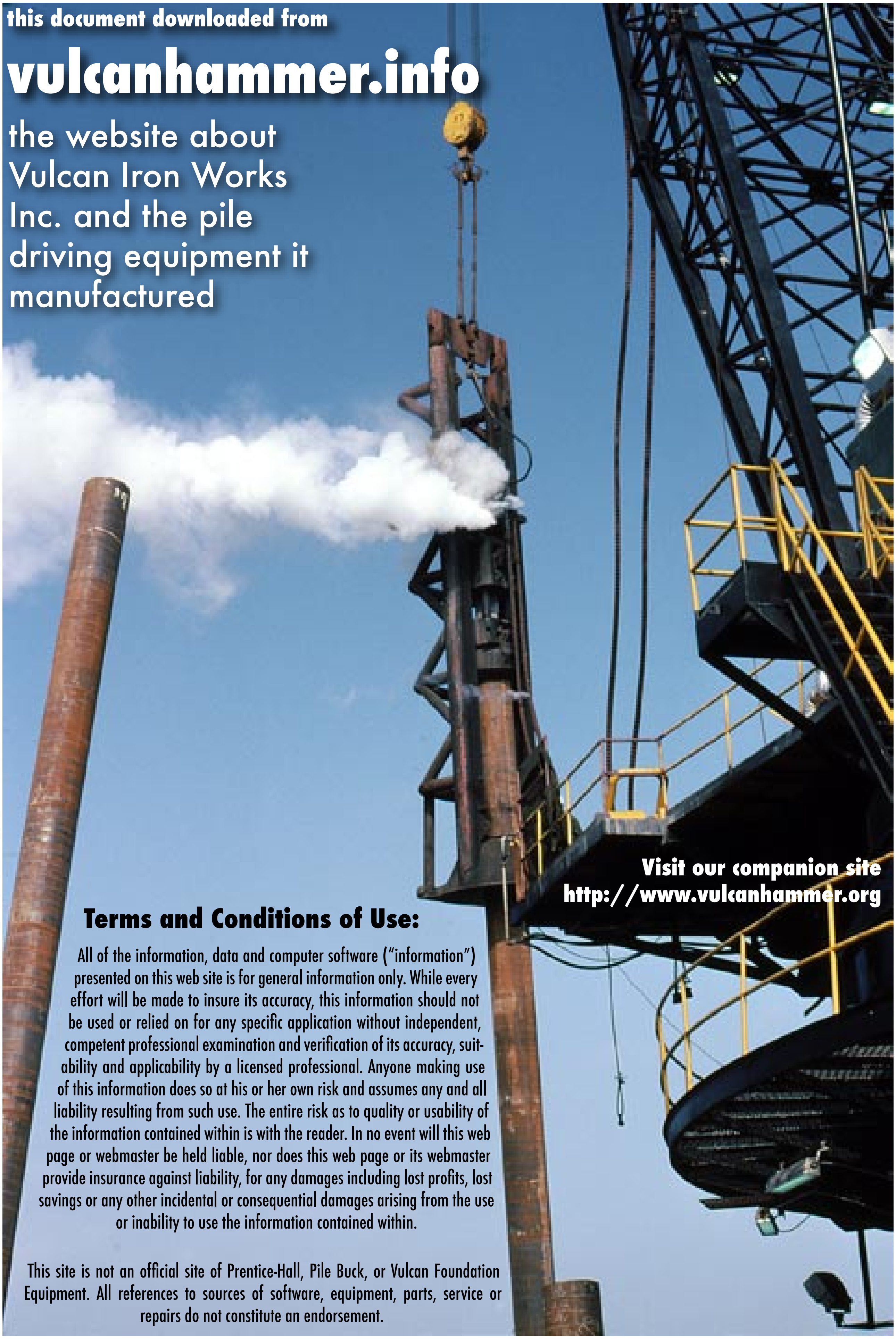


INFLUENCE OF CYCLIC LOADING ON AXIAL PILE RESPONSE

H. G. Poulos, University of Sydney

SUMMARY

This paper reviews existing data on the effects of cyclic degradation and loading rate on skin friction and soil modulus for axially loaded piles. Some of these data are used in a theoretical analysis of cyclic axial response, and the effects of such factors as cyclic load level, number of cycles, loading rate and group effects are investigated. Group effects are shown to have a very significant influence on both the ultimate load capacity and cyclic pile stiffness. Finally, a procedure is described whereby the behaviour of a pile subjected to variable cyclic loading can be estimated.

1. INTRODUCTION

An important feature of pile foundations for offshore structures is the cyclic nature of the loading (both axial and lateral). Considering the possible consequences of failure of such piles, suprisingly little is known about the response of piles to cyclic axial loading. A number of experimental investigations have been carried out and these generally indicate that "two-way" cyclic loading (involving load reversal) has a far more significant effects in reducing pile capacity and pile stiffness than does "one-way" cyclic loading. On the other hand, the relatively high frequency of wave loading and the resulting rapid rate of load application tends to cause an increase in both load capacity and pile stiffness.

Only a few attempts have been made to incorporate either cylcic degradation or loading rate effects (or both) into analyses of axial response (Matlock and Foo, 1979; Boulon et al, 1980; Poulos, 1979a; 1981b). The present paper describes a further evolution of the analyses presented in the latter two papers. Recent data on the factors governing cyclic degradation and loading rate effects are reviewed first and some of the areas still requiring further investigation are highlighted. An analysis incorporating both cyclic degradation and loading rate effects is then described and an extension to allow for pile group effects is discussed. Some solutions are presented to indicate the effects on cyclic axial response of such factors as cyclic load level, loading rate and number of piles in the group. Finally, an approximate procedure for evaluating the effects of variable cyclic loading on pile response is suggested. As in the previous papers, two main aspects are considered:

(a) the influence of cyclic loading on the ultimate axial capacity of a pile

(b) the influence of cyclic loading on the axial stiffness of the pile-soil system.

2. CYCLIC DEGRADATION EFFECTS

Laboratory and field data show that cyclic loading may cause a reduction in load capacity and an increase in settlement of piles. Data collected by Bea et al (1980) suggests only a maximum 10-20% reduction in load capacity, but shows a definite trend for increasing pile head settlement with increasing number of cycles and level of cyclic load level. They suggest that the sum of static and cyclic axial load be kept below 80% of ultimate capacity in order to avoid large cumulative settlements. Small-scale laboratory and field data however suggests that reductions in ultimate load capacity significantly greater than 20% may occur, particularly if load reversal occurs during cycling. The extent of such reductions also depends on the soil type, and therefore data on piles in clay and piles in sand will be reviewed separately.

2.1 Piles in Clay.

Two main mechanisms may be postulated to explain the effects of cyclic loading on piles in clay:

(i) changes in pore pressure in the soil adjacent to the pile

(ii) realignment of the clay particles adjacent to the pile.

Laboratory tests carried out by Steenfelt et al (1981) showed that a drop in shaft adhesion during a "two-way" cyclic test was accompanied by a gradual rise in excess pore pressure, but that when there was no significant rise in pore pressure, there was no evidence of a reduction in shaft capacity. Puech et al (1980) found no significant changes in pore pressure during cyclic loading of a pile in loose compressible silt, although some reduction in skin friction appears to have occurred. A small-scale field test by Grosch and Reese (1980) on a pile in soft clay showed an overall decrease in pore pressure during cycling, prior to (or together with) a decrease in skin friction. Fluctuations in pore pressure began immediately on initiation of reduction in skin friction capacity and were greatest during the periods of greatest reduction. Failure was considered to be located entirely in the soil within a zone of about 2 mm width and not at the pile-soil interface. The soil in this zone was over-consolidated due to pile insertion and subsequent reconsolidation, and hence was considered to dilate as the clay particles rotate and become realigned. Grosch and Reese considered that the primary mechanism of cyclic load-transfer reduction is the destruction of interparticle bonds and realignment of the soil structure parallel to the direction of shear strain. In view of the limited information on the role of both postulated machanisms, it would therefore appear prudent at this stage to adopt a more phenomenological approach to describe cyclic degradation effects on both pile load capacity and deformation.

A consistent feature of cyclic response of piles is that "two-way" cyclic loading (involving stress reversal) results in a much more dramatic reduction in pile load capacity than "one-way" cyclic loading (Holmquist and Matlock, 1976; Steenfelt et al, 1981), with reductions in skin friction of up to 75% being recorded for extremely large cyclic load or displacement amplitudes. Such findings have led to the development of a degradation model by Matlock and Foo (1979) in which cyclic degradation only occurs if plastic reversals of strain occur. However, model tests by Poulos (1981a) reveal that one-way cyclic loading will also cause a reduction in ultimate skin friction, and that a more satisfactory basis for cyclic degradation may be the cyclic displacement of the pile.

In order to express degradation effects conveniently, the concept of degradation factors has been introduced, the degradation factor being defined as

$$D = \frac{\text{property after cyclic loading}}{\text{property for static loading}} \qquad (1)$$

The degradation factors for skin friction, ultimate base resistance and soil modulus are denoted as D_τ, D_b and D_E respectively.

The variation of the skin friction degradation factor D_τ with cyclic pile displacement is shown in Fig.1. These results are derived from tests on model piles 20 mm diameter and 250 mm long in remoulded clay (Poulos, 1981a) and the cyclic displacement is expressed in dimensionless form as ρ_c/d, where ρ_c = half-amplitude of cyclic displacement and d = pile diameter. Tests on model piles of various diameters indicate that this normalization of cyclic displacement is applicable on a model scale, but it is not known whether the results would apply for full-size piles. However, data presented by Aurora et al (1981) suggests that the static pile displacement for full slip, ρ_{st}/d, increases more-or-less linearly with diameter, with ρ_{st}/d ranging between about 0.005 and 0.025. As discussed below, the cyclic pile displacement to cause a particular degree of cyclic degradation of skin friction appears to be related to ρ_{st} so that it seems reasonable to assume that cyclic displacement is also related to the diameter. Fig.1 reveals that no degradation of skin friction occurs unless the (half-amplitude) cyclic displacement exceeds about 0.2% of the diameter. Thereafter, increasing cyclic displacement results in degradation and loss of skin friction, although the degradation factor appears to reach a limiting value for cyclic displacements in excess of about 1.5% of the diameter. The degradation factor depends on the number of cycles (in contrast to the assumption made earlier by Poulos, 1981b), but the majority of degradation occurs in the first 10 or 20 cycles. This was also noted in tests by Grosch and Reese (1980). However, it should be emphasized that the cyclic displacement required to initiate degradation may vary considerably from those indicated in Fig.1. For example, Grosch and Reese (1980) have found, from tests on a 1 in diameter model aluminium pile in a soft in-situ clay, that cyclic displacements ρ_c in excess of $\pm 0.02d$ were required, and that a limiting degradation factor was reached when ρ_c exceeded about $\pm 0.06d$. It would appear that the "critical" cyclic displacement at which cyclic degradation commences, ρ_{cc}, is related to the displacement required for full slip in a static load test, ρ_{st}. In

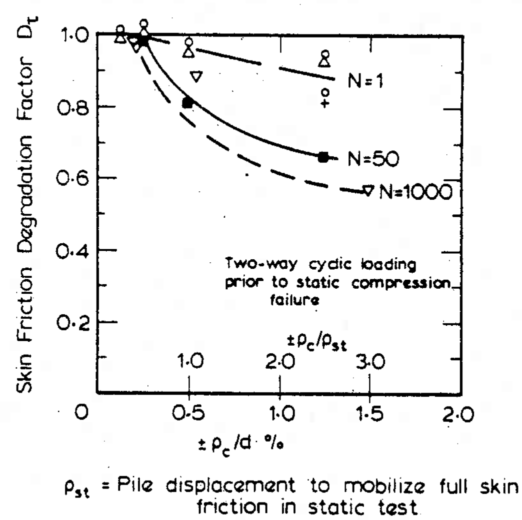


FIG.1 SKIN FRICTION DEGRADATION FACTOR FOR MODEL PILES IN HURSTVILLE CLAY

(After Poulos, 1981 a)

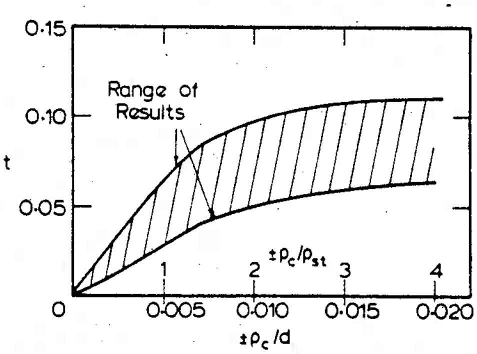


FIG.2 DEGRADATION PARAMETER t DERIVED FROM MODEL PILE TESTS IN HURSTVILLE CLAY

the model tests of Poulos (1918a) ρ_{st} was about 0.005d, whereas in the tests of Grosch and Reese (1980), ρ_{st} was about (0.04 to 0.05)d. The corresponding values of $\rho cc/_{\rho st}$ are therefore similar, being of the order of 0.4 to 0.5 in both cases. Consequently, the degradation factor is probably best related to the ratio of the cyclic displacement ρ_c to the static displacement for full development of skin friction, ρ_{st}. This approach has, in effect, been adopted by Poulos (1981b) who considers the degradation factors to be dependent on a cyclic "strain" ratio, in which the reference strain is related to the static shear strain to failure. Fig.1 also shows the abscissa in terms of $\pm^{\rho}c/\rho_{st}$, and this would appear to be the most useful form of the skin friction degradation relationship to adopt for practical purposes. Comparison between Fig.1 and the degradation curve previously adopted by Poulos (1981b) reveals that the latter is far more severe, with a degradation factor of about 0.3 for a value of cyclic strain ratio (or $^{\rho}c/\rho_{st}$) of 1.5.

Data on modulus degradation from cyclic triaxial tests by Idriss et al (1978) indicated that the modulus degradation factor D_E could be approximated as follows:

$$D_E = N^{-t} \qquad (2)$$

where N = number of cycles

t = a degradation parameter which is a function of cyclic strain.

It has been found from model tests that Eq.2 can also be applied to the degradation of soil modulus for piles, and that the parameter t can be related to the dimensionless cyclic displacement $\pm\rho_{c/d}$, or preferably, to $\pm^{\rho}c/\rho_{st}$. The relationship derived from the model pile tests of Poulos (1981a) is shown in Fig.2; because of experimental scatter, the range of t values is shown. Comparison between Fig.2 and the previously-assumed relationship used by Poulos (1981b) shows that the latter results in considerably greater modulus degradation, although that relationship can be adjusted by altering the reference shear strain.

Unfortunately, no direct data is yet available on the cyclic degradation of ultimate base resistance of a pile in clay, as most tests to date have concentrated on cyclic effects on skin friction. In the absence of other evidence, it is suggested that Figs.1 and 2 can be used to estimate the amount of degradation of ultimate base pressure and soil modulus at the base, provided that the value of ρ_{st} is now taken as the displacement required to mobilize the ultimate base resistance; this will generally be significantly greater than the displacement to mobilize the ultimate skin friction, so that the amount of cyclic degradation at the base will generally be less than along the shaft.

Finally, it should be remarked that data derived from small-scale model pile tests may tend to reflect the effects of particle reorientation more than pore pressure effects, since any excess pore pressure developed during cyclic loading will dissipate much more rapidly than in a full-scale field situation. The extent to which pore pressure effects influence cyclic degradation of full-scale piles (particularly large-diameter offshore piles) remains to be investigated.

2.2 Piles in Sand.

The limited information available on the effects of cyclic loading on piles in sand indicates that remarkable reductions in load capacity and pile stiffness can occur. Chan and Hanna (1980) describe laboratory model tests which demonstrate that pile failure can occur with cyclic loads of 30% of the ultimate static load for one-way loading or even smaller values for two-way cyclic loading. Permanent settlement of the pile may continue to increase, even after a very large number of cycles. Model tests by Gudehus and Hettler (1981) show that failure can occur under one-way cycling for a maximum load as small as 10% of the ultimate static value; this failure is characterized by increasing deflection with increasing numbers of cycles (incremental collapse). At higher cyclic load levels (e.g. 30% of ultimate), failure occurs within about 40 cycles. Field and laboratory tests reported by Van Weele (1979) broadly confirm the conclusions reached in the above model tests. In one test, with an average load of 20-30% of the static failure load and a cyclic component of equal magnitude, failure occurred after about 3000 load cycles, while in another test, cycling between zero load and about 25% of the ultimate static load caused failure after 10 000 cycles. It was deduced that degradation of base resistance was more severe than degradation of skin friction, and close examination of the sand near the tip showed appreciable crushing of the grains. For design purposes, Van Weele suggests that the ultimate peak load under cyclic conditions is approximated as the sum of 25-33% of the static end-bearing and 60-70% of the static ultimate friction.

In all the above cases, failure is characterized by a continued accumulation of permanent displacement, resulting in movements of the order of one pile diameter after many cycles. Van Weele attributes this to the continuous re-arrangement of particles (and the possible crushing of highly-stressed particles) and argues that deformation may continue to increase with increasing load cycles without reaching a final and constant value. Thus, the consideration of permanent displacement and its accumulation with increasing load cycles appears to be of prime importance in assessing cyclic axial response of piles in sand. Nevertheless, the amount of cyclic degradation of skin friction has been shown to be dependent on the cyclic displacement, as is also the case for piles in clay. Small-scale laboratory tests have been carried out by the author on jacked aluminium piles (20 mm diameter, 250 mm long) in medium dense silica sand consolidated to various overburden pressures using displacement-defined two-way cyclic loading with a mean zero load. The skin friction degradation factor D_τ is found to be relatively insensitive to overburden pressure and overconsolidation ratio, and the relationship obtained between D_τ and dimensionless cyclic pile displacement $\rho_{c/d}$ is shown in Fig.3 for N = 10 cycles. Also shown on the horizontal axis is the cyclic pile displacement ρ_c normalized with respect to the static displacement of full slip ρ_{st}, which for these tests averaged 0.025d. The general characteristics of cyclic degradation of skin friction are similar to those for piles in clay (Fig.1), and again the majority of degradation appears to take place within the first 10 cycles.

Detailed data on the degradation of soil modulus has not yet been obtained for piles in sand. The cyclic stiffness of a pile tends to decrease with increasing numbers of cycles, but it is not yet clear whether the expression in Eq.2 can be applied in this case. Moreover, no data on the degradation of ultimate base resistance is available, although the tests of Van Weele suggest that this degradation may be important. Consequently, it must be concluded that, at this time, there is a dearth of experimental data on the effects of cyclic loading on piles in sand, although indications are that they can be more critical than for piles in clay.

3. LOADING RATE EFFECTS

Bjerrum (1973) and Bea et al (1980) have summarized the results of field tests on piles in clay which clearly indicate that the rate of application (or the time to failure) has a significant effect on pile load capacity. The more rapid the loading rate, the greater the pile capacity, and an approximately linear increase in load capacity with the logarithm of loading rate is observed. Typically, the load capacity increases by between 10 and 20% per decade increase in loading rate. Laboratory tests on model piles in clay also confirm these values (Poulos, 1981a). Similar effects have been noted on pile stiffness by Gallagher and St. John (1980) and Kraft et al (1981) in their field tests. Thus, in quantifying the effects of loading rate, a reasonable procedure appears to be to apply a loading rate factor D_R to the values of ultimate skin friction, ultimate base resistance and soil modulus, in which

$$D_R = 1 - F_\rho \log (\lambda/\lambda_r) \qquad (3)$$

where F_ρ = rate coefficient (typically 0.1 to 0.25)

λ = actual loading rate

λ_r = reference loading rate (e.g. for static load test).

In situations where relatively rapid cyclic loading is being applied to a pile, (such as with offshore piles subjected to wave loading) the beneficial effects of high loading rate maybe offset by the degradation of load capacity due to the cycling of the load, and the ultimate load capacity may be less than or more than the ultimate static capacity. For example, in the tests conducted by Kraft et al (1981), the combined effects of one-way cycling and rapid loading rate resulted in a load capacity which exceeded the static value by up to 20%. Thus, it is necessary to consider both cyclic and rate effects simultaneously in order to assess the ultimate load capacity of offshore piles.

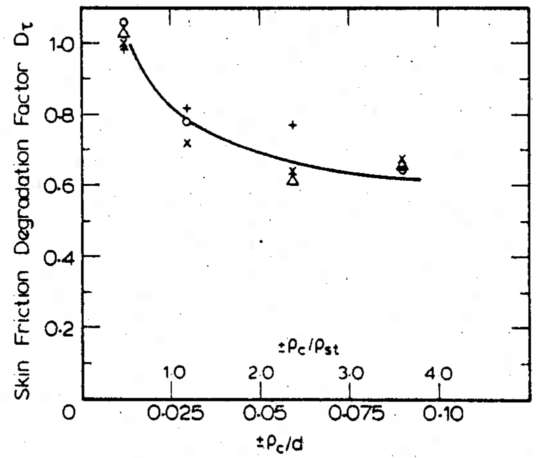


FIG.3 CYCLIC DEGRADATION OF SKIN FRICTION FOR PILE IN NORMALLY-CONSOLIDATED SILICA SAND N=10 CYCLES

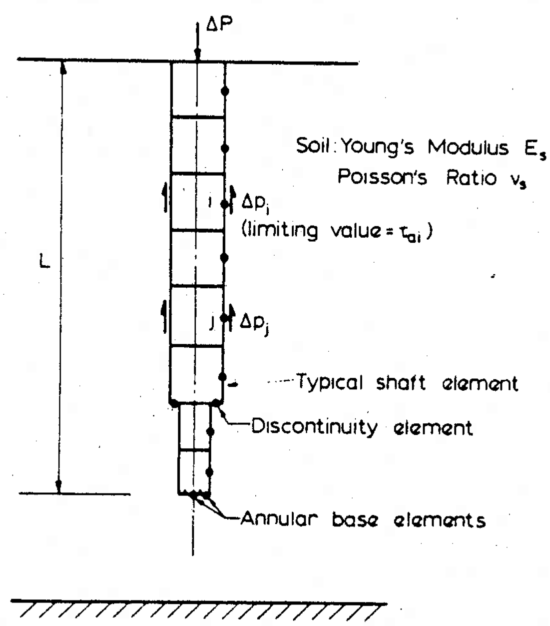


FIG.4 DISCRETIZATION OF PILE

There is no published evidence on the effects of loading rate on piles in sand. Laboratory static triaxial tests show that the shear strength of sand is largely unaffected by loading rate (in contrast to clays which are influenced in a similar manner to piles in clay). Thus it would seem that no rate effects could be relied upon for piles in sand, so that cyclic loading would serve only to cause degradation of pile load capacity and stiffness; if this is so, the significance of cyclic loading effects on piles in sand may indeed be much greater than for piles in clay.

4. THEORETICAL ANALYSIS

An analytical procedure for incorporating the effects of cyclic loading rate on axial pile response has been described by Poulos (1981b). The analysis for a single pile will be summarized briefly and then an extension to allow analysis of circular groups of piles will be discussed.

4.1 Single Piles.

The analysis is a simplified form of boundary element analysis in which the pile is discretized into a series of elements, cylindrical for the shaft and annular elements for the base and any discontinuities in diameter along the shaft (see Fig.4). The soil is considered initially to be a linearly elastic continuum. By considering equilibrium and compatability between the vertical movements of the pile and soil at each element, relationships may be derived between the increment in pile-soil interaction stresses, the incremental pile vertical displacement, and the applied axial load increment. In the case of static loading, these relationships may be solved for the interaction stress increments and the incremental pile deflection, from which the over-all values can be obtained by addition to the existing values. Allowance can be made for pile-soil slip or soil yeild at an element by specifying an upper limit to the value of the interaction stress at each element, and carrying out an interative analysis. Nonhomogeneity of the soil along and beneath the pile may also be taken into account approximately, in the manner described by Poulos (1979b). The ability to consider soil nonhomogeneity in a convenient and economical fashion is essential when considering a cyclically loaded pile, as degradation of the soil modulus and skin friction will occur at different rates along the pile, so that, even in an initially homogeneous soil mass, non-uniform distributions of modulus and skin friction will generally exist along the pile.

In extending the above analysis to incorporate cyclic loading, it is most convenient to carry out an analysis to determine the response of a pile after a number of cycles N of uniform magnitude (maximum load P_{max}, minimum load P_{min}), with the soil parameters being adjusted to reflect the effects of cyclic loading at the end of the load sequence. The following procedure is followed:

(1) First estimates of soil modulus E_s, ultimate skin friction τ_a and ultimate base resistance p_{bu} are chosen for each element (e.g. the values for static loading).

(2) The pile is analyzed for the maximum load P_{max}, and the distributions of shear stress and displacement along the pile are determined.

(3) The pile is similarly analyzed for the minimum load P_{min}.

(4) The cyclic displacement ρ_c at each element is determined by subtracting the minimum value (Step 3) from the maximum value (Step 2).

(5) For the specified number of cycles and the cyclic displacement, the degradation factors D_τ, D_b and D_E are determined from relationships such as those in Figs.1 and 2, and Eq.2.

(6) Revised values of ultimate skin friction, ultimate base resistance and soil modulus are determined by multiplying the static values by the appropriate degradation factor and the rate factor D_R (Eq.3). These revised values are compared with the existing values, and if the difference is greater than a specified tolerance, Steps 2 and 6 are repeated until the desired degree of convergence is obtained.

(7) The cyclic deflection, the mean deflection, and the values of ultimate skin friction and base resistance corresponding to N cycles of load (range P_{max} to P_{min}) are thus obtained, from which the available ultimate load capacity after cycling may be readily calculated.

In order to obtain more rapid convergence, it is desirable to choose reasonable first estimates of the soil parameters, and this can be achieved by carrying out successive incremental analyses in which the first estimates of the parameters for each increment are the values for the previous increment.

4.2 Pile Groups.

If the piles in a group are symmetrically arranged around the circumference of a circle, as is the case for offshore pile groups, all piles will behave identically under axial loading and consequently the single pile analysis can very easily be extended to analyze the behaviour of the piles in the group. The only modification required is in the calculation of the soil displacements at each element, which now include components due to the surrounding piles in the group in addition to the subject pile itself. These components can be evaluated from Mindlin's equations of elasticity, as described by Poulos (1968). Solution of the resulting displacement compatability equations, the checks for pile-soil slip and yield, and the incorporation of cyclic degradation and rate effects, are then carried out as for a single pile. A computer program, CYCPL7, has been written to carry out this analysis, this program being an extension of an earlier program (CYCPL6) for single piles.

The significance of group effects will be discussed in detail below, but in brief, since cyclic degradation is dependent on cyclic displacement, and since the cyclic displacement of a pile in a group is greater than for an isolated single pile, it follows that the effects of cyclic loading on a pile group will be more severe than on a single pile (for the same cyclic load per pile).

5. SOME THEORETICAL RESULTS

To illustrate some of the characteristics of piles under cyclic axial loading, solutions have been obtained for a hypothetical case in which steel tube piles are driven into a deep deposit of relatively stiff clay. The piles are 72 m long, 1.4 m diameter and have a 50 mm wall. The soil is assumed to have a constant undrained shear strength of 200 kPa, a constant ultimate skin friction of 50 kPa (in both compression and tension), an ultimate base resistance of 1.8 MPa in compression and 0.1 MPa in tension and a constant Young's modulus of 50 MPa. The degradation characteristics for both shaft and base are assumed to be as shown in Figs.1 and 2, and the loading rate effects are given by Eq.3 with a rate coefficient F_ρ of 0.1. Each pile has a static compressive load P_0 of 6 MN applied to it (corresponding to a static safety factor of about 3) and a cyclic component $\pm P_c$.

The following factors are investigated:

(i) the influence of cyclic load amplitude P_c and number of cycles N

(ii) the influence of number of piles

(iii) the influence of loading rate.

Fig.5 plots, for a single pile, the computed ultimate load capacity after cycling, P_{uc}, against the cyclic load amplitude P_c, for N=1000 cycles. As would be expected, P_{uc} decreases as P_c increases, and if P_c = ±8.5 MN, failure will occur during cyclic loading. At small values of P_c, the ultimate load capacity exceeds the static value of 18.6 MN because of rate effects.

Also shown in Fig.5 are the corresponding relationships between P_{uc} and P_c for groups of 4 and 8 piles. The load capacity after cycling is substantially reduced because of group effects, and failure during cycling occurs at a cyclic load of about ±6.5 MN. For P_c in excess of about ±4 MN, the value of P_{uc} does not decrease with increasing P_c because the limiting degradation factor is reached along the length of the pile. The significance of group effects can best be appreciated by considering the safety factor SF against ultimate failure for a typical cyclic load of P_c = ±3 MN (i.e. a total maximum load of 9 MN). Referring to Fig.5 for static conditions, SF is 18.6/9.0 = 2.07, and for a single pile under cyclic loading, SF rises to about $\frac{21.9}{9}$ = 2.3 because of rate effects. However, for a group of 4, SF is reduced to 13.9/9 = 1.5 and for a group of 8, SF reduces even further to 12.5/9 = 1.4. Such reductions have obvious important implications in pile design.

Fig.6 shows stress and load distributions for a single pile and a pile in a group of 4 and 8 piles. As the number of piles increases, smaller shear stresses are developed near the top of the pile and more load is transferred to the lower portion and to the base. Consequently, the nature of the bearing stratum will be of greater importance for a pile group than for a single pile.

Fig.7 shows the decrease in pile load capacity and stiffness with increasing numbers of cycles for a cyclic load of ±3 MN. The ultimate load will eventually asymptote to the value corresponding to

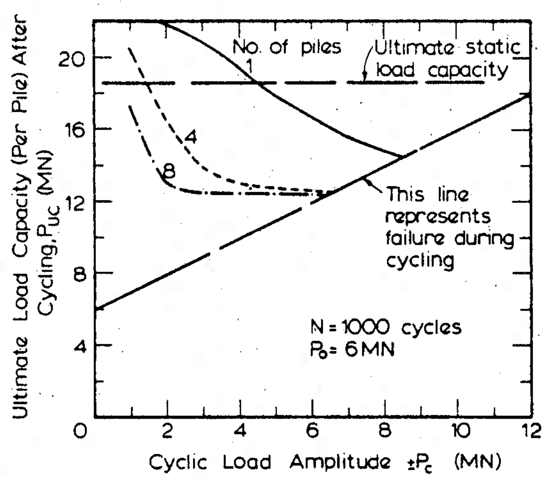


FIG.5 EFFECT OF CYCLIC LOADING ON ULTIMATE LOAD CAPACITY

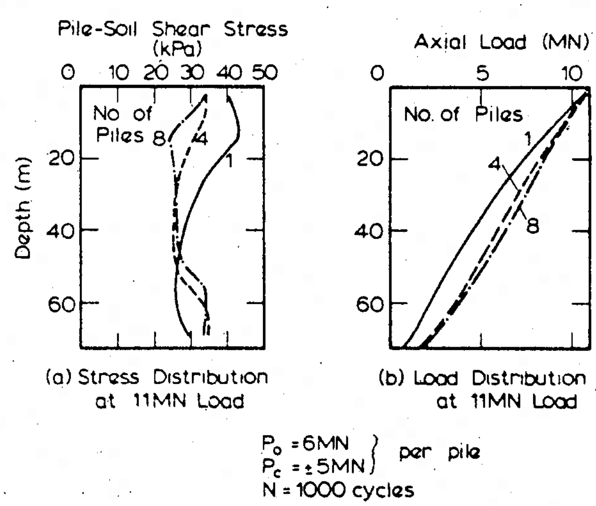


FIG.6 INFLUENCE OF NUMBER OF PILES ON STRESS AND LOAD DISTRIBUTIONS

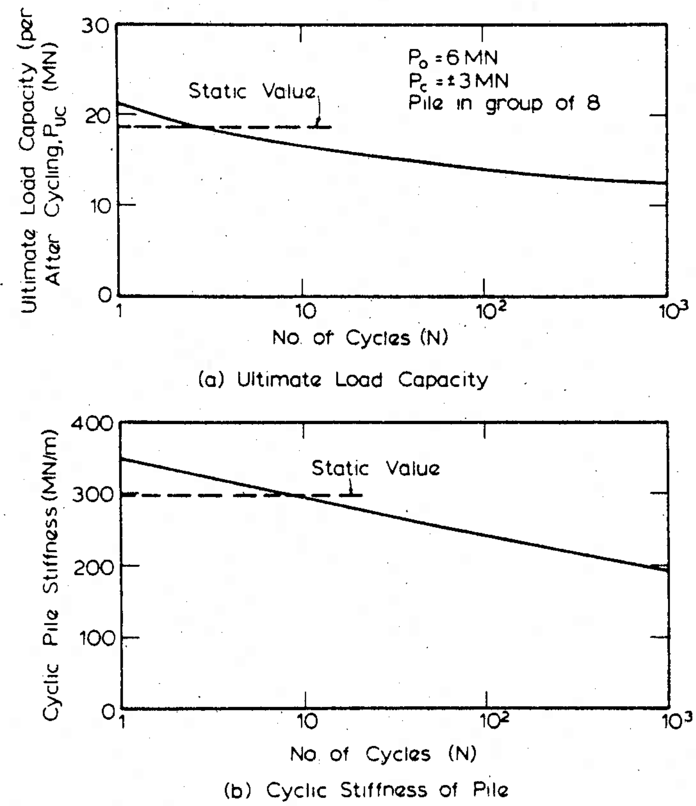


FIG.7 EFFECT OF NUMBER OF CYCLES ON PILE CAPACITY AND STIFFNESS

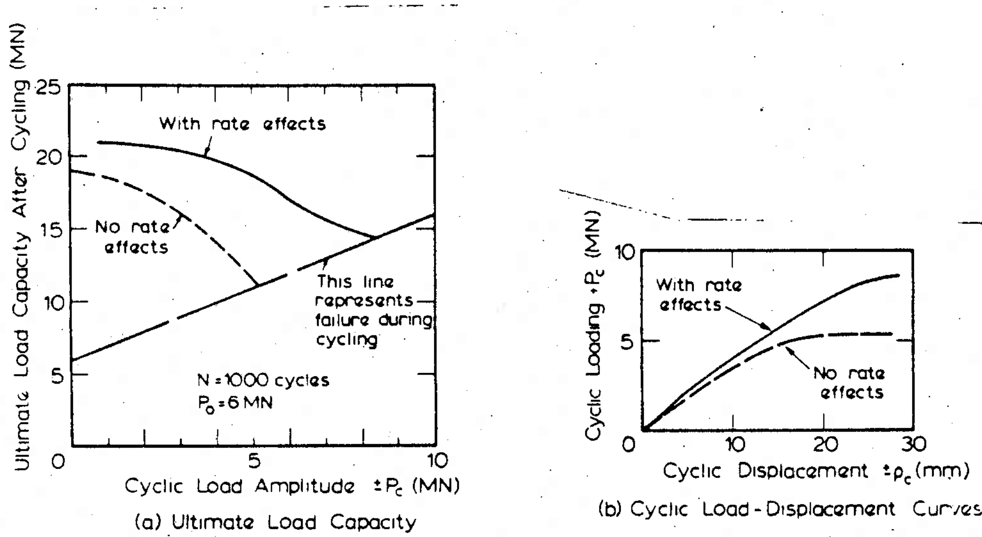


FIG.8 EFFECTS OF LOADING RATE-SINGLE PILE

the limiting degradation factors for skin and base resistances, but the cyclic pile stiffness decreases more or less linearly with the logarithm of number of cycles.

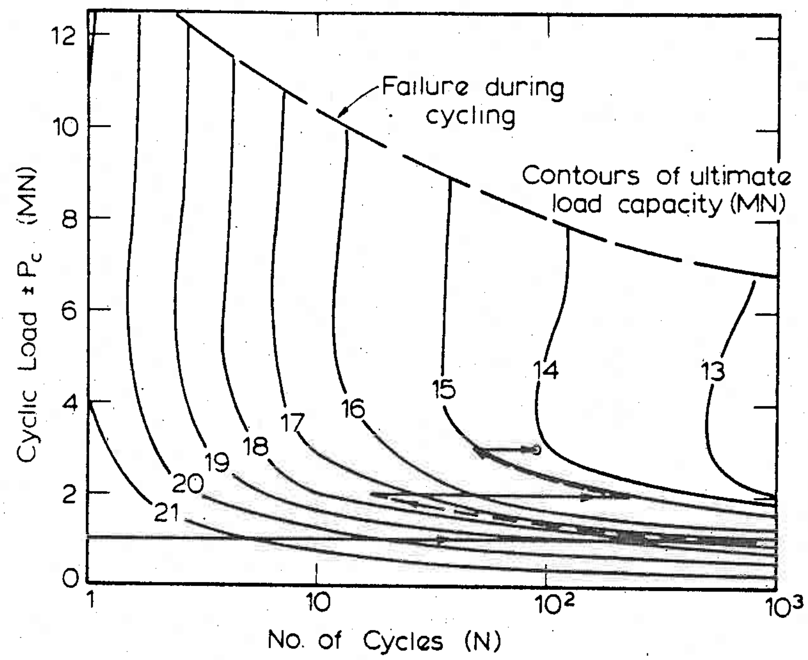
The important influence of including loading rate effects in the analysis is illustrated in Fig.8 for a single pile. With rate effects, pile failure will occur after about ±8.5 MN; however, if no rate effects are present, a cyclic load of only about ±5.2 MN will cause failure after 1000 cycles.

The combined effects of cyclic load level and number of cycles may be conveniently represented in the form of a contour diagram (Fig.9) in which contours of load capacity after cycling, P_{uc}, are plotted against cyclic load P_c and number of cycles N. Such a diagram can be prepared by carrying out a series of analyses in which the cyclic load level is progressively increased for a given number of cycles, and various numbers of cycles are considered. The use of this diagram for estimating the effects of variable cyclic loading (e.g. "storm" loading sequences) is discussed below.

6. VARIABLE CYCLIC LOADING

The above solutions for cyclic response assume that the cyclic load amplitude remains constant for the specified number of cycles. For the design of offshore foundations, cyclic loading due to wave action is difficult to characterize by an equivalent number of cycles of uniform cyclic load amplitude and it is usual to consider a number of "parcels" of cyclic loading involving various numbers of cycles of cyclic load of different amplitudes. In order to obtain an estimate of the effect of such a cyclic loading sequence, an approximate graphical procedure has been devised which is similar in principle to the procedure described by Andersen (1976) for computing shear strains in a clay.

From the solutions obtained for cyclic loading of constant amplitude, a series of contours of equal ultimate load capacity can be drawn on a plot of cyclic load level versus number of cycles, as shown in Fig.9 for the example considered in the previous section. The "parcels" in the cyclic loading sequence are then considered in turn, assuming that the order of application of the parcels does not influence the final result.



± P_c MN	N	P_{UC} MN	N_e
1.0	900	17.5	900
2.0	200	15.1	217
3.0	40	14.2	90

FIG.9 CONTOURS OF ULTIMATE LOAD CAPACITY AFTER CYCLING. PILE IN GROUP OF 8. P =6MN. CONSTRUCTION FOR VARIABLE CYCLIC LOADING

For the first cyclic load amplitude, P_1, the ultimate load capacity P_{uc1} at the end of the appropriate number of cycles N, is determined from this plot. At the next cyclic load level, P_2, the equivalent number of cycles, N_{e2}, to give this same ultimate load capacity P_{uc1} is then estimated from the contours. The second parcel of load (N_2 cycles) is taken as occurring from N_{e2} to ($N_{e2} + N_2$) cycles of load of amplitude P_2. At this point, the ultimate load capacity will be P_{uc2}, and the equivalent number of cycles, N_{e3}, to give this ultimate load capacity at the next cyclic load level, P_3, is determined. The procedure is repeated until the entire cyclic load sequence is considered.

To illustrate the application of the above procedure, the following cyclic loading sequence is considered:

900 cycles of ± 1.0 MN
200 cycles of ± 2.0 MN
40 cycles of ± 3.0 MN

The construction for this sequence are shown in Fig.9. At the end of the sequence, the estimated load capacity is 14.2 MN. The above sequence corresponds to approximately 90 cycles of load of amplitude ±3 MN, and hence the cyclic stiffness at the end of the loading sequence is most easily obtained by determining the value for 90 cycles of ±3 MN (in this case, about 242 MN/m).

The applicability of the above procedure remains to be verified, but it does appear to provide a simple and reasonably logical means of estimating the response of a pile to variable cyclic loading, or at the very least, of determining an equivalent number of cycles of a given cyclic load amplitude for which to carry out a detailed analysis of the pile response.

7. CONCLUSIONS

Cyclic loading of a pile is characterized by two opposing phenomena; cyclic degradation effects which tend to decrease both pile capacity and stiffness, and loading rate effects which tend to increase these quantities. The analysis described herein is a development of earlier analyses, and takes both phenomena into account. In particular it utilizes recent data relating the cyclic degradation of skin friction and soil modulus at a point on the pile to the cyclic displacement at that point. Another important extension to the analysis is the incorporation of group effects. Since these tend to increase displacements, they also tend to cause more severe degradation. The consequences of group effects are demonstrated by an example which indicates that the maximum cyclic load (per pile) which can be applied decreases significantly as the number of piles in a group increases. The example also illustrates the important influence of loading rate in increasing both load capacity and cyclic stiffness of the pile.

A procedure is suggested for determining the behaviour of a pile subjected to variable cyclic loading, using the solutions for constant cyclic load magnitude. Such a procedure should prove useful for assessing the effects of storm wave loading on offshore piles and pile groups.

Much work remains to be done before a proper understanding of the effects of cyclic loading can be achieved; this applies in particular to piles in sand which appear to display a tendency to continuously accumulate permanent settlements, even at relatively low levels of cyclic load.

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REFERENCES

ANDERSEN, K. (1976). "Behaviour of Clay Subjected to Undrained Cyclic Loading." Proc. Conf. Behaviour of Offshore Structures (BOSS), Trondheim, Vol.1, pp.392-403.

AURORA, R.P., PETERSON, E.H. and O'NEILL, M.W. (1981). "Model Studies of Long Piles in Clay." Proc. 13th OTC Conf., Houston, Paper OTC 4149, pp.331-338.

BEA, R.G., AUDIBERT, J.M.E., and DOVER, A.R. (1980). "Dynamic Response of Laterally and Axially Loaded Piles." Proc. 12th OTC Conf., Houston, Paper OTC 3749, pp.129-139.

BJERRUM, L. (1973). "Problems of Soil Mechanics and Construction on Soft Clays and Structurally Unstable Soils." State of the Art Rep., 8th Int. Conf. S.M. & F.E., Moscow, Vol.3, pp.111-159.

BOULON, M., DESRUES, J., FORAY, P. and FORGUE, M. (1980). "Numerical Model for Foundation Under Cyclic Loading, Application to Piles." Int. Symp. Soils Under Cyc. and Trans. Loading, Swansea, Wales. A.A. Balkema, Rotterdam, pp.681-694.

CHAN, S.F. and HANNA, T.H. (1980). "Repeated Loading on Single Piles in Sand." Jnl. Geot. Eng. Divn. ASCE, Vol.106, No.GT2, pp.171-188.

GALLAGHER, K.A. and ST. JOHN, H.D. (1980). "Field Scale Model Studies of Piles as Anchorages for Buoyant Platforms." Eur. Off. Pet. Conf. and Exhibition, London, England.

GROSCH, J.J. and REESE, L.C. (1980). "Field Tests of Small-Scale Pile Segments in a Soft Clay Deposit under Repeated Axial Loading." Proc. 12th OTC Conf., Houston, Paper OTC 3869, pp.143-151.

GUDEHUS, G. and HETTLER, A. (1981). "Cyclic and Monotonous Model Tests in Sand." Proc. 10th ICSMFE, Stockholm, Vol.2, pp.211-214.

HOLMQUIST, D.V. and MATLOCK, H. (1976). "Resistance - Displacement Relationships for Axially-Loaded Piles in Soft Clay." Proc. 8th OTC Conf., Houston, Paper OTC 2474, pp.554-569.

IDRISS, I.M., DOBRY, R. and SINGH, R.D. (1978). "Nonlinear Behaviour of Soft Clays During Cyclic Loading." Jnl. Geot. Eng. Divn. ASCE, Vol.104, No.GT12, pp.1427-1447.

KRAFT, L.M., COX, W.R. and VERNER, E.A. (1981). "Pile Load Tests: Cyclic Loads and Varying Load Rates." Jnl. Geot. Eng. Divn, ASCE, Vol.107, No.GT1, pp.1-19.

MATLOCK, H. and FOO, S.C. (1979). "Axial Analysis of Pile Using a Hysteretic and Degrading Soil Model." Proc. Conf. Num. Methods in Offshore Piling, I.C.E., London, pp.165-185.

POULOS, H.G. (1968). "Analysis of the Settlement of Pile Groups." Geotechnique, Vol.18, pp.449-471.

POULOS, H.G. (1979a). "Development of an Analysis for Cyclic Axial Loading of Piles." Proc. 3rd Int. Conf. Num. Meth. Geomechs, Aachen, Vol.4, pp.1513-1530.

POULOS, H.G. (1979b). "Settlement of Single Piles in Non-Homogeneous Soils." Jnl. Geot. Eng. Divn. ASCE, Vol.105, No.GT5, pp.627-641.

POULOS, H.G. (1981a). "Some Aspects of Skin Friction of Piles in Clay Under Cyclic Loading." Geot. Eng., Vol.12, No.1, pp.1-17.

POULOS, H.G. (1981b). "Cyclic Axial Response of Single Pile." Jnl. Geot. Eng. Divn, ASCE, Vol.107, No.GT7, pp.41-58.

PUECH, A. and JEZEQUEL, J.F. (1980). "The Effects of Long Time Cyclic Loadings on the Behaviour of a Tension Pile." Proc. 12th OTC Conf. Houston, Paper OTC 3870, pp.153-162.

STEENFELT, J.S., RANDOLPH, M.F. and WROTH, C.P. (1981). "Model Tests on Instrumented Piles Jacked into Clay." Proc. 10th Int. Conf. S.M. & F.E., Stockholm, Vol.2, pp.857-864.

VAN WEELE, A.F. (1979). "Pile Bearing Capacity Under Cyclic Loading Compared with that Under Static Loading." Proc. 2nd Int. Conf. Beh. Off. Structures, London, pp.475-488.