VIBRATED VM-PILES (H-BEAMS WITH GROUT INJECTION). DESIGN, TESTING AND MONITORING

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This paper describes the first time use of Rüttel-Injectionspfahle in the Netherlands. These piles, that can be described as vibrated H-beams with grout injection along the shaft, were new to the Dutch foundation practise and had to be tested both prior to the installation (design investigation tests) and after installation (acceptance tests) in order to meet the demands of the building authority. The design of the piles was based on the results of static load tests and on the results of Cone Penetration Tests (CPT's) carried out at various stages of the works. The performance of the piles was validated by monitoring results that were obtained during construction. Subsequently the results were interpreted to increase the understanding of the behaviour of pile groups subject to tension forces.

INTRODUCTION

Part of the North/South line metro extension in Amsterdam, the Netherlands, is the realisation of the RAI - Europaplein station. This station is situated in front of the RAI complex, the most important exhibition and congress centre of the Netherlands. The metro tunnel and station are being constructed in a building pit with sheet piling, underwater excavation and underwater concrete. As a result of the high groundwater levels tension piles are necessary during construction. Once the station is completed a number of piles will remain in tension, while others become compression piles.

The station floor is situated at 11m below ground level. The design pile loads vary from 1100kN compression load to 700kN tension load. As a result pile tip levels varied from 28 to 31m below ground level, approximately 20m below the bottom of the excavation.

Since a stiff load-settlement behaviour of the piles was required by the design, both in tension and compression, and the impact on the RAI congress centre should be as minimal as possible, the client decided to use Rüttel-Injectionspfahle which are commonly used in parts of Germany. These piles, that can be described as vibrated H-beams with grout injection along the shaft, were new to Dutch foundation practise and had to be tested both prior to the installation (design investigation tests) and after installation (acceptance tests) in order to meet the demands of the building authority.

Cone Penetration Tests (CPT's) were carried out before and after excavation and again after installation of the piles. The piles where installed form a piling rig that was placed on a bridge situated on top of the sheet piles (fig. 1). Six piles were monitored with strain gauges at different locations along their shaft. The continuous monitoring started directly after the underwater concrete was in place and lasted until most of the inner structure of the station was completed.

Figure 1: Piling rig

SOIL CONDITIONS

The sub-surface at the building location consist of approximately 5m of made ground, predominantly fine sands, overlaying Holocene clay, peat and sand layers with a thickness of approximately 20m. At this depth a Pleistocene sand layer with a thickness of more than 80m is encountered. The cone resistance in these sand layer vary from 15MPa (medium dense sands) to over 40MPa (dense to very dense sands).

A typical CPT result is shown in figure 2.

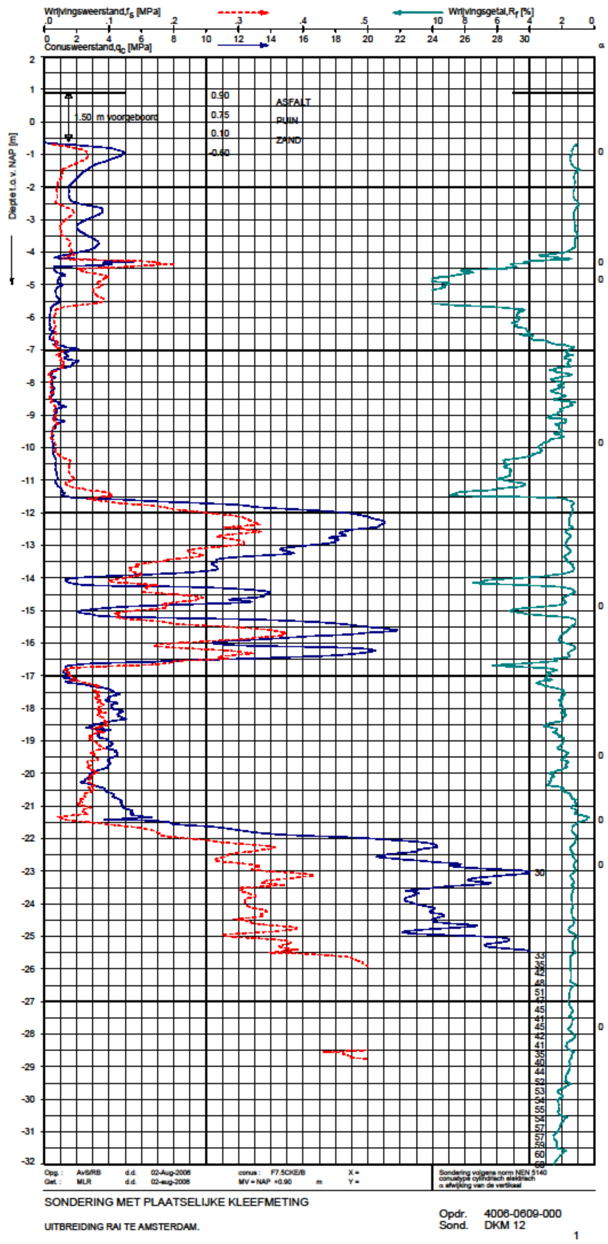

Figure 2: Typical CPT at site

DESIGN INVESTIGATION TESTS

In order to determine the geotechnical pile characteristics, three test piles were constructed after which a test load (design investigation test) was executed. Because the piles are subject to both tension and compression loads, the tests were executed to suite both load types and were carried out according to the Dutch standard for pile tests (NEN 6745).

Execution

The test piles consisted of HE-B 240 profiles with a length of approximately 35m. Alongside the pile a HDPE-pipe was assembled through which the grout was pumped. The pile toe is a special construction to accommodate the outflow of the grout. To measure the pile characteristics the piles were equipped with strain gauges. These were installed in an extra pipe, that post installation of the strain gauges was grouted. In order to maximise the accuracy of the transformation of results to the pile the extra pipe was welded to the Rüttel-Injection pile. Five strain gauges were installed per pile.

The reaction forces of the tension piles were derived through two coupled steel profiles into the ground next to reaction piles. The reaction forces of the compression piles are led directly into the reaction piles. The forces were set up with four hydraulic cylinders and measured with load cells, both analogue and digital. The displacement of the pile head was also measured by means of two digital and one analogue device. Every apparatus was calibrated or controlled to reduce the chance of defective occurrences.

The load scheme of the tension tests was identical to the load scheme of the compression tests. The load is applied in steps of 500kN with a minimum duration of one hour. When the maximum load of 3000kN was reached the load was decreased with the same steps of 500kN.

The measured data are:

- digital values of the forces and movements of the piles, every 2 seconds,
- observations of the strain gauges,
- observations of the analogous displacements of the pile head,
- penetration rate of the piles (Figure 3)

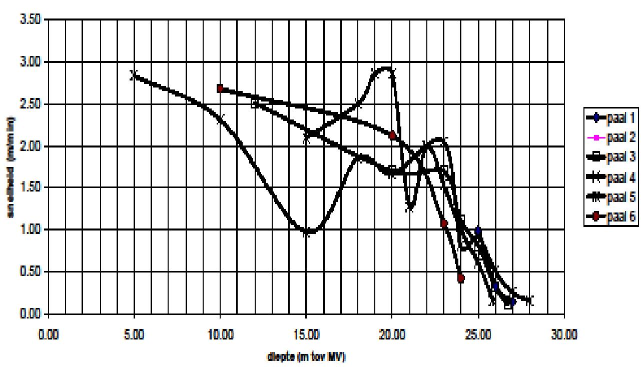

Figure 3: Penetration rate testing piles

Every test load, with maximal load of 3000kN, complied with a maximum allowable creep of < 0.2mm/hour.

INSTALLATION OF THE PILES

A total of approximately 1000 Rüttel-Injection piles were installed. Before interpreting the data obtained from the monitored piles and the multiple CPT's, a better understanding of the pile system is desirable. This paragraph will discuss the installation process and the applied equipment.

Installation process

The working order of the construction of a Rüttel-Injection pile is as follows:

1. Assembling of pipes and cables on the pile,
2. Lifting the pile with an assisting crane and placing into in the travertine. (The lifting is accomplished making use of a grommet which is secured by welded measures on the pile head),
3. Placing the vibrator on the pile and locking of the pile guide,
4. Connecting the pipe on the grouting pump,
5. Filling of the store tank with approximately 400 litres grout,
6. Positioning of the pile,
7. Vibrating the pile while simultaneously injecting with grout,
8. After certain metres opening of the pile guide,
9. Vibrating of the pile from height,
10. Pulling off and cutting the pipe at pile head level,
11. Proceeding to the next pile.

Equipment

The equipment used for the work was:

- Scaffold:
  Sennebogen 6100 HD on travertine,
- Assisting crane:
  Hitachi KH 180,
- Vibrator:
  ICE 36 RFts

MONITORING OF THE PILES

During the execution of the metro station six piles were monitored, three in the middle of the pile group and three near the side. The objective was to obtain understanding of the distribution of force along the shaft of the pile during various building stages. The measurements started after the installation of the piles was completed and the underwater concrete was in place, but before dewatering of the building pit. There were a few adjustments made compared to the investigation tests.

1. The design of the pile toe was optimised, the construction on behalf of the outflow of the grout had been modified,
2. Pile tip level varies from 28m to 32m below NAP, pile lengths from 17m to 21m.
3. The piles were constructed in a water filled building pit (bottom level NAP -11m) in contrast to the investigation tests where the piles where installed from ground level.

Instrumentation

The strain gauges chosen were according to the Vibrating Wire principle, also an adjustment in comparison with the test piles. The choice was determined on local circumstances, e.g. a wet environment, variation in temperatures, long cable lengths and a long monitoring period. From experiences the "VW Spotweldable Strain Gauge" was chosen with a "mid range", to monitor both tension and compression forces.

Before the installation of the strain gauges the surface was prepared. This was done by removing the wax layer after which the protection partitions are placed. Next the strain gauges are fixed with a point welder at 7 to 9 different spots along the pile shaft. Subsequently the sensors were provided with a waterproof coating and a measure log. The steel protection partitions were to prevent stripping the resin fixing (by pushing the soil aside). Both sensors and cables were poured in a resin fixing of polyurethane.

The strain sensors were calibrated with "batch calibration", the measurements in Hertz can be converted into micro-strain. The tension loads were calibrated to 700kN.

To simplify the installation of the piles no extra cable length was supplied to the piles. Therefore special underwater connectors were used instead. After installation of the piles the cables were guided over the building pit floor alongside the sheet piles to the upper side of the building pit from where the cables were lead to a central data logger unit.

Results

The following results are measured:

- Forces along the shaft of the pile,
- Elevation of the underwater concrete floor
- Elevation of the sheet piling

The most important objective was to obtain understanding of the distribution of force along the shaft of the pile. Figure 4 shows the forces acting on different spots along the pile shaft. Six different figures of this kind are made.

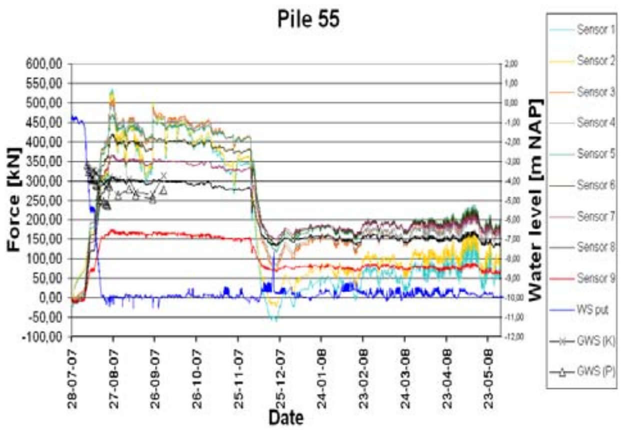

Figure 4: Forces along the shaft

From these figures the maximum tension force can be found. The values are shown in table 1.

Table 1: Maximum tension forces

Pile nr. [-]	Middle / side building pit	Maximum tension force [kN]
52	Side	390
55	Middle	540
122	Side	370
125	Middle	515
282	Side	345
284	Middle	500

- The influence of the temperature on the forces of the piles

The forces in the tension piles show a clear waving behaviour (after the dewatering of the pit), which corresponds with day and night rhythm. During monitoring the temperature in the logger unit was measured. The result is shown in figure 5. The temperature is only showing a trend, not the actual outside temperature.

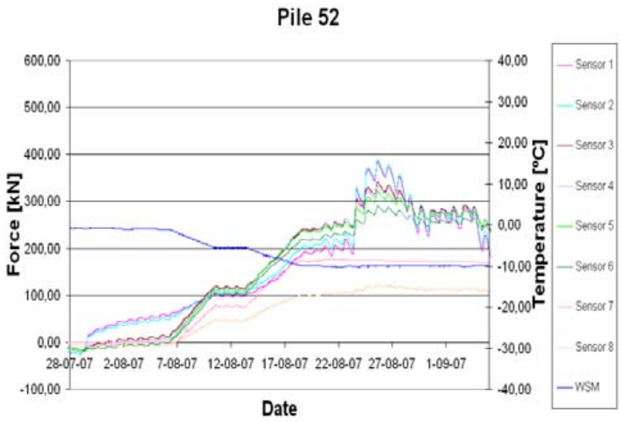

Figure 5: Relation temperature and pile forces

- Drill depth, drill duration, pile installation speed, grout discharge and grout pressure,

To control the grouting process the above mentioned variables were measured during execution. Table 2 presents the measured quantities.

Table 2: Variables during execution

Pile nr. [-]	Instal-lation depth [m]	Installa-tion time [min:sec]	Speed of installation [m/h]	Grout discharge [l/min]]	Grout pressure [bar]
52	18.17	8:03	Ca. 125	36.00	Ca. 2.5
55	18.97	8:00	Ca. 125	38.75	Ca. 2.5
122	15.77	7:02	Ca. 125	38.39	Ca. 2.5
125	18.27	8:17	Ca. 125	33.80	Ca. 2.5
282	17.57	7:55	Ca. 125	37.89	Ca. 2.5
284	17.48	7:40	Ca. 125	37.83	Ca. 2.5

- Rising underwater concrete,

During the dewatering of the building pit the heaving of the underwater concrete 'raft' was measured. The rising of the raft equals the rising of the pile heads, the arching effects of the concrete were limited since the pile to pile distance was limited to 2.8m and the concrete had a thickness of approximately 1m. At loads that varied from 240 to 390kN per pile, a heave of approximately 1 to 4mm occurred.

- Water level in the surroundings.

Finally the water level near the building site was monitored. The blue line in figure 6 is the water level inside the building pit and the green and red line are water levels outside the building pit.

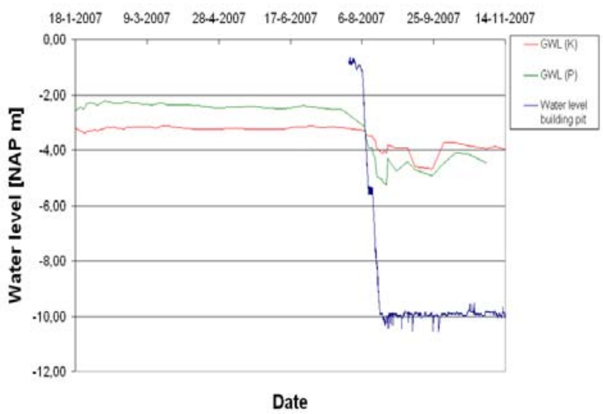

Figure 6: Water levels

Interpretation

The monitoring results have been interpreted in relation to the actual and calculated loads in the piles, the load distribution along the pile shafts and the distribution of the loads in the pile group.

- Actual maximum tension loads compared to calculated maximum tension loads.

By means of a simple calculation the maximum tension loads can be derived. Vertical equilibrium is reached when the water pressure at the bottom of the underwater concrete equals the weight of the underwater concrete and the reaction forces developed by the tension piles. The thickness of the under water concrete was, according to normal tolerances, 1m ±0.1m. Table 3 shows the calculated values next to the measured values.

Table 3: Calculated and measured maximum tension loads

Pile nr. [-]	Calculated maximum tension load (Rep. value) [kN]	Measured maximum tension load [kN]
52	390	390
55	539	540
122	390	370
125	460	515
282	398	345
284	467	500

The differences between the calculated and measured loads are minimal and can be explained taken the tolerances of the underwater concrete into account.

- Load distribution

Along the shaft of the tension piles different spots were measured. Figure 7 shows the distribution of the pile loads along the shafts of the 6 piles that were instrumented.

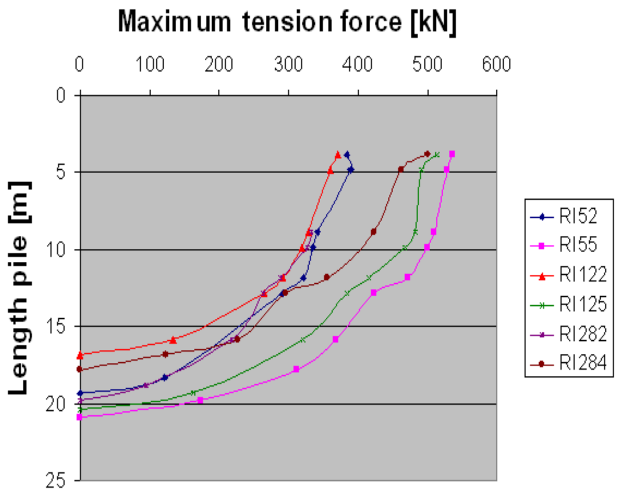

Figure 7: Load distribution in the piles

The distribution of the load along the shaft in percentages is shown in figure 8.

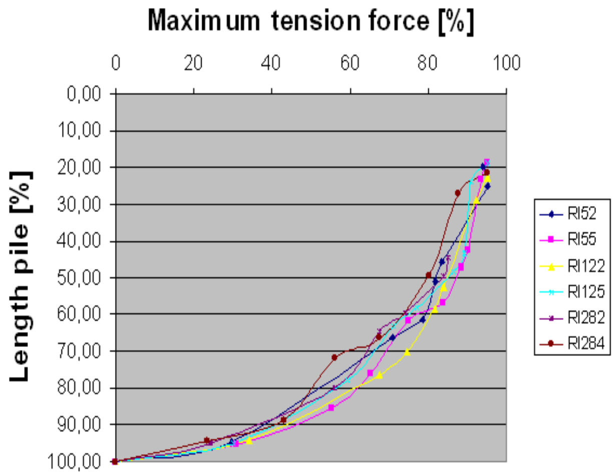

Figure 8: Load distribution in the piles in percentages of the maximum load

One can see that in the top section of the piles the tension load equals the maximum load. Nearly 50% of the tension load is still present in the remaining 20% of the piles just above the pile tip level. This can be explained in relation to the soil conditions, since the soil layers responsible for the bearing capacity are situated in the lower parts of the piles.

- Pile group effect

Since the piles were monitored in the actual building pit it is possible to determine the influence of the pile group on the bearing capacity of the piles.

The bearing capacity of a single vibrated VM-pile depends largely on the load transfer capability of the grout cover. This capability is a function of the grout properties, the (over)pressure under which the grout layer is formed and the local soil conditions. An estimate of the bearing capacity of a single pile can therefore be calculated using the known grout pressure. For this estimate it is assumed that the friction is maximal on the separation surface between grout and soil.

The grout pressure (with $\gamma_{g;d}$ = 22 kN/m^3) at depth h is:

$$\sigma_g = \gamma_{g;d} \cdot h \qquad (1)$$

And the hydrostatic water pressure at depth h is:

$$\sigma_h = \gamma_w \cdot h_1 \qquad (2)$$

The difference between the pressures is considered to be the effective pressure between grout and soil. From here the maximum shear stress for depth h is:

$$\tau_{max} = (\sigma_g - \sigma_w) \cdot \tan(\varphi'_d) \qquad (3)$$

Where:

φ'_d is the design value of the internal friction angle in °

The maximum tension load is equal to the sum of the maximum shear stresses along the shaft of the grouted pile.

Table 4 presents the calculated single pile capacities, together with the design value of the pile in a pile group. The design value in this particular project (load conditions) is 1.33 times the characteristic value. The measured loads in the piles matched these characteristic values.

Table 4: Calculated design values

Pile nr. [-]	Calculated single pile, design value [kN]	Calculated pile in group, characteristic value [kN]	Calculated pile in group, design value [kN]
52	1063	390	520
55	1257	539	840
122	772	390	520
125	1191	460	824
282	1126	398	530
284	883	467	754

The Dutch design guideline for tension piles, CUR2001-4, calculates the maximum tension force based on the measured cone resistance and a pile factor α_t, which depends largely on the installation process of the pile type. For driven precast piles a typical value of α_t = 0.007. For "standard" driven VM-piles this factor = 0.012. The rule for a tension pile is formulated as follows:

$$F_{r;trek;d} = \int_0^L (O_{p;gem} \cdot p_{r;z;d})dz \qquad (4)$$

The shaft friction ($p_{r;z;d}$) for single piles is formulated as:

$$p_{r;z;d} = \alpha_t \cdot q_{c;z;d} \qquad (5)$$

Applying this formula to pile groups two factors (f_1 and f_2) are introduced to take the compression of the soil due to installation, respectively the relaxation due to the tension force of the pile group, into account. The shaft friction is then formulated as follows.

$$p_{r;z;d} = \alpha_t \cdot q_{c;z;d} \cdot f_1 \cdot f_2 \qquad (6)$$

The factor f_1 is negligible if the distance between the piles is larger then 7 times the equivalent diameter, as is the fact for this project.

Using the results of the CPT's executed after excavation the α_t can be determined with the results of the calculation based on grouting pressure. The determination is shown in table 5.

Table 5: Determination α_t

Pile nr. [-]	Calculated single pile, cone resistance, design value [kN]	α_t
52	1063	0.0122
55	1257	0.0125
122	772	0.0118
125	1191	0.0123
282	1126	0.0122
284	883	0.0119

The average α_t factor is 0.012, which is the same as for driven VM-piles.
With the α_t the pile in a pile group can be determined. This has to be compared with the measured value to determine the group effect f_2. Only the middle piles are used because these piles receive the highest loads.

Table 6: Determination f_2 with formula 6

Pile nr. [-]	Pile toe depth [m]	Measured single pile, cone resistance, design value (char. value*1.33) [kN]	f_2
55	-30.0	718	0.60
125	-29.5	685	0.60
284	-27.0	665	0.76

One has to keep in mind that the factor f_2 is not constant in relation to the depth. Deeper along the pile shaft there is more pressure reduction due to the influence of the reduction in the above layers. Factor f_2 behaves as in figure 9. The points are calculated f_2's regarding the CUR 2001-4. The mean value of the measured f_2 values (0.65) is visualized by the vertical line.

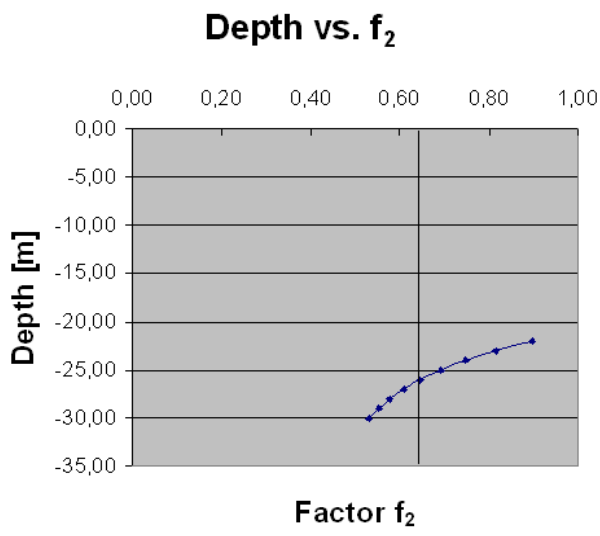

Figure 9: Development f_2 to the depth

In this case the group effect due to the tension loads on the pile group (overlapping of the influence area) is a reduction of the tensile pile capability of approximately 35%. It can be concluded that for this project the reduction based on the CUR 2001-4 guideline is in compliance with the monitoring results.

CONE PENETRATION TESTS

During various building stages of the building pit several CPT's were executed. The different cone resistances are shown in figure 10. Two clear phenomena can be discovered from the data.

1. Reduction of the cone resistance in the first sandy layer due to excavating soil above the layer,
2. Reduction of the cone resistance in the deep sandy layers due to loosening of the dense soil by pile installation, this includes the reduction of the horizontal coefficient of soil pressure to a coefficient occurring in normal consolidated soil ($K_0 = 1 - \sin(\varphi')$).

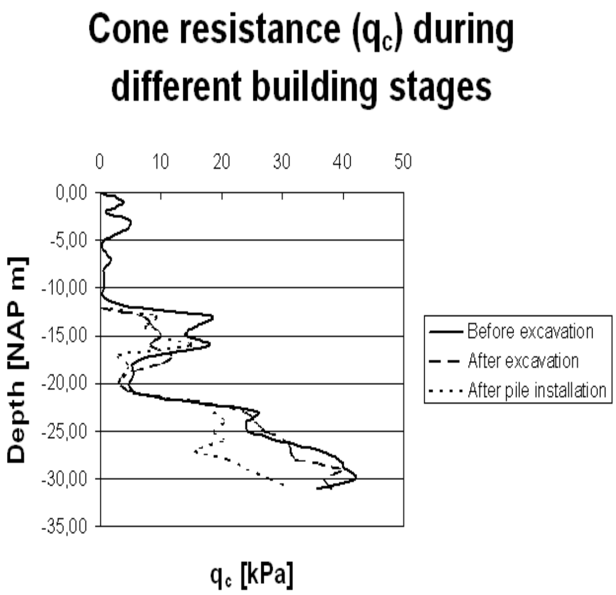

Figure 10: Cone Penetration Tests

CONCLUSIONS

Normally in the Netherlands pile loads are not monitored during construction. The design of the piles is based on CPT results and pile tests are only deemed necessary to determine pile characteristics for new piling systems. Design investigation tests are mostly being performed under special site conditions (test piles). The disadvantage of such tests could be either an under- or over estimate of the actual pile behaviour of production piles (due to different circumstances during execution, different ground levels and the lack of the group effects). This article creates more insight in these mechanisms which also have to taken into account during the design. The analyses of the mechanisms are done for typical Dutch soils.

Designing tension piles is a difficult process because many variables are unknown. To avoid uncertainties safe values are presumed for every variable. Validating the design with monitored piles during construction can lead to design rules / standards with less uncertainty.

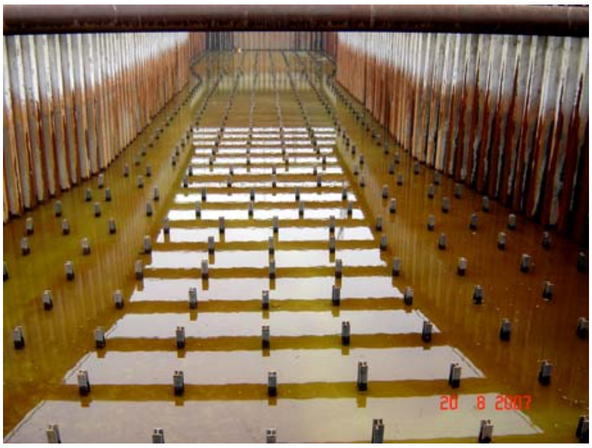

Figure 11: The building after almost complete dewatering

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