A case study of piling project and testing in Poland

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ABSTRACT: This paper presents a case study of piling project and testing of a huge Commercial Centre in the south-western region of Poland. The overall pile driving works involved more than 2500 RC piles of a total length over 33 000 m. Assumption that the bearing capacity of a pile driven into cohesive soil may increase significantly in time (set-up effect), was the reason for the contractor to take the risk to accelerate the testing procedure. Usually, when the load test result indicates insufficient bearing capacity, the testing procedure may be repeated after a period required by the codes of practice. The possible later increase of pile bearing capacity adds up to additional safety margin for the design. In the case of sandy soils, reported by Jardine et al (2006) values of capacity increase amounting to app. 20% do not affect much pile bearing capacity and the design procedure. It is important to state that some authors have observed an opposite effect called relaxation, which can appear in silty soil. The authors of the paper, however, have never noticed this effect. On the contrary, the numerous static and dynamic testing of foundation piles designed for Auchan Commercial Centre in Raciborz (Poland) have proved a significant time-dependent increase of bearing capacity of piles driven in silt (reaching app. 67%).

1 INTRODUCTION

The present case study refers to the pile foundation works of a supermarket hall in Racibórz, in the south-western part of Poland. The pile foundation took place on the area of a former water basin, which had been filled in with the industrial and municipal wastes in 1997.

The foundation on piles included the pile foundation support of the steel structure and of a floor area of 155 m×130 m = 19500 square meters.

The piles under the floor slab were spaced at 3-metre square grid, the spacing of the hall columns approached 28.0 m, and 3–4 piles were designed to support each column.

More than 2500 driven RC piles altogether were supposed to be used to construct the pile foundation. The designed reinforced concrete piles had the cross-section of 30×30 cm and their length varied between 11.0 and 13.0 m. Pile toe was located on the levels ranging from 176.2 m to 178.2 m above mean sea level (AMSL).

The design load for the piles under the floor was app. 300 kN, for the piles under the hall columns – 400 ÷ 600 kN.

The design load capacity of piles ranged from 500 kN to 660 kN.

2 GEOTECHNICAL CONDITIONS

After crossing the deposits and aggregate muds located at the bottom of the water basin, the piles sank into irregular strata of medium sands and gravels with the admixture of silts, sometimes into the silt interbeddings.

The pile load capacity is due to the resistance of the strata found below the aggregate muds: sands and gravels with the admixture of silts and silt interbeddings. The superficial layer composed by fills was supposed to contribute to the additional negative skin friction to the piles.

A global factor of safety of 2.5 was considered to be adequate, which took into account the additional load on the pile (negative friction), resulting from the ground settlement.

A typical soil strata profile is shown on Fig. 1.

- geotechnical stratum 0 – fill,
- geotechnical stratum Ia – soft and very soft aggregate muds – liquidity index $I_L = 0.3 \div 0.6$,
- geotechnical stratum Ib – silty clays and clayey sands – soft, verging on the firm state – liquidity index $I_L = 0.2 \div 0.3$,
- geotechnical stratum IIa – firm silts – liquidity index $I_L = 0.0 \div 0.1$; locally – silty clays,

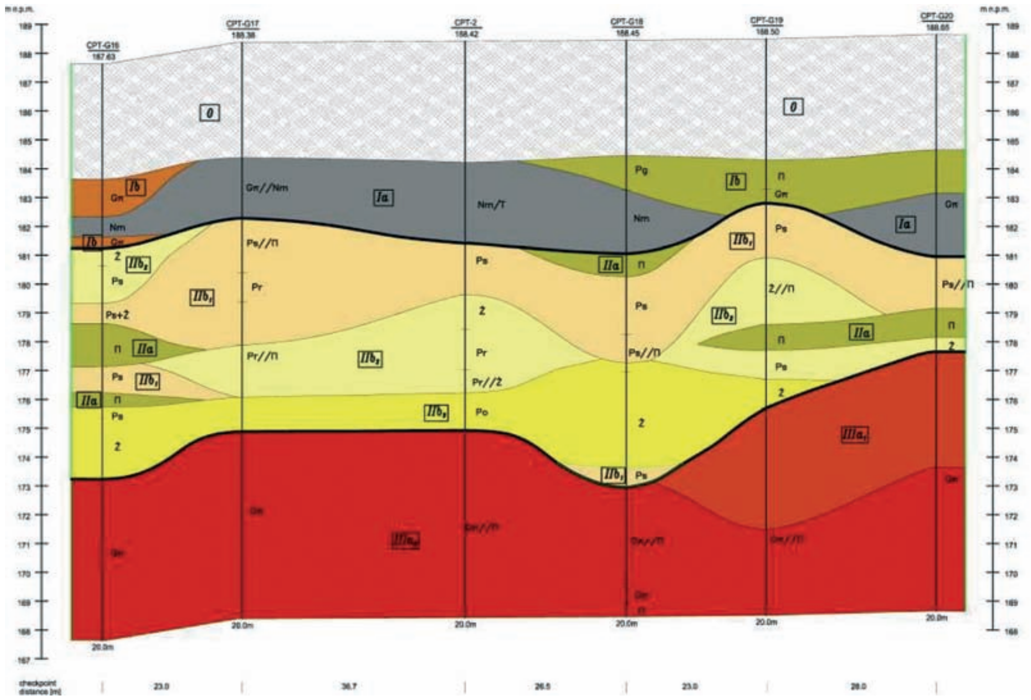

Figure 1. A typical soil strata profile based on CPT results.

- geotechnical stratum IIb_1 – non-cohesive formations (mainly gravels and medium sands) in a loose state– relative density index $I_D < 0.33$ with silt interbeddings,
- geotechnical stratum IIb_2 – non-cohesive formations (mainly gravels and medium sands) in a medium-compact state – relative density index $I_D = 0.4 \div 0.5$ with silt interbeddings,
- geotechnical stratum IIb_3 – non-cohesive formations (mainly gravels and medium sands) in a medium-compact and compact state – relative density index $I_D = 0.6 \div 0.8$ with silt interbeddings,
- geotechnical stratum $IIIa_1$ – firm silty clays and silts – liquidity index $I_L = 0.2$,
- geotechnical stratum $IIIa_2$ – silty clays and silts, in a firm state, verging on half-compact state,– liquidity index $I_L = 0.0$,
- geotechnical stratum IIIb – silty and fine sands in a medium-compact stat – relative density index $I_D = 0.4$ with silt interbeddings.

3 EXECUTION OF PILE WORKS

The insistence of the client on the prompt pile work execution, together with the concurrent delay for putting up the whole building, made it impossible to follow the appropriate procedure, in accordance with which the test piles should have been installed and the test measurements of pile load capacity should have been carried out.

As a result, the load capacity of piles, calculated on the basis of the pile penetration per blow, became the sole point of reference for the major part of the pile work, including the transport and installation of piles.

It was optimistically assumed, that the load capacity of piles would increase significantly, after the piles have been installed into the ground.

4 PILE TEST RESULTS

Dynamic pile load tests were carried out for the piles which had been installed from 4 to 6 days before. The measurements were repeated on the 55^{th} day after the piles had been installed. Load capacity has been calculated on the basis of the pile penetration per blow and by means of dynamic load tests.

Hammer energies used for the pile driving and for the dynamic load tests were very similar (the same hammer mass, falling height varying generally from 0.4 m to 0.6 m).

The values of the load capacity of piles have been juxtaposed in Table 1.

Measured values of the pile load capacity are characterised by a considerable discrepancy. In the first series, for the piles with 11.0 m in length ($L = 11.0$ m), the obtained values ranged from about 610 kN to 810 kN, for the piles with $L = 12.0$ m – from 610 kN to 890 kN, and for the piles with $L = 13.0$ m– from 550 kN to 650 kN.

In the second series, the obtained results were as follows: for the piles with $L = 11.0$ m, the load capacity varied between 640 kN and 920 kN; for the piles with $L = 12.0$ m – between 700 kN and 970 kN, and, finally, for the piles with $L = 13.0$ m – between 660 kN and 860 kN. The discrepancy of the load capacity values can be attributed to the heterogeneity of the geological structure.

The load capacity values obtained from the dynamic load tests, are marked by quite good

Table 1. Pile load capacity (stage one)

Pile No.	Set-up period	Pile length	Pile penetration per blow (last 20 cm of driving)	Pile load capacity according to Sorensen and Hansen's formula	Load capacity from dynamic load tests (CAPWAP)
	t	L	w	RD	Ru
	[days]	[m]	[t/m/blows]	[kN]	[kN]
1	6	12.0	5t/0.5/8	733	629
	55			733	695
2	6	12.0	5t/0.5/7	663	804
	55			663	868
3	6	12.0	5t/0.4/5	415	789
	55			415	815
4	6	12.0	5t/0.5/6	589	693
	55			589	889
5	6	12.0	5t/0.5/6	589	817
	55			589	973
6	12	12.0	5t/0.6/3	535	650
	61			535	795
7	5	12.0	5t/0.4/7	544	682
	46			544	805
8	4	12.0	5t/0.4/9	658	612
	53			658	830
9	5	11.0	5t/0.3/11	601	631
	54			601	645
10	5	11.0	5t/0.4/13	862	813
	54			862	917
11	5	11.0	5t/0.5/10	870	607
	54			870	687
12	6	11.0	5t/0.4/9	666	654
	54			666	669
13	5	13.0	5t/0.4/7	540	555
	54			540	664
14	5	13.0	5t/0.4/7	540	670
	54			540	799
15	5	13.0	5t/0.4/7	540	690
	54			540	863

agreement with the load capacity values calculated on the basis of the Sorensen and Hansen's formula. It was thus attempted to take advantage of the pile load capacity values estimated in that way, in order to compare them with the dynamic load tests results.

5 LOAD CAPACITY INCREASE AND CORRECTION OF PILE DESIGN

The strata, which have the decisive impact on the pile load capacity and the change of the pile load capacity in time, were sands and gravels with the admixture of silts. The fill deposit above the bottom of the basin had no influence on the change of the pile load capacity in the considered time interval.

Very soft aggregate muds at the bottom of the basin, with thickness of about 2.0 m, had no significant importance from the point of view of the pile load capacity and its change in time, either. The piles numbered from 1 to 15 show a gain of about 20% in the load capacity, in the period starting from about the 5th day until the 55th day after the piles had been driven in the ground. When we assume the formula given by Skov and Denver (1988), describing the change of the pile load capacity values in time, it is easy to predict the load capacities measured in the second series, 55 days after their driving.

$$Q/Q_0 - 1 = A \cdot \log_{10}(t/t_0) \quad (1)$$

- Q – load capacity in time (t)
- Q_0 – the load capacity at the moment of the first test
- A – the empirical constant
- t – time elapsed from the moment of the pile installation
- t_0 – the time of the first load capacity test

A 20% load capacity increase for the piles driven in sands, assuming $t_0 = 0.5$ day, may be calculated for the constant $A = 0.24$. The typically used constant $A = 0.20$ gives the increase of 17%. When we consider heterogeneous geology, the interbeddings of silt, sand and gravel, as well as the measured load capacities of all the piles, it turns out that it is adequate to assume the constant $A = 0.20$, which leads to a good mean result. The average of the load capacity increments for all of the piles is 16%.

The load capacity values measured in the first series pointed to the fact that their increments in proportion to the load capacities calculated on the basis of Sorensen and Hansen's formula assume the values which are typically observed in soil types identified at the level of the hall foundation (it was later confirmed by the second series measurements). In that way, the pile load capacities proved to be not sufficient. As a consequence, the pile foundation design was corrected.

In the hall columns pile foundations, which had been constructed before the load capacity measurements, the number of piles was enlarged, and the length of the additional piles was increased.

The other piles were lengthened from the primary calculated 11.0 m and 13.0 m, to 14.0 m and 15.0 m respectively. As a result, the pile base level was about 3.0 m in the silt stratum.

6 CHECK OF PILE LOAD CAPACITY AFTER PILE DESIGN CORRECTION

On the lengthened piles and piles driven in silt (numbers 16–26), 13 days after installation, the load capacity was tested. The results have been presented in Table 2.

Table 2. The results of the measurements of the pile load capacity (stage two)

Pile No	The time elapsed between the driving of the pile and the test	Pile length	Pile penetration of per blow (last 20 cm of driving)	Pile load capacity according to Sorensen and Hansen's formula	Load capacity according to dynamic load tests (CAPWAP)
	t	L	w	RD	Ru
	[days]	[m]	[t/m/blows]	[kN]	[kN]
16	77	14.0	5t/0.5/8	717	1291
17	13	13.0	6t/0.4/9	761	1459
18	13	13.0	5t/0.4/12	795	1189
19	13	14.0	6t/0.4/14	1002	1083
20	17	14.0	6t/0.4/6	558	1106
21	13	14.0	6t/0.4/7	628	838
22	5	15.0	6t/0.2/7	337	697
23	17	15.0	6t/0.2/8	374	786
24	45	11.0	5t/0.2/7	293	569
25	13	12.0	5t/0.5/8	733	1006
26	14	13.0	6t/0.4/13	972	1369

The measured load capacity values varied between 570 kN and 1370 kN. The substantial discrepancy of load capacity values could be observed already at the time of pile installation. In line with Sorensen and Hansen's formula, the values ranging from 290 kN to 970 kN were obtained. No second series of measurements was carried out on those prolonged piles. It is possible, however, to venture predicting the pile load capacity, relying on the results obtained on the basis of pile penetration per blow values and the dynamic load tests capacity. Such prediction is by nature approximate, hence it was assumed, that the load capacity calculated on the basis of the values of the pile penetration per blow was, at the same time, the load capacity of a pile on the first day after driving. The exact analysis of the geology of particular piles was also abandoned. The average increase of the load capacity of piles number 16 to 26, in the period of 13 days after they had been installed, amounted to 68%. When we substitute $A = 0.6$ in equation (1), taking into account the period from the first to the 13th day after the pile driving, we obtain a significant load capacity increase, reaching about 67%.

7 CONCLUSIONS

The check of the mean values of pile load capacity made it possible to predict, with no great difficulty, the load capacity of the installed piles already at the time of their driving, relying on the values of the pile penetration per blow. Such rough and ready check, however, does not always bring about good results, therefore, it is indispensable to carry out control tests in order to identify the exact soil conditions, which is imperative from the point of view of the pile load capacity. Here, the identification of soil conditions was essential for the proceeding of the pile work. Under the pressure of the pile work deadline, the values of pile load capacity were supposed to be predicted, in not well-characterized geological conditions and in a shorter time than required in regulations referring to pile load capacity tests. At the same time, it turned out to be justified to estimate the lower values of load capacity increase, at least in the geological conditions which had not been identified with respect to the pile load capacity.

Eventually, such all too optimistic estimation of pile load capacity led to the increase of the pile work cost. It was required to enlarge the number of piles in places, where piles had been driven in before the dynamic load tests measurements. Due to the great number of piles according to the design, it was necessary to start the transportation of piles before the completion of the full geological survey. As a result, some of the piles had to be returned to the factory. Longer piles had to be delivered to the construction site, instead.

In 2007 the commercial centre was extended. The experience gained a year before certainly made it easier to come up with a new design and to execute the pile work. The observation of the values of pile penetration per blow, without load capacity measurements, was enough to fulfil the terms of the contract. Load capacity tests were carried out towards the end of the pile work, in order to comply with the formal pile work regulations.

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