CONCLUSIONS

While the primary objective of this paper is to report interesting factual data obtained under an unusual set of conditions, some tentative conclusions may be justified to summarize the findings:

1) It would appear that the soil structure in the top clay deposit was quite completely disturbed or remolded at a distance of 3 inches from the pile but was not measureably affected at a distance of several feet from the pile.
2) In the bottom clay deposit no significant loss in shearing resistance was apparent in any of the four borings regardless of the distance from the pile. In fact available evidence would indicate that shearing resistance may have been increased for reasons not apparent in the present study.
3) The structure of the clay in the intruded core in one open end pile investigated, indicated complete remolding of the soil resulting in a decreased shearing resistance after pile driving to the remolded value found in the laboratory.
4) Remolding of the soil close to the pile was apparently greater in the case of an open end pile than was found for a closed end indicating that disturbance may be related to the type of displacement which may occur.
5) The apparent disturbance of the soil structure varied widely at different depths. This variation was particularly apparent in comparison between the top and bottom clay deposits as identified by physical characteristics measured in supplementary tests. There are also some indications that this difference in behavior may be related to the depth involved and a possible change in the type of displacement at variable depths. These indications are suggested as subjects for future study.

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TESTS ON SMALL SIZED MODEL PILES

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M. FARHI

SUMMARY OF THE FRENCH REPORT

Purpose.

To research for an imposed and increasing total load, the repartition between point resistance and lateral friction resistance.

Device.

The pile is composed by brass tube elements of 42 mm. diameter (external) and 97 mm. long each. These elements are welded without fins on couplings. Each element bears 2 diametrically opposite SR 4 gages. The whole SR 4 being alined on two generatrix of the pile.

An SR 4 cell ends the point. All electric wires come out amidst a loading head through a petrosene "cork".

The external pile surface is coated by a "resinous paint" on which fine sand has been sprinkled.

Principle.

The SR 4 indicate at their depth the remaining load applied on the pile. The difference with the imposed total load is the lateral friction resistance. During the tests consecutive cycles of loading and unloading occured with the same total load and with the former increasing.

Material.

We used dry Fontainebleau sand with vibratuin set-up.

Tests are intended on wet sand, with interposition of very compressible materials so that we may study negative friction.

Results.

1) For every imposed total load, with the consecutive cycles it occurs settlement increment which are decreasing.

After few cycles, there is still an irreversible settlement (hysteresis), but the settlement increment is negligible.

For instance a pile of 0,80 m length, bearing 200 kg. total load, at the 10th cycle had a settlement of 172 μ and 135 μ rebounding during the unloading.

2) The lateral friction resistance and point resistance (R_e and R_p) appeared both at the same time.
3) The Lateral friction distribution is not parabolic as the theoretic distribution, when ultimate bearing capacity is reached.
4) The F_ℓ/F_p ratio decreases when the total load

$F_\ell + F_p$ increases.

5) For two piles of different length and same total imposed load, the lateral friction curves do not superpose.

For the shortest pile, the F_ℓ / F_p ratio is smaller and the point resistance F_p higher than

a) the point resistance of the longest pile
b) the resistance remaining on the longest pile at the depth of the point of the shortest pile.

Horizontal component of earth pressure.-

Let ω = material density
χ = pile perimeter
h = depth
H = pile length
φ = internal friction angle

we have
$$F_\ell = \int_0^H \omega \chi h A \cdot dh$$

where A is a pure dimensionless number.

M. LEHUEROU-KERISEL has shown in the case of a pinched driven pile that A is constant, when ultimate bearing capacity is reached, along the whole pile, and is only a function of φ

Its value being
$$\mathrm{tg}\left(\frac{\pi}{4} + \frac{\varphi}{2}\right) e^{(1,77 + \varphi)\,\mathrm{tg}\,\varphi} \sin\varphi$$

The above relation, which defines A, derived, we obtain
$$A = \frac{d F_\ell}{d h} \cdot \frac{1}{\omega \chi . h}$$

The experimental results show that before ultimate bearing value for a certain length pile:

1st) A reaches a maximum near the mid length whatever be this one.

2nd) The A curve, as a function of h, is deformed when the total load increases.

3d) At a certain depth A increases with the total load

4th) The greatest values found for A are close to those given by M. LEHUEROU-KERISEL.

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ABOUT AN OBSERVED CASE OF NEGATIEVE FRICTION ON PILES

J. FLORENTIN
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SUMMARY OF THE FRENCH REPORT

The piles of 0,80 m diameter, upon which a shed is founded, were driven through a fine sand fill, 4 m. thick, and a layer of very soft clay, about 14 m. thick, in process of consolidation. The clay lies above a compact marl (probably cenomanian) slightly decompressed on about 1 m. (fig. A)

Field observations.

1) The shortest piles, probably driven only to decompressed marl, settled considerably, up to 30 cm. in 3 years. (fig. B)

2) The previously levelled soil, settled under the fill and its own weight. After 3 years, the settlement in the centre was about 10 cm. less than around piles, resulting in an overload. (fig. C).

Laboratory tests.

Moisture contents are plotted on fig. A and consolidation curves on Fig. D.

Atterberg's limits averaged to :

Soft clay	L.L. = 50 %	I.P. = 30 %
Marl	L.L. = 52 %	I.P. = 28 %

Therefore, soft clays behave as viscous liquids.

Compressibility measured by the oedometer varies progressively with the depth.

It seams that we deal with the consolidation of the clay under its own weight.

The internal friction, measured by HVORSLEV's apparatus was 23°.

For marl, at 18 m. $\varphi = 19°$ C = 1,5 Kg/cm^2 (on decompressed samples, cohesion drops to 0,500 Kg/cm2) at 19 m. $\varphi = 20°$ C = 3 Kg/cm^2

For sand $\varphi = 34°$.

Computing overloading.

Buoyancy accounted, the weight of the piles 18 m. long and 0,80 m. diameter, is about 11 T.; they were loaded to 30 T. During static tests, to 50 T., nothing happened.

Sinking can only be explained by "negative friction" on piles.

Owing to clay's settlement, we had supposed in sand, an "active pressure equilibrium". For soft clay, we had supposed either an active or an hydrostatic pressure equilibrium.

On fig. E, we have plotted the hydrostatic pressures of soil, the densities being calculated with buoyancy. Each area (S) expressed in metric tones, corresponding for each layer, to 1 m. of perimeter.

For each layer, the overloading by negative friction is

$$\pi \cdot d \cdot S\, A\, \mathrm{tg}\,\varphi ,$$

with d = diameter
φ = angle of friction soil-pile, quite equal to friction soil-soil
A = coefficient of active earth pressure in the Boussinesq-Resal equilibrium, with A = 1 for hydrostatic equilibrium.

The negative friction thus calculated would be of 117 T. in case of hydrostatic equilibrium, and 43 T. in case of an active equilibrium of pressure, in the soft clay, with $\varphi = 23°$. The truth lies between the two, because it is difficult to think that in the soft clay, stresses can be both hydrostatic and have a tangential component against the pile.