LOADING TESTS ON BENOTO PILES

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Introduction

In the late 1960's a 10,000-ton granary silo was built in *Kaposvár* (Hungary). The foundation was designed with *Franki* piles 12 m long, assumed to have bearing capacities of 1.3 MN each. The loading test, however, did not confirm this value: the piles settled significantly and nearly failed under loads as low as 0.7 to 0.9 MN. Therefore the only way to save the foundation was to increase the bearing capacity of the piles by preloading the ground surface, applying to this purpose the dead weight of the silo (Kézdi, 1971).

In 1975, the construction of another silo was begun in the vicinity of the already successfully stabilized one. Soil investigations were carried out and a suggestion for foundation design was made by the Department of Geotechnique, Technical University, Budapest. A characteristic result of soil exploration and investigation is represented in Fig. 1. The subsoil is a typical transition soil, loose and saturated with water, hence of low bearing capacity and highly compressible. Owing to the great thickness and comparatively low permeability of the layers, these soils behave even in the case of an average rate of construction as closed systems; filling the silo with grain is a definitely sudden application of load. In such a case exhaustion of the bearing capacity (i.e., soil failure) is not accompanied by volume change. Allowing for the hazard of a sudden loading, the shear strength parameters were determined by undrained (i.e. constant-volume) tests without leaving time for consolidation.

The strata shown in the figure are followed by a 0.6 to 2.3 m thick layer of silty fine sand and gravelly sand. From the depth 17.7 to 18.7 m below the ground surface only cohesive soils (silt, clay), here and there very hard, were found.

The phreatic water level at the site is rather high, it is near the ground surface.

To meet the soil conditions, Benoto piles with lengths $l = 18.5$ m and 90 cm diameter were proposed by the Department of Geotechnique.

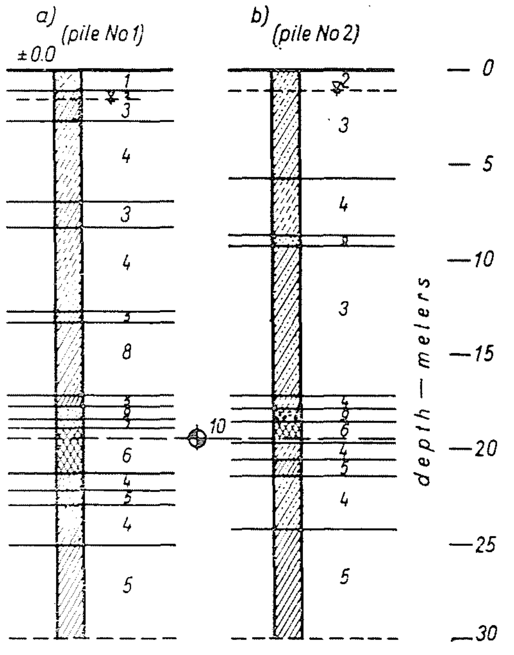

Fig. 1. Boring logs at the site of test piles: a — boring No. 1; b — boring No. 2; 1 — fill; 2 — groundwater table; 3 — silty fine sand; 4 — silt; 5 — clay; 6 — hard clay; 7 — organic; 8 — silty fine sand; 9 — sand, gravel, 10 — plane of the pile base

The calculated ultimate bearing capacity was $P_t = 4.3$ MN; from this, in the ultimate conditions the share of the tip resistance and skin friction was 2.0 MN and 2.3 MN, respectively. As limiting bearing capacity $P_h = 1.4$ MN has been given.

The financial aid jointly offered by the builder and the designer gave us a possibility to use special instrumentation for the standard loading test. Specially designed measuring cells have been built in, indicating the pressure transferred at the pile base, yielding valuable information on the actual value of the tip resistance.

Pile loading test

Prior to beginning to build the foundation, two piles were subjected to load test. The diagram of the load tests on piles 1 and 2 is shown in Fig. 2. The results of the load test are summarized in Table 1. The "ultimate load" could be determined in several ways:

a) as a value associated with the settlement $y = 3$ cm;
b) as a value associated with the transition of the diagram into an inclined straight line;

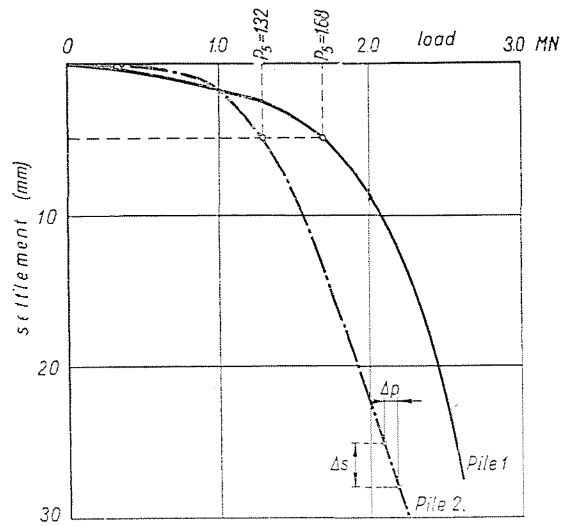

Fig. 2. Loading test diagram

Table 1

Evaluation of the load test			
Data		Pile No. 1	Pile No. 2
Length of pile (m)		19.5	19.5
Diameter (m)		0.9	0.9
Pile load (MN)	a)	2.65	2.26
	b)	2.62	2.26
	c)	2.80	2.58
	d)	2.75	2.50

c) according to Skempton's diagrams;

d) according to the Norvegian code of practice.

As ultimate force $P_t = 2.5$ MN was obtained, corresponding to the limit load:

$$P_H = 0.6\ P_t = 1.5 \text{ MN}.$$

Because of group action, this value had to be reduced in dependence on the pile spacing. For pile spacings equal to or greater than 2.5 m, the value of the reduction factor is 0.9 and below that, 0.8. That is, $P_H = 1.35$ MN, or $P_H = 1.20$ MN.

Measurement of the tip resistance

For the purpose of measurements the measuring cells constructed by the *Hungarian Institute for Building Science* were applied which in several instances have successfully been used. The permissible pressure of the cells was 2 MN/m^2. The cells had to be inserted under the pile-tips so as to provide direct contact with the soil and to avoid cell displacement by the first batch of concrete placed in the borehole.

Therefore the cell has been rigidly attached to the reinforcing steel of the pile and prior to placing it into position the bottom of the borehole has been filled with sand. The cells were placed in the soil immediately before concreting. Unfortunately, because of insufficient concrete supply (the concrete mixing plant did not work yet), no continuous and rapid concreting of the pile was possible.

Prior to placing the concrete, the water column rose to nearly 19 m in the borehole. The fine grains getting into the ground-water during excavation of the borehole remained there in suspension. Due to the delayed concreting, sedimentation of soil particles took place which adversely affected the state of soil under the pile tip.

After the completion of concreting, the cells have been zeroed with piles unloaded; then the piles were loaded in increments and the stresses induced were read.

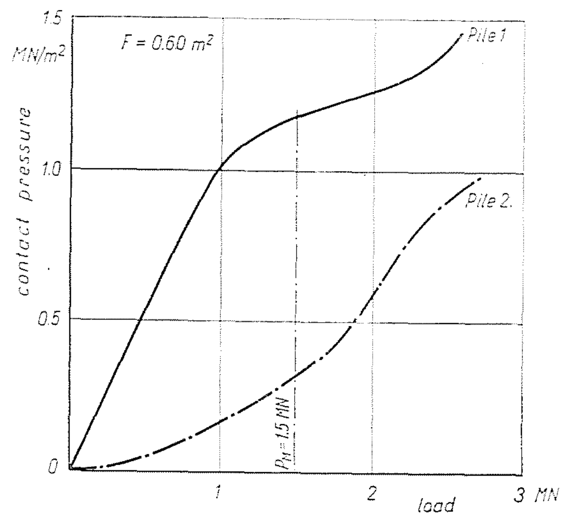

Fig. 3. Base pressure versus load

Fig. 3 shows the development of the base pressure in course of loading piles 1 and 2. The two diagrams are somewhat different in nature. Under pile No. 1, initially the base pressure increased rapidly, and in the first loading stage even linearly, from which it could be concluded that the skin friction was minimum. In the case of higher loads, the rate of base pressure increase diminished, i.e., the skin friction took up an increasing part of the load.

The base pressure in pile No. 2 hardly increased initially. Accordingly, the greatest part of the load was carried by the pile skin. A heavier load (about 1.2 MN) was needed to provoke a higher rate of increase in the base pressure. Following that, the rate of increase was almost the same as that measured on pile No. 1.

By comparing the measurement results with those of model tests reported in publications it can be stated that the stress development found under pile No. 2 is, in general, the more frequent one. The uncommon behaviour of pile No. 1 might be ascribed on the one hand to faulty construction, on the other hand, this behaviour might be foreseen from the boring log given in Fig. 1. The soil strata along the skin of pile No. 1 are seen to be of more cohesive character, providing lower friction values than in the environment of pile No. 2. In contrary, the higher tip resistance found for pile No. 1 can be ascribed to the presence of a thick, hard layer of blue-grey clay.

Variation of tip resistance in dependence on time, load and settlement

The measurement values evaluated will be those for pile No. 2 deemed to be more reliable. The first and most appropriate representation would be a load-dependent plotting. Presuming a uniformly distributed base pressure, the force at the pile tip (i.e. the tip resistance) is investigated. The remaining value from the total load is the skin friction. The distribution of these two forces is shown in Fig. 4.

It is evident that in the range of small loads the function of the skin friction is crucial. The tip resistance increases only slowly, after the skin friction is fully mobilized through the settlement of the pile. The ultimate value associated with failure would correspond to the results calculable from static equilibrium formulae.

In Fig. 5 the percentage distribution of the pile force and the ratio of the two factors (i.e., the tip resistance and the skin friction) are plotted against the total load. The tip resistance of pile No. 2 is seen to uniformly increase up to a load of about 2 MN. Afterwards the rate of increase diminishes and the curve tends to a horizontal asymptote. This means that in the environment of the pile tip the shear resistance of the soil is exhausted and no further load can be carried by the soil but only at cost of a significant penetration. In the figure, the results measured on pile 2 (curve *a*) have been compared with those obtained in tests by Mansur and Kaufmann in 1956. They measured the tip resistance of piles of similar length whose percentage distribution is

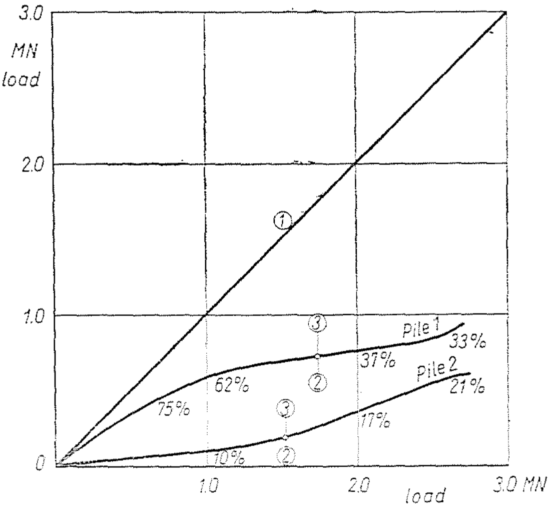

Fig. 4. Distribution of the pile load. 1 — total pile reaction; 2 — tip resistance; 3 — skin friction

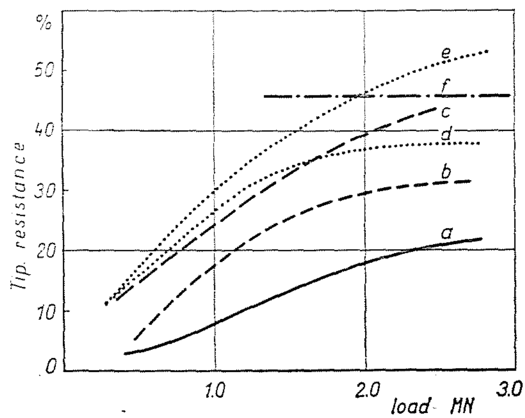

Fig. 5. Percentage distribution of the pile load. P_t — total pile reaction, P_c — tip resistance, P_k — skin friction, a — pile No. 2, b — Kaufman's measurement, pile $l = 19.8$ m, c — Kaufman's measurement, pile $l = 20.1$ m, d — slurry trench wall, Óbuda, e — slurry trench wall, Újpalota, $l = 6.8$ m $F = 2.2 \times 0.55$ m^2, f — Kaposvár, value calculated for pile No. 2 (according to Meyerhof's formula)

shown by curves *b* and *c* drawn in dashed line. The difference is explained by the different soil conditions and pile diameters, but the character of the curves is the same. Also the results of basement pressure measurements, obtained in 1970 on diaphragm walls (dotted curves *d* and *e*) are given in the figure.

According to our measurements, in failure state the skin friction was roughly equal to the theoretical value 2.3 MN. The tip resistance was, however, much lower.

This might be explained by the fact that in a highly compressible soil no complete failure could develop. Will the settlement of the pile continue, so the tip resistance will increase, theoretically to infinity; in this case, however, the settlement would increase beyond every limit permissible.

Therefore, it is more practicable to calculate the stress distribution as a function of settlement.

According to the theoretical formulation by KÉZDI (1957, 1970) for cases where both tip resistance and skin friction are expected, the relationship may be written as follows:

$$P = P_1 + P_2,$$

here

P_1 = tip resistance,
P_2 = skin friction

and

$$P_1 = a \, . \, s \, . \, d \left(l\gamma + \frac{4P_2}{n^2 d^2 \pi} \right)$$

$$P_2 = K_0 d\pi \frac{1^2}{2} \gamma \tan \varphi \left(1 - e^{-k \frac{s}{s_0 - s}} \right)$$

where

a — proportionality factor to be calculated from end section of load-settlement diagram of pile:
$a = \dfrac{\Delta P}{\Delta s}$ (Fig. 2); in our case $a = 30$;
d — diameter of pile;
l — length of pile;
γ — mean soil density;
n — constant assumed with a value of 3 according to measurements;
K_0 — coefficient of earth pressure assumed with an intermediate value between "at rest" and "passive" states;
$\tan \Phi$ — coefficient of friction involving also the effect of adhesion (in the actual case $\Phi = 13°$);
k — proportionality factor to be read off the shear-strain diagram (in this case 2);
s_0 — settlement associated with full mobilization of skin friction (according to measurement results, 40 mm or so).

Herewith, the distribution of forces associated with s_0:

$$P_1 = 0.56 \text{ MN},$$

$$P_2 = 1.87 \text{ MN}.$$

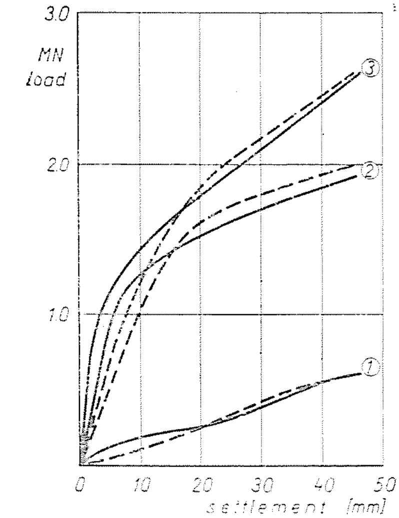

Fig. 6. Variation of skin friction and tip resistance vs. settlement. 1 — tip resistance; 2 — skin friction; 3 — total pile reaction; – – – – calculated; ——— measured

Values P_1 and P_2 calculated for other s values are represented in Fig. 6. The measurement and calculation results are shown by continuous and dashed lines, respectively.

As far as the tip resistance is concerned, the two values are in a rather good agreement. In the range of small settlements the measured value of skin friction is lower than the calculated one.

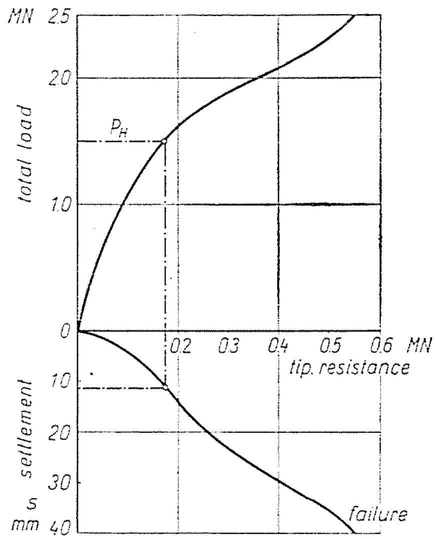

Fig. 7. Relationship between tip resistance, settlement and pile load (pile No. 2)

This finding again seems to support the statement that piles under small loads owe their load capacity — especially in soft, cohesive soils — mainly to the skin friction, while the tip resistance only begins to increase after a penetration of a few centimeters, and reaches its maximum value near failure.

The behaviour of the tip resistance versus pile settlement has been investigated by making use of the measurement results on pile No. 2. In Fig. 7, both the tip resistance and the total pile load are plotted versus settlement. Initially the tip resistance is seen to gradually increase with the settlement, to slow down near failure and at last the curve tends to a sloping asymptote. It may be observed that the pile force increases slower vs. tip resistance above the given limit load. This means that the greater part of the skin friction became already mobilized, and the margin of safety resides in the tip resistanc e

Measurement of forces in piles under the building

Measuring cells were placed in certain points to investigate the forces in piles. Pressure cells (o.d. 323 mm, wall thickness 7.5 mm, height 70 mm) were constructed by the Department of Geotechnique. Two pairs of strain gauges were stuck diagonally to the inner surface of the cells (Fig. 8). 8-core electric cables leading out of cells were watertight sealed. Moreover the cells were completely filled with a special watertight material. Then they were placed on a thick steel plate fixed to the top of piles.

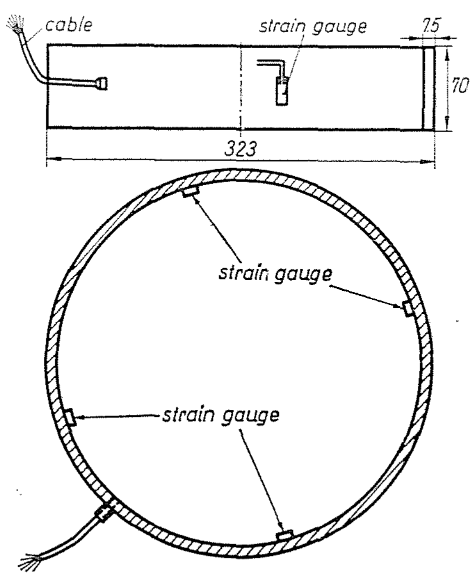

Fig. 8. Sketch of a pressure cell

Table 2

Record of measurements

	Pile No. 1		Pile No. 2*	
	Reading	Load MN	Reading	Load MN
Basic measurement		0		0
August 1976	−0.005	0.008	−0.010	0.016
September 7, 1976	−0.137	0.219	—	—
October 28, 1976	−0.139	0.225	—	—

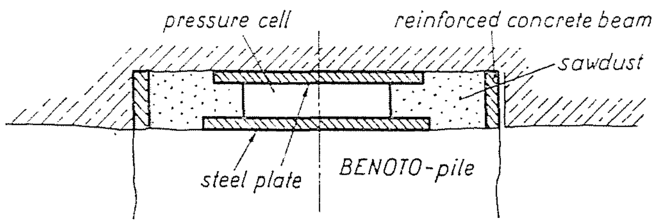

Fig. 9. Placing pressure cells between the beam and the pile head

Five piles have been chosen for the measurements. Their heads were specially constructed so that pressure cells could be placed without disturbing the reinforcement of the upper beam.

The cells were encased in timber.

The gap between the beam and piles was filled with sawdust that transfers no load. The separation aimed at loading the piles exclusively through the pressure cells.

After being calibrated in laboratory, the cells were placed on the pile heads and zeroed.

On September 7th, 1976 the construction work had been nearly finished, thus 90% of dead weight could be taken into account.

As on October 28th, 1976 the silo was already completed, the total weights have been calculated. Test results are shown in Table 2.

The lay-out and reference points are seen in Fig. 10.

Measurements proved the measured forces uniformly distributed on the piles ranging from 0.225 to 0.24 MN corresponding to the dead weight of silo, were in good agreement with calculated loads.

Pile No. 3		Pile No. 4		Pile No. 5	
Reading	Load MN	Reading	Load MN	Reading	Load MN
	0		0		0
−0.009	0.014	−0.003	0.005	−0.006	0.009
−0.176	0.252	−0.144	0.229	−0.140	0.228
−0.143	0.228	−0.150	0.24	−0.140	0.228

* Pressure cell damaged during construction work.

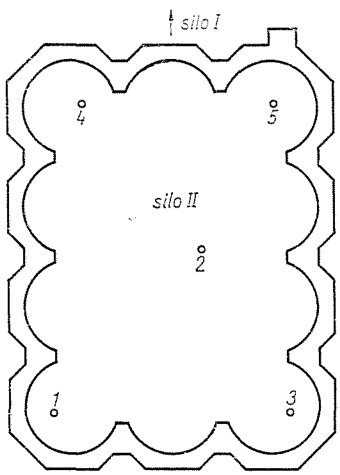

Fig. 10. Location of measuring points

Conclusions

Measurement results led to the conclusion that the large diameter piles investigated owed their load capacity mainly to skin friction, while the tip resistance was much lower than expected. This fact could be attributed to the soil conditions and to the method of construction of the piles.

Also in this case it became clear that the practice of applying a common safety factor for the skin friction and the tip resistance in the theoretical formula is wrong. It is advisable instead to take always the full value or at least 90 per cent of the skin friction and to specify the tip resistance as a function of the permissible settlement.

The extraordinary care in respecting piling specifications load capacity depends on should be particularly emphasized. It is very important to start concreting immediately after finishing the excavation of borehole, and to proceed continuously.

Measurements proved the weight of silo to be uniformly distributed on piles and the measured forces to correspond to the values calculated from the dead weight of building.

The final purpose of measurements was to investigate the effect of a sudden load increase in filling the silo. Up to now the silo has not been filled yet.

Summary

Subsidence of a silo structure on piles motivated to carry out Benoto pile tests to check skin friction and tip resistance. Specially designed pressure cells have been applied, showing pile load capacity to be mainly due to skin friction, while tip resistance was much lower than expected. Piling specifications must be carefully respected, and no overall safety factor can be applied.

References

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