LATERALLY LOADED PILES IN AN EMBANKMENT OF DRY SAND

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ABSTRACT

This article deals with the foundation design and construction of a sound barrier system for the Noord-Zuid subway line in Amsterdam. A foundation on open ended steel tubular piles had to be applied, resulting from high lateral loads due to the possible impact of crashing vehicles. An extensive soil investigation program was executed which consisted (a.o.) of Cone Pressure Meter tests. CPM tests were used to improve the lateral pile capacity calculations which resulted in a more economic design. Plaxis calculations were performed to analyze the lateral pile behavior close to slopes.
Steel tubular piles were installed by means of vibro driving to minimize construction noise. Installation proved to be difficult, as the piles had to be driven through very dense sand layers of an existing road embankment. On certain locations, full soil plugging during driving within the piles was observed, which is quite unusual for large tubular piles.

Keywords: sound barrier, piles, lateral, pressiometer, Plaxis, drivability

INTRODUCTION

In crowded urban areas in the Netherlands, the extended use of roads and railroads increases the noise level for the environment. The installation of sound barrier systems plays a vital role in (new) infrastructure projects and causes significant and necessary noise reduction for residents.

One of the finishing phases of the Noord-Zuid subway line in Amsterdam is the construction of a sound barrier system. This article describes the design and the construction of the foundation of a sound barrier system in the Northern part of Amsterdam, with a total length of approx. 2.6 km along the Nieuwe Leeuwarderweg (NLW) between highway A10 North and the Buiksloterdijk. The project location is presented in Fig 1. Sound barrier height varies from 1.0 m to 8.3 m. The red line in the figure shows the location with the highest barriers.

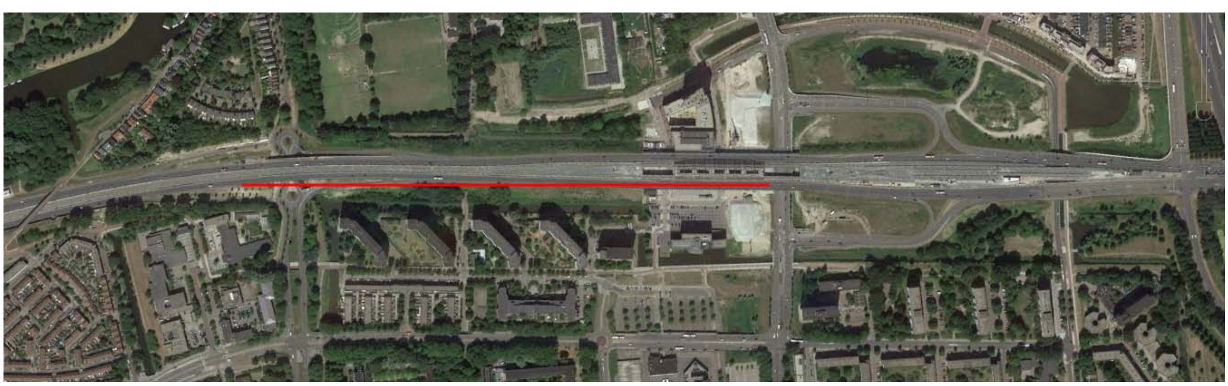

Fig. 1: location of proposed sound barrier system along the Nieuwe Leeuwarderweg. In red the location with the highest barrier (source: Google Earth)

The NLW was constructed in the year 1968 and consists over a substantial distance of a 10 m high embankment of sand. Several additions to the embankment have been constructed in the year 2005. The sound barriers are placed on top of this embankment. Further to the south of the project, the NLW has a deeper level and is constructed between sheet piles. The sound barrier will be placed directly on the sheet piles on these locations. This article focusses on the 8.3 m high barriers on the high embankments.

FOUNDATION SYSTEM

The sound barrier has a double function in this project: apart from the proposed noise reduction for the surrounding residential buildings and schools, the structure also functions as a traffic barrier. One of the client demands was that the complete system was able to resist lateral loads due to vehicles crashing into the barrier. As a result of this, high lateral loads and bending moments are acting on the foundation. The lateral loads combined with considerable axial loads and possible negative skin friction (drag down forces) caused by residual settlements of the NLW embankment requires a special foundation solution. The most economical solution was found in open ended steel tubular piles. Big advantage of this system is the application of a concrete plug in the top of the pile, where a structural connection between superstructure and pile is made. Due to the stringent client demands with respect to construction noise, it was decided to install the piles by means of vibro driving with an High Frequency, Variable Moment (HF-VM), vibro hammer. The foundation system is schematically shown in Fig. 2. Pile diameter varied due to the height and location of the sound barrier. Minimum pile diameter was 508 mm with a wall thickness of 7 mm. Maximum pile diameter was 914 mm with a wall thickness of 9 mm (slenderness D/t approx. 100). This article focusses on the large pile diameters.

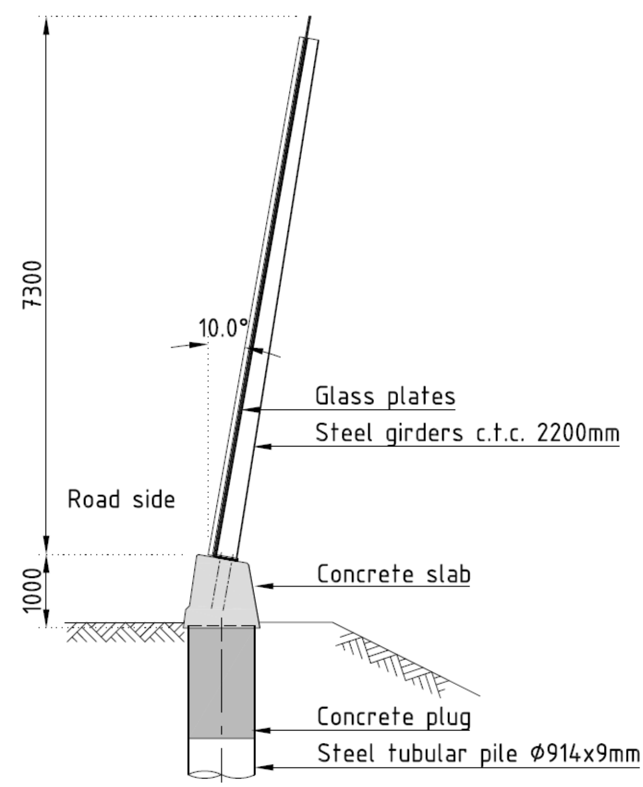

Fig. 2: Foundation system with open ended steel tubular piles

SOIL INVESTIGATION

Soil investigation programmes were carried out in different stages of the project and consisted in total of approx. 80 Cone Penetration Tests (CPT), 2 Cone Pressio Meter (CPM) tests with 4 tests per location, 4 soil borings and several laboratory tests. The laboratory program consisted of determination of unit

weight and water content, Consolidated Undrained Triaxial testing and oedometer testing on selected cohesive samples.
Typical spacing between test locations varied from 25 to 50 m, depending on observed variability of soil conditions.

The soil conditions are dominated by the existing NLW embankment which totally consists of sand material. Underneath, the typical Amsterdam soil conditions can be found, consisting of soft holocene layers (marine clay and the so called 'wadzand' formation), underlain by the 1st and 2nd sand layer. A thin layer of clayey sand (Alleröd) divides these sand layers. A typical profile is given in table 1.

Table 1. Typical Soil Profile

Layer no.	Depth [m below GL] From	to	Material
1	0.0	11.0	Sand, dense to very dense (old embankment)
2	11.0	12.0	Marine clay, firm
3	12.0	16.0	Sand and Silt (wadzand formation)
4	16.0	18.0	Sand, medium dense, silty
5	18.0	20.5	Sand, dense
6	20.5	21.0	Sand, clayey (Alleröd)
7	21.0	--	Sand, very dense

Ground water level is at approx. 10.0 m below ground level. Pile tip levels were chosen in the 1st sand layer with pile lengths up to 20 m. Interaction calculations were performed to determine the deepest level where negative skin friction would occur.

Lateral pile behavior was analyzed in an elasto-plastic model based on the theory of a beam supported by springs. The elastic behavior of the soil is usually described by means of the subgrade modulus K_h according to the theory of Ménard (1963). For piles with a radius larger than 300 mm, this equation is given by:

$$\frac{1}{K_h} = \frac{1}{E_m} \cdot \left[1.3 \cdot R_0 \cdot \left(\frac{2.65 \cdot R}{R_0}\right)^{\alpha} + \alpha \cdot R\right] \qquad [1]$$

Where E_m = Ménard modulus, R_0 = reference pile radius (300 mm), R = pile radius and α = rheological coefficient.

The Ménard modulus is usually derived from the CPT cone resistance q_c. For sand, typical E_m values are 0.7 q_c to 1.0 q_c. In clays, this conversion factor varies between 2.0 to 3.0. This approach leads to safe but sometimes unrealistic low values. For this project it was decided to perform Cone Pressure Meter tests (CPM) to derive more realistic Ménard moduli enabling the project team to optimize the foundation design.

Pressuremeters are devices for carrying out in situ testing of soils and rocks for strength and stiffness parameters. They are generally cylindrical, long with respect to their diameter, part of this length being covered by a flexible membrane. The CPM is a combination of a normal CPT and a 'standard' pressuremeter and is entered into the ground by pushing.
Once in the ground, increments of pressure are applied to the inside of the membrane forcing it to press against the material and so loading a cylindrical cavity. A test consists of a series of readings of pressure and the consequent displacement of the cavity wall and the loading curve so obtained may be analyzed using standard solutions for cylindrical cavity expansion and contraction. Typical CPM results for this project are presented in Fig. 3.

From the CPM tests, the parameters G_{ini} (initial shear modulus), G_{ur} (unloading / reloading shear modulus) and c_u (undrained shear strength, cohesive soils only) can be derived. The Ménard modulus E_m can be derived from the shear moduli using the common relationship based on the Poisson's ratio ν. For this project, it was decided to use the initial shear modulus, derived from the initial loading curve, as input for the lateral pile calculations. For short term loading of crashing vehicles a 'dynamic' factor of 3.0 was applied to the static moduli, representing the stiffer soil behavior under very short loading conditions.

The sand material in the embankment has the largest contribution to the lateral stiffness of the foundation system, but also the marine clays at a lower level were tested. Elaboration of test results was performed according to the theory of Bolton (1999). The results are summarized in Table 2. Applied correlation factors are 1.0 for sands and 3.0 for clays, being the highest recommended factors.

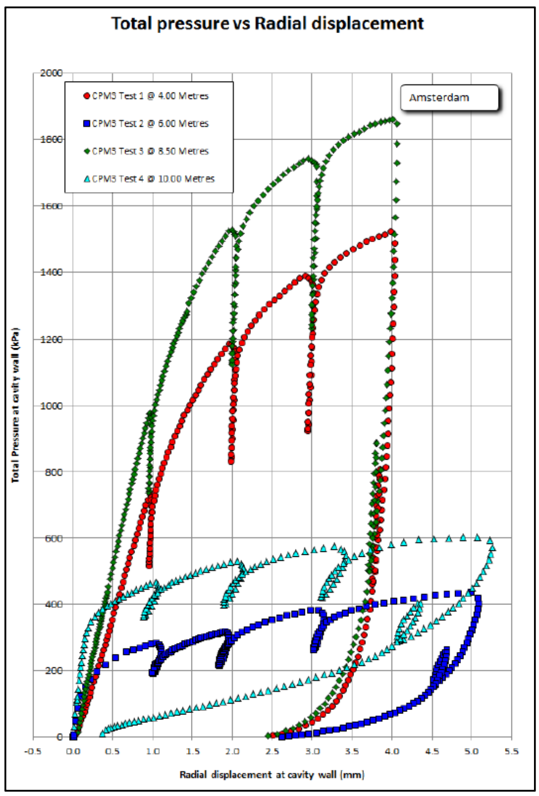

Fig. 3: CPM test results

Table 2. Ménard Moduli from CPM tests and correlation with cone resistance

Test no.	Depth [m]	Material	Cone resistance [MPa]	$E_{m,ini}$ (CPM) [MPa]	E_{ur} (CPM) [MPa]	E_m (correlation) [MPa]
CPM95-102	2.0	Sand	7.5	26.0	167.1	7.5
	3.5	Sand	20.0	24.4	144.7	20.0
	5.5	Clay	0.5	13.8	13.7	2.0
	7.5	Clay	0.5	12.0	38.8	2.0
CPM84-103	4.0	Sand	15.0	27.0	168.4	15.0
	6.0	Clay	0.5	3.4	19.9	2.0
	8.0	Sand	13.0	34.1	193.6	13.0
	9.5	Clay	1.0	12.0	13.3	2.0

The results show that large differences occur between the moduli based on correlation with cone resistance and moduli from CPM tests, even when the highest correlation factors are used. For the soils considered here, it is very conservative to use the ‘standard’ textbook correlation factors, especially lower or mean factors. Equation [1] shows that the relation between Ménard modulus and subgrade modulus is linear. A conservative estimate of the Ménard modulus will therefore lead to a very low value of the subgrade modulus and subsequent an unnecessarily large pile diameter and/or wall thickness. The investment of additional soil investigation by means of CPM testing pays off in a later phase of the project by means of a significant reduction on steel mass.

FINITE ELEMENT METHODS

The NLW embankment slope has an angle of approx. 27 degrees with the horizontal plane (2H/1V). This slope has to produce the passive resistance for piles subject to lateral wind forces and/or lateral forces from the impact of vehicles. A typical cross section with the NLW slope and the sound barrier system is given in Fig. 4. Piles are situated just near the slope. A water pond is present near the toe of this slope, which further reduces the soil resistance.

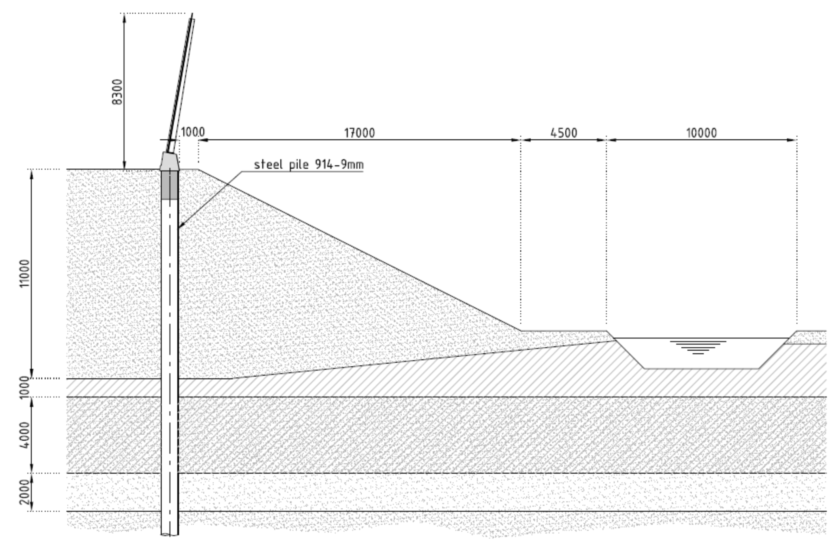

Fig. 4: sound barrier near slope of NLW embankment

Most computer programs based on spring supported beams have a tendency to overpredict the passive soil resistance of soil with steep slopes. Manual adjustments to this resistance are therefore often required, ranging from a reduction factor based on engineering judgement to totally neglecting the lateral resistance over the slope.

To get a closer insight in the influence of the slope on the passive resistance for this specific load case, Finite Element Method calculations were carried out with program Plaxis 2D. For comparison, calculations were carried out with a horizontal ground level and with a slope as given in Fig. 4. The differences in pile deflections and bending moments between the two sets of calculations were substantially high. Piles were first modelled as so called ‘embedded piles’, which simulates discrete elements at a certain centre to centre distance in a 2D plane. For comparison, the piles were also modelled as continous plates, where pile stiffness is corrected for the real centre to centre distance.

The results of the two sets of calculations are shown in table 3. An example of the Plaxis output for a pile near a slope is shown in figure 5.

Table 3. Results Plaxis calculations

Run no.	Situation	Pile condition	Maximum bending moment [kNm]	Deflection top of pile [mm]
1	Horizontal ground level	Embedded pile	149	15.5
2	Slope with water pond	Embedded pile	134	48.0
3	Horizontal ground level	Plate	112	3.5
4	Slope with water pond	Plate	115	14.7

The differences between the pile head deflection for a horizontal ground level and the 'slope with water pond' condition is remarkable high. Both approaches show that a reduction of approx. 70% of soil resistance applies to the 'slope with water pond' condition.

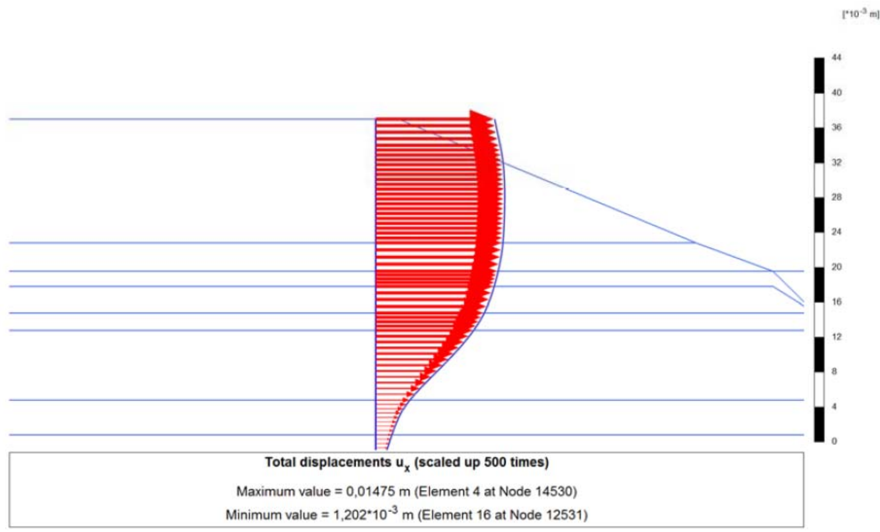

Fig. 5: Plaxis output for pile near slope

It was finally decided to perform all other calculations for large slopes with a spring based model and a horizontal ground level and an efficiency factor of 0.3 on the soil parameters. This means a reduction of 70% of the passive soil resistance over the height of the slope.

PILE DRIVING

As stated before, steel piles had to be installed by means of high frequency vibro driving. Piles were maximum 20 m long with a slenderness D/t of about 100. Drivability assessments had shown that soil resistance was relative high, due to the presence of thick layers of well compacted dry sand in the NLW embankment, but piles could be installed with a vibro hammer with an centrifugal force of approx. 1600 kN. In all assessments, coring behavior was assumed, where Soil Resistance during Driving (SRD) consists of inner and outer wall friction and tip resistance at the pile rim. This is common practice when driving open ended tubular piles with diameters larger than 600 mm. (Full) plugging during pile driving is hardly observed with piles of this diameter and was therefore not modelled.

The process of vibro pile driving on top of the embankment is shown in Fig. 6. During execution of the works, the SRD was rapidly increasing with increasing height of the embankment.

The sudden increase in SRD can be explained by Fig. 6., which shows the calculated SRD as a function of depth below Ground Level. The difference between coring SRD (inner and outer wall friction, dotted line) and plugging SRD (outer friction and full end bearing, striped line) is clearly visible. Plugging will lead to a drastic increase in SRD and therefore reduced pile advancement.

Fig. 6: Vibro driving of steel piles on top of the embankment

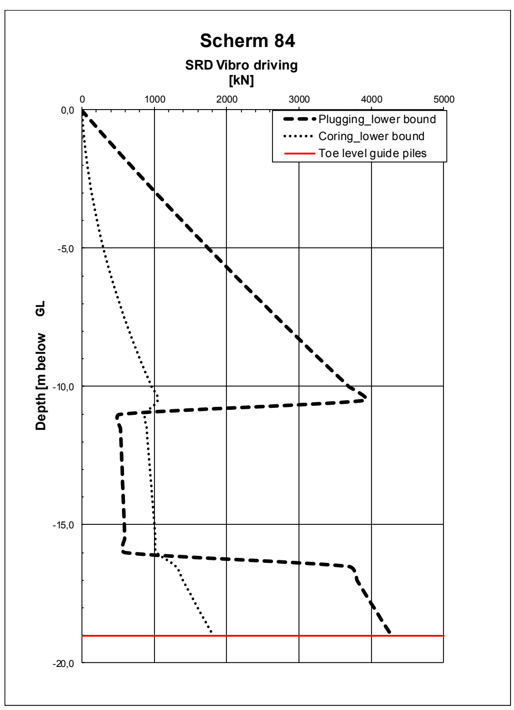

Fig. 7: difference between coring SRD and plugging SRD

At certain locations, obstacles (concrete lumps, stones, etc.) were found at a deep level, causing early refusal. At these locations it was required to move the position of the pile. At some other locations, it was observed that the soil level inside the pile after driving was up to 9 m lower. Naturally, this was observed relatively late in the pile driving process. Obviously, soil plugging had occurred within the pile, causing a severe increase in SRD which resulted in damage to the pile head due to the massive energy transfer to the pile without sufficient pile penetration. The observed soil level reduction and damage to the pile head are shown in figure 8.

Fig. 8: soil plugging within a pile and pile top damage due to extensive vibro driving

Several mitigation measures were tried, such as the use of additional counter weights, the use of a larger vibro hammer, lubrication by means of adding small amounts of water inside and outside of the pile wall. Due to the strict client demands with respect to noise and vibration, pile installation by means of impact driving was not allowed.
Final solution was found in relief drilling by means of augering. The pile plug was removed to a level of approx. 6 m above pile toe level. The inner skin friction was decreased to acceptable values and piles could be driven to the design level.

CONCLUSIONS

- The use of tubular piles have proven to be a reliable and economic foundation type, especially when dealing with large lateral forces.
- The presence of slopes close to the pile location influences the lateral behavior of piles in a serious way, especially with high and steep slopes. To avoid uneconomic designs, it is highly recommended to perform Finite Element Analyses to get a better insight in the exact lateral pile behavior rather than use standard solutions.
- CPM testing can be beneficial in obtaining more reliable values of Ménard moduli and subsequent horizontal subgrade moduli, rather than using a standard correlation with cone resistance; the investment of this detailed soil investigation pays of further in the project because of the savings in pile diameter and/or wall thickness of the steel piles.
- (vibro) pile driving in dry sands may lead to early plugging behavior during driving, even with large pile diameters. This effect can lead to a sudden and dramatic increase in soil resistance during driving resulting in refusal and/or pile damage. Detailed drivability analyses with plugging behavior are recommended in these cases.

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