The smaller the plot the larger the problems: inadequacies of site investigation in the Hague

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Abstract: The standard site investigation (SI) undertaken in the Netherlands for a small plot intended for a house, in the centre of The Hague, would consist of two cone penetration tests (for foundations) and two auger boreholes (for contaminated ground). Small sites may not be suited to all foundation solutions owing to varying manoeuvrability and size of piling rigs. In a vacant lot of 5m by 10m only a hollow steel tube-piling rig could access the site. The contractor chose this rig in preference to the design-recommended driven concrete piles, which required a much larger rig. The insertion of the steel tubes caused severe vibrations and subsidence of neighbouring structures to such a degree that the municipal inspector halted the work.

The question arose: could this have been predicted? New buildings occupying sites nearby with similar ground conditions appear to have been successfully built. Extra 'site investigation' in the municipal archives showed that, at very little cost, information on the subsurface could be obtained. Further information was available concerning foundation proposals for new buildings and measures undertaken to protect existing neighbouring buildings.

One could argue that the municipality should have warned the architect about these measures and demanded they be given due consideration. Such a situation of municipal involvement happened in Dubai 15 years previously where the municipality questioned the results of a site investigation as a result of comparing them with earlier nearby investigations. Possibly such a precedent cannot be a function of all municipalities. The SI standard should be extended to include information within a 100m radius of the site relating to site investigation and foundations.

Résumé: La reconnaissance de terrain traditionnellement entreprise aux Pays-Bas pour une petite parcelle de terre destinée à une maison au centre de La Haye, consiste de 2 profiles pénétrométriques (pour les fondations) et de 2 forages à la tarière (pour la détection de sols contaminés). Les petits sites ne sont pas favorables à tout type de construction vu la taille des machines à enfoncer les pieux et leur marge de manœuvre. Dans le cas considéré, une parcelle de 5m par 10m, seulement une machine pour enfoncer des tubes d'acier creux pouvait accéder le site où une maison se trouvait déjà. L'entrepreneur a préfère cette solution aux pieux de béton battus préconisés par le consultant géotechnique. L'insertion des tubes creux a engendré des vibrations et des tassements de terrains dans les propriétés avoisinantes tels que la municipalité a arrêté les travaux.

Une question s'est posé: est-ce-que cela aurait pu être évité? De nouvelles bâtisses semblaient avoir été construites avec succès sur des sites apparemment similaires. Une recherche dans les archives municipales a montré, qu'à petits coûts, des informations supplémentaires sur le sous-sol pouvaient être obtenues. D'autres informations étaient disponibles sur les propositions de fondations de nouveaux bâtiments et sur les mesures de protection à prendre pour protéger les propriétés avoisinantes.

On peut argumenter que la municipalité aurait du avertir l'architecte des précautions à prendre et demander à ce qu'elles soient considérées. Une telle situation avait eu lieu 15 ans auparavant à Dubaï. La municipalité avait questionné les résultats d'une reconnaissance de terrain par comparaison avec une reconnaissance menée préalablement sur un site voisin. Un tel précédent ne peut probablement pas être une fonction de toute municipalité. La reconnaissance de terrain standard devrait inclure toute information disponible sur le sous-sol et les fondations dans un rayon de 100m autour du site à prospecter.

Keywords: site investigation, cone penetration tests, piles, vibrations, soil structure interaction, geology of cities

INTRODUCTION

Small plots in city centres present more construction problems than a new urban development. The problems start with the site investigation where access may be limited and site investigation procedures and reporting are too much related to a more open environment. Based on a case history for a four storey terraced house in The Hague recommendations are made to expand the requirements of a site investigation. Site investigation requirements in the Netherlands over the last 20 years have been expanded to include investigation for contaminants in the subsurface. It is strange that other environmental hazards are not adequately addressed in the Dutch situation, namely disturbances and possible damage that may result from the construction works, possibly the most critical stage being installation of pile foundations.

Site investigation requirements for a small plot of five by twenty metres are two cone penetration tests of which one includes both q_c, end bearing and f, sleeve friction, measurements (so as to identify the soils). These are used to calculate the pile bearing capacities. In addition it is common practice to examine neighbouring structures for existing damage such as differential settlement and cracks in the plaster work, often in combination with a photographic record and installation of simple tell-tales over existing cracks to measure any possible movement during installation of piles.

A common solution is to resort to 'vibration-free' piles the most common being screw (augered) piles and another advertised as vibration free consisting of a hollow cylindrical steel tubes which are, however driven into the soils using a rig which is similar to driving casing in a shell and auger type operation. The latter was adopted at this site because the screw pile rig would be difficult to manoeuvre onto the site. Despite the claims to being vibration free the latter piles not only caused substantial vibration disturbance but also caused active and substantial settlement of adjoining structures.

The following hazards are considered during foundation installation: vibrations causing 1. annoyance to adjoining residents, 2. structural damage, 3. compaction of sub-soil causing subsidence, 4. liquefaction resulting in loss of foundation support, and 5. erosion of sublayers due to extraction of soil to insert open ended tubular piles.

Methods exist to predict both movement and disturbance, though they seldom seem to be included in site investigation reports. Despite compaction techniques, such as vibroflotation, often being used to improve the density of soils the effect of the vibrations are not usually considered. This is in spite of published case histories showing increased cone resistance profiles as a result of pile driving activity. Liquefaction susceptible layers can be identified from the cone penetration test but no case histories exist of such phenomena occurring during pile driving. The liquefaction study, though may indicate soils that easily become 'fluid' say under bailing operations in open ended piles causing sub-surface erosion.

A recent combined study by the foremost institutions in the Netherlands has examined uncertainties in prediction methods based on expert experience, simple prediction methods and more complex (computer driven) methods. These showed that showed that the level of uncertainty remained high and that expert experience and simple prediction methods appear to offer a lower level of uncertainty when predicting damage and disturbance from pile driving. (Anon, 2003).

Methods for analysing vibrations due to piling are not new and also not relegated to obscure learned papers as documentation exists which allows for prediction which can be accomplished by, say, an engineering geologist having to write a site investigation report for piled foundations.

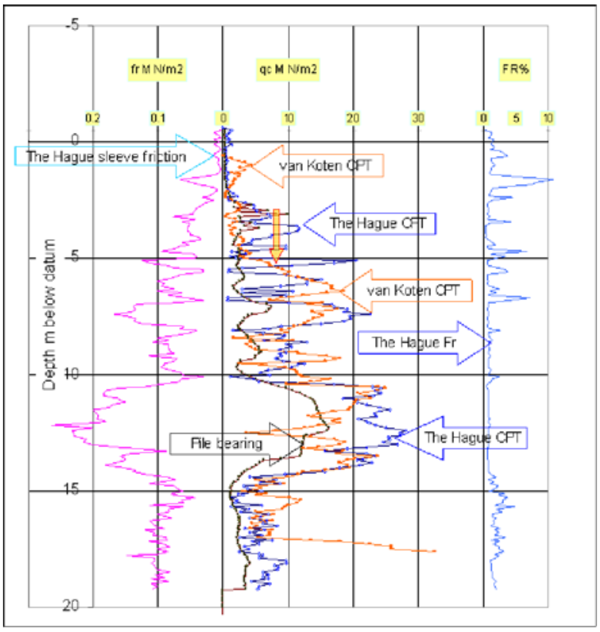


Figure 1. Cone penetration test profiles from The Hague and q_c profile of van Koten 1992 with position of pile driving.

PREDICTIVE METHODS

The predictive methods described below apply to installation of piles as the design methods are both well known and form part of standard procedure, namely, determining bearing capacity, depth of pile and number of piles. The method in the Netherlands is often referred to as the 'Koppejan method' that uses the CPT, static cone penetration profile. A typical profile for bearing capacity of a 20cm pile is shown in Figure 1 for the site in The Hague. A second method is used by the piling contractor to determine the number of hammer blows required for pile installation, often for different hammers and pile sizes. This results in a 'pile calendar' showing the number of blows for 10 cm intervals of depth. The method is often referred to as the 'wave equation'. The methods described below examine environmental impact of pile installation because of the close proximity of buildings and their residents to pile installation operations.

Vibration analysis

The mathematical method used for analysis is from van Koten, 1972. In this instance use is made of a subsequent publication by van Koten, 1992, translated from Dutch into English "Movements of the ground and buildings as a result of pile driving". A worked example has been reproduced here (and translated from the Dutch) as the example corresponds to the situation in The Hague. The results can also be compared with velocity measurements made by local municipal engineers.

The example: Pile driving is required on a site next door to a house consisting of foundations without piles and having a ground area of 7x10m. The cone penetration test profile (van Koten and The Hague site) is given in Figure 1.

The cone resistance, q_c, of the layer wherein driving requires extra effort is 8Mpa. The elastic modulus E at that depth is greater than $10.q_c$. For firm soil layers the elastic modulus at low-stress ground-waves is from 150 to 200 MPa. The cross-section the piles is 0.32x0.32 metre square. The maximum vertical velocity of the house at a distance of 14 m from the pile foot is given in table 1:

Table 1: Parameters, definitions, equations and calculations for determination of vertical velocities induced by pile driving on a building adjoining the construction site.

symbol	description	formula	source(s) for value	value	units
q_c	cone tip resistance		CPT profile	8	MPa
D	equivalent pile diameter		pile dimension	0.32	m
E	elastic modulus soil		text	200	MPa
ρ	density soil		table of values	1500	kg/m^3
		ρ for calculations where N=kgm/s²		0.0015	s²MPa/m²
R	distance pile foot and observation point			5	m
l	minimum building dimension			7	m
L	wave length	$2tc_s=(2\pi/0.1).D/[2(1+\nu)]^{½}$	$(2\pi/0.1)/[2(1+\nu)]^{½}=40$, D=0.32	12.8	m
t	half wave period	π/ω			s
ω	circle frequency	$(0.1/D).[E/\rho]^{½}$	$(0.1/0.32)\cdot[200/15]^{½}$	114.11	Hz
f	frequency	ω/2π		18.161	Hz
c_s	shear wave velocity soil	$[E/\{2(1+\nu)\rho\}]^{½}$	$[200/\{2(1+0.3)\cdot0.0015\}]^{½}$	226.46	m/s
ν	Poisson's ratio			0.3	
c_c	compression wave velocity	$[E/\rho]^{½}$		365.15	m/s
Rafs	reduction due to wave decay	$e^{(-0.5R)}$	exp(-0.5·14)	0.8	
Rb	reduction due to λ	$e^{(-0.5l/L)}$	exp(-0.5·7/{40·0.32})	0.76	
λ	influence factor:minimal building dimension/wave length	l/L	7/12.8		
V	maximum vertical velocity observation point	$0.03\,(q_cD/ER).[E/\rho]^{½}\,R_{afs}R_b$			m/s
		$0.03\,(q_cD/ER).c_cR_{afs}R_b$	0.03·{8·0.32/(200·14)}·200·0.5·0.8[1]	0.0166	m/s
Vh		$0.005\,(q_cH/ER).[E/\rho]^{½}\,R_{afs}R_b$	0.005·(8·6/200·14)·230·0.5·0.9[1]	0.0284	m/s

[1] Values used by van Koten can appear confusing when calculating wave velocities using the E value and density ρ. He uses a higher E value with correspondingly low wave velocities which do not satisfy the wave velocity equation for c_c or c_s. Presumably the higher E value is taken because 'ground is more rigid at depth'.

The values from the above analysis and that from measurements taken on site to measure the vibrations on applying an impact to a partially inserted pile gave measurements lower than those predicted above. The velocities are plotted in a chart shown in figure 2. The classification used in the chart is extended to damage that can occur from the vibrations:

A: Structural collapse
B: Local damage
C: Fractures form in masonry
D: Initial signs of fracture formation
E: Little influence on normal buildings
F: No influence
G: No influence

The results of this study would confirm that damage could have been predicted by pile driving operations and confirmed by on site measurements.

Layering can influence wave propagation and attenuation with respect to resonance resulting from reflection of the seismic waves within a layer. A recent report in a web-site journal 'Infrasite' announced the following:

"Geotechnical investigation has shown that the subsurface along the Eindhoven orbital motorway (A2/A67) alternates with hard and soft layers. Such a ground structure is quite sensitive for vibrations. These vibrations could cause damage on buildings in the immediate neighbourhood. The Rijkswaterstaat (state public works department) will carry out tests to determine

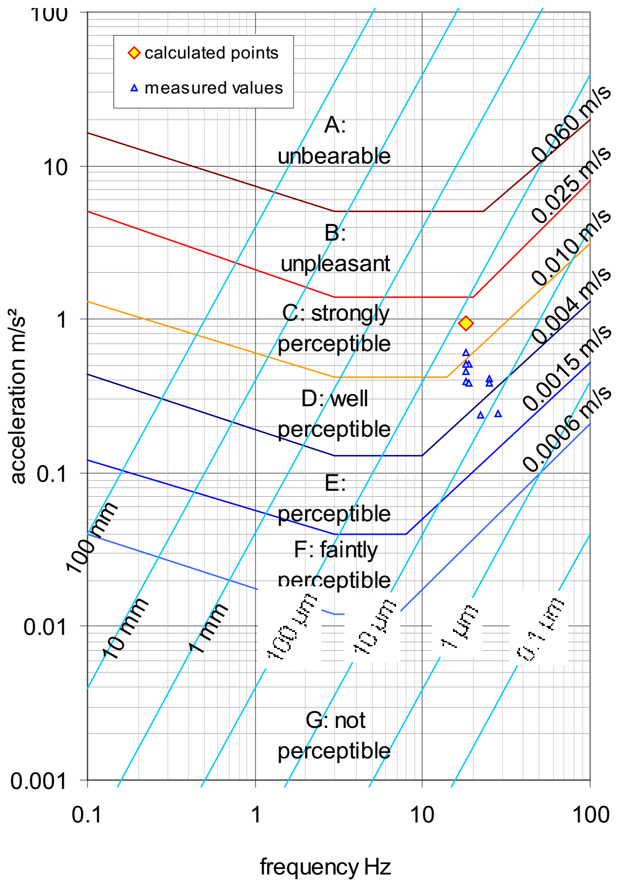


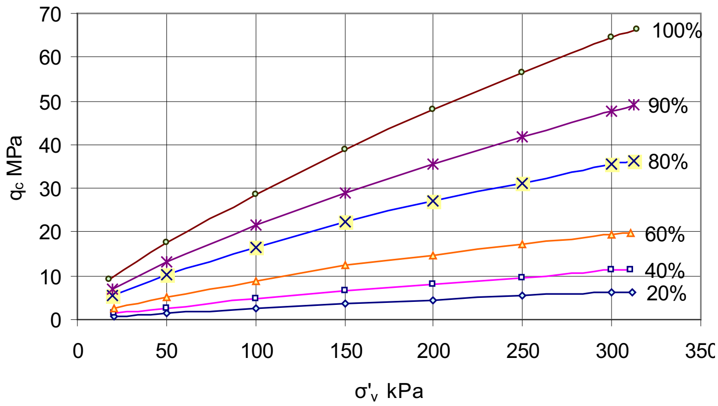
Figure 2: Chart from van Koten 1992 with superimposed values from the predicted values and those measured at the site.

what damage vibrations and pile-driving would result. On the basis of these tests the Rijkswaterstaat will be able to contain nuisance and damage to a minimum. In addition all buildings within 100m of the work will be examined for their structural quality to ensure that any subsequent claims for damage can be shown to have occurred after the start of construction.”

Dynamic compaction

The above approach examines possible nuisance to residents and damage to their buildings near or adjoining the construction site. The analysis assumes the ground to be, more or less, an elastic medium and does not consider possible compaction that may occur due to vibrations. Studies relating to pile driving compacting the soil have been made more to examine the increased hammer energy required to install subsequent driven piles. On The Hague site this possibility did exist as the house adjoining the site started to subside as piling progressed which would not have been predicted by the foregoing methods. One approach to predict compaction is to correlate the cone penetration test end bearing qc and friction ratio Rf values with relative density. Such graphs have been produced by Schmertman (1973), Searle (1979) and inferred from Robertson (1990) The correlation with Schmertman appear to produce values exceeding 100% Dr, (relative density). The Schmertman chart has been modified, the example shown in Figure 3 is from Lunne & Chrisopherson, 1983 for electrical qc values. To obtain densities a further chart in Figure 4 relates unit weight with relative density Dr.

Relative densities could be derived from cone tests using the q_c-Rf chart by Searle 1978. The Hague CPT values when plotted on Searle’s chart suggest the relative densities to be realistic. The chart misleads with regard to soil type indicating a degree coarser fraction for the soil type. This may be that Searle’s chart is based on the mechanical cone. Robertson’s chart has been super-imposed. His chart is based on the electrical cone and corresponds well with the type of soil fraction one would expect in The Hague. Robertson’s more recent ‘normalized’ chart



$$D_r=\log(31q_c\sigma'_v{}^{-0.82})/\log(1.037\sigma'_v{}^{-0.0013})$$

Figure 3. Relative Density based on Lunne & Christoffersen 1983 modified from Schmertman 1973

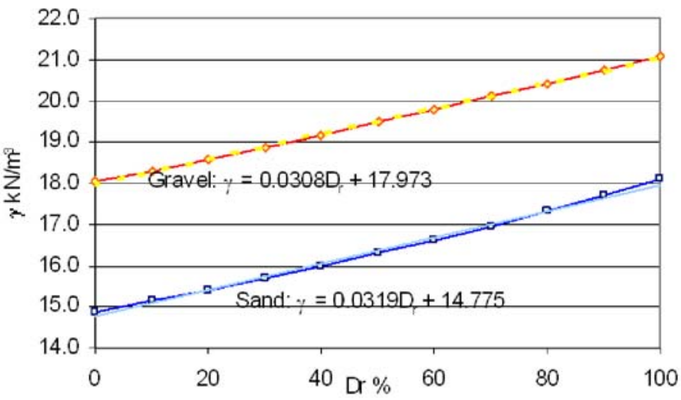


Figure 4. Unit weight versus relative density, CROW 2004

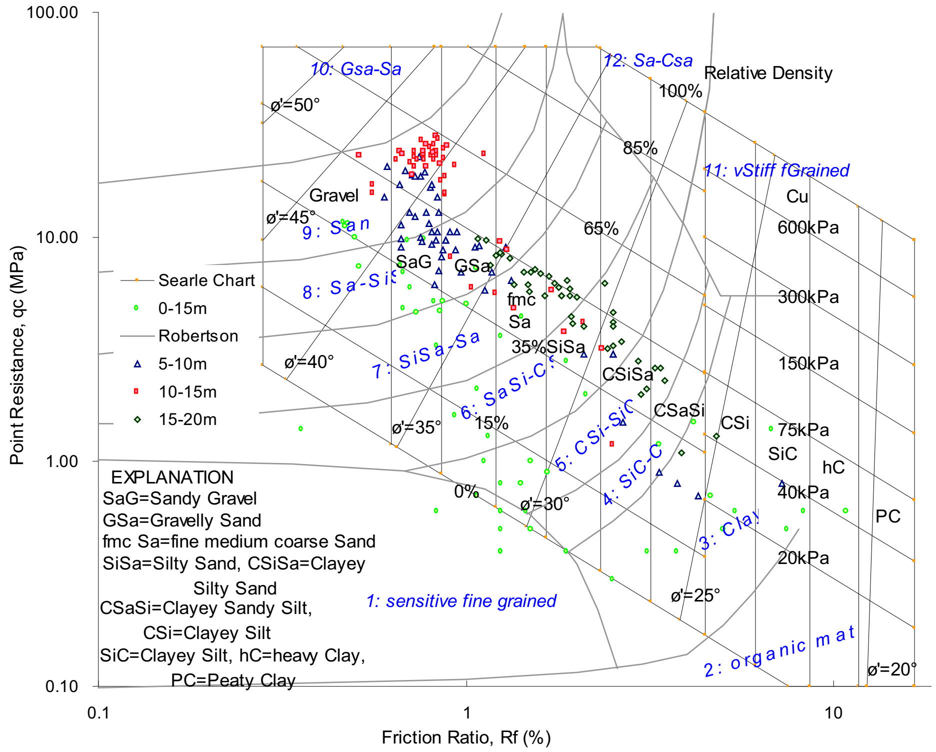
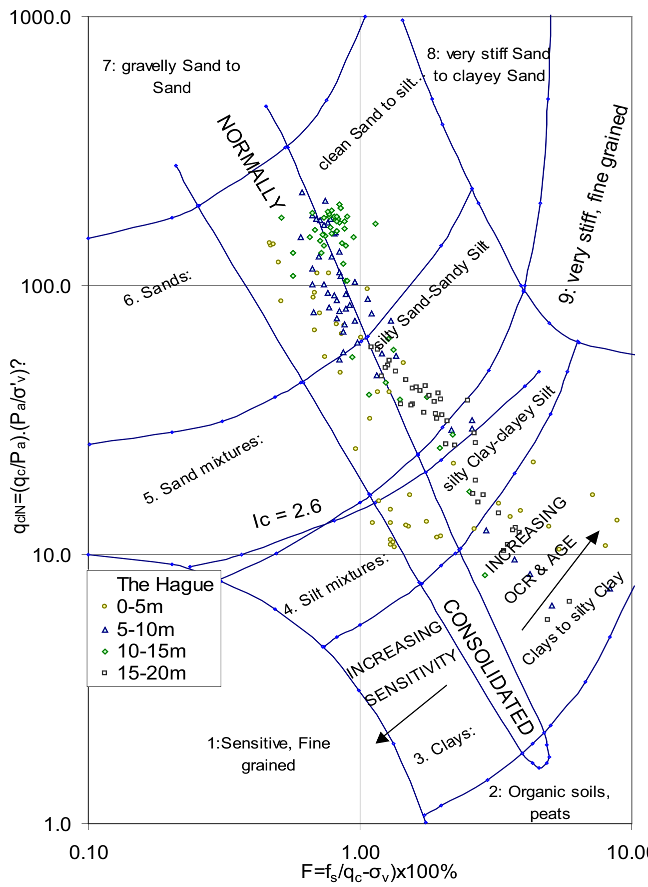


Figure 5. CPT Chart from Searle 1979 with Robertson 1990 superimposed and q_c Rf values from The Hague CPT.



Note: if(I_c <2.6, n=0.5, n=0.75), I_c= [(3.47-logQ)² +(logF+1.22)²]^{½}
(Wride et al., 2000) Q=q_t-σ_{vo}/σ'_{vo}, q_t=q_c, q_t=q_c +(1-d²/D²)u,
d (load cell support)=D (diameter cone).
For sands can approximate q_c=q_t, (Robertson, 1990).
P_a=100kPa.

Figure 6. Normalized CPT end bearing and friction ratio chart from Wride et al 2000 with superimposed CPT values from The Hague site

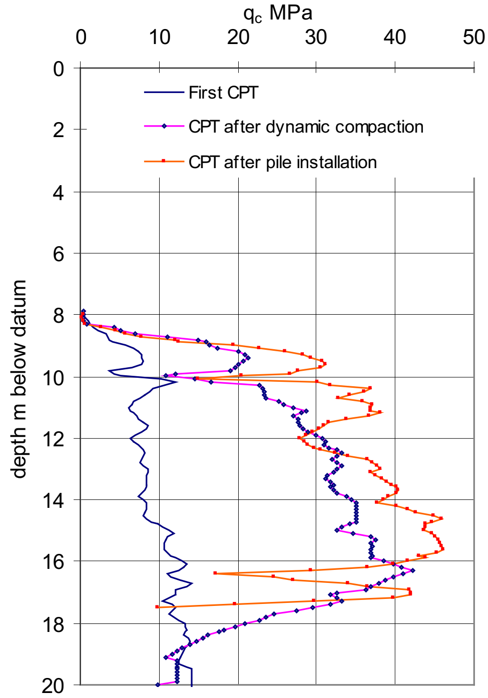


Figure 7. Cone penetration tests IJmeer lake railway bridge foundations van Rossum 1985

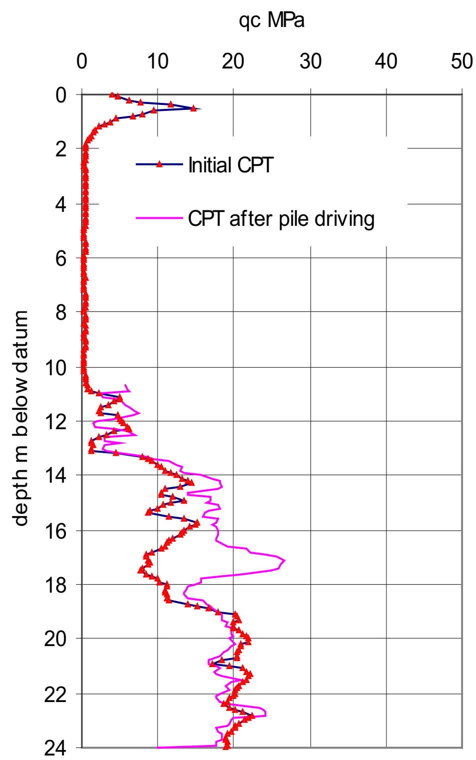


Figure 8. CPT increase as a result of installation two piles 1.65m distant in Rotterdam, van Weele & Schellingerhout 1991

allowing for vertical effective stress is shown in Figure 6 with the CPT data once again plotted depicting similar clusters for the 5-10m depth zone and the 10-15m depth zone. The 5-10m zone plots as a cluster as sand in the normally consolidated region indicating this zone could possibly become more dense on pile driving.

Actual densities would allow for an estimate of potential compaction under pile driving. Two CPT profiles were examined from the literature showing the increased q_c values as a result of pile driving in Figure 7 for a bridge across the Ijmeer east of Amsterdam and Figure 8 for foundations in Rotterdam. In both cases the soil type is assumed to be sand. To estimate the density increase in these profiles figure 3 is used to determine relative density in tandem with the chart in figure 4 relating relative density with unit weight. To balance the unit weight for determining the vertical effective stress with the unit weight derived from relative density (which requires the effective stress) an iterative procedure was adopted. The empirical best-fit equations (shown in the figures) were derived to speed the process. Unit weights were obtained for every 0.1m interval of soil depth corresponding with the resolution obtained from CPT profiles. In Figures 9 and 10 rough correlations were made to obtain an indication as to what density increase was obtained as a result of pile driving. This was used to estimate potential density increases for the sands at The Hague site. For the 'normally consolidated' sands between 5 and 10m up to 140mm settlement was estimated.

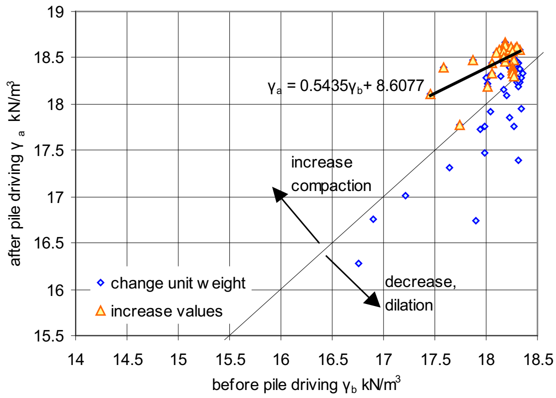


Figure 9. Before and after pile driving density correlation for Ijmeer bridge site.

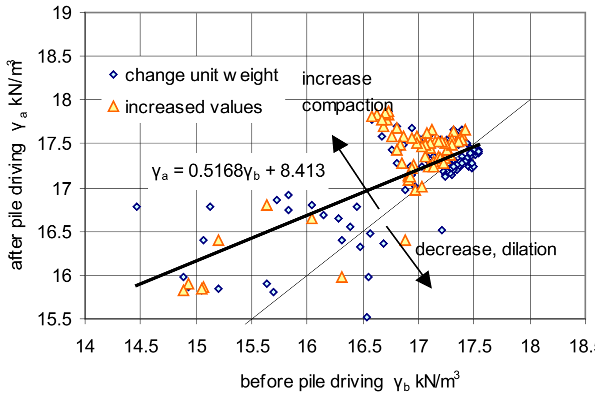


Figure 10. Before and after pile driving unit weight correlation for Rotterdam site

The procedure described above indicates that the order of settlement predicted is similar to that which occurred during piling operations.

Observations on site during the driving phase suggest that settlement occurred by undermining support of existing shallow footings as sand flowed into the open ended steel tubes. Judging, however, by the complaints from residents in the immediate vicinity of the piling operation and the above estimate suggests that dynamic compaction probably took place.

Liquefaction and flow

A third hazard could be liquefaction in which the vibrations would again cause the sand to compact but compaction would be prevented as the ground water would not be able to drain. Figure 6 was used in a major research project CANLEX, Canadian Liquefaction Experiment to examine susceptibility of soils layers to liquefaction. The sand layers at dam sites in Canada believed to be prone to liquefaction plotted within zones 6 and 5 in the 'increasing sensitivity' zone. The site in The Hague appears to be sufficiently consolidated to resist liquefaction, plotting in the normally consolidated zone and extending into the 'increasing over-consolidation' zone.

Unfortunately, presumably because problems were not expected, no careful record has been kept during the pile installation of either pile calendars as or when difficulties arose. Discussions with experienced engineers on foundations in the Netherlands stated that such piling should be installed with water up to ground level to prevent inflow of sand. The pile was driven by a falling weight on the gravel placed at the base of the pile, or by a falling weigh on an anvil attached to the top of the pile. When using the gravel for driving water would have to be lowered in the pile to allow the hammer to drop freely without hydraulic resistance. Lowering the water table and causing an inflow would probably make pile driving less strenuous but at the expense of local subsidence. Placing the piles only a short distance further away (about 1.5 to 2m) would possibly have avoided the worst influences on adjoining buildings.

CONCLUSIONS: URBAN SITE INVESTIGATION IMPROVEMENTS

In the early 1980s the author was involved in site investigation in Dubai. At one stage the municipal engineers questioned the results of a site investigation carried out under the auspices of the author. They wanted to know why, from an earlier investigation, piled foundations to about four to five metre depth was advised for a light industrial building and this time, a few years later, at an adjoining plot only shallow footings were advised. Inspection of the CPT logs indicated that the earlier investigations indeed showed that piles were needed and the subsequent investigation that shallow foundations would be sufficient. To placate the municipality an additional investigation was offered free of charge. During discussions with the municipal engineers they offered a possible explanation for the differences: not too distant from the site, construction of the clock tower underpasses had been in progress for over a year resulting in installation of sheet piles and lowering of groundwater. Both these operations, especially the latter, would compact the soil to such extent that piles would not be required.

Could a similar scenario have occurred in The Hague? Inspection of site investigation reports for recent infill construction work nearby revealed CPTs with similar profiles. The next obvious step is to examine the type of piles used. It soon became apparent that the foundations for an adjoining old building were underpinned with piles, presumably to ensure no adverse settlement would result when installing piles for the new structure. Should The Hague municipal engineers have forewarned the developers or architect that such problems could arise? Either way one could only conclude that more effort should be made to predict installation hazards both on the basis of predictive models and the investigation of readily available records such as site investigation reports and construction drawings deposited with the municipality.

The simplest approach would be that when submitting an application for construction in an urban environment the applicant must show that such records had been inspected and have suitable copies made and appended to the application. Additionally very little extra effort is required to determine predictions of pile driving influence on the immediate environment on the basis of existing cone penetration test methods used for determining pile depth and bearing capacity.

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