Test-Loading on 8 Large Bored Piles in Sand

Essais de Chargement des 8 Pieux Moulés avec Grand Diamètre en Sable

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SYNOPSIS. In this paper the results of 8 axial test-loadings on large bored piles in sand are presented. The skin friction was eliminated from the pile heads downwards to 2.5 m above the pile bases. Beneath this level the sand had medium strength (dutch cone penetration resistance about 15 MN/m^2). The elimination of skin friction was done by filling up bentonite suspension in an annular space round the pile shafts. The results of the measurements show that the load-settlement behaviour is somewhat deteriorated by cutting enlarged bases. For piles with 1.1 m to 1.5 m in (constant) diameter and those with enlarged bases with 1.5 m to 2.1 m in diameter no remarkable differences were observed in the load-settlement behaviour resp. no dependence of diameter. The comparison between piles of 6 m length and piles of 13 m length showed almost no difference in load-settlement behaviour.

INTRODUCTION

The German Pile Committee has worked out a code of practice for large bored piles. For the determination of skin friction and point resistance only test loadings were accepted, as theoretical methods are not reliable until now. It will be reported on a selection of these tests herein. (For further test results see: Jelinek et al. in this volume.)

With test loadings of large bored piles in sand failure occurs at settlements of 20% to 30% of the pile diameter (see e.g. Vesic (1967)) which normally can not be reached with economical means. Therefor such failure loads are almost unknown and consequently it is unknown which safety factors should be applied on such a failure load, to obtain allowable settlements under working loads. Therefor it was decided to determine skin friction and point resistance as a function of the settlement.

At settlements of 2 to 3 cm, which are normally allowed in Germany under working loads for buildings not very sensitive to differential settlements, the skin friction is almost always fully mobilized and the safety against failure is guaranteed by the point resistance only. To avoid settlements greater than 2 to 3 cm, point pressures at a fictitious failure at a settlement of 15 cm were defined (see Table 3), because application of the usual safety factor of 2 yielded the mentioned values of allowable settlement and because 15 cm was the utmost settlement which could be reached in the tests.

PLANNING OF THE TESTS

Principles

Because of limited funds the number of tested piles and the number of the investigated parameters were restricted.

- Therefor the tests were only performed in sands of medium strength with a Dutch cone penetration resistance of about 15 MN/m^2. (In case of a weaker subsoil loading test must be performed, they can be performed in stronger subsoil to make use of better bearing behaviour.)
- The pile bases were embedded well below the ground water level to eliminate the effect of capillary cohesion.
- In the tests a pile embedment of l_o = 2.5m was simulated, which is a minimum normally applied in Germany. Above l_o the skin friction was eliminated by filling bentonite suspension into an annular space round the pile shafts (see Fig. 2)

The latter arrangement was applied because the separation of point pressure and skin friction by strain measurements in the pile shafts proved to be rather inaccurate, on the other hand load cells at the pile base covering the whole cross section as those used by Whitaker and Cooke (1966) or Jelinek et al. (1977) are expensive, but above all no type of load cell was available operating below the ground water table with guaranteed confidence. As the separation of the skin friction at the length l_o and of the point pressure was impossible and as the point pressure dominated the skin friction in most cases, the latter was included in the point pressure and denoted $q_{2.5}$ to indicate this.

According to the concept described above the influence of three parameters on the load-settlement-behaviour were studied:

Pile diameter
Pile length resp. lateral overburden
Enlarging the pile bases.

Therefor the piles of Table I had been constructed.

Pile No.	Lenght l	Diameter Shaft D_S	Diameter Base D_F	Summary value of cone resistance	Soil beneath pile base
	m	m	m		
1	13	1.1	1.1	13.5	Sand $d_{60}/d_{10} \approx 3$
2	13	1.1	1.58	14.5	
3	14	1.5	1.5	13.5	
4	13	1.5	2.04	12.0	
5	6	1.1	1.1	17.0	Silty sand $d_{60}/d_{10} \approx 6$
6	6	1.1	1.53	16.5	
7	6	1.5	1.5	16.0	
8	6	1.5	2.1	17.0	

TABLE I

SUBSOIL EXPLORATION

The testing area was investigated by boring and sounding. Different from a conventional view sounding results are seen as a measure of the strength of the soil, because particularly static cone penetration tests do not only represent the density of the soil, but also the grain size distribution, grain roughness and structural resistance (see Franke 1973).

For a zone from the pile base to 2.5 m below a summary value of the cone resistance was estimated taking into account that this summary value must be reduced the more, the closer lower values of cone resistance are to the pile base (see Table I)

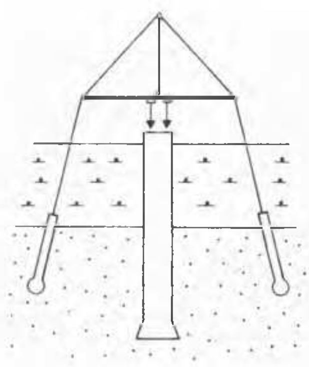

Fig. 1 Test Loading Device of Frankipfahl Baugesellschaft (schematically)

TEST LOADING DEVICE

As a counterfort for four hydraulic jacks of 5 MN each a steel framework was used, which was held by 4 or 8 anchor piles (see Fig. 1)

Between the counterfort and the jacks 8.5 cm thick Neoprene bearings were placed, to keep unintentional horizontal forces small and to be able to get an idea of their magnitude by measuring the deformations of the bearings which proved to be small.

An electrical load cell on one of the jacks was installed to control the accuracy of the manometer readings; in this case it could be seen that the mean error was about 10%.

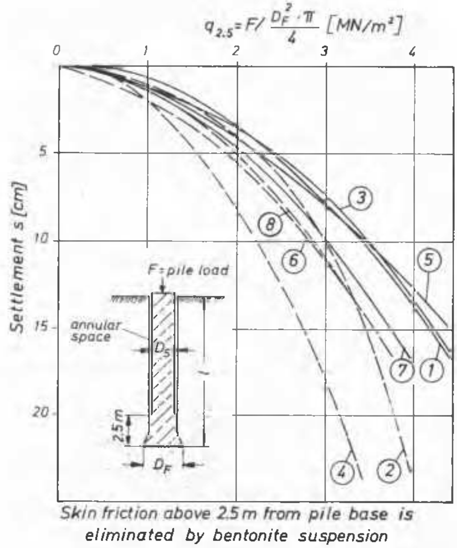

Fig. 2 Point pressure ($q_{2.5}$) versus settlement

TEST RESULTS

Point Pressure-Settlement-Curves

In Fig. 2 the test results are presented in graphs of the point resistance $q_{2.5}$ versus settlement without un- and reloading branches. (The test results are documented in detail by Franke and Garbrecht 1976). The results of pile 4 will be interpreted with care, as the construction procedure to enlarge the pile base was particularly difficult so that the bearing behaviour was more deteriorated than is unavoidable. As already mentioned the failure load was never reached. It also proved impossible to estimate the failure load by the usual extrapolation methods e.g.

according to that of van der Veen (1953) or of Fig. 6 in Franke (1976), because the load-settlement curves are actually more parabolic.

Influence of Enlarging the Base

In Table II the point pressures $q_{2.5}$ are shown for different mean settlements $\bar{s}$, comparing piles with enlarged bases and those

$\bar{s}$	Piles with constant diameters		Piles with enlarged bases		$\Delta q_{2.5}$
[cm]	$q_{2.5}$ [MN/m²]	Δs [cm]	$q_{2.5}$ [MN/m²]	Δs [cm]	[MN/m²]
1	1.0	± 0.4	0.8	± 0.5	0.2
2	1.4	± 0.5	1.2	± 0.6	0.2
3	1.7	± 0.6	1.5	± 0.7	0.2
4	2.0	± 0.65	1.8	± 0.7	0.2
5	2.9	± 1.7	2.6	(± 1.0)	0.3

TABLE II

with constant diameters. Furthermore the differences in the point pressures $\Delta q_{2.5}$ for the two pile types are shown. (The deviation Δs from the mean settlements $\bar{s}$ indicate a satisfactory accuracy of these test results.) From Table II the conclusion can be drawn that enlarging a base deteriorates the bearing behaviour compared to the piles of constant diameters. Qualitatively this can also be seen directly from Fig. 3.

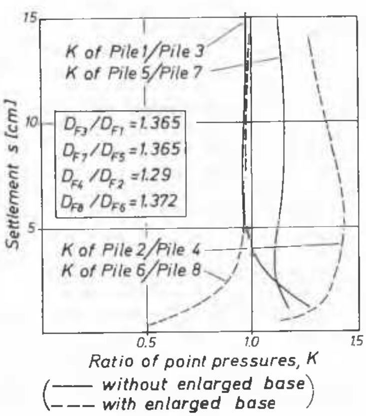

Fig. 3 Ratio K of point pressures ($q_{2.5}$) versus settlements for piles similar in length and a ratio of pile base diameters from 1.3 to 1.4

Influence of Diameter

In sand of medium density no effect of the pile diameter on the bearing behaviour can be expected at settlements exceeding 1 cm, only in sand of high density the bearing capacity decreases with pile diameter (see e.g. Kerisel et al. 1965). In Fig. 3 the ratio K of the point pressures $q_{2.5}$ of piles of equal length at equal settlements are shown. K approximates 1 at settlements greater 5 cm for halve of the piles.

For piles 2 and 4 K deviates because of the difficulties with the enlargement of the base of pile 4 as mentioned above. The deviation in case of piles 5 and 7 is probably due to difficulties in the construction procedure too. At small settlements up to 1 cm the piles with constant diameters show a behaviour approximately according to elastic theory ($q_{2.5}/q_{2.5} = D_2/D_1$). Piles with enlarged bases show a completely different behaviour, which can not yet be explained satisfactorily.

Influence of Pile Length

Comparing the point-pressure-settlement-curves of piles with a length of 6 m and 13 m, it is seen, that there are no significant differences as well as for piles without as for piles with enlarged bases.

CONCLUSIONS

It reveals that the load-settlement behaviour is somewhat deteriorated by forming enlarged bases. For piles with 1.1 m to 1.5 m in (constant) diameter and those with enlarged bases with 1.5 m to 2.1 m in diameter no remarkable differences occured in the load-settlement behaviour. The comparison between piles of 6 m length and piles of 13 m length showed almost no difference in load-settlement behaviour.

Due to the test loadings the point pressure of Table III was taken for the projected code of practice (DIN 4014, Teil 2).

For cases, where the embedded length of the pile in sand is greater than l_o = 2.5 m, the projected code of practice DIN 4014, Teil 2, recommends values for skin friction which can only be used at settlements greater 2 cm and for that part of the pile length which exceeds l_o = 2.5 m. For settlements smaller than 2 cm linear interpolation is permitted.

Settlement	Point pressure	
s	Piles with constant diameters	Piles with enlarged bases
cm	MN/m^2	MN/m^2
1	0.9	0.65
2	1.4	1.0
3	1.9	1.2
15	4.0 [a]	2.7 [a]

[a] Fictitious failure value at settlements of 15 cm

TABLE III

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