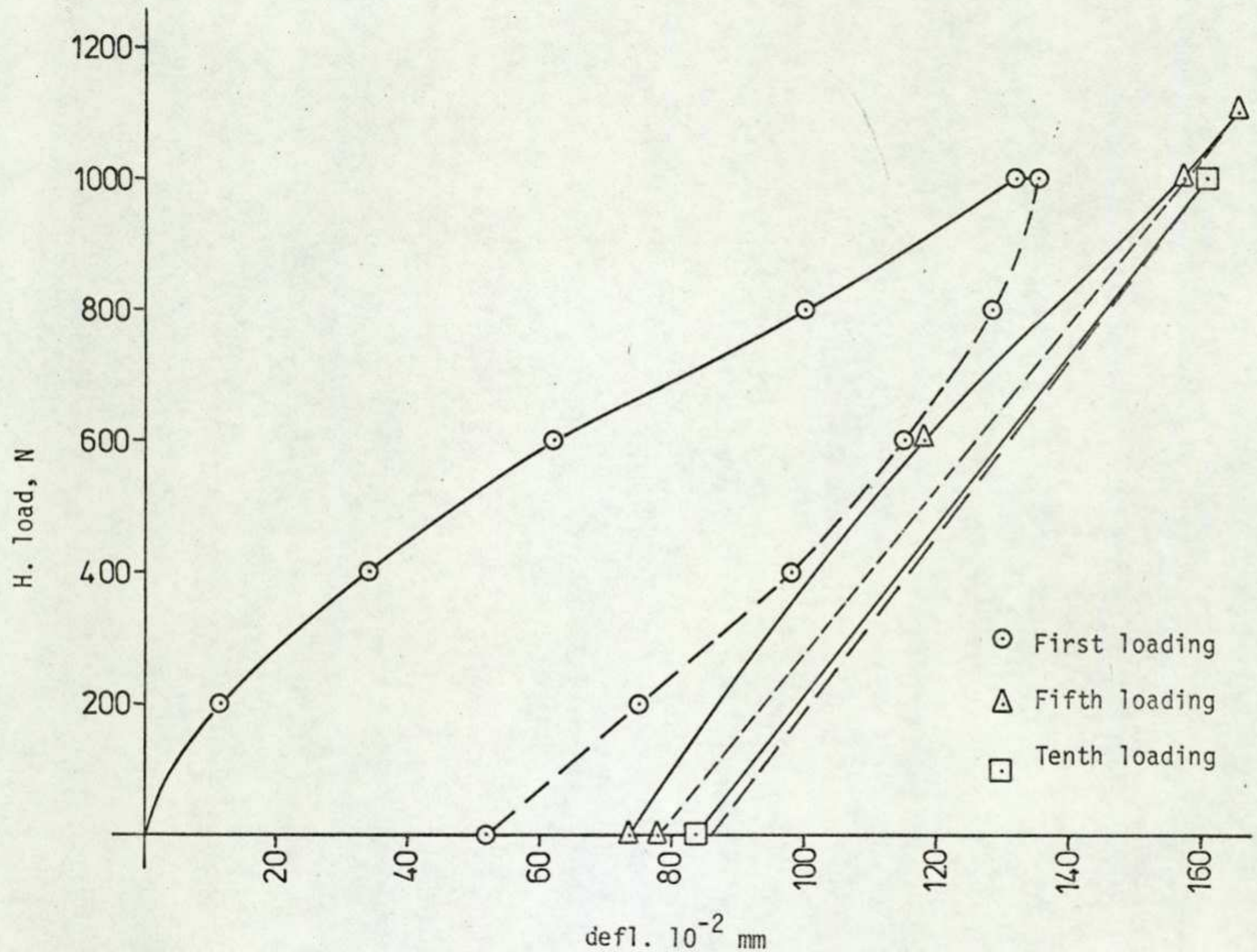


Fig. 7.48 Load Vs defl. - pile group

Test No. 3

3 x 3
9

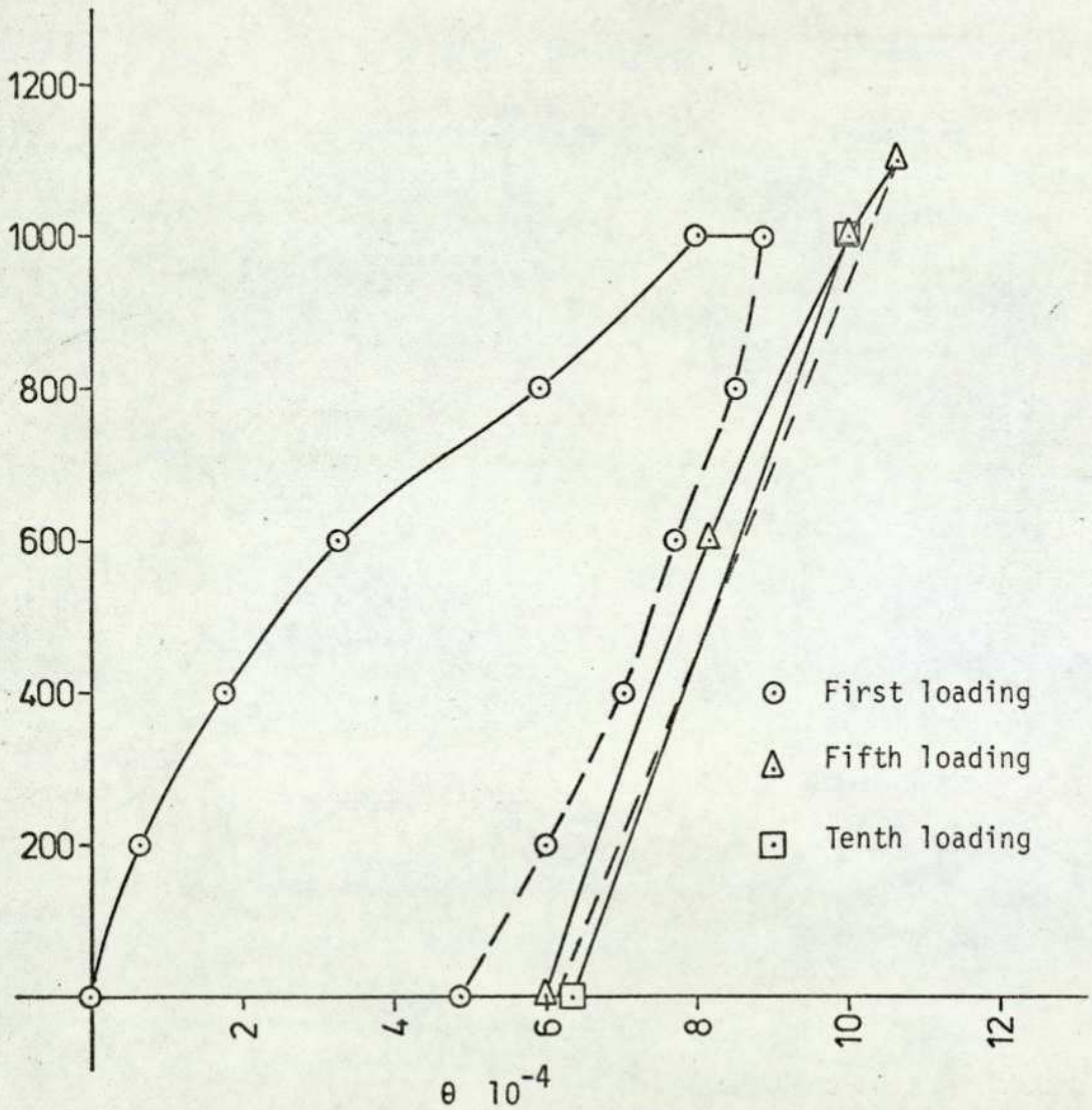
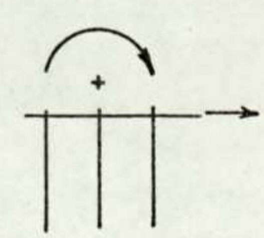


Fig. 7.49 Load Vs Rotation of the Cap.

pile group



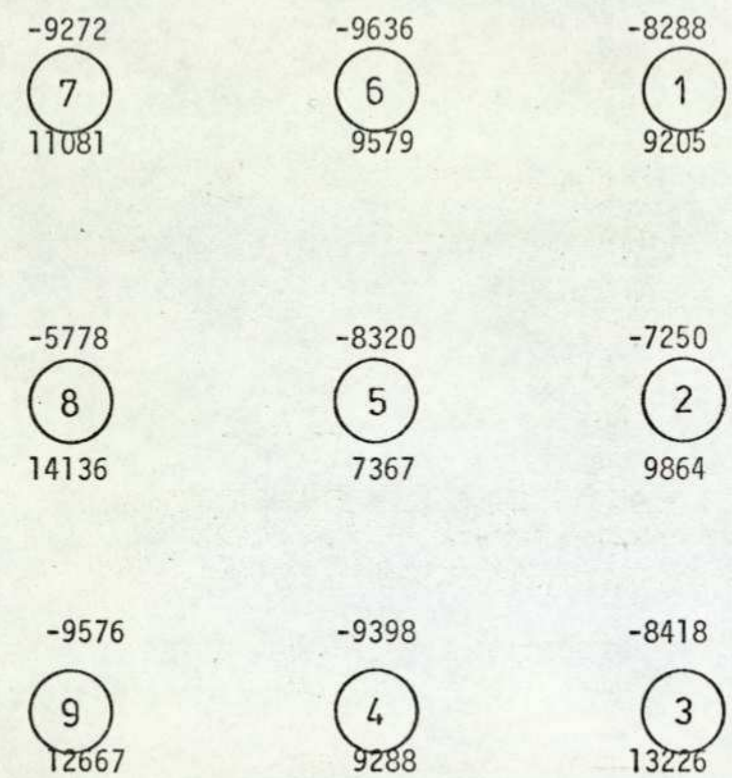
Test No. 3

Test No. 3

3 x 3

9V

H. load, 1000N



Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.50 Distribution of the maximum negative and positive bending moments in the piles.

Pile Group

H. Load,1000 N

First loading

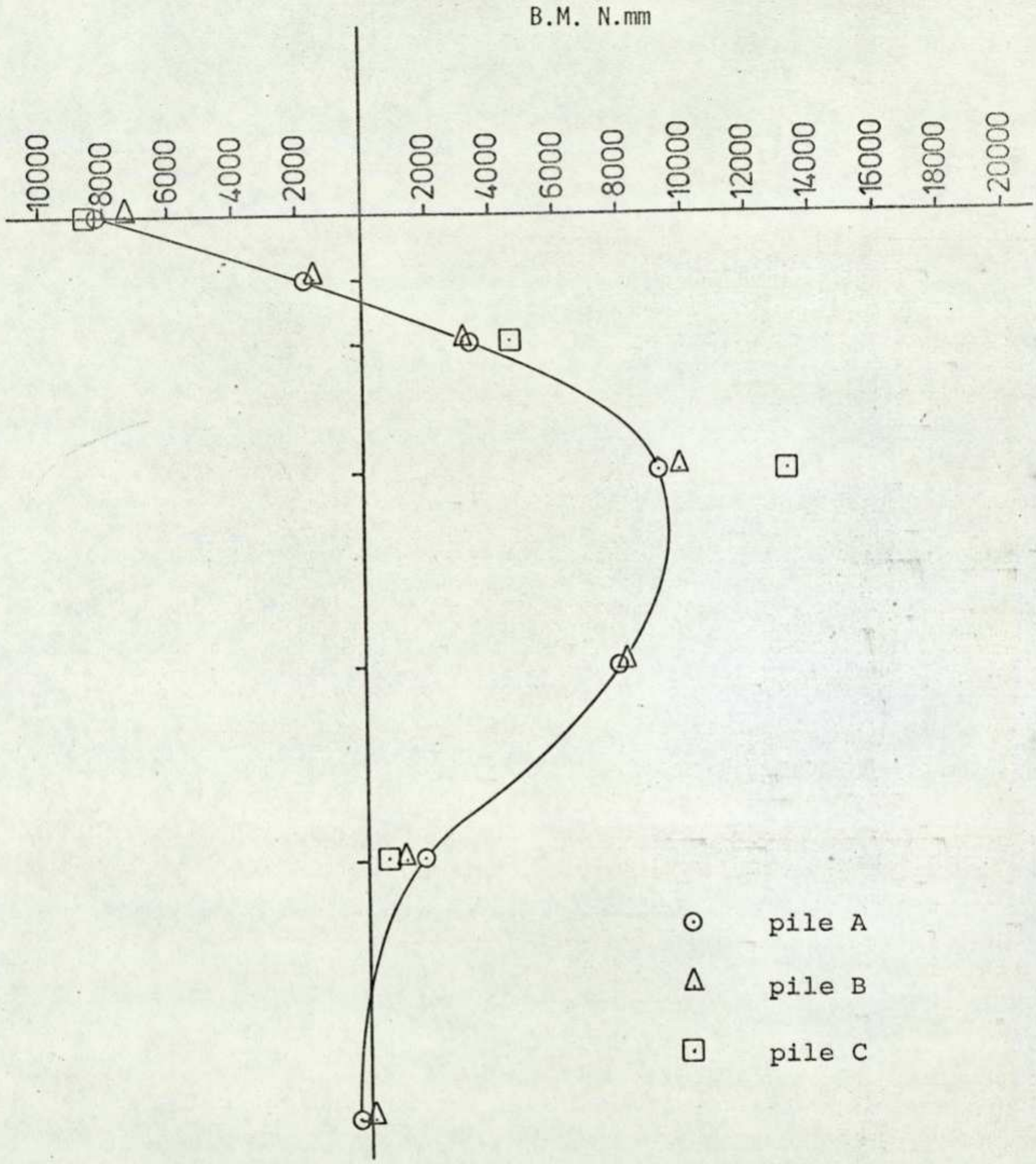


Fig. 7.51. Moment Vs Depth

Test No. 3

Pile Group

H. Load, 1000 N

First loading

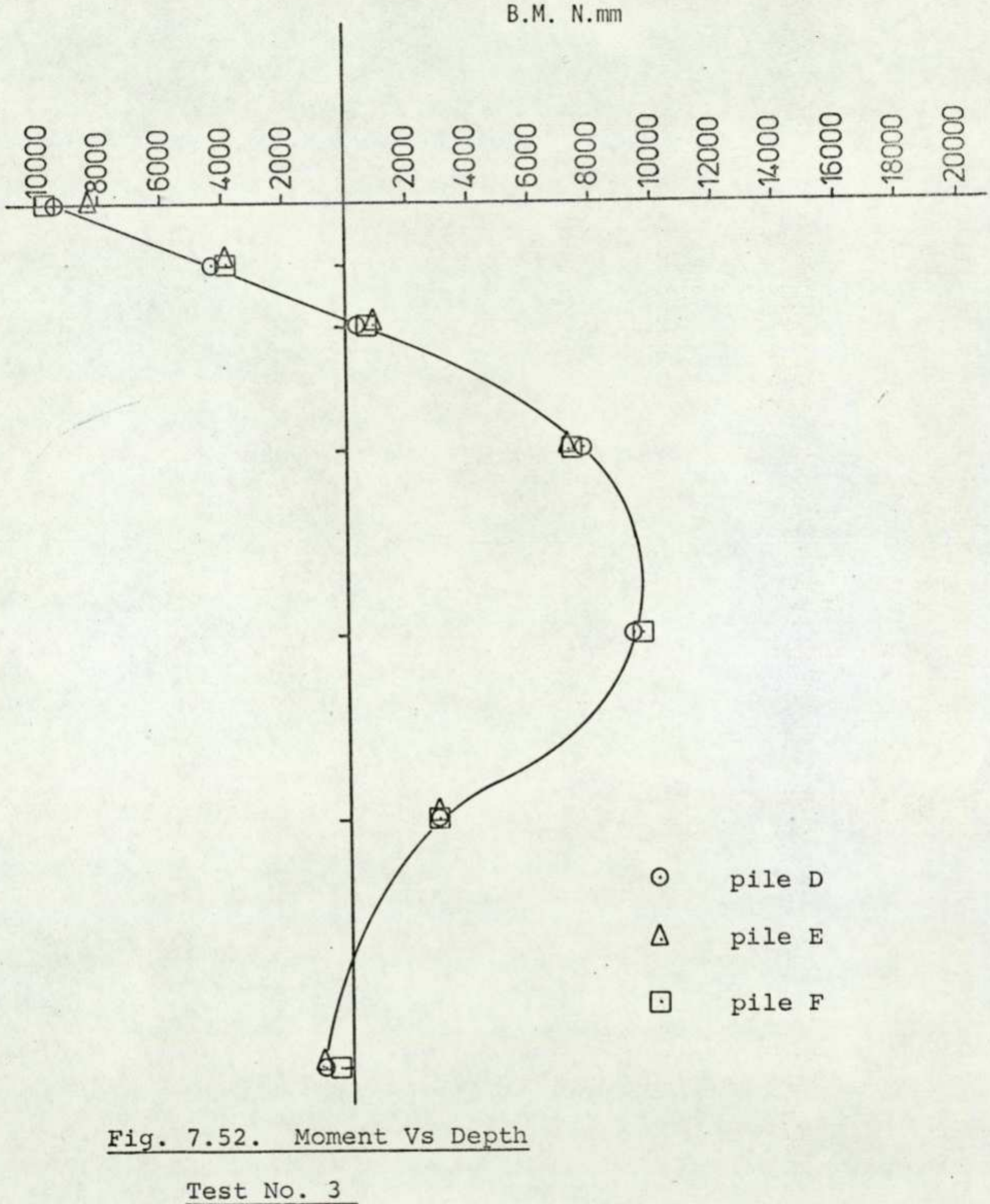


Fig. 7.52. Moment Vs Depth

Test No. 3

Pile Group

H. Load, 1000 N

First loading

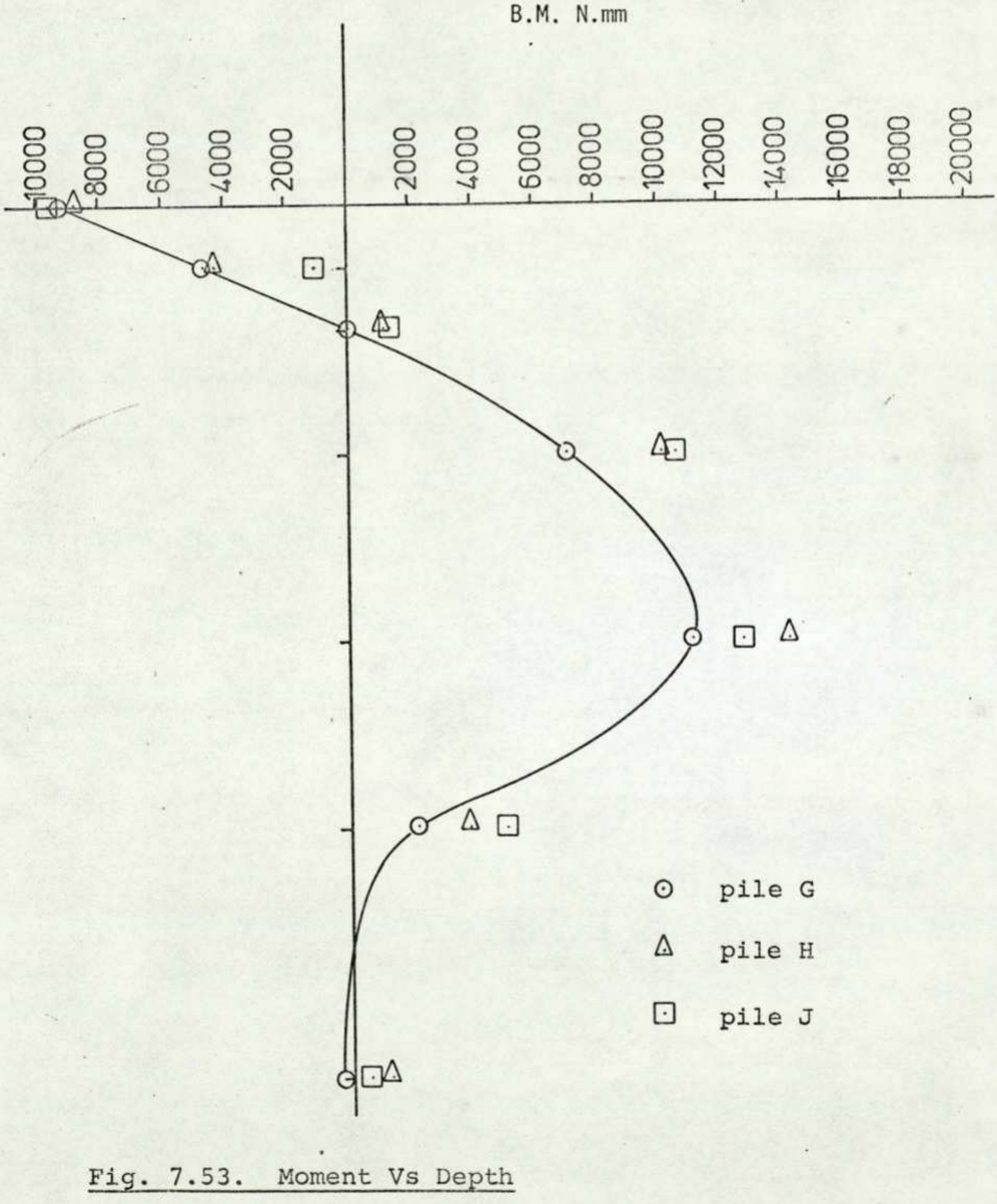


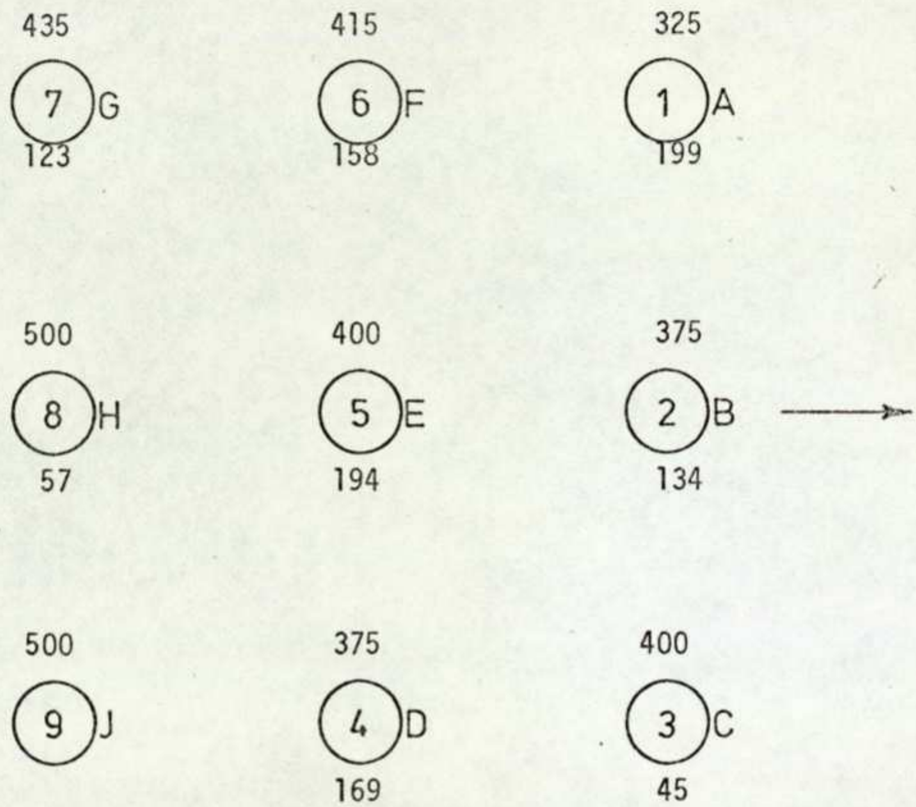
Fig. 7.53. Moment Vs Depth

Test No. 3

Test No. 4

3 x 3

+3B, 6V 15^{o}



Driving Resistance, kg

Total movement upwards, 0.01 mm

A, B, C +B 15^{o}
D, E, F, G, H & J V

Fig. 7.54 Driving resistance and upwards movement in the piles during driving

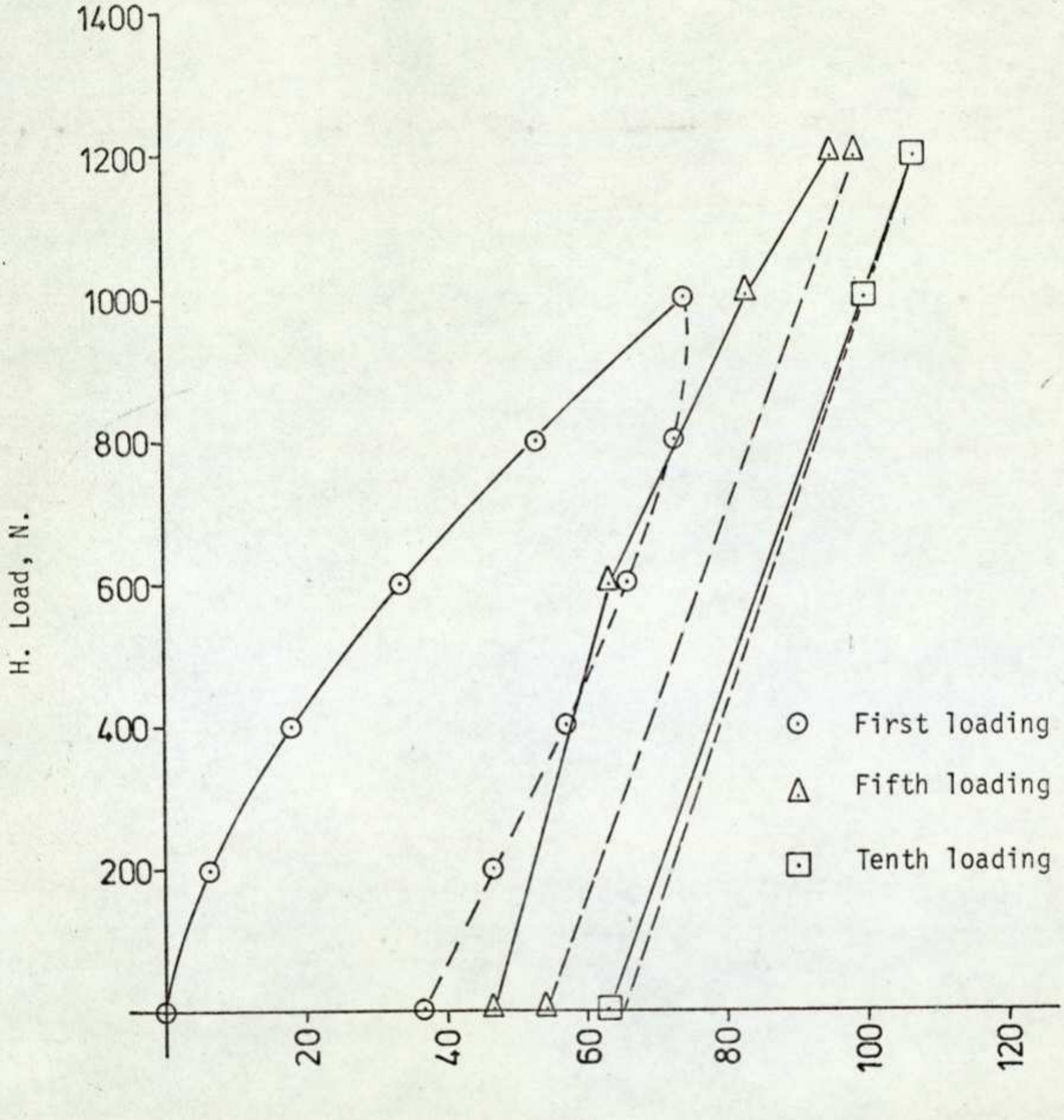
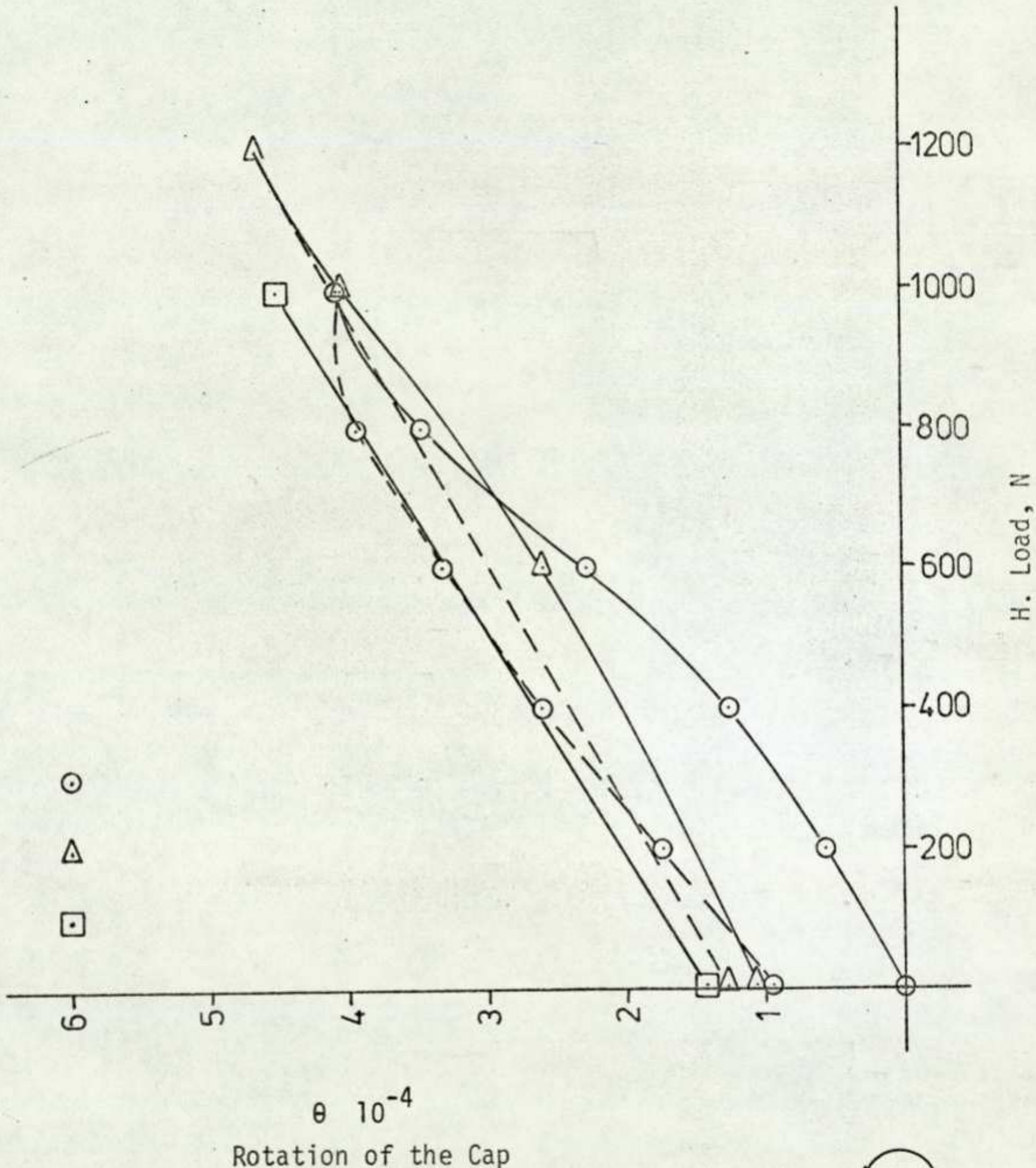


Fig. 7.55 load vs defl. - pile group

Test No. 4

3 x 3
+3B 15^{0}
6V



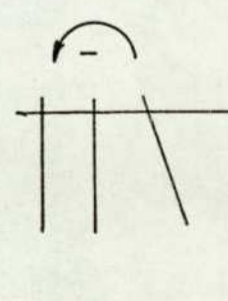


Fig. 7.56 Load Vs Rotation of the Cap

Pile Group

Test No. 4

Test No. 4

3 x 3

+3B, 6V , 15^{0}

H. Load, 1000N

-7198	-8580	-7650
7	6	1
5603	5562	5760
-7296	-6528	-7125
8	5	2
7440		8273
-7581	-8636	-5612
9	4	3
6692	5418	10114

Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.57 Distribution of the maximum negative and positive bending moments in the piles.

Pile Group

H. Load, 1000 N

$B = 15^{\circ}$

First loading

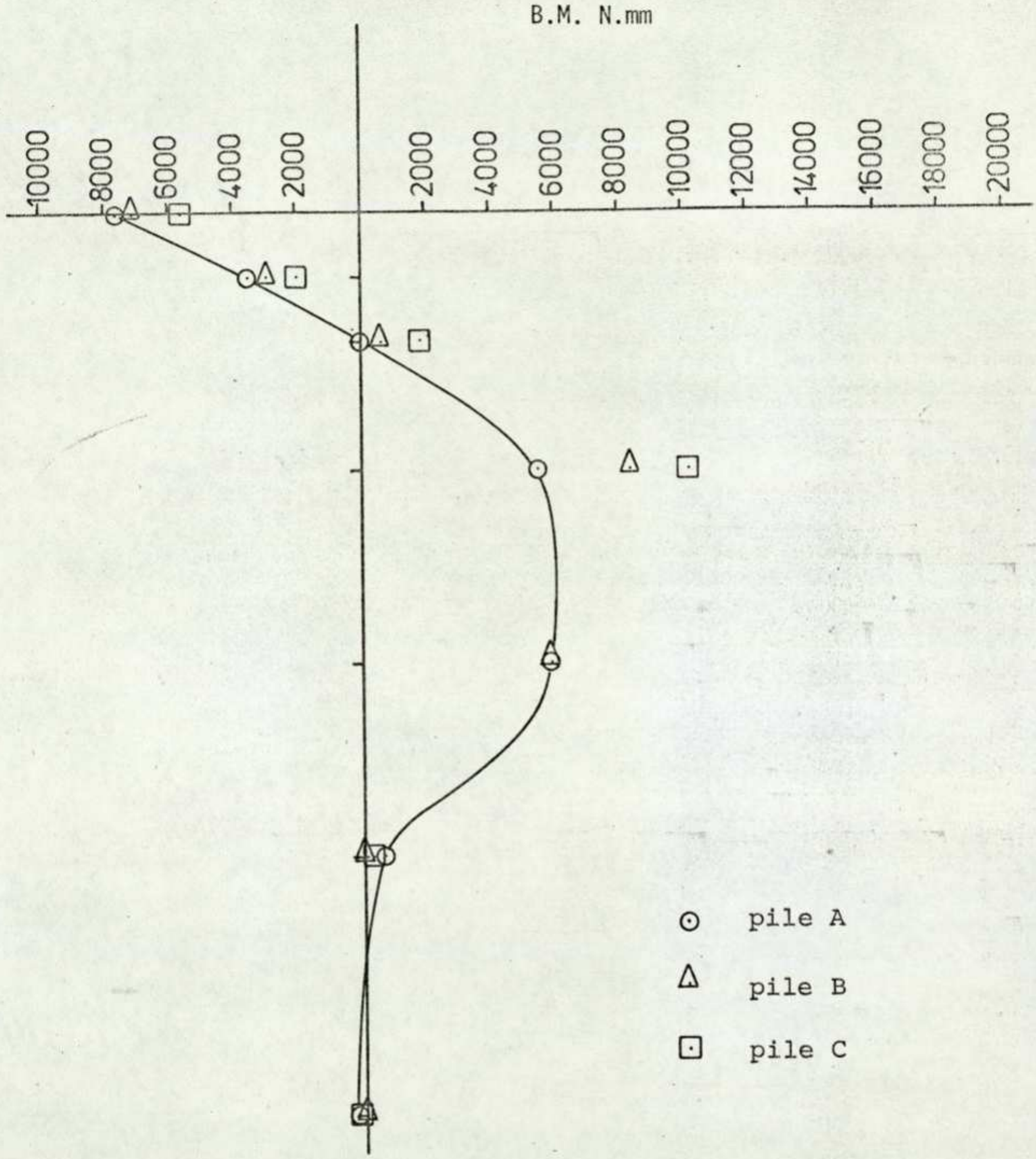


Fig. 7.58. Moment Vs Depth

Test No. 4

Pile Group

H. Load, 1000 N

B = 15°

First loading

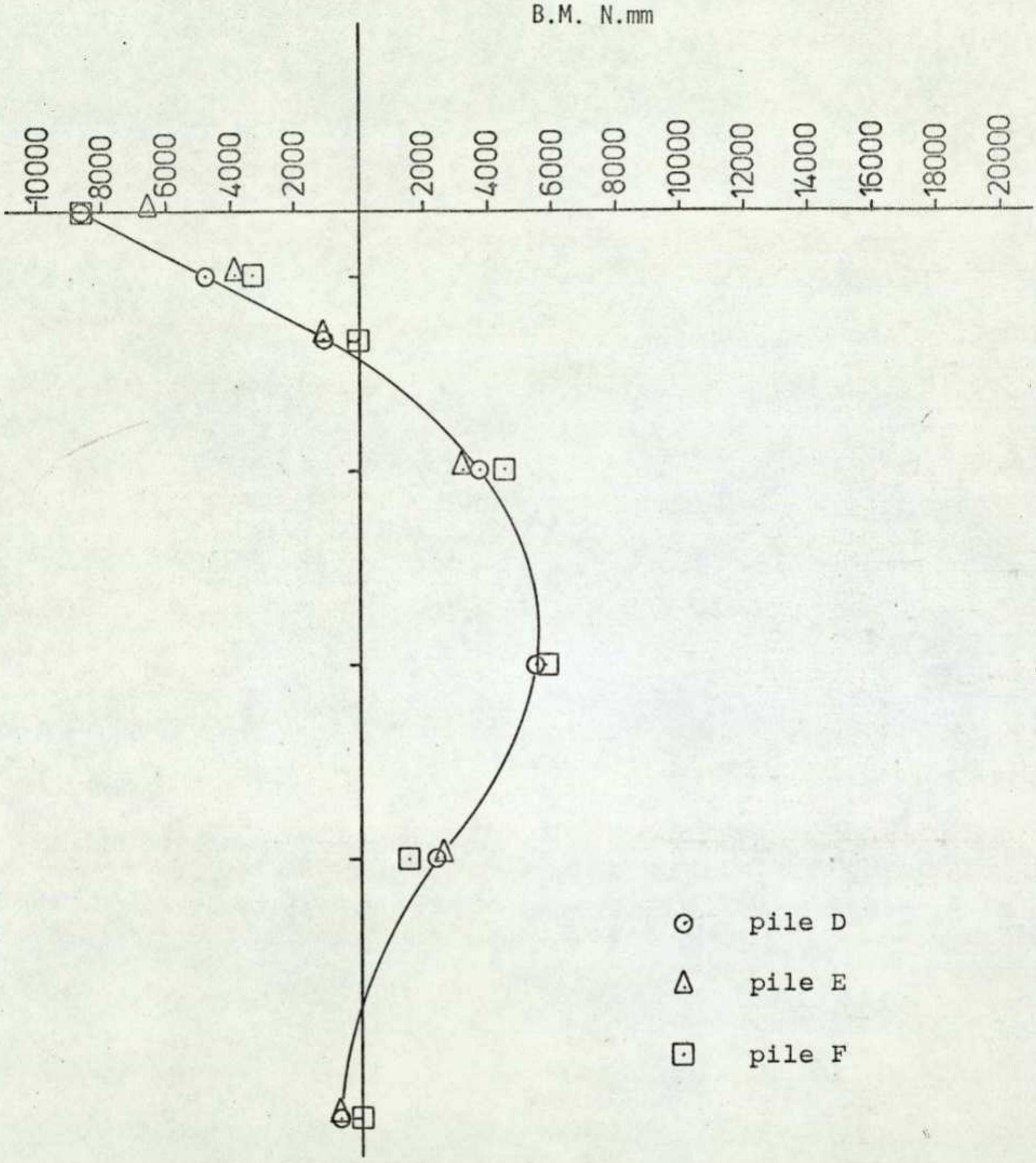


Fig. 7.59. Moment Vs Depth

Test No. 4

Pile Group

H. Load, 1000 N

B = 15°

First loading

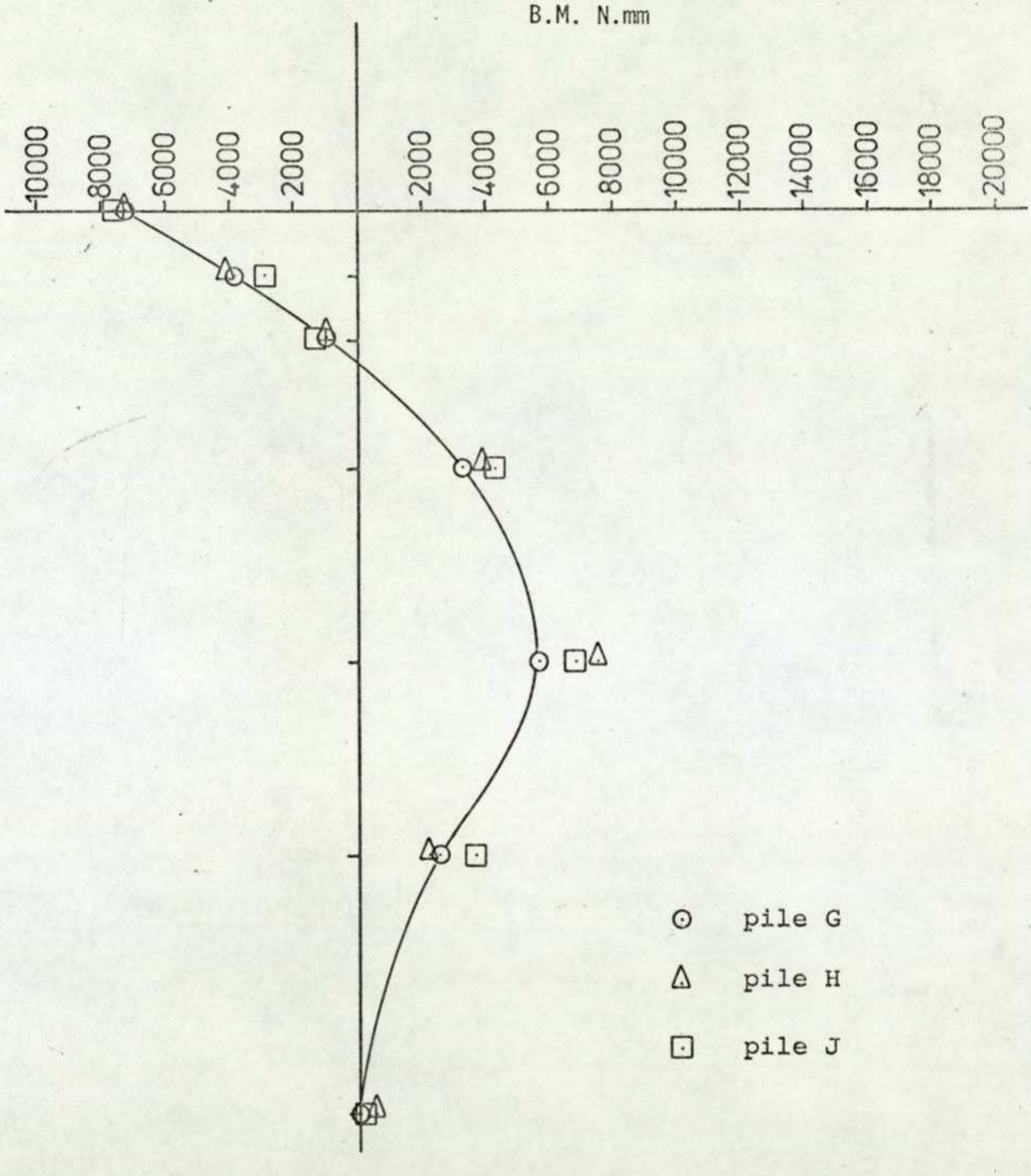


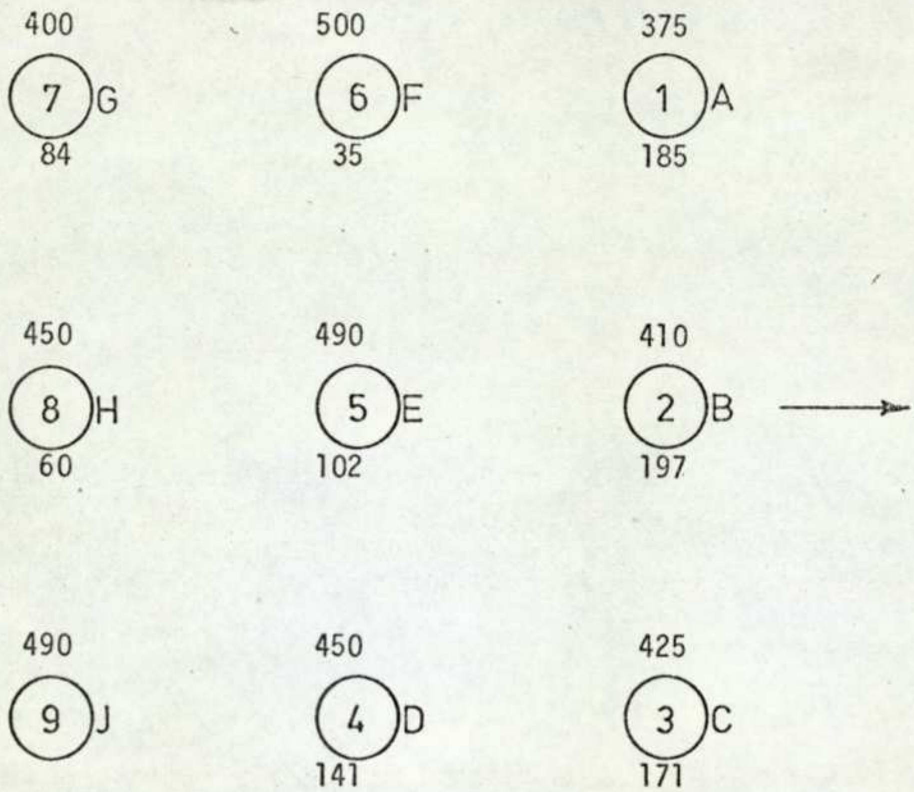
Fig. 7.60. Moment Vs Depth

Test No. 4

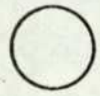
Test No. 5

3 x 3

+6B, 3V 15°



Driving Resistance, kg



Total movement upwards, 0.01 mm

A, B, C, D, E & F +B 15°

G, H & J V

Fig. 7.61 Driving resistance and upwards movement in the piles during driving

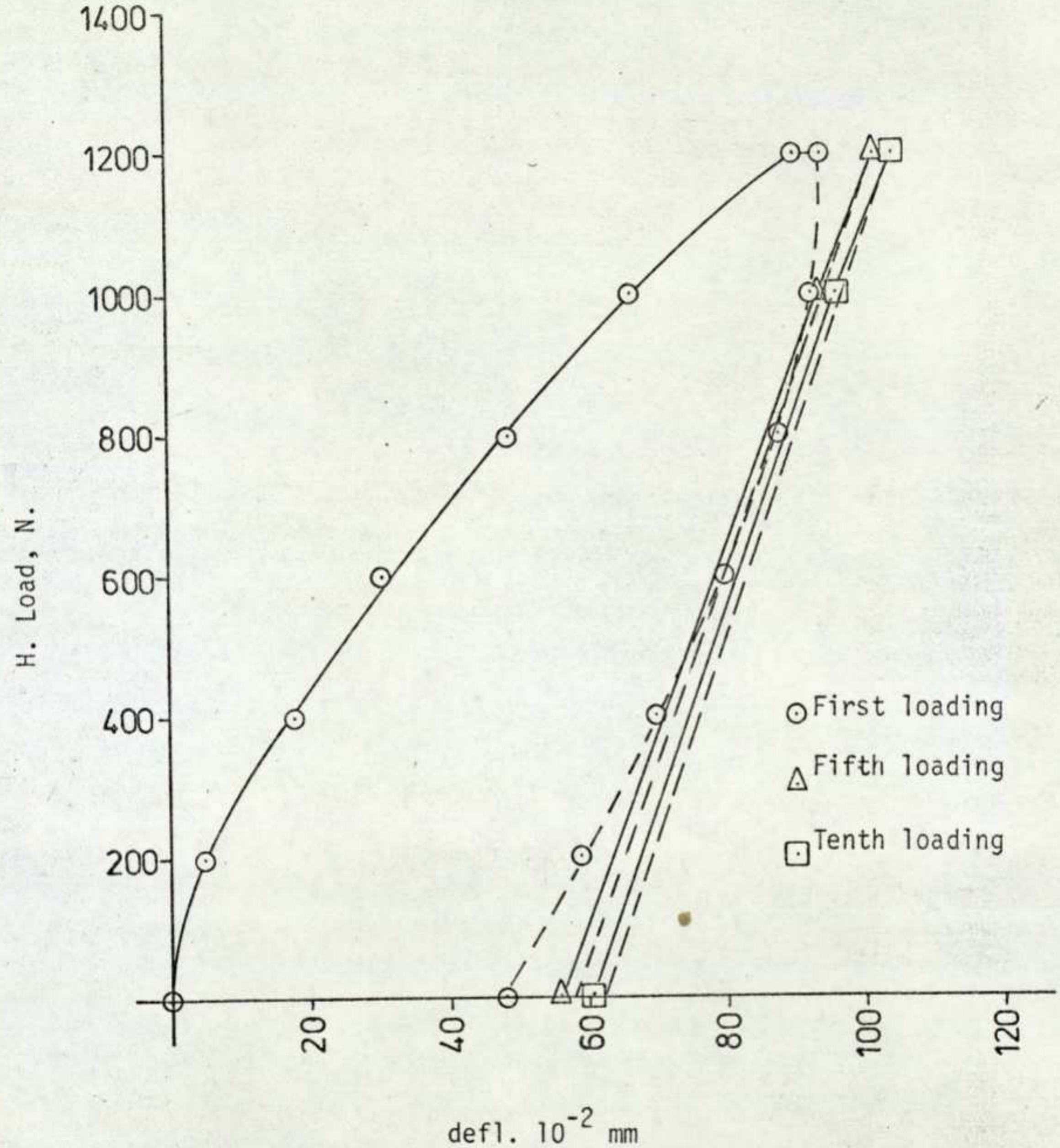


Fig. 7.62 load Vs defl. - pile group

Test No. 5

3 X 3
+6B, 3V 15^{0}

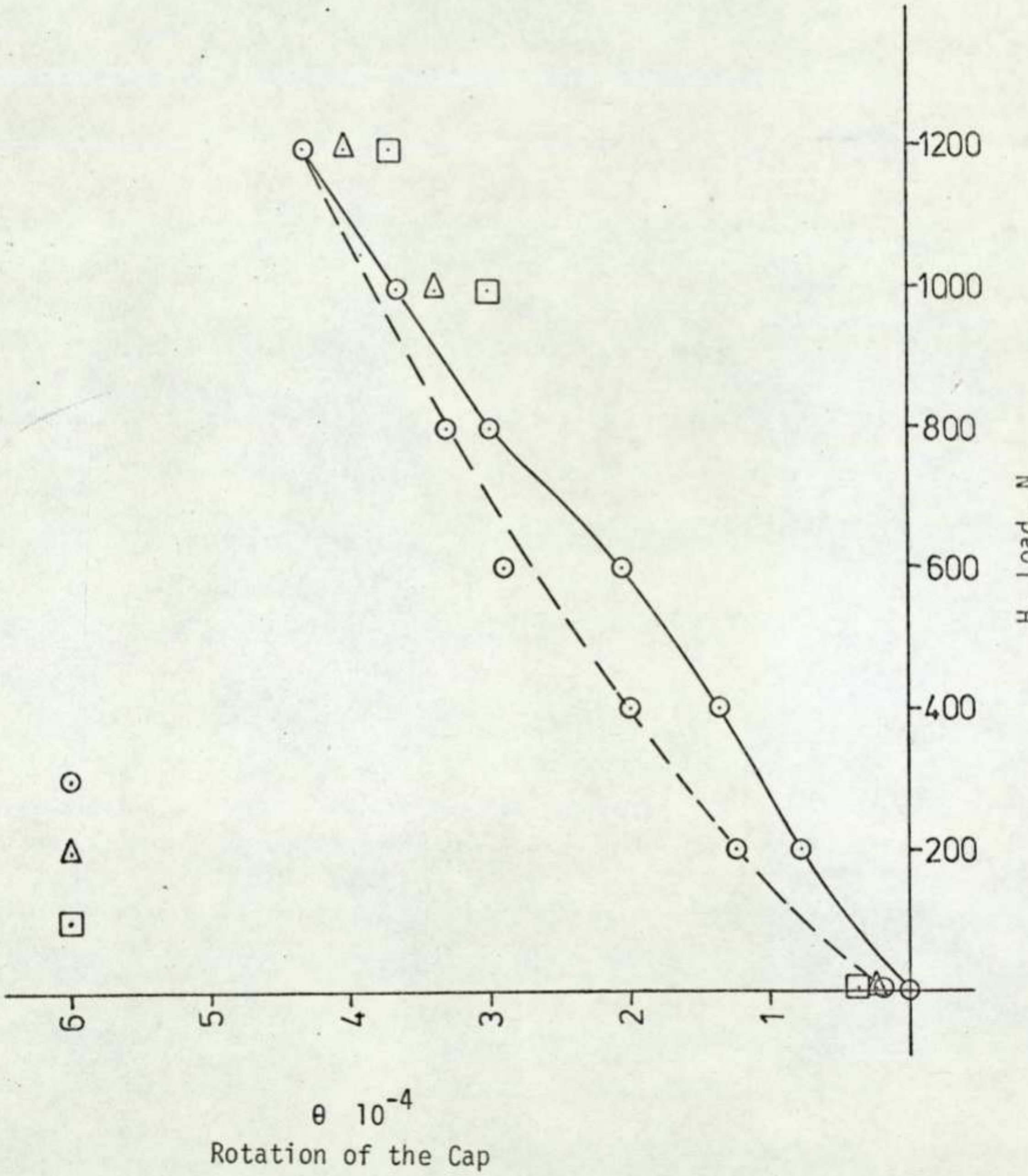


Fig. 7.63 Load Vs Rotation of the Cap- Pile Group

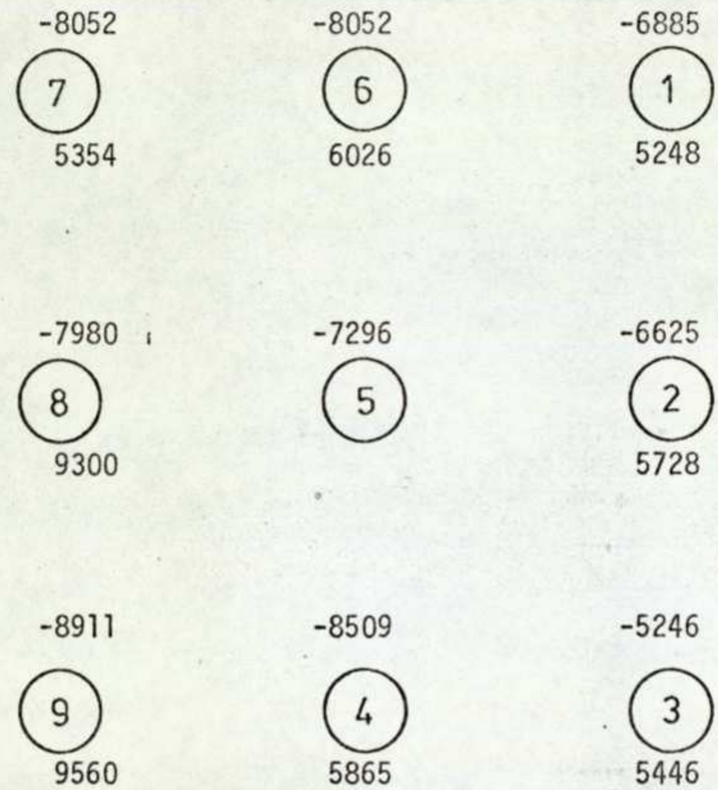
Test No. 5

Test No. 5

3 x 3

+6B, 3V, 15^{o}

H. Load, 1000N



Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.64 Distribution of the maximum negative and positive bending moments in the piles.

Pile Group

H. Load, 1000 N

B = 15°

First loading

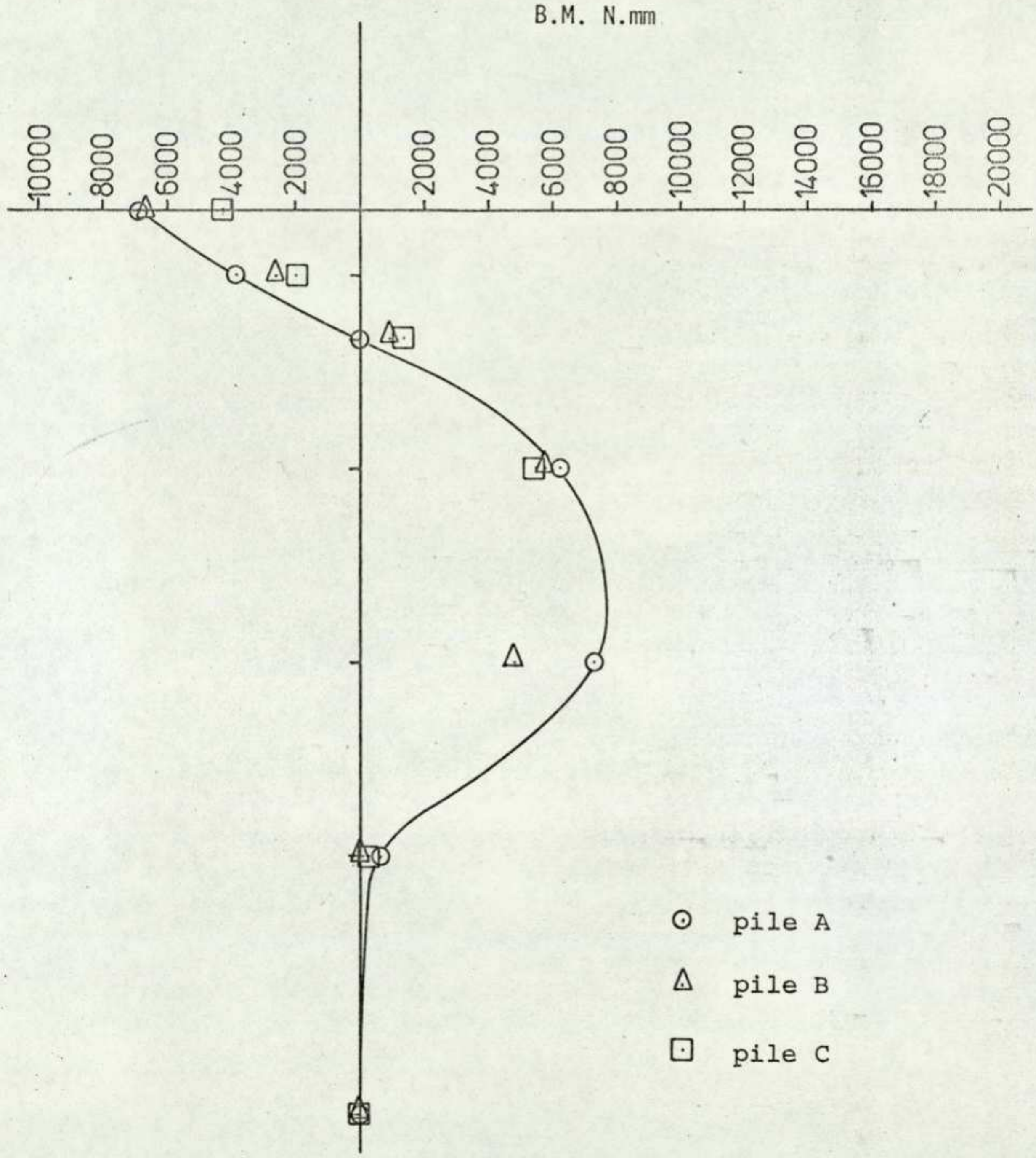


Fig. 7.65. Moment Vs Depth

Test No. 5 $3 \times 3 + 6B, 3v$

Pile Group

H. Load, 1000N

B = 15

First loading

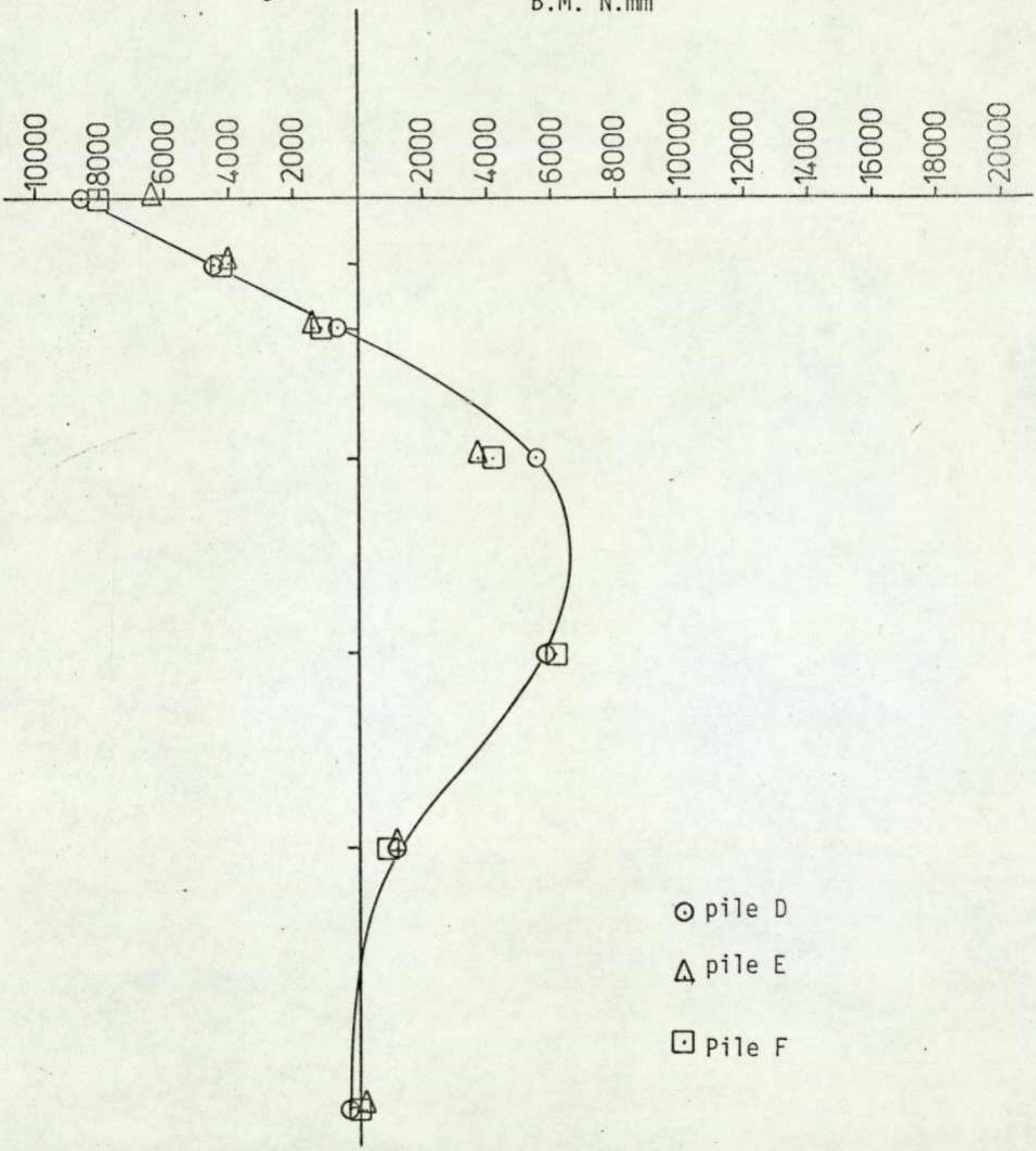


Fig. 7.66 Moment Vs Depth

3 x 3
+6B, 3V

Test No. 5

H. Load,1000N

B = 15^0

First loading

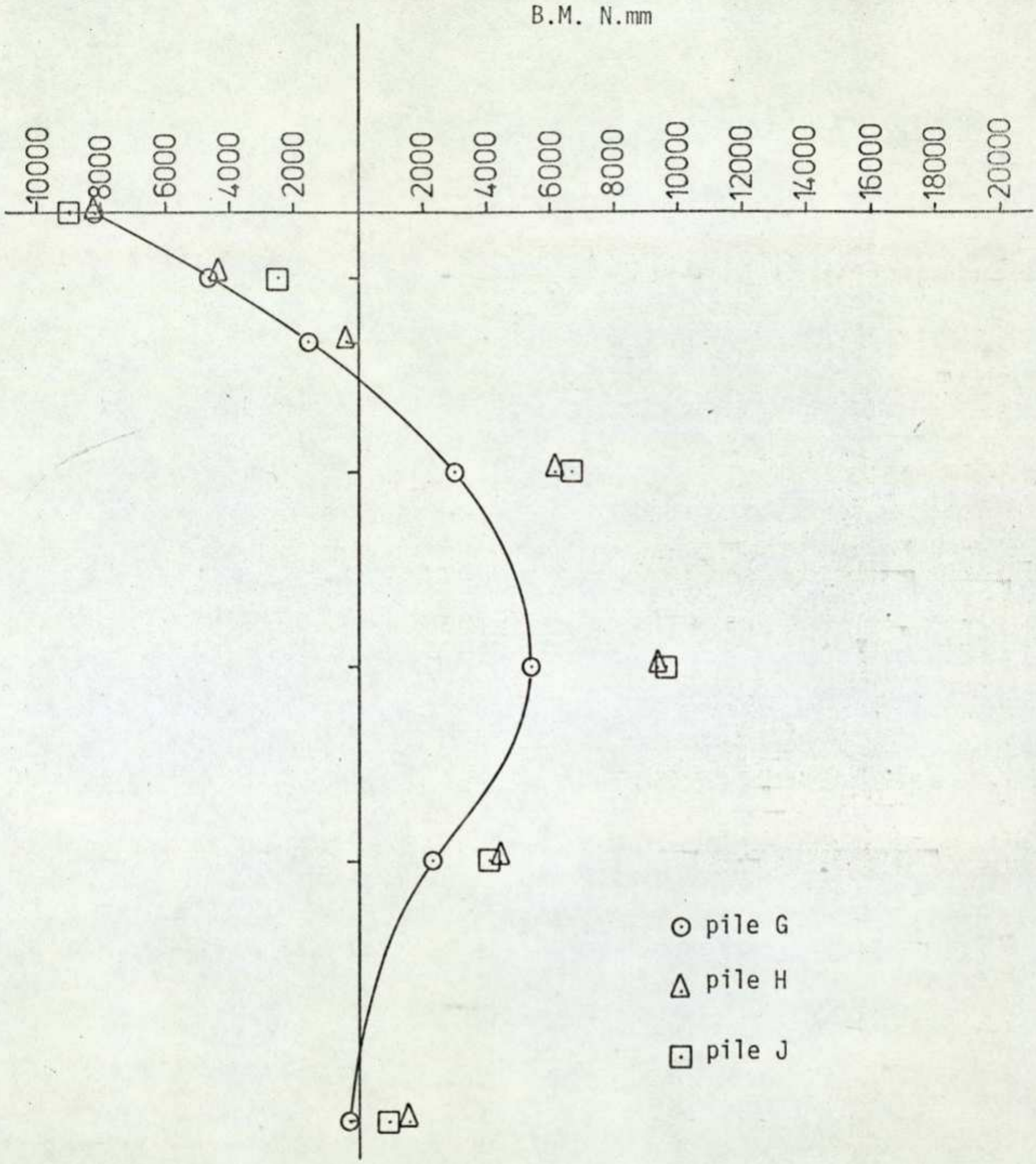


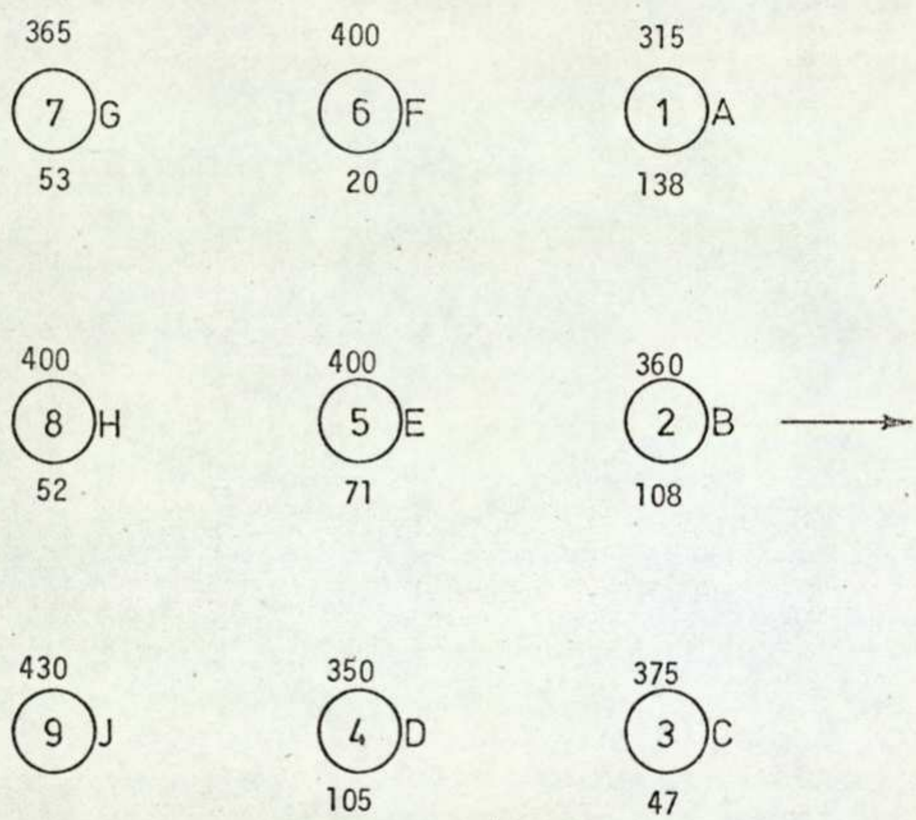
Fig. 7.67 Moment Vs Depth

Test No. 5

Test No. 6

3 x 3

+3B, 3V, -3B 15^{0}



Driving Resistance, kg

Total movement upwards, 0.01 mm

A, B, C	+B 15^{0}
D, E, F	V
G, H, J	-B 15^{0}

Fig. 7.68 Driving resistance and upwards movement in the piles during driving

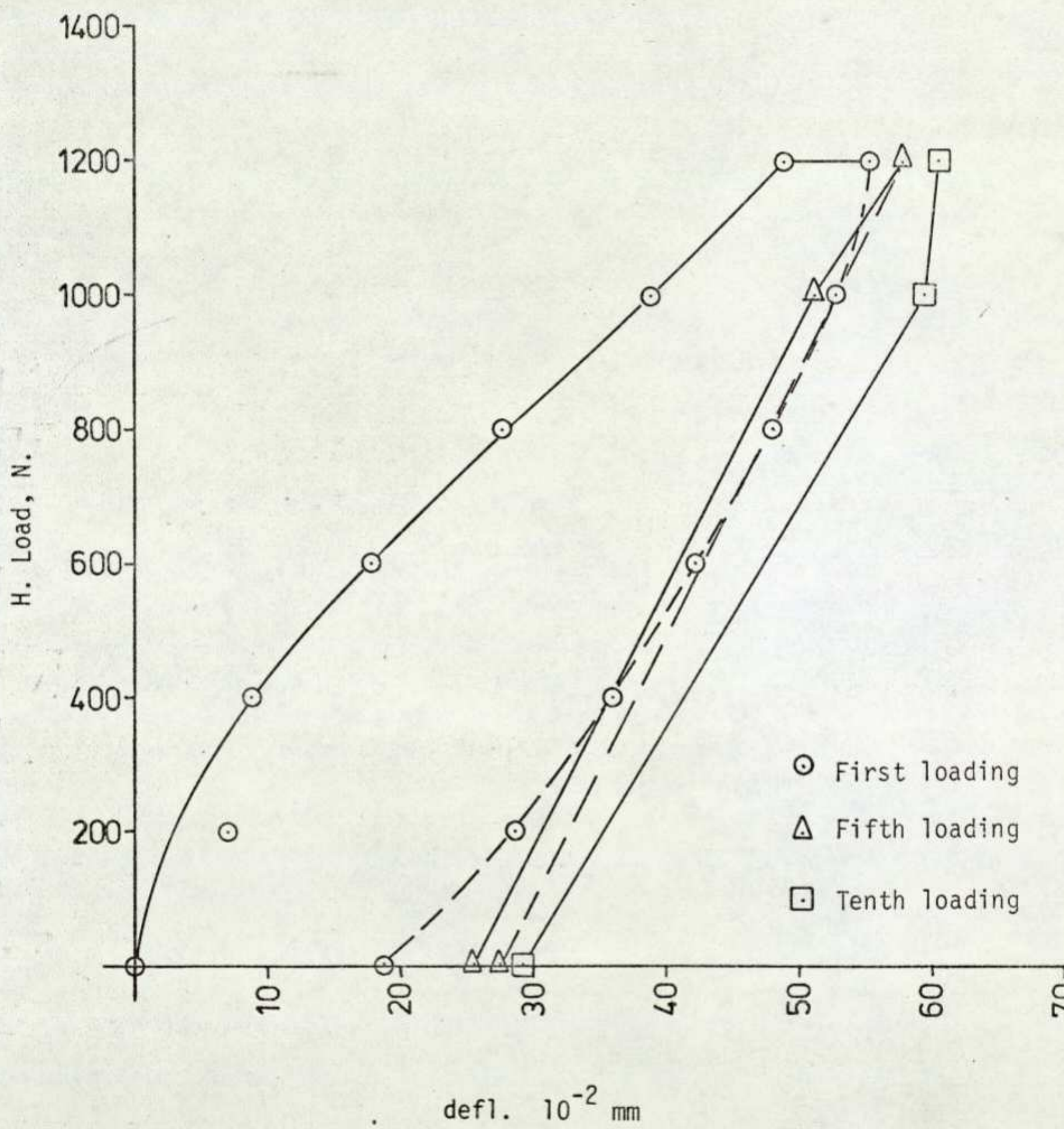


Fig. 7.69 Load Vs defl. - pile Group

Test No. 6

3 x 3

+3B, 3 , - 3B

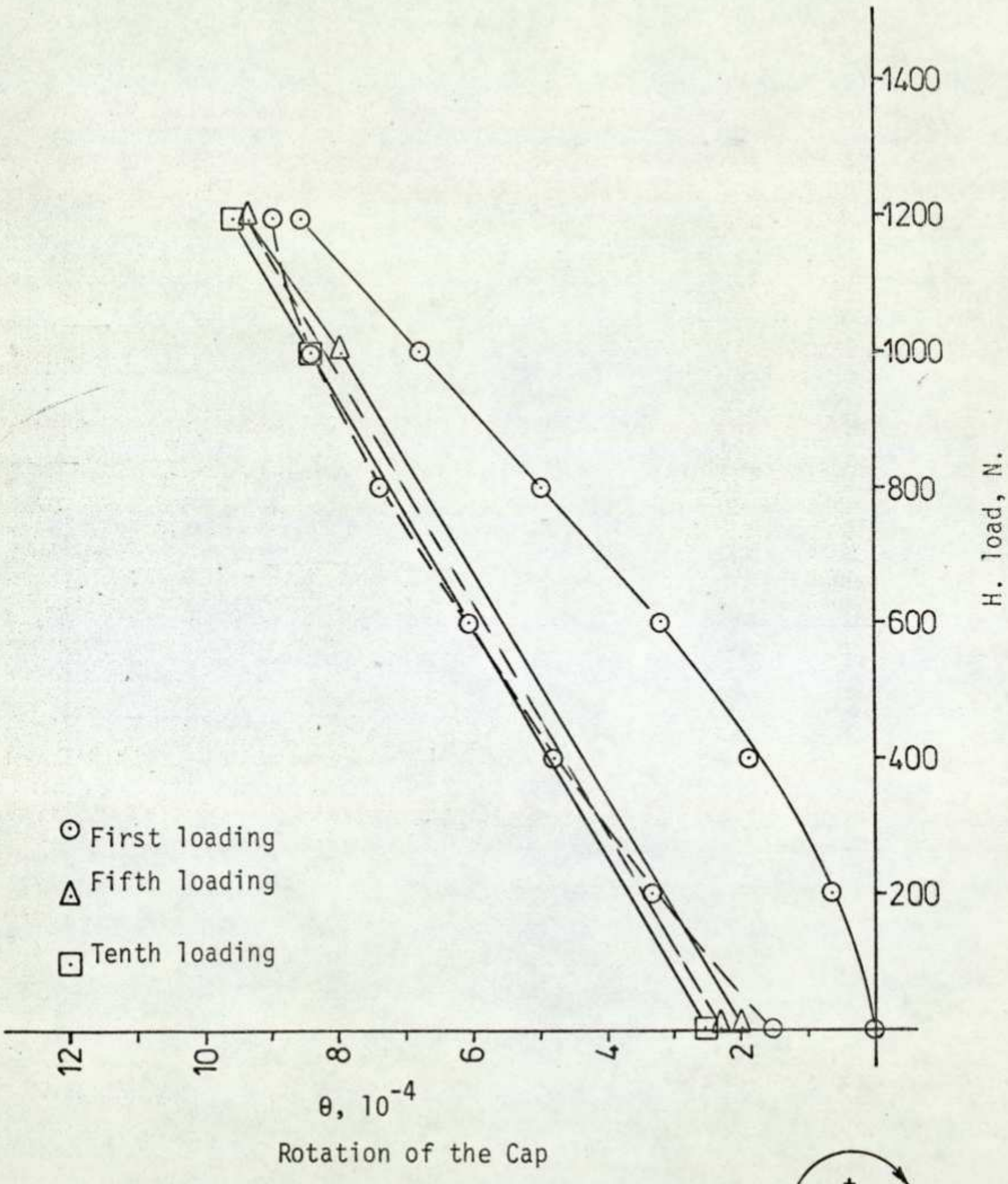


Fig. 7.70 Load Vs Rotation of the Cap

PILE GROUP

Test No. 6

Test No. 6

3 x 3

+3B, 3 , -3B, 15^{o}

H. Load, 1000N

-6466	-6600	-6630
7	6	1
3611	3848	4866
-6498	-5888	-6500
8	5	2
5952	2502	5409
-6650	-6731	-6466
9	4	3
5497	3313	7780

Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.71 Distribution of the maximum negative and positive bending moments in the piles.

Pile Group

H. Load, 1000N

B = 15^0

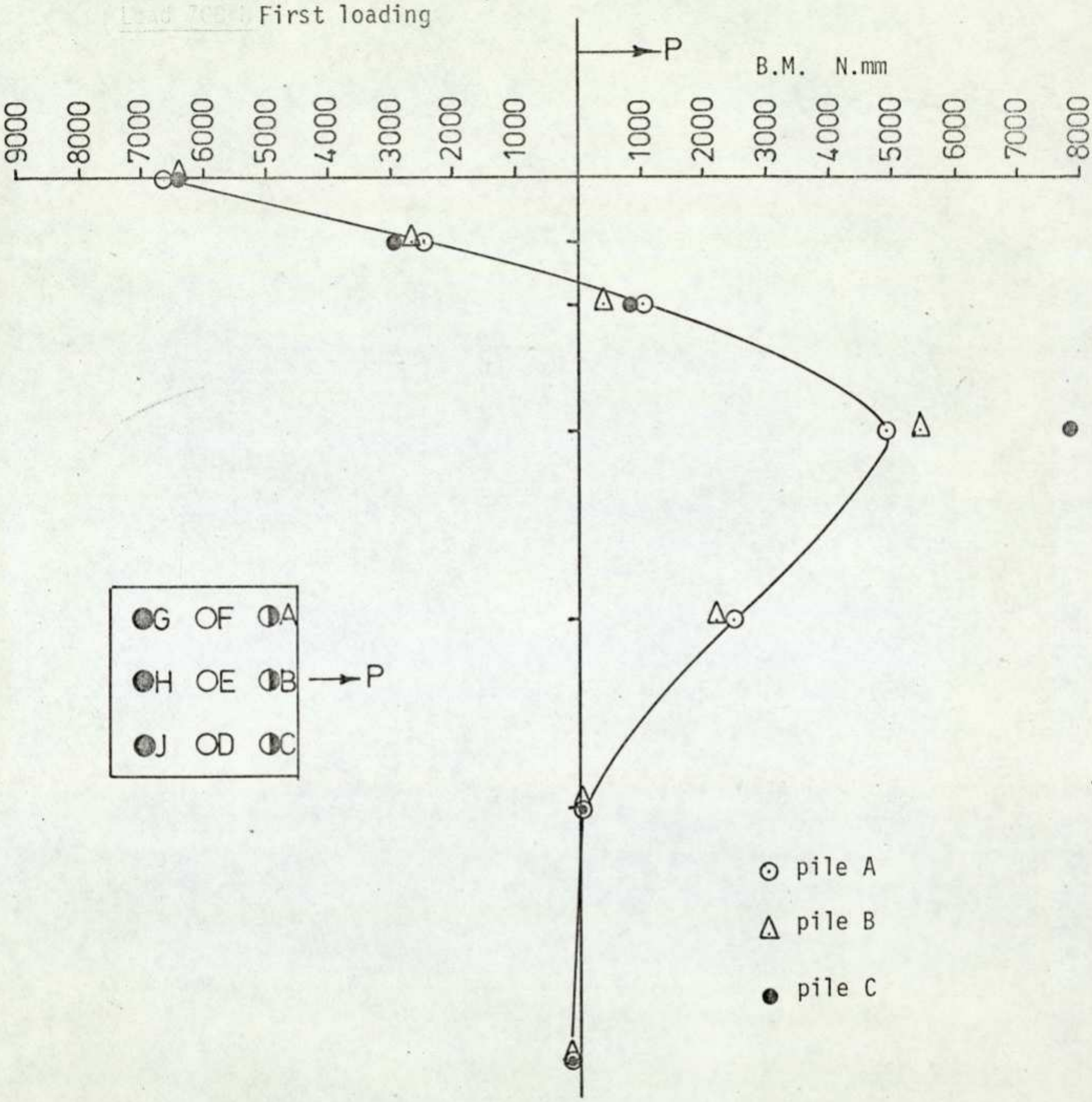


Fig. 7.72 Moment Vs Depth

Test No. 6

Pile Group

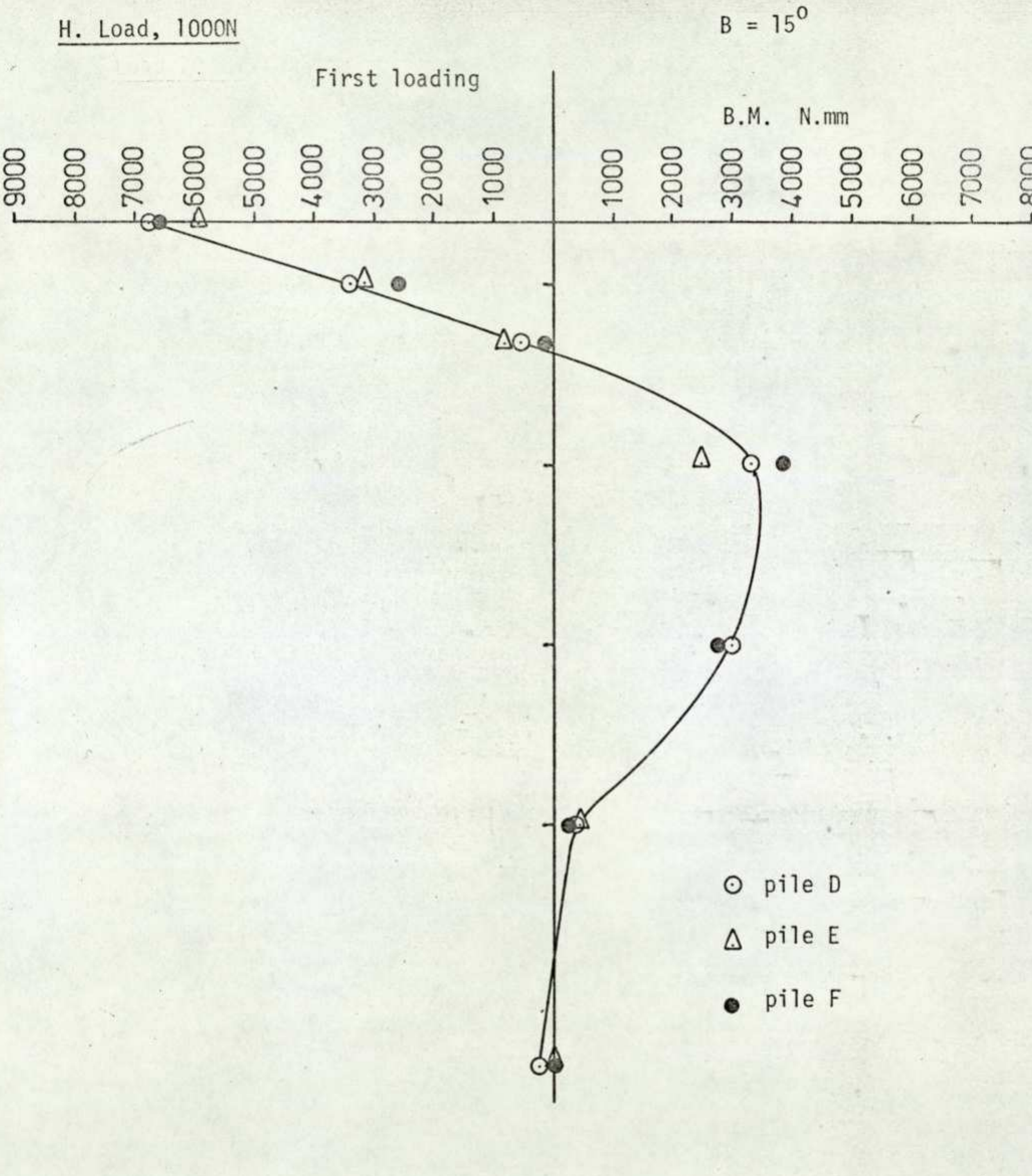


Fig. 7.73 Moment Vs Depth

Test No. 6

Pile Group

H. Load, 1000N

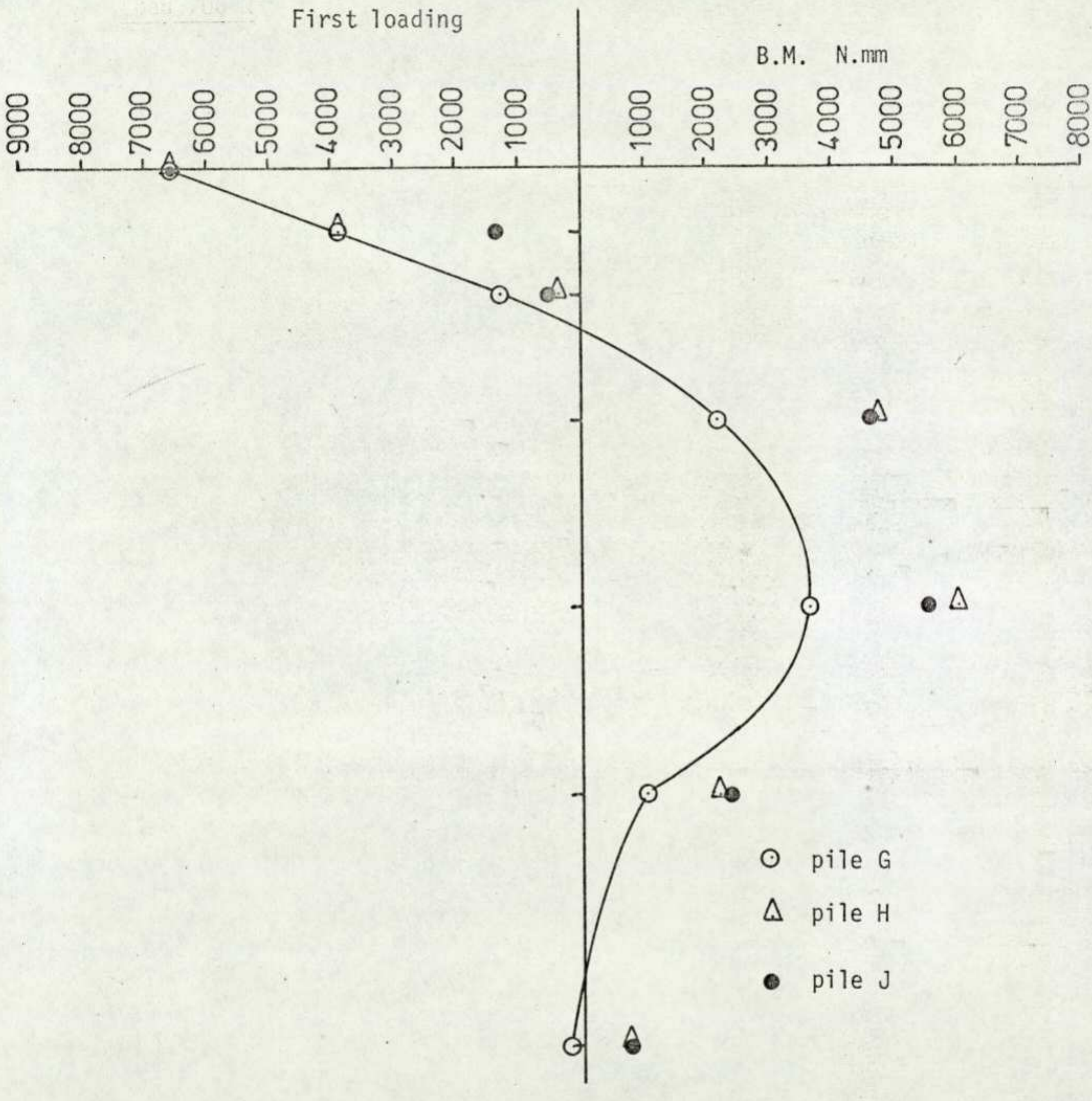


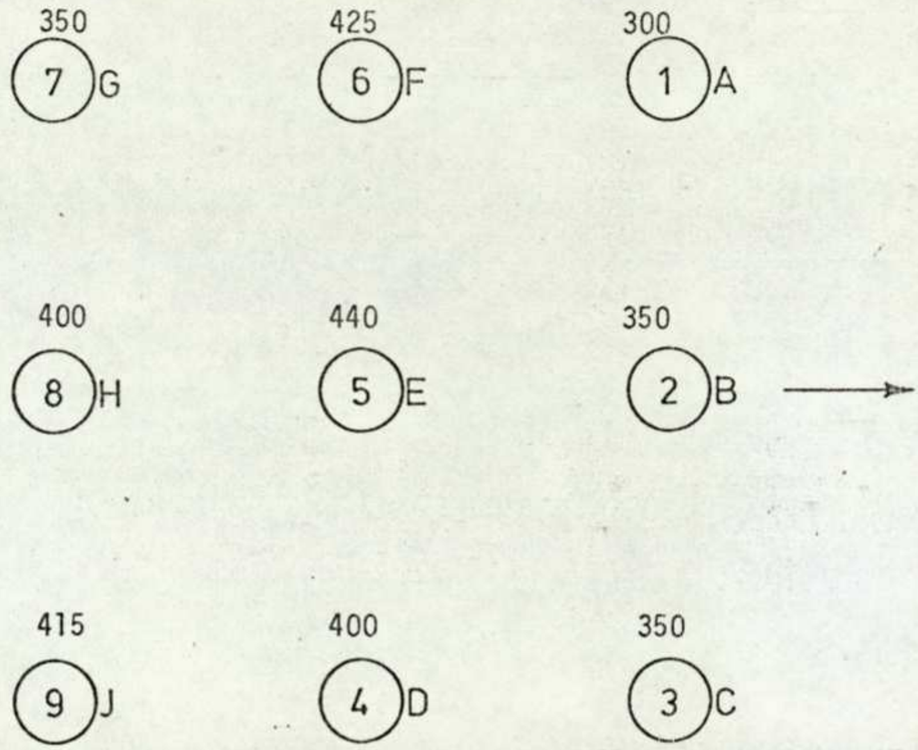
Fig. 7.74 Moment Vs Depth

Test No. 6

Test No. 7

3 x 3

+6B, -3B 15^{0}



Driving Resistance, kg

A, B, C, D, E, F +B 15^{0}

G, H, J -B 15^{0}

Fig. 7.75 Driving resistance and upwards movement in the piles during driving

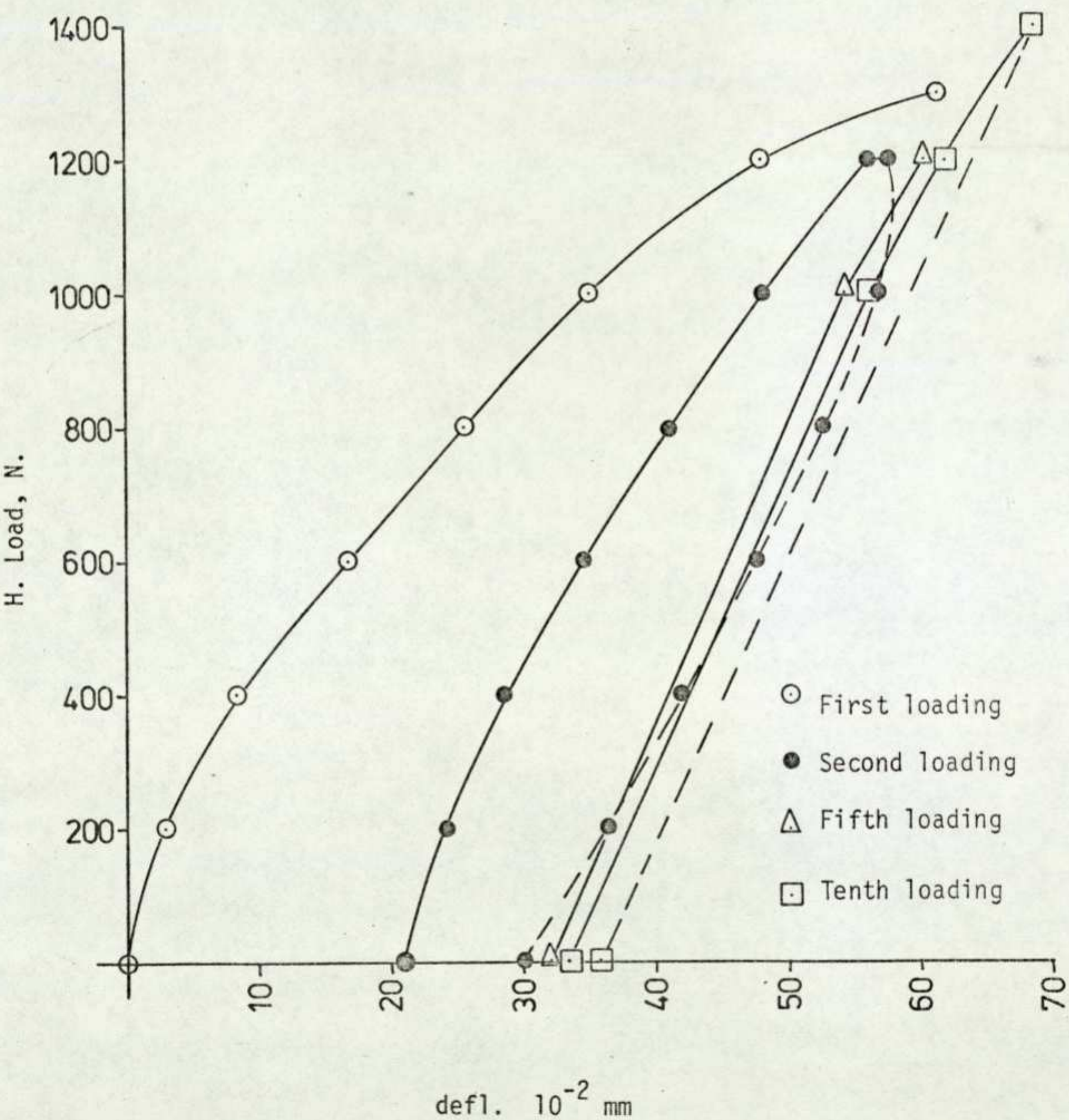


Fig. 7.76 Load Vs Defl. - pile group

Test No. 7

3 x 3

+4B, -3B 15^0

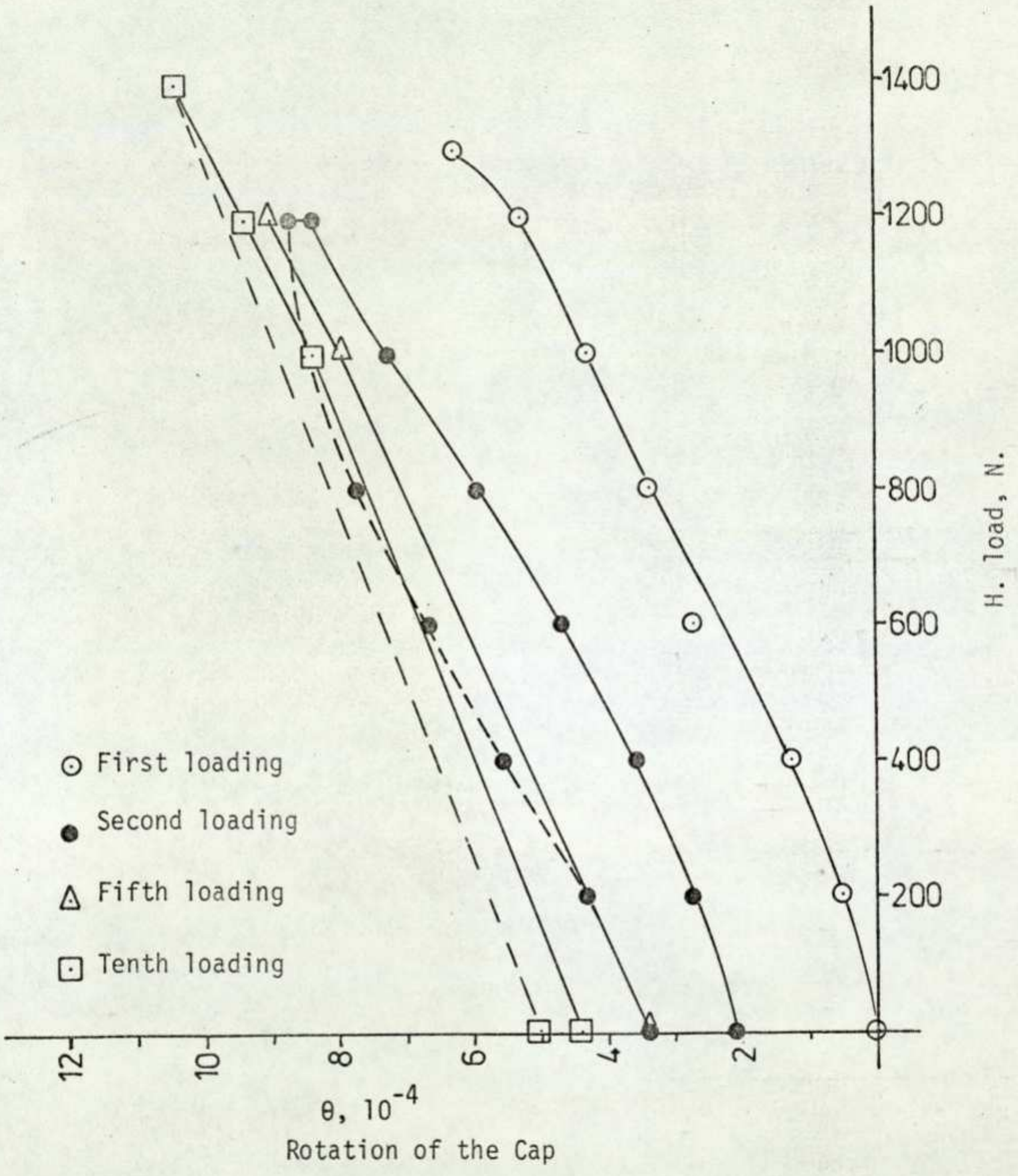


Fig. 7.77 Load Vs Rotation of the Cap

Pile Group

Test No. 7

Test No. 7

3 x 3

+6B, -3B, 15^{0}

H. Load, 1000N

-5734	-5940	-4973
7	6	1
2844	3399	3682
-5586	-5120	-5000
8	5	2
5952	2919	5409
-5586	-4826	-4270
9	4	3
5019	4240	3890

Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.78 Distribution of the maximum negative and positive bending moments in the piles.

Pile Group

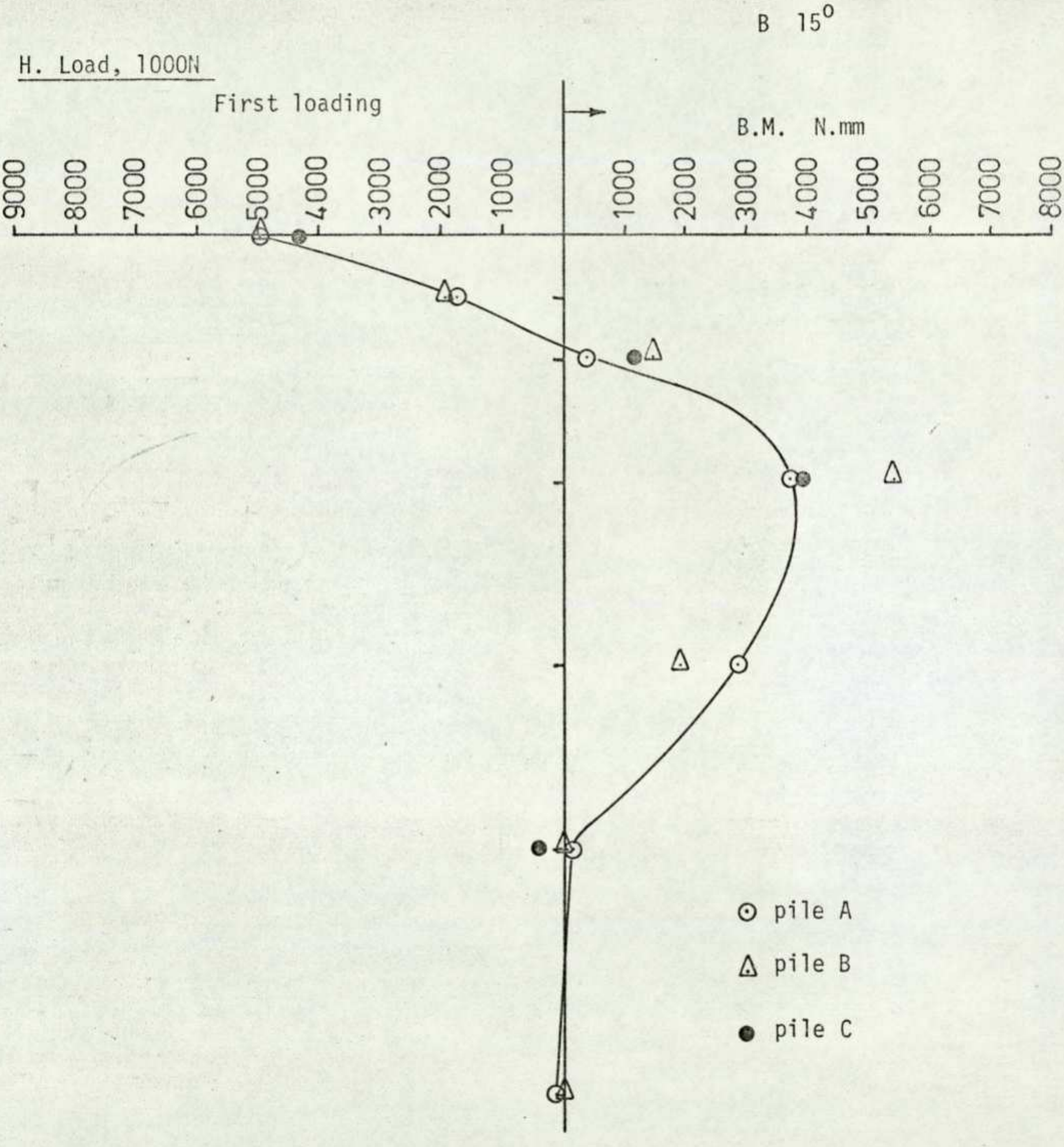


Fig. 7.79 Moment Vs Depth

Test No. 7

Pile Group

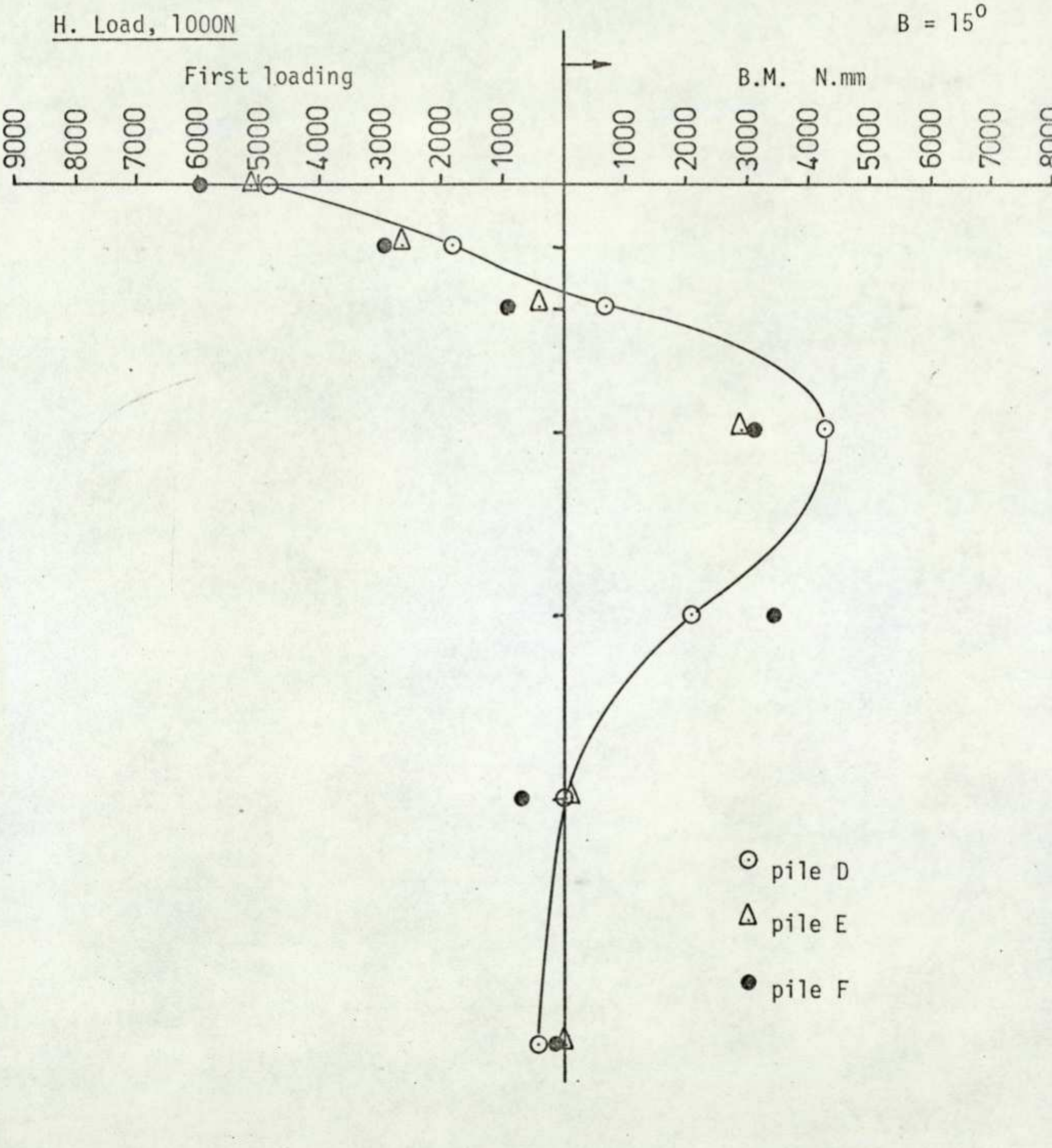


Fig. 7.80 Moment Vs Depth

Test No. 7

Pile Group

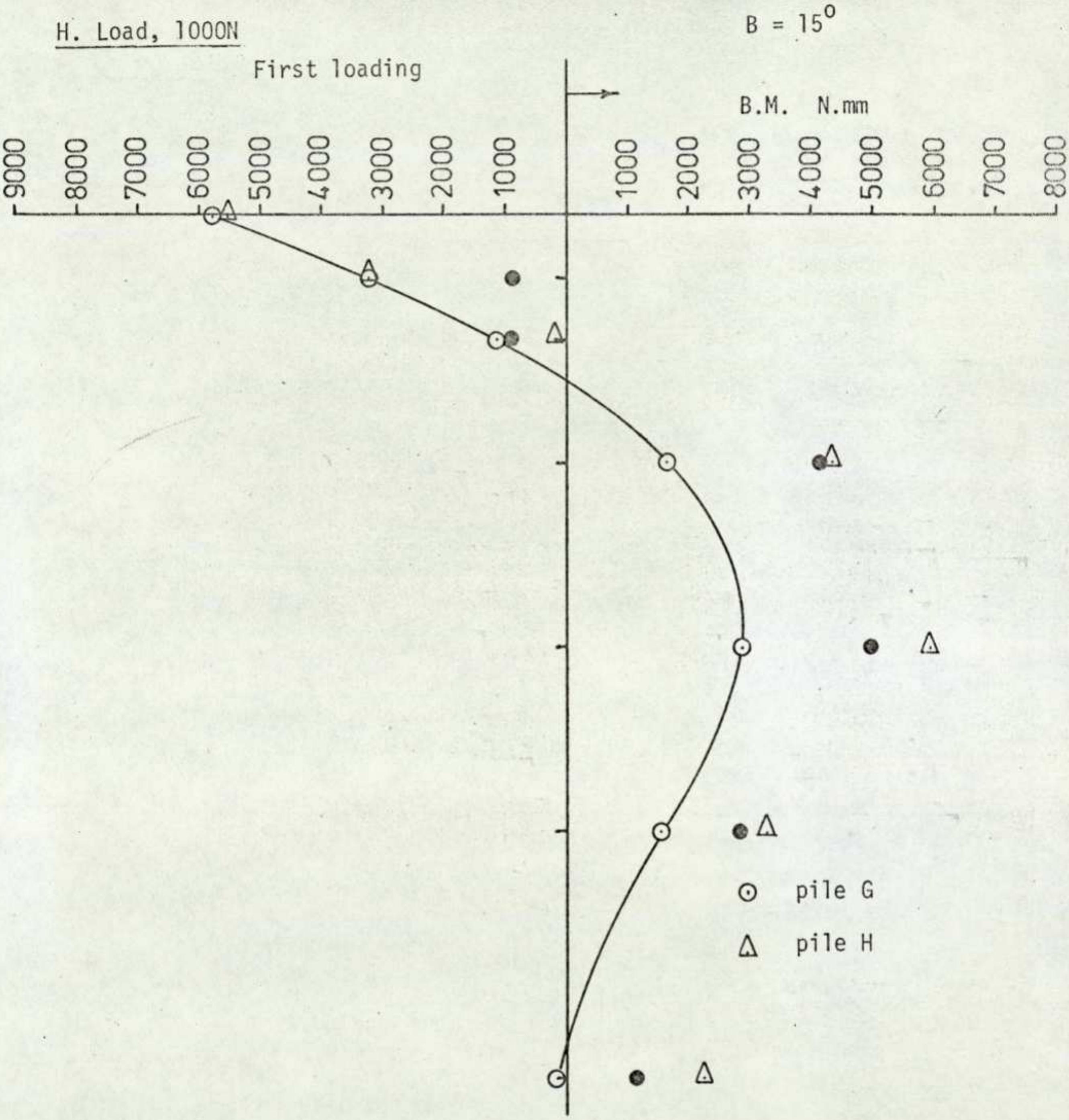


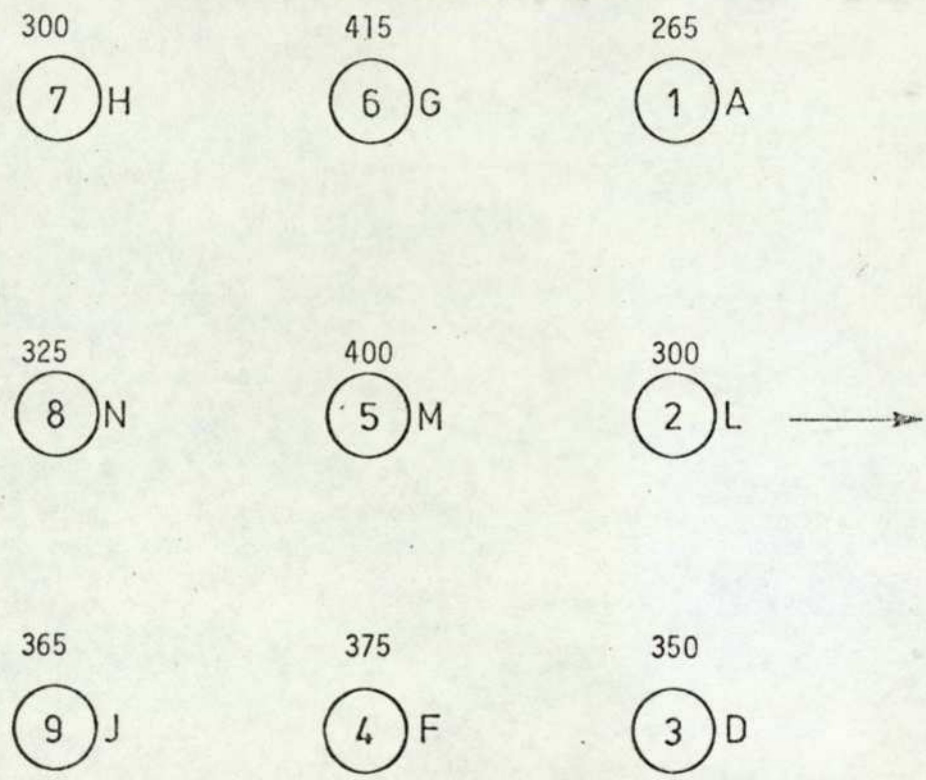
Fig. 7.81 Moment Vs Depth

Test No. 7

Test No. 10

3 x 3

+3B, 3V, -3B 30^0



Driving Resistance, kg

A, L, D	+B 30^0
F, M, G	V
H, N, J	-B 30^0

Fig. 7.82 Driving resistance in the piles.

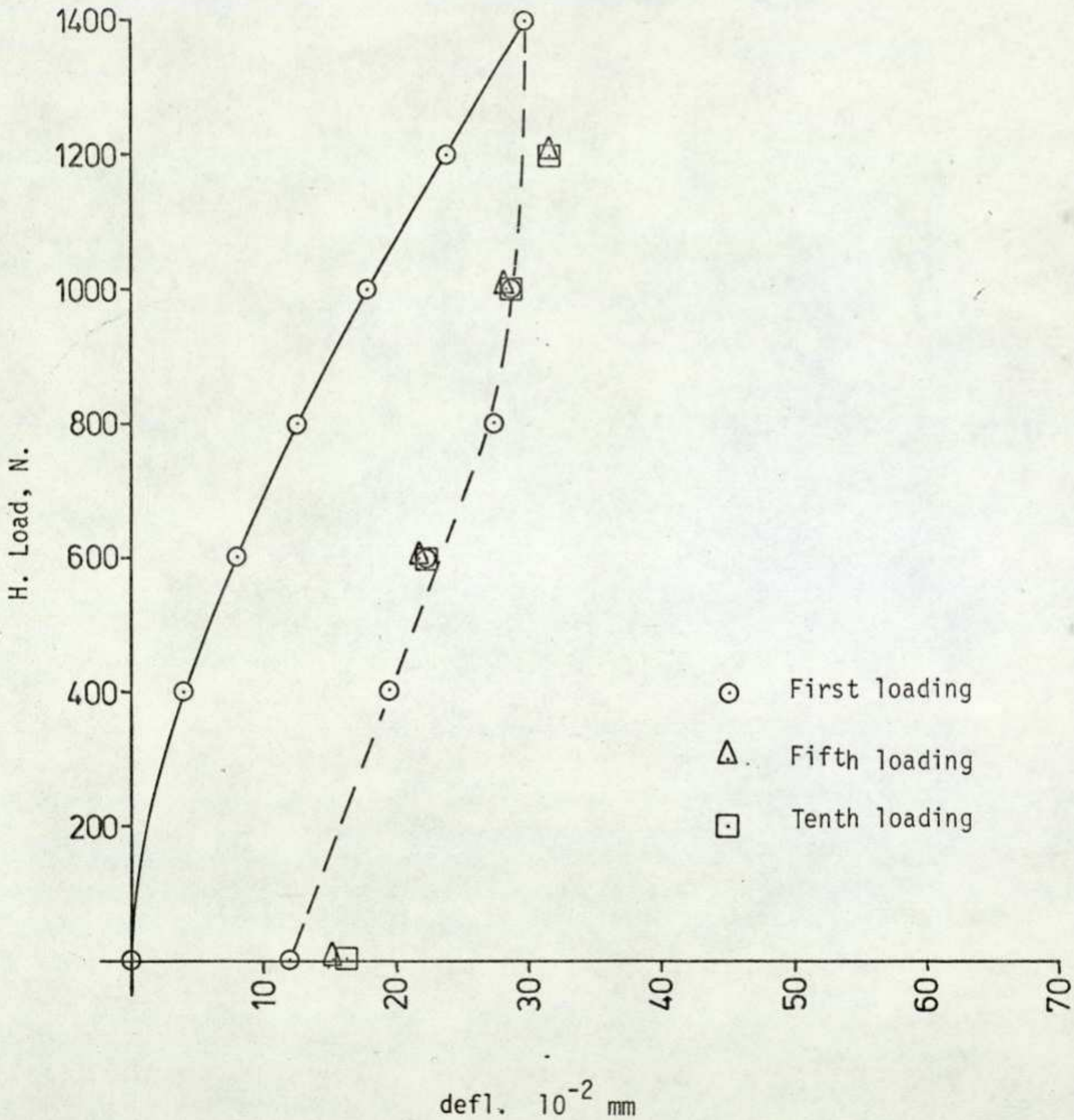
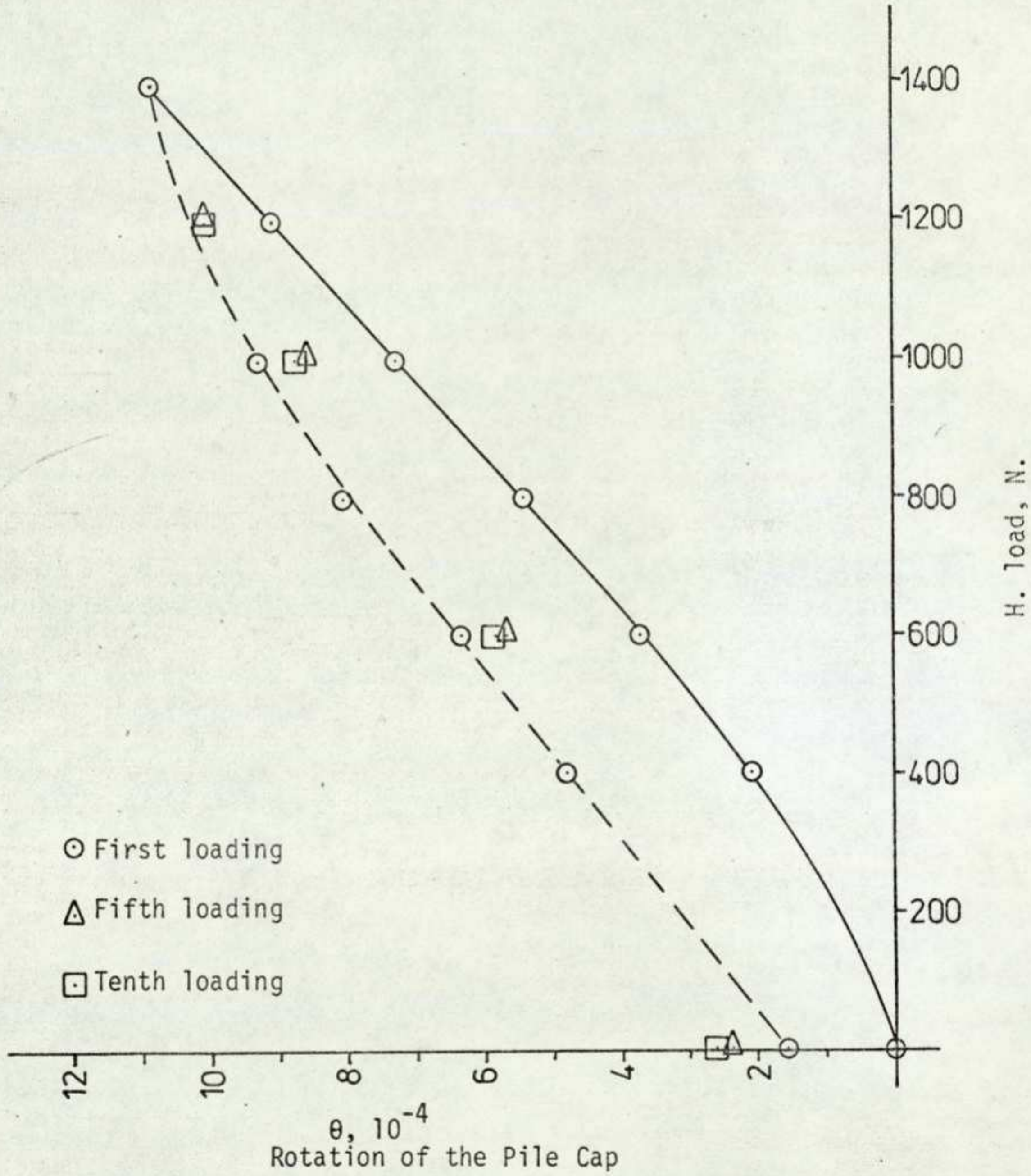


Fig. 7.83 Load Vs defl. - pile group

Test No. 10



<u>Fig. 7.84 Load Vs Rotation of the Pile Cap</u>

<u>Test No. 10</u>

Test No. 10

3 x 3

+3B, 3V , -3B, 30^{0}

H. Load, 1000N

-4666	-5332	-4086
7	6	1
3445	2034	2508
-3024	-5200	-5772
8	5	2
	2470	1800
-4899	-5412	-4494
9	4	3
3363	2024	3275

Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.85 Distribution of the maximum negative and positive bending moments in the piles.

Pile Group

H. Load, 100N

First loading

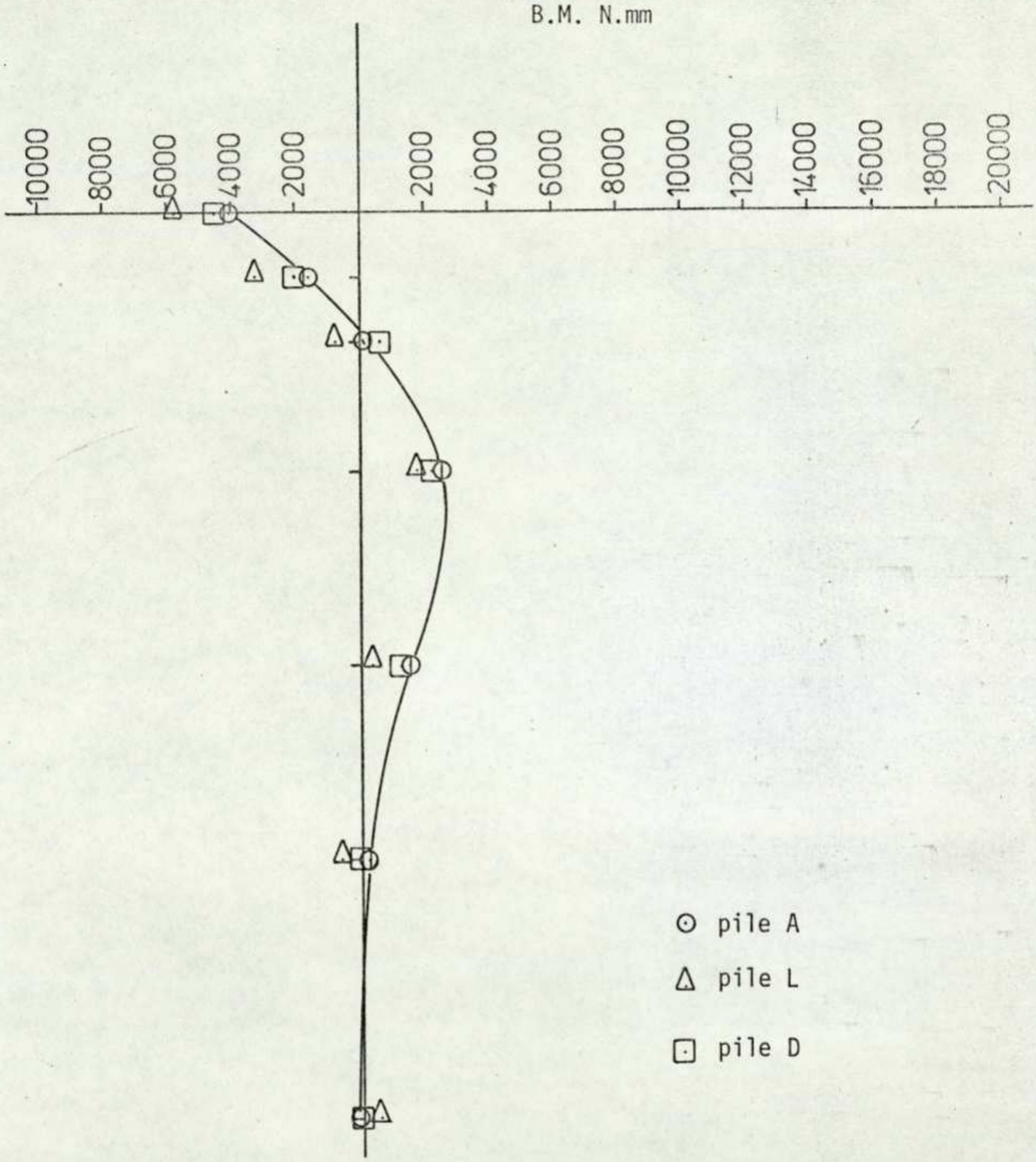


Fig. 7.86 Moment Vs Depth

Test No. 10

H. Load, 1000N

First loading

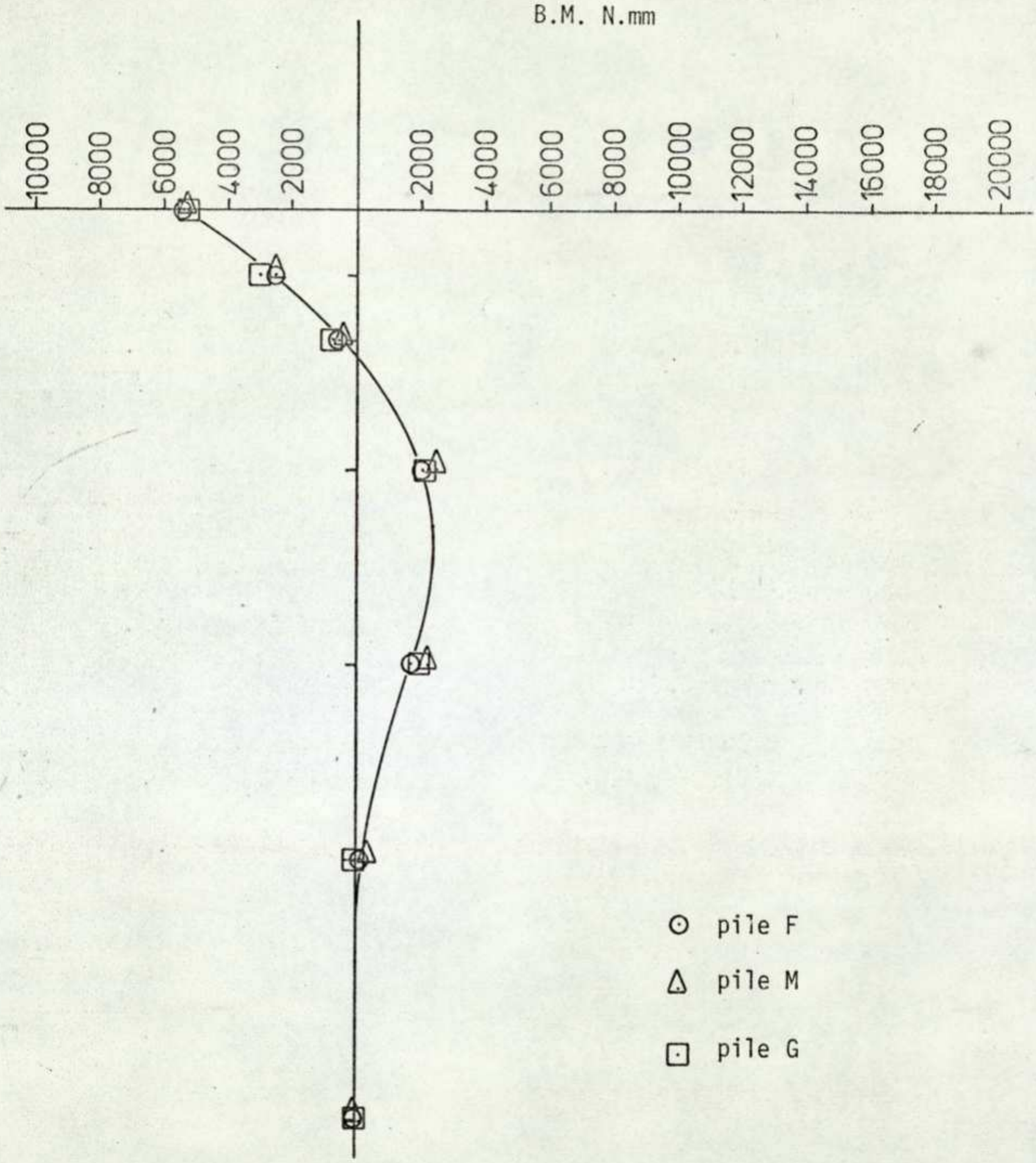


Fig. 7.87 Moment Vs Depth

Test No. 10

Pile Group

H. Load, 1000N

First loading

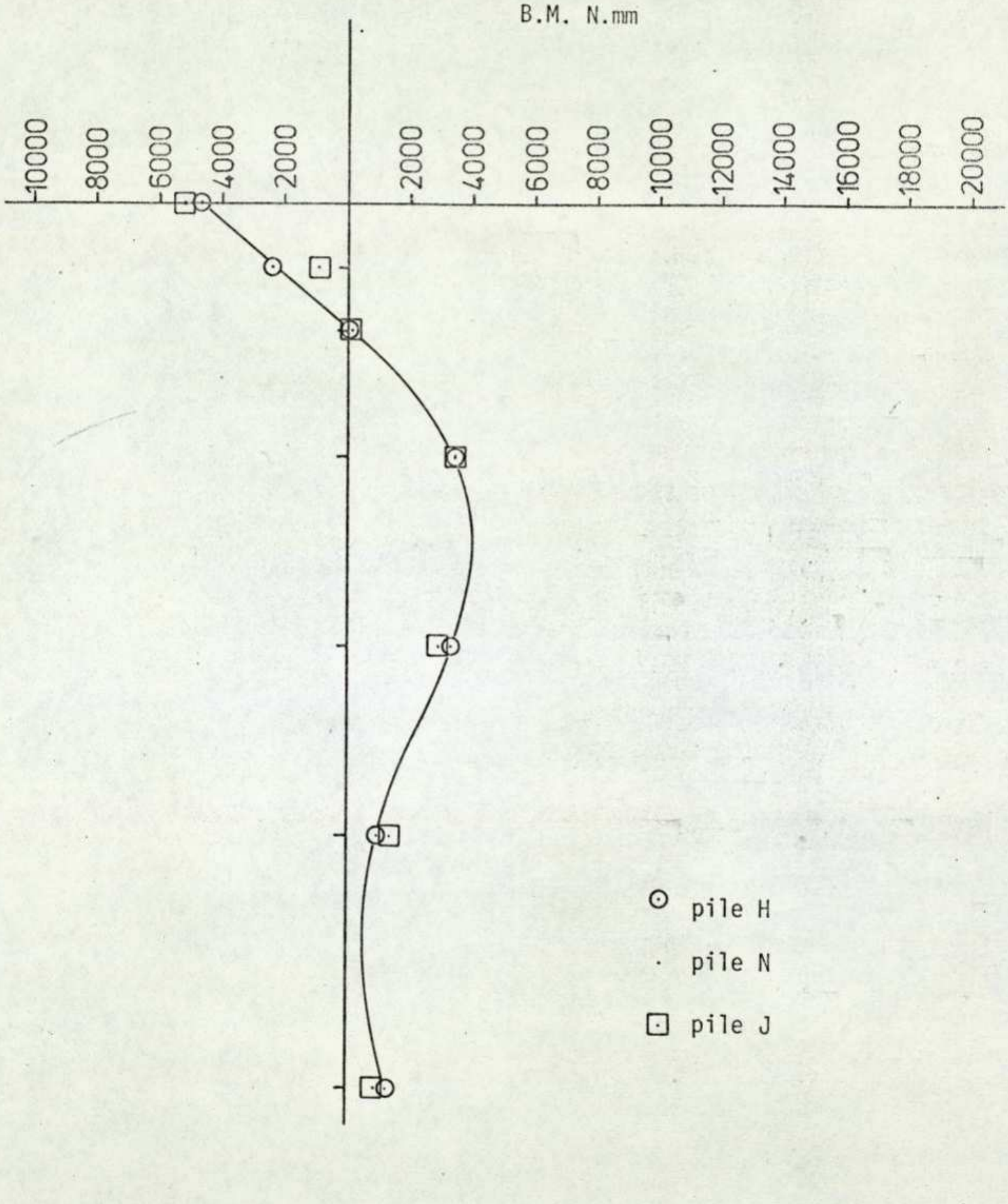


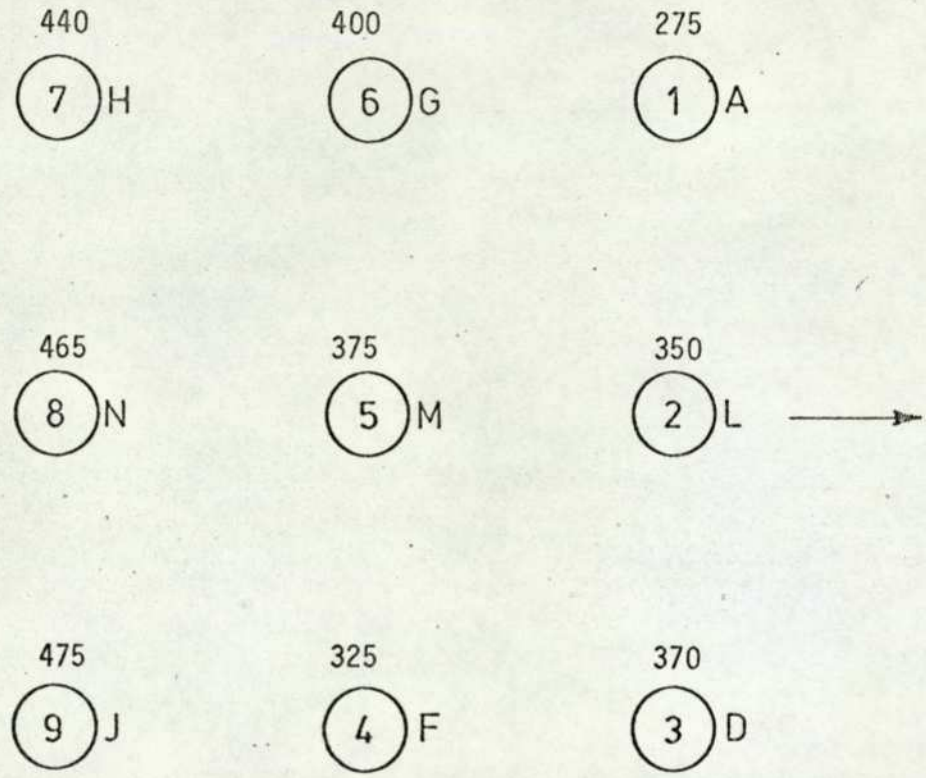
Fig. 7.88 Moment Vs Depth

Test No. 10

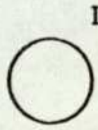
Test No. 11

3 x 3

+3B 30°
6



Driving Resistance, kg



A, L, D +B 30°

F, M, G, H, N, J V

Fig. 7.89 Driving resistance in the piles

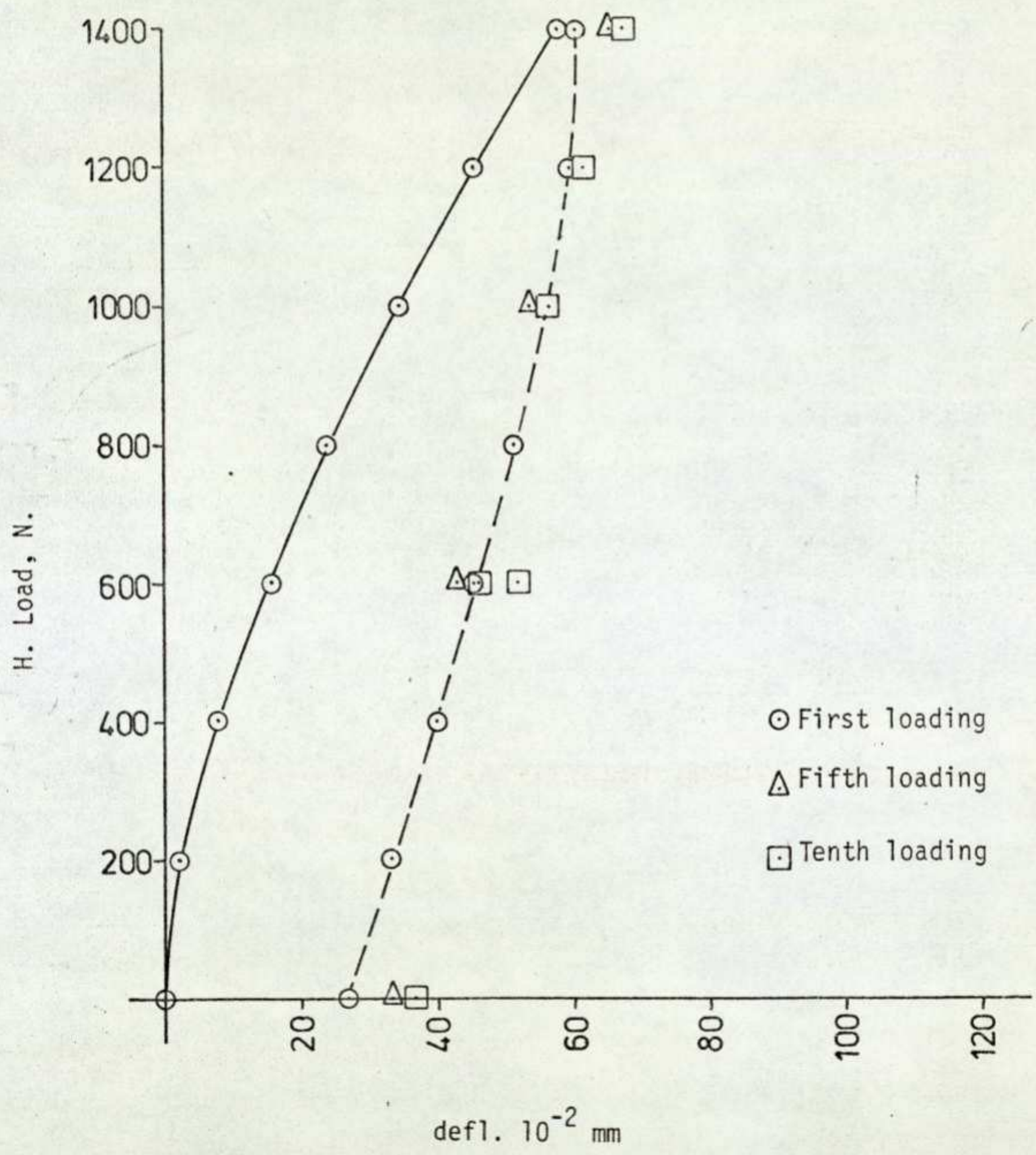


Fig. 7.90 load Vs defl. - Pile Group

Test No. 11

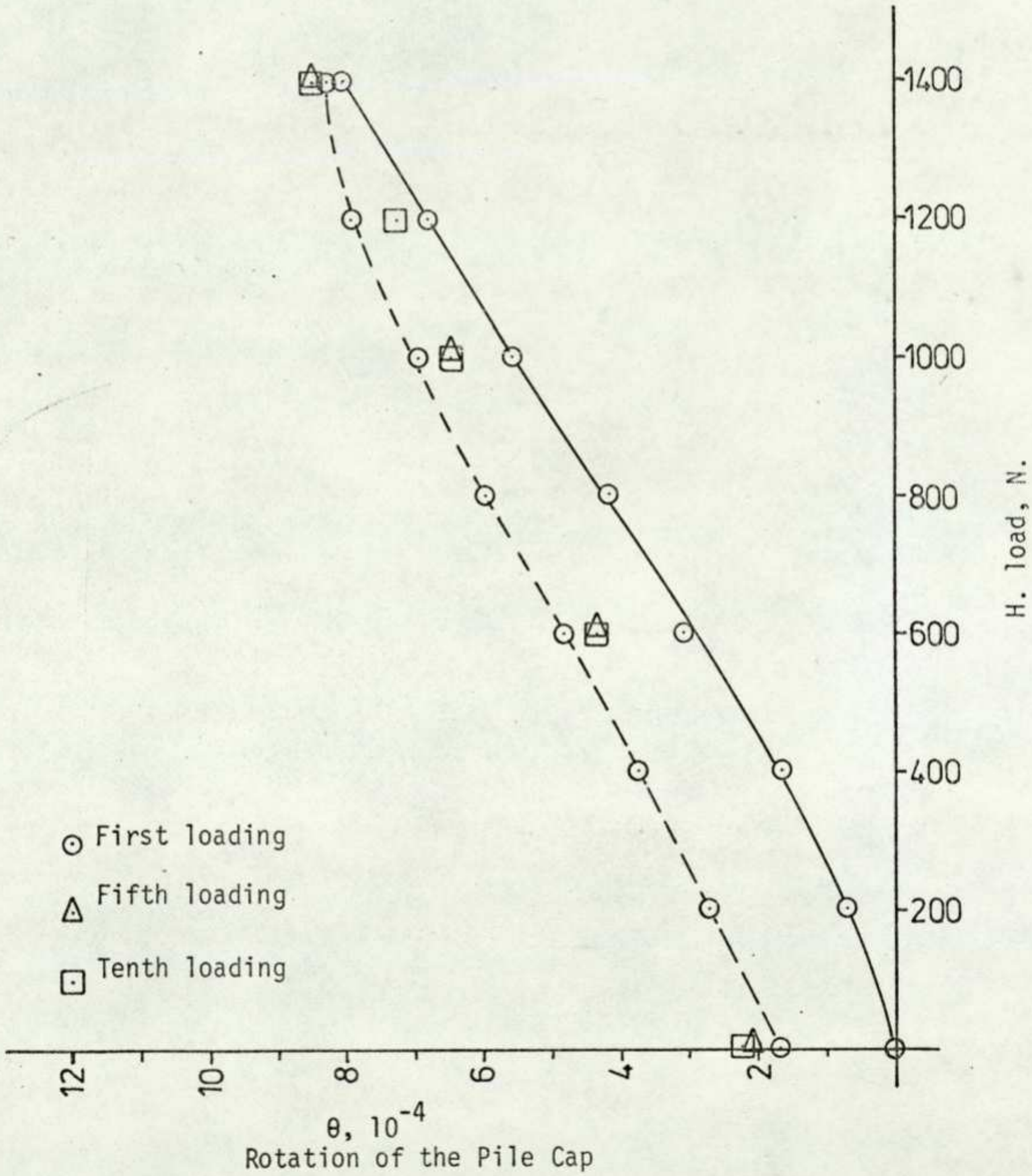


Fig. 7.91 Load Vs Rotation of the Pile Cap

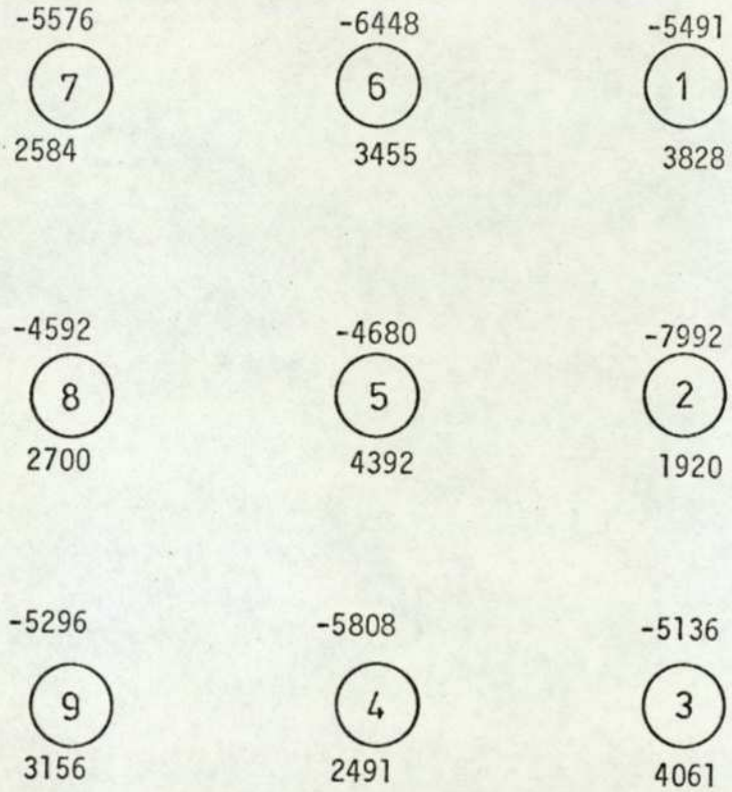
Test No. 11

Test No. 11

3 x 3

+3B, 6V , 30^0

H. Load, 1000N



Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.92 Distribution of the maximum negative and positive bending moments in the piles

H. Load, 1000N

First loading

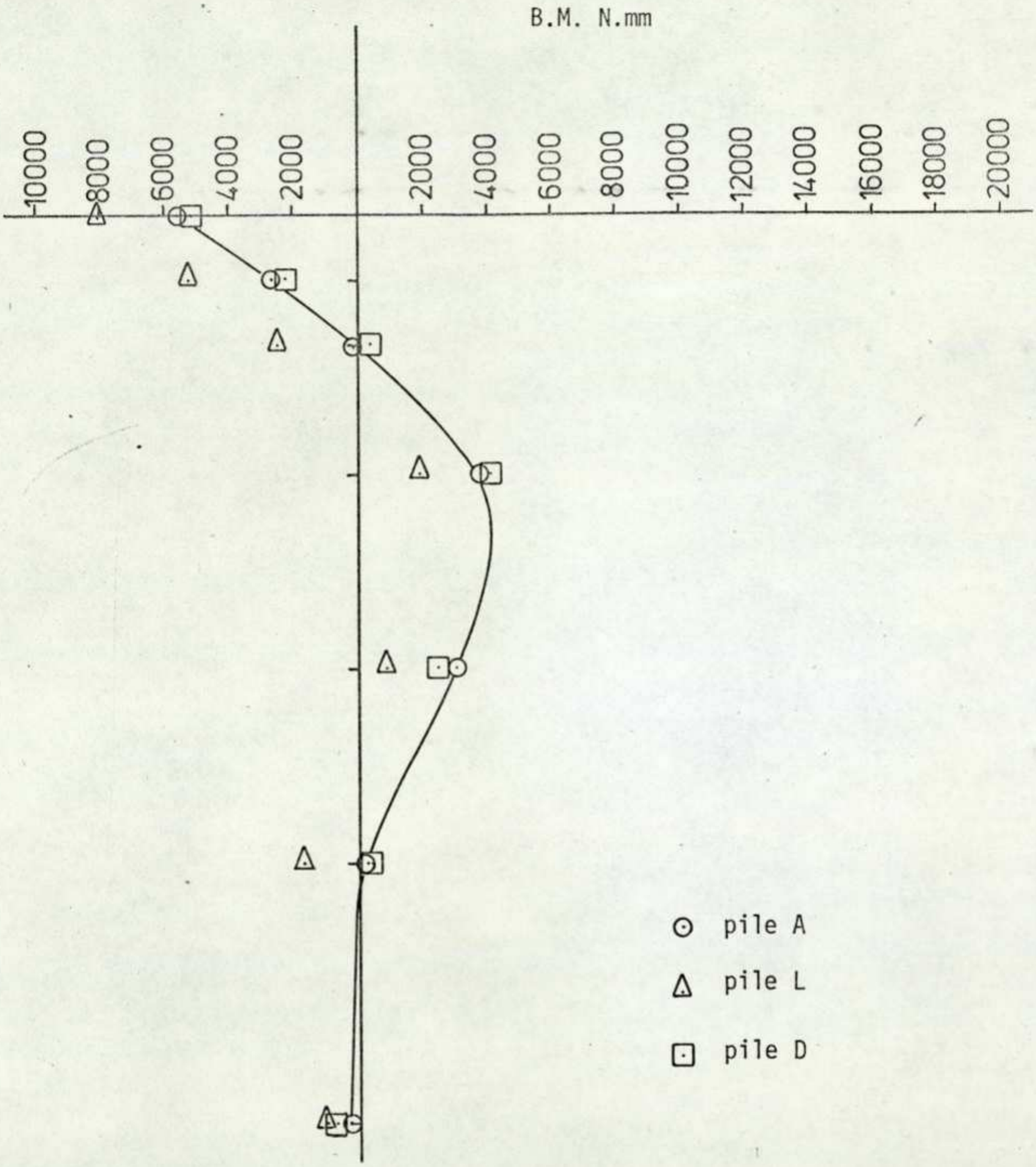


Fig. 7.93 Moment Vs Depth

Test No. 11

H. Load, 1000N

First loading

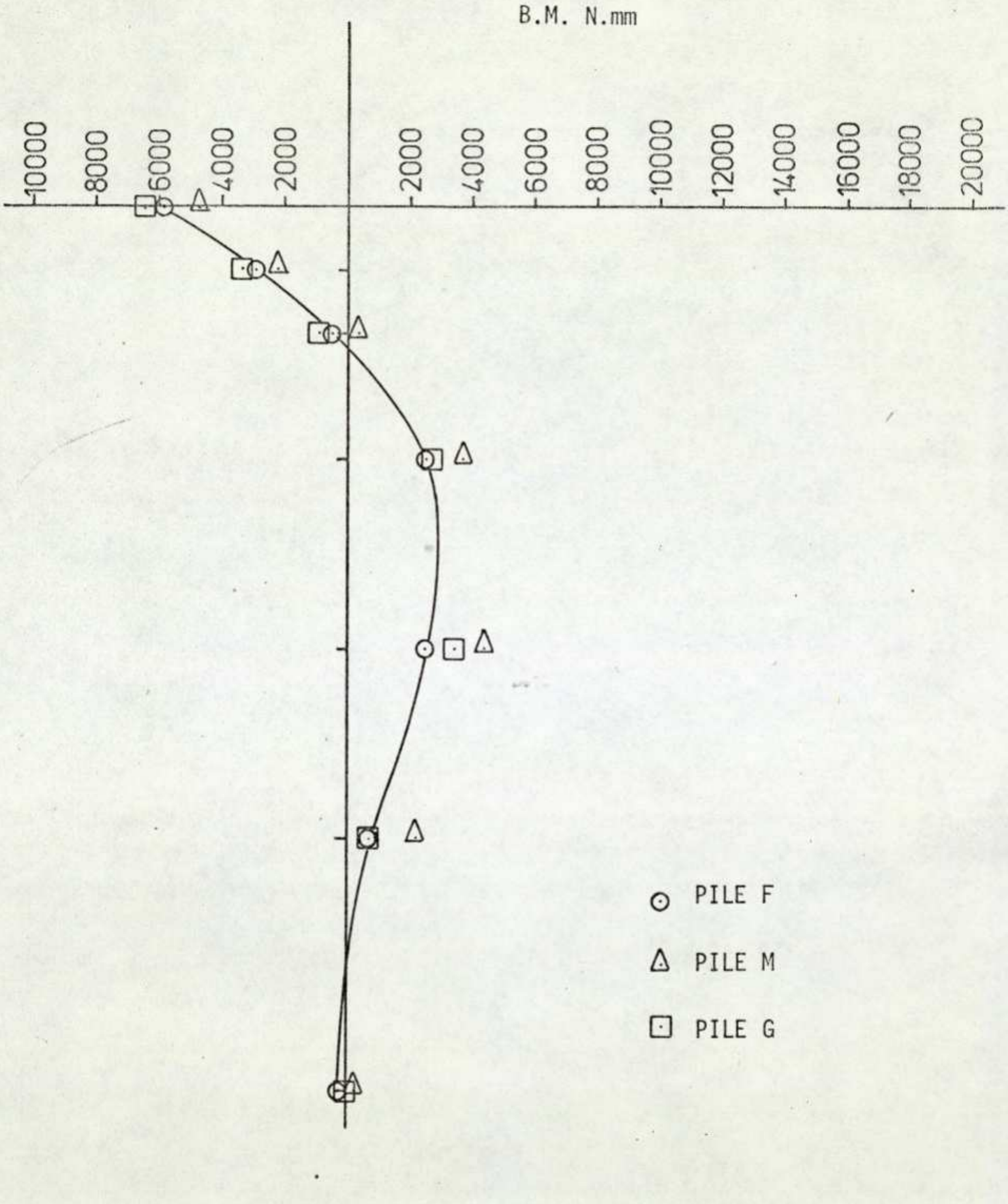


Fig. 7.94 Moment Vs Depth

Test No. 11

H. Load, 1000N

First loading

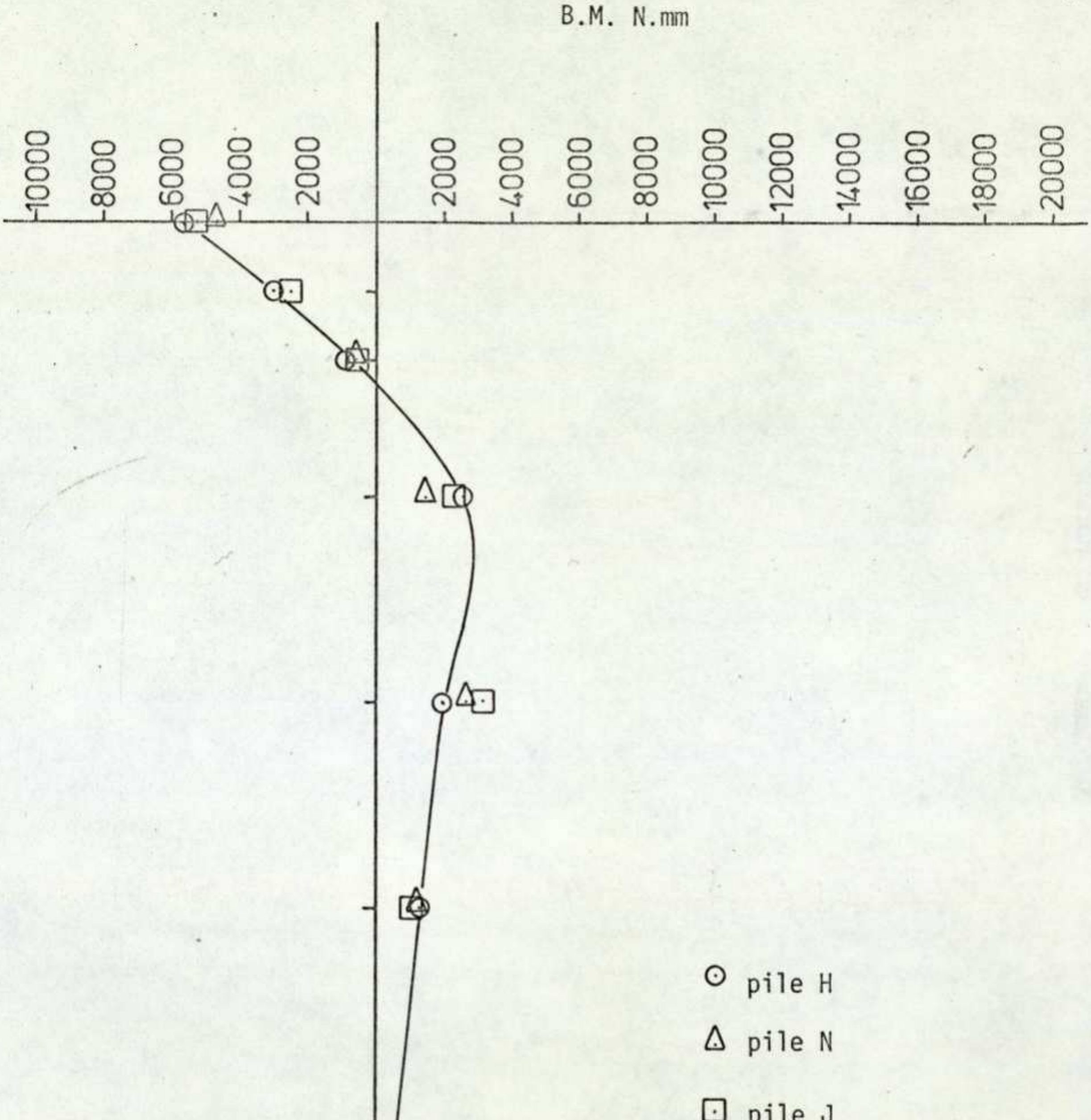


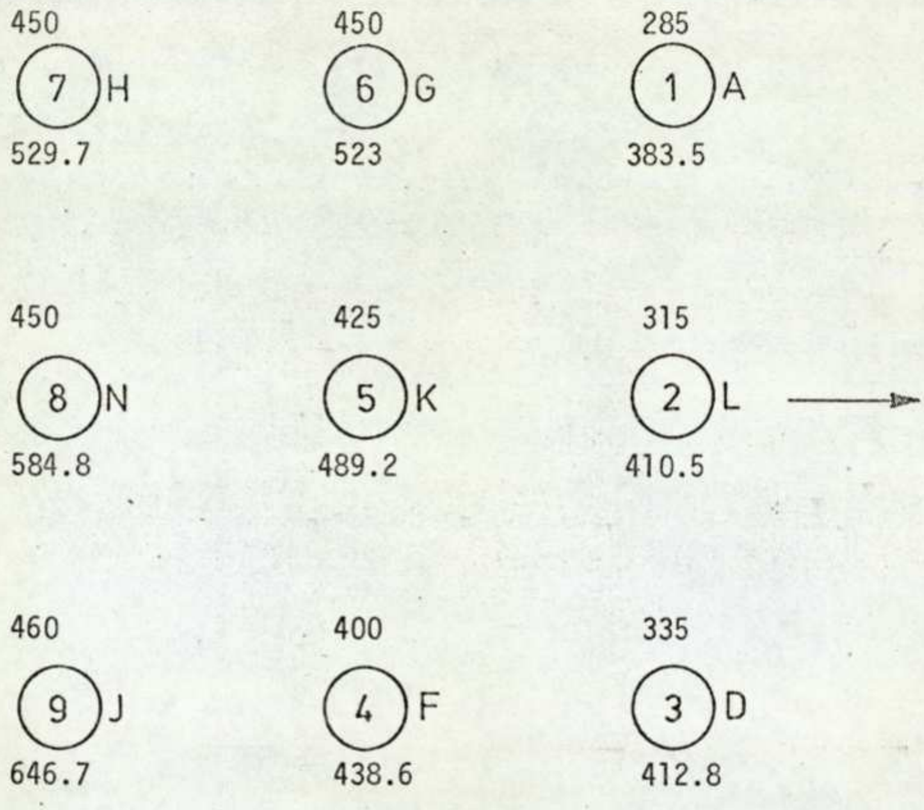
Fig. 7.95 Moment Vs Depth

Test No. 11

Test No. 15

3 x 3

9V



Driving Resistance, kg

Re-Driving Resistance, kg

Σ Re-Driving Force = 4418.8 kg

Fig. 7.96 Driving and Re-Driving resistance in the piles

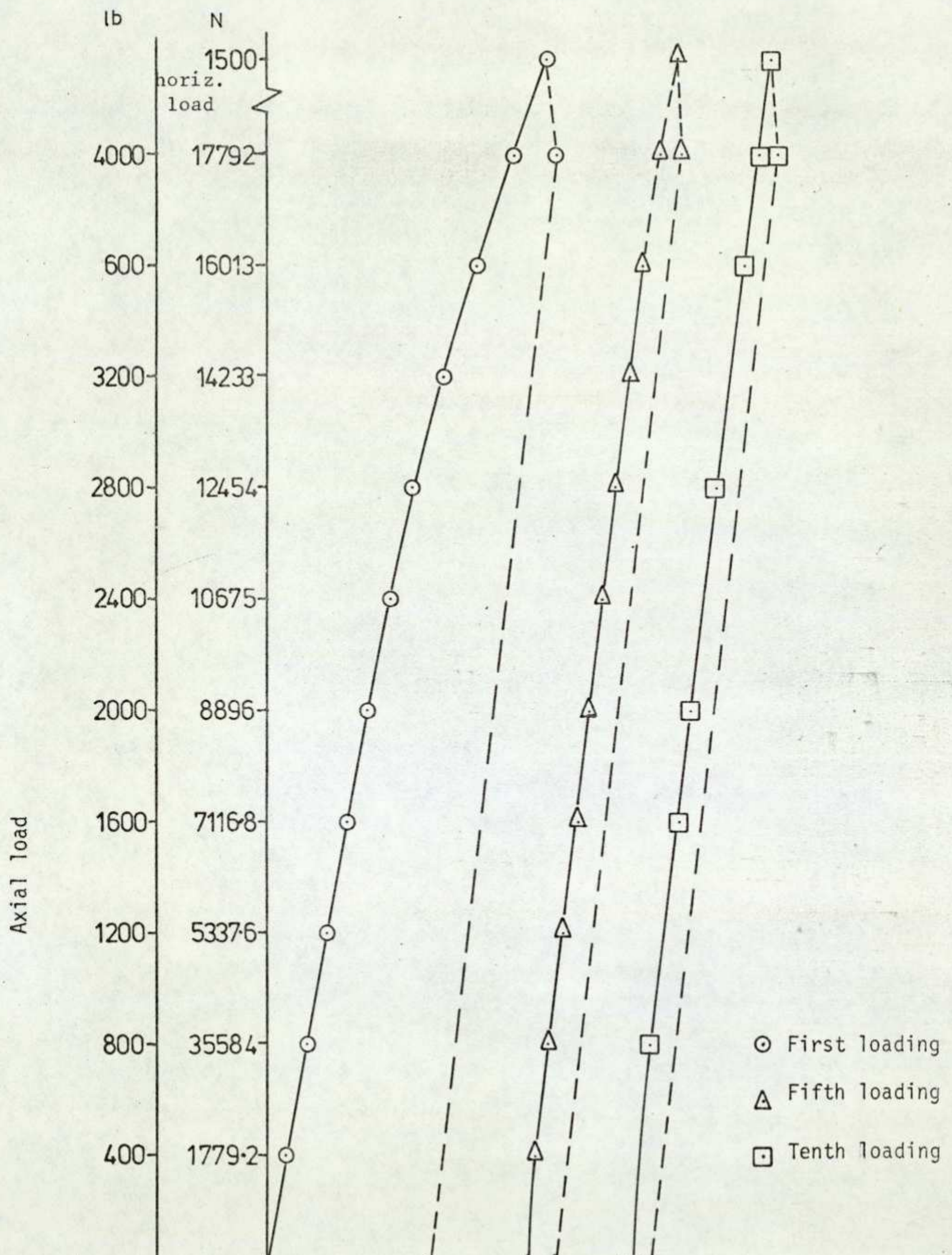


Fig. 7.97 Load Vs Settlement - Test No. 15

Test No. 15

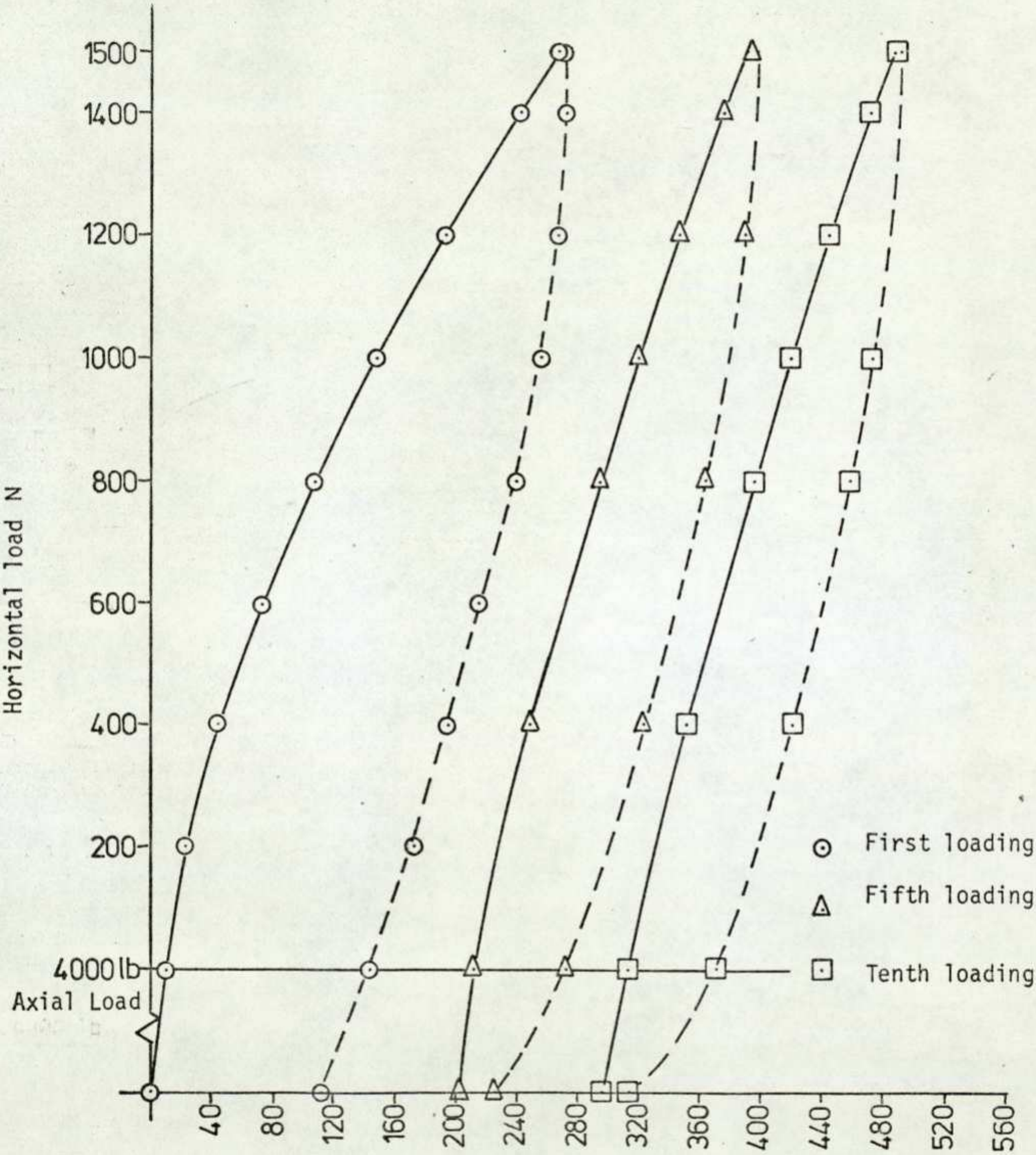
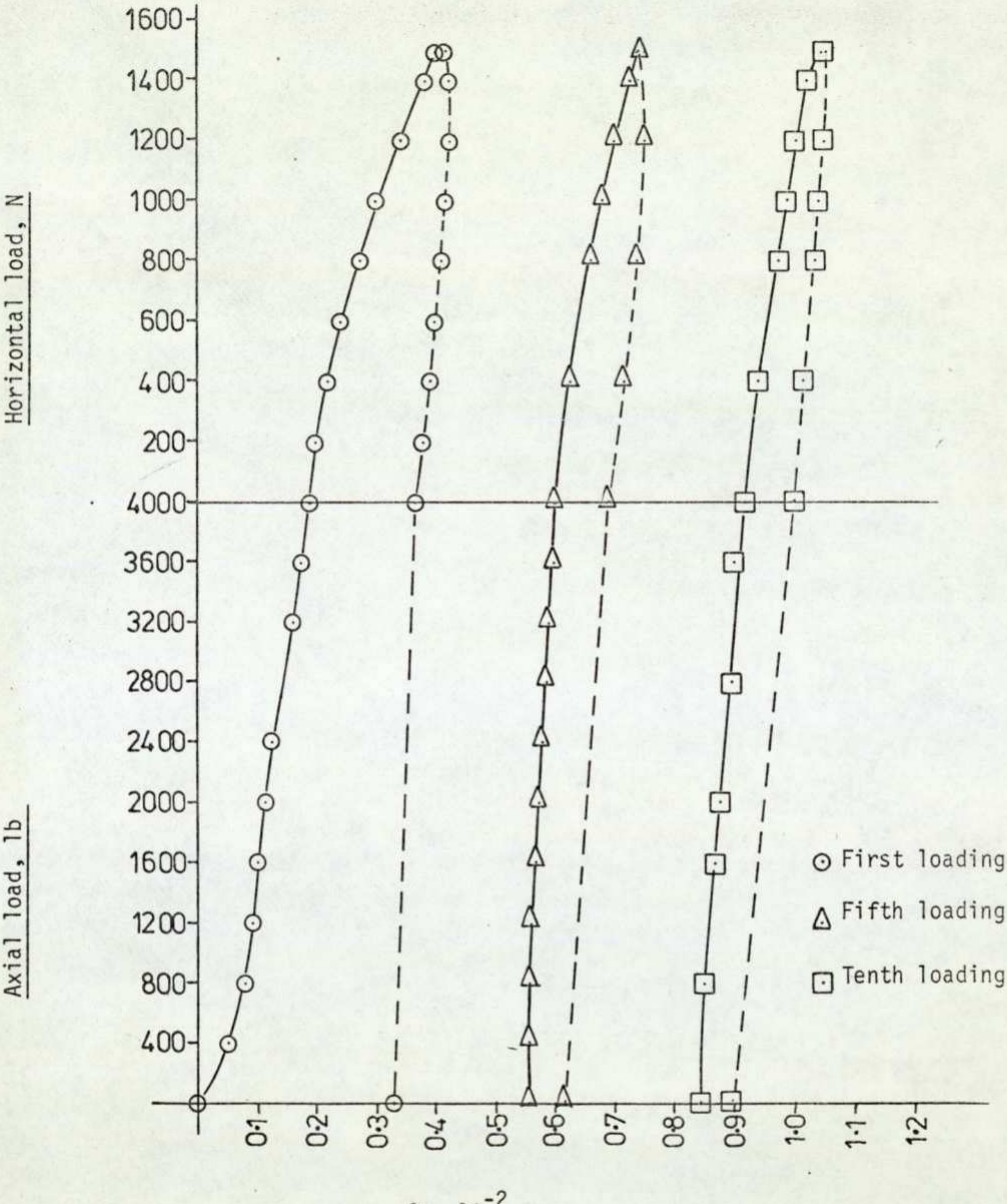


Fig. 7.98 load Vs defl. - pile Group

defl. in x - direction

Test No. 15



Rotation of the Cap

Fig. 7.99 Load Vs Rotation of the Pile Cap

Test No. 15

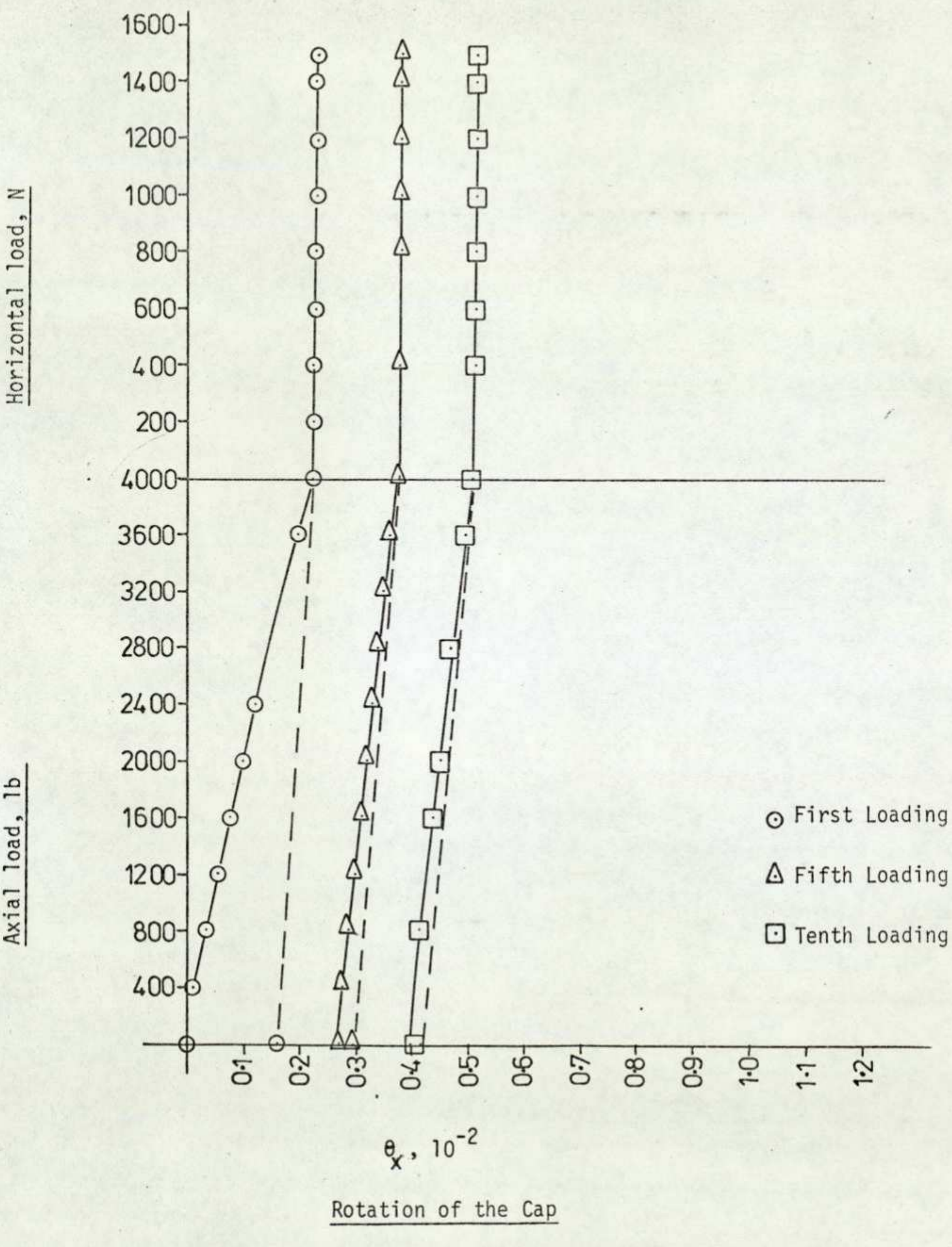
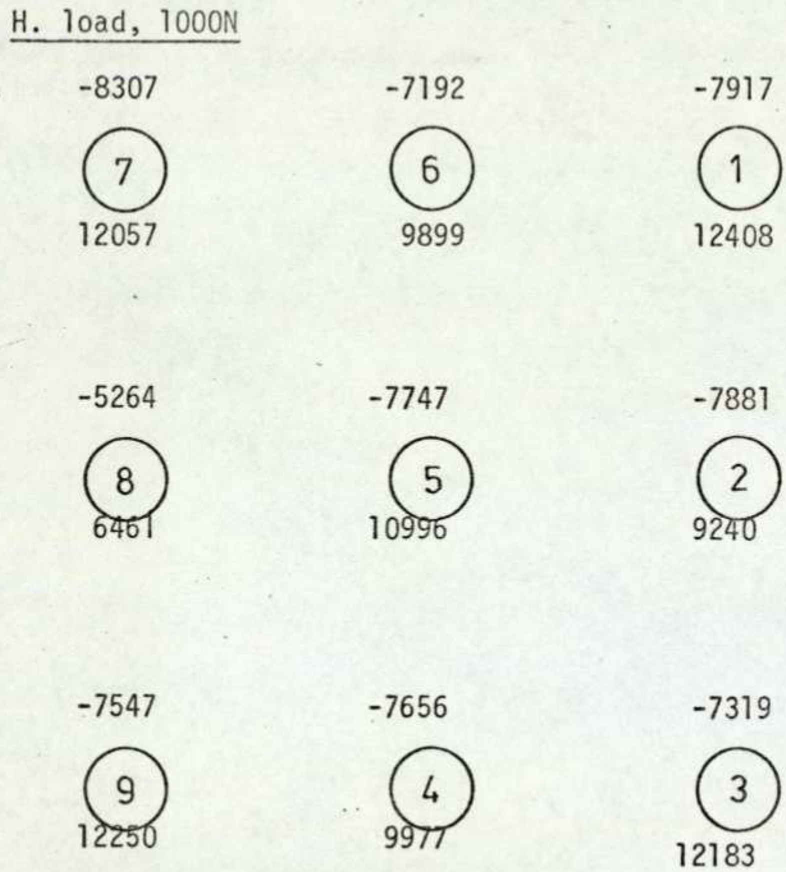


Fig. 7.100 Load Vs Rotation of the Pile Cap

Test No. 15

3 x 3
9V



Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.101 Distribution of the maximum negative and positive moments in the piles

Pile Group

H. Load, 1000N

First loading

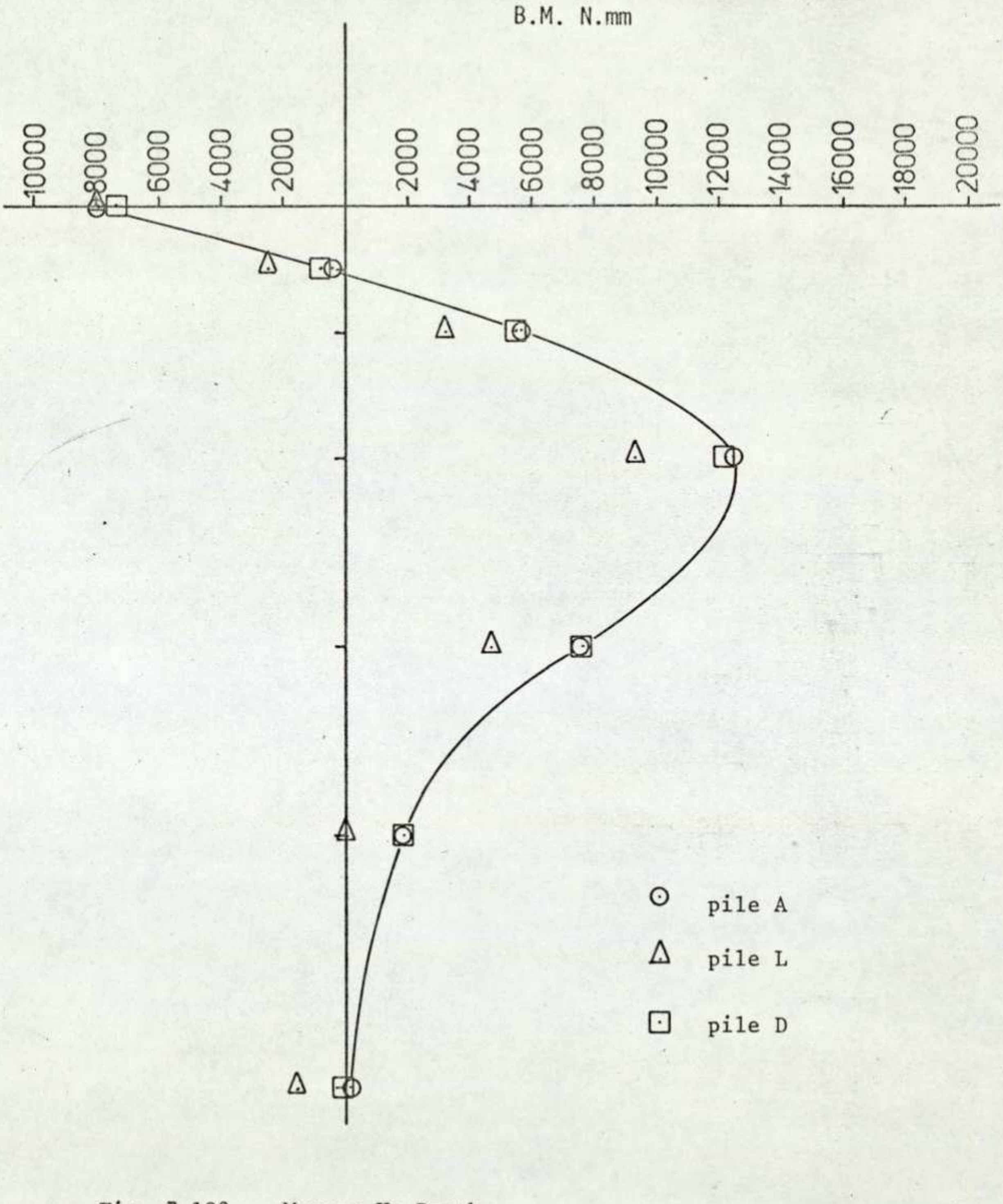


Fig. 7.102 Moment Vs Depth

Test No. 15

Pile Group

H. Load, 1000N

First loading

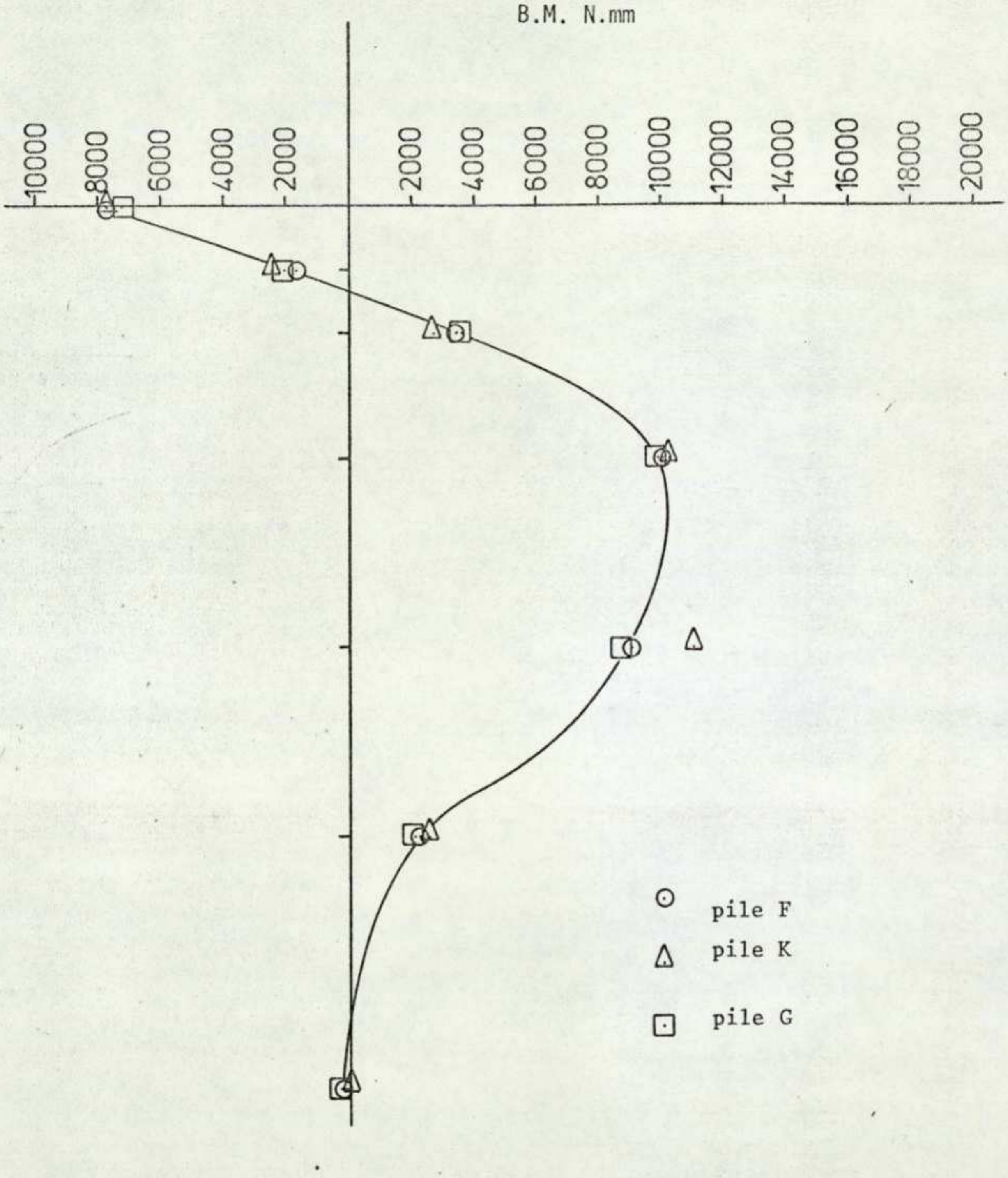


Fig. 7.103 Moment Vs Depth

Test No. 15

Pile Group

H. Load, 1000N

First loading

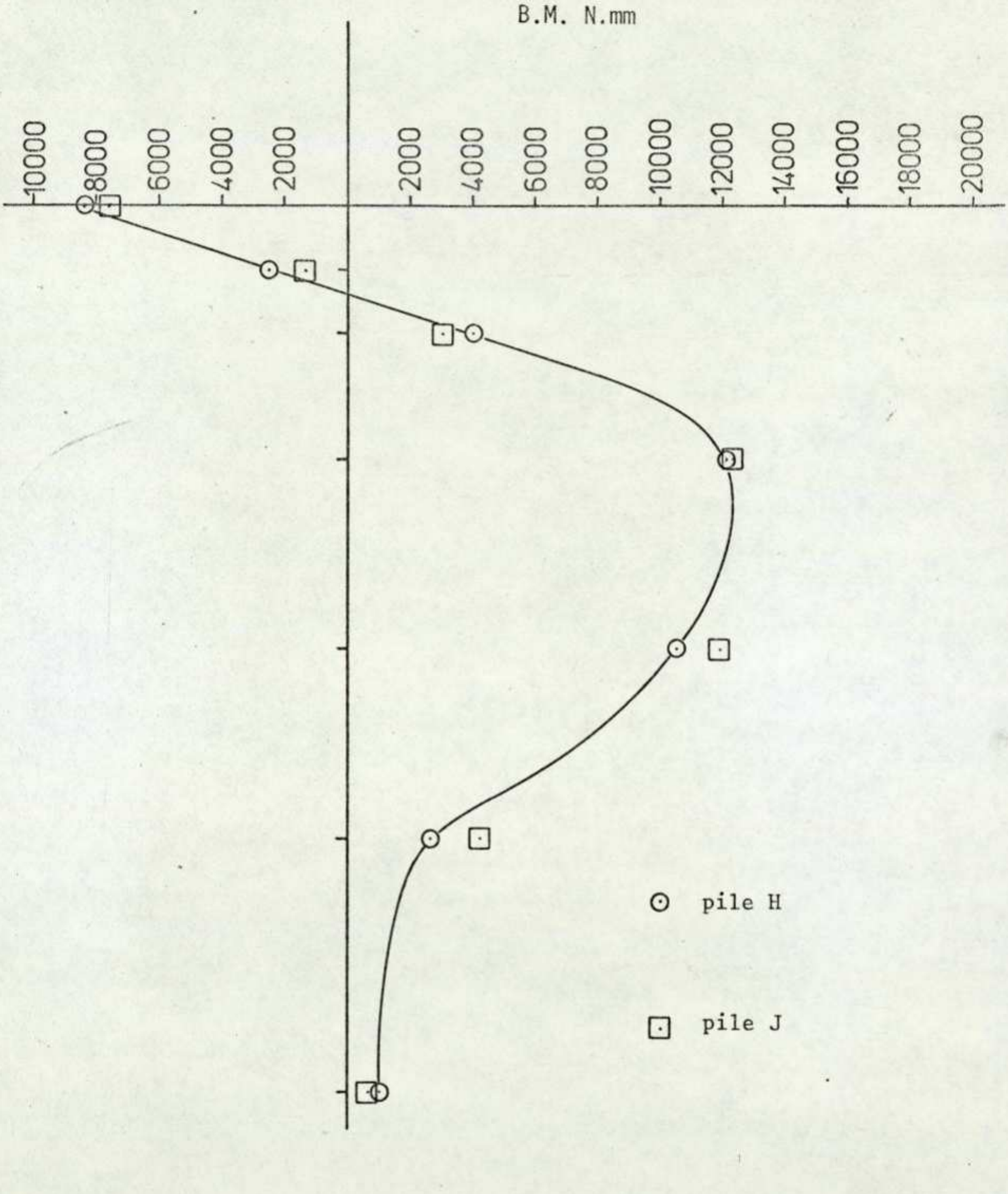


Fig. 7.104 Moment Vs Depth

Test No. 15

Test No. 16

3 x 3

+3B 15^{0}

3V

-3B 15^{0}

435	500	340
7 H	6 G	1 A
475.7	652.2	315.2
475	475	375
8 N	5 K	2 L
646.7	562.4	347.4
515	400	400
9 J	4 F	3 D
826.3	399.1	455.3

Driving Resistance, kg

Re-Driving Resistance, kg

Σ Re-Driving Force = 4680.3 kg

Fig. 7.105 Driving and Re-Driving resistance in the piles

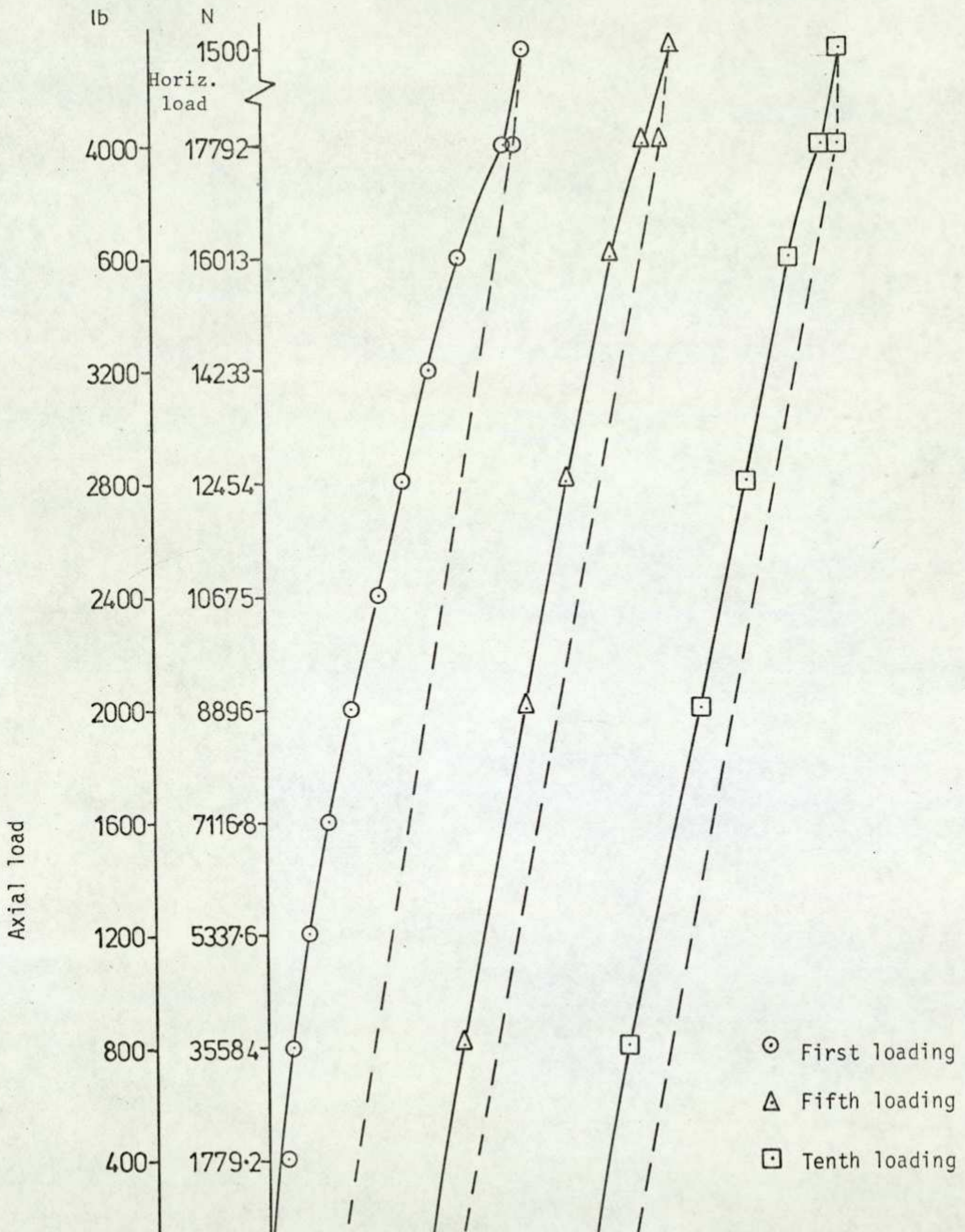


Fig. 7.106 · Load Vs Settlement - Test No. 16

Test No. 16

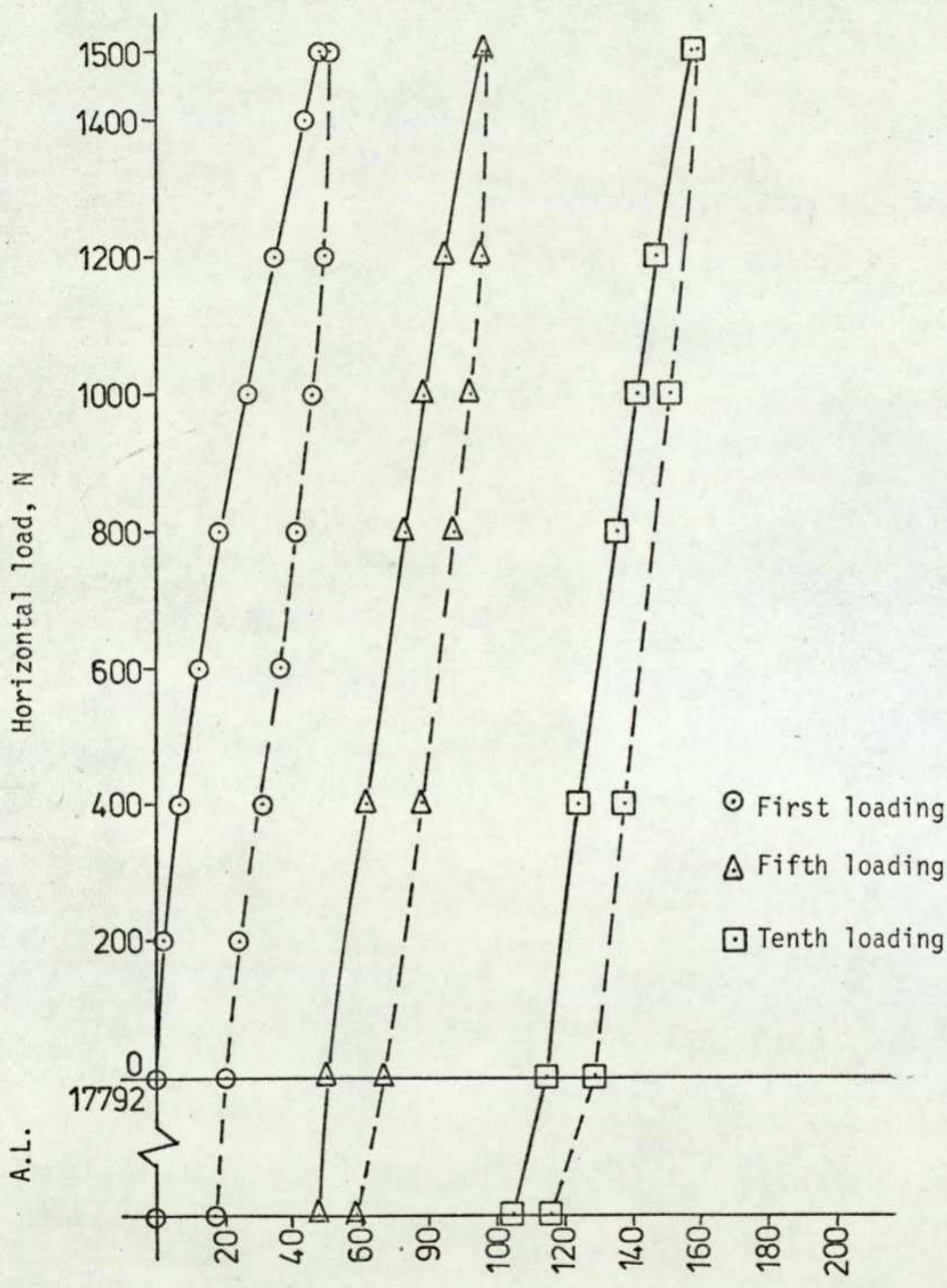
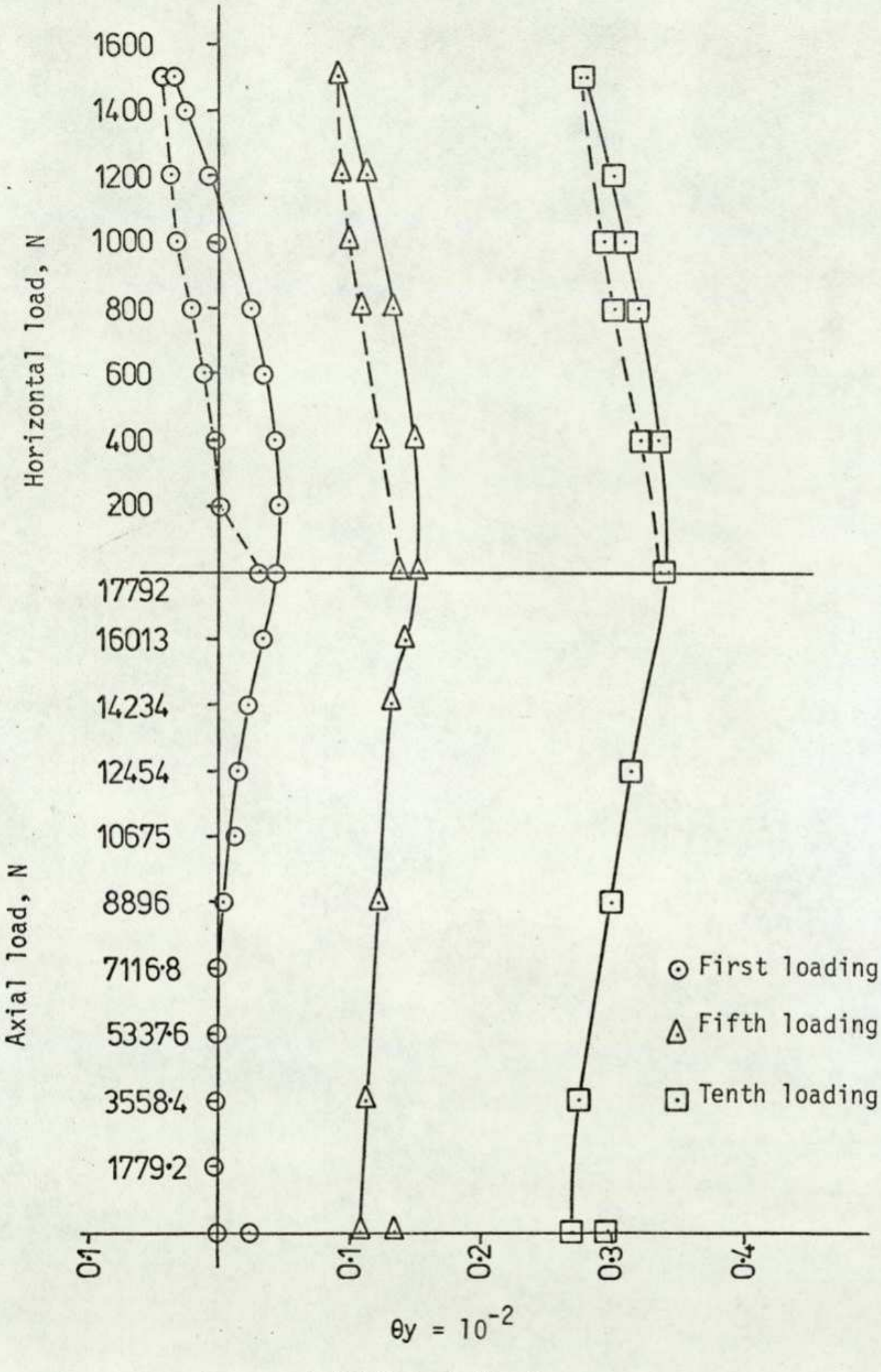


Fig. 7.107 load Vs defl. - pile group

Test No. 16



Rotation of the Cap

Fig. 7.108 load Vs Rotation of the pile cap

Test No. 16

3 x 3

+3B, 3V, -3B 15^{0}

H. load, 1000N

-5121	-4836	-5619
7	6	1
6028	3254	3828
-4032	-4734	-5550
8	5	2
5676	3477	2520
-5164	-4620	-5008
9	4	3
6245	3326	4192

Maximum negative bending moment, N.mm

Maximum positive bending moments, N.mm

Fig. 7.109 Distribution of the maximum negative and positive bending moments in the piles.

H. Load, 1000N

First loading

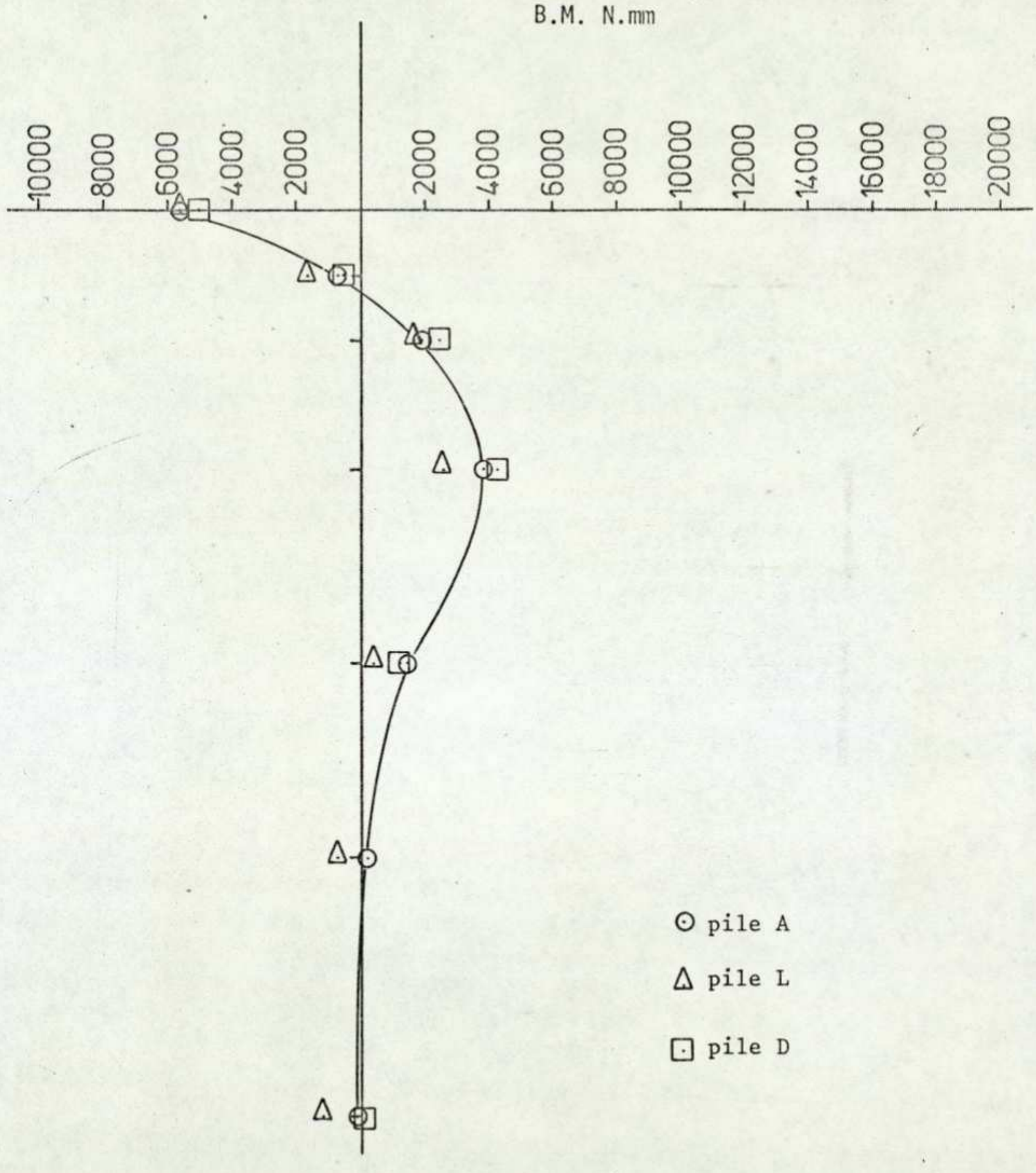


Fig. 7.110 Moment Vs Depth

Test No. 16

Pile Group

H. Load, 1000N

First loading

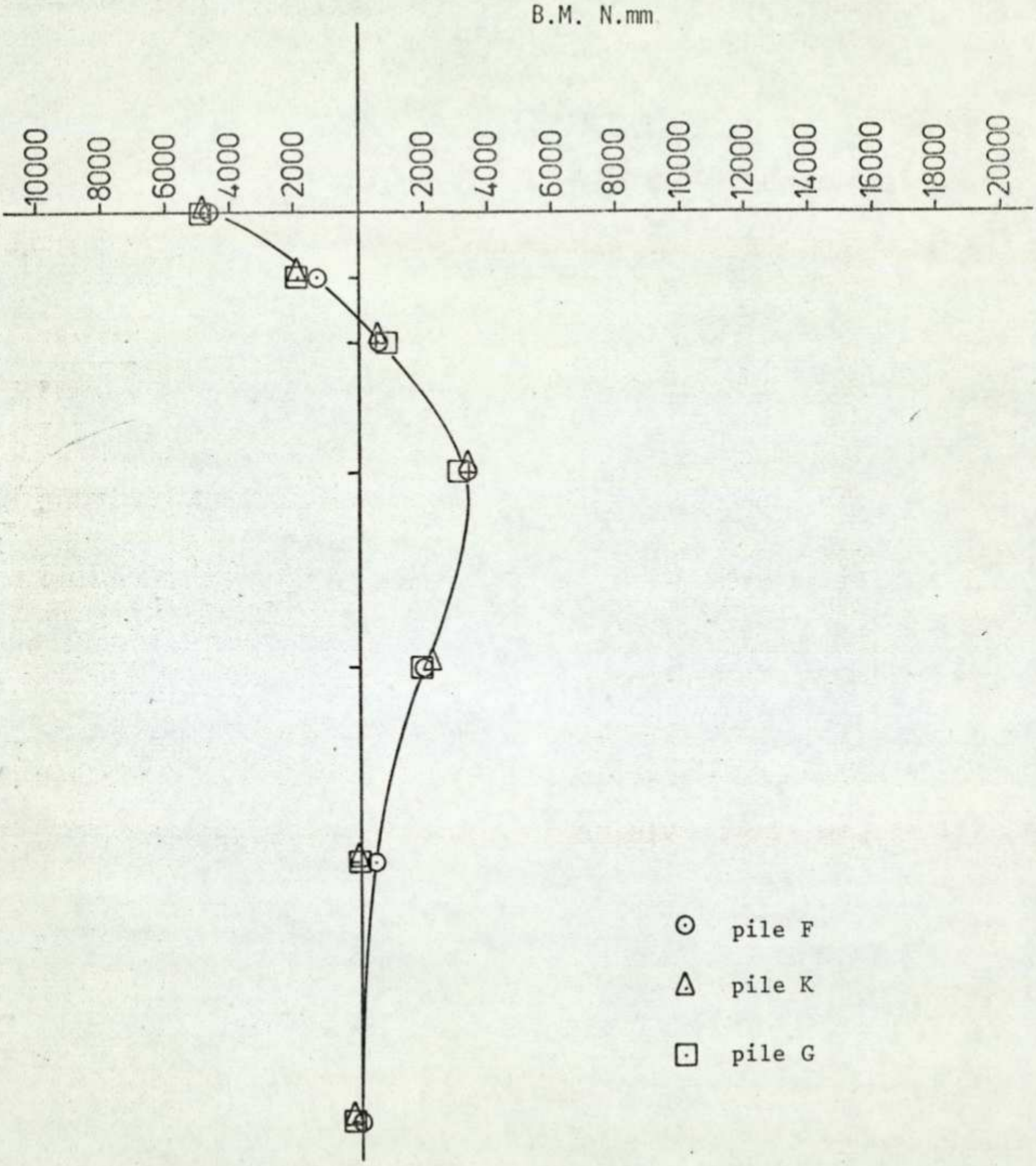


Fig. 7.111 Moment Vs Depth

Test No. 16

H. Load, 1000N

First loading

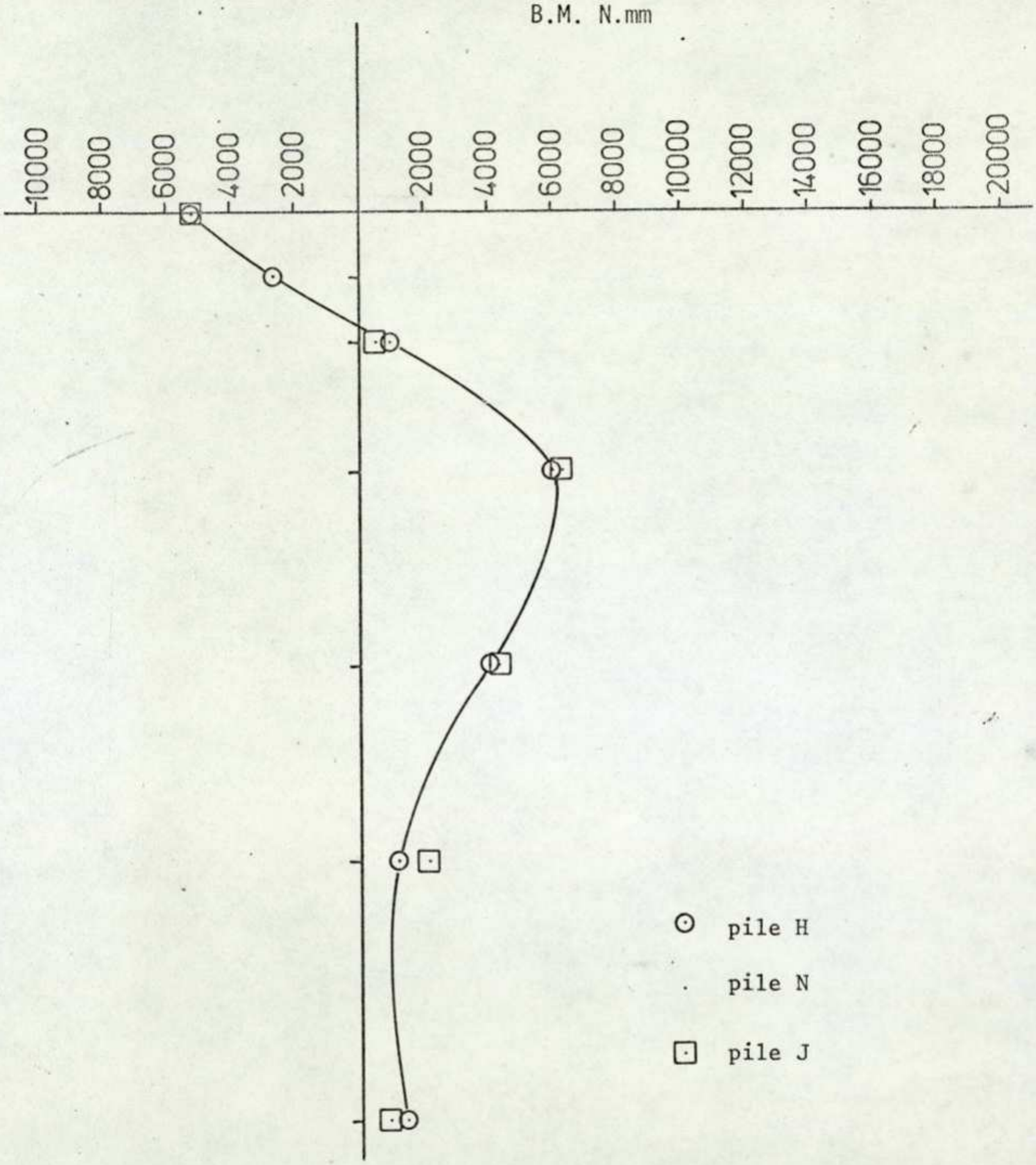


Fig. 7.112 Moment Vs Depth

Test No. 16

Table No. 7.11

Test No. / Loading No.		Transverse Displacements , mm — Transverse load , N						
		0	20cosΘ	40cosΘ	60cosΘ	80cosΘ	100cosΘ	120cosΘ
13A	1	0	0.0775	0.38	0.925	1.605	2.335	3.115
(Θ=0°)	5	0	0.06	0.373	0.888	1.333	1.743	2.183
	10	0	-	0.40	-	1.23	1.61	1.995
14A	1	0	0.0052	0.049	0.169	0.494	1.024	1.63
(Θ=15°)	5	0	-	0.055	0.218	0.579	1.118	1.635
	10	0	-	0.061	-	0.599	1.068	1.531
14B	1	0	0.184	0.483	0.862	1.292	1.784	2.299
(Θ=15°)	5	0	0.209	0.432	0.661	0.912	1.167	1.447
	10	0	-	0.405	-	0.873	1.079	1.343
14C	1	0	0.0145	0.065	0.104	0.298	0.37	0.627
(Θ=30°)	5	0	-	0.013	0.085	0.203	0.384	0.645
	10	0	-	0.013	-	0.196	0.370	0.620
14D	1	0	0.107	0.283	0.526	0.766	1.049	1.285
(Θ=30°)	5	0	-	0.155	0.245	0.339	0.448	0.559
	10	0	-	0.153	-	0.325	0.45	0.53

13 A Single vertical pile , 0°
14 A Single batter pile , + 15°
14 B Single batter pile , - 15°
14 C Single batter pile , + 30°
14 D Single batter pile , - 30°

Table No. 7.12

Applied axial load = 1334.5 N

Test No.	Axial Displacements , mm		
	Loading No.		
	First	Fifth	Tenth
13A	0.29	0.1533	0.135
14A	1.024	0.711	0.694
14B	1.042	0.991	1.00
14C	1.389	1.391	1.35
14D	1.367	1.064	1.15

13A Single vertical pile , 0°

14A Single batter pile , $+15^{\circ}$

14B Single batter pile , -15°

14C Single batter pile , $+30^{\circ}$

14D Single batter pile , -30°

Table No. 7.13

Test No.	Loading No.	Rotation, 10^{-2}						
		Transverse Load, N						
		0	20cosΘ	40cosΘ	60cosΘ	80cosΘ	100cosΘ	120cosΘ
13A	1	0	0.0346	0.1519	0.3385	0.5769	0.8192	1.081
(Θ= 0°)	5	0	0.0308	0.1615	0.3308	0.5115	0.6692	0.8462
	10	0	-	0.15	-	0.4731	0.6308	0.7923
14A	1	0	0.0118	0.038	0.1059	0.2686	0.4902	0.7431
(Θ=15°)	5	0	0.0137	0.049	0.1392	0.31	0.5314	0.7490
	10	0	-	0.0451	-	0.3177	0.5137	0.7216
14B	1	0	0.0726	0.2098	0.3706	0.5451	0.7490	0.9490
(Θ=15°)	5	0	0.1	0.2078	0.32	0.4431	0.5667	0.6941
	10	0	-	0.1941	-	0.4255	0.5302	0.6569
14C	1	0	0.0255	0.0706	0.1471	0.2490	0.3922	0.5726
(Θ=30°)	5	0	-	0.049	0.1118	0.2216	0.3686	0.5561
	10	0	-	0.053	-	0.2149	0.3471	0.54
14D	1	0	0.07451	0.2	0.3824	0.551	0.742	0.9341
(Θ= 30°)	5	0	-	0.1216	0.2	0.2824	0.3686	0.4706
	10	0	-	0.1137	-	0.2706	0.3843	0.4451

13A Single vertical pile , 0°
14A Single batter pile , + 15°
14B Single batter pile , - 15°
14C Single batter pile , + 30°
14D Single batter pile , - 30°

Table No. 7.14

Test No.	Bending Moments, N.mm						
	Station No.						
	1	2	3	4	5	6	7
T. load = 80cosΘ N							
13A	7276	11235	13915	14214	10760	0	- 375
14A	4627	8331	10447	8475	1341	- 500	- 123
14B	7102	11036	14134	14776	6973	- 500	- 370
14C	6241	9738	10569	5867	536	- 500	0
14D	6994	10387	12659	12603	5364	0	- 370
T. load = 100cosΘ N							
13A	9844	14980	18634	19570	15602	1000	- 625
14A	7102	12010	15117	14125	5096	- 500	- 246
14B	9146	14174	18312	19557	10460	- 500	- 616
14C	8500	12876	14502	9561	2414	-1000	123
14D	9038	13525	16592	16732	8046	0	- 493
T. load = 120cosΘ N							
13A	12305	18725	23353	25132	20444	1500	- 875
14A	9899	16014	20033	20209	9387	- 500	- 493
14B	11298	17420	22368	24555	13946	- 500	- 739
14C	10652	16122	18681	14776	5900	- 500	0
14D	10975	16446	20156	20643	10460	0	- 616

13A Single vertical pile , 0°
14A Single batter pile , $+ 15^{\circ}$
14B Single batter pile , $- 15^{\circ}$
14C Single batter pile , $+ 30^{\circ}$
14D Single batter pile , $- 30^{\circ}$

Table No.7.15

Test No.	H.load,N	Bending Moments, N.mm						
		Station No.						
		1	2	3	4	5	6	7
	80	8346	12091	14762	16274	8339	- 500	- 250
6A	100	10379	15087	18634	20806	11298	- 500	- 500
	120	12412	17976	22264	25338	15064	0	- 625
	80	7276	11128	13915	16686	13450	2000	- 625
8A	100	8988	13803	17545	21630	18561	3500	-1000
	120	10887	16478	21054	26574	23941	5500	-1375
	80	6099	9416	10769	7004	538	- 500	0
9A	100	7597	11877	13915	9888	1345	-1000	- 125
	120	9095	14231	17061	12978	2421	-1000	- 125

6A Single vertical pile , 0^{o}

8A Single batter pile , $+ 15^{o}$

9A Single batter pile , $- 15^{o}$

Table No. 7.16

Test No.	Loading No.	H. Displacements , mm						
		H. load , N						
		0	200	400	500	600	700	800
	1	0	0.12	0.3				
1	2	0	0.1	0.28	0.4			
	3	0	0.1	0.25	0.35	0.545	0.875	
	1	0	0.14	0.365	0.515			
2	2	0	0.115	-	0.38	0.535		
	3	0	0.12	0.315	0.37	0.48	0.715	
	1	0	0.06	0.175	0.245	0.345	0.442	
8	5	0	-	-	-	0.222	-	0.332
	10	0	-	-	-	0.2185	-	0.315
	1	0	0.463	1.25	1.695	2.265		
9	5	0	-	0.758	-	1.213	1.462	
	10	0	-	-	-	1.14	1.335	
	1	0	0.0525	0.4425	-	0.52	-	0.624
12	5	0	0.0245	0.1025	-	0.256	-	0.3025
	10	0	0.023	0.1025	-	0.20	-	0.293

Test No. 1	Pile group 2x2		2V	-2B	15^{o}
Test No. 2	Pile group 2x2	+2B	2V		15^{o}
Test No. 8	Pile group 2x2	+2B		-2B	15^{o}
Test No. 9	Pile group 2x2		4V		0^{o}
Test No. 12	Pile group 2x2	+2B	2V		30^{o}

Table No. 7.17

Test No.	Loading No.	Rotation of the cap , 10^{-2}						
		H. load , N						
		0	200	400	500	600	700	800
	1	0	-0.0115					
1	2	0	-0.0077	-	-0.0019			
	3	0	-0.0058	-0.0212	-0.0023			
	1	0	-0.0147	-0.0353	-0.0441			
2	2	0	-0.0088	-	-0.0353	-0.0382		
	3	0	-0.0162	-0.03	-0.0368	-0.042	-0.022	
	1	0	-0.0227	-0.0481	0.0634	-0.0752	-0.0822	
8	5	0	-	-	-	-0.0698	-	-0.0913
	10	0	-	-	-	-0.0636	-	-0.0853
	1	0	0.0675	0.2069	0.288	0.405		
9	5	0	-	0.0973	-	0.1542	0.1927	
	10	0	-	-	-	0.1371	0.1688	
	1	0	0.0238	0.082	-	0.106	-	0.1341
12	5	0	0.0159	0.03864	-	0.0717	-	0.0932
	10	0	0.016	0.0372	-	0.06157	-	0.08554

Table No. 7.18

H. Load = 600N

Test No.	Pile No.	Station No. 1	2	3	4	5	6	7
		Bending Moments, N.mm						
1	1	-5565	127	3870	6552	1703	- 127	- 222
	2	-4375	889	4606	7797	1251	282	- 127
	3	-5061	-1485	2394	819	-	237	0
	4	-4428	0	3140	5698	2480	- 250	0
2	1	-6321	-2048	1403	5699	-	387	- 119
	2	-7115	-1905	2345	4917	2772	0	-
	3	-6750	-2241	1950	5300	3813	250	0
	4	-7504	-2147	688	2625	260	-1152	-1477
8	1	-6758	-3375	- 625	3814	3456	125	- 125
	2	-6000	-2667	395	6682	2584	0	0
	3	-6832	-3599	- 990	0	-	880	- 123
	4	-7239	-3900	- 807	2915	3483	494	0
9	1	-6758	2000	8750	17358	13696	2000	-1500
	2	-7000	2032	9212	20683	13056	1251	-1089
	3	-6588	- 667	5656	10892	-	3740	- 369
	4	-6223	130	5784	14443	14190	3705	- 448
12	1	-8428	-4507	-1255	4092	4992	1250	- 250
	2	-4622	-1926	133	3930	4231	858	116
	3	-8580	-4815	-2635	578	3270	2934	120
	4	-7564	-4875	-2048	1627	3949	2264	113

Table No. 7.19

Test No.	Loading No.	H. Displacements, mm						
		H.Load , N						
		0	200	400	600	800	1000	1200
	1	0	0.115	0.34	0.62	1.00	1.32	
3	5	0	-	-	0.442	-	0.8375	
	10	0	-	-	-	-	0.7675	
	1	0	0.0625	0.18	0.335	0.53	0.74	
4	5	0	-	-	0.17	-	0.365	0.485
	10	0	-	-	-	-	0.365	0.435
	1	0	0.05	0.18	0.305	0.49	0.665	0.90
5	5	0	-	-	-	-	0.3775	0.4575
	10	0	-	-	-	-	0.3525	0.4375
	1	0	0.07	0.0875	0.1775	0.274	0.3875	0.495
6	5	0	-	-	-	-	0.251	0.314
	10	0	-	-	-	-	0.30	0.31
	1	0	0.03	0.085	0.1675	0.256	0.35	0.4775
7	5	0	-	-	-	-	0.2215	0.28
	10	0	-	-	-	-	0.22	0.28
	1	0	-	0.04	0.08	0.125	0.1775	0.2365
10	5	0	-	-	0.064	-	0.1305	0.1615
	10	0	-	-	0.061	-	0.126	0.153
	1	0	0.023	0.0815	0.16	0.24	0.346	0.4524
11	5	0	-	-	0.094	-	0.2015	-
	10	0	-	-	0.0915	-	0.195	0.245

Table No. 7.20

Test No.	Loading No.	Rotation of the cap , 10^{-2}						
		H. Load , N						
		0	200	400	600	800	1000	1200
	1	0	0.0063	0.0173	0.0322	0.059	0.0802	
3	5	0	-	-	0.0214	-	0.0403	
	10	0	-	-	-	-	0.0377	
	1	0	-0.0055	-0.0126	-0.0229	-0.0346	-0.0409	
4	5	0	-	-	-0.0154	-	-0.029	-0.0359
	10	0	-	-	-	-	-0.0308	-0.0344
	1	0	-0.0079	-0.0138	-0.0204	-0.0299	-0.0365	-0.0432
5	5	0	-	-	-	-	-0.03113	-0.0377
	10	0	-	-	-	-	-0.0263	-0.0334
	1	0	-0.0063	-0.0188	-0.0316	0.0492	-0.067	-0.0846
6	5	0	-	-	-	-	-0.058	-0.0719
	10	0	-	-	-	-	-0.058	-0.0690
	1	0	-0.005	-0.0125	-0.027	-0.0333	-0.0427	-0.0522
7	5	0	-	-	-	-	-0.0411	-0.0519
	10	0	-	-	-	-	-0.0388	-0.0488
	1	0	-	-0.0208	-0.0372	-0.0541	-0.0727	-0.0906
10	5	0	-	-	-0.033	-	-0.0616	-0.0764
	10	0	-	-	-0.0316	-	-0.061	-0.0739
	1	0	-0.007	-0.0164	-0.031	-0.0414	-0.0556	-0.0677
11	5	0	-	-	-0.0227	-	-0.434	-
	10	0	-	-	-0.0222	-	-0.0422	-0.051

Table No. 7.21

H.Load = 1000 N

Test No.	Pile No.	Bending Moments , N.mm						
		Station No.						
		1	2	3	4	5	6	7
3	1	-8288	-1750	3375	9205	7936	1750	- 375
	2	-7250	-1651	3158	9864	8160	1251	0
	3	-8418	-1866	4666	13226	-	660	- 123
	4	-9398	-4290	404	7685	9288	2841	
	5	-8320	-3906	680	7367	-	2838	-1000
	6	-9636	-3840	625	7400	9579	2708	- 366
	7	-9272	-4693	0	6962	11081	2166	- 220
	8	-8778	-4294	1190	10063	14136	3843	1250
	9	-9576	-1111	1309	10535	12667	4998	570
4	1	-7650	-3500	- 125	5392	5760	625	- 250
	2	-7125	-2921	658	8273	5712	139	- 121
	3	-5612	-2000	1838	10114	-	220	0
	4	-8636	-4810	-1076	3710	5418	2347	- 560
	5	-6528	-3906	-1088	3197	-	2580	- 250
	6	-8580	-3328	156	4440	5562	1425	0
	7	-7198	-3829	-1012	3276	5603	2421	0
	8	-7296	-4068	- 850	3938	7440	2196	500
	9	-7581	-2889	-1190	4165	6692	3570	380
5	1	-6885	-3000	125	5129	5248	375	0
	2	-6625	-2667	921	5728	4760	278	0
	3	-5246	-2000	1273	5446	-	220	0
	4	-8509	-4420	- 673	5433	5805	1112	- 336
	5	-7296	-4158	-1360	3753	-	1161	0
	6	-8052	-4096	-1250	4144	6026	855	- 366
	7	-8052	-4570	-1518	3003	5354	2293	- 330
	8	-7980	-4407	- 340	6125	9300	4392	1500
	9	-8911	-2444	-1428	6615	9560	4046	950

Table No. 7.22

H. Load = 1000 N

Test No.	Pile No.	Bending Moments, N.mm						
		Station No.						
		1	2	3	4	5	6	7
6	1	-6630	-2500	1000	4866	2432	0	- 125
	2	-6500	-2667	395	5409	2176	0	- 121
	3	-6466	-2933	848	7780	-	0	0
	4	-6731	-3380	- 538	3313	2967	371	- 224
	5	-5888	-3150	- 816	2502	-	387	0
	6	-6600	-2560	156	3848	2781	285	0
	7	-6466	-3829	-1265	2184	3611	1019	- 220
	8	-6498	-3729	- 340	4375	5952	2196	750
	9	-6650	-1333	- 476	4655	5497	2380	760
7	1	-4973	-1750	375	3682	2816	125	- 125
	2	-5000	-1950	1448	5404	1904	0	0
	3	-4270	-1866	1131	3890	-	- 440	0
	4	-4826	-1820	672	4240	2064	0	- 448
	5	-5120	-2646	- 408	2919	-	129	0
	6	-5940	-2944	- 938	3108	3399	713	- 122
	7	-5734	-3211	-1139	1638	2844	1529	- 110
	8	-5586	-3277	170	4375	5952	3294	2250
	9	-5586	- 888	- 952	4165	5019	2856	1140
10	1	-4086	-1753	0	2508	1536	250	- 125
	2	-5772	-3262	- 744	1800	369	- 610	590
	3	-4494	-2054	667	3275	1282	0	0
	4	-5412	-2534	- 620	2024	1713	147	0
	5	-5200	-2730	- 494	2470	2196	381	137
	6	-5331	-3000	- 768	2034	1974	0	0
	7	-4666	-2369	170	3445	3297	1006	1339
	8	-3024	-	681	438	900	1018	802
	9	-4899	- 888	118	3363	2930	1405	892

Table No. 7.23

H.Load = 1000N

Test No.	Pile No.	Bending Moments , N.mm						
		Station No.						
		1	2	3	4	5	6	7
11	1	-5491	-2629	- 126	3828	2944	250	- 250
	2	-7992	-5243	-2480	1920	861	-1708	-1062
	3	-5136	-2183	400	4061	2436	368	- 693
	4	-5808	-2914	- 465	2458	2491	734	- 240
	5	-4680	-2210	371	3640	4392	2159	274
	6	-6448	-3375	- 896	2712	3455	755	0
	7	-5576	-2933	- 848	2584	1978	1342	446
	8	-4592	-	- 568	1424	2700	1273	573
	9	-5296	-2444	- 588	2402	3156	1405	357

Table No. 7.24

Test No. / Loading No.		V. Displacements , mm / V. Load , N						
		0	3558	7117	8896	14234	16013	17792
15	1	0	0.15	0.2963	0.3715	0.6458	0.765	0.897
	5	0	0.0688	0.1705	0.2158	0.3588	0.4088	0.4728
	10	0	0.0668	0.17	0.2155	-	0.4075	0.4613
16	1	0	0.0463	0.1113	0.1513	0.2945	0.349	0.4313
	5	0	0.065	-	0.18	-	0.3338	0.3925
	10	0	0.067	-	0.1995	-	0.3608	0.4183

Test No. 15 Pile group 3x3 9V

Test No. 16 Pile group 3x3 +3B 3V -3B 15^{o}

Table No. 7.25

Test No.	Loading No.	H. Displacements , mm						
		H. Load , N						
		0	400	800	1000	1200	1400	1500
15	1	0	0.3325	0.965	1.375	1.825	2.315	2.565
	5	0	0.3	0.825	1.07	1.333	1.63	1.799
	10	0	0.3	0.8125	1.05	1.295	1.565	1.738
16	1	0	0.066	0.1865	0.265	0.3475	0.4375	0.4825
	5	0	0.0925	0.205	0.265	0.33	-	0.435
	10	0	0.0925	0.205	0.265	0.325	-	0.4275

Test No. 15 Pile group 3x3 9V

Test No. 16 Pile group 3x3 +3B 3V -3B 15^{o}

Table No. 7.26

Test No.	Loading No.	Rotations of the cap , 10^{-2}						
		V. Load , N						
		0	3558	7117	8896	14234	16013	17792
					θ_y			
15	1	0	0.0808	0.1029	0.1135	0.161	0.1731	0.1907
	5	0	0.0040	0.0162	0.0217	0.0367	0.0425	0.0483
	10	0	0.006	0.027	0.0362	-	0.0673	0.074
16	1	0	-0.0029	0.00096	0.0048	0.02346	0.03346	0.04904
	5	0	0.00385	-	0.0135	-	0.03365	0.04423
	10	0	0.00423	-	0.02923	-	0.05712	0.0725
					θ_x			
15	1	0	0.035	0.0788	0.0975	0.1618	0.195	0.222
	5	0	0.0138	0.0395	0.0483	0.0788	0.0913	0.1078
	10	0	0.016	0.045	0.057	-	0.1	0.1138

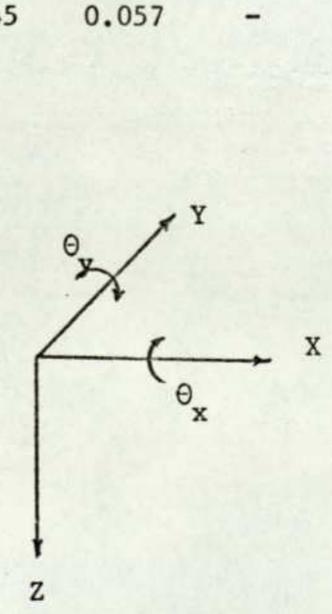


Table No. 7.27

Test No.	Loading No.	Rotations of the cap, 10^{-2}						
		H. Load, N						
		0	400	800	1000	1200	1400	1500
					Θ_y			
15	1	0	0.0294	0.0823	0.1175	0.15308	0.1929	0.217
	5	0	0.0246	0.0548	0.0779	0.1001	0.1221	0.1385
	10	0	0.021	0.0558	0.0721	0.0914	0.1115	0.1279
16	1	0	-0.0052	-0.0236	-0.0496	-0.0567	-0.0759	-0.0827
	5	0	-0.0023	-0.0202	-0.0294	-0.0394	-	-0.0589
	10	0	-0.009	-0.024	-0.0323	-0.041	-	-0.0637
					Θ_x			
15	1	0	0.0043	0.008	0.0073	0.0125	0.0093	0.0115
	5	0	0.0055	0.0048	0.0033	0.0073	0.0048	0.006
	10	0	0.0078	0.005	0.006	0.0063	0.007	0.006
16	1	0	0.00075	0.00125	0.001	0.0005	0.00125	-
	5	0	-0.012	-0.0098	-0.0108	-0.0112	-	-0.011
	10	0	0.00325	0.00125	0.002	0.00175	-	0.00475

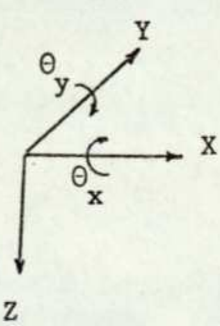


Table No. 7.28

H. Load = 1000 N

Test No.	Pile No.	Bending Moments, N.mm						
		Station No.						
		1	2	3	4	5	6	7
15	1	-7917	- 376	5648	12408	7552	1875	125
	2	-7881	-2447	3224	9240	4674	0	-1534
	3	-7319	- 770	5465	12183	7436	1838	0
	4	-7656	-1647	3410	9977	9031	2201	- 240
	5	-7747	-2380	2704	10213	10996	2500	- 123
	6	-7192	-1875	3584	9899	8761	2139	- 226
	7	-8307	-2482	4070	12057	10550	2683	1116
	8	-5264	-	4994	6461	6300	3054	916
	9	-7547	1333	3058	12250	11946	4213	714
16	1	-5619	- 751	1883	3828	1408	125	- 125
	2	-5550	-1631	1612	2520	369	- 732	-1180
	3	-5008	- 642	2399	4192	1282	245	0
	4	-4620	-1267	620	3326	2024	440	0
	5	-4734	-1839	615	3477	2146	0	- 123
	6	-4836	-1875	896	3254	1851	252	- 113
	7	-5121	-2594	1019	6028	3956	1174	1339
	8	-2016	-	2838	767	- 600	2036	1489
	9	-5164	- 444	588	6245	4283	2107	892

Chapter 8

A comparison between the theoretical and experimental test results

8.1 Introduction

The purpose of this chapter is to compare the results obtained using the computer programmes PILE and PILEGROUP with the experimental observations .

A comparison is also presented with the results of a computer analysis based on the surface integral method of Butterfield and Banerjee (1971) . For reasons which will be discussed later , this method is not satisfactory in this case .

Comparisons have been made between the theoretical and experimental values for displacement and rotation of the pile cap and distribution of bending moment along the length of the pile , in the cases of both single piles and pile groups .

8.2 Comparison of the results of tests on single piles

Comparisons have been made between the theoretical and experimental values for displacement and rotation of the cap and for bending moment in the piles in Test 13A (vertical pile) and in Tests 14A to 14D (raked piles) .

The theoretical analysis was discussed in chapter 3 and is the basis of the computer programme PILE , listed in Appendix 1 .

8.2.1 Displacement and rotation of the pile cap

The initial slope of the transverse load displacement relationship , and the maximum transverse displacement on first loading were used to obtain the elastic and plastic soil constants from the computer programme PILE . The displacements and rotations for each load increment were then computed and comared with the measured values (see Figs. 8.5 to 8.14).

There is generally very good agreement , although the theoretical values are generally rather too large at small loads .

8.2.2 Distribution of bending moment along the piles due to horizontal load

Fig. 8.15 shows the theoretical relationship between bending moment and depth for different horizontal loads in Test 13A (vertical pile) . It may be seen that the shape of the bending moment diagram is generally similar at all loads , but that the position of maximum moment is slightly lower for larger horizontal loads .

Figs. 8.16 to 8.20 show the theoretical and measured bending moments plotted against depth for each of the single pile tests. The differences betwwen the maxmum theoretical and measured bending moments are given in Table 1 below :

Test No.	Inclination of raked pile	Difference between measured and calculated maximum moment
14D	-30^{o}	1.8%
14B	-15^{o}	0.8%
13A	0^{o}	8.4%
14A	$+15^{o}$	8.2%
14C	$+30^{o}$	0.2%

Table 1 Difference between measured and theoretical maximum moment , expressed as % of the theoretical value .

8.3 Comparison of the results of tests on pile groups

The programme PILE was used to determine the elastic and plastic soil constant and the initial stiffness coefficients B2 to B6 . Comparisons were then made beteen the measured values of displacements and rotations of the pile caps and moments in the piles , and the corresponding theoretical values obtained from programme PILEGROUP , as follows :

a - Four - pile groups (2 x 2) , under horizontal load only (Tests 1 , 2 , 8 , 9 and 12)

b - Nine - pile groups (3 x 3) , under horizontal load only (Tests 4 , 5 , 7 , 10 and 11)

c - Nine - pile groups (3 x 3) , under vertical and horizontal load (Tests 15 and 16)

The theoretical analysis was discussed in chapter 4 and is the basis of the computer programme PILEGROUP , listed in Appendix 11 .

8.3.1 Group of four vertical piles (Test 9)

Preliminary investigations showed that the displacements and in particular the pile cap rotations were very sensitive to the value of the axial stiffness coefficient B1 . After some experiment , the value of 2000 N/mm was found to give the best agreement . this is only about 44% of the value measured in Test 13A on a single vertical pile .

Two cases were then investigated :

Case 1 The value of the elastic and plastic soil stiffness constants and the initial stiffness coefficients (B2 to B6) for all piles were computed (by programme PILE) from the results of the Test 13A on a single vertical pile . B1 was taken to be 2000 N/mm .

Case 2 The values of the elastic and plastic soil stiffness constant were increased by 30% for piles in the front row . Coefficients B1 to B6 were computed as in case 1 .

The results of these analyses are given in Figs. 8.24 , 8.25 , 8.30 and 8.31 . From these results it may be seen that :

a - Case 2 gives rather better agreement than case 1 .

b - Case 2 gives excellent agreement with the measured displacements but overestimates the rotation by about 40%

c - Case 2 overestimates the maximum positive bending moments in the piles by from 10% to 30% and correspondingly underestimates the negative bending moment at the ground surface .

8.3.2 Four - pile groups with raked piles (Tests 1 , 2 , 8 and 12)

Three cases were investigated :

Case 1 The elastic and plastic soil stiffness constants and the initial stiffness coefficients B2 to B6 for all piles were computed from Test 13A on a single vertical pile . The axial stiffness coeficient B1 was given the value 4600 N/mm which is also the value computed from Test 13A .

Case 2 The elastic and plastic soil stiffness constants were increased by 30% for the front row of piles . The coefficients B1 to B6 were computed as in case 1 .

Case 3 The soil stiffness constants and the stiffness coefficients B2 to B6 for each pile were taken from the corresponding test on a single pile with the same inclination . The coefficient B1 was given the value 4600 N/mm .

From these results it may be seen that :

a - Results of case 2 showed no significant difference from the results of case 1 .

b - Case 3 overestimates the dispacements and the bending moments in the piles .

8.3.3 Nine - pile groups under horizontal loading

(Tests 4 , 5 , 7 , 10 and 11)

Three cases were investigated as follows :

Case 1 The soil stiffness constants and the stiffness coefficients B2 to B6 for all piles were taken from Test 13A on a single pile . The coefficient B1 was given the value 4600 N/mm (obtained also from Test 13A)

Case 2 As in case 1 , but the soil stiffness coefficients were increased by 30% for positively raked piles and reduced by 30% for negatively raked piles . Results of this case showed no significant difference from the results of case 1 above .

Case 3 The soil stiffness constants and the stiffness coefficients B2 to B6 were taken from the corresponding test on a single pile with the same inclination . The coefficient B1 was given the value 4600 N/mm (obtained from Test 13A)

The results of these investigations are given in Figs. 8.33 to 8.52 .

Case 3 underestimates the displacements of the pile caps .

8.3.4 Nine - pile groups under vertical and horizontal loading

(Tests 15 and 16)

The following three cases were examined :

Case 1 Soil stiffness constants and stiffness coefficients B2 to B6 were taken from Test 13A (single vertical pile) . The coefficient B1 was given the value 2000 N/mm in Test 15 (all vertical piles) and 4600 N/mm in Test 16 (containing raked piles) .

Case 2 As in case 1 above , but the soil stiffness coefficients were increased by 30% for the front row of piles and decreased by 30% for piles in the rear row .

The results of this case showed no sigificant difference from these of case 1 .

Case 3 Soil stiffness constants and stiffness coefficients B2 to B6 were obtained from Test 13A . The coefficient B1 for the central pile was given the value 2000N/mm (for Test 15) or 4600 N/mm (for Test 16) . The values for the other piles were computed in proportion to the resistance measured during redriving .

From these results it may be seen that :

a - Case 3 showed no significant difference for the vertical displacements from the results of case 1 .

b - Case 3 overestimates the horizontal displacement for Test 15 .

c - Case 3 overestimates the rotation for Test 15 and underestimates the rotation for Test 16 .

8.4 Comparison of the test results with a linear elastic solution

A comparison was made between the experimental observation and the results of an analysis using the computer programme PGROUP prepared by the Highway Engineering Comuter Branch of the Department of the Environment . This programme uses the surface integral method of Butterfield and Banerjee (1971) and presumes that the materials of both the soil and the piles are uniform linear elastic solids. The method has been successfully used to analyse pile groups in clay (Ghosh , 1975) . For reasons which will be discussed later , it was not thought likely that the method would prove useful for pile groups in sands , but it was felt that the attempt should be made .

The results of two tests were analysed :

a - Test 13A (a single vertical pile) .

b - Test 15 (a nine - pile group loaded vertically and horizontally) .

8.4.1 Analysis of a test on a single vertical pile

The horizontal and vertical displacements measured in Test 13A were each back - analysed to determine the values of Young's Modulus which would give the same displacements in the elastic model .

Where this process has been used for piles in clays , the two values obtained have been found to differ by not more than 30% . In this case , however , the ratio of the modulus obtained from horizontal displacement to that required to give the correct vertical displacement was 0.008 to 1 . Moreover , even using the smaller of these values of the modulus , the depth of the maximum computed bending moment was only about 0.14 of the embedded length compared with 0.27 of the embedded length measured in Test 13A . The measured bending moments were also considerably larger than predicted . (see Figs. 1 and 2)

8.4.2 Analysis of a test on a group of nine vertical piles

A similar analysis was made on the results of Test 15 , the results of which are shown in Figs. 8.3 and 8.4 .

From this analysis the following conclusion may be drawn :

a - The ratio of the values of the elastic moduli obtained from the measured horizontal and vertical displacements was 0.0484 to 1 .

b - The position of the maximum theoretical bending moment in no case agreed with the position measured in Test 15 .

c - The measured values of the negative moments in the piles at the soil surface were six to seven times the theoretical values .

d - In all piles the measured maximum positive moments were greater than the theoretical values .

8.4.3 Discussion of the results

The method is clearly unsuitable for analysis of piles and pile groups in sands . Although this was not unexpected , the extent of the discrepancies was surprising . The reasons for these are however easily seen .

The horizontal displacement under horizontal load depends on the stiffness of the soil over the first few pile diameters from the surface .

The vertical displacement under vertical load , on the other hand , depends mainly on the stiffness of the soil under the tips of the pile . This soil is much stiffer than the soil near the surface ,

a - because it is deeper and the stiffness of the soil increases with depth , and

b - because the soil beneath the pile tips has been compacted during driving .

The method used cannot account for this since the soil modulus is presumed to be constant with depth . It is understood that the programme is being extended to allow for soils in which the elastic modulus increases with depth . This modified method may give better agreement with the experimental results , although it will still be unable to allow for the inelastic behaviour of the soil and the effect of compaction during pile driving .

8.5 General conclusions

The method outlined in chapter 3 and 4 can allow for the non - linear soil behaviour and the variation of soil stiffness with depth . Allowance cannot , however be made for the interaction of closely spaced piles in a group , except by empirical adjustment of the coefficients . In this respect , the most important quantity is the axial pile stiffness (coefficient B1) . The computed rotation of the cap is particularly sensitive to changes in its magnitude .

Where the piles were raked , so that the tips were far apart , good agreement was reached using the value of B1 measured in a single pile test . Where all the piles in the group were vertical so that the tips were close together , it was necessary to reduce the value of B1 by more than 50% .

Some small additional improvement was obtained by modifying the other soil stiffness constants in some cases .

The tests on single raked piles proved less satisfactory than the test on a single vertical pile as a means of predicting the stiffness of the raked piles in the groups . The reason for this is not obvious , but may be related to the effect on the stiffness of the locked - in moments observed during driving and discussed in section 7.2.1.c above.

The method used in the computer programme PGROUP is , at least in its present form , unsuitable for the analysis of pile groups in sands .

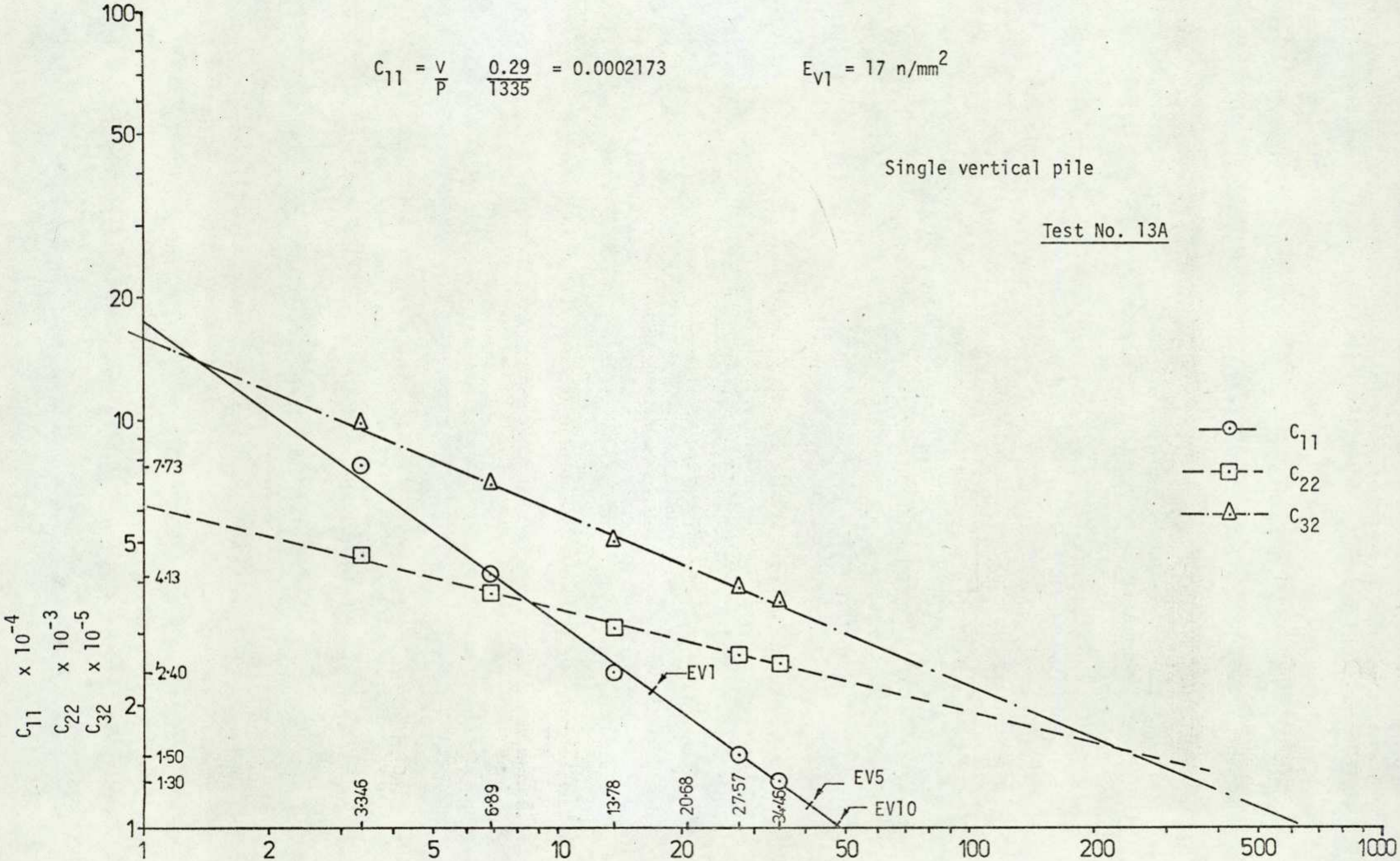


Fig. 8.1.a Flexibility coefficients Vs Young's modulus of soil

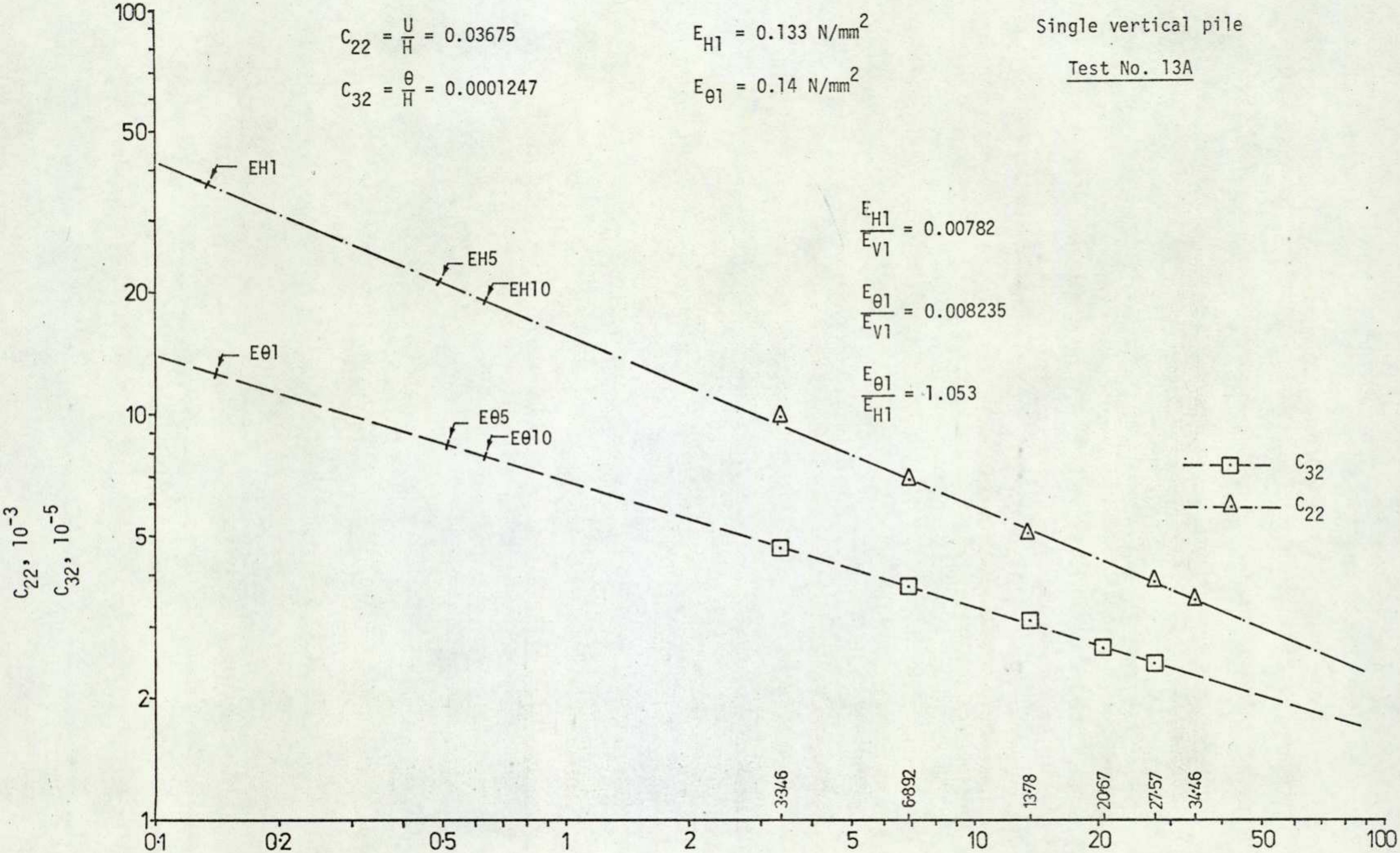


Fig. 8.1.b Flexibility coefficients Vs Young's modulus of soil

Test No. 13A

Single Vertical pile

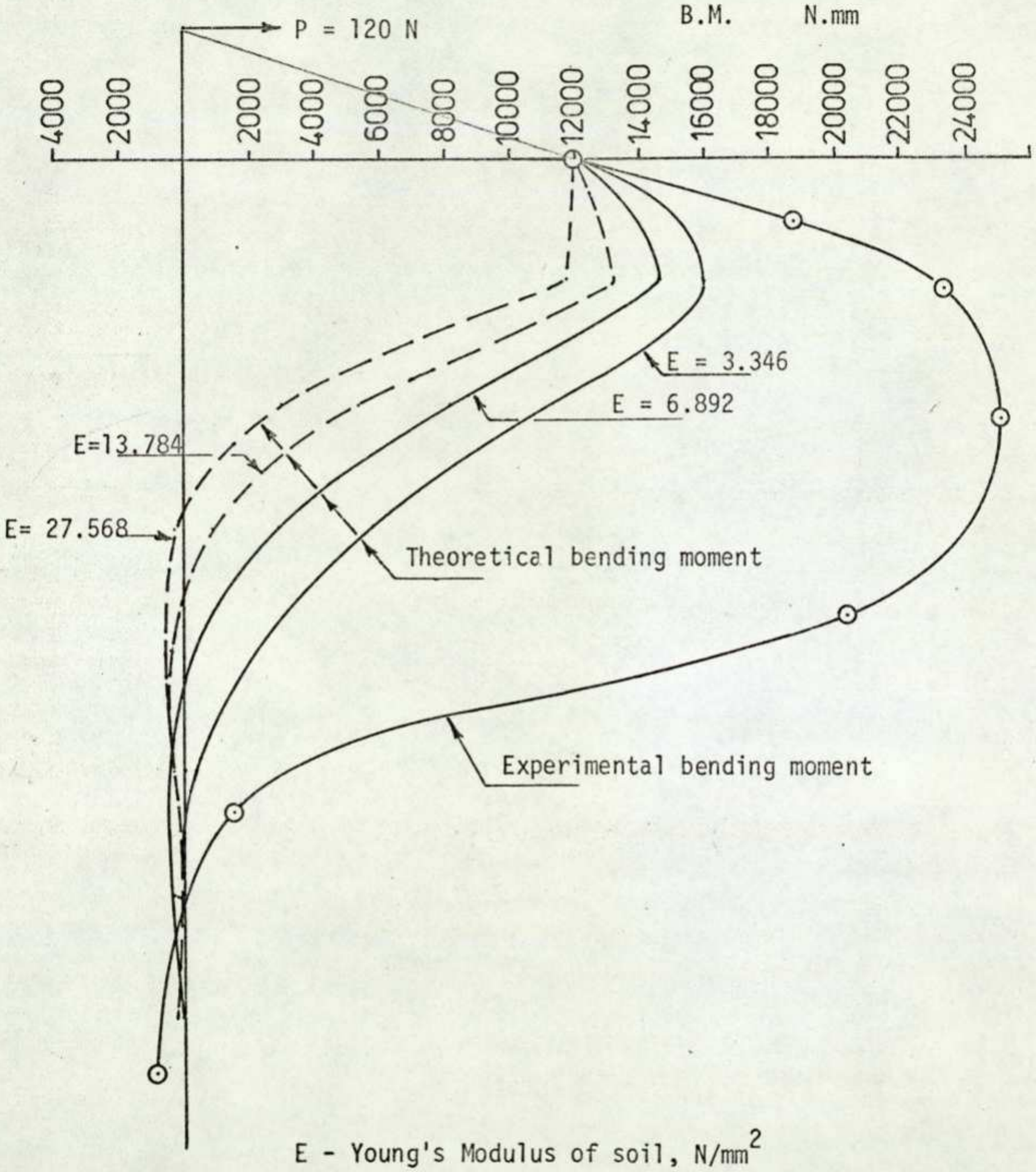


Fig. 8.2 Theoretical and experimental bending moment Vs Depth

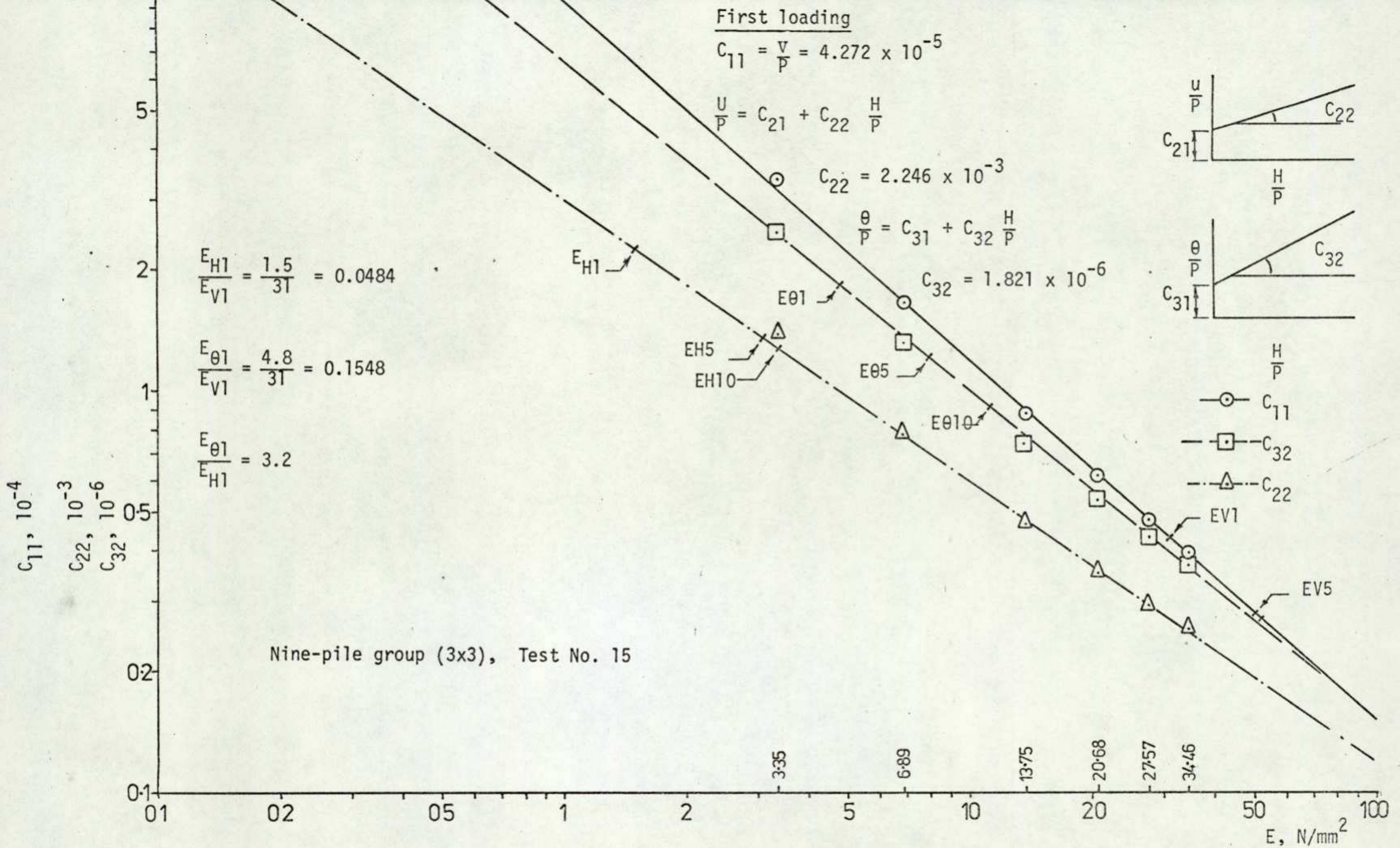
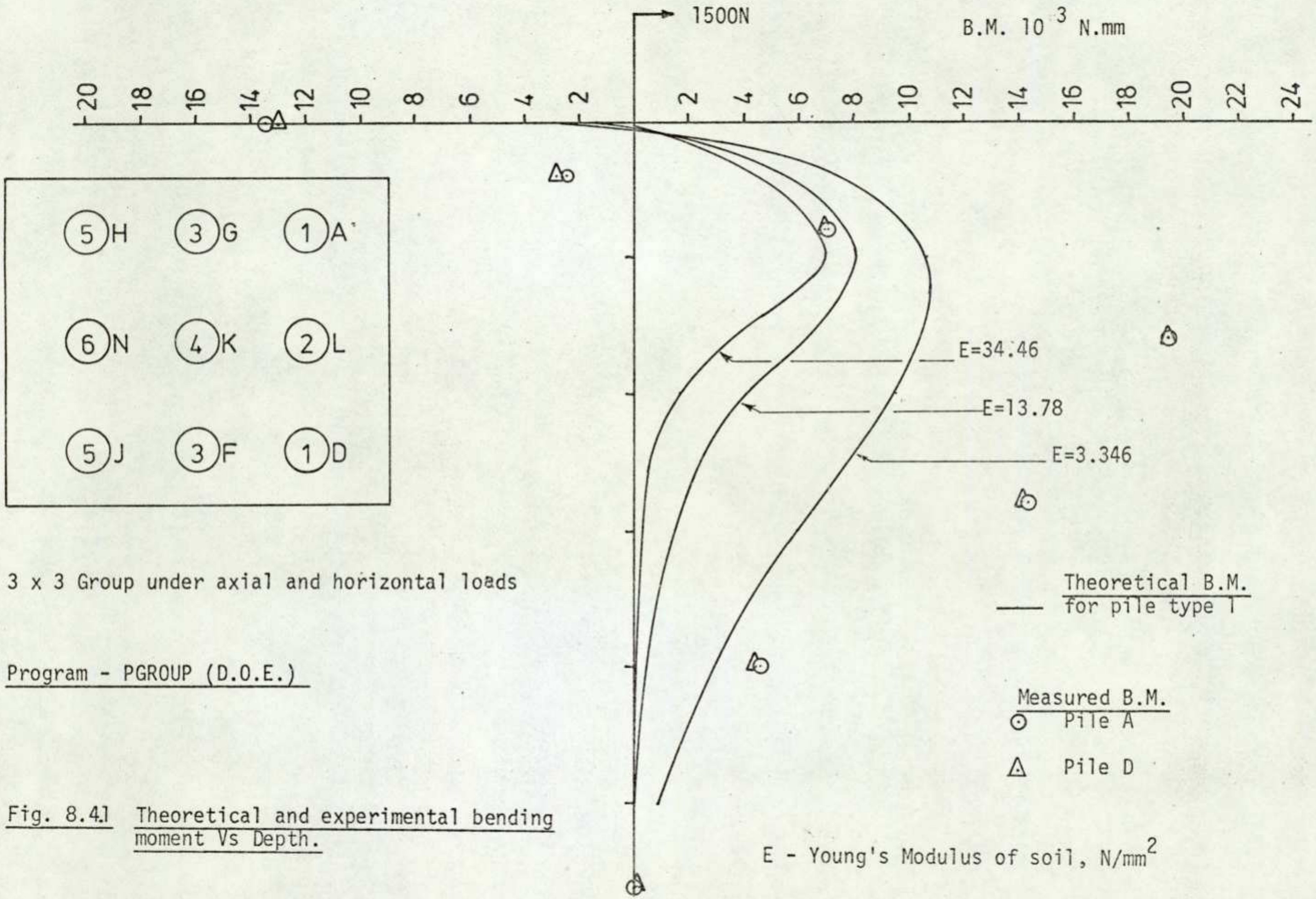


Fig. 8.3. Flexibility coefficients Vs Young's Modulus of Soil



3 x 3 Group under axial and horizontal loads

Program - PGROUP (D.O.E.)

Fig. 8.41 Theoretical and experimental bending moment Vs Depth.

Test No. 15

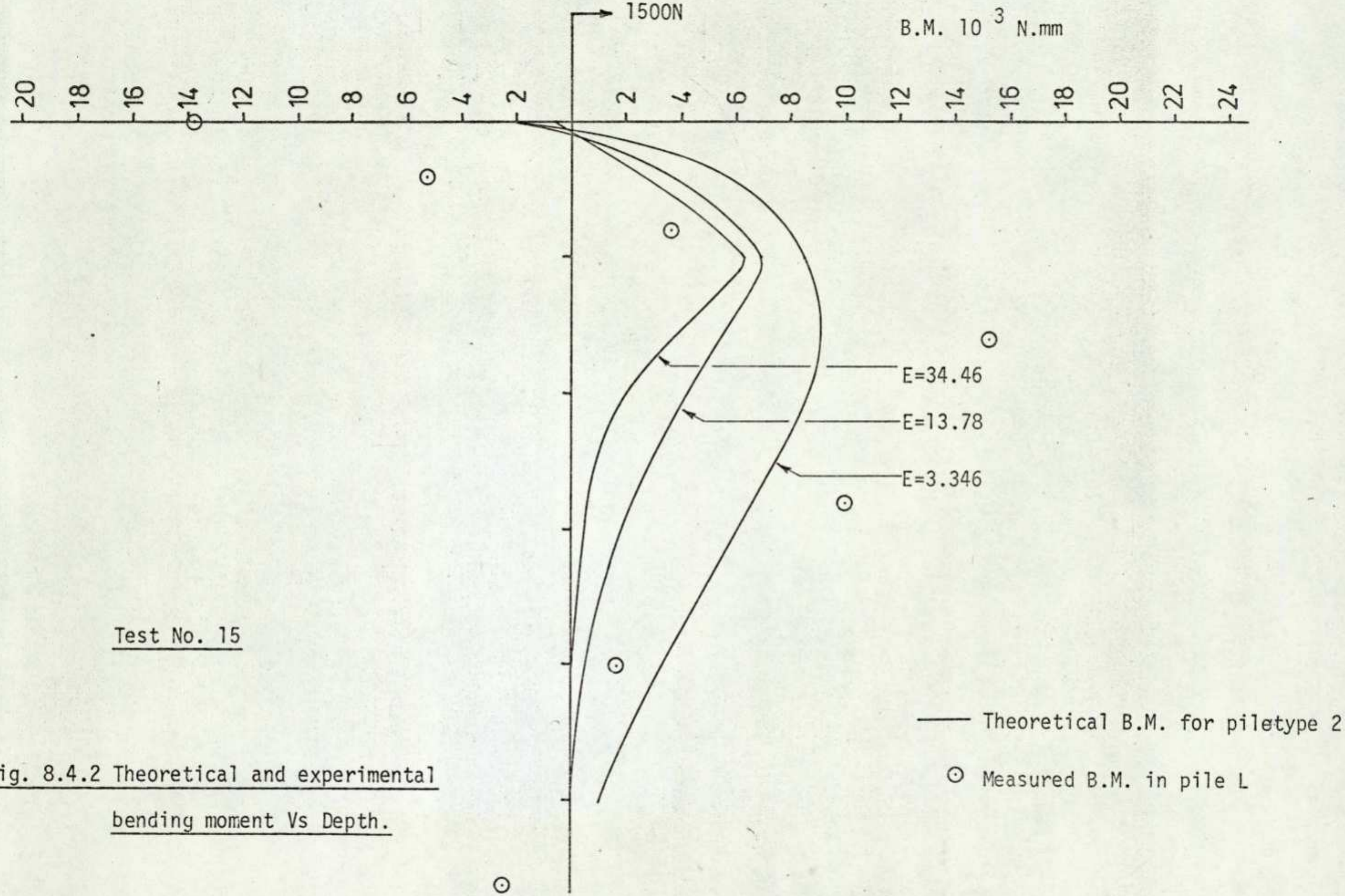
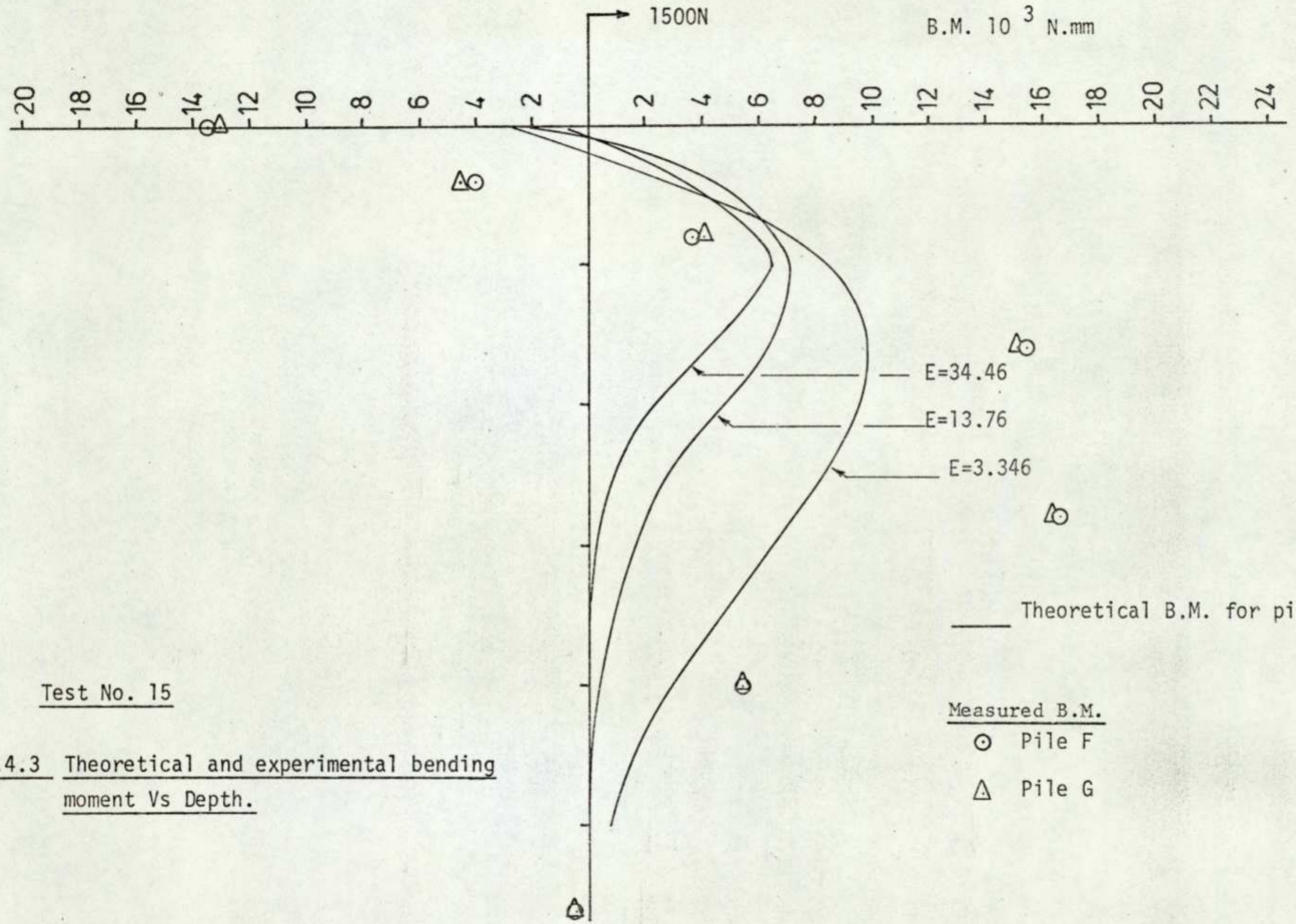


Fig. 8.4.2 Theoretical and experimental bending moment Vs Depth.



Test No. 15

Fig. 8.4.3 Theoretical and experimental bending moment Vs Depth.

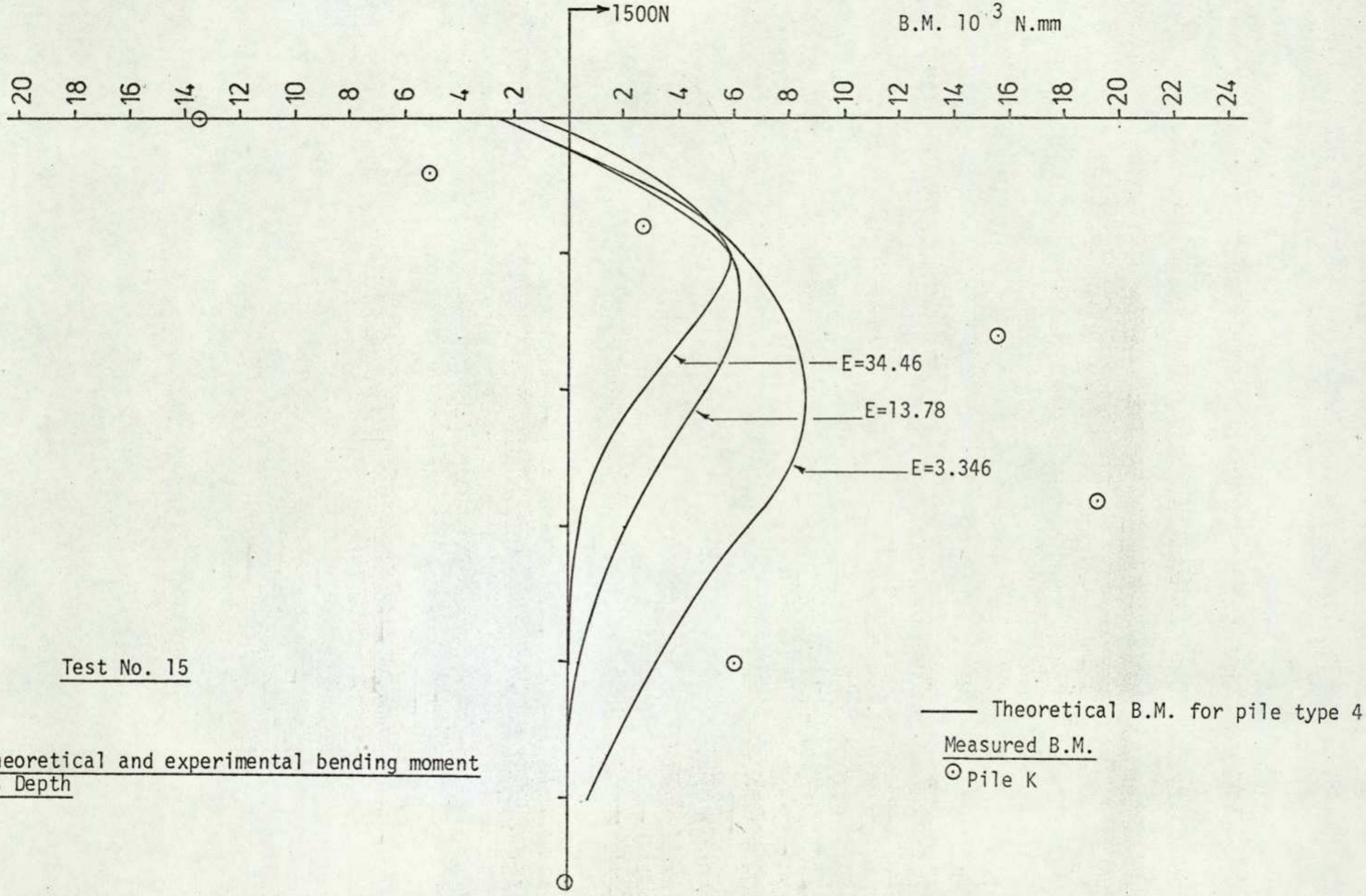
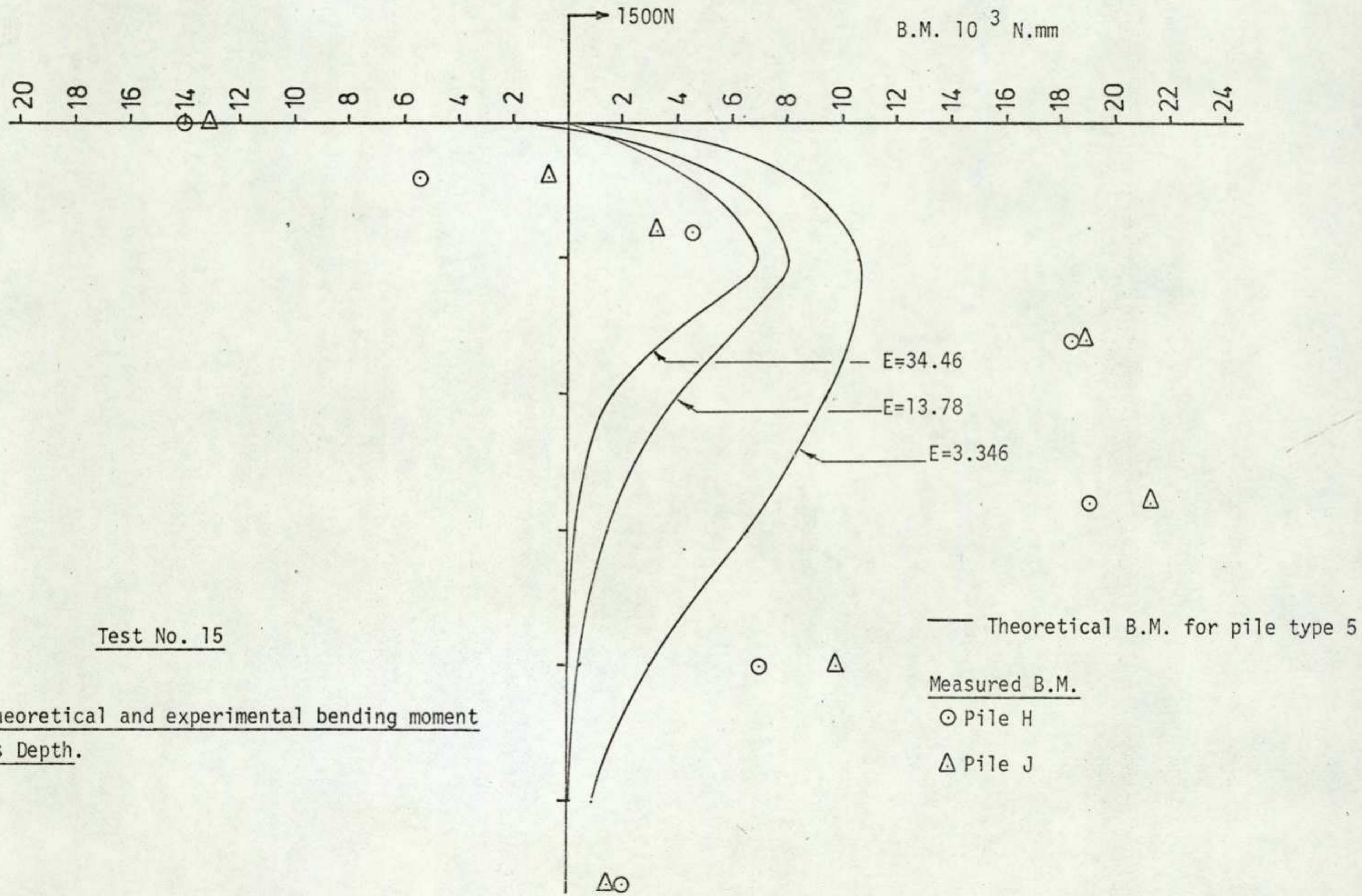


Fig. 8.4.4 Theoretical and experimental bending moment Vs Depth



Test No. 15

Fig. 8.4.5 Theoretical and experimental bending moment Vs Depth.

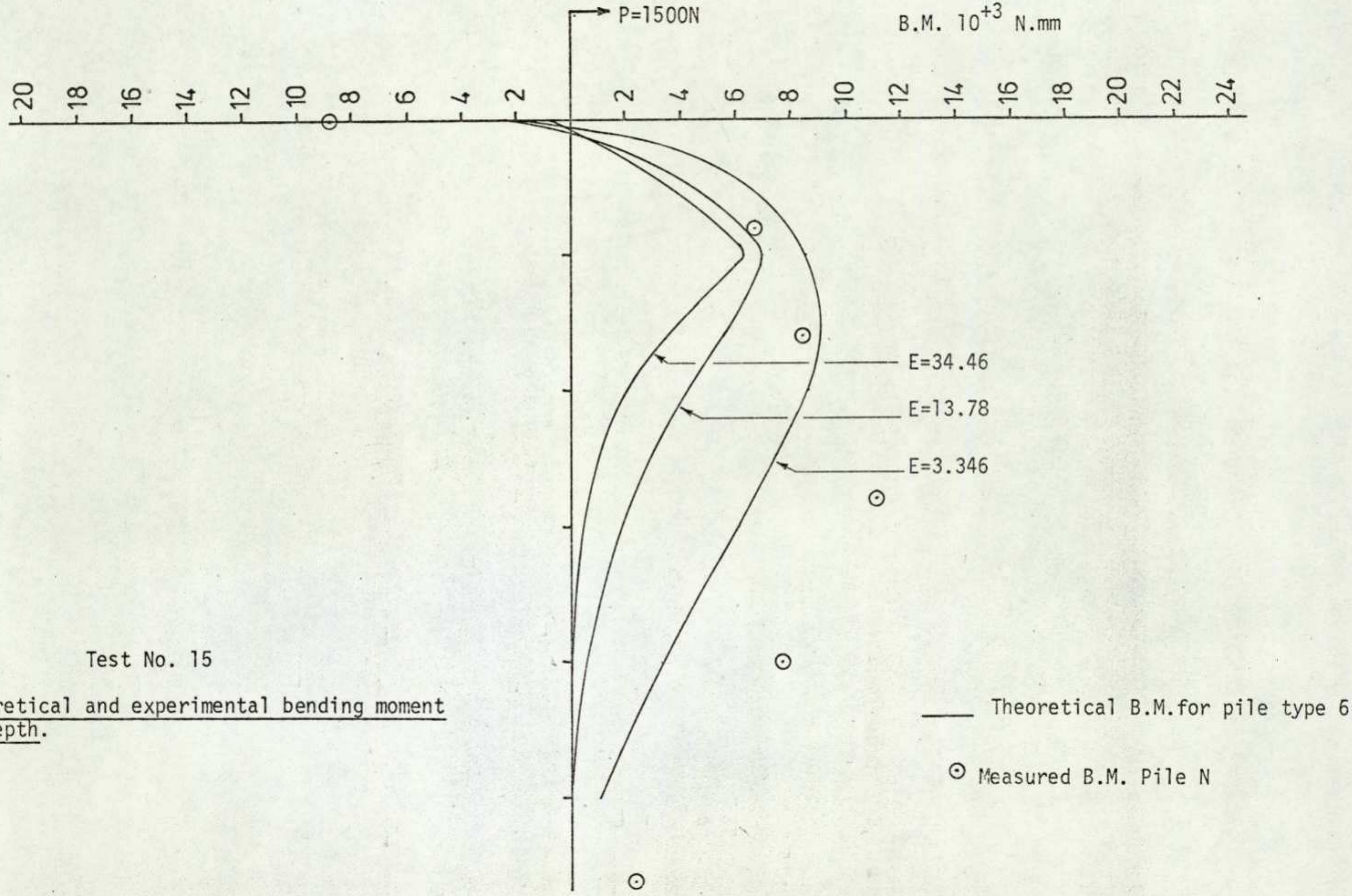


Fig.8.4.6 Theoretical and experimental bending moment Vs Depth.

Single Vertical pile

Test No. 13A

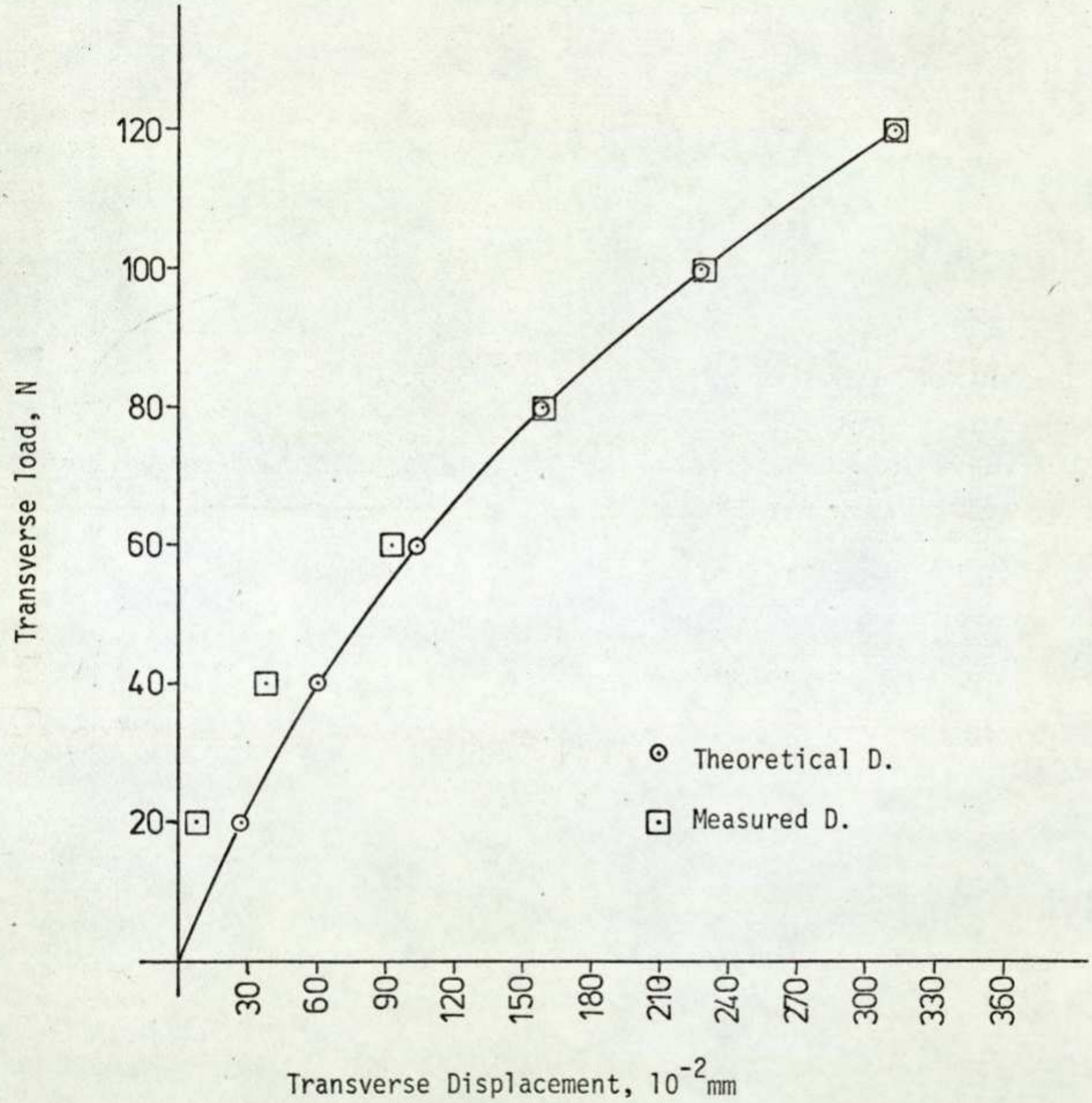


Fig. 8.5 T. Load Vs Theoretical and experimental T. Displacement

Single Vertical pile

Test No. 13A

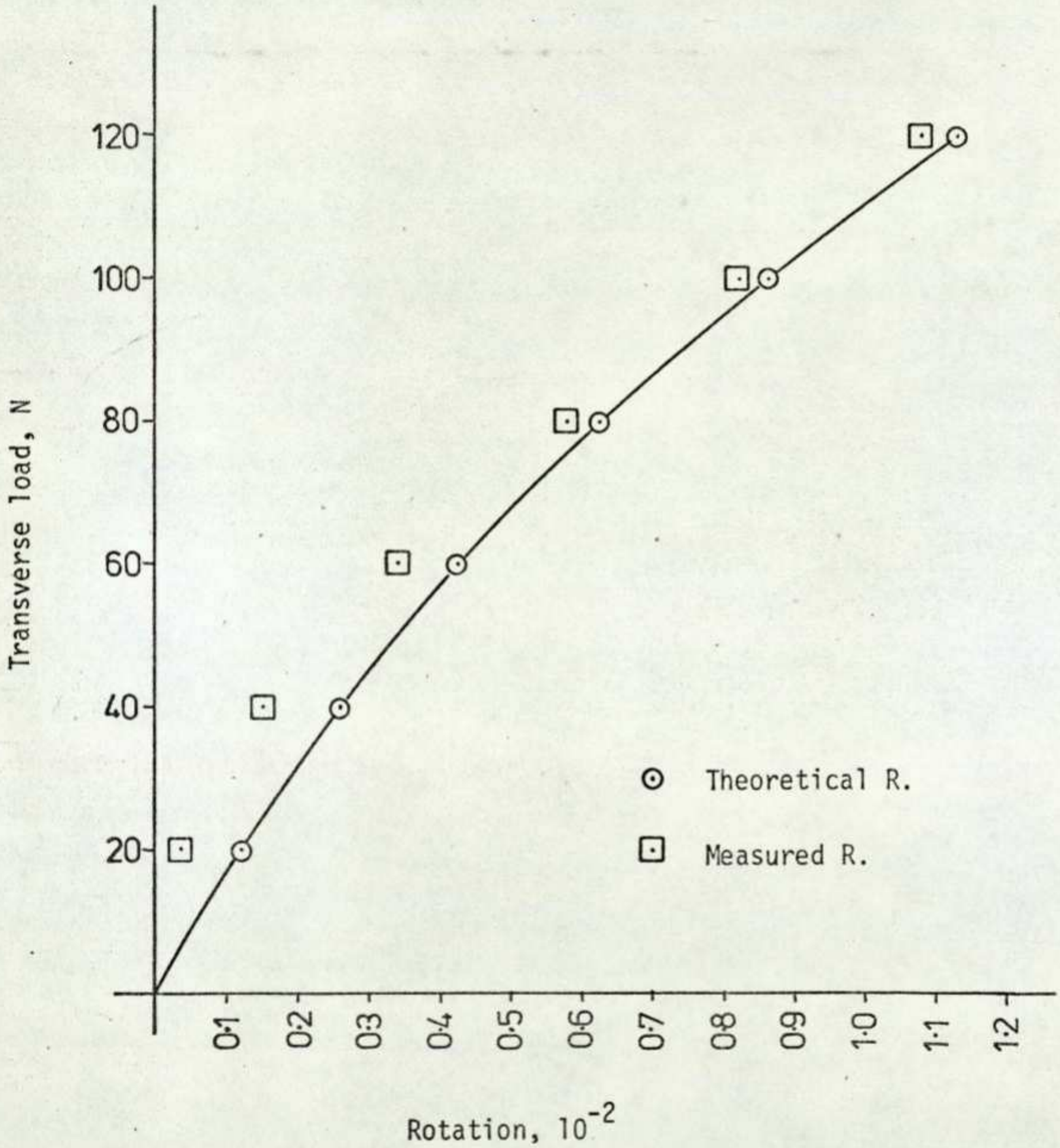


Fig. 8.6 T. load Vs Theoretical and experimental rotation of the cap

Single Batter pile $(+15^0)$

Test No. 14A

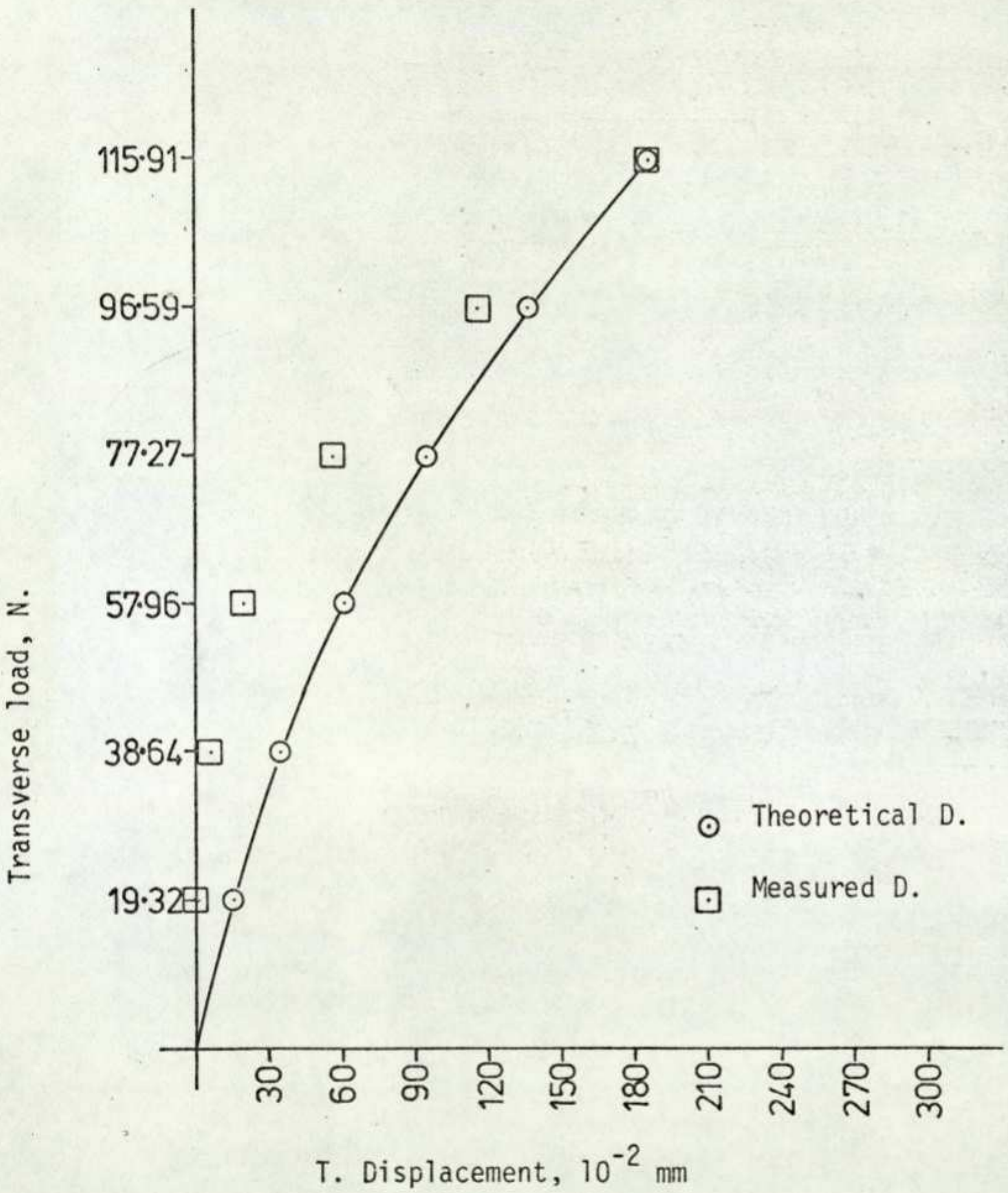


Fig. 8.7 T. Load Vs Theoretical and experimental T. Displacement

Single Batter pile (+15^{0})

Test No. 14A

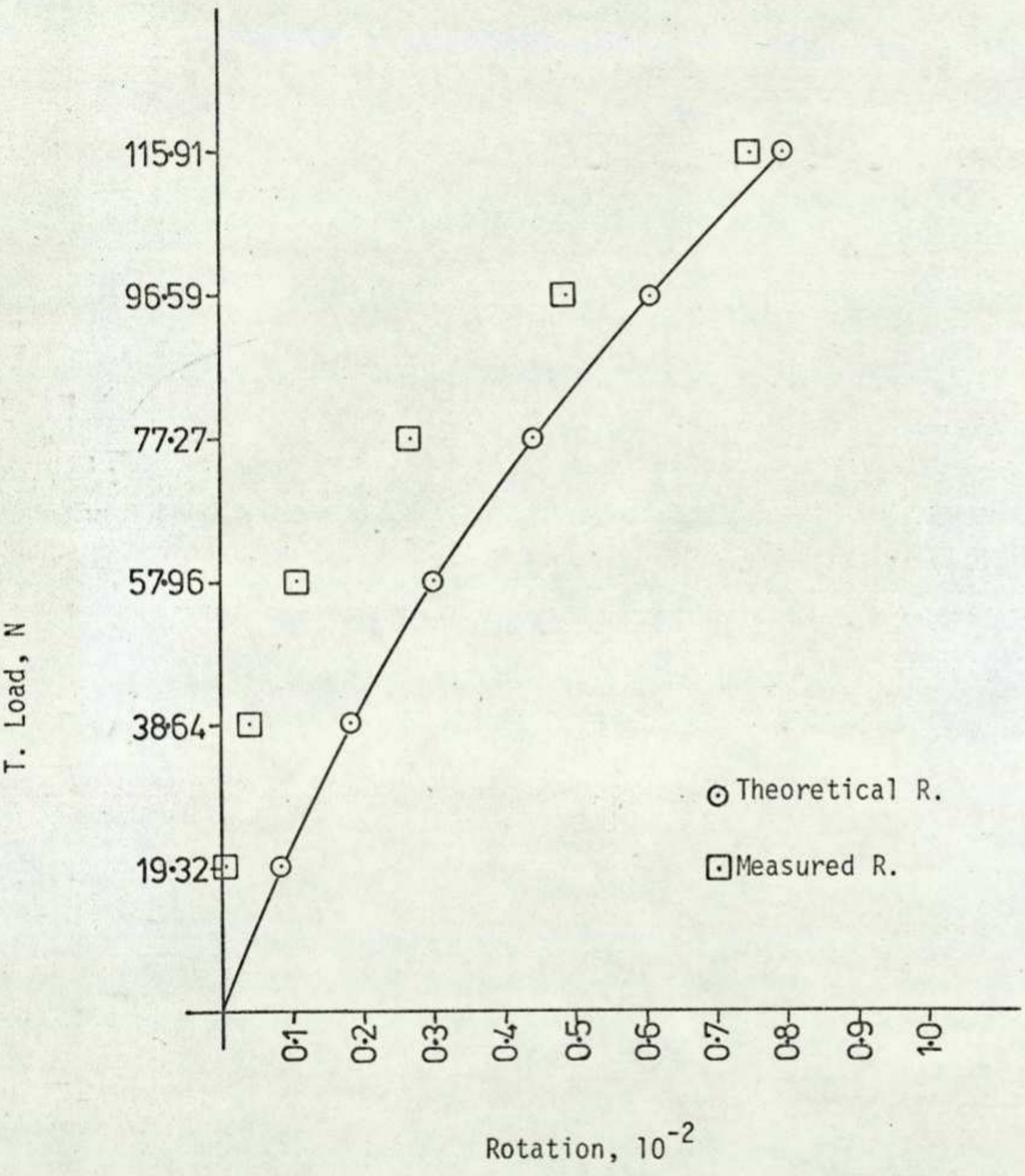


Fig. 8.8 T. Load Vs Theoretical and experimental rotation of the cap

Single Batter pile (-15^{0})

Test No. 14B

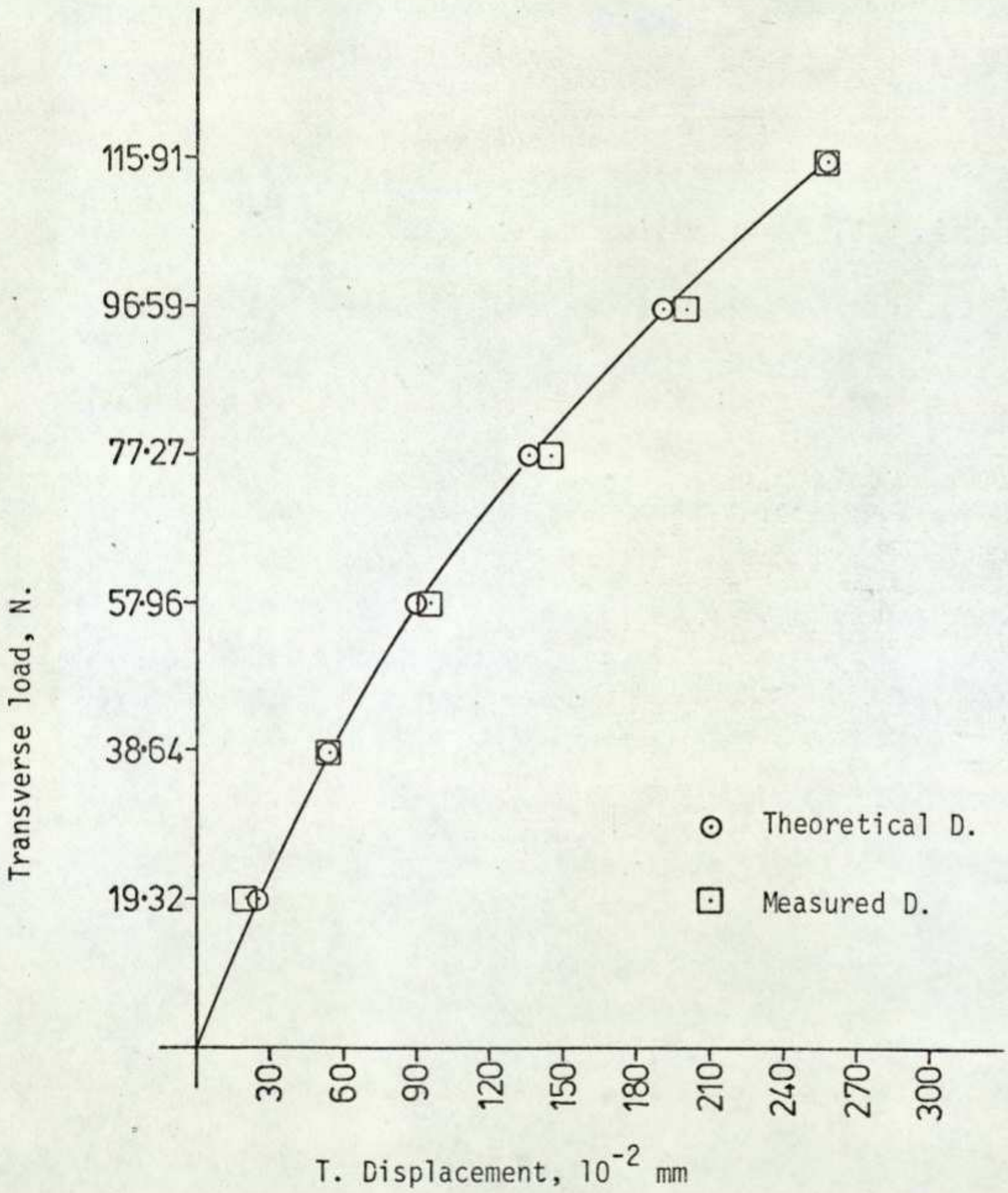


Fig. 8.9 T. Load Vs Theoretical and experimental T. Displacement

Single Batter pile (-15^0)

Test No. 14B

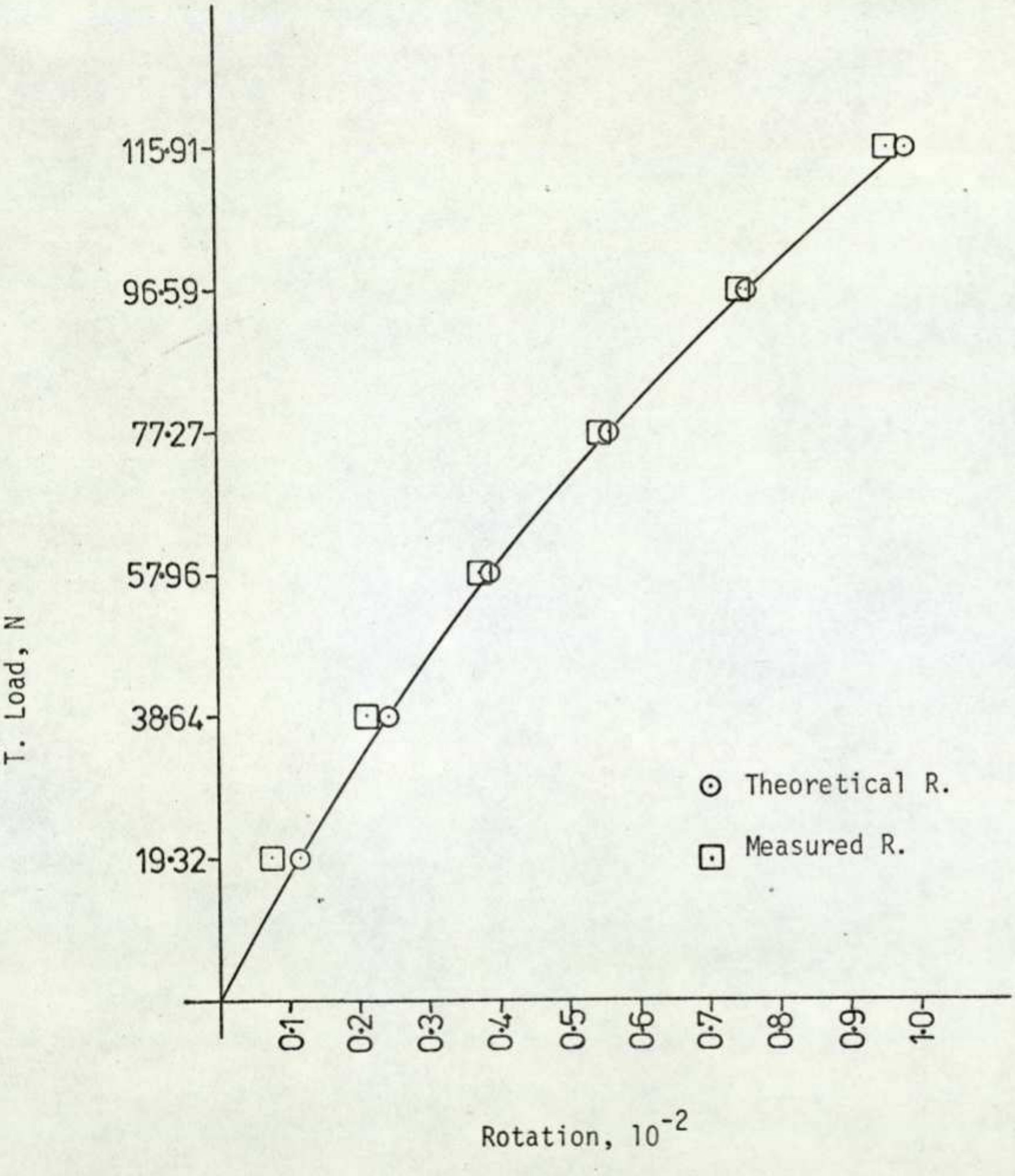


Fig. 8.10 T. load Vs Theoretical and experimental rotation of the cap

Single Batter Pile (+30^{o})

Test No. 14c

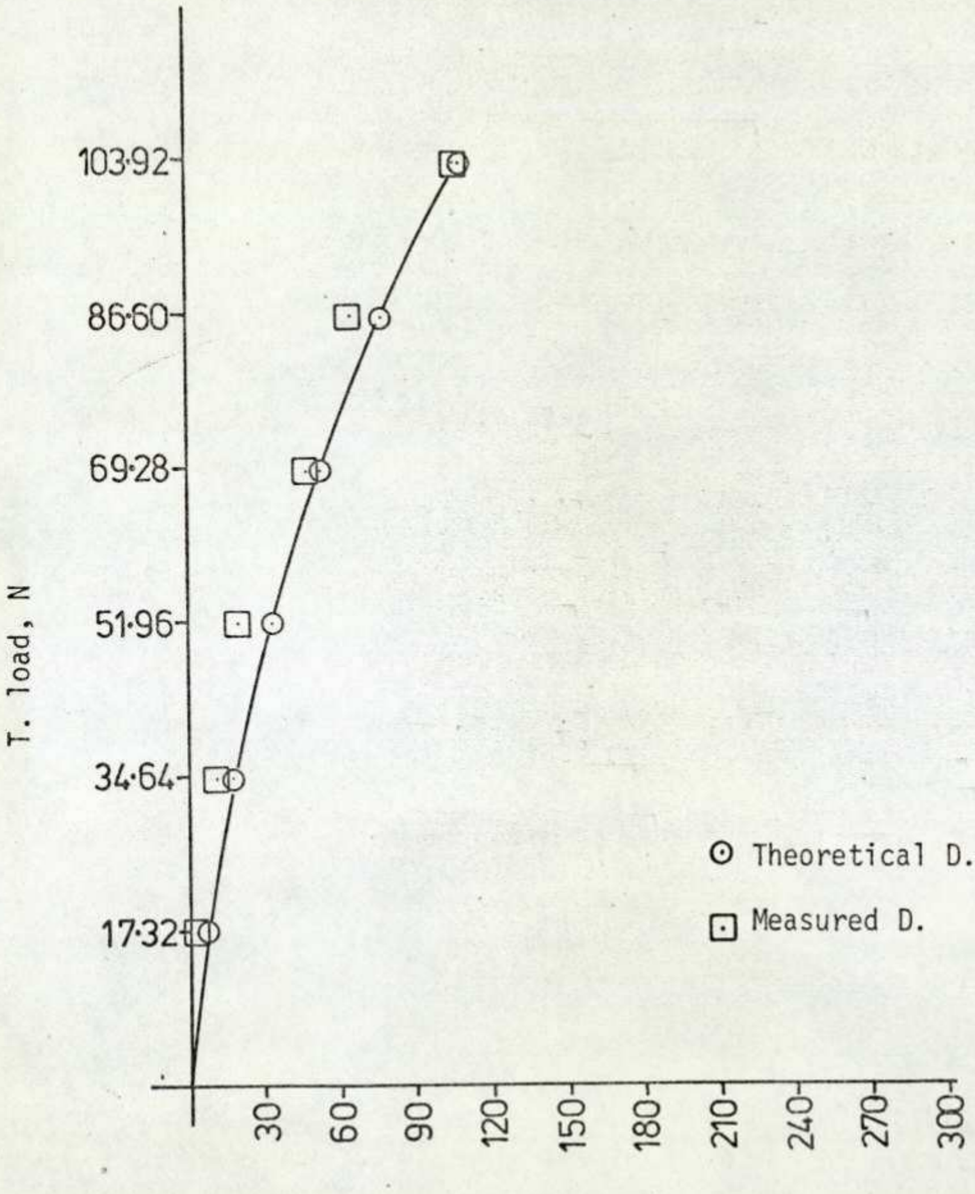


Fig. 8.11 T. Load Vs Theoretical and experimental T. Displacement

Single Batter pile (+30^0)

Test No. 14C

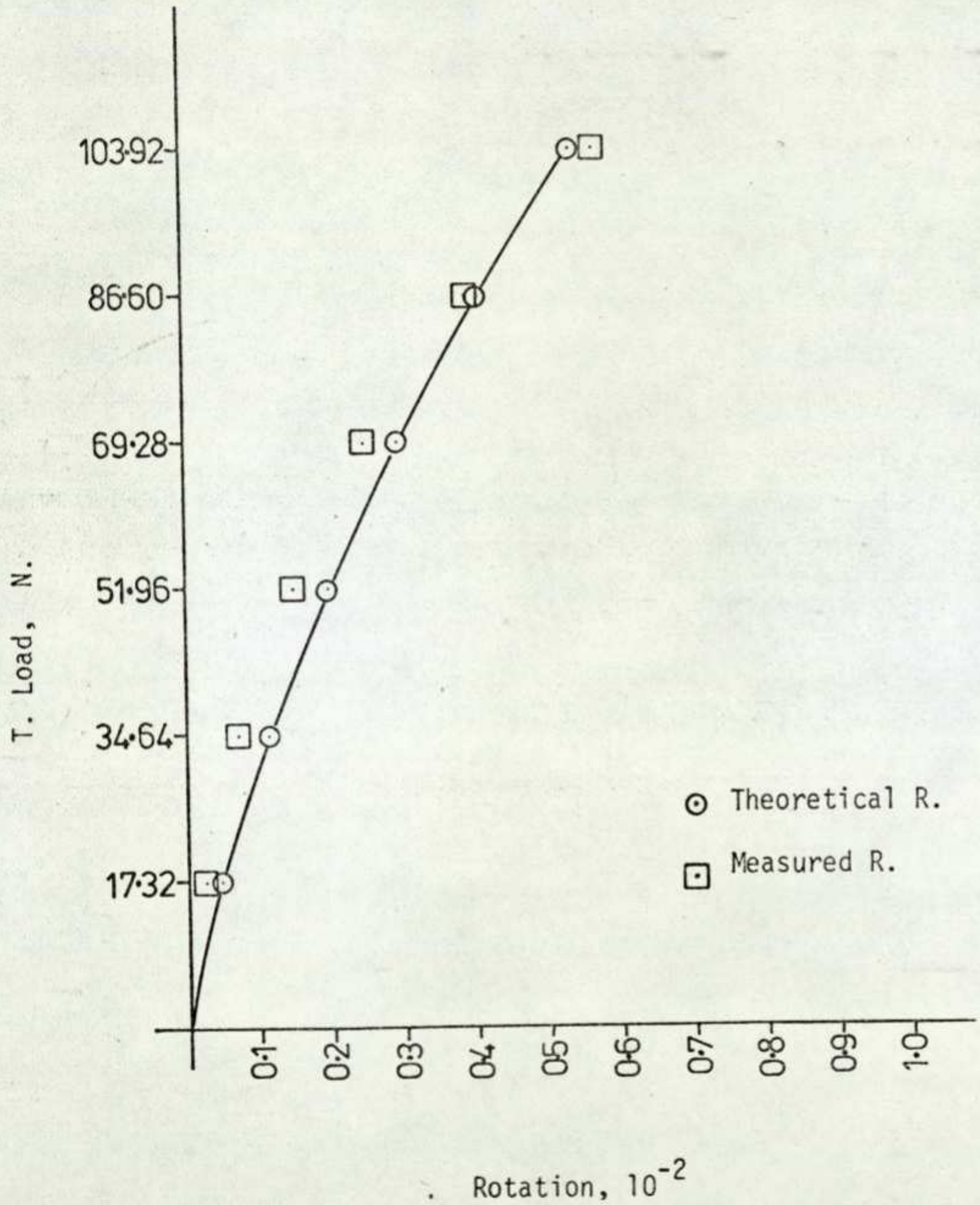


Fig. 8.12 T. Load Vs Theoretical and experimental rotation of the Cap

Single Batter pile (-30^0)

Test No. 14D

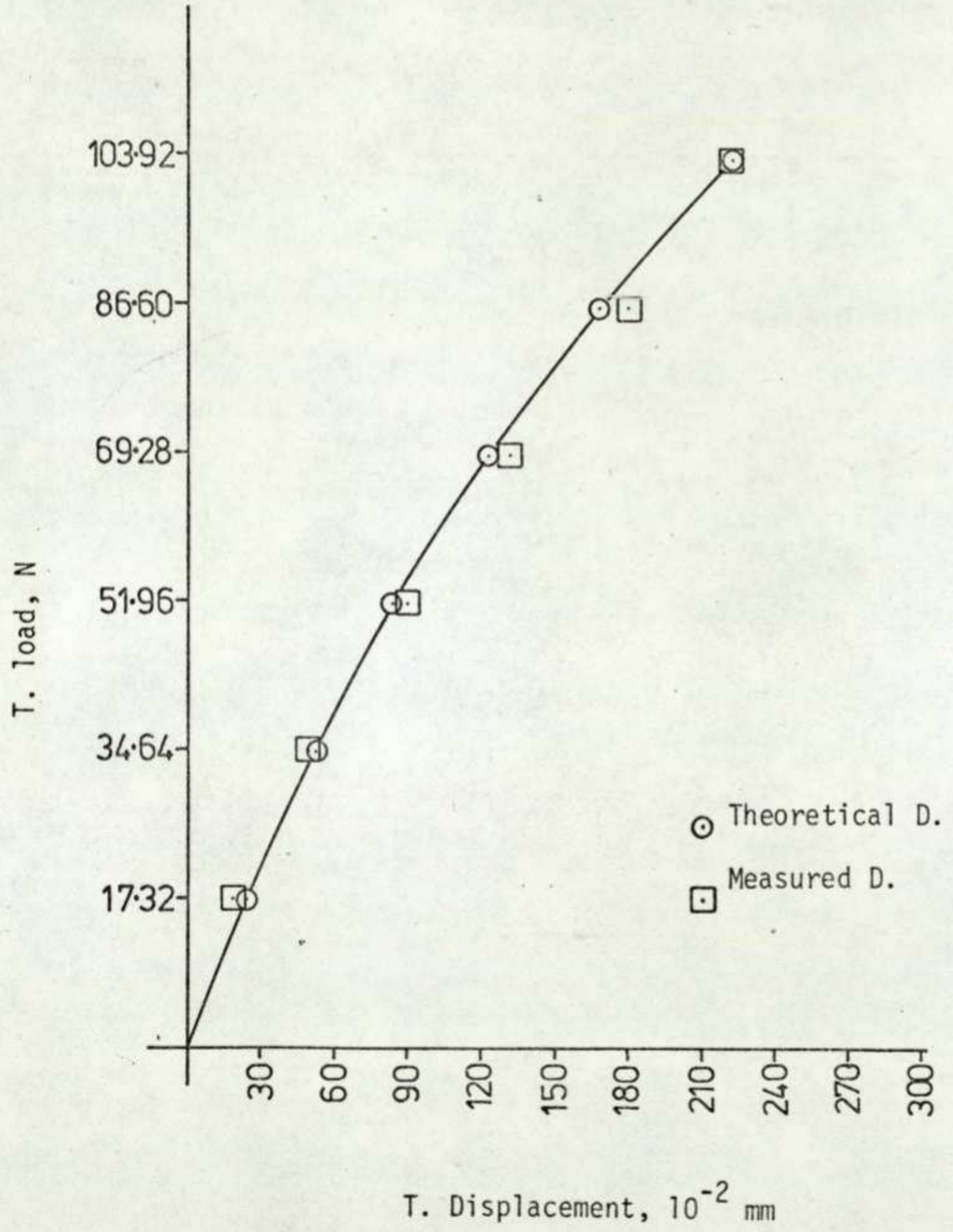


Fig. 8.13 T. Load Vs Theoretical and Experimental T. Displacement

Single Batter pile (-30^0)

Test No. 14D

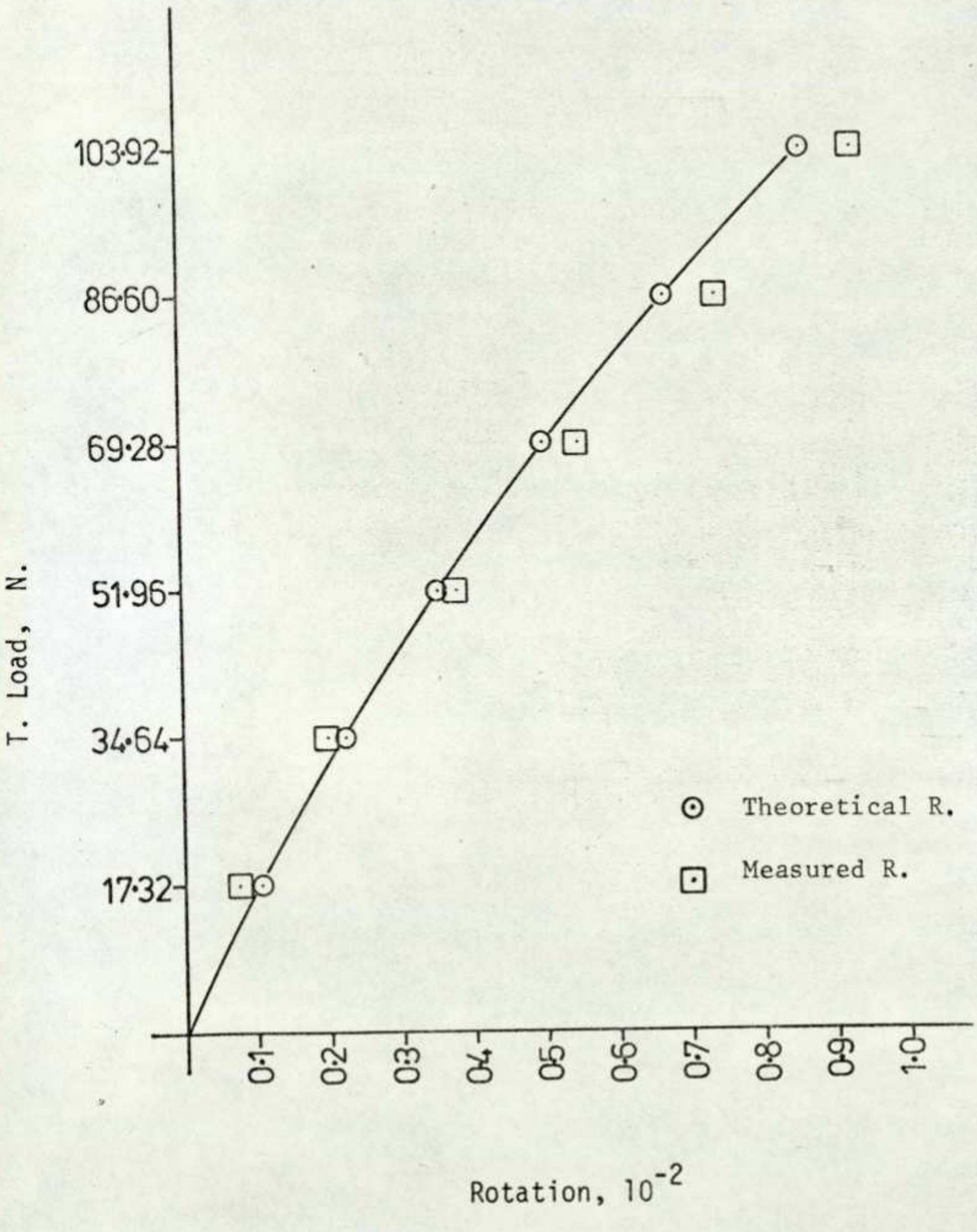
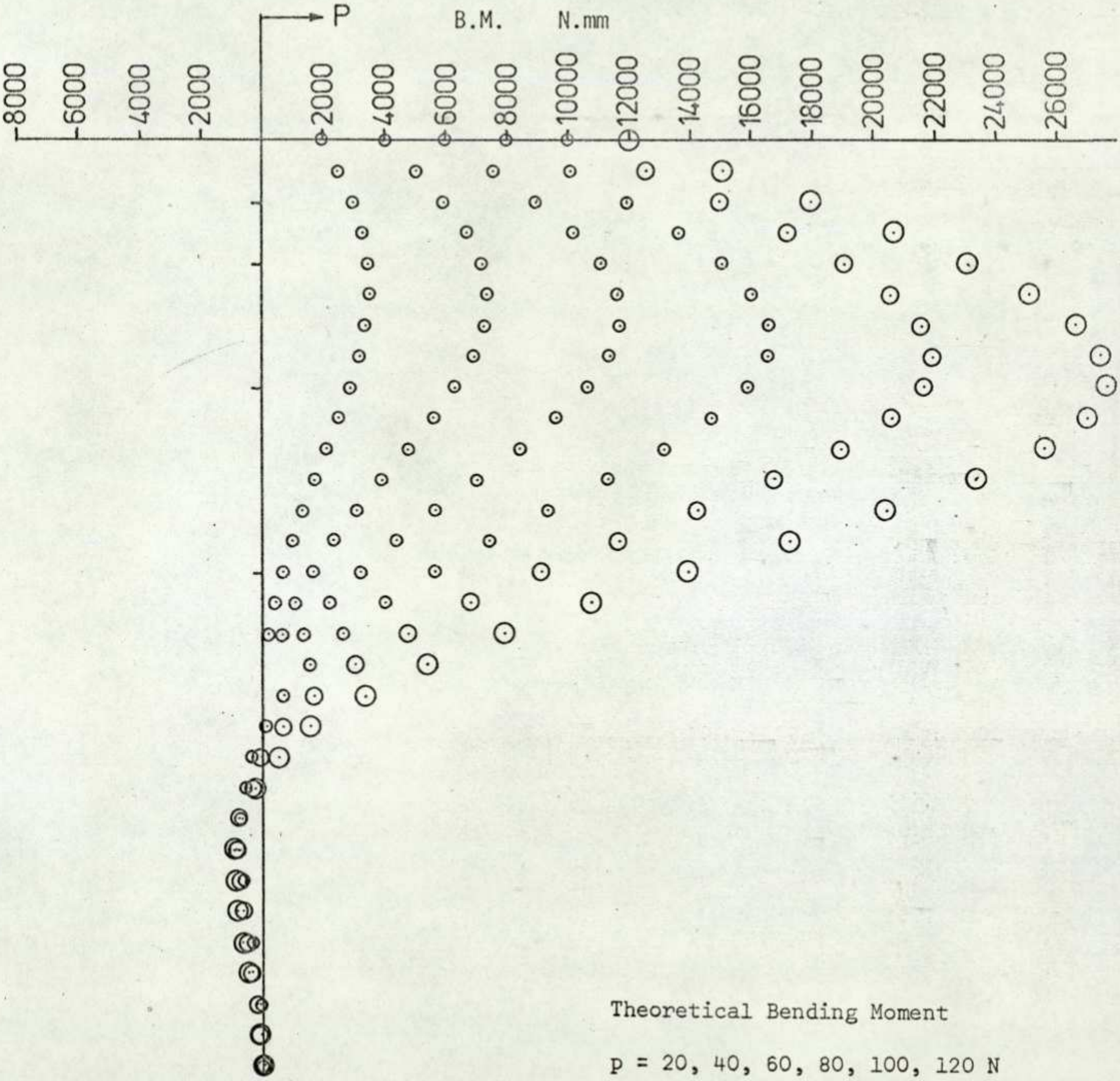


Fig. 8.14 T. Load Vs Theoretical and experimental rotation of the cap

Single Vertical pile



Test No. 13A

Fig. 8.15. Theoretical Moment Vs Depth.

Test No. 13A

Single Vertical pile

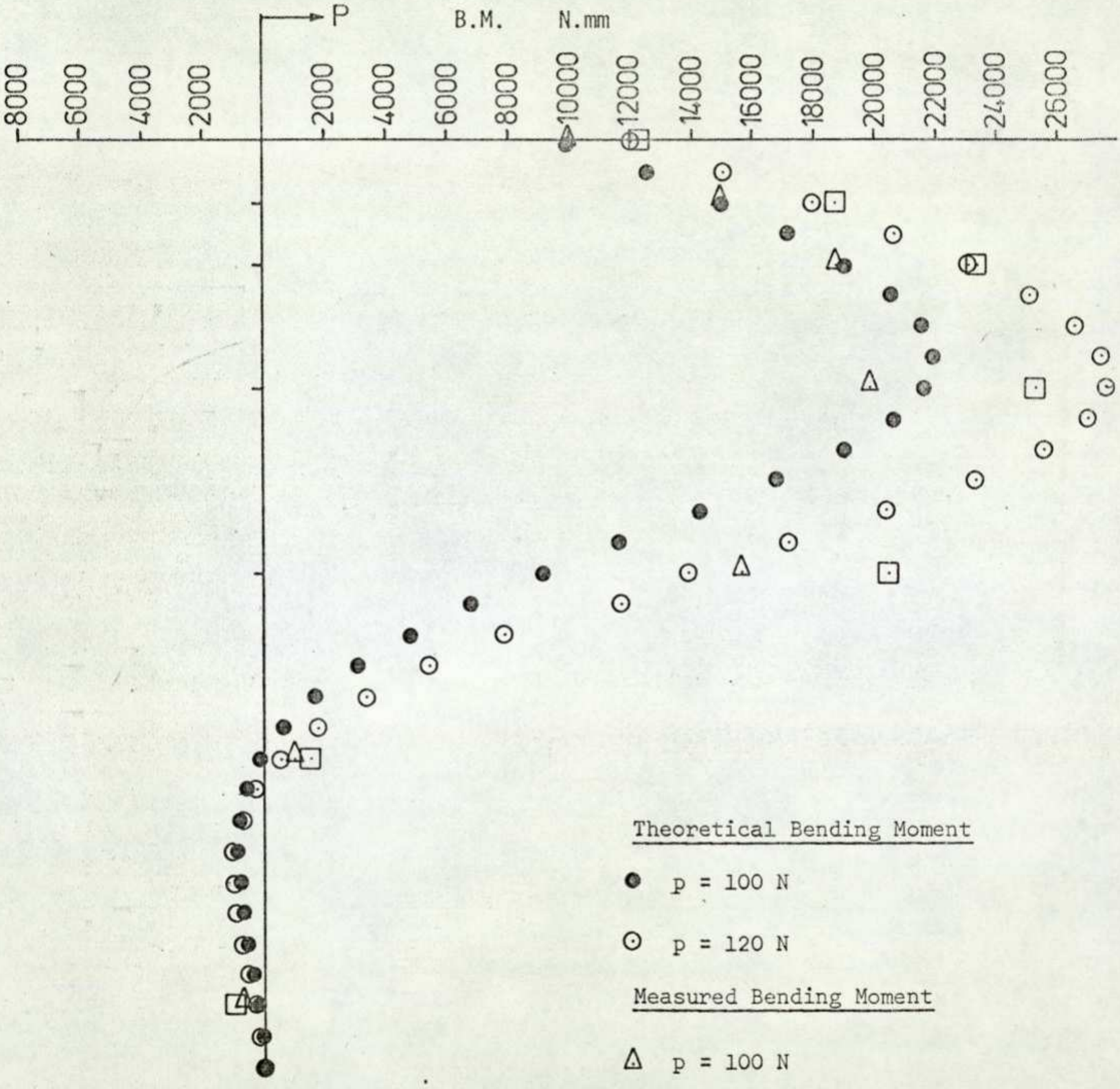


Fig. 8.16. Theoretical and experimental bending moment Vs Depth.

Test No. 14A

Single Batter pile (+ 15°)

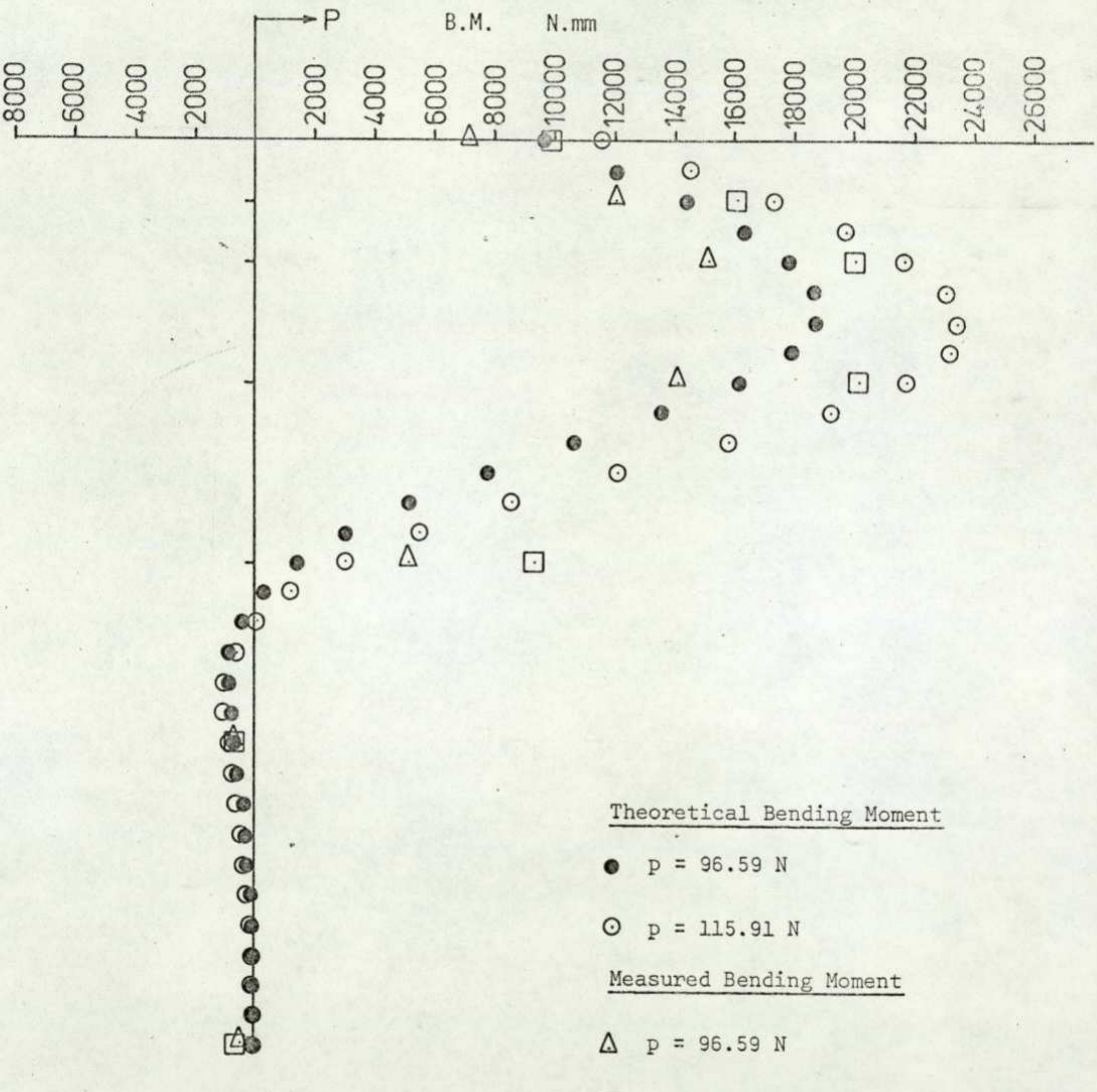


Fig. 8.17. Theoretical and experimental bending moment Vs Depth.

Test No. 14B

Single Batter pile (- 15°)

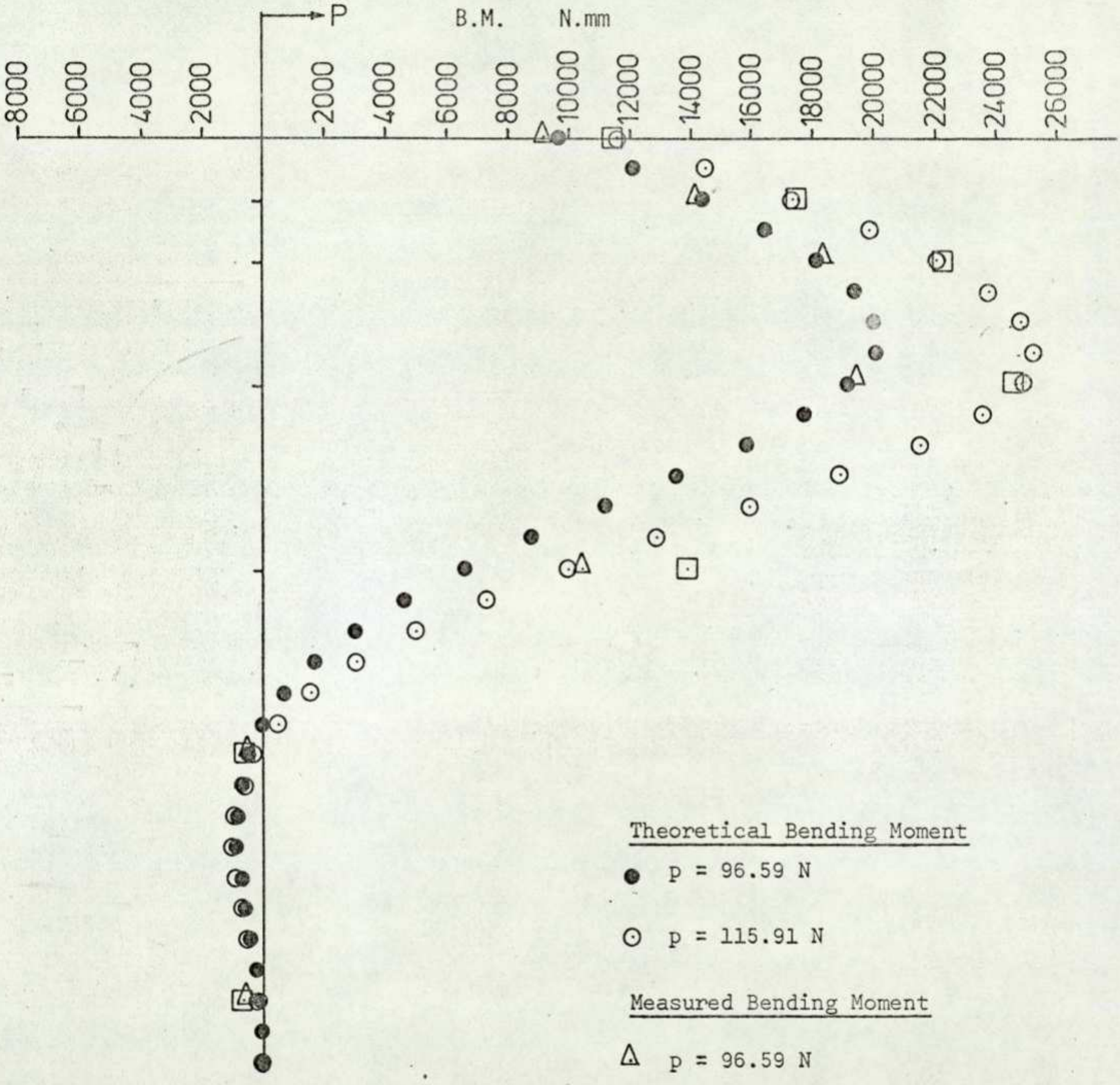


Fig. 8.18. Theoretical and experimental bending moment Vs Depth

Test No. 14C

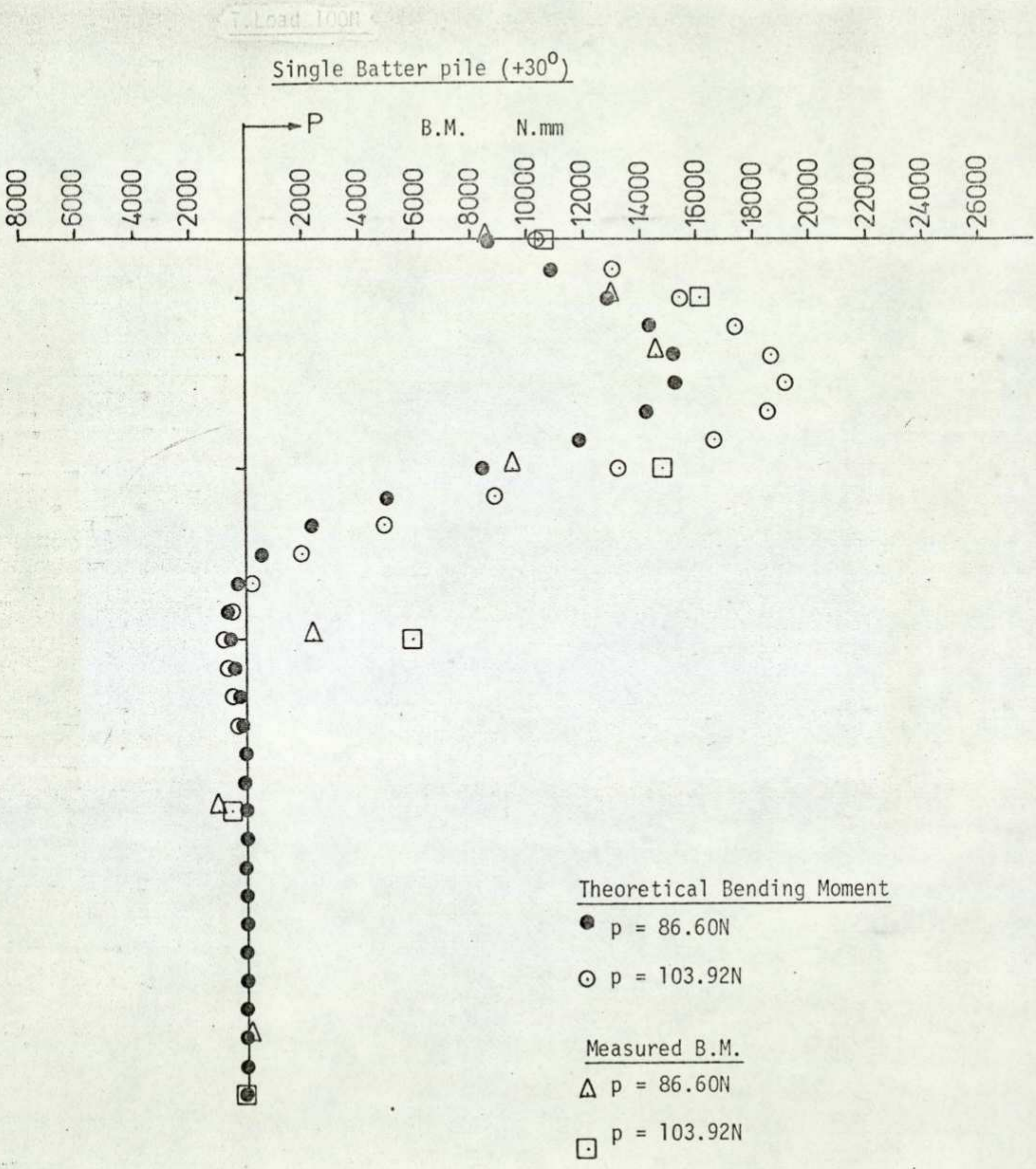


Fig. 8.19 Theoretical and experimental bending moment Vs Depth

Test No. 14D

Single Batter pile (-30^0)

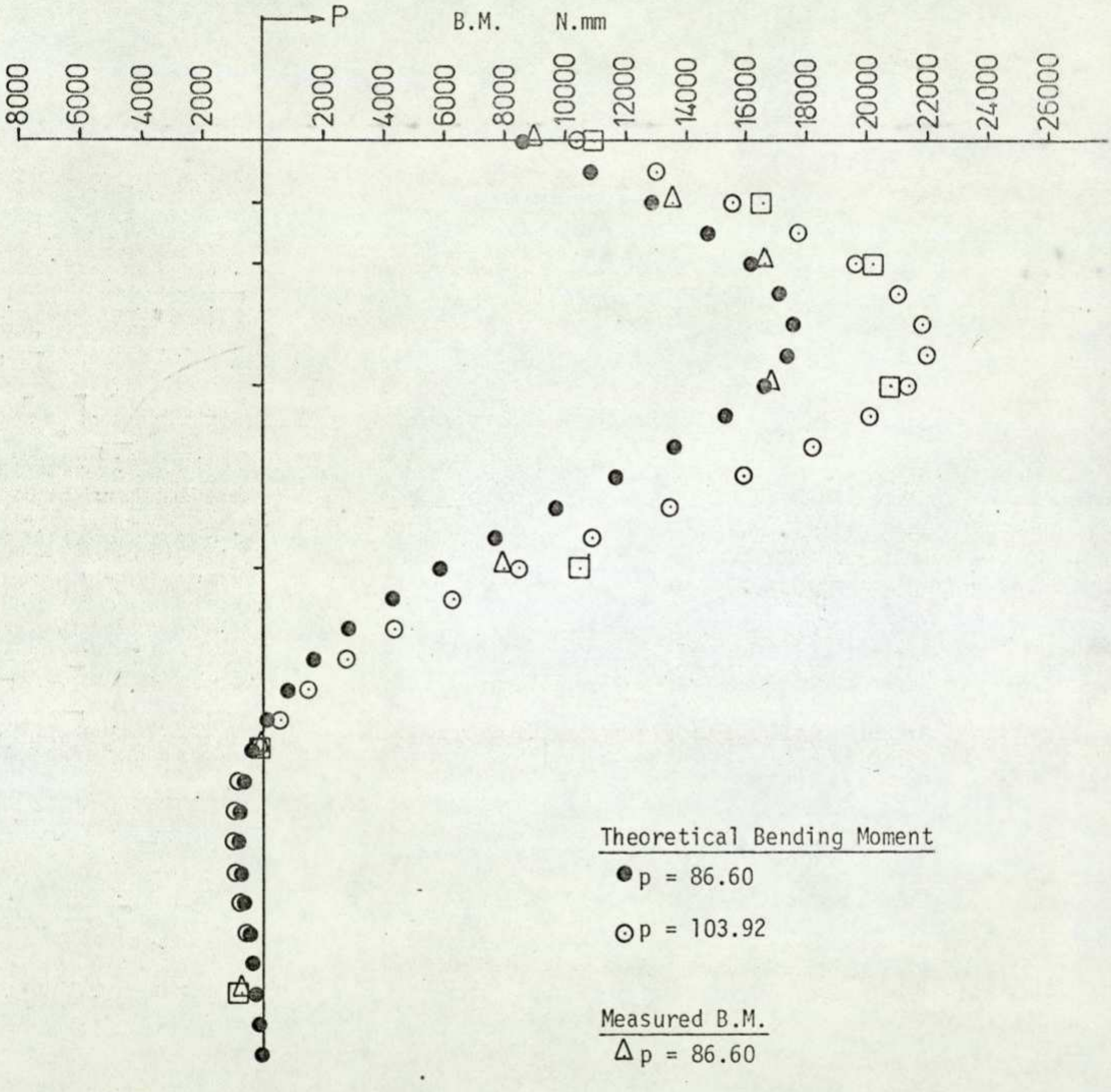


Fig. 8.20 Theoretical and experimental bending moment Vs Depth

Four - Pile Group (2x2)

Test No. 1

2V, -2B, 15^0

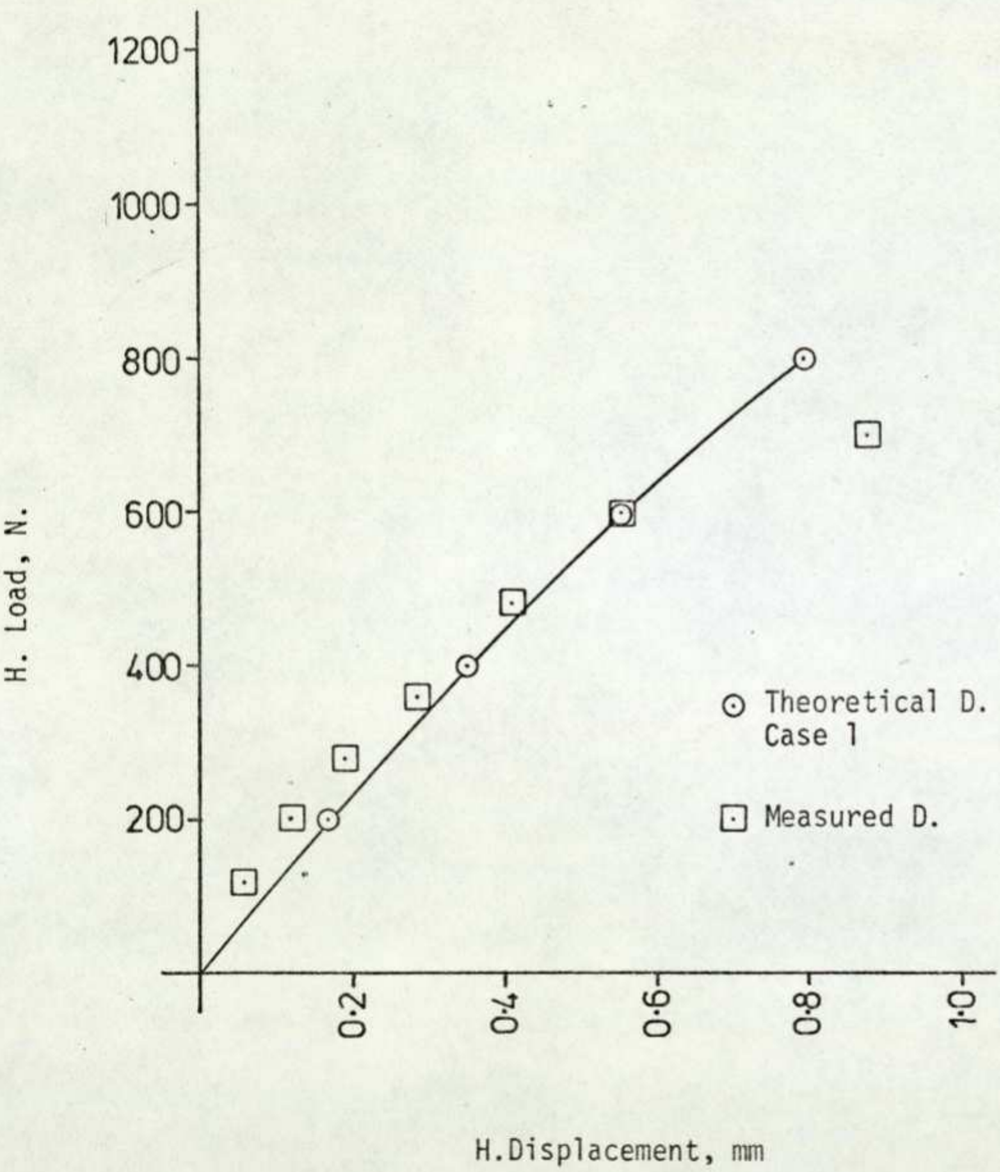


Fig. 8.21 H. load Vs Theoretical and experimental displacement

Four-Pile Group (2x2)

Test No. 2

+2B, 2V 15°

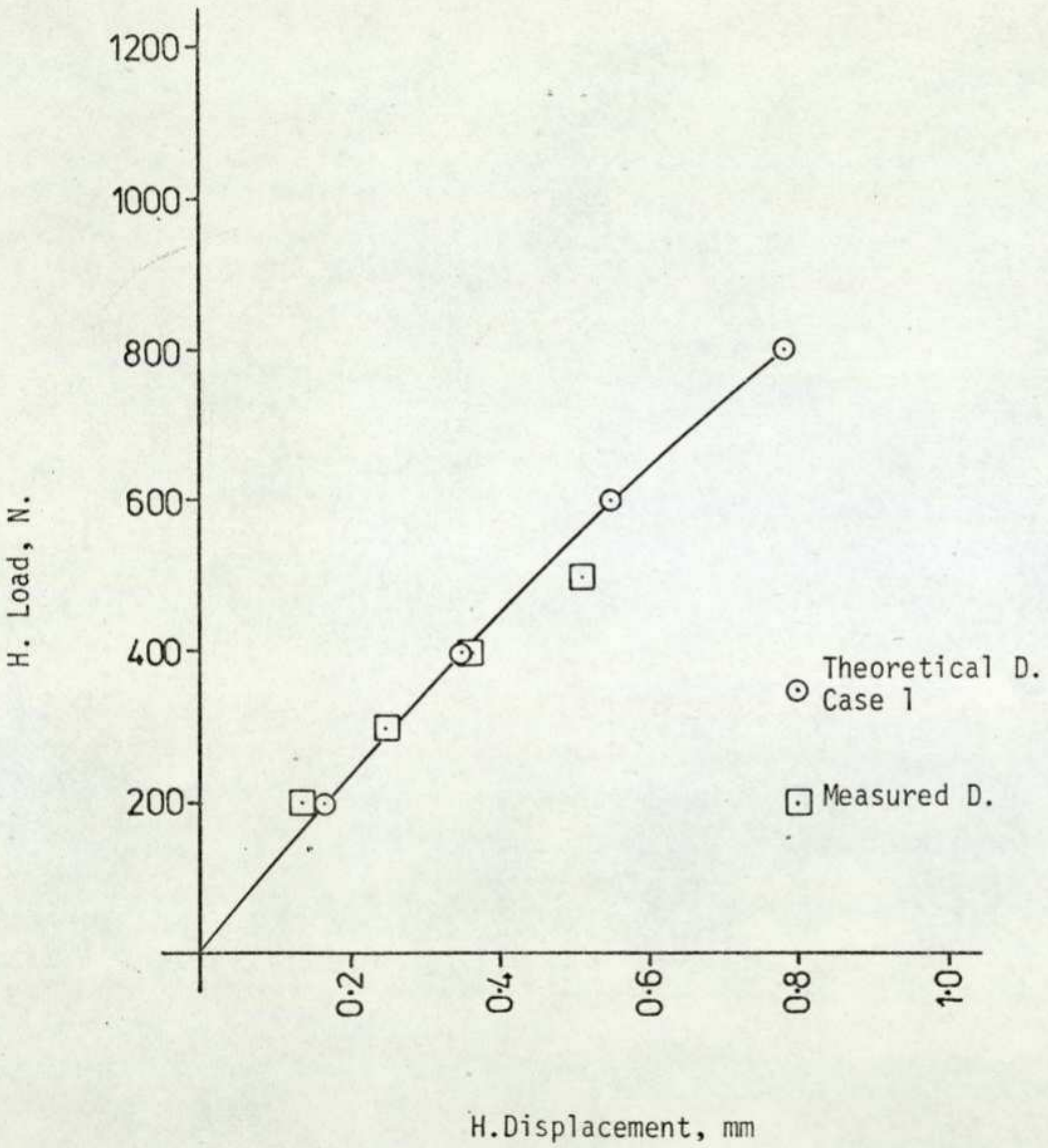


Fig. 8.22 H. load Vs Theoretical and experimental displacement

Four-Pile Group (2 x 2)

Test No. 8

+ 2B, - 2B, 15^{0}

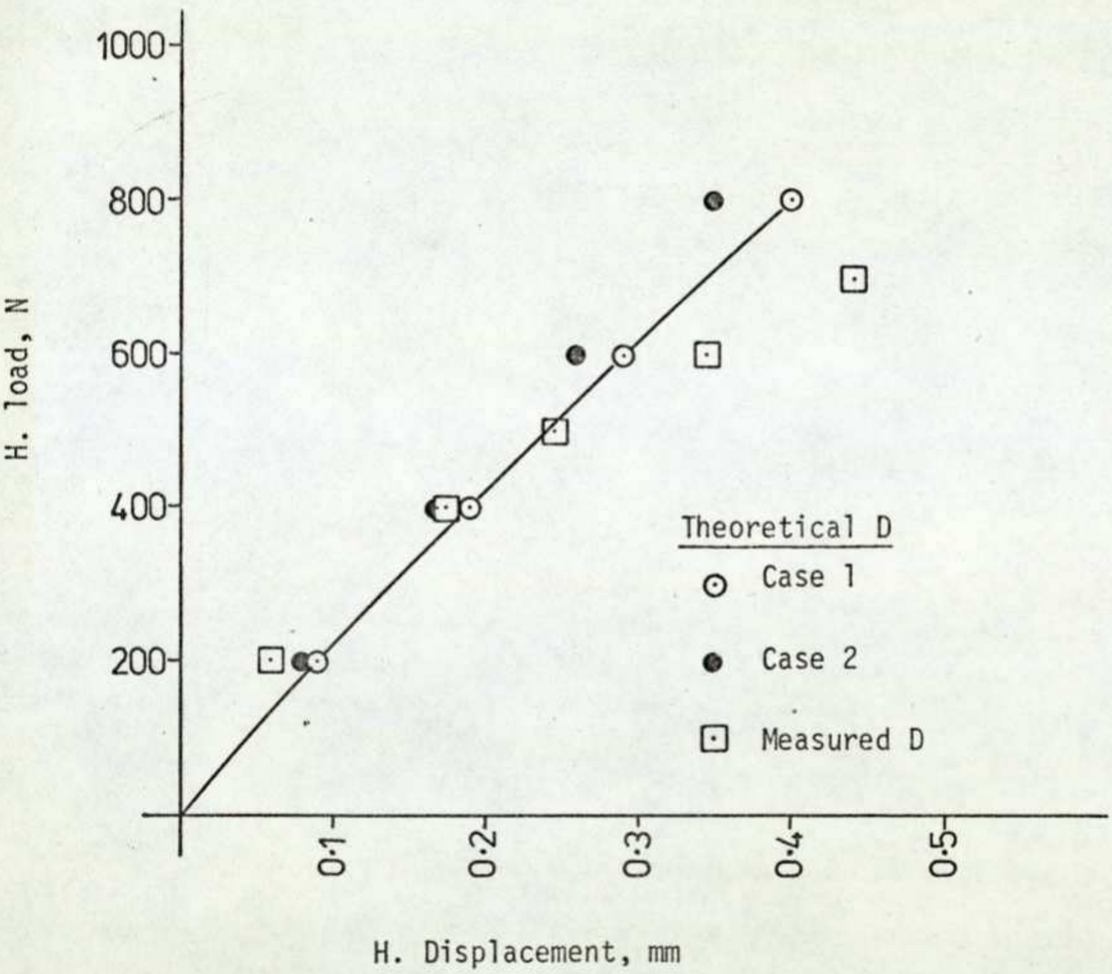


Fig. 8.23 H. load Vs Theoretical and experimental displacement

Four-Pile Group (2 x 2)

Test No. 9

4 V

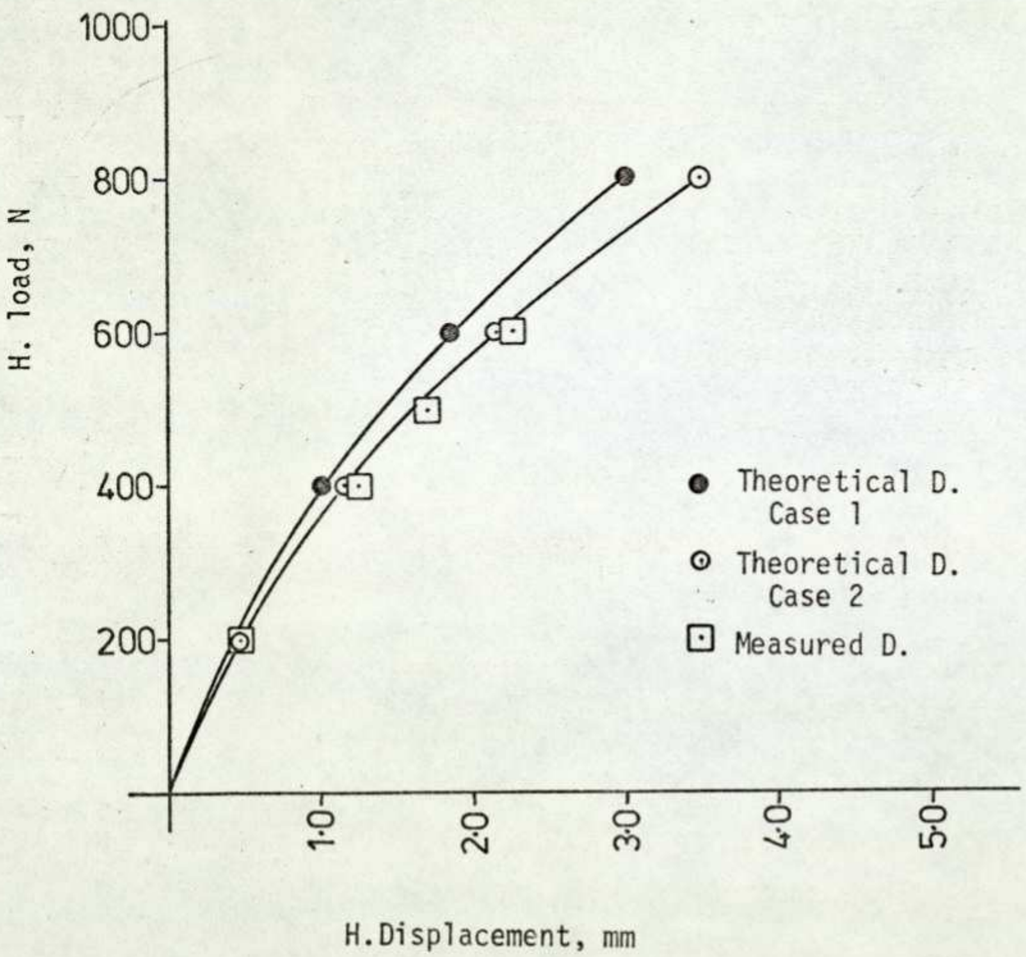


Fig. 8.24 H. load Vs Theoretical and experimental displacement

Four-Pile Group (2 x 2)

Test No. 9

4V

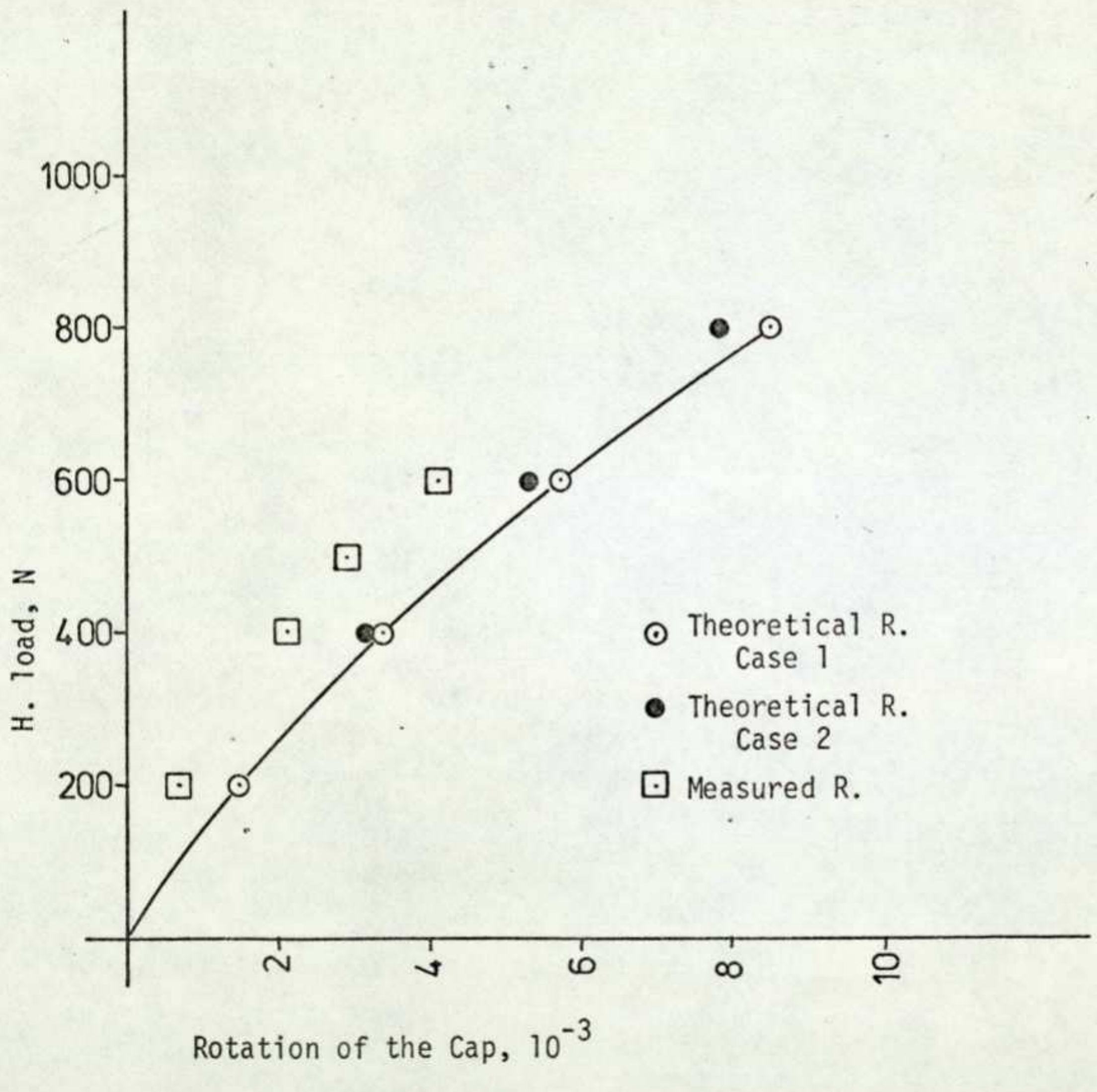


Fig. 8.25 H. Load Vs Theoretical and experimental rotation of the cap

Four-Pile Group (2 x 2)

Test No. 12
+2B, 2V 30^0

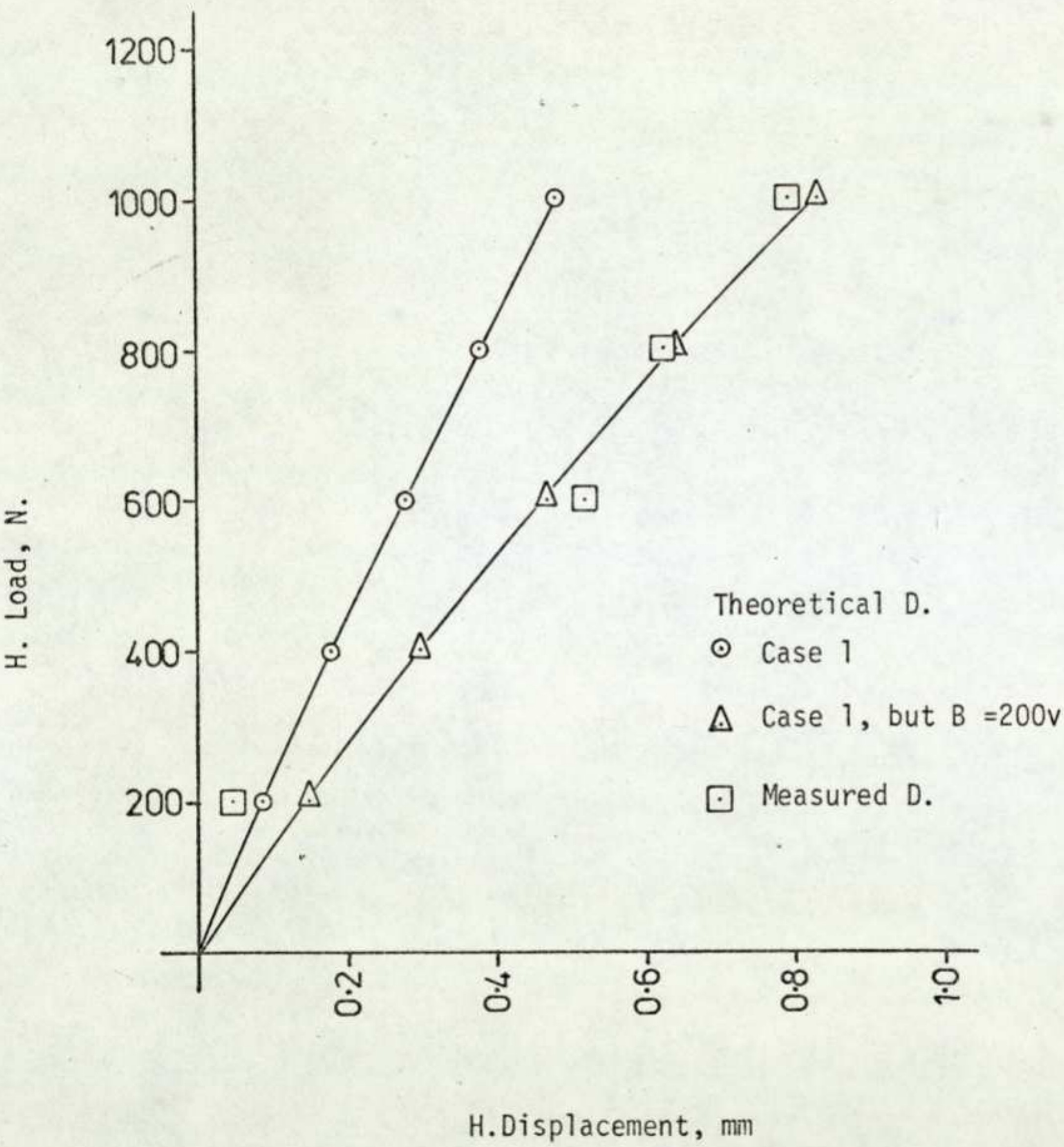


Fig. 8.26 H. load Vs Theoretical and experimental displacement

Four-Pile Group (2x2)

Test No. 1

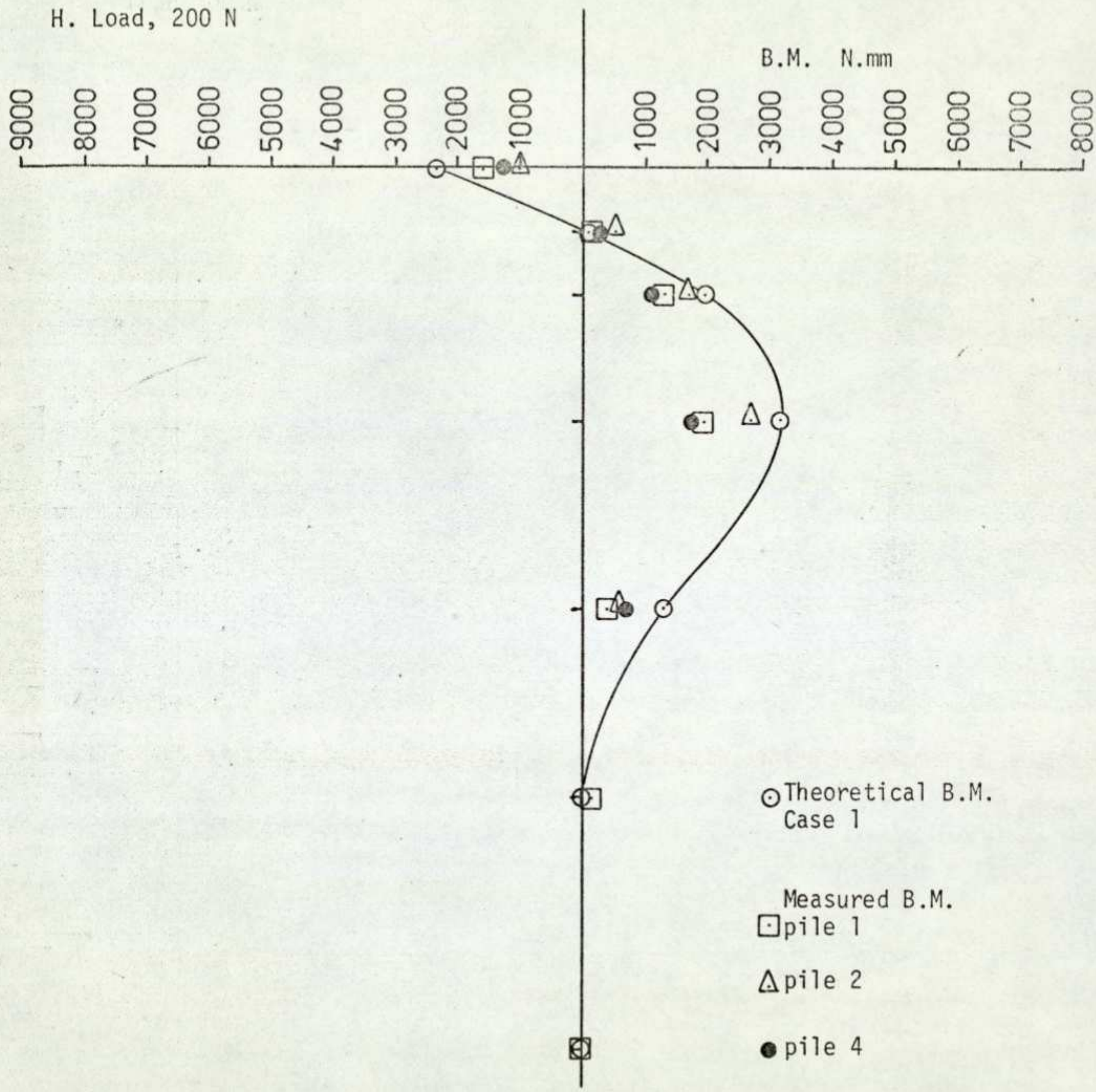


Fig . 8.27 Theoretical and experimental bending moment Vs Depth

Four-Pile Group (2 x 2)

Test No. 2

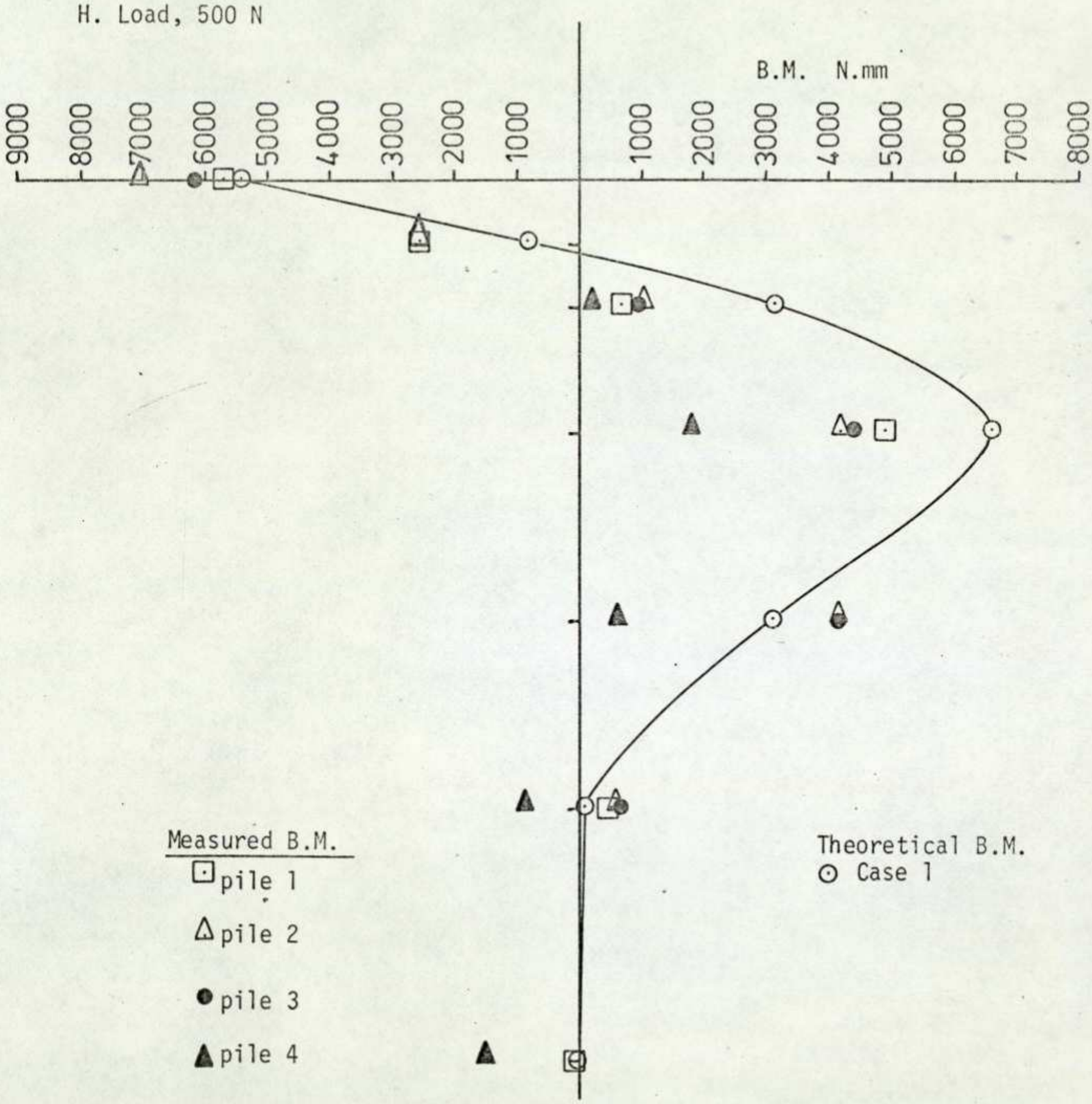


Fig. 8.28 Theoretical and experimental bending moment Vs Depth

Four-Pile Group (2 x 2)

Test No. 8

H. Load = 600 N

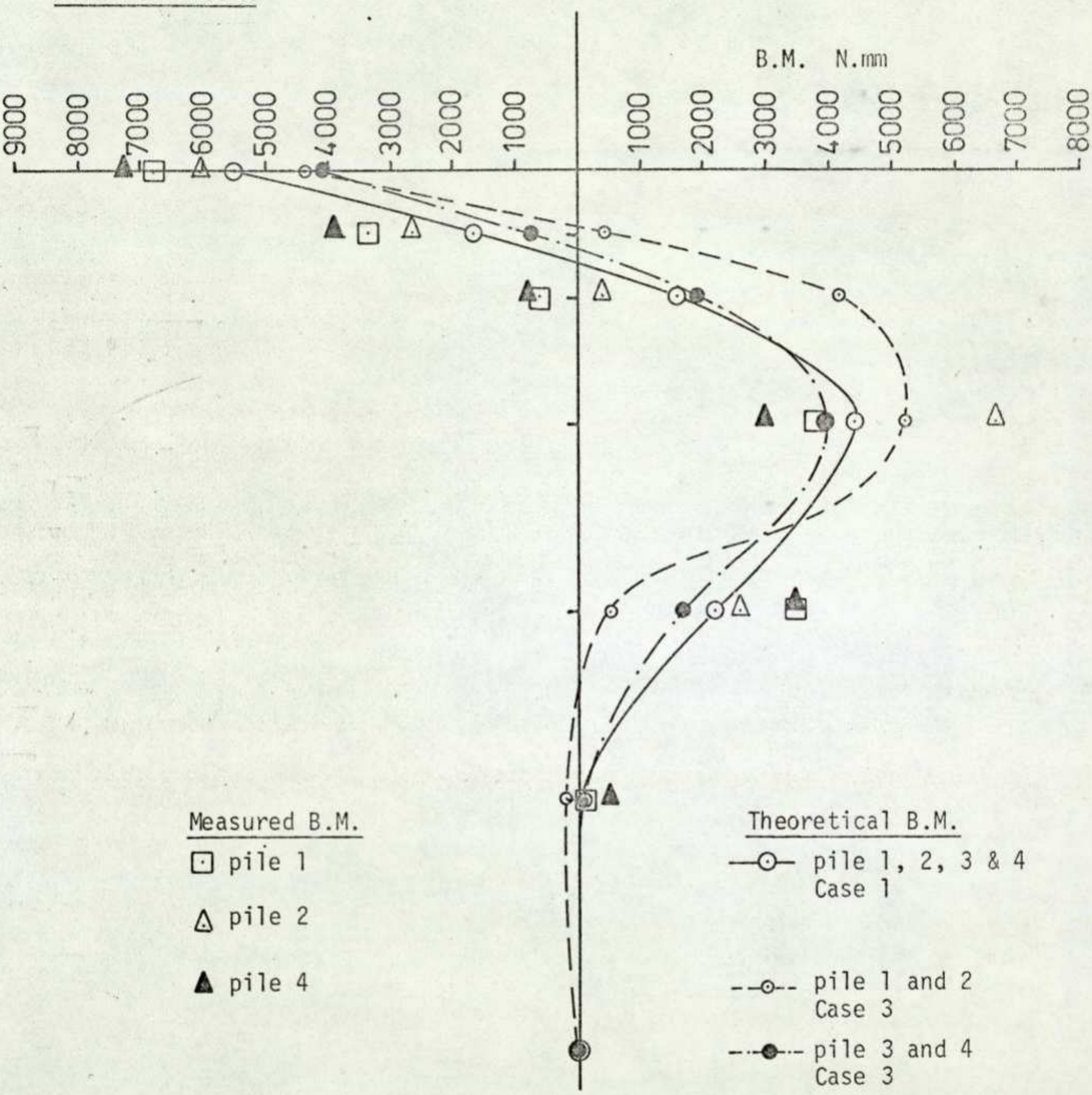


Fig. 8.29 Theoretical and experimental bending moments Vs Depth.

Four-Pile Group (2 x 2)

Test No. 9

H. Load, 600N

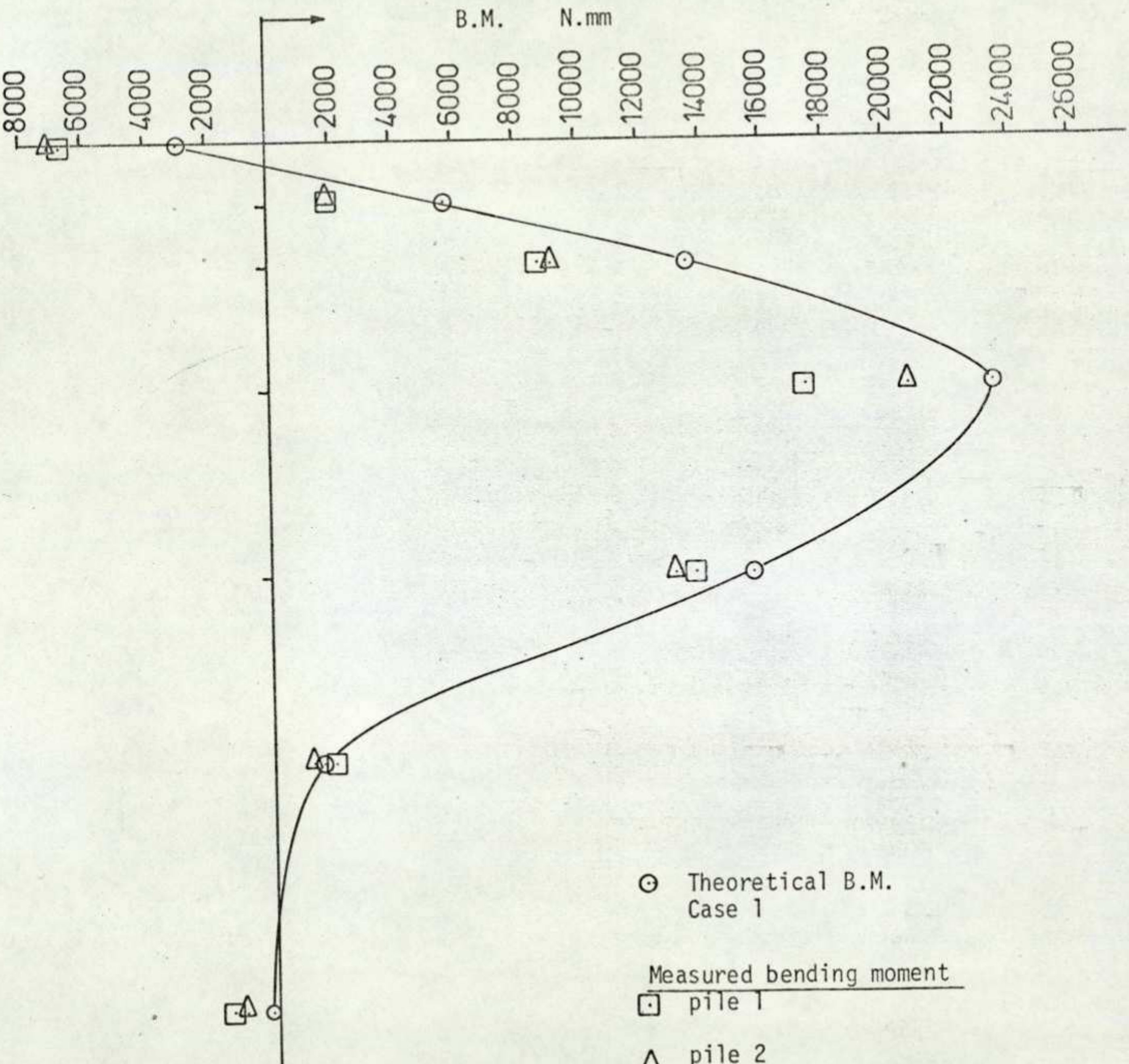


Fig. 8.30 Theoretical and experimental bending moments Vs Depth

Four-Pile Group (2x2)

T.Load 100N

Test No. 9

H. Load, = 600 N

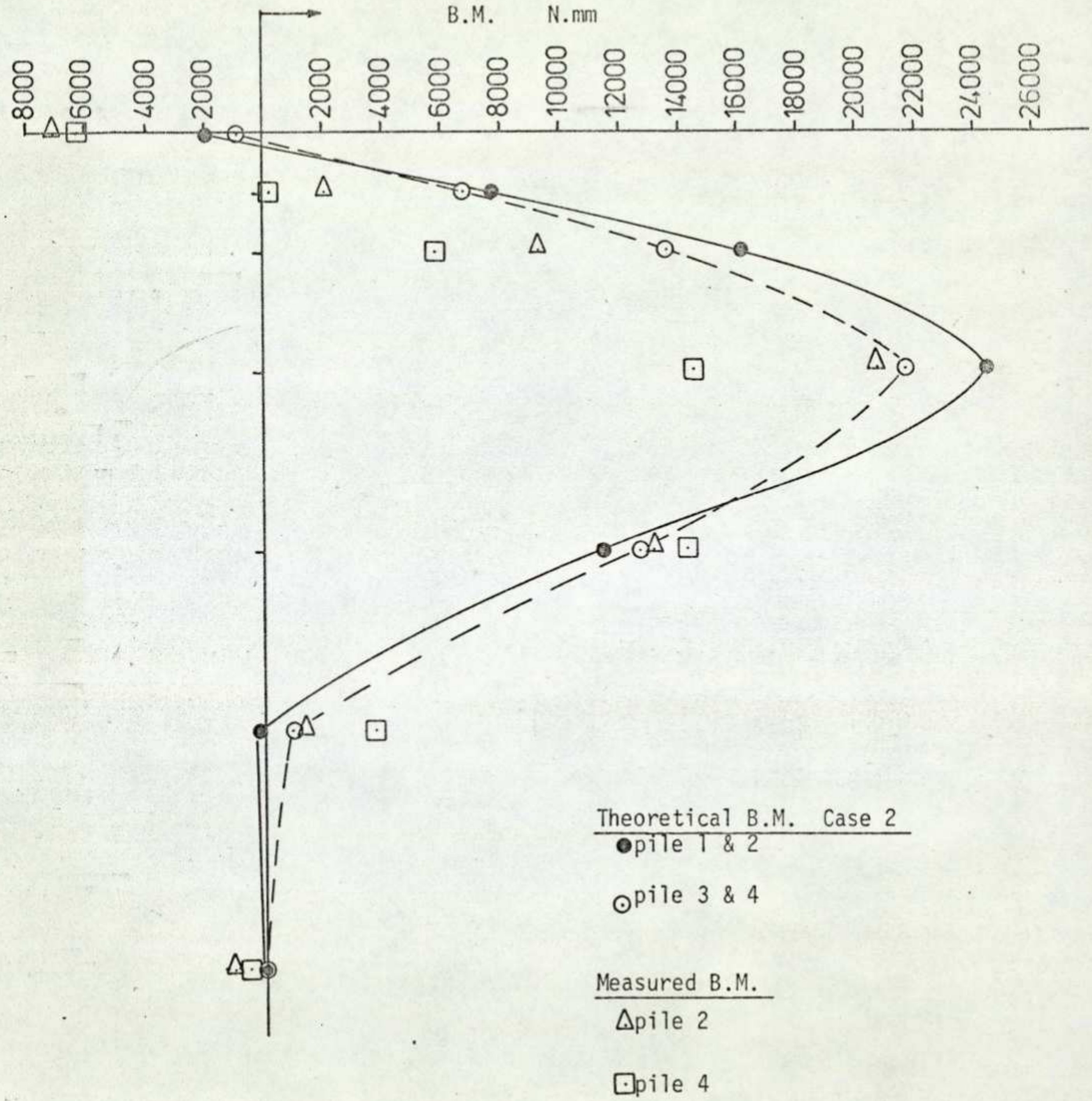


Fig. 8.31 Theoretical and experimental bending moments Vs Depth

Four-Pile Group (2 x 2)

Test No. 12

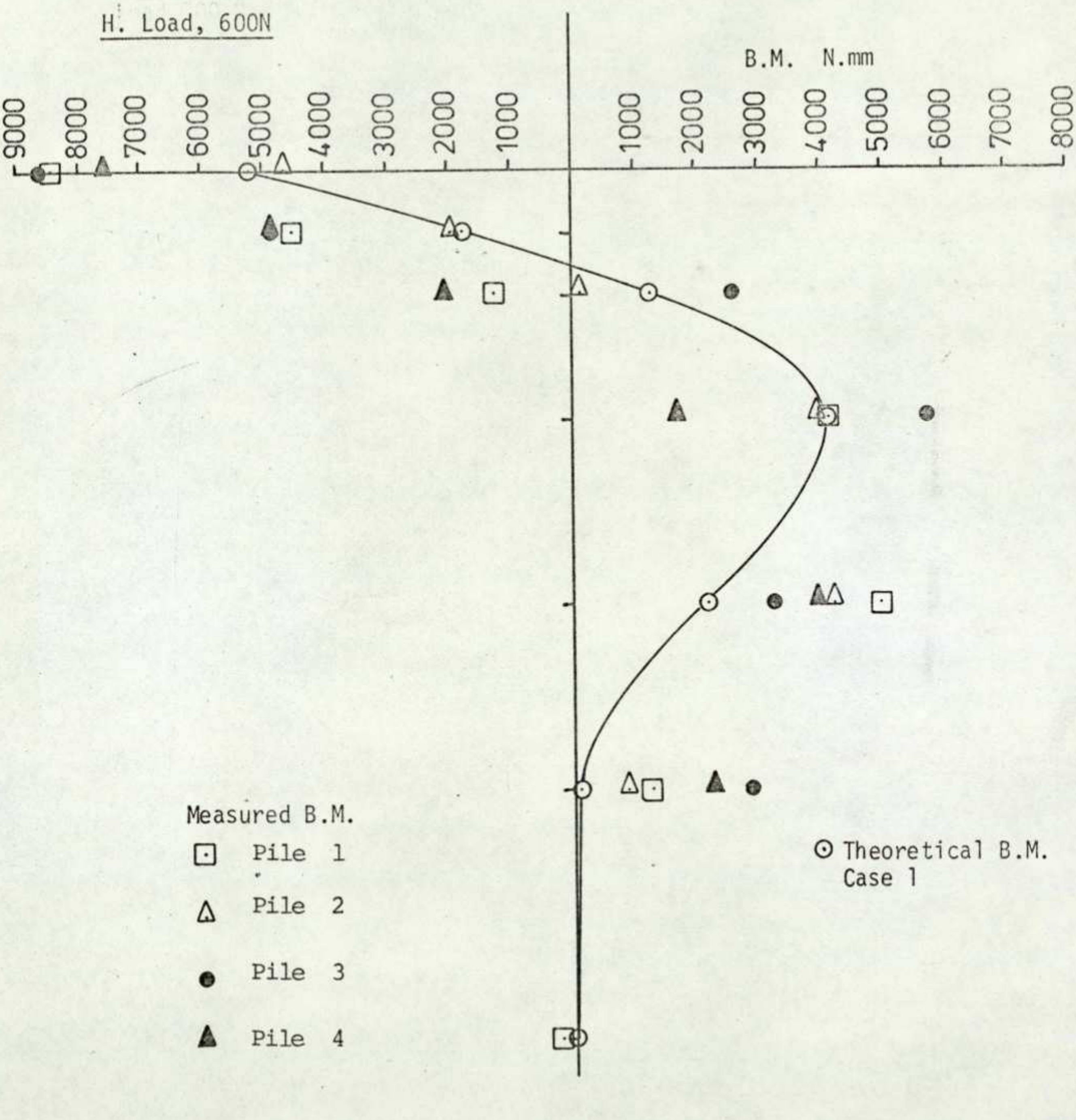


Fig. 8.32 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3 x 3)

Test No. 4

3B, 6V 15^{o}

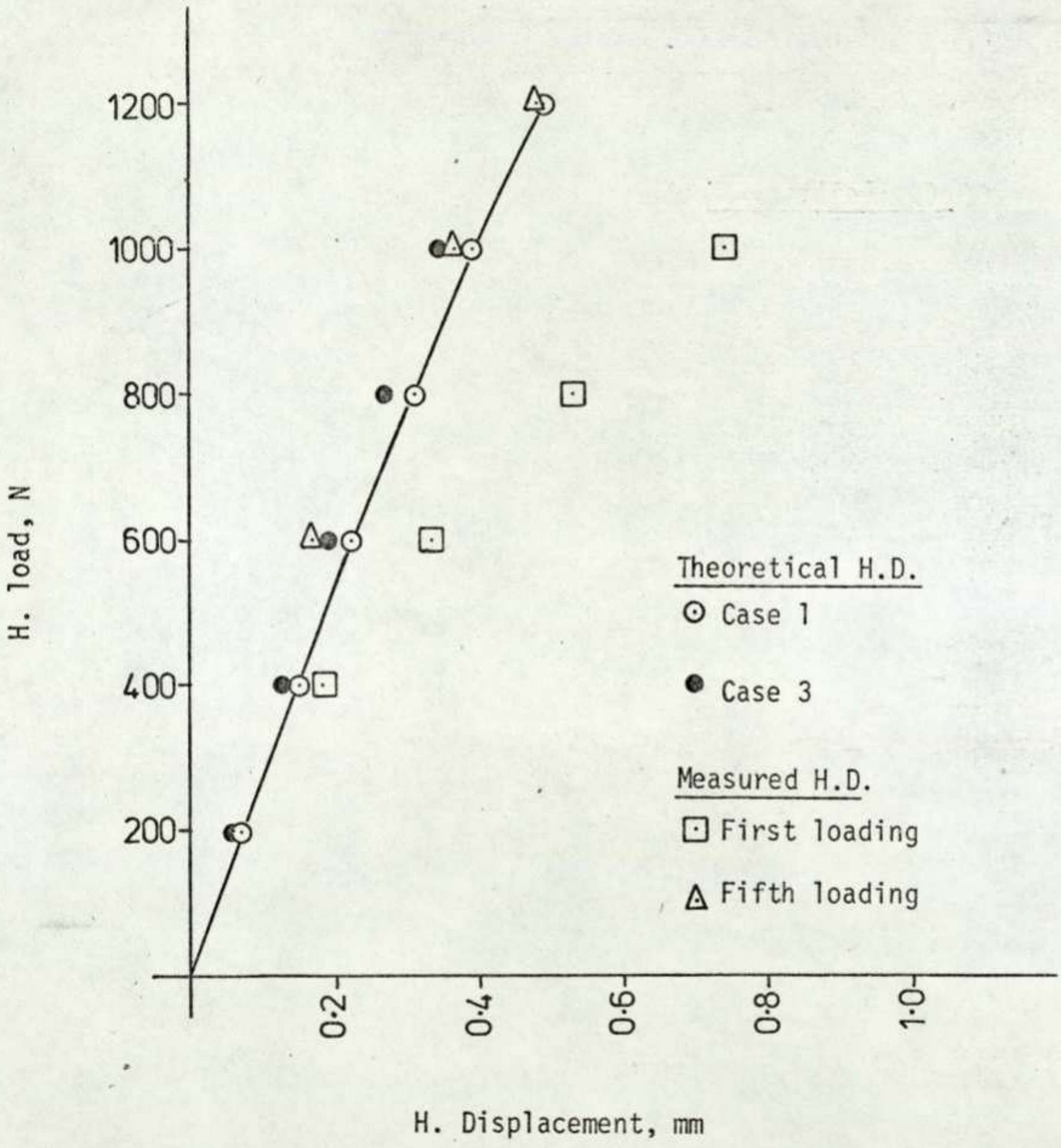


Fig. 8.33 H. load Vs Theoretical and experimental displacement

Nine-Pile Group (3x3)

H. Load, 1000 N

Test No. 4

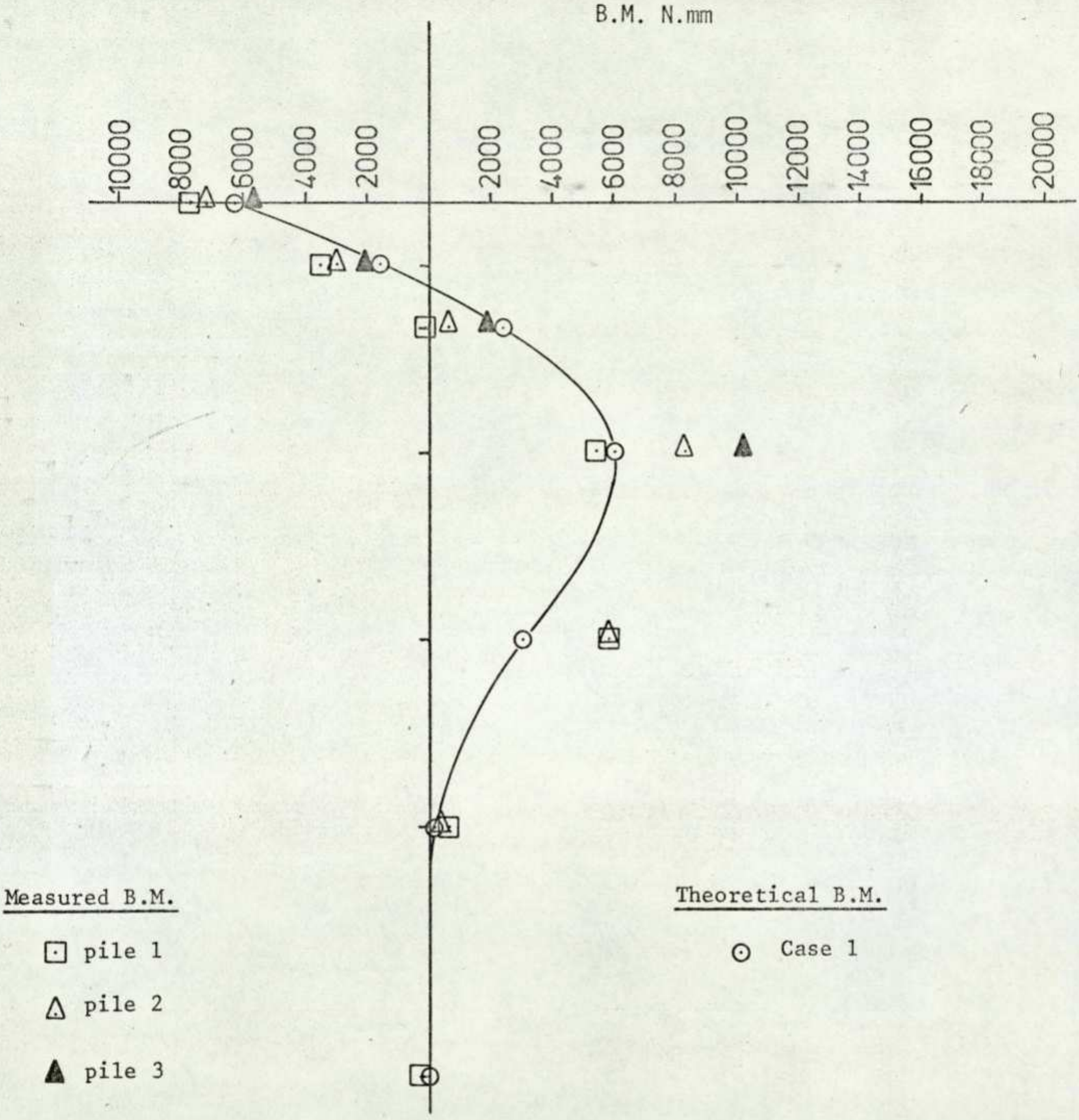


Fig. 8.34 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

H. Load, 1000N

Test No. 4

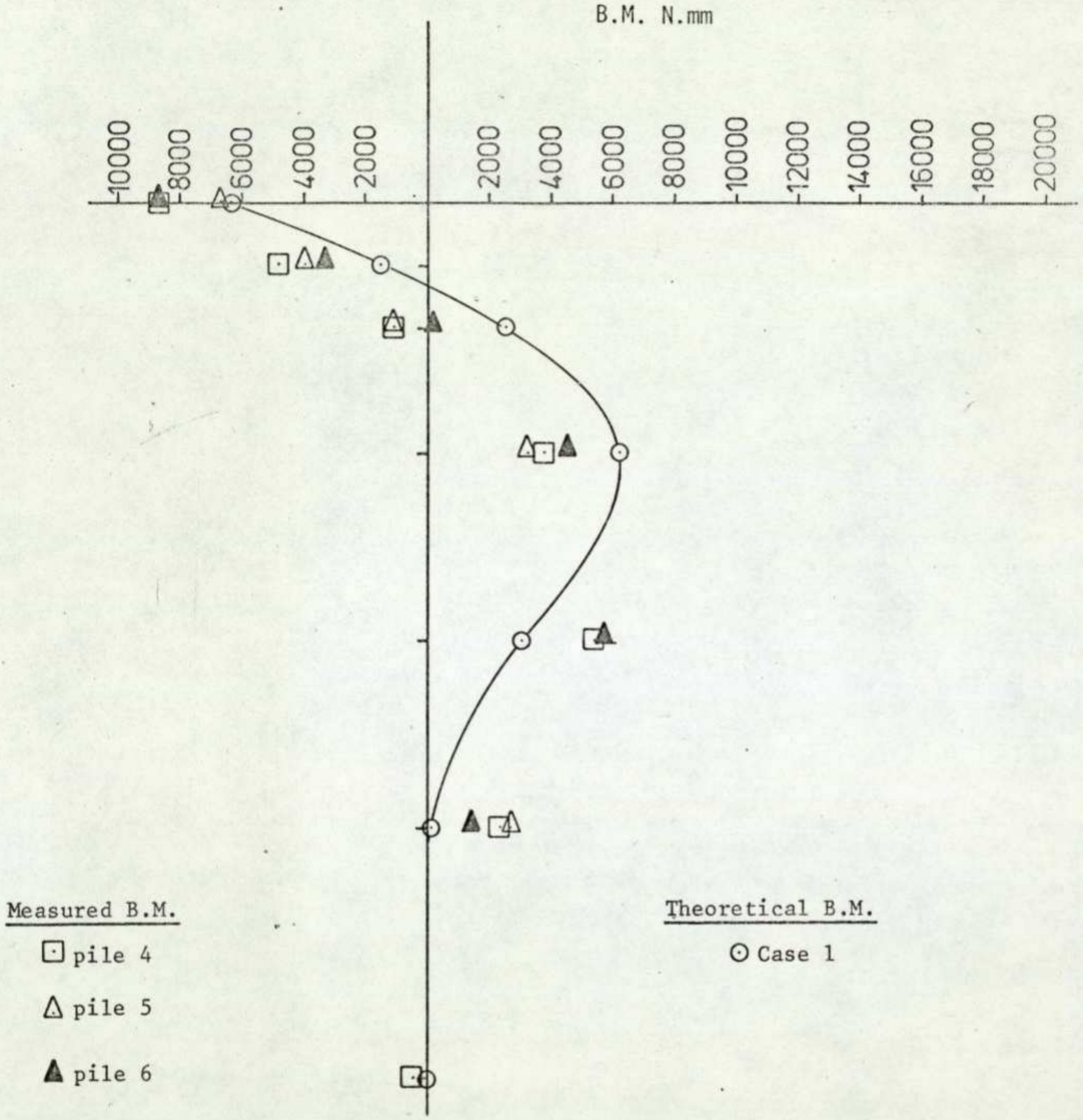


Fig. 8.35 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

H. Load, 100N

Test No. 4

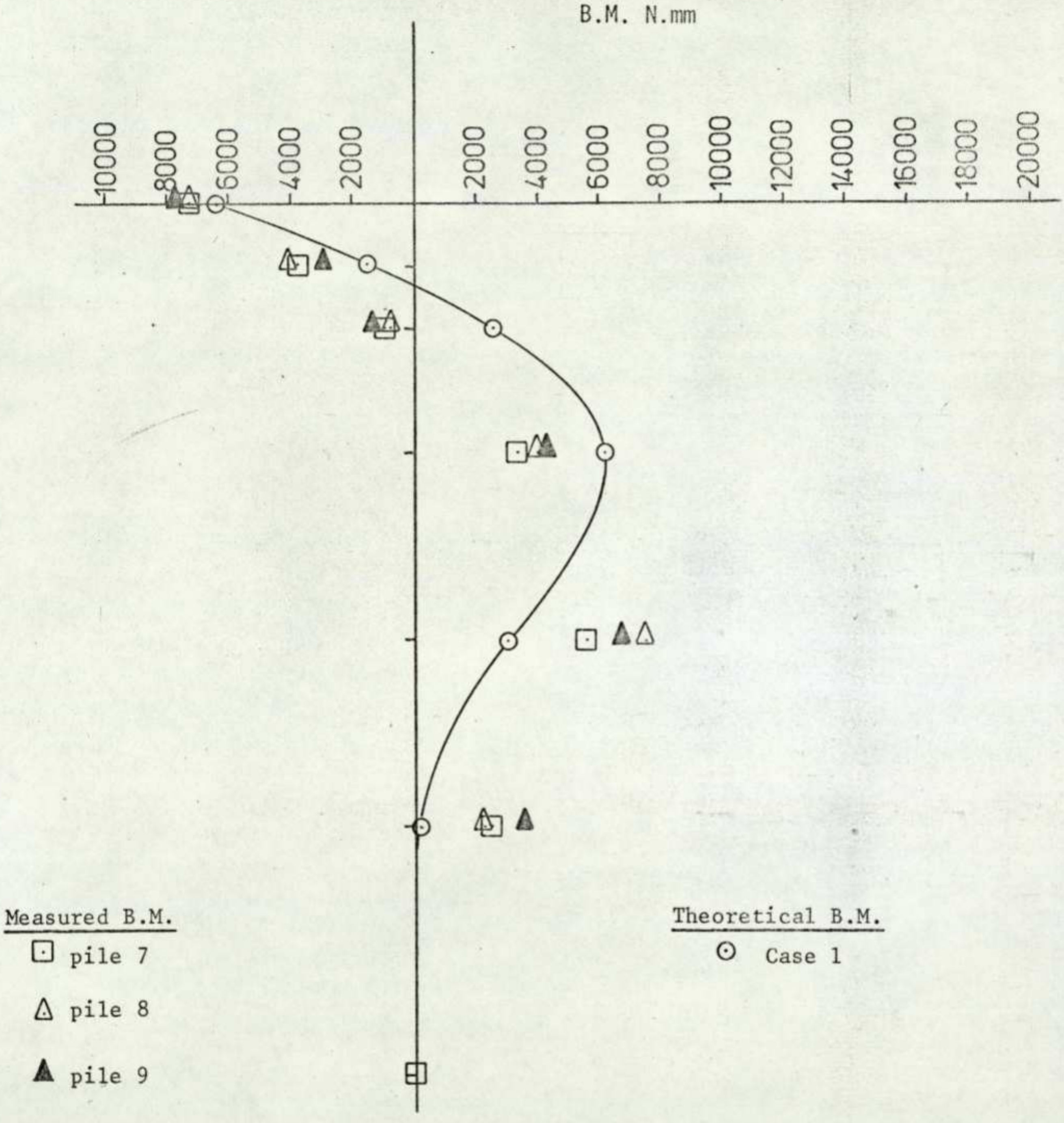


Fig. 8.36 Theoretical and experimental bending moment Vs Depth

Nine- pile group (3 x 3)

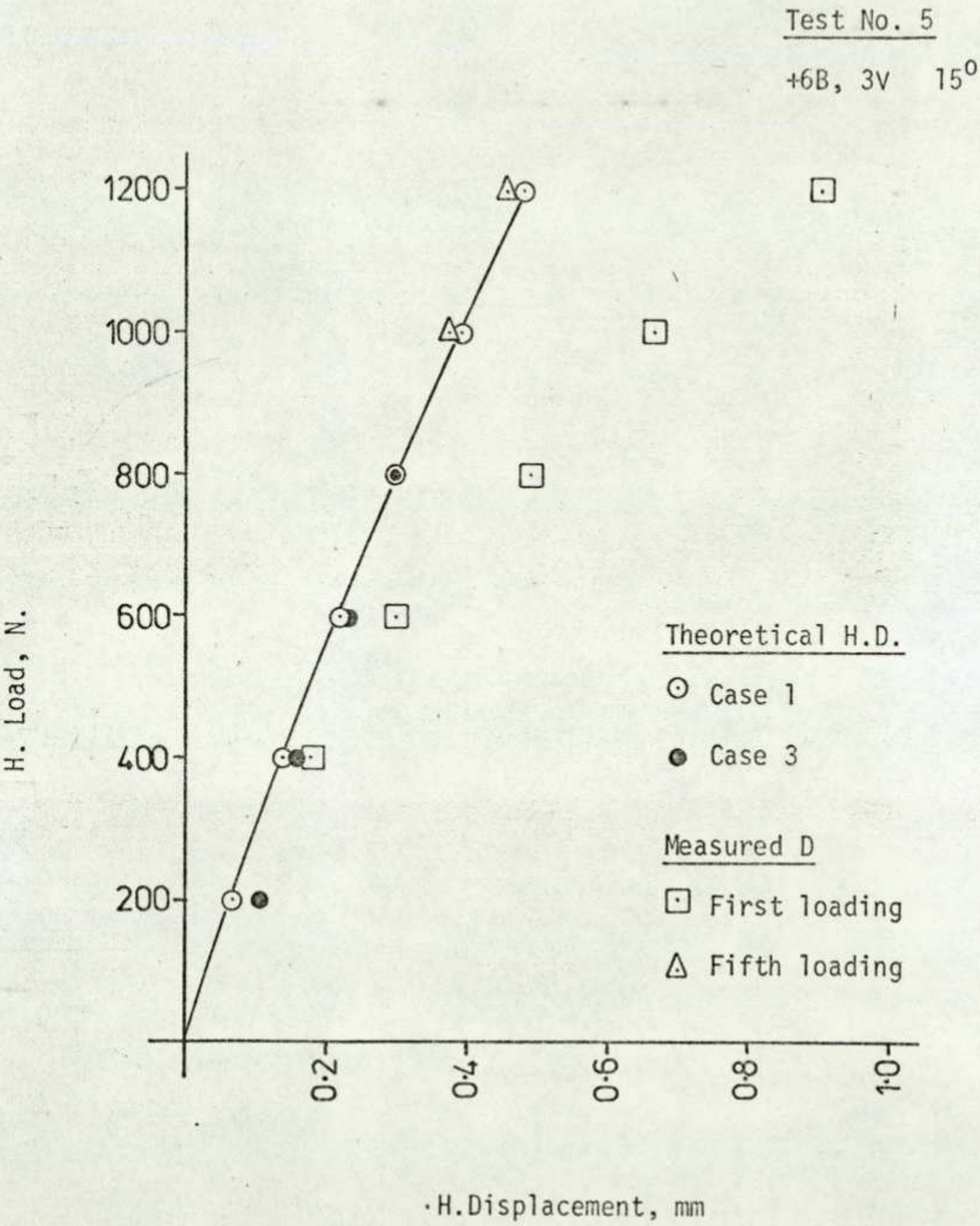


Fig. 8.37 H. load Vs Theoretical and experimental displacement

Nine-Pile Group (3x3)

H. Load, 600N

Test No. 5

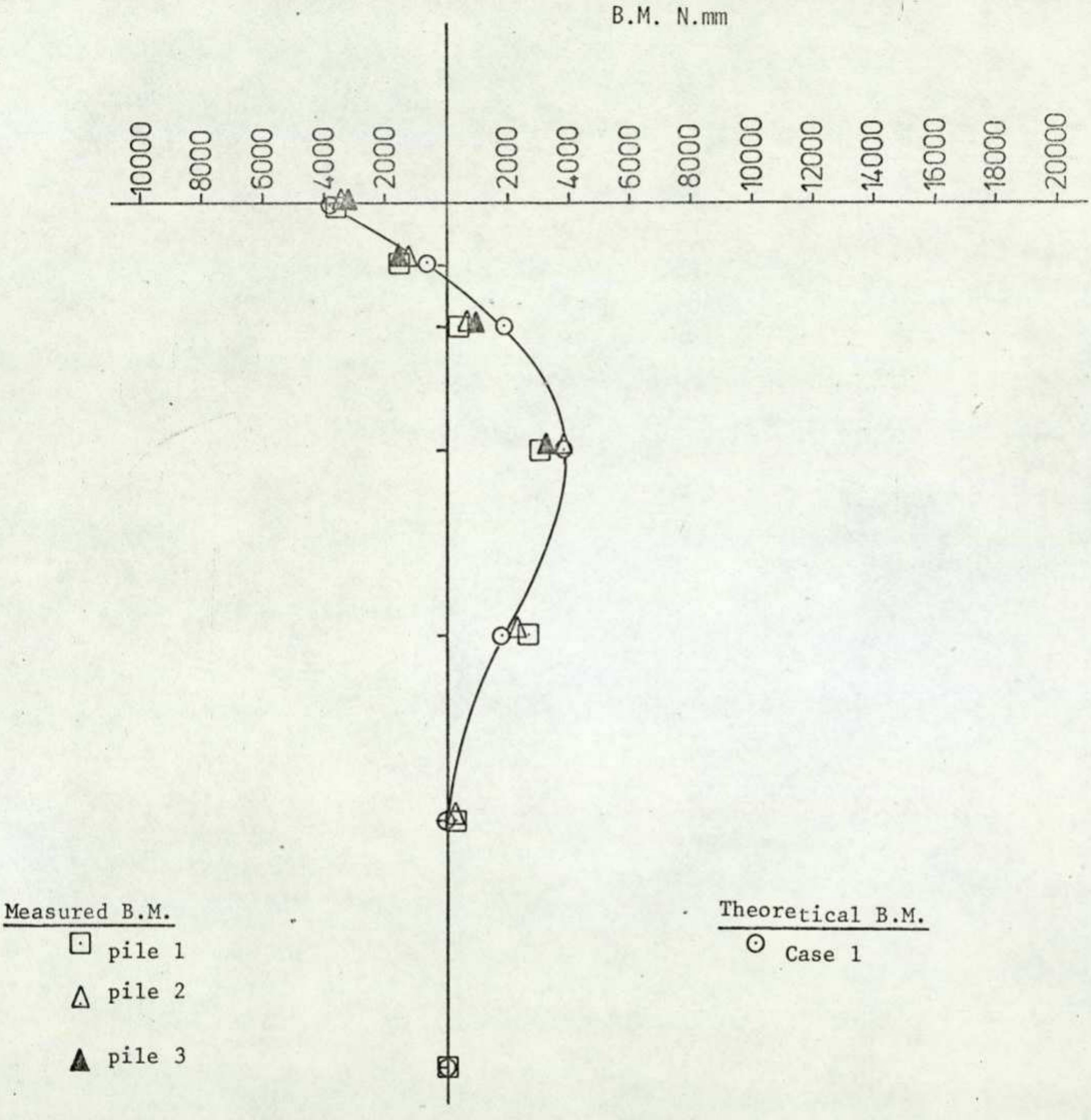


Fig. 8.38 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

H. Load, 600N

Test No. 5

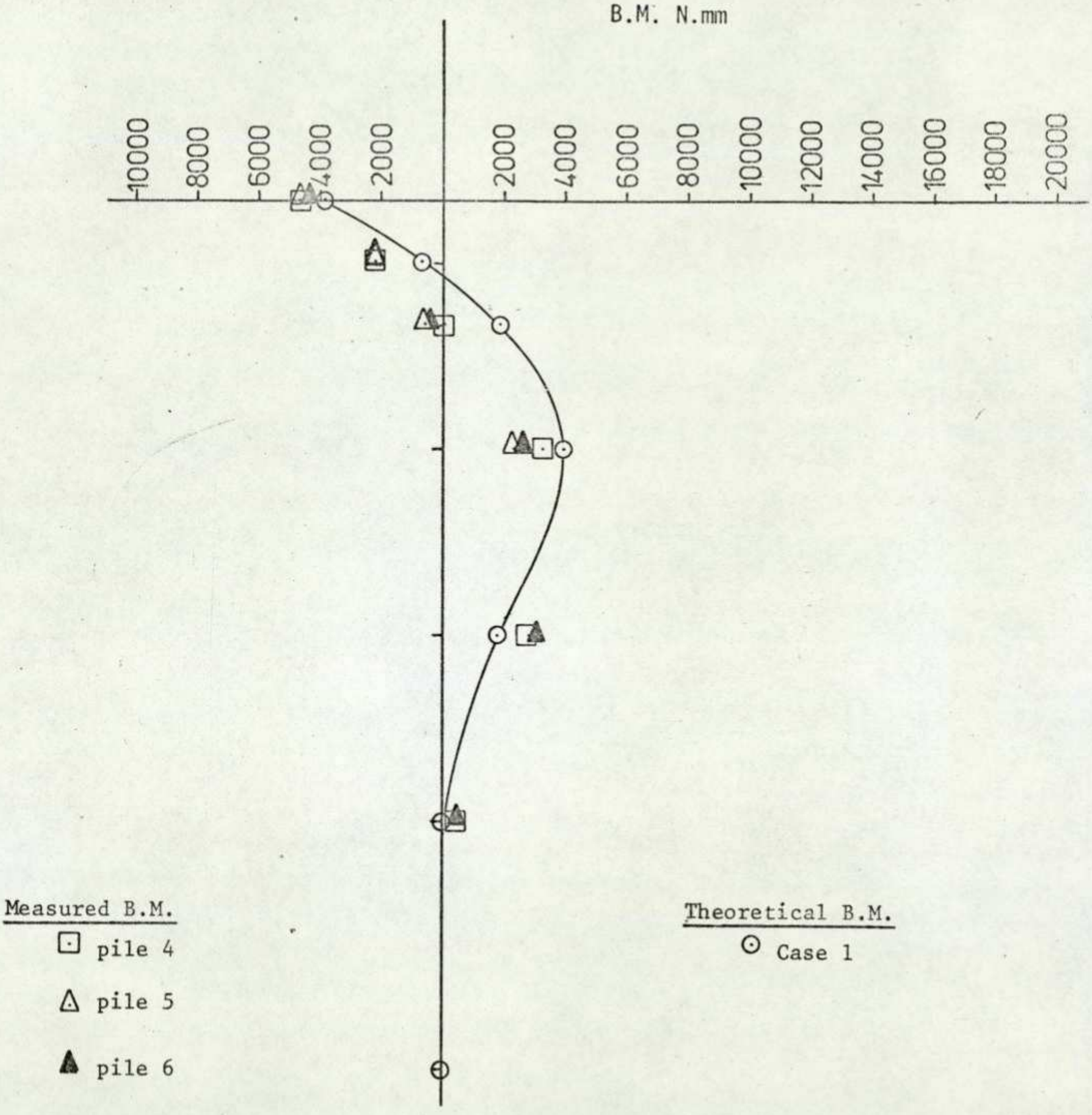


Fig. 8.39 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

H. Load, 600N

Test No. 5

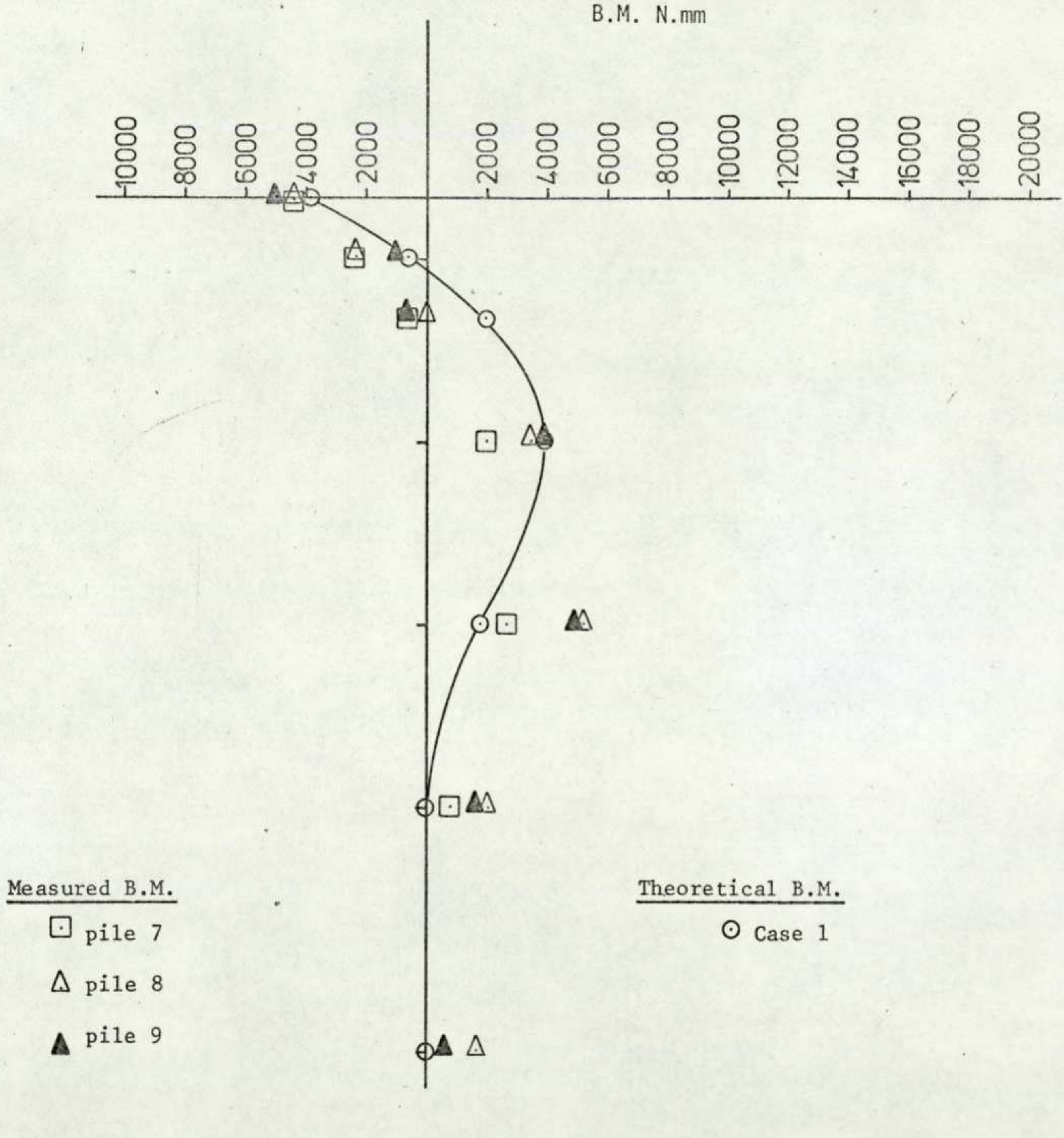


Fig. 8.40 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3 x 3)

Test No. 7

+6B, -3B 15^0

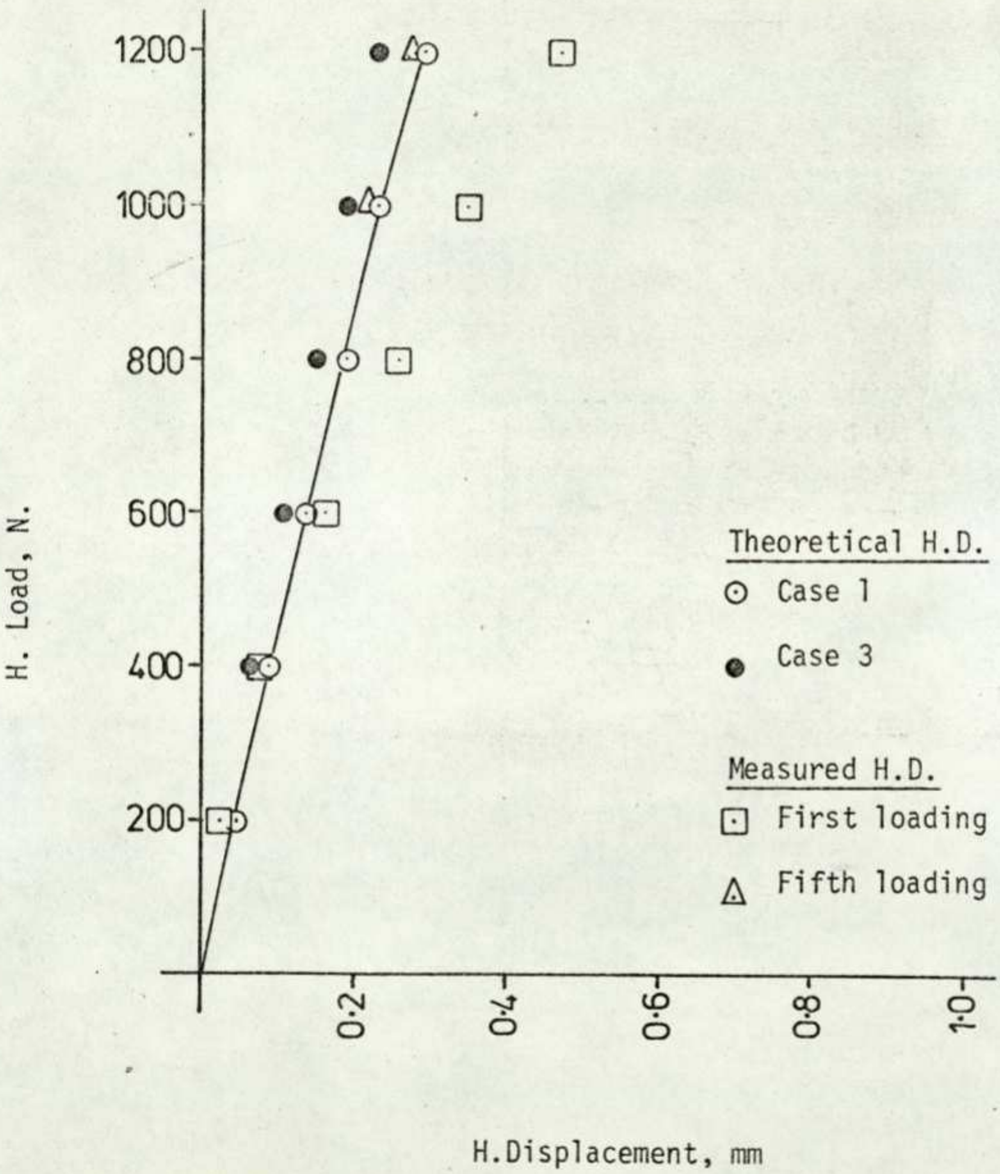


Fig. 8.41 H. load Vs Theoretical and experimental displacement

Nine-Pile Group (3x3)

H. Load,1000N

Test No. 7

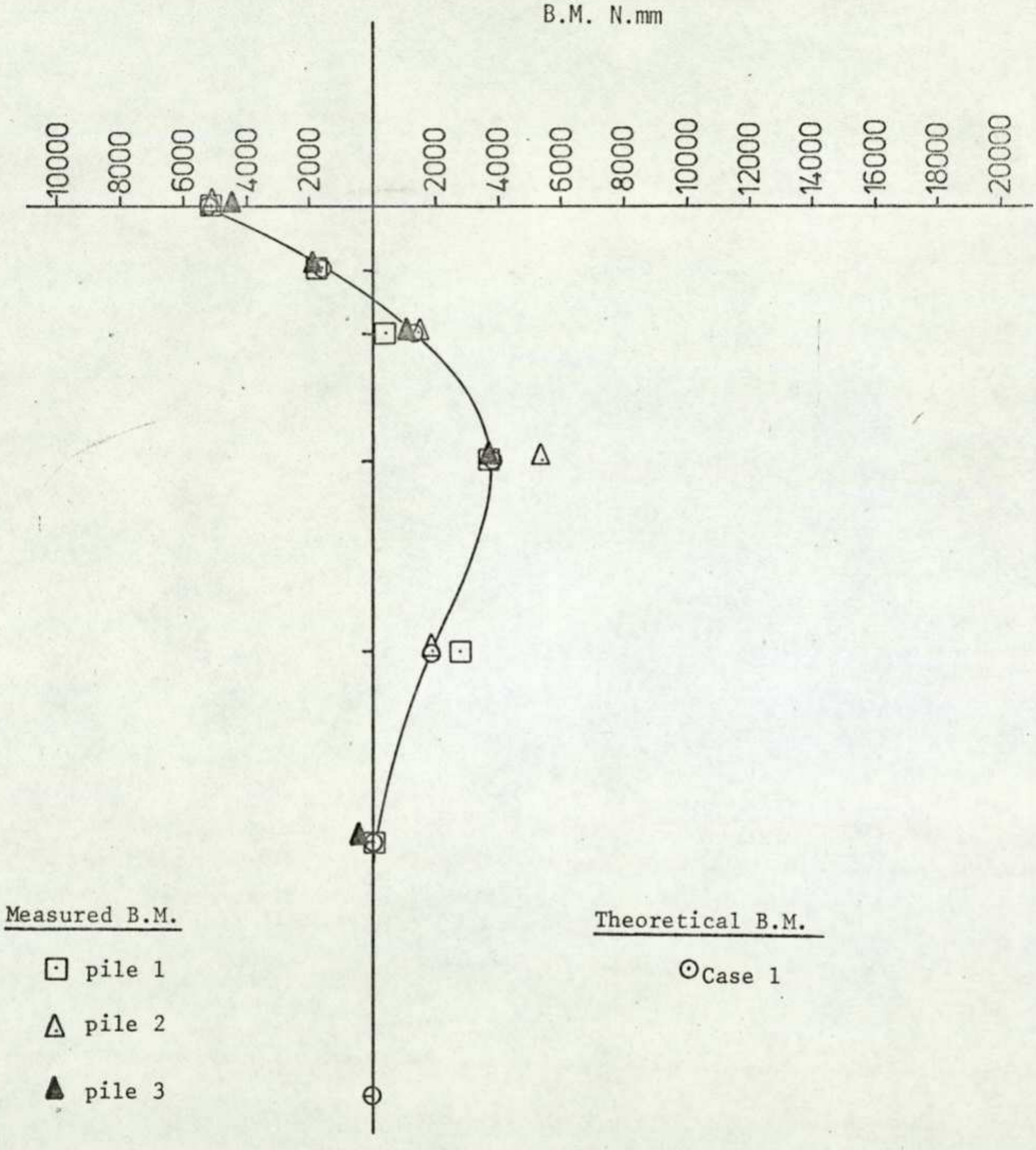


Fig. 8.42 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

H. Load,1000N

Test No. 7

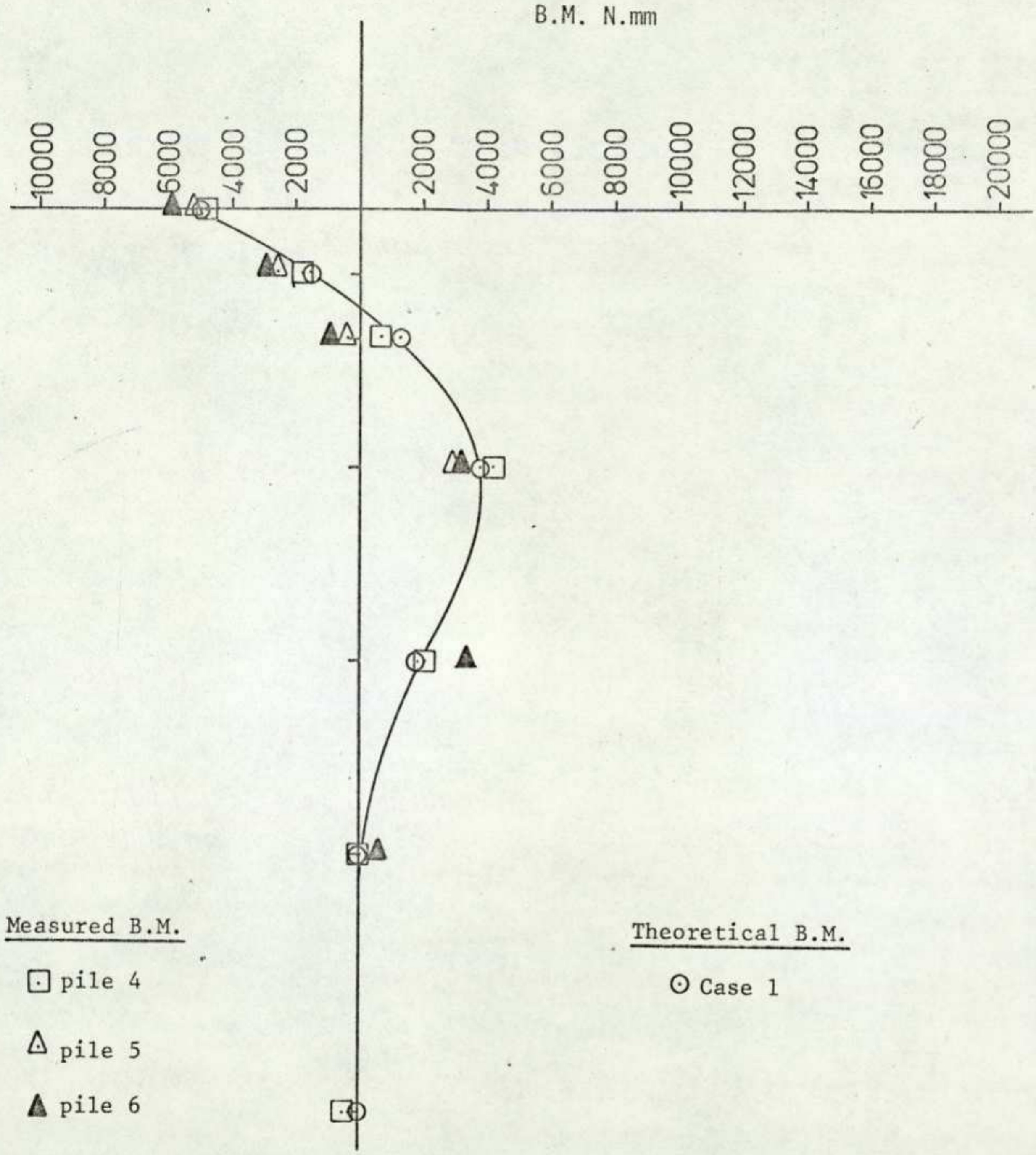


Fig. 8.43 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

H. Load, 1000N

Test No. 7

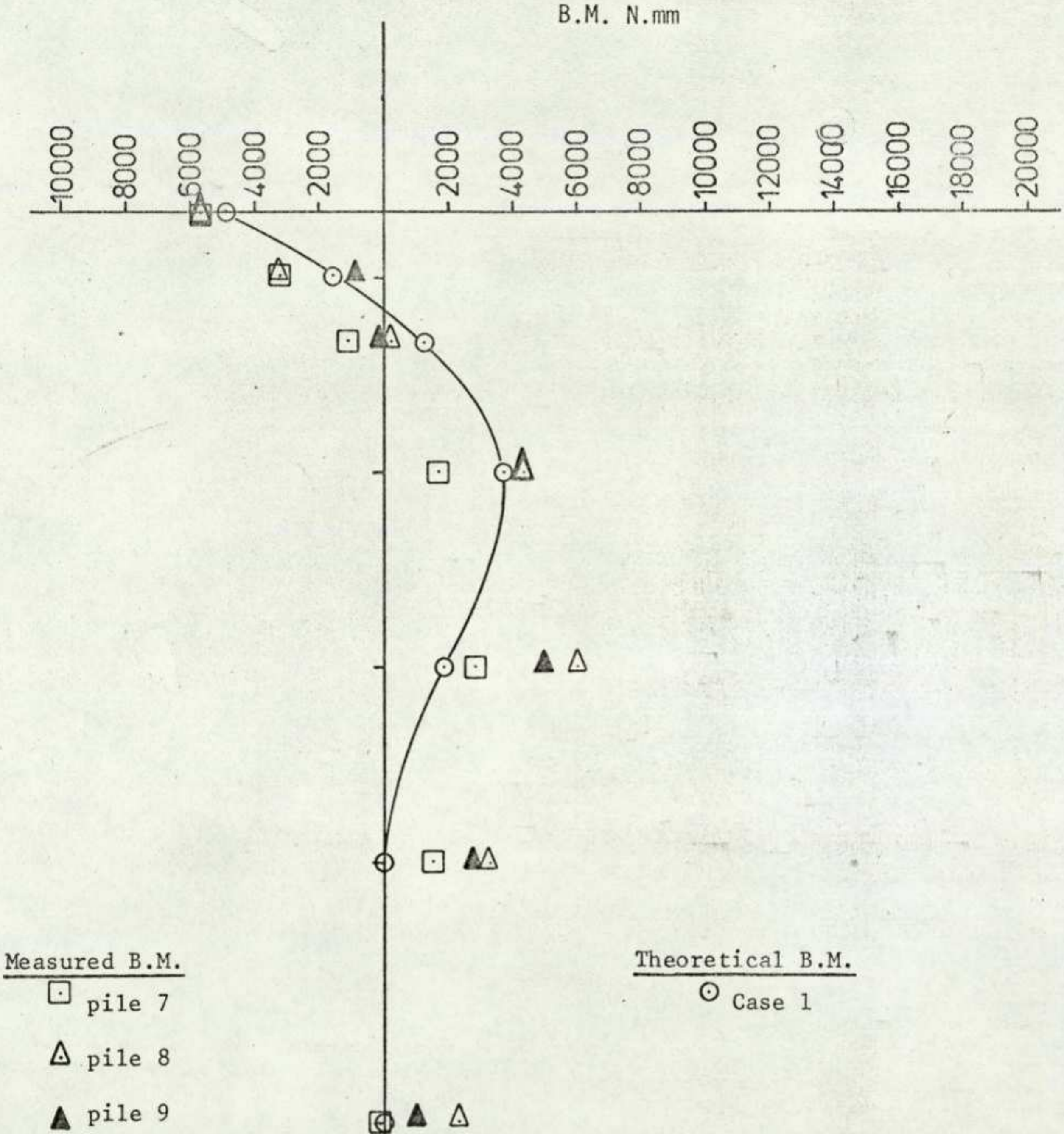


Fig. 8.44 Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

Test No. 10

+3B, 3V, -3B, 30^0

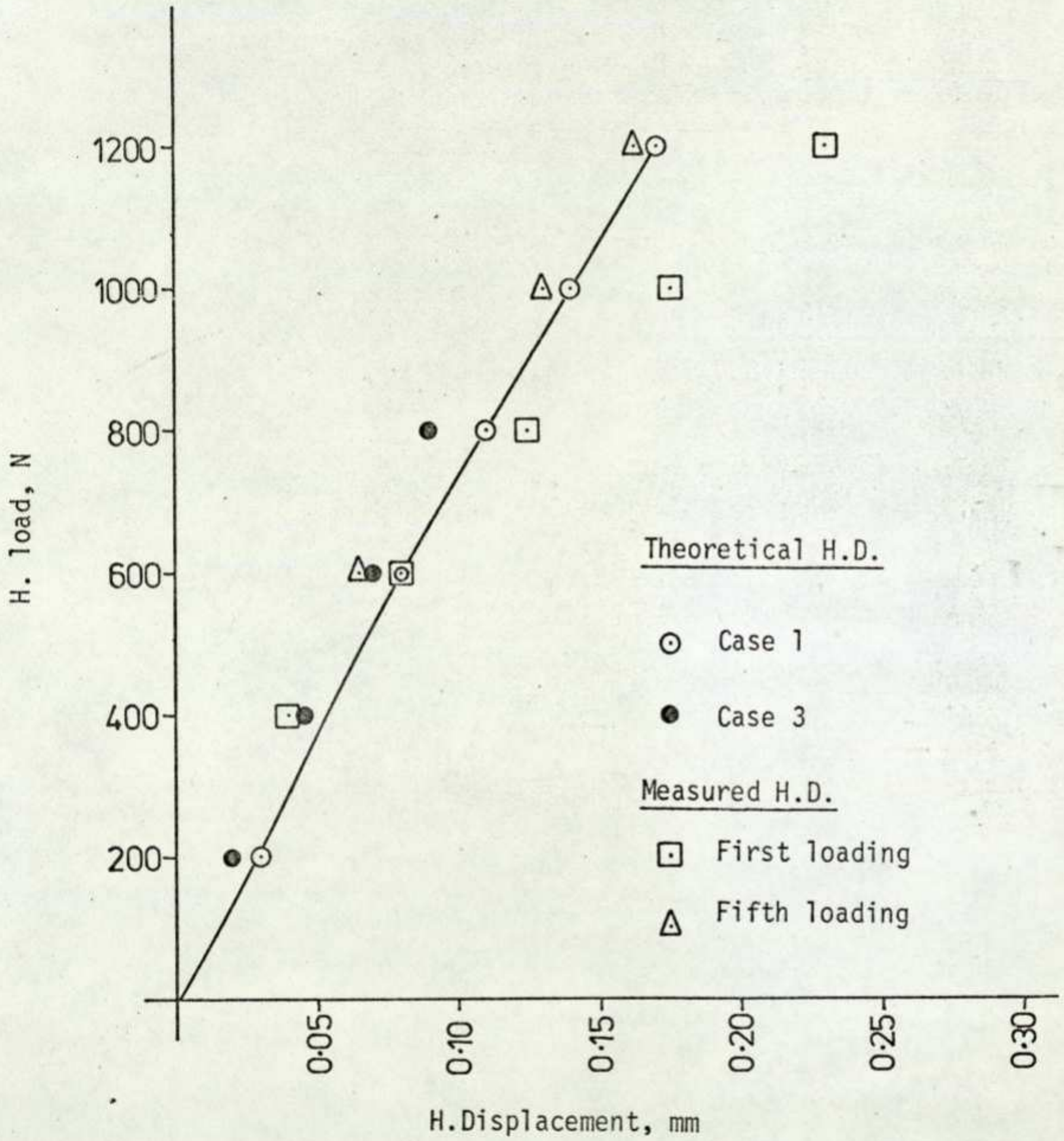


Fig. 8.45 H. Load Vs Theoretical and experimental displacement

Nine-pile Group (3 x 3)

Test No. 10

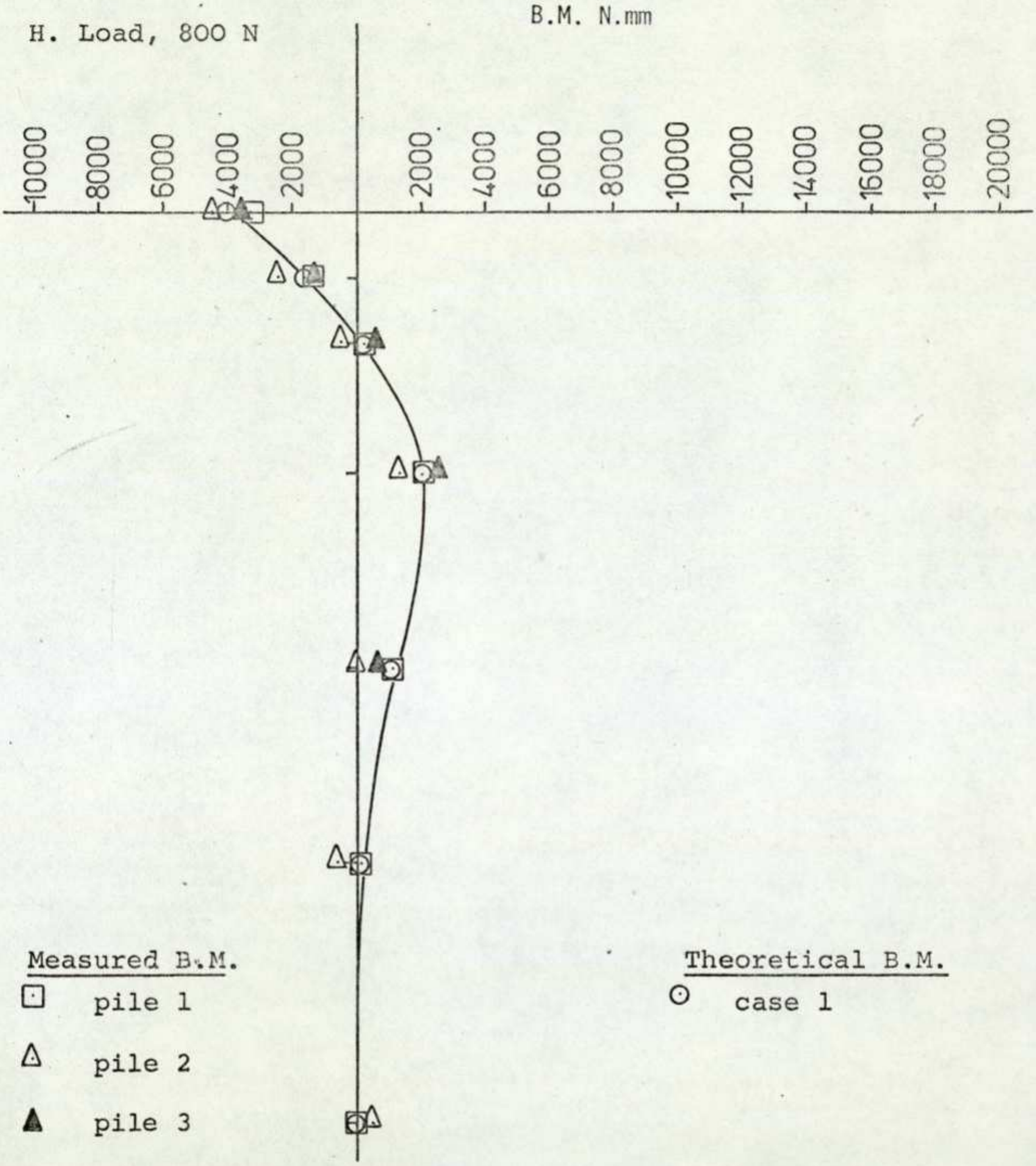


Fig. 8.46. Theoretical and experimental bending moment Vs Depth

Nine-pile Group (3 x 3)

Test No. 10

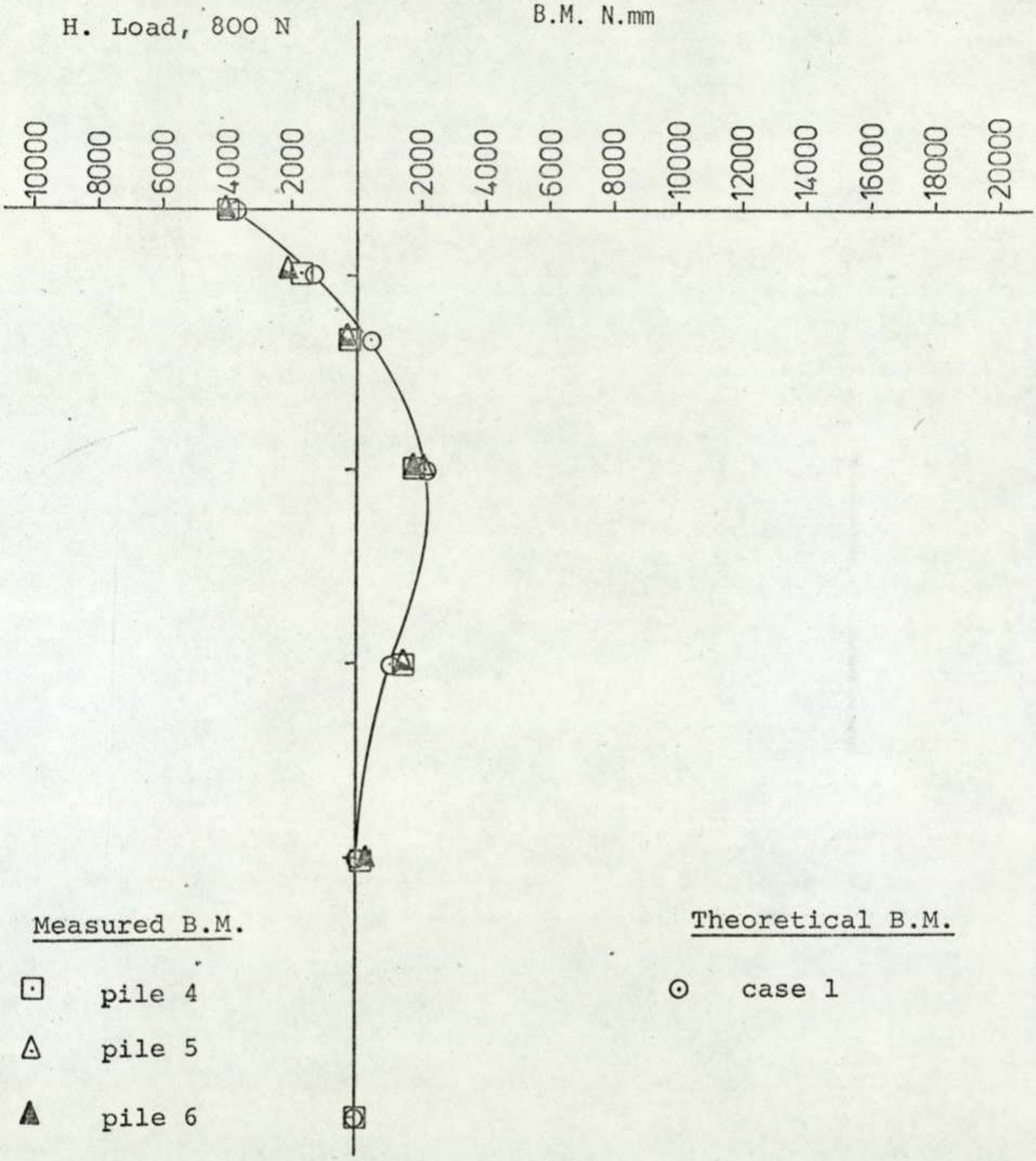


Fig. 8.47. Theoretical and experimental bending moment Vs Depth

Nine-pile Group (3 x 3)

H. Load, 800N

Test No. 10

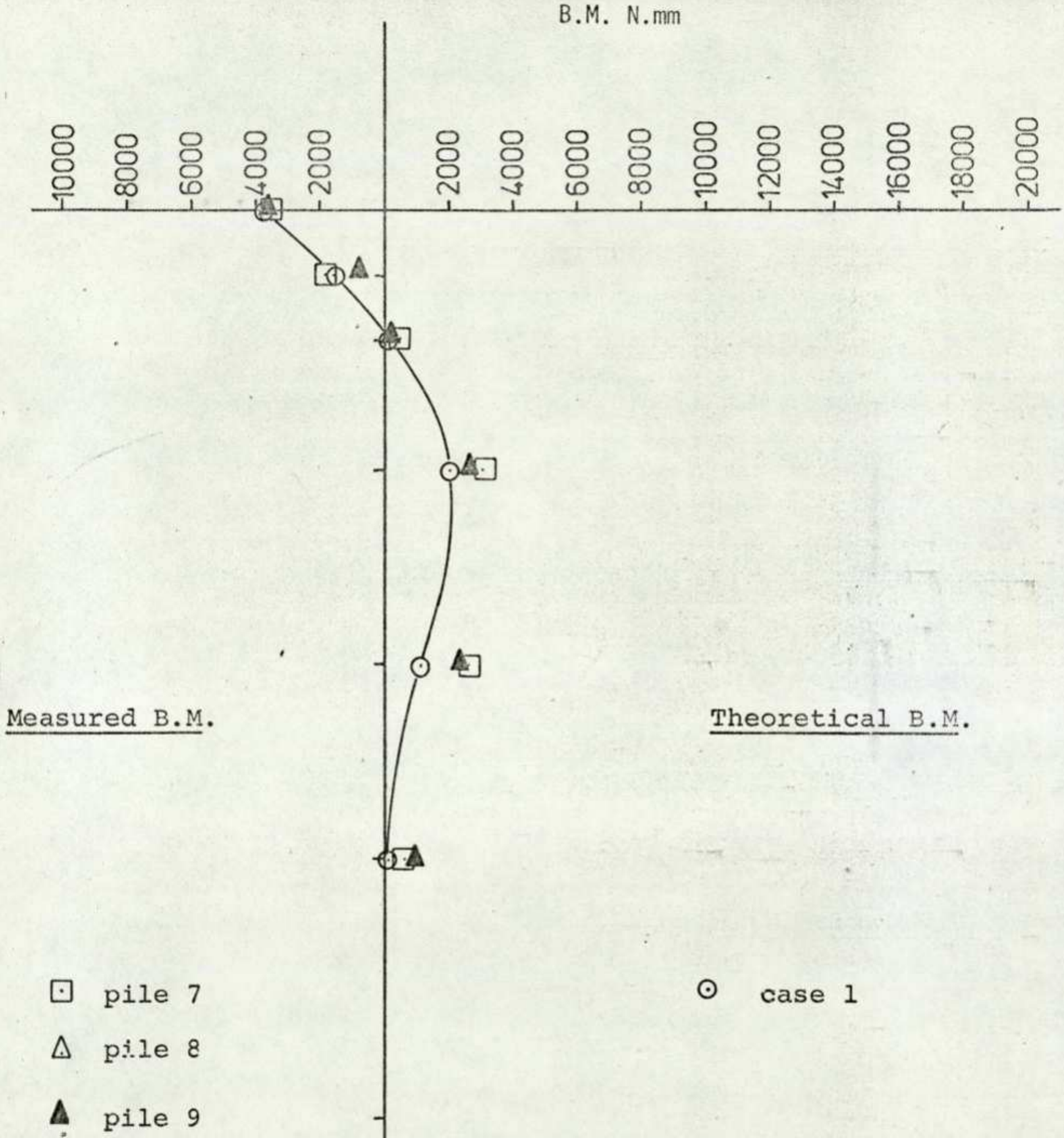


Fig. 8.48. Theoretical and experimentalbending moment Vs Depth

Nine-Pile Group (3 x 3)

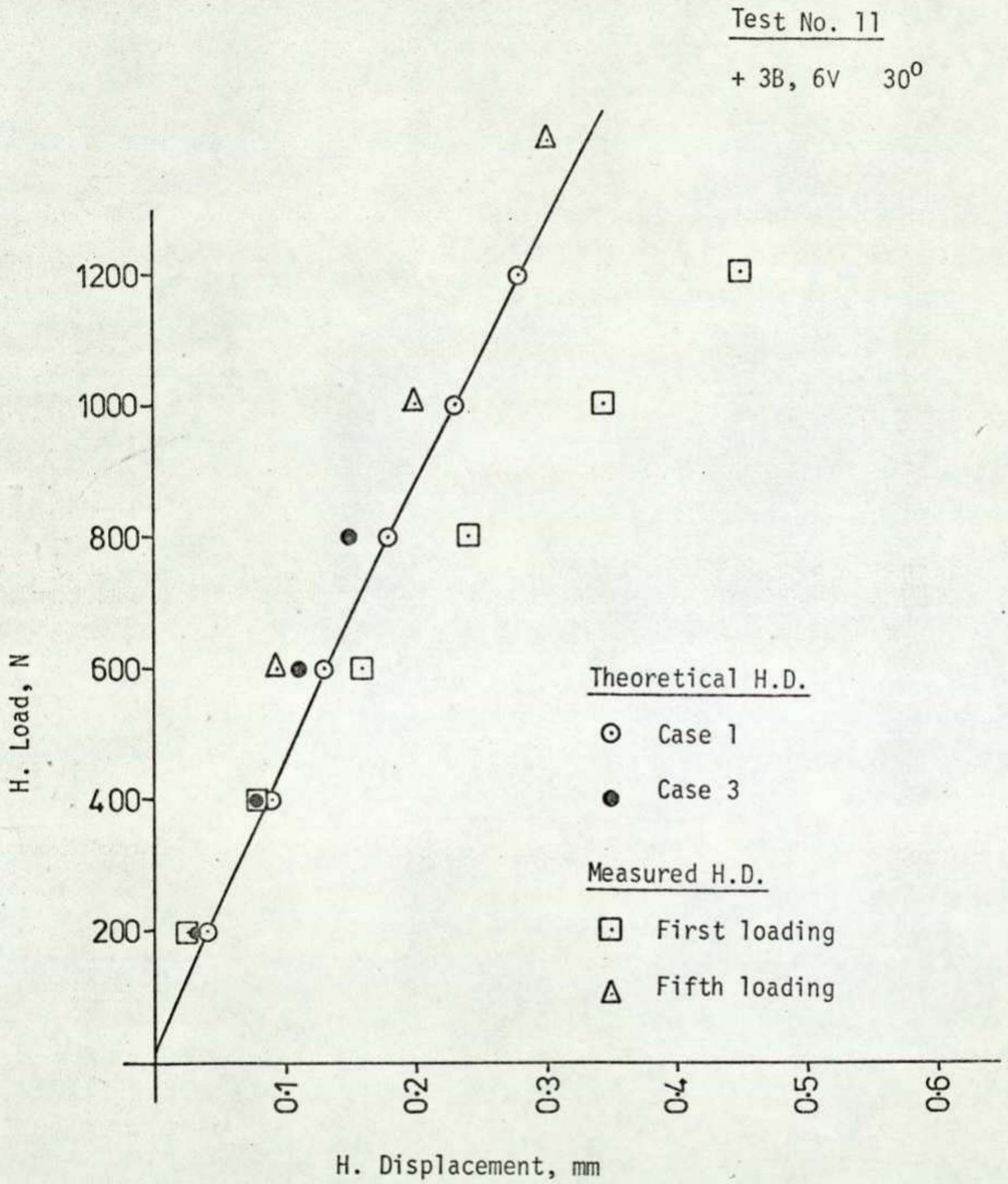


Fig. 8.49 H. Load Vs Theoretical and experimental displacement

Nine-pile Group (3 x 3)

Test No. 11

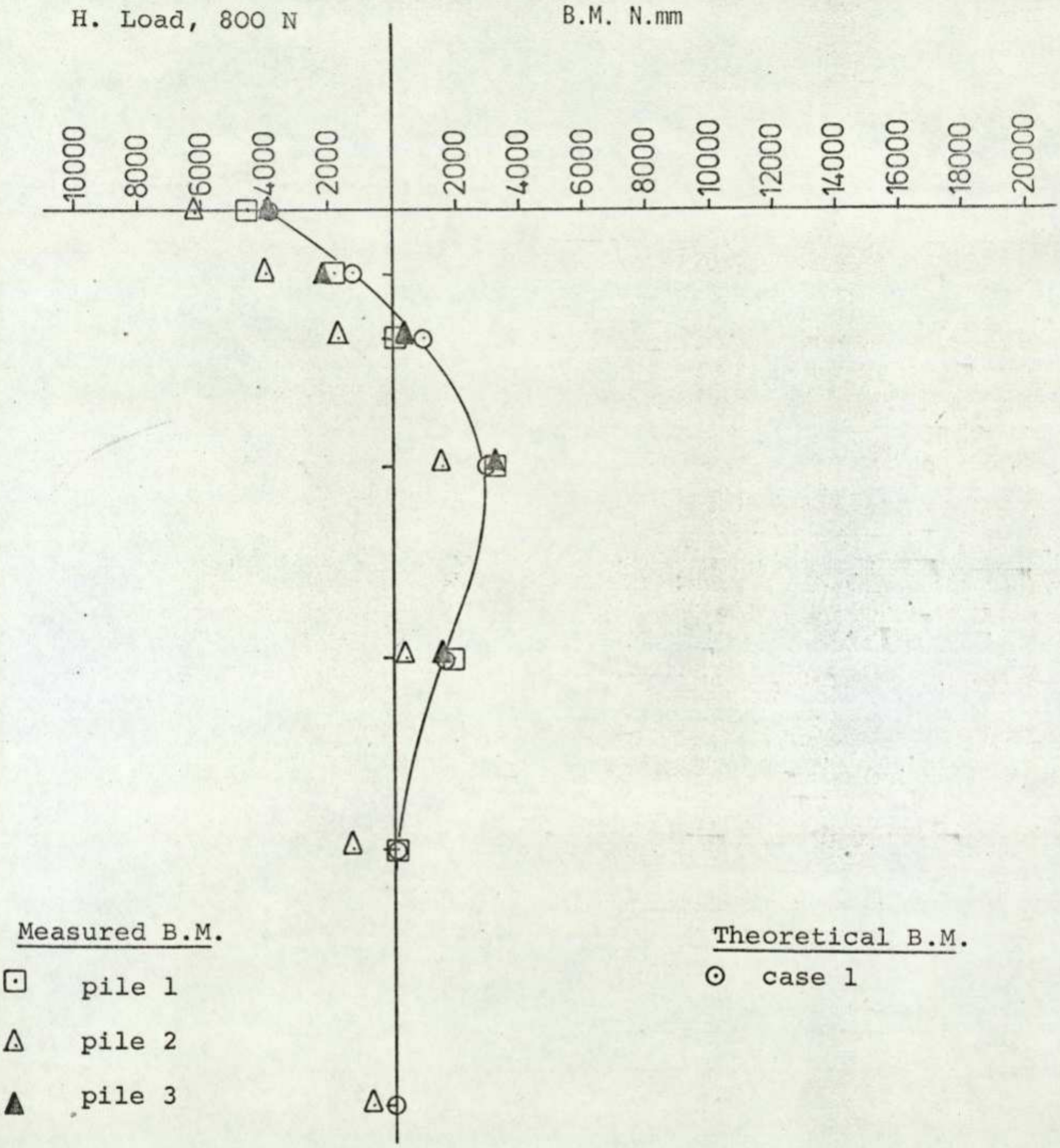


Fig. 8.50. Theoretical and experimental bending moment Vs Depth

Nine-pile Group (3 x 3)

Test No. 11

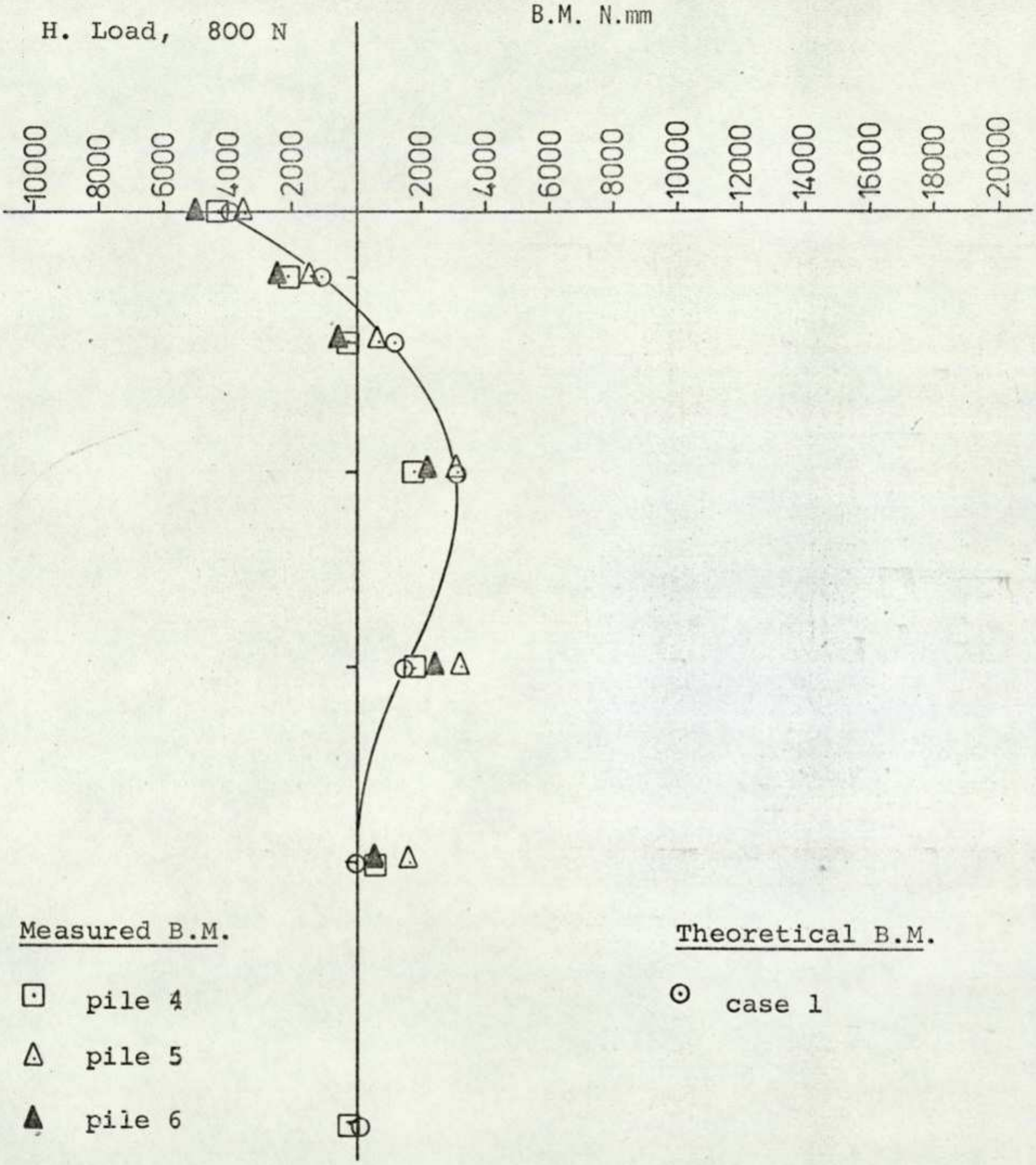


Fig. 8.51. Theoretical and experimentalbending moment Vs Depth

Nine-pile Group (3 x 3)

Test No. 11

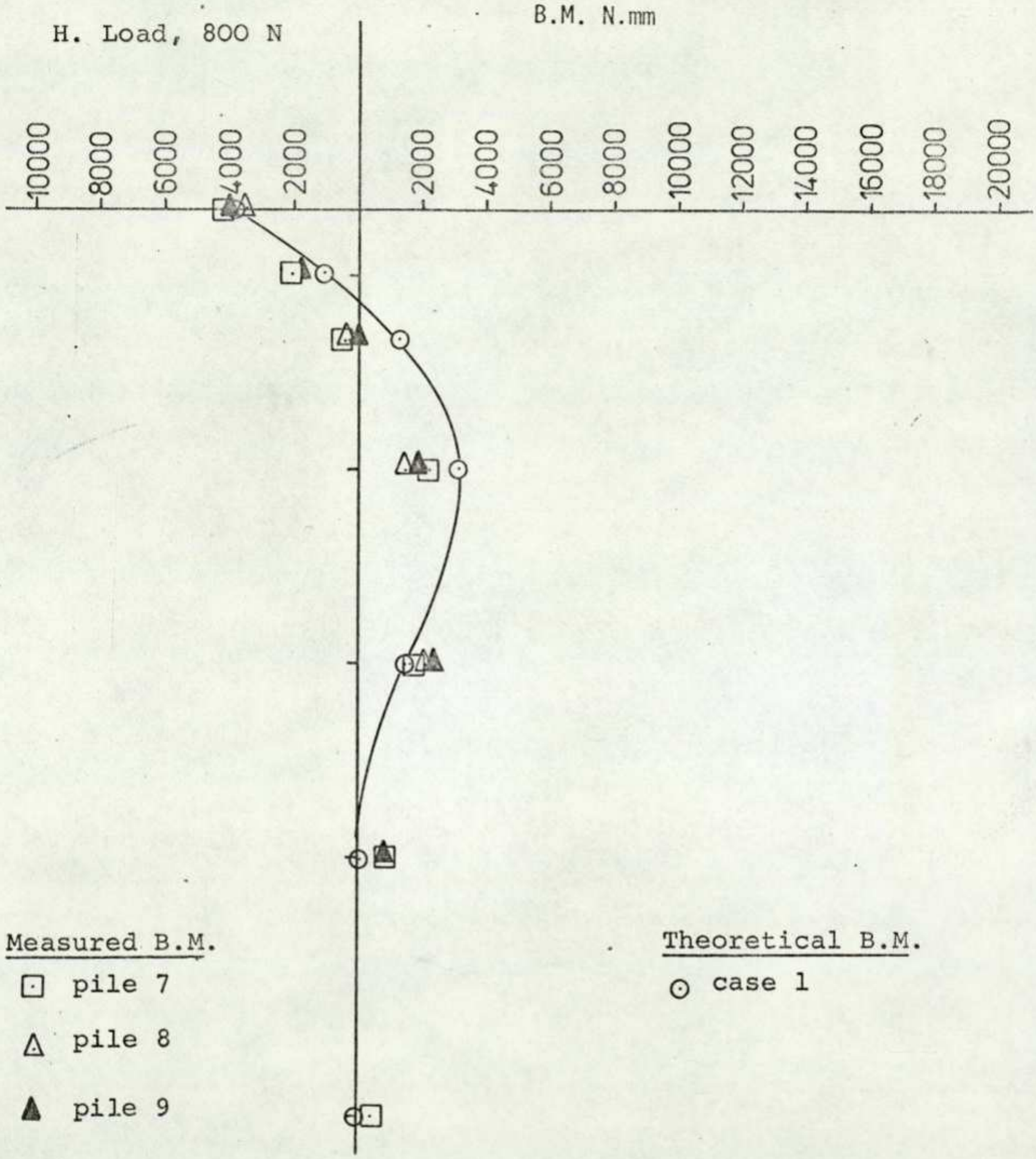
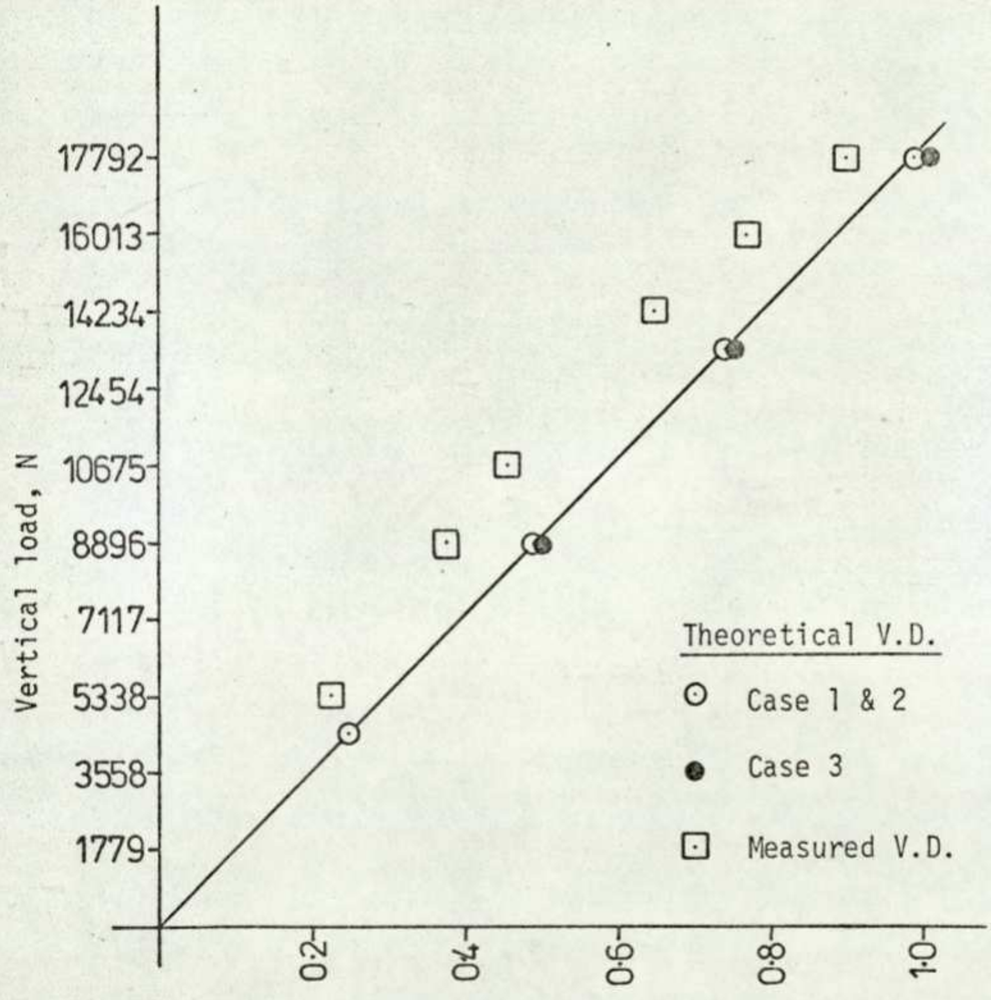


Fig. 8.52. Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

Test No. 15

9 V



Vertical displacement, mm

Fig. 8.53 Vertical load Vs Theoretical and experimental vertical displacement

Nine-Pile Group (3x3)

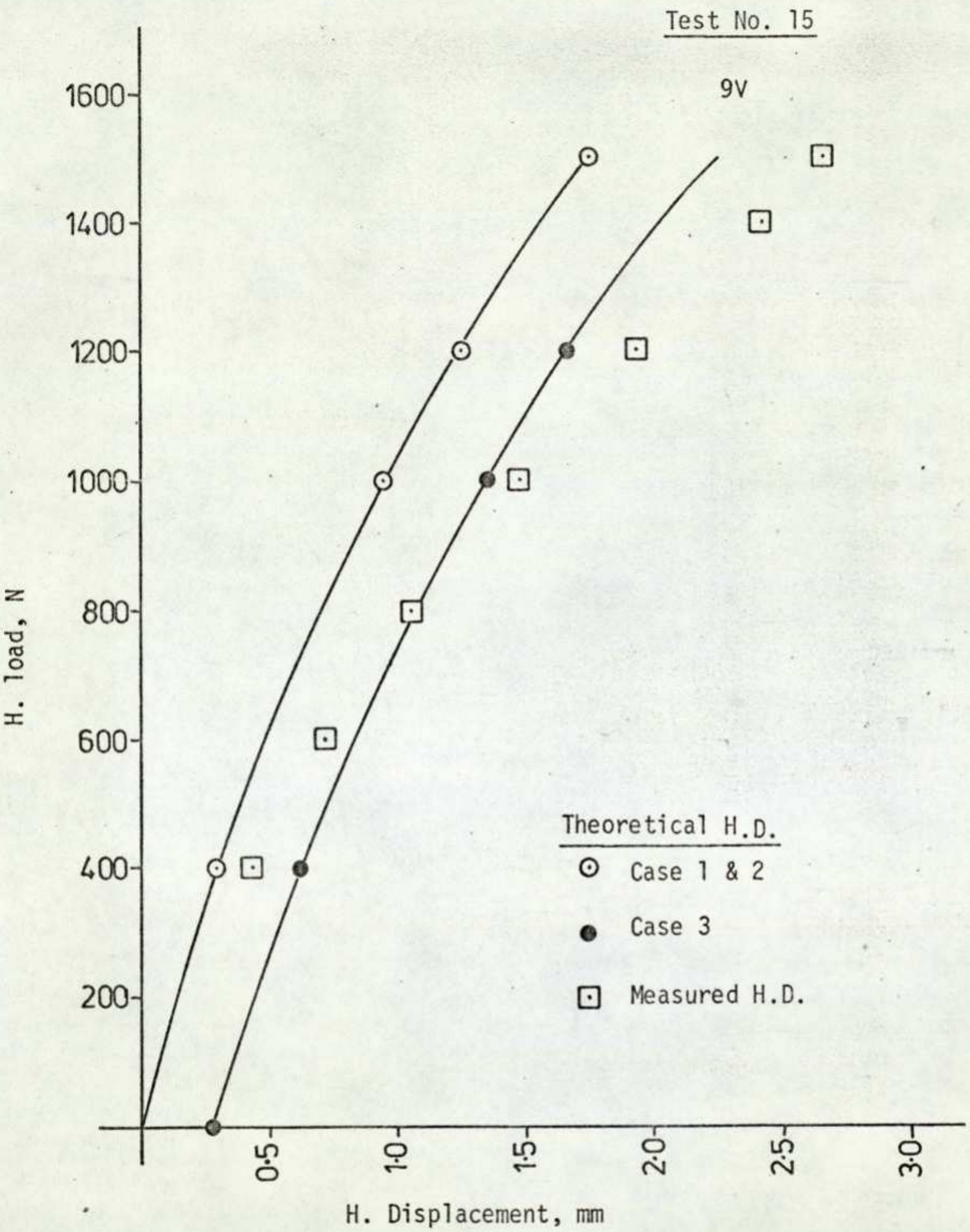


Fig. 8.54 H. load Vs Theoretical and experimental H.displacement

Nine - pile Group (3 x 3)

Test No. 15

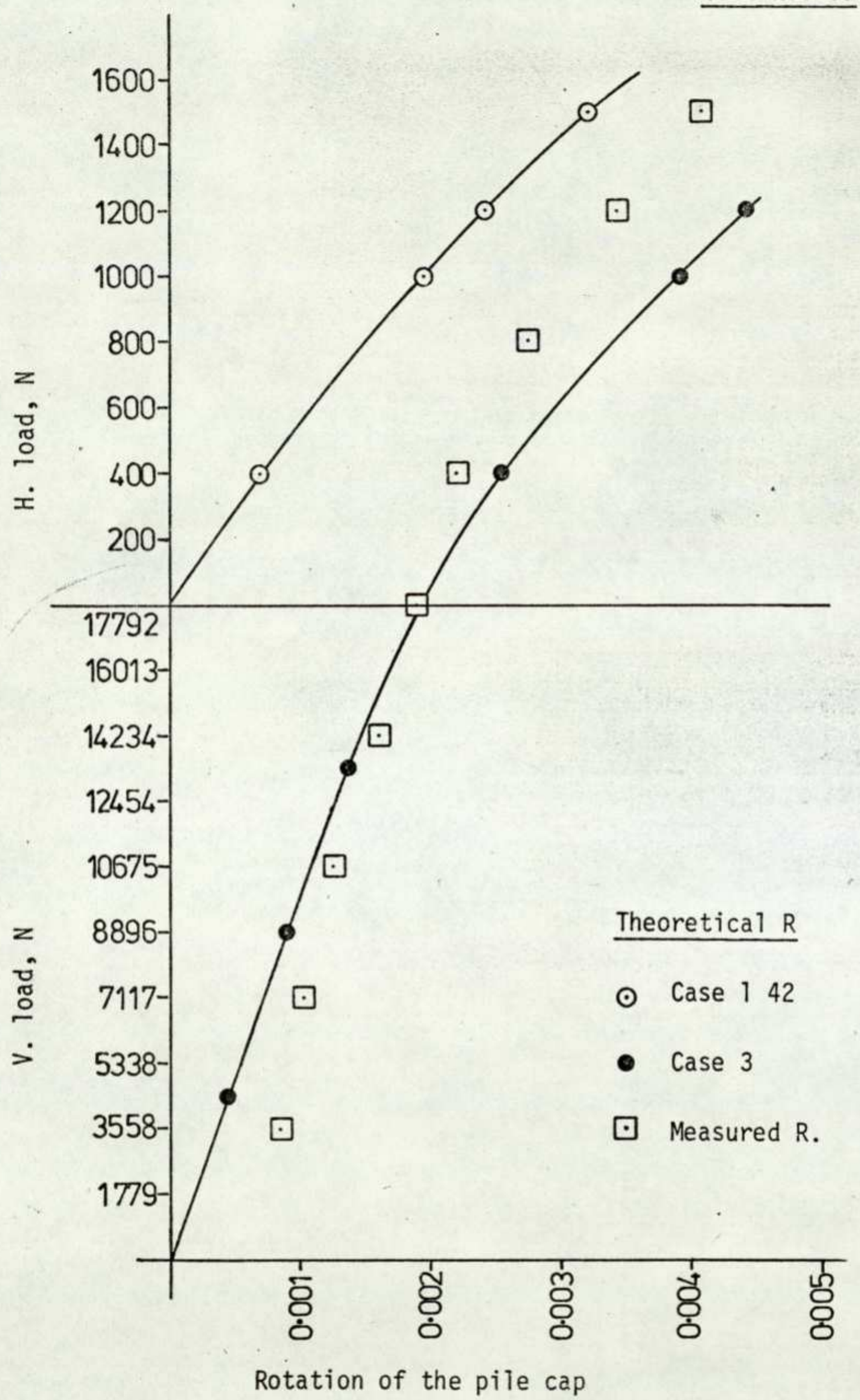


Fig. 8.55 H. & V. loads Vs Theoretical
and experimental rotation of the pile cap

Nine-pile Group (3 x 3)

Test No. 15

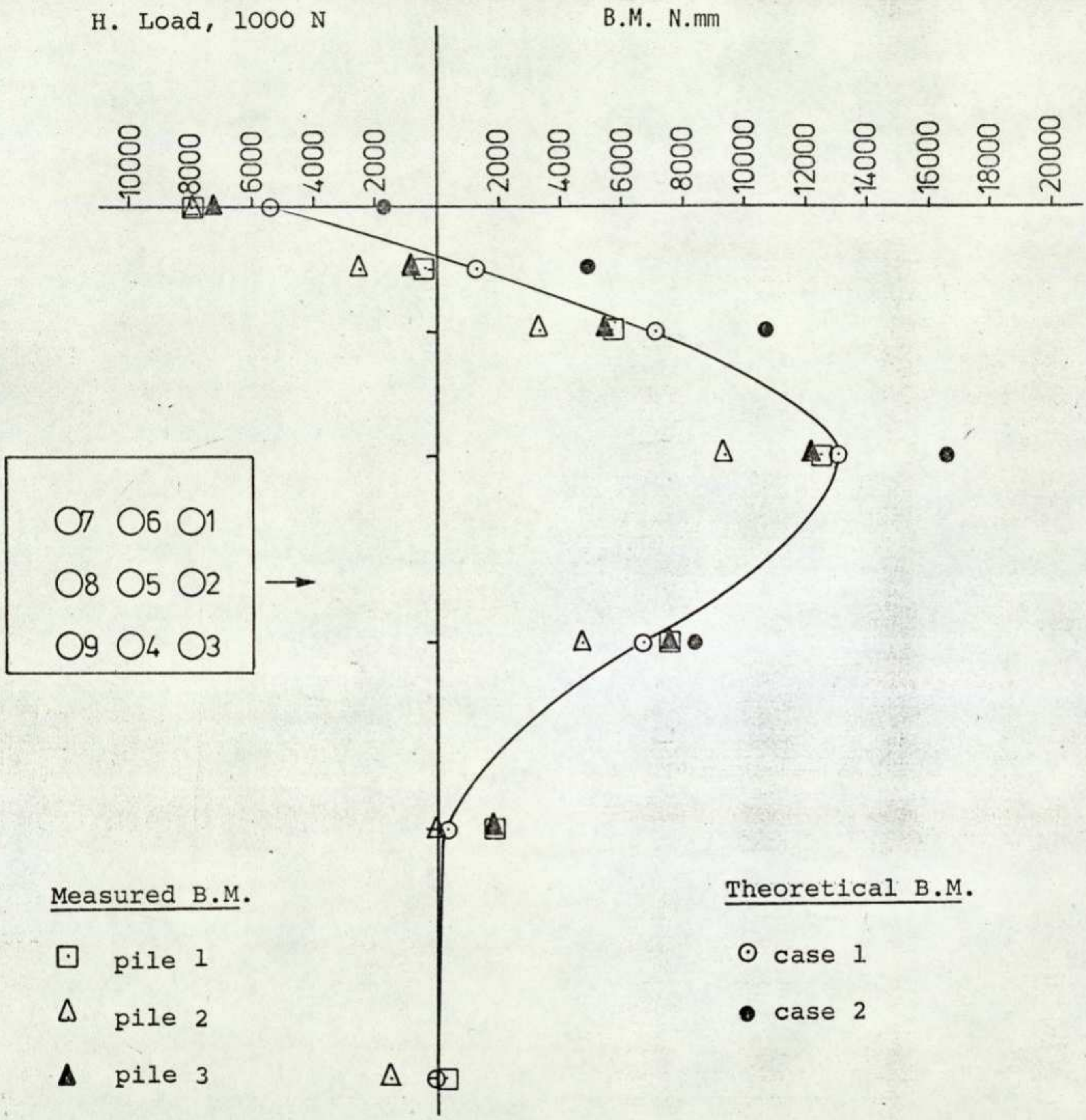


Fig. 8.56. Theoretical and experimental bending moment Vs Depth

Nine-pile Group (3 x 3)

Test No. 15

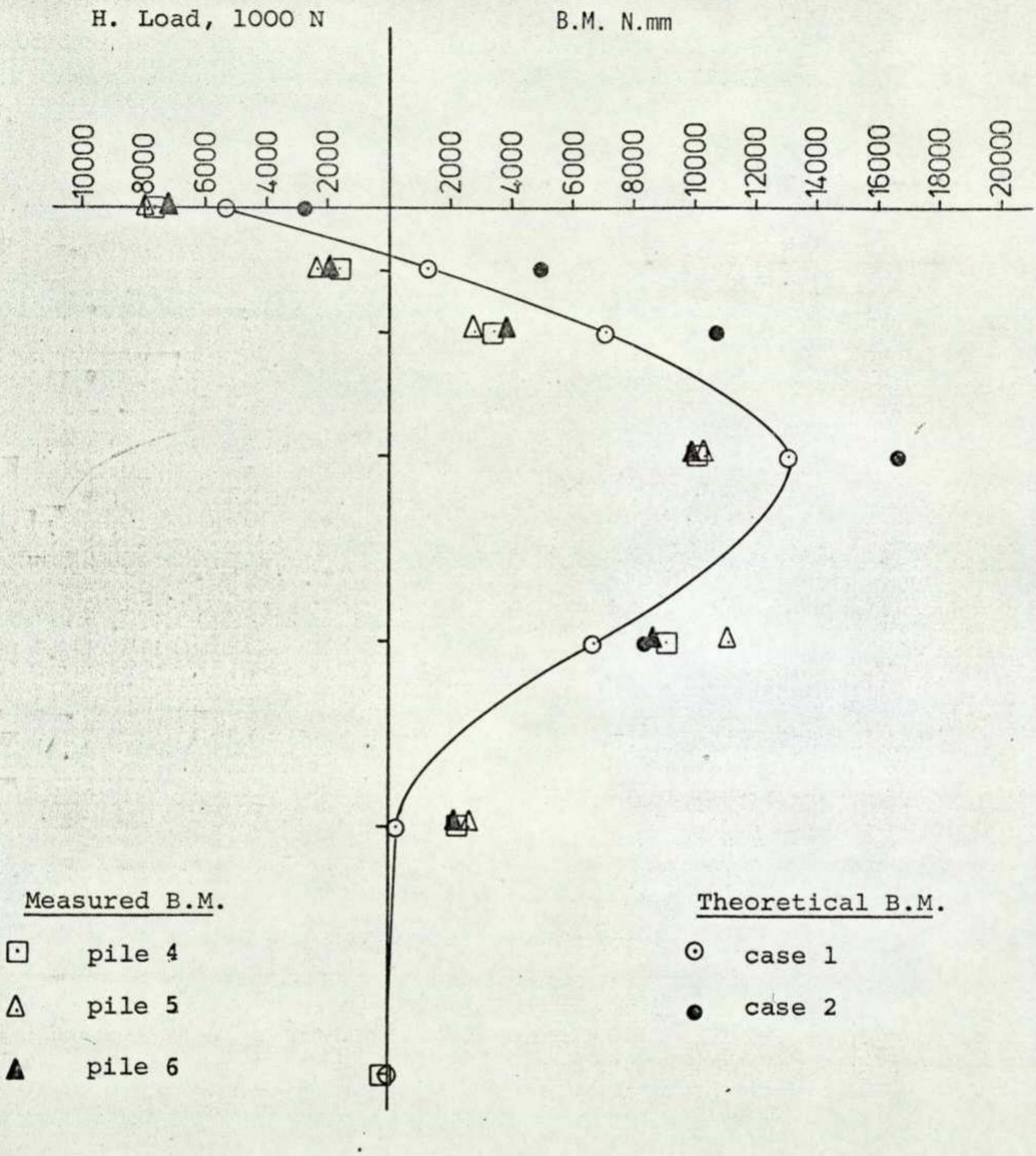


Fig. 8.57. Theoretical and experimental bending moment Vs Depth

Nine-pile Group (3 x 3)

Test No. 15

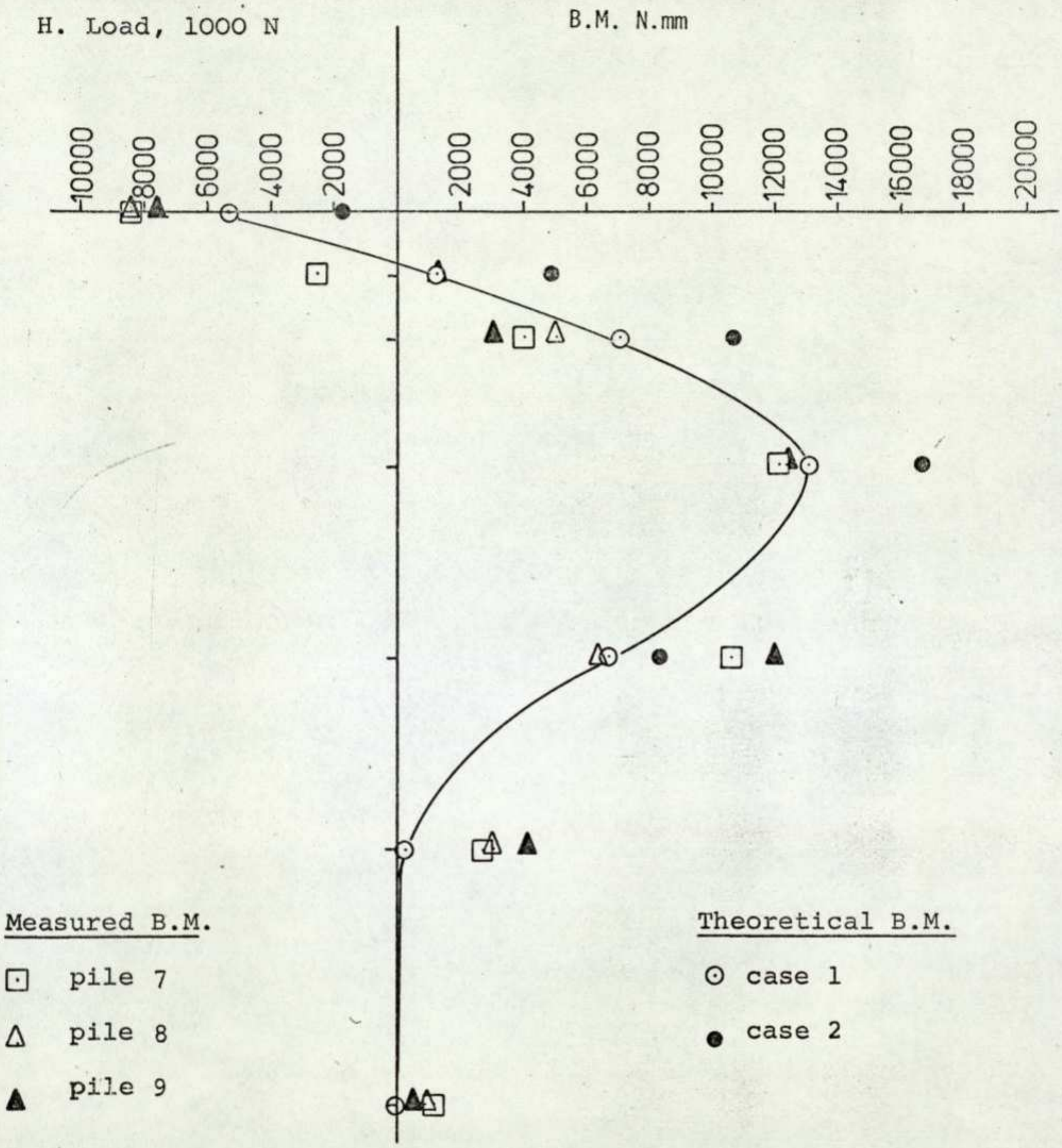
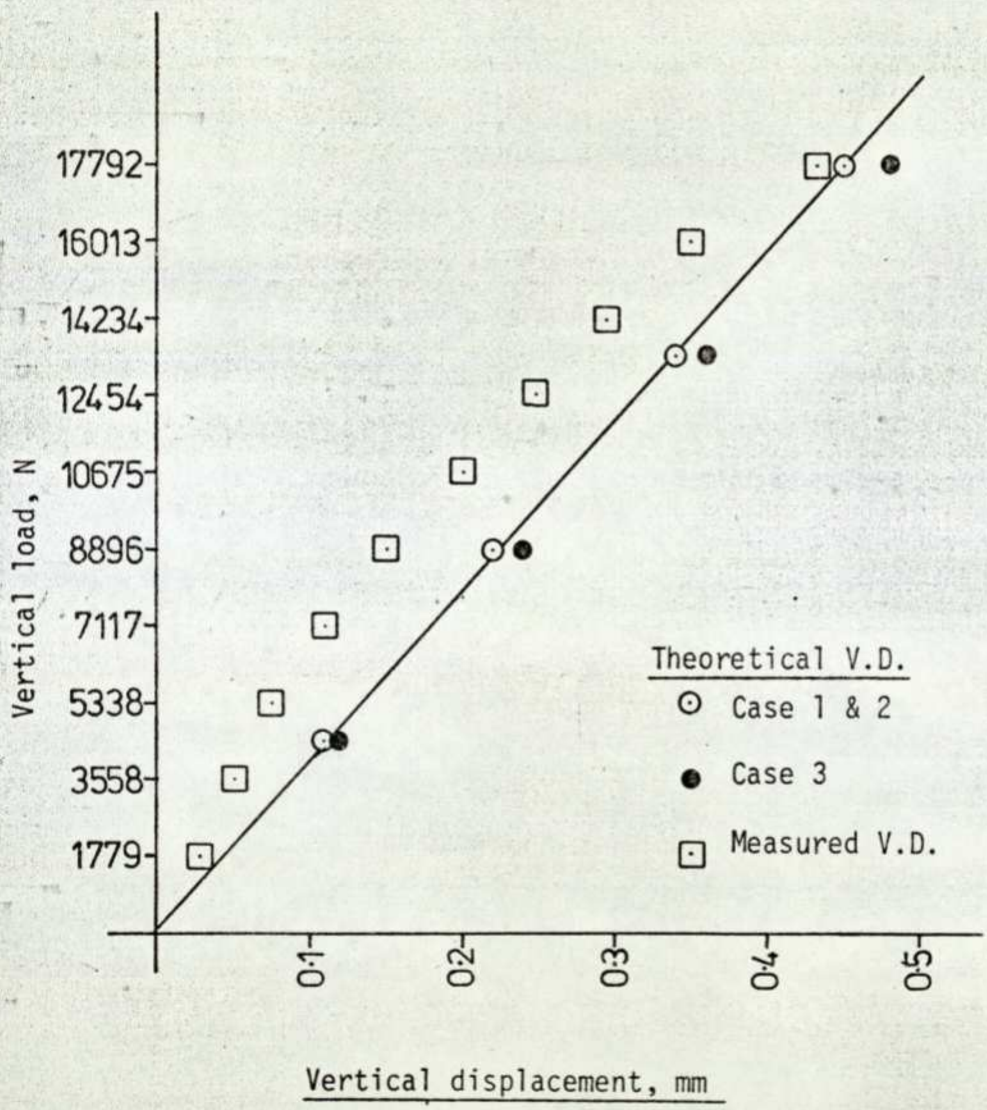


Fig. 8.58. Theoretical and experimental bending moment Vs Depth

Nine-Pile Group (3x3)

Test No. 16

+3B, 3 , -3B 15^0



Vertical displacement, mm

Fig. 8.59 Vertical Load Vs Theoretical and experimental vertical displacement

Nine-Pile Group (3 x 3)

Test No. 16

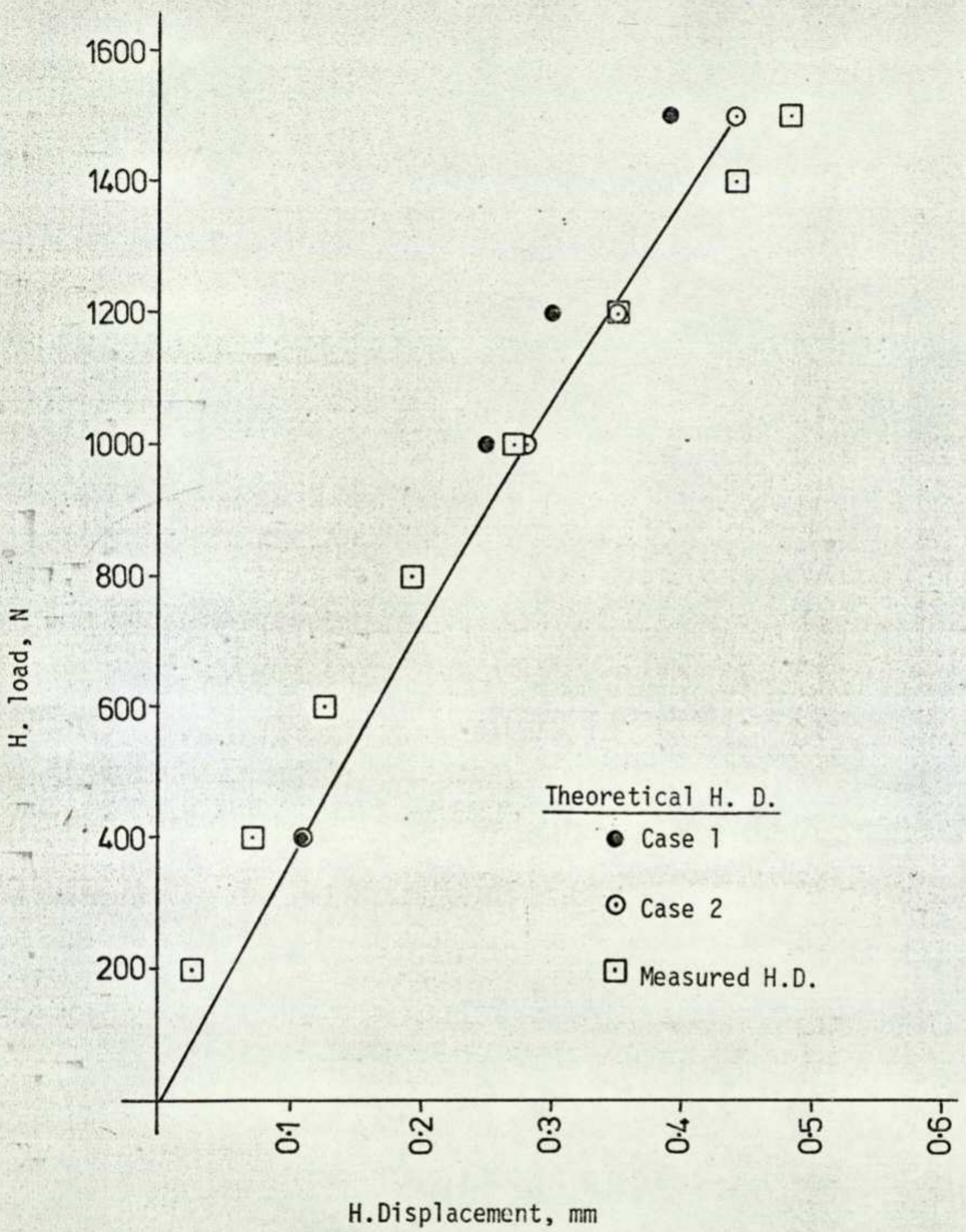


Fig. 8.60 H. load Vs Theoretical and experimental H.displacement

Nine-Pile Group (3x3)

Test No. 16

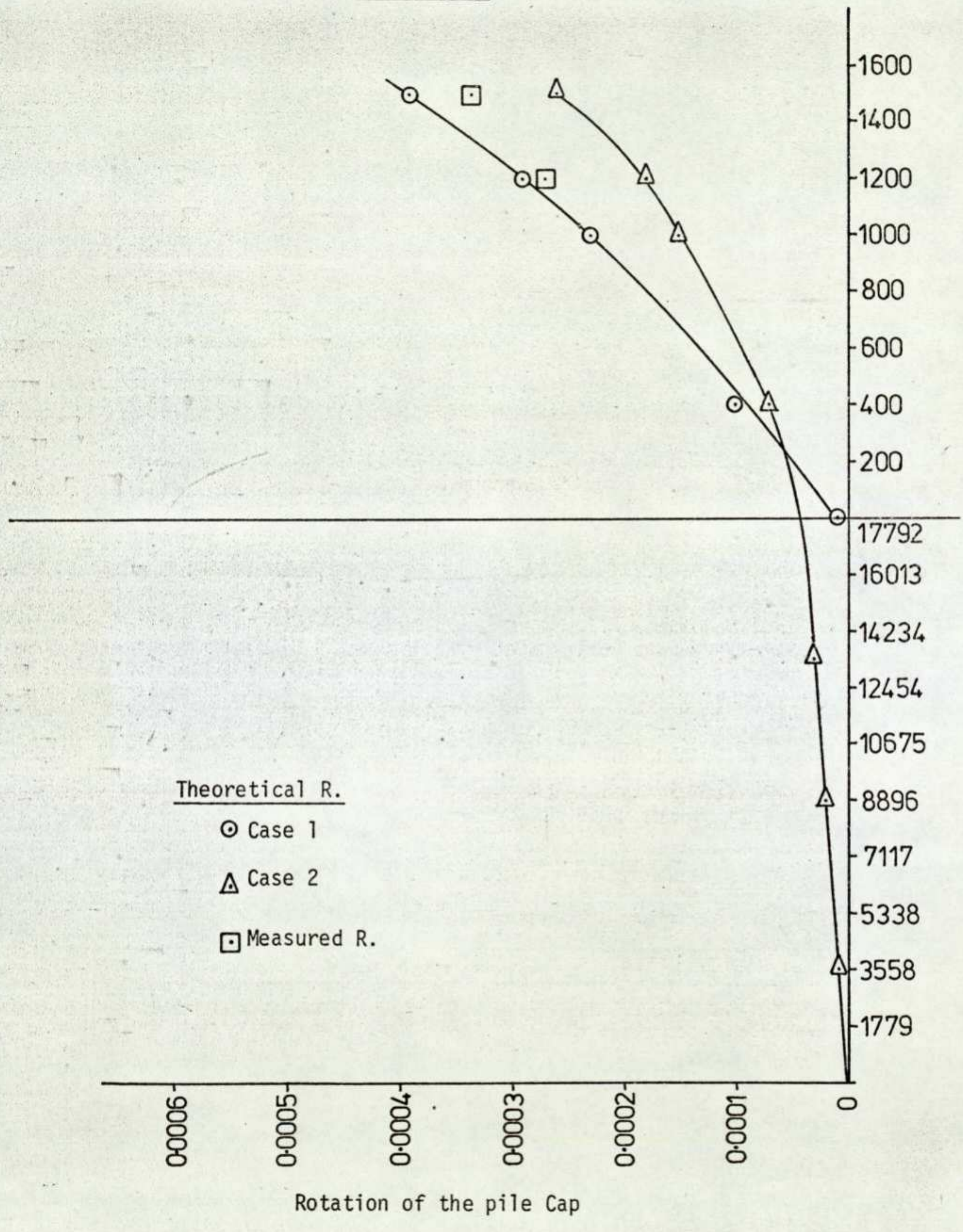


Fig. 8.61 V.&H. loads Vs Theoretical and experimental rotation of the pile cap

Nine - pile Group (3 x 3)

Test No. 16

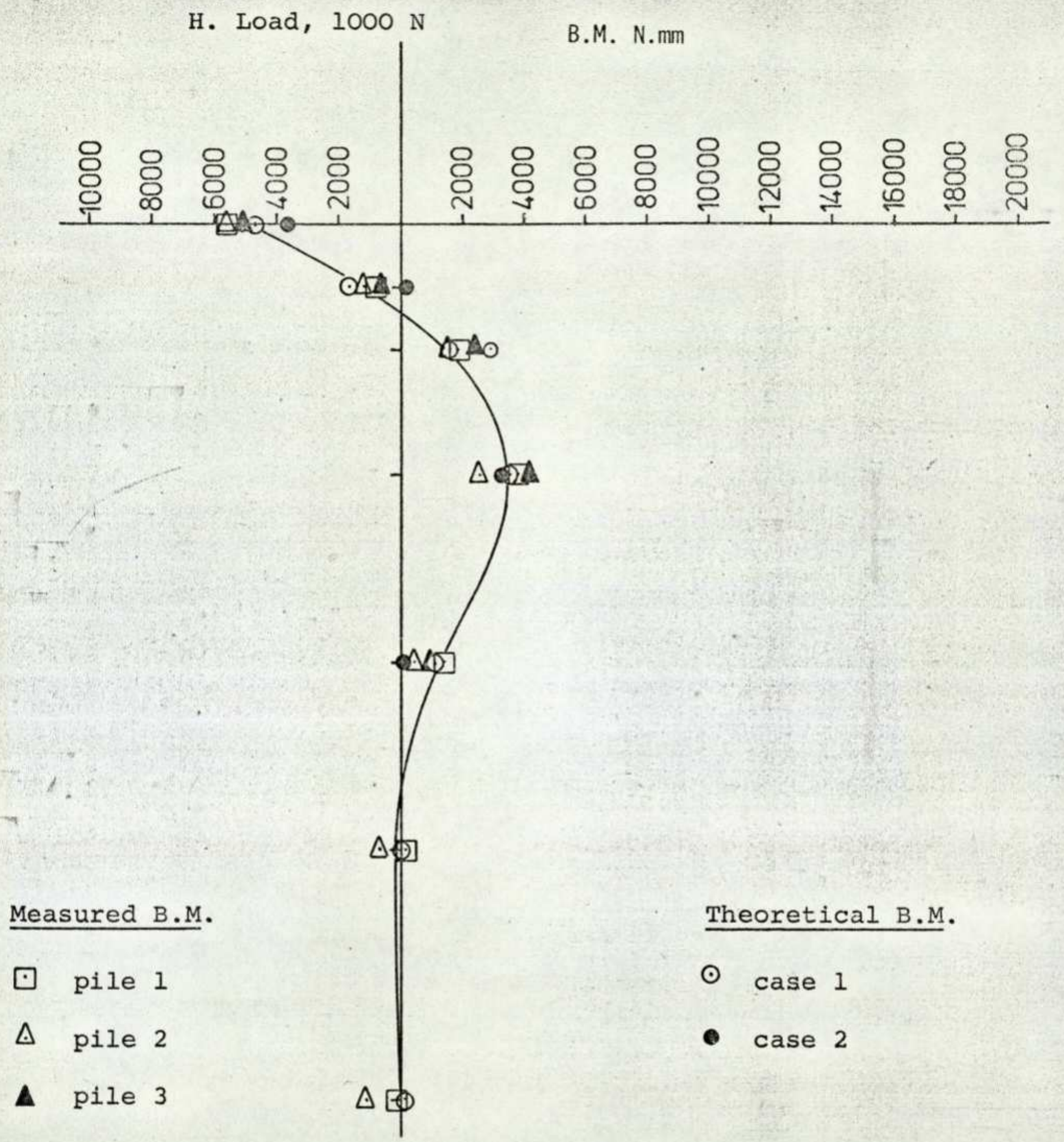


Fig. 8.62. Theoretical and experimental bending moment Vs Depth

Nine-pile Group (3 x 3)

Test No. 16

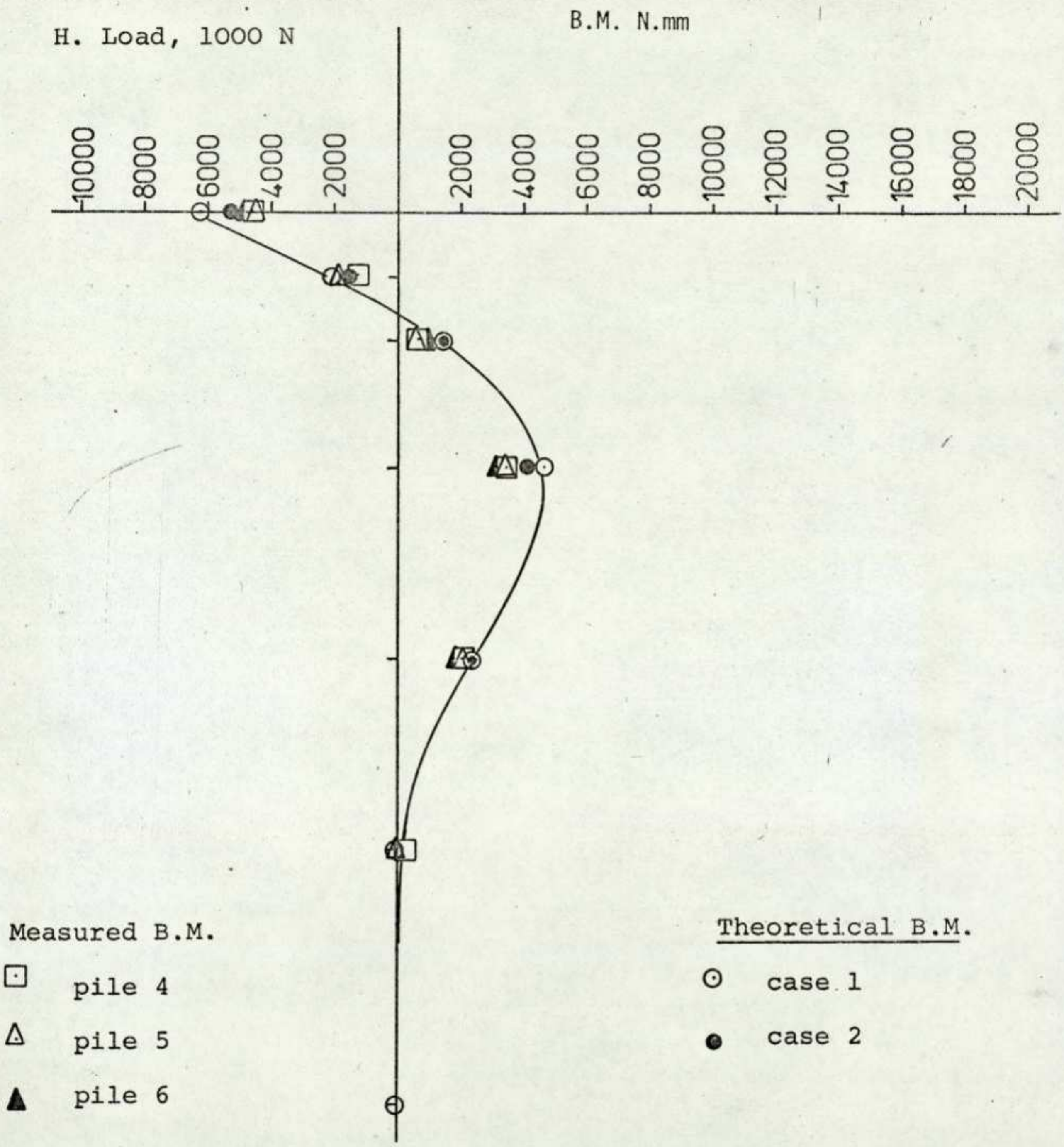


Fig. 8.63. Theoretical and experimental bending moment Vs Depth

Nine-pile Group (3 x 3)

Test No. 16

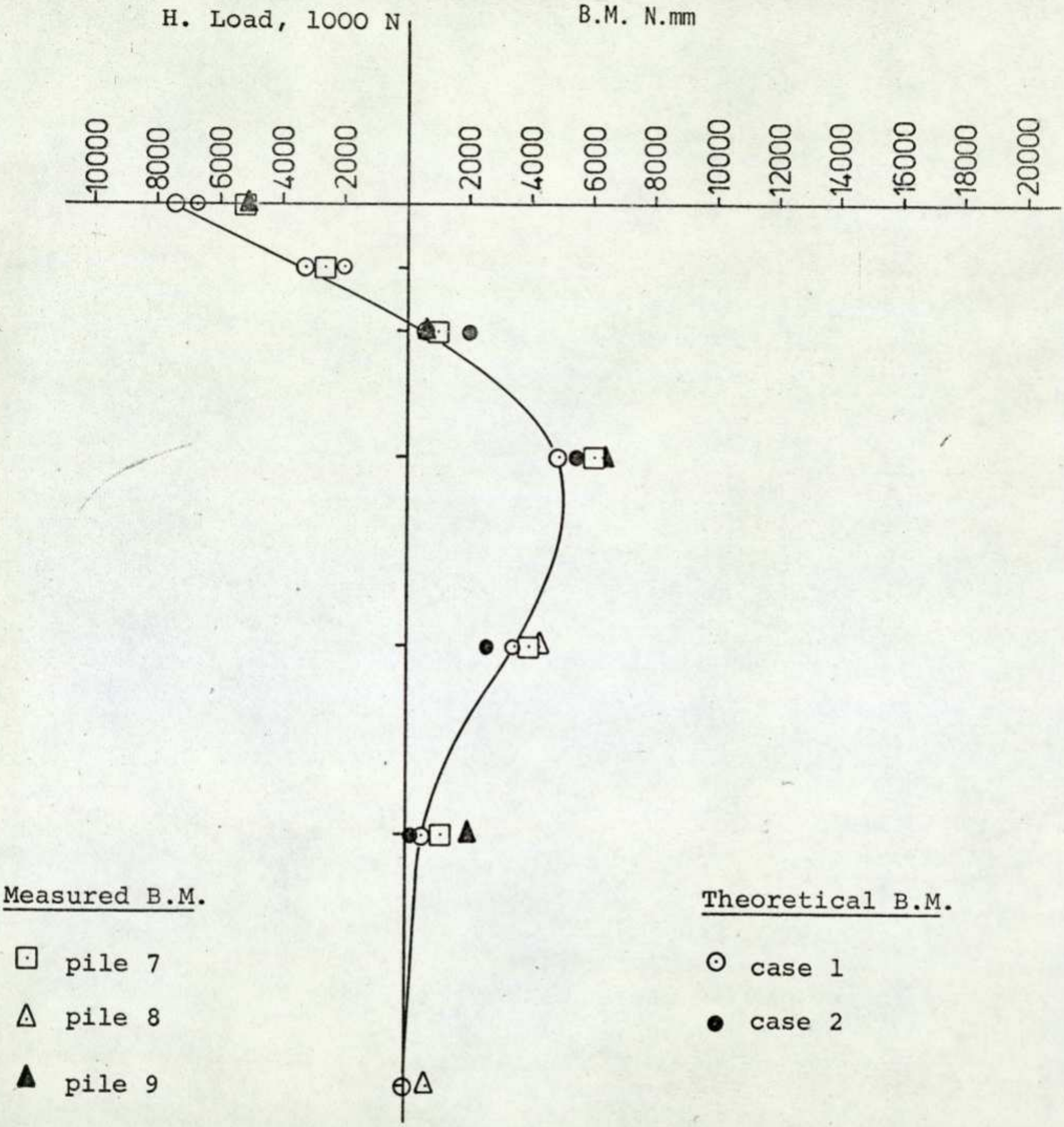


Fig. 8.64. Theoretical and experimental bending moment Vs Depth

Table 8.1

Single vertical pile (Test 13A)

Results from programme - PGROUP (D . O . E .)

Young's modulus of soil , E N/mm^2	Pile group flexibility coefficients C_{11} 10^{-3}	C_{22} 10^{-2}	C_{32} 10^{-4}
3.346	0.7733	0.9774	0.4632
6.892	0.4132	0.6836	0.3727
13.784	0.2405	0.5033	0.3098
27.568	0.1498	0.3857	0.2640
34.460	0.1303	0.3573	0.2522

C_{11} - Vertical displacement due to unit vertical load

C_{22} - Horizontal displacement due to unit horizontal load

C_{32} - Rotation due to unit horizontal load

Cap displacements

Young's modulus of soil , E N/mm^2	Load case No.	Vertical displacement mm	Horizontal displacement mm	Rotations (radians)
3.346	1	1.029	0.0	0.0
	2	0.0	1.173	0.00558
6.892	1	0.5496	0.0	0.0
	2	0.0	0.8203	0.004472
13.784	1	0.3199	0.0	0.0
	2	0.0	0.6039	0.003718
27.568	1	0.1993	0.0	0.0
	2	0.0	0.4629	0.003169
34.460	1	0.1734	0.0	0.0
	2	0.0	0.4288	0.3026

Load case No. 1 - Vertical load = 1330 N

Load case No. 2 - Horizontal load = 120 N

Table 8.2

Single vertical pile (Test 13A)

Results from programme - PGROUP (D . O . E .)

Total moment at top of each pile segment , N.mm

Load case No. 2 - Horizontal load = 120 N

No. of shaft segments	Young's modulus of soil , N/mm^2 3.346	6.892	13.784	27.568	34.460
1	12000	12000	12000	12000	12000
2	16000	14600	13200	11700	11200
3	11300	8150	5350	2890	2210
4	5800	2840	848	-258	-439
5	2110	290	-419	-443	-375
6	327	-416	-384	-140	-75.3
7	-194	-344	-150	-1.16	16
8	-115	-110	-274	-10.1	10.5

Table 8.3

The elastic and plastic soil stiffness constant and the initial stiffness coefficients B2 to B6

Type of pile	Latest elastic constant	Latest plastic constant	Initial stiffness coefficients B2 = B3 N/mm	B4 = B5 N/mm	B6 N/mm
V	37255.734	257.793	246.4482	8.6645	38.58775
$+15^o$	149022.937	397.571	444.9917	10.7391	58.21975
-15^o	40627.666	306.569	256.1439	8.7862	39.63635
$+30^o$	1141638.83	545.254	944.6711	14.1529	98.46435
-30^o	32715.237	306.569	232.497	8.484	37.057

V - Single vertical pile

$+15^o$ - Single batter pile ($+15^o$)

-15^o - Single batter pile (-15^o)

$+30^o$ - Single batter pile ($+30^o$)

-30^o - Single batter pile (-30^o)

Chapter 9

Conclusions and suggestions for further investigations

9.1 Conclusions

As a summing up , it can be concluded :

a - Single piles

1 - The transverse displacements and rotations of the pile cap on first loading were reduced with increasing inclination to the vertical . This reduction was significantly greater for piles with a positive batter .

2 - The transverse and rotational stiffness after cyclic loading were increased for vertical piles with a negative batter but were almost unchanged for piles with positive batter . The bulk of the increase in stiffness occurred in the first few load cycles .

3 - Cyclic loading caused a significant permanent displacement of the pile head . In the case of vertical piles and negative batter piles , this displacement was in the direction of the tranverse load . It was in the contrary direction for positive batter piles under both axial and transverse loading .

4 - The ratio of maximum axial to transverse loads does not significantly affect the transverse or rotational stiffness of the pile head .

5 - The position of the maximum moment was about one quarter of the embedded length below the soil surface in the case of the vertical pile . The maximum moment was slightly higher for raked piles , particularly those with a positive rake .

6 - The maximum bending moment in the negatively inclined piles was nearly equal to that in the vertical pile . A positive inclination reduces the maximum moment significantly .

7 - In the case of the piles with a positive batter , the bending moments were reduced very sharply below the point of maximum moment .

b - Pile groups

1 - Where the groups contained raked piles , the horizontal and vertical displacements of the pile caps on first loading were considerably smaller than in the case of a group of vertical piles .

2 - Where the groups contained raked piles , the rotations of the pile caps were considerably reduced and in the opposite direction to that in the case of a group of vertical piles .

3 - There was a significant increase in the horizontal and rotational stiffness of the groups as a result of cyclic loading . Most of this increase in stiffness occurred during the first few load cycles .

4 - Cyclic loading caused significant permanent displacements and rotations of the pile caps , much of which occured within the first few load cycles .

5 - In the case of a group of vertical piles there was some small rotation of the cap under a central vertical load , as a result of the variation in axial stiffness of the piles produced by the driving sequence . Raking the piles reduced this rotation .

6 - The existance of the vertical load had no significant effect on the horizontal stiffness of the pile head .

7 - For four - pile groups , the position of the maximum positive bending moment was at about one quarter of the embedded length for vertical piles . In the case of raked piles the position was significantly lower , particularly in the case of piles with a negative rake .

For nine - pile groups , the position of the maximum positive bending moment was about one fifth of the embedded length for vertical piles . The position of maximum positive moment was significantly lower where the piles were raked . This was particalarly noticeable in the case of piles in the centre and rear rows .

8 - The mean negative moment at the soil surface , and the mean of the maximum positive moments were smaller where the group contained raked piles than where the piles were all vertical .

9 - The negative bending moments at the soil surface in the piles of each group had nearly the same value . However , the maximum positive moments depended on the position of the pile in the group , and on the inclination in the case of a raked pile .

10 - The vertical load did not significantly affect the bending moments caused by horizontal loads .

11 - In the case of a group of vertical piles , the driving resistance was increased with increasing number of piles in the group , with the largest value for the latest driven pile . Where the groups contained raked piles , the driving resistance was increased within the same row , but it was increased or decreased for the next row depending on the number of piles and degree of inclinations of the neighbouring rows .

12 - Time dependent displacements , rotations and bending moments of pile groups were generally insignificant .

13 - There was good agreement between the computer prediction and the observed behaviour of the models , although this method cannot allow directly for interaction between closely spaced piles .

14 - Some allowance for interaction can be made by empirical adjustment of the pile and soil stiffness coefficients , of which the most important is the axial stiffness coefficient (B_1) .

15 - Where the piles were raked , so that the tips were far apart , good agreement was reached using the value of B_1 measured in a single pile test . Where all the piles in the group were vertical so that the tips were close together , it was necessary to reduce the value of B_1 by more than 50 % .

16 - The tests on single raked piles proved less satisfactory than the test on a single vertical pile as a means of predicting the stiffness of the raked piles in the groups .

17 - The surface integral method proved unsatisfactory for the analysis of pile groups in sands since it assumed the elastic modulus to be constant with depth . It is possible that an adaptation allowing the elastic modulus to vary with depth might prove more successfully .

9.2 Suggestions for further investigations

The following suggestion are made for further investigations :

1 - The extension of the investigation to larger groups .

2 - The extension of the investigation to groups with pile caps in contact with or embedded in the soil surface .

3 - The investigations of the behaviour of groups with raked piles in cohesive soils .

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APPENDIX 1

List of Computer Programme (PILE)

SUBFILE PILE

```
      MASTER PILE
      COMMON P1(10),BM1(10),R(40,5),S(40,5),P(40),
     2Y(40),BM(40),TITLE(10)
      READ(1,100) TITLE
100   FORMAT(10A6)
      READ(1,101) Z,B,EI,M,K
101   FORMAT(3F0.0,2I0)
      N=M+2
      DO 1 I=1,K
1     READ(1,102) P1(I), BM1(I)
102   FORMAT(2F0.0)
      READ(1,103) AM,PO,MG,L
103   FORMAT(2F0.0,2I0)
      READ(1,104) Y1E,Y1P,AN,PN
104   FORMAT(4F0.0)
      WRITE(2,105) TITLE,Z,B,EI
105   FORMAT(1H1///6X,21HLATERALLY LOADED PILE//
     26X,10A8///6X,9HPILE DATA//6X,11HPILE LENGTH,
     319X,F10.3,3X,1HM/6X,12HPILE BREADTH,18X,
     4F10.3,3X,1HM/6X,14HSTIFFNESS (EI),16X,
     5F10.3,3X,7HKN-SQ.M/)
      WRITE(2,106)M,K
106   FORMAT(6X,15HNUMBER OF PARTS,20X,I5/6X,
     225HNUMBER OF LOAD INCREMENTS,10X,I5///)
      WRITE(2,107)
107   FORMAT(6X,9HLOAD DATA//6X,5HSTAGE,5X,2HP1,
     210X,3HBM1/)
      DO 2 I=1,K
2     WRITE(2,108) I,P1(I),BM1(I)
108   FORMAT(I9,2F12.5)
      WRITE(2,109) AM,PO,MG,L
109   FORMAT(///6X,9HSOIL DATA//
     26X,26HINITIAL ELASTIC CONSTANT =,F10.1/
     36X,26HINITIAL PLASTIC CONSTANT =,F10.1/
     46X,22HNODE AT GROUND LEVEL =,I5/
     56X,22HNUMBER OF ITERATIONS =,I5///)
```

```
      H=Z/M
      C1=B*H**4/EI
      C2=H**2/EI
      C3=H*C2
      CALL FORMR(N,1)
      KN,KE,KP=0
    3 DO 4 I=1,N
    4 Y(I)=0.0
      K1=K
      IF(KN.LE.1)      K1=1
      DO 6 J1=1,K1
      I=J1
      IF(KN.LE.1)      I=K
      L1=L
      IF(KN.EQ.0)      L1=1
      DO 5 J=1,L1
      CALL RHS(I,N,H,C2,C3)
      CALL SOIL(N,B,H,AM,PU,MG,C1)
    5 CALL SOLN(N)
      IF(KN.LE.1)      GO TO 6
      CALL MOMENT(I,N,C2)
    6 CALL PRINT(I,N,KN,H,AM,PU,C3)
      IF(KN.NE.0)      GO TO 8
      DIF=Y1E-Y(2)
      ALIM=Y1E/100
      IF(ABS(DIF).LE.ALIM)      GO TO 7
      IF(DIF.LT.0.0)      AM=AM*2**AN
      IF(DIF.GT.0.0)      AM=AM/2**AN
      AN=AN/2
      KE=KE+1
      IF(KE.GT.12)      STOP
      GO TO 3
    7 KN=1
      GO TO 3
```

```
    8 IF(KN.EQ.2)           GO TO 10
      DIF=Y1P-Y(2)
      ALIM=Y1P/100
      IF(ABS(DIF).LE.ALIM)        GO TO 9
      IF(DIF.LT.0.0)        PU=PU*2**PN
      IF(DIF.GT.0.0)        PU=PU/2**PN
      PN=PN/2
      KP=KP+1
      IF(KP.GT.12)          STOP
      GO TO 3
    9 KN=2
      GO TO 3
   10 CALL FORMR(N,2)
      DO 12 I=11,12
      DO 11 J=1,N
   11 Y(J)=0.0
      CALL RHS(I,N,H,C2,C3)
      CALL SOIL(N,B,H,AM,PU,MG,C1)
      CALL SOLN(N)
      CALL MOMENT(I,N,C2)
   12 CALL PRINT(I,N,KN,H,AM,PU,C3)
      STOP
      END
```

```
      SUBROUTINE FORMR(N,KA)
      COMMON P1(10),BM1(10),R(40,5),S(40,5),P(40),
     2Y(40),BM(40),TITLE(10)
      DO 1 I=1,3
      DO 1 J=1,5
    1 R(I,J)=0.0
      DO 2 I=4,N
      R(I,3)=6.
      R(I,2),R(I-1,4)=-4.
    2 R(I,1),R(I-2,5)=1.
      IF(KA.EQ.2)          GO TO 3
      R(1,3),R(1,5),R(2,3),R(3,3)=1.0
      R(1,4)=-1.0
      R(2,4),R(3,4)=-2.0
      GO TO 4
    3 R(4,1)=0.0
      R(1,3)=0.5
      R(1,4)=0.1666666667
      R(2,2),R(2,3),R(3,1)=1.0
      R(1,5),R(2,5),R(3,3),R(3,5)=-1.0
      R(2,4),R(3,2),R(3,4)=2.0
    4 R(N,1),R(N,3)=2.
      R(N-1,3)=5.
      R(N-1,4)=-2.
      RETURN
      END
```

```
      SUBROUTINE SOIL(N,B,H,AM,PU,MG,C1)
      COMMON P1(10),BM1(10),R(40,5),S(40,5),P(40),
     2Y(40),BM(40),TITLE(10)
      DO 1 I=1,N
      DO 1 J=1,5
 1    S(I,J)=R(I,J)
      DO 3 I=MG+2,N
      ID=I-MG-1
      YI=ABS(Y(I))
      IF(YI.LE.0.0000001)      GO TO 2
      X=(AM*YI/(PU*B))
      S(I,3)=S(I,3)+PU*H*ID*TANH(X)*C1/YI
      GO TO 3
 2    S(I,3)=S(I,3)+AM*H*ID*C1/B
 3    CONTINUE
      RETURN
      END
```

```
      SUBROUTINE RHS(I,N,H,C2,C3)
      COMMON P1(10),BM1(10),R(40,5),S(40,5),P(40),
     2Y(40),BM(40),TITLE(10)
      DO 1 J=1,N
    1 P(J)=0.0
      IF(I.GT.10)       GO TO 2
      P(1)=BM1(I)*C2/2+P1(I)*C3/6
      P(2)=BM1(I)*C2+P1(I)*C3
      P(3)=BM1(I)*C2+P1(I)*C3*2
      GO TO 4
    2 IF(I.EQ.12)       GO TO 3
      P(1)=-1.0
      P(2)=1.0
      P(4)=-1.0
      GO TO 4
    3 P(1)=H
    4 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE SOLN(N)
      COMMON P1(10),BM1(10),R(40,5),S(40,5),P(40),
     2Y(40),BM(40),TITLE(10)
      DO 1 I=1,N-2
      S(I,3)=1.0/S(I,3)
      DO 1 J=1,2
      DO 1 K=1,2
    1 S(I+J,3+K-J)=S(I+J,3+K-J)-S(I+J,3-J)*S(I,3)*
     2S(I,3+K)
      S(N-1,3)=1.0/S(N-1,3)
      S(N,3)=S(N,3)-S(N,2)*S(N-1,3)*S(N-1,4)
      DO 2 I=1,N-2
      DO 2 J=1,2
    2 P(I+J)=P(I+J)-S(I+J,3-J)*S(I,3)*P(I)
      Y(N)=(P(N)-S(N,2)*S(N-1,3)*P(N-1))/S(N,3)
      Y(N-1)=(P(N-1)-S(N-1,4)*Y(N))*S(N-1,3)
      DO 4 I1=2,N-1
      I=N-I1
      DO 3 J=1,2
    3 P(I)=P(I)-S(I,3+J)*Y(I+J)
    4 Y(I)=P(I)*S(I,3)
      RETURN
      END
```

```
      SUBROUTINE MOMENT(J,N,C2)
      COMMON P1(10),BM1(10),R(40,5),S(40,5),P(40),
     2Y(40),BM(40),TITLE(10)
      IF(J.GT.10)        GO TO 1
      BM(1)=BM1(J)
      BM(2)=(Y(2)-2*Y(3)+Y(4))/C2
      GO TO 3
    1 BM(1)=Y(1)/C2
      IF(J.EQ.12)        GO TO 2
      BM(2)=(1.0-2*Y(3)+Y(4))/C2
      GO TO 3
    2 BM(2)=(-2*Y(3)+Y(4))/C2
    3 DO 4 I=3,N-2
    4 BM(I)=(Y(I)-2*Y(I+1)+Y(I+2))/C2
      BM(N-1)=0.
      RETURN
      END
```

```
      SUBROUTINE PRINT(I,N,KN,H,AM,PU,C3)
      COMMON P1(10),BM1(10),R(40,5),S(40,5),P(40),
     2Y(40),BM(40),TITLE(10)
      IF(I.GT.10)          GO TO 1
      YR=Y(1)/H
      WRITE(2,100) I,Y(2),YR
100   FORMAT(///6X,8HSTAGE = ,I2,6X,
     224HPILE HEAD DISPLACEMENT =,F10.7/22X,
     320HPILE HEAD ROTATION =,F14.7//)
      WRITE(2,101) AM,PU
101   FORMAT(22X,25HLATEST ELASTIC CONSTANT =,F12.3/
     222X,25HLATEST PLASTIC CONSTANT =,F12.3///)
      GO TO 2
1     PR=Y(2)/C3
      WRITE(2,102) I,PR,BM(1)
102   FORMAT(///6X,8HSTAGE = ,I2,6X,
     217HPILE HEAD FORCE =,F13.4/22X,
     318HPILE HEAD MOMENT =,F12.4//)
2     IF (KN.LE.1)         GO TO 4
      WRITE(2,103)
103   FORMAT(/6X,4HNODE,6X,6HMOMENT,4X,
     212HDISPLACEMENT/)
      IF(I.LE.10)        Y0=Y(2)
      IF(I.EQ.11)        Y0=1.0
      IF(I.EQ.12)        Y0=0.0
      WRITE(2,104) BM(1),Y0
104   FORMAT(8X,1H1,2F12.7)
      DO 3 I1=2,N-1
3     WRITE(2,105) I1,BM(I1),Y(I1+1)
105   FORMAT(I9,2F12.7)
4     CONTINUE
      RETURN
      END
```

APPENDIX 11

List of Computer Programme (PILEGROUP)

```
      MASTER PILEGROUP
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     2/TWO/E(9,21),G(10,6)
     3/THREE/A(9,6,6),B(9,6,6),ABAT(6,6),BAT(6,6)
     4/FOUR/C(6,6),D(6,6)
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
     6/SIX/H(6),G1(6),RG(6),U(9,6),F(9,6)
      READ(1,98) TITLE
98    FORMAT(10A8)
      READ(1,99) NP,NL,NM,NI,LIST,Z
99    FORMAT(5I0,F0.0)
      DO 1 I=1,NP
      READ(1,100) (E(I,J),J=1,8)
1     READ(1,100) (E(I,J),J=10,17)
100   FORMAT(8F0.0)
      DO 2 I=1,NL
2     READ(1,100) (G(I,J),J=1,6)
      WRITE(2,101) TITLE,NP,NL,NM
101   FORMAT(///6X,20HPILE GROUP PROGRAM 2//6X,
     232HNON-LINEAR LATERAL SOIL REACTION//
     36X,10A8//6X,15HNUMBER OF PILES,20X,I1/6X,
     421HNUMBER OF LOAD STAGES,13X,I2/6X,
     524HNUMBER OF PARTS PER PILE,10X,I2)
      WRITE(2,102) NI,Z
102   FORMAT(6X,30HNUMBER OF ITERATIONS IN *PILE*,
     2I6/6X,32HHEIGHT OF CAP ABOVE SOIL SURFACE,
     3F7.3///6X,9HPILE DATA//)
      DO 3 I=1,NP
      WRITE(2,103) I,(E(I,J),J=1,5)
103   FORMAT(6X,7HPILE NO,I2/6X,
     222HHEAD CO-ORDINATES: X =,F7.3,5H; Y =,F7.3,
     35H; Z =,F7.3/6X,23HDIRECTION FROM X AXIS =,
     4F6.2,10H; BATTER =,F6.2)
```

```
      WRITE(2,104) (E(I,J),J=6,8)
104   FORMAT(6X,8HLENGTH =,F7.3,11H; BREADTH =,F7.3,
     217H; STIFFNESS(EI) =,F7.3)
      WRITE(2,105) (E(I,J),J=10,17)
105   FORMAT(6X,23HELASTIC SOIL CONSTANT =,F10.3/
     26X,23HPLASTIC SOIL CONSTANT =,F10.3/6X,
     332HINITIAL STIFFNESS COEFFICIENTS:-/12X,4HB1 =
     4,F10.3,6H; B2 =,F10.3,6H; B3 =,F10.3/12X,
     54HB4 =,F10.3,6H; B5 =,F10.3,6H; B6 =,F10.3//)
      E(I,9)= Z/COS(E(I,5)*0.01745327)
      E(I,18)=E(I,6)/NM
      E(I,19)=E(I,7)*E(I,18)**4/E(I,8)
      E(I,20)=E(I,18)**2/E(I,8)
      E(I,21)=E(I,18)*E(I,20)
3     CONTINUE
      WRITE(2,106)
106   FORMAT(//6X,10HLOAD DATA:/6X,5HSTAGE,14X,1HX,
     211X,1HY,11X,1HZ)
      DO 4 I=1,NL
4     WRITE(2,107)I,(G(I,J),J=1,6)
107   FORMAT(6X,I2,6X,3HF =,3F12.3/14X,3HM =,3F12.3)
      N=NM+2
      CALL FORMR
      DO 5 J=1,6
      DO 5 K=1,6
5     C(J,K),D(J,K)=0.0
      DO 9 I=1,NP
      DO 6 J=1,6
      DO 6 K=1,6
6     A(I,J,K),B(I,J,K),BAT(J,K),ABAT(J,K)=0.0
```

```
      CALL FORMA(I)
      CALL FORMB(I)
      DO 7 J=1,6
      DO 7 K=1,6
      DO 7 L=1,6
 7    BAT(J,K)=BAT(J,K)+B(I,J,L)*A(I,K,L)
      DO 8 J=1,6
      DO 8 K=1,6
      DO 8 L=1,6
 8    ABAT(J,K)=ABAT(J,K)+A(I,J,L)*BAT(L,K)
      DO 9 J=1,6
      DO 9 K=1,6
 9    C(J,K)=C(J,K)+ABAT(J,K)
      DO 10 I=1,6
      DO 10 J=1,6
 10   D(I,J)=C(I,J)
      CALL INVERT
      DO 11 K=1,6
 11   H(K),G1(K)=0.0
 12   DO 21 J=1,NL
      K1=0
      AL1=ABS(G(J,1))*0.01
      AL3=ABS(G(J,3))*0.01
 13   DO 14 K=1,6
      RG(K)=G(J,K)-G1(K)
 14   G1(K)=0.0
      DO 15 K=1,6
      DO 15 L=1,6
 15   H(K)=H(K)+D(K,L)*RG(L)
      DO 19 I=1,NP
      DO 16 K=1,6
      U(I,K)=0.0
      DO 16 L=1,6
 16   U(I,K)=U(I,K)+A(I,L,K)*H(L)
```

```
      DO 17 K=1,6
      F(I,K)=0.0
      DO 17 L=1,6
17    F(I,K)=F(I,K)+B(I,K,L)*U(I,L)
      Y1=U(I,2)
      HR=-U(I,6)
      CALL PILE(I,K1,Y1,HR,E(I,18))
      F(I,2)=Y(I,2)/E(I,21)
      F(I,6)=-Y(I,1)/E(I,20)
      DO 18 K=1,6
      DO 18 L=1,6
18    G1(K)=G1(K)+A(I,K,L)*F(I,L)
19    CONTINUE
      IF(K1.EQ.1)               GO TO 20
      R1=ABS(G(J,1)-G1(1))
      R3=ABS(G(J,3)-G1(3))
      IF(R1.LT.AL1.AND.R3.LT.AL3)      K1=1
      GO TO 13
20    CONTINUE
      WRITE(2,110)J,(G(J,K),K=1,6),G1,H
110   FORMAT(///6X,9HLOAD CASE,I3/
     236X,1HX,11X,1HY,11X,1HZ/
     36X,21HAPPLIED LOAD:   FORCE ,3F12.5/
     421X,6HMOMENT,3F12.5//
     56X,21HPILE REACTION: FORCE ,3F12.5/
     621X,6HMOMENT,3F12.5//
     76X,21HPILE CAP DISPLACEMENT,3F12.5/
     815X,8HROTATION,4X,3F12.5///)
21    CONTINUE
      STOP
      END
```

```
      SUBROUTINE FORMR
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
      DO 1 I=4,N
      R(I,3)=6.0
      R(I,2),R(I-1,4)=-4.0
    1 R(I,1),R(I-2,5)=1.0
      R(4,1)=0.0
      R(1,3)=0.5
      R(1,4)=0.1666666667
      R(2,2),R(2,3),R(3,1)=1.0
      R(1,5),R(2,5),R(3,3),R(3,5)=-1.0
      R(2,4),R(3,2),R(3,4)=2.0
      R(N,1),R(N,3)=2.0
      R(N-1,3)=5.0
      R(N-1,4)=-2.0
      RETURN
      END
```

```
      SUBROUTINE FORMA(I)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     2/TWO/E(9,21),G(10,6)
     3/THREE/A(9,6,6),B(9,6,6),ABAT(6,6),BAT(6,6)
      SNA=SIN(E(I,4)*0.01745327)
      SNB=SIN(E(I,5)*0.01745327)
      CSA=COS(E(I,4)*0.01745327)
      CSB=COS(E(I,5)*0.01745327)
      A(I,1,1)=CSA*SNB
      A(I,1,2)=CSA*CSB
      A(I,1,3)=-SNA
      A(I,2,1)=SNA*SNB
      A(I,2,2)=SNA*CSB
      A(I,2,3)=CSA
      A(I,3,1)=CSB
      A(I,3,2)=-SNB
      DO 1 J=1,3
      A(I,4,J)=E(I,2)*A(I,3,J)-E(I,3)*A(I,2,J)
      A(I,5,J)=E(I,3)*A(I,1,J)-E(I,1)*A(I,3,J)
      A(I,6,J)=E(I,1)*A(I,2,J)-E(I,2)*A(I,1,J)
      DO 1 K=1,3
      A(I,J+3,K+3)=A(I,J,K)
    1 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE FORMB(I)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     2/TWO/E(9,21),G(10,6)
     3/THREE/A(9,6,6),B(9,6,6),ABAT(6,6),BAT(6,6)
      B(I,1,1)= E(I,12)
      B(I,2,2)= E(I,13)
      B(I,3,3)= E(I,14)
      B(I,5,5)= E(I,15)
      B(I,2,6),B(I,6,2)= E(I,17)
      B(I,3,5),B(I,5,3)= -E(I.17)
      B(I,6,6)= E(I,16)
      RETURN
      END
```

```
      SUBROUTINE INVERT
      COMMON/FOUR/C(6,6),D(6,6)
      DO 4 K=1,6
      DO 1 J=1,6
    1 IF(J.NE.K)            D(K,J)=D(K,J)/D(K,K)
      DO 2 I=1,6
      IF(I.EQ.K)            GO TO 2
      DO 2 J=1,6
      IF(J.EQ.K)            GO TO 2
      D(I,J)=D(I,J)-D(K,J)*D(I,K)
    2 CONTINUE
      DO 3 I=1,6
    3 IF(I.NE.K)            D(I,K)=-D(I,K)/D(K,K)
      D(K,K)=1.0/D(K,K)
    4 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE PILE(I,K1,Y1,HR,H)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
      DO 1 J=1,NI
      CALL RHS(Y1,HR,H)
      CALL SOIL(I)
      CALL SOLN(I)
      IF(K1.EQ.0.OR.J.NE.NI)            GO TO 1
      CALL MOMENT(I,Y1)
      CALL PRINT(I,Y1)
    1 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE RHS(Y1,HR,H)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
      P(1)=-Y1+HR*H
      P(2)=Y1
      P(3)=0.0
      P(4)=-Y1
      DO 1 J=5,N
    1 P(J)=0.0
      RETURN
      END
```

```
      SUBROUTINE SOIL(I)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     2/TWO/E(9,21),G(10,6)
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
      DO 1 J=1,N
      DO 1 K=1,5
    1 S(J,K)=R(J,K)
      DO 2 J=3,N
      DP=(J-2)*E(I,18)-E(I,9)
      IF(DP.LE.0.0)            GO TO 2
      YJ=ABS(Y(I,J))
      IF(YJ.LE.0.1E-6)         GO TO 3
      X=(E(I,10)*YJ/(E(I,11)*E(I,7)))
      S(J,3)=S(J,3)+E(I,11)*DP*TANH(X)*E(I,19)/YJ
      GO TO 2
    3 S(J,3)=S(J,3)+E(I,10)*DP*E(I,19)/E(I,7)
    2 CONTINUE
      RETURN
      END
```

```
      SUBROUTINE SOLN(IP)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
      DO 1 I=1,N-2
      S(I,3)=1.0/S(I,3)
      DO 1 J=1,2
      DO 1 K=1,2
    1 S(I+J,3+K-J)=S(I+J,3+K-J)-S(I+J,3-J)*S(I,3)*
     2S(I,3+K)
      S(N-1,3)=1.0/S(N-1,3)
      S(N,3)=S(N,3)-S(N,2)*S(N-1,3)*S(N-1,4)
      DO 2 I=1,N-2
      DO 2 J=1,2
    2 P(I+J)=P(I+J)-S(I+J,3-J)*S(I,3)*P(I)
      Y(IP,N)=(P(N)-S(N,2)*S(N-1,3)*P(N-1))/S(N,3)
      Y(IP,N-1)=(P(N-1)-S(N-1,4)*Y(IP,N))*S(N-1,3)
      DO 4 I1=2,N-1
      I=N-I1
      DO 3 J=1,2
    3 P(I)=P(I)-S(I,3+J)*Y(IP,I+J)
    4 Y(IP,I)=P(I)*S(I,3)
      RETURN
      END
```

```
      SUBROUTINE MOMENT(I,Y1)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,7
     2/TWO/E(9,21),G(10,6)
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
      BM(1)=Y(I,1)/E(I,20)
      BM(2)=(Y1-2*Y(I,3)+Y(I,4))/E(I,20)
      DO 1 J=3,N-2
    1 BM(J)=(Y(I,J)-2*Y(I,J+1)+Y(I,J+2))/E(I,20)
      BM(N-1)=0.0
      RETURN
      END
```

```
      SUBROUTINE PRINT(I,Y1)
      COMMON/ONE/TITLE(10),N,NP,NL,NM,NI,LIST,Z
     2/TWO/E(9,21),G(10,6)
     5/FIVE/Y(9,40),R(40,5),S(40,5),P(40),BM(40)
      PR=Y(I,2)/E(I,21)
      WRITE(2,100) I,PR,BM(1)
100   FORMAT(///6X, 7HPILE NO,I2,6X,
     218HPILE HEAD FORCE  =,F12.4/21X,
     318HPILE HEAD MOMENT =,F12.4//)
      IF(LIST.GT.0)       GO TO 3
      BMIN,BMAX=0.0
      DO 2 I1=1,N-1
      IF(BM(I1).LT.BMAX)        GO TO 1
      BMAX=BM(I1)
      MMAX=I1
      GO TO 2
1     IF(BM(I1).GT.BMIN)        GO TO 2
      BMIN=BM(I1)
      MMIN=I1
2     CONTINUE
      WRITE(2,101)BMAX,MMAX,BMIN,MMIN
101   FORMAT(21X,18HMAXIMUM MOMENT    =,F12.4/
     221X,18HAT NODE NUMBER      ,I9/
     321X,18HMINIMUM MOMENT    =,F12.4/
     421X,18HAT NODE NUMBER      ,I9///)
      GO TO 5
3     WRITE(2,102)
102   FORMAT(/6X,4HNODE,6X,6HMOMENT,4X,
     212HDISPLACEMENT/)
      WRITE(2,103) BM(1),Y1
103   FORMAT(8X,1H1,2F14.7)
      DO 4 I1=2,N-1
4     WRITE(2,104) I1,BM(I1),Y(I,I1+1)
104   FORMAT(I9,2F14.7)
5     CONTINUE
      RETURN
      END
```