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THE UNIVERSITY OF ALBERTA

AN EVALUATION OF THE USE OF SPUD PILES FOR GRAVITY STRUCTURES

by

FRANK WILLIAM BOULTBEE MORISON

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH

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Supervisor

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Date....21 April 86..........

ABSTRACT

Some arctic offshore gravity structure designers have proposed using short rigid ("spud") piles to increase the resistance of the structure to horizontal loads. An evaluation of the effectiveness of this concept has been carried out. This evaluation initially involved studying how gravity structures are currently designed for horizontal loads and determining the lateral capacity of single pile. Subsequently the literature review proceeded to attempting to evaluate the horizontal capacity of groups of piles. It was decided that the utility pile groups had in increasing the lateral capacity of gravity structures depended upon the ultimate capacity of the pile groups under horizontal loading.

Tests on model piles and pile groups in a bed of compacted clay were carried out. The objective of the tests was to compare the capacity of groups with a similarly loaded single pile, not to estimate the capacity of prototype groups.

Tests were carried out on 4, 9, and 16 pile square groups in a compacted low plasticity clay. The pile heads were fixed rigidly and the group was made to undergo a simple horizontal translation. Total load on the group was measured. The spacing between the piles was varied between 1.5 and 4.0 diameters, centre to centre.

The capacity of single piles tested was consistent with predicted values. However it was found that the efficiency

of the large groups tested was very low. The results show that for all configurations tested, behaviour is controlled by the pile dimensions. Under some conditions interference between piles in a laterally loaded group can be greater than previously believed. The results of these tests raise questions about the effectiveness of the concept of enhancing lateral resistance of offshore structures utilizing piles.

Acknowledgements

This thesis would not have been written if not for the help of a very great many people. First, I would like to thank my supervisor, Dr. N.R. Morgenstern, who suggested the topic and guided my efforts along the way. I shall always value the discussions we have had and the insight he brought to this problem. Without him, I would be still unaware of the value of what I have accomplished.

Financial support for the research came from the National Science and Engineering Research Council through the research funds of Dr. Morgenstern. In additon I would like to acknowledge the support afforded myself by the University of Alberta, and by the Government of Alberta, through its scholarship program.

The process of my education has benefitted through my interaction with many of the staff and students of the Department of Civil Engineering, too numerous to mention here. Howver I would like to mention two of my fellow graduate students, Tai Wong and Patrick Collins. Tai, as all who have met him know, is an endless source of reference in almost every subject, especially if a computer can be involved in some way. Patrick, dispensed generously of his expertise with Textform, without him this thesis could not have completed. The plots of the results of the research were done with the "PLOTIT" routines written by Tom Casey.

I would also like to thank Gerry Cyre and his staff in the labs. Without their help none of lab program could have been completed. Jeff Patsula and Ralph Kuhn conducted the experiments for test suite #2.

This degree has been completed while I was on an educational leave of absence from EBA Engineering Consultants Ltd. Some of the figures in this thesis were produced by the EBA drafting staff.

Finally, but certainly not least of all, I must acknowledge the debt I owe to my parents. Without their love and encouragement throughout a total of 22 years of scholastic endeavour this thesis could not have been begun let alone completed.

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1. INTRODUCTION

As hydrocarbon exploration in the southern Beaufort Sea begins to meet with success, design and research efforts are being directed increasingly at production concerns. Design and construction of production stuctures will be subjected to a number of enviromental loadings and constraints.

Among the constraints are the requirement to avoid on-site construction, short weather windows in which to schedule construction, and the possibility of pack ice invading the site and preventing access. These factors have led to a preference for a prefabricated structure ~~which~~ could be floated to the site and set down quickly.

Enviromental loads include waves, currents, tidal surges, and winds. However the most critical load is from ice. An extreme event could be an impact by a floating ice island up to 50m thick. Current estimates are that an ice feature of this size could apply a horizontal load of up to 24.5 MN per metre of structure width (Gulf Canada Resources Ltd.,1983). Obviously a load of this magnitude makes the lateral stability of the structure a significant problem.

It is generally considered that gravity structures, rather than the conventional pile supported jacket structures found in other offshore oilfields, will be used in the Beaufort. A gravity structure is essentially a structure with a shallow foundation. It is termed a gravity structure because its stability against overturning and sliding come from its own wieght. However in soft cohesive soils it is possible, if not probable, that the sliding resistance developed by the structure will be insufficient to resist the load applied by an extreme ice feature impact.

One proposed exploration structure, the SOHIO "SAMS" (Gerwick et al, 1983, Bea et al., 1983, 1984) proposes the use of 'spuds': short vertical piles beneath the structure. These piles are installed through sleeves in the structure in order to increase the resistance to horizontal loads. The SAMS is still a gravity structure; the piles play no role in supporting vertical load. The sleeve connections between the strucure and the piles prevent them from providing vertical resistance.

The SAMS was to be an octagonal structure 105m wide. It was to be stabilized with up to 36 spuds placed around the perimeter. The designers felt, after suitable analysis, that the 2.13m diameter piles were spaced sufficiently apart to allow each pile to mobilize very close to its resistance as an isolated pile.

Because of its role as an exploration structure, the SAMS was designed to resist foundation loads due to ice impact of between 360MN and 580MN or between 4.3MN/m and 5.5MN/m of structure width. As design of production structures progresses, it will become logical to ask whether the spud concept can be extended to resist the higher loads that these structures must be designed for. To resist higher loads greater numbers of spuds will be required, at closer spacings. At this point the designer must become concerned about whether he can depend on each pile to resist the same ultimate load as it would if it were isolated. If he cannot, he needs to know the magnitude of reduction that must be made and whether there is a limit to the benefits that can be attained by the use of piles.

It is the purpose of this thesis to attempt to answer these questions. In the first three chapters a review of some of the various components of the problem will be undertaken. Subsequent chapters will discuss tests on model pile groups carried out for this study.

In Chapter 2, some of the theories and practical experience in the design of an isolated single pile under horizontal loading will be reviewed. Chapter 3 will review the components of the stability of gravity structures with conventional foundations. Then, in Chapter 4, a review of what is known about pile group behaviour will be made, concentrating on the ultimate lateral load capacity of piles in groups.

Because the available literature does not adequately address the concept of how proximity to other piles affects the ultimate resistance, especially for larger groups of short piles, it was decided to undertake model tests. Chapter 5 discusses the justification for and philosophy behind the tests. The apparatus which was designed and constructed for the tests is described. Finally in Chapter 6 the results of the tests are described and the impact of the results on the viability ~~of~~ the spud concept is discussed.

2. HORIZONTAL RESISTANCE OF SINGLE PILES

2.1 FAILURE MECHANISMS

The failure mechanism of a single pile depends on the combination of applied moment, applied horizontal force, and assumed soil reaction. All piles except for rigid fixed head piles, will have at least one point of rotation where the soil reaction will act in different directions. Depending upon the members length, strength and its condition of end fixity, the pile could fail in one of five different modes described by Broms(1964). These modes of failure are shown on Figure 2.1(a-e), and are described in the following section.

2.1.1 Short Fixed Head Piles (Figure 2.1a)

This pile exhibits the simplest behaviour. In this case the strugural capacity is not exceeded in any part of the pile. The pile translates through the soil as a rigid body and the resistance is determined by the strength of the soil alone.

2.1.2 Short Free Headed Pile (Figure 2.1b)

As with the first type, this pile does not have its flexural capacity exceeded. However because it has no end fixity, it must derive rotational restraint from the soil. While the pile top translates in the direction of the load, the pile also rotates and the tip translates in the opposite

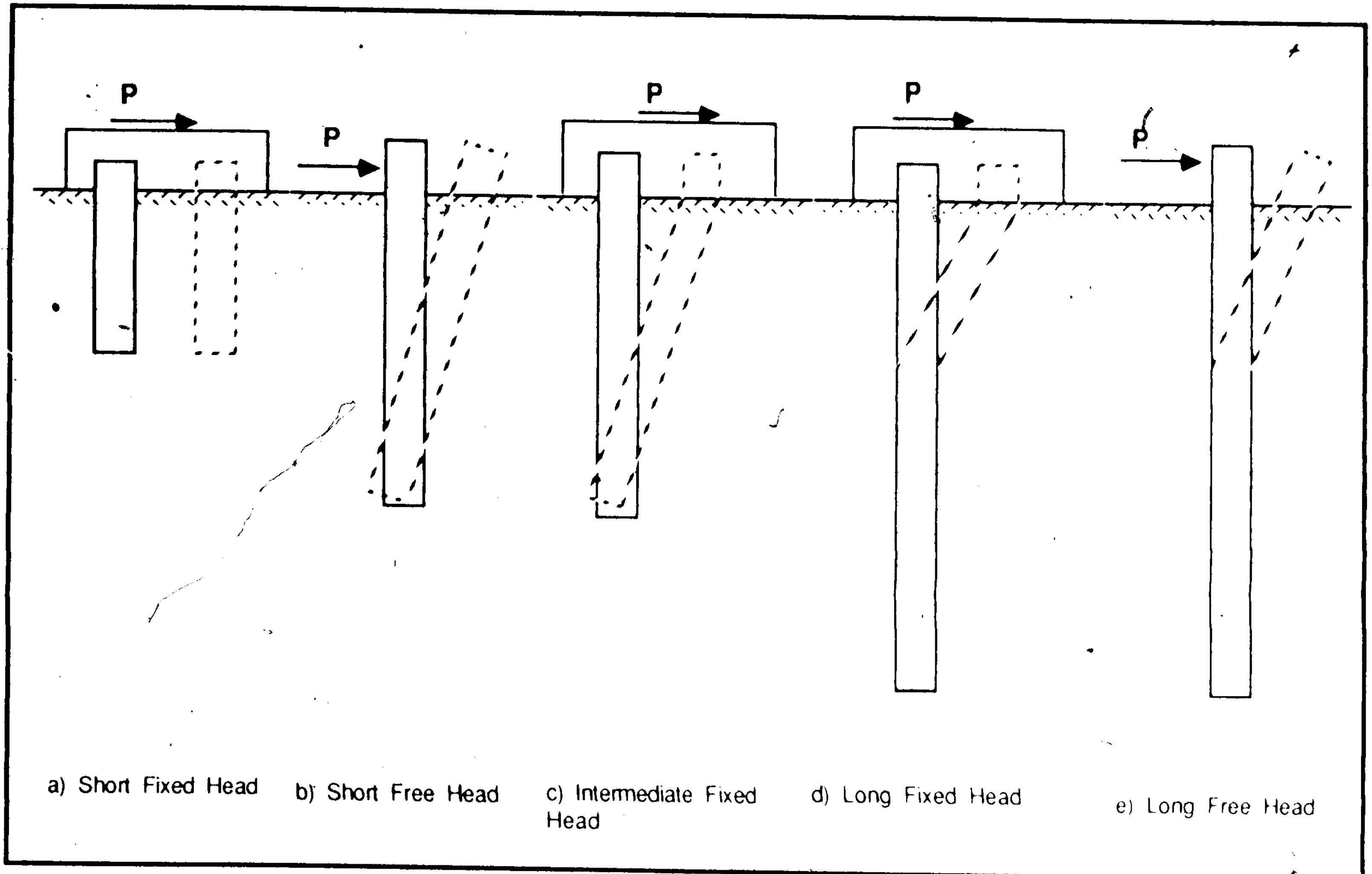


FIGURE 2.1 FAILURE MODES FOR LATERALLY LOADED PILES

direction. The resistance is again dependent on the strength of the soil, but now is also dependent on its stiffness. This is so, because near the point of rotation the displacement of the pile will be small, and the degree to which the strength of the soil is mobilized will depend on its stiffness.

2.1.3 Intermediate Fixed Head Pile (Figure 2.1c)

In this case the moment capacity at the head of the pile is exceeded and a plastic hinge forms. Therefore the pile will rotate until additional restraint is provided by the soil. Horizontal capacity is now dependent on moment capacity of the pile cap as well as soil strength and soil stiffness. Sample calculations show that the yield moment of even the heaviest walled steel pipe piles will be exceeded if the pile is longer than 10 to 15 diameters in all but very soft soils.

2.1.4 Long Fixed Head Pile (Figure 2.1d)

Two plastic hinges have now formed, one at the pile cap and one in the soil. Above the lower plastic hinge the pile rotates and translates in the direction of the load, while below the lower plastic hinge, the pile rotates slightly to develop soil reaction to resist the moment. Capacity is dependent on pile moment capacity, capacity at the cap, soil strength and soil stiffness. Relatively larger pile displacements make soil stiffness a parameter of less importance.

2.1.5 Long Free Head Pile (Figure 2.1e)

A plastic hinge forms in the soil only. As with the long fixed head pile, the upper portion translates and rotates, while the lower portion rotates only. Capacity depends on pile moment capacity, soil strength, and soil stiffness.

2.2 METHODS OF CALCULATING HORIZONTAL CAPACITY

If a soil reaction distribution is assumed it becomes a fairly simple matter of determining a combination of ultimate horizontal load and moment. Various authors have worked out solutions for simple soil reaction distributions, and these are given in Table 2.1.

There are significant differences between the methods of the various authors because of different assumptions made to make the problem statically determinate. As an example, Broms' solution for a short free-headed pile in a soil where the reaction has a triangular distribution gives a load 33 percent more than the Poulos and Davis solution for a pile 20 diameters long with the load applied at the soil surface (Poulos and Davis, 1980).

Alternative more sophisticated analyses treat the soil reaction along the pile as a series of nonlinear springs. Relationships describing the behaviour of these springs are

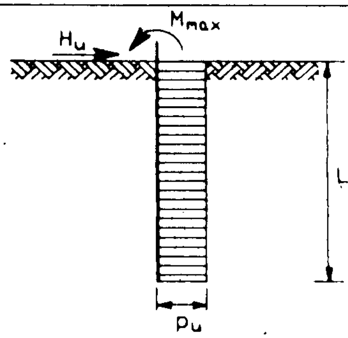
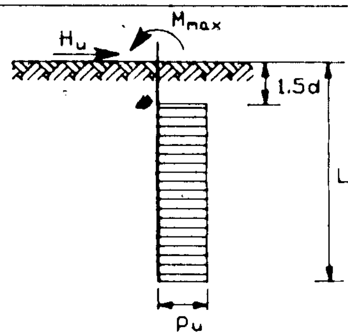
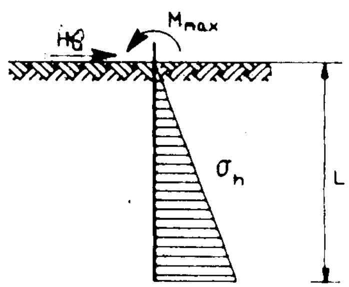
PILE TYPE	SOIL REACTION ASSUMED	ULTIMATE LATERAL LOAD	MAXIMUM MOMENT	REFERENCE
SHORT FIXED HEAD		$H_u = p_u L$	$M_{max} = 0.5HLp_u$	
SHORT FIXED HEAD		$H_u = p_u d(L - 1.5d)$	$M_{max} = H_u(0.5L + 0.75d)$	BROMS (1964a)
SHORT FIXED HEAD		$H_u = 0.5\sigma_h L^2 d$	$M_{max} = 0.67H_u L$	BROMS (1964b)

Table 2.1 ULTIMATE LATERAL LOAD FOR SIMPLE SOIL REACTION DISTRIBUTIONS

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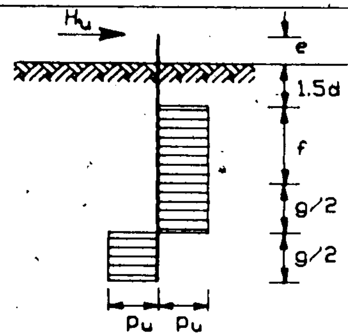
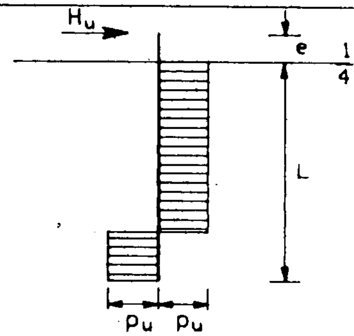
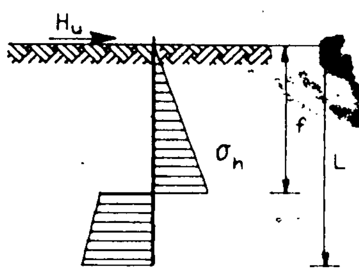
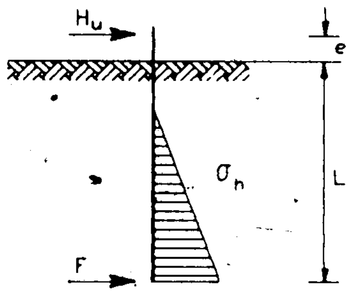
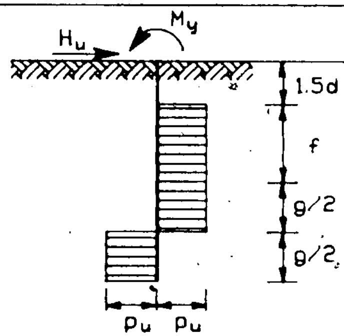
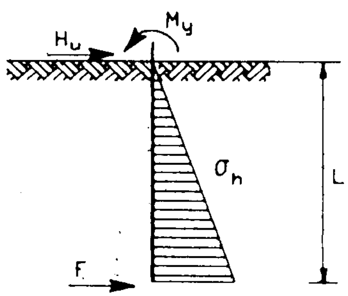
PILE TYPE	SOIL REACTION ASSUMED	ULTIMATE LATERAL LOAD	MAXIMUM MOMENT	REFERENCE
SHORT FREE HEAD		$H_u = p_u f$ (SOLVE FOUR EQUATIONS SIMULTANEOUSLY)	$M_{max} = 0.25 p_u d g^2$ $M_{max} = H_u(e + 1.5d + 0.5f)$ $L = (1.5d + f + g)$	BROMS (1964a)
SHORT FREE HEAD		$\frac{1}{4}$ $\frac{H_u}{p_u dL} = \sqrt{\left(1+\frac{2e}{L}\right)^2 + 1} - \left(1+\frac{2e}{L}\right)$	$M_{max} =$ $\frac{1}{4}\left[1 - \left(\frac{2H_u}{p_u dL}\right) - \left(\frac{H_u}{p_u dL}\right)^2\right]$	POULOS & DAVIS (1980)
SHORT FREE HEAD		$H_u = d\sigma_h(f^2 - 0.5L^2)$ $\frac{1}{L}\left[4f^3 + 6e\left(\frac{f}{L}\right)^2 - 3e\right] - 2 = 0$		POULOS & DAVIS (1980)

Table 2.1 ULTIMATE LATERAL LOAD FOR SIMPLE REACTION DISTRIBUTIONS (CONTINUED)

6458-215

PILE TYPE	SOIL REACTION ASSUMED	ULTIMATE LATERAL LOAD	MAXIMUM MOMENT	REFERENCE
SHORT FREE HEAD		$H_u = \frac{\sigma_h dL^3}{6(e + L)}$	$M_{max} = H_u(e + 0.67f)$ $f = \sqrt{\frac{2H_u}{d\sigma_h}}$	BROMS (1964b)
INTERMEDIATE FIXED HEAD		$H_u = p_u df$ (SOLVE THREE EQUATIONS SIMULTANEOUSLY)	$M_y = \frac{p_u dg^2}{4} + p_u df(1.5d + 0.5f)$ $L = (1.5d + f + g)$	BROMS (1964a)
INTERMEDIATE FIXED HEAD		$H_u = \frac{\sigma_h dL^3 - M_y}{6L}$	$\frac{1}{12}\sigma_h f^3 d < M_y$ $f = \sqrt{\frac{2H_u}{d\sigma_h}}$	BROMS (1964b)

6458-212

Table 2.1 ULTIMATE LATERAL LOAD FOR SIMPLE REACTION DISTRIBUTIONS (CONTINUED)

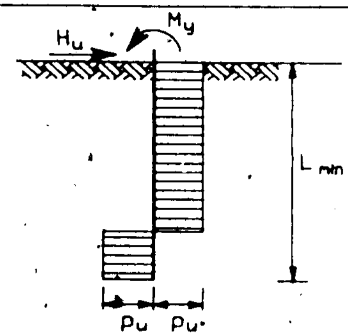
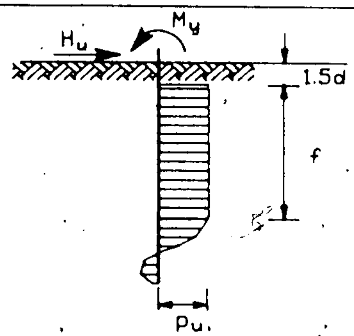
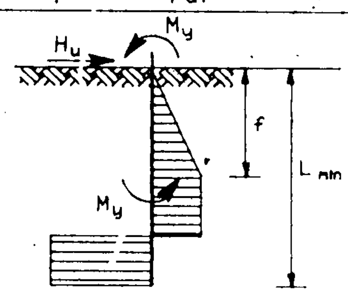
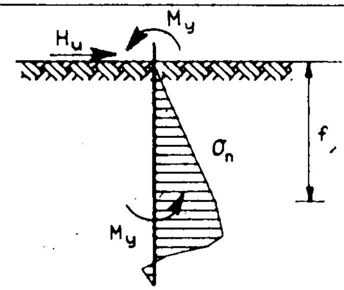
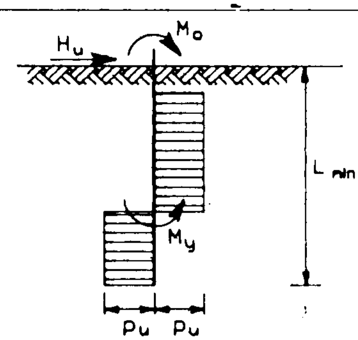
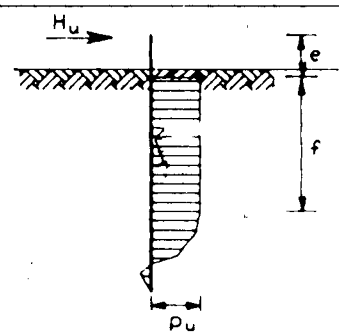
PILE TYPE	SOIL REACTION ASSUMED	ULTIMATE LATERAL LOAD	MAXIMUM MOMENT	REFERENCE
LONG FIXED HEAD		$H_u = 2\sqrt{M_y p_u}$	$L_{min} = \frac{2H_u}{p_u}$	JANBU (1981)
LONG FIXED HEAD		$H_u = \frac{2M_y}{(1.5d + 0.5f)}$ $H_u = p_u d f$		BROMS (1964a)
LONG FIXED HEAD		$H_u = (\frac{9}{2} M_y^2 d\, \sigma_h)^{1/3}$	$f = \left[\frac{2H_u}{d\, \sigma_h}\right]^{1/2}$ or $f = \frac{3M_u}{H_u}$ $L_{min} = 1.5f$	JANBU (1981)

Table 2.1 ULTIMATE LATERAL LOAD FOR SIMPLE REACTION DISTRIBUTIONS (CONTINUED)

6458-213

PILE TYPE	SOIL REACTION ASSUMED	ULTIMATE LATERAL LOAD	MAXIMUM MOMENT	REFERENCE
LONG FIXED HEAD		$H_u = (\frac{9}{2} M_y^2 d\, \sigma_h)^{1/3}$	$f = \left[\frac{2H_u}{d\,\sigma_h}\right]^{1/2}$	BROMS (1964b)
LONG FIXED HEAD		$H_u = \sqrt{2(M_y - M_o)p_u}$	$L_{min} = \frac{H_u}{p_u} + \sqrt{M_y/p_u}$	JANBU (1981)
LONG FIXED HEAD		$H_u = p_u d f$	$M_y = H_u(e + 1.5d + 0.5f)$	BROMS (1964a)

6458-216

Table 2.1 ULTIMATE LATERAL LOAD FOR SIMPLE REACTION DISTRIBUTIONS (CONTINUED)

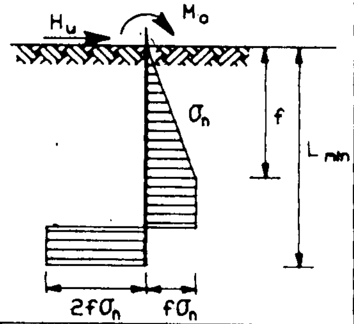
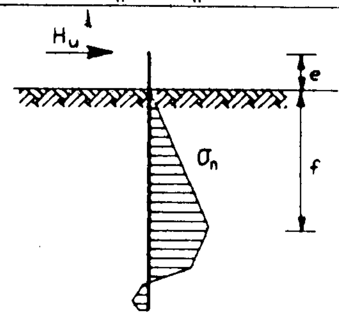
PILE TYPE	SOIL REACTION ASSUMED	ULTIMATE LATERAL LOAD	MAXIMUM MOMENT	REFERENCE
LONG FREE HEAD		$H_u = \left[1.13 d \sigma_n (M_y - M_o)^2\right]^{1/3}$	$L_{min} = f + \left[\frac{3M_y}{f d \sigma_n}\right]^{1/2}$ $f = 1.5 \left(\frac{M_y - M_o}{H_u}\right)$	JANBU (1981)
LONG FREE HEAD		$H_u = 0.5 \sigma_n d f$ $H_u = \frac{M_y}{(e + 0.67f)}$		BROMS (1964b)

Table 2.1 ULTIMATE LATERAL LOAD FOR SIMPLE REACTION DISTRIBUTIONS (CONTINUED)

6458-214

often termed P-Y curves. Programs have been written to model the pile-soil interaction, and can include non-linear and second order effects in both pile and soil behaviour(e.g. Nordal et al,1982).

The common method of determining P-Y curves in North American practice is based on research reported by Matlock, Reese, and their co-workers (Matlock,1970; Reese et al.,1974; Reese et al.,1975; Reese and Welch,1975; Sullivan et al.,1979). P-Y curves have been determined empirically for four types of soils (soft clays, stiff fissured clays above and below the water table, and sands) by laterally loading instrumented test piles. P-Y curves determined by this method are intended to simulate the fact that the soil is a continuous medium. Except for soft clays, the method for each soil type is based on tests at only one site. The method for soft clay is based on results at two sites.

Other methods of determining coefficients of lateral reaction are available. For instance there is a method developed by Grande and Nordal(1979) based on work conducted on equilibrium stress fields around piles. The P-Y curves are determined by solving for an equilibrium stress field using strength and stiffness parameters determined for the soil by laboratory or insitu tests. If the soil for which one is designing a laterally loaded pile is dissimilar from the soils at the sites used to develop the empirical rules, this method may be preferable.

Beneath a gravity structure, piles will act most effectively against lateral loads if they behave as short fixed head piles. The resistance of these piles depends on soil resistance only and they are intended to utilize the available soil strength.

On a sand foundation, the structure can be made heavy enough that the design horizontal load can be resisted by friction. Stub piles are a potential solution to a sliding problem only in clay. Therefore what follows will concentrate on determining the ultimate resistance of a cohesive ($\phi=0$) soil to a laterally loaded pile.

2.3 SOIL RESISTANCE TO A SINGLE PILE

2.3.1 Theoretical Considerations

The ultimate resistance of cohesive soil to a pile moving through it is usually expressed as:

$$p_u=c_uN_pd \qquad [2.1]$$

where p_u is the ultimate resistance resistance per unit length (FL^{-1})

d is the pile width or diameter (L)

c_u is the undrained shear strength (FL^{-2})

N_p is a pile resistance factor

Various theoretical values of N_p have been proposed. Two classes of N_p exist, those calculated assuming the pile is at great depth and those calculated for the near surface condition, where three dimensional effects are important.

2.3.1.1 Resistance at Great Depth

At great depth the soil is asssumed to move only in a horizontal plane, generally being thought to flow around the pile. Most theoretical solutions treat the laterally loaded pile as a strip footing at great depth and then neglect the effects due to gravity. Using slip line theory, a number of resistance factors (N_p) can be determined, depending on variations in shape and surface roughness. A number of cases given by Broms(1964) are shown in Figure 2.2. The values of N_p vary from 8.28 to 12.56.

A slip line solution for circular piles has been obtained by Randolph and Houlsby(1984). Values for N_p are very similar to those obtained for square piles. N_p=9.14 for a smooth pile and 12.56 for a rough pile. These solutions are also summarized on Figure 2.2.

As might be expected Figure 2.2 indicates that the roughness at the pile surface can have a significant effect on the magnitude of the ultimate resistance, up to 35 percent for the diamond shaped pile. Vertical plates provide the greatest resistance, followed, in order, by circular and diamond shaped sections. Unfortunately the plate is not a practical section for a pile.

The theory of the capacity of deep foundations developed by Meyerhof(1951) provides a good basis for understanding how the ultimate capacity of the horizontally loaded pile is developed. Using this theory values of N_p=8.28 for smooth square piles and N_p=8.85 for rough square

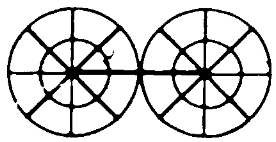
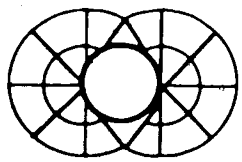
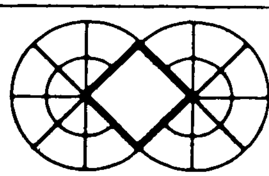
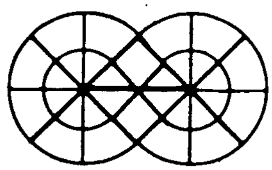
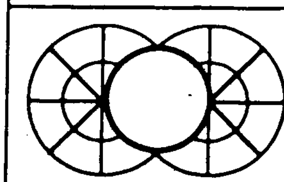
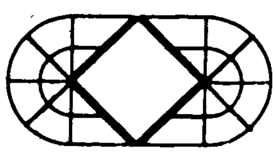
SLIP PATTERN FIELD	SURFACE	RESISTANCE FACTOR N_p
	ROUGH	12.56
	ROUGH	11.94
	ROUGH	11.42
	SMOOTH	11.42
	SMOOTH	9.14
	SMOOTH	8.28

FIGURE 2.2 SLIP LINE SOLUTIONS FOR PILE OF VARIOUS SHAPES (modified from Broms, 1964, and Randolph and Houlsby, 1984)

piles are obtained.

For square piles (Hansen, 1961) calculated a value of 8.12 for N_p Also for a square pile Reese (1958) used a simple block analysis to obtain $N_p = 12$. He later (Reese, 1975) revised his analysis to include wedges, and obtained $N_p = 11$.

An alternative analysis, useful for compressible soils, likens the failure mechanism to the expansion of a cylindrical cavity. For an elastic perfectly plastic material, Meyerhof (1951) obtained a value:

$$N_p = \frac{4}{3}\left(\log_e \frac{E_c}{3c_u} + 1.75\right) \qquad [2.2]$$

For normal values of E/c_u values of N_p range from about 7 to 9.

Randolph and Houlsby (1984) consider suction on the back of the pile, or the possibility of free water in the gap behind the pile. Combining these factors with the cavity expansion solution they bracket the ultimate resistance by:

$$\frac{\sigma_{ho}}{c_u} + 7 < N_p < \frac{\sigma_{ho} + p_a}{c_u} \qquad [2.3]$$

For normally consolidated soils the lower bound is about 9, while for heavily overconsolidated soils the lower bound could be approximately 7.5.

If one wishes to consider non-linearity or sensitivity of the soil, the method described by Ladanyi (1967) can be utilized to give different results for the cavity expansion pressure. Especially for sensitive clays values of N_p will be lower.

In summary, theoretical calculations based on conditions of general shear indicate values of N_p between 8 and 12, while calculations based on cavity expansion, vary from about 6 to almost 9. This is a considerable range and verification based on field and model tests will be discussed in a later section.

2.3.1.2 Resistance Near Surface

Near the soil surface the soil is not constrained to flow horizontally around the pile, but may also move in an upward direction. This leads to lower values of N_p near the surface.

Reese(1958) considered the passive resistance that a wedge of soil the same width as the pile would provide. Included in the resistance is the shear developed on the sides of the wedge. The expression obtained for N_p at depth H from surface for a smooth pile is:

$$N_p = 2 + \frac{\gamma H}{c_u} + 2.8\frac{H}{d} \qquad [2.4]$$

and for a rough pile:

$$N_p = 3 + \frac{\gamma H}{c_u} + 2.8\frac{H}{d} \qquad [2.5]$$

where H is the depth below surface

γ is the soil unit weight (buoyant unit weight for offshore piles)

and other symbols are as before.

Hansen(1961) made a similar analysis for a smooth pile but for the more general case where $\phi>0$. To obtain solutions for a rough pile he introduced terms from the solution for the passive resistance of a rough wall based on log spiral analyses. For smooth piles the expression he obtained reduces to Reese's equation for $\phi=0$, but for rough piles he obtained

$$N_p=2.57+\frac{3.63H}{d}\qquad[2.6]$$

Hansen proposed an empirical formula to "marry" together the "near surface" and the "at depth" expressions. Reese implies a bilinear relationship, where the results of equation 2.4 or 2.5 would govern until they exceed a value of N_p calculated for great depth.

2.3.2 Load Tests

There is some published data on the ultimate capacity of a laterally loaded pile. In general, however, most load tests have concentrated on determining the modulus of soil reaction or pressure-deflection curves (p-y curves) and the piles were not loaded to the failure of the soil. Also most large scale or prototype scale tests were made on long piles and only the upper portion of the piles were deflected sufficiently to cause failure of the soil. Most of the tests reported were performed on free headed piles. This allows the pile section to rotate as well as translate. Thus the conditions in the tests do not exactly match the assumptions upon which the preceding theory was based.

Broms(1964) reports that McKenzie(1955) determined a value of N_p=8 from experiments. Donovan(1959; personal communication,1983) performed model tests and obtained N_p of about 8 for high plastic clay and about 11 for a low plastic clay.

Matlock (1970) made tests with a 12.75 inch pile in soft clays and determined that ultimate capacity at depth was $9c_u$. He also obtained an expression for N_p near the surface in a similar form to that obtained by Reese theoretically:

$$N_p = 3 + \frac{\gamma H}{c_u} + J\frac{H}{d} \qquad [2.7]$$

J is an empirical constant which appears to be specific to each site. Matlock determined values of J to be 0.25 and 0.50 for two sites. These values do not agree with the theoretical derivation (J=2.8) which seems to imply that the vertical failure planes forming the sides of the wedge do not become fully developed. Still, for full sized piles, the maximum value for N_p is reached within 3 to 4 pile diameters.

Reese et al.(1975) performed load tests with 6 inch and 24 inch diameter piles in submerged stiff fissured clays. On the basis of these tests he recommended using theoretical values for N_p, modified by a depth dependent, empirically determined 'A' value. i.e.

$$N_p^* = A(H)N_p \qquad [2.8]$$

where N_p^* is the resistance value to be used,

H is the depth, and

N_p is determined from equation 2.3 near surface, or is equal to 11 at depth

The 'A' value ranges from 0.2 at surface to 0.6 at depth. This leads to a maximum value of N_p of 6.6. However it should be noted that the tests produced ultimate resistance of the soil only for shallow depths.

Reese and Welch (1975) report the results of lateral loading of piles in stiff clay above water. For these tests they found that N_p could be predicted to be:

$$N_p = 3 + \frac{\gamma H}{c_u} + 0.5\frac{H}{d} \qquad [2.7]$$

maximum N_p=9.

Again it should be noted that the ultimate resistance was attained only to a depth of about 1m. Since the pile diameter was 0.760m, N_p at this depth was about 3.8.

Meyerhof(1981) reports model tests where the resistance of free headed piles was about $3c_uLd$. This corresponds to a resistance of $8c_ud$ developing above and below the centre of rotation. He (personal communication, 1983) recommends a value of N_p=8 as a good conservative value.

Janbu(1981) is more conservative. For circular piles N_p=6 is the recommended value. This capacity is developed at depths greater than 10 pile diameters. N_p for square piles can be 30 to 50 percent higher. No data is presented or referenced to substantiate this value for N_p. It may be that the Norwegian experience is influenced by the prevalence of sensitive clays in many parts of the country.

2.3.3 Load-Deflection Response

Although the scope of this thesis is generally concerned with ultimate resistance of piles to lateral loads, the load-deflection response of the piles, especially during post peak behaviour, is important when assessing the potential for progressive failure of these large structures.

A cause for concern in this case would be a load-deflection curve which exhibited a significant drop from peak resistance to residual resistance at large deflections.

Matlock(1970) recommended load-deflection curves for use with soft clays. He fitted an exponential function to the initial part of the load-deflection curve, with a horizontal line defining the peak resistance. No resistance reduction is observed for static loading. Under cyclic loads it is noted that resistance is reduced at large displacements. The drop in resistance at large displacements occurs only near the surface when soil deformation around the pile is three dimensional so that N_p is less than 9.

Reese and Welch(1975) found similar results for a stiff fissured clay. Reese et al.(1974) found no reduction in resistance at large displacement for piles in sand.

Conversely, Reese et al.(1975) recommend load-displacement curves of the same shape as the stress-strain curves of the clay in submerged stiff clays. At depth, residual resistance is predicted to be less than 10% of the peak resistance and at the surface residual resistance is zero. Intuitively, one would not expect such a drop in resistance for a laterally loaded pile. As the pile section moves through the soil it is continually coming up against previously unsheared soil. In fact, the data presented by Reese et al. do not appear to support their proposal for such drastic reductions in resistance. Except very near the surface (within two pile diameters) resistance does not drop significantly. It seems likely that reduction in resistance at larger displacements is only associated with soil that fails by moving in an upward direction.

Theoretical considerations appear to support the view that no significant reductions in resistance are possible. Methods of obtaining load-deflection curves from the theory of cavity expansion developed by Ladanyi(1963,1967) can be used to gain insight into pile load-deflection relationships. Even with a clay of infinite sensitivity (C_r=0) no reduction in the expansion pressure is predicted although the magnitude of the expansion pressure is affected by the post-peak resistance.

Working at the problem from the other direction, Ladanyi(1972) also developed a method of determining soil stress-strain curves from pressure-expansion curves. If one attempts to derive a soil stress-strain curve from a hypothetical pressure-expansion curve that exhibits a drop in resistance, the resulting curve drops below the strain axis. This is obviously an impossible situation.

2.4 SUMMARY

The capacity of a pile loaded in a horizontal direction may be limited by either soil resistance or by the structural capacity of the pile. For the greatest effectiveness in augmenting the resistance of a gravity structure to horizontal loads, the pile will be sized so that the soil resistance is the limiting factor. A quick decision about the size of pile required can be made by refering to the equations of Table 1.1.

Determining the ultimate soil resistance depends on the factor N_p. Theoretical evaluation of N_p can give factors between 7 and 12. Load tests on models indicate that normally consolidated or lightly overconsolidated clays can develop a resistance of $9c_u$. Heavily overconsolidated clays should probably only be counted upon to provide a resistance of $7\text{-}8c_u$ depending upon their brittleness or sensitivity.

3. GRAVITY STRUCTURE FOUNDATION DESIGN

The purpose of this chapter is to examine the stability of a typical proposed structure using conventional bearing capacity analyses. It is assumed that the structure has skirts beneath it in the same manner as existing structures in the North Sea. All analyses assume a basically two dimensional behaviour.

From an overall stability point of view, there are four items which need to be investigated. These are:

1. safety against bearing capacity failure,
2. safety against overturning,
3. safety against sliding,
4. adequacy of the skirt system (related to item 3).

3.1 BEARING CAPACITY

Calculation of bearing capacity is a much studied problem. Conventional theory has its basis in Prandtl's(1921) solution for an indenter on the surface of a perfectly plastic medium. In extending the solution to the conditions encountered in foundation design, semi-empirical and theoretical modifications to the solution have been proposed to account for embedment of the foundation, lateral and eccentric loading conditions, foundation shape, soil compressibilty, plus other factors. e.g. Meyerhof (1951, 1953,1963), Vesic(1973,1975).

3.1.1 Effect of Horizontal Load

It is sometimes overlooked that the horizontal components of applied foundation loads can have a significant effect on the ultimate vertical loads that the foundation is capable of supporting. The influence of horizontal load is illustrated on the stability envelope shown on Figure 3.1. The stability envelope is based on equations recommended by Vesic(1975); other researchers are more conservative.

Curve 1 on Figure 3.1 is for a foundation at mudline with the load applied at that level. That is , there is no eccentricity to the load. At the point of sliding failure it can be seen that the ultimate vertical load is 70 percent of the ultimate load with no horizontal component.

3.1.2 Effect of Foundation Embedment

If the foundation level is extended below the mudline, (perhaps with piles or deeply penetrating skirts) increases in bearing capacity can be attained. However in terms of net load, increases due to depth can be reduced by the effect of load eccentricity because now the footing depth is below the point of load application. The results are illustrated by curve 2 of Figure 3.1. Therefore piles or skirts would be ineffective in improving the safety of the structure against bearing capacity failure.

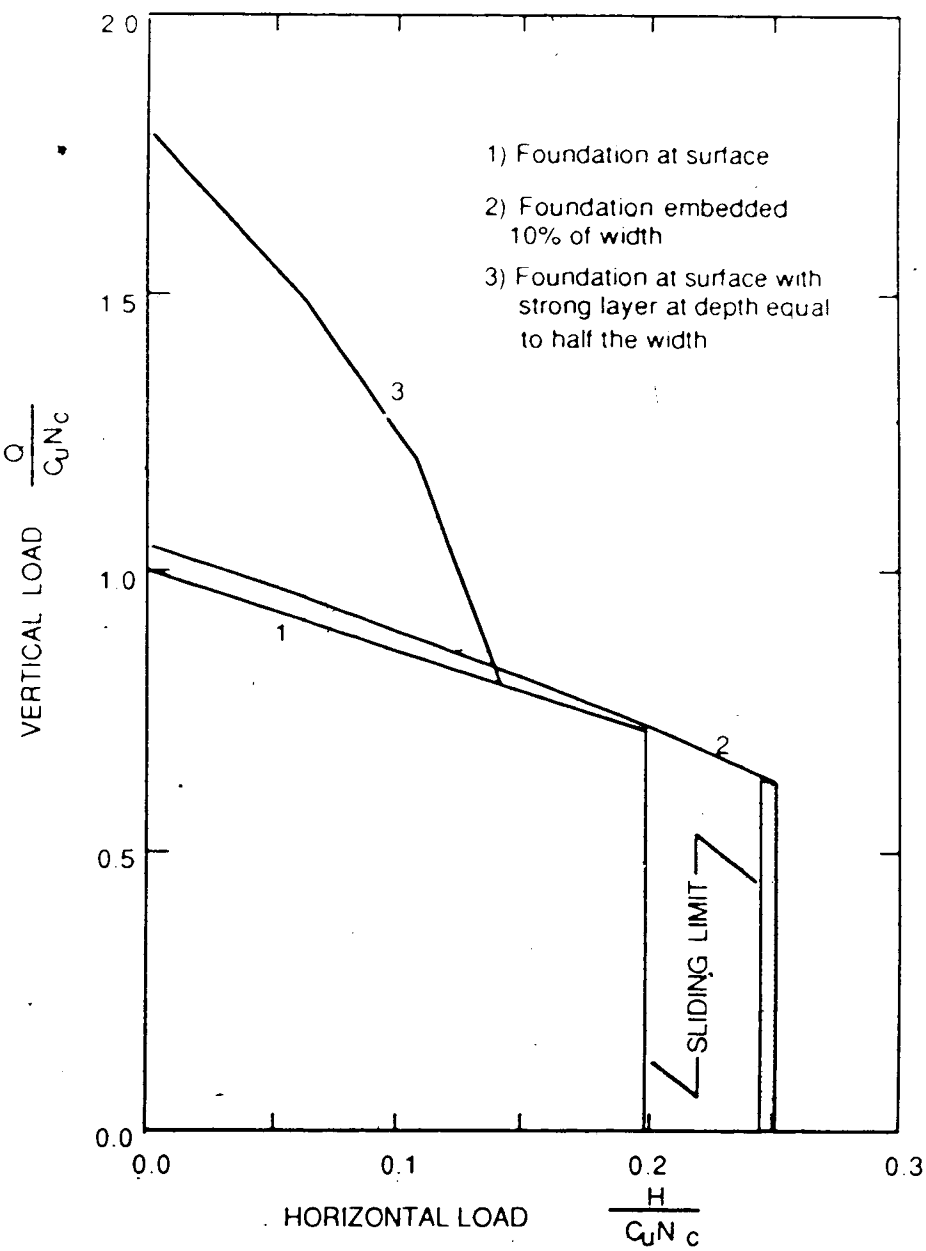


FIGURE 3.1 STABILITY ENVELOPE FOR A GRAVITY STRUCTURE ON COHESIVE SOIL

3.1.3 Effect of Strong Layer at Depth

Because of the large size of the gravity structure it is likely that stronger materials would be encountered within one foundation width (B) below the foundation depth. These materials will increase the safety of the structure against bearing capacity failure.

Solutions for bearing capacity of a soil underlain by a stronger base (Meyerhof and Chaplin, 1953; Brown and Meyerhof, 1969; Meyerhof, 1974) indicate that considerable gains in bearing capacity can be obtained, at least for vertical loads. Because of the size of the potential structures, soils at depths of 50m to possibly 150m would contribute to resistance against bearing capacity failure. Soils at these great depths will almost certainly be much stronger than the surficial soils. In the Beaufort Sea they will probably be ice bonded.

For situations involving horizontal loads it is by no means certain that increases in bearing capacity will be as great. Meyerhof and Hanna (1978) suggest that the following semi-empirical formula originally proposed for layered systems loaded vertically is equally applicable for inclined loads:

$$q_v = q_{v1} + (q_{v2} - q_{v1})\left(1 - \frac{H_1}{H_f}\right)^2 \qquad [3.1]$$

where

q_{v1} is the bearing capacity on a deep layer of the (weaker) uppper stratum,

q_{v2} is the bearing capactiy on a deep layer of the (stronger) lower stratum,

H_f is the depth of soil involved in the failure, and

H_w is the depth of the weak layer.

H_f is said to be equal to the foundation width B for vertical loads in cohesive soils and decreases for inclined loads. Inspection of equation 3.1 shows that if H_f is reduced then increases in bearing capacity due to the presence of a strong layer at depth are also reduced.

Meyerhof and Hanna do not suggest a method of estimating the variation of H_f with load inclination. The following theoretical derivation is offered as a means of investigating possible effects of lateral loads on the bearing capacity of a foundation. Its use has not been experimentally justified.

Meyerhof(1953) presents a solution for bearing capacity of a horizontal strip foundation with inclined load. For a surface footing on clay the solution becomes particularly simple.

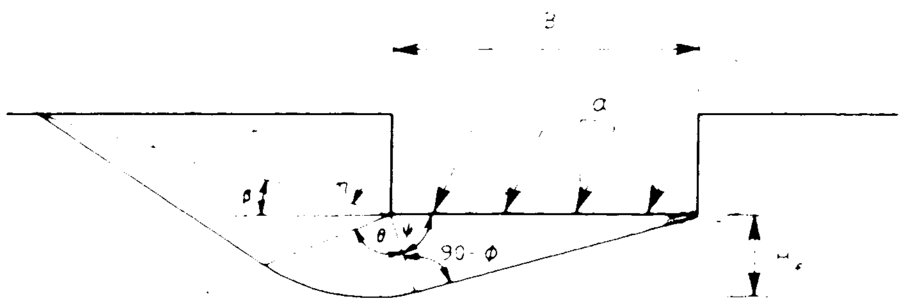
Theoretically, (see Figure 3.2):

$$H_f = B\cos\psi \qquad [3.2]$$

and

$$\tan\alpha = \frac{\cos(2\psi)}{\sin(2\psi) + 3\pi/2 - 2\psi + 1} \qquad [3.3]$$

Equations 3.2 and 3.3 predict that H_f is 0.707B for a vertical load. It has been observed that the actual value of H_f for a vertical load is 1.0B. Possibly, this discrepancy can be related to the true frictional nature of all soil.



FROM MEYERHOF (1953):

$$q'_v = \frac{c + p'_p \tan\phi}{\cos\phi}\left[\sin(2\psi - \phi) + \sin\phi\right] + p'_p$$

$$q'_v = \frac{c + p'_p \tan\phi}{\cos\phi}\cos(2\psi - \phi)\cot\alpha$$

$$p'_p = \left[(c + p_1 \tan\phi)e^{2\theta\tan\phi} - c\right]\cot\phi$$

$$\theta = \pi + \beta - \eta - \psi$$

FOR A SURFACE FOOTING ON CLAY:

$$\phi = 0$$
$$\beta = 0$$
$$\eta = 0$$

THE EQUATIONS BECOME:

$$q'_v = c \cdot \sin(2\psi) + p'_p$$

$$q'_v = -c \cdot \cos(2\psi)\cot\alpha$$

$$\theta = 3\pi/4 + \psi$$

$$p'_p = 2\theta c + c$$

THIS GIVES: $\tan\alpha = \dfrac{-\cos(2\psi)}{\sin(2\psi) + \frac{3\pi}{2} - 2\psi + 1}$, AND

$$\frac{H_f}{B} = \cos\psi$$

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FIGURE 3.2 DETERMINATION OF THE RELATIONSHIP BETWEEN H_f/B AND LOAD INCLINATION (α)

(The theory does predict that H_t is greater for soil with $\phi>0$.) To adjust the theory to observed behaviour it is proposed that equation 3.2 become:

$$H_t = B\cos\psi \qquad [3.4]$$

The results of equations 3.3 and 3.4 combined lead to the relationship between load inclination α and H_t shown on Figure 3.3.

Figure 3.3 shows that H_t becomes 0 and failure occurs by pure sliding when α=21.6. This is consistent with the Vesic load inclination factor for strips which implies a maximum load inclination of α=17.6

The third curve on Figure 3.1 shows the variation of the ultimate vertical load where there is a stronger stratum with an undrained strength four times that of the surface material at a depth of B/2. The figure indicates that although significant increases in bearing capacity are attained for nearly vertical loads, there is little or no increase in vertical bearing capacity if the horizontal load is large.

3.2 OVERTURNING

Because proposed structures have wide bases and the water depths on the Beaufort Shelf are relatively shallow, the load is applied relatively close to foundation level and overturning is not likely to be a problem. If the resultant vertical force on the foundation falls within the middle third of the base, this is a sufficient condition for the

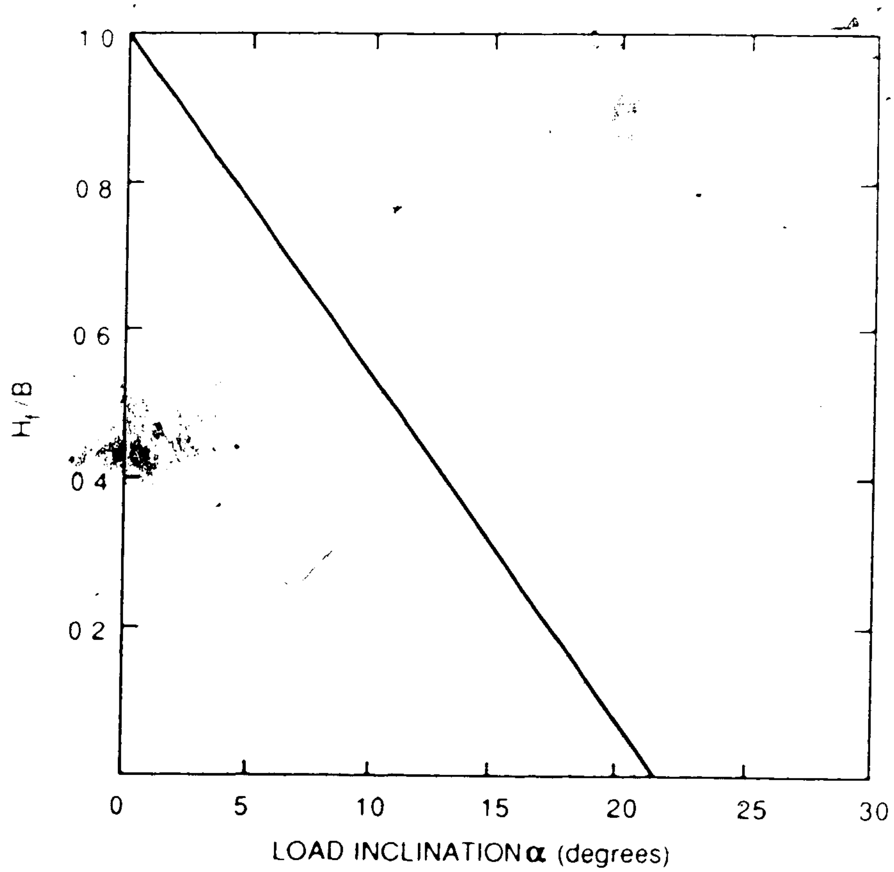


FIGURE 3.3 RELATIONSHIP BETWEEN LOAD INCLINATION AND H_f/B FOR SURFACE FOOTINGS ON CLAY

prevention of overturning. It can be shown that in the very worst case the corresponding factor of safety against overturning about the toe is 1.5.

3.3 SLIDING

Design for sliding is generally very straight forward. In cohesive soils, if complete base contact can be guaranteed, resistance against sliding is estimated to be:

$$R_u = c_u A \qquad [3.5]$$

The problem of design against sliding becomes one of ensuring that the full contact area A is mobilized. The conventional method of ensuring this is with 'skirts'. These are steel or concrete projections on the base of the structure which are supposed to be placed together close enough so that shear failure will occur below their tips.

If the base of the foundation is below the mudline, then additional resistance to sliding can be obtained from the passive resistance of the soil above foundation elevation. For a D_f/B ratio of 0.1 this additional resistance could vary from 20 to 50 percent or more for a very large structure on very soft soil. The sliding failure portion of the stability envelope of Figure 3.1 is based on a soil with c_u=100kPa and γ'=10kN/m^3. The variation shown is for structures widths of 100m and 200m. The increase in sliding resistance over the surface foundation for these cases averages about 25 percent.

Theoretically a foundation placed at depth would also mobilize shearing along the sides of the foundation unit. The magnitude of this resistance depends on the adhesion between the soil and the structure. Traditionally it has been ignored in design.

3.4 DESIGN OF SKIRTS

Young et al.(1975) review possible sliding failure modes for structures with skirts. These are shown on Figure 3.4.

In uniform soils modes a, b, and c apply. The spacing and depth of the skirts are chosen to ensure that mode c, a sliding base failure, will occur.

Murff and Miller(1979) analyse mechanism a using an upper bound method. They determine that the force F_x required to cause failure of each shear key (skirt) as:

$$F_x = 2c_u \sqrt{\frac{F_z h}{c_u} + h^2} \qquad [3.6]$$

where

F_x is the horizontal force exerted by the skirt on the soil wedge,

F_z is the vertical force applied to the wedge as a result the structure's weight,

h is the depth of the skirt, and,

c_u is the undrained strength of the clay

Mechanism b (deep passive failure) will generate a very large resistance. For perfectly smooth skirts and base, the

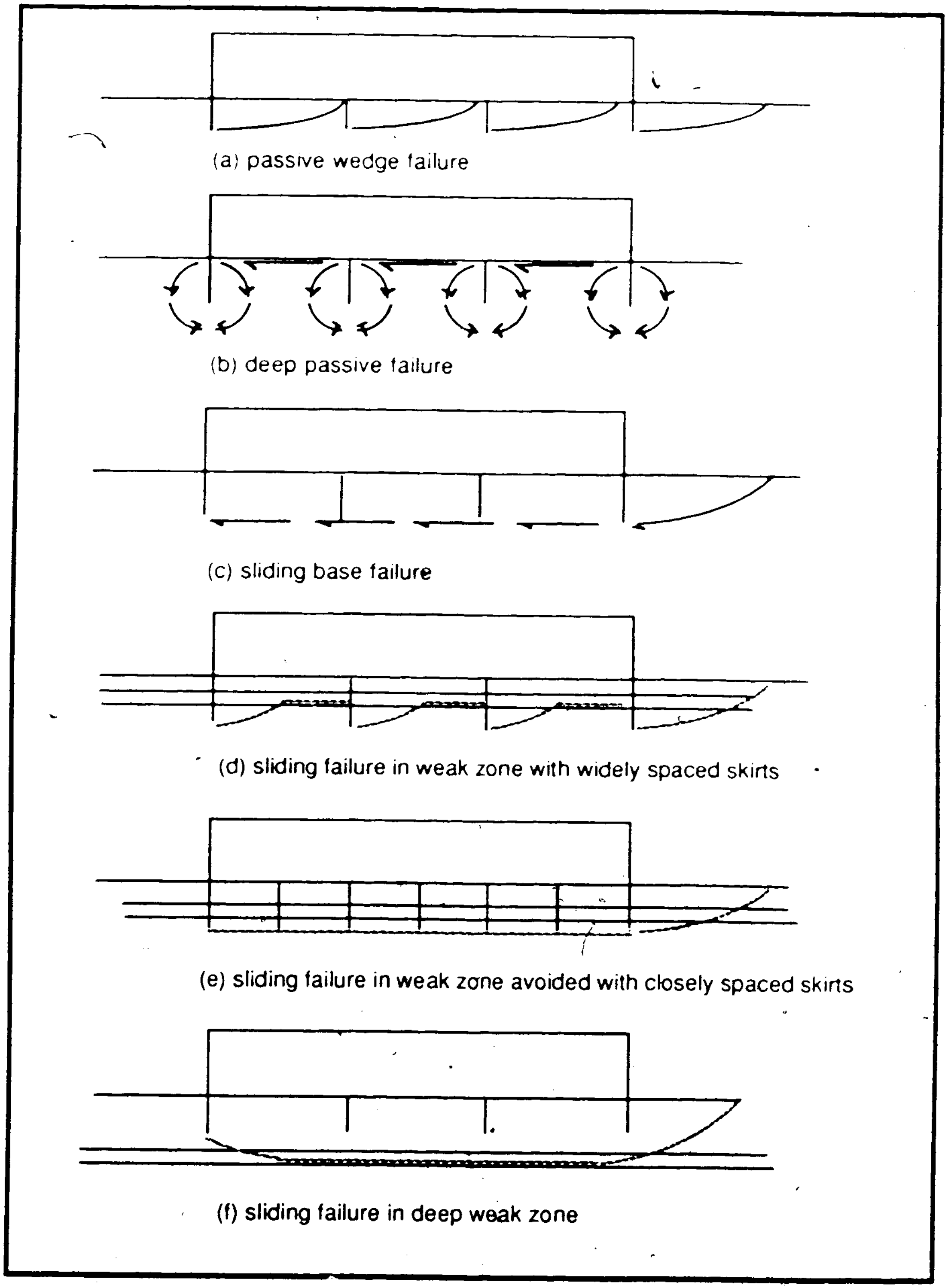


FIGURE 3.4 SCHEMATIC ILLUSTRATION OF SOME POSSIBLE FAILURE MODES FOR SLIDING RESISTANCE
(adapted from Young et al. ,1975)

resistance developed is $9.5c_uh$ per skirt.

For sliding beneath the tips (mechanism c) the resistance r_u developed will be equal to:

$$r_u = c_u L \qquad [3.7]$$

Figure 3.5 compares the resistances predicted by each of the above expressions. Mechanism b becomes critical only for unlikely combinations involving high foundation (vertical) pressures and widely spaced skirts.

Typical depths of skirts are reported to be 1m to 4.5m (Young et al., 1975; Andersen et al., 1979). At their deepest and with a small structure (B=100m), considering the passive resistance would only increase the total sliding resistance by about 10 percent. This small resistance increase is not considered in design.

3.5 POSSIBLE BENEFITS OF PILES

From the previous discussion on sliding it can be seen that the maximum attainable resistance is limited to the product of the shear strength of the soil at the pile or skirt tip and the base area. To this resistance may be added a component due to the passive pressure along the front of the structure.

It may be possible to install piles beneath a structure in a way that they will force the shear plane beneath their tips the same way that skirts do. This would appear to be the maximum increase possible. Because piles would be installed to greater depths development of the passive

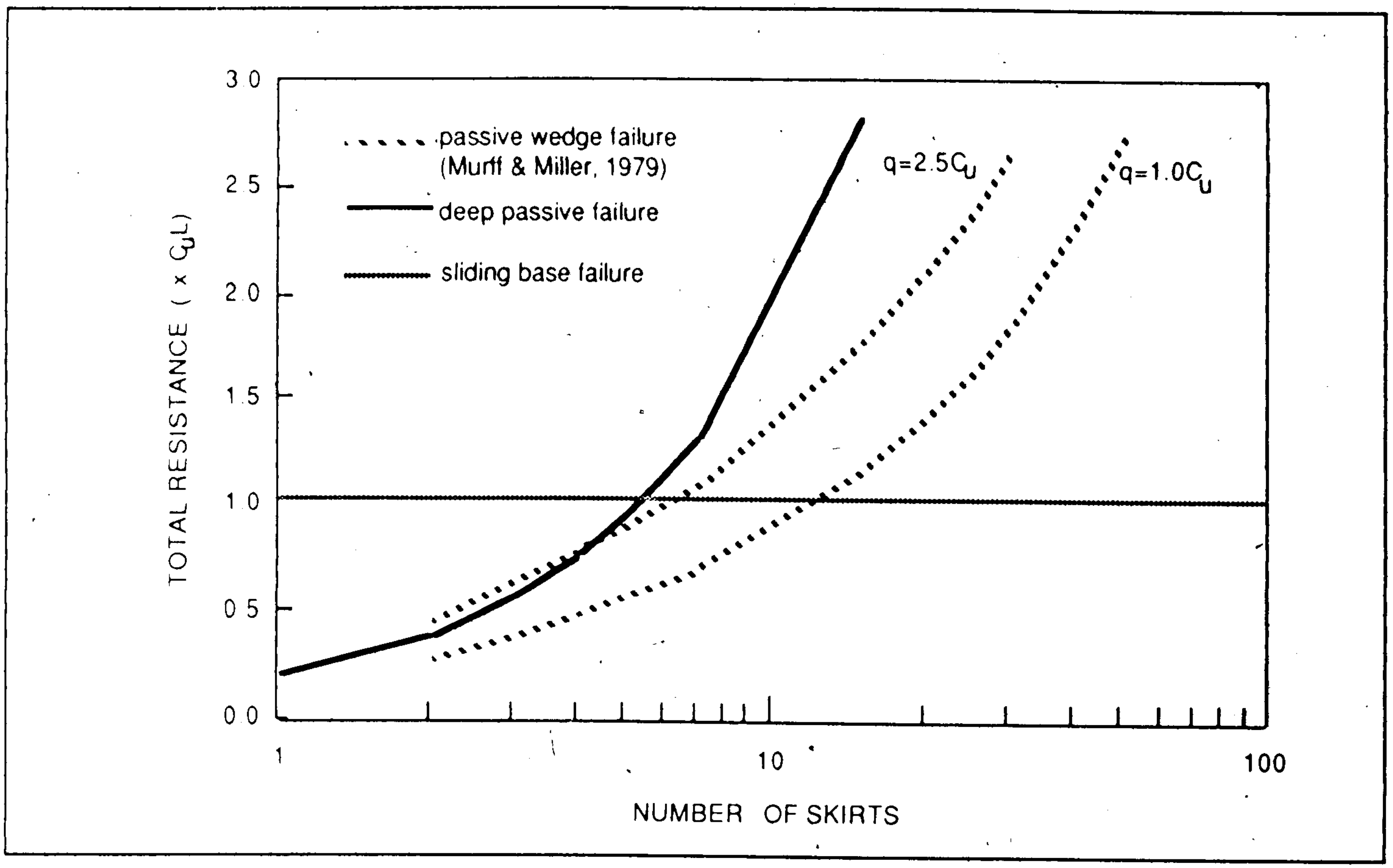


FIGURE 3.5 COMPARISON OF ULTIMATE SLIDING RESISTANCE PREDICTED BY VARIOUS FAILURE MECHANISMS
skirt depth 2% of structure width (L)

resistance along the front of the foundation could be a factor utilized in design. If mobilization could be ensured then shear along the side of the block of piles could also be taken into account.

It is envisioned that the spud piles used in this concept will typically be approximately 15m long. For structures with widths of 100m to 200m, the typical increase in sliding resistance is about 25 percent in uniform soils. Bearing capacity would be completely unaffected. Only in a very rare situation would this small increase justify the expense of installing the piles.

Piles may be more effective in areas where an increase in strength with depth is encountered. An example would be a deep profile of normally consolidated clay. Another situation in which piles could be used is a site where a weak layer exists. Piles may be able to be used to force the failure plane beneath their tips into more competent soil as is illustrated on Figure 3.5d.

To establish the effectiveness of these design possibilities it is necessary to establish whether it is possible to develop a block type failure of a group of piles and if the expected resistance will be attained. This will be evaluated in the next chapter.

4. ULTIMATE LATERAL RESISTANCE OF PILES IN GROUPS

In this chapter the ultimate load behaviour of piles in groups will be reviewed. It is important to try to determine at what spacings piles are unaffected by adjacent piles in the group, the magnitude of interference at closer spacings, and whether any change of failure mode (e.g. individual pile action vs. block failure) takes place. As was the case for the discussion of single pile resistance, this discussion will be generally limited to pile groups in cohesive soils.

4.1 MODERN DESIGN PRACTICE

The method described by Focht and Koch(1973) is probably the most commonly used procedure of analysing pile groups subject to lateral loads. The method involves determining the deflection of a pile in a group by adding the deflection calculated by the p-y curve method for a single pile and the additional deflection for the group calculated by elastic methods described by Poulos(1971).

When the elasto-plastic group deflection is found the most heavily loaded pile is reanalysed with a p-y curve modified so that the deflection of the single pile and the group match. Generally the y-axis is stretched. The 'p' value is also reduced if it is felt that some piles are in a "shadow zone" of another pile. A rational method for choosing a 'p' factor has not been given. Currently designers are tentatively using factors between 0.7 and 1.0.

Variations of the method exist (e.g. O'Neill, 1983) but none appear to rationally treat the problem of a reduction in lateral resistance. There seems to be general acceptance that when piles are in line with the load, reduction of resistance will occur, but when they are transversely oriented no variation of resistance occurs.

4.2 THEORY AND PRACTICAL RESULTS FROM RELATED PROBLEMS

4.2.1 Adjacent Strips on Clay

It was first decided to compare a row of piles to adjacent strip footings. The limits of their behaviour indicate efficiencies of 1.0 for both very close and very distant spacings.

The resistance of n very widely spaced footings of width B is n times the bearing capacity of a single footing:

$$n \times (c_u N_c B) = n c_u N_c B \qquad [4.1]$$

At the other limit of the problem, the footings are brought so close as to be a contiguous single footing of width nB. The resistance of the footing is:

$$c_u N_c (nB) = n c_u N_c B \qquad [4.2]$$

Thus the resistance is the same whether the footings are very widely spaced or immediately adjacent. It may be postulated that footings that are very close but not touching may involve the unloaded spaces between them in the failure zone and thus increase the total resistance.

Analyses carried out by Stuart(1962), Mandel(1965) and West and Stuart(1965) show that this effect is negligible in $\phi=0$ material, although it may be important in sands.

This suggests that the ultimate resistance of a single row of piles should be independent of spacing.

4.2.2 Group Action: Vertically Loaded Piles

It was thought that it might be helpful to the understanding of laterally loaded piles to review group action in vertically loaded piles.

Although efficiency formulas based on spacing exist (for instance the Converse-Labarre formula, Chellis,1962), Kezdi(1975) dismisses them as "not resting on a sound basis" and recommends the method of Terzaghi and Peck(1967). They (Terzaghi and Peck) assume that for close spacings the piles and mass of soil enclosed in the group will act as a block and that the capacity of the group may be determined by considering bearing capacity at the base of the group, plus shear on the sides of the block. If this exceeds that calculated by taking the product of the single pile capacity and the number of piles, then the latter is used.

This model of behaviour is supported by tests by Whitaker(1957) although agreement in calculated capacities is not always particularly good. A definite change in failure mode was observed in his tests. At close spacings block failure occurred, and at greater spacings each pile appeared to act as a single pile. The spacing at which this change in mode occurred was affected by group size; groups containing more piles behaved as a block to larger spacings between the piles. This is predicted by the Terzaghi and Peck method. Whitaker found that for spacings at which individual pile failure occurred, efficiencies were still below unity. It appears that this could be explained because the piles tended to fail progressively, with outside piles in the group failing first. Efficiencies were reduced as the number of piles in the group increased.

It is thought that this concept of block vs. individual pile failure should be a useful concept in explaining and predicting laterally loaded pile behaviour. At close spacing the pile group should behave as a large caisson or large diameter single equivalent pile. This implies that the total resistance of the group can be calculated by summing shear resistance over the base and sides of the equivalent caisson together with passive resistance over the frontal area. Tensile forces acting over the rear of the caisson are likey to be negligible. At larger spacings the piles should fail individually, and the group efficiency should be near unity.

4.3 EXPERIMENTS AND THEORY WITH PILES IN GROUPS

4.3.1 Piles in Rows

4.3.1.1 Theory and Experiments of Matsui et al.

Ito and Matsui(1975) developed an analysis to predict the force exerted on a row of piles used to stabilize landslides. They assume a sliding surface as shown in Figure 4.1. Rankine active pressure is assumed to act on the soil between the piles. Certain other assumptions are made about the direction of principal stresses in the soil mass.

They obtain, for a purely cohesive soil, the following expression for the force per pile (per unit length):

$$p=c\{D_1(3\log\frac{D_1}{D_2} + \frac{D_1 - D_2}{D_2}\tan\frac{\pi}{8}) - 2(D_1 D_2)\} + \gamma z(D_1 - D_2) \qquad [4.3]$$

where D_1 = centre to centre distance between piles, and

D_2 = clear spacing between piles.

In their discussion of this paper, De Beer and Carpentier(1977) took issue with some of the assumptions of the direction of the principal stresses and obtained the following modified expression:

$$p=c\{D_1(2\log_e\frac{D_1}{D_2} + \frac{D_1 - D_2}{D_2}\tan\frac{\pi}{8}) - \frac{3}{2}(D_1 D_2)\} + \gamma z(D_1 - D_2) \qquad [4.4]$$

The difference is not great. The relationship between p and D2/D1 for each expression are shown on Figure 4.2.

De Beer and Carpentier also point out that there are limits to the applicability of these expressions. At large

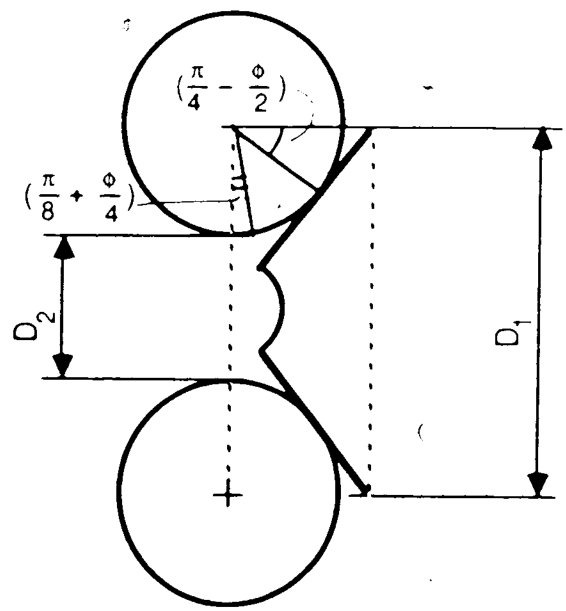


FIGURE 4.1 FAILURE PATTERN FOR A ROW OF PILES ASSUMED BY ITO & MATSUI (1975)

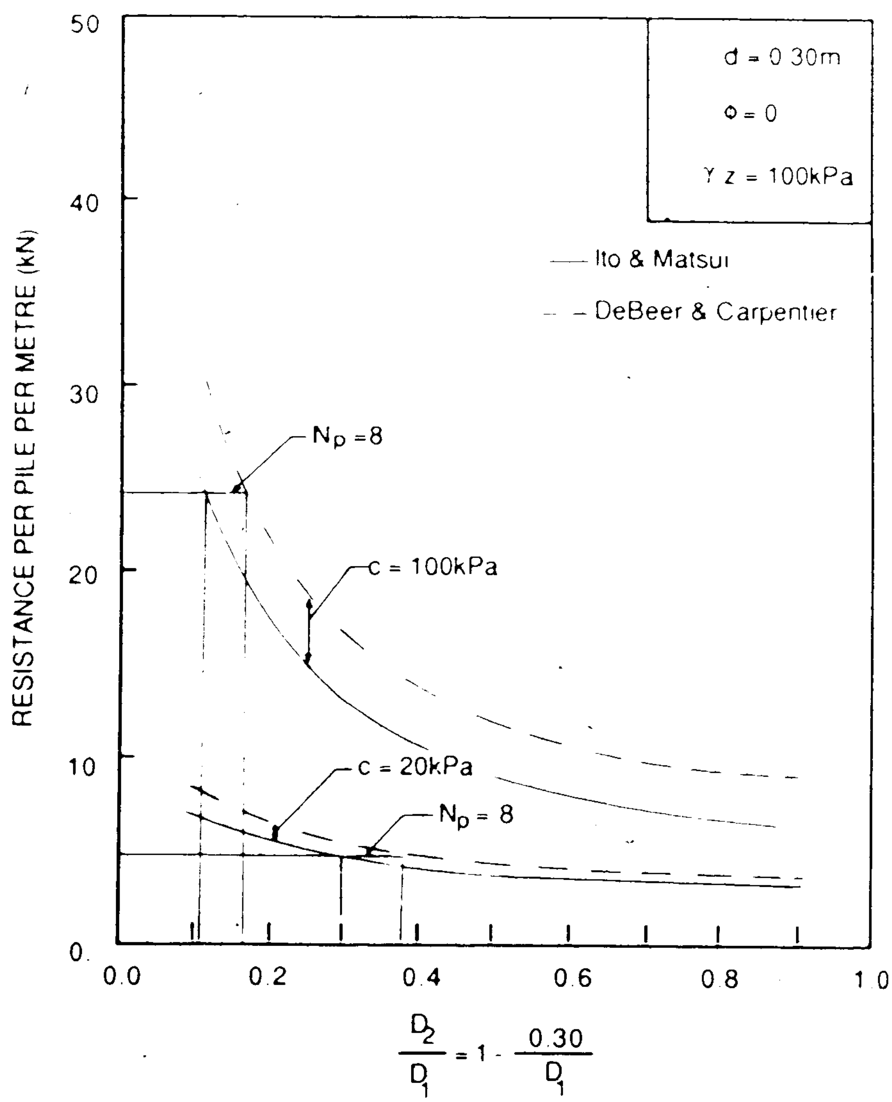


FIGURE 4.2 COMPARISON OF THE RESISTANCE OF A LINE OF PILE AS CALCULATED BY ITO & MATSUI (1975) AND BY DEBEER & CARPENTIER (1977)

spacings the row of piles should each act as a single pile, and the pressure should be $c_u N_p$. At very close spacings the row of piles should behave as a wall and the load would be limited by passive pressure theory. They recommend as a range of applicability centre to centre spacings from 3d to 5d (D2/D1 between 0.66 and 0.8).

Referring to Figure 4.2, it can be seen that predicted forces on a pile drop below $8c_u$ at centre to centre spacings of 1.1d to 1.6d (D2/D1 between 0.1 and 0.38) depending on the value of c_u and the theory used.

Matsui et al.(1982) performed model tests to check the theories. They pushed a mass of soil through a row of piles. The soil was confined in all directions. They plotted the resulting load deflection data semi-logarithmically with displacement on the log scale. They found the data plotted in a bilinear fashion and defined failure at the intersection of the two straight line portions. Failure load defined in this way was found to be in excellent agreement with the theory of Ito and Matsui. The ultimate load measured in the test was usually approximately 60 percent higher than the failure load.

Broms(1983), in a discussion of Matsui et al., proposed a simpler analysis which effectively predicted the ultimate resistance of the row of model piles in clay. His expression, which he determined from shallow foundation theory is:

$$p=dc_u(5.14 + 2\alpha\frac{D}{D_1}) \qquad [4.5]$$

where α is the cohesion coefficient and,

all other variables are as before.

This theory is especially attractive because it gives (when α=1.) a value of N_p=7.14 when the piles are widely spaced.

4.3.1.2 Experiments of Cox et al.(1984)

Cox et al.(1984) have performed experiments with rows of three and five stub piles. The groups were tested in-line (load parallel to axis of the group) and side by side (load perpendicular to the axis of the group). The clay used was very soft, it had an undrained shear strength of 2 kPa. The pile length varied from 2 to 8 pile diameters.

In contrast to the results obtained by Matsui et al., it was found the that group efficiencies for the side by side groups was less than one. The minimum efficiency for piles spaced 1.5d centre to centre was approximately 0.76. The tests indicated that full efficiencies are obtained with spacings of between three and four diameters centre to centre. This result was independent of whether there were three or five piles in the group.

In-line groups exhibited efficiencies from 0.54 for centre to centre spacings of 1.5d to 0.95 at a centre centre spacing of 6.0d. Extrapolation of the data indicates that

100% efficiency would be obtained for a centre to centre of between 9 and 10. It was observed that the efficiency of the five pile groups was lower than that of the three pile group at the same spacing.

4.3.2 Tests of Model Piles in Groups

4.3.2.1 Tests of Donovan (1957)

Broms(1964) reports tests by Donovan indicating that when piles were spaced at less than two diameters centre to centre the soil and the piles behaved as a block and that there was no reduction in resistance at spacings greater than four pile diameters. Donovan(personal communication, 1983) described these tests as being performed with a direct shear device, using 1/4 inch wooden dowels. Tests were performed at integral spacings in both a highly plastic and a slightly plastic soil. It is probable that only a 2x2 group was used.

4.3.2.2 Tests of Prakash and Saran(1967)

Prakash and Saran performed model tests on aluminum piles 9mm in diameter and 290mm in length. The relative stiffness of these piles was intermediate between a rigid pile and a flexible pile. Four and nine pile square groups were tested. The piles were tested with a rigid pile cap, which was permitteed to rotate during the test. Tests were performed at centre to centre spacings of 3, 4, and 5 pile diameters. Deflections were limited to 6mm at the pile cap, which was a deflection that appeared to not quite mobilize the ultimate resistance of the pile group.

The authors interpret their results to show that resistance becomes independent of spacing at 5 to 6 pile diameters. However resistance per pile at equal deflections is only about 60% of single pile resistance for 2x2 groups and 45% for 3x3 groups. What the results actually appear to show is that interference is substantial at the pile spacings tested, and that extrapolation of the data indicates that interference will remain significant to very large pile spacings.

<u>4.3.2.3 Tests of Meyerhof(1981)</u>

Meyerhof tested groups of two piles (2x1 in the direction of the load) and 4 piles (2x2), both free headed and with a rigid pile cap. The piles were roughened steel tubes 13mm in diameter and were up to 300mm long. Centre to centre spacing was 40mm.

Meyerhof found that group efficiency was roughly unity, except for the 2x2 groups with a pile cap, where block failure governed the load.

4.3.3 Field Tests

Bogard and Matlock(1983) suggest a new method of analysing circular offshore pile groups. In it they introduce the concept of an imaginary pile which takes on the outside dimensions of the group. They develop p-y curves based on the imaginary pile and superimpose deflections of individual piles to obtain the p-y curve of an individual pile on the context of the group.

Depending on spacing the ultimate capacity is limited by the ultimate capacity of the imaginary pile or by the ultimate capacity of the single piles. This design procedure is utilizing the previously discussed concept of block vs. individual pile failure.

The design procedure is compared to tests intermediate between laboratory models and field installations: 6 inch steel pipe piles were used in circular groups containing 5 or 10 piles. Good agreement was obtained between measured and predicted deflections and bending moments in the pile.

4.3.4 Analytical Results

Yegian and Wright(1973) performed a two-dimensional finite element analysis of a laterally loaded cylindrical pile section in a hyperbolic model soil. They first calculated the p-y curve of a single pile and then the p-y curves for single rows or files of piles.

For a group of two piles, spaced two diameters apart, when loaded perpendicular to the group axis the efficiency was calculated to be 90 percent, while when loaded parallel to the group axis efficiency was about 70 percent.

An infinite row spaced at 3 pile diameters centre to centre was also analysed. In this case it was found that a row loaded perpendicular to the group axis would resist load with an efficiency of 1.0, while the group loaded parallel to the group axis had an efficiency of about 50 percent.

4.4 SUMMARY

Pertinent results of this review are presented in Table 4.1. The table shows that there is some disagreement about how pile groups behave under lateral load. Some researchers have obtained results that indicate that at sufficiently close spacings that a pile group will fail as a block and at larger spacings have efficiencies close to unity (eg. Donovan, Meyerhof, Bogard and Matlock) while some other researchers obtained results which showed that group efficiencies are much less than unity for much larger spacings (eg. Prakash and Saran).

Ito, Matsui and their co-workers have developed a theory, which they have substantiated with model tests, that predicts that the group efficiencies of a row of piles will increase with decreased spacing, the efficiencies being greater than unity. This analysis is for a two dimensional stress state.

Yegian and Wright give finite element analysis results, also for two dimensional behaviour which shows that the group efficiency of a row of piles is lower than one. There is broad agreement between their work and the tests of Cox et al., except that those tests allowed three dimensional behaviour.

As for a single file of piles the analytical results of Yegian and Wright again agree with the test results of Cox

Table 4.1 Summary of Literature Review for Laterally Loaded Pile Groups

Type of Problem	Findings	Reference
Adjacent strip footings	Bearing Capacity on undrained clays is virtually unaffected by adjacent footings	Stuart(1962), Mandel(1965), West&Stuart(1965)
Vertically loaded Terzaghi&Peck(1967)	differentiate between block failures pile groups at close spacings and individual pile failure at more distant spacings. this is the accepted design method	Kezdi(1975)
	concept generally supported by tests by Whitaker	Whitaker(1957)
Piles in rows (side by side) (plane strain)	conflicting results: Ito&Matsui developed theory which predicts increase in capacity over single pile, supported by their experiments. finite element analyses suggest decrease in capacty relative to single pile	Ito&Matsui(1975) DeBeer& Carpentier(1977) Matsui et al (1982) Broms(1983)
Yegian&Wright(1973)		
(3D action)	group capacity less than single pile efficiency as low as .76, full efficiency attained at spacings of 3-4 c/c.	Cox et al. (1984)
Piles in rows Yegian&Wright(1973) (in line) (plane strain)	finite element analyses suggest de- crease in capacity relative to single pile. lower efficiency than for side by side rows	
(3D action)	group capacity less than single pile, efficiency as low as 0.54, full efficiency attained at spacings of approx. 10c/c.	Cox et al. (1984)
Laterally loaded groups (models)	conflicting results: 2x2 groups fail individually at full efficiency at spacings greater than 3-4 block behaviour under 2 dia. c/c. square groups of 4 or 9 pile exhibit maximum efficiency of .60 and .45 respectively at 5 dia. c/c. Increase with spacing small indicating that interference extends to large spacings.	Donovan(1957) Meyerhof(1981) Donovan(1957) Prakash& Saran(1967)
(field size tests)	circular groups behave as individual piles or an equivalent pile depending on spacing	Bogard& Matlock(1983)

et al. and indicate severe declines in the ultimate resistance. Meyerhof on the other hand has results for a group of two piles loaded in line for which reduction in ultimate load was negligible.

5. DESCRIPTION OF TEST PROGRAM

5.1 THE NEED FOR A TEST PROGRAM

As can be seen from Chapter 4 there exists much disagreement about the behavior of laterally loaded pile groups. As well there have been no tests which can be considered to have modelled the conditions under which stub piles under a gravity structure will be expected to operate.

To adequately design a system to stabilize a gravity structure against a horizontal load it is important to know the following:

1. is it possible to obtain a block failure around the piles and will the resistance be equivalent to what would be expected if the base of the foundation were at the base of the piles? (i.e. Will the full shear strength on the base plus passive pressure on the front be mobilized?).
2. what spacing of piles will create a block failure?
3. what will be the group efficiency at other pile spacings?

The literature search did not adequately answer these questions. The results reported in previous studies were not consistent in all cases. Additionally the situation of a group of short rigid piles has not been adequately examined in the past. Because of this gap in the literature a model test program was decided upon.

5.2 OBJECTIVES OF THE MODEL TEST PROGRAM

In general, the objectives of the model tests are to provide the required information identified in the previous section. However, the scope was made more specific than that.

The specific objectives were to,

1. obtain efficiency factors for groups of short rigid piles under conditions similar to being under a gravity structure, and to see how they varied with group size and pile spacing,
2. to observe the mode of soil deformation when the groups of piles were loaded horizontally in order to gain insight into the failure mechanisms.

5.3 DESCRIPTION OF THE TESTING APPARATUS

The objective in designing the testing apparatus was to enable a pile group to be translated in a horizontal direction while the heads of the piles are fixed against rotation. It was also felt desirable to be able to apply a vertical pressure to the surface of the soil in order to promote a plane strain mode of failure rather than a three dimensional mode. A condition of vertical restraint would occur beneath a gravity structure.

A preliminary test on a very small scale model showed that the deformation around a group extended 3 group widths ahead of the group and two widths to the side. Prakash(1962) recommends that the wall of the bin should be 8 to 12 pile diameters ahead of the pile and 3 to 4 times the diameter of the pile to the side, irrespective of the size of the group.

The model piles were 15.8mm (5/8") diameter cold rolled seamless steel tubing, installed to a depth of 170mm. Considering these criteria it was decided that a bin 1m square would be sufficient to test a group having a maximum size of 200mm. A bin size of 1m by 1m by 500mm deep was chosen.

Essentially the apparatus is a large direct shear device. The schematic diagram of Figure 5.1 illustrates how the load is applied. The bin of soil is caused to move horizontally on linear bearings. The piles are prevented from moving with the box or rotating by the pile cap. Consequently, the piles are forced through the soil. The load required to prevent the piles from moving is measured at the pile cap. The displacement of the bin of soil, which is the same as the displacement of the piles through the soil, is measured by a pair of LVDT's.

The pile cap assembly, which is illustrated by two photographs on Plate 5.1, was consructed to slide along two parallel hardened steel rods on Thomson bushing bearings. The inner portion of the pile cap was originally designed to slide in a vertical direction with a hardened steel rod in a brass bushing. This feature was removed before the tests were begun, for reasons discussed in the following paragraph. Upper and lower removable templates held the piles in place. A selection of templates were fabricated so

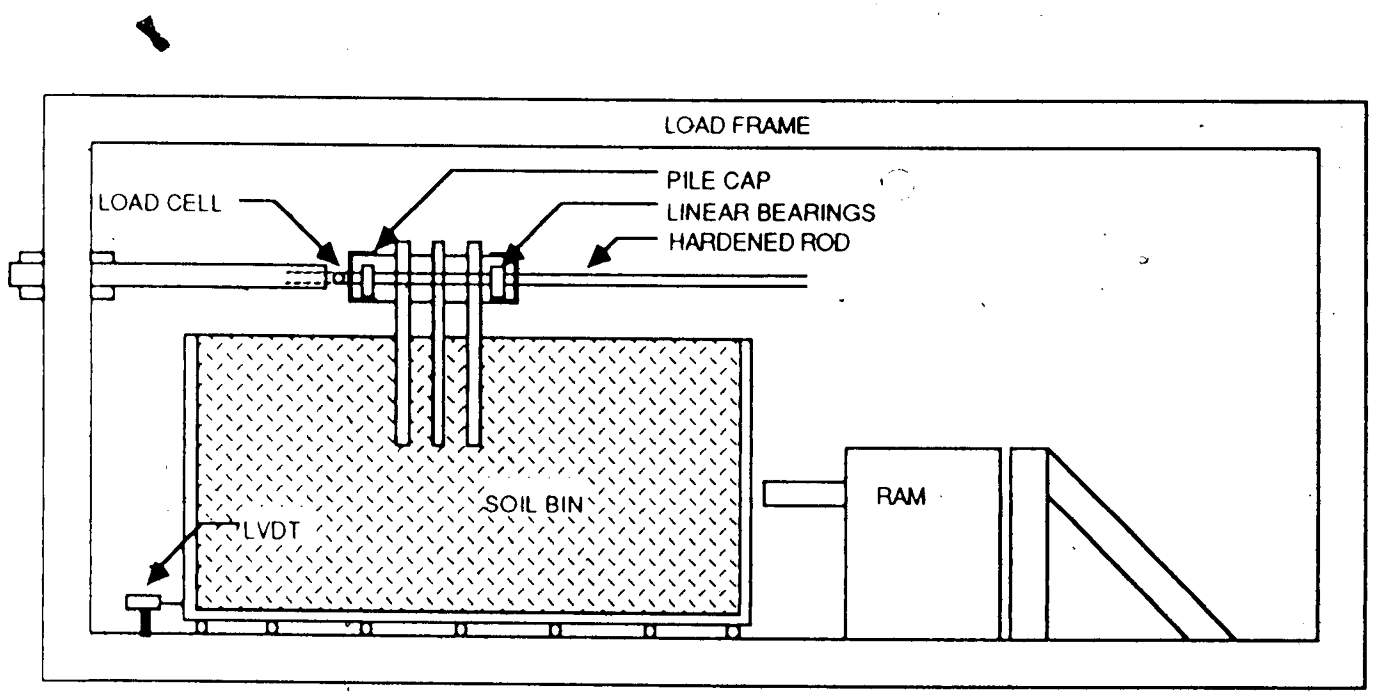
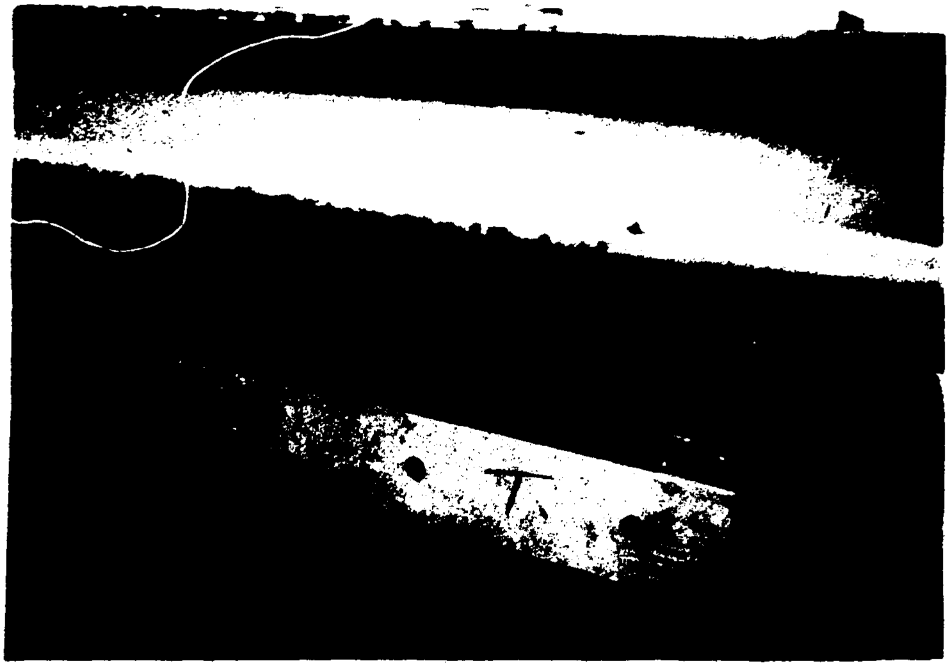


FIGURE 5.1 SCHEMATIC DIAGRAM OF THE TESTING APPARARATUS



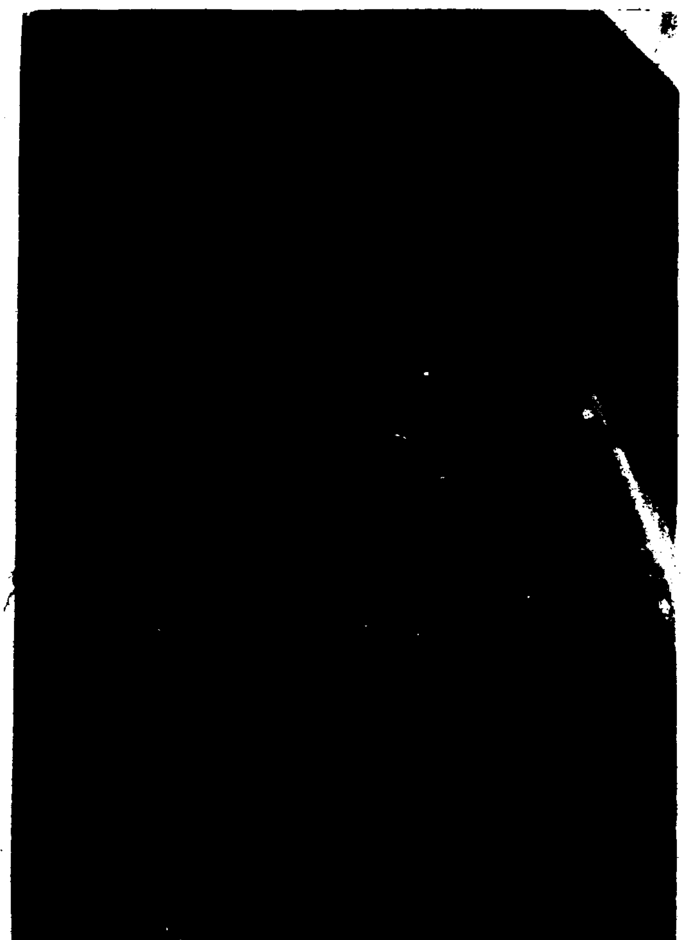


Plate 5.1 Two photographs showing the pile cap assembly

that each group size and spacing could be tested. The cap is restrained by a bar connected to a fixed portion of the apparatus. The bar contains a load cell so that the load required to force the piles through the soil is measured

The original design of the pile cap apparatus allowed its inner portion to move in the vertical direction. It was planned to have the lower template rest on the soil surface. By applying a load to the top of the inner section of the pile cap, the soil surface around the piles was to be loaded. In this way the vertical stress conditions beneath a gravity structure were to be modelled.

It was found however that the pile cap could rotate a small amount. This led to a substantial variation in the resistance generated by the bottom template sliding on the surface of the soil. The apparatus was modified to raise the pile cap off the surface of the soil. Three dimensional deformation was therefore allowed to occur in the stressed soil.

5.4 SOIL

Athabasca Clay was used in the experiments. This soil is classified CL under the Unified Soil Classification System. It has a Plastic Limit of 21% and a Plasticity Index of 22. Proctor Optimum Moisture is 22% and the maximum dry density is 1650kg/m^3. The strength of compacted clay was found to be (during test suite #1) sensitive to moisture content as illustrated by Figure 5.2.

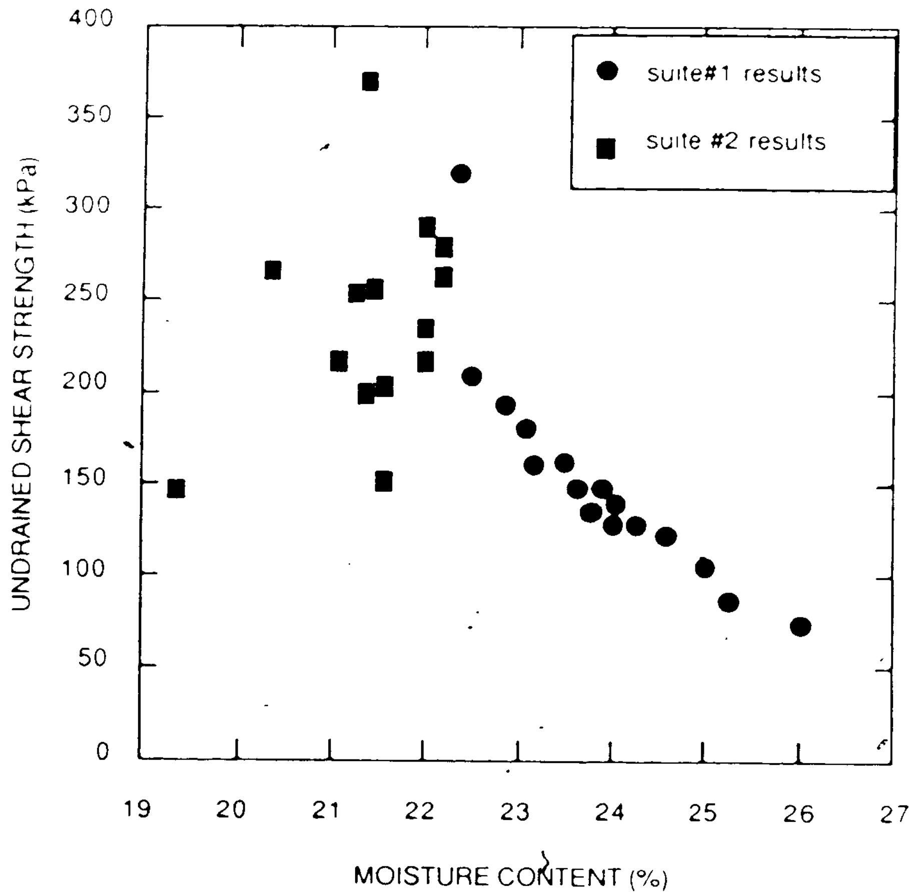


FIGURE 5.2 UNCONFINED COMPRESSIVE STRENGTH vs. MOISTURE CONTENT

5.5 SOIL PREPARATION

The soil was mixed to a moisture content of 24%. Using an "air operated pogo-stick" dynamic compactor the soil was compacted into the soil bin in 100mm lifts. The compactive effort applied consisted of a minimum of four passes with the dynamic compactor, until it appeared that a state of maximum density for that moisture content had been achieved. Tests with a laboratory vane indicated that the soil was in a relatively uniform state. Between tests, approximately half of the soil was removed and then recompacted into place. The variation in the shear strength - moisture content relationship of Figure 5.2 suggests that control on soil compaction was poorer during test suite #2.

5.6 DATA ACQUISITION

The load on the pile cap was measured with an Interface model 1310AJ load cell. Displacements were obtained by measuring the movement of the bin of soil with LVDTs.

Output from the transducers was routed into a Fluke model 2240B data aquisition system. Readings were recorded on paper tape.

5.7 TESTING PROCEDURE

After the soil was placed, it was screeded to provide a level surface. Guide holes were drilled to facilitate the insertion of the model piles. The guide holes were approximately 12mm in diameter which made them undersized compared to the piles. The guide holes were advanced to a depth approximately 5mm less than the final installed length of the piles to ensure that soil contact was continuous at the base of the group.

Two suites of tests were conducted. For the first suite the lower template was approximately 5mm above the soil surface. It was found that soil piled up in front of the piles and came into contact with the lower template before an ultimate load was measured. As described in more detail in Chapter 6, the load continued to increase throughout the course of each test. Tests were generally conducted until a total displacement of 10mm to 15mm was attained. The bin was displaced at a rate of 0.25mm/min.

For the second suite of tests, changes were made to rectify the perceived problems. The elevation of the lower template was raised to 25mm. Recognizing that larger displacements were going to be required to mobilize ultimate resistance the tests were conducted to total displacements of between 40mm and 45mm. In conjunction with this, the displacement rate was raised to 1.1mm/min

There was generally space for two tests in each bin of compacted soil. In test suite #1, usually two pile groups were tested. Two single pile tests were conducted during the program. During the second test suite, tests of a single pile and one group were conducted in a bin of soil. The exception to this procedure during test suite #2 was when, because of the small size of the groups, a single pile, and the 2x2 groups spaced at 1.5 and two diameters centre to centre were all tested in the same bin.

After the test was completed the strength of the soil was measured with a laboratory vane shear device. In addition the soil was sampled with thin walled tubes. The strength of the sampled soil was later tested in unconfined compression. Finally samples for moisture content determination were taken from the soil around the piles.

Approximately one half of the soil was removed from the bin and recompacted into place. The depth of soil removed was about 150% of the length of the piles.

5.8 LIMITATIONS AND DIFFICULTIES WITH THE MATERIAL

Despite choosing relatively small rods to be model piles, and choosing what was believed to be the minimum tolerable size of soil bin relative to the maximum group size, there was still a considerable volume of soil to be placed. In terms of volume of soil it is one of the most ambitious model testing programs in a clay soil ever to be attempted. The mass of the soil was approximately 1 tonne.

Problems were encountered preparing and handling the material. The method of placement which was chosen was the only practical one available. The compromises which had to be made resulted in a material which differed substantially from anything which might occur naturally, and which was less than ideal in terms of the model testing. Some of the differences of the soil from an ideal (natural) material are identified in this section. Potential difficulties in interpreting the results of the tests will pointed out.

Standard methods for mixing clay at the University of Alberta are limited to batch sizes of 4 to 5kg. Larger batches were mixed by using a small concrete mixer. This worked quite well, although there were concerns about the uniformity of mixing. It was found that compaction completed the mixing process and a uniform product was obtained.

Obtaining a soft, uniform soil was the objective during soil preparation and placement. This objective was attempted by trying to keep the soil as wet as possible and by imparting a large amount of compactive energy. By compacting the soil heavily the soil would be moved close to the "zero air voids line" in dry density - moisture content space. This is the upper limit to the dry density attainable at a given moisture content. By ensuring that all the soil was close to this upper limit, variation in dry density, and therefore, strength was reduced. This method of achieving a uniform soil bed was felt to be a great deal simpler, and as effective as any method of imparting a specific measured energy.

The high level of compactive effort did however produce a stronger soil than desired. A soft soil was desired for two reasons:

1. a soft soil causes the model piles to behave in a similar manner to the full scale prototype,
2. a softer soil reduces the loads, allowing use of lighter

members in the design of the apparatus.

It was planned to use the wettest soil practical. The upper limit for handling appeared to be approximately 25 percent moisture. Problems were encountered achieving this moisture content. Although the material was kept covered with plastic during storage, it was found that a considerable amount of drying took place during the lengthy mixing process and during placement. In place moisture contents were generally between 22% and 23% in a test series conducted during the summer and between 21% and 22% during a test series conducted during the winter when the relative humidity in the building was lower. Despite frequently coating all soil surfaces with a fine spray, drying continued throughout each test program. Measured in place moisture contents became lower toward the end of each program.

The objective of these tests was to model undrained (ϕ=0) loading conditions. Since the compacted soil was unsaturated, a frictional component to the strength of the soil may exist. This is not expected to affect the results significantly since the soil was compacted wet of the Proctor Optimum moisture content. Experience with compacted Athabasca clay has shown that the friction angle under total stress is less than 3° under low confining pressures.

The heavy compaction used on the soil imparted a very high K_o. This condition does not affect the ultimate capacity of laterally loaded piles, although the load deflection curve may well be affected. It has been assumed that the behaviour of both single piles and groups are equally affected. No work has been done during this study to justify this assumption.

Another effect of compaction is that the soil becomes horizontally layered. This can conceivably lead to horizontal planes of weakness along which the soil could fail preferentially. If this were to occur it could reduce the capacitiy of the larger groups that derive a larger proportion of the their resistance from shear beneath their base. Fortuneately, no indication of this preferential shear on an existing plane of weakness was observed when the test were performed. Therefore the test results are not believed to be significantly affected by this problem.

6. RESULTS FROM TEST PROGRAM

6.1 SINGLE PILE

6.1.1 Load-Deflection Curves

Load-deflection curves for the first tests of the single piles are shown on Figures A.1-A.2. Curves for the single pile tests conducted during the second test program are shown on Figures B.1-B.11. Load continues to increase throughout the test.

During the first test suite, as the pile was pushed through the soil, soil would pile up in front of it until it came in contact with the lower template. It was thought that this caused an increased confinement on the soil, which in turn caused an increase in resistance. In addition, there is a component of friction on the base of the plate.

The results of the second suite of tests also show the same steady rise in resistance. These tests were taken to large displacements, some to a final displacement of 40 to 50mm, about 3 pile diameters. Since the apparatus had been modified to prevent interference from the lower template it is apparent that interference was not the major factor in the continuing resistance gain.

The resistance build up is probably the result of a combination of some or all of the following factors:

1. the ductility of the soil used in the tests,
2. the geometry of the failure mechanism, which precludes

resistance loss with continuing displacement. This is discussed in section 2.3.2.1.

3. pile up of the soil in front of the pile.

The failure load was determined by fitting straight lines to the two portions of the curve and extending them to find the point of intersection.

As a check an alternative method of determining failure of the soil was used. This involved plotting load as a function of the natural logarithm of displacement. A typical curve is shown on Figure 6.1. The curve has four sections:

1. a concave upward portion corresponding to the initial linear portion of the actual curve,
2. a linear portion corresponding to the initial curved portion of the actual load displacement curve,
3. a second linear portion at a lower slope,
4. finally a second curved portion portion which is concave upward.

Other tests analysed in this way (eg. Matsui et al., 1982) do not exhibit the fourth portion of the curve. It is a product of the steady increase in load which these tests exhibited after failure.

Failure is designated as the point where the two linear portions of the curve meet. Agreement is good between the two methods of interpreting the data. The semi-log plot has the advantage that the failure point can be determined with less ambiguity.

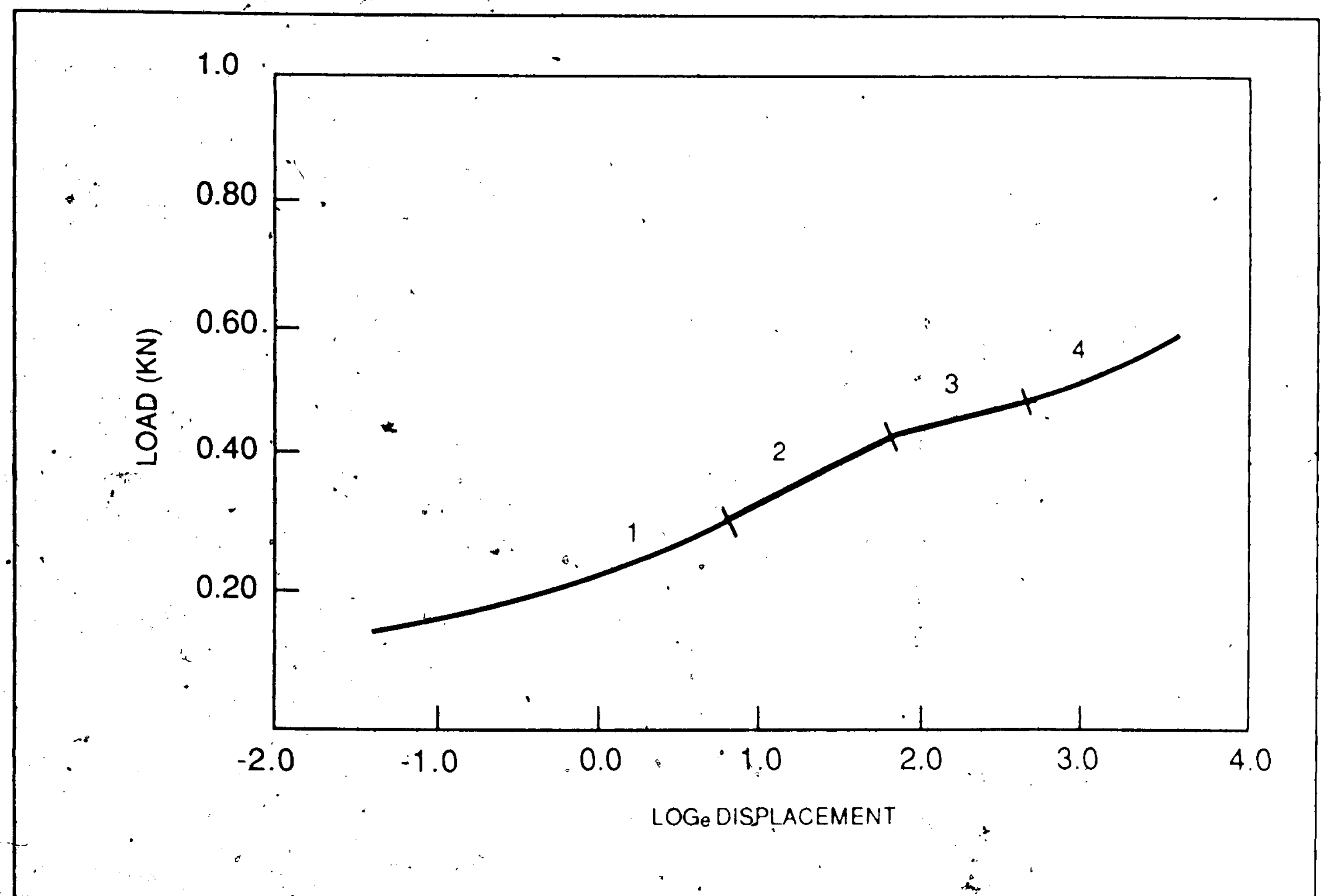


FIGURE 6.1 TYPICAL PLOT OF LOAD AGAINST LOG DISPLACEMENT

6.1.2 Failure Load

Figure 6.2 shows the variation of single pile resistance plotted against the estimated shear strength of the soil. As discussed in section 6.2.2 the strength of the soil is estimated by assessing vane shear tests, unconfined compression tests, and the moisture content of the soil around the piles.

Most of the data is for the second suite of tests, since only two single pile tests were carried out in the first suite. Although the results show considerable variabilty, there is a correlation between shear strength and the single pile resistance. The two tests from the first series, conducted in soil with an estimated average strength of 75kPa provided an average resistance of 0.73kN. At an average strength of 100kPa the resistance of the piles appears to average 0.83kN.

The single pile resistances referred to above are based on the inflection point of the load - log displacement curve. This value is lower than the ultimate resistance at very large deflections. Matsui et al. (1982) noted that for their tests the ultimate load was approximately 60 percent higher than the 'failure' load as defined on the semi-log plot. Most of the tests of second suite were carried out to displacements of at least two diameters. The resistance developed at that displacement was measured and compared to the corresponding failure load as defined by the semi-log plot. Interestingly, the resistande at two pile diameters

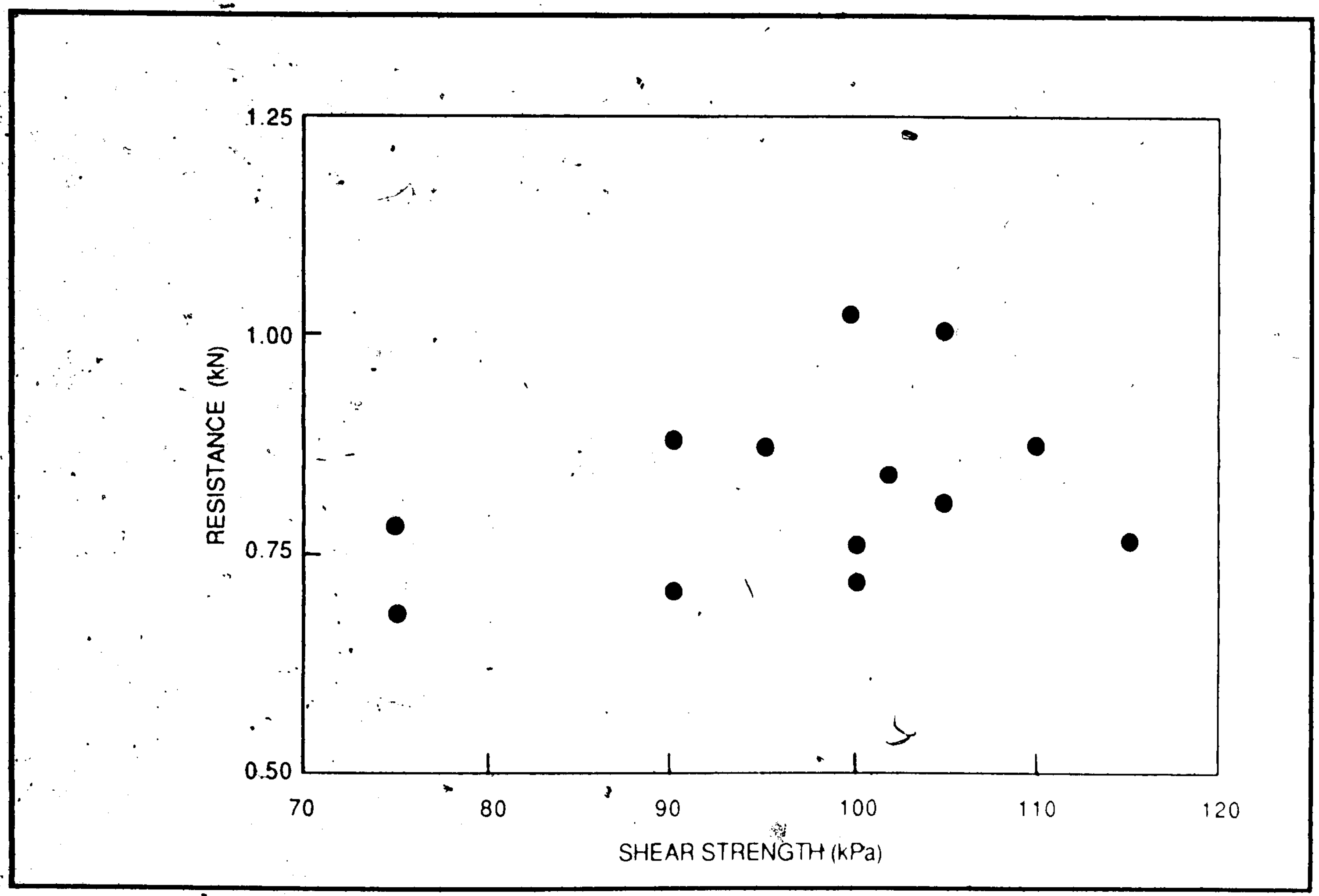


FIGURE 6.2 SINGLE PILE RESISTANCE vs. SHEAR STRENGTH

displacement averaged 60 percent higher than the failure load. It was decided on the basis of these two factors that the ultimate resistance of the single piles could be estimated as 1.6 times the failure resistance.

Matlock(1970) has developed one of the most complete descriptions of how lateral resistance is developed near the surface by laterally loaded piles. The results obtained during this study were compared with Matlock's work to establish whether they are reasonable. If the estimate for ultimate resistance discussed above is accepted, then the ultimate resistance in 75kPa soil is 1.17kN, and in 100kPa soil it is 1.33kN. These values were compared to the results obtained by Matlock(1970) to establish whether whether they reasonable.

The frontal projection of each pile had an area of $2.69x10^{3}m^{2}$. It can be calculated that the average pressure on the pile was $5.80c_u$ for the 75kPa soil and $4.94c_u$ for the 100kPa soil. These are average values for the entire length of the pile. It is expected that N_p is lower at the soil surface and increases along the length of the pile. Matlock developed an empirical relationship for the development of resistance with depth. It was stated earlier in this thesis as equation 1.7:

$$N_p=3 + \frac{\sigma_v}{c_u} + J\frac{H}{d}$$

at the surface $N_p=3$, and for 75kPa soil, if J=0.5, then at the tip,

$$N_p=3 + \frac{(.170)(2.03)(9.81)}{75} + 0.5\frac{(.170)}{(.0158)} = 8.42$$

The average value of N_p over the length of the pile is 5.71. This is slightly lower than what was was measured, possibly indicating a lower value of the 'J' parameter. Matlock found values of 0.5 and 0.25 for two different sites. It is interesting that the lower value of J was measured for the stronger soil. The same effect could be an explanation for the lower average N_p value deduced for stronger soils in these tests.

This calculation highlights a difference between the behaviour of the model piles and a full scale pile. For these tests the magnitude of the third term of equation 2.7 is much greater than the second term. This means that the computed value of N_p is significantly affected by the value assumed for 'J'. For full scale piles the term which contains 'J' is the least significant term, so that the value assumed is not important.

The above discussion assumes that there is validity in comparing the "ultimate" resistance values as defined for these tests and the ultimate resistance as measured by Matlock in his tests. Virtually all of the single pile tests continued to pick up load until the test was halted. The methodology used here chose the ultimate resistance as 1.6 times failure, a value which roughly corresponds to a displacement of two pile diameters. The value was used because factors such as soil build up in front of the pile certainly appear to be affecting the resistance at that order of displacement. In addition, a displacement of two pile diameters would certainly be unacceptable for the design of most structures.

6.1.3 Deflection

Measured displacements to failure were 16% and 25% of the pile diameter during the first suite of tests. During the second suite of tests the displacements averaged approximately 35% of the pile diameter. Broms(1964) states that maximum resistance should be attained at 20% of the pile diameter. In a similar vein, Matlock(1970) suggests that the deflection at ultimate resistance is given by:

$$y_u=20\epsilon_c d \qquad [6.1]$$

where ϵ_c is the strain at half the ultimate strength in an undrained compression test. A value of ϵ_c =0.010 is recommended for most cases. This leads to a value of y_u=0.2d, 20 percent of the pile diameter.

These recommendations of Broms and Matlock provide estimates for the displacement to ultimate resistance. In the tests reported for this thesis only the designated failure was reached at displacements of this magnitude. One explanation for this difference is the very ductile response of the soil used. The value of ϵ_c observed in unconfined compression tests on this material was between approximately 0.06 and 0.09. Using these values in equation 6.1 gives a displacement at ultimate averaging 1.5 pile diameters, a value closer to the observed behaviour.

6.1.4 Soil Deformation

As the test progressed a gap would open up behind the pile and the soil would bulge upward in front of the pile. At approximately the point of designated failure a vertical crack would appear and propagate outward from the pile in a direction perpendicular to the direction of the load application. The crack would continue to grow in length as the test continued. After the test was completed, it could be observed that both the gap behind the pile and the crack extended to the full depth of the pile.

During the first suite of tests the bulge would rise until it came into contact with the lower template. As the test continued the contact area increased in extent, spreading from the initial point of contact. This effect certainly added to the resistance measured because of the increase in friction on the template. Since the template prevented the soil from heaving it locally increased the vertical stress in front of the pile. Equation 2.7 shows that the pile resistance is dependent upon the vertical stress, so it is expected that this also led to an increase in measured resistance.

For the second suite of tests the lower template was raised so that soil contact with the pile cap was prevented. During the test the pile up in front of the pile reached a height of approximately 15mm.

6.2 GROUP RESULTS - TEST SUITE #1

6.2.1 Raw Test Results

Individual load-deflection curves for each test are shown on Figures A.3 to A.15. In the same way as the single pile tests, load continues to build up throughout.

Failure was initially determined by fitting two straight lines to the load deflection curve in the same way as the single pile tests. The data was also plotted with the natural logarithm of displacement as the abscissa. This method of presenting the data produced curves of the same shape as the curves for the single pile.

It was found, as in the case of the single pile, that the semi-log plot produced values for the failure load similar to those obtained by fitting straight lines to the load-deflection curve. Because the semi-log plot provided a failure point that was consistently defineable values determined by this method were used.

The loads at which failure occurred are summarized in Table 6.1, and shown graphically on Figure 6.3. A general trend of increasing resistance with increasing spacing can be seen.

Table 6.1 Summary of Single Pile and Pile Group Load Tests - Suite #1

Date	Size	Spacing	Failure Load (kN)	Displacement at Failure (mm)	Undrained Shear Strength (kPa)	Normalized Load (kN)
84-09-10	single	n/a	0.68	2.9	75	0.68
84-09-23	single	n/a	0.68	3.6	65	0.78
84-09-21	2x2	1.5	1.6	3.6	75	1.6
84-09-17	2x2	2	1.7	3.7	75	1.7
84-09-11	2x2	3	2.0	3.4	65	2.3
84-09-23	2x2	4	1.9	2.9	65	2.2
84-08-28	3x3	1.5	1.3	3.5	75	1.3
84-09-10	3x3	1.5	1.5	3.6	75	1.5
84-09-14	3x3	2	2.3	4.2	80	2.2
84-09-11	3x3	3	2.6	3.4	65	3.0
84-09-14	3x3	4	3.2	3.0	80	3.0
84-09-17	4x4	1.5	2.6	4.0	75	2.6
84-09-21	4x4	2	2.8	4.1	75	2.8
84-09-29	4x4	3	4.3	3.4	90	3.6
84-10-03	4x4	4	5.4	4.4	105	3.8

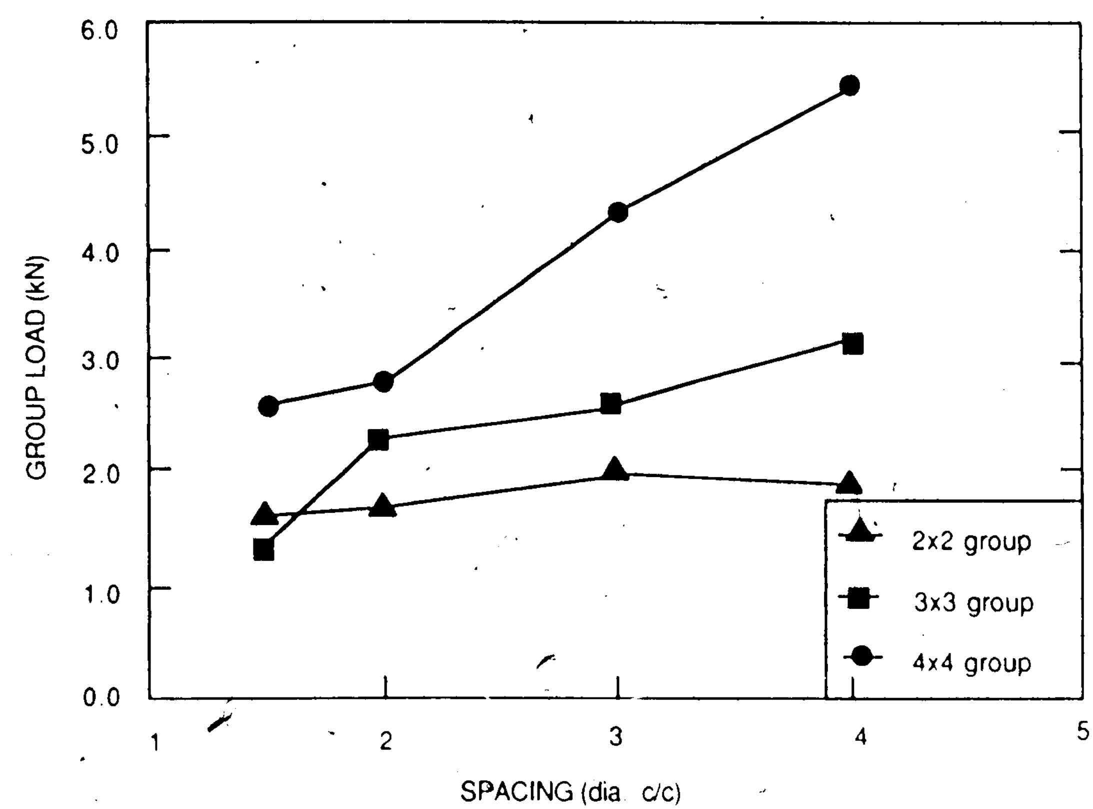


FIGURE 6.3 GROUP RESISTANCE vs. PILE SPACING
RAW RESULTS
test suite #1

6.2.2 Normalized Test Results

6.2.2.1 Estimation of Average Clay Properties

Because slight drying of the soil took place during the course of the test program, the strength of the soil varied between tests. To account for this, the measured resistances were normalized to a shear strength of 75kPa. To do this it was necessary to estimate the strength of the soil in the bin for each test.

Three methods were used to estimate the shear strength of the soil.

1. The soil strength was measured directly in the bin with a laboratory vane shear device. Seven measurements were usually taken from each soil bin.
2. One or two samples were taken from each bin with a 38mm I.D. thin walled sample tube. These samples were tested in unconfined compression. The results of these tests were used both to estimate the strength of the soil in that bin and to develop the relationship betweeen moisture content and strength.
3. Samples for moisture content determination were taken from the soil within the pile group after the piles were removed. Using the correlation between moisture content and strength determined from the unconfined compression samples taken from test suite #1, an estimate of the strength was obtained.

Table 6.2 shows the average vane shear strength, unconfined compression test results, and the average moisture content and the shear strength deduced from that. The final column shows the shear strength estimated from all these data.

6.2.2.2 Normalized Load Test Results

Failure loads were normalized by dividing the failure load by the ratio of shear strength of that particular soil to the reference shear strength of 7.5kPa. The normalized failure loads are given in Table 6.1 and shown on Figure 6.4. Two conclusions are immediately obvious: group resistance increases only very slightly as the spacing of piles increases over three diameters centre to centre. Additionally, efficiency at four pile diameter spacing is low and it is lower for larger groups: maximum efficiency is 75 percent for 4 pile groups and drops to 32 percent for 16 pile groups.

6.3 GROUP RESULTS - TEST SUITE #2

6.3.1 Raw Test Results

The individual load-deflection curves are shown on Figures B.12 to B.23. The results are generally consistent with the curves shown in Appendix A despite the change in test procedure. There is some variation in the load-deflection response between the two test suites. This will be discussed further in section 6.4 Most tests were carried out to displacements of over 40mm and exhibited increasing resistance throughout the test. Surprisingly the

Table 6.2 Summary of Soil Properties for each Test Bed - Suite #1

Date of Tests	Average Vane Shear Strength (kPa)	Average Unconfined Shear Strength (kPa)	Average Moisture Content (%)	Shear Strength Inferred From Moisture Content (kPa)	Shear Strength Chosen For Normalization (kPa)
84-09-10	68	74	23.5	74 - 86	75
84-09-11	60	64	23.2	83 - 90	65
84-09-14	75	74	23.2	83 - 90	80
84-09-17	83	88	24.0	63 - 72	75
84-09-21	71	75	23.6	73 - 85	75
84-09-23	67	77	24.0	63 - 72	65
84-09-29	89	82	23.1	85 - 99	90
84-10-03	107	94	22.3	102 - 123	105

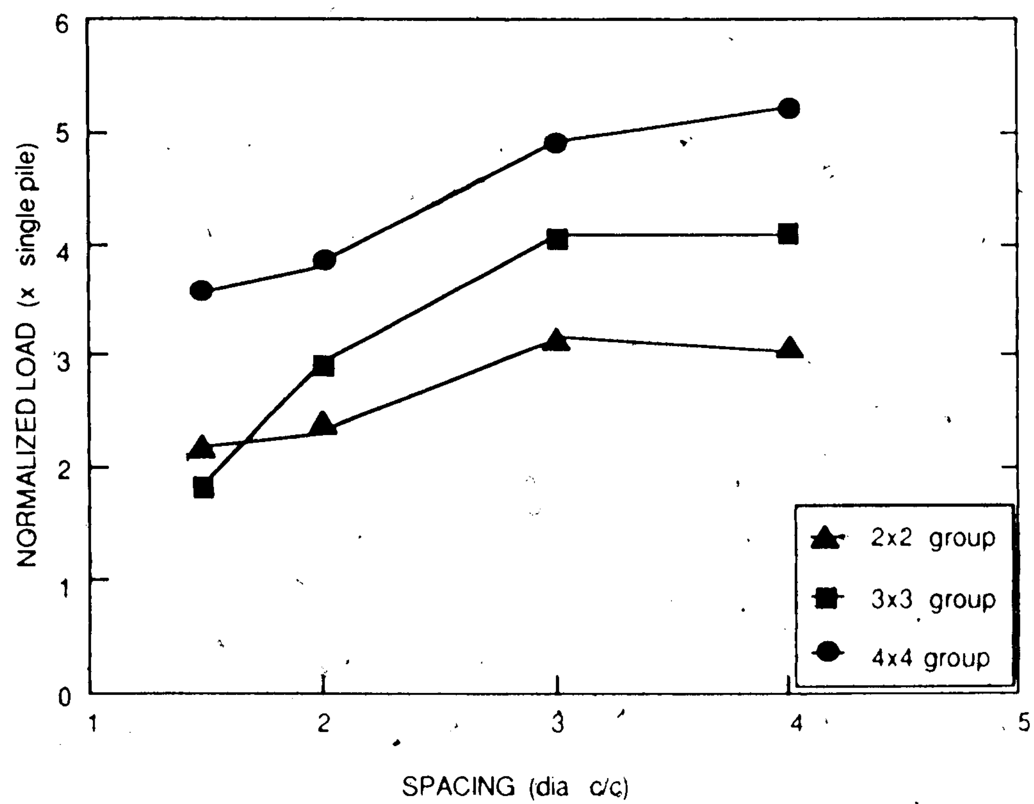


FIGURE 6.4 GROUP RESISTANCE vs. PILE SPACING NORMALIZED BY SINGLE PILE RESISTANCE suite #1 results

exceptions weere the tests on the largest of the 4x4 groups (spacings of 3.0d and 4.0d) which exhibited a small drop in resistance at large displacement.

Failure was again defined two ways, with the log displacement method used as the preferred method due to its precision. A summary of the failure loads and the displacement at which they occurred may be found in Table 6.3.

6.3.2 Normalized Test Results

Rather than normalize the group results by the strength of the soil in the bin, the resistance of each group has been expresed as a multiple of the resistance measured for the single pile test performed in the same soil bin. The method is simpler and less prone to judgemental errors.

Since the 2x2 groups at spacings of 1.5d and 2.0d were tested in the same soil bin, both are compared against single pile test of Figure B11.

The results are plotted on Figure 6.5. The results are similar to the results shown on Figure 6.4. The combined results of both suites of tests are plotted on Figure 6.6.

6.4 LOAD-DEFECTION BEHAVIOUR AND SOIL DEFORMATION

6.4.1 Load-Deflection Behaviour

Designated failure of the single piles tested in suite #1 took place at displacements of 2.6mm and 3.2mm. The displacement at which designated failure took place for pile

Table 6.3 Summary of Pile Load Tests - Suite #2

Date	Size	Spacing	Failure Load (group) (kN)	Failure Load (single pile) (kN)	Estimated Undrained Shear Strength (kPa)	Group Load as Multiple of Single Pile	Displacement at Failure group (mm)	Displacement at Failure single pile (mm)
85-04-04	2x2	1.5d	1.75	0.77	115	2.27	6.0	5.1
85-04-04	2x2	2.0d	2.13	0.77	115	2.77	6.8	5.1
85-04-01	2x2	3.0d	2.46	0.88	90	2.80	5.2	5.1
85-03-20	2x2	4.0d	2.38	0.87	110	2.74	5.3	5.2
85-03-13	3x3	1.5d	2.76	0.87	95	3.17	4.9	5.9
85-02-19	3x3	2.0d	2.48	0.84	100	2.95	5.6	6.2
85-03-06	3x3	3.0d	3.23	0.72	100	4.49	4.2	4.2
85-02-13	3x3	4.0d	3.46	0.94	100	3.68	5.2	10.7
85-02-28	4x4	1.5d	3.46	1.00	105	3.46	6.2	7.0
85-02-22	4x4	2.0d	3.39	0.80	105	4.24	4.8	5.0
85-02-07	4x4	3.0d	3.66	0.76	100	4.82	4.8	5.3
85-02-05	4x4	4.0d	3.54	0.69	90	5.13	4.2	5.6

Table 6.4 Summary of the Properties of Each Test Bed - Suite #2

Date of Tests	Average Vane Shear Strength (kPa)	Average Unconfined Shear Strength (kPa)	Average Moisture Content (%)	Shear Strength Inferred from Moisture Content (kPa)	Estimated Average Shear Strength (kPa)
85-02-05		75	22.5	83-107	90
85-02-07			22.1	88-118	100
85-02-13	75	136	22.1	88-118	100
85-02-19	102	128	22	90-120	100
85-02-22	104	128	21.9	92-123	105
85-02-28	95	122	21.5	98-113	105
85-03-06	104	105	22.1	88-118	100
85-03-13	99	102	22.5	83-107	95
85-03-20	106	304	21.6	95-130	110
85-04-01	90		22.5	83-107	90
85-04-04			21.4	100-135	115

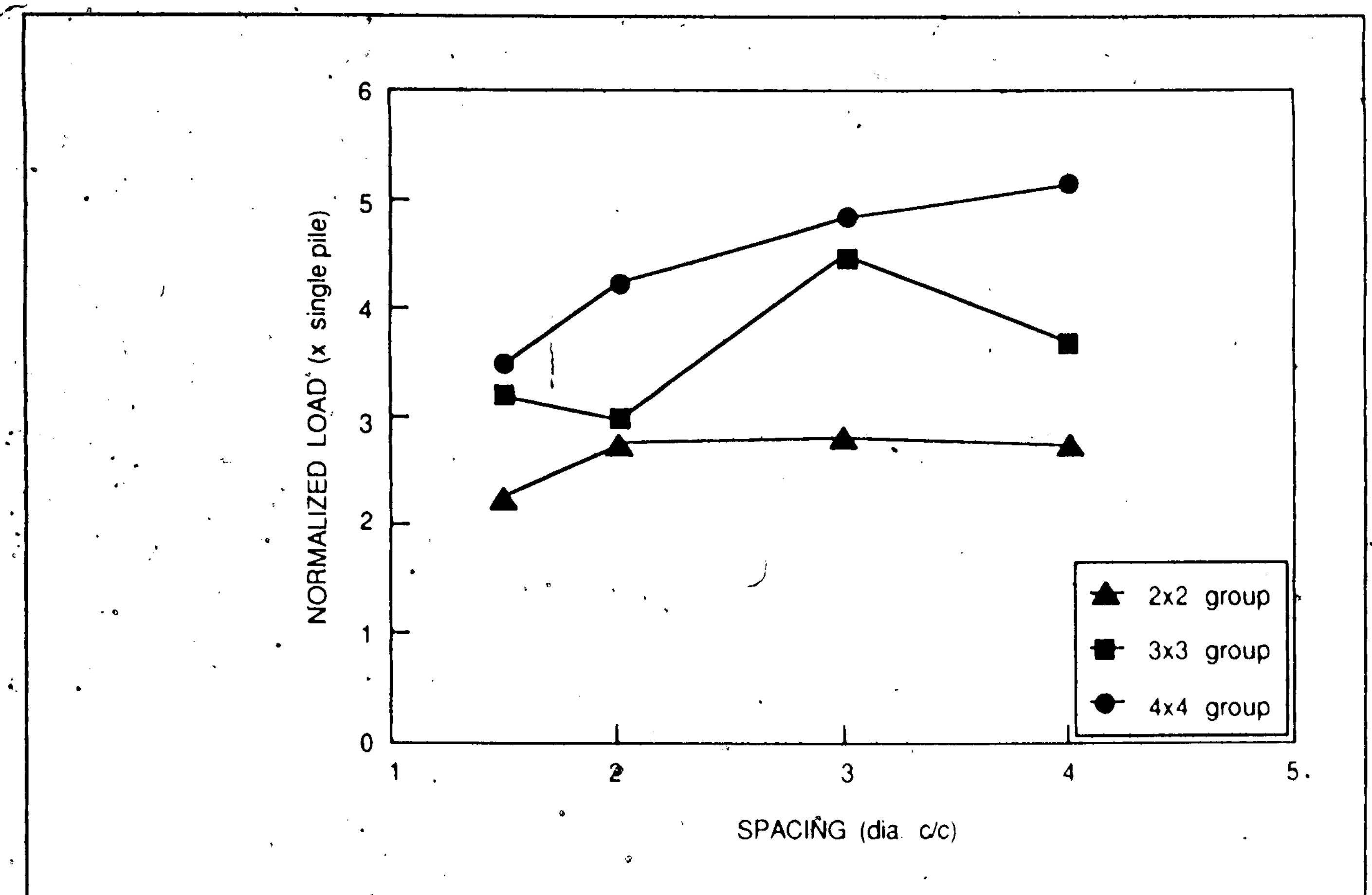


FIGURE 6.5 GROUP RESISTANCE vs. PILE SPACING
NORMALIZED BY SINGLE PILE RESISTANCE
test suite #2

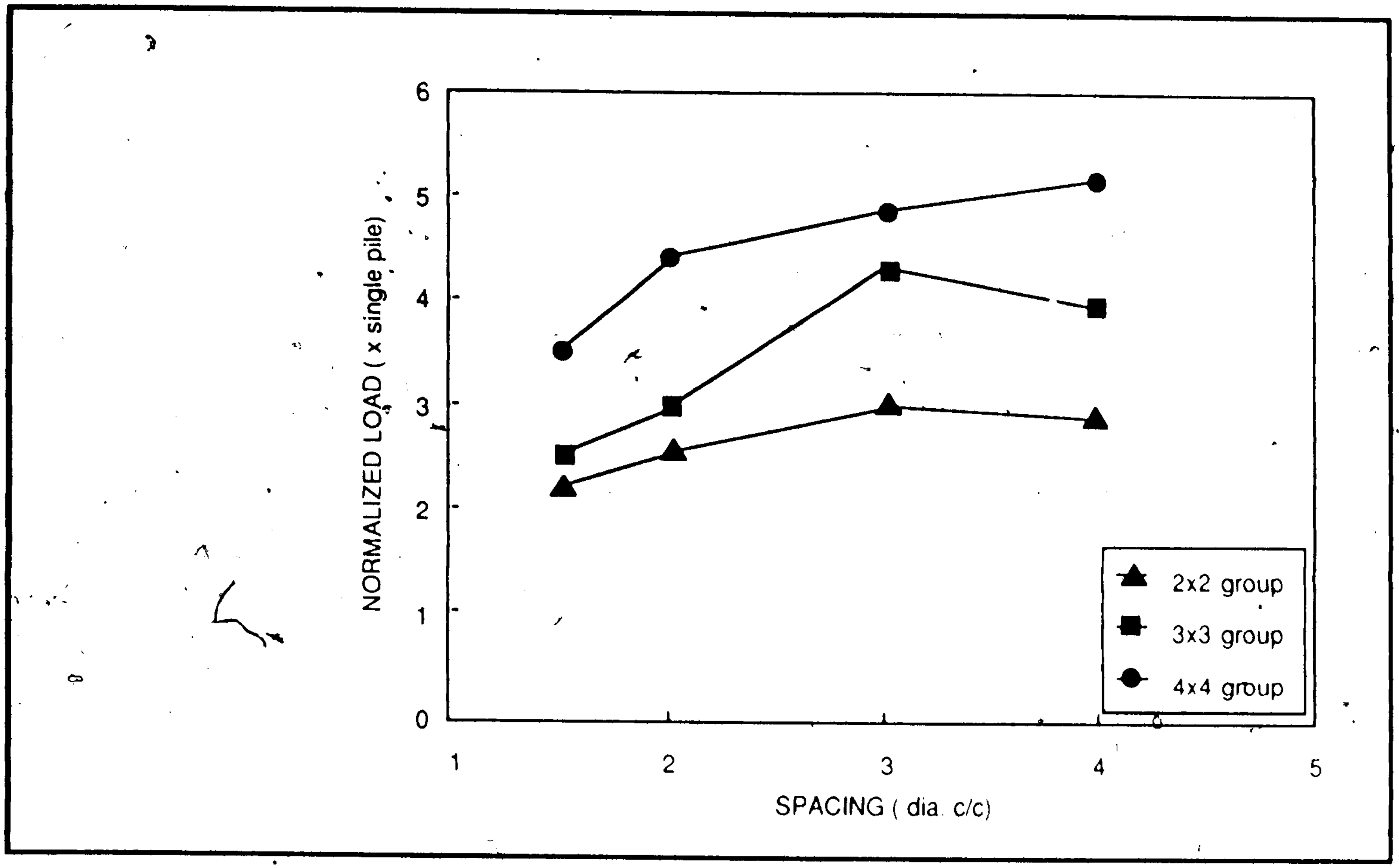


FIGURE 6.6 GROUP RESISTANCE vs. PILE SPACING NORMALIZED BY SINGLE PILE RESISTANCE average results

groups generally ranged from 3.5mm to 4.4mm. Displacement at failure is plotted as a function of group breadth on Figure 6.7. Although there may be a slight tendency for the displacement at failure to increase with group size, it is not statistically significant. It appears that the failure mechanism is related to the single pile rather than the group.

The single piles tested in suite #2 reached designated failure at an average displacement of 5.61mm. The groups failed at an average displacement of 5.20mm, slightly lower than the single piles. A plot of failure displacement against group breadth for suite #2, although not shown in this thesis, would demonstrate that the variables are completely independent. The two differences between the tests that might be related to the different failure displacements are the displacement rate and the moisture content in the soil. (Because the soil did not contact the pile cap until after failure had occurred, the position of the lower template does not seem to be a factor)

Soil moisture content varied throughout each test suite. No dependence of failure displacement on moisture content was observed within a test suite. It is tentatively concluded that the difference can be related to the different displacement rates.

Because the point of failure appeared to correspond with the opening of the gap behind the pile, it is likely that the behaviour can be related to the loss of tension on

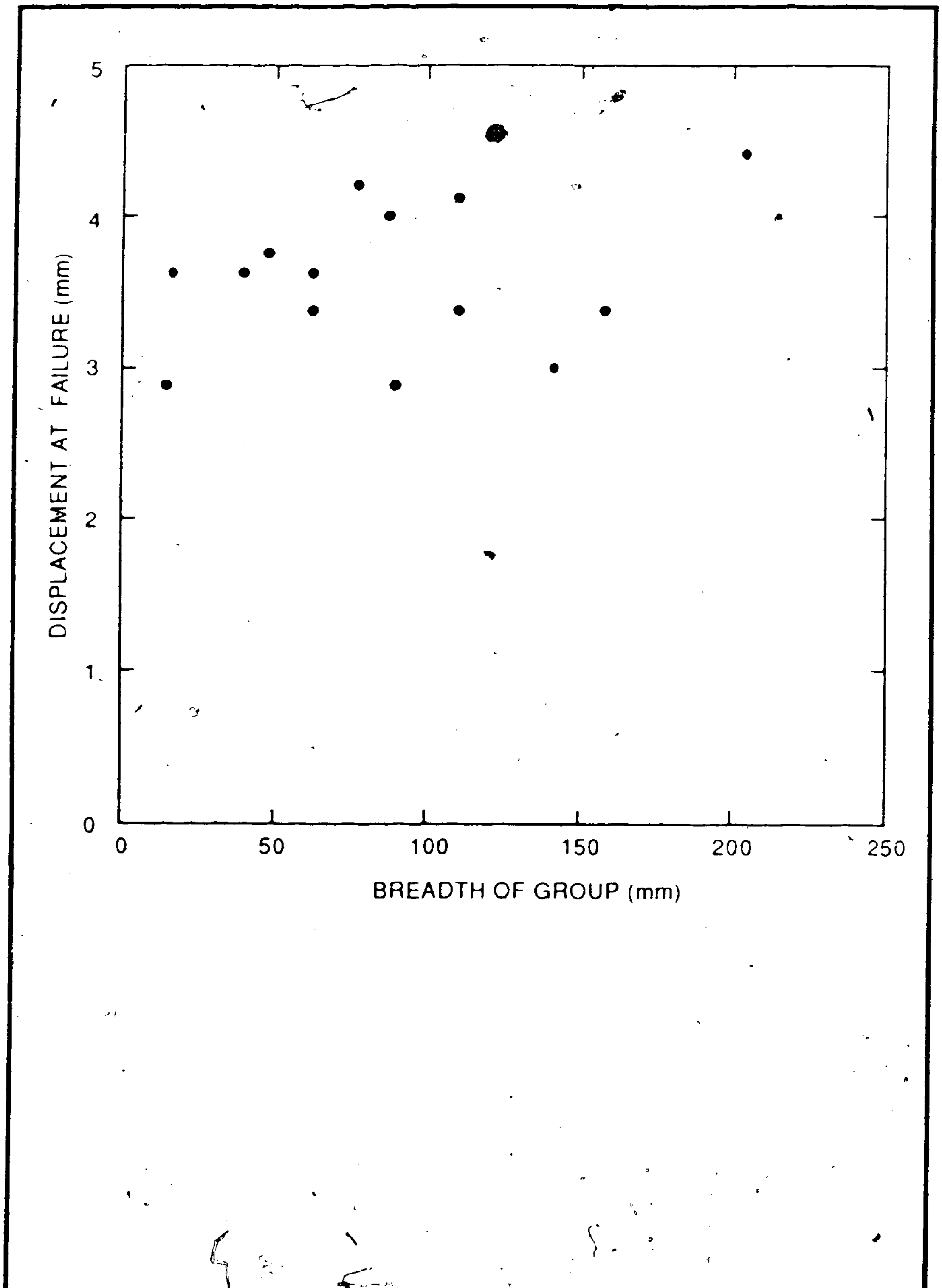


FIGURE 6.7 DISPLACEMENT OF PILE OR PILE GROUP AT FAILURE PLOTTED AGAINST BREADTH OF THE GROUP

the back of the pile. It may be that the soil can sustain higher tensile strains for shorter periods of time.

On Figure 6.8 the load deflection curves of six selected tests from suite #1 are plotted together, with the load axis normalized with respect to the failure load. The curves plot virtually on top of each other, indicating that the load-deflection behaviour is the same, independent of group size and spacing.

This does not appear to be the case for the suite #2 results. Figure 6.9 shows a selection of normalized curves from those tests. It can be seen that although failure occurs at the same point in each test the larger groups continue to pick up load at a greater rate than single piles and the groups at 1.5 and two diameter spacing. The greatest difference between the two modes occurs between displacements of 1.5 and 2.5 pile diameters, after which the gap begins to close. Relatively speaking the larger groups pick up more load in the initial post failure stages and progressively less at larger displacements. As mentioned before, some of the largest groups tested appeared to have attained an ultimate resistance. It was found that groups spaced at three and four diameters experienced the largest post-failure load increase, while the groups spaced at 1.5 diameters generally behaved like the single piles. The groups spaced at two diameters formed an intermediate case.

The reason for the variation betwen test suites must be related to either the displacement rate of the different

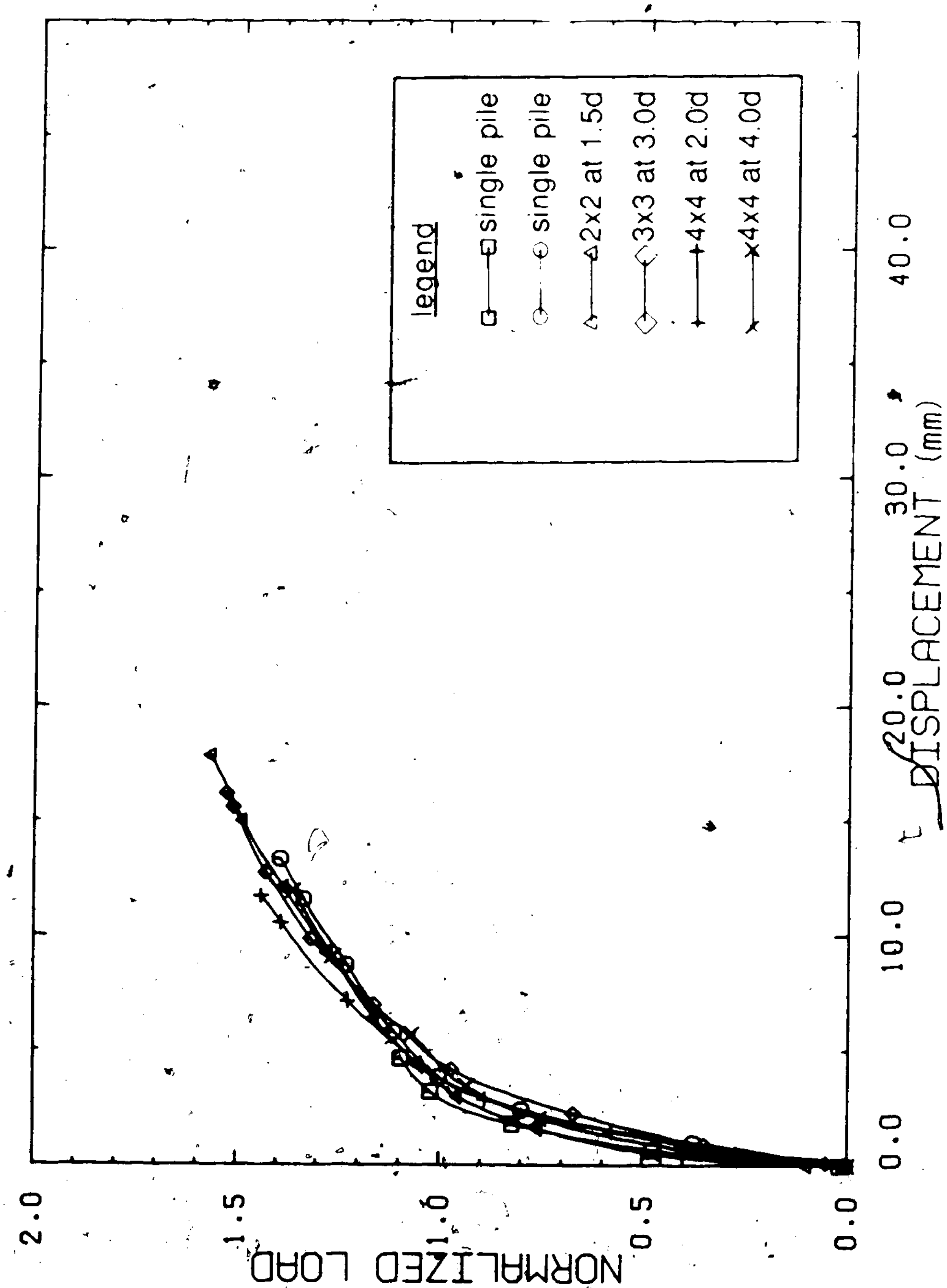


Figure 6.8 Normalized load-deflection curves from test suite #1.

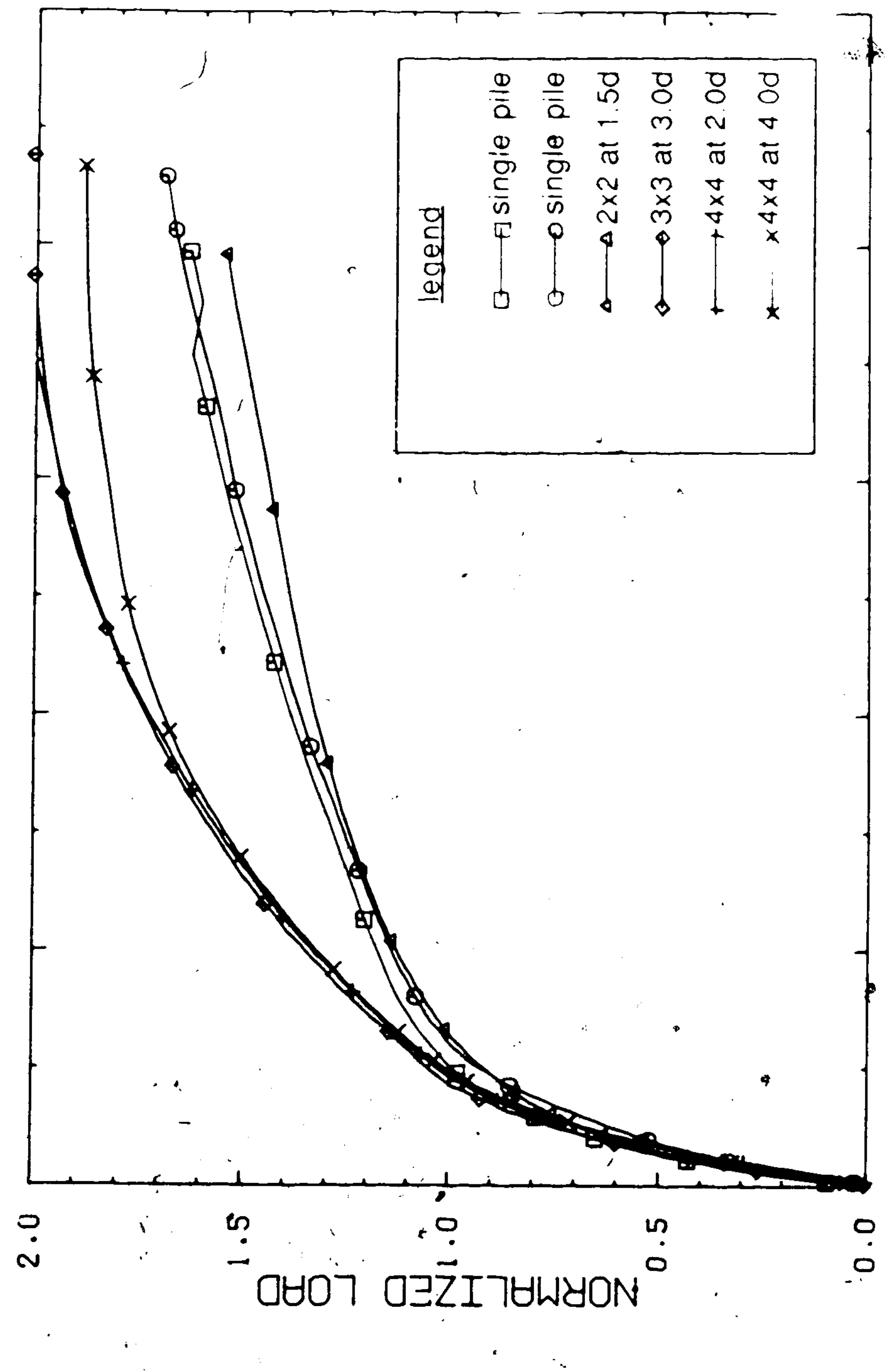


Figure 6.9 Normalized load-deflection curves from test suite #2.

location of the lower template. Differences in strength due to moisture content variations are not believed to be responsible for the different response for the same reason as before. The post-failure response of the larger groups in test suite #2 is reasonably similar to the response observed throughout the first test suite. The smaller groups in the second test suite developed lower post-failure resistance. This result is consistent with the theory that interference from the lower template caused an increase in measured group loads.

The effect of the template does not fully explain observed behaviour, however. The template did not interfere with soil deformation in any of the group tests. It may be that there is some mode of behaviour common to the larger groups which was obscured by the presence of the lower template during test suite #1.

6.4.2 Soil Cracking and Deformation

The observed behaviour of the soil was consistent in all tests, and appeared to be independent of the size of the group or of the pile spacing.

Figure 6.10 is a sketch of the pattern of deformation after the test of a 4x4 group. It is typical of all of the pile groups tested. A gap formed behind each pile in the rearmost row of the group and a crack developed between each pile in that row. The crack extended laterally from the side of the group a distance which was approximately equal to the

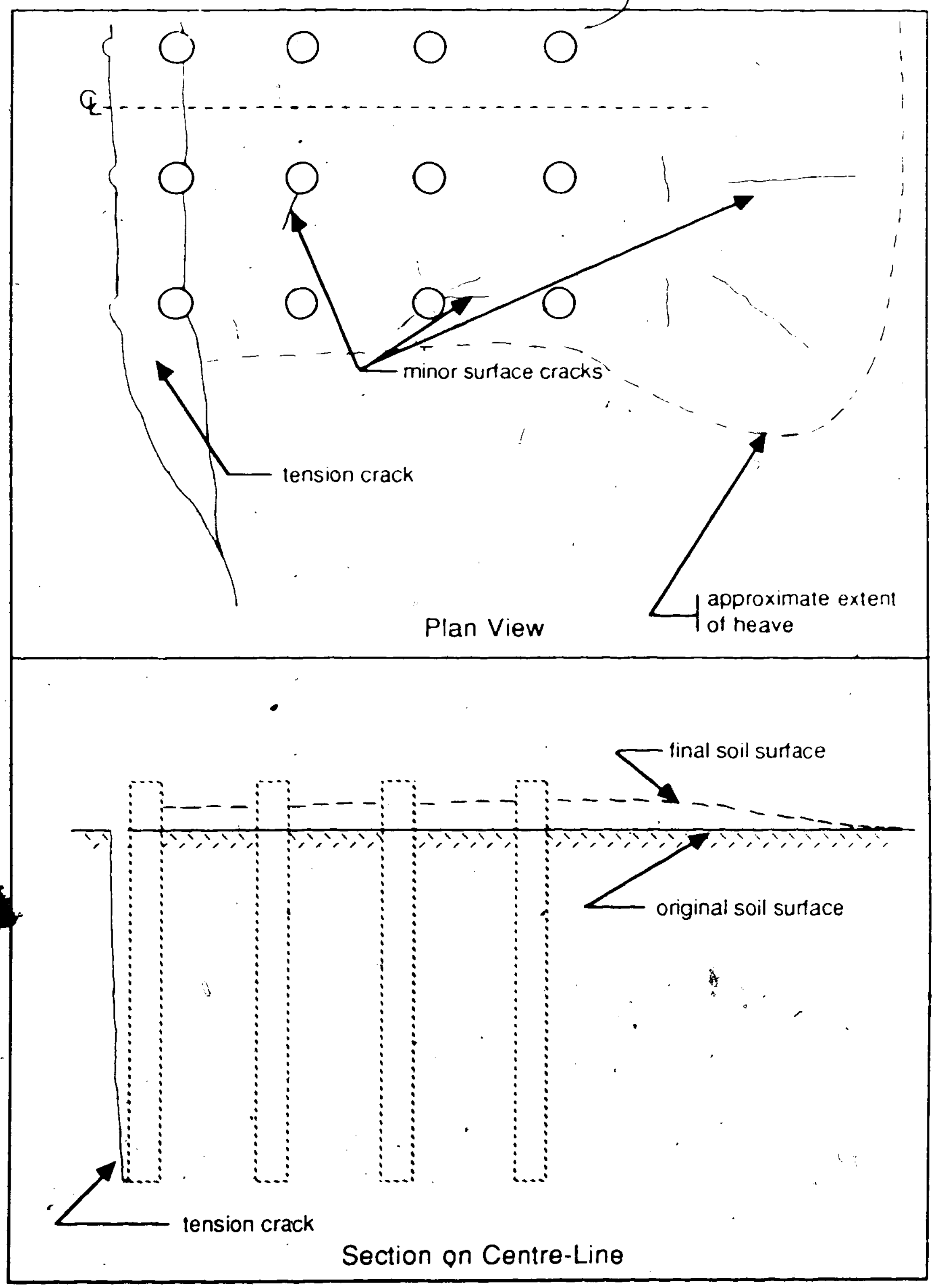


FIGURE 6.10 SKETCH OF THE SOIL DEFORMATION PATTERN AROUND A 4x4 PILE GROUP

breadth of the group. Both the gaps and the crack extended to the full depth of the piles. The soil remained in close contact with all the other piles in the group.

Within the area of the group and approximately one pile length in front, the soil was observed to heave. The transition from the region of heaved soil to undisturbed soil was gradual and usually only very minor surface cracking occurred.

6.5 DISCUSSION OF THE RESULTS

6.5.1 Comparison with Other Results

There are similarities with the observed behaviour of vertical groups reported by Whitaker. There appears to be two parts to the curve of resistance against pile spacing, a portion where the resistance increases rapidly with spacing then a flat portion. Unlike the vertically loaded groups the horizontally loaded group behaviour cannot be related to block action.

In the flatter portion of the curve the efficiencies of horizontally loaded groups are much lower than the efficiency of the vertically loaded groups.

The efficiencies observed in these tests correspond quite well with the results reported by Prakash and Saran. For the 2x2 groups a higher efficiency was observed in these tests (0.75 vs. 0.60), in the 3x3 group test the efficiency observed was approximately equal for both experiments (0.48

in these tests, and 0.45 reported by Prakash and Saran).

Although the results of Cox et al.(1984) indicate that interference in in-line loaded single files of piles is greater than previously used in design, they do not begin to indicate the extent of the interference that occurs when the group is extended to two dimensions.

6.5.2 Possible Failure Mechanisms

The low values for group resistance observed is both striking and disconcerting. The measured resistances are well below either a block failure mechanism (described in section 4.2.2) or that of individual single piles.

Figures 6.11 to 6.13 compare the measured resistance with the capacity predicted by some different mechanisms that may be postulated. The two dotted horizontal lines indicate the capacity of the group if the efficiency with respect to the single pile capacity was 1.0. Failure is defined as it was previously, as the inflection point on the load log-displacement plot. The lower line should be compared to the actual capacity at failure. The upper horizontal line is the estimated "ultimate" capacity which is 60 percent higher than the failure resistance.

The resistances predicted for the block which enclosed the pile group are plotted for comparison using rigid plasticity assumptions. Three lines are plotted, reflecting various degrees of conservatism which might be used in predicting the resistance. The top-most line represents

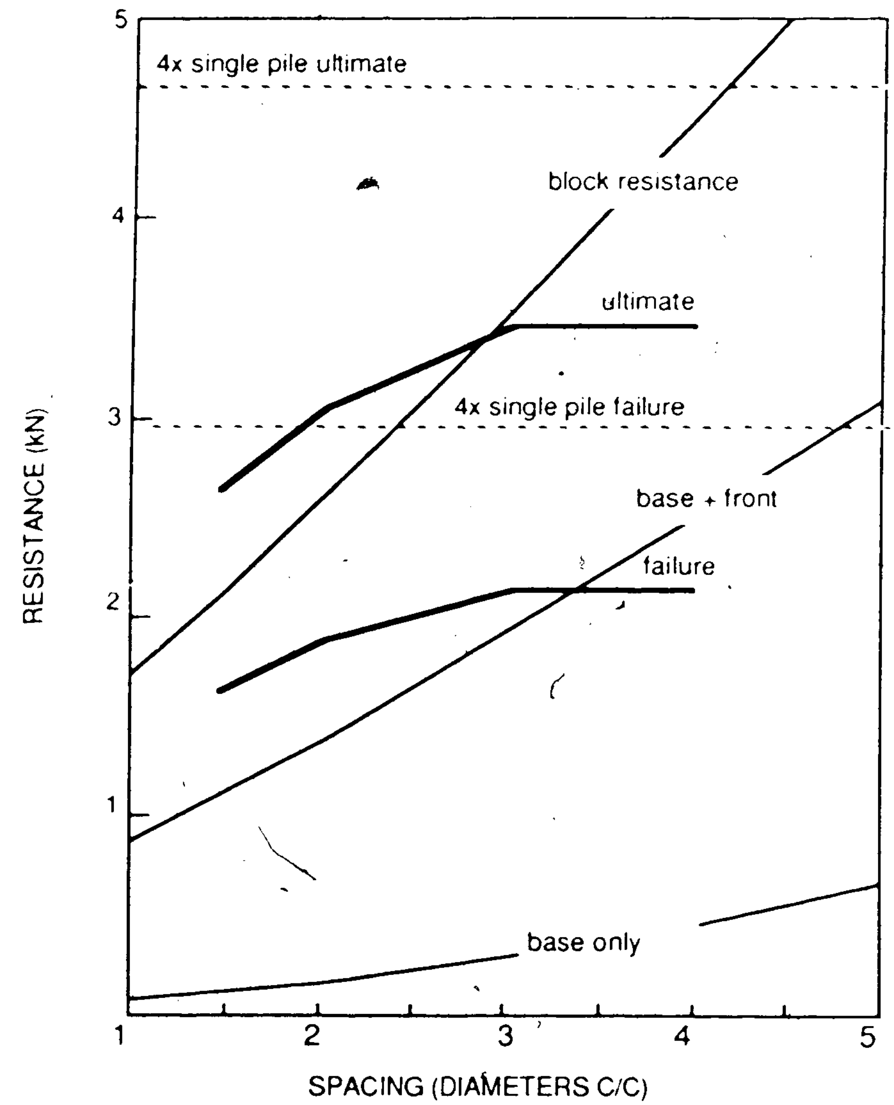


FIGURE 6.11 COMPARISON OF MEASURED GROUP RESISTANCE WITH POSSIBLE FAILURE MECHANISMS
2x2 groups

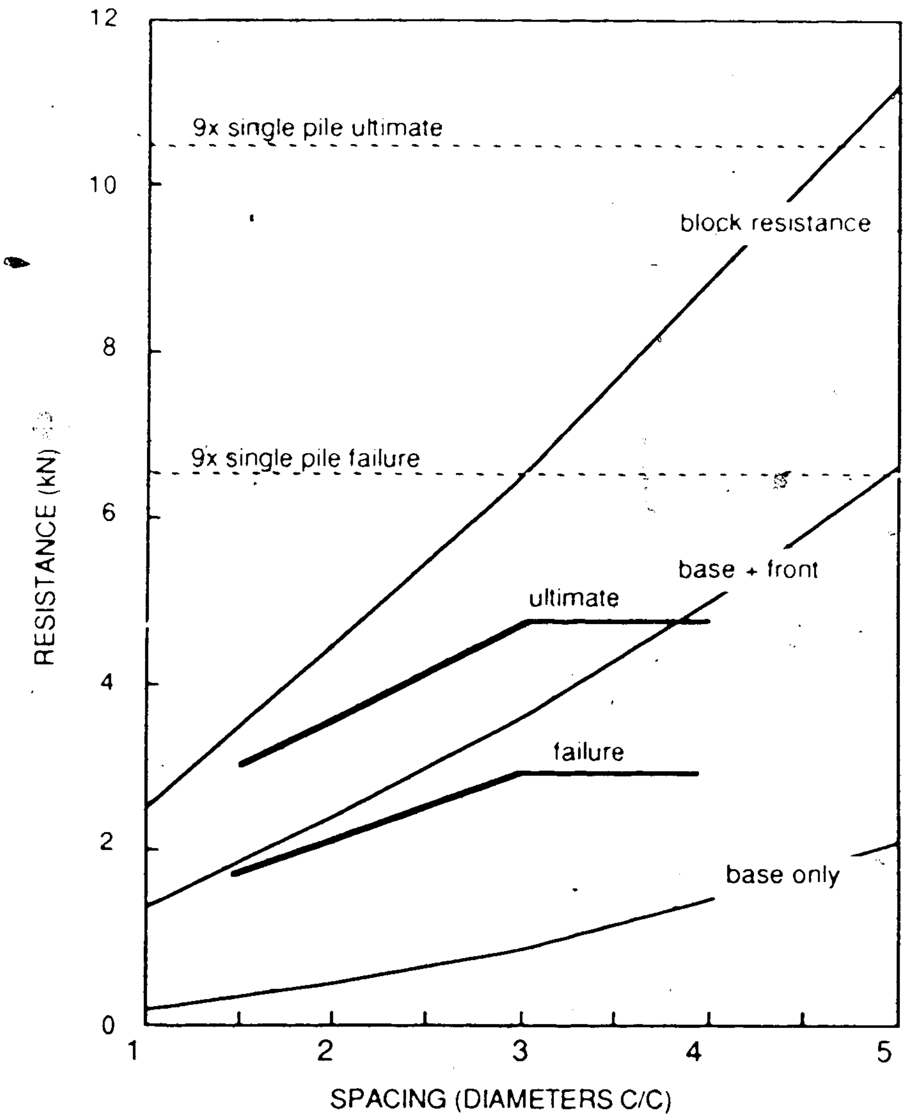


FIGURE 6.12 COMPARISON OF MEASURED GROUP RESISTANCE WITH POSSIBLE FAILURE MECHANISMS
3x3 groups.

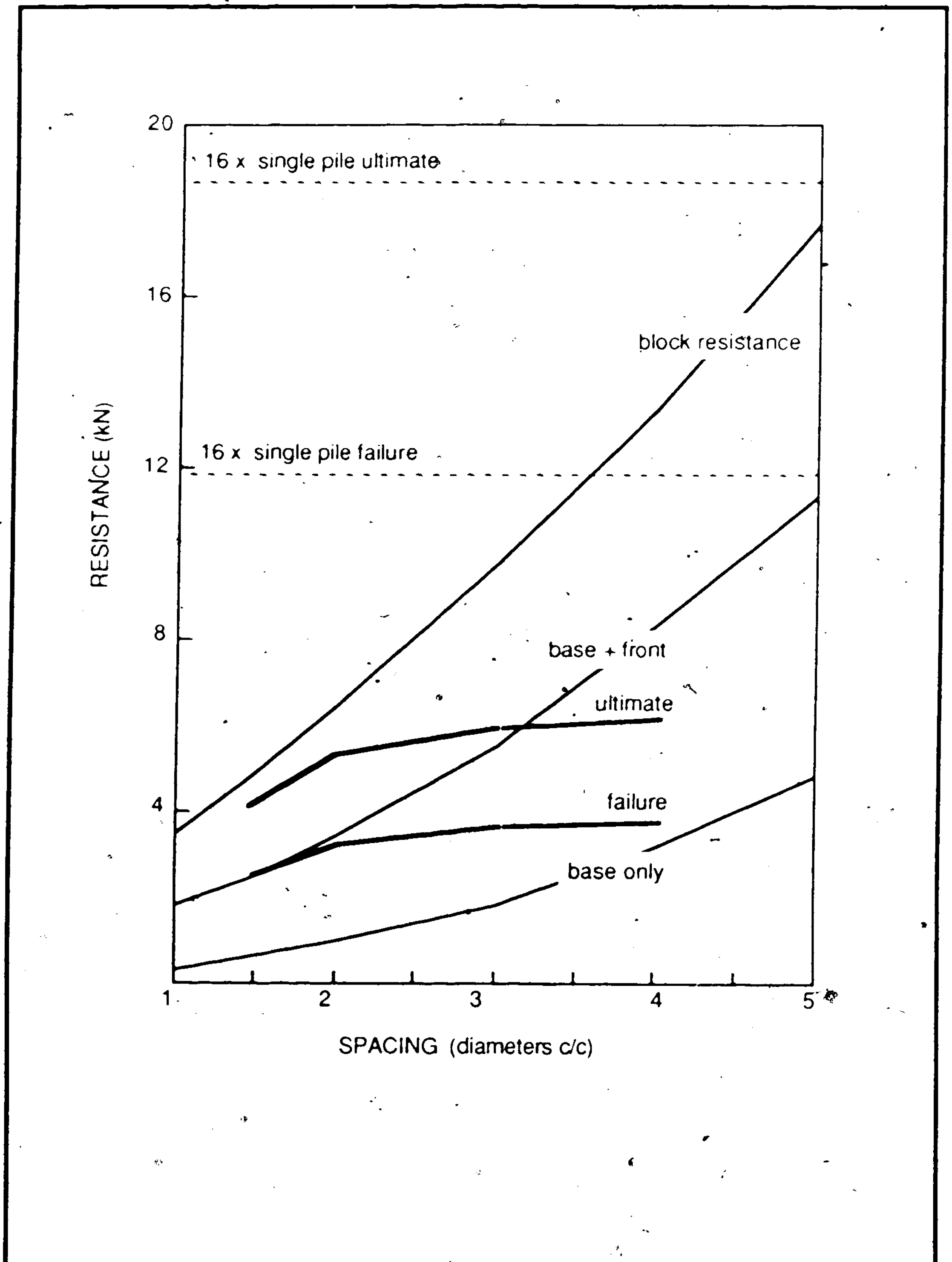


FIGURE 6.13 COMPARISON OF MEASURED GROUP RESISTANCE WITH POSSIBLE FAILURE MECHANISMS
4x4 groups

mobilizing passive pressure over the frontal area ($2c_u$), and mobilizing shearing resistance (c_u) along the area of the sides and the base.

A more conservative estimate neglects the shearing on the sides of the block and includes only the frontal passive pressure and shearing on the base. The third estimate considers shearing on the base of the block only.

Figure 6.11 shows these results for the 2x2 groups. The trend of the ultimate resistances shows a good agreement with what is predicted for the block mechanism up to the 3.0d spacing, but then drops below the predicted resistance for the 4.0d spacing. Figure 6.12 shows a similar result for the 3x3 group. Ultimate resistance drops below the more conservative prediction for the block, which accounts only for the base and front. Finally Figure 6.13 shows that an extrapolation of the 4x4 results will drop below even the base only resistance.

On all the figures the actual resistances are far below the line showing full efficiency. Extrapolation to spacing at which the pile resistance will be unaffected by group effects is difficult. It is apparent that a fairly high spacing will be required, greater than indicated by previous research. It is also apparent that the spacing required for full efficiency will be greater for larger pile groups.

Some characteristics of the pile group behaviour make it appear that the group is behaving as a block. Chiefly these are some elements of the mode of soil deformation: the cracking of the soil behind the group, the fact that there was no gap behind any but the rearmost piles in the group, and the heaving motion of the soil in front of the group. Other characteristics, the consistency of the displacement at failure despite variations in group size, and the heaving of the surface of the soil within the confines of the group, for example, give the appearance that the group behaviour is controlled by mechanisms based on the single pile.

It is difficult to assess what the variation in post-failure behaviour observed in test suite #2 indicates. It is postulated that the larger increase in load observed for the larger groups is due to the shape of the group being wide relative to its depth so that different components of the groups resistance, which might be mobilized at larger displacements (e.g. shear on the base of the group), becomes dominant. If this is so, it implies that the soil is behaving as a coherent mass up to the largest pile spacing tested. Extending this argument, at very large pile spacings the shape of the load-deflection curve would revert to that of the single pile. It may also be postulated however, (although this seems less likely) that the change in behaviour observed is caused by the change from a single mass to multiple piles. Under this scenario, the piles at 1.5 diameter spacing behave like an equivalent pile and produce that kind of a load deflection curve, while at larger spacings the load-deflection curve is characteristic of a pile group.

At spacings of three and four pile diameters each group develops only slightly more than the resistance of the front row. This suggests that the second and following rows are being shadowed by the first of piles. This has been observed by Cox et al.(1984) for a single file, but these results indicate that shadowing can extend to adjacent files as well. A suggestion may be made that the shadow effect is a result of the failure of the soil ahead of piles in the second and subsequent rows into the gap left behind the leading pile. Since no gap was observed behind any piles but the extreme trailing row this does not seem to be the complete explanation. The soil in the group appears to displace as a relatively intact unit, yet shearing resistance does not appear to be mobilized over the boundaries of this mass.

A possible explanation for this is that shear planes develop progressively on the boundaries. The soil used in these tests has a low E/c ratio. This property may have prevented uniform mobilization of shear strength on the boundaries of the enclosing block, resulting in progressively failure. The deformation observed within the group makes clear that at least some differential mobilization of shear strength along the boundaries of the block is taking place.

The development of the tension crack behind the group is likley the event that is producing the designated failure resistance. It may be possible that the development of this crack prevents the full development of shear planes around the group. The softness of the soil may also be a factor in preventing shear planes from developing at the displacements to which the groups were tested.

An elastic analysis described by Poulos and Davis(1980) predicts that the stiffness of the load-deflection curve is very much less for larger groups when expressed as an average load per pile. If, consistent with the observations made during these tests, the assumption is made that displacement to failure is constant, irrespective of group size, then the efficiency of each group can be predicted. The predictions are shown on Figure 6.14. Actually measured group efficiencies have also been plotted on the figure. Although agreement is not good, certain trends are similar. Group efficiency is very low and increases only very slowly for larger spacings between piles. Group efficiency declines as the number of piles in the group increases. The dissimilarity between the elastic theory and the performance of the model groups is that the actual efficiencies are almost twice as high as the theoretical efficiencies. The walls of the soil bin provide a rigid boundary condition in close proximity to the pile group which may explain the apparently stiffer response of these tests.

It appears that the pile groups develop resistance somewhat in accordance with elastic theory for groups, yet then fail at the same displacement, which is related to the single pile. The mechanism for this is unclear.

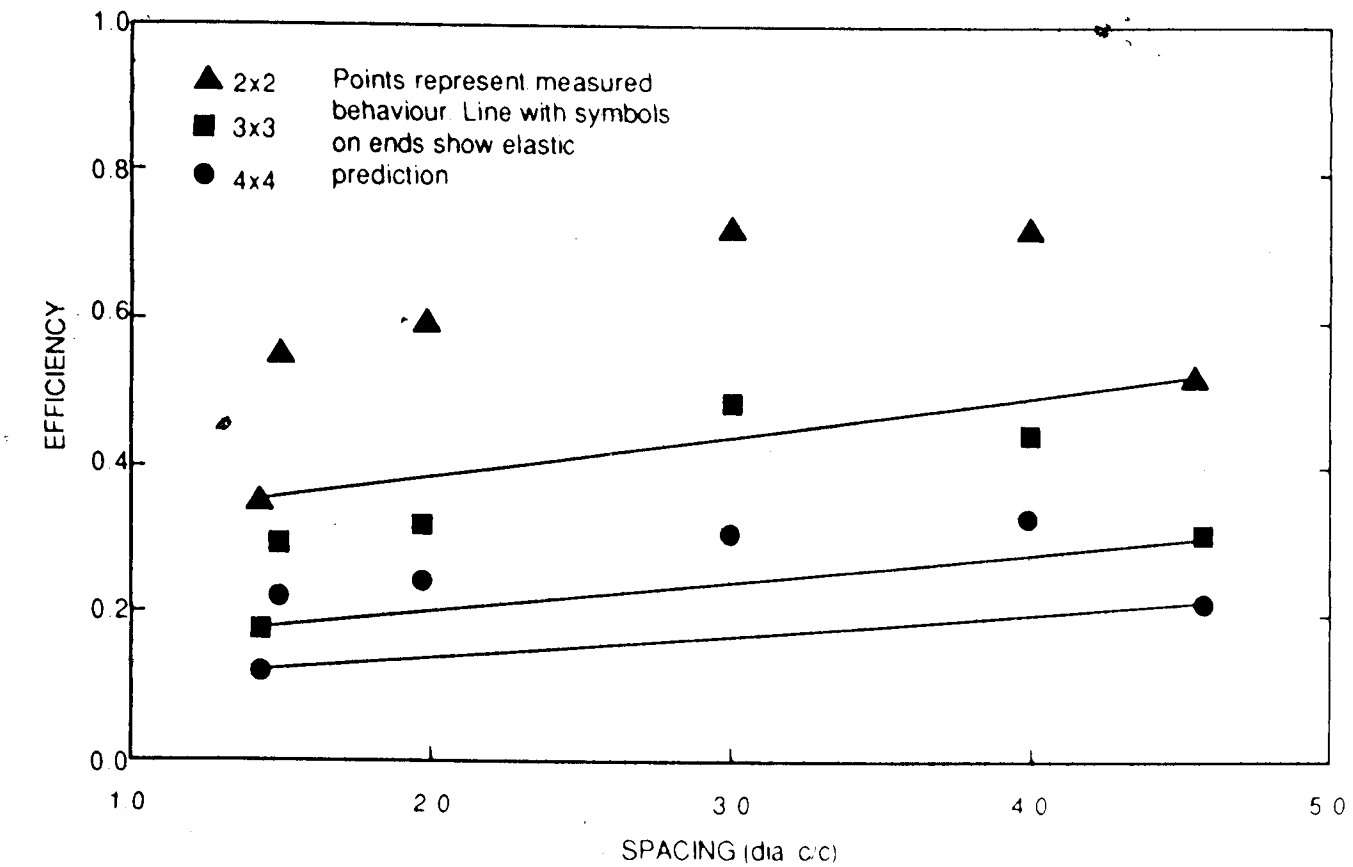


FIGURE 6.14 PILE GROUP EFFICIENCY COMPARED WITH PREDICTION OF ELASTIC THEORY WITH ASSUMPTION OF CONSTANT DISPLACEMENT TO FAILURE

7. CONCLUDING REMARKS

7.1 USING PILES TO INCREASE SLIDING RESISTANCE

Chapter 3 discusses the possible advantages of using piles beneath gravity structures to increase lateral stability. It was assumed that the effect of piles can be considered similar to the skirts or shear keys which are usually used beneath gravity structures.

Piles may have some advantages over skirts in developing shear resistance. Because they are not built on to the base of the structure there is more flexibility in designing for varying soil conditions. Concerns about whether the pile or skirt can be installed to the design depth are lessened. Even more advantageously, piles can be designed for deeper tip penetrations than skirts. This means that stronger soils, if they occur at depth, can be utilized in the design.

Also in Chapter 3 the possible benefits of installing piles to greater depths than normally attained by the use of skirts was investigated. Piles between 1 and 2m in diameter cannot be installed to deeper than 20m with the expectation that they will behave as short rigid piles. The maximum increase in sliding resistance in uniform soils for this length of pile would amount to only about 25 percent. More substantial increases (in percentage terms) would only be possible for smaller structures utilizing very large diameter, thick walled sections. In most cases the ultimate bearing capacity under inclined loads will not be affected.

For site stratigraphies where an increase in soil strength can be expected within the depth of the piles, greater benefits could be expected. In these cases it would be expected that the usual design goal would be to develop a sliding resistance close to that which could be obtained by forcing the shear plane beneath the pile tips. It was to determine whether this goal was attainable that the experiments that form this thesis were performed.

7.2 GROUP INTERACTION IN LATERALLY LOADED PILES

The hypothesis was made at the beginning of this work that the ultimate horizontal resistance of a group of piles could be bounded, at distant spacings, by the sum of the resistances of the individual piles, and at close spacings by the resistance of the block which enclosed the group. Indications from previous work was divergent, but most investigators seemed to indicate that this was true, and that the transition from a single pile failure mode to a block mode was quite distinct.

The indication from model testing performed for this thesis indicates that this is not true, at least for the conditions of these experiments. On the contrary , it appears that neither the block capacity predicted from perfectly plastic mechanisms nor the single pile capacity can be mobilized. Extrapolating the trend of the results indicates that complete independent action of the piles could not take place until the piles were spaced on the order of 20 diameters centre to centre. Yet even at the spacings used in the model tests the measured resistance was below the predicted resistance of the enclosing block.

Even though the results of these tests were unexpected, they are not really in conflict with previous model tests since most had been carried out on groups of four or fewer piles. Efficiencies measured for the four pile groups in these tests were relatively high, it was for the larger groups that the very low efficiencies were measured. Cox et al.(1984) noticed a similar result during their tests on rows loaded in-line, with larger groups exhibiting lower efficiencies.

Despite this, the results of the tests differ from accepted judgement of the capacity of horizontally loaded pile groups. Potentially, the capacity of many laterally loaded groups may have to be re-evaluated. It is important that the results of these model tests are fully understood.

7.3 POSSIBLE CAUSES FOR LOW MEASURED CAPACITY

Various characteristics of the test results have been discussed in an effort to explain the behaviour observed. It has not been possible to arrive at any conclusions. Further investigation is recommended, concentrating on the possible failure mechanisms discussed in section 6.5.2. The apparent discrepancy between observed characteristics of soil deformation, some indicating individual pile behaviour, some more consistent with block behaviour, creates the most difficulty in interpreting the results. However it is believed that special consideration of the effects of the tension crack formation on the development of the horizontal capacity of the group would be fruitful. Other possible mechanisms suggested include a "shadow" effect from the front row of piles, development of progressive failure iver the boundaries of the enclosing group, and the combination of a displacement related failure mechanism with a strain controlled development of load.

It is suggested that a numerical analysis program should be carried out. The analysis should initially attempt to reproduce the observed mechanism. For this reason a program which is capable of modelling tension crack development would be required.

The results of such an analysis should serve to clarify many of the issues raised by these tests. In particular, once confidence in the analysis was established by matching the model test results, the analysis could be extended to analysing proposed full scale structures. Parametric analyses could be conducted to establish the conditions under which a full scale groups of piles would experience the low efficiencies observed for the model.

7.4 MODEL TESTS IN COMPACTED CLAY

Section 5.8 discusses the problems encountered preparing the soil. A primary recomendation for subsequent researchers is to avoid having to handle such a large amount of material. Proper control of the properties of the material, notably its moisture content, becomes extremely difficult. Placement becomes a long arduous process.

A major objective of the model testing program was to determine mechanisms and modes of behaviour. This was accomplished. The testing program has shown that under some conditions overall translation of the soil mass can take place to larger spacings than previously appreciated. Development of tension cracks around larger groups has been shown to occur. As mentioned before, some elements of the deformation behaviour are difficult to interpret. These will be resolved when the the low efficiencies measured are explained.

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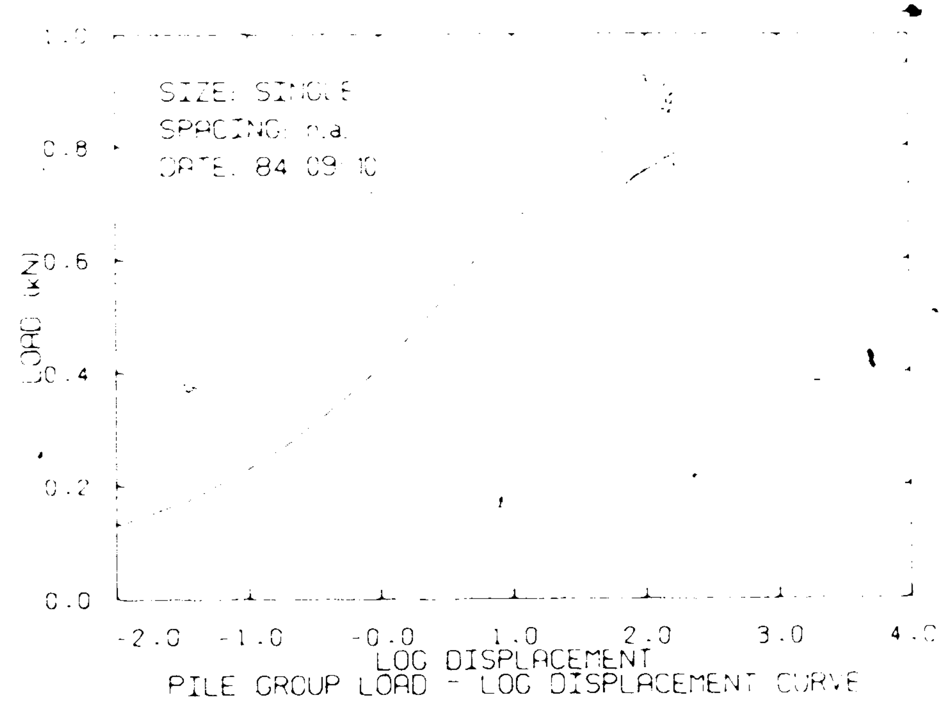
APPENDIX A

Load-Displacement Curves Test Suite #1

On the following pages are presented the load-displacement curves for the single piles and pile groups tested during the first suite of tests. These tests were conducted from August to October 1984. A total of eight bins of soil were prepared and fifteen load tests were conducted. Two of these tests were on a single pile, and the remaining thirteen were conducted on groups. Twelve configurations were tested. The three by three group at the 1.5 diameter spacing was tested twice. The first test on this pile group was preliminary and the properties of the soil were not completely determined.

Each figure presents the load-displacement curve for the test plotted in two ways: the upper figure presents the load as a function of the natural logarithm of displacement and the lower curve presents the curve on a natural scale. The reader is referred to section 6.1.1 for a discussion of how the load-displacement curves were interpreted.



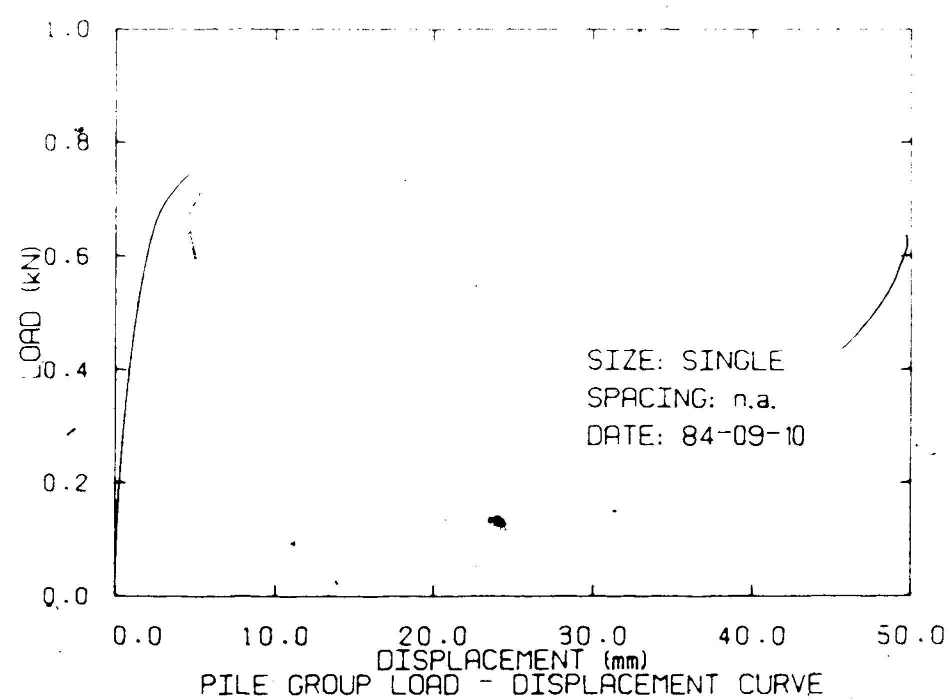
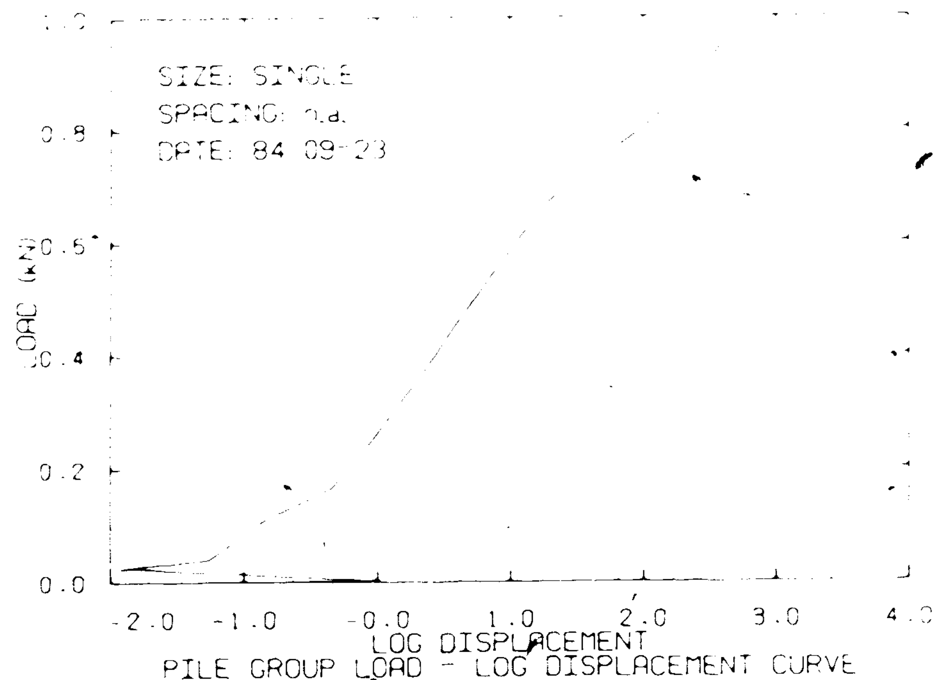


Figure A.1 Load-Deflection Curve for Single Pile Test #1



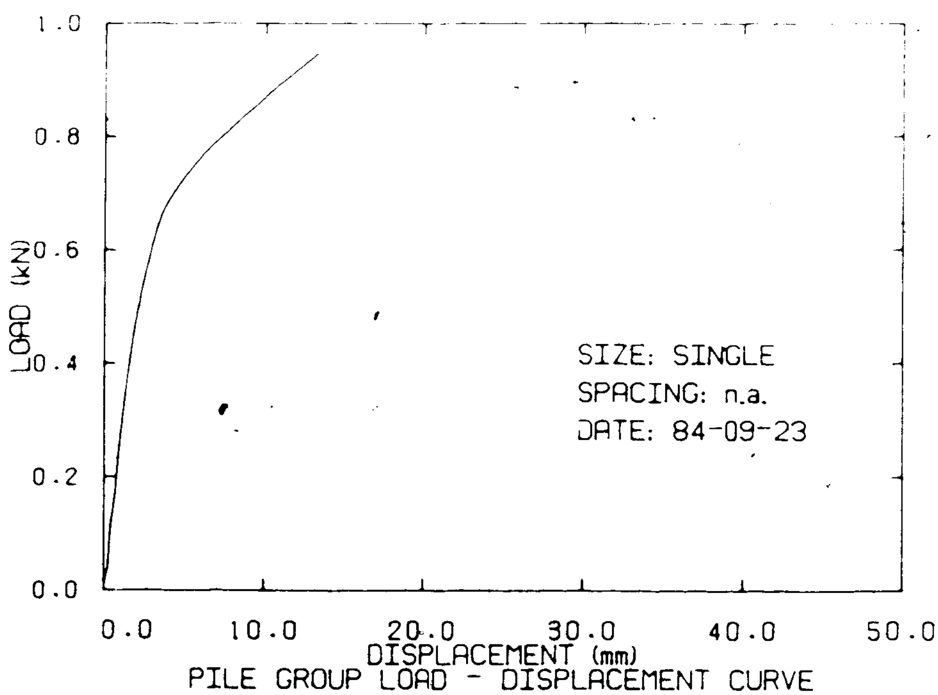
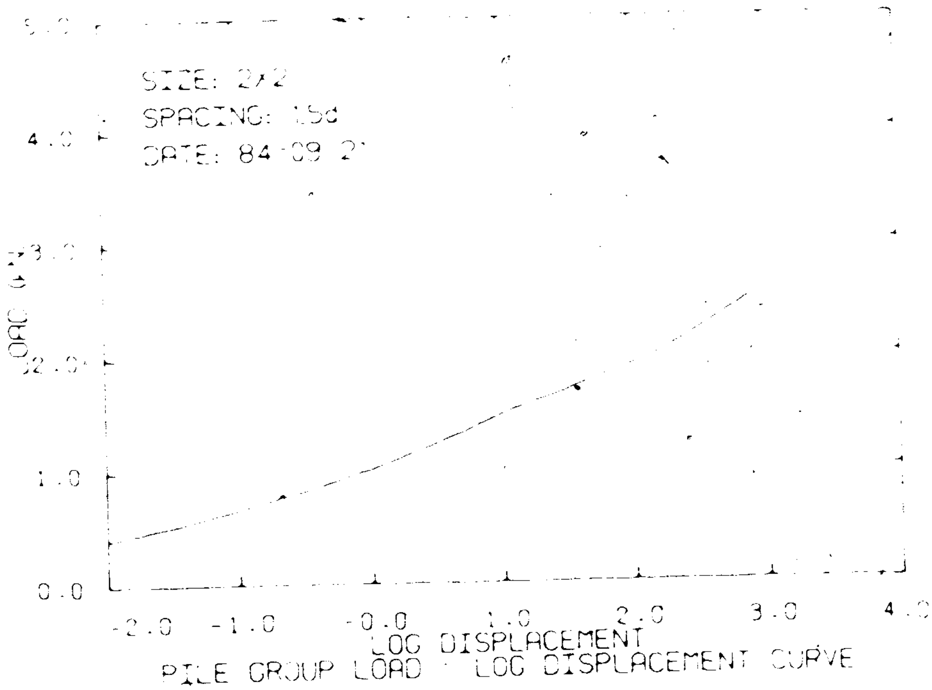


Figure A.2 Load-Deflection Curve for Single Pile Test #2



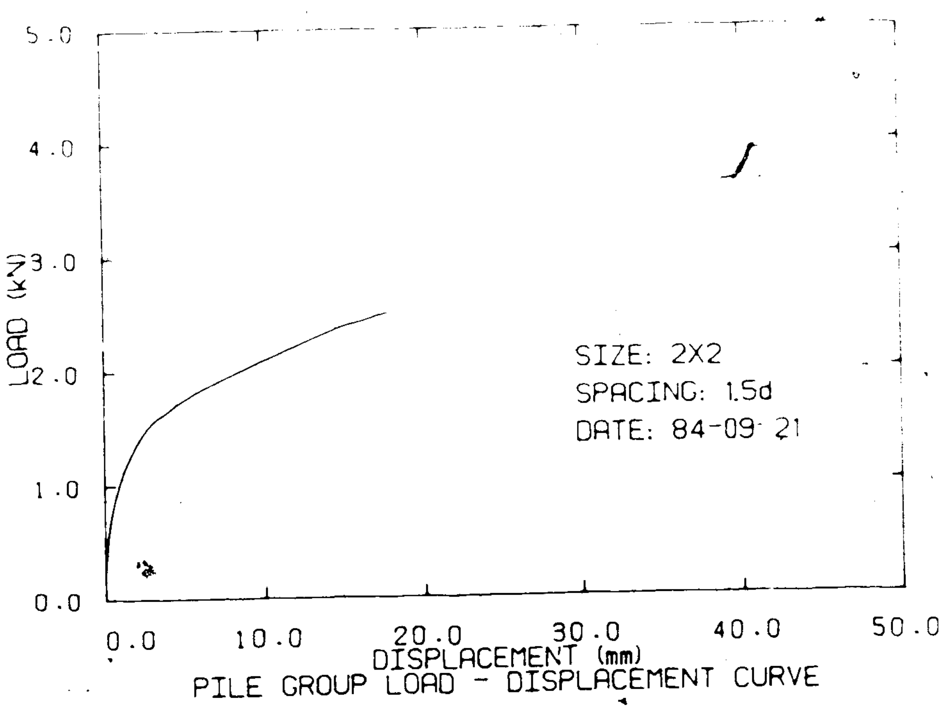
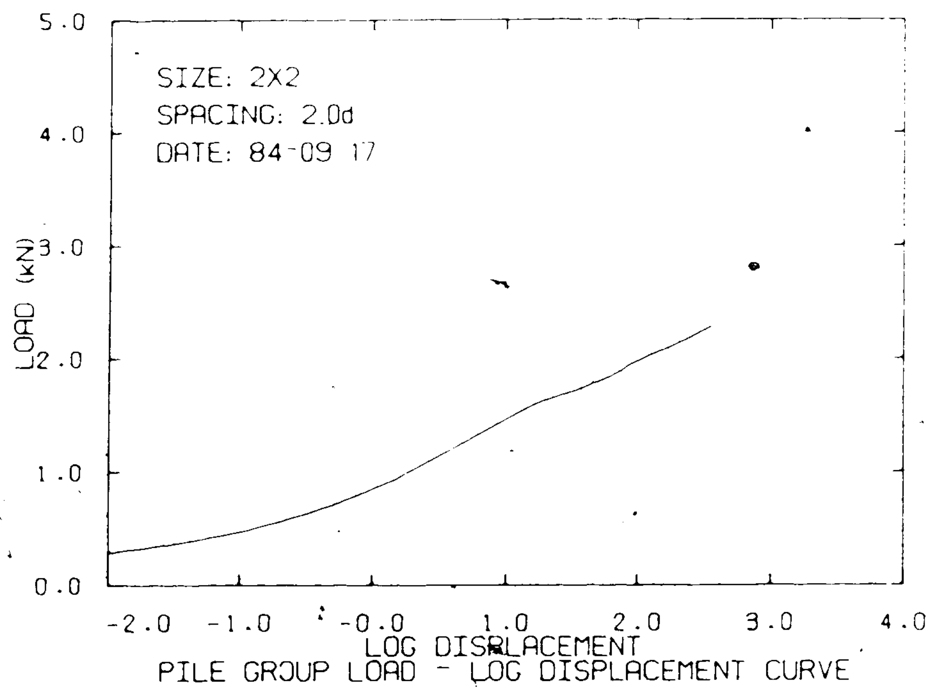


Figure A.3 Load-Deflection Curve for Pile Group 2x2 at 1.5d



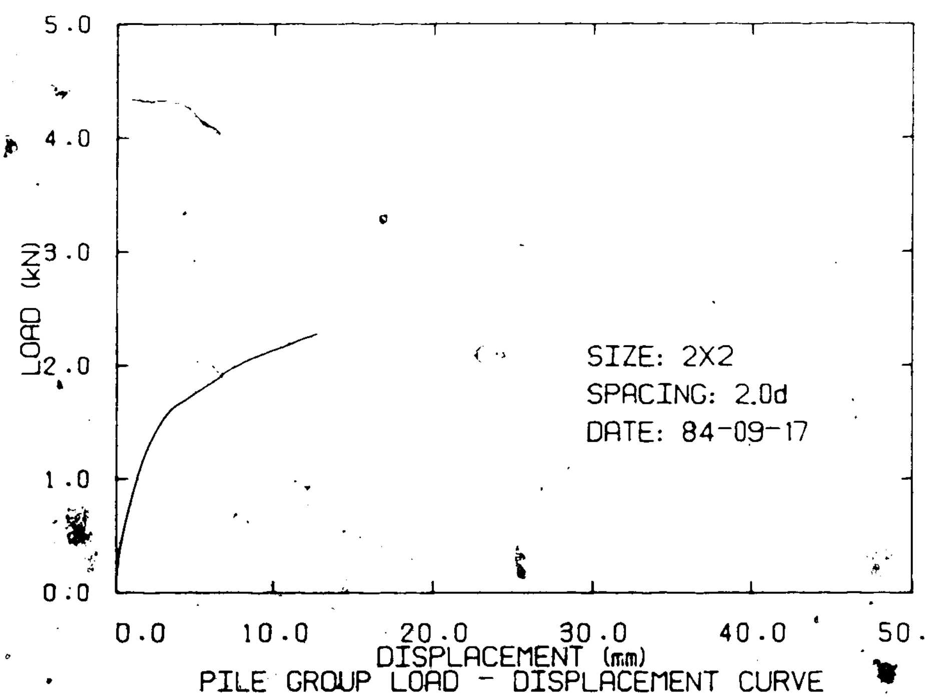
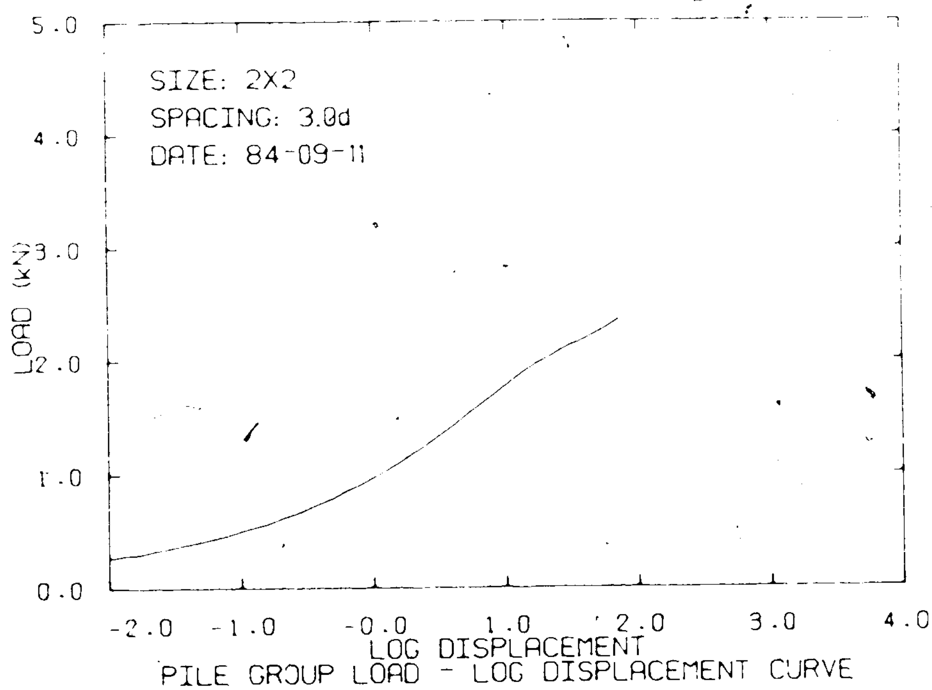


Figure A.4 Load-Deflection Curve for Pile Group 2x2 at 2.0d



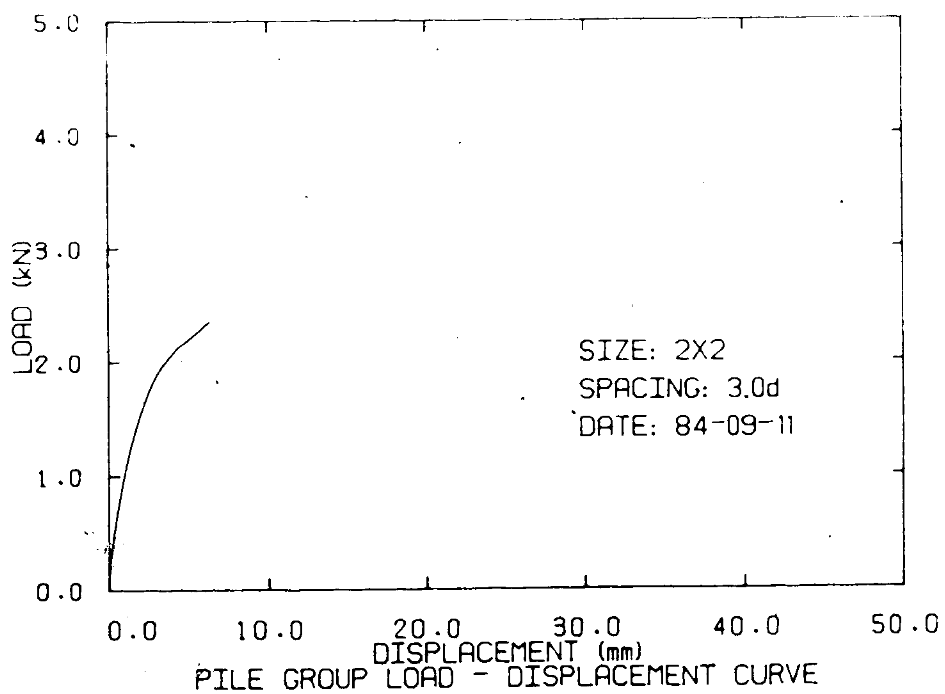
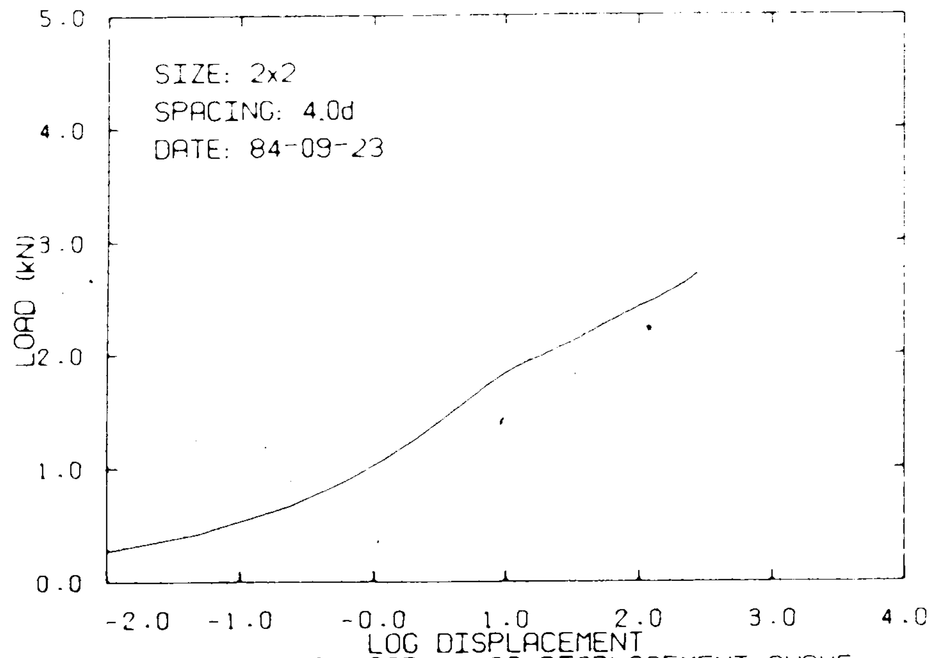
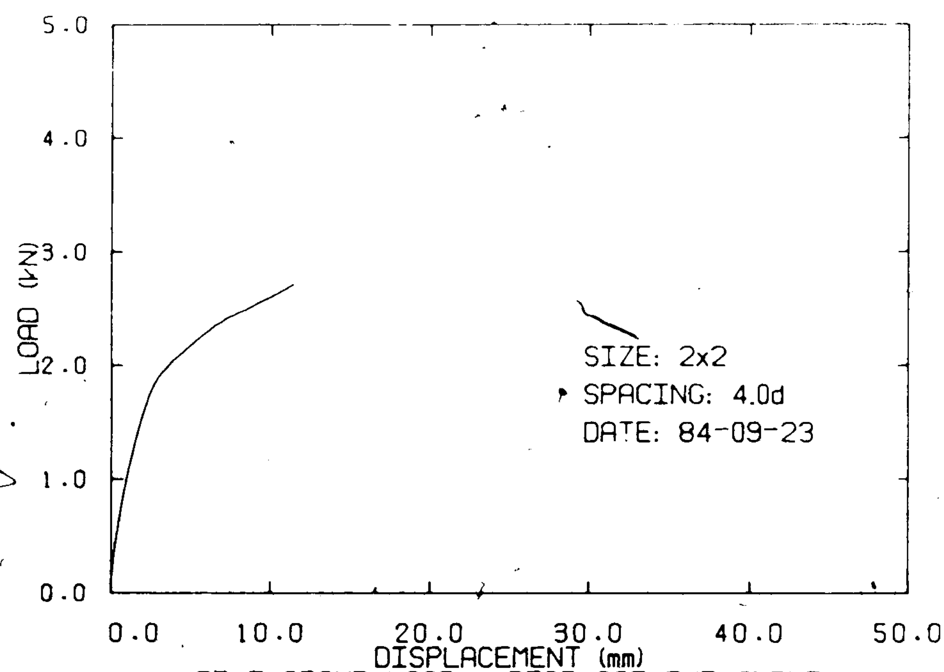


Figure A.5 Load-Deflection Curve for Pile Group 2x2 at 3.0d

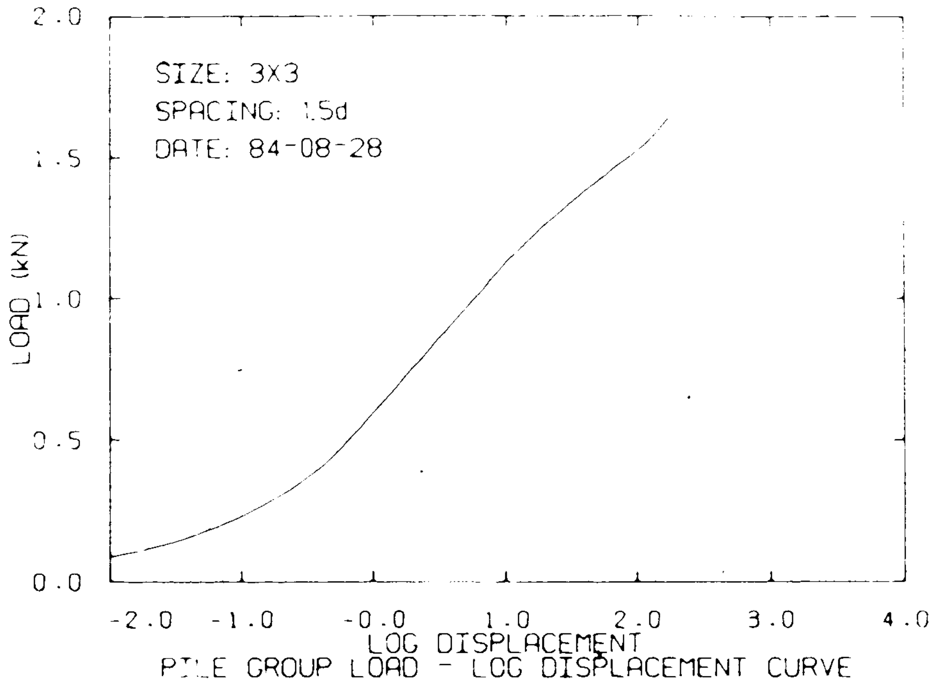


PILE GROUP LOAD - LOG DISPLACEMENT CURVE



PILE GROUP LOAD - DISPLACEMENT CURVE

Figure A.6 Load-Deflection Curve for Pile Group 2x2 at 4.0d



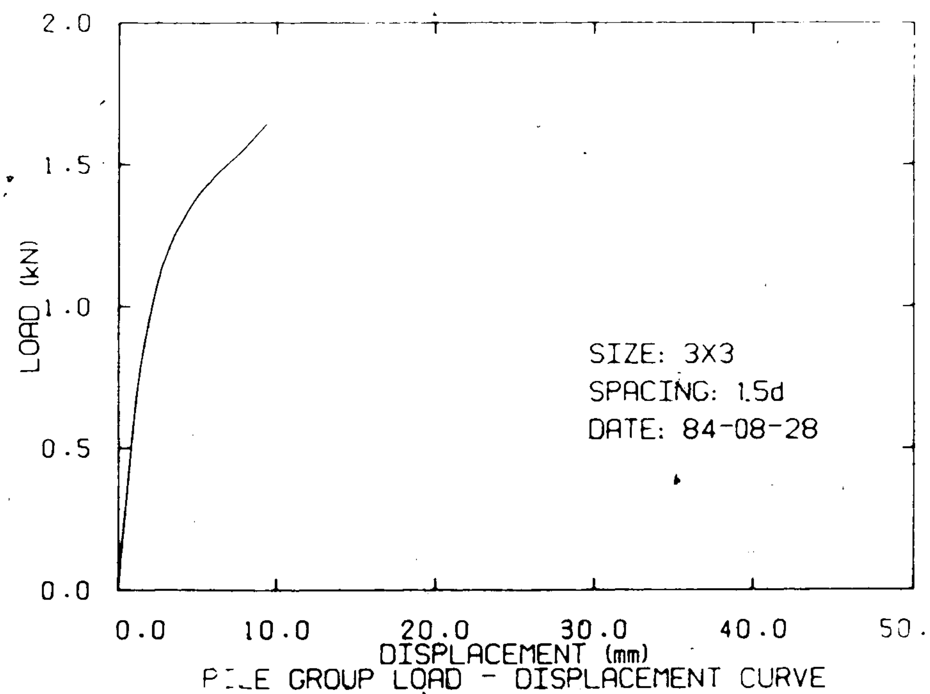
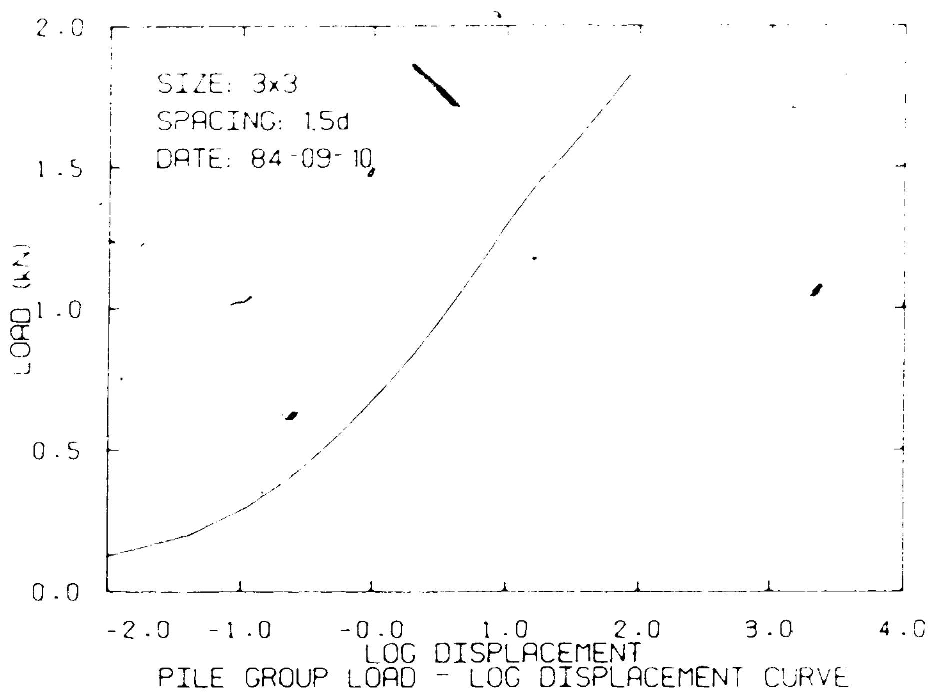


Figure A.7 Load-Deflection Curve for Pile Group 3x3 at 1.5d preliminary



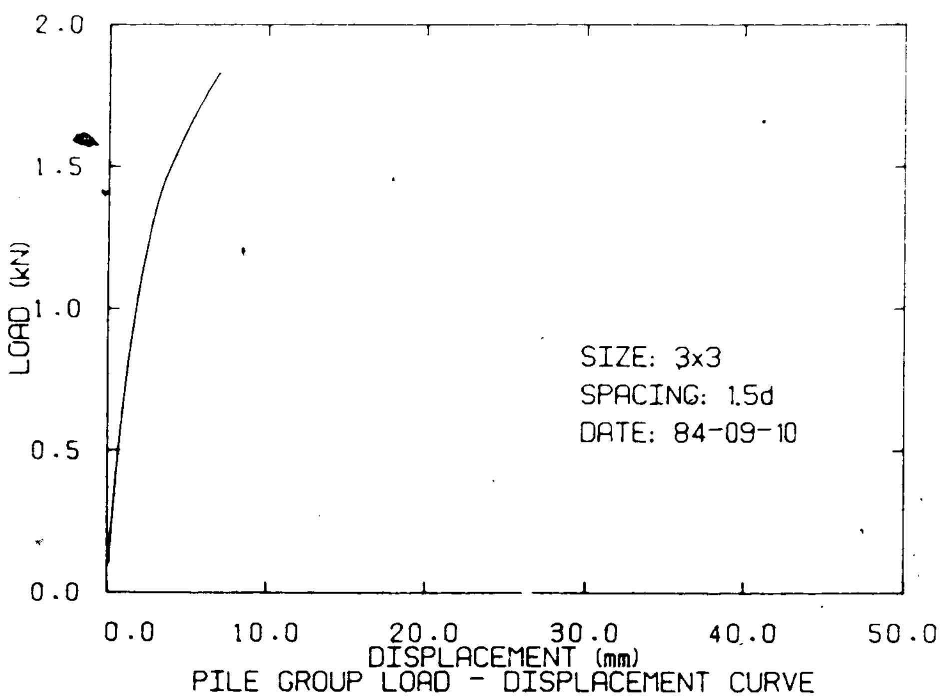
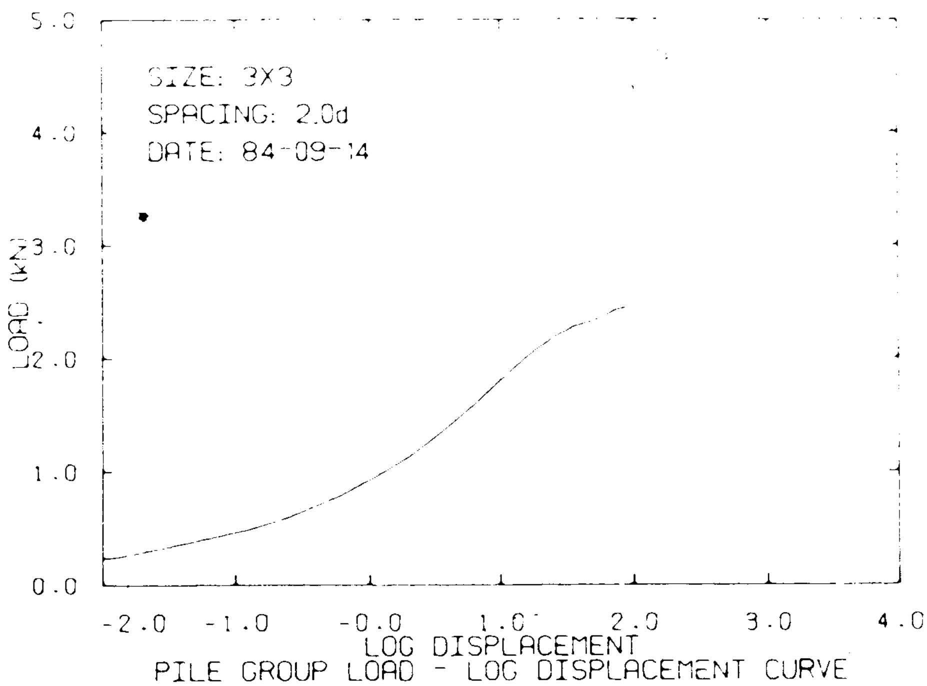


Figure A.8 Load-Deflection Curve for Pile Group 3x3 at 1.5d



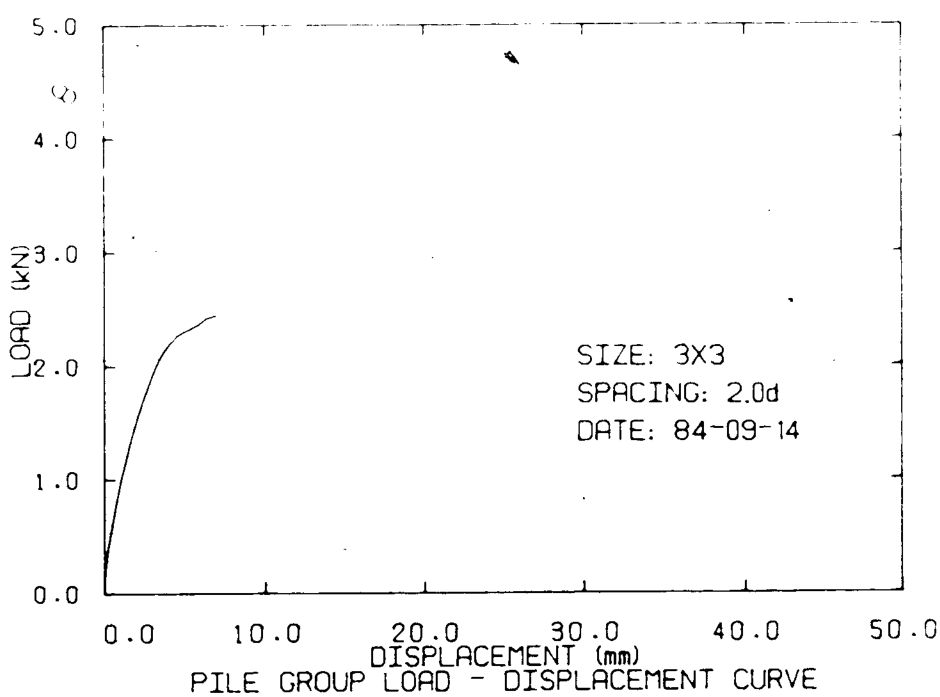
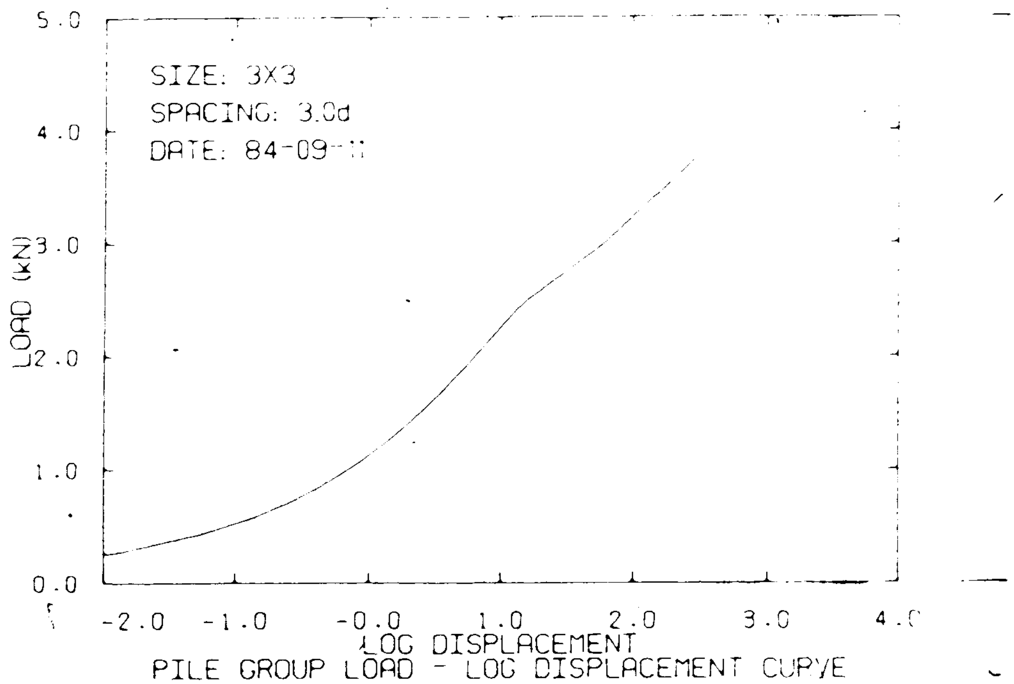


Figure A.9 Load-Deflection Curve for Pile Group 3x3 at 2.0d



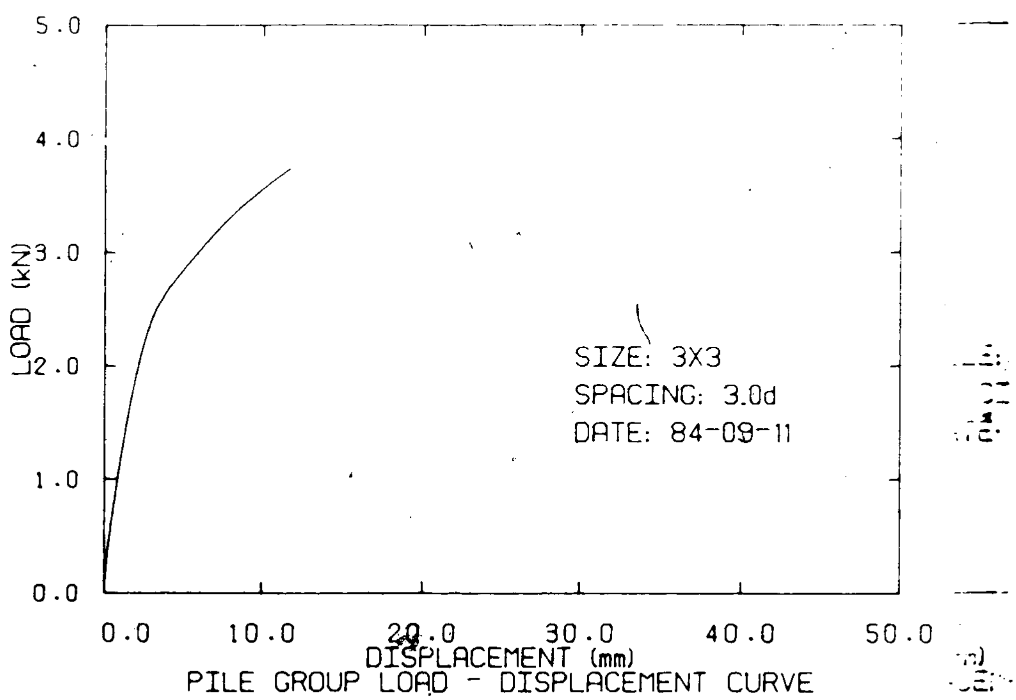


Figure A.10 Load-Deflection Curve for Pile Group 3x3 at 3.0d

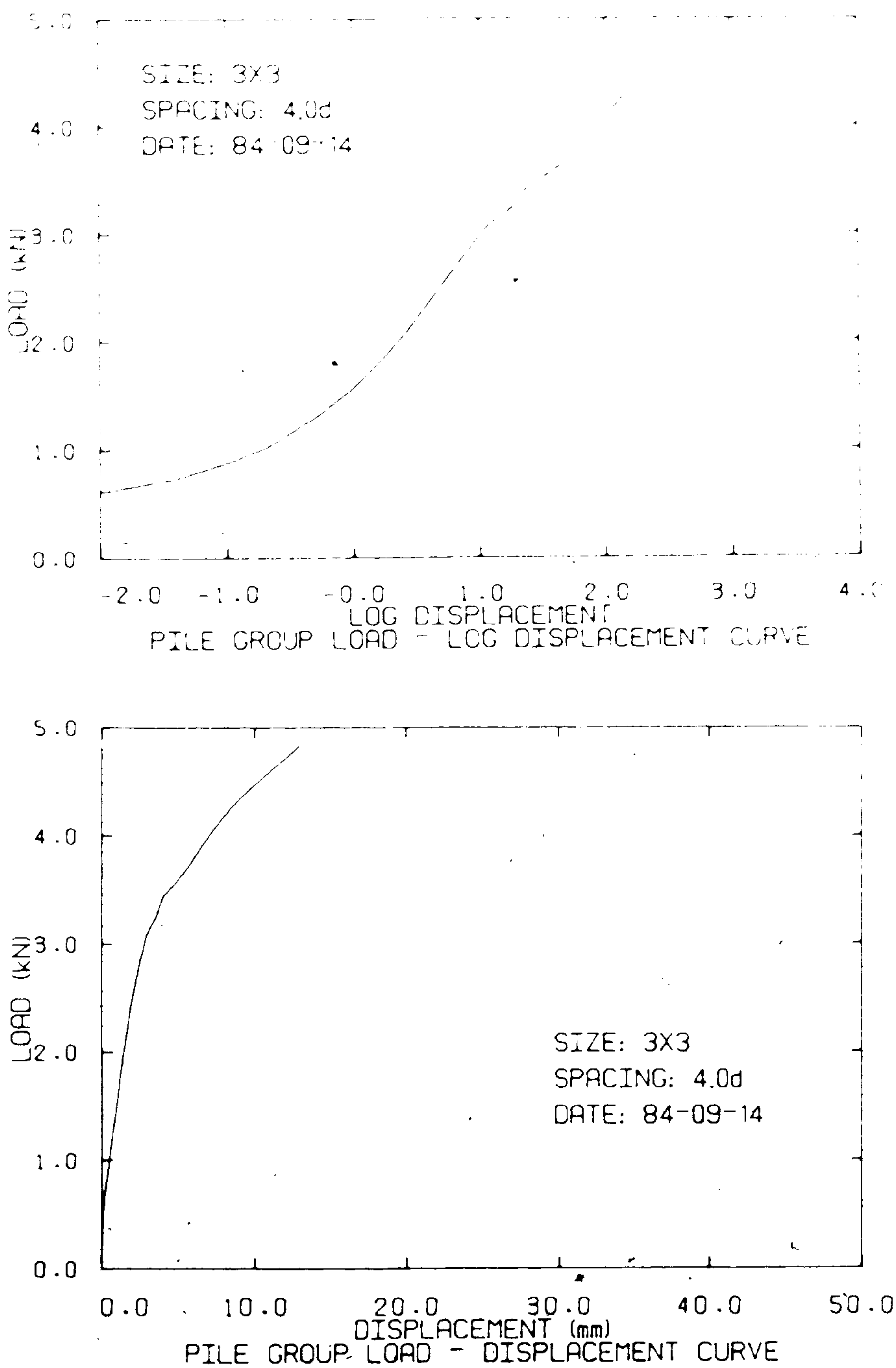
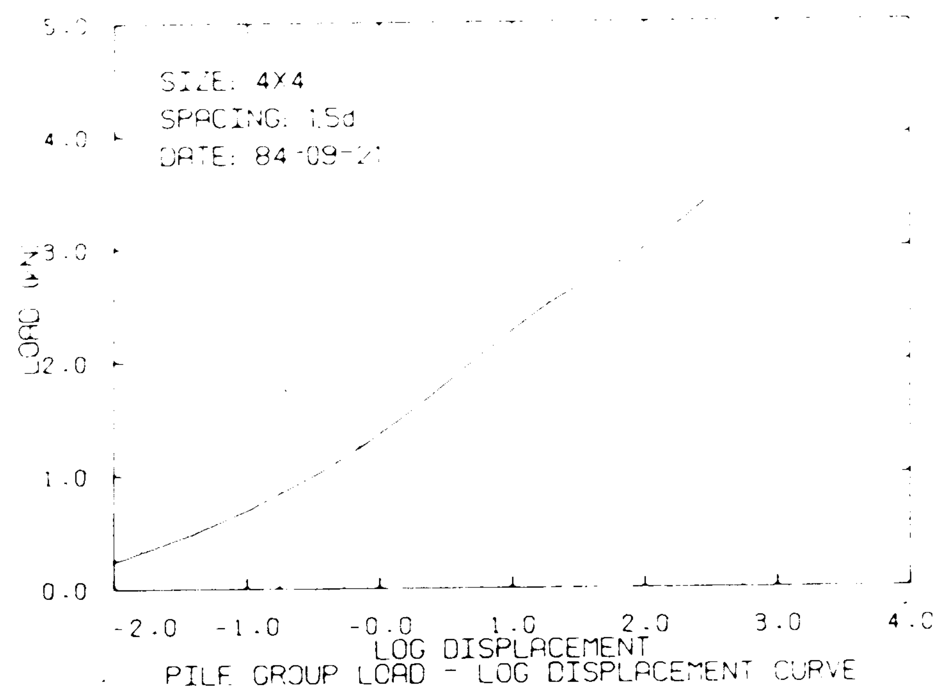


Figure A.11 Load-Deflection Curve for Pile Group 3x3 at 4.0d



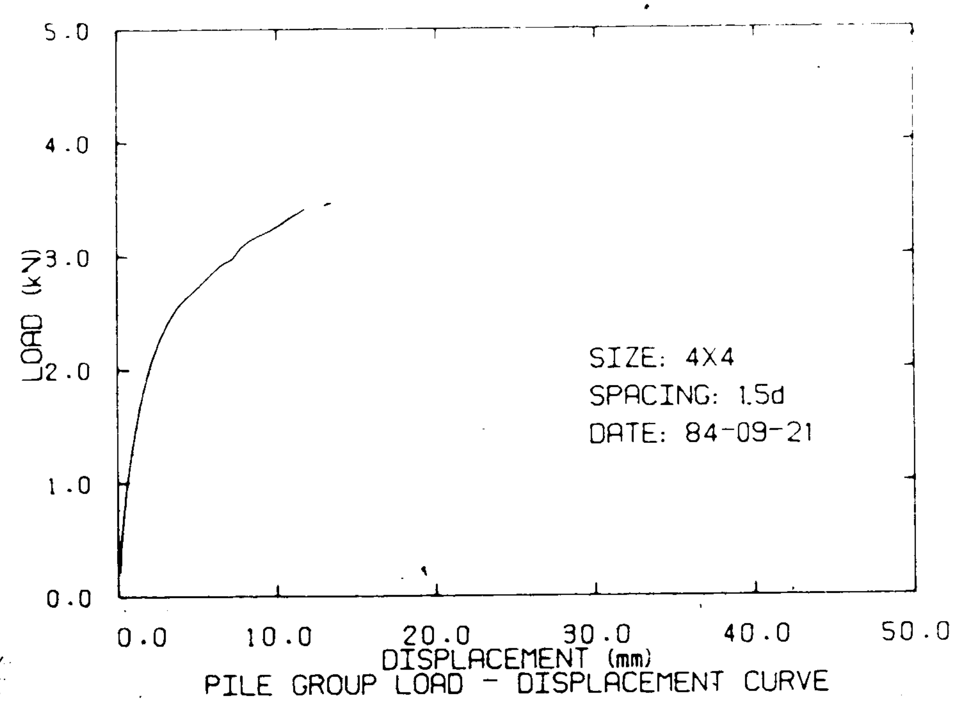
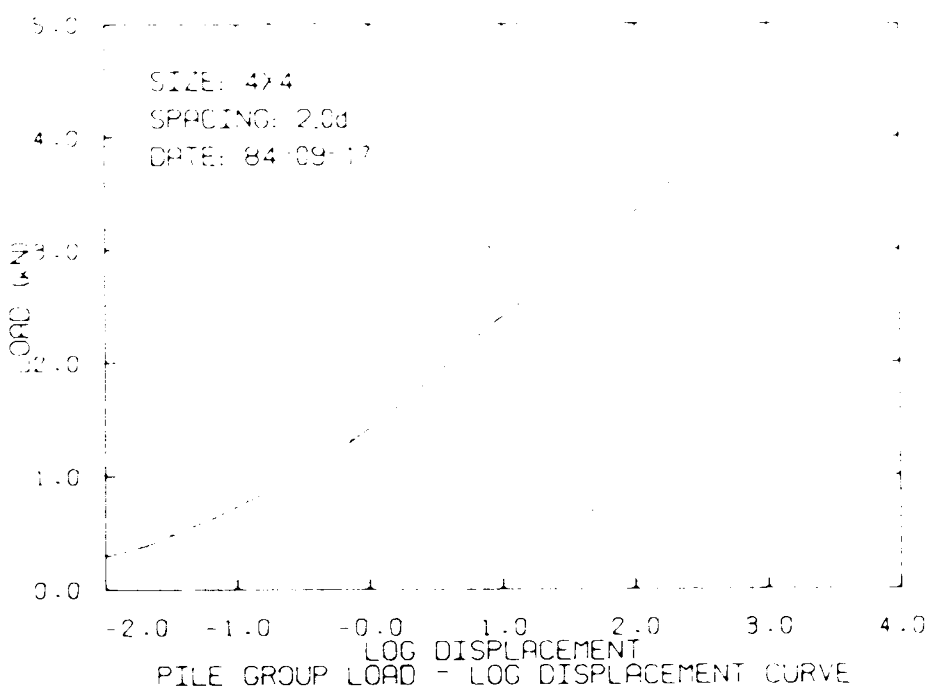


Figure A.12 Load-Deflection Curve for Pile Group 4x4 at 1.5d



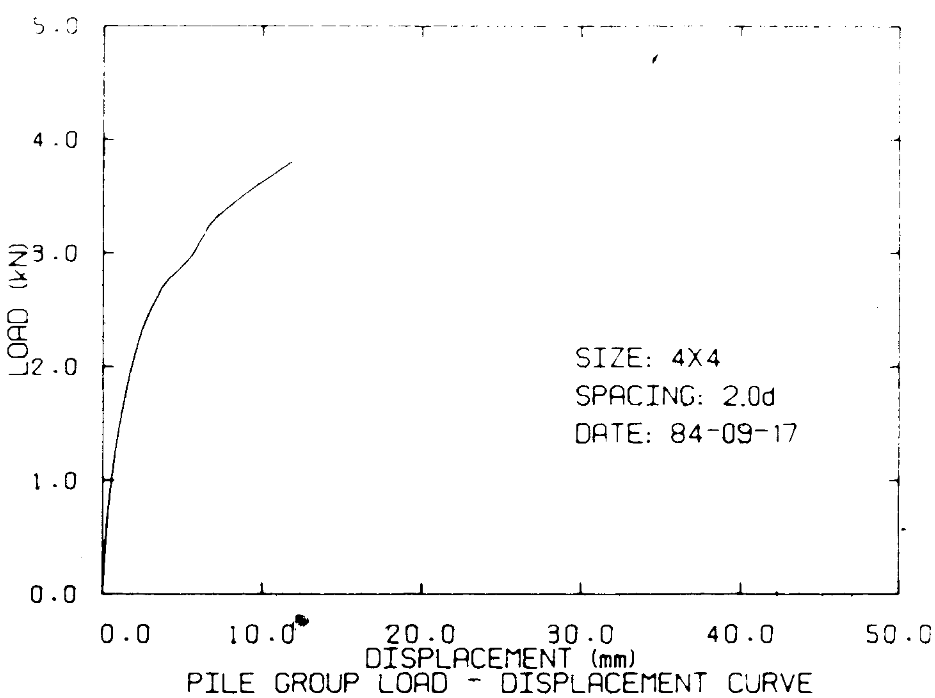
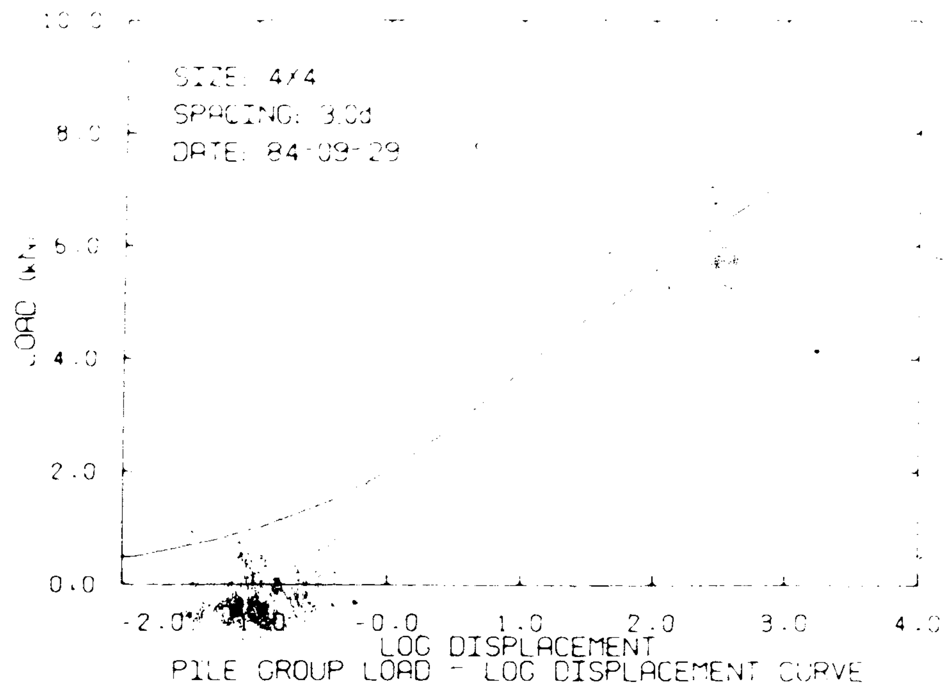


Figure A.13 Load-Deflection Curve for Pile Group 4x4 at 2.0d



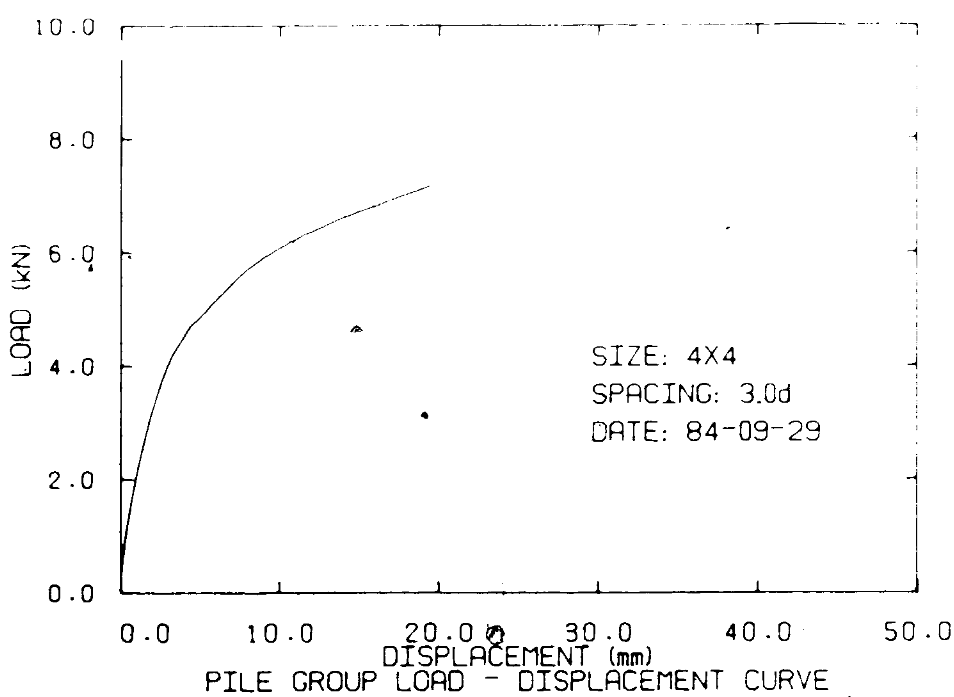


Figure A.14 Load-Deflection Curve for Pile Group 4x4 at 3.0d

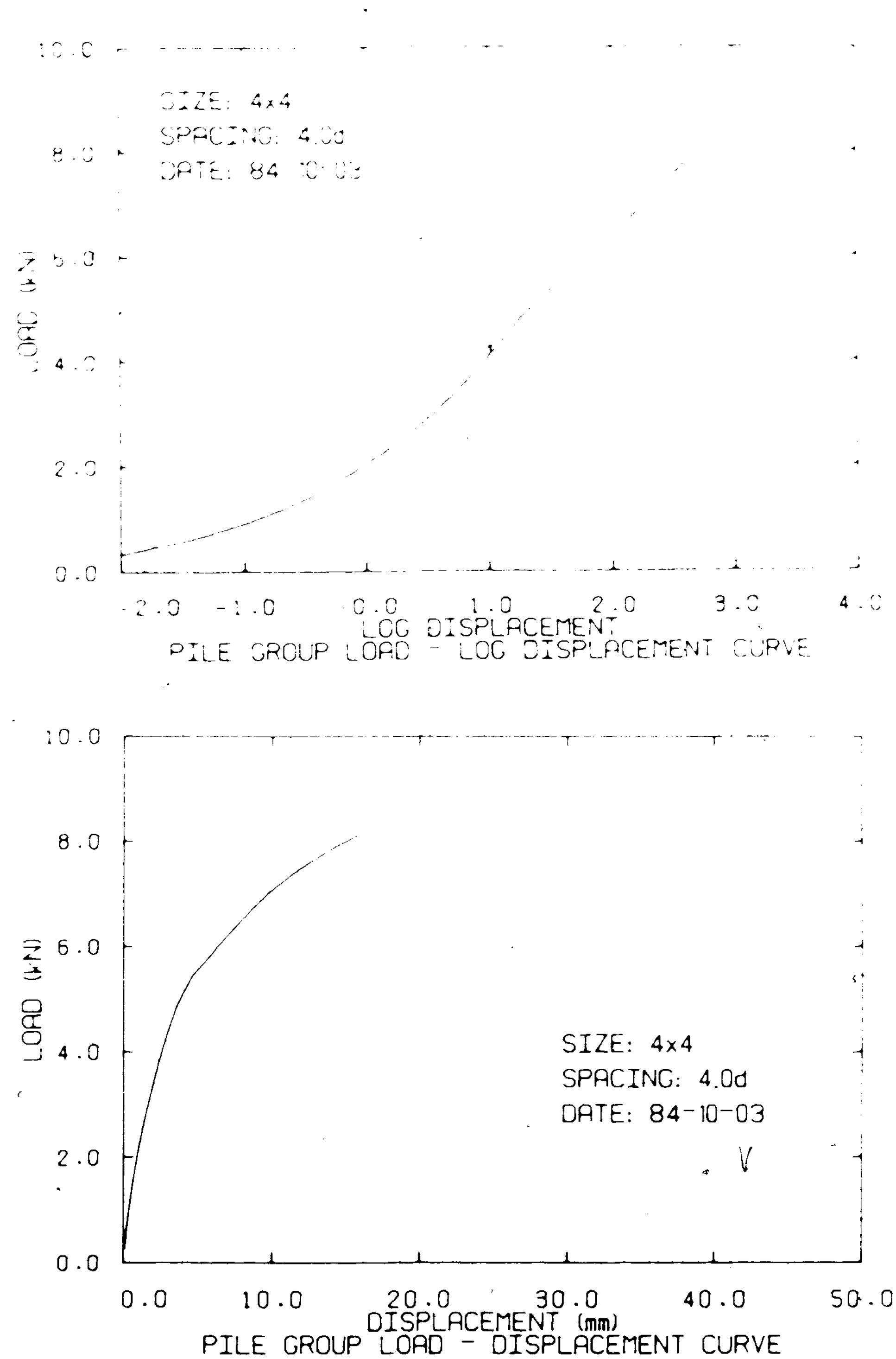


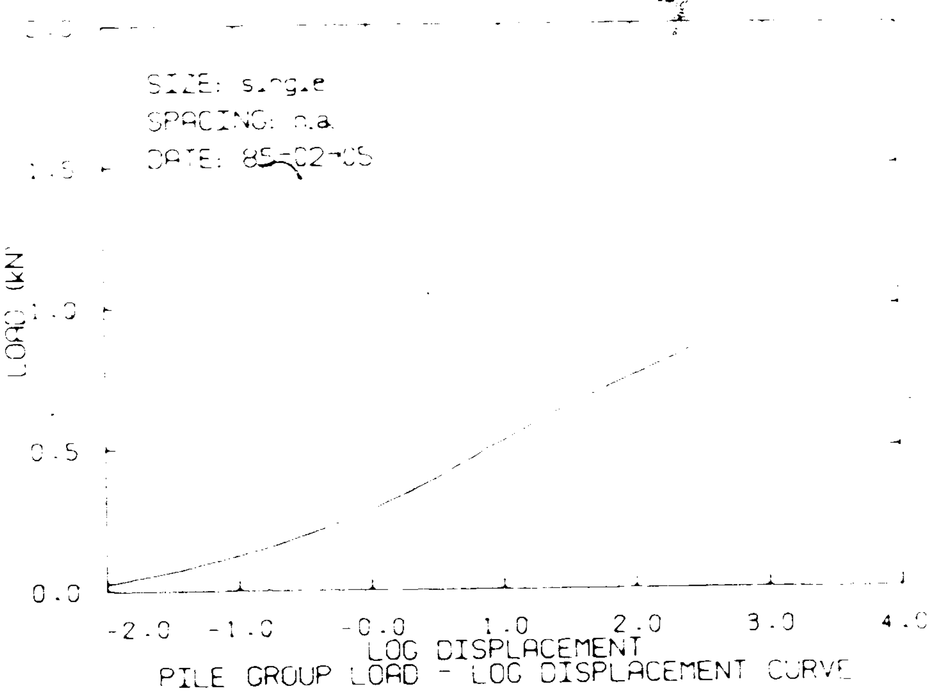
Figure A.15 Load-Deflection Curve for Pile Group 4x4 at 4.0d

APPENDIX B

Load-Displacement Curves Test Suite #2

On the following pages the load-displacement curves for the single piles and pile groups tested during the second suite of tests are presented. These tests were conducted from February to April 1985. A total of 11 bins of soil were prepared and 23 load tests were conducted. In each bin a single pile was tested. Then a pile group was tested. On April 4, 1985 a single pile, and the 2x2 groups spaced at 1.5 and two diameters were tested. The single pile tests are presented first on Figures B.1 to B.11. The group tests follow on Figures B.12 to B.23. The group test and its companion single pile test were always conducted on the same day so that the single pile test corresponding to a specific group test can be found from the test date reported. The single pile tests, Figures B.1 to B.11 are presented in the order of the date of test. The pile group tests are ordered by group size, 2x2 to 4x4, and within each group size, by spacing, 1.5 to 4.0 diameters.

Each figure presents the load-displacement curve for the test plotted in two ways: the upper figure presents the load as a function of the natural logarithm of displacement and the lower curve presents the curve on a natural scale. The reader is referred to section 6.1.1 for a discussion of how the load-displacement curves were interpreted.



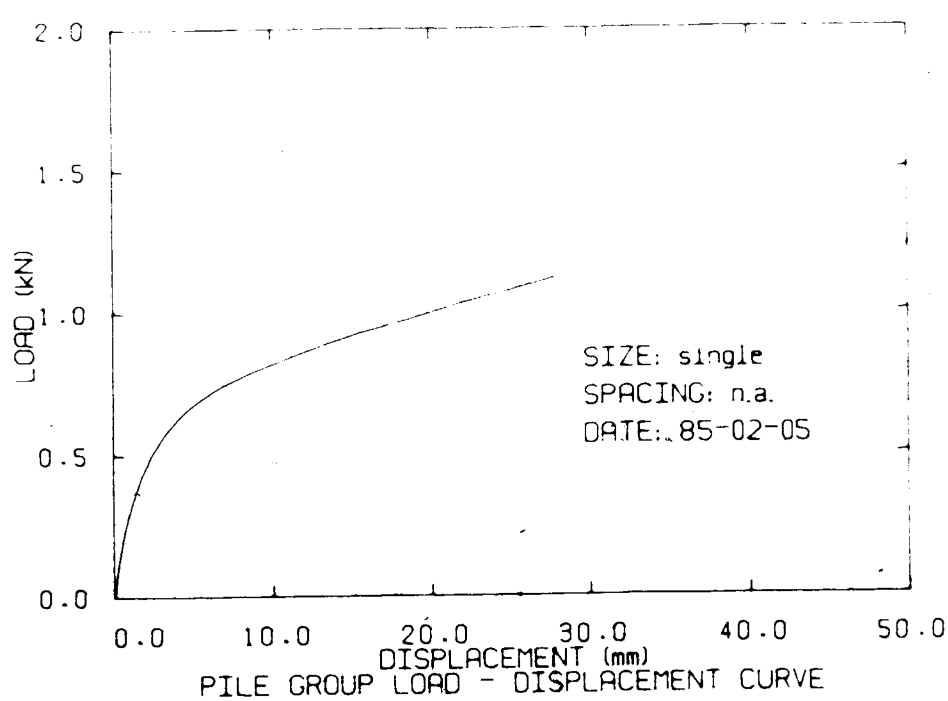
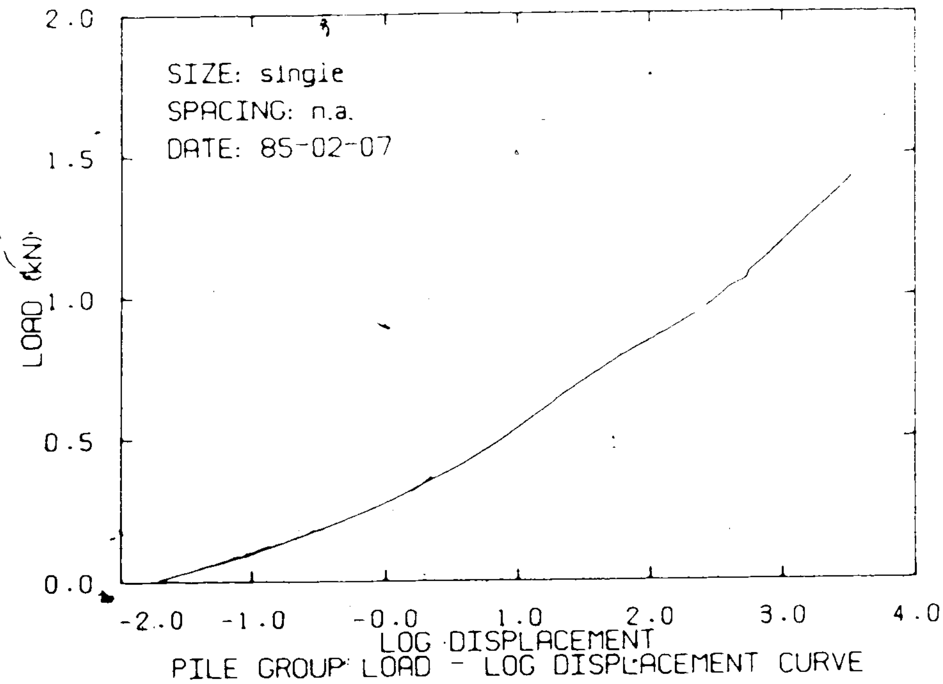


Figure B.1 Load-Deflection Curve for Single Pile Test 85-02-05



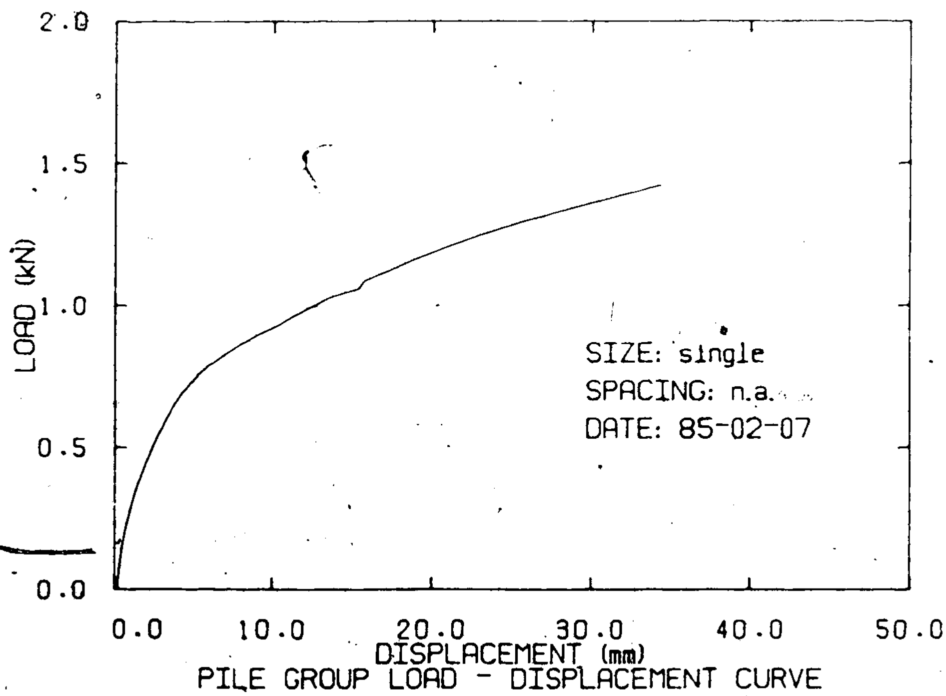
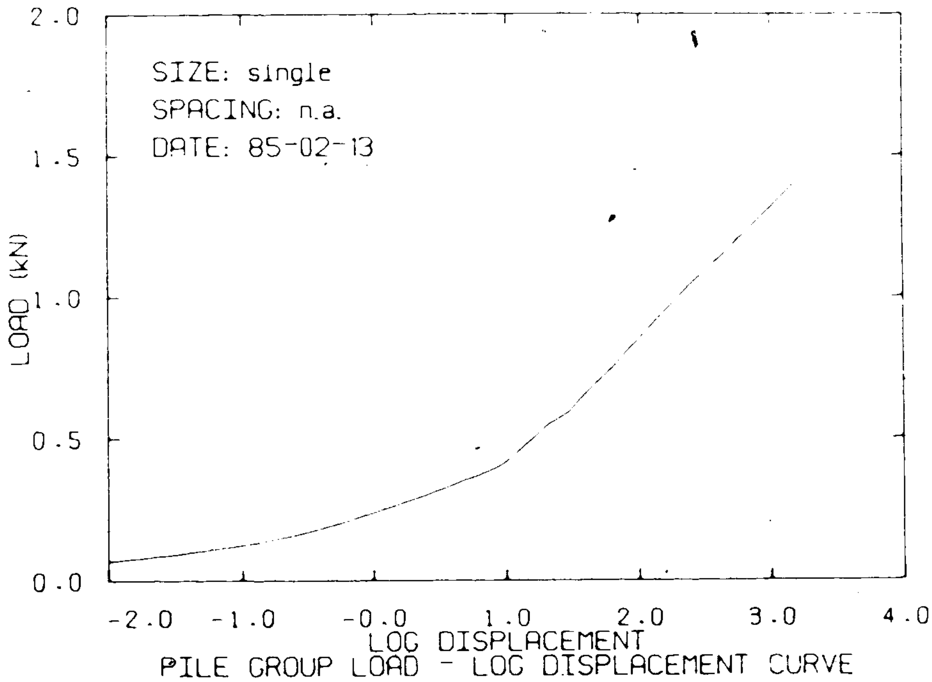


Figure B.2 Load-Deflection Curve for Single Pile Test 85-02-07



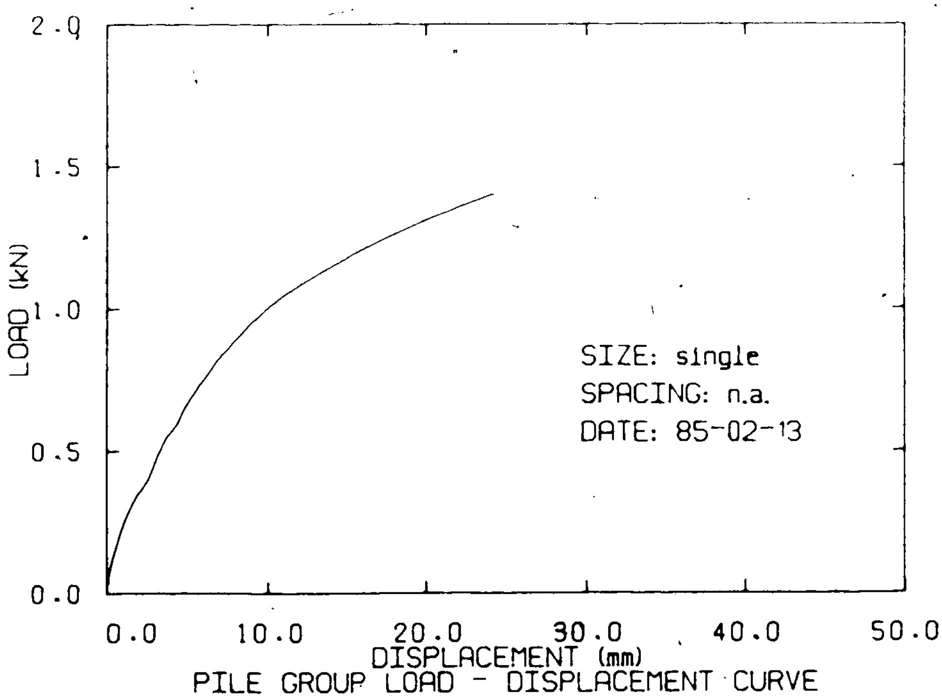
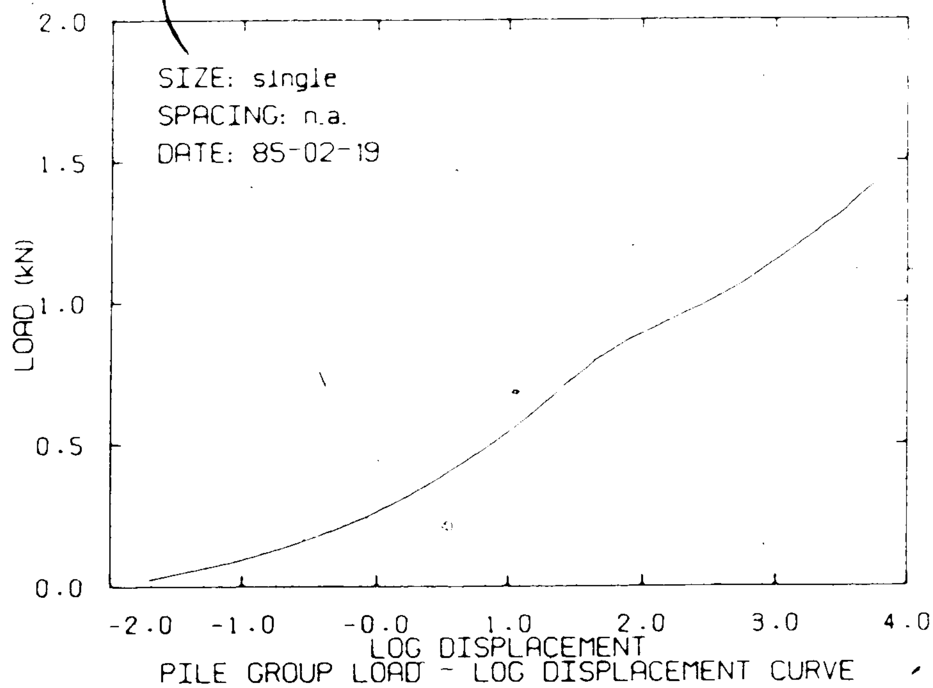


Figure B.3 Load-Deflection Curve for Single Pile Test 85-02-13



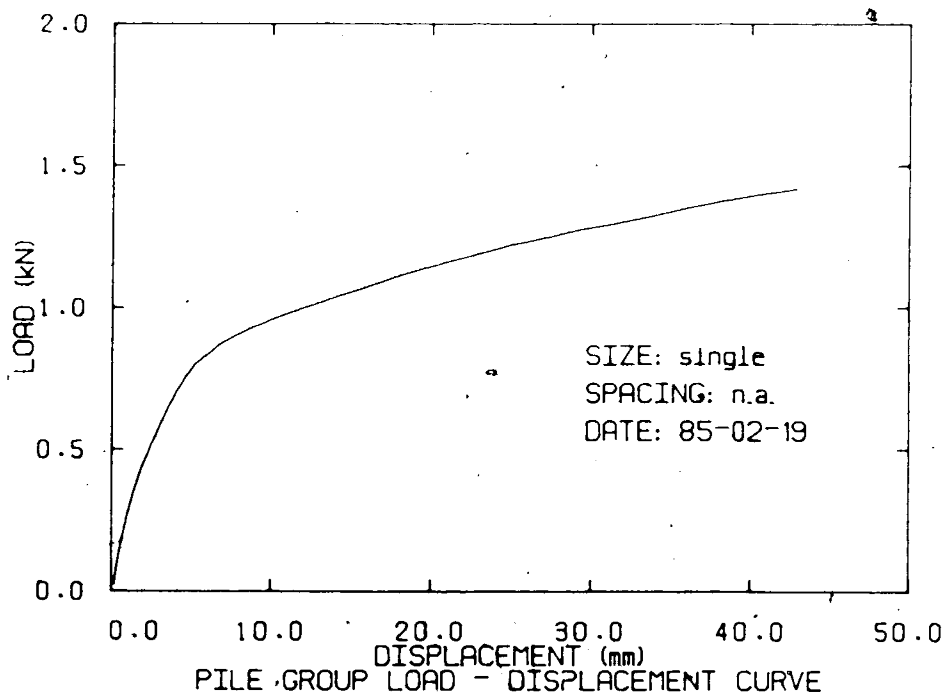
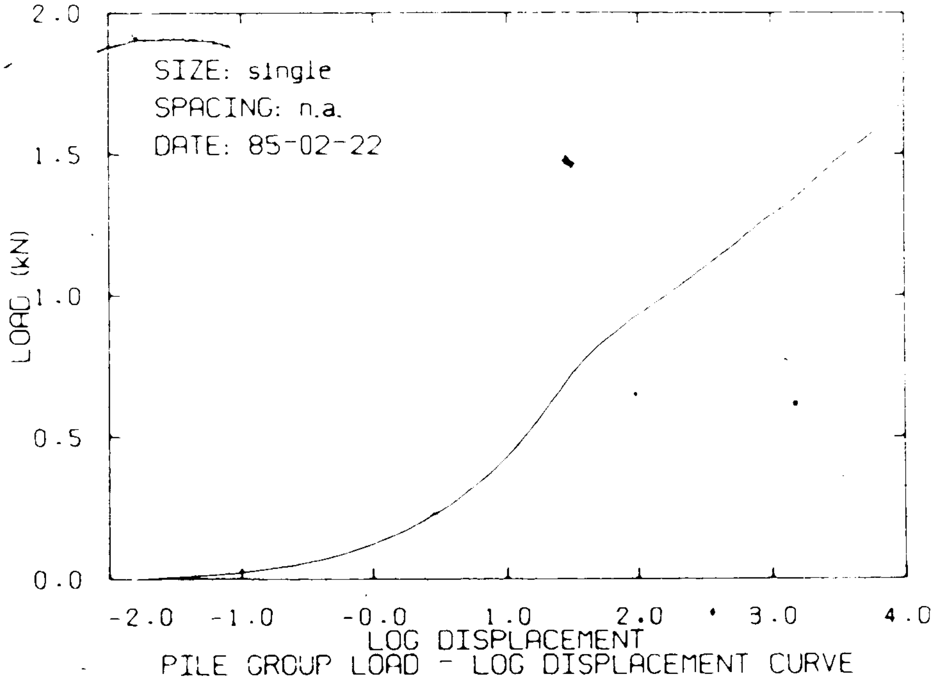


Figure B.4 Load-Deflection Curve for Single Pile Test 85-02-19



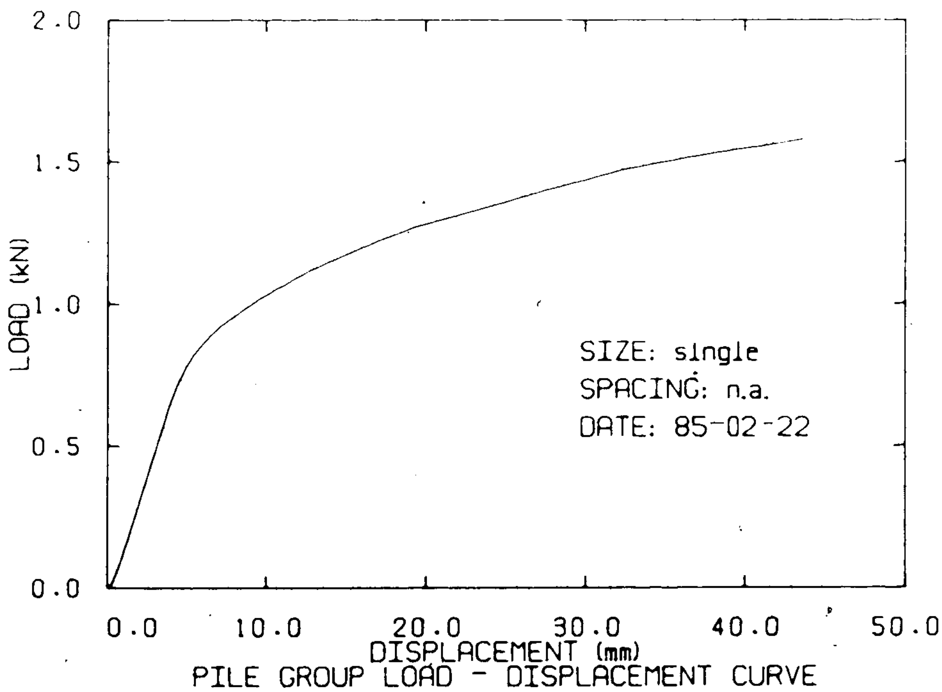
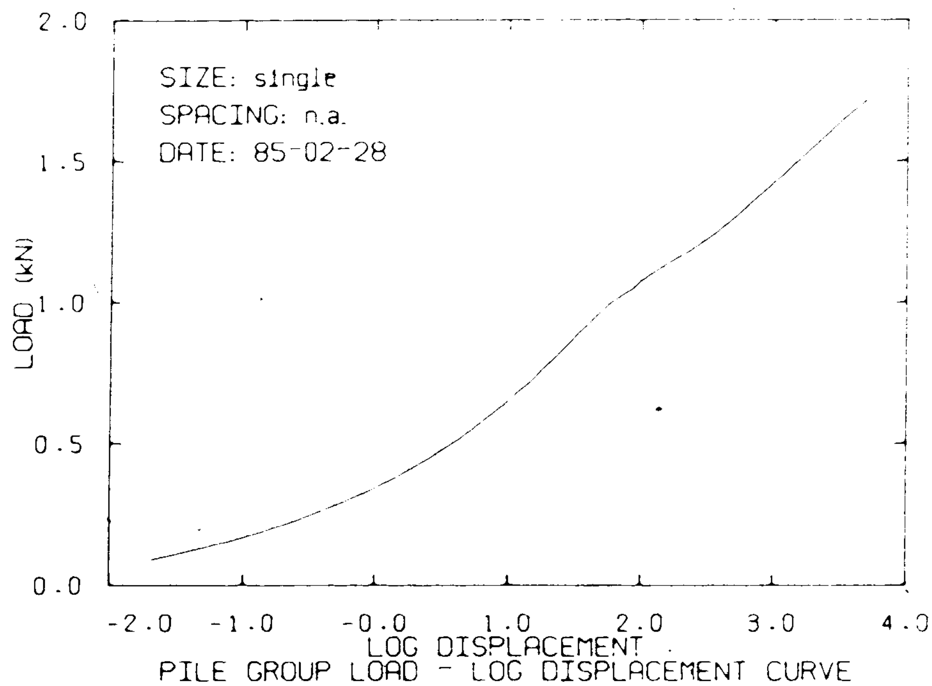


Figure B.5 Load-Deflection Curve for Single Pile Test 85-02-22



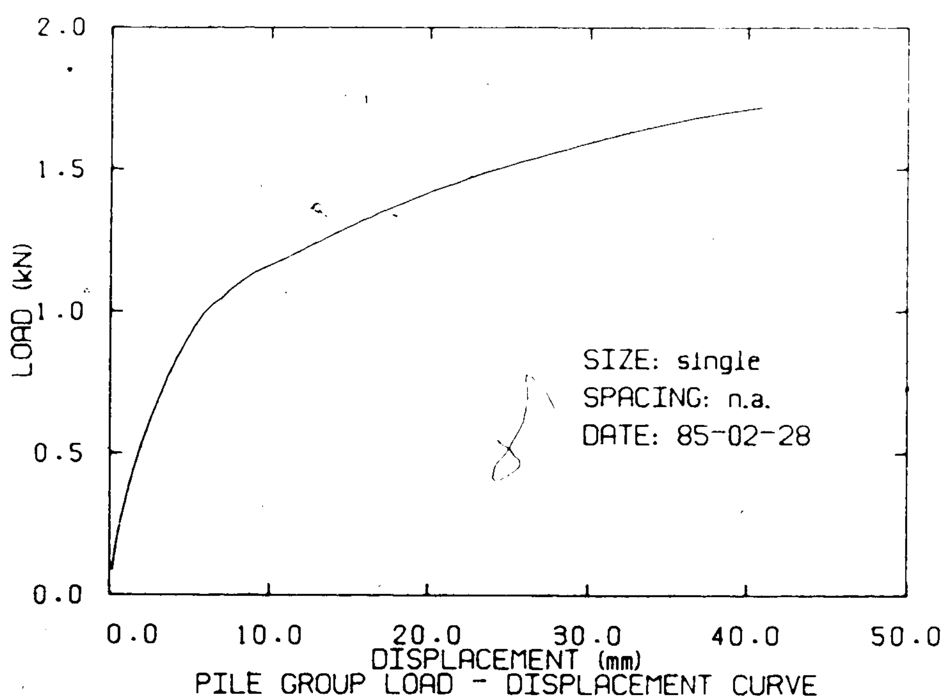
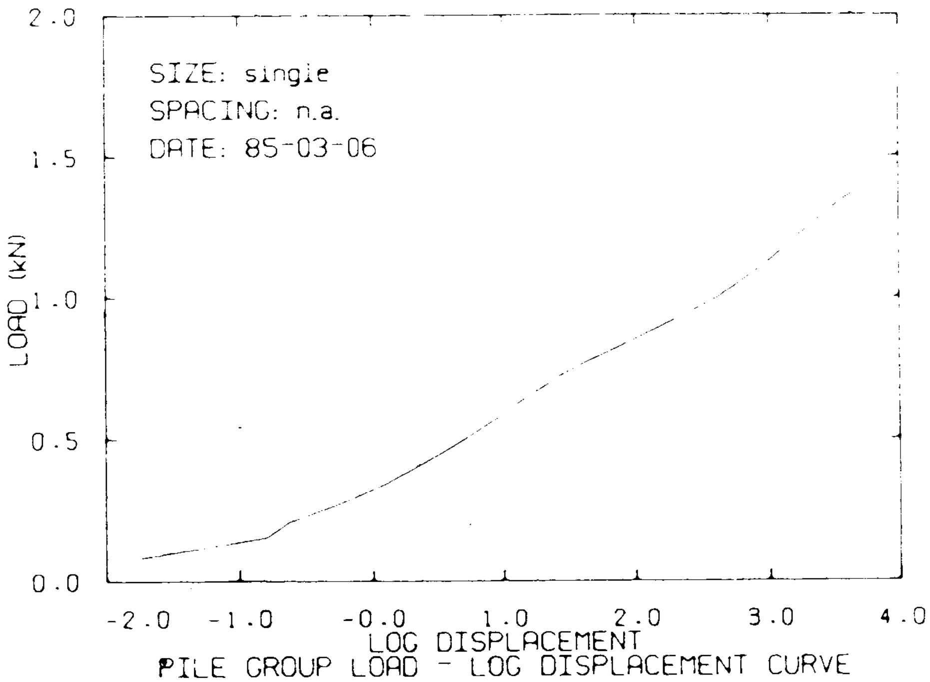


Figure B.6 Load-Deflection Curve for Single Pile Test 85-02-28



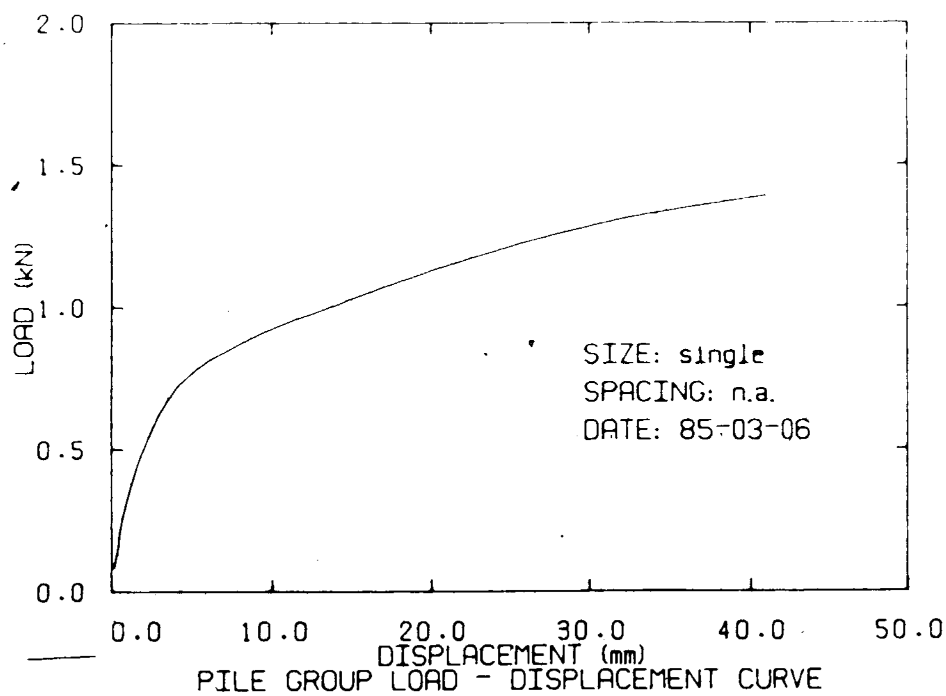
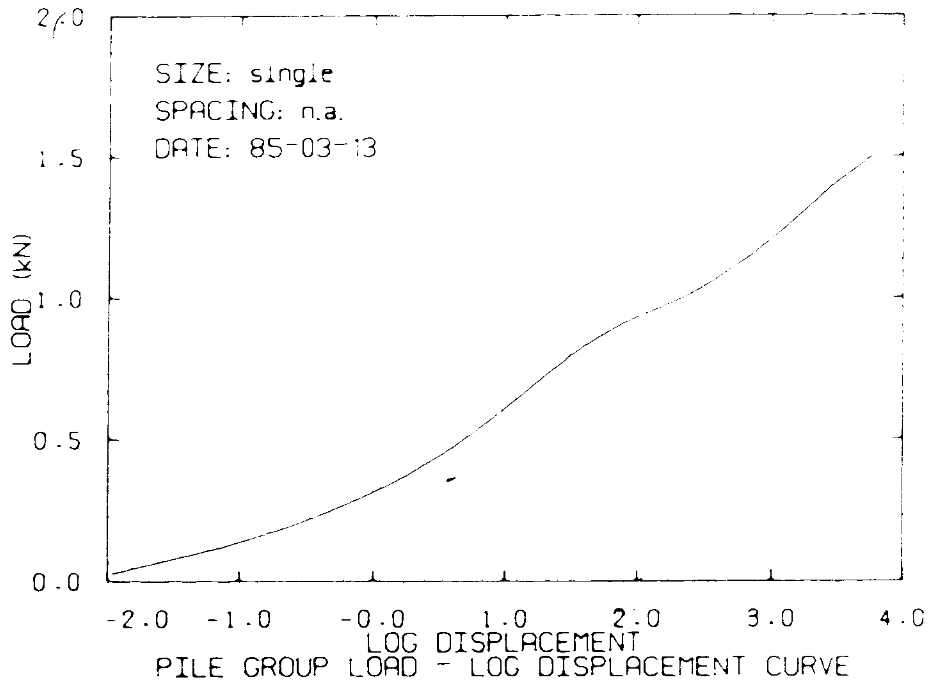


Figure B.7 Load-Deflection Curve for Single Pile Test 85-03-06



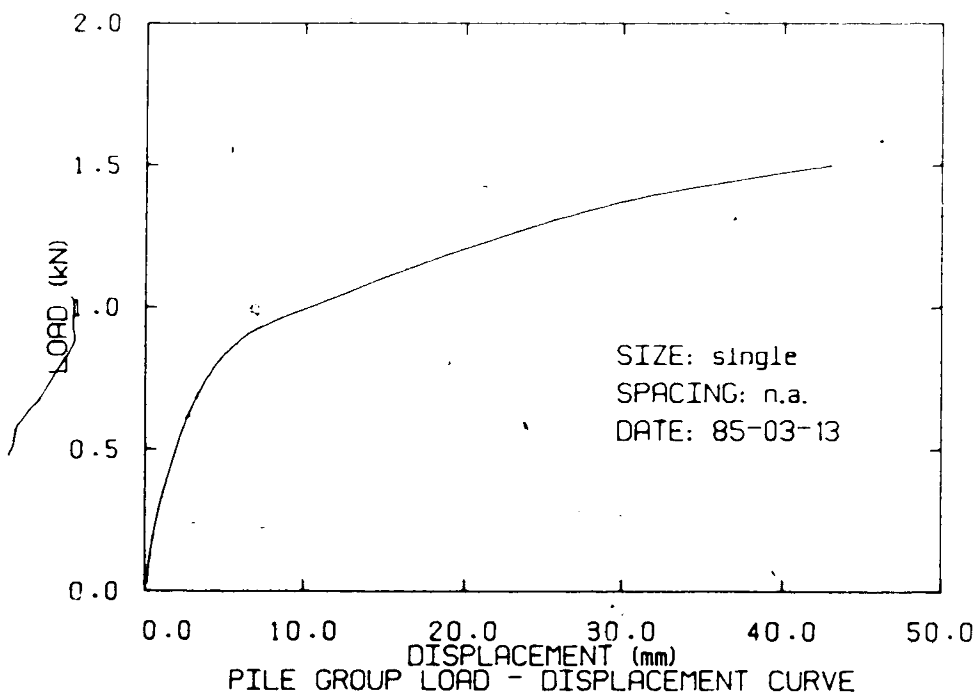
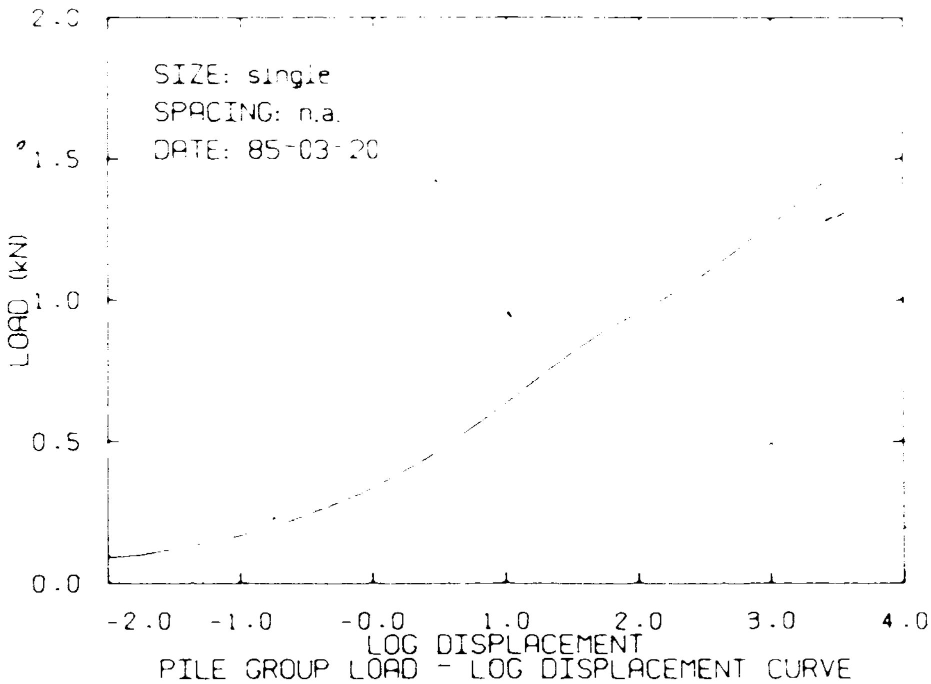


Figure B.8 Load-Deflection Curve for Single Pile Test 85-03-13



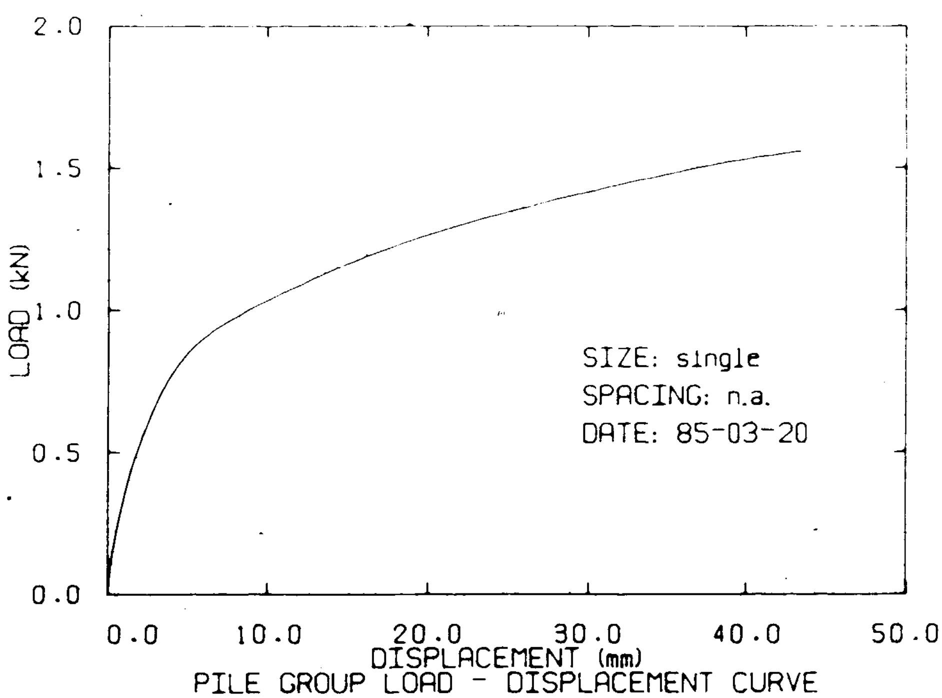
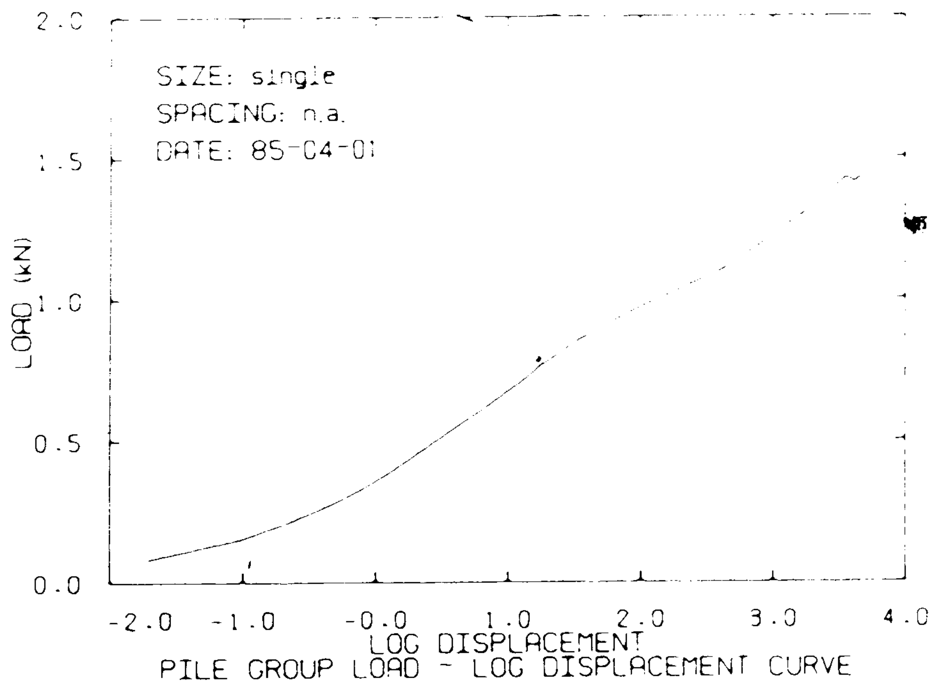


Figure B.9 Load-Deflection Curve for Single Pile Test 85-03-20



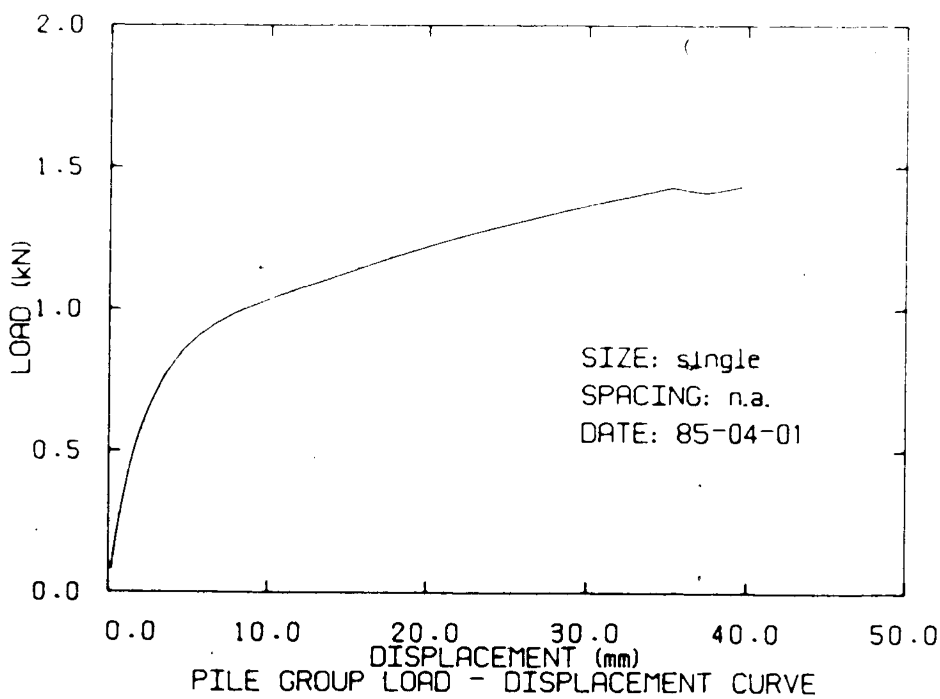
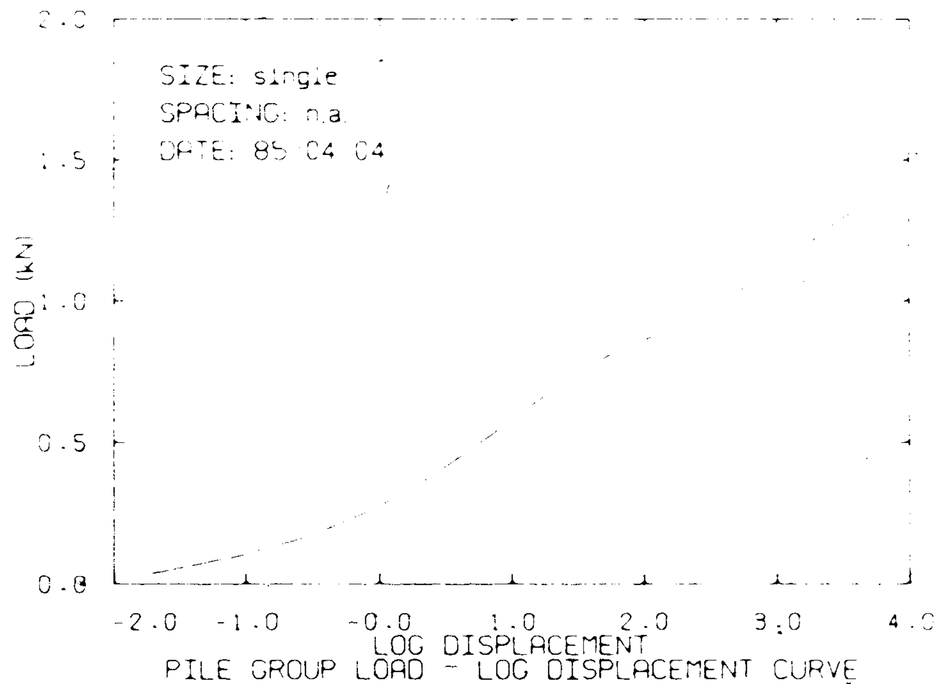


Figure B.10 Load-Deflection Curve for Single Pile Test 85-04-01



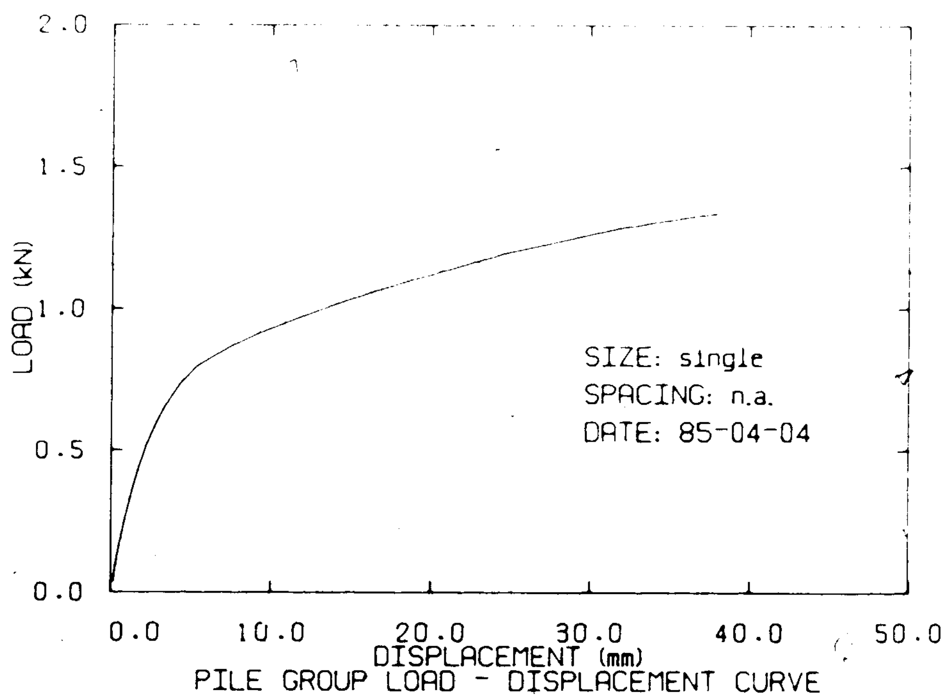
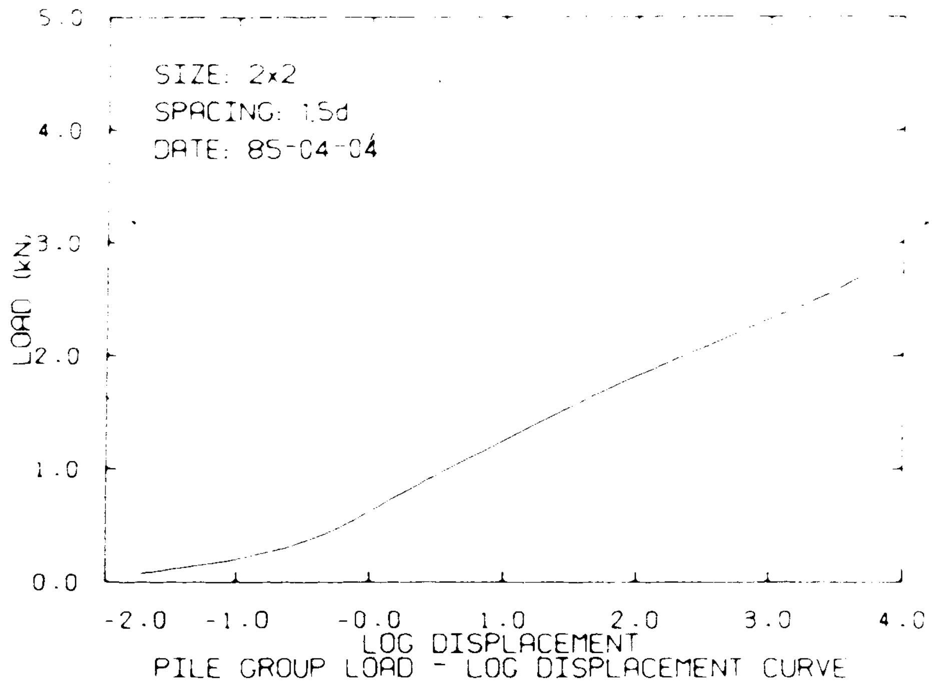


Figure B.11 Load-Deflection Curve for Single Pile Test 85-04-04



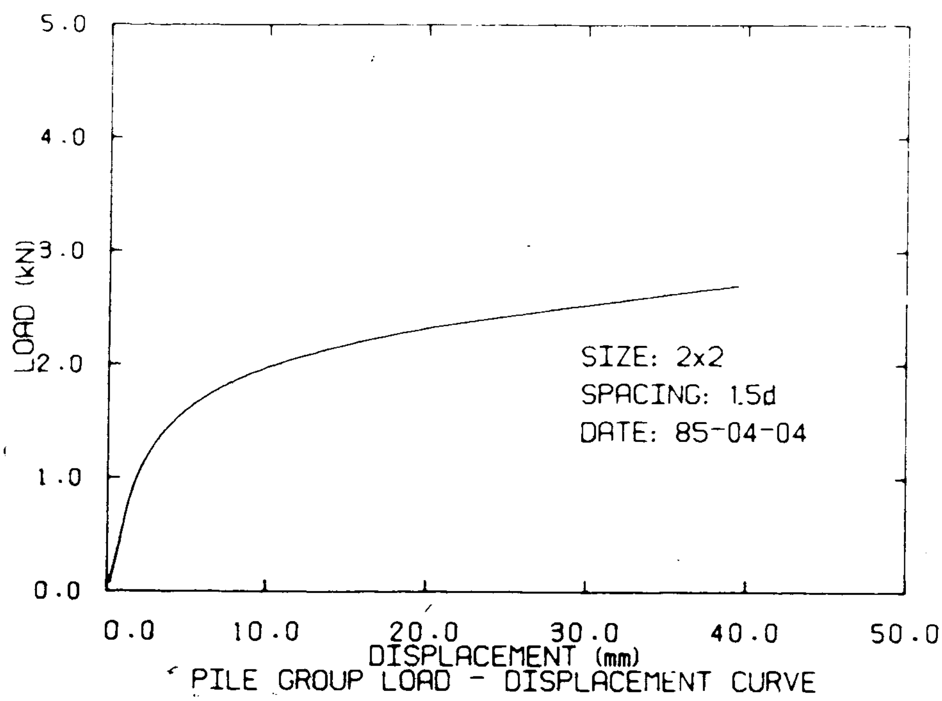
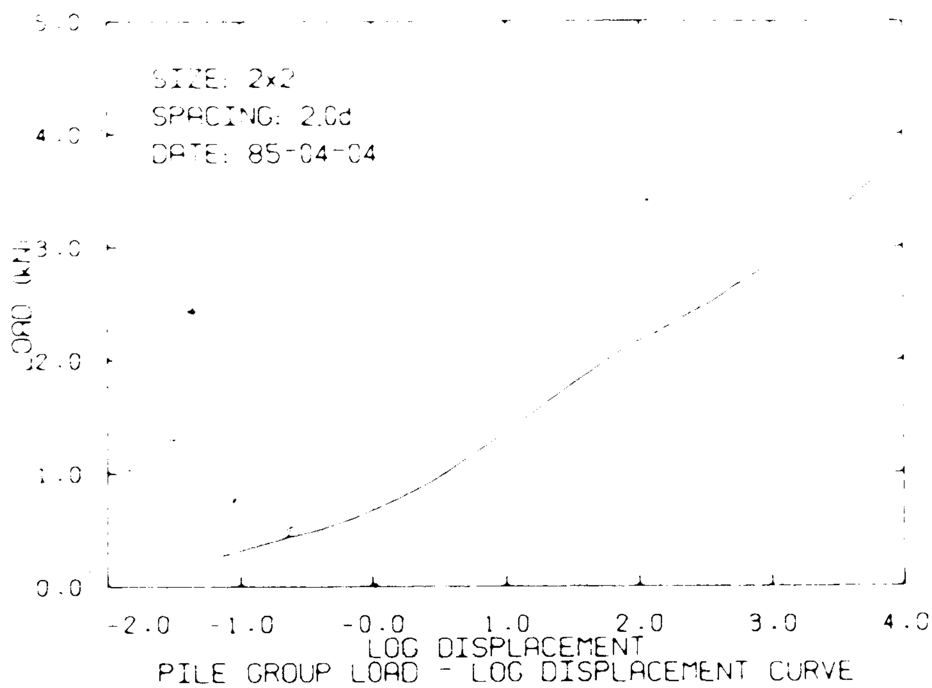


Figure B.12 Load-Deflection Curve for Pile Group 2x2 at 1.5d



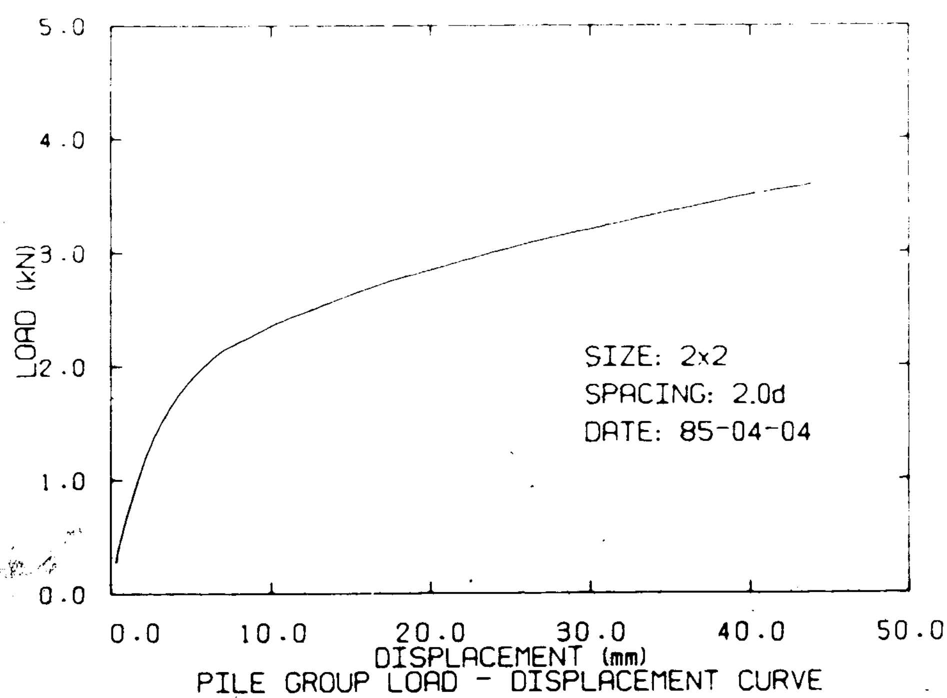
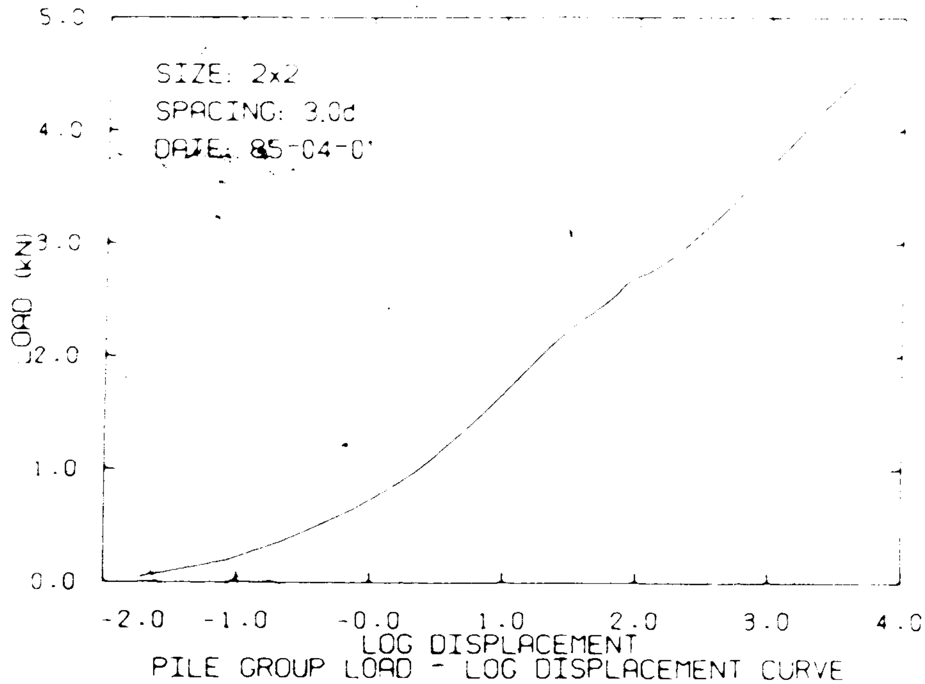


Figure B.13 Load-Deflection Curve for Pile Group 2x2 at 2.0d



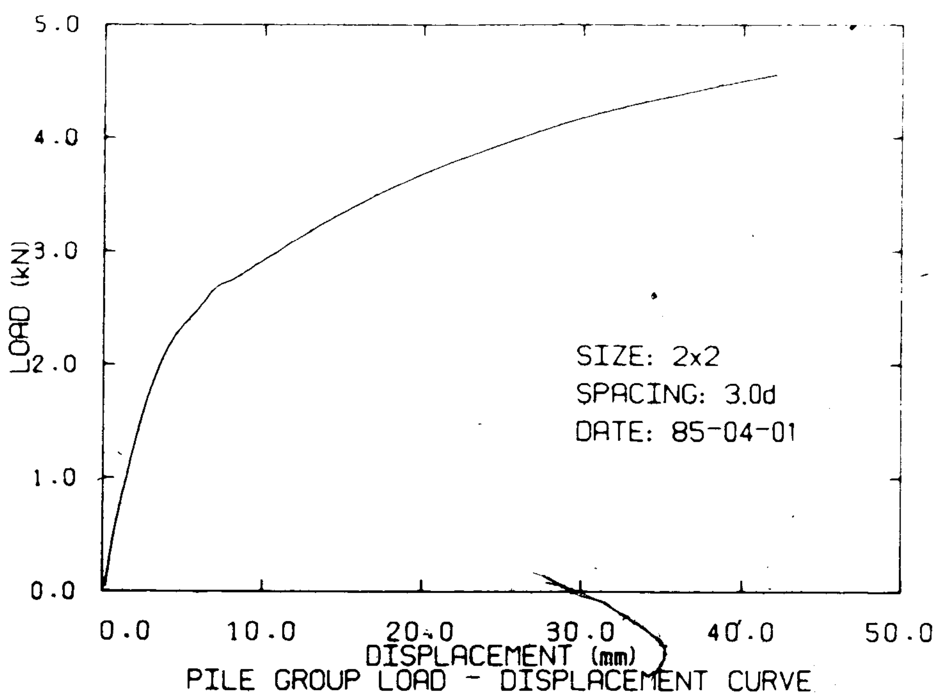
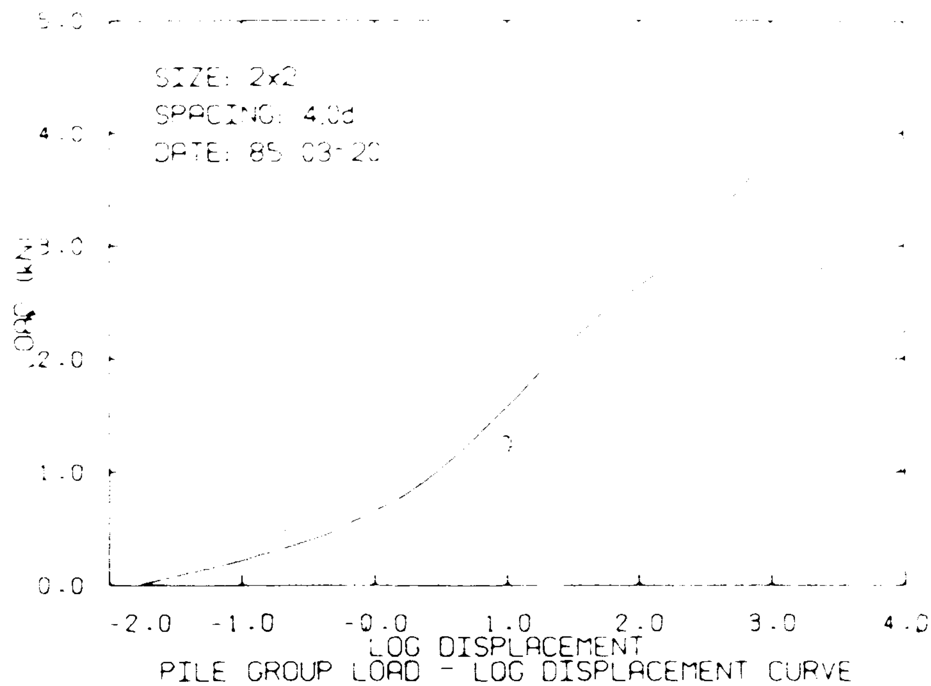


Figure B.14 Load-Deflection Curve for Pile Group 2x2 at 3.0d



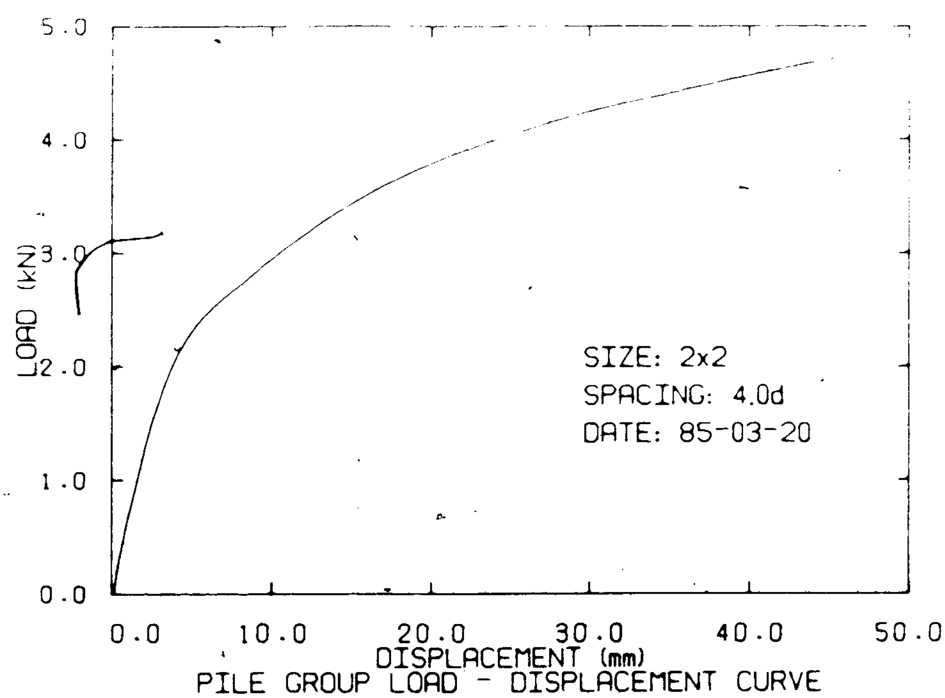
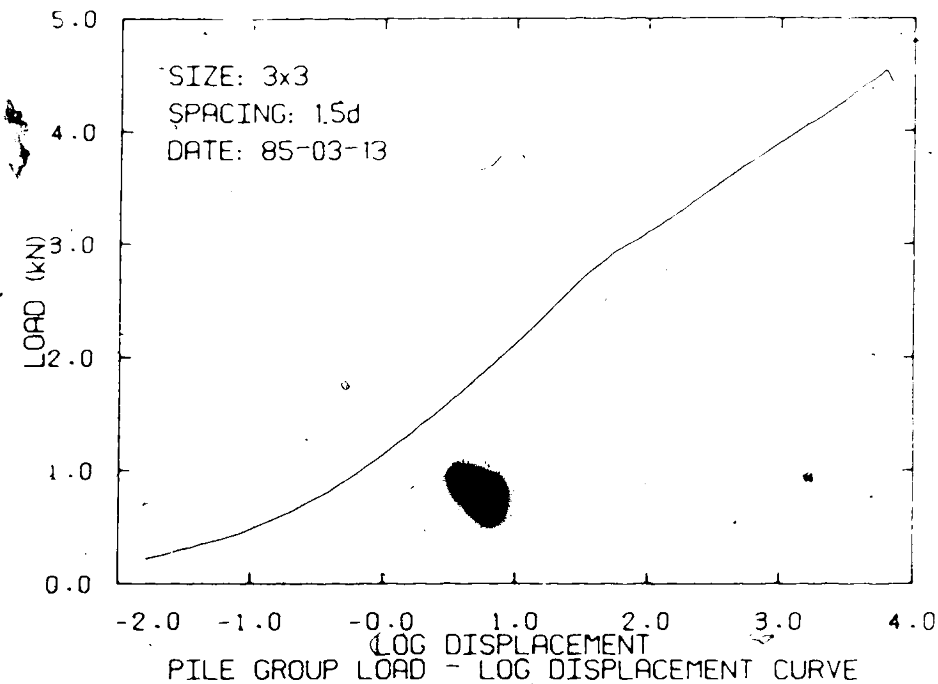


Figure B.15 Load-Deflection Curve for Pile Group 2x2 at 4.0d



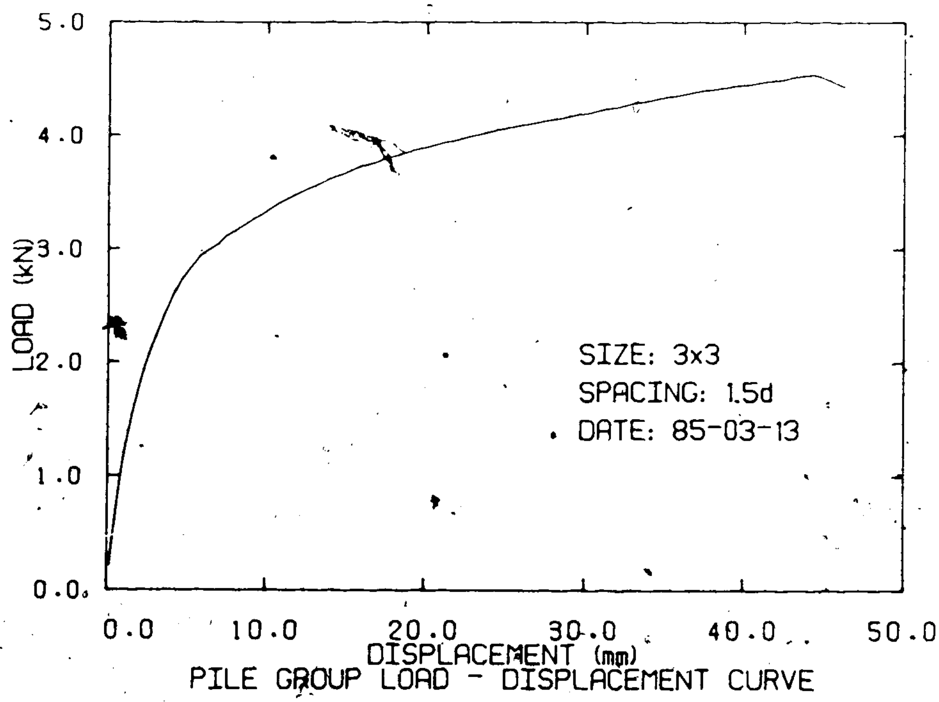
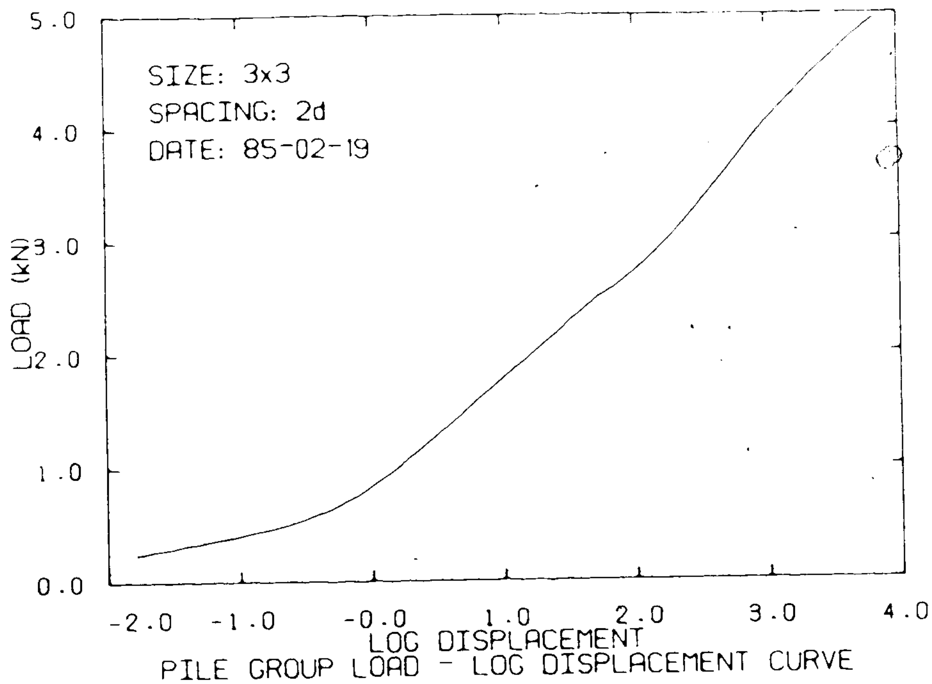


Figure B.16 Load-Deflection Curve for Pile Group 3x3 at 1.5d



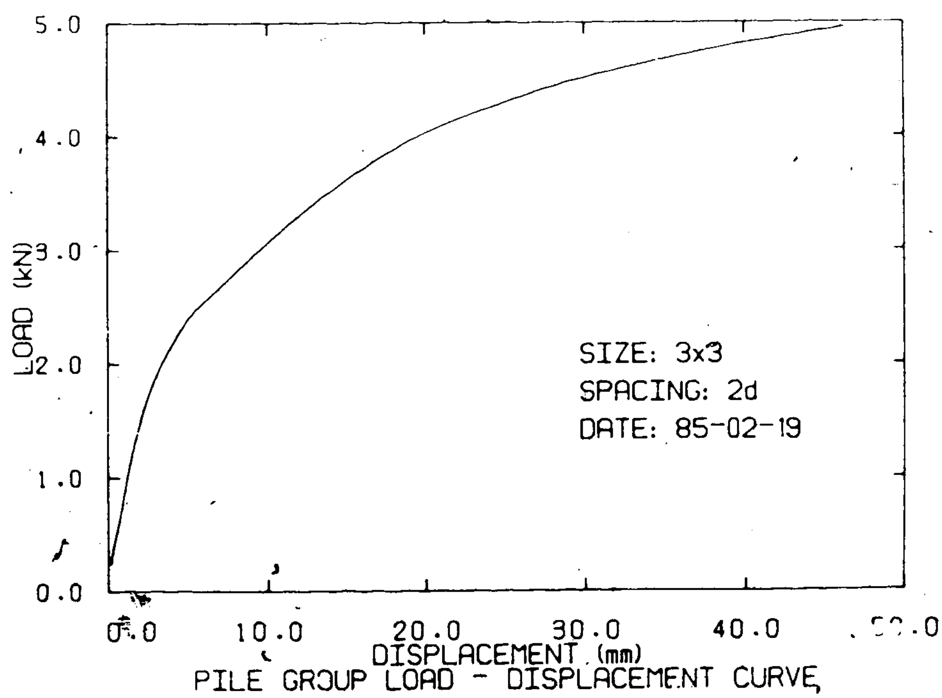
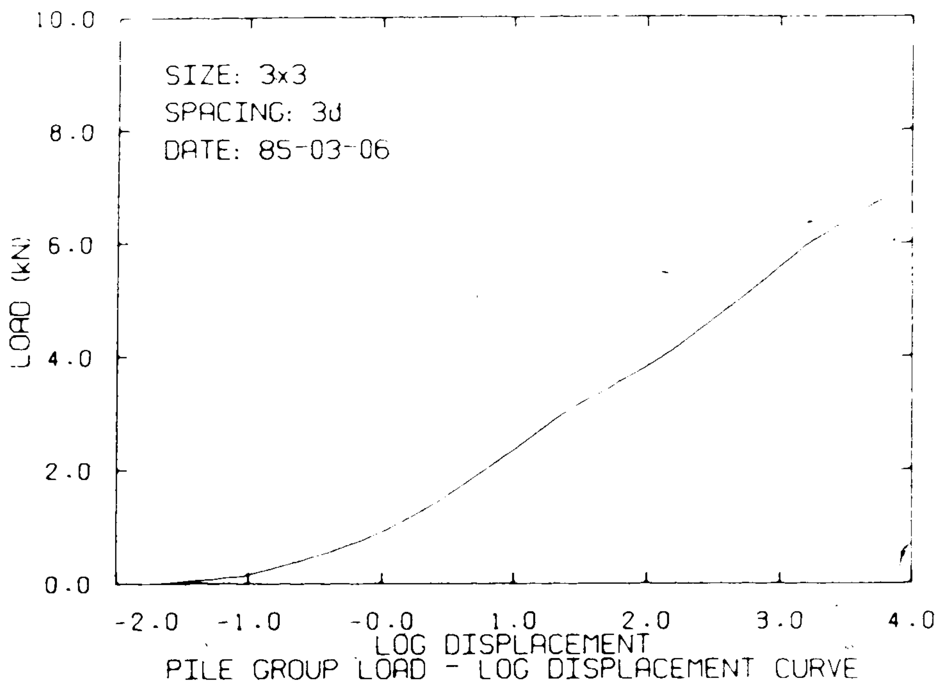


Figure B.17 Load-Deflection Curve for Pile Group 3x3 at 2.0d



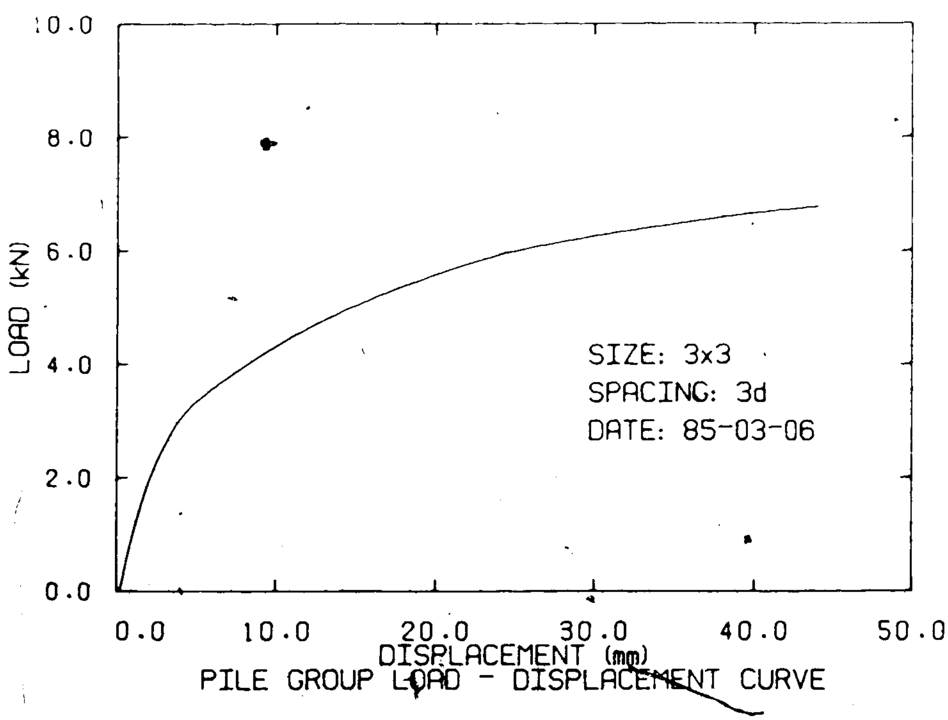
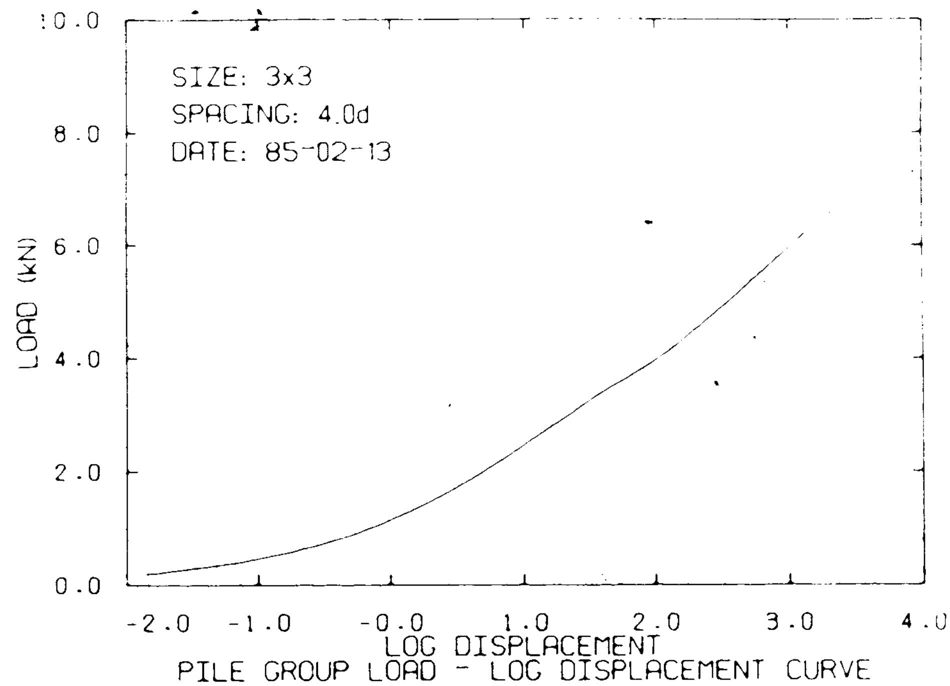


Figure B.18 Load-Deflection Curve for Pile Group 3x3 at 3.0d



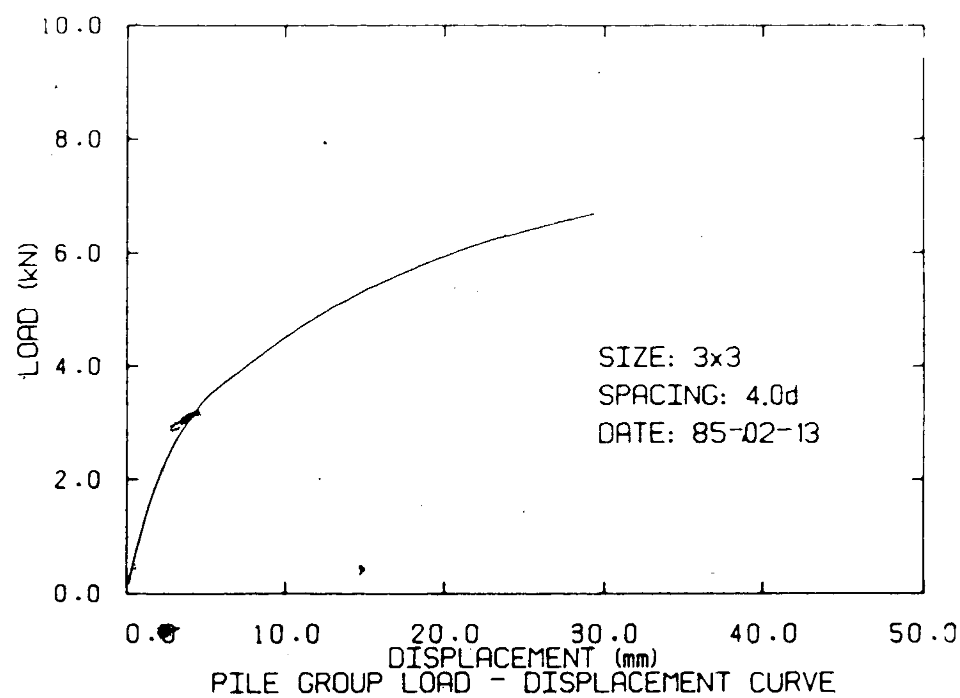
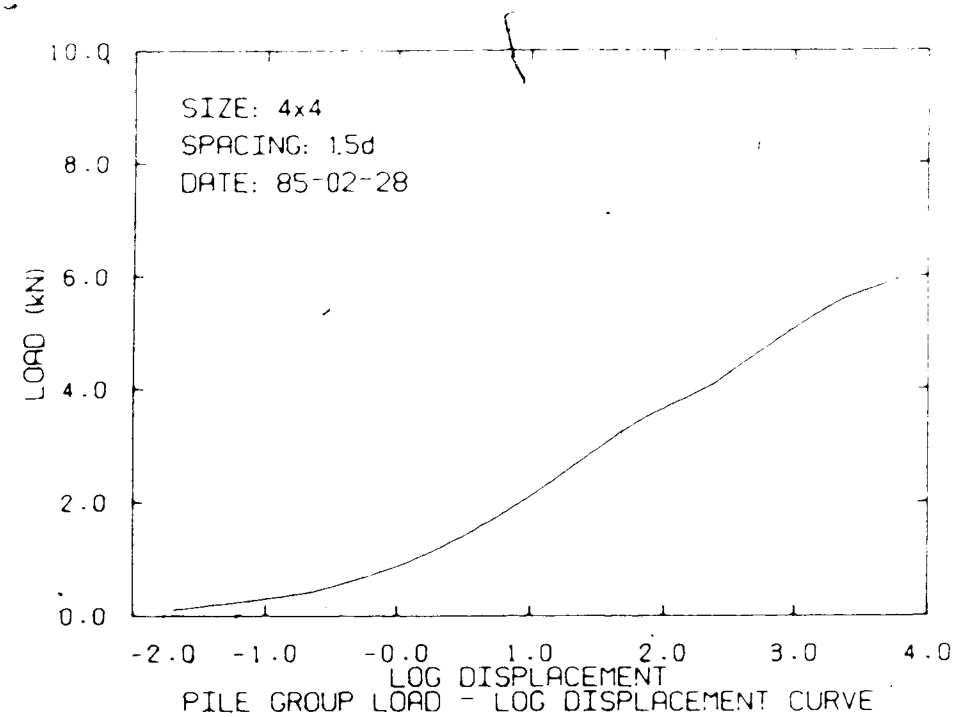


Figure B.19 Load-Deflection Curve for Pile Group 3x3 at 4.0d



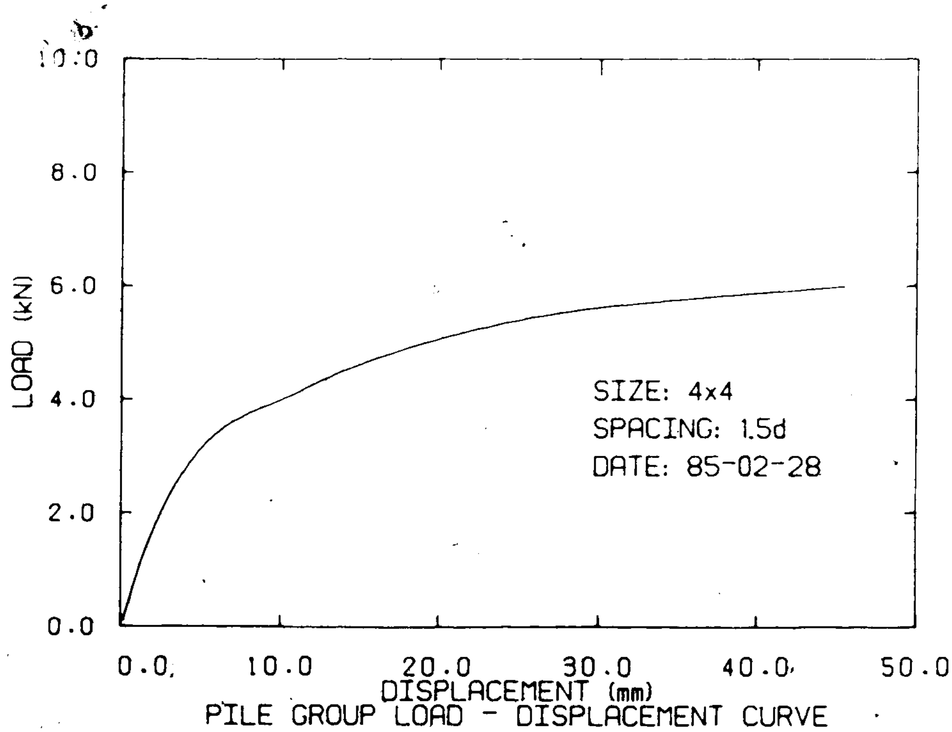
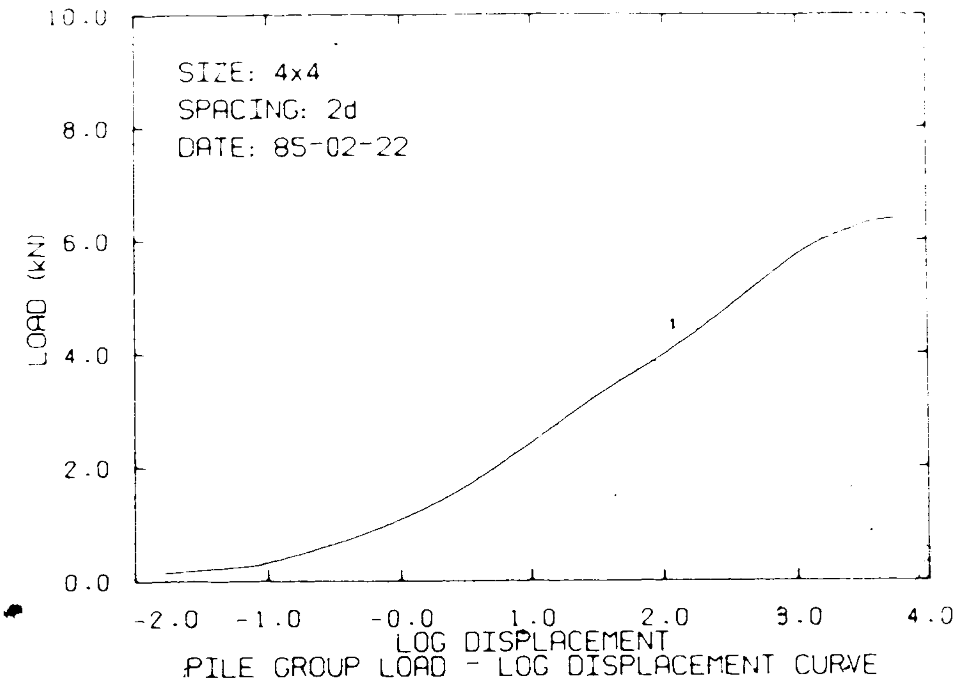


Figure B.20 Load-Deflection Curve for Pile Group 4x4 at 1.5d



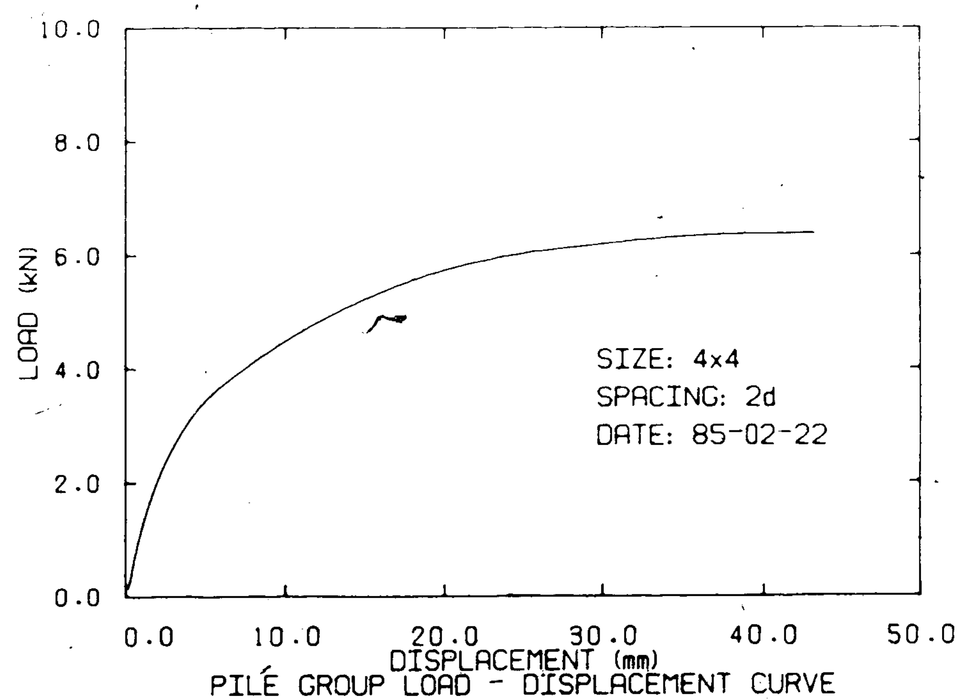
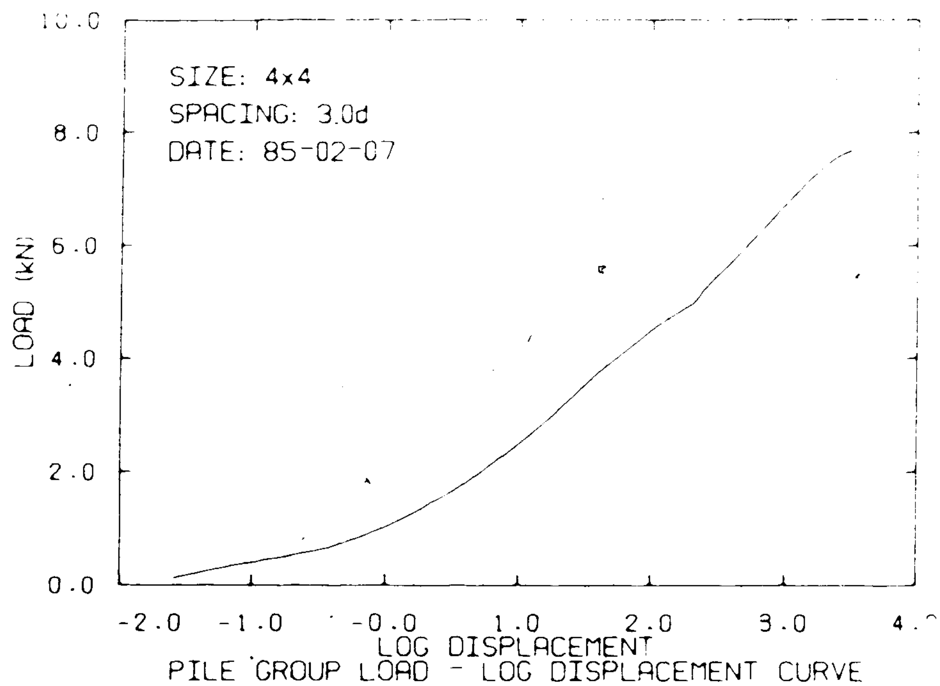


Figure B.21 Load-Deflection Curve for Pile Group 4x4 at 2.0d



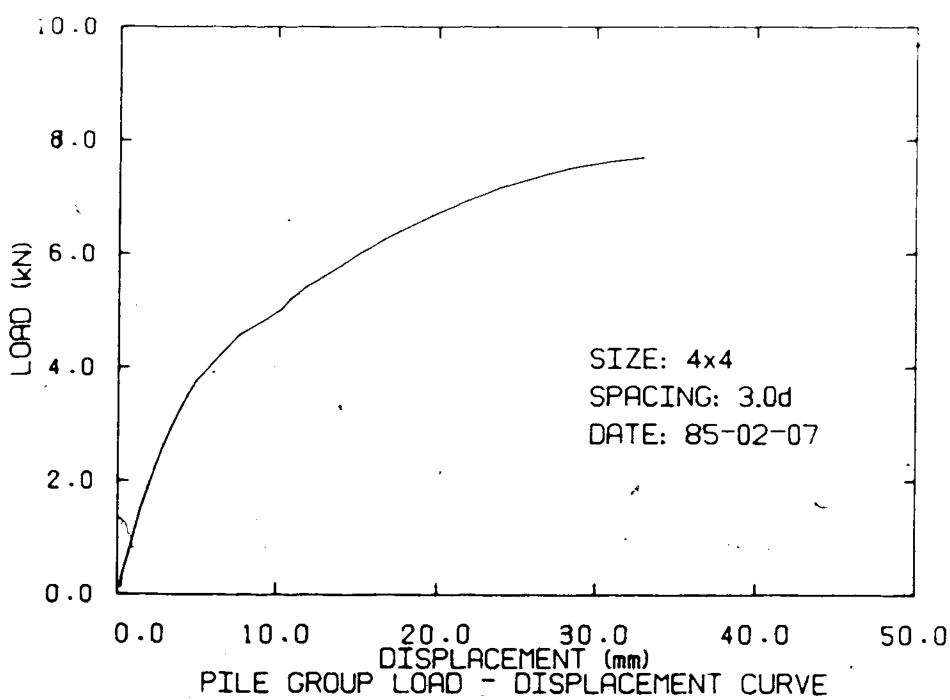
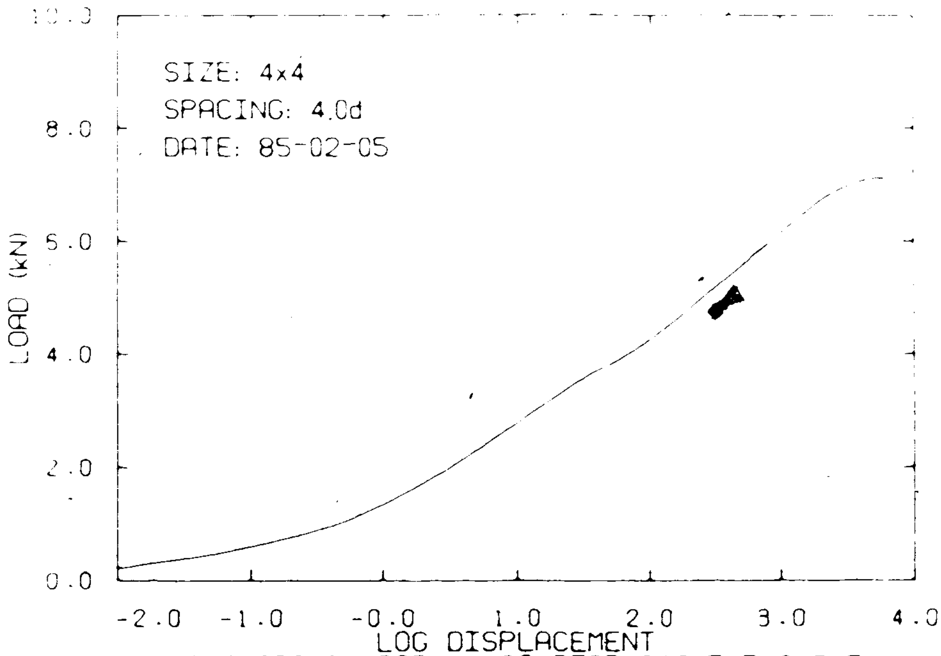
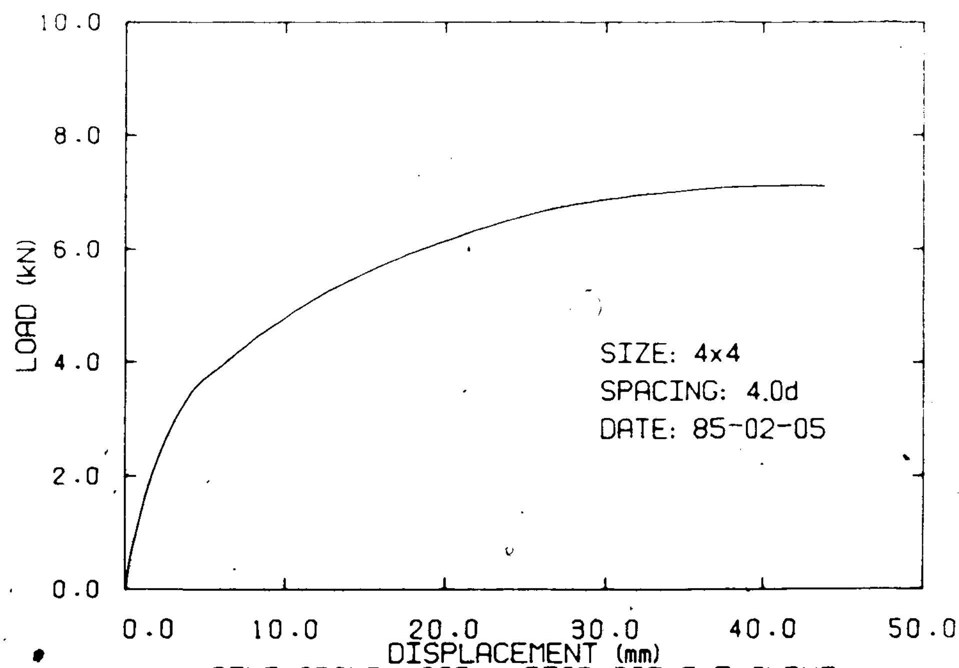


Figure B.22 Load-Deflection Curve for Pile Group 4x4 at 3.0d



PILE GROUP LOAD - LOG DISPLACEMENT CURVE



PILE GROUP LOAD - DISPLACEMENT CURVE

Figure B.23 Load-Deflection Curve for Pile Group 4x4 at 4.0d