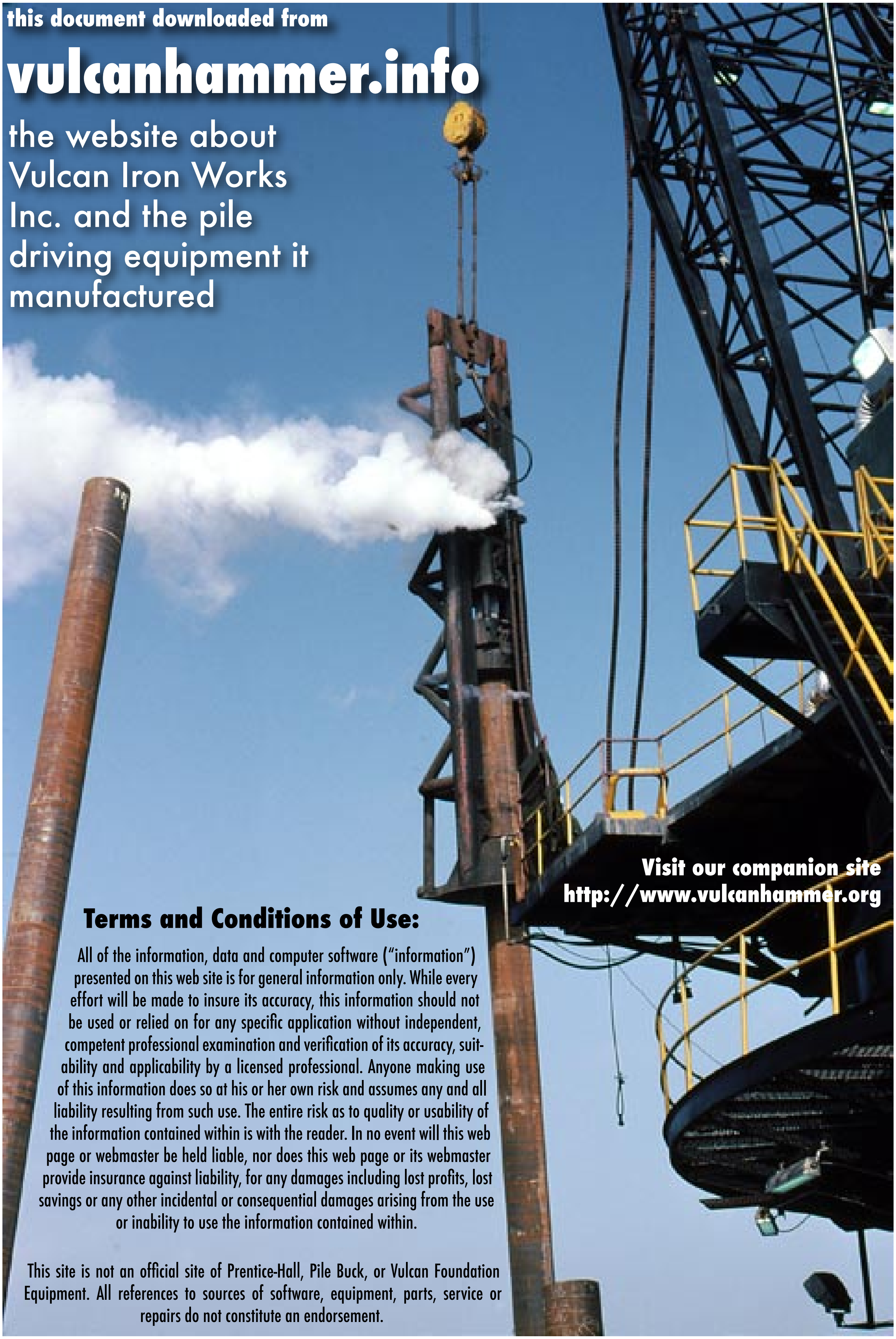

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Relationships between wall friction, displacement velocity and horizontal stress in clay and in sand, for pile driveability analysis

by E. P. HEEREMA*

simple laboratory test arrangement is described which simulates the action of a steel pile wall in the soil during driving, and which is suitable for determination of relationships between wall friction, horizontal stress at the pile/soil interface, and pile wall velocity, in sand and in clay. These tests were run in order to provide more realistic input parameters for pile driveability studies (e.g. wave equation analyses).

The results of the tests demonstrate that during pile driving in sand a simple Coulomb friction behaviour is exhibited: the friction force is linearly dependent on normal stress and independent of velocity. Conversely, in clays, it is found that wall friction is, in an unexpected manner, dependent on the stress condition at the clay/pile interface, on the undrained shear strength of the clay adjacent to the pile, and on the velocity at which the pile wall displaces.

Comment is given on the commonly held view that clay remoulding is the cause of the phenomenon that static friction during driving is lower than bearing friction after set-up.

Introduction

PRESENT KNOWLEDGE about pile-driving behaviour is rather limited. The wave-equation computer program has proven to be a most valuable analysis tool, but the quality of a driveability prediction depends fully on the correctness of the assumed magnitude and behaviour of the soil resistance.

Soil resistance behaviour during driving is as yet hardly accessible to a theoretical approach, soil being a very complex material. Yet more understanding of the fundamental behaviour of soil resistance during driving is necessary to make better pile driveability predictions.

Analysis of pile-driving in current practice bases dynamic (driving) resistance on an ultimate static bearing capacity calculation, or a version thereof in which the resistance is reduced by either taking a percentage or by using remoulded shear strength values. The driving resistance is linked to the static value by means of an empirical correlation factor (Smith's "damping" value[1]). In the author's opinion [illegible] has no direct rela-

tal soil stress, pile wall velocity, and soil parameters.

In an effort towards this end, an apparatus has been developed to simulate in the laboratory the movement of a steel pile wall along the soil during driving. In this test apparatus, a flat steel plate is pressed against a soil sample and moved back and forth by a hydraulic oscillator. Amplitude and frequency can be varied over a wide range; forces and displacements are continuously recorded. By means of these tests, relationships between wall friction, velocity, and horizontal stress can be investigated in sand and in clay. The tests have led to a better understanding of resistance behaviour during pile-driving; using the derived relationships for computer back-analyses of observed driving behaviour, much can be learnt about the magnitude of actual resistance during driving, so aiding in future predictions. This was thought important in view of Heerema Engineering Service's extensive involvement in offshore pile-driving activities in the North Sea, where correct assessment of pile configuration and equipment is essential to economic platform installation.

The relationships. found between wall friction, horizontal soil stress against the pile wall, and pile wall velocity, as well as the general conclusions drawn in this Paper, have been primarily formulated for offshore applications. Fundamentally, however, they are not limited to that field of foundation engineering.

Description of the test apparatus

The test apparatus is shown schematically in Fig. 1*a*. In an attempt to disturb the soil sample as little as possible, it is left in the steel tube in which it was collected. The sampling tube together with the sample is cut in half length-wise. Thus the half-specimen has a face of 50mm; its height is about 150mm. It is placed in a vertical position in a half-cylindrical support frame. Against the face of the soil sample, a flat stiff steel plate is pressed which covers the sample face almost completely. As the soil specimen is also enclosed from above and below relatively high horizontal stresses can be applied to it before failure. Because of the confinement, it does not dry out dur-

the steel plate through a wheel mounted on a load lever. As this wheel has ball bearings, its friction may be neglected. Because of the way it is mounted, the hydraulic system cannot withstand horizontal forces, so that all horizontal load applied by the lever arm is transferred to the soil sample's face. The horizontal load can be varied by altering the load on the lever.

Two series of tests were carried our — horizontal load variation tests, and velocity variation tests. These were carried out in January and November 1976, at TNO Delft laboratories, The Netherlands.

Results of friction tests in sand

The sand samples used in these tests were from Chevron's Ninian Field in the North Sea, the sand being fine and with only a small silt content. The sand was wetted and as a result it remained well collected inside the test apparatus.

(a) Horizontal load variation tests in sand

Horizontal loads were varied from 50 to $240kN/m^2$ while the amplitude and frequency of the plate motion were maintained at 12.5mm and 1.6Hz respectively (sinusoidal motion). Frictions were measured at zero crossings, so that the velocity there was $2\pi af=0.126$ m/s. The result of such a test are shown in Fig. 2. It ca be seen that the friction-horizontal stres relationship is very regular; the frictio angle δ between the pile wall and th sand is 25°. This is no new figure. It customary to take $\delta=\phi-5°$, where ϕ the effective angle of internal friction the sand. (ϕ was not measured for th sample, but should be about 30°).

(b) Velocity variation tests in sand

Velocities were varied between 7×10 and 0.6 m/s, while the horizontal lo was kept constant. Fig. 3 shows results of such a test, the horizontal lo here being $85kN/m^2$.

This figure shows clearly that the f tion force is not velocity-dependent at and therefore the sand may be cons ered to behave as a simple Coulo material. In terms of Smith damping ues[1], this would imply $J=0$. For c parison the commonly used value c for wall friction in sand is 0.164

and low; many were silty or sandy. samples were recovered from four itions: Kontich, Belgium ("Boom"); Chevron's Ninian Field; Occident- more Field; and Unionoil's Heather latter three locations are in the and northern North Sea.

ndrained shear strengths varied from /m² to 620kN/m² and the depths recovery below the sea bed ranged 2 to 70m.

Horizontal load variation tests lay

rizontal loads were varied from 10 to kN/m² (on the harder samples), gen- to the point where the clay failed. and frequency were again and 1.6Hz, respectively. The of such tests are shown in Fig. 4. appears that the friction/normal stress tionship is not linear in clay. For shear ngths (c_u) up to about 150kN/m² the fit of the power of σ_h is 0.6; for c_u and 250kN/m², 0.7; for c_u around 450 m², 0.8; and for the hardest clay (c_u ve 600kN/m²) the best power is 1, its behaviour approaches that of sand other "normal" materials.

or computational simplicity of the tionship which will be derived further this article, the value of 0.7 has selected as the "standard" power the horizontal stress. It is an average e from the tests, and is representative those shear strengths most commonly untered in the North Sea.

izontal load variation tests on hus generally described by the onship:

$$\tau = a_t \cdot \sigma_h^{0.7} \qquad \ldots (1)$$

st-analyses of driving experiences to horizontal stresses which are gen- in or near the range of horizontal ses applied in the laboratory tests.

servations on slip surface aviour during horizontal load ation tests in clay

hough the measured friction force not change by wear of the soil sur- only a few oscillations were made every load step. The soil surface did e somewhat shiny as a consequence many oscillations. When continu- oscillating, a moderate temperature of the steel plate could be felt; the should, of course, occur during al pile driving.

ere was no apparent remoulding of clay in the tests; the pattern of the y clay face did not change. The only plane is the pile-soil interface itself. fore, one could suggest that due to highly superficial occurrence the slip e needs almost no time to consoli- and the horizontal stress might be idered to be an effective stress. This ust ed by the fact that if the appar- to rest for an hour or so with izontal load applied, the friction not increase.

owever, the observed non-linear rela- ship between friction and applied ontal stress indicates that the effect- stress does not increase at the same as the total applied stress. The lower on coefficient at high horizontal stress

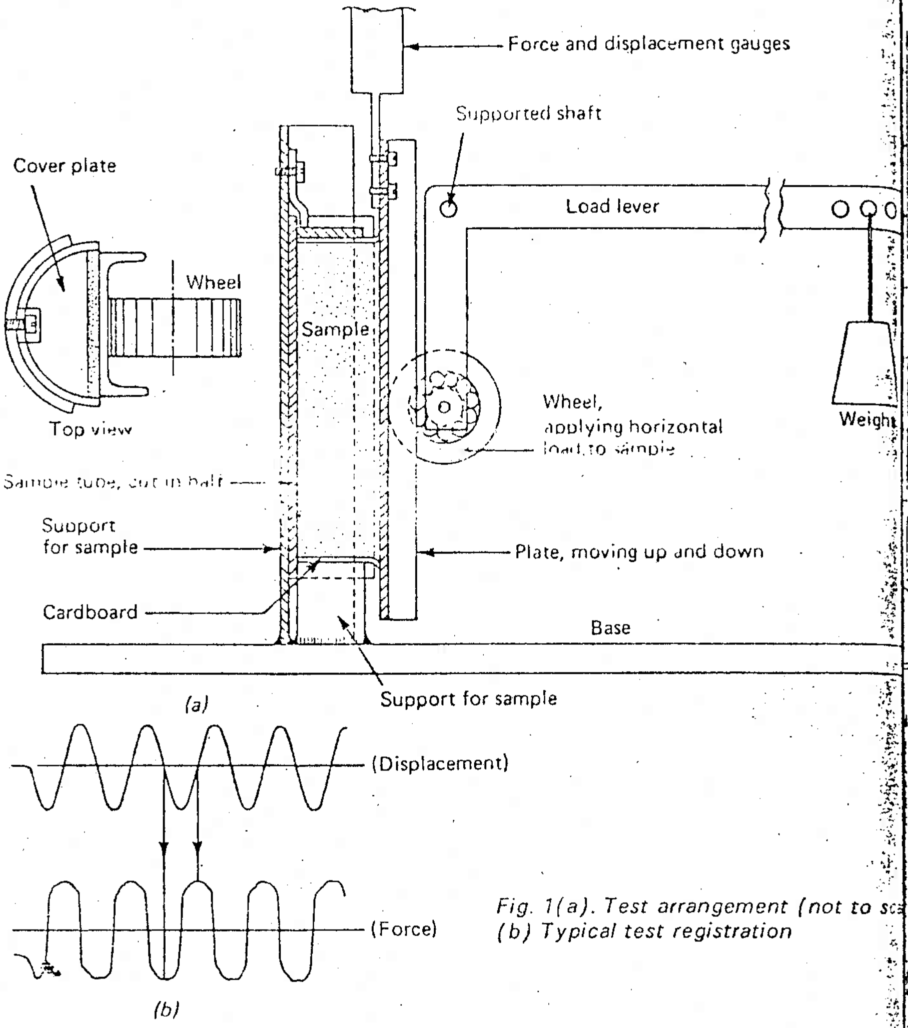


Fig. 1(a). Test arrangement (not to sc *(b) Typical test registration*

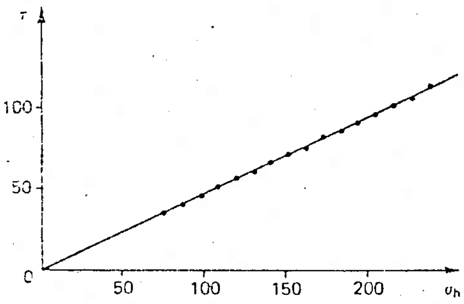


Fig. 2. Horizontal load variation test on a sand sample. τ and σ_h in kN/m²

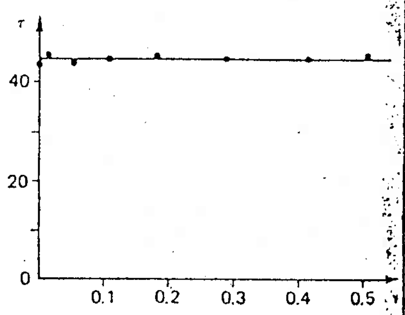
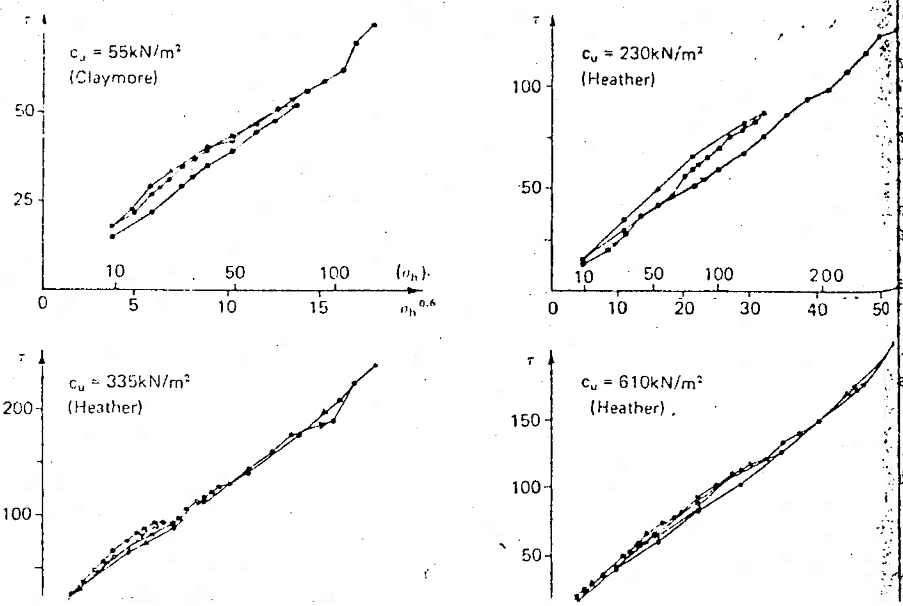
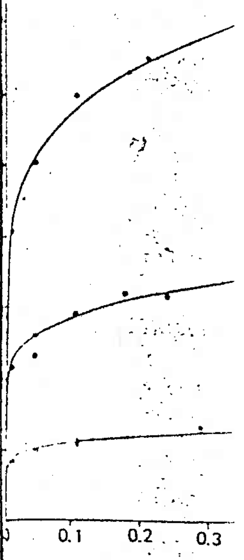
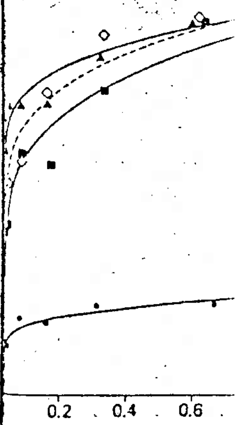


Fig. 3. Velocity variation test on a sanc sample. σ_h = 85kN/m², v in m/s





5(a & b). Velocity var = 31.5, 87 and 184kN/



5(a & b). Velocity vari different applied load.

curve, it might be water's behaviour is i playing a lesser role ses.

ith regard to the diff h the horizontal stres likely that the te gher power for the gher shear strengths ed by the decreasin s.

an attempt to learn viour, the slip plane face) was wetted w cases. This resulted g friction reduction; rently soon driven on returned to the well-confined environ pile wall is located, riven out of the slip neither would it fin plane, so that the we considered to be a e.

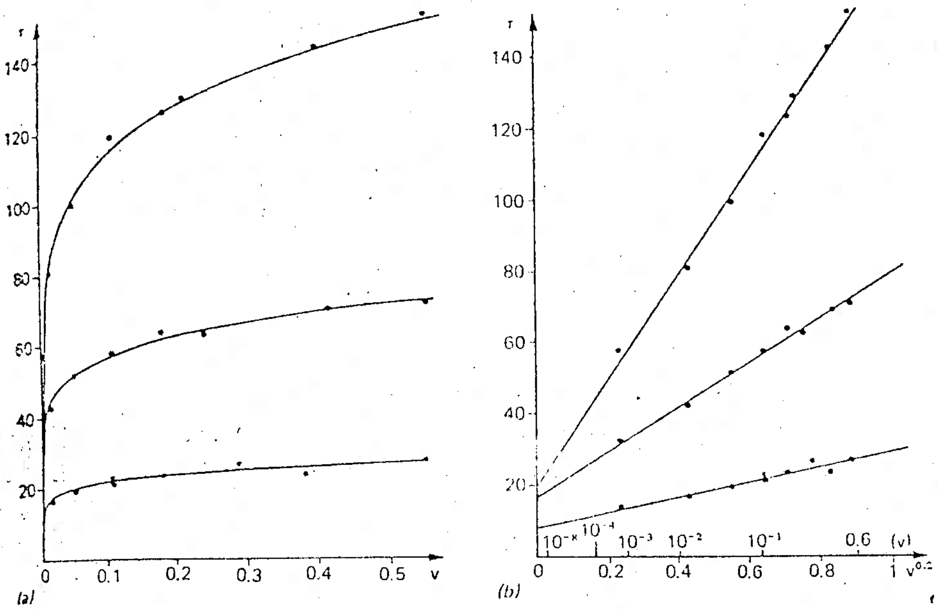


Fig. 5(a & b). Velocity variation tests on a clay sample $c_u = 260kN/m^2$ (Kontich); $\sigma_h = 31.5$, 87 and $184kN/m^2$, respectively. τ in kN/m^2, v in m/s

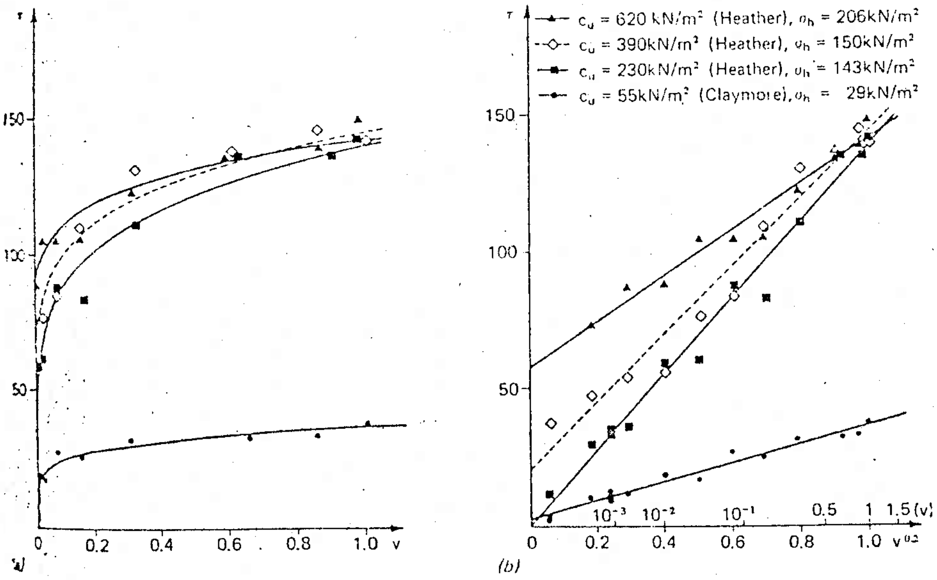


Fig. 6(a & b). Velocity variation tests on clay samples with different shear strengths (and different applied loads). τ in kN/m^2, v in m/s

curve, it might be suggested that the water's behaviour is in a sense "elastic", playing a lesser role at small applied stresses.

With regard to the different powers to which the horizontal stress must be raised, it is likely that the tendency towards a higher power for the horizontal stress at higher shear strengths is mainly determinded by the decreasing moisture contents.

In an attempt to learn more about this behaviour, the slip plane (pile wall/soil interface) was wetted with water in a few cases. This resulted in a very short-lasting friction reduction; the water was apparently soon driven out so that the friction returned to the normal value. In the well-confined environment in which a real pile wall is located, water could not be driven out of the slip plane so easily;

zontal load was kept constant.

At an amplitude of 12.5mm, this implies frequencies between 10^{-5} and 13Hz. For the lowest velocity, this would mean a complete oscillation would take 28 hours; to prevent this, the amplitude was decreased and also most of the oscillation was carried out at a higher frequency, making only the zero crossings at the low frequency. This was necessary to save time and prevent the sample from drying out.

Only a few oscillations per frequency were made. The different frequencies were run in an irregular sequence to compensate for effects of wear, if there were any.

During pile driving, peak velocities of the pile wall can be as high as about 3 m/s at low blow counts. The duration of the peak velocities is however relatively very short; the average velocity of the pile wall during passage of the most significant

velocities, and very weakly velocity-dependent at high velocities. It is also noteworthy that at velocities very near to zero, the friction is very small.

Fig. 5*b* shows that the relationship can well be described by

$$\tau = a_2 + a_3 (v)^{0.2} \qquad \ldots(2)$$

The power 0.2 appears to be the right value over the whole range of tested clays.

Fig. 6 shows the results of a number of velocity variation tests on samples with different shear strengths (and different applied horizontal loads). Apparently a_2 and a_3 in Eqn. (2) are dependent on the shear strength value.

The observed strange velocity-dependence of the friction might be explained as follows. If the pile wall was perfectly smooth (Fig. 7*a*), the friction between pile wall and soil would probably not be velocity-dependent. But every pile surface, as was the steel plate used in the laboratory, is somewhat roughened and not perfectly flat. Under horizontal stress the clay will adjust itself to the minuscule irregularities of the steel surface (Fig. 7*b*). Now when the steel surface displaces (Fig. 7*c*), tiny gaps will be formed locally, causing relative underpressures. As a consequence of these resulting pressure differentials the clay will try to adjust itself to the new shape. If the displacement is slow, there is enough time for the clay's plastic reaction, so that the effective friction can also be low. But as velocity increases, the adjustment becomes progressively more difficult (too slow), and the apparent friction becomes larger. At a certain stage the velocities become so large that there is almost no adjustment of the clay at all anymore — only slippage, so that the magnitude of the velocity becomes relatively unimportant. This would explain the flattening of the friction-velocity curves at higher velocities.

The harder the clay, the less it adjusts itself to the roughnesses and consequently its velocity-dependence is less explicit.

Sand, finally, has relatively large particles compared to the minuscule irregularities of the pile's surface, can provide easy access and flow of water, and particularly lacks plastic flow behaviour in the way the clay is suggested to have. These factors could explain why sand friction is not velocity-dependent.

In the tests it was noted that samples with sand and clay intermixed in pockets had an intermediate behaviour.

As regards the pile wall surface, test piles have shown that the steel surface is polished by the soil to some extent, removing the roughness caused by rust.

Derivation of the general skin friction relationship for clay

The aim of the velocity variation tests is to find an expression for velocity-dependence which is suitable for use in the wave-equation computer program. It is logical to start with the common relationship for dynamic and static friction (τ_d and τ_u) introduced by Smith:

In Eqn. (1), a_1 is a constant assumed to be dependent on the displacement velocity and on the cohesive shear strength.

If Eqn. (4) is re-written in the form of Eqn. (1):

$$\tau_d = \sigma_h^{0.7}\left\{\frac{\tau_o}{\sigma_h^{0.7}} + \frac{\tau_o J}{\sigma_h^{0.7}} v^n\right\} \quad \ldots (5)$$

then the factor $\sigma_h^{0.7}$ determines the relationship with horizontal stress, and the terms $\frac{\tau_o}{\sigma_h^{0.7}}$ and $\frac{\tau_o J}{\sigma_h^{0.7}}$ are only determined by the shear strength c_u. To find their relationships with c_u, the values $\frac{\tau_o}{\sigma_h^{0.7}}$ and $\frac{\tau_o J}{\sigma_h^{0.7}}$ must be determined from the individual tests and drawn against c_u. Each velocity test provides a number of friction values through which a line can be drawn described by Eqn. (2). Comparison of Eqns. (2) and (5) shows that the value $\frac{\tau_o}{\sigma_h^{0.7}}$ from each test is equal to $\frac{a_2}{\sigma_h^{0.7}}$ and the value $\frac{\tau_o J}{\sigma_h^{0.7}}$ is equal to $\frac{a_3}{\sigma_h^{0.7}}$.

Figs. 9 and 10 show the relationships of $\frac{\tau_o}{\sigma_h^{0.7}}$ and $\frac{\tau_o J}{\sigma_h^{0.7}}$ against c_u as determined from the velocity tests:

$$\frac{\tau_o}{\sigma_h^{0.7}} = 0.0029\ c_u - 0.32;$$

$$\frac{\tau_o J}{\sigma_h^{0.7}} = -0.0041\ c_u + 4.44 \quad \ldots (6)$$

The horizontal load variation tests have so far only been used to determine the power 0.7 of σ_h.

From Eqns. (1) and (5) follows

$$\frac{\tau_o}{\sigma_h^{0.7}} + \frac{\tau_o J}{\sigma_h^{0.7}} v_1^{0.2} = a_1$$

Assuming the term $\frac{\tau_o}{\sigma_h^{0.7}}$ to be known from the relationship of Fig. 9, a value of $\frac{\tau_o J}{\sigma_h^{0.7}}$ can be determined from the horizontal load variation tests; vice versa, assuming $\frac{\tau_o J}{\sigma_h^{0.7}}$ to be known from the relationship of Fig. 10, a value of $\frac{\tau_o}{\sigma_h^{0.7}}$ can be determined. The horizontal load variation tests thus serve mainly as confirmation of the already determined relationships of $\frac{\tau_o}{\sigma_h^{0.7}}$ and $\frac{\tau_o J}{\sigma_h^{0.7}}$ with c_u.

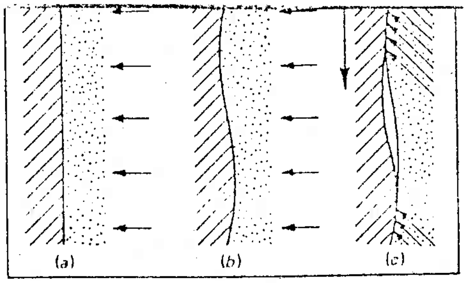


Fig. 7. Pile wall – soil interaction (simplified suggestion)

Figs. 9 and 10 show a large scatter in values. This is however not uncommon in soil tests; an important factor in the scatter is, for example, the shear strength value used. Soil reports illustrate the wide scatter usually found in shear strength measurements.

From Eqns. (5) and (6) the final relationship between friction, horizontal stress, displacement velocity, and cohesive shear strength follows:

$$\tau_{dyn} = \sigma_h^{0.7}[\,(-0.0041\ c_u + 4.44)v^{0.2} + (0.0029\ c_u - 0.32)\,] \quad \ldots (7)$$

Min. range of validity (tested range):

$55 < c_u < 620$ kN/m²,
$8 \times 10^{-7} < v < 1$ m/s,
$10 < \sigma_h < 490$ kN/m².

Note: for conversion of these factors to another measuring system, multiply 0.0041 and 0.0029 by $(f)^{-0.7}\ (g)^{-0.2}$, and 4.44 and 0.32 by $(f)^{+0.3}\ (g)^{-0.2}$ in which:

f is the conversion factor: 1 kN/m² = f of the other system;
g is the conversion factor: 1 m/s = g of the other system.

Eqn. (7) can replace Smith's "damping" factor in the wave equation computer program. Input for each pile part should no longer be τ_o and J, but σ_h and c_u.

In sand layers, a fictitious c_u of 1 083 kN/m² must be put in to eliminate all velocity-dependence of the friction. Eqn. (7) then becomes $\tau = 2.82\ \sigma_h^{0.7}$, so for $\sigma_h^{0.7}$ in the computer input the value 0.355 τ must be used.

Figs. 11, 12 and 13 illustrate the wall friction behaviour of clay according to Eqn. (7) as functions of horizontal stress,

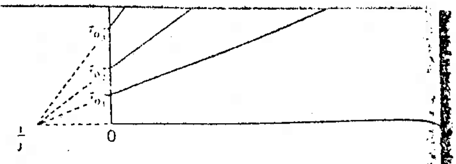


Fig. 8. Graphical representation of $\tau_d = \tau_o\ (1 + Jv^n)$

of velocity, and of shear strength resp tively. The tested ranges (drawn w thicker lines) should be kept in mi when, for example, interpreting the v condition in Fig. 13. The negative fric for soft clay is of course an unreali consequence of a simplified formulat But at an extremely low velocity (2 10^{-6} m/s or 7 mm/hr) the friction comes positive for $c_u = 0$, and at 1.7m hr the friction becomes positive for c 25 kN/m² (very soft to soft clay).

It should also be kept in mind t these tests were primarily intended studying friction behaviour during p driving, so that the fully static situa is not important in this context. Nev theless some conclusions can be dra about soil conditions under static bear situations, and these will be conside later in this article.

Comparison of the obtained frictio velocity relationship with Smith's "damping" relationship

Fig. 14 shows these two relationsh in one graph. The "damping" relations (straight line) is drawn for τ_o=0 (according to API[3]) and J=0.656 s (0.2 s/ft, the commonly applied value clay). Shear strength is 250kN/m². new relationship (curved line) is dra for σ_h=375kN/m² to approximately the "damping" relationship.

Is a comparison between the newly rived friction-velocity relationship Smith's "damping" relationship relev

Smith's "damping" values are a re of correlations between observed dri behaviour and measured static bea

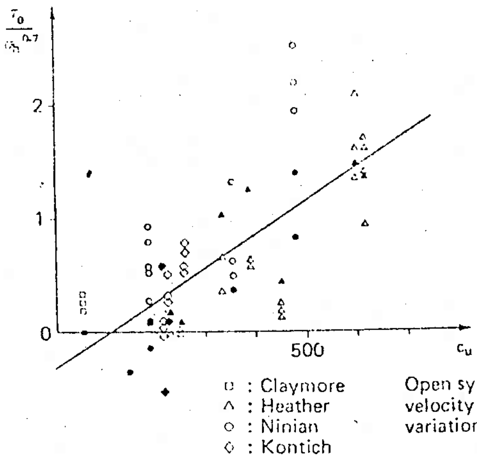


Fig. 9. "Static" component of general friction formula

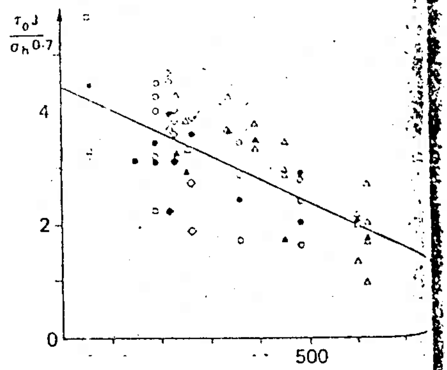


Fig. 10. "Dynamic" component of gene friction formula

capacity. Smith's derivation procedure may be expected to have been the following: drive a test pile; perform compressive and tensile load tests on it; introduce the measured resistance values in the wave equation computation; by trial and error, find a value for J that leads to the actually measured blow count.

Therefore, the values for J that have been derived in this way and are commonly used today are a means to jump from static friction in bearing to dynamic friction during driving. They do not represent actual conditions during driving; a false (too high) static value during driving is used.

The useful application, therefore, of the "Smith" damping values is to make a correlation between observed blow count and the static bearing capacity that may be expected later. Recognising the fact that the costs of offshore static load tests are usually prohibitive, the method should be given fair consideration.

Care must be taken, however, when relating driving behaviour to static bearing capacity using Smith's damping factors. The validity is limited for several reasons. Difference in pile plugging behaviour during driving and in the static bearing condition is, for example, one reason. (Detailed discussion of these aspects is beyond the scope of this article; see [4]). Another argument against relating driving behaviour to bearing capacity is that it assumes the friction set-up factor after driving to be equal to that of the test piles from which the "damping" values were derived, regardless of shear strength, penetration, or horizontal stress conditions.

The relationships illustrated in this article do not provide an answer to the question as to what the bearing friction will be after set-up. On the contrary, they illustrate that friction during driving and friction after set-up are wholly different. Assume that in Fig. 14 the straight line represents an application of Smith's "damping" relationship such that it correlates well with both the static bearing capacity after set-up and the blow count during driving of a given case; the newly derived relationship represented by the curved line in the figure correlates reasonably well with the "damping" relationship at an "average" velocity (1.75 m/s; the dynamic frictions are the same). Then the static friction during driving according to the new relationship appears to be only roughly one-fifth of the static value that will be valid after set-up. This value of one-fifth is valid only for this shear strength, and is dependent on the validity of Eqn. (7) for $v=0$.

Of course, it is also assumed that the laboratory test arrangement is sufficiently representative for actual pile-driving circumstances. But as an indication this ratio of one-fifth is significant. This case illustrates that static friction during driving in clay is considerably smaller than after set-up. As the horizontal load variation tests have pointed out that friction is determined by horizontal stress, we must conclude that apparently horizontal

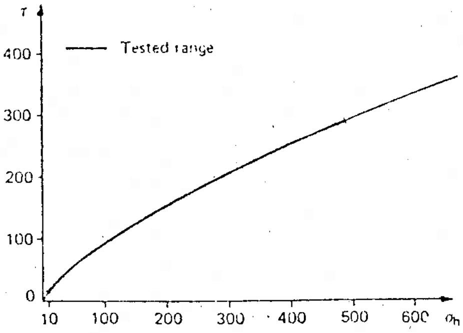


Fig. 11. Derived relationship between wall friction and horizontal stress, for $c_u = 300kN/m^2$ and $v = 1m/s$. τ and σ_h in kN/m^2

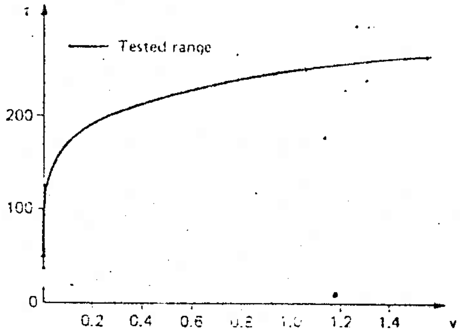


Fig. 12. Derived relationship between wall friction and velocity, for $c_u = 300kN/m^2$ and $\sigma_h = 400kN/m^2$. τ in kN/m^2, v in m/s

(the regain of friction after pile-driving has stopped). The phenomenon that the static value during driving is lower is commonly attributed to clay remoulding. It is reasoned that in this manner the clay loses strength and, therefore, exerts less friction on the pile wall.

The results obtained from the laboratory tests point out that clay remoulding cannot be the main cause of friction decrease during driving:

(*1*) If the clay is remoulded, leading to a lower shear strength, the friction which it exerts at the high average velocities occurring during the pile wall displacement (1-2 m/s) does not decrease as long as the horizontal stress remains the same (see Fig. 13).

(*2*) The considerable difference between static friction during driving and bearing friction after set-up which was deduced in the previous section (1:5 for that particular example) is too large to be attributed to remoulding of the clay only: shear strengths of North Sea clays do not reduce to that extent when remoulded.

(*3*) Of course, the clay is remoulded to some extent when a pile tip is forced into it, but after that the clay is probably not further remoulded significantly, the slip plane being the pile/clay interface itself as in the laboratory tests.

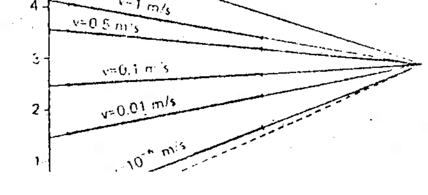
It could be suggested that water in the shear plane might bear a large proportion of the total horizontal ground stress, so that the friction is reduced. However, if the water is extracted from the surrounding clay there is no reason why this does not also occur in the laboratory tests. And it cannot be assumed that sea water finds access to large depths along the pile shaft against the soil pressure.

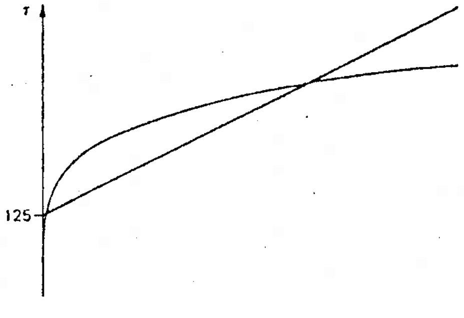
A final argument in favour of the hypothesis that friction decrease during driving is caused by horizontal stress decrease can be found by examining the behaviour of piles driven in sand. Open-ended steel piles driven wholly in sand, from which the soil plugs are removed, show low blow counts after restarting, even beyond the depth where point resistance may be small due to sand disturbance by the plug removal operation. Back-analyses of such driving cases show that during driving the pile's outside friction is considerably smaller than the bearing friction which may be expected later. In sand, a lower outside friction during driving definitely proves the occurrence of a lower horizontal stress. If a lower horizontal stress occurs in sand during driving, there is no reason why this would not also be the case in clay.

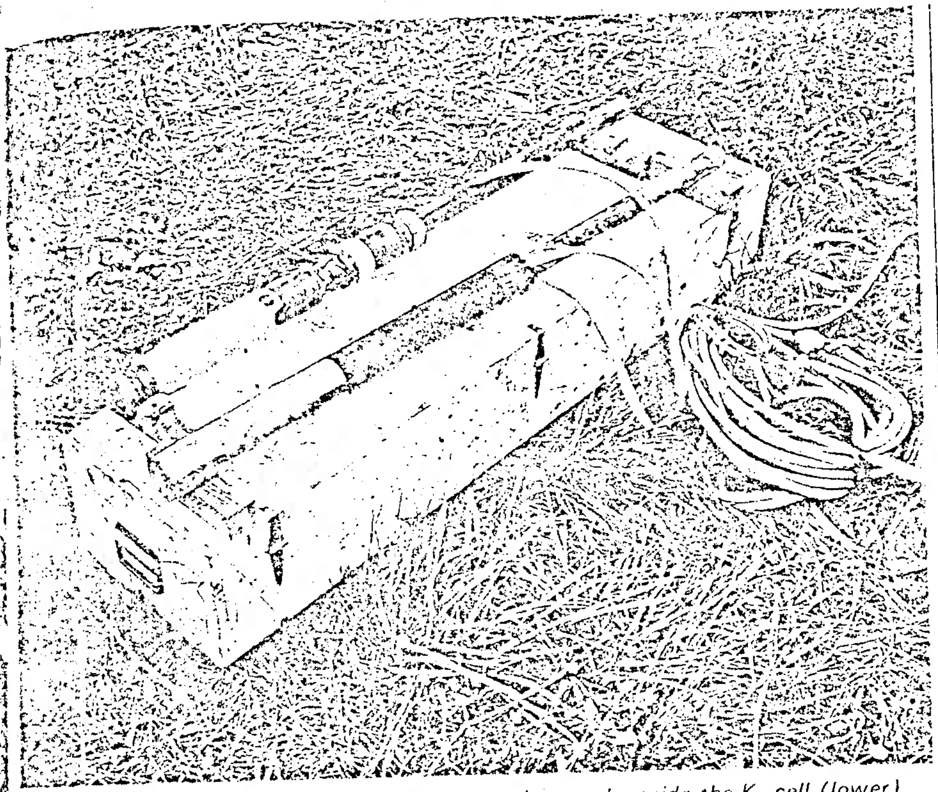
The reduction of horizontal stresses against the pile wall during driving may be explained mainly by irregularities of the pile wall pushing the soil outwards and taking some soil downwards, and by elastic expansion and transverse vibration of the pile, pushing the soil outwards. A horizontal soil arch may thus be formed around the pile, temporarily, as any static load test points out.

As the fact that static friction during driving is lower than after set-up, is not caused by remoulding, it is fundamentally incorrect to use the remoulded shear strength of the clay for driveability predictions. This does not mean to say that no acceptable predictions can be made using the remoulded shear strength; empirical methods based on correlations with remoulded shear strength may prove to work satisfactorily. But the incorrect starting point should provide a warning

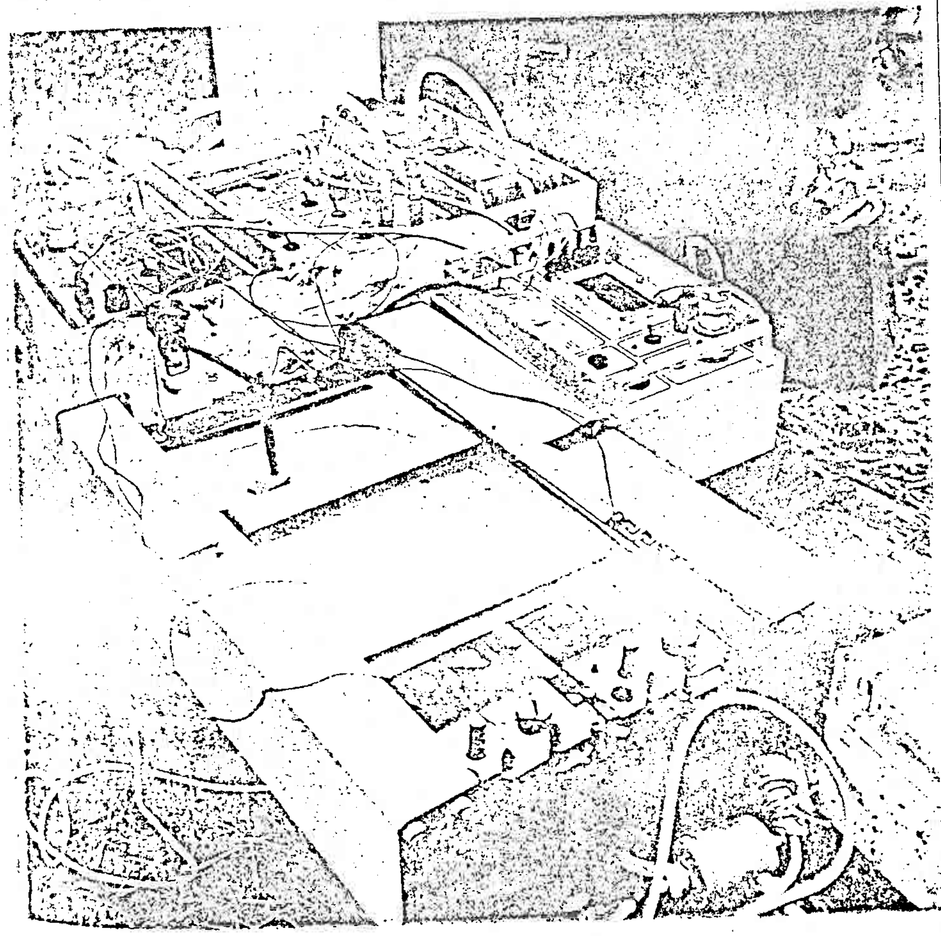
(concluded on page 65)







Self-boring pressuremeter shown without its membrane, alongside the K_u cell (lower)



Stress-strain curve recorded on the X-Y plotter during a test

Relationships between wall friction, displacement velocity and horizontal stress in clay and in sand, for pile driveability analysis

(continued from page 51)

that where a good correlation is found this should be considered to be rather a coincidence, as the procedure carries the risk of not being generally applicable.

Conclusions

The laboratory tests described in this article lead to remarkable relationships between wall friction, velocity, horizontal stress, and shear strength for piles during driving.

Main results are:

(*i*) For sand with a fairly low silt content there is no velocity-dependence of the friction at all ($J=0$).

(*ii*) For clay the friction is very strongly velocity-dependent at low velocities and very little velocity-dependent at high velocities.

(*iii*) For clay the friction is strongly related to horizontal stress. The friction coefficient decreases as the total horizontal ground stress increases.

(*iv*) From the laboratory tests the general relationship Eqn. (7) can be derived between friction, velocity, horizontal stress, and shear strength for piles during driving in clay.

(*v*) The commonly used "damping" values in driveability analyses appear not to be representative for the conditions during pile driving. It is made clear that their proper function is to correlate dynamic friction during driving somewhat crudely with the static bearing friction expected after set-up.

(*vi*) The reason why static friction during driving is lower than after set-up does not lie in remoulding of the clay. The phenomenon is wholly determined by temporary relief of the horizontal stress that the soil exerts on the pile wall — probably an "arching" effect.

The results of these relatively unsophisticated tests have given rise to significant improvements in the understanding of skin friction behaviour during pile driving, providing a basis for improved predictions of pile driveability.

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2. *De Beer, E., Lousberg, E., Wallays, M., Carpentier, R., De Jaeger, J., & Paquay, J.* (1977):

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been established to provide an analytical and computing consultancy service for

iety of soil models that have been tested in practice. Additionally, PMGA provide a service for the analysis of piles and pile