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VERIFICATION OF THE BEARING CAPACITY OF PILES

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Abstract. The location of the buses in the basement of the new bus station in Bratislava, supplemented by two further parking floors below, shifted the foundation joint of the new bus station to a depth of approximately 14 m. The maneuvering space for the buses required unusually large distances between the supporting columns. In extreme cases, it was necessary to transfer a vertical load of more than 30,000 kN from the column to the subsoil. Only a relatively thin layer of Quaternary gravel remained between the footing bottom and the Neogene soils. The whole area was designed to be based on deep foundations. As this was a construction of the 3rd geotechnical category, it was necessary to verify the bearing capacity and quality of the piles. The applied test methods (especially the dynamic load tests) confirmed the correct design and implementation of piles.

Keywords: Dynamic load tests, extreme foundation demands, integrity tests, pile control calculations, pile design.

1. Introduction

The complex, including the bus station and commercial and administrative areas, is approximately rectangular in shape with ground plan dimensions of 420 x 175 m (see Figure 1). The majority of buildings have an average of 5 floors. The dominant feature of the complex is a 125 m high administrative building with 32 floors. The construction site is located on a flat area with a range of altitudes of 136.40 m a.s.l. up to 137.20 m a.s.l. and an average height of 136.80 m a.s.l. [1]

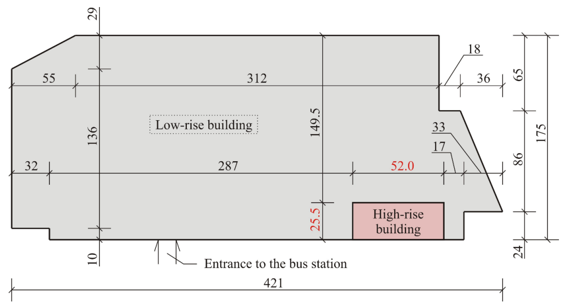

Figure 1. Ground plan of a construction pit with dimensions 420 × 175 m.

2. Geological Conditions on Site

The engineering geological investigation included 40 boreholes (some of which reached a depth of up to 37 m) and a large number of dynamic penetration tests. From a comprehensive evaluation of the investigation works, several subsoil models with a comparative level of ± 0.00 = 137.50 m a.s.l. were created for the characteristic parts of the area. A typical geological model for pile design had the following sequence of layers:

0.0 – 1.1 m fill;

1.1 – 6.2 m clay with intermediate plasticity, stiff (F6);

6.2 – 14.6 m gravel badly grained (G2);

14.6 – 22.2 m sand clayey (S5);

22.2 – 23.1 m clay with intermediate plasticity, mostly solid (F6);

23.1 – 23.5 m clay with high plasticity, solid (F8);

GWL – 6.6 m below the surface.

According to the penetration probes, the quaternary gravel layers were evaluated as medium compacted with a scattering of values $I_D = 0.36$ to 0.65 and modulus of deformation $E_{def} = 55$ to 147 MPa. However, in deeper positions, loose layers with $I_D = 0.21$ to 0.34 and $E_{def} = 22$ to 49 MPa were also found in the gravel. The upper parts of Neogene sediments are comprised of sand with an admixture of fine-grained soil also to sandy clay. Sands with an admixture of fine-grained soil were evaluated by dynamic penetration tests as medium compacted ($I_D = 0.41$ to 0.50 and $E_{def} = 23$ to 29 MPa).

In laboratory tests, Neogene layers were evaluated as soils with a predominant solid consistency ($I_C = 1.04$ to 1.31; mean $I_C = 1.16$). Given the summary evaluation of Neogene soils, it is necessary to point out the average values: $\phi' = 27°$; $c' = 19$ kPa; $E_{oed} = 37$ MPa. More precisely, the oedometric modules for the three depth zones that were adopted for the design of the piles were evaluated as follows:

- for depth zones 14 – 18 m below ground $E_{oed} = 21.1$ MPa;
- for depth zones 18 – 26 m below ground $E_{oed} = 33.2$ MPa;

- for depth zones 26 – 35 m below ground E_{oed} = 40.1 MPa.

For various soil classes, the bulk density and shear strength of soils were also elaborated by laboratory analysis:

- for clay with high plasticity (F8-CH): $\gamma = 20.3$ kN/m^3; $\phi' = 19.8°$; $c' = 21$ kPA;
- for clay with medium plasticity (F6-CI): $\gamma = 20.7$ kN/m^3; $\phi' = 24.7°$; $c' = 16$ kPA;
- for silt with medium plasticity (F5-MI): $\gamma = 20.2$ kN/m^3; $\phi' = 28.6°$; $c' = 16$ kPA;
- for sandy clay (F4-CS): $\gamma = 20.2$ kN/m^3; $\phi' = 29.9\circ$; $c' = 10$ kPA;

3. Verification of Piles Design

In the predominant part of the ground plan area, there are clayey soils just below the foundation joint, or indeed clayey sand. All these soils are characterized by a relatively smaller deformation modulus, which would cause a large settlement of the foundation plate. In addition, in the floor plan of the construction pit, a zone with worse soil properties was discerned in the middle part. This area caused significant problems in the early stages of design. The whole building had to be based on a foundation plate to protect against the groundwater level. In the larger part of the floor plan, a 0.7 m thick foundation plate was designed; in the places of the pillars, strip-shaped depressions with a thickness of 1.1 to 2.2 m were formed under concentrated loads, and which were supported by piles. Taking into account technological possibilities, CFA type piles were selected. The support of the foundation plate with piles made it possible to transfer large loads to greater depths. Reducing the settlements was only achieved at the cost of relatively long piles.

Altogether, more than 600 piles with very high shape variability have been designed. The required short production deadlines placed high demands on the organization of work. In terms of assortment, there were 32 types of piles with a variable length of 6.0 to 19.0 m, with a diameter of 900 or 1200 mm. By taking into account different load conditions and fluctuations in the groundwater level, some piles can also be stressed by tensile effects. Due to the large volume of piling work, a detailed audit of the design of piles was carried out, aimed not only at reaching the required bearing capacity but also at minimising the expected settlement [2].

For the given combinations of loads taken from the structural engineers of the upper structure, the vertical bearing capacity of the piles under all columns was recalculated by the classical theory using ČSN 73 1002 and the assumed settlement by the nonlinear theory as according to Masopust. In the case of piles with a diameter of ∅ 900 mm, 6.0 to 15.5 m long, the settlement in the range of mobilisations of mantle friction was between 7.5 – 9.6 mm, in the case of piles diameter ∅ 1200 mm, 14.5 to 16.5 m long there was a settlement corresponding between the mobilisations of the mantle friction in the interval 11.6 – 11.8 mm. For the assumed loading of the piles with vertical forces, the settlement of piles ∅ 900 mm long 6.0 to 15.5 m in the range of 3 – 24 mm and similarly piles of ∅ 1200 mm long 14.5 to 16.5 m length of 14 – 20 mm was calculated. The designers of the upper structure determined the permissible settlement to be 25 mm. The foundation designs had to be verified by a load test for the construction of the 3rd geotechnical category.

4. Verification of Pile Manufacturing

With an excavation depth of up to 14.7 m, the buoyancy effects of groundwater in some places reached more than 8 m. To this must be added unusually high point loads. Therefore, after the audit of the pile design, it was necessary to verify whether the implementation of the foundation met the requirements of the project. As part of the quality control of the construction work, the following piles inspections were carried out:

- inspection records of fabrication of piles;
- control of pile heads;
- integrity check (so-called PIT tests);
- dynamic load tests of piles.

All piles had their production records checked, and from which any anomalies could be detected. No insufficiency was found during this inspection. Similarly, the pile heads were inspected, ready for connection to spread foundations. During this stage, harm heads were found in three piles [3]. The unsatisfactory part of the head was excavated to a healthy full profile of the pile (it was approximately 600 mm) and concreted with a concrete grade of strength to a class one degree higher than the original pile.

A pile integrity test (PIT) was carried out on systemic piles [4]. A hammer blow weighing 1 kg on the pile head will cause acoustic waves to propagate to the foot of the pile. Changes in the shape and quality of the material of the pile body create reflections, which are observed as returning waves to the pile head. During the inspection, the projected length of the piles was compared with the determined length and also at the same time with the age of the piles. Piles with a diameter of ∅ 900 and 1200 mm, 6.0 to 18.0 m long, were verified, while the piles had a test time of 5 to 69 days.

58 PIT tests were performed on the site. In only one case was a non-standard acoustic wave (at a depth of 9 m) detected [5]. Therefore, the test was repeated the following day with the participation of the investor's technical supervision, and without confirming an anomaly that would indicate a violation of the homogeneity of the pile shaft. The initial non-standard

result was explained by the existence of a working joint under the pile head, which was concreted to the projected level later in the second phase. On a complete evaluation of the PIT tests, it was possible to state that all tested piles had the prescribed length and none of the piles showed a narrowing of the diameter or a break in the shaft.

4.1. Dynamic Load Tests

The most important form of piles verification were dynamic load tests. Compared to static load tests, they are simpler, cheaper, but above all faster. Their disadvantage is that they do not allow a direct measuring of the working diagram of the piles. The magnitude of the dynamic force (impact from the free fall of the weight), the magnitude of the instantaneous deformation of the pile head, and also the rate of this deformation were all recorded during the test. The test was evaluated by the CAPWAP method (Case Pile Waves Analysis Program). The requirement of strict adherence to the construction work schedule did not make it possible to create space for verification of the bearing capacity of piles by classical static load tests. In countries where piling technologies and methods for monitoring the achievement of design parameters are being developed, dynamic load tests are widespread.

A pulse wave of voltage is induced in the pile by a specially adjusted weight (Figure 2). The strain gauges located at the top of the pile measure the force that causes the pile to move after the impact. Another way to measure force is to record the acceleration of the shock wave with an accelerometer. The course of speed is determined by integration. The results of this impact velocity and the impedance of the pilot are the second way to measure force. Unlike pure force measurement, this calculation contains an acceleration vector. By comparing these two impedance measurements, two waves of forces are obtained; at the same time, one wave of forces is directed downwards towards the base of the pile and the other force is directed upwards, made manifest by friction on the casing and resistance on the base of the pile. The maximum bearing capacity of the pile is calculated from the measured skin friction and the resistance of the pile base.

Dynamic load tests (PDA) were performed in two terms on 4 non-systemic piles. The tests were performed by the company DUBA Testing CZ, Plzeň [6]. From the detailed evaluation by the CAPWAP program, in Table 1 and 2 there are presented the summarized outputs.

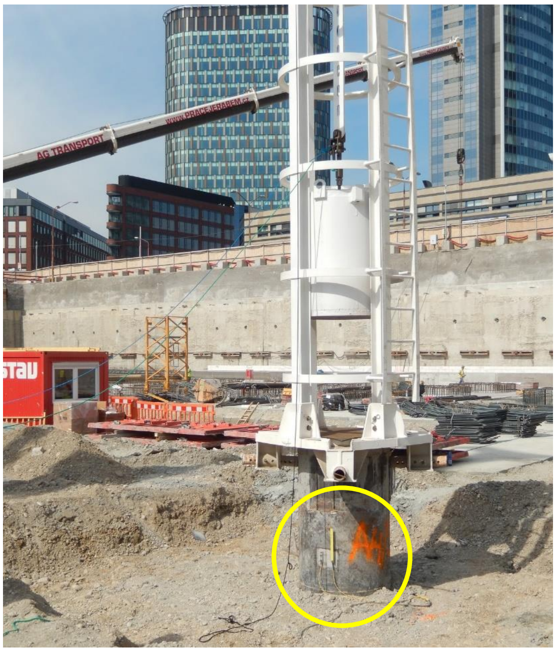

Figure 2. Dynamic load test of the pile and location of strain gauges.

Notes:

- Pile length is given as length below ground level.
- The total length of the piles with the extension was:
  - ▹ pile A4/SP1 – 14.0 m;
  - ▹ pile B3/SP4 – 17.1 m;
  - ▹ pile A3/SP2 – 13.25 m;
  - ▹ pile B2/SP3 – 15.8 m.
- Max. s in Table 2 is the maximum settlement of the pile.
- Comment on the bearing capacity of piles:
  - ▹ for the A4/SP1 pile, the bearing capacity of the pile base was not fully mobilised;
  - ▹ for the B3/SP4 pile, the bearing capacity of the pile base and the skin friction were not fully mobilised;
  - ▹ for the A3/SP2 pile, the bearing capacity of the pile base and the skin friction were not fully mobilised;
  - ▹ for the B2/SP3 pile, the bearing capacity of the pile base was not fully mobilised.

From the evaluated load-dependence curves, it was possible to determine the expected settlement of the tested piles under static load. On some construction works, where static and dynamic load tests of piles were performed in parallel, very good similarities of both tests were confirmed. This attests to the sufficient reliability and credibility of the results of the dynamic load tests. From the evaluation of dynamic tests, it is also possible to determine the expected settlement of a particular pile for different intensity of static load (see Table 3).

A very important positive circumstance that results from the dynamic load test should be considered the

Pile No.	Length of pile (m)	Fall height (m)	Settlement of the pile (mm)
A4/SP1	12.5	1.76	3.0
B3/SP4	15.0	1.96	3.5
A3/SP2	11.25	1.76	2.5
B2/SP3	13.3	1.96	3.5

Table 1. Permanent settlement of test piles after 5 blows with a load.

Test date	Pile No.	Age of piles (days)	∅ of pile (mm)	Legth of pile (m)	max. s (mm)	Bearing capacity (kN)		
						Skin friction	Pile base	Total
4.7.18	A4/SP1	35	900	12.5	8.0	4770	1160	5930
4.7.18	B3/SP4	35	1200	15.0	7.0	6550	2650	9200
25.7.18	A3/SP2	28	900	11.25	6.5	5300	1300	6600
25.7.18	B2/SP3	28	1200	13.3	6.5	6630	2100	8730

Table 2. Evaluation of test piles by PDA.

evaluation that for all 4 tested piles the full value of the bearing capacity at the pile base, and in two cases the full value of the mobilised skin friction, was not reached. This means that the test piles still held out the possibility of increasing the load capacity. At the same time, however, it should be noted that this increased bearing capacity will be achieved at the expense of a relatively more intensive settling. The prognosis of bearing capacity, but especially of the expected settlement by exceeding the tested values, is not based on explicitly verifiable data. Extrapolation of the evaluated measurements should be considered risky and it was not recommended to consider it.

Pile No.	Bearing capacity after CAPWAP (kN)	Load (kN)	Assumed settlement (mm)
A4/SP1	5930	2800	2.2
		4000	3.1
		5600	8.1
B3/SP4	9200	4500	1.6
		6300	2.7
		9000	6.8
A3/SP2	6600	3000	2.2
		4000	3.0
		6000	6.8
B2/SP3	8730	4500	2.0
		6300	2.9
		8500	5.1

Table 3. Assumed settlement from static load.

Experience with dynamic tests of piles placed in clayey soils has shown that, under static load, a final pile settlement of two times more than the one calculated from the dynamic test can be expected. Data in the last column Table 3, even with such an increase, shows they met the required maximum permissible settlement for the upper structure.

The data obtained from the dynamic load tests were finally compared with the audit of the pile foundation project, in which control calculations of the bearing capacity and settlement of 32 piles of various dimensions placed in variable geological conditions were performed. For all tested piles on the construction site, it was possible to confirm a considerable margin not only in bearing capacity but especially in the assumed installation of piles in comparison with the control calculations.

5. Conclusion

Dynamic load tests are not yet considered in Slovakia as tests the results of which can be accepted without discussion. In Western Europe, these tests are accepted as routine tests. The spatial distribution of dynamic load tests, as well as PIT tests (integrity), could be considered sufficiently representative for the establishment of the entire bus station area. All performed verifications and tests confirmed the fulfilment of the project assumptions.

The performed dynamic load tests confirmed the higher bearing capacity of the tested piles than that required by the project for load transfer from the upper structure. Given this fact, it was recommended to continue making piles according to the project. Interiors are currently being completed throughout the building. The covid-19 pandemic caused a slight time adjustment for the completion of the whole site (Figure 3). No anomalies have been detected by monitoring so far. It has been shown that dynamic load tests can also be used in our given conditions as equivalent to static load tests, while their implementation is significantly faster and cheaper.

Figure 3. View of the completed area of the bus station.

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References

[1] *Project documentation for the establishment of a multifunctional complex of the NIVY bus station.* (in Slovak).

[2] P. Turček. *Forecast of settlement of th TWIN CITY complex in Bratislava (area B3.1 and B3.2).* Expert assessment. T-G, Bratislava, 04.04.2018, 6 p. (in Slovak).

[3] M. Balucha. *Opinion of the designer on the rehabilitation of pile heads.* SPAI, Bratislava, 1 p. (in Slovak).

[4] F. Kout. *Multifunctional building Bus station Mlynské Nivy.* Pile foundation. Accompanying report for checking the integrity of piles using the PIT method. Zakládání staveb, a.s. Praha. 19.09.2018. (in Czech).

[5] J. Šperger. *The technologist's statement on the results of the PIT test of the A2-10 pile.* Zakládání staveb, Praha, 05.10.2018, 1 p. (in Czech).

[6] Collective. *Report on measuring the bearing capacity of piles using the PDA method on the TWIN CITY construction site, bus station Mlynské Nivy, Bratislava.* DUBA Testing CZ, s.r.o., Plzeň, 04.-09.07.2018 and 25.07.-06.08.2018. 2x18 pp.