Load Tests on Large Bored Piles in Sand

Essais de Charge de Gros Pieux Forés dans du Sable

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SYNOPSIS Some results from the test loading of d = 1.5 m bored piles in sand at two bridges are presented. On the basis of strain measuring it was possible to obtain the size and rate of shaft friction and point resistance mobilization for piles, where the ultimate load was reached. For one pile the test loading was repeated several times with pumping from deep wells in order to assess the effect of ground water level and of pile age. The influence on the evaluation of observed creep deformations etc. is discussed as well as some construction observations, which are considered important for the pile capacity.

INTRODUCTION

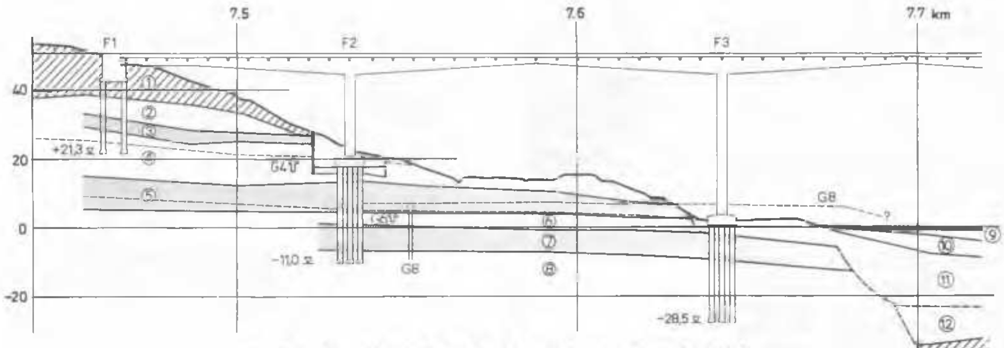


FIG. 1 Vejle Fjord Bridge, South Side

1-13	Glacial Boulder Clay
2-4-6-8	Tertiary Mica Sand w = 22-28% φ/c = 31/16 to 34/10 °/kPa
3-5-7	Tertiary Mica Clay w = 29-33% φ/c = 23/39 to 31/16 °/kPa
9	Mud, marine
10	Sand, post-glacial
11	Slide Soils
12	Sand, glacial

The three southern pillars of the Vejle Fjord Bridge, Denmark, are situated on an approx. 55 m high slope and founded on d = 1.5 m bored piles (fig. 1). The slope is built up of at least 70 m of tertiary sand and clays, topped by approx. 10 m of glacial boulder clay. The tertiary soils are mostly micaceous fine to medium sand with interbedding of overconsolidated mica clay.

Because of evidence of slide activity and superficial soil creep the design incorporated large diameter piles to resist horizontal loading. Piles were bored in a temporary casing to avoid vibrations and prevent lateral soil deformations during construction. Also other measures such as slope grading, retaining wall with soil anchors etc. were applied.

The d = 1.5 m piles at foundation 3 (fig. 2) were bored through 10 m mica clay and into micaceous fine-sand to depth 28 m. The bottom was extended by underreaming to d = 2.3 m. During construction the artesian water level was lowered by deep well pumping.

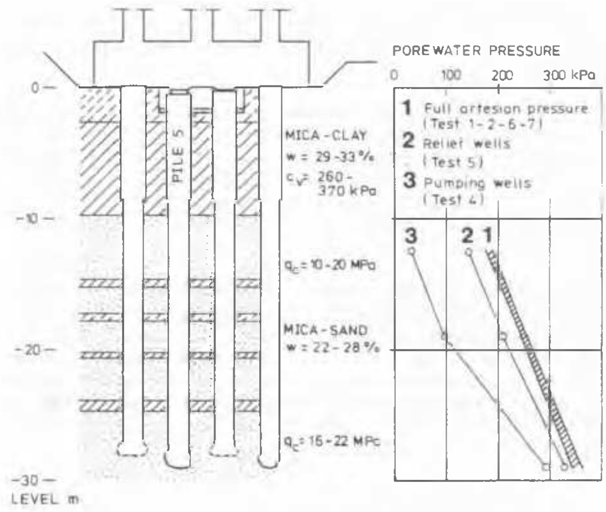


FIG. 2 Foundation 3, Cross Section

The Sathorn Bridge over Chao Phraya in Bangkok, Thailand, is in fact three parallel bridges for the two roadways and a mass transit line. Each bridge has two stream pillars founded on d = 1.5 m bored piles. The water depth is 13-18 m, and the piles reach from level 0 to -46 m (fig. 3). The subsoil is stiff Bangkok Clay to level -25, followed by sand to great depths. Around level -38 m is a 5-6 m interbedding of stiff clay which is penetrated by the piles.

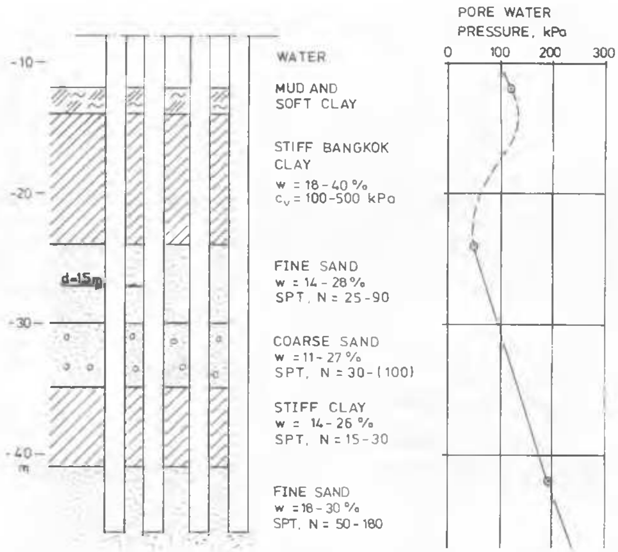


FIG. 3 Sathorn Bridge, Foundation on Bored Piles

DESIGN CONSIDERATIONS

The use of large diameter bored piles in sand is hampered by the difficulties of assessing the bearing capacity. Current methods of calculation lead to a wide range of pile capacities. At the two bridges in question this was further accentuated by the ground water conditions. At Vejle Fjord Bridge, foundation 3, there is an artesian water table approx. 7 m above sea level, while at the bridge site in Bangkok heavy ground water extraction has resulted in a ground water level approx. 25 m below sea level in the actual sand layer.

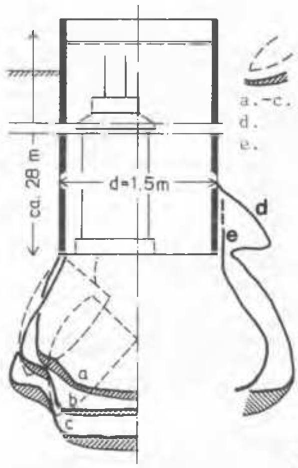
By test loading of large diameter piles the mobilization of reaction is very costly, unless the testing is postponed until sufficient adjacent piles are available. A load test at this stage to have any meaning requires preparation for increased capacity in case of an unsatisfactory result.

At Sathorn Bridge openings were left in the footings for possible extra piles. At Vejle Fjord preparations were made for possible extention of the foundations. In addition the pump wells needed for the construction phase were designed as part of a permanent ground water lowering system, which was assumed able to increase the pile bearing capacity if required.

PILE CONSTRUCTION

The Vejle Fjord piles were dug out with a 3-bladed clamshell operated by a telescopic rod. Simultaneously with the boring temporary casing were jacked down from level 0 to the underreaming level at -27 m, with telescoping at level -8 m. Water was added to maintain the water level inside the casing at ground level, while the ground water table was lowered by pumping from 3 deep pump wells from +7 m to between -2 and -7 m measured at pile tip level.

A civil engineer with a diver's licence bravely undertook to inspect most of the finished borings at Vejle Fjord prior to the casting.



Teoretical blade positions
Soft material
a.-c. Typical bottom profiles
d. Double bell
e. Casing excentricity

FIG. 4 Diver Inspection of Pile Borings

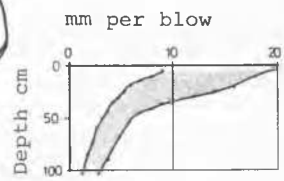


FIG. 5 Borros Rammer Drilling from Pile Tip Level.

Two major observations were noted (fig. 4): The bell obtained with the clamshell generally was much lower than anticipated, and generally a "bench" was left at the wall creating a central depression. Secondly, and more serious, considerable amounts of disturbed material were left at the top of the bench and in the central depression. After cleaning the bottom with a small wire-operated bucket, the soft bottom could only be reduced to around 5-10 cm, and it was not possible to remove the bench sediments.

In contrast the bell wall always seemed very firm and stable.

Fig. 5 shows the resistance of the finesand below the pile tip after casting of the pile according to Borros Ram drilling through encased tubes. The results seem to confirm that loose material is not (fully) consolidated by the casting pressure.

The Sathorn Bridge piles were uncased except for the uppermost part, and the borehole was stabilized with circulating bentonite slurry. Boring was carried out by rotating the boring pipe which at the end carried 3 wings with steel bits. The loosened material was removed through the pipe (reverse circulation), and the slurry was pumped back after passing clay and sand separators.

The method offered relatively fast boring. Samples from the borehole wall brought up by diver from a borehole which, by purpose, was left open for several days, showed only a thin slurry cover in spite of the large pressure difference of water inside and outside boring.

The equipment also offered the possibility of cleaning the borehole bottom by airlift in the tremie pipe, thus removing the sediments of the reinforcement period of 4-5 hours.

The piles for both bridges were cast by the tremie method. Concrete plasticizers and retarders were added to maintain slump in a 5 hour casting period.

LOAD TEST RESULTS

Fig. 6 shows results of 6 load tests on pile 5, foundation 3, at Vejle Fjord, carried out at pile ages from 5.5 to 13.8 months and with different ground water levels.

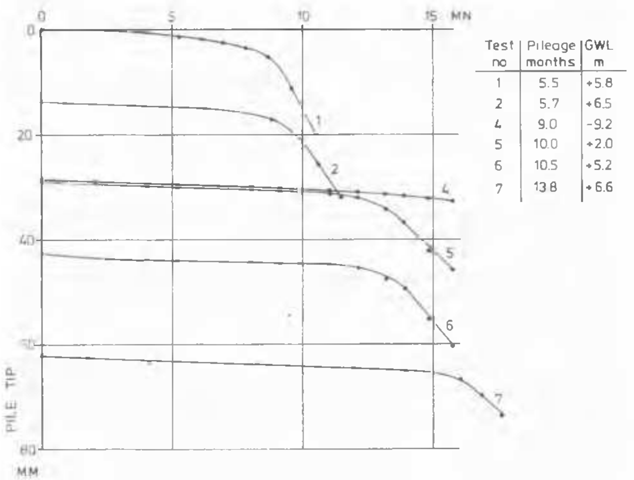


FIG. 6 Load Tests on Pile 5

The beneficial effect of ground water lowering is clearly demonstrated (tests 4 and 5).

Considering the 4 tests with no ground water lowering (tests 1-2-6 and 7), the increase in Q_{ult} may be explained as either accumulated tip resistance or by an ageing effect.

At fig. 7 the load-settlement curves for test 1 and 7 are plotted together with the load-strain curves obtained by tell-tale measuring. The compression curves appear to be composed of straight line segments, a, b and c on the figure, indicating load intervals where the center of soil resistance does not move.

The slope of these lines may be obtained by simple calculation of elastic strains if it is assumed that the pile load is taken by:

line a: adhesion in clay with adhesion center in middle of the pile section in clay.
line b: friction in sand with friction center in the middle of the pile section in sand, excluding the underreaming height.
line c: point resistance only.

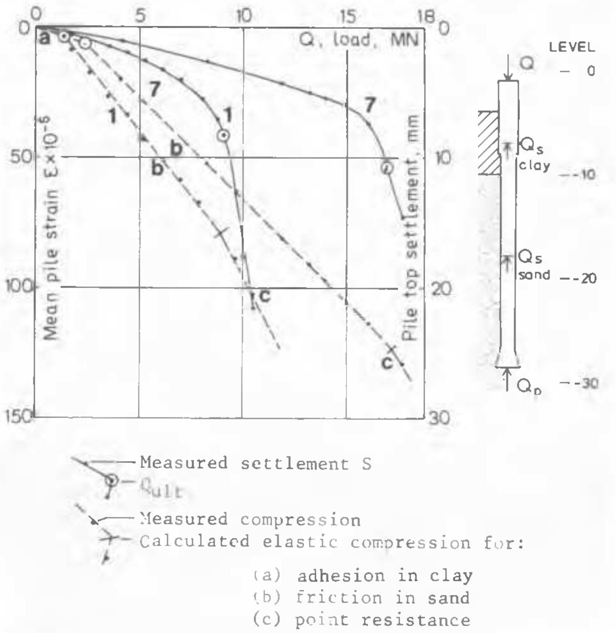


FIG. 7 Pile 5, Load Test 1 and 7

By parallel displacement of the calculated lines they will coincide with the test curves. The break points thus indicate the loads where the clay adhesion is exhausted (point a/b) and where the sand friction is exhausted, and point resistance takes over (point b/c).

The values of adhesion and friction thus established (for 6 tests on pile 5) are found at the below table:

TEST no	1	2	4	5	6	7
Pile age months	5.5	5.8	9	10	10.5	13.8
u_w at -19 m kPa	248	255	98	210	242	256
LOAD-SETTL. CURVE						
Q_{ult} MN	9.0	9.6	17-19	13.1	13.4	16.9
LOAD-STRAIN CURVE						
Clay, $q_{s,ult}$ kPa	41	41	53	50	78	75
Sand, $q_{s,ult}$ kPa	95	102	>190	156	144	186
Sa+Cl, $Q_{s,ult}$ MN	8.9	9.5	>16	14.1	14.0	17.1

TABLE 1 Vejle Fjord, Pile 5, Summary of Tests

From the near merging of the break point b/c and the ultimate load, determined from the load-settlement curve (Q_{ult} defined as the load with a settlement twice the settlement obtained by

80% of the load), it is concluded that practically no point resistance was effective at failure. The point resistance mobilization by loading in excess of failure was slow, between 1 and 2 MN per cm tip penetration for all tests. Such low values suggest that underreaming in sand may have no - or even negative - effect on the pile capacity.

The sand friction increase by ground water lowering was confirmed, however the ultimate load was not reached by the largest GWL-lowering.

The above results confirm that the considerable increase with pile age of Q_{ult} is caused mainly by sand friction increase. From 5.5 to 13.8 months the increase was approx. 90% (from 95 to 186 kPa). The ageing effect allowed the abandonment of a planned permanent ground water lowering.

By the Sathorn Bridge a similar analysis was not possible as heavy creep deformations were involved during prolonged load steps and as the ultimate load was not reached. However, the slope of the strain curves indicated that point resistance and shaft friction in the lower sand was not yet mobilized at the maximum load of 12 MN. The corresponding mean friction (level -19 to -38 m) was 135 kPa.

LOAD TEST CONSIDERATIONS

For large bored piles it is considered important to determine the load distribution along the pile for the various loads and especially to separate the shaft resistance and point resistance as the two parts are mobilized at much different rates. The general methods are strain measurements with strain gauges (Vejle) or tell-tales (Vejle + Sathorn).

However, experiences from the two sites show that a considerable amount of creep (and shrinkage?) strains are involved, complicating the evaluation.

At Sathorn Bridge, tell-tale measurements for the upper, constant-load pile part (pile in water) showed a creep deformation (mean for 5 piles) during 48 hour loading with 10 MN of $\varepsilon = 11 \times 10^{-6}$.

At fig. 8 curve A indicates a pile stress distribution calculated on the basis of assumed values of pile compression, E_b-modulus and relationship between settlement and shaft friction and between settlement and point resistance.

If an additional pile compression during the loading is considered caused by stress redistribution only, it is necessary to assume an increased point resistance (curve B) and consequently a reduced friction to explain the compression increase.

In reality, however, creep deformation may be the cause and the effect is the mobilization of extra friction. This can only take place at the expense of point resistance (curve C).

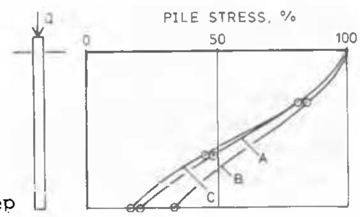


FIG. 8 Effect of Creep

It is very likely that som previously publishe tests may exaggerate the point resistance as tl test conditions did not allow an assessment of creep.

Also other effects may be important. At one of the Vejle Fjord piles strain gauge measurements were carried out during the construction period, cfr. fig. 9, where the mean result for gauges near the top is plotted. It appears that the strains were around 50 x 10^{-6} for the first months after casting, where the pile carried no load except during two "fast" test loadings, which according to tell-tale measuring produced negligible creep. However, the strains for the subsequent construction pile load (around 5 MN) agree with probable, calculated elastic strains.

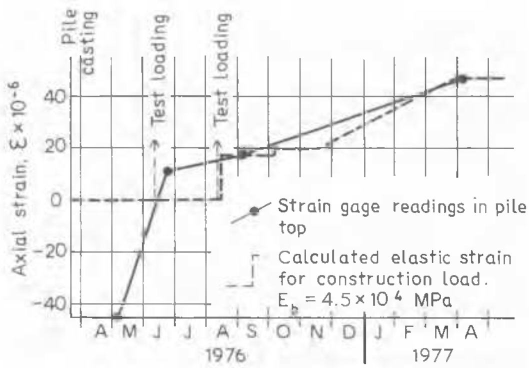


FIG. 9 Vejle Fjord, Construction Period Pile Strains

The considerable straining of the young pile probably has induced considerable shaft friction and pile strains.

It is concluded that the friction value generally obtained by a test loading should be considered only a mean value of the friction increase caused by the test load for the concerned pile or pile section. Creep and residual deformation from previous tests may have introduced positive and negative friction, which is not registered.

Another conclusion is a confirmation of Terzaghi's warning against exaggeration of calculations in geotechnics.

CONCLUSIONS

1. The testing showed a considerable increase with pile age of ultimate shaft friction.

2. An increase of ultimate shaft friction by ground water lowering was confirmed.

3. The point resistance for clamshell bored piles was almost negligible at settlements corresponding to ult. load definition, even if underreaming was carried out.

4. A considerable amount of creep and shrinkage deformations may occur and seriously influence the evaluation of shaft friction.