STORM-INDUCED CYCLIC LOADING OF ANCHOR PILES IN CLAY

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ABSTRACT

The response of anchor piles in clay under sustained, storm-induced cyclic loading has been investigated in a series of rapid static and cyclic model pile load tests supplemented by static and cyclic laboratory tests used to define the properties of the reconstituted foundation soil. These experimental results are evaluated and compared to similar data from the literature in order to provide insight with regard to some critical design aspects, namely the degradation of cyclic stiffness, the accumulation of permanent displacements and the ultimate pile capacity under combined static and cyclic pullout loads. Drainage conditions at the pile-soil interface, as well as, applied load magnitudes and number of cycles are identified as the main factors which control the pile response. The effect of these factors is described by simple analytical expressions and charts that can be used for the design of anchor piles in offshore environment.

KEYWORDS

Model Piles, Clay, Cyclic Loads, Tension.

INTRODUCTION

Anchor piles is a common means for the foundation of many offshore structures, including Tension Leg Platforms (TLPs) for deep water oil production and exploration. At calm weather conditions, the loads applied on the pile head are static with a predominant axial tensile component; during a storm, pile loads fluctuate around a mean value at a rather slow frequency, in the range of 0.1 Hz or less. This combination of static and cyclic axial loads may lead to accumulation of pile head displacement either at gradually decreasing rates or at gradually increasing rates which lead to failure of the foundation. These two distinct modes of response have been identified in many previous model and in-situ pile load tests and are commonly refered as "shakedown" and "incremental collapse" respectively.

The axial cyclic response of piles became the subject of systematic research recently, mostly in connection with the need to build oil production platforms at large water depths and hostile environments. Despite this, the available literature on the subject is rich both in experimental studies (e.g. Kraft *et al.*, 1981; Mattlock *et al.*, 1982; Karlsrud and Haugen, 1985; Jaime *et al.* 1990; Dunnavant *et al.*, 1990) and in theoretical analyses (e.g. Karlsrud *et al.*, 1986; Poulos, 1988; Malek *et al.*, 1988; Bea, 1992). However, the emphasis so far has been placed upon piles in compression used to support steel jacket anf gravity type platforms. Piles in tension have received relatively little attention and their design is essentially based on concepts derived for piles in compression.

Table 1. Summary of Pile Load Test Conditions

Test	Q_{av}/Q_u	$Q_c/2Q_u$	No. of Cycles	Time after Unloading (h)	Test Type
1a	-	-	-	27	Rapid Static
2a	-	-	-	27	Rapid Static
3a	-	-	-	110	Rapid Static
4a	-	-	-	105	Rapid Static
5a	0.52	0.17	1006	54	Cyclic
6a	0.59	0.39	39	84	Cyclic
7a	0.57	0.31	50	80	Cyclic
8a- I	0.50	0.12	200	52	Cyclic
8a- II	0.52	0.23	200	52	Cyclic[1]
8a-III	0.53	0.30	200	52	Cyclic[1]
8a-IV	0.53	0.37	165	52	Cyclic[1]
9a	-	-	-	166	Rapid Static

[1] Cyclic tests with preshearing. No drainage was allowed upon load change.

Table 2. Properties of Nivaa Clay (D.G.I., 1991)

Liquid Limit (%)	w_L	35.0
Plastic Limit (%)	w_P	15.0
Plasticity Index (%)	I_P	20.0
Grain Unit Weight (KN/m^3)	γ_s	27.3
Permeability[1] (m/s)	k	8×10^{-11}
Coeff. of Consolidation[1] (m^2/s)	cv	10^{-7} to 10^{-8}
Undr. Shear Strength Ratio	C_u/σ'_{vc}	0.30

[1] from 1-D consolidation tests

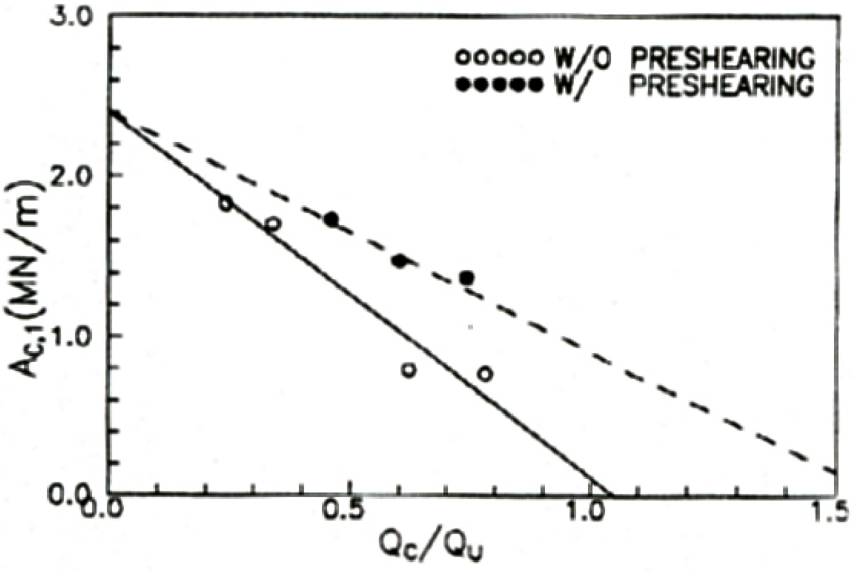

Fig. 1. Effect of cyclic load magnitude on cyclic stiffness

To investigate the response of anchor piles in clay under sustained cyclic loading, a number of cyclic model pile load tests has been performed at the Danish Geotechnical Institute, in an artificial bed of reconstituted normally consolidated clay (D.G.I., 1991a). The cyclic tests have been supplemented by rapid static pullout tests performed in order to provide refference values of the static ultimate pile capacity, and also static and cyclic laboratory tests for the evaluation of the response of the foundation soil. Data from these tests, as well as from tests published in the literature are evaluated here with respect to three critical aspects of pile design: the degradation of cyclic stiffness, the accumulation of permanent displacements and the ultimate pile capacity under combined monotonic and cyclic loads.

To aid the presentation, the definition for some basic parameters used to describe the pile response is summarized below. Denoting with Q_{max} and Q_{min} the maximum and minimum values of the tensile axial load applied on the pile head during a load cycle N, and with d_{max} and d_{min} the respective values of the pile head displacement, the cyclic stiffness of the pile and the different components of load and displacement are defined as:

Permanent Displacement	$d_{p,N} = (d_{max}+d_{min}))/2$
Cyclic Displacement	$d_{c,N} = d_{max}-d_{min}$
Average Load	$Q_{av,N} = (Q_{max}+Q_{min})/2$
Cyclic Load	$Q_{c,N} = Q_{max}-Q_{min}$
Cyclic Stiffness	$A_c = Q_c/D_c$

EXPERIMENTAL PROGRAMME

The experimental programme has been planned and executed such as to provide a consistent set of data that can be analysed to give insight to the mechanisms controlling the response of anchor piles, and included the following tests:

- Five (5) *rapid static pullout tests*, in order to estimate the static ultimate capacity of the model piles (Q_u) for loading rates comparable to the ones imposed by storm-induced cyclic loading. The tests were performed with displacement control and an approximate time to failure of 8 to 13 s.

- Four (4) *cyclic loading tests in tension*, with a loading frequency of 0.1Hz, a static load ofsset (Q_{av}) between 50% and 60% of the static pullout capacity obtained from the rapid static tests, and various cyclic load amplitudes.

- Four (4) *post-cyclic rapid static pullout tests* in order to estimate the degradation of static pullout capacity due to sustained cyclic loading.

- Eight (8) undrained *cyclic triaxial tests* with the aim to assess the cyclic response of the clay used as foundation soil.

All pile load tests were performed on a closed-end model alluminum pile with length L=40 cm and diameter D=1.9 cm, jacked into clay cakes of remoulded and normally consolidated Nivaa clay. This is a natural meltwater clay of Denmark, that has been extensively used in previous model test series at D.G.I. (1991b). To simulate in situ stress conditions, the clay cakes were allowed to consolidate under overburden pressures of 220 and 260 KPa, and were consequently unloaded prior to the execution of the tests. With this procedure, effective stresses in the soil after unloading remain approximately equal to the applied overburden, due to negative pore pressure build up. With time, however, negative pore pressures dissipate reducing the effective stresses and changing the clay cakes from normally to over-consolidated; hence, model testing had to be performed within a short time after the removal of the overburden to minimize this effect.

Table 1 summarizes the conditions of the initial static and the cyclic pile load tests which are reviewed here, while Table 2 summarizes some basic classification and strength parameters for remoulded and normally consolidated Nivaa clay determined from previously reported tests (D.G.I., 1991b).

EVALUATION OF CYCLIC STIFFNESS

Effect of Load Magnitude

The cyclic stiffness of axially loaded long friction piles is mainly related to the geometry of the pile and the shear modulus of the soil around the pile shaft. Thus, it is reasonable that the cyclic stiffness is a function of the applied cyclic loads or displacements, such as the shear modulus is a function of the applied cyclic shear stresses or strains. In fact, previous experimental studies have shown that, during one-way cyclic loading in tension or in compression, the cyclic stiffness decreases with increasing cyclic load amplitude while it is practicaly independent from the applied static load offset.

The available pile load tests in Nivaa clay have been peformed under a more or less constant static load offset and consequently they can only be used to investigate the effect of cyclic load magnitude. This is shown in Fig. 1 which relates the cyclic stiffness measured at the first load cycle $A_{c,1}$, to the cyclic load ratio Q_c/Q_u, where Q_u is the ultimate pile capacity under rapid static pullout loading. It is observed that the cyclic stiffness decreases significantly with increasing cyclic load amplitude, especially in tests 5a, 6a, 7a and 8a-I where the piles had not been subjected to any previous cyclic loading (cyclic preshearing). In a simple form, the test data may be described as:

$$A_{c,1}\ (\mathrm{KN/m}) = 2400 - 2300\ Q_c/Q_u \quad \text{(w/o preshearing)} \tag{1a}$$

or

$$A_{c,1}\ (\mathrm{KN/m}) = 2400 - 1500\ Q_c/Q_u \quad \text{(w/ preshearing)} \tag{1b}$$

From a practical view point, it is also interesting to compare the cyclic to the static load-displacement relationships obtained from the pile load tests. This is shown in Fig. 2, where cyclic displacements in the first load cycle are plotted against cyclic load amplitudes. It is observed that the average load-displacement curve from the rapid pullout test, drawn in the same figure, may be considered as a reasonable average fit to the data from tests with and without cyclic preshearing. A similar conclusion had been derived earlier by Karlsrud and Haugen (1985) based on data from one-way cyclic tests and rapid static tests in tension and in compression, performed in a natural overconsolidated deposit of Haga clay.

Effect of Number of Cycles

Fig. 3 shows the variation with number of load cycles of the cyclic stiffness measured in tests 5a, 6a, 7a and 8a-I with no cyclic preshearing. For a unique presentation of all tests, the cyclic stiffness after N cycles ($A_{c,N}$) has been normalised with respect to the cyclic stiffness in the first cycle ($A_{c,1}$). It is observed that the ratio $A_{c,N}/A_{c,1}$ may increase slightly or may decrease with number of cycles depending upon the intensity of applied shaking. The slight increase is observed for tests 5a and 8a-I with peak axial load Qp ($=Q_{max}$) equal or less than 69% of static pullout capacity under rapid loading Q_u, while the decrease is observed for tests 6a and 7a with Q_p higher than 88% of Q_u. In the later case, the decrease in cyclic stifness is gradual at the initial stages of loading, but accelerates rapidly after a certain number of load cycles.

Assuming a power relationship between the cyclic stiffness and the number of cycles at the initial stages of cyclic loading, the data in Fig. 3, are expressed as:

$$A_{c,N} = A_{c,1} \cdot N^{-\alpha} \tag{2}$$

The values of the exponent α estimated from all cyclic load tests in Nivaa clay are plotted in Fig. 4 versus the peak cyclic load ratio Q_p/Qu. It is observed that α varies between -0.10 and 0.20, increasing systematicaly with increasing peak cyclic load ratio.

Effect of Drainage Conditions

In a previous study, Briaud and Felio (1986) used a similar power relationship to fit experimental data from a number of one-way cyclic pile load tests (in tension or in compression) published in the literature,

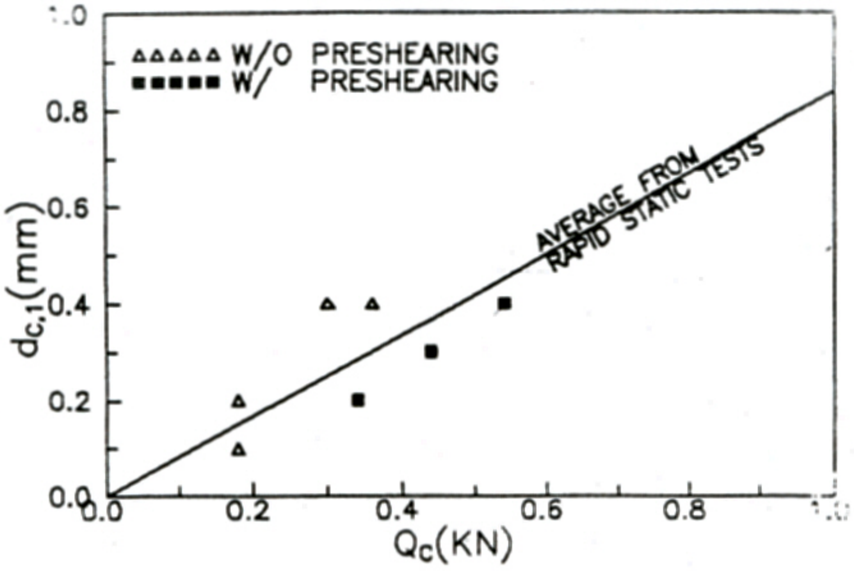

Fig. 2. Cyclic versus static load-displacement relationship.

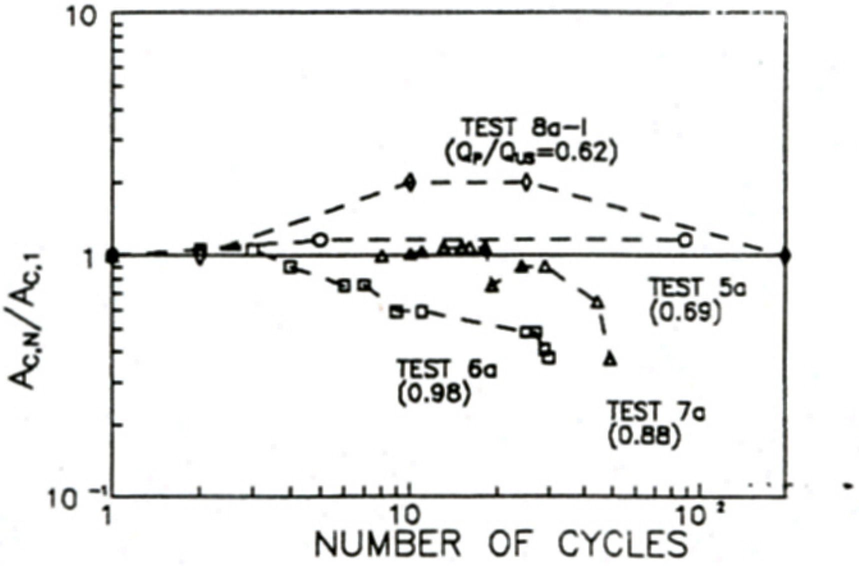

Fig. 3. Effect of number of cycles on cyclic stiffness.

with various pile geometries, loading frequencies and clay properties. Most of the tests examined in that study show a gradual degradation of cyclic stiffness with number of cycles which is simulated by Eq. 2, with the exponent α computed as:

$$\alpha = 0.19\ Q_p/Q_{us} \tag{3}$$

where Q_{us} denotes the static pullout capacity of the piles under conventional slow loading rates. The data in Figs. 3 and 4 contradict in part the data presented by Briaud and Felio since they show that a hardening in the cyclic stiffness is also possible under certain conditions.

One possible explanation of this discrepancy is that the exponent α in Eq. 2 is not only a function of the peak cyclic load magnitude but it also depends on the amount of drainage that takes place in the clay

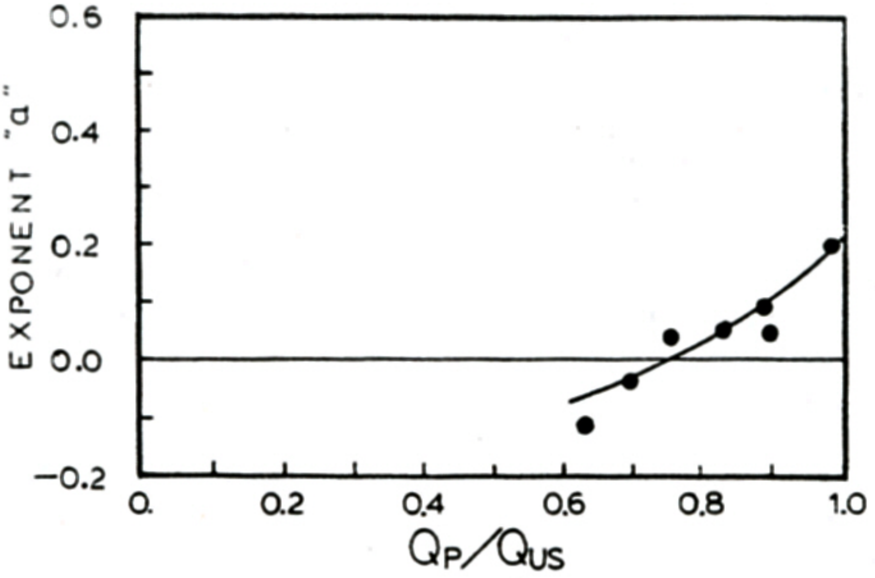

Fig. 4. Variation of exponent **α** (Eq. 2) with peak cyclic load.

around the pile during each load cycle. For large diameter piles and high loading frequencies, excess pore pressures build up around the pile shaft as a result of cyclic loading, and consequently the respective effective stresses and the soil stiffness are reduced. The opposite may occur for small diameter piles and low loading frequencies where excess pore pressures can dissipate within each load cycle and consequently the soil stiffness may increase slightly as a result of the pseudo-overconsolidation imposed by cyclic loading under drained conditions.

To check this assumption, the data presented by Briaud and Felio, as well as the data from the pile load tests in Nivaa clay are shortened in Table 3 with respect to the non-dimentional time for radial consolidation, with time replaced by the period of cyclic loading:

$$T_h = c_h / f r_o^2 \tag{4}$$

where c_h is the coefficient of consolidation within a horizontal plane, f is the frequency of cyclic loading and r_o is the pile radius. The value of c_h was not reported for any of the tests in Table 3 and it was arbitrarily chosen as 10^{-7} m^2 /s, i.e. equal to the coefficient of consolidation c_v from 1-D consolidation tests on Nivaa clay (Table 2).

Three groups of data may be distinguished in this way, with an order of magnitude difference from each other in the estimated values of T_h:

- Group A, with $T_h = 0.44 \times 10^{-4}$ to 1.04×10^{-4}
- Group B, with $T_h = 1.01 \times 10^{-3}$ to 1.80×10^{-3}
- Group C (Nivaa Clay), with $T_h = 1.05 \times 10^{-2}$

The variation of exponent **α** in Eq. 2 with the peak load ratio Q_p/Q_{us} is shown in Fig. 5, separately for each group of tests. For the tests in Nivaa clay, the ultimate static capacity for slow loading rates Q_{us} was deduced after a 20% reduction of the static capacity measured in the rapid static Q_u; the amount of reduction was estimated approximately based on published experimental studies of the effect of loading rate on the axial capacity of friction piles in clay (e.g. Bea and Audibert, 1979; Kraft *et al.*, 1981; Doyle and Pelletier, 1985). There is considerable scater in the data, but it can be clearly observed that **α** is grossly reduced from test group A to test group C, i.e. for increasing radial consolidation time T_h. In the contrary, the critical value of the peak cyclic load ratio required to trigger the degradation phenomenon increases with T_h, from about 0.40 for test groups A and B to about 0.90 for test group C.

Table 3. Pile Load Tests used to determine the Effect of Drainage on Cyclic Stiffness Degradation

Clay	Pile Diameter (cm)	Loading Frequency (Hz)	T_h[1] (sec/m^2)
Group A			
- *Cran*	27.4	0.070	0.76×10^{-4}
- *Haga*	15.2	0.167	1.04×10^{-4}
- *Empire*	35.7	0.070	0.44×10^{-4}
Group B			
- *Upsala I*	25x25	3.30×10^{-3}	15.2×10^{-4}
- *Upsala II*	32x35	2.78×10^{-3}	10.1×10^{-4}
- *Bjorktorp*	25x25	2.78×10^{-3}	18.1×10^{-4}
Group C			
- *Niva*	1.95	0.10	105.2×10^{-4}

[1] In the absence of relevant data, it was assumed that $c_h = 10^{-7}$ m^2/s

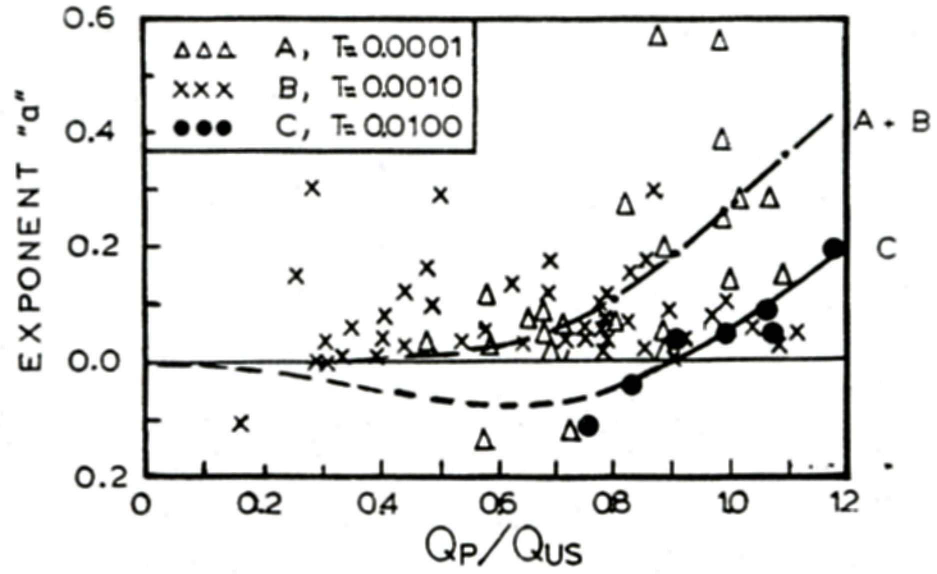

Fig. 5. Effect of peak cyclic load magnitude and drainage conditions on the exponent α (Eq. 2).

It is important to notice that all pile load tests examined in Fig. 5 have been performed in clayey sites and consequently the differences in T_h from one group of tests to the other were mainly due to different pile diameters and loading frequencies. In practical offshore applications, however, these factors have much less variability and consequently hardening or degradation of cyclic stiffness will also depend upon the in situ coefficient of consolidation c_h and in extend of the soil consistency (e.g. sand or silt vs clay). As an example we may refer to the test data for a predominantly cohesionless and relatively permeable soil site, presented by Puech (1982), where no degradation in cyclic stiffness has been observed for peak cyclic load ratios Q_p/Q_{us} as high as 60% even though the pile diameter and the loading frequency were similar to those of Group A in Table 3.

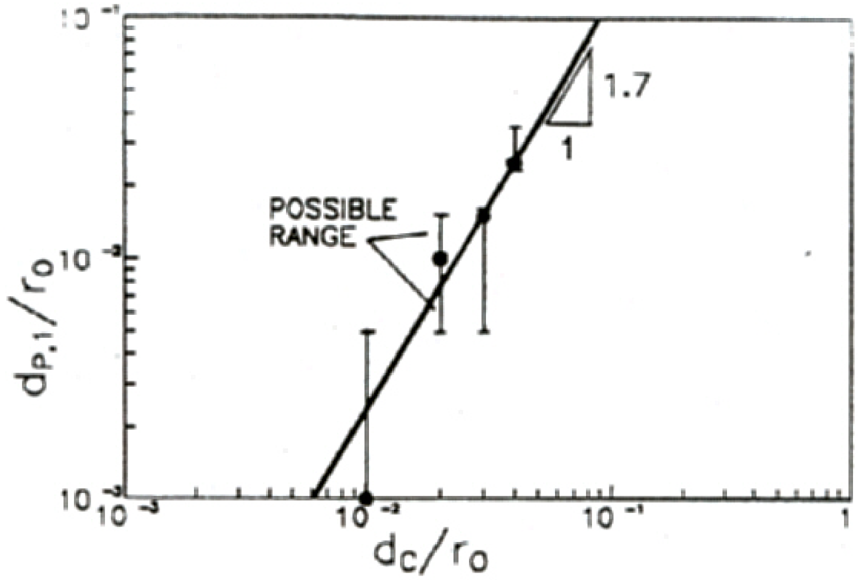

Fig. 6. Relationship between cyclic and permanent pile head displacements in the first load cycle.

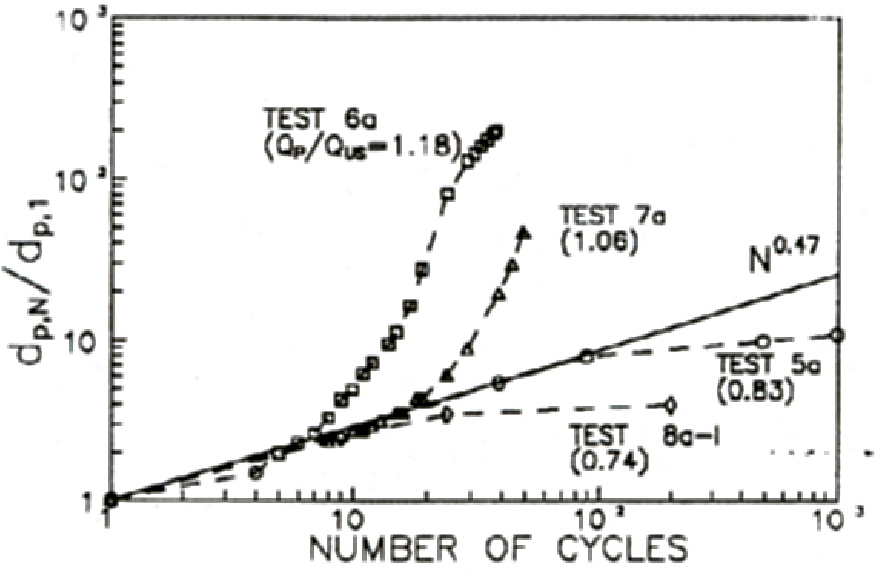

Fig. 7. Accumulation of premanent pile head displacement with number of cycles.

PERMANENT DISPLACEMENT ACCUMULATION

One way cyclic axial loading of piles, in tension or in compression, is commonly accompanied by the accumulation of permanent pile head displacements, as a result of plastic strain accumulation in the nearby soil. The cyclic tests in Nivaa clay provide data which can be used for the systematic evaluation of two basic factors which affect the magnitude and the rate of accumulation of permanent displacements; cyclic displacement and number of cycles. The equally critical effect of the average load offset cannot be investigated based on these data since all tests have been performed under essentially equal average loads.

Fig. 6 correlates permanent displacements of the pile head after the first cycle of loading ($d_{p,1}$) to the respective cyclic displacement ($d_{c,1}$) in a double logarithmic graph. For a generalised presenattion of the test data, which will not depend directly upon the size of the test piles, both displacements have been normalized with respect to the pile radius r_o. It is observed that permanent displacements increase with increasing applied cyclic displacements; in simple terms, this relationship may be expressed as:

$$d_{p,1}/r_o = A\,(d_c/r_o)^{1.70} \qquad (5)$$

where A=6.00 is a constant which is potentially related to the applied average load, the soil conditions, the loading frequency, etc.

The effect of number of cycles on permanent displacement accumulation is shown in Fig. 7, based on data from cyclic tests 5a, 6a, 7a and 8a-I which have not been subjected to cyclic preshearing. For a unique presentation of all tests, permanent displacements after N cycles ($d_{p,N}$) have been normalized with respect to the permanent displacement after the first cycle ($d_{p,1}$), and plotted as a double logarithmic graph. Two distinct modes of response are observed:

- In tests 5a and 8a-I, with moderate shaking ($Q_p/Q_{us} < 0.83$), permanent displacements evolve initially according to a linear logarithmic relationship with respect to number of cycles, and tend to stabilize at later stages of the test (Shakedown).
- In tests 6a and 7a, with intense shaking ($Q_p/Q_{us} > 1.06$), permanent displacements accumulate initially as in tests 5a and 8a-I, but after a certain cycle the rate of permanent displacement accumulation undergoes a rapid increase which ultimately leads to extremely large displacements (Incremental Collapse).

Regardless of the intensity of cyclic loading, the initial part of the permanent displacement accumulation curves in Fig. 7 may be described by a single power expression:

$$d_{p,N} = d_{p,1}\,N^b \qquad (6)$$

with b = 0.47. A similar expression, but with b=0.07±0.05, had been proposed earlier by Bea and Audibert (1979) to describe the permanent displacement accumulation in one-way cyclic pile load tests in clay with peak load ratios Q_p/Q_{us} less than 0.90. The data reported in that study concern mostly piles in compression and this may be one reason for the substantialy lower value obtained for the exponent b. It is possible that other factors may have also contibuted to the difference. This, however, is not known with certainty due to the limited information which are given about the reviewed pile load tests.

CYCLIC UPLIFT CAPACITY

From the previous presentation it is evident that, under certain conditions, the rate of change in permanent and cyclic displacements may become unacceptably large and lead to failure even at peak loads less than the static ultimate capacity of the pile. To define the basic mechanisms associated with failure under combined static and cyclic loading, Fig. 8 summarizes the change with the number of cycles of the cyclic head displacement, the cyclic stiffness and the permanent head displacement measured in test 7a, where the pile collapsed after about 50 uniform load cycles. For a uniform presentation all measurements quantities are normalized with respect to the respective value at the first load cycle.

Focusing upon the initial 20 cycles, it is observed that the stiffness and the displacement of the pile remain practically constant, while the permanent displacement accumulates according to a linear double logarithmic relationship (Eq. 6). The onset of "incremental collapse" is clearly identified at the end of this initial stage, since it marks the beggining of an abrupt deterioration of the cyclic stiffness and a respective increase in the rate of cyclic and permanent displacement increase in the following cycles.

The same mode of cyclic failure was also observed in test 6a which, similar to Test 7a, was not subjected to any cyclic preshearing. In test 8a-IV, which was subjected to about 600 cycles of cyclic preshearing, failure occured due to a smooth build up of excessive pile head displacements, which was not

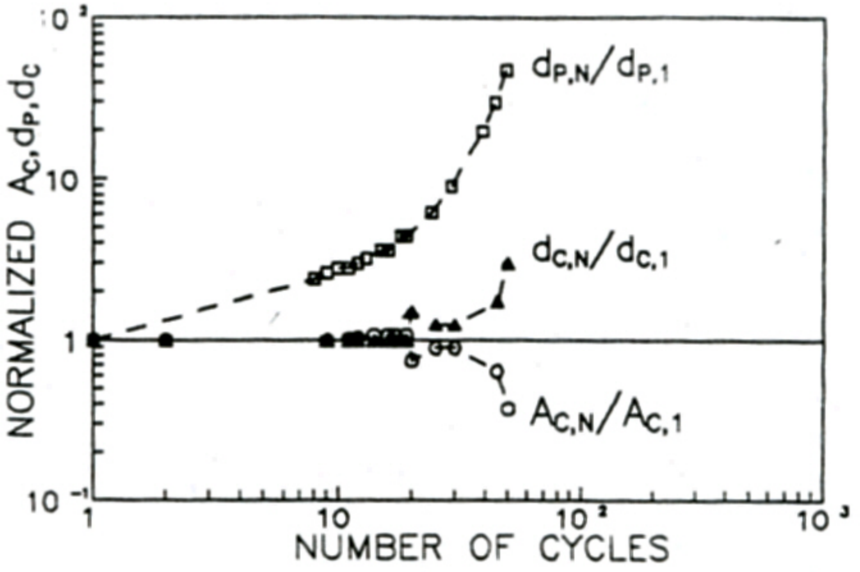

Fig. 8. Variation of permanent displacement, cyclic stiffness and cyclic displacement in Test 7a.

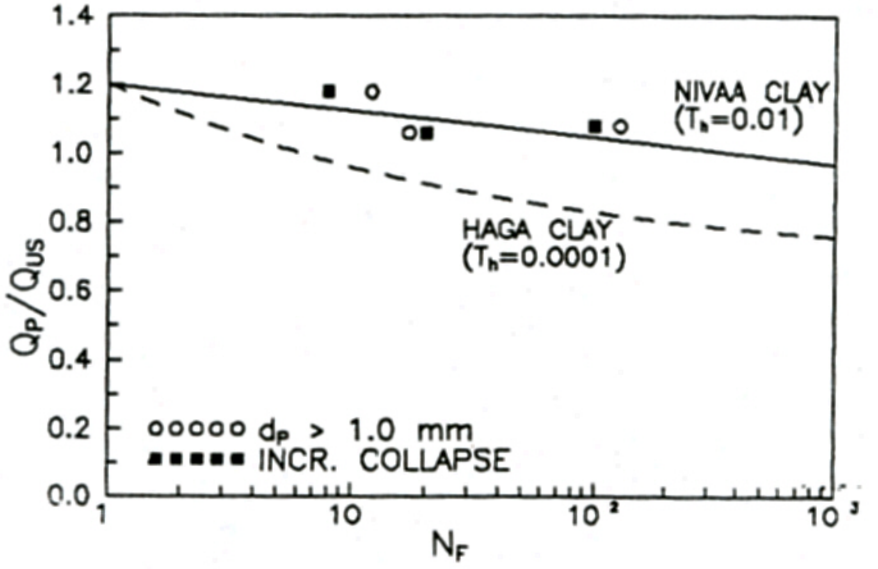

Fig. 9. Variation of peak uplift capacity with number of cycles.

accompanied by any obvious degradation in the cyclic stiffness. This mode of failure is certainly different from that demonstrated in Fig. 8 and it is questionable whether it should be also defined as "incremental collapse".

Fig. 9 relates the peak cyclic load ratio Q_p/Q_{us} to the number of uniform load cycles N_F required to cause pullout failure of the model piles. Due to the different modes identified previously, N_F is defined in two ways; at the onset of incremental collapse and at a maximum pile head displacement equal to one pile radius. To evaluate the effect of drainage conditions on the cyclic uplift capacity, Fig. 9 also shows the average curve obtained by Karlsrud and Haugen (1985) from cyclic pile load tests in tension and in compession, in Haga Clay. It is reminded that these tests were classified in group A of Table 3, i.e. among the pile load tests with the least drained conditions during cyclic loading and the most pronounced degradation of cyclic stiffness.

Similar to what has been observed earlier in connection with the cyclic resistance, the degradation of cyclic capacity with number of cycles exhibited by the tests in Haga Clay is faster than that exhibited by the tests in Nivaa Clay. This observation implies that the effect of drainage during cyclic loading is not only limited to the cyclic stiffness of the pile but it may also extend to several other aspects of the cyclic pile response including the failure mechanisms. In general, it appears that the capacity of piles to sustain cyclic loads and displacements is reduced as the degree of drainage allowed to take place during the period of loading is decreased.

CONCLUSIONS

In conclusion, the previous analysis of test data from monotonic and cyclic tests on anchor piles in Nivaa clay verified previous findings obtained mostly from pile load tests in compression but also provided new insight to the mechanisms controlling the cyclic pile response. In summary, it has been shown that,

at the initial stages of loading:

- The cyclic stiffness is mainly a function of the applied cyclic load amplitude and can be approximately estimated from monotonic pullout tests at comparable loading rates. The effect of number of cycles is relatively small at this stage, and may lead either to slight hardenning or softenning of the cyclic stiffness, depending upon the intensity of cyclic loading and the drainage permitted parallel to cyclic loading.

- The permanent displacement of the pile head is a function of the applied cyclic displacement amplitude, and accumulates at a gradually decreasing rate with number of cycles that may be described by a linear log-log relationship.

At later stages of loading:

- The response of the pile may follow the same trends described for the initial stages of loading (shakedown), or it may experience a rapid deterioration (incremental collapse). To predict the mode of response, one must consider both the intensity of cyclic loading and the amount of drainage that takes place during a load cycle.

Some, at least, of the previous conclusions have been derived from a rather limited number of tests and consequently they require further experimental verification before they can be used in practical design applications. At present, additional theoretical and experimental research is under way at N.T.U.A. for a thorough evaluation of the different aspects of cyclic response of enchor piles and its correlation with the cyclic response of the surrounding soil.

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